Nuclear Development
244 Chestnut Street, Salem, NJ 08079

PSEG
Power LLC

ND-2012-0048
September 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. 64, Stability of Subsurface Materials and Foundations

References: 1) PSEG Power, LLC Letter No, ND-2012-0031 to USNRC, Submittal of Revision 1 of the Early Site Permit Application for the PSEG Site, dated May 21, 2012

2) RAI No. 64, SRP Section: 02.05.04 – Stability of Subsurface Materials and Foundations, dated August 8, 2012 (eRAI 6607)

The purpose of this letter is to provide a response to the request for additional information (RAI) provided in Reference 2 above. This RAI addresses the 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 1.

Enclosure 1 provides our response for RAI No. 64, Question Nos. 02.05.04-22 through 02.05.04-26. Our response to RAI No. 64, Question Nos. 02.05.04-23 and 02.05.04-25 will result in a revision to the SSAR. Enclosure 2 contains the proposed revisions of the SSAR. Enclosure 3 includes the new regulatory commitment established in this submittal.

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

D079
NRO

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

Sincerely,

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

Enclosure 1: Response to NRC Request for Additional Information, RAI No. 64, Question Nos. 02.05.04-22 through 02.05.04-26, 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 New Reactor Licensing (w/enclosures)
USNRC Region I, Regional Administrator (w/enclosures)

PSEG Letter ND-2012-0048, dated September 20, 2012

ENCLOSURE 1

Response to RAI No. 64

Question Nos.
02.05.04-22
02.05.04-23
02.05.04-24
02.05.04-25
02.05.04-26

Response to RAI No. 64, Question 02.05.04-22:

In Reference 2, the NRC staff asked PSEG for information regarding the Stability of Subsurface Materials and Foundations, as described in Subsection 2.5.4 of the Site Safety Analysis Report. The specific request for Question 02.05.04-22 was:

> ***Supplement to RAI 41, Question 02.05.04-7***
>
> *In response to RAI 41, Question 02.05.04-7, you explained how possible variations in the estimated Ko (ratio of vertical to horizontal stress) were accounted for by using multiple test confining pressures for RCTS Testing. Since RCTS tests results were not used to estimate modulus reduction and damping variation with shear strains, and in compliance with 10 CFR100.23(d)(4) and conformance to NUREG-0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," please explain how variations in the estimated Ko were accounted for when using Darandeli equations. Also, please justify using Ko=.5 shown in calculation package ESP811_PSEG_CALC_2251_ESP_GT_006_REV_2.*

PSEG Response to NRC RAI:

The Darendeli equations were run using a single value of Ko for all four layers. Ko is an input which affects the mean effective confining pressure parameter that is an input to the Darendeli equations (Reference RAI-64-22-1). A calculation was performed to explore the effect of different Ko values on the calculated modulus reduction and damping variation with shear strain. A Ko value of 0.5 is commonly assumed for normally consolidated soils. The subsurface soils are overconsolidated as discussed in the response to RAI No. 41, Question 02.05.04-14; however, the degree of overconsolidation is not known. Therefore, three values of overconsolidation ratio (OCR) - 2, 4 and 6 - were used to compute modulus reduction and damping variation with shear strain using the Darendeli equations for comparison with original values.

Figures RAI-64-22-1 through RAI-64-22-8 show the results of the calculation by comparing plots of the G/G_{max} and D variation with shear strain for each of the three OCR cases against the original values from calculation package ESP-811_PSEG_CALC_2251-ESP-GT-006, Rev_2. As can be seen on the figures, there is only a slight increase in the G/G_{max} and D values for the same shear strain values.

The response to RAI No. 64, Question 02.05.04-25, discusses the effect of the different G/G_{max} values on estimated settlements and concludes it is not significant.

References:

RAI-64-22-1 Darendeli, Mehmet B., (August, 2001), "Development of a New Family of Normalized Modulus Reduction and Material Damping Curves", Dissertation, The University of Texas at Austin.

Associated PSEG Site ESP Application Revisions:

None

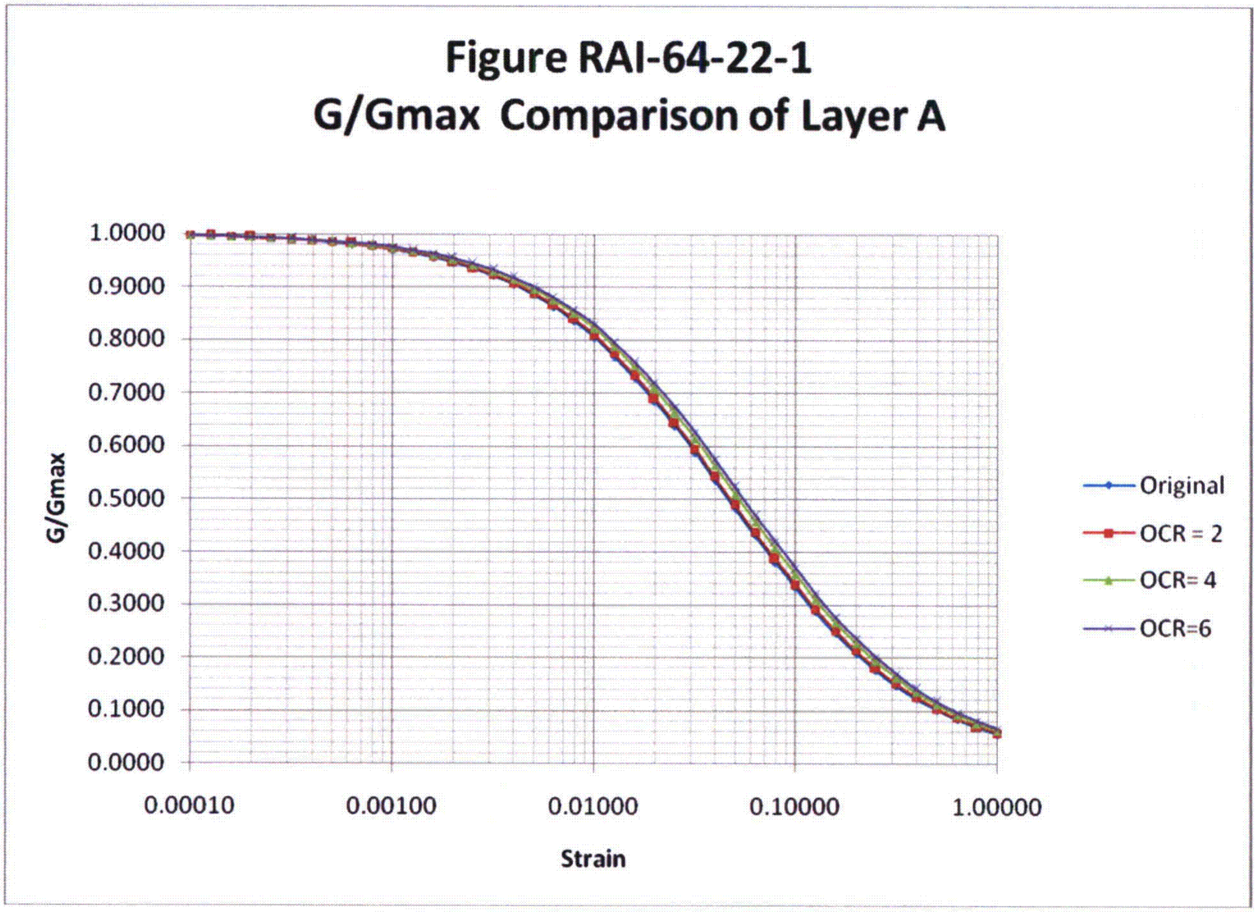


Figure RAI-64-22-1
G/Gmax Comparison of Layer A
1.0000
0.9000
0.8000
0.7000
0.6000
0.5000
0.4000
0.3000
0.2000
0.1000
0.0000
G/Gmax
0.00010
0.00100
0.01000
0.10000
1.00000
Strain
Original
OCR = 2
OCR= 4
OCR=6

