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244 Chestnut Street, Salem, NJ 08079

PSEG
Power LLC

ND-2012-0005
January 20, 2012

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

Subject: **PSEG Early Site Permit Application**
Docket No. 52-043
Response to Request for Additional Information, RAI No. 41, Stability of Subsurface Materials and Foundations

References:
1) PSEG Power, LLC letter to USNRC, Application for Early Site Permit for the PSEG Site, dated May 25, 2010

2) RAI No. 41, SRP Section: 02.05.04 – Stability of Subsurface Materials and Foundations, dated December 8, 2011 (eRAI 6153)

3) PSEG Power, LLC Letter No. ND-2012-0001 to USNRC, Response to Request for Additional Information, RAI No. 41, Stability of Subsurface Materials and Foundations, dated January 6, 2012

The purpose of this letter is to respond to the request for additional information (RAI) identified in Reference 2 above. This RAI addresses Stability of Subsurface Materials and Foundations, as described in Subsection 2.5.4 of the Site Safety Analysis Report (SSAR), as submitted in Part 2 of the PSEG Site Early Site Permit Application, Revision 0.

Enclosure 1 provides our response for RAI No. 41, Question Nos. 02.05.04-7, 02.05.04-9, 02.05.04-13, 02.05.04-14, 02.05.04-15, and 02.05.04-18. The responses to the remaining RAI questions were provided in Reference 3.

Enclosure 2 includes the revisions to SSAR Subsection 2.5.4 resulting from our response to RAI No. 41, Question No. 02.05.04-9. Enclosure 3 includes the new regulatory commitments established in this submittal.

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NRO

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

I declare under penalty of perjury that the foregoing is true and correct. Executed on the 20th day of January, 2012.

Sincerely,

James Mallon

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

Enclosure 1: Response to NRC Request for Additional Information, RAI No. 41, Questions Nos. 02.05.04-7, 02.05.04-9, 02.05.04-13, 02.05.04-14, 02.05.04-15, and 02.05.04-18, SRP Section: 02.05.04 – Stability of Subsurface Materials and Foundations

Enclosure 2: Proposed Revisions, Part 2 – Site Safety Analysis Report (SSAR), Subsection 2.5.4 - Stability of Subsurface Materials and Foundations

Enclosure 3: 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-2012-0005, dated January 20, 2012

ENCLOSURE 1

RESPONSE to RAI No. 41

QUESTION Nos.
02.05.04-7
02.05.04-9
02.05.04-13
02.05.04-14
02.05.04-15
02.05.04-18

Response to RAI No. 41, Question 02.05.04-7:

In Reference 2, the NRC staff asked PSEG for information regarding Stability of Subsurface Materials and Foundations, as described in Subsection 2.5.4 of the Site Safety Analysis Report. The specific request for Question 02.05.04-7 was:

> *SSAR Section 2.5.4.2.1.3.4 states that a Ko=.5 was used to calculate horizontal effective stresses on samples for RCTS Testing. The applicant mentioned that Ko=.5 is considered a typical value for generally normally consolidated soils. In compliance with 10 CFR 100.23 (d) (4) and in conformance to NUREG-0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," please provide additional details to justify selecting this value, especially when SSAR Section 2.5.4.10.3 states that the soils in the Vincentown formation and below are considered to be over-consolidated.*

PSEG Response to NRC RAI:

The RCTS testing is performed at five different confining pressures related to an estimated in-situ pressure calculated by the following equation (considering isotropic conditions):

$$\sigma_m = (\sigma_v.(1+2(K_o)))/3$$

Where:

σ_m = the mean confining pressure
σ_v = the vertical pressure
K_o = the ratio of vertical to horizontal stress

The mean confining pressure is initially calculated using K_o equal to 0.5 and considering isotropic conditions. Because the value of K_o may not be known for a particular sample, values of confining pressure for RCTS testing are taken as 0.25, 0.5, 1, 2 and 4 times the calculated mean confining pressure based on sample vertical pressure. The purpose of using multiple test confining pressures is to allow for variations in the estimated K_o value. The multiples of 2 and 4 times the calculated mean confining pressure imply K_o values of 1.5 and 3.5, respectively, considering isotropic conditions. Thus the RCTS test represents soil behavior at a wide range of possible consolidation conditions.

Associated PSEG Site ESP Application Revisions:

None.

Response to RAI No. 41, Question 02.05.04-9:

In Reference 2, the specific request for Question 02.05.04-9 was:

> *SSAR Section 2.5.4.2.2.1.5 states that for engineering purposes the Vincentown and Hornerstown formations are combined into one engineering layer due to their similar engineering properties. In compliance with 10 CFR 100.23 (d) (4) and in conformance to NUREG-0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," and in order to reach this conclusion, please provide additional details regarding properties from both layers and how overall properties were weighted. Also, please justify that both formations would behave similarly, especially when the Vincentown formation is classified as mostly a silty sand layer while the Hornerstown has a considerable increase in fine content.*

PSEG Response to NRC RAI:

For engineering purposes, the Vincentown and Hornerstown Formations are grouped into one unit based on the formations having similar engineering characteristics including Unified Soil Classification System (USCS) classifications, percent fines, standard penetration test (SPT) penetration resistances (N-values) and shear wave velocities, V_s. The basis for considering these two formations together is discussed below and summarized in Table RAI-41-9-1.

SSAR Table 2.5.4.2-2, Sheets 9 and 10 of 18, present the USCS classifications based on laboratory tests of soil samples from the Vincentown and Hornerstown Formations obtained in the NB and EB borings. For the Vincentown Formation, the USCS classification was determined on 33 samples. Of these, 30 are classified as poorly graded sand, silty sand or clayey sand (USCS classifications SP, SM, SC, SC-SM or SP-SM). The remaining three samples are silts and clays and have USCS classifications MH, ML and CL. For the Hornerstown Formation, the USCS classification was determined on 14 samples. The samples tested are classified as poorly graded sand, silty sand and clayey sand (USCS classifications SM, SP-SM and SC).

The percent of silt and clay fines was determined on 31 samples from the Vincentown Formation. Excluding the three samples having USCS classification MH, ML and CL, which have over 50 percent silt and clay fines, the average percent fines in the sandy soils is 24 percent. The average percent fines of 13 samples obtained in the Hornerstown Formation is also 24 percent.

The SPT N-values and corresponding geologic formations for tests performed in the EB and NB series borings are included in Tables RAI-8-1 through RAI-8-16 in the response to RAI No. 8. From those Tables, the average SPT N-values are 45 blows per foot (bpf) and 52 bpf in the Vincentown Formation and Hornerstown Formation, respectively.

Soil dynamic properties were determined from geophysical logging using the P-S suspension seismic velocity logging method performed in borings NB-1, NB-8, EB-3, and EB-8G. The simulated downhole travel times were used to calculate the shear wave and compression wave velocities for discrete depths without consideration of geologic formations. The results are shown in SSAR Figures 2.5.4.7-6A, B, C, and D which also contain the geologic formation delineations.

Based on the results of the four P-S logging surveys, the shear wave velocity measured in the Vincentown, Hornerstown and Navesink Formations [identified as layer 1 on SSAR Figure 2.5.4.7-8(a)] ranges from 2036 to 2584 feet per second as shown on SSAR Table 2.5.4.7-3. To determine the average shear wave velocity for the individual formations, the simulated downhole travel times were reevaluated for the Vincentown and Hornerstown Formations using the methods discussed in SSAR Subsection 2.5.4.7.4.1. As shown on Table RAI-41-9-1, the shear wave velocities determined from analysis of the P-S suspension velocity data of the Vincentown Formation are 2101 feet per second and 2233 feet per second for the Hornerstown Formation.

