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Subject: RAI Letter #6 Geology Response 3 of 3

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RAI Figure 2.5.3-2C. Photograph Illustrating Downward Termination of a "Clastic Dike."

2.5.4-1 SSAR Section 2.5.4.2.2 states that information has been taken from the 14 borings and 10 cone penetrometer tests performed during the ESP subsurface investigation. However, Section 2.2.1 of Appendix 2.5A, "Geotechnical and Laboratory Testing Data Report," to SSAR Section 2.5 states that 12 borings, designated B-1001 through B-1011 and B-1013, were drilled at the site. Please clarify this inconsistency and also describe how the other 2 borings were taken.

Response:

The number of borings drilled during the ESP subsurface investigation is fourteen (14), as briefly stated in Section 2.5.4.2.2 and further described in Section 2.5.4.3.2. These borings can be alternatively described as follows:

- Eleven (11) borings (B-1001, B-1002, B-1004 through B-1011 and B-1013) were drilled and sampled at regular depth intervals. The termination depths of these borings ranged from 98.9 ft (B-1009) to 304 ft (B-1004).
- One (1) boring (B-1003) was continuously cored through soil and rock to a depth of 1,338 ft.
- Boring B-1002A was drilled without sampling to a depth of 105 ft, and C-1005A was drilled without sampling to a depth of 60 ft. These two borings were drilled to allow the performance of suspension P-S velocity logging to measure shear wave velocities above the Utley Limestone where drilling fluid loss was observed.

A summary of the 14 ESP borings is shown on SSAR Table 2.5.4-7, and their locations are shown in SSAR Figure 2.5.4-1.

Section 2.2.1 on Page 2.5A-5 of Appendix 2.5A provides essentially the same information described above and in Sections 2.5.4.2.2 and 2.5.4.3.2 of the SSAR. The first paragraph of Section 2.2.1 of Appendix 2.5A talks about the 12 borings with sampling/coring (same as the first two bullets of this response). The third paragraph of Section 2.2.1 of Appendix 2.5A talks about the 2 borings without sampling/coring (same as the last bullet of this response).

In summary, there's no inconsistency between Sections 2.5.4.2.2 and 2.5.4.3.2 of the SSAR and Section 2.2.1 of Appendix 2.5A.

2.5.4-2 SSAR Section 2.5.4.2.1 states that the Upper Sand Stratum (Group 1 soils) will be completely removed and replaced with compacted structural fill prior to the construction of VEGP Units 3 and 4. SSAR Section 2.5.4.5.3 describes the sources and quality control of the structural fill.

a. Please explain whether the excavation and backfill will cover only the foot-print of the power block or extend to certain distance from the foundation footprint.

b. If the site excavations will not extend to significant distances to the side of the plant, shouldn't the seismic hazard calculations be carried to the free-ground surface including the Barnwell Group in the base-case site soil columns? What is the basis for this column analysis which presumes that the fill extends uniformly in all horizontal directions, while the actual excavation and backfill will extend only in the immediate vicinity of the plant? (Page 2.5.2-39)

c. SSAR Section 2.5.4.5.3 states that backfill will be placed with as much as 25-percent fines. This is significantly higher fines content than used for building foundations. How will compaction controls be implemented for such materials?

Response:

a) Two backfilled excavations will be associated with the safety-related footprints of Units 3 and 4, one for each unit, and will extend beyond their respective power block footprints. The minimum lateral extent of each excavation has been established by determining the stress zone as defined by a 1(H):1(V) slope extending from the bottom of the turbine, containment, and auxiliary building foundations at approximate bottom of foundation elevations of El. 216 ft msl for the turbine building and El 180 ft msl for the containment and auxiliary buildings to the top of the Lisbon Formation (Blue Bluff Marl) at 130 ft msl. The stress zone at the top of the Lisbon Formation will extend approximately 86 ft (El 216-El 130) horizontally beyond the footprint of the power block structures. The turbine building foundation governed this horizontal extension (since it is the higher foundation), and the 86 ft extension was conservatively used for all four sides of the excavation. The entire excavation, including the power block footprint, stress zone, and areas beyond the stress zone will be backfilled with compacted structural fill.

b) The site excavations will extend to significant horizontal distances from the structures. With the base of the excavation extending approximately 86 feet outside the building footprints as described above, and with excavation side slopes at 2(H):1(V), the structural fill will extend more than 180 ft beyond the containment and auxiliary buildings at their foundation level, and will extend more than 250 ft beyond the edge of the turbine building at its foundation level.

c) Sand and silty sand (SM) with no more than 25 percent fines obtained from on-site sources were used as structural backfill for Units 1 and 2, as described in VEGP Unit 1 & 2 FSAR Section 2.5.4.5.2.1. The same structural backfill criterion will be used for Units 3 & 4.

Compaction controls for placement of the backfill will be implemented through an independent soil testing firm. As identified in SSAR Section 2.5.4.5.3, this testing firm will maintain an on-site soils testing laboratory to control the quality of the fill material and the degree of compaction. Compaction will be monitored through field density testing performed at a minimum frequency of one test per 10,000 square ft per lift of placed fill. More detailed testing compaction control criteria will be developed during the COL.

2.5.4-3 According to Table 2 of Appendix A, "Boring Data," to Appendix 2.5A of the SSAR, only 4 borings (B-1002, B1003, B1004 and B-1005) went through the Blue Bluff Marl material (Group 2 Zone) and reached the Lower sand Stratum (Group 3 Zone - coarse- to fine sand with interbedded thin seams of silt and/or clay). Since the top layer of soil (Group 1 soil) will be removed and backfilled with compacted structural fill prior to the construction of VEGP, Units 3 and 4, only the information collected from these 4 borings can be used for the investigation of Group 2 and Group 3 soil that are supposed to be the primary load-bearing component of safety-related facilities. Please provide justification for the following:

With the data from 4 borings and no significant samples taken in Group 3 zone, what is the basis for the development of geotechnical parameters of Groups 2 and 3 layers?

SSAR Section 2.5.2.5.1.2 indicates that base case soil velocity profiles together with their uncertainty were developed from the available data. If only 12 borings were taken at the site, and most of these borings did not extend to depths below 91 m (300 ft), how were these parameters developed?

Are there any indications of soft zones, such as those encountered at the Savannah site, in the upper soils of the profile above the Blue Bluff Marl which may be collapsible under a seismic event? Even though soils under the foundation footprint are to be removed, how far to the side of the plant does collapsibility become unimportant?

Soft soils were indicated in the lower soils below the Blue Bluff Marl. Standard Penetration Test (SPT) blow counts for the lower sands in B-1002 are indicated to be as low as 10 bpf (Page "4 of 6" of Soil Test Boring Record of Appendix A to Mactec's report). Please explain what is the implication of such low values even though the average blow count through this material is indicated to be about 60 bpf?

Response:

a) Three (3) ESP borings (B-1002, B-1003 and B-1004) completely penetrated the Blue Bluff Marl (Lisbon Formation), and another nine (9) ESP borings extended partly into the marl. Borings B-1002 and B-1004 extended through the Still Branch Formation and penetrated into the Congaree Formation. Boring B-1003 extended all the way through the Lower Sand Stratum to bedrock at 1338-ft depth. Boring B-1005 did not completely penetrate the Blue Bluff Marl. From the borings in Blue Bluff Marl, 58 SPT N-values were obtained from Group 2 and 3 layers along with the corresponding SPT samples. Twelve tube samples were also obtained. Borehole geophysical tests including suspension P-S velocity logging were performed in the three borings that completely penetrated the marl.

The following laboratory test results were obtained from the SPT and tube samples:

33 natural moisture contents

19 Atterberg limits

19 grain-size curves

28 unit weights

15 undrained shear strengths (from unconsolidated-undrained triaxial tests)

The geotechnical parameters were derived primarily from the ESP field and laboratory test results. Parameters from VEGP Units 1 & 2, were also taken into consideration. We note that geotechnical properties of layers below the Congaree Formation will be of no engineering consequence for the design of plant foundations, as these layers are too deep.

Four borings will extend to a depth of 250 and two borings will extend to a depth of 400 ft during the COL subsurface investigation. These borings will provide additional data related to Group 2 and 3 layers.

b) The base case shear wave velocity profile in the Lisbon Formation, Still Branch Formation and in the upper portion of the Congaree Formation was derived from the results of the three suspension P-S velocity logging tests performed in these strata. One of the suspension P-S velocity logging tests extended into bedrock below the Lower Sands, and the results were used to derive the base case shear wave velocity profile below the top of the Congaree Formation.