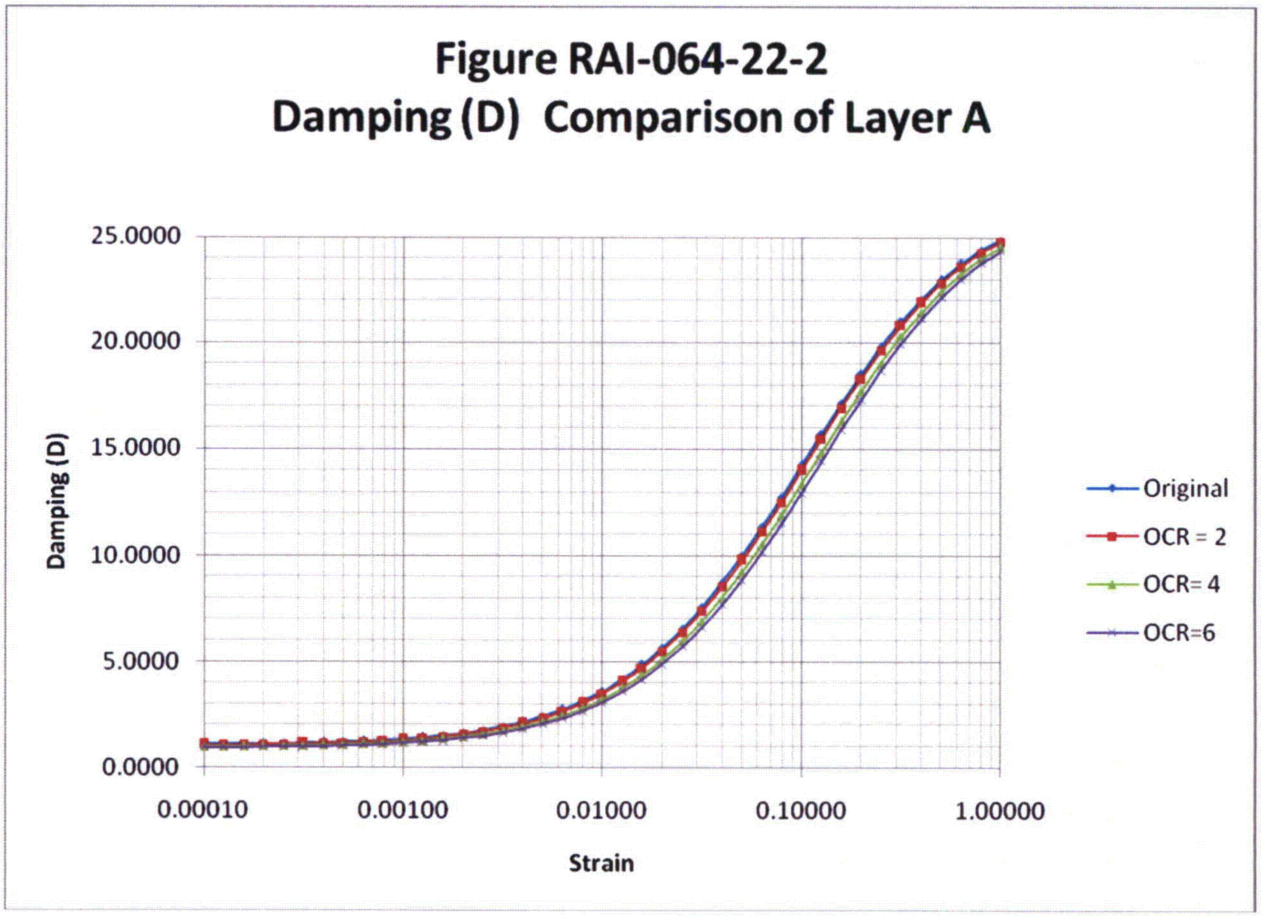


Figure RAI-064-22-2
Damping (D) Comparison of Layer A
25.0000
20.0000
15.0000
10.0000
5.0000
0.0000
Damping (D)
0.00010
0.00100
0.01000
0.10000
1.00000
Strain
Original
OCR = 2
OCR= 4
OCR=6

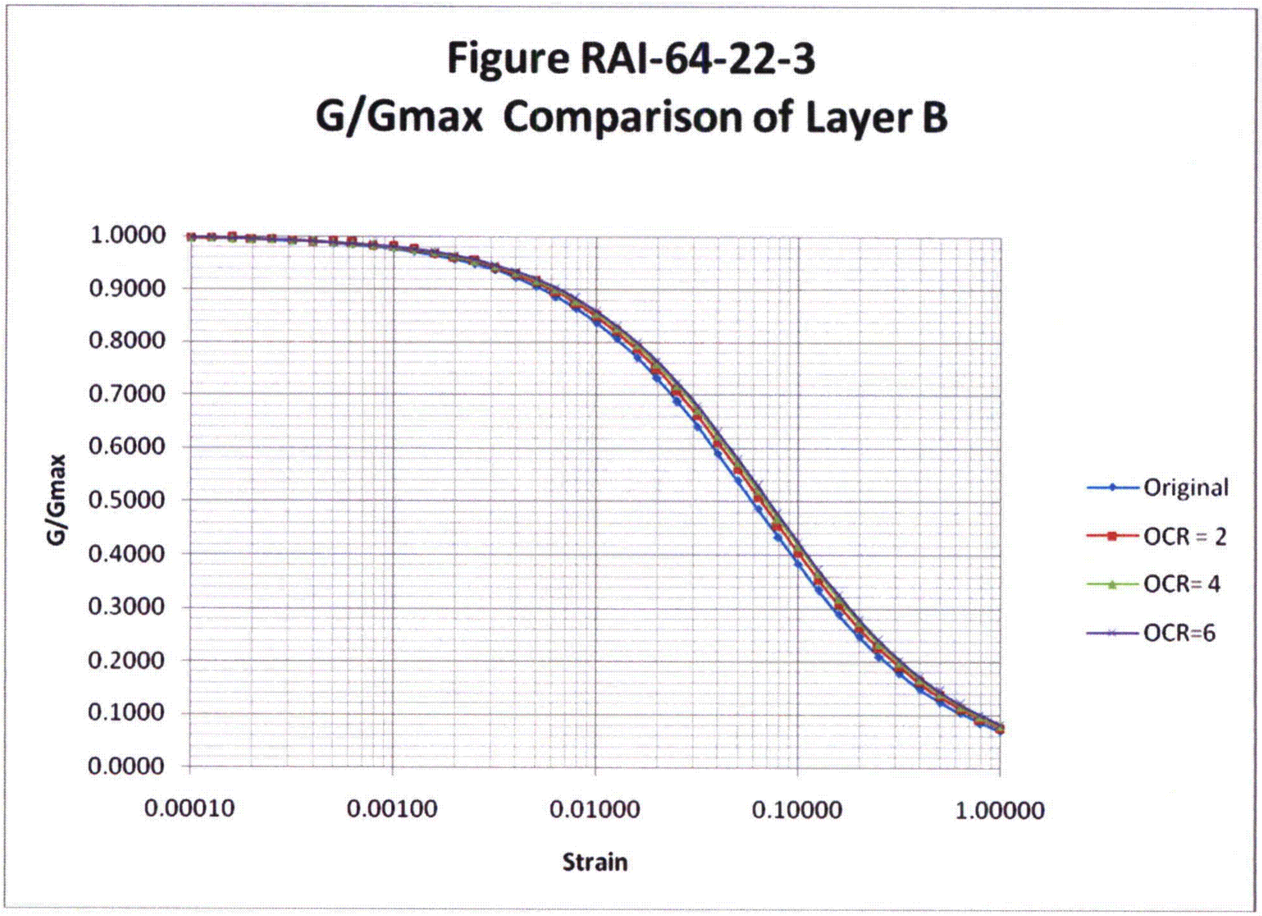


Figure RAI-64-22-3
G/Gmax Comparison of Layer B
1.0000
0.9000
0.8000
0.7000
0.6000
0.5000
0.4000
0.3000
0.2000
0.1000
0.0000
G/Gmax
0.00010
0.00100
0.01000
0.10000
1.00000
Strain
Original
OCR = 2
OCR= 4
OCR=6

