PMTurkeyCOLPEm Resource

From: Comar, Manny
Sent: Thursday, November 01, 2012 5:03 PM
To: TurkeyCOL Resource
Subject: FW: DRAFT RAI Responses FPL Turkey Point 6 & 7 for eRAI 6006 - Stability of Subsurface Materials and Foundations
Attachments: Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-17 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-10 (eRAI 6006).pdf

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From: Franzone, Steve [mailto:Steve.Franzone@fpl.com]
Sent: Friday, October 19, 2012 11:48 AM
To: Comar, Manny
Cc: Burski, Raymond; Maher, William; Franzone, Steve
Subject: RE: DRAFT RAI Responses FPL Turkey Point 6 & 7 for eRAI 6006 - Stability of Subsurface Materials and Foundations

Manny,

To support a future public meeting, FPL is providing draft revised responses for eRAI 6006 (RAI questions 02.05.04-10 and 02.05.04-17) in the attached files:

If you have any questions, please contact me.
Thanks
Steve Franzone
NNP Licensing Manager - COLA
"Never give in--never, never, never, never, in nothing great or small, large or petty, never give in except to convictions of honour and good sense. Never yield to force; never yield to the apparently overwhelming might of the enemy. " Sir Winston Churchill, Speech, 1941, Harrow School
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Hearing Identifier: TurkeyPoint_COL_Public
Email Number: 703

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MESSAGE	1637	11/1/2012 5:03:31 PM
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Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-10 (eRAI 6006).pdf	303989	

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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: 02.05.04-17 (eRAI 6006)

The calculation for “Site Response and Strain Compatible Properties Calculation” Rev. 001 describes the procedure used to calculate stresses in the liquefaction analysis. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," and Regulatory Guide (RG) 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites" please clarify the following regarding the methodology used to calculate the CSR (cyclic stress ratio):

a. Clarify how the method used for determining SR_{DRS} meets the ground motion level requirements for liquefaction analysis per 10 CFR 50, Appendix S. The *GRMS* initially resulted in a *PGA* of less than 0.1g and was scaled upwards per RG 1.208. Since the method used for determining the SR_{DRS} is the same as the *GRMS*, describe how this method provides stress ratio values that are comparable to those calculated using a *PGA* value of at least 0.1g.

b. Describe how the amplitude ratio $A_R(f)$, defined by (ARS_{10-5})/(ARS_{10-4}) and used in the determination of the weighting factor w, correlates to the ratio of the in-situ *stress ratios* resulting from site response analysis using the ARS_{10-5} and ARS_{10-4} as input spectrums.

c. The weighting factor *w* applied to the stress ratios SR_{10-4} and SR_{10-5} for the determination of SR_{DRS} is based on the average of the weighting factor *W(f).* Justify using an average value of *W(f)* over all frequencies, and describe how this is a conservative approach.

d. Describe how ARS_{10-5} and ARS_{10-4} are used as input to the *RVT* for site response, and how this approach correctly accounts for duration effects as compared to time series inputs for the determination of Cyclic Stress Ratio (*CSR)*. Please justify and provide the technical basis of this approach, including any assumptions.

e. Justify use of equations (77) and (78) from Idriss and Boulanger (2008) for determining q_{c1Ncs} values, and how the resulting values are conservative compared to the methods outlined in RG 1.198 using your calculated I_c values.

FPL RESPONSE:

Part a:

Clarify how the method used for determining SR_{DRS} meets the ground motion level requirements for liquefaction analysis per 10 CFR 50, Appendix S. The GRMS initially resulted in a PGA of less than 0.1g and was scaled upwards per RG 1.208. Since the method used for determining the SR_{DRS} is the same as the GRMS, describe how this method provides stress ratio values that are comparable to those calculated using a PGA value of at least 0.1g.

The factor of safety (FOS) against liquefaction is computed by dividing the strength (capacity) of the soil available to resist liquefaction (cyclic resistance ratio or CRR) by the stresses (demand) in the soil caused by the earthquake (cyclic stress ratio, CSR, or simply SR as used in the question). For Units 6 & 7, the evaluation of the soil strength was primarily based on cone penetration test (CPT) results. The methodology that was applied to the computation of FOS values in FSAR Subsection 2.5.4.8 utilized the CSR values obtained from the site-specific ground response (P-Shake) analysis. This RAI response re-evaluates the FOS against liquefaction by directly computing the CSR values using the Seed simplified equation (FSAR Subsection 2.5.4 Reference 219) with a peak ground acceleration (PGA) of 0.1g.

FSAR Figure 2.5.4-238, which is reproduced as Figure 1 in this response, is a compilation of the four CPT data sets and presents the FOS values that are based on the CSR values from P-Shake analysis as a function of elevation. The three lowest FOS values that correspond to the CPT measurements at three different elevations are selected from this figure for comparison purposes. The FOS of 1.92 at El. -137.7 ft, 2.11 at El. -231.5 ft, and 2.16 at El. -252.3 ft (NAVD 88) are tabulated below, along with their corresponding CSR values of 0.047, 0.043, and 0.043.

For re-evaluation purposes, the CSR values for the same data points are recalculated using a PGA of 0.1g. Using Equations 1, 2a and 2b of FSAR Subsection 2.5.4 Reference 219, the corresponding CSR values are computed as 0.060, 0.064, and 0.065, respectively. Thus, the CSR values based on a PGA of 0.1g are increased compared to those directly obtained from the site-specific P-Shake analysis. As a result, because there is an inversely proportional relationship between the CSR and FOS values, substituting the CSR values from 0.1g will reduce the corresponding FOS values. Thus, the FOS values computed using a PGA of 0.1g at El. -137.7 ft, -231.5 ft, and -252.3 ft are 1.50, 1.42, and 1.43, respectively. As indicated in FSAR Subsection 2.5.4.11, the minimum allowable FOS of 1.25 was conservatively selected as the trigger value for the liquefaction analysis of site soils. Given that RG 1.198 considers soils with a FOS value less than 1.1 as liquefiable, there is about 14% conservatism employed in the analysis (1.1 versus 1.25). Nonetheless, the FOS values of 1.50, 1.42, and 1.43 exceed the minimum allowable FOS of 1.25.

CPT	El. (ft)	P-Shake		PGA of 0.1g						
		CSR	FOS	Current depth (ft)	Finished depth (ft)	Total overburden pressure (ksf)	Effective overburden pressure (ksf)	Stress reduction coefficient r_d	CSR $a=0.1g$	FOS $a=0.1g$
C-601	-137.7	0.047	1.92	137.6	163.2	21.93	13.34	0.56	0.060	1.50
C-701	-231.5	0.043	2.11	230.1	257.0	33.21	18.76	0.56	0.064	1.42
C-701	-252.3	0.043	2.16	250.9	277.8	35.70	19.96	0.56	0.065	1.43

In summary, using a PGA of 0.1g in calculation of CSR values will reduce the FOS values against liquefaction. However, as observed above, the latter FOS values exceed the minimum allowable FOS of 1.25. Thus, no modifications are proposed to the approach presented in the FSAR.

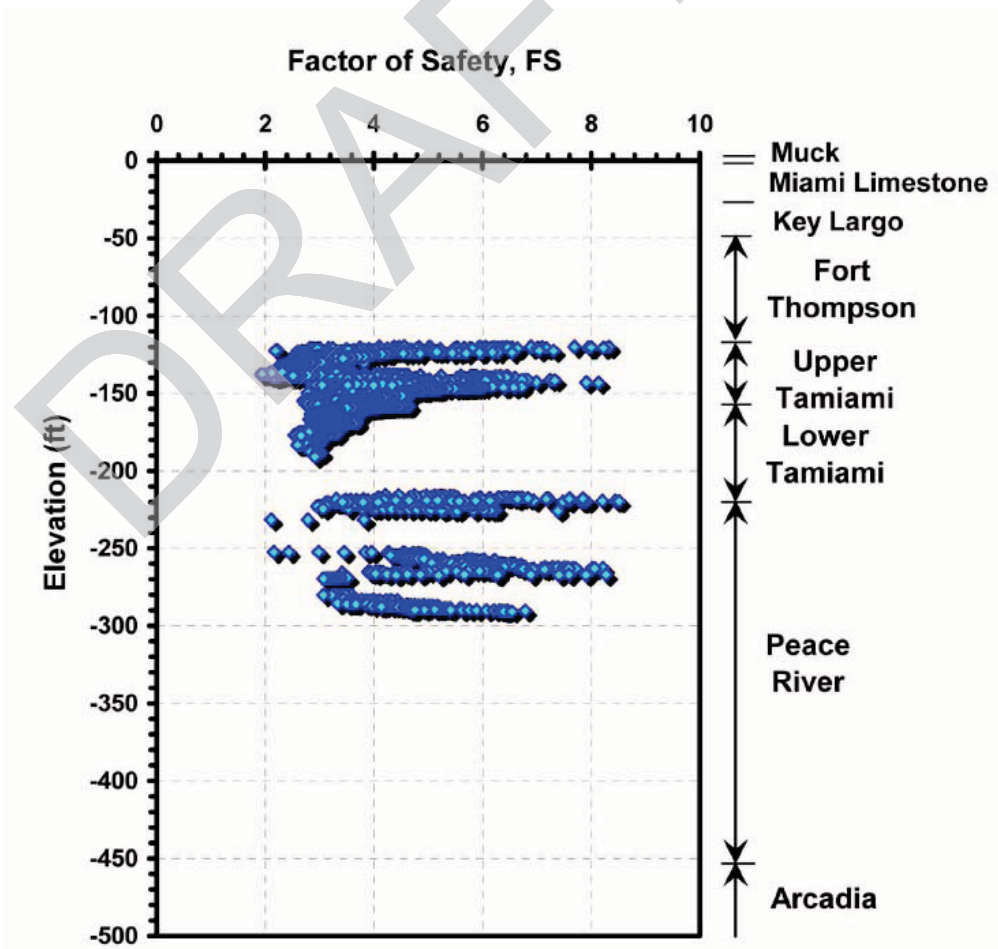

Figure 1
Factor of safety against liquefaction based on CPT values
(reproduced from FSAR Figure 2.5.4-238)

Part b:

Describe how the amplitude ratio $A_R(f)$, defined by (ARS_{10-5})/(ARS_{10-4}) and used in the determination of the weighting factor w, correlates to the ratio of the in-situ stress ratios resulting from site response analysis using the ARS_{10-5} and ARS_{10-4} as input spectrums.

The weighting factor approach results in stress ratios that are compatible with the design response spectrum (DRS), that is larger than stress ratios compatible with 1E-4 seismic motion, and lower than stress ratios compatible with 1E-5 seismic motion. The amplitude ratio $A_R(f)$ correlates positively to the weighting factor $W(f)$ as shown in Figure 2. Stress ratios are calculated using the following equation:

$$SR_{DRS} = \left(SR_{10-4}\right)^{1-\omega}\left(SR_{10-5}\right)^{\omega}$$

Where ω is the average $W(f)$ over the entire frequency range. Note that the larger the value of ω the smaller the contribution of SR_{10-4} and the larger the contribution of SR_{10-5}. Therefore, the amplitude ratio A_R also correlates positively to the resulting stress ratios SR_{DRS}.

