PMTurkeyCOLPEm Resource

From: Comar, Manny
Sent: Thursday, November 01, 2012 5:06 PM
To: TurkeyCOL Resource
Subject: FW: DRAFT RAI Responses FPL Turkey Point 6 & 7 for eRAI 6006 Vibratory Ground Motion - Part 4 of 4
Attachments: Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-3 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-5 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-7 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-9 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-16 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-18 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-2 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-11 (eRAI 6006).pdf

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From: Franzone, Steve [mailto:Steve.Franzone@fpl.com]
Sent: Monday, September 17, 2012 8:46 PM
To: Comar, Manny
Cc: Maher, William; Burski, Raymond
Subject: DRAFT RAI Responses FPL Turkey Point 6 & 7 for eRAI 6006 Vibratory Ground Motion - Part 4 of 4

Manny,
To support a future public meeting, FPL is providing draft revised responses for eRAI 6006 (RAI questions 02.05.04-2, 02.05.04-3, 02.05.04-5, 02.05.04-7, 02.05.04-9, 02.05.04-11, 02.05.04-16, 02.05.04-18) in the attached files.

DRAFT RAI Responses FPL Turkey Point 6 & 7 for eRAI 6006 Vibratory Ground Motion - email 4 of 4 dated 20120917 and no more e-mail transmittals.

If you have any questions, please contact me.
Thanks
Steve Franzone
NNP Licensing Manager - COLA
" Words may show a man's wit, but actions his meaning" ~ Benjamin Franklin
561.694.3209 (office)
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Subject: FW: DRAFT RAI Responses FPL Turkey Point 6 & 7 for eRAI 6006 Vibratory Ground Motion - Part 4 of 4
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Files	**Size**	**Date & Time**
MESSAGE	1536	11/1/2012 5:05:58 PM
Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-3 (eRAI 6006).pdf	252972	
Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-5 (eRAI 6006).pdf	238799	
Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-7 (eRAI 6006).pdf	235330	
Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-9 (eRAI 6006).pdf	486320	
Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-16 (eRAI 6006).pdf	326485	
Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-18 (eRAI 6006).pdf	320939	
Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-2 (eRAI 6006).pdf	328459	
Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-11 (eRAI 6006).pdf	289984	

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NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-3 (eRAI 6006)

FSAR Subsection 2.5.4.2.2 indicates that adjustments are made to the subsurface investigation including changes to the field testing locations and to the types. Also, the applicant made adjustments to depths and frequencies of sampling. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," and Regulatory Guide (RG) 1.132, "Site Investigations for Foundations of Nuclear Power Plants," please provide further information on how and to what extent these adjustments vary from the recommendations provided in RG 1.132 and justify its acceptance for characterizing site subsurface conditions.

FPL RESPONSE:

Regulatory Guide (RG) 1.132 (Revision 2) provides guidance on conducting subsurface explorations including investigation methods, location and depth of exploration points, and in-situ tests. Section C.1 of the guide states that the "…site investigation program will be site dependent; such a program should be tailored to the specific conditions of the site using sound professional judgment." The guide also acknowledges that the program should be flexible and adjusted as the investigation proceeds.

As indicated in FSAR Subsection 2.5.4.2.2.1, the guidance in RG 1.132 was used as the basis for planning the site-specific subsurface investigation for Turkey Point Units 6 & 7. Local geologic information from the subsurface investigation for Turkey Point Units 3 & 4 was used in planning the investigation. In the powerblock, the boring layout included a minimum of one boring or cone penetration test (CPT) per structure and one boring or CPT per 10,000 square feet of structure plan area. Planned drilling methods included mud rotary for geotechnical boreholes. Triple-tube wire-line coring was used to sample rock. Overall, the subsurface investigation program as summarized in FSAR Subsection 2.5.4.2.2 included 88 geotechnical borings, 22 groundwater observation wells, 4 CPTs, and 2 test pits. The supplemental response to RAI 02.05.04-10 compares the numbers and depths of borings, CPTs, etc. performed for Turkey Point Units 6 & 7 with those performed at other planned two unit AP1000 sites (Levy County, Vogtle and V.C. Summer) and concludes that there is no typical amount of exploration — the amount depends on the site conditions. Surface and downhole geophysical surveys (as described in Subsection 2.5.4.4) were also conducted. These surveys included borehole logging (natural gamma, long and short normal resistivity, spontaneous potential, caliper, and deviation), P-S suspension velocity logging, downhole seismic velocity logging, and an integrated surface geophysical survey for evaluation of potential dissolution features.

After the start of field work, adjustments to the subsurface investigation program were made to account for site specific conditions, for both accessibility and subsurface issues. These adjustments included exploration methods, borehole locations, borehole deletions, sampling frequencies, and exploration depths. All changes to the exploration program were documented with either a revision to the specification or through the submission, by the subcontractor, of a Supplier Deviation Disposition Request (SDDR) form. This form, and its use, was provided in the subsurface investigation specification. A summary of adjustments reported with the SDDR process is presented in Table 1. All adjustments made were consistent with RG 1.132 guidance.

During the initial drilling activities, the Tamiami Formation underlying the Fort Thompson Formation was found to be less dense than anticipated from review of previous subsurface data. For this reason, CPT soundings were added to the exploration program with a revision to the specification. The CPTs were advanced into the Tamiami and Peace River Formations to aid in characterizing these materials. Execution of the CPT program necessitated coring the overlying Key Largo and Fort Thompson at these locations. Additionally, one boring, B-701, was extended into the underlying Arcadia Formation to confirm the characterization of this material.

The depth of exploration utilized the guidance in RG 1.132. As stated in FSAR Subsection 2.5.4.2.2.3, the borings beneath the reactors and key structures extended to 250 feet with one boring beneath each reactor to at least 400 feet. The deepest boring (B-701) was extended to a depth of 615.5 feet. The supplemental response to RAI 02.05.04-10 notes that the maximum required boring depth at the site based on RG 1.132 guidelines is about 285 ft.

RG 1.132 Section 4.3.1 states that at least one continuously sampled boring should be used for each safety-related structure. Generally, soil was sampled at 2.5-foot intervals to 15 feet and then 5-foot intervals until rock coring began (when SPT refusal was encountered) or at a depth of about 35 feet. The Key Largo and Fort Thompson Formation limestones were then cored continuously. SPT sampling was conducted at approximately 10-foot intervals in the sands and silts of the underlying Tamiami and Peace River Formations. In the deepest boring, where the Arcadia Formation was encountered, the rock was cored continuously. In summary, the rock formations were sampled continuously. The soil was sampled at close intervals near the surface and then at an increased interval at greater depths, reflecting the lessening variability of soils with increasing depth. The selected sampling intervals enabled satisfactory characterization of the materials encountered.

RG 1.132 Section 4.3.1.2 states that boreholes with depths greater than 100 feet should be surveyed for deviation. As stated in FSAR Subsection 2.5.4.4.3, deviation measurements were conducted in the 10 uncased boreholes in which borehole geophysical logging was performed. The depths of these deviation data in the boreholes ranged from approximately 157 to 610 feet as provided in Table 5 of Appendix D – Geovision Downhole and P-S Logging Report in Volume 2 of FSAR Subsection 2.5.4, Reference 257. The deviation of the borehole from the vertical does not impact the characterization of the materials encountered.

Adjustments made during field work accounted for differing surface and subsurface conditions. These changes provided enhancements to the original exploration program to supplement characterization of site conditions. None of the adjustments made to the field testing locations, test methods, testing frequencies and test depths vary from the recommendations in RG 1.132. The exploration program met the intent of RG 1.132 and met the guidelines of the RG except, as previously discussed in this response, no boring was continuously sampled in the deeper soils, and only the borings used for borehole geophysical logging were surveyed for deviation.

Table 1 - Summary of SDDR Issues Related to Adjustments in Subsurface Exploration Program

SDDR NO. (25409-102-YD4-CY00-XXXXX)	Subject	Cause of Adjustment	Associated with Safety-Related Structures
00007	Drilling fluids permitted while rock coring, except in association with wells	Improve sample recovery and integrity	yes
00008	PS velocity logging modified to eliminate the upper 20 to 30 feet	Soft ground conditions required casing which precluded obtaining P-S logging data	yes
00015	Relocate borings B-613, B-614, and B-615	Minimize environmental impact	no
00017	Relocate borings B-734, B-735, B-736, and B-737 and wells OW-735U, OW-735L	Minimize environmental impact	no
00018	Relocate borings B-806 and B-807, wells OW-636U, OW-636L	Minimize environmental impact	no
00019	Relocate boring B-621, wells OW-621U and OW-621L	Minimize environmental impact	no
00020	Relocate boring B-805, wells OW-805U and OW-805L	Location inaccessible, submerged land	no
00024	Relocate borings B-634, B-635, B-636, and B-637	Minimize environmental impact	no
00025	Delete boring B-801, relocate borings B-812, B-813, and B-814, and wells OW-802U, OW-802L, OW-812U and OW-812L	Deleted boring due to inaccessibility (submerged land), relocated borings to minimize environmental impact	no
00026	Delete borings B-638, B-803 and B-804	Deleted borings due to inaccessibility (submerged land)	no
00027	Relocate boring B-802, wells OW-802U and OW-802L	Minimize environmental impact	no
00028	Install wells OW-802L and OW-805L in geotechnical boreholes instead of separate borings	Soft ground conditions restricted access to original well locations	no
00030	Relocate test pits TP-601 and TP-701	Minimize environmental impact	no
00032	Relocate boring B-813	Minimize environmental impact	no
00033	Change secondary seismic method from crosshole to downhole	More appropriate method for site-specific geology	yes
00039	Install 2 additional wells OW-606D and OW-706D, conduct slug tests	Evaluate hydrogeologic properties of Tamiami Formation	no

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

None

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-041

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-5 (eRAI 6006)

FSAR Section 2.5.4.2.1.3.11 states that two different relationships were used to calculate the high strain elastic modulus for fine and coarse grained soils. For fine-grained soils the first correlation is based on the use of an empirical Su and the second is based on the use of Vs. Since SPT N-values were used to calculate Su, and these were considered unreliable because of artesian conditions, and Vs obtained from small strain tests is not suitable to be used for the “high strain” case, please explain, in accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," what correlation was ultimately used to obtain the values for high strain elastic modulus included in FSAR table 2.5.4-209. Also, please explain why other test methods, such as laboratory testing as suggested by RG 1.138, were not applied given the large variability and uncertainty of SPT results.

FPL RESPONSE:

The high strain elastic moduli of fine- and coarse-grained soils were estimated using the available data from in-situ measurements and laboratory testing (as suggested by RG 1.138). For these materials, the high strain elastic modulus (E) can be derived based on various methodologies, including the relationships with shear strength (FSAR Equation 2.5.4-5), N_{60}-value (FSAR Equation 2.5.4-8), or low strain modulus (FSAR Equations 2.5.4-6 and 2.5.4-7).

As indicated in FSAR Subsection 2.5.4.2.1.3.11, the E values were derived from shear strength for the Lower Tamiami, and N_{60}-value for the Upper Tamiami and Peace River formations. The information regarding the estimation of the design N_{60}-values and shear strength, both derived from cone penetration test (CPT) results, is provided in the RAI 02.05.04-2 and RAI 02.05.04-8 responses, respectively. As noted in the RAI 02.05.04-2 response, four CPTs were advanced through the Upper and Lower Tamiami Formations and into the Peace River Formation. CPT measurements were taken at a depth interval of 0.07 ft from depths of approximately 120 to 290 ft, corresponding to a total of 7,304 data points. Unlike the SPT results, the CPTs show a more distinct and characteristic pattern with depth (FSAR Figure 2.5.4-214). As FSAR Section 2.5.4.8.3 indicates, the CPT measurements are much less susceptible to soil disturbance from hydraulic gradients. Under these conditions, the N_{60}-values (or, more precisely, equivalent N_{60}-values) and the shear strength, both derived from CPT results, are considered reliable and suitable to be used for computation of E. As noted in the revised response to RAI 02.05.04-2, the high relative density of the Upper Tamiami and the hardness of the Lower Tamiami (as demonstrated by the CPT results) are reflected in the high shear wave velocity measurements obtained in these strata, namely average values of 1400 ft/sec and 1600 ft/sec, respectively.

In addition, further evaluation pertaining to the computation of E was made by utilizing the relationship between high and low strain modulus. As indicated in FSAR Subsection 2.5.4.2.1.3.11, the low strain elastic (E_L) and shear (G_L) moduli are computed using FSAR Equations 2.5.4-6 and 2.5.4-7, respectively, based on the design shear wave velocity of each stratum. The ratio of high strain to low strain modulus is obtained from the shear modulus reduction curves that are based on RCTS test results, and is determined at a strain level between 0.25 and 0.5 percent, i.e., 0.375 percent. Because the axial and shear strain values are similar and interchangeable, the ratio of G/G_L is the same as the ratio of E/E_L. Consequently, the E value for each stratum was computed by multiplying the respective low strain elastic modulus with the ratio of E/E_L at 0.375 percent strain. Comparison of the results using these approaches showed that E values based on low strain modulus were consistently higher than those derived using the N_{60}-value and shear strength using CPT results. Thus, to be conservative, the E values developed from the CPT-derived N_{60}-values and shear strength were used, with minor rounding, for design. Although low strain elastic modulus values are often associated with dynamic applications, and high strain elastic modulus values are often associated with static applications, the terms "dynamic" modulus and "static" modulus used to describe low and high strain applications, respectively, can be misleading. Dynamic applications such as earthquakes and vibrating machine foundations, for instance, can produce strains that are orders of magnitude higher than the 10^{-4} percent strain frequently used to describe low strain, while strains associated with static settlement of rock can be orders of magnitude lower than the 0.25 to 0.5 percent frequently used to describe high strain. The estimated low levels of strain in the Key Largo and Fort Thompson Formations (about 0.005 percent) produced by the loading from the nuclear island are discussed in the revised response to RAI 02.05.04-6.

Also as discussed in the RAI 02.05.04-8 response, retrieved samples of these materials can be expected to be extremely disturbed considering the depth. Greater reliance is placed on deriving elastic modulus from empirical correlations with in-situ tests results where the stress relief that causes the sample disturbance is mostly absent. Thus, the laboratory testing of "undisturbed" samples of these materials was purposely limited because of the potential for sample disturbance, which would lead to inaccurate laboratory test results.

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

None

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-7 (eRAI 6006)

FSAR Table 2.5.4-208 shows that only one triaxial test was completed on an intact soil sample from Tamiami Formation sandy silt (Boring B-630). Also, the recommended effective cohesion and effective friction value for Lower Tamiami in FSAR Table 2.5.4-209 is solely based on the result of this sample. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," and Regulatory Guide (RG) 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," please justify why additional triaxial tests are not needed to fully characterize the shear strength parameters of these soils.

FPL RESPONSE:

The Tamiami Formation, described in FSAR Subsection 2.5.4.2.1.2, was found to be, on average, about 117 ft thick (FSAR Table 2.5.4-201). The formation is subdivided into the Upper Tamiami Formation (Stratum 5) and the Lower Tamiami Formation (Stratum 6). The Upper Tamiami is described as dense to very dense silty sand with varying amounts of gravel which transitions into the Lower Tamiami, described as very stiff to hard, sandy silt with minor amounts of silty clay. This transition can be seen in FSAR Figure 2.5.4-216 in which fines content is plotted against depth. Laboratory test results plotted on this figure clearly show the sandy material in the Upper Tamiami and the silty material with fines contents above 50 percent in the Lower Tamiami.

As stated in the RAI 02.05.04-8 question for sandy formations, “Laboratory tests of these material samples can be expected to be extremely disturbed considering the depth and behavior of materials.” The revised response to RAI 02.05.04-8 provides details of the triaxial testing on the Lower Tamiami Formation, and states that the laboratory testing of samples of these materials was purposely limited because of the assumed sample disturbance during recovery of the intact samples. For the very deep soils of the Tamiami and Peace River Formations, greater reliance is placed on deriving shear strength results from empirical correlations with in-situ tests results where the stress relief that causes the sample disturbance is mostly absent.

The information regarding the estimation of the design N_{60}-value, derived from cone penetration test (CPT) results, and the subsequent estimation of the shear strength and friction angle for the Tamiami formation is provided in the RAI 02.05.04-2 and RAI 02.05.04-8 revised responses, respectively.

As noted in the RAI 02.05.04-2 response, four CPTs were advanced through the Upper and Lower Tamiami Formations and into the Peace River Formation. CPT measurements were taken at intervals of 0.07 ft from depths of approximately 120 to 290 ft, corresponding to a total of 7,304 data points. Unlike the SPT results, the CPTs show a more distinct and characteristic pattern with depth (FSAR Figure 2.5.4-214). As FSAR Subsection 2.5.4.8.3 indicates, the CPT measurements are much less susceptible to soil disturbance. Under these conditions, the N_{60}-values and the shear strength, both derived from CPT results, are considered reliable and suitable to be used for design. As noted in the revised response to RAI 02.05.04-2, the high relative density of the Upper Tamiami and the hardness of the Lower Tamiami (as demonstrated by the CPT results) are reflected in the high shear wave velocity measurements obtained in these strata, namely average values of 1400 ft/sec and 1600 ft/sec, respectively.

In summary, no additional laboratory triaxial tests are needed because the strength parameters of the Tamiami and Peace River Formations have been adequately defined from CPT data. The conservatism of these strength parameters is confirmed by the high shear wave velocity measurements obtained in these strata.

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

None

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-9 (eRAI 6006)

In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please describe how shear and compressive wave velocity values are selected for design in Table 2.5.4- 209. Provide correlations for shear and compressive wave velocity between Table 2.5.4-209 and Table 2.5.4-215 to indicate what percentile is used for recommended values. Given the large deviations, especially on Key Largo and Fort Thompson formations (see Figures 2.5.4-218 and 2.5.4-19 and Table 2.5.4- 215), explain how the selected single value for each stratum statistically reflects the entire layer.

FPL RESPONSE:

The measured shear wave velocity (V_S) and compression wave velocity (V_P) values from each of 10 Suspension P-S velocity logging tests are plotted against depth in FSAR Figures 2.5.4-218 and 2.5.4-219, respectively. Figure 1(a) of this response provides a clearer picture of the variation of V_S within the Key Largo and Fort Thompson Formations. This figure shows the average and standard deviation of V_S in these formations at each measured depth interval in the 10 V_S borings. This figure is the basis for the V_S profile used for the site response analysis shown in Figure 1(b). Figure 1(b) is FSAR Figure 2.5.4-220. Figure 1(a) will be added to the FSAR. Figure 2 in this response is a typical plot of the measured values of V_S and V_P in one of the 10 borings. It shows the difference between the higher values in the Key Largo (25 to 50 feet depth) and the Fort Thompson (50 to 113 feet depth).

Using all of the test results, V_S values were averaged over typically 10-foot vertical depth intervals for the power block area, and a mean V_S profile with low/high end boundaries (mean plus/minus one standard deviation) was obtained as shown in the FSAR Table 2.5.4-215. The same approach was applied to the V_P measurements, and the resulting mean profile with low/high end boundaries is also included in FSAR Table 2.5.4-215. This methodology was followed for the Miami Limestone between depths of 0 and 30 feet, for the Key Largo Limestone between depths of 20 and 50 feet, for the Fort Thompson Formation between depths of 50 and 120 feet, for the Tamiami Formation between depths of 120 and 210 feet, and for the Peace River Formation between depths of 210 and 450 feet. Note that the interval from 20 to 30 feet depth was included in the analyses for both the Miami Limestone and the Key Largo Limestone as explained in the next paragraph. As an example, a summary of the V_S model for the Key Largo Limestone is presented in the table below.

Depth (ft)	**V_s (ft/sec) Mean**	**V_s (ft/sec) Mean - Std. Dev.**	**V_s (ft/sec) Mean + Std. Dev.**
20 to 30	4691	2580	6455
30 to 40	5871	4785	7675
40 to 50	6834	5302	8194
Average	**5799**	4222	7441

V_S readings were taken at 1.64-foot intervals in each boring, i.e., there were six readings per 10-foot depth. The depth interval between 30 and 40 feet in the above table is used to explain how the mean and standard deviations in the above table were derived. All 10 V_S borings covered this depth interval. Using the 10 V_S readings, a mean V_S value with plus/minus one standard deviation was computed and was assigned at each 1.64-foot interval. By averaging the six mean V_S values that fall in the 10-foot increment from 30 to 40 feet depth, the mean V_S of 5871 feet per second was obtained. The minimum of the six mean minus standard deviation V_S values was assigned as the low end V_S boundary (4785 feet per second), whereas the maximum of the six mean plus standard deviation V_S values is assigned as the high end V_S boundary (7675 feet per second). The mean V_s of 5799 feet per second for the Key Largo Limestone was computed by taking the average of these three mean V_S values, each corresponding to a 10-foot vertical depth interval. A best-estimate V_S of 5800 feet per second is recommended as shown in FSAR Table 2.5.4-209. Note that this value is close to the average value of V_S in the Key Largo Formation in Figure 1(b) in this response.

In deeper strata, when only one boring remained (as in part of the Peace River Formation and all of the Arcadia Formation), the standard deviation could not be computed because there was only a single V_S value measured at each 1.64-foot interval. The variation and uncertainty in V_S values in these and underlying strata were accounted for in the randomization process described in FSAR Subsection 2.5.2.5.2 and illustrated in FSAR Figure 2.5.2-239. Note also that FSAR Table 2.5.4-201 indicates the top of the Key Largo Limestone as El. –27 feet (North American Vertical Datum 1988 [NAVD 88]), which corresponds to a depth of approximately 27 feet, i.e., above 27 feet depth, the readings reflect the softer Miami Limestone. Therefore, including the mean V_S obtained between depths of 20 and 30 feet in the analysis reduces the average value somewhat.

Also, for the overlying Miami Limestone, the V_S measurements were averaged starting from the ground surface over 5-foot vertical intervals to 10 feet depth, below which 10-foot vertical intervals were used. For all other formations, the V_S measurements were averaged over 10-foot vertical intervals.

As mentioned earlier, for the Tamiami Formation, the V_S measurements were averaged between 120 and 210 feet depth. FSAR Table 2.5.4-201 indicates that the top of the Tamiami Formation is at El. –115 feet and the bottom is at El. –215 feet, corresponding to the depths of approximately 115 and 215 feet, respectively. The mean V_S increases from 1769 feet per second between 200 and 210 feet depth to 2235 feet per second between 210 and 220 feet depth. The reason for such an increase is the early presence of the Peace River Formation between 210 and 220 feet depth. For design purposes, it is reasonable to exclude the V_S measured between the depths of 210 and 220 feet for obtaining best-estimate values since it does not properly reflect the best estimate Vs value of either of the strata. In general, the Upper Tamiami, which extends to 160 feet depth, has a lower V_S than the Lower Tamiami. The best-estimate V_S values of 1400 and 1600 feet per second are recommended for the Upper and Lower Tamiami Formations, respectively.

The Poisson's ratios (μ) derived from the 10 Suspension P-S velocity logs were analyzed with a methodology similar to that used in the V_S assessment. An example of the μ model (mean with low/high end boundaries) is presented for the Key Largo Limestone in the table below.

Depth (ft)	**μ Mean**	**μ Mean - Std. Dev.**	**μ Mean + Std. Dev.**
20 to 30	0.33	0.24	0.41
30 to 40	0.30	0.23	0.35
40 to 50	0.29	0.23	0.35
Average	**0.31**	0.23	0.37

Table 4-11 of FSAR Reference 2.5.4-217 presents the values of μ in the range of 0.24 and 0.45 for limestone rock. Compared to these published values, it is reasonable to use a μ of 0.31 for the Key Largo Limestone.

Using the average V_S (5799 feet per second) and the Poisson's ratio (0.31) in FSAR Eq. 2.5.4-1, which defines the relationship between V_S, V_P, and μ, the average compression wave velocity is computed as 11,051 feet per second. A best-estimate V_P of 11,000 feet per second was selected for the Key Largo Formation as shown in FSAR Table 2.5.4-209. Following the same methodology, the best-estimate values for V_S, V_P, and μ were selected for other formations and are presented in FSAR Table 2.5.4-209.

This response to the RAI describes how the V_S, V_P, and μ values given in FSAR Table 2.5.4-209 were derived. The V_S and V_P in that table are best-estimate values. The statistical variation in V_S and V_P with increasing depth is shown in FSAR Table 2.5.4-215. As discussed in FSAR Subsection 2.5.2.5.2, FSAR Table 2.5.4-215 is the source and starting point from which the randomized V_S profiles were developed for the site response analyses.

RG 1.132 provides guidelines on the maximum depth of boring, d_{max}, required for an investigation of safety-related structures, based on the dimensions and design loading of the structure. Based on these guidelines, d_{max} is approximately 285 feet below the existing ground surface. Prior to starting the field investigation, the boring depth beneath the center of each reactor building was conservatively selected as 400 feet. Boring B-601 beneath the center of the Unit 6 reactor building was drilled to 419 feet to enable V_S measurements to be obtained to about 400 feet using Suspension P-S Velocity Logging equipment. During the investigation, information was obtained that indicated the top of the Arcadia Formation might be encountered at around 450 feet depth. Boring B-701 beneath the center of the Unit 7 reactor building was modified to about 616 feet depth, i.e., over double the RG 1.132 requirement, to obtain approximately 600 feet of V_S measurements, including about 150 feet in the Arcadia Formation.

Below 600 feet, V_S measurements down to almost 12,000 feet depth were derived from sonic testing of deep wells performed previously as described in FSAR Subsection 2.5.4.2.1.2.10. The locations of the deep wells range from about 60 to 115 miles from the site as shown in FSAR Figure 2.5.4-210. FSAR Figure 2.5.1-232 shows a cross-section down to about 6000 feet depth through the area where many of the deep wells are located, and indicates the geology from about 600 to 6000 feet depth is fairly consistent from the site to beyond 115 miles. It is expected that the geology below 6000 feet will be equally consistent. Where there was multiple sonic log information, generally below about 4000 feet, the standard deviation of the data was computed as well as the mean as shown in FSAR Figure 2.5.4-211. Where there was only one V_S profile, as in the plant area between about 400 and 600 feet depth and for the well logs between about 600 and 4000 feet depths, the variation of V_S was generated using 60 randomized profiles as described in FSAR Subsection 2.5.2.5. The standard deviation of $\ln(V_S)$ (the natural logarithm) as a function of depth was used to define the variation in the randomization process. The input profiles of the median and plus/minus one standard deviation of the V_S are shown in FSAR Figure 2.5.2-236. The randomized V_S profile for the complete 12,000 feet depth showing the variation in V_S at every depth is given in FSAR Figure 2.5.2-239.

DCD Section 2.5.4.5.3, titled Site Foundation Material Evaluation Criteria discusses the criteria that qualify the site as uniform. It states, "If a site can be classified as uniform, it qualifies for the AP1000 based on analyses and evaluations performed to support design certification without additional site-specific analyses." The section gives examples of sites that are considered uniform. The example that applies to the Turkey Point site rock strata is, "For a layer with a low strain shear wave velocity (V_S) greater than or equal to 2500 feet per second, the layer should have approximately uniform thickness and should have a dip of no greater than 20 degrees, and the V_S at any location within the layer should not vary from the average velocity within the layer by more than 20 percent." This example is also given in Table 2-1 of the DCD. The Key Largo and Fort Thompson Limestone Formations at the Turkey Point site consistently have V_S values well in excess of 2500 feet per second, are uniform in thickness across the site, and do not dip. However, V_S values at some depths within both the Key Largo and Fort Thompson Formations do vary from the average velocity in the stratum by more than 20 percent.

DCD Section 2.5.4.5.3.1 indicates that many sites that do not meet the uniform site criteria are acceptable for the AP1000. As described in that section, the driving force behind defining uniform site criteria is the design of the nuclear island (NI) basemat. As noted in the DCD, this basemat is designed specifically for bearing pressures on the mat of 120 percent of those of the uniform soil properties case. If the non-uniformity of the subsurface conditions makes these pressures exceed 120 percent, then the site may be unacceptable. The non-uniformity is basically the non-uniformity of the subgrade modulus (or spring constant) in the soil/rock below the basemat, which can cause non-uniform deformation and consequent overstressing of the basemat. Non-uniform thickness of strata and dipping strata can both clearly cause variation of subgrade modulus across the mat. More than 20 percent variation of V_S about the average is a much less obvious cause of non-uniform subgrade modulus. This is discussed below for the Key Largo and Fort Thompson Formations.

For Turkey Point Units 6 & 7, there will be approximately 19 feet of concrete fill beneath each NI basemat, which will act as a buffer against the effects of any non-uniformity in the rock formations below. The Key Largo Limestone extends below the concrete fill with an average thickness of about 15 feet, with little variation in thickness across the NIs. This limestone along with about 10 feet of the underlying Fort Thompson Limestone will be grouted down to El. –60 feet beneath the NI foundation within the surrounding concrete diaphragm wall as described in the response to RAI 02.05.04-1. The grouting program is intended to reduce the hydraulic conductivity of the Key Largo Limestone and the Fort Thompson Limestone within the diaphragm wall to limit flow rates during dewatering. This grout will be injected in three or four phases and is intended to fill the large majority of voids within the limestone. The grout is expected to be designed for a strength of 500 to 1000 psi. If a design strength of 750 psi is assumed, then the computed V_S of the grout is close to but somewhat higher than the V_S values for the Key Largo and Fort Thompson Limestones. (Grout and concrete typically have higher V_S values than natural rock with the same strength.) One effect of the grouting program will be to produce a much more homogeneous material, which will have considerably less variation in properties, including V_S. Thus, below the NI mat, there will be over 40 feet thickness (19 feet of concrete fill and 25 feet of grouted rock) with little variation in V_S within each layer.

The ungrouted Fort Thompson Limestone extends from El. –60 feet down to about El. –115 feet with minimal variation in thickness. Figure 2 shows a measured V_S profile (B-704) from El. –60 feet to El. –115 feet. This profile is typical of the V_S profiles in the other seven V_S borings beneath the NI foundations. Average V_S from El. –60.7 feet to El. –113.2 feet in B-704 is 4480 feet per second (with standard deviation of 975 feet per second) compared with the overall average V_S from all borings in the Fort Thompson Limestone of 4250 feet per second. Thus, for ungrouted Fort Thompson Limestone, although individual V_S values are sometimes beyond 20 percent of the average in any chosen boring, the average V_S for each boring is relatively consistent, and thus the subgrade modulus values (an approximate function of Vs^2) below the NI will be consistent. Therefore, if an analysis was performed to evaluate the stresses in the basemat due to variation in spring constant, little variation would be computed. Moreover, the spring constants based on average V_S values of concrete fill (6270 feet per second), Key Largo Limestone (5800 feet per second) and Fort Thompson Limestone (4250 feet per second) are extremely high, i.e., the rock is extremely stiff, resulting in negligible deformation under loading conditions. The estimated average combined settlement of the concrete fill, Key Largo Limestone and Fort Thompson Limestone under the DCD design static loading of 8.9 ksf is about 0.03 inches. Even if the average Vs values in each boring were not consistent, the differential deformations, and hence variations in stresses in the basemat, would still be negligible.

In summary regarding variation of V_S within each layer, the site cannot be classified as uniform according to the DCD based on the variation of the V_S measurements in the Key Largo and Fort Thompson Formations. Part of the Key Largo Limestone beneath the NI foundation will be replaced with concrete fill and the remainder will be grouted. The top portion of the Fort Thompson Limestone beneath the NI foundation will also be grouted. Although the remaining portion of the Fort Thompson Limestone has individual V_S values that are beyond 20 percent of the average in any chosen boring, the average V_S for each boring is relatively consistent, and thus the subgrade modulus values below the NI will be consistent. It can thus be simply demonstrated that the V_S variations will not cause non-uniform stresses in the basemat. As a result, the site is acceptable for the AP1000.

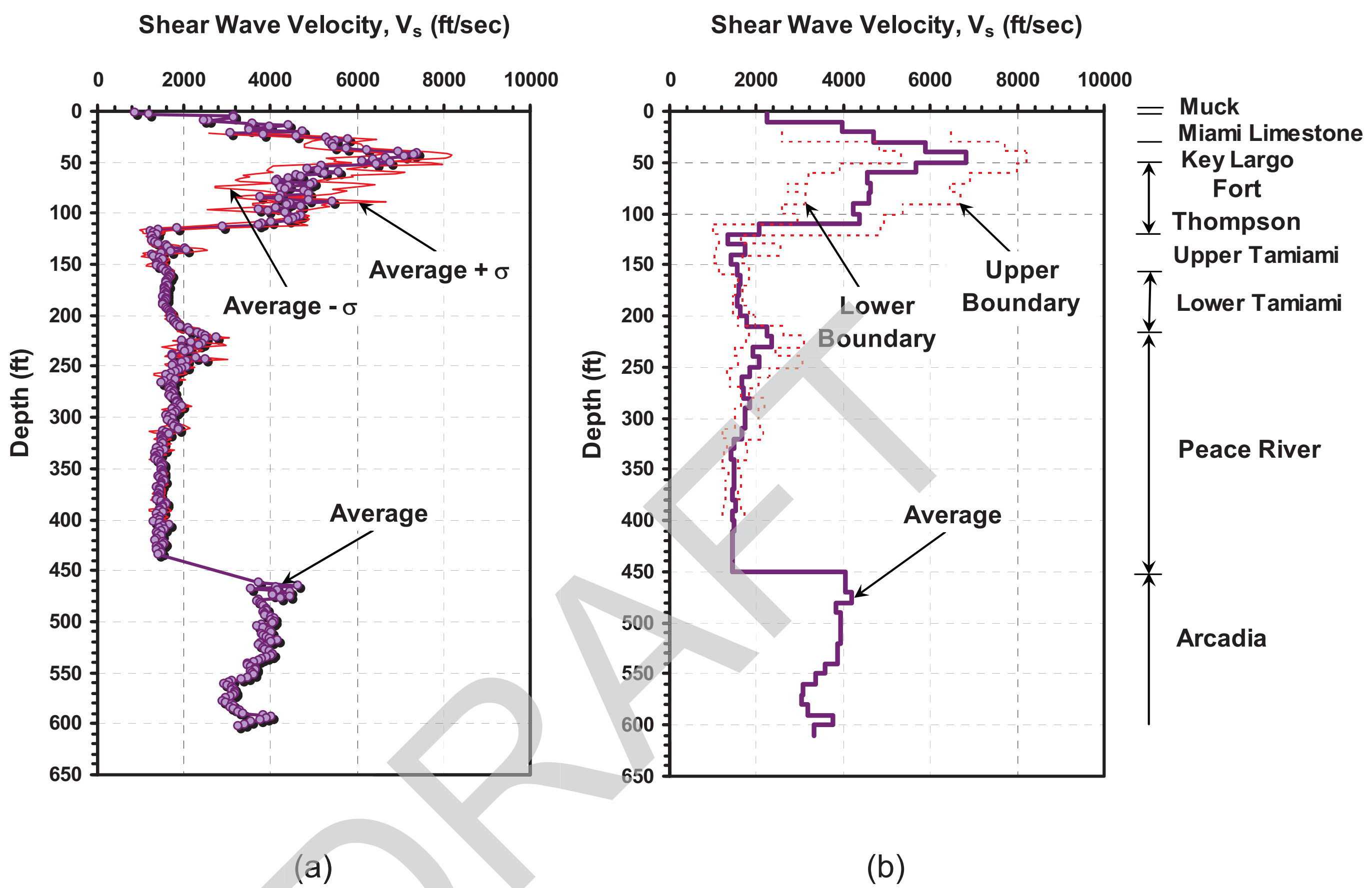

Figure 1. Shear Wave Velocities (a) Average and Standard Deviation of the Measured V_s; (b) Recommended Upper Boundary, Lower Boundary, and Average V_s

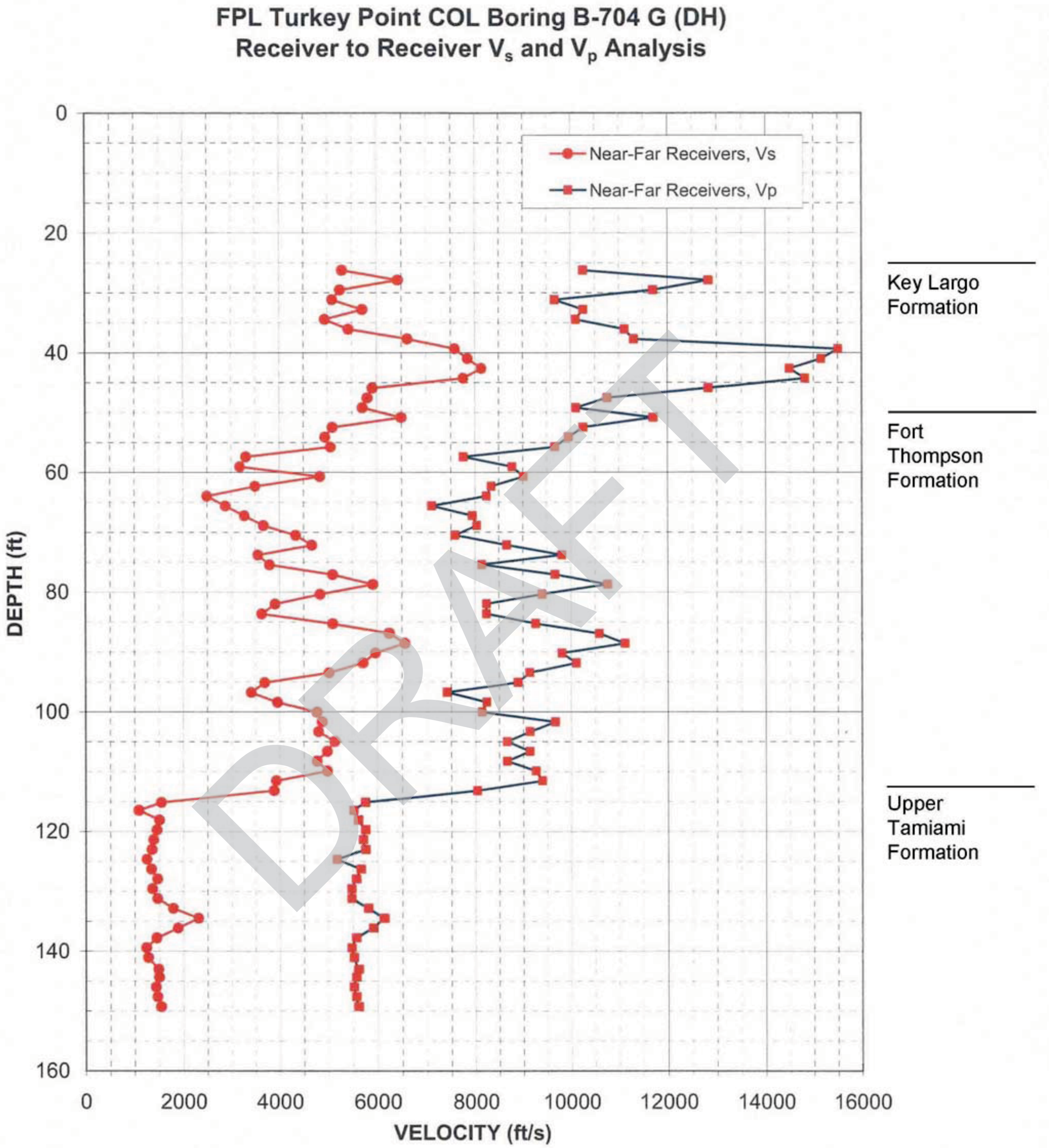

Figure 2. Shear and Compressive Wave Velocities Measured in Boring B-704 (from Volume 2 of FSAR Reference 257)

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

Figure 1(a) will be added to the FSAR as Figure 2.5.4-218 (b). Present Figure 2.5.4-218 will become Figure 2.5.4-218 (a).

Subsection 2.5.4.2.1.3.14, third sentence, will be modified as follows:

Figure 2.5.4-218 **(a)** is a plot of all of the measured shear wave velocities to depths of 400 and 600 feet at Unit 6 and Unit 7, respectively. **Figure 2.5.4-218 (b) shows the average and standard deviation of all of the shear wave velocity measurements at each measurement depth.**

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-16 (eRAI 6006)

Section 2.5.4.7.3.3 “Shear modulus and Damping for Rock”, indicates that the damping for rock is taken as 1%. The damping shown in Figure 2.5.2-249, which describes the soil properties used to develop the GMRS, indicates that a damping value of 0.5% was used in the analyses. In accordance with NUREG- 0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please provide clarification as to the actual level of damping used in the analyses and provide a basis for its selection considering the large variability in RQD shown in Figure 2.5.4-215.

FPL RESPONSE:

This RAI is very similar to RAI 03.07.01-19 (eRAI 6432) and thus the response given here follows closely the response to RAI 03.07.01-19.

FSAR Figure 2.5.2-249 (Sheet 2 of 2) is reproduced in this response for illustrative purposes. This figure represents the full soil and rock column that includes approximately 30 feet of structural fill (with surface at El. +25.5 feet NAVD 88) above approximately 25 feet of Miami Limestone, overlying about 90 feet of rock consisting of the Key Largo and Fort Thompson Formations. The rock is underlain by soil of the Tamiami and Peace River Formations to about 475 feet depth. The Arcadia Formation, consisting of very weak rock mixed with some soil extends to about 640 feet depth, the limiting depth of the site subsurface investigation. The actual levels of damping used in the analyses are the values shown in FSAR Figure 2.5.2-249 (Sheet 2 of 2). The basis for selecting the value for each formation is described in the following paragraphs.

FSAR Subsection 2.5.4.7.3.3 indicates that the Miami Limestone is considered sufficiently weak to have strain-dependent modulus and damping values. FSAR Table 2.5.4-216 shows the damping ratio (D) in percent versus shear strain values. D remains constant at 0.6 percent from 0.0001 to 0.03 percent strain. In the SHAKE analysis, shear strain did not exceed 0.03 percent, and so D is constant at 0.6 percent in FSAR Figure 2.5.2-249 (Sheet 2 of 2). D equals 1 percent at all strain levels for the Key Largo and Fort Thompson Formations (Strata 3 and 4) as stated in FSAR Subsection 2.5.4.7.3.3, which notes that damping in these formations is not strain dependent.

The only other rock formation noted in FSAR Subsection 2.5.4.7.3.3 is Stratum 8, the Arcadia Formation. This formation is included with the Key Largo and Fort Thompson Formations in FSAR Subsection 2.5.4.7.3.3 as being non-strain-dependent and having D constant at 1 percent. The Arcadia Formation is much weaker than the Key Largo and Fort Thompson Formations. FSAR Table 2.5.4-209 indicates an unconfined compressive strength of 100 psi compared with 1500 and 2000 psi for the Key Largo and Fort Thompson Formations, respectively. Even the strain-dependent Miami Limestone has double the strength of the Arcadia Formation. Thus, for the Arcadia Formation, consideration was given to using the D versus shear strain values of the Miami Limestone (Oolite) given in FSAR Table 2.5.4-216. However, since the Arcadia Formation is the lower portion of the

Hawthorn Group, with the overlying Peace River Formation (FSAR Subsection 2.5.1.1.1.2.1.1) forming the upper portion, it was considered more appropriate to use the D versus shear strain values of the Peace River Formation for the Arcadia Formation. FSAR Subsection 2.5.4.7.3.2 will be modified to indicate that the Peace River Formation damping values are also used for the Arcadia Formation. (FSAR Subsection 2.5.4.7.3.1 will be similarly modified to address shear modulus degradation curves.)

The constant damping ratio of the material below the Arcadia Formation, (i.e., below about 640 feet depth in FSAR Figure 2.5.2-249, Sheet 2 of 2), is 0.32 percent based on the median value of kappa and associated uncertainty.

The damping ratio versus shear strain relationship derived for mudstone was selected for the Miami Limestone, and the damping ratio versus shear strain relationship for natural soil measured from RCTS testing was selected for the Arcadia Formation (both relationships are shown on FSAR Figure 2.5.4-235). The strength and rigidity of the limestone in the Key Largo and Fort Thompson Formations indicated that damping ratio is independent of strain level, and a value of 1 percent was selected, consistent with the damping ratio selected for other rock sites (e.g., North Anna Unit 3 and VC Summer Units 2 & 3).

The following provides a basis for the selection of damping ratio values for the various rock strata considering the large variability in RQD shown in FSAR Figure 2.5.4-215. Each rock stratum is defined based on age, mineral composition, depositional mode, etc. Although there may be significant variability of a parameter (strength, shear wave velocity, RQD, etc.) within a stratum, a single damping ratio versus strain relationship is selected for each stratum. If the variability was clearly defined within the stratum (e.g., high RQD at the top of the stratum and low RQD at the bottom), the stratum could be sub-divided into separate strata and different damping ratio versus strain relationships assigned to each. However, with the Turkey Point rock strata, the variability is generally random, and the strata are not sub-divided.

The uncertainties and variation in the damping ratios (reflected in the variations in parameters such as RQD) were taken into account in the randomization process. FSAR Figure 2.5.2-238 shows the variation assumed in the randomization process for the damping ratio versus strain for the Arcadia Formation.

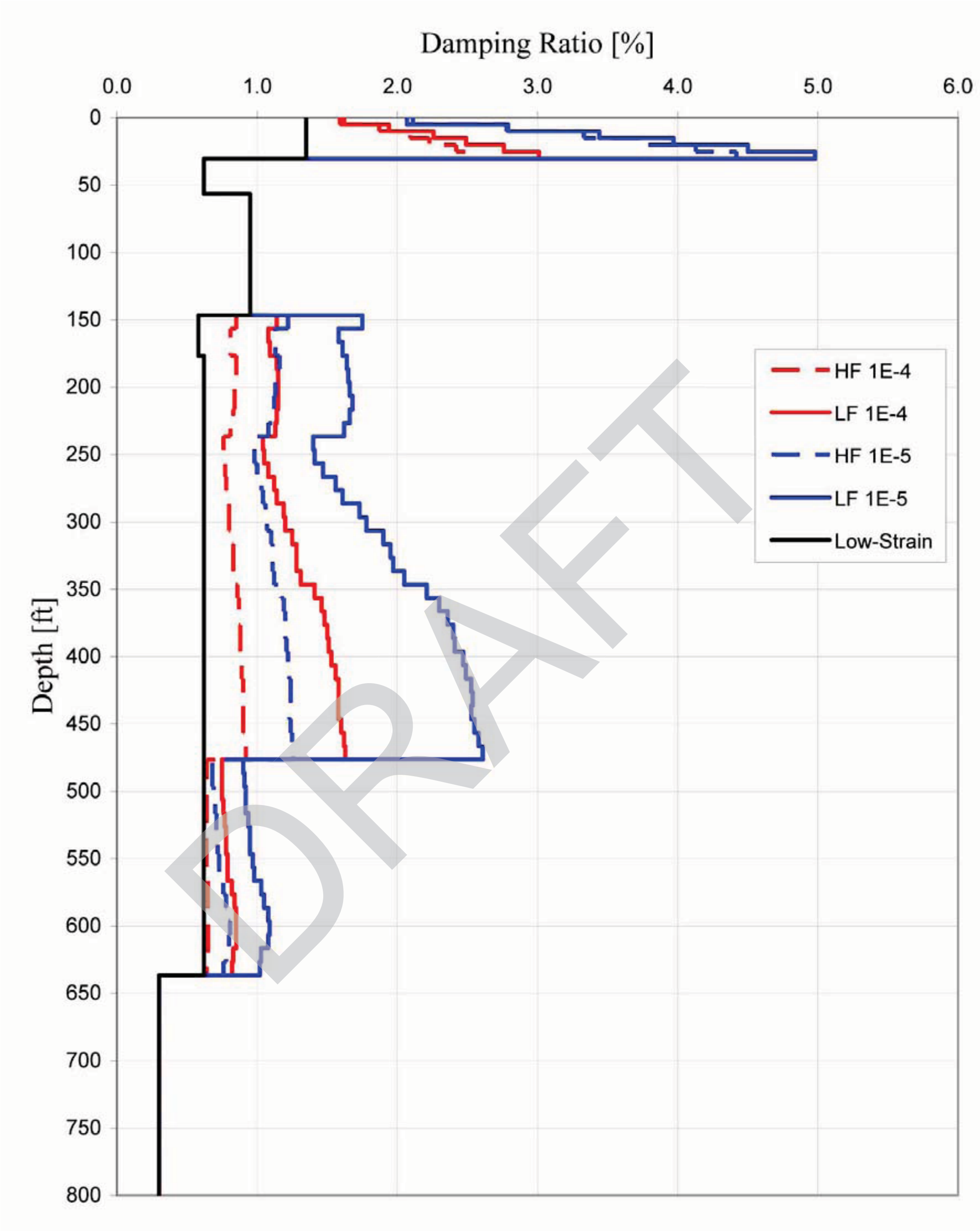

Figure 2.5.2-249 Median Profiles of Strain-Compatible Soil Damping (Upper 800 feet) (Sheet 2 of 2)

(Source: FSAR Revision 3)

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

The following changes will be made in a future FSAR revision.

FSAR Subsection 2.5.4.7.3.1, Fifth and Sixth Paragraphs:

Due to the similarity of the grain size distribution and the materials, the recommended shear modulus degradation of Stratum 7 is the same as for Stratum 6, i.e., natural soil deeper than 159 ft depth in Figure 2.5.4-233. **This modulus degradation curve is also selected for Stratum 8 which consists of very weak rock and is part of the same geological formation (Hawthorn Group) as Stratum 7.**

Rock Strata 3~~, 4~~ and ~~8~~**4** are considered not subject to modulus degradation, as described in Subsection 2.5.4.7.3.3.

FSAR Subsection 2.5.4.7.3.2, Last Sentence:

Figure 2.5.4-235 shows the selected values of D versus shear strain for tested Strat~~um~~**a** 5, 6, and 7, i.e., the natural soil curve used for all three strata. **This D versus shear strain curve is also selected for Stratum 8.**

FSAR Subsection 2.5.4.7.3.3, First Paragraph, Second Sentence:

For Strata 3~~, 4~~ and ~~8~~**4**, the shear modulus is considered non-strain dependent based upon the competency of the rock.

FSAR Subsection 2.5.4.7.3.3, First Paragraph, New Last Two Sentences:

Reference 259 does not include a damping curve for mudstone/shale. The curve in Figure 2.5.4-235 was developed based on interpolation/extrapolation of other damping curves included in Reference 259.

FSAR Subsection 2.5.4.7.3.3, Third Paragraph, Second Sentence:

For site-specific work, damping of 1 percent is adopted for Strata 3~~, 4~~ and ~~8~~**4**, and bedrock shear modulus is considered to remain constant (i.e., no degradation) in the strain range of 10^{-4} percent and 1 percent.

FSAR Subsection 2.5.4.7.3.3, Last Paragraph, New Last Sentence:

Subsection 2.5.2.5.2 addresses the uncertainty in shear modulus and damping values modeled in the site response analysis.

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-18 (eRAI 6006)

FSAR Section 2.5.4.10.2 describes bearing capacity calculations for the nuclear island foundation. This section states that FSAR Equation 2.5.4-15 was used to calculate the ultimate bearing capacity of the reactor and auxiliary buildings. Also the calculation for COL bearing capacity and settlement analyses states that 20 % of the unconfined compressive strength was used instead. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please clarify on the actual methodology used to calculate the ultimate bearing capacity and justify its applicability. Also, please explain how the unconfined compressive strength parameter values in Table 2.5.4-209 were selected given the large range of values presented in FSAR Figure 2.5.4-217 and Table 2.5.4-207.

FPL RESPONSE:

This response is provided in three parts—part 1 to discuss the methodology used to calculate ultimate bearing capacity, part 2 to discuss how unconfined compressive strength (U) values were developed, and part 3 to demonstrate that the rock has adequate bearing capacity for the applied NI loads, even if the minimum measured compressive strengths are assumed.

1. Methodology to Calculate Ultimate Bearing Capacity

<u>Method (a) Used for the Nuclear Island</u>

The nuclear island (NI) is founded on about 19 feet of concrete fill underlain by the Key Largo and Fort Thompson Limestone formations. An allowable bearing capacity of not more than 20 percent of U of the rock can be used according to FSAR subsection 2.5.4, Reference 221. This is a value based on several building codes for drilled piers in all types and strengths of rock. This value is necessarily conservative because of the inherent danger of punching failure of an end-bearing pier. Such punching failure in rock cannot occur for a large mat foundation.

The design U for the Key Largo Limestone is 1.5 ksi from FSAR Table 2.5.4-209; 20 percent of 1.5 ksi = 300 psi ~ 43 ksf. The design U for the Fort Thompson Limestone is 2.0 ksi from FSAR Table 2.5.4-209; 20 percent of 2.0 ksi = 400 psi ~ 58 ksf.

The lower of these two values, i.e., 43 ksf, was selected as the allowable bearing capacity of the rock beneath the NI. This is an allowable value. For bearing capacity, the allowable value is typically about one-third of the ultimate value. Thus, the ultimate bearing capacity of the rock can be taken as about 130 ksf.

Method (b) Used for Structures on Miami Limestone

As discussed in FSAR Subsection 2.5.4.10.2, the ultimate bearing capacity for a foundation bearing on weak rock is evaluated using FSAR Equation 2.5.4-15, shown here as Equation 1, with the terms as defined in the FSAR.

$$q_{ult} = c\ N_c\ C_{f1} + \gamma\ D_f\ N_q + 0.5\gamma\ B\ N_\gamma C_{f2} \qquad \text{Equation 1}$$

This analysis was conducted on the Miami Limestone layer; however, the equation was simplified as follows:

$$q_{ult} = cN_cC_{f1} \qquad \text{Equation 1a}$$

This simplification is conservative since it neglects the contribution of the second two terms.

FSAR equations 2.5.4-17 and 2.5.4-18 are shown here as Equations 2 and 3, respectively.

$$N_\phi = \tan^2(45+\phi/2) \qquad \text{Equation 2}$$

$$N_c = 2\ N_\phi^{0.5}(N_\phi+1) \qquad \text{Equation 3}$$

where:

ϕ= friction angle

Rock mass cohesion (c) and friction angle are needed for input into Equation 1. Since there were no laboratory test results available to derive rock mass cohesion or friction angle for Miami Limestone, a generic value was used from FSAR Reference 272. For limestone with 10- to 20-millimeter clay infillings, $c = 2.3$ ksf and $\phi = 14$ degrees. Using a length-to-breadth (L/B) ratio of 2, the shape factor, C_{f1}, from FSAR Equation 2.5.4-16b = 1.12.

Therefore, using Equations 2, 3, and 1a above,

$$N_\phi = \tan^2(45+\phi/2) = \tan^2(45+14/2) = 1.64$$
$$N_c = 2\ N_\phi^{0.5}(N_\phi+1) = 2 \times 1.64^{0.5}(1.64+1) = 6.76$$
$$q_{ult} = 2.3 \text{ ksf} \times 6.76 \times 1.12 = 17.42 \text{ ksf}$$

Using a factor of safety of 3,

$$q_{all} = 17.42/3 = 5.8 \text{ ksf}$$

This value can be compared with the method (a) approach outlined above for the NI. For the Miami Limestone, U of 200 psi is given in FSAR Table 2.5.4-209. Twenty percent of this strength is 40 psi (5.76 ksf). Results of these two methods compare favorably. Note that the Miami Limestone will be excavated out and replaced with concrete fill before construction of the NI foundation.

Similarly, the 43 ksf obtained for the Key Largo Limestone using method (a) can be compared with the value derived using the method (b) approach. The friction angle is conservatively assumed to be the same as for the Miami Limestone, 14 degrees. Cohesion of the rock mass is conservatively approximated as 10 percent of U = 0.10 x 1.5 ksi = 150 ksi = 21.6 ksf.

Therefore, using Equation 1a,

$$q_{ult} = 21.6 \text{ ksf} \times 6.76 \times 1.12 = 163.54 \text{ ksf}$$

Using a factor of safety of 3,

$$q_{all} = 163.54/3 = 54.51 \text{ ksf}$$

While the results of these methods compare favorably, the lower value of 43 ksf was chosen for design.

2. Development of Unconfined Compressive Strength Values

FSAR Subsection 2.5.4.2.1.3.10 provides a discussion of the results from the U testing on rock core samples from three of the rock strata cored. Test results are summarized in FSAR Table 2.5.4-207 and shown in FSAR Figure 2.5.4-217. Recommended U values for these strata are provided in FSAR Table 2.5.4-209. It can be seen that the recommended values are less than the average values from FSAR Table 2.5.4-207. The best estimate compressive strength value selected for the Key Largo Formation (1500 psi) is considerably less than the average strength measured on the core samples (2729 psi). For the Fort Thompson Formation, the selected best estimate strength (2000 psi) is less than but closer to the average strength measured on the core samples (2269 psi). The more conservative best estimate strength for the Key Largo Formation was selected because this is the formation directly beneath the concrete fill beneath the NI foundation. Note that the Key Largo Formation beneath the NI will be grouted with cementitious grout in order to reduce its hydraulic conductivity prior to construction dewatering. This grouting will both increase the overall strength and help reduce the strength variability of the rock by filling in the voids in the material.

3. Bearing Capacity Based on Rock Strength

The allowable bearing pressure of 43 ksf is based on 20 percent of the best estimate compressive strength of the Key Largo Limestone. FSAR Equation 2.5.4-15, as described in Part 1 of this response, was only used to demonstrate that the 43 ksf is a conservative value.

The AP1000 DCD gives a static average bearing demand of 8.9 ksf over the NI footprint (average allowable static bearing capacity) and a maximum bearing demand of 35 ksf at the edge of the NI foundation (dynamic bearing capacity). The 43 ksf allowable bearing capacity satisfies these demands. Note that the 35 ksf value is based on a peak ground acceleration of 0.30g. The peak ground acceleration at the Turkey Point site is less than 0.10g and so the actual maximum bearing demand will be significantly less than 35 ksf.

There were 31 compressive strength tests performed on samples from the Key Largo Limestone. The minimum measured strength was 309 psi at El. –27.5 feet. The allowable bearing capacity based on this strength is 0.2 x 309 = 61.8 psi = 8.9 ksf. There were 46 compressive strength tests performed on samples from the Fort Thompson Limestone. The minimum measured strength was 172 psi at El. –63.3 feet. The allowable bearing capacity based on this strength is 0.2 x 172 = 34.4 psi = 4.95 ksf. This sample is about 47 feet below the bottom of the NI mat foundation. Assuming an approximately 1V:1H distribution of load through the concrete fill and the grouted rock below the NI foundation, and assuming load will not distribute beyond the diaphragm wall (described in the final section of this response) that will be constructed around the NI foundation, the average load at El. –63.3 feet is about 52 percent of the applied load, i.e., 4.63 ksf. In short, even the minimum compressive strengths measured in the rock formations applied at the sample depths provide adequate allowable bearing capacity for the average static bearing demand.

4. Bearing Capacity Based on Bearing Capacity Equation

In clay soils, cohesion is 50 percent of unconfined compressive strength, U. For the Key Largo Limestone, 10 percent of U was assumed. The angle of internal friction is not a property typically derived for moderately strong rock. The value of 14 degrees assumed is considered to be reasonable and conservative. If the second two terms in bearing capacity Equation 2.5.4-15 had not been neglected, the computed value of bearing capacity would have been extremely large, given that the depth of embedment, D_f, in the second term of the equation is about 39 feet for the NI, and the width of the foundation, B, in the third term of the equation is about 88 feet (minimum) for the NI.

The USACE Rock Foundations Manual states in Section 6-12b, “In cases where the shear failure is likely to develop along planes of discontinuity or through highly fractured rock masses…cohesion cannot be relied upon to provide resistance to failure.” The grouted rock within the diaphragm wall beneath the NI foundation and the underlying Fort Thompson Limestone are neither fractured nor have planes of discontinuity, and thus, cohesion will play a large factor in determining bearing capacity. Note that if cohesion is not used in Equation 1, and the only strength parameter assumed for the rock is the angle of internal friction of 14 degrees, the factor of safety against bearing failure is still greater than 3 for the average static bearing demand (8.9 ksf) of the NI.

5. Overall Stability Considerations

The proposed plant is stable from a bearing capacity/sliding standpoint. At the beginning of construction, a reinforced concrete diaphragm wall will be installed down to El. –60 feet, with the bottom of the wall about 10 feet below the top of the Fort Thompson Limestone. This wall will be about 30 feet outside the perimeter of the NI foundation. This type of wall is typically 3 to 4 feet thick and reinforced to full depth with either reinforcing bars or steel H-beams. The wall installation will be followed by a grouting program with primary, secondary, tertiary, and quaternary stages. This will involve grouting the Key Largo and Fort Thompson Limestone formations between about El. –35 feet and El. –60 feet within the perimeter of the diaphragm wall. Grout hole spacing will be a maximum of 10 feet within the area of the diaphragm wall.

The purpose of the grouting program is to minimize the permeability of the rock between El. –35 feet and El. –60 feet to allow dewatering but will also eliminate all but the smallest voids in the grouted volume. After the volume within the diaphragm wall is dewatered, the muck, Miami Limestone, and parts of the Key Largo Limestone will be removed down to El. –35 feet. Concrete fill with a design strength of 1500 psi will then be placed to about El. –16 feet, extending laterally out to the diaphragm wall. The NI will be constructed on the concrete fill. Granular backfill compacted to at least 95 percent of modified Proctor dry density (ASTM D 1557) will be placed between the walls of the NI structures and the diaphragm wall to about El. 0 feet and then up to final grade at about El. +25 feet as shown in FSAR Figure 2.5.4-222.

The NI has a design loading of 8.9 ksf, which is about 62 psi. It sits on about 19 feet of 1500-psi concrete, which is underlain by about 25 feet of grouted Key Largo and Fort Thompson Limestone with pre-grout strengths of 1500 and 2000 psi, respectively. The ungrouted Fort Thompson Limestone, which has an allowable bearing capacity of about 58 ksf, extends another 55 feet below the grouted zone.

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

FSAR Section 2.5.4.2.1.3.10 will be revised as follows in a future COLA revision.

2.5.4.2.1.3.10 Rock Unconfined Strength

Rock core samples from three of the rock strata cored (the Key Largo Limestone, Fort Thompson Formation, and Arcadia Formation) are tested for unconfined compressive strength. Although the Miami Limestone is a rock, its texture does not lend itself to typical rock coring and the use of SPT to sample this formation is common in South Florida. **Unconfined compressive strength for the Miami Limestone given in Table 2.5.4-209 is based on results of unconfined compression tests on samples of Miami Limestone obtained during the investigation for Units 3 and 4 (Reference 710 of Subsection 2.5.1) and other strength test results for this material published in the literature.**

Results of the unconfined strength tests performed on 31 samples from the Key Largo Limestone, 46 samples from the Fort Thompson Formation, and three samples from the Arcadia Formation are summarized on Table 2.5.4-207 and shown on Figure 2.5.4-217.

FSAR Section 2.5.4.10.2 revised as follows in future COLA revision.

2.5.4.10.2Units 6 & 7 Bearing Capacity Evaluation

The ultimate bearing capacity, q_{ult}, of a foundation is calculated using Reference 225:

$$q_{ult} = c\ N_c\ \zeta_c + q\ N_q\ \zeta_q + 0.5\ \gamma'\ B\ N_\gamma\ \zeta_\gamma \qquad \text{Equation 2.5.4-14}$$

Category I seismic structures bear on lean concrete placed on the rock of Key Largo Limestone (Stratum 3). For foundations bearing on rock, Reference 272 is used to calculate bearing capacity.

Using Reference 272, the ultimate bearing capacity (q_{ult}) formula for a footing on weak rocks with little fracturing is calculated as:

$$q_{ult} = c\ N_c\ C_{f1} + \gamma\ D_f\ N_q + 0.5\gamma\ B\ N_\gamma C_{f2} \qquad \text{Equation 2.5.4-15}$$

Where:

c = rock mass cohesion

γD_f = overburden pressure at base of foundation

γ = unit weight of rock

D_f = depth from ground surface to base of foundation

B = width of foundation

N_c, N_q, and N_γ are bearing capacity factors for rock

C_{f1} and C_{f2} are shape factors that replace ζ shape factors in Equation 2.5.4-14.

From Table 5.4 of Reference 272,

$C_{f1} = C_{f2} = 1.0$ for L/B>6 strip foundation — Equation 2.5.4-16a

$C_{f1} = 1.12$, $C_{f2} = 0.9$ for L/B=2 — Equation 2.5.4-16b

$C_{f1} = 1.05$, $C_{f2} = 0.95$ for L/B=5 — Equation 2.5.4-16c

$C_{f1} = 1.25$, $C_{f2} = 0.85$ for square foundation — Equation 2.5.4-16d

$C_{f1} = 1.2$, $C_{f2} = 0.7$ for circular foundation — Equation 2.5.4-16e

Where,

L = length of footing.

From Equation 5.8 of Reference 272,

$N_\phi = \tan^2(45+\phi/2)$ — Equation 2.5.4-17

$N_c = 2 N_\phi^{0.5}(N_\phi+1)$ — Equation 2.5.4-18

$N_\gamma = 0.5 N_\phi^{0.5}(N_\phi^2-1)$ — Equation 2.5.4-19

$N_q = N_\phi^2$ — Equation 2.5.4-20

Equation 2.5.4-15 can be simplified to:

$q_{ult} = cN_cC_{f1}$ — Equation 2.5.4-15a

This simplification is conservative since it neglects the contribution of the second two terms.

Since there were no laboratory test results available to derive rock mass cohesion or friction angle for Miami Limestone, a generic value was used from FSAR Reference 272. For limestones with 10 to 20 mm clay infillings, c = 2.3 ksf and ϕ = 14º. Using Equation 2.5.4-215a gives an allowable bearing capacity of 5.8 ksf, including a factor of safety of 3.

Alternatively, an allowable bearing capacity of not more than 20 percent of the unconfined compressive strength (U) of the rock can be used, according to FSAR Reference 221. For the Miami Limestone, a U of 200 psi is given in FSAR Table 2.5.4-209. Twenty percent of this strength is 40 psi (5.76 ksf). The results of the two methods compare favorably.

The foundation bearing capacities of the Category 1 seismic structures are considered similarly. The design U for the Key Largo Limestone is 1.5 ksi from Table 2.5.4-209 with 20 percent of 1.5 ksi = 300 psi ~ 43 ksf. This allowable capacity compares favorably to the value of 54.5 ksf which is calculated using Equation 2.5.4-15a (conservatively assuming a friction angle that is the same as for the Miami Limestone = 14^{o} and a cohesion of 10 percent of the U, i.e., 21.6 ksf); the lower value of 43 ksf is used.

Foundation bearing capacities are calculated using the average material properties in Table 2.5.4-209 and Equations 2.5.4-14 through 2.5.4-20. A summary of the allowable bearing capacities (using FOS = 3.0) of Seismic Category I structures (nuclear island) is given in Table 2.5.4-217. Analysis results show that for the Seismic Category I structures (including both units), the allowable static bearing capacity is 43 ksf, which greatly exceeds the anticipated average allowable bearing capacity of ~~8.6~~ **8.9** ksf specified in the DCD.

The above bearing capacity formulation is based on the assumption that the strata within the zone of foundation deformation are uniform with depth in terms of shear strength properties. While recognizing that the site strata are interlayered, the properties of the soil and rock are conservatively selected to provide for a representative bearing capacity.

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-2 (eRAI 6006)

FSAR Section 2.5.4.2.1.3.2.2 states that N60 was obtained by applying a correction factor, CE, to the energy ratio. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," and Regulatory Guide (RG) 1.132, "Site Investigations for Foundations of Nuclear Power Plants," please justify why other correction factors (e.g., overburden pressure, borehole diameter, rod length and sampling method) were not included in the N-value correction process. Also please describe how the recommended SPT design values in FSAR Table 2.5.4-209 were obtained and how each single value for each stratum could properly and statistically reflect the entire layer variations, as shown on Figure 2.5.4-213 and Table 2.5.4-204

FPL RESPONSE:

As stated in FSAR Subsection 2.5.4.2.1.3.2.2, the N-value correction was made for the SPT hammer energy based on the average hammer efficiency of the specific equipment using an energy correction factor ($C_{\acute{\epsilon}}$). Other minor corrections were also made for borehole diameter (C_B), sampler type (C_S), and rod length (C_R) using the formula in Equation 1 (FSAR References 225 and 219). As indicated in the COLA Revisions, these values will be added to Equation 2.5.4-2 in the FSAR.

$$N_{60} = N\, C_{\acute{\epsilon}}\, C_B\ \ C_S\, C_R \qquad \text{(Equation 1)}$$

The values of these additional correction factors are provided in FSAR Subsection 2.5.4 References 219 and 225. N_{60}-values are typically used in correlations to derive friction angle and other engineering properties.

One additional N-value correction that can be made is for the overburden pressure (C_n), resulting in an $(N_1)_{60}$-value. This value is typically used in liquefaction analyses. The liquefaction analysis for the Turkey Point Units 6 & 7 site is discussed in FSAR Subsection 2.5.4.8. Note that because the N-value data for Strata 5 and 6 and the upper portions of Stratum 7 were discounted due to the partial hydraulic gradient disturbances, $(N_1)_{60}$-values and a discussion of C_n were not used or presented in the FSAR.

The measured and corrected N-values are presented in FSAR Figure 2.5.4-212 and Figure 2.5.4-213, respectively. Where SPT refusal was encountered, the N-value was conservatively taken as 100 blows per foot (bpf). The N_{60}-values presented in FSAR Table 2.5.4-209 represent the best estimate values and are not meant to show variations. In the next paragraphs, the methodology used to derive the recommended N_{60}-values is explained.

Miami Limestone (Stratum 2)

The N_{60}-values presented in FSAR Figure 2.5.4-213 show a wide scatter, varying from 0 to 100 bpf with depth. The average N_{60}-value is 29 bpf (FSAR Table 2.5.4-204), and the median value is 19 bpf. An N_{60}-value of 20 bpf is recommended.

Key Largo Limestone and Fort Thompson Limestone (Stratum 3 and Stratum 4)

Where rock coring was not possible, limited sampling using the SPT was conducted. This sampling was mainly performed in the upper few feet of the Key Largo Limestone as a continuation of the SPT sampling in the overlying Miami Limestone before switching to rock coring in the Key Largo Limestone. About 3 percent of the sampling of the Fort Thompson formation was performed with SPT, following rock coring attempts with little or no recovery. Thus, N_{60}-values in FSAR Figure 2.5.4-213 are more representative of the lower-bound strength of the materials encountered. No N_{60}-value recommendation was made for these rock formations.

Upper and Lower Tamiami (Stratum 5 and Stratum 6)

The N_{60}-values derived from SPT measurements show a wide scatter, varying from 0 to 100 bpf in the Upper Tamiami and from 3 to around 100 bpf in the Lower Tamiami (FSAR Table 2.5.4-204). There is no obvious correlation with elevation. Silty sands and sandy silts that range in depth from 120 to 220 feet would normally be dense to very dense with consistently high N_{60}-values. As discussed in FSAR Subsection 2.5.4.8.2, blow counts of less than 20 bpf, and particularly less than 5 bpf, are most probably due to sample disturbance caused by an unbalanced hydraulic head. An assessment of groundwater measurements indicated an upward hydraulic gradient with approximately 1 to 2 feet of piezometric head difference between wells screened in different zones. It seems likely that this hydraulic gradient has contributed to at least partial blowout of the bottom of the hole prior to/during SPT sampling on many if not most of the samples. The average N_{60}-value is 27 and 23 bpf for the Upper Tamiami and Lower Tamiami Formations, respectively (FSAR Table 2.5.4-204). Based on depth, the average N_{60}-values should be at least double these values. The high relative density of the Upper Tamiami and the hardness of the Lower Tamiami are reflected in the high shear wave velocity measurements obtained in these strata, namely average values of 1400 feet per second and 1600 feet per second, respectively.

Due to the scatter and low N-values recorded for the Tamiami Formation, four cone penetration tests (CPTs) were performed through the Tamiami Formation after drilling through the overlying rock. Unlike the SPT results, the CPTs show a more distinct and characteristic pattern with depth (FSAR Figure 2.5.4-214). For the Upper Tamiami, the CPT corrected tip resistance (q_t) values range from about 100 to 200 tons per square foot (tsf), with an average of about 160 tsf. The q_t values in the Lower Tamiami range from about 80 to 150 tsf, with an average of about 110 tsf. To better evaluate the soil properties, the measured N-values were then compared to those back-calculated using the correlation with q_t. Figure 2-32 from Kulhawy & Mayne (Reference 1) substantiates a general trend between q_t/N and fines content, based on data available from various sources, and provides the best estimate relationship with increasing fines content. The ratio of q_t/N typically varies between 4 and 5 for clean sands and between 3.5 and 4.5 for silty sands, with a fines content of about 12 percent. Note that for a given q_t value, the back-calculated N-values become higher with increasing fines content. Thus, for a given q_t value, the smaller the fines content, the smaller the equivalent N-value is.

Using a q_t/N ratio of 4 and an average q_t of 160 tsf for the Upper Tamiami, an N_{60}-value of 160/4 = 40 bpf is estimated. Similarly, using a q_t/N ratio of 3.5 and an average q_t of 110 tsf for the Lower Tamiami, an N_{60}-value of 110/3.5 ~ 31 bpf is obtained. However, considering the relatively high shear wave velocity values observed in these formations (FSAR Figures 2.5.4-218 and 2.5.4-220), even these N_{60}-values derived from the CPT results are deemed to be conservative. Nevertheless, N_{60}-values of 40 bpf for the Upper Tamiami and 32 bpf for the Lower Tamiami are recommended. Note that the angle of internal friction of 35 degrees selected for the Upper Tamiami is a conservative value for an N_{60}-value of 40 bpf and a shear wave velocity of 1400 feet per second. The 35 degrees would be a reasonable choice based on the average N_{60}-value of 27 from the measured N-values. The undrained shear strength of 4 ksf selected for the Lower Tamiami is a realistic value for an N_{60}-value of 32 bpf but a very conservative value for a shear wave velocity of 1600 feet per second.

Peace River (Stratum 7)

While the very top portion of this stratum exhibited low N-values, attributed to high artesian conditions, the lower portion of the stratum generally exhibited SPT refusal; resulting N-values were capped at 100 bpf. Thus, N_{60}-values of 100 bpf presented in FSAR Figure 2.5.4-213 are conservative. Considering the entire stratum, the average of the N_{60}-values is 72 bpf (FSAR Table 2.5.4-204), and the median is 74 bpf. An N_{60}-value of 75 bpf is recommended.

Limerock Fill

An N_{60}-value of 30 bpf is assumed for the limerock fill. Additional information regarding the gradation of the fill material can be found in the RAI 02.05.04-15 response.

This portion of the response describes how the N_{60}-values given in FSAR Table 2.5.4-209 were derived. These are best estimate values and do not reflect the variation of the N_{60}-values within each layer. This variation is demonstrated in FSAR Figure 2.5.4-213. The mean value for each layer +/-1 standard deviation is given in Table 1.

Table 1. Mean Value for Each Layer with Standard Deviation

Stratum (no. of tests)	**N_{60}-value in bpf from SPTs**			
	Mean	Std Deviation (σ)	Mean - σ	Mean + σ
Miami Limestone (619)	29	24	5	53
Upper Tamiami (253)	27	17	10	44
Lower Tamiami (72)	23	16	7	39
Peace River (64)	72	27	45	99

Table 1 shows statistically the large variation in measured N-values in the various strata as depicted in FSAR Figure 2.5.4-213 and Table 2.5.4-204. The variation of N-values within each stratum and its impact on stability analyses are discussed in the following paragraphs.

Miami Limestone (Stratum 2)

The large variation in N_{60}-values is to be expected given the partly cemented nature of the material. None of the engineering properties (including strength and high and low strain stiffness) presented in FSAR Table 2.5.4-209 were derived from the N-value, and thus any structural stability analysis that includes the Miami Limestone is not impacted by the variation of N-value. As noted in FSAR Subsection 2.5.4.5.1, the Miami Limestone below the nuclear island (NI) will be removed and replaced by concrete fill, and thus the Miami Limestone is not a factor in bearing capacity, settlement, or sliding analysis of the NI.

Upper and Lower Tamiami Formations (Stratum 5 and Stratum 6)

Unlike the Miami Limestone, these deep soil formations are expected to have consistent properties, and thus the large variations shown in N-values are not expected. The Tamiami Formation is a Miocene deposit, ranging in age from 1.6 to 5.3 million years. A soil deposit becomes more consistent with increasing depth and age. This is because the local differences caused by depositional or weathering variations are evened out as the soil is compressed under an ever increasing overburden. This is evidenced by the consistency of the shear wave velocity (V_s) measurements of the 10 Suspension P-S Velocity Logging borings performed in the Tamiami Formation (all completely penetrated the Upper Tamiami, and 5 penetrated the Lower Tamiami.) As noted in the response to RAI 02.05.04-11,

> *The Tamiami Formation does exhibit a generally increasing V_s profile with depth, as illustrated in FSAR Figure 2.5.4-220 and tabulated in Table 2.5.4-215. FSAR Figure 2.5.4-216 shows the fines content of the Tamiami Formation also increases with depth, and the increase of V_s with depth is probably more a function of this increase in fines rather than the overburden effect, which becomes less pronounced with increasing depth. The correlation with fines content appears to also apply to the Peace River Formation, where there is a slight decrease of V_s with depth; FSAR Figure 2.5.4-216 shows a steady decrease of fines content with depth.*

The consistency of the Tamiami Formation is also demonstrated by consistency of the CPT results in FSAR Figure 2.5.4-214, where tip resistance, sleeve friction, and pore pressure measurements follow a well-defined path.

As described earlier in this response, many of the N-values measured in the Tamiami Formation were considered to be affected by in-situ disturbance, giving reduced N-values. This accounts in large part for the variation of N-values seen in FSAR Figure 2.5.4-213. The parameters that form the basis of the bearing capacity analysis for the NI (angle of internal friction, ϕ, for the more granular Upper Tamiami, and undrained shear strength, c_u, for the much finer grained Lower Tamiami) were considered reasonable and conservative for these materials. The ϕ = 35 degrees for the Upper Tamiami is typical for medium dense sands with N-values, according to Bowles (Reference 2), in the 10 to 15 bpf range.

The c_u = 4 ksf assigned to the Lower Tamiami is lower than expected from a material of its age and depth and with the consistently high V_s and CPT tip resistances recorded. For settlement, the strains in the Tamiami were so low that the low strain values of elastic modulus derived from Vs were the dominant factor. At the depths of the Tamiami Formation, sliding is not a factor. In summary, the large variations in N-values shown in the Tamiami Formation are not considered to be accurate or realistic. Regardless, the Tamiami Formation parameters used in the stability analyses for the NI were not based on N-values.

Peace River (Stratum 7)

As noted above, while the very top portion of this stratum exhibited relatively low N-values, attributed to high artesian conditions, the lower portion of the stratum generally exhibited SPT refusal; resulting N-values were capped at 100 bpf. FSAR Figure 2.5.4-213 shows scatter of N-values in the middle portion of the stratum but within the range of scatter typically associated with the SPT test. Even including the lower values recorded in the upper portions of the stratum, the (mean – σ) value is 45 bpf, which is a dense sand (N > 30 bpf). The ϕ = 40 degrees given as a best estimate value for this stratum in FSAR Table 2.5.4-209 is still appropriate for N = 45 bpf. This stratum is so deep that it has no significant effect on the bearing capacity and settlement of the NI.

This response is PLANT SPECIFIC.

References:

1. Kulhawy, F.H., and Mayne, P.W., *Manual on Estimating Soil Properties for Foundation Design*, Report No. EL-6800, Electric Power Research Institute (EPRI), August 1990.
2. Bowles, J.E., *Foundation Analysis and Design*, Third Edition, McGraw-Hill, New York, 1982.

ASSOCIATED COLA REVISIONS:

FSAR Subsection 2.5.4.2.1.3.2.2 will be revised as follows in a future COLA revision:

2.5.4.2.1.3.2.2 N-Value Correction

Field SPT N-values are adjusted for SPT hammer energy, **borehole diameter (C_B), sampler (C_S), and rod length (C_R)**. This adjusted N-value, N_{60}, is determined using the following equation (Reference**s** 219 **and 225**):

$N_{60} = N\ C\varepsilon\ \mathbf{C_B\ C_S\ C_R}$ Equation 2.5.4-2

Where,

N = field measured SPT blow count
$C\varepsilon$ = hammer energy correction factor
C_B = borehole diameter correction factor
C_S = sampler correction factor
C_R = rod length correction factor

The SPT N-value used in correlations with engineering properties is a value traditionally based on 60 percent hammer efficiency. All 10 of the drill rigs employed in this subsurface investigation for SPT sampling use automatic hammers, which typically have efficiencies greater than 60 percent. SPT hammer energy measurements are made for each drilling rig/hammer employed, in accordance with ASTM D 6066 (Reference 220), and the hammer energy measurements (expressed as energy transfer ratios, or ETRs) are obtained. {As shown in Table 2.5.4-203, average ETRs range from 79.6 percent to 88.0 percent. The resulting energy correction factor, C_ε (expressed as ETR/60%), ranges from 1.33 to 1.47, also as shown in Table 2.5.4-203. N_{60}-values (from Equation 2.5.4-2) from each boring are corrected using the appropriate C_ε value. **Additional information on the correction factors for rod length, boring diameter, and soil sampler are provided in References 219 and 225.** The resulting SPT N-values are termed N_{60}. For the liquefaction analysis, an additional correction factor **for overburden pressure is** ~~are~~ applied.}

A summary of all N_{60}-values with depth is shown on Figure 2.5.4-213 and **in** Table 2.5.4-204.

FSAR Subsection 2.5.4.2.1.3.2.3 will be revised as follows in a future COLA revision:

2.5.4.2.1.3.2.3 Design N-Values

Table 2.5.4-209 presents N_{60}-values selected for design for each stratum, both within and outside the power block.

Note that as explained in Subsection 2.5.4.2.1.3.3, four cone penetration tests (CPTs) were performed through the Tamiami Formation. The CPT corrected tip resistance (q_t) was then used to derive the N_{60}-values based on the best estimate relationship of q_t/N with increasing fines content from Kulhawy & Mayne (1990) (Reference 282). Using an average q_t of about 160 tsf and a q_t/N ratio of 4 for the Upper Tamiami, a N_{60}-value of 40 bpf is estimated. Similarly, using an average q_t of about 110 tsf and a q_t/N ratio of 3.5, a N_{60}-value of 32 bpf is estimated for the Lower Tamiami. Considering the substantial depth of the Tamiami Formation, these N_{60}-values derived from the CPT results provide a better representation of the subsurface conditions in comparison with the SPT N_{60}-values in Table 2.5.4-204. However, considering the relatively high shear wave velocity values observed in these formations (FSAR Figures 2.5.4-218 and 2.5.4-220), even these N_{60}-values derived from the CPT results are probably conservative.

FSAR Subsection 2.5.4.8.2 will be revised as follows in a future COLA revision:

2.5.4.8.2 Liquefaction Resistance Based on SPT Data

As indicated on Figures 2.5.4**-212**~~-237~~ **and 2.5.4-213**, there is a very wide scatter of **un**corrected and ~~un~~corrected N-values. **The N_{60}-values vary** from 0 to 100 blows/foot in the upper Tamiami Formation and from ~~less than 5~~**3** to around ~~80~~ **100** blows/foot in the lower Tamiami Formation. Where SPT sampling encountered refusal, the N-value is capped at 100, so the actual range of penetration resistance is higher than these values indicate. There is no obvious correlation between N-value and elevation in these strata. Silty sands and sandy silts that range in depth from 120 to 220 feet would normally be dense to very dense with consistently high N_{60}-values. Blow counts of less than 20 blows/foot and particularly less than 5 blows/foot (including the zero values) are most probably due to sample disturbance. Subsection 2.4.12 describes the upward vertical hydraulic gradient observed in the water level measurements. It seems likely that this hydraulic gradient has contributed to at least partial blowout of the bottom of the hole prior to/during SPT sampling on many if not most of the samples. To evaluate where N-values are not representative of actual in-situ density conditions, the corrected N-values are compared to the CPT corrected tip resistance. The ratio of q_{c1}/N_1 for clean sands is typically 4 to 5 and for silty sands 3.5 to 4.5 based on the work presented in Reference ~~222~~ **282**. Figure 2.5.4-237 indicates the N-values relative to the predicted range based on the ratio of q_{c1}/N_1. As can be seen in the figure, very few of the N-values fall into the predicted range, supporting the theory that these blow counts are significantly affected by the hydraulic gradient. Therefore, the measured N-values are not used in the calculation of liquefaction potential in favor of the measured CPT and Vs results that are more consistent with each other and with expected values for deposits of similar age, depth, and overburden.

A new reference will be added to FSAR Subsection 2.5.4.13 as follows in a future COLA revision:

2.5.4.13 References

282. Kulhawy, F.H., and Mayne, P.W. "Manual on Estimating Soil Properties for Foundation Design," Report No. EL-6800, Electric Power Research Institute (EPRI), August 1990.

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-11 (eRAI 6006)

Figure 2.5.4-218 presents a plot of shear wave velocity measurements. Below the Fort Thompson formation, the soils are variously described as silty sands or silts and clays. However, the velocities do not show any change with depth. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please indicate the data that was used to construct this figure and explain the uniformity in shear wave velocity below the Fort Thompson formation.

FPL RESPONSE:

The shear wave velocity (V_s) profiles presented in FSAR Figure 2.5.4-218 are a compilation of the 10 suspension P-S velocity data sets, from B-601(DH), B-604(DH), B-608(DH), B-610(DH), B-620(DH) in Unit 6, and B-701(DH), B-704G(DH), B-708(DH), B-710G(DH), B-720G(DH) in Unit 7. As indicated in FSAR Subsection 2.5.4.4.2.1, these are the receiver to receiver V_s data that are given in FSAR Subsection 2.5.4, Reference 257. Figure 1(a) of this response provides a clearer picture of the variation of V_S within the Tamiami and Peace River Formations. This figure shows the average and standard deviation of V_S in these formations at each measured depth interval in the V_S borings. This figure is the basis for the V_S profile used for the site response analysis shown in Figure 1(b). Figure 1(b) is FSAR Figure 2.5.4-220. As described in the supplemental response to RAI 02.05.04-9, Figure 1(a) will be added to the FSAR.

As discussed in FSAR Subsection 2.5.1.2.2, the Tamiami Formation is Pliocene age while the Peace River Formation is Pliocene-Miocene age. V_s in soils is generally a function of overburden pressure, and so some increase in V_s would be expected with increasing depth. For these relatively homogeneous soils that were deposited millions of years ago and gradually consolidated under increasing overburden pressure, a relatively consistent shear wave velocity (V_s) profile is expected, with some increase with depth. The Tamiami Formation does exhibit a generally increasing V_s profile with depth, as illustrated in Figures 1(a) and 1(b) and tabulated in FSAR Table 2.5.4-215. FSAR Figure 2.5.4-216 shows the fines content of the Tamiami Formation also increases with depth, and the increase of V_s with depth may be more a function of this increase in fines rather than the overburden effect, which becomes less pronounced with increasing depth. The correlation with fines content appears to also apply to the Peace River Formation, where there is a slight decrease of V_s with depth; FSAR Figure 2.5.4-216 shows a steady decrease of fines content with depth. For the Peace River Formation, any increase of V_s due to increasing overburden pressure appears to be more than offset by the decrease in V_s due to the decrease in fines content.

In summary, the V_s data in FSAR Figure 2.5.4-218 are compiled from the receiver to receiver data from the 10 suspension P-S velocity data sets. There is some variation of V_s with depth, but this appears to be more related to fines content than overburden pressure.

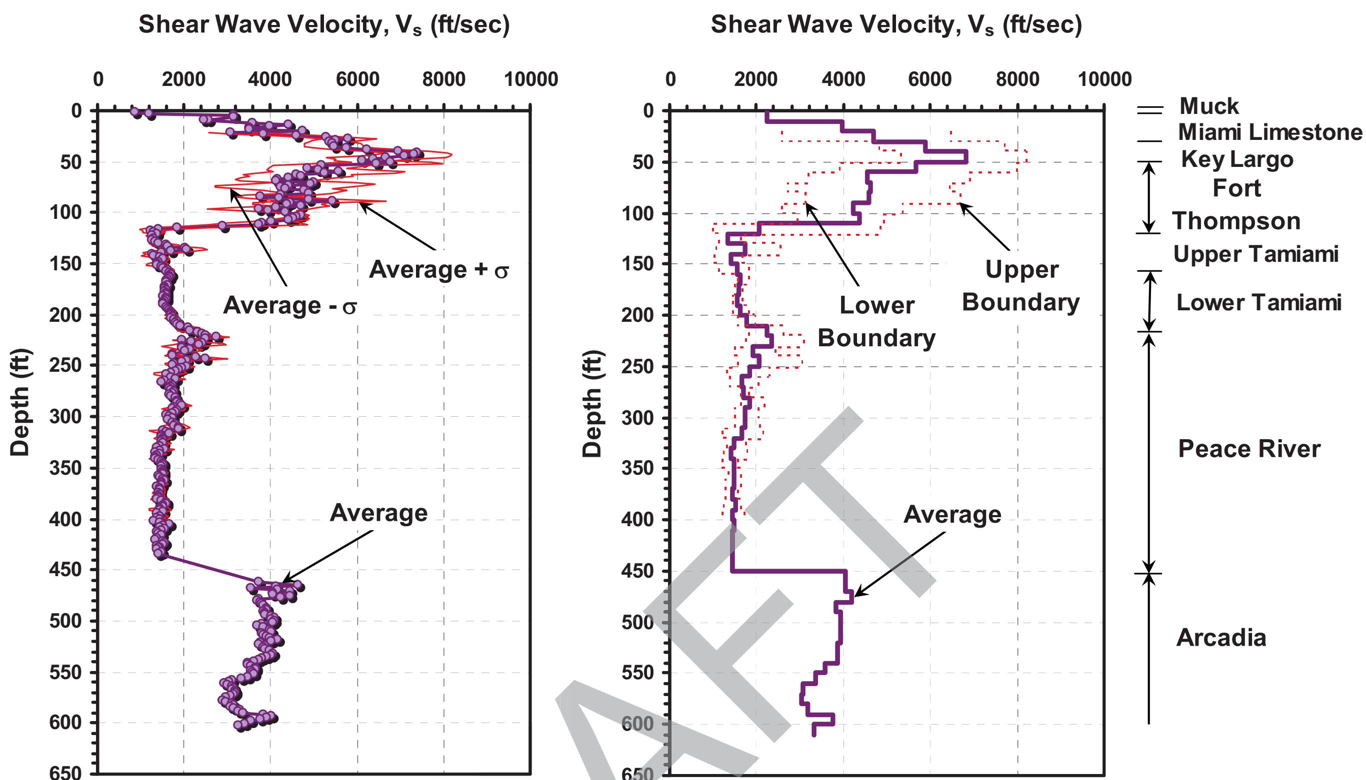

Figure 1. Shear Wave Velocities (a) Average and Standard Deviation of the Measured V_s; (b) Recommended Upper Boundary, Lower Boundary, and Average V_s

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

None

ASSOCIATED ENCLOSURES:

None