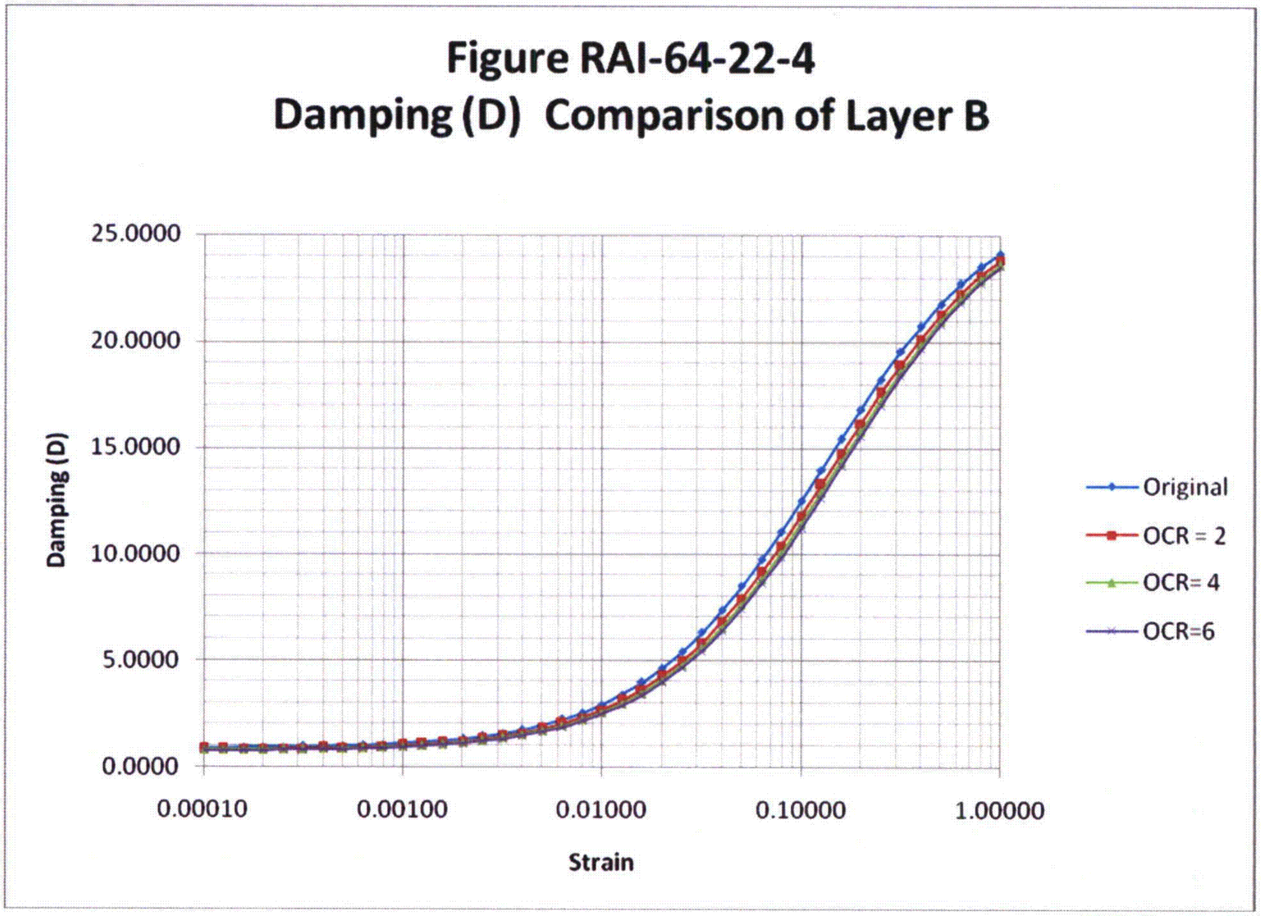


Figure RAI-64-22-4
Damping (D) Comparison of Layer B
25.0000
20.0000
15.0000
10.0000
5.0000
0.0000
Damping (D)
0.00010
0.00100
0.01000
0.10000
1.00000
Strain
Original
OCR = 2
OCR= 4
OCR=6

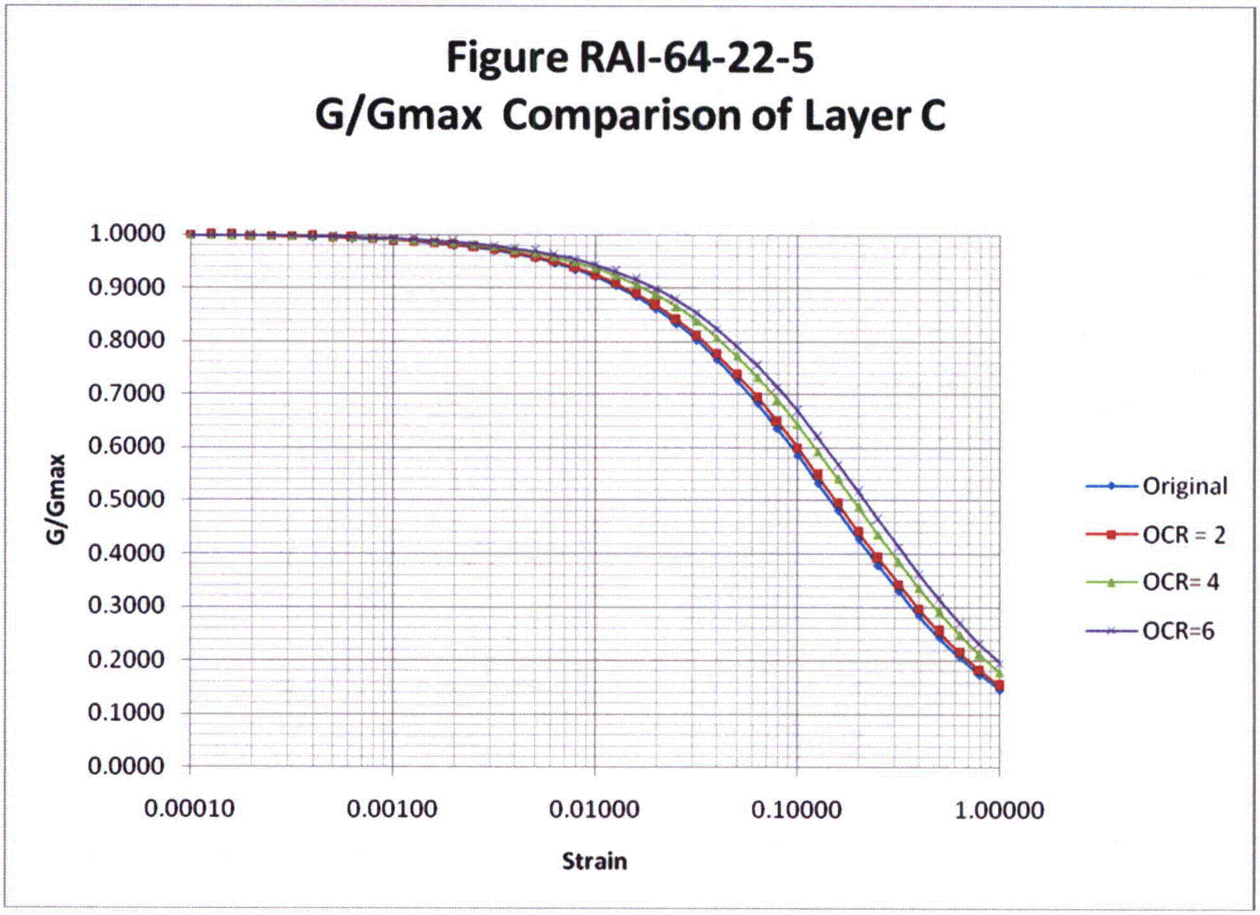


Figure RAI-64-22-5
G/Gmax Comparison of Layer C
1.0000
0.9000
0.8000
0.7000
0.6000
0.5000
0.4000
0.3000
0.2000
0.1000
0.0000
G/Gmax
0.00010
0.00100
0.01000
0.10000
1.00000
Strain
Original
OCR = 2
OCR= 4
OCR=6