As summarized on Table RAI-41-9-1, soils of the Vincentown and Hornerstown Formations have similar USCS classifications, percent fines, field SPT N-values and shear wave velocities, justifying combination of these formations for engineering purposes. Based on these similarities, design values presented in SSAR Table 2.5.4.2-8 were determined by considering data obtained from the Vincentown and Hornerstown Formations with no weighting.

In preparing the response to this question, it was noted that SSAR Subsection 2.5.4.2.2.1.5, pages 2.5-228 and 2.5-229, referenced the incorrect sheets of Table 2.5.4.2-2. These pages will be modified as described below.

Associated PSEG Site ESP Application Revisions:

SSAR Subsection 2.5.4.2.2.1.5 will be revised to correct the references to specific sheets of Table 2.5.4.2-2 as shown in Enclosure 2.

Table RAI-41-9-1
Summary of Vincentown and Hornerstown Properties
EB and NB Series Borings

	Vincentown Formation	Hornerstown Formation
USCS Classification		
SM	23	11
SC-SM	3	-
SP-SM	1	1
SC	3	2
MH	1	-
ML	1	-
CL	1	-
Percent Fines[(a)]		
Average	24	24
Maximum	36	36
Minimum	12	9
Median	22	24
Field SPT N-values		
Average	45	52
Maximum	100	100
Minimum	5	18
Median	30	42
Shear Wave Velocity		
Average (ft/sec)	2101	2233

(a) Percent fines of samples with USCS classifications SM, SC, SC-SM and SP-SM in the Vincentown and Hornerstown Formations.

Response to RAI No. 41, Question 02.05.04-13:

In Reference 2, the specific request for Question 02.05.04-13 was:

> *SSAR Section 2.5.4.7.5 indicates that the applicant did not use RCTS test results to characterize the degradation property of foundation bearing soils because of sample disturbances of the cemented soil layers. Darendeli equations were instead used to estimate modulus reduction and damping variation with shear strain. In compliance with 10 CFR 100.23 (d) (4) and in conformance to NUREG-0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," please justify the validity of such equations and how they could represent actual degradation properties of the soils at the site, and discuss whether it is a conservative approach when used in site seismic response analysis.*

PSEG Response to NRC RAI:

The equations to produce modulus reduction and damping variation with shear strain used in the SSAR (Darendeli equations) were the result of research work at the University of Texas under the direction of Dr. Ken Stokoe. The validity of these equations is supported by comparisons with data from the DOE Savannah River Site as reported in SSAR Reference 2.5.4.7-10. The subsurface conditions at the Savannah River Site are similar to those at the PSEG Site, consisting of layered Tertiary and Cretaceous sediments.

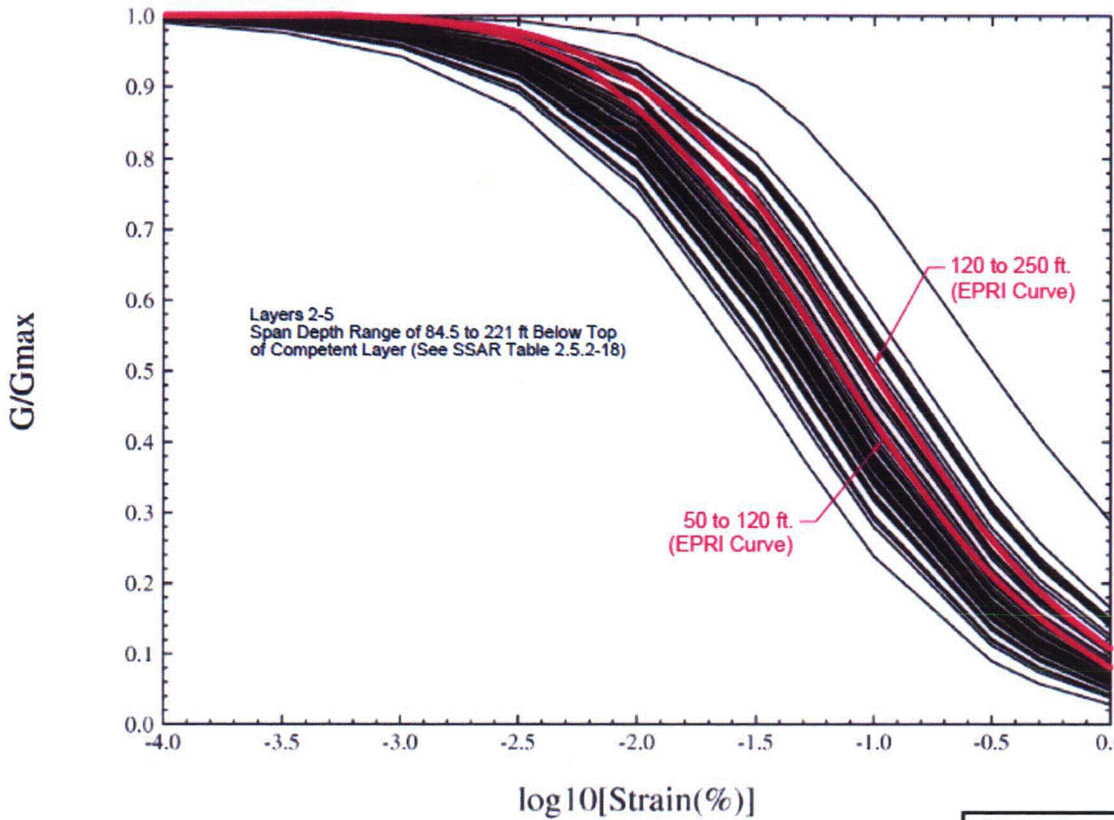
The results of the RCTS testing on samples from the Vincentown, Hornerstown, and Navesink Formations at the PSEG Site, as discussed in SSAR Subsection 2.5.4.7.5, were interpreted as inconsistent and potentially affected by presence of cemented layers. There are no other data on degradation and damping variation characteristics of the site soils available. Based on the general similarity of subsurface soil types at the PSEG Site and the Savannah River Site and the successful use of the Darendeli equations with those soil types, use of the calculated curves is a reasonable approach to representing degradation properties of the soils at the PSEG Site.

The calculated curves were used in conjunction with other factors to develop 60 synthetic profiles for use in the GMRS analysis as described in SSAR Subsection 2.5.2.5.2.1. The synthetic profiles are shown in SSAR Figures 2.5.2-38 and 2.5.2-39. The plots on those figures have been separated into Figures RAI-41-13-1 through RAI-41-13-4 and the EPRI generic curves for the appropriate depth range of the layers on the SSAR figures have been overlaid. As can be seen, the EPRI curves are within the range of the synthetic profiles.

The calculated degradation curves were also used in calculating an elastic modulus for the settlement analysis as described in SSAR Subsection 2.5.4.10.3. As shown in Figures RAI-41-13-5 through RAI-41-13-8, the mean lines of the calculated degradation curves are either slightly lower than the generic EPRI curves for the layer depth range, or in the case of Layer C, approximately in the same position. This indicates that elastic modulus values computed using the generic EPRI curves would be equal to or greater (stiffer) than those from the calculated curves used to develop the SSAR settlement estimates. Thus, using the values developed from the calculated curves, the settlement is equal to or greater than it would be using the EPRI curves - a conservative approach.

Associated PSEG Site ESP Application Revisions:

None.



Reference: Base from SSAR Figure 2.5.2-38, EPR1 Curves from SSAR Reference 2.5.4.7-3.

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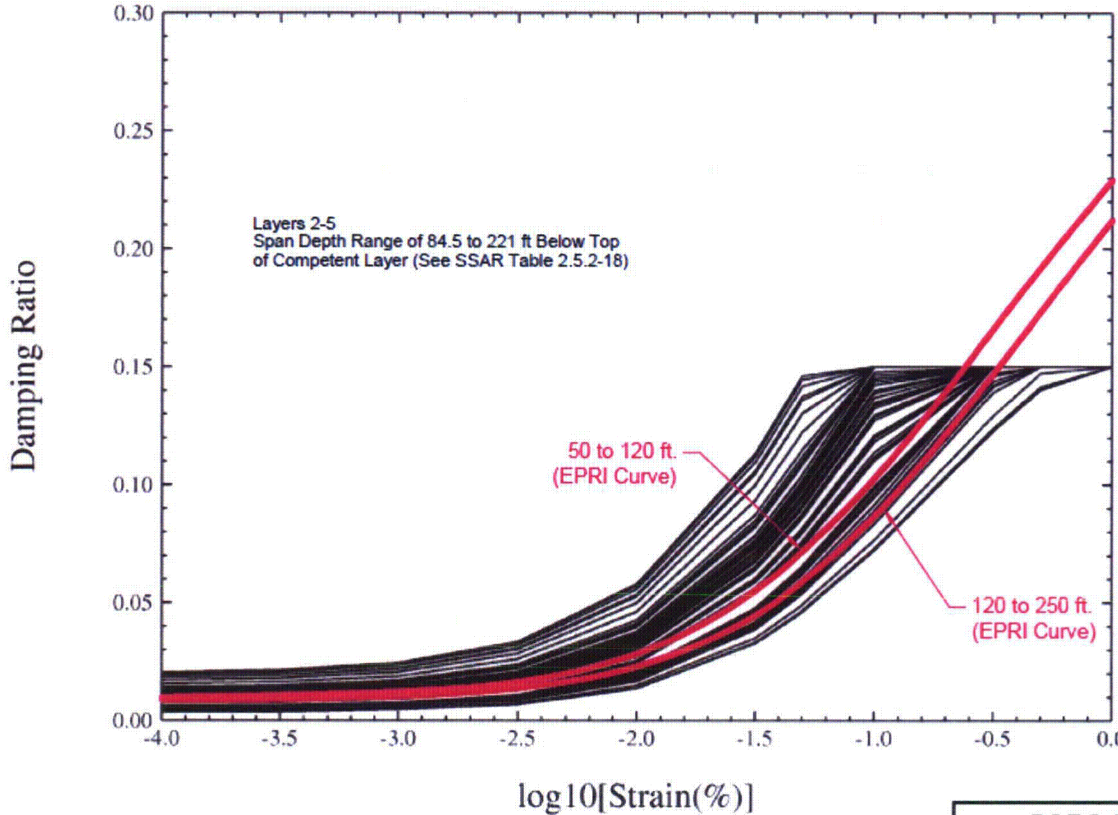
PSEG SITE ESPA
Part 2, Site Safety Analysis Report

Comparison of EPRI Curves to Synthetic Modulus Degradation Curves Layers 2-5

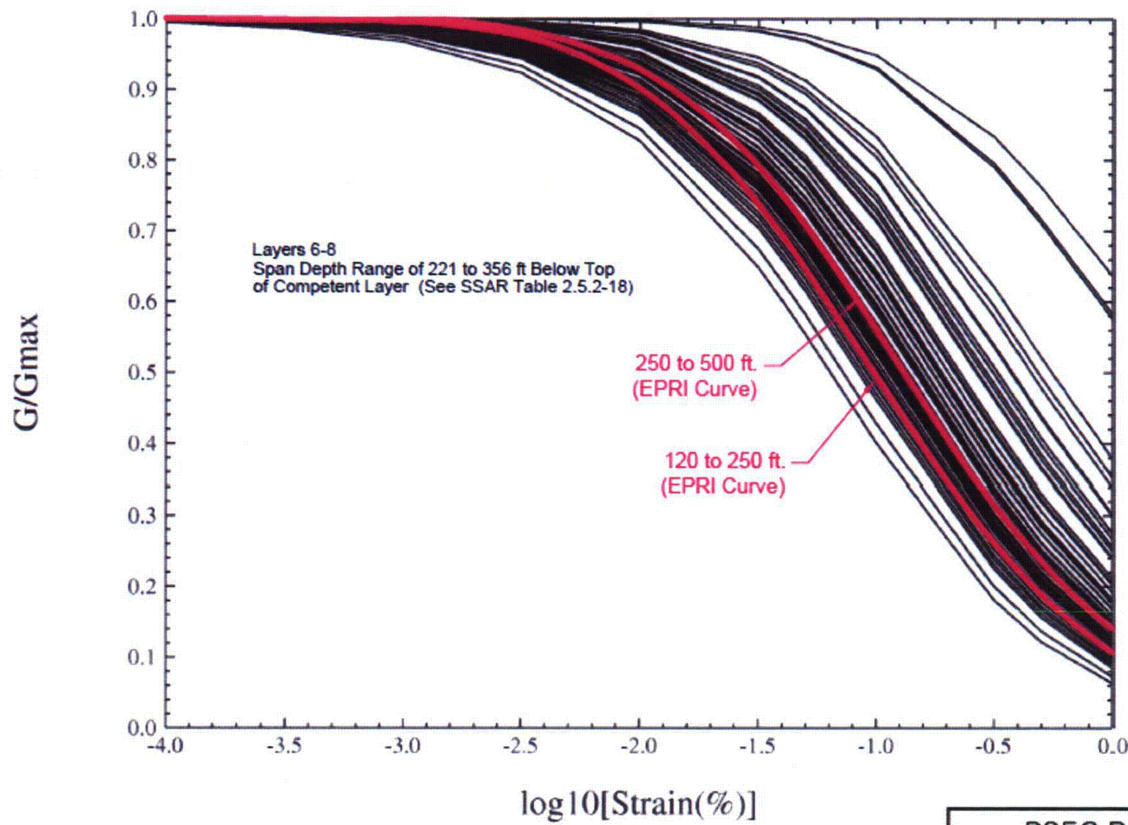
FIGURE RAI-41-13-1

Rev 0



Reference: Base from SSAR Figure 2.5.2-38, EPR1 Curves from SSAR Reference 2.5.4.7-3.

PSEG Power, LLC PSEG SITE ESPA Part 2, Site Safety Analysis Report
Comparison of EPRI Curves to Synthetic Damping Degradation Curves Layers 2-5 FIGURE RAI-41-13-2 Rev 0



Reference: Base from SSAR Figure 2.5.2-39, EPR1 Curves from SSAR Reference 2.5.4.7-3.

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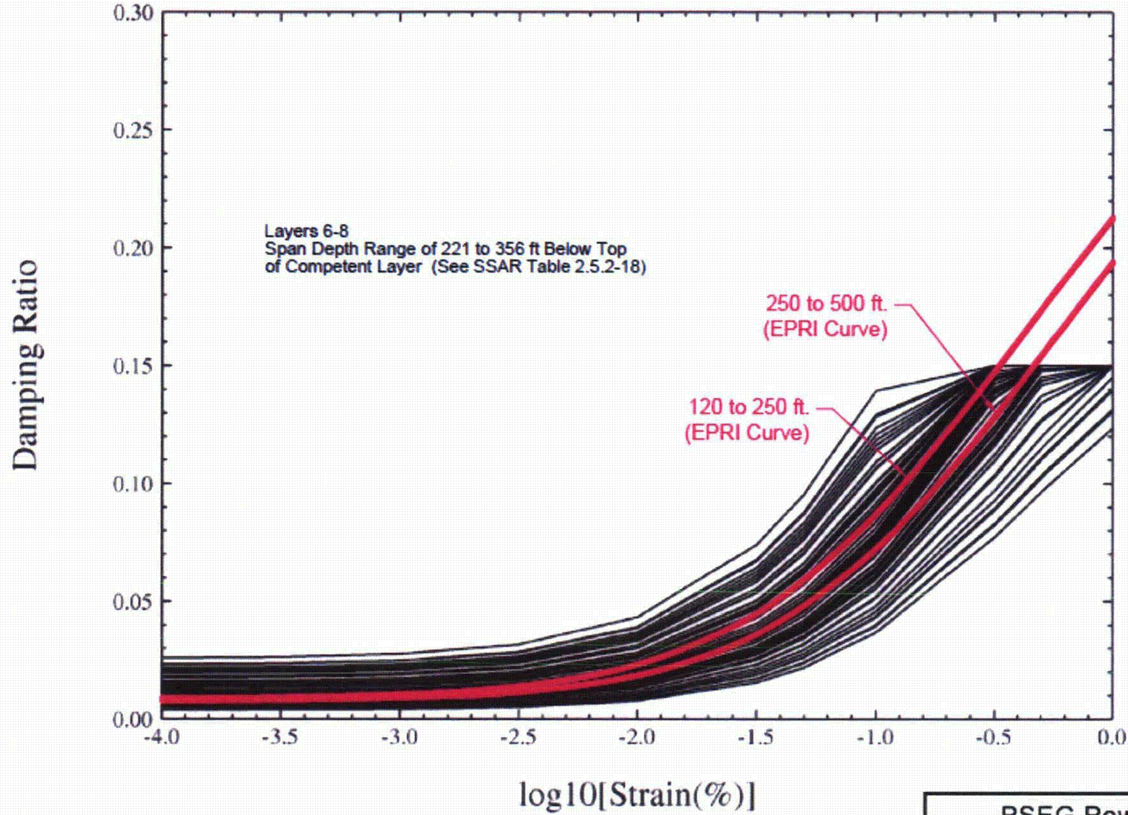
Part 2, Site Safety Analysis Report

Comparison of EPRI Curves to Synthetic Modulus Degradation Curves Layers 6-8

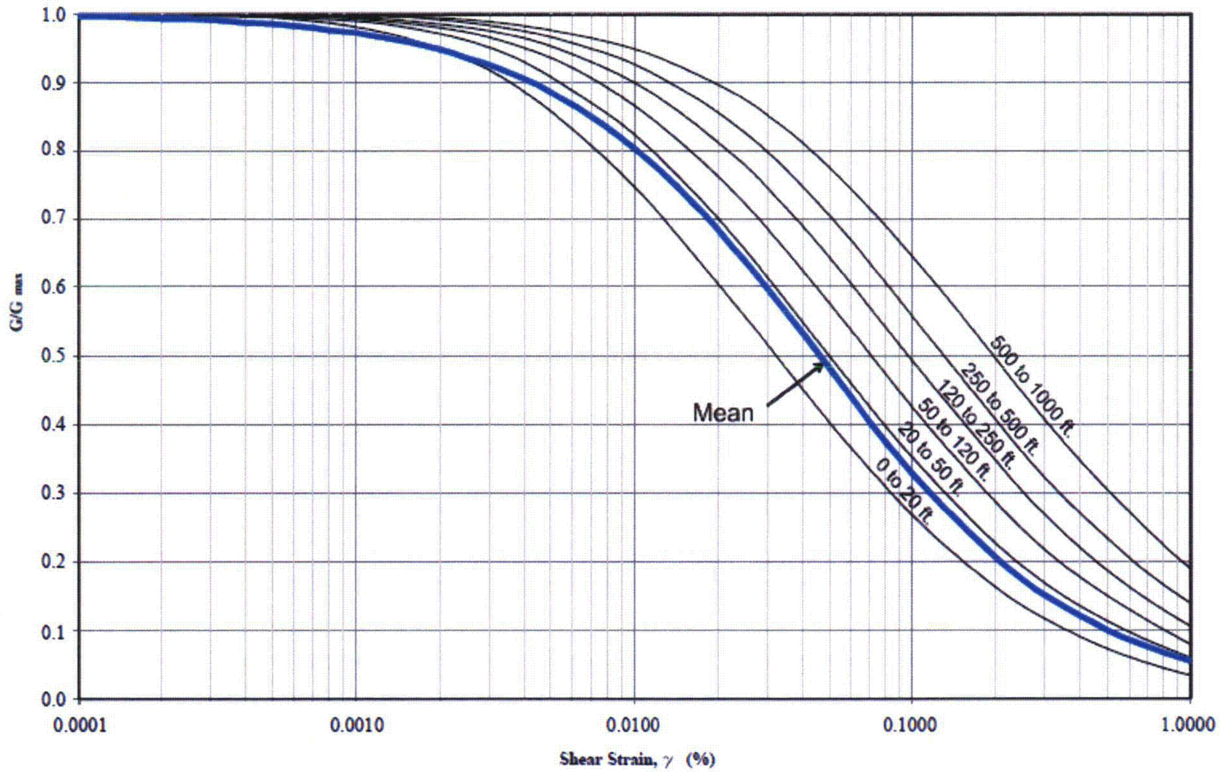
FIGURE RAI-41-13-3

Rev 0



Reference: Base from SSAR Figure 2.5.2-39, EPR1 Curves from SSAR Reference 2.5.4.7-3.

PSEG Power, LLC PSEG SITE ESPA Part 2, Site Safety Analysis Report
Comparison of EPRI Curves to Synthetic Damping Degradation Curves Layers 6-8 FIGURE RAI-41-13-4 Rev 0



Layer A is from 76 to 160.5 ft Below Ground Surface (See SSAR Figure 2.5.4.7-8a)

Reference: EPR1 Curves from SSAR Reference 2.5.4.7-3.

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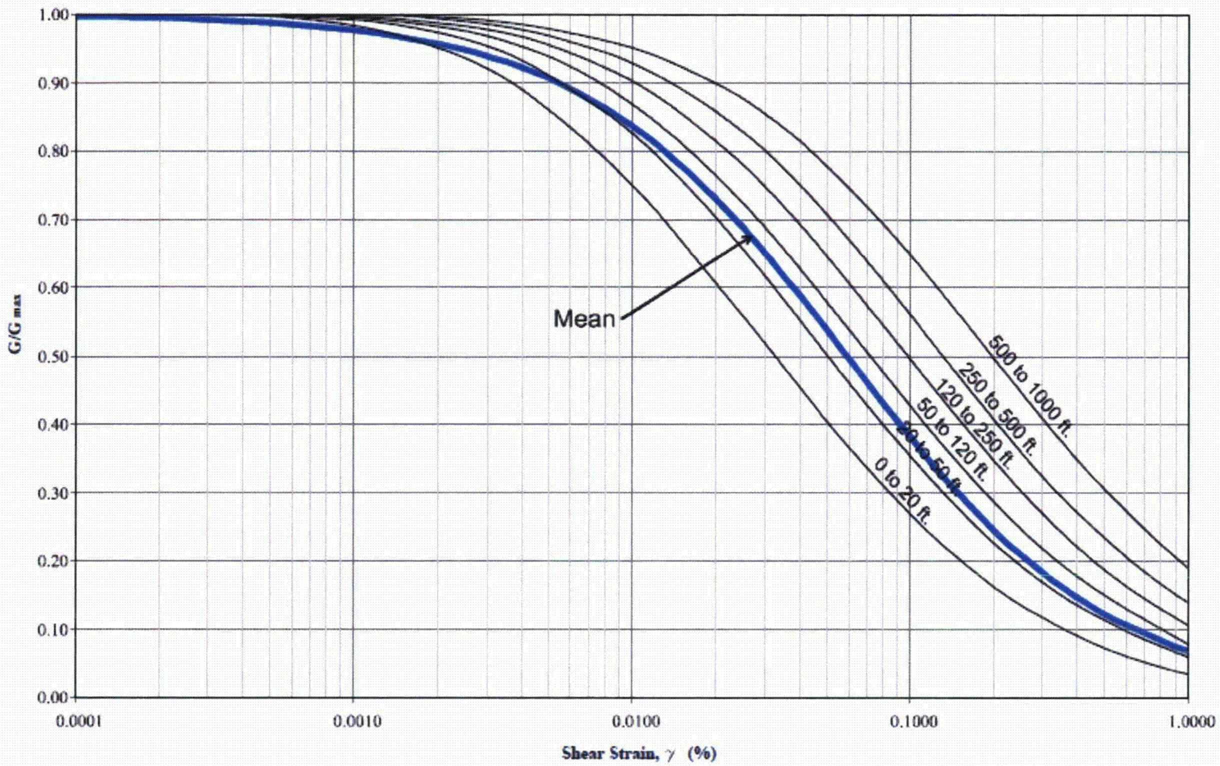
Part 2, Site Safety Analysis Report

Comparison Calculated Modulus Reduction Curves to EPRI Curves, Layer A

FIGURE RAI-41-13-5

Rev 0



Layer B is from 160.5 to 297 ft Below Ground Surface (See SSAR Figure 2.5.4.7-8a)

Reference: EPR1 Curves from SSAR Reference 2.5.4.7-3.

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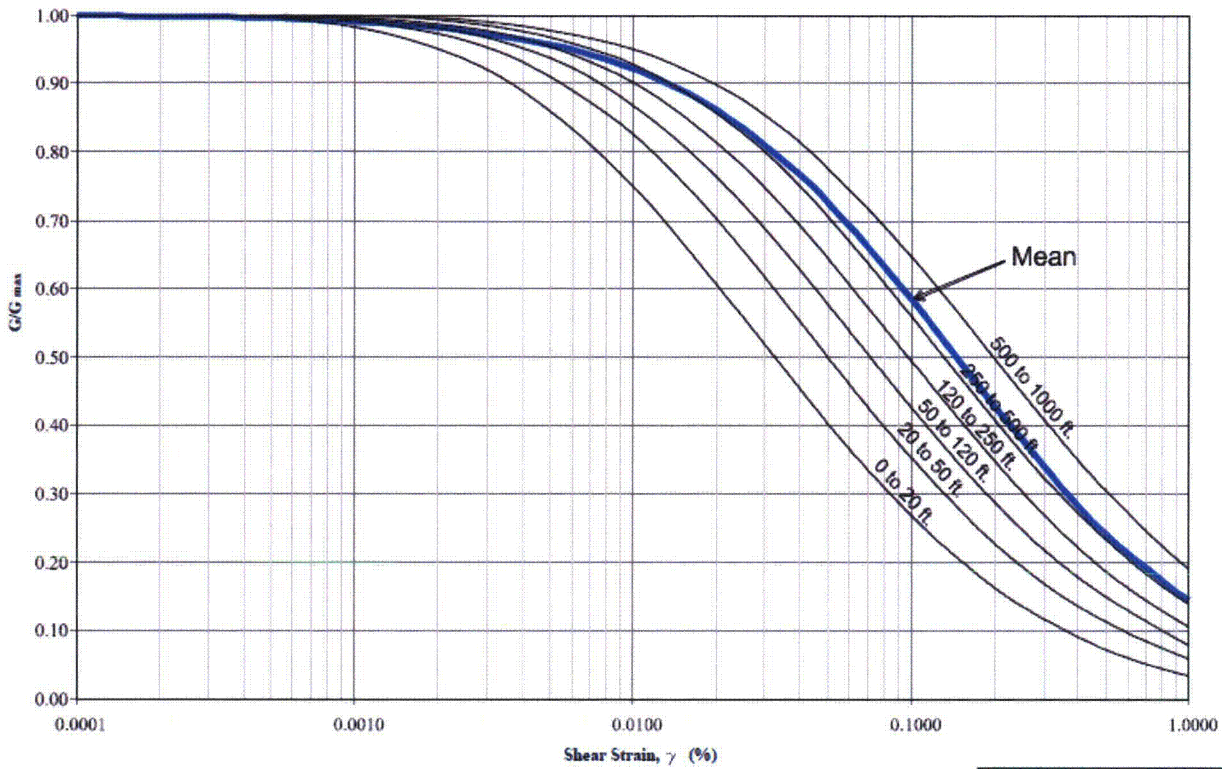
Part 2, Site Safety Analysis Report

Comparison Calculated Modulus Reduction Curves to EPRI Curves, Layer B

FIGURE RAI-41-13-6

Rev 0



Layer C is from 297 to 432 ft Below Ground Surface (See SSAR Figure 2.5.4.7-8a)

Reference: EPR1 Curves from SSAR Reference 2.5.4.7-3.

PSEG Power, LLC

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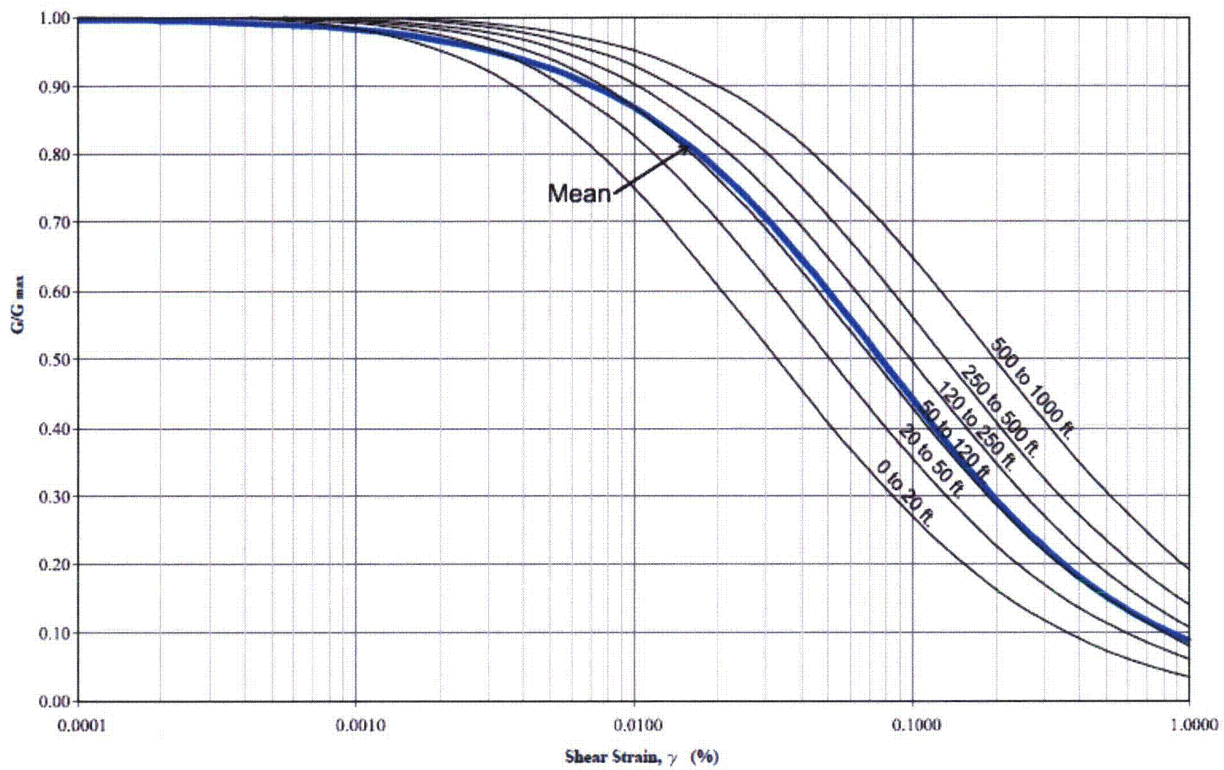
Part 2, Site Safety Analysis Report

Comparison Calculated Modulus Reduction Curves to EPRI Curves, Layer C

FIGURE RAI-41-13-7

Rev 0



Layer D is from 432 to 514 ft Below Ground Surface (See SSAR Figure 2.5.4.7-8a)

Reference: EPR1 Curves from SSAR Reference 2.5.4.7-3.

PSEG Power, LLC

PSEG SITE ESPA

Part 2, Site Safety Analysis Report

Comparison Calculated Modulus Reduction Curves to EPRI Curves, Layer D

FIGURE RAI-41-13-8

Rev 0

Response to RAI No. 41, Question 02.05.04-14:

In Reference 2, the specific request for Question 02.05.04-14 was:

> *SSAR Section 2.5.4.10.3, "Settlement Analysis," states that the Vincentown formation and below soils will deform elastically because of the sandy composition of soils and over-consolidated nature of clays. In compliance with 10 CFR 100.23 (d) (4) and in conformance to NUREG-0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," please provide additional information to support this statement, especially when the pre-consolidation pressures were not obtained from one dimensional consolidation tests for these clay type soils. Also, please clarify if drained elastic modulus values were calculated for clay type soils to assess long term conditions.*

PSEG Response to NRC RAI:

The Englishtown and Woodbury Formations are present between approximate elevations -292 ft and -372 ft under the PSEG Site based on boring NB-1. These formations are clay soils with some sand. The interpretation of these formations as being overconsolidated is drawn from the geologic history of the area.

The formations were deposited in a marine environment approximately 70 to 80 Million years ago (Ma). As discussed in the Salem Generating Station Updated Final Safety Analysis Report (UFSAR) (SSAR Reference 2.5.4.1-11) and the Hope Creek UFSAR (SSAR Reference 2.5.4.1-10), erosion removed materials above the Navesink Formation and later above the Vincentown Formation. Removal of weight by erosion produces an overconsolidated condition in clays.

SSAR Figure 2.5.1-3b illustrates the stratigraphic column for New Jersey and Pennsylvania and shows that two geologic formations are considered present between the top of the Navesink Formation and the base of the Vincentown Formation. These formations are not identified in the boring records for the Salem, Hope Creek or PSEG sites and were interpreted in SSAR Reference 2.5.4.1-11 as having been removed by erosion. Similarly, the upper surface of the Vincentown Formation exhibits effects of erosion as discussed in SSAR Subsection 2.5.4.1.2.3.2.

In addition to removal of material by erosion, sea levels have fluctuated up and down during the millions of years since the Englishtown and Woodbury Formations were deposited (Reference RAI-41-14-1). A lowering of sea level after the time of initial deposition also produces an overconsolidating effect because of the difference between buoyant unit weight and total unit weight.

The result of sea level changes and removal of materials by erosion produced vertical stresses that exceed the present day vertical stresses; thus the interpretation that the clays of the Englishtown and Woodbury Formations are overconsolidated.

The method of computing an elastic modulus based on shear wave velocity does not produce a "drained" or an "undrained" elastic modulus, only a material modulus based on shear deformations that would occur as a load is applied. Whether the elastic modulus is determined from an undrained or drained shear test, the value still represents a soil's immediate response to loading, not a long-term response. Based on the settlement calculations, the materials in the Englishtown and Woodbury Formations (Layer C in the settlement calculation) contribute only approximately 14 percent of the total settlement calculated using the elastic methods.

References:

RAI 41-14-1 Browning, James V., Kenneth G. Miller, Peter J. Sugarman, Michelle A. Kominz, Peter P. McLaughlin, Andrew A. Kulpecz and Mark D. Feigenson. "100 Myr record of sequences, sedimentary facies and sea level change from Ocean Drilling Program onshore coreholes, US Mid-Atlantic coastal plain", Basin Research, 20, pp 227-228, 2008.

Associated PSEG Site ESP Application Revisions:

None.

Response to RAI No. 41, Question 02.05.04-15:

In Reference 2, the specific request for Question 02.05.04-15 was:

> *SSAR Section 2.5.4.10.2 states that for the bearing capacity calculations, a friction angle of 37 degrees was selected based on N 60 values and a unit weight of 125 pounds per cubic foot (lbs/ft^3) was selected based on a weighted average of unit weights from the Vincentown, Hornerstown, Navesink and Mount Laurel formations. In compliance with 10 CFR 100.23 (d) (4) and in conformance to NUREG-0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," for the selection of the internal friction angle, please clarify why N 60 instead of (N1) 60 values were used, provide the correlation that was ultimately used, and compare these values with those obtained by triaxial testing. Regarding the unit weight, please justify selecting 125 pounds per cubic foot (lbs/ft^3) especially when the referenced values given in SSAR Table 2.5.4.2-9 were all below such number.*

PSEG Response to NRC RAI:

The $(N_1)_{60}$ values were used in selecting the internal friction angle as noted in the response to RAI No. 41, Question 02.05.04-10, part (b). The internal friction angle is used in calculating bearing capacity. The method for using the $(N_1)_{60}$ values was to first calculate an effective friction angle from the average $(N_1)_{60}$ value for the Vincentown plus Hornerstown Formations (combined), Navesink Formation and Mt. Laurel Formation by the following equation (Reference RAI-41-15-1):

> ϕ' = Square Root $(15.4(N_1)_{60})$ + 20, where:
> ϕ' is the effective friction angle, and
> $(N_1)_{60}$ is the N-value corrected for field procedures, energy and overburden pressures

The calculated values were compared to available laboratory test results, and a value selected for use in the calculations. Table RAI-41-15-1 shows the comparison of effective friction angles computed from the equation, the laboratory tests and the value selected for use in bearing capacity calculations.

With respect to the unit weight used in the bearing capacity calculation, values given in SSAR Table 2.5.4.2-8 for the Vincentown plus Hornerstown Formations (combined), Navesink Formation and Mt. Laurel Formation were used to calculate an average unit weight as follows:

The unit weight in SSAR Table 2.5.4.2-8 for each layer was multiplied by the thickness of the layer, the results summed and the total divided by the total thickness of the layers to provide a weighted average. Layer thicknesses from boring NB-1 were used. The operations are summarized in Table RAI-41-15-2. As shown in the table, the resulting weighted average unit weight is 126 pounds per cubic foot (pcf); this was rounded to 125 pounds pcf for use in calculations.

References:

RAI 41-15-1 Federal Highway Administration (FHWA), Geotechnical Engineering Circular No. 5, Evaluation of Soil and Rock Properties, p 184, April, 2002.

Associated PSEG Site ESP Application Revisions:

None.

Table RAI-41-15-1a
Effective Friction Angle Information

Formation Name	Col 2: $(N_1)_{60}$ *avg*	Col 3: Square root (15.4 times $(N_1)_{60}$)	Effective friction angle, ø' (degrees) Col 3 +20
Vincentown + Hornerstown	35	23.2	43.2
Navesink	45	26.3	46.3
Mt. Laurel	54	28.8	48.8

Table RAI-41-15-1b
Effective Friction Angle Comparison to Laboratory Tests

Formation Name	Value from Table RAI-41-15-1a (degrees)	Values from Lab testing (degrees) [a]	Value used in Bearing Capacity Calculation (degrees)
Vincentown + Hornerstown	43.2	37.8, 44.4, 30.1, 34.1	37
Navesink	45.7	No value	37
Mt Laurel	48.8	20.4 [b]	37

[a] values from SSAR Table 2.5.4.2-4
[b] test result included high value for effective cohesion as well.

Table RAI-41-15-2
Weighted Unit Weight Determination

Formation Name	Design Unit Weight (pounds per cubic foot)	Layer Thickness (ft)	Unit weight times layer thickness
Vincentown+ Hornerstown	118.5	59 [a]	6991.5
Navesink	123.6	24	2966.4
Mt Laurel	131	102	13362
Totals	N/A	185	23319.9
Weighted Average Unit Weight = 23319.9/185 = 126			

[a] Portions of Vincentown Formation above Elevation -67 ft which will be removed during construction are not included in layer thickness.

Response to RAI No. 41, Question 02.05.04-18:

In Reference 2, the specific request for Question 02.05.04-18 was:

> *SSAR Table 2.5.4.6-3 shows a summary of groundwater drawdowns at existing structures within the Vincentown Formation after one year of dewatering. Calculation Package PSEG 2251-ESP-GT-009-4, Figure 2251-ESP-GT-009-4, shows contour maps depicting these drawdowns overlaid onto a general layout plan of existing HCGS and SGS plants. In compliance with 10 CFR 100.23 (d) (4) and in conformance to NUREG-0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," please discuss the impact of different groundwater levels across the structure foundation on differential settlements and stability of existing HCGS and SGS safety related structures.*

PSEG Response to NRC RAI:

The estimated total future settlements, resulting from the planned dewatering activities, near the center of existing safety-related and non-safety related structures of the Hope Creek Generating Station (HCGS) and the Salem Generating Station (SGS) are presented in SSAR Subsection 2.5.4.6.3.1. The estimated future differential settlements across the safety-related structures, as a consequence of differences in dewatering drawdown, which vary with distance from the proposed excavation, are presented and discussed in this response.

The settlement estimates in SSAR Subsection 2.5.4.6.3.1 were determined in Calculation 2251-ESP-GT-009 for existing structures located within the calculated zone of influence from construction dewatering for the new plant excavation. Both safety and non-safety related structures are considered in the calculation. Settlements are based on drawdowns shown in SSAR Figures 2.5.4.6-3 and 2.5.4.6-4, using elastic methods as discussed in SSAR Subsections 2.5.4.6.3.1.1 and 2.5.4.10.3. For this response, only the safety related structures of the existing HCGS and SGS (the HCGS and SGS Nuclear Islands, and the HCGS and SGS Intake Structures) are considered. These structures have mat foundations bearing on granular fill or concrete fill that extends to the Vincentown Formation.

Because the calculation approach used elastic methods, and the soil properties of the Vincentown and underlying formations were considered to be the same under all structures, the settlement is proportional to the drawdown amount and varies linearly with drawdown. To estimate differential settlements across the structures, this proportionality was used. Drawdowns occur in both the Hydraulic Fill and the Vincentown Formations as shown on SSAR Table 2.5.4.6-3. Comparing the respective drawdowns, the drawdown in the Vincentown Formation accounts for 85 to 90 percent of the total drawdown. For purposes of this response, the drawdown in the Vincentown Formation was used in conjunction with the total settlement values for the approximate center of the HCGS and the SGS Nuclear Islands (presented in SSAR Subsection 2.5.4.6.3.1.1) to estimate differential settlements of these structures. Because the

HCGS and SGS Intake Structures have small areal dimensions compared to the scale of the drawdown, and because of their distance from the new plant excavation, the variation in drawdowns across these structures is too small to estimate; therefore, no differential settlements are reported for those structures.

The following approach was used to estimate the differential settlement.

1. The gradient of the drawdown was estimated from SSAR Figures 2.5.4.6-4.
2. The gradient was used to estimate the drawdown at the edges of the structures. Settlements at the approximate center of the structures from SSAR Subsection 2.5.4.6.3.1.1 were used with the drawdown gradient to estimate settlements at the edges of the structures.

Table RAI-41-18-1 shows estimated drawdowns at the edges of the HCGS and SGS Nuclear Islands in addition to those at the approximate center and the associated settlements. The differential settlement is taken as the difference between the maximum and minimum settlements.

As shown on Table RAI-41-18-1, future differential elastic settlements, resulting from differences in the magnitude of drawdown from the dewatering, under the HCGS and SGS safety-related structures are approximately 0.1 inch or less. Differential settlements of this magnitude are not anticipated to negatively impact the HCGS or SGS safety related structures.

Associated PSEG Site ESP Application Revisions:

None.

Table RAI-41-18-1

Summary of Drawdown and Vertical Settlement After One Year of Dewatering for Construction

Structure	Minimum Drawdown in Vincentown Formation (a) (ft)	Maximum Drawdown in Vincentown Formation (a) (ft)	Average Drawdown (Center) in Vincentown Formation (b) (ft)	Minimum Elastic Settlements of Vincentown Formation and Deeper Formations (c) (in)	Maximum Elastic Settlements of Vincentown Formation and Deeper Formations (c) (in)	Average Elastic Settlements (Center) of Vincentown Formation and Deeper Formations (d) (in)	Differential Settlement (in)
Hope Creek Nuclear Island	15.2	21.8	18.5	0.25	0.35	0.3	0.10
Salem Nuclear Island	6.0	9.0	7.5	0.08	0.12	0.1	0.04

(a) Minimum and maximum drawdown (edges of the structure) in the Vincentown Formation was determined using SSAR Figure 2.5.4.6-4 to estimate the drawdown gradient across the structure.

(b) Drawdown in the Vincentown Formation from SSAR Table 2.5.4.6-3.

(c) Settlements are elastic settlement for the Vincentown and lower formations. Settlements presented herein are based on linear proportionality of drawdown in the Vincentown formation and settlement due to use of elastic methods.

(d) Settlement due to drawdown in the Hydraulic Fill and Vincentown Formation shown on SSAR Table 2.5.4.6-3.

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ENCLOSURE 2

Proposed Revisions
Part 2 – Site Safety Analysis Report (SSAR)

Subsection 2.5.4 – Stability of Subsurface Materials and Foundations

Marked Up Pages

2.5-228
2.5-229

The Vincentown Formation was encountered in all of the borings performed for this ESPA investigation. The Vincentown Formation serves as the bearing stratum for the adjacent Salem and Hope Creek generating stations and will serve as the bearing stratum for the new plant. Based on borings performed for the ESPA, thickness of the Vincentown Formation ranges from 35 to 93 ft. The elevation of the top of the Vincentown Formation ranges from elevation -33 to -91 ft. NAVD in the borings performed for this ESPA. The Vincentown Formation consists primarily of a greenish-gray, fine to medium grained silty sand with some zones of clayey sand. The mineral glauconite, which imparts the greenish color, was observed in most samples. Previous studies indicate that glauconite typically comprises less than 10 percent of the sand fraction of the Vincentown Formation, but can vary up to 20 percent. Based on drilling characteristics and recovered samples, friable to indurated (cemented) zones of 0.1 to 3.0 ft. in thickness are present throughout this formation. Previous studies, including geologic mapping of the HCGS excavation, have described the indurated zones as calcareous sandstone and limestone. An upper weathered or possibly reworked zone was observed in some of the borings. Where encountered, this upper weathered zone generally exhibited a lower degree of induration and was reddish-brown in color (likely due to oxidation).

Replace with "Sheet 9 and 10 of 18" per Question 02.05.04-9.

The Hornerstown Formation was encountered in all of the borings [...] exploration. Based on the borings, the Hornerstown Formation ran[...] 22 ft. The Hornerstown Formation conformably underlies the Vincentown Formation and primarily consists of a greenish-gray to dark green silty and clayey, quartz and glauconitic sand with indurated zones, similar to the overlying Vincentown Formation. The contact between the Vincentown and Hornerstown formations was observed to be gradational. This contact was identified due to an increase in fines (silt and clay), and glauconite content.

Static laboratory indices were determined for 40 SPT samples and seven intact samples of the Vincentown and Hornerstown formations collected during the ESPA subsurface investigation (Table 2.5.4.2-2, ~~Sheet 6 and 7 of 15~~). Laboratory testing, including sieve analysis with hydrometer, sieve analysis (No. 200 wash), Atterberg limits, specific gravity and moisture content, were performed to determine the soil index properties of the Vincentown and Hornerstown formations.

Samples of the Vincentown and Hornerstown formations are generally classified as silty sands (SM, SP-SM) and, less commonly, clayey sand (SC, SC-SM), silt (ML, MH) and clay (CL). The moisture content of tested samples ranges from 9 to 40 percent, with an average of 30 percent, and a median value of 30 percent. The fine-grained component of the Vincentown and Hornerstown formations (silt and clay; minus 200 sieve) ranges from 9 to 96 percent, with an average of 27 percent, and a median value of 23 percent. A grain size distribution envelope developed from 40 grain size distribution curves performed for the ESP investigation is presented as Figure 2.5.4.2-1.

Nine of the 22 samples submitted for Atterberg limits tests indicate no value for the liquid limit, and non-plastic for the plastic limit. For the remaining samples, the liquid limit ranges from no value to 36. The average value of the liquid limit is 26, and the median value is 25. The plastic limit ranges from non-plastic to 27. The average and median value of the plastic limit are 20 and 19, respectively. The plasticity indices range from non-plastic to 12. The average and median plasticity indices are 6. The average and median value of the liquid limit test, plastic limit test, and the calculated plasticity indices are based on tests having values for the liquid limit and plastic limit. The specific gravity ranges from 2.61 to 2.75, with an average of 2.70, and a median value of 2.70. Soil index properties for the individual tests are shown in Table 2.5.4.2-2 (Sheet 6 and 7 of 15).

Soil index properties of the Vincentown Formation reported in the HCGS UFSAR were reviewed to determine if index properties determined in the ESPA are similar. Based on review of the HCGS UFSAR, the liquid limit ranges from 27 to 47, with an average of 36, and a median value of 35. The plasticity indices of the tested samples range from 6 to 20, with an average and median value of 11. The natural moisture of the tested samples ranges from 21 to 42 percent, with an average and median value of 30 percent. The specific gravity of the tested samples ranges from 2.60 to 2.73, with an average and median value of 2.68. Void ratios calculated for 100 percent saturation ranged from 0.55 to 1.06. Figure 2.5.4.2-1 shows the grain size envelope for the Vincentown and Hornerstown formations determined in the ESPA is consistent with the grain size envelope for the Vincentown Formation reported in the HCGS UFSAR. Based on review, soil index properties reported in the HCGS UFSAR are found to be comparable to soil index properties determi~~ned for the ESPA~~. Soil index values reported in the HCGS UFSAR are shown in Table 2.5.4.2-2 ~~(Sheet 8 of 15)~~. Design values for the soil index properties are presented in Table 2.5.4.~~2-8~~.

Representative intact samples of the Vincentown and Hornerstown strata collected during the ESPA subsurface investigation were submitted for strength and consolidation testing.

Three CU triaxial compression tests were performed on intact sa Hornerstown formations for the ESPA. Tests were performed on classification of SM. Results of CU tests indicate average shear and Φ = 20° for total stress, and c' = 0.40 tsf, and Φ' = 37° for eff properties for the individual tests performed for the ESPA are presented on Table 2.5.4.2-4.

Replace with "Sheet 11 of 18" per Question 02.05.04-9.

Shear strength properties determined for the ESPA were compared with CU tests performed for the HCGS UFSAR. CU test results from the HCGS UFSAR indicate shear strength values ranging from Φ = 23° to 37° for total stress, and Φ' = 31° to 43° for effective stress. The CU tests performed for the HCGS UFSAR were one-point tests with the cohesion intercepts, c and c' assumed to be 0. Comparison of the strength test results between the ESPA samples and the HCGS UFSAR is not made due to the difference in test methods. Design shear strength values for the Vincentown and Hornerstown formations determined from CU tests performed for the ESPA are presented in Table 2.5.4.2-8.

The total unit weight determined from 13 intact samples of the Vincentown and Hornerstown formations was calculated to range from 110.9 to 130.2 pcf. The unit weight was calculated from the dry density and moisture content determined from intact samples selected for strength and consolidation testing. Results of individual tests performed for the ESPA investigation are shown in Table 2.5.4.2-6. A summary of unit weights for the Vincentown and Hornerstown formations reported in the DMR is shown in Table 2.5.4.2-7. Based on review of the DMR, unit weights calculated for the ESPA are consistent with unit weights of the Vincentown and Hornerstown formations reported in the DMR. The design unit weight for the Vincentown and Hornerstown formations, based on test results from the ESPA exploration and the DMR, is presented in Table 2.5.4.2-8.

Field SPT N-values range from 5 to greater than 100 bpf. The average SPT N-value for this layer is 47 bpf. The median SPT N-value is 33. N-values greater than 100 bpf were treated as

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ENCLOSURE 3

Summary of Regulatory Commitments

ENCLOSURE 3

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 2.5.4 to incorporate the changes in Enclosure 2 in response to NRC RAI No. 41, Question No. 02.05.04-9.	This revision will be included in a future update of the PSEG ESP application.	Yes	No