Ronald B. Clary
General Manager
New Nuclear Deployment

SCE&G®
A SCANA COMPANY

April 29, 2009
NND-09-0064

U.S. Nuclear Regulatory Commission
Document Control Desk
Washington, DC 20555

ATTN: Document Control Desk

Subject: Virgil C. Summer Nuclear Station (VCSNS) Units 2 and 3 Combined License Application (COLA) - Docket Numbers 52-027 and 52-028 Supplement 1 to the Response to NRC Request for Additional Information (RAI) Letter No. 031

Reference: 1) Letter from Ravindra G. Joshi (NRC) to Alfred M. Paglia (SCE&G), Request for Additional Information Letter No. 031 Related to SRP Section 2.5.4 for the Virgil C. Summer Nuclear Station Units 2 and 3 Combined License Application, dated February 10, 2009.
2) Letter from Ronald B. Clary (SCE&G) to the Document Control Desk, Response to NRC Request for Additional Information (RAI) Letter No. 031, dated March 12, 2009.

The enclosure to this letter provides the South Carolina Electric & Gas Company (SCE&G) supplemental response to the following RAI items included in the above referenced NRC letter: 02.05.04-3, 4, 6, 8, 9 and 11. The enclosure also identifies any associated changes that will be incorporated in a future revision of the VCSNS Units 2 and 3 COLA.

Should you have any questions, please contact Mr. Al Paglia by telephone at (803) 345-4191, or by email at apaglia@scana.com.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on this 29th day of April, 2009.

Sincerely,

Ronald B Clary

Ronald B. Clary
General Manager
New Nuclear Deployment

D083
NRO

AMM/RBC/am

Enclosure

c (w/o enclosure):
Luis A. Reyes
Ravindra G. Joshi
Chandu Patel
John Zeiler
Stephen A. Byrne
Ronald B. Clary
Bill McCall
Kenneth J. Browne
Randolph R. Mahan
Kathryn M. Sutton
Amy M. Monroe
Courtney W. Smyth
John J. DeBlasio
Grayson Young

c (with enclosure):
FileNet

NRC RAI Letter No. 031 Dated February 19, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-3

FSAR Subsection 2.5.4.4.4.1 (pgs 2.5.4-18 and 19) discusses the results of shear and compression wave velocity tests in the rock foundation layer. FSAR Figure 2.5.4-224 depicts the shear wave velocity measurements obtained at Units 2 and 3 from the P-S suspension logging method. The right side of the figure represents the variability in the shear wave velocities measured at different locations in the Unit 3 power block area. Variability in shear wave velocity between El. 310 to El. 355 (foundation grade) does not appear to meet two uniformity requirements cited in the AP1000 DCD. Specifically, in the AP 1000 DCD Tier 1, Table 5.0-1 states that, for a layer with a low strain shear wave velocity greater than or equal to 2500 feet per second, the layer should have approximately uniform thickness, a dip not greater than 20 degrees, and less than 20 percent variation in the shear wave velocity from the average velocity in any layer. Also, in AP1000 Tier 2, Subsection 2.5.4.5.3.1, the DCD states that the key attribute for acceptability of the site for an AP1000 is the bearing pressure on the underside of the basemat, such that a site having local soft or hard spots within a layer, or layers, will not meet the criteria for a uniform site.

In order for the staff to assess the adequacy of the site, please explain how the variations in shear wave velocities in the upper elevations of Layer V meet the AP1000 site uniformity requirements.

VCSNS RESPONSE:

In order to provide responses to the questions, it is beneficial to summarize the proposed excavation and backfilling program that will be used for the nuclear island foundations.

The base of each nuclear island foundation will be located at about El. 360 ft. FSAR Subsection 2.5.4.5.2 indicates that the top of sound rock is at an average of about El. 355 ft at the locations of each nuclear island (although this is conservatively interpreted for Unit 3 where top of sound rock is slightly higher). However, as would be expected in in-situ bedrock, there is some variation across the foundation footprints of both units, with sound rock extending above El. 355 ft (and above the foundation base at El. 360 ft) at some locations and more weathered rock extending below El. 355 ft at other locations.

Sound rock can be defined in different ways. FSAR Section 2.5.4.2.2.1 defines the Layer V sound rock as having an RQD of at least 50% but typically exceeding 70%. However, when excavating the site, RQD is not a practical measure to use. A more practical approach is to define sound rock as rock that cannot be ripped by a large dozer or excavated with a large trackhoe. FSAR Section 2.5.4.4.4.1 defines sound rock as rock that is non-rippable with a very large ripper. For a Caterpillar Single Shank No. 11 Ripper on a D11N dozer, the upper limit for an igneous or metamorphic rock corresponds to a seismic (compression) wave velocity of about 11,000 ft/sec (equivalent to a shear wave velocity of about 6,500 ft/sec). Comparison of the depth to sound rock using the 70% RQD criterion and the 11,000 ft/sec compression wave velocity criterion shows agreement to within an average of about 5 ft depth in Unit 2 and 3 ft depth in Unit 3. (These average values indicate the 70% RQD criterion is met at a higher elevation than the compression wave velocity criterion.) As shown on Figure 2.5.4-224, the shear wave velocity of the sound rock increases rapidly with depth to above 9,000 fps.