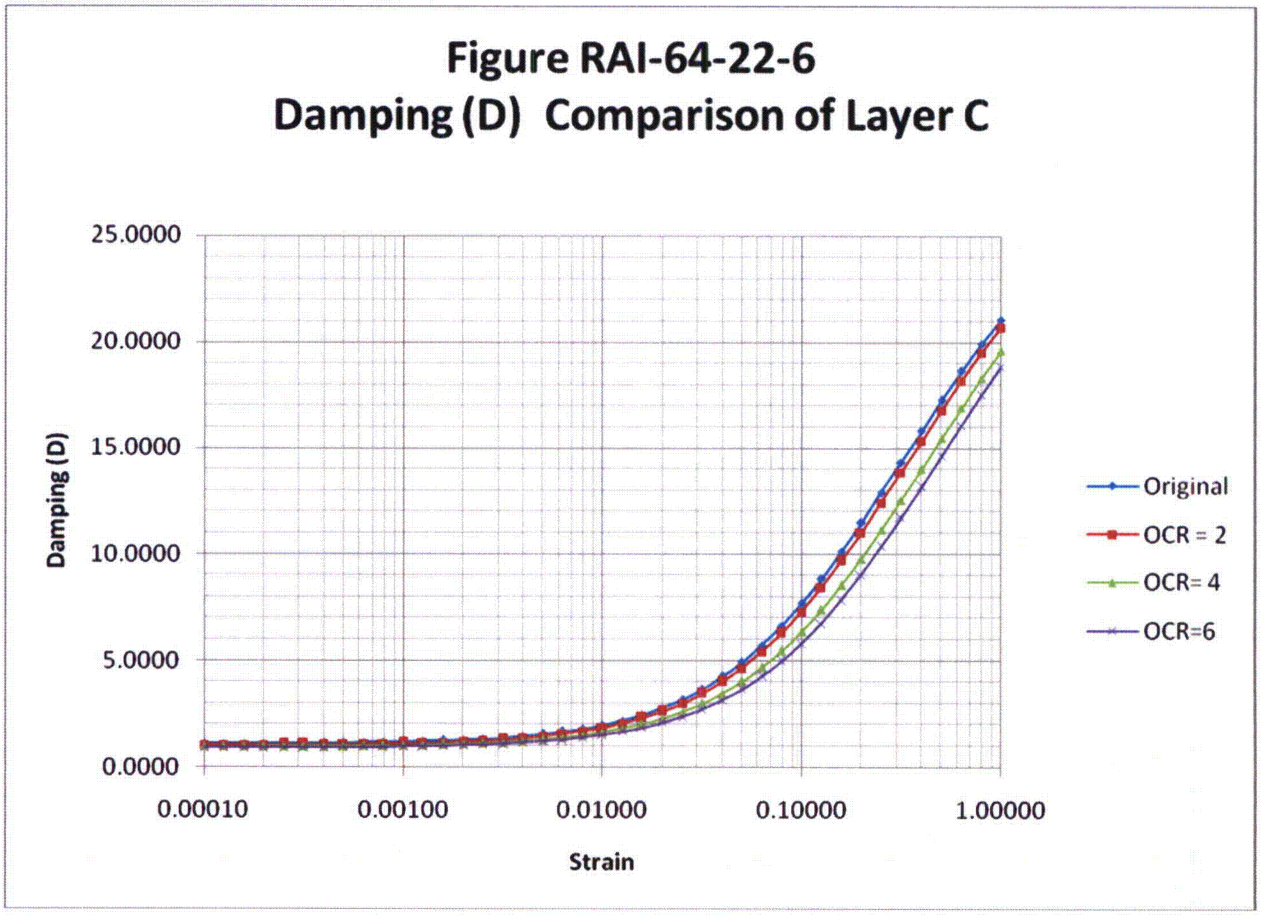
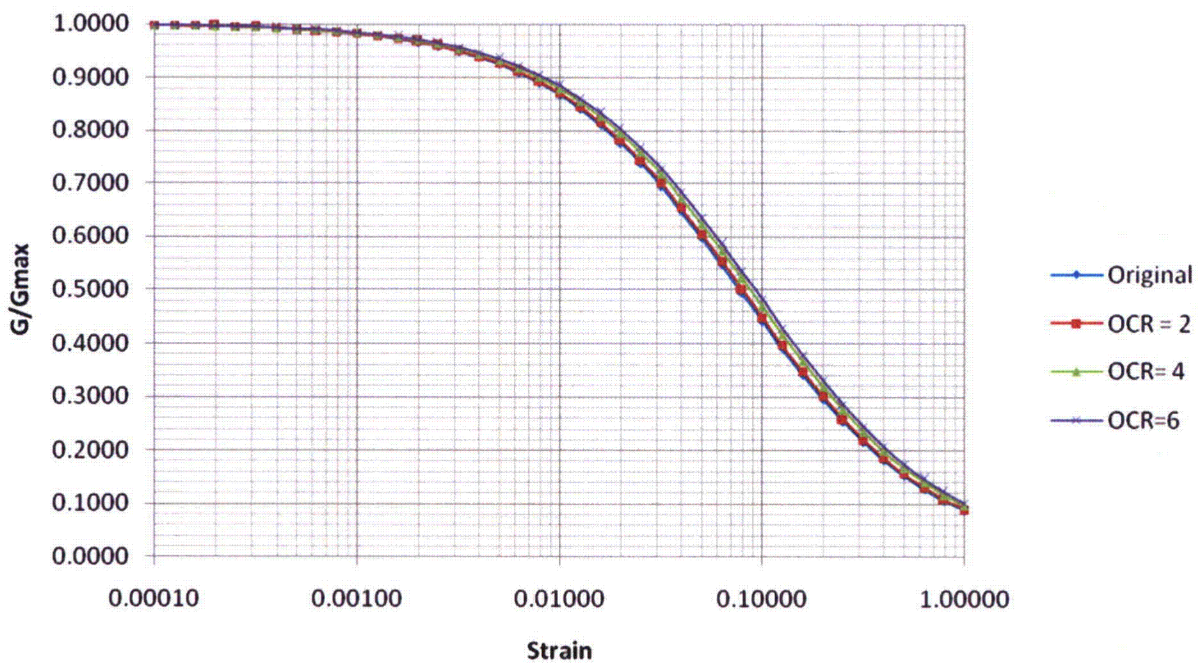


Figure RAI-64-22-6
Damping (D) Comparison of Layer C
25.0000
20.0000
15.0000
10.0000
5.0000
0.0000
Damping (D)
0.00010
0.00100
0.01000
0.10000
1.00000
Strain
Original
OCR = 2
OCR= 4
OCR=6

Figure RAI-64-22-7
G/Gmax Comparison of Layer D



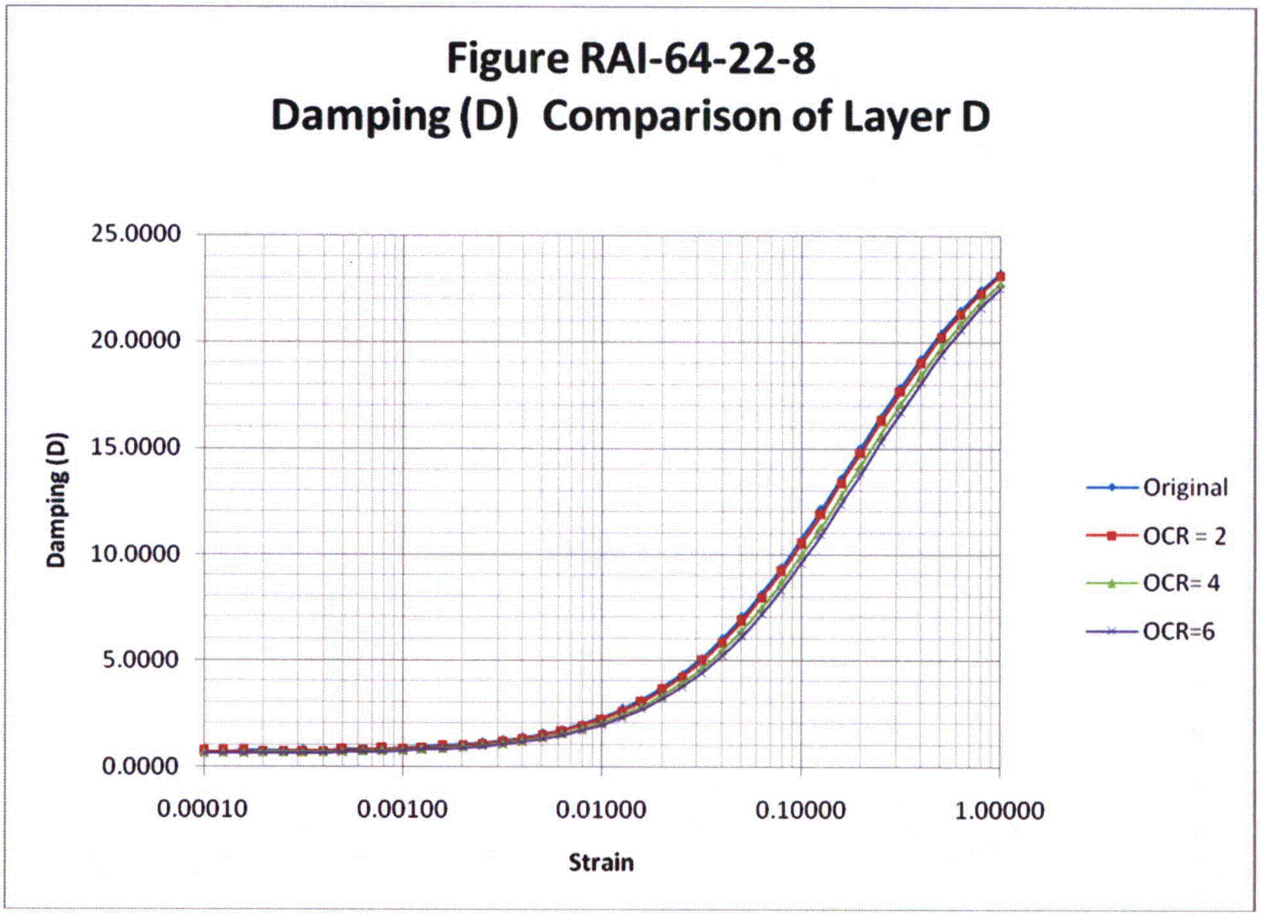


Figure RAI-64-22-8
Damping (D) Comparison of Layer D
25.0000
20.0000
15.0000
10.0000
5.0000
0.0000
Damping (D)
0.00010
0.00100
0.01000
0.10000
1.00000
Strain
Original
OCR = 2
OCR= 4
OCR=6

Response to RAI No. 64, Question 02.05.04-23:

In Reference 2, the NRC staff asked PSEG for information regarding the Stability of Subsurface Materials and Foundations, as described in Section 2.5.4 of the Site Safety Analysis Report. The specific request for Question 02.05.04-23 was:

> ***Supplement to RAI 41, Question 02.05.04-8***
>
> *In response to RAI 41, Question 2.5.4-8, you stated that you relied on SPT N-Values, USCS designation and Vs data to demonstrate that soils of the Vincentown and Hornerstown formations were similar laterally across the site and thus, will have similar soil engineering properties. Average field SPT N-Values for the Vincentown and Hornerstown formations were 37 bpf for NB-series borings and 57 bpf for EB-Series borings. In compliance with 10 CFR 100.23(d)(4) and conformance to NUREG-0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," please explain how these formations were considered to be laterally uniform when considerable variations in average SPT N-Values exist between NB and EB borings. Also, please explain why a design value of 47 bpf was used for the Vincentown and Hornerstown formations in SSAR Table 2.5.4.2.8 and justify how the selected single value statistically reflects the entire layer.*

PSEG Response to NRC RAI:

As concluded in the response to RAI No. 41, Question 02.05.04-8, the USCS designation of soils tested in the Vincentown and Hornerstown formations, shear wave velocities measured on materials of the Vincentown and Hornerstown formations and field N-values of the samples from the Vincentown and Hornerstown formations all indicate that materials of these formations are similar across the site and would have similar engineering properties. Both the SSAR and the response to RAI No. 41, Question 02.05.05-8, describe and discuss the formations in terms of similarities and do not present the formations as being laterally uniform, (having identical properties), but in terms of being laterally consistent. With respect to the field N-values, the average from the NB-series borings, while lower than the average from the EB-series borings, still indicates a dense sand.

The average SPT field N-values in the NB-series and EB-series borings are 37 and 57 blows per foot (bpf), respectively. The average field N-values suggest that the relative densities of soils of the Vincentown and Hornerstown formations in the area of the EB-series borings are slightly greater than soils in the area of the NB-series borings. Both of these average field N-values indicate that soils in the EB-series and NB-series borings are dense and would behave similarly. The field N-values recorded in the Vincentown and Hornerstown formations in the EB-series and NB-series borings were used in determining the design field SPT N-value of 47 bpf shown on SSAR Table 2.5.4.2-8. The design field SPT N-value of 47 bpf would provide similar, but slightly higher soil shear strength properties, than the average field SPT N-value of 37 bpf determined in the NB-series borings.

Figure RAI-64-23-1 illustrates the vertical variation of field N-values within the Vincentown and Hornerstown formations, grouped by EB-series and NB-series borings. In SSAR Subsection 2.5.4.5, the top of a competent layer was defined at elevation -67 feet (NAVD 88), near the upper surface of the Vincentown Formation. Figure RAI-64-23-1 shows the position of the competent layer. The plot on Figure RAI-64-23-1 shows loose to medium dense soils in the Vincentown Formation above the competent layer, with average field SPT N-values ranging from approximately 5 to 25 bpf. Materials above the top of the competent layer are to be excavated during construction of the power block. Figure RAI-64-23-1 shows that the average field SPT N-values below the top of the competent layer generally increase with depth in the NB-series and the EB-series borings. The average field SPT N-values recorded in the Vincentown and Hornerstown formations beneath the top of the competent layer have medium dense to very dense relative densities with average field SPT N-values ranging from approximately 25 to 60 bpf for the NB-series borings, and average field SPT N-values ranging from approximately 25 to 85 bpf for the EB-series borings.

The PSEG Site is the area explored by the NB-series borings. Based on the lower average field SPT N-value of 37 bpf determined in the NB-series borings and the planned location of the plant, the design field SPT N-value shown on SSAR Table 2.5.4.2-8 will be revised to reflect only the NB-series borings. The design field SPT N-value shown in SSAR Table 2.5.4.2-8 for the Vincentown and Hornerstown formations will be revised from 47 to 37 bpf. The design corrected SPT N_{60} value will be revised from 70 to 56 bpf and the design corrected SPT $(N_1)_{60}$ value will be revised from 35 to 32 bpf.

The design SPT $(N_1)_{60}$ value is only used in estimating a soil friction angle for use in bearing capacity calculations. In the response to RAI 41, Question 02.05.04-15, the use of an empirical equation to determine the effective internal friction angle of a soil was discussed. That equation uses the normalized SPT resistance $(N_1)_{60}$ value. The design effective friction angle reported on SSAR Table 2.5.4.2-8 (37 degrees) is less than the value calculated by the empirical equation using either the previous design $(N_1)_{60}$ value of 35 bpf or the revised design $(N_1)_{60}$ value of 32 bpf, so no change in the design effective friction angle value is necessary. In addition, the minimum $(N_1)_{60}$ value of 26 bpf results in an estimated effective friction angle of 40 degrees, which exceeds the design effective friction angle reported on SSAR Table 2.5.4.2-8.

The SPT $(N_1)_{60}$ value is used in evaluating potential for soil liquefaction; however, the individual SPT $(N_1)_{60}$ values from the samples in each boring are used in this calculation, not an average or design SPT $(N_1)_{60}$ value. Thus, modifying an average or design value does not affect the calculations for potential liquefaction.

Based on the average field SPT N-value comparison to elevation, the revised field SPT N-value of 37 bpf is reasonable for the entire Vincentown and Hornerstown layer for purposes of the ESP Application. Additional subsurface information is obtained during the COLA phase of the project and is targeted to the specific technology planned and the locations of the safety-related structures. The additional exploration will obtain more SPT data. Thus, in the COLA phase, the distribution of N-values laterally and vertically within the Vincentown and Hornerstown formations can be further evaluated to more fully characterize the engineering properties of the Vincentown and Hornerstown formations and their potential variations laterally and vertically.