As noted in SSAR Section 2.5.2.5.1.2.2, the randomization model that captures the uncertainty involved in the base case shear wave velocity profile for the in-situ soils used the logarithmic standard deviation of shear wave velocity as a function of depth set to values obtained from soil randomization performed at the SRS site.

c) "Soft zones" with SPT N-values ≤ 5 bpf were encountered in the upper soils of the profile at ESP boreholes B-1001, B-1004, B-1005 and B-1006. For such soils below the water table, it is probable that they would liquefy under certain seismic events, resulting in several and perhaps even many inches of settlement of the surface above the liquefied material.

The planned location of the nuclear island relative to the upper sands acknowledges the potential lack of stability of these sands. Our response to RAI 2.5.4-2(a) provides further details about the extent of the soil replacement in the power block area.

d) SPT-38 at 189 ft in boring B-1002 disclosed N = 9 bpf, and is the only N-Value taken in the Lower Sand Stratum during the ESP subsurface investigation that indicates a loose relative density. This result was obtained in the Still Branch formation that is over 40 million years old, and that has a present overburden pressure approaching 7 tons/ft^2. Sands of this age and depth cannot be in a loose condition under normal circumstances. Infilling of a cavity could result in a low blowcount, but there is no evidence of cavities in this formation.

The measured shear wave velocity in B-1002 was 1,320 ft/sec at 188.7 ft depth, and 1,200 ft/sec at 190.3 ft depth. These values indicate dense to very dense sands with typical N-values in the 45 to 50 bpf range, similar to the N-values measured in the soils above and below 189 ft. The caliper log for B-1002 showed a very constant diameter of about 3.75 in. at, above and below 189 ft. In short, the geophysical measurements taken at 189 ft depth in B-1002 show no physical or strength abnormalities.

The most plausible explanation for the low blowcount is that the SPT was taken through disturbed material at the bottom of the drill hole. This could be from the cuttings not being properly flushed out of the hole before sampling. Alternatively, a temporary imbalance of water pressure inside and outside the borehole could have caused some flow of the sand at the bottom of the boring. The sieve analysis performed on the SPT sample at 189 ft depth indicates a fine sand with less than 7% fines, indicating a material susceptible to flow.

In summary, the isolated very low N-value in the Still Branch formation was most probably due to poor sampling rather than loose in-situ material.

2.5.4-4 SSAR Section 2.5.2.5.1.2 states that the backfill shear wave velocities were determined from measurements made on the existing backfill at the site under Units 1 and 2 as summarized in Tables 2.5.4-10 and 2.5.4-11. As indicated in these tables, the shear velocities in the top layers of backfill are well below 305 mps (1,000 fps). How were shear wave velocity values generated at depths of 15 m (50 ft) or more below the top of the backfill? Were effects of confinement considered?

Response:

SSAR Section 2.5.2.5.1.2.1.1 states that, "Shear-wave velocity was not measured for the compacted backfill during the ESP subsurface investigation (APPENDIX 2.5A). Interpolated values based on measurements made on fill for existing Units 1 and 2 **(Bechtel 1984)** are used instead." The RAI implies that shear wave velocity measurements were made on the in-situ backfill for Units 1 and 2. In fact, the "measurements made on fill for existing Units 1 and 2" were laboratory measurements using resonant column tests. No in-situ shear wave velocity measurements were made for the compacted backfill before or during the ESP subsurface investigation. The shear wave velocity profile for the backfill was developed from the equations:

$G_{max} = 1000 \cdot K_2 \cdot (\sigma'_m)^{0.5}$

$V_s = (G_{max}/\rho)^{1/2}$

or

$V_s = (G_{max} \cdot g/\gamma)^{1/2}$

where: G = dynamic or low-strain shear modulus (psf)

σ'_m = mean principal effective stress (psf)

K_2 = a parameter reflecting primarily the effect of void ratio or relative density and the strain amplitude of the motions. The value of K_2 was determined to be 80 from resonant column test results.

V_s = shear wave velocity (fps)

ρ = density of the backfill sample

γ = unit weight of the backfill sample (pcf)

g = acceleration of gravity = 32.2 ft/s^2.

2.5.4-5 SSAR Section 2.5.4.7.2.1 (Page 2.5.4-27) indicates that the EPRI 1993 soil degradation relationships were used to perform SHAKE analyses and derive the shear modulus reduction factors. It is the NRC staff's understanding that the appropriateness of using the EPRI 1993 curves for fine-grained soils is not obvious since they were generally developed for sands and but not fine grained silts or clays. (The degradation models at the Savannah River site were generated from laboratory testing of in-situ soils.) Please explain the significance of using such models for fine-grained soils on the computed results?

Response:

The degradation curves included in EPRI 1993 cover the range of soils from gravels to high plasticity clays, and thus are appropriate for fine-grained soils. The curves for fine-grained soils are presented in Figures 7.A-16 (shear modulus reduction curves) and 7.A-17 (damping ratio curves) in terms of soil plasticity, and require the use of the plasticity index (PI). Our response to RAI 2.5.4-17 provides further details on how the degradation curves included in the SSAR were derived from the EPRI (1993) curves.

The soil degradation relationships for fine-grained soil (and also coarse-grained soils) used in the SSAR will be verified by laboratory testing of in-situ soil samples during the COL subsurface investigation.

2.5.4-6 Regarding the ground water control, SSAR Sections 2.5.4.5 and 2.5.4.6 state that (1) the total depth of construction excavation to the Blue Bluff Marl bearing stratum will range from approximately 80 to 90 ft (4 to 27 m) below ground surface (SSAR Section 2.5.4.5.1), (2) the groundwater generally occurs at a depth of about 60 ft (18 m) below the existing ground surface (SSAR Section 2.5.4.6.1), and (3) due to the relatively impermeable nature of the Upper Sand Stratum, sump-pumping of ditches will be adequate to dewater the soil. Please explain what dewatering procedures and what criteria will be developed to "minimize effects on the surrounding area and the existing

power block"? (The impact of the simple use of sumps and pumps on any existing area of the site will depend on the extent of time during which drawdown will occur.)

Response:

The dewatering program as identified in SSAR Section 2.5.4.6 was developed based on the similar subsurface conditions to those encountered at Units 1 and 2 and based on the success of the dewatering program employed for Units 1 and 2. The planned dewatering program will utilize a series of ditches that drain to a sump or sumps. The sump(s) will be equipped with pumps to discharge the ground water inflow. The pumps will also have capacity to discharge storm water inflow. The ditches will be advanced below the progressing excavation. However, the dewatering program for Units 3 and 4 differs from Units 1 and 2 in that the excavation below the design groundwater elevation of 165 feet MSL will not remain open as long as it was for Units 1 and 2. At Units 1 and 2, various safety-related power block structures were founded on or in the Blue Bluff Marl, requiring the drawdown until backfilling around these structures exceeded EL. 165 ft MSL. All safety-related structures for Units 3 and 4 will be founded in the compacted backfill at or above an elevation of about 180 feet MSL. This founding level is about 15 feet above the design groundwater level. Once the Upper Sand stratum has been excavated and the Blue Bluff Marl level prepared, the excavation may be backfilled. The dewatering system will be maintained during the placement of compacted backfill. However, upon achieving the design founding level groundwater inflow should not impact construction and the drainage ditches and sump(s) will be maintained to control storm water inflow only. This will enable the excavation below the groundwater level to remain open for a much shorter length of time than was required for Units 1 and 2.

The duration that the excavation below the groundwater remains open will be determined by the constructor. Nevertheless, even if the excavation remains open for an extended period, the drawdown effects on the existing power block will be minimal. At the excavation site, the groundwater will be lowered about 45 feet; from El. 165 ft to El. 120 ft. Conservatively neglecting the cone of depression that the groundwater level will follow outside the excavation and projecting the 45-foot drop in groundwater level to the existing power block, the effective stresses beneath the power block will increase. However, this increase in effective stress (45 ft x 62.4 pcf = 2,310 psf) at the projected bottom of the backfill will have little impact on settlement of the existing power block structures. The safety-related structures in the existing power block are either founded in compacted fill or in/on the Blue Bluff Marl. Data for the existing Units 1 and 2 granular backfill (Bechtel 1985-see list of references at the end of the response to this RAI) disclosed measured SPT N-values greater than 100 bpf. The design elastic modulus for this very dense backfill was 1,500 ksf. Likewise, the Blue Bluff Marl is characterized by design values of 80 bpf for the measured SPT N-value and 10,000 ksf for the elastic modulus. In addition, the preconsolidation pressure for the very hard Blue Bluff Marl is estimated at 80,000 psf. Given these stress-strain characteristics of the foundation materials, the potential drawdown effects (primarily settlement) associated with the excavation dewatering at Units 3 and 4 are expected to be minimal.