After excavation using a very large ripper (and/or trackhoe) at the nuclear island footprint, there will still be some portions of sound (non-rippable) rock left above foundation base level. This will be removed down to foundation base level using controlled blasting as summarized in FSAR Subsection 2.5.4.5.2.2. FSAR Figure 2.5.4-202 (top of sound rock contours) indicates that, for Unit 3, the top of sound rock is nearly level across the nuclear island footprint, and does not extend much below El. 360 ft. Individual boring logs show the minimum elevation of top of sound rock is about El. 355 ft. For Unit 2, the lowest elevation of the sound rock is about El. 343 ft or about 17 ft below the base of the foundation. As noted in FSAR Section 2.5.4.10.1.1, where needed, the concrete placed between the top of sound rock and the base of the nuclear island will have a strength of about 5,000 psi. One reason for selecting this strength is that, according to Boone (2005), concrete with this strength has a shear wave velocity of around 9,000 fps, i.e., close to that of the in-situ rock.

To summarize, beneath the approximately 160 ft x 255 ft nuclear island foundation, after excavation, the top of sound rock will range from foundation level to about 17 ft below foundation level. Concrete with a shear wave velocity of around 9,000 fps will extend from the top of sound rock to the bottom of the foundation to provide a uniform bearing surface. At Unit 3, the maximum thickness of concrete will be about 5 ft. Where sound rock exists above foundation level, it will be removed by controlled blasting. Sound rock is defined as being non-rippable with very large equipment. The shear wave velocity of the material in this state is nominally 6,500 fps, but the data show that once this level of soundness/lack of weathering is encountered, the shear wave velocity increases rapidly with depth.

In response to this specific RAI:

- The DCD indicates the bearing layer should have uniform thickness. Layer V (the bearing layer) is bedrock and extends from an average elevation of about El. 355 ft to below the bottom of the deepest boring at about El. 75 ft.

- The DCD states that the dip of the bearing layer should not be more than 20 percent. Layer V is a massive pluton of igneous rock that has no dip. As noted above, the excavated surface of the sound rock has almost no variation beneath the Unit 3 nuclear island. Up to 17 ft of weathered material will be removed below Unit 2, and will be backfilled with high strength concrete. The dip limit is not an issue at this site.

- The DCD states that there should be less than 20 percent variation in the shear wave velocity from the average velocity in any layer. The only materials beneath the nuclear island foundation will be the sound rock of Layer V and, where required, a layer of high strength concrete with a shear wave velocity close to the average velocity of the sound rock. Figure 2.5.4-224(b) shows very little variation in the Layer V shear wave velocity below Unit 3 foundation level. Figure 2.5.4-224(a) shows more variation in the Layer V shear wave velocity below Unit 2 foundation level. As discussed above, up to 17 ft of the lower velocity material will be removed and replaced with high shear wave velocity concrete. Some zones of lower velocity rock will remain, but in almost all cases, these zones have a shear wave velocity of over 7,500 fps, near the upper limits of the shear wave velocity levels for rock considered in the DCD.

- In regards to soft layers beneath the nuclear island foundation, none were encountered.

References:

Boone, S.D. (2005). A Comparison Between the Compressive Strength and the Dynamic Properties of Concrete as a Function of Time, MS Thesis, University of Tennessee, Knoxville.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

No COLA changes have been identified as a result of this response.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 031 Dated February 19, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-4

FSAR Subsection 2.5.4.5.3.1 (pg 2.5.4-22) specifies that structural fill is either concrete or granular fill and that concrete fill is used mainly at the bottom of the excavations for the seismic Category I nuclear island foundation.

In order for the staff to assess adequacy of the structural fill, please discuss the following aspects in regard to the concrete fill material:

(a) Target properties of the concrete fill materials and expected uniformity of these properties.

(b) Relationships between the concrete fill and the compacted structural fill.

(c) Extent of concrete fill.

(d) Governing design standard for the concrete fill.

VCSNS RESPONSE:

(a) As noted in FSAR Subsection 2.5.4.10.1.1, where needed, the concrete fill placed between the top of sound rock and the base of the nuclear island will have a strength of about 5 ksi. One reason for selecting this strength is that, according to Boone (2005), concrete with this strength has a shear wave velocity of around 9,000 fps, i.e., close to that of the in-situ rock. As noted in FSAR Section 2.5.4.10.1.1, this concrete has an allowable bearing capacity of 144 ksf, less than the Layer V rock and more than the Layer IV, and much greater than the maximum bearing pressure (static and dynamic) from the nuclear island.

Because of its high strength, this concrete will have a high cement content and thus a relatively high heat of hydration. This high heat of hydration generated will require the concrete to be placed in relatively thin lifts to avoid cracking. The concrete will likely also have a high flyash content to minimize the heat of hydration.

The surface of the concrete fill will be left in a roughened state prior to pouring the mat foundation for the nuclear island to ensure that a coefficient of sliding of at least 0.7 is achieved between the concrete surfaces.

(b) and (c) Five (5) ksi concrete fill will be used only beneath the nuclear island, where needed to establish the foundation bearing level. At other locations, fill concrete with a strength of approximately 3 ksi may be used to smooth or level the rock surface before placing structural fill. Compacted granular structural fill will be used, where needed, beneath the other major power block structures. The relative concrete and structural fill locations are shown on FSAR Figures 2.5.4-220 through 2.5.4-223.

Based on the top of sound rock profiles (FSAR Figure 2.5.4-202) and the boring logs, the Unit 2 concrete fill will have a maximum thickness of about 17 ft while at Unit 3, the maximum thickness of concrete fill will be about 5 ft. At both units, the concrete fill will extend a minimum of 5 ft out from beyond the footprint of the nuclear island. At Unit 2, a bench will be provided where needed. Structural fill will be placed above and adjacent to the concrete fill at both units.