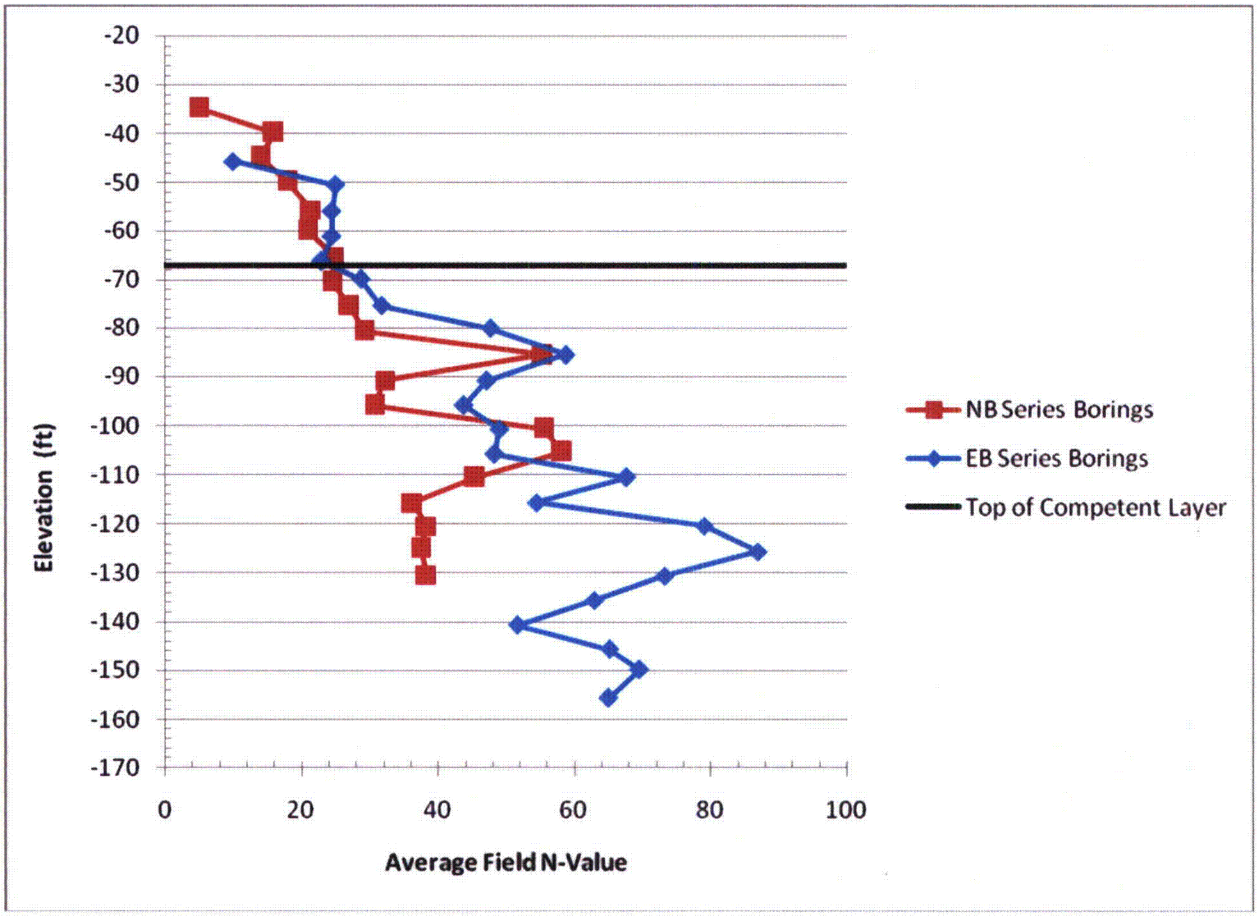


Figure RAI-64-23-1-1. Average Field N-values for Vincentown and Hornerstown Formations vs. Elevation.

Associated PSEG Site ESP Application Revisions:

SSAR Table 2.5.4.2-8 will be revised as shown in Enclosure 2.

Response to RAI No. 64, Question 02.05.04-24:

In Reference 2, the NRC staff asked PSEG for information regarding the Stability of Subsurface Materials and Foundations, as described in Section 2.5.4 of the Site Safety Analysis Report. The specific request for Question 02.05.04-24 was:

> ***Supplement to RAI 41, Question 02.05.04-11***
>
> *As part of the response to RAI 41, Question 2.5.4-11, you explained how soil strength properties for the Navesink formation were obtained. You mentioned that this formation's friction angle was obtained based on an empirical correlation included in the FHWA's geotechnical manual, which uses SPT N-Values as the main input. In compliance with 10 CFR 100.23(d)(4) and conformance to NUREG- 0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," please clarify if the equation used was the one referenced as part of the response to RAI 41, Question 2.5.4-15. Also, please explain why a design value was not included in SSAR Table 2.5.4.2-8 and justify the adequacy of the friction angle given the absence of lab testing and the sole reliance on empirical correlations.*

PSEG Response to NRC RAI:

As described in the response to RAI No. 41, Question 02.05.04-15, and as shown on Table RAI-41-15-1a, the empirical correlation provided in the FHWA's geotechnical manual (Reference RAI-64-24-1) was used to determine the friction angle of the Navesink Formation. A value of 46.3 degrees was calculated. For design use, as shown in Table RAI-41-15-1b, a conservative value of 37 degrees was used in bearing capacity calculations.

Only strength properties determined from laboratory shear strength tests were reported on SSAR Table 2.5.4.2-8. Therefore, values for design drained and undrained friction angle of the Navesink Formation were not included.

It is common engineering practice to evaluate the effective stress or drained friction angle of granular materials from in-situ penetration tests via a correlation to a measured test parameter such as an SPT Value (Reference RAI-64-24-1). The soils of the Navesink Formation are primarily granular and have low fines content. In addition, the friction angle calculated from the empirical formula was conservatively reduced. Therefore, the design effective friction angle for the Navesink Formation is conservative and adequate for the ESP Application.

Additional borings and tests are performed during the COLA phase that provide information for further evaluation of shear strength properties of the Navesink Formation soils. Soil shear strength properties determined in the ESP phase will be modified during the COLA phase, if appropriate.

References:

RAI-64-24-1 Federal Highway Administration (FHWA) (2002). "Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5", FHWA Report No. FHWA-IF-02-034, p. 184.

Associated PSEG Site ESP Application Revisions:

None.

Response to RAI No. 64, Question 02.05.04-25:

In Reference 2, the NRC staff asked PSEG for information regarding the Stability of Subsurface Materials and Foundations, as described in Section 2.5.4 of the Site Safety Analysis Report. The specific request for Question 02.05.04-25 was:

> ***Supplement to RAI 41, Question 02.05.04-13***
>
> *As part of the response to RAI 41, Question 2.5.4-13, you justified using Darendeli's equations to characterize PSEG's site dynamic properties. You mentioned that these equations were applicable to the PSEG site given the similarities between the PSEG and Savannah site soils. You also mentioned that site settlement estimates were based on the elastic modulus derived using such equations. In compliance with 10 CFR 100.23(d)(4) and conformance to NUREG-0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," please:*
>
> *i. Provide additional details on the similarities between the PSEG and Savannah soils in order to justify the validity of Darendeli equations to be used to characterize the PSEG site soils.*
> *ii. Explain how the use of these curves was considered appropriate and conservative to estimate site specific settlements. Also, justify using dynamic instead of static properties for this analysis.*

PSEG Response to NRC RAI:

1. The language in the response to RAI No. 41, Question 2.5.4-13, was not intended to mean that use of the Darendeli equations was contingent upon soils at the PSEG Site being "similar" to those at the Savannah River site. Rather, the intent was to show that when the Darendeli equations were used to compute modulus reduction and material damping curves for the Savannah River site soil profile, the results compared favorably with results based on RCTS testing at that site.

   Darendeli's work (Reference RAI-64-25-1) was based on results from 123 RCTS tests performed on 110 soil samples obtained from sites in Northern California, Southern California, South Carolina, and Lotung, Taiwan. Two sites in South Carolina were included, one of which was the Savannah River site. Eleven samples from the Savannah River site were included in the test data base used by Darendeli. The results of Darendeli's work are presented as equations with multiple parameters. Most of the parameters used in the Darendeli equations are mean values of model parameters determined from the research and are not dependent on soil type or properties. The soil-specific parameters are mean confining pressure, Plasticity Index (PI), and Over-Consolidation Ratio (OCR). Of these parameters, Darendeli found that mean confining pressure and PI have more effect than OCR.

Based on the large data base of test results on varying soil types analyzed by Darendeli, the use of his equations and model parameters for estimating normalized modulus reduction and material damping curves that an RCTS test performed on the same materials would produce is considered valid.

2. Using the shear wave velocity (Vs) to estimate the elastic modulus of soil is a well-known method (e.g., Reference RAI-64-25-2, p 150 - 151). The modulus that is estimated from Vs is a low-strain modulus and must be adjusted for shear strain effects that are consistent with settlement. The referenced method presents an equation to estimate the reduction factor which is referred to in the reference as E/E_0 based on the ratio of applied bearing pressure to ultimate bearing pressure:

   $$E/E_o = 1 - (q/q_{ult})^{0.3} \text{ where:}$$

   E = reduced modulus for higher shear strain,
   E_o = modulus at low shear strain,
   q = applied bearing pressure, and
   q_{ult} = ultimate bearing capacity.

   Note that because the value E/E_o is a ratio and because E and E_o are both related to shear modulus, G and the low-strain shear modulus, G_{max} by the same factor, the ratio E/E_0 is the same for G/G_{max}

Using an applied bearing pressure of 15,000 pounds per square foot (psf), which is typical for several reactor technologies, and the calculated ultimate bearing capacity of 420,000 psf in SSAR Subsection 2.5.4.10, the calculated E/E_o value is 0.63. SSAR Subsection 2.5.4.10 listed values for the G/G_{max} reduction factor of 0.4 for materials above elevation -300 ft. (NAVD88) and 0.5 for materials below that elevation. Use of a higher reduction factor results in an increase in the modulus value used for calculating settlement. Because the settlement is inversely related to modulus, a higher modulus produces less calculated settlement. Thus, the SSAR values of settlement are conservative with respect to the FHWA methodology.

Comparing the SSAR reduction factor values given in SSAR Subsection 2.5.4.10.3 to the modulus reduction curves presented in SSAR Figures 2.5.4.7-21, 2.5.4.7-23, 2.5.4.7-25, and 2.5.4.7-27, values at a shear strain of 10^{-3} (or 0.1%) shear strain are found to vary between 0.33 and 0.59 for the layers used in the dynamic analysis. The values in SSAR Figure 2.5.4.7-27 were applied also to Layer E for the settlement calculation. A comparison of estimated settlement made using the values from the SSAR figures for the appropriate layers in the settlement calculation shows an increase of approximately 10 percent, an amount considered suitable for consideration in an ESPA.

In the response to RAI No. 64, Question 02.05.04-22, variations in K_o are discussed with respect to the G/G_{max} curves. The results from that work showed increases in the G/G_{max} reduction factor at a strain level of 10^{-3}. Increases in the reduction factor, as discussed above, lead to lower calculated settlement. Thus, the settlements reported in SSAR Subsection 2.5.4.3 are conservative with respect to variations in K_o.

The low-strain G_{max} (and by extension, E_o) values do represent dynamic conditions. The reduced values used for higher shear strains represent static conditions. Thus, the settlement calculations did use a static modulus value.

Settlements presented in the ESPA are subject to modification in the COLA after the specific technology is selected and the excavation backfill material properties are known.

References:

RAI-64-25-1 Darendeli, Mehmet B., (August,2001), "Development of a New Family of Normalized Modulus Reduction and Material Damping Curves", Dissertation, The University of Texas at Austin.

RAI-64-25-2 Federal Highway Administration (FHWA) (2002), "Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5", FHWA Report No. FHWA- IF-02-034, p. 184.

Associated PSEG Site ESP Application Revisions:

Review of SSAR Subsection 2.5.4.10.3 found a typographical error in the statement of shear strain applicable for use in settlement. The value was stated as 10^{-3} percent, and it should be 10^{-3}. SSAR Subsection 2.5.4.10.3 will be revised as shown in Enclosure 2.

Response to RAI No. 64, Question 02.05.04-26:

In Reference 2, the NRC staff asked PSEG for information regarding the Stability of Subsurface Materials and Foundations, as described in Section 2.5.4 of the Site Safety Analysis Report. The specific request for Question 02.05.04-26 was:

> ***Supplement to RAI 41, Question 02.05.04-14***
>
> *As part of the response to RAI 41, Question 2.5.4-14, you stated that you relied on the area's geologic history (erosion and sea level changes) to justify describing site soils as overconsolidated. Laboratory testing was not performed to obtain consolidation data. In compliance with 10 CFR 100.23(d)(4) and conformance to NUREG-0800, Standard Review Plan, Section 2.5.4, "Stability of Subsurface Materials and Foundations," please provide the following details to further justify your conclusion regarding the soil's behavior:*
>
> 1. *Please indicate if laboratory tests on site subsurface soils will be performed to assess consolidation properties during the COLA phase.*
> 2. *Explain why the information from ESP subsurface investigations was not used to assess consolidation properties (e.g. Atterberg limits) to support your conclusions.*

PSEG Response to NRC RAI:

1. During the COLA exploration, additional borings will be drilled to meet the requirements of RG 1.132 with respect to spacing and depth. Intact samples will be obtained by appropriate methods, as allowed by the characteristics of the soils. Laboratory testing, conducted in accordance with RG 1.138, will include consolidation testing for materials having a high percentage of fine-grained particles (i.e., silt and clay).

2. Figure RAI-64-26-1 shows a chart from the United Facilities Criteria, Soil Mechanics (UFC-3-220-10N) (Reference RAI-64-26-1); that relates the Liquidity Index (a parameter derived from Atterberg limits tests) and preconsolidation pressure for varying degrees of sensitivity. The preconsolidation pressure is the highest vertical pressure to which a soil layer has been subjected in the past. Considering that the geologic age of the Englishtown, Woodbury, and Merchantville formations (those formations with clayey zones), is more than 72 million years, it would be expected that the preconsolidation pressures would at least be equal to the present overburden pressures. The present effective overburden pressures include the weight of materials placed to form Artificial Island in the area of the PSEG Site between approximately 1900 and 1990. Due to the elapsed time since Artificial Island was formed, excess pore pressures from its formation would be fully dissipated.

Table RAI-64-26-1 summarizes the Liquidity Indices from the seven Atterberg limits tests that were performed on samples from the Englishtown, Woodbury, and Merchantville formations as part of the ESPA exploration, the estimated existing overburden effective pressures based on site conditions and values of preconsolidation pressures derived from Figure RAI-64-26-1. As can be seen, the existing effective overburden pressures are inconsistent (but typically greater) with respect to the preconsolidation pressures estimated from Figure RAI-64-26-1. These results were interpreted during the ESP work as indicating use of Atterberg limits tests to further assess consolidation properties was not likely to be reliable for the soils in the Englishtown, Woodbury, and Merchantville formations.

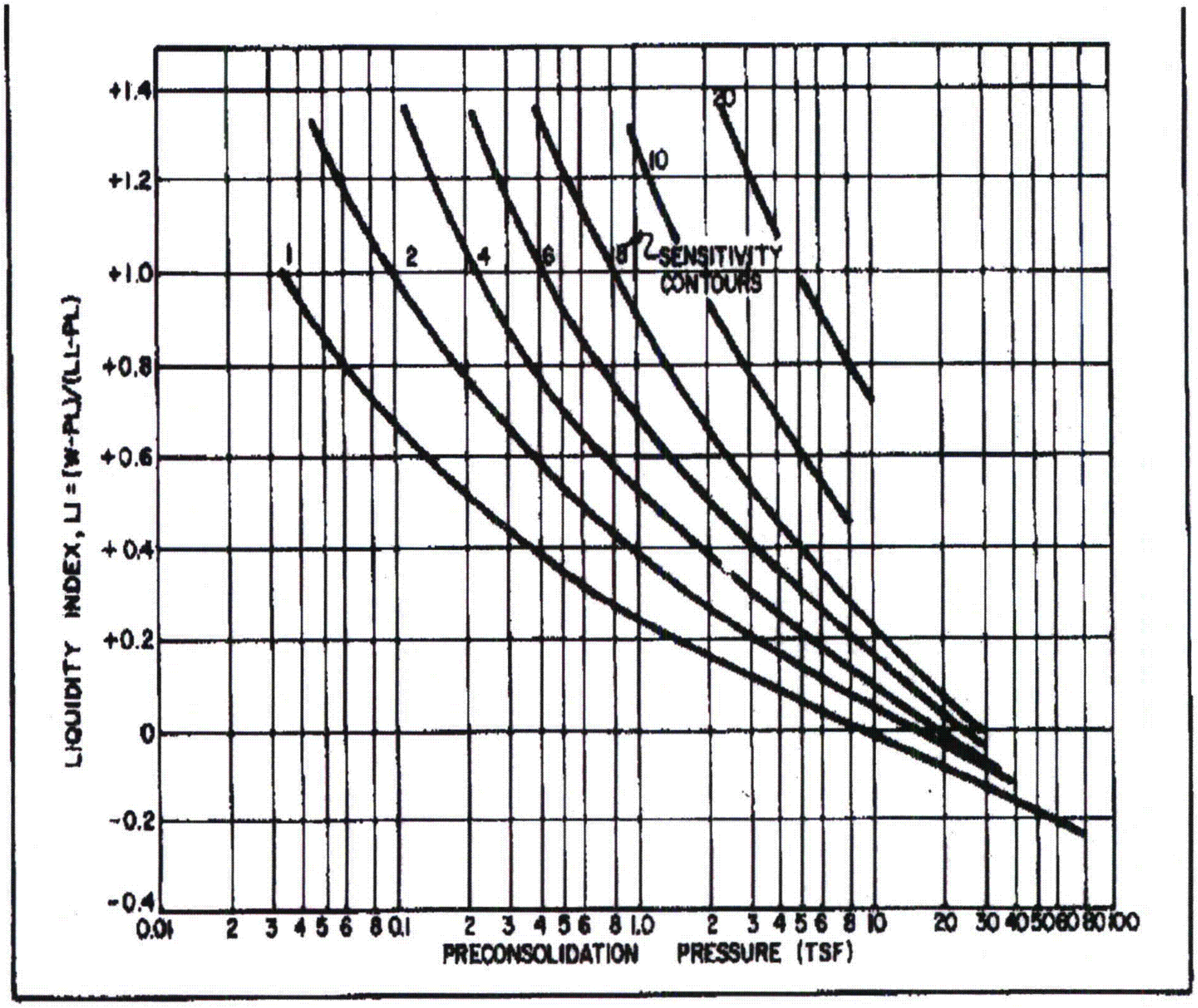


FIGURE 3
Preconsolidation Pressure vs. Liquidity Index

Figure RAI-64-26-1. Preconsolidation Pressure vs. Liquidity Index

Table RAI-64-26-1. Comparison of Liquidity Index Values and Estimated Consolidation Pressures

Sample	Sample Depth, ft	Formation	Moisture Content, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index, %	Liquidity Index	Estimated Preconsolidation Pressure from Figure RAI-64-26-1, tons per square foot (TSF)(1)					Estimated Vertical Effective In-situ Pressure at sample depth, (TSF)
								S =1	S = 2	S = 4	S = 8	S = 10	
NB-1, SS-55	311	Englishtown	25	36	16	20	0.45	0.3	0.7	1.5	4.0	8.0	9.6
NB-1, SS-57	331	Englishtown	28	51	20	31	0.26	0.9	2.1	4.0	8.5	N/A	10.3
EB-3, SS-59	351	Englishtown	28	32	20	12	0.67	0.1	0.3	0.6	2.0	4.0	11.1
NB-1, SS-62	381	Woodbury	31	73	20	53	0.21	1.4	2.9	5.0	10.0	N/A	11.8
EB-3, SS-64	400	Woodbury	30	75	21	54	0.17	1.9	3.9	6.0	13.0	N/A	12.5
NB-1, SS-65	411	Merchantville	31	43	21	22	0.45	0.3	0.7	1.5	4.0	8.0	12.8
EB-3, SS-68	441	Merchantville	25	36	18	18	0.39	0.4	1.0	2.0	5.1	N/A	12.9

(1) S = Soil sensitivity; N/A indicates no values in source figure

References:

RAI-64-26-1 United States Department of Defense, June 8, 2005, "Unified Facilities Criteria (UFC) – Soil Mechanics", UFC 3-220-10N, p 7.1-142.

Associated PSEG Site ESP Application Revisions:

None.

PSEG Letter ND-2012-0048, dated September 20, 2012

ENCLOSURE 2

Proposed Revisions

Part 2 – Site Safety Analysis Report (SSAR)
Subsection 2.5.4 – Stability of Subsurface Materials and Foundations

<u>Marked-up Pages</u>
2.5-276
2.5-337

Table 2.5.4.2-8 (Sheet 2 of 4)
Design Values for Static Engineering Properties of Subsurface Materials[f]

Parameter	Formation: Vincentown and Hornerstown	Formation: Navesink	Formation: Mount Laurel	Formation: Wenonah and Marshalltown
Range of Thickness, feet	51 to 112	19 to 26	102 to 112	18 to 40
Average Thickness, feet	72.4	22.3	107.1	35.4
Range of Top Elevation, feet NAVD [c]	-70 to -33	-133 to -121	-157 to -145	-259 to -250
Average Top Elevation, feet NAVD	-57	-127	-151.5	-256
USCS Symbol	SP-SM,SC-SM,SM,SC,MH,ML	SC-SM/SM/SC	SC-SM/SM/SC	SM/SC/CL
Natural Moisture, %	30	22	20	23
Unit Weight, (pcf)	118.5	123.6	131.0	125
Liquid Limit, (LL)	26	27	27	29
Plastic Limit, (PL)	20	16	18	15
Plasticity Index (PI)	6	11	9	14
Field SPT N-value, bpf[d]	~~47~~ 37	72	91	41
N_{60}, bpf[d][e]	~~70~~ 56	108	137	61
$(N_1)_{60}$, bpf[d][g]	~~35~~ 32	45	54	28
Undrained Shear Strength (c_u), tsf	ND	ND	ND	ND
Total stress internal friction angle, Φ	20	ND	13	ND
Total stress cohesion intercept, c , tsf	1.275	ND	7.640	ND
Effective stress internal friction angle, Φ'	37	ND	20	ND
Effective stress cohesion intercept, c', tsf	0.400	ND	4.810	ND
Compression Index, C_c	ND	ND	ND	ND
Recompression Index, C_r	ND	ND	ND	ND
Pre-consolidation Pressure, P_c (psf)	ND	ND	ND	ND

Revise values per Question No. 02.05.04-23

$$G_{max} = V_s^2 \rho \qquad \text{(Equation 2.5.4.10-2)}$$

where:

G_{max} = small-strain shear modulus (psf)
$\rho = \gamma_T/g$
g = gravitational constant = 32.2 ft/sec^2
γ_T = total unit weight of soil (pcf),and
V_s = shear wave velocity of soil (ft/sec)

Delete per Question No. 02.05.04-25

To account for the reduction of G with increasing shear strain, G_{max} is reduced using plots of the ratio G/G_{max} against shear strain, as described in Subsection 2.5.4.7.4. A typical shear strain for settlements is 10^{-3} ~~percent~~ (Reference 2.5.4.10-5). For purposes of the analysis, a reduction ratio of 0.4 is used above -300 ft. NAVD and 0.5 below that point. Poisson's ratio is taken as 0.3 above -300 ft. NAVD and 0.2 below. These values are less than values interpreted from the geophysical logging discussed in Subsection 2.5.4.7 and are based on general recommendations by the Federal Highway Administration (Reference 2.5.4-10-5). The layers, top elevations, unit weights, average shear wave velocities, shear modulus, Poisson's ratio, and elastic modulus used are shown in Table 2.5.4.10-1.

To consider differential settlement, settlement at the center of the base mat, a side point, and a corner are calculated. The differential settlement over the distance is calculated by dividing the distance into the settlement difference. Differential settlement occurs due to lateral changes in soil layer thicknesses or soil properties, and due to the difference in applied stresses below a corner and the center of the loaded area. As discussed in Subsections 2.5.4.1 and 2.5.4.7, the subsurface layers are subhorizontal and have similar thicknesses and properties across the site. Thus, the only contributor to differential settlement is the difference in applied stress conditions under the mat corner and the center.

As discussed in Subsection 2.5.4.5, the base mats for the technologies are constructed with their bottoms at an upper bound level of elevation -2.1 ft. NAVD, and a lower bound level of -47.4 ft. NAVD. However, excavation for all technologies extends to approximate elevation -67 ft. NAVD, to reach the competent layer. Rebound due to removal of soil in the excavation process occurs and recompression of the base of the excavation would occur upon application of the backfill loading. The weight of concrete fill placed below a base mat applies stresses of about the same as the weight of soil removed, or greater. Subsurface deformations related to the weight of the concrete fill are essentially completed before loads from the reactor building are added to the base mat because of the elastic character of the settlement. As a conservative approach for the calculations, the static bearing pressure is taken to be applied at the top of the competent layer (elevation -67 ft. NAVD).

The Janbu analysis method resulted in slightly greater estimated settlement than the Timoshwnko and Goodier analysis method. The estimated settlement from the Janbu analysis described above is 1.6 in. for the center of the mat, and 1 in. for a side of the mat for the average modulus values. For the U.S. EPR mat dimension, the side to center differential settlement is 0.25 in. over 50 ft. Soils respond to load application in an elastic manner, therefore, much of the settlement occurs as the base mat and building wall loads are applied.

2.5.4.10.3.1 Deflection Monitoring

A settlement monitoring program will be conducted during construction of the facility to determine the magnitude of settlement that has occurred during construction to-date, and to better establish

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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 2.4.5 and SSAR Table 2.5.4.2-8 to incorporate the changes in Enclosure 2 in response to NRC RAI 64, Questions 02.05.04-23 and 02.05.04-25.	This revision will be included in a future update of the PSEG ESP application.	Yes	No