<u>References</u>

(Bechtel 1985) Bechtel Power Corporation, Geotechnical Verification Work-Report of Results, Vogtle Electric Generating Plant, August 1985.

2.5.4-7 SSAR Section 2.5.4.2.2.2 states that 15 unconsolidated undrained (UU) tests were performed on Blue Bluff Marl samples and that the measured undrained shear strength ranged from 150 to 4,300 psf. Both of these values are significantly lower than the 10,000 psf design value. Please justify the wide range of values and why they differ substantially from the values measured previously for Units 1 and 2. In addition,

elaborate on how the Standard Penetration Test N-values measured during the ESP investigations support the use of the 10,000 psf design value.

Response:

Laboratory measurements of undrained shear strength for the Blue Bluff Marl (Lisbon Formation) made as part of the ESP subsurface investigation yielded values ranging from 150 to 4,300 psf. A total of 15 UU tests were performed using 1 confining pressure corresponding to the overburden pressure. These values were deemed low because of qualitative and quantitative reasons outlined in the next paragraphs.

Qualitative factors that suggest that the laboratory measured undrained shear strength for the Lisbon Formation is low:

- CPTs could not be pushed below the Barnwell Group and into the Lisbon Formation because the soils were too hard.
- Shelby tubes could not be pushed into the Lisbon Formation without being damaged, suggesting a hard formation.
- Samples obtained by pitcher barrel were likely disturbed by the sampling, storage and transportation process.
- A design undrained shear strength value of 10,000 psf is adopted for the Lisbon Formation in the VEGP Unit 1 & 2 FSAR.
- Adoption of the 10,000-psf design value in the ESP is supported by other ESP site specific data, as described in the next paragraphs.

Empirical Correlation with PI – Peck et al. (1974) suggest the following correlation between plasticity index (PI) and the ratio of undrained shear strength (s_u) over vertical effective stress (p) of *normally consolidated* soft clays (Equation 4.8 on p. 93 of Peck et al. 1974):

$s_u/p = 0.1+0.004PI$

Using the value of 25% for PI we get

$s_u/p = 0.1+0.004x25 = 0.2$

If we consider the ground water depth at 60 ft, we can take the vertical effective stress at the top of the Lisbon Formation (depth taken as 90 ft) as

p = 115x60+(115-62.4)x30 = 8,478 psf

and

s_u = 0.2x8,478 = 1,696 psf, say 1.7 ksf

A similar calculation at the bottom of the Lisbon Formation (depth taken as 150 ft) yields

p = 115x60+(115-62.4)x90 = 11,634 psf

and

$s_u = 0.2 \times 11{,}634 = 2{,}327$ psf, say 2.3 ksf

In other words, 1.7 ksf to 2.3 ksf would be reasonable estimates of undrained shear strength at the top and bottom of the Lisbon Formation, ***if*** the Lisbon Formation were normally consolidated. Because the Lisbon Formation is highly overconsolidated, we can state that 1.7 to 2.3 ksf represent very low estimates of undrained strength for the Lisbon Formation, and can be used to invalidate the low laboratory measured values.

Empirical Correlation with SPT N-value - The undrained strength, s_u, was calculated from Terzaghi's correlation with the SPT N-value (Fig. 1.22, p. 38 of Winterkorn & Fang 1975 - see list of references at the end of the response to this RAI). This correlation is given by

$s_u = N/8$ (ksf)

where: N = SPT N-value in blows per foot (bpf)

If we use the average N-value of $N_{avg} = 83$ bpf we get

$s_u = 83/8 \cong 10$ ksf

We note that the split barrel sampler did not penetrate the full 12 inches (1 foot) during most of the SPT sampling in the Lisbon Formation. In these cases the blow counts for the actual sampler penetration were linearly extrapolated to N-values corresponding to 12 inches of penetration. These linearly extrapolated N-values exceeded 100 bpf in most cases. A cutoff value of 100 bpf was used in the computation of N_{avg} = 83 bpf, thus resulting in a conservative estimate of N_{avg}.

Empirical Correlation with Shear Wave Velocity - The correlation shown below between shear wave velocity, Vs, and cone tip resistance, q_t (figure on p. 103 of Mayne 2006 - see list of references at the end of the response to this RAI):

$V_s = 1.75(q_t)^{0.627}$

where: V_s = shear wave velocity (m/s)

q_t = cone tip resistance (kPa)

Using the average value of 2,354 fps ≅ 717 m/s for V_s reported in Section 2.5.4.4.2.1 we get

$q_t = (717/1.75)^{1/0.627} \cong 14{,}454$ kPa ≅ 302 ksf

Mayne (2006) suggests the following correlation between undrained shear strength, s_u, and cone tip resistance, q_t (figure on p. 62 of Mayne 2006):

$s_u = (q_t - \sigma_{vo})/15$

where: σ_{vo} = total vertical stress (ksf)

Using $\gamma = 115$ pcf, then σ_{vo} at the bottom of the Lisbon Formation, i.e., 150 ft depth is

$\sigma_{vo} = 150 \times 115 = 17{,}250$ psf = 17.25 ksf

and we get

$s_u = (302-17.25)/15 \cong 19$ ksf

The value of σ_{vo} is smaller at top of the Lisbon Formation, and will result in a slightly higher s_u–value.

Table 2 of Senapathy et al. (2001) (See list of references at the end of the response to this RAI.) summarizes values of G_{max}/s_u from 15 clay sites. The values ranged from 535 to 1,539 with a median value of 828, and average value of 892.

We know that

$G_{max} = (V_s)^2 \cdot \gamma/g$

Using V_s = 2,354 fps (average for the Lisbon Formation), and γ=115 pcf, then the average G_{max} for the Lisbon Formation is

$G_{max} = 2{,}354^2 \cdot 115/32.2 \cong 19{,}790{,}000$ psf =19,790 ksf

If we use the minimum and maximum values of G_{max}/s_u reported by Senapathy et. al. (2001), we obtain:

$s_u = 19{,}790/535 \cong 37$ ksf, for $G_{max}/s_u = 535$

and

$s_u = 19{,}790/1{,}539 \cong 12.9$ ksf, for $G_{max}/s_u = 1{,}539$

In summary, the low undrained shear strength measured in the laboratory for the Lisbon Formation is likely due to disturbance introduced by the sampling, and sample storage and transportation process. Field evidence (impossibility of pushing Shelby tubes) and evidence from other available data (SPT N-values and geophysical test results) justify the use of s_u = 10,000 psf for the Lisbon Formation. This value was also adopted in the VEGP Unit 1 & 2 FSAR.

References:

(Mayne 2006) Mayne, P.W, *Site Characterization by Seismic Piezocone,* Georgia Instiute of Technology, Atlanta, GA, 2006.

(Senapathy et al. 2001) Senapathy, H., Clemente, J.L.M. and Davie, J.R., “Estimating dynamic shear modulus in cohesive soils”, Proceedings International Conf. Soil Mechanics & Geotechnical Engrg., Istanbul, Turkey, 2001.

(Winterkorn and Fang 1975) Winterkorn, H.F. and Fang, H.Y., *Foundation Engineering Handbook*, Van Nostrand Reinhold Co., New York, NY, 1975.

2.5.4-8 SSAR Section 2.5.4.2.2.2 states, “Previous laboratory test results indicate the Blue Bluff Marl to be highly preconsolidated ... the preconsolidation pressure of the Blue Bluff Marl stratum was estimated to be 80,000 psf. Settlements due to loadings from new structures would be small due to this preconsolidation pressure.”

a) Provide a description of the “previous” laboratory testing methods and results.

b) Justify the assumption of an undrained shear strength of 16,000 psf as UU Test results range from 150 to 4,300 psf. Were consolidation tests performed to verify this assumption?

c) The preconsolidation pressure for the Blue Bluff Marl is given as 80,000 psf and is based on the plasticity index values (which ranged from 2 to 70 with an average value of 25) and a PI of 25, which results in a su/p (undrained shear strength / effective preconsolidation pressure) ratio of 0.2. Provide a complete description of the Skempton (1957) method used to determine the ratio of 0.2. Also, justify the use of 0.2 for the ratio in view of the wide range of PI values. In addition, justify the estimated preconsolidation pressure for the Blue Bluff Marl based on the wide range of PI values.

d) Justify your conclusion, "settlements due to loadings from new structures would be small due to this preconsolidation pressure," in view of settlements for the current Units 1 and 2 and also with regard to the su/p ratio of 0.2, which indicates that the soil is under consolidated (0.25 is an indication of normally consolidated soil).

Response:

a) The original data and interpretation are contained in Bechtel (1974b). The responses provided here are largely excerpts from this reference. Laboratory tests included one hundred and ninety one 1-point unconsolidated-undrained (UU) triaxial tests, and thirty eight consolidation tests.

The 1-point UU tests disclosed undrained shear strength values in the range of 260 psf to 500,000 psf. Thirty of the UU tests, i.e., 15.7% of all measurements, disclosed undrained shear strength values lower than 10,000 psf. Twenty five of the UU tests that disclosed undrained shear strength values lower than 10,000 psf also disclosed larger strains at failure, which was considered to be due to disturbance either during sampling or preparation of test specimens. This leaves five tests on good quality samples, i.e., 2.6% of all measurements disclosed undrained shear strength values lower than 10,000 psf. Thus, adoption of a 10,000-psf design value can be considered conservative based on the available data.

The consolidation tests were performed using vertical pressures that reached 64 ksf for all thirty eight specimens. Most of the test results (void ratio versus vertical effective stress curves) showed very flat curves that indicated that the preconsolidation pressure had not been achieved.

b) The 16,000 psf value is the average undrained shear strength value based on previous laboratory test results contained in Bechtel (1974b) for Units 1 and 2. The 16,000 psf average value was calculated from the 1-point unconsolidated-undrained (UU) triaxial tests that disclosed undrained shear strength of less than 50,000 psf. This average includes the results of one hundred and eighty five tests, because only six tests disclosed undrained shear strength of more than 50,000 psf. Results from tests performed on samples deemed disturbed (twenty five of them, as explained above) were included in this average.

c) The Skempton (1957) method used to determine the ratio $s_u/p = 0.2$ is the same method described in our response to RAI 2.5.4-7, i.e.,

$s_u/p = 0.1+0.004PI$

Using the average value of 25% for PI we get

$s_u/p = 0.1+0.004x25 = 0.2$

If we use $s_u = 16,000$ psf and $s_u/p = 0.2$, we obtain

$p = 16,000/0.2 = 80,000$ psf

We know that the vertical effective stress in the Lisbon Formation ranges from 8,478 psf at the top of the layer to 11,634 psf at the bottom of the layer (see our response to RAI 2.5.4-7). Thus, the calculated value of p = 80,000 psf is 6.9 to 9.4 times larger than the vertical effective stress. We can conclude that the Lisbon Formation is highly overconsolidated with overconsolidation ratios (OCRs) in the range of 6.9 to 9.4.

Use of the average value PI=25% is justified as a common geotechnical practice. It would be unreasonable to consider outlying measured values in these types of calculations. However, let's consider the extreme low measured value of PI=2%, and follow the same procedure used for PI=25%. We obtain the following:

$s_u/p = 0.1+0.004x2 = 0.108$

If we use s_u = 16,000 psf and $s_u/p = 0.108$, we obtain

$p = 16,000/0.108 \cong 148,000$ psf

Using the vertical effective stress in the Lisbon Formation ranging from 8,478 psf at the top of the layer to 11,634 psf at the bottom of the layer (see our response to RAI 2.5.4-7), the calculated value of p = 148,000 psf is 12.7 to 17.5 times larger than the vertical effective stress. We could conclude that the Lisbon Formation is highly overconsolidated with overconsolidation ratios (OCRs) in the range of 12.7 to 17.5.

If we consider the extreme high measured value of PI=70%, and follow the same procedure used for PI=25%, we obtain the following:

$s_u/p = 0.1+0.004x70 = 0.38$

If we use s_u = 16,000 psf and $s_u/p = 0.38$, we obtain

$p = 16,000/0.38 \cong 42,000$ psf

Using the vertical effective stress in the Lisbon Formation ranging from 8,478 psf at the top of the layer to 11,634 psf at the bottom of the layer (see our response to RAI 2.5.4-7), the calculated value of p = 42,000 psf is still 3.6 to 5 times larger than the vertical effective stress. We could conclude that the Lisbon Formation is highly overconsolidated with overconsolidation ratios (OCRs) in the range of 3.6 to 5.

We also note that most of the consolidation tests results on thirty eight samples of the Lisbon Formation reported in Bechtel (1974b) showed very flat curves that indicated that the preconsolidation pressure exceeded 64,000 psf. Thus, the 80,000-psf preconsolidation pressure estimated from the empirical correlation is reasonable.

d) As a starting point for this part of the response, we note that the Lisbon Formation is highly overconsolidated, as outlined in part c of this response. There is no evidence (previous or obtained during the ESP subsurface investigation) to suggest that the Lisbon Formation is underconsolidated.

Our conclusion that settlements due to loadings from new structures would be small due to this preconsolidation pressure is based on the fact that heavily overconsolidated soils are known to develop small settlements, and these settlements take place during placement of structural loads, i.e., during construction. This behavior was observed during settlement monitoring for Units 1 and 2, as described in the "VEGP Report on Settlement" prepared by Bechtel in 1986. The same behavior is expected for Units 3 and 4.

2.5.4-9 SSAR Section 2.5.4.2.5.2 cites Bowles (1982) as the reference for determining the effective angle of internal friction for site soils. It is not clear how the effective angle of internal friction was calculated using this reference. Provide an example of a calculation and justify the accuracy of the results in view of the range of N-values.

Response:

The angle of shearing resistance, ϕ, of the granular Upper Sand and Lower Sand Strata at the site was estimated from an empirical correlation with SPT N-values (Bowles 1982). Table 3-2 on p. 100 of Bowles (1982) was used. This table provides ranges of ϕ-values as a function of ranges of SPT N-values. A review of Table 3-2 on p. 100 of Bowles (1982) reveals that the ranges of ϕ-values are usually much narrower than the corresponding ranges of SPT N-values, and there is also some overlapping of proposed ranges. Engineering judgment was used in the selection of appropriate ϕ-values, as explained in the next paragraphs.

The average SPT N-value for the Upper Sand Stratum adjusted for hammer efficiency is N_{avg} = 25 bpf, which falls in the range of $10 \leq N \leq 40$. The corresponding range of ϕ-values on Table 3-2, p. 100 of Bowles (1982) is $35° \leq \phi \leq 40°$. VEGP Unit 1 & 2 FSAR Table 2.5.4-2 recommends $\phi = 34°$ for the Upper Sand Stratum. We used $\phi = 34°$, as shown on SSAR Table 2.5.4-1.

The average SPT N-value for the Lower Sand Stratum adjusted for hammer efficiency is N_{avg} = 62 bpf, which falls in the range of $20 \leq N \leq 70$. The corresponding range of ϕ-values on Table 3-2, p. 100 of Bowles $38° \leq \phi \leq 43°$. VEGP Unit 1 & 2 FSAR Section 2.5.4.2.3 states that SPT N-values for the Lower Sand Stratum ranged from 70 to more than 100 bpf. While $38° \leq \phi \leq 43°$ would be reasonable to use considering the high SPT N-values, we decided to adopt a more conservative $\phi = 34°$ for the Lower Sand Stratum. This value is shown on SSAR Table 2.5.4-1 and matches the value for the Compacted Structural Fill contained on VEGP Unit 1 & 2 FSAR Table 2.5.4-8.

2.5.4-10 Provide relative densities for Blue Bluff Marl.

Response:

As stated in our response to RAI 2.5.4-14, although the Blue Bluff Marl frequently contains less than 50% of fine material, it has the appearance and characteristics of a calcareous claystone or siltstone and is described as a hard, slightly sandy, cemented, calcareous clay. Its design undrained shear strength is 10 ksf, and its preconsolidation pressure could be as high as 80 ksf, i.e. this is a highly overconsolidated material. Thus, the marl performs as hard clay or soft rock, not as a granular material, and relative density does not apply to these types of materials.

2.5.4-11 SSAR Section 2.5.4 states that high strain elastic modulus for Upper Sand and Lower Sand Strata were derived based on the Davie and Lewis (1988) relationships. Explain why these relationships are applicable to the ESP soil strata. What is the scientific consensus on Davie and Lewis' relationship between SPT values and elastic modulus, as well as the relationship between undrained shear strength and elastic modulus? How extensively are these relationships used?