(d) Because of the high quality of concrete required in this situation, the mix design for the Unit 2 and Unit 3 concrete fill will be governed by ACI 318, Building Code Requirements for Structural Concrete.

References:

Boone, S.D. (2005). A Comparison Between the Compressive Strength and the Dynamic Properties of Concrete as a Function of Time, MS Thesis, University of Tennessee, Knoxville.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

The following paragraph will be added to the end of FSAR Subsection 2.5.4.2.5.1.

The sliding coefficient is tangent δ, where δ is the friction angle between the rock and the material it is bearing against, i.e., concrete in this case. Based on Reference 234, tangent δ = 0.7 was adopted for Layers III, IV and V rock. Where concrete fill is placed on top of the rock beneath the nuclear island (Subsection 2.5.4.5.3.1), the surface of the concrete fill will be left in a roughened state prior to pouring the mat foundation for the nuclear island to ensure that a coefficient of sliding of at least 0.7 is achieved between the concrete surfaces.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 031 Dated February 19, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-6

FSAR Subsection 2.5.4.5.3.2 (pg 2.5.4-23) describes common fill and states that such fill, from the excavated residual and saprolitic soils of layers I and II, will be compacted to 90% of the maximum dry density as determined by ASTM D 1557. This material was determined to be unacceptable for structure backfill and is potentially subject to liquefaction. Also, FSAR Figures 2.5.4-221 and 223 indicate that common fill is in close proximity to the turbine and radwaste buildings.

In order for the staff to assess the compaction criteria for common fill, please provide justification for selecting 90% as the minimum required compaction. Please also provide justification or analyses to ensure that the use of 90% in close proximity to structures and adjacent to structural backfill will not adversely affect soil density to the point that the minimum shear wave velocity falls below minimum requirements.

VCSNS RESPONSE:

Although FSAR Figures 2.5.4-221 and 223 indicate that common fill is in close proximity to the turbine and radwaste buildings, they also show that the common fill is outside the zone of loading influence of these buildings. This is demonstrated by considering that the effects of the loading produced by the bulb of pressure beneath a structure can be approximated by a load distribution of 2-vertical to 1-horizontal beneath the foundation (Perloff, 1975). The structural fill beneath the turbine, annex, and radwaste buildings extends down to sound rock and approximately 50 feet horizontally beyond the foundation footprint of these buildings. The maximum vertical distance to sound rock is about 60 feet below any major power block building foundation. Thus, any common fill or native soil and weathered rock beyond the structural fill will experience no loading effects. Just as the common fill and native soil and weathered rock are conservatively beyond the zone of loading influence beneath the major power block structures, the shear wave velocity of the common fill and native materials will not impact the shear wave velocity profiles beneath those structures.

The compacted saprolite fill is not considered suitable for use as structural fill beneath major structures because it can have a high mica content, which can possibly lead to excessive settlements under high loading and saturated conditions. The compacted saprolite's liquefaction potential is very small, as described in the response to RAI 02.05.04-8.

While 95% modified Proctor compaction is usually specified for structural fill beneath structures, roadways, etc., 90% modified Proctor compaction is normally specified for backfill surrounding structures and for backfill around buried pipes or conduits. Thus, specifying 90% modified Proctor compaction is considered adequate for common fill material that will not be supporting significant applied loads. Four modified Proctor tests were performed on saprolite samples taken from onsite test pits (COLA Part 11). The computed total densities for 100% modified Proctor compaction ranged from about 123 to 126 pcf, with an average of about 125 pcf. For 90% modified Proctor compaction, this translates to total densities ranging from about 110 to 114 pcf, with an average of about 112.5 pcf.

References:

Perloff, W.H. (1975). "Pressure Distribution and Settlement," in Foundation Engineering Handbook, Winterkorn, H.F, and Fang, H-Y, Editors, Van Nostrand Reinhold, New York.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

No COLA changes have been identified as a result of this response.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 031 Dated February 19, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-8

In FSAR Section 2.5.4.8 (pg 2.5.4-30), a liquefaction analysis was not performed for the common backfill, justified by the fact that the common backfill was outside any load bearing structures adjacent to the nuclear island. However, the common fill abuts the structural fill.

In order for the staff to assess liquefaction potential, please provide additional information to clarify whether there are areas where the common fill extends from the structural fill to the edge of the plant site. To the extent that the common fill extends to the edge of the plant site, please also address any potential concerns with flow liquefaction or lateral spreading in the common fill. Finally, please provide information to address the possible effects that flow liquefaction failure may have on the structural fill.

VCSNS RESPONSE:

The existing in-situ saprolite, partially weathered rock and moderately weathered rock will all be excavated from beneath the power block areas of Units 2 and 3. As is discussed in the response to RAI 02.05.04-6, the excavation extends far enough beyond the structure locations such that these soils and weathered rock are outside the zone of loading influence of the major power block structures. This is illustrated in Figures 2.5.4-221 and 223. Common fill is used only to backfill the temporary excavations in some areas outside the zone of loading influence. Thus, common fill will always be bounded by structural fill or in-situ soil and weathered rock and will never extend to the edge of the site. Therefore, even if the common fill were to liquefy, it could not flow or lead to lateral spreading. (In FSAR Figures 2.5.4-220 and 2.5.4-222, common fill is shown extending to the edge of the profile. These profiles were drawn in this way since, at the time, a final decision had not been made about having separate excavations for each unit, or one common excavation.)