Response:

a) The relationship for sand, E= 36N ksf, was derived based on elastic modulus E versus SPT N-value relationships reported in the literature and influenced by a specific case history of measured settlement of a chimney foundation on medium dense to dense quartz sands and gravels described in the paper (Davie and Lewis 1988). Bechtel's experience has shown that this relationship provides reasonable predictions of settlement when compared to measured settlements for a wide range of foundation sizes on granular materials varying from clean to silty sands and gravels. The Upper Sand Stratum is a medium dense silty sand, and the Lower Sand Stratum is a generally very dense silty sand. Thus, it is anticipated that the relationship can be successfully applied to these sands.

b) *SPT and Elastic Modulus*

Davie and Lewis (1988) provides a summary of various estimates of elastic modulus E for granular soils from SPT N-values in the literature. The table below shows the computed E values based on N = 25 bpf (design value for the Upper Sand Stratum corrected for hammer efficiency), and N = 62 bpf (design value for the Lower Sand Stratum corrected for hammer efficiency).

Reference	Relationship	E, ksf	
		N = 25 bpf	N = 62 bpf
Bowles (1987)	E = 10 (N + 15) ksf	400	770
D'Appolonia et al. (1970)	E = 432 + 21.2N ksf	962	1,746
Parry (1971)	E = 100N ksf	2,500	6,200
Schmertman (1970) and Schmertman et al. (1978)	E = 30N to 50N ksf	750 to 1,250	1,860 to 3,100
Yoshida and Yoshinaka (1972)	E = 42N ksf	1,050	2,604
Median		**1,006**	**2,232**
Davie and Lewis (1988)	E = 36N ksf	900	2,232
Note: The references shown above are cited in Davie and Lewis (1988) and are listed at the end of the response to this RAI.			

As can be seen, the Davie and Lewis (1988) E-value is somewhat lower than the median from the other references for N= 25 bpf and almost identical to the median for N = 62 bpf. Since, as noted in part a) of this response, the E = 36N ksf relationship has provided reasonable predictions of settlement when compared to measured settlements, and the relationship gives predictions that are close to the median of other E and N relationships, then it is expected that this relationship has reasonable scientific consensus.

Undrained Shear Strength and Elastic Modulus

The relationship between a clay's undrained shear strength (s_u) and elastic modulus E is widely recognized. As noted in Davie and Lewis (1988), a large range of E/s_u values have been reported in the literature, from as low as 50 (Skempton 1951-see list of references at the end of the response to this RAI) to as high as 2,500 (D'Appolonia et al. 1971), and thus scientific consensus may be difficult to achieve. To get this wide range of E/s_u values found in the literature into some context, one can refer to Figure 5 from Duncan and Buchignani (1976-see list of references at the end of the response to this RAI) which shows $E = Ks_u$ where $K = E/s_u$ is a function of plasticity index (PI) and overconsolidation ratio (OCR) of the clay. For a normally consolidated clay (OCR = 1), K ranges from about 130 at very high PI to 1,500 at very low PI. For a very heavily overconsolidated clay (OCR = 10), K ranges from about 30 at very high PI to 400 at very low PI. The design PI for the Blue Bluff Marl is 25, and thus the K value will be in the PI<30 area. Figure 5 of Duncan and Buchignani (1976) shows that K= 600 is comfortably in the range of values in the PI<30 area. [Bear in mind that the curves on Figure 5 of Duncan and Buchignani (1976) are purely empirical and based on limited data. The figure should be used as a guideline only.]

The E/s_u = 600 used in Davie and Lewis (1988) is in good agreement with values derived by Williams and Focht (1982-see list of references at the end of the response to this RAI) from numerous case histories of mat foundations on Beaumont Clay, the material in the Davie and Lewis paper.

c) These relationships have been used by Bechtel to estimate settlement of major structures for numerous power plant projects both in the US and overseas. These power plants have been founded on a wide range of granular and cohesive materials.

References [All references are cited in Davie and Lewis (1988), except for Duncan and Buchignani (1976)]

Bowles (1987) Bowles, J.E., "Elastic Foundation Settlements on Sand Deposits", *Journal of Geotechnical Engineering*, ASCE, Vol. 113, No. 8, 1987.

D'Appolonia et al. (1970) D'Appolonia, D.J., D'Appolonia, E. and Brissette, R.F., "Settlement of Spread Footings on Sand", Discussion Closure, *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 96, SM2, 1970.

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Schmertman et al. (1978) Schmertman, J.H., Hartman, J.P. and Brown, P.R., "Improved Strain Influence Factor Diagrams", *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 104, GT8, 1970.

Skempton, A.W. (1951) Skempton, A.W., "The Bearing Capacity of Clays", *Building Research Congress*, pp. 180-189, 1951.

Williams and Focht (1982) Williams, C.E. and Focht, J.A., "Initial Response of Foundations on Stiff Clay", *ASCE Convention*, New Orleans, LA, 1982.

Yoshida and Yoshinaka (1972) Yoshida, I. and Yoshinaka, R., "A Method to Estimate Modulus of Horizontal Subgrade Reaction for a Pile", *Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 12, No. 3, 1972.

2.5.4-12 SSAR Table 2.5.4-1 presents average static engineering properties of the subsurface material. Explain how the value for the unit weights for the different soils were obtained. Based on the discussion in the last paragraph on page 2.5.4-10, the average values are higher than those listed in the table. Also, explain why the plasticity index, liquid limit, and plastic limit values are different from those discussed on page 2.5.4-5 for the Blue Bluff Marl.

Response:

Unit Weight - Unit weight test results for selected soil samples collected during the ESP subsurface investigation are summarized in SSAR Table 2.5.4-4, and are reproduced below. These tests were performed mostly on Lisbon Formation samples, but also on some of the fine-grained samples encountered in the Lower Sand Strata.

Boring Number	Depth (ft)	Formation	γ (pcf)	Boring Number	Depth (ft)	Formation	γ (pcf)
B-1002	92.0	Lisbon	103.6	B-1004	154.5	Lisbon	117.4
			102.4				119.3
	103.5	Lisbon	114.3		164.5	Lisbon	117.4
			114.5				125.6
	113.5	Lisbon	132.8		177	Lisbon	124.7
			132.9				124.6
	123.5	Lisbon	140.2				131.8
	133.5	Lisbon	118.0		188.5	Lisbon	120.4
			118.1				120.6
B-1003	93	Lisbon	115.7		198.5	Lisbon	128.1
			115.8				128.2
	104.7	Lisbon	111.5			**Range:**	102.4-140.2
	121.7	Lisbon	122.5			**Average:**	119.7
	141.7	Lisbon	126.1			**Median:**	118.7
B-1004	144.0	Lisbon	105.1	B-1003	165.7	Still Branch	121.7
			105.2		315.7	Congaree	119.4
			114.2		350.7	Snapp	128.3

Additionally, from VEGP Unit 1 & 2 FSAR Table 2.5.4-4 and Sections 2.5.4.10.2 and 3.7.B.1.4, the following moist/saturated unit weight values are recommended for the different layers:

Compacted Structural Fill - γ_{moist} = 123 pcf

γ_{sat} = 133 pcf

Upper Sand Stratum – γ_{moist} = 115 pcf

γ_{sat} = 115 pcf

Lisbon Formation (Blue Bluff Marl)– γ_{sat} = 115 pcf

Lower Sand Stratum – γ_{sat} = 115 pcf

Considering that the values recommended in the VEGP Unit 1 & 2 FSAR are based on a much larger number of tests, we used the values shown in the VEGP Unit 1 & 2 FSAR. We note that these values will be reassessed during the COL subsurface investigation, where a much larger number of tests will be performed.

Atterberg Limits - Natural moisture content (ω_N) and Atterberg limit test results for selected Lisbon Formation samples collected during the ESP subsurface investigation are given in Table 2.5.4-4 and are summarized on the next page. The results indicate average and median values of liquid limit (LL) below 50% which corresponds to low plasticity soils. It's noted that several samples disclosed high plasticity soils. Also, the natural moisture content is mostly near the plastic limit, which corresponds to the hard consistency disclosed by the SPT N-values. Bechtel (1974b) indicates PI-values for the Lisbon Formation ranging from 2 to 70% with an average of 25%. The range of PI-values shown on the table below ($0 \leq PI \leq 58\%$) is similar to that reported in Bechtel (1974b). Considering that the range and average values reported in Bechtel (1974b) are based on a much larger number of tests, we used PI_{avg} = 25%. The values shown on the line of the table below labeled "Use" are the values shown on SSAR Table 2.5.4-1. We note that these values will be reassessed during the COL subsurface investigation, where a much larger number of tests will be performed.

Boring Number	Depth (ft)	Formation	ω_N (%)	PL (%)	LL (%)	PI (%)
B-1002	92.0	Lisbon	52.1	37	72	35
	103.5	Lisbon	56.5	22	34	12
	113.5	Lisbon	25.5	19	29	10
	123.5	Lisbon	13.5	17	22	5
	133.5	Lisbon	28.6	25	32	7
	153.5	Lisbon	23.3	21	34	13
B-1003	88	Lisbon	67.4	42	93	51
	93	Lisbon	30.6	32	54	22
	104.7	Lisbon	40.6	51	83	32
	121.7	Lisbon	28.0	NP	NP	NP
	141.7	Lisbon	25.9	28	46	18
B-1004	144.0	Lisbon	44.6	38	59	21
	153.5	Lisbon	30.1	27	43	16
	163.5	Lisbon	25.1	22	31	9
	177.0	Lisbon	20.8	22	31	9
	188.5	Lisbon	29.0	27	34	7
	198.5	Lisbon	26.2	21	31	10
B-1006	123.5	Lisbon	53.7	43	99	56
B-1010	98.5	Lisbon	49.9	36	94	58
		Range:	**13.5-67.4**	NP-51	NP-99	NP-58
		Average:	**35.3**	29	51	22
		Median:	**29.0**	27	43	16
		Use:	**N/A**	25	51	26
NOTE:	Bechtel (1974b) reports 2≤PI≤70 for the Lisbon Formation with average value of 25%.					

2.5.4-13 SSAR Section 2.5.4.7 states that: "The EPRI curves were extended beyond the 1 percent strain values reported in EPRI (Technical Report (TR)-102293 1993) to 3.3 percent using values provided by Silva (2006)." Provide Silva's values, justification for use of these values, and a detailed description on how the shear modulus and damping curves were extended.

Response:

Even though the EPRI curves were extended beyond the 1 percent strain values reported in EPRI (1993), the maximum strains calculated during the site amplification analyses remained below 1 percent. The same applies to the SRS curves. SSAR Sections 2.5.2.5.1.5, 2.5.4.7.2.1 and 2.5.4.7.2.2 will be revised in the next revision of the ESP application along with SSAR Tables 2.5.4-12 and 2.5.4-13, and SSAR Figures 2.5.4-9 through 2.5.4-12 to show the degradation curves (EPRI and SRS) stopping at 1% cyclic shear strain.

2.5.4-14 Since the Blue Bluff Marl has a relatively high variable fines content (24-77 percent) and saturation level (14-67 percent) and since there is also a potentially high ground motion level at the site, justify why liquefaction analyses were not performed.

Response:

The response is divided into two parts. The first examines whether the Blue Bluff Marl (Lisbon Formation) can be considered potentially liquefiable based on material type and age. The second part assumes that the material is potentially liquefiable based on type and age, and looks at field strength and shear wave velocity results to determine if the marl could liquefy based on these results.

Material Type and Age

Type: Soil liquefaction is a process by which loose or medium dense, granular, saturated deposits lose a significant portion of their shear strength due to porewater pressure buildup resulting from cyclic loading, such as that caused by an earthquake. Although the Blue Bluff Marl frequently contains less than 50% of fine material, it has the appearance and characteristics of a calcareous claystone or siltstone and is described as a hard, slightly sandy, cemented, calcareous clay. Its design undrained shear strength is 10 ksf, and its preconsolidation pressure could be as high as 80 ksf, i.e. this is a highly overconsolidated material. Thus, the marl is not loose or medium dense, it performs as hard clay or soft rock, not as a granular material, and although it is saturated since it is below the ground water, its structure is so compressed that it does not have the free water characteristic of a saturated granular material. In short, the Blue Bluff Marl is not a material with liquefaction potential, regardless of the ground motion level.

Age: Youd et al. (2001-see list of references at the end of the response to this RAI), "Youd and Hoose (1977-see list of references at the end of the response to this RAI) and Youd and Perkins (1978-see list of references at the end of the response to this RAI) noted that liquefaction resistance increases markedly with geologic age. Sediments deposited within the past few thousand years are generally much more susceptible to liquefaction than older Holocene sediments; Pleistocene sediments are even more resistant; and pre-Pleistocene sediments are generally immune to liquefaction." Pre-Pleistocene sediments are sediments older than about 2 million years. The Blue Bluff Marl is estimated to be late middle Eocene age, i.e., about 40 to 41 million years old. From the foregoing, even if the marl were potentially liquefiable based on its material characteristics, it should be immune from liquefaction based on its age.

Field Test Results

Youd et al. (2001) describe computation of safety factor against liquefaction based on standard penetration test (SPT) N-values, cone penetrometer test (CPT) tip resistance, and shear wave velocity. For the N-values, tip resistances and shear wave velocities, there are values above which the material is considered non liquefiable, i.e., the computed factor of safety against liquefaction is theoretically infinite, regardless of ground motion level.

N-Values: Youd et al. (2001) indicates that, for a sand with 35% or more fines, soils with a corrected N-value of over about 21 are not liquefiable. To correct the N-value, the value measured in the field is corrected for several factors – the two principal correction factors are the overburden stress and the energy efficiency of the SPT hammer. Based on the overburden pressure at the mid-depth of the marl, and the average energy efficiencies of the hammers used for the borings, the corrected N-value will be 40 to 45% of the measured N-value. Thus, the corrected N-value of 21 translates to an uncorrected N-value of about 50. Of the 58 N-values measured in the marl for the ESP investigation, 5 were below 50, ranging from 27 to 46 (SSAR Table 2.5.4-5). Thus, if the marl were a potentially liquefiable material, a liquefaction analysis would be run for these 5 samples. (An initial analysis of these 5 samples show FS values in excess of the accepted 1.35 value in all cases.)

CPT Values: All of the CPTs that were able to penetrate to the marl met refusal at or near the top of the stratum. Thus, measured tip resistances showed the material to be non liquefiable.

Shear Wave Velocities: The typical shear wave velocities in the marl ranged from 1,400 to 2,650 ft/sec (SSAR Table 2.5.4-6). When corrected for overburden, these values range from about 990 to 1,680 ft/sec. Youd et al. (2001) indicates that, for a sand with 35% or more fines, soils with a corrected shear wave velocity in excess of about 625 ft/sec are non liquefiable.

Conclusions

Based on material type and age, the Blue Bluff Marl does not have the potential to liquefy. If this conclusion is neglected and the SPT, CPT and shear wave velocity measurements in the marl are analyzed to determine factor of safety against liquefaction, the data show that the CPT and shear wave velocities consistently indicate non-liquefiable materials; the SPT data show that over 90% of the N-values indicate non liquefiable materials, and the remaining N-values show satisfactory factors of safety.

References

(Youd et al 2001). Youd, T.L., et al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," *Journal of Geotechnical and Environmental Engineering, ASCE,* Vol. 127, No. 10, 2001.

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(Youd and Perkins 1978). Youd, T.L and Perkins, D.M., "Mapping of Liquefaction-Induced Ground Failure Potential," *Journal of Geotechnical Engineering, ASCE,* Vol. 104, GT 4, 1978.

2.5.4-15 SSAR Section 2.5.4.10.2 states that: "For the large mat foundations that support the major power plant structures, general considerations based on geotechnical experience indicate that total settlement should be limited to 2 in., while differential settlement should be limited to ¾ in. (Peck et al. 1974). For footings that support smaller plant components, the total settlement should be limited to 1 in., while the differential settlement should be limited to ½ in. (Peck et al. 1974)."

a) Provide justification for adopting the Peck et al. (1974) settlement and differential settlement values as guidelines.

b) What are the main causes for exceeding these settlement values at the foundation levels for Units 1 and 2? What kind of measures will be taken to prevent settlements and differential settlements for the new units?

c) Justify the use of an average bearing pressure of 5 ksf for the settlement analyses of compacted fills.

Response:

a) The Peck et al. (1974) total settlement guidelines of 1 inch for column footings and 2 inches formats are widely accepted and used by the geotechnical community. It is known that when foundation settlements are limited to these values, then differential settlements are minimized, and good structural performance follows. On the other hand, if these limiting settlement values are exceeded, it does not necessarily have adverse effects on structures. This is particularly true for large mat foundations, which can efficiently distribute structural loads to the soil. A good example is the large mats for Units 1 and 2 where the calculated settlements of the containment buildings ranged from 4 to 4.3 inches.

b) The settlement guidelines based on Peck et al. (1974) were not used for Units 3 and 4. The approach used for Units 3 and 4 consisted of estimating settlements for power-block structures, and using them as design values. A detailed settlement monitoring program was established, and monitored settlements were compared to the design values. The “VEGP Report on Settlement” prepared by Bechtel in 1986 provides comparisons of measured versus calculated settlements, and concludes that the calculated or design values were not exceeded. Reanalysis and/or corrective measures would be employed in the event that monitored settlements exceeded the design values. This same approach will be followed for Units 3 and 4, and Sections 2.5.4.10.2 and 2.5.4.11 will be revised accordingly in the next revision to the ESP application.

c) The value of 5 ksf was used for illustrative purposes as no design value was available during the ESP. The calculation will be revised using design values during the COL.

2.5.4-16 SSAR Section 2.5.4.10 provides two general scenarios for bearing capacity and settlement analyses. However, in order to meet the requirements of 10 CFR Parts 50 and 100, the stability of all planned safety-related facilities should be analyzed including bearing capacity, rebound, settlement, and differential settlements under deadloads of fills and plant facilities, as well as lateral loading conditions. Please provide justification for not addressing the above information for each planned safety-related structure.

Response:

This information will be provided as part of the COL application, when more details regarding the bearing capacity, rebound, settlement, and differential settlements etc., are available. This level of detail is not available during the ESP application process.

2.5.4-17 SSAR Section 2.5.4.7 states that EPRI Procedure TR-102293 was used to develop the shear modulus and damping curves based on the site shear wave velocities and plasticity index values. Please provide a complete description, including sample calculations, to show how the shear modulus and damping curves were developed and how uncertainties in the site parameters were incorporated into their development.

Response:

The shear wave velocity is used to calculate the low strain dynamic shear modulus (G_{max}) only, according to Equation (9) shown in our response to RAI No. 2.5.4-7. The shear modulus reduction curves show the reduction in G_{max} as the shear strain increases during a seismic event, i.e., the EPRI (1993) curves simply show the ratio G/G_{max} versus shear strain, regardless of the initial value of G_{max}.

The shear modulus reduction and damping ratio curves for cohesionless materials were based on confining pressure at depth, or simply depth. The shear modulus reduction and damping ratio curve for the Lisbon Formation was based on the plasticity index (PI).

The next paragraphs contain a summary of how the shear modulus reduction and damping ratio curves shown on SSAR Figures 2.5.4-9 and 2.5.4-11 were derived from the EPRI (1993) curves.

It is noted that shear modulus reduction and damping ratio curves will be obtained using undisturbed samples collected during the COL subsurface investigation.

Compacted Fill

The compacted backfill extends from the ground surface to a depth of 86 ft. The three EPRI (1993) curves for shear modulus reduction at shallow depths (0-20ft, 20-50ft, and 50-120ft) presented on Figure 7.A-18 of EPRI (1993) were used.

The three EPRI (1993) curves for damping ratio at shallow depths (0-20ft, 20-50ft, and 50-120ft) presented on Figure 7.A-19 of EPRI (1993) were used.

Lisbon Formation (Blue Bluff Marl)

The shear modulus reduction values were obtained by using PI = 25% and interpolating between the curves shown on Figure 7.A-16 of EPRI (1993) for PI = 10 and PI = 30.

The damping ratio values were obtained by using PI = 25% and interpolating between the curves shown on Figure 7.A-17 of EPRI (1993) for PI = 10 and PI = 30.

Lower Sand (Still Branch Formation)

The Still Branch Formation extends from a depth of 146 ft to a depth of 213 ft. The curve for depths of 120 ft to 250 ft shown on Figure 7.A-18 of EPRI (1993) was used to obtain the modulus reduction values.

The curve for 120 ft to 250ft shown on Figure 7.A-19 of EPRI (1993) was used to obtain the damping ratio values.

Lower Sand (Congaree Formation)

The Congaree Formation extends from a depth of 213 ft to a depth of 328 ft. The curve for depths of 250 ft to 500 ft shown on Figure 7.A-18 of EPRI (1993) was used to obtain the modulus reduction values.

The curve for 250 ft to 500 ft shown on Figure 7.A-19 of EPRI (1993) was used to obtain the damping ratio values.

Lower Sand (Snapp Formation)

The Snapp Formation extends from a depth of 328 ft to a depth of 435 ft. The curve for depths of 250 ft to 500 ft shown on Figure 7.A-18 of EPRI (1993) was used to obtain the modulus reduction values.

The curve for 250 ft to 500 ft shown on Figure 7.A-19 of EPRI (1993) was used to obtain the damping ratio values.

Lower Sand (Black Mingo Formation)

The Black Mingo Formation extends from a depth of 435 ft to a depth of 474 ft. The curve for depths of 250 ft to 500 ft shown on Figure 7.A-18 of EPRI (1993) was again used to obtain the modulus reduction values.

The curve for 250 ft to 500 ft shown on Figure 7.A-19 of EPRI (1993) was used to obtain the damping ratio values.

Deep Sands

The 500-1,000 ft curve shown on Figure 7.A-18 of EPRI (1993) was used to obtain the modulus reduction values for soils below 500 ft depth, since the soil extends to just below 1,000 ft depth.

The curve for 500 ft to 1000 ft shown on Figure 7.A-19 of EPRI (1993) was used to obtain the damping ratio values for the deep sands below 500 ft.

Rock

All rock is assumed to behave elastically and therefore will not degrade with strain. In other words, G/G_{max} = 1 for all shear strain levels in rock.

For randomization purposes, the shear modulus reduction curves were extended beyond the 1 percent strain values reported in EPRI (1993) to 3.3 percent using values provided by Silva (2006). See our response to RAI 2.5.4-13 for a discussion of Silva (2006).

Based on inspection of SSAR Figure 2.5.4-11, the low strain damping ratio of soils is on the order of 0.5 percent, which generally increases to 0.6 percent to 2 percent for strain compatible conditions. Rock, which would be expected to have lower damping than soil, was assumed to behave as a linearly elastic material with 1 percent damping.

Uncertainties in the site parameters were incorporated during the randomization process. Each layer in the profile is associated with the appropriate base case shear modulus reduction curve and damping ratio curve shown on SSAR Figures 2.5.4-9 through 2.5.4-12. The shear modulus reduction and damping ratios were randomized at one strain level using log-normal distributions with median values given by the values in the corresponding base-case curves and logarithmic standard deviations taken from the statistical summaries obtained by Costantino (1997) for natural soils. For the engineered backfill, these standard deviations were reduced by 1/3 to account for a more homogeneous soil mass. The shear modulus reduction and damping ratios at other strains are generated from the randomized values obtained above, using a hyperbolic parametric form. This approach produces realistic curves with logarithmic standard deviations that approximate the Costantino (1997) values over a wide range of strains. The normal random variables associated with the log-normal shear modulus reduction and damping ratios are taken as having a correlation coefficient of -0.75.

2.5.4-18 SSAR Section 2.5.4.10.1 provides a brief description of the allowable bearing capacity value, which is based on Terzaghi's bearing capacity equations modified by Vesic (1975). Please provide a more detailed description of how the allowable bearing capacity value was obtained that includes the actual calculations.

Response:

Following is an explanation of the bearing capacity calculation, a description of the subsurface section used in the bearing capacity calculations, and a typical bearing capacity calculation.

Methodology - The net bearing capacity of shallow foundations, i.e., the bearing capacity beyond the existing overburden pressure, will be calculated according to the following equation (Equation 3.11, p. 128 of Vesic 1975):

$$q_o = c \cdot N_c \cdot \zeta_c + q \cdot (N_q - 1) \cdot \zeta_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma \cdot \zeta_\gamma \quad (1)$$

where: q_o = net ultimate bearing pressure (ksf)

c = soil cohesion (ksf)

q	=	effective overburden pressure at bottom of foundation level (ksf)
γ	=	unit weight of soil (kcf) – per Equation 2
B	=	foundation width (ft)
L	=	foundation length (ft) – not included in Equation (1)
N_q, N_q, N_q	=	bearing capacity factor from Table 3.1, p. 127 of Vesic (1975)
ζ_c, ζ_q, ζ_γ	=	foundation shape factors from Table 3.2, p. 129 of Vesic (1975)

The unit weight of soil to be used in Equation 1 is given by

$\gamma = \gamma_{sub} + (z_w/B)\cdot(\gamma_m - \gamma_{sub})$ (Equation 3.35, p. 138 of Vesic 1975) (2)

where: γ_m = moist unit weight of soil above the water table (kcf)

γ_{sub} = submerged unit weight of soil below the water table (kcf)

z_w = vertical distance from bottom of foundation to the ground water table (ft)

NOTE: Use $z_w = 0$ in Equation (2), if the ground water table is above the bottom of foundation.
If $z_w > B$, then use $z_w = B$ in Equation (2).

The allowable bearing capacity (qa), not considering settlement, is given by

$q_a = q_o/FS$ (3)

where: FS = factor of safety against bearing failure (FS=3 for static loading, and FS=2 for dynamic loading)

It is noted that Equation (1) applies to a homogeneous profile where the soil thickness is much larger than the width of the foundation. If the foundation is placed on a "strong" layer (compacted granular structural fill) that is underlain by a "weaker" layer (Lisbon Formation or Blue Bluff Marl that acts as a cohesive material), such as is the case for all structures to be analyzed here, the lower "weaker" layer can affect the bearing capacity.

Vesic (1975) gives the following equation (Equation 3.41, p. 142 of Vesic 1975) for the case where the upper layer is cohesionless with $25^o \leq \phi \leq 50^o$, which applies to the compacted structural fill:

$q_o = q_o'' \cdot \exp\{0.67\cdot[1+(B/L)]\cdot(H/B)\}$ (4)

where:

q_o'' = ultimate bearing pressure per Equation (1) of foundation sitting on the surface of the Lisbon Formation (ksf)

H = thickness of compacted structural fill between the bottom of the foundation and the top of the Lisbon Formation (ft)

The assumption is still made that q_o'' is calculated for a Lisbon Formation layer that has thickness much larger than the width of the foundation. This assumption will result in conservative q_o'' bearing capacity values in the case of the foundations for the containment structures, where the thickness of the Lisbon Formation is 63 ft, and the Lower Sand Stratum below is as strong as or stronger than the Lisbon Formation.

It is noted that the bearing capacity value, q_o, obtained from Equation (4) cannot exceed the value obtained from Equation (1).

Subsurface Section Used in the Bearing Capacity Calculations – The subsurface section and engineering properties used in the bearing capacity calculations is summarized in the table below.

Layer	Shear Strength		Unit Weight (kcf)	
	c (ksf)	ϕ (degrees)	Moist (γ_m)	Saturated (γ_{sat})
Compacted Structural Backfill (0 to 83 ft)	0	34	0.120	0.130
Lisbon Formation (83 to 146 ft)	10	0	N/A	0.115
Lower Sand Stratum (Below 146 ft)	0	34	N/A	0.115
NOTE: The ground water table was taken at a depth of 55 ft below finish grade, i.e., at about El. 165 ft.				

Typical Bearing Capacity Calculation – We will look at the bearing capacity of square foundations placed at a depth D_f = 4 ft below finish grade. Consider square foundations with width: B = 5 ft and B = 80 ft. If the Lisbon Formation summarized on the profile in the above table is ignored, and the foundations are placed at a depth D_f = 4 ft below grade on the compacted structural fill with c = 0.0 ksf and $\phi = 34^o$, then:

$N_c = 42.16$ $N_q = 29.44$ $N_\gamma = 41.06$

$\zeta_q = 1+(29.44/42.16) = 1.70$ $\zeta_q = 1+\tan 34° = 1.67$ $\zeta_\gamma = 0.60$

$q = 4 \cdot 0.130 = 0.52$ ksf

Values of γ according to Equation (2) are as follows:

B (ft)	D_f (ft)	z_w (ft)	γ (kcf)
5	4	$z_w = 55-4 = 51$ $z_w > B$ Use $z_w = B = 5$	$\gamma = (0.130-0.0624)+(5/5)\cdot(0.120-(0.130-0.0624)) = 0.120$ kcf
80	4	$z_w = 55-4 = 51$ $z_w < B$ Use $z_w = 51$	$\gamma = (0.130-0.0624)+(51/80)\cdot(0.120-(0.130-0.0624)) = 0.101$ kcf

The ultimate bearing pressure calculated according to Equation 1, ignoring the Lisbon Formation, is

$q_o' = 0.0\cdot 42.16\cdot 1.70+0.52\cdot(29.44-1)\cdot 1.67+0.5\cdot 0.130\cdot 5\cdot 41.06\cdot 0.60 \cong 0+24.7+8.0 = 32.7$ ksf, for B = 5 ft

$q_o' = 0.0\cdot 42.16\cdot 1.70+0.52\cdot(29.44-1)\cdot 1.67+0.5\cdot 0.101\cdot 80\cdot 41.06\cdot 0.60 \cong 0+24.7+99.5 = 124.2$ ksf, for B = 80 ft

The effect of the Lisbon Formation on this calculated allowable bearing pressure will be considered next.

For square foundations of width B = 5 ft or B = 80 ft placed at the top of the Lisbon Formation with c = 10 ksf and $\phi = 0^o$:

$N_c = 5.14$ $N_q = 1.0$ $N_\gamma = 0.0$
$\zeta_c = 1+(1.0/5.14) = 1.19$ $\zeta_q = 1+\tan 0^o = 1.0$ $\zeta_\gamma = 0.60$
$q = 55 \cdot 0.120+(83-55)\cdot(0.130-0.0624) = 8.49$ ksf

The ultimate bearing pressure of the foundation resting on top of the Lisbon Formation calculated according to Equation 1 is:

q_o" $= 10 \cdot 5.14 \cdot 1.19+8.49 \cdot (1.0-1) \cdot 1 = 61.1 + 0 = 61.1$ ksf

and the ultimate bearing pressure calculated according to Equation 4 for the foundation resting at a depth D_f = 4 ft, i.e., at a vertical distance H=83-4=79 ft above the top of the Lisbon Formation is

$q_o = 61.1 \cdot \exp\{0.67 \cdot [1+(5/5)] \cdot (79/5)\} > 32.7$ ksf, use 32.7 ksf for B = 5 ft

$q_o = 61.1 \cdot \exp\{0.67 \cdot [1+(80/80)] \cdot (79/80)\} > 124.2$ ksf, use 124.2 ksf for B = 80 ft

The results above indicate that the bearing capacity is not affected by the presence of the Lisbon Formation for square foundations with B ≤ 80 ft,. The static allowable bearing pressures can be taken as

$(q_a)_{static} = 32.7/3 = 10.9$ ksf for B = 5 ft

$(q_a)_{static} = 124.2/3 = 41.4$ ksf for B = 80 ft

These allowable bearing pressures do not take into consideration foundation settlements.

2.5.4-19 SSAR Section 2.5.4.11 does not provide the complete design criteria or actual design methods that will be employed in the geotechnical review. Please provide justification for not providing the above information.

SSAR Section 2.5.4.11 provides two factors of safety for slope stability with references to Section 2.5.5.2. Neither of these factors of safety is listed in Section 2.5.5. Please explain their omission.

Response:

This information will be provided as part of the COL application, when the complete design criteria and actual design methods are available. This level of detail is not available during the ESP application process.

Section 2.5.5 will be revised in the next revision of the ESP application to include the factors of safety for slope stability.

2.5.4-20 SSAR Section 2.5.4 does not provide the relationship of foundations to the underlying materials in the form of plot plans and profiles. In addition, foundation stability with respect to groundwater conditions is not described, and detailed dewatering plans are also missing. Please provide justification for not providing the above information.

Response:

This information will be provided as part of the COL application when more details regarding the foundations' interaction with the site including detailed dewatering plans are available. This level of detail is not available during the ESP application process.

Southern Nuclear Operating Company

AR-07-0801

Enclosure 2

Geologic Data Files

on

Compact Disc (CD)

NOTE: The following files are contained on Enclosure 2 CD:

1. 252-16_mmax.xls (Provided electronically as requested in RAI 2.5.2-16)
2. 252-16_geom.txt (Provided electronically as requested in RAI 2.5.2-16)
3. 252-16_mmax.pdf (Copy of number "1" above formatted for ADAMS)
4. 252-16_geom.pdf (Copy of number "2" above formatted for ADAMS)