More detailed excavation and backfill plans, currently being developed, utilize separate excavations for each unit and incorporate temporary retaining walls (soldier piles and lagging) to support the excavation in areas where there is insufficient room for a sloped excavation due to location of the heavy lift crane foundation, module fabrication pads, and construction lay down areas that are required at grade (El 400 feet) before any backfill is placed. The temporary retaining walls will be removed by removing lagging and releasing tiebacks in stages, from bottom to top, as the structural fill is placed. This will result in structural fill and a small amount of common fill being compacted in near-vertical contact with the native soil and weathered rock.

Although cases of liquefaction in hydraulic fills are well documented, there are no known reports of liquefaction in well-compacted engineered fill. The common fill will consist of a silty sand compacted to at least 90% of the maximum dry density obtained from the modified Proctor test (ASTM D 1557). The uppermost 17 ft, or more, of structural and common fill will be above the estimated maximum ground water table and hence cannot liquefy. Because the structural fill will generally extend all the way to plant grade (elev. 400 feet) in areas where a temporary retaining wall is used, there will be only limited areas where common fill extends below the water table. Although the common fill beneath the water table will have lost the structure and aging effects of the in-situ saprolite, it will have gained the effects of significant and consistent compaction with heavy equipment. It is this combination of compaction and relatively limited use that makes it highly unlikely that the common fill will liquefy under the design basis earthquake loading.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

No COLA changes have been identified as a result of this response.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 031 Dated February 19, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-9

FSAR Section 2.5.4.8 (pg 2.5.4-30) states that a liquefaction analysis for the structural backfill adjacent to the nuclear island was not performed, citing that the backfill was dense and well compacted and would not liquefy. Regulatory Guide 1.206 states that "[i]f the foundation materials at the site adjacent to and under safety-related structures are soils or soils that have a potential to become saturated and the water table is above bedrock, the applicant should provide an appropriate state-of-the-art analysis of the potential for liquefaction at the site." The nuclear island depends on the stability of the backfill for resistance to sliding and rocking. FSAR Figure 2.5.4-242 presents values of the predicted peak ground accelerations, and those for saprolite are relatively high.

In order for the staff to adequately evaluate liquefaction potential at the site, please perform a liquefaction analysis to confirm stability of the structural fill.

In addition, for the saprolitic soils, provide further justification for the selected peak ground acceleration values and factors of safety used for the liquefaction analysis.

VCSNS RESPONSE:

SCE&G would like to take exception to the following sentence contained within this RAI: "The nuclear island depends on the stability of the backfill for resistance to sliding and rocking.". This statement is presented as a fact and is not considered accurate for the VCSNS site. Specifically, for a hard rock site which will have concrete fill and the nuclear island structural foundation placed directly on rock, lateral stability from the adjacent backfill (side fill) is not required or credited in accordance with the Westinghouse AP1000 seismic models, provided a coefficient of friction of 0.70 is maintained for the base slab interfaces (refer to DCD 3.8.5.5.3).

Further response to the specific questions of this RAI follow:

The structural fill will be a well-graded medium to coarse sand compacted to at least 95% of the maximum dry density obtained from the modified Proctor test (ASTM D 1557). There are no reported cases of liquefaction of this type of material compacted to this degree.

The value N_{60} = 30 bpf for this material was assumed in the FSAR in order to provide somewhat conservative estimates of settlements for structures on fill. However, the response to RAI 02.05.04-1 demonstrates that this is a conservative value, and that N_{60}

> 40 bpf is a more realistic value. It can be shown that structural fill with N_{60} = 40 bpf will not liquefy under the design basis earthquake at the V.C. Summer site, as follows.

The attached Figure 2 of Youd et al (2001) shows the Cyclic Stress Ratio (CSR) plotted against the corrected SPT blow count. The 3 curved lines represent the cut-off for liquefiable and non-liquefiable soils – each line represents a different fines content. Points on the plot that fall to the left of the line are potentially liquefiable and a calculation must be made to compute the factor of safety against liquefaction based on several factors such as peak ground acceleration and earthquake magnitude.

Points that fall to the right of the line represent soils that are too dense to liquefy. For the selected structural fill, fines contents of 5 and 10 percent were measured, and thus it is reasonable and slightly conservative to use the furthest right of the 3 lines. Now the corrected blow count in Figure 2 is $(N_1)_{60}$ which is N_{60} corrected for overburden pressure. This correction is made to standardize the blow count because measured N-values increase with overburden stress. In the case of the N_{60} = 40 bpf, the depth at which the N_{60} = 40 bpf occurs is not defined. For the compacted fill, the N_{60} = 40 bpf can be considered to be the same as $(N_1)_{60}$ = 40 bpf, since, in reality, the measured N-value in the 40 ft of fill would increase with depth, and the N-value then corrected for overburden stress is 40 bpf.

$(N_1)_{60}$ = 40 bpf is to the right of the selected line in Figure 2 and thus the compacted fill is considered non liquefiable. Note that $(N_1)_{60}$ = 30 bpf also falls to the right of the selected line.

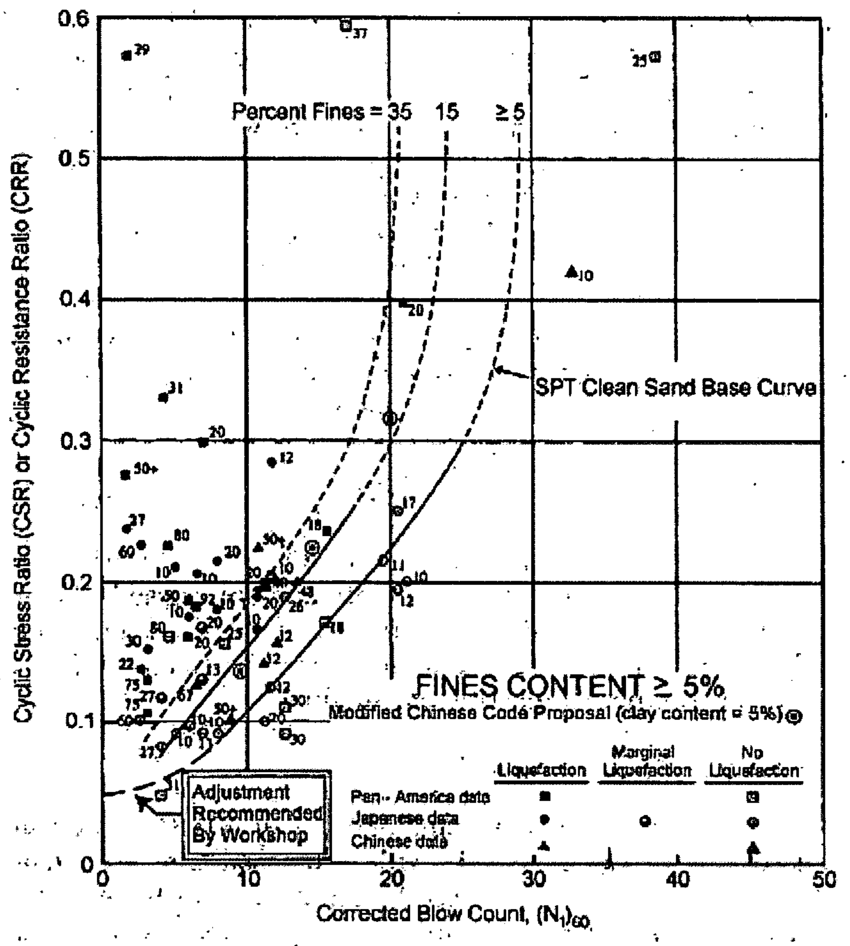


FIG. 2. SPT Clean-Sand Base Curve for Magnitude 7.5 Earthquakes with Data from Liquefaction Case Histories (Modified from Seed et al. 1985)

(from Youd et al, 2001)

References:

Youd, T.L., et al (2001) "Liquefaction Resistance of Soils: Summary of Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," *ASCE Journal of Geotechnical and Environmental Engineering*, Volume 127, No.10, October. (COLA FSAR Reference 252).

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

No COLA changes have been identified as a result of this response.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 031 Dated February 19, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-11

FSAR Figure 2.5.4-226 shows the mean plus and minus 1 standard deviation shear wave velocity profiles for Units 2 and 3 below basemat elevation 355. It is assumed that the data shown in Figure 2.5.4-224 was used to develop Figure 2.5.4-226. It is observed from Figure 2.5.4-224 that two borings, B-201 and B-206, exhibit the lowest shear wave velocities recorded between El. 315 and 355 of the four shear wave velocity profiles shown. A review of the locations of these borings shows that Borings B-201 and B-206 are within the footprint of the nuclear island, but B-207 and B-211 are considerably outside of the NI footprint. Averaging the four shear wave velocity profiles (borings B-201, B206, B207 and B-211) yields a higher mean shear wave velocity profile than if only B-201 and B-206 were used.

In order for the staff to assess shear wave velocity data, please provide additional information to address the effects on the site response if only B-201 and B-206 were used to define the shear wave velocity profile.

VCSNS RESPONSE:

The RAI correctly assumes that the data shown in FSAR Figure 2.5.4-224 were used to develop Figure 2.5.4-226. In these figures, the shear wave velocity (V_s) data are from the four Unit 2 boreholes in which V_s was measured, i.e., B-201 and B-206 within the nuclear island, and B-207 and B-211 outside the nuclear island. These figures are included at the end of this response. In addition, the figures have been redrawn using the V_s data from only the two nuclear island borings, and are included at the end of this response (referred to here as Figures 2.5.4-224RAI and 2.5.4-226RAI).

As noted in the RAI, the average values of V_s between about El. 315 ft and El. 355 ft are less for the two nuclear island profiles than for all four profiles. The questions to be answered when considering these differences are (a) are the differences significant from a site response standpoint, and (b) from geology considerations, should only the V_s values measured beneath the nuclear island be used?

(a) Examining Figure 2.5.4-226RAI, the average V_s below about El. 330 ft is 9,200 fps or more. At velocities of 9,200 fps and more, it is generally recognized that no amplification/attenuation through the rock occurs. Thus, from an amplification/attenuation standpoint, the rock below El. 330 ft does not need to be considered. From about El. 330 ft to the top of sound rock at El. 355 ft, the average V_s is about 8,000 fps. Thus, there is a 25 ft thickness of rock with a V_s

value about 13% less than 9,200 fps. Considering the huge mass of high velocity rock involved (see (b)) and the wavelength of the shear waves (hundreds or thousands of feet, depending on layer thickness considered), 25 ft thickness of 8,000 fps rock (defined as hard rock in the AP1000 DCD) should have minimal amplification effects.

(b) The data and analyses presented in the FSAR incorporate a geologic conceptual model of the site based on the geologic mapping described in FSAR subsections 2.5.1.1 and 2.5.1.2, on the borehole lithologic and geophysical data presented in COLA Part 11 and on geotechnical data discussed in FSAR subsection 2.5.4. As stated in the FSAR (subsection 2.5.1.2.3), the VCSNS Units 2 and 3 site is located within the Winnsboro Pluton. This felsic plutonic complex consists of several phases that include granodiorite, quartz diorite, migmatite, and pegmatite dikes. This coarse to medium grained intrusive complex is dated by Rb-Sr and K-AR methods at about 300 Ma (FSAR Reference 2.5.1-240). Both field mapping and core samples indicate that the Winnsboro plutonic complex intruded amphibolite-grade metamorphic country rock composed primarily of complexly interlayered and folded biotite and hornblende gneiss and amphibolite schist. Based on the lithologies and its areal extent (FSAR Figures 2.5.1-220 and -224), the pluton appears to be a deeply rooted, large rock mass. Based upon these data and the conceptual model of the site as being underlain by a large mass of crystalline rock, four borings were drilled in the vicinity of each nuclear island to better obtain an estimate of the mean V_s and its variance near the reactor foundations.

As noted in the RAI, the average values of V_s between about El. 315 ft and El. 355 ft are less for the two Unit 2 nuclear island profiles than for all four profiles. The decrease in V_s is correlated with a zone of subhorizontal, slightly weathered fractures observed in the rock core and noted on the boring logs as a decrease in RQD values. These subhorizontal fractures at Unit 2 are not observed at Unit 3. The results of the P-S suspension logging survey in the four boreholes near Unit 2 appear to represent conditions of the rock mass beneath that unit. An average of the eight borehole surveys at Units 2 and 3 will likely represent a mean V_s within the granitic pluton and its variance across the Units 2 and 3 site.

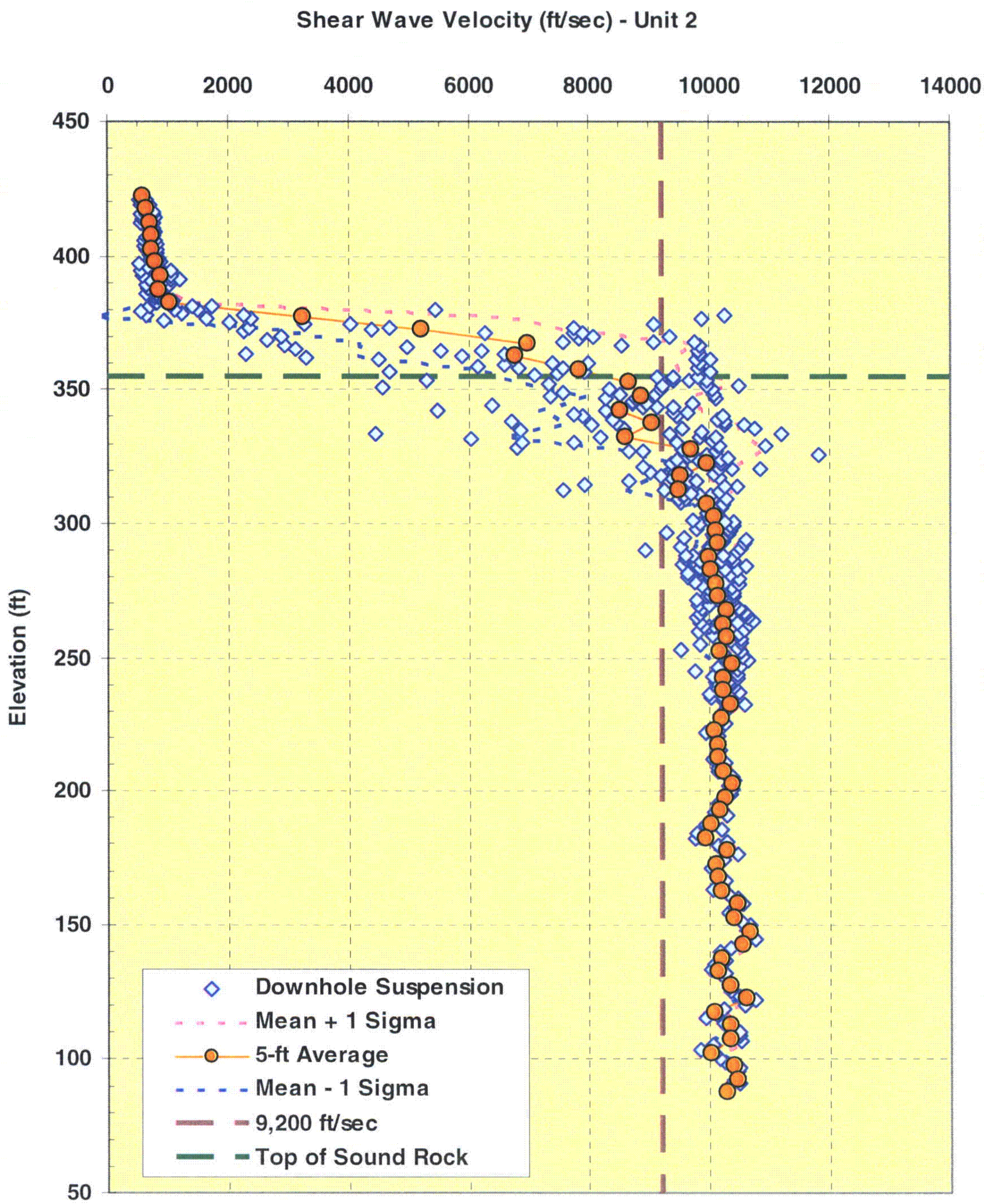
This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

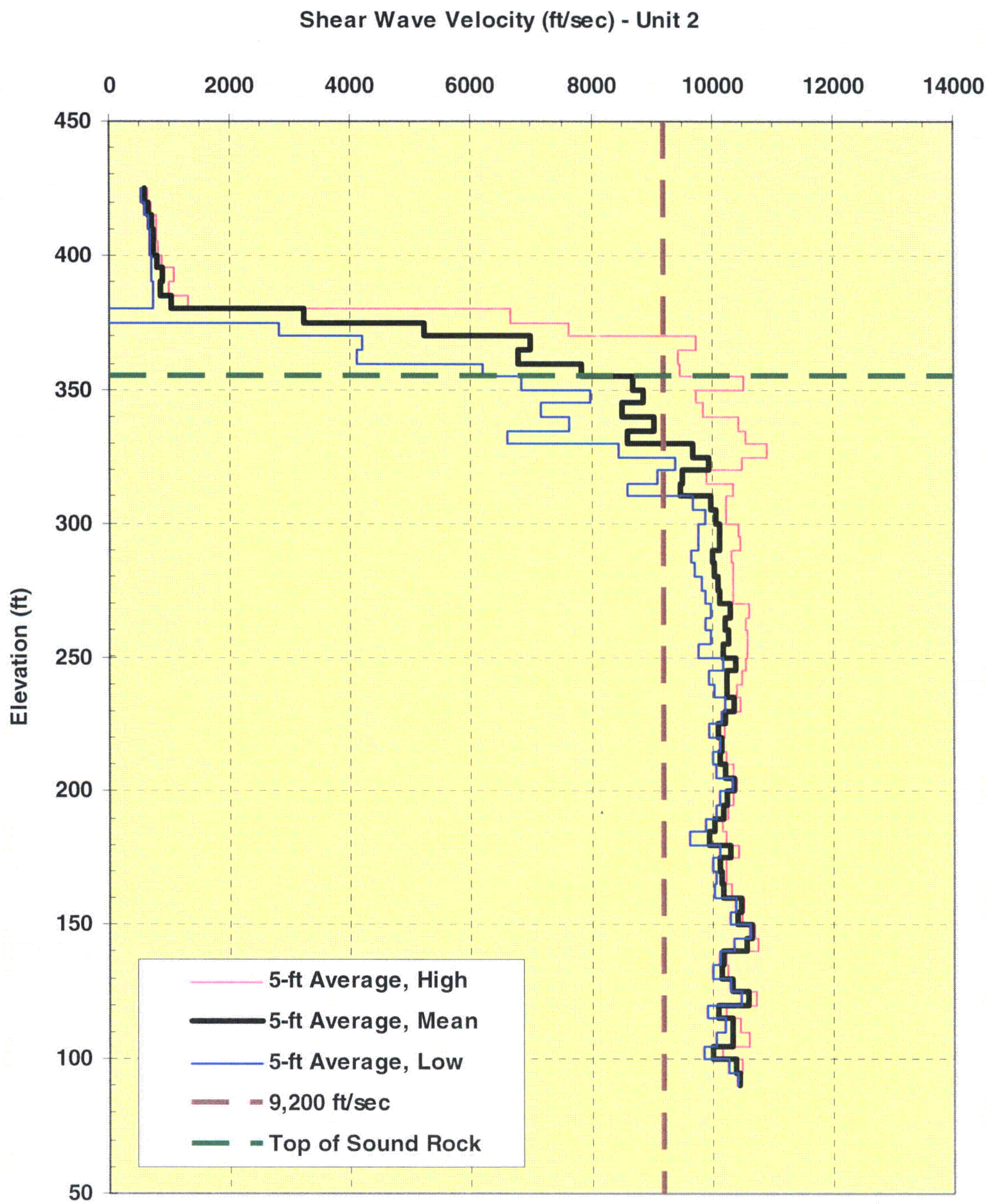
No COLA changes have been identified as a result of this response.

ASSOCIATED ATTACHMENTS:

FSAR Figures 2.5.4-224 and 226;
Figures 2.5.4-224RAI and 226RAI



Measured Unit 2 shear wave velocity in 4 borings (2 beneath nuclear island, 2 outside nuclear island (FSAR Figure 2.5.4-224)



Unit 2 shear wave velocity in 4 borings (2 beneath nuclear island, 2 outside nuclear island, showing average per 5 ft, and +/- 1 standard deviation (FSAR Figure 2.5.4-226)

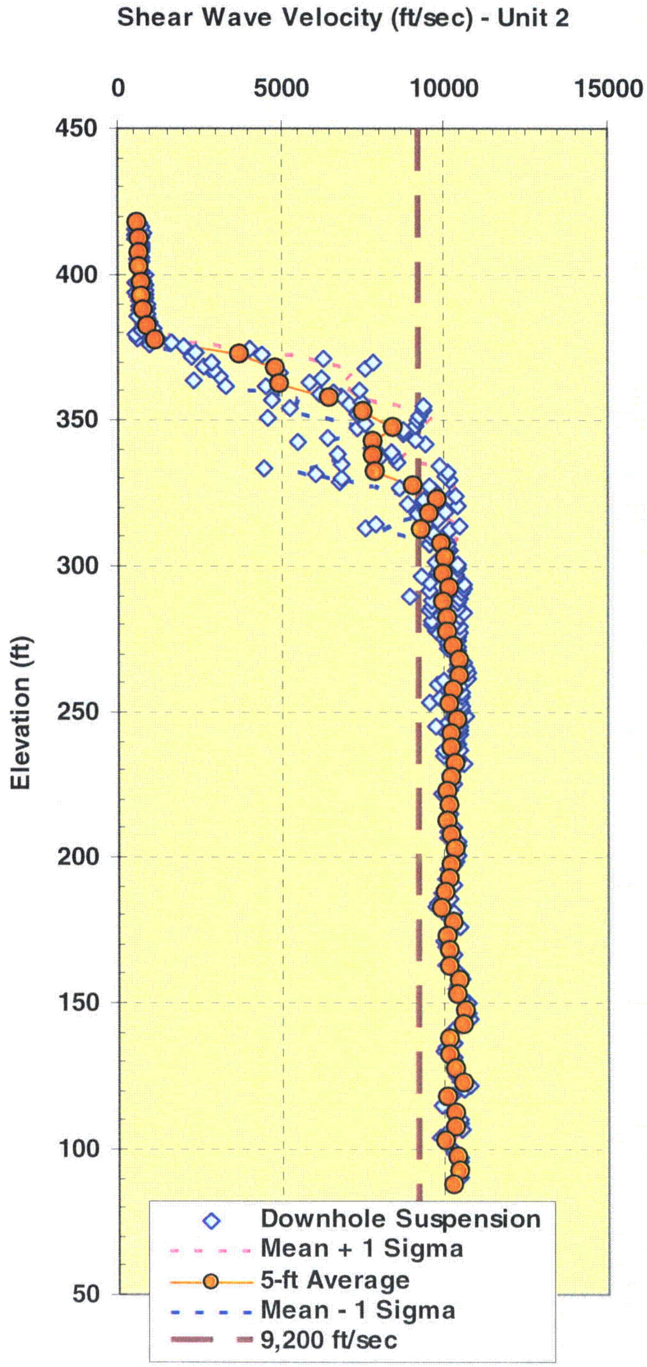


Figure 2.5.4-224RAI - Measured Unit 2 shear wave velocity in 2 borings beneath nuclear island

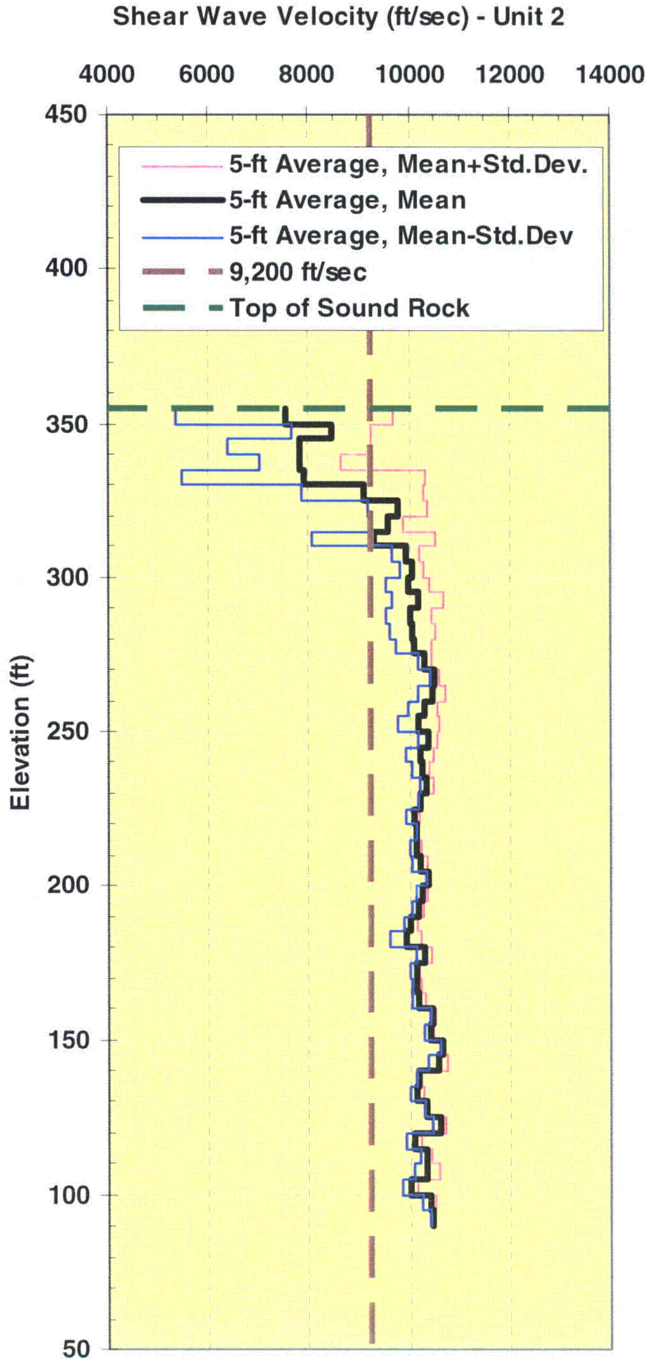


Figure 2.5.4-226RAI - Unit 2 shear wave velocity in 2 borings beneath nuclear island, showing average per 5 ft, and +/- 1 standard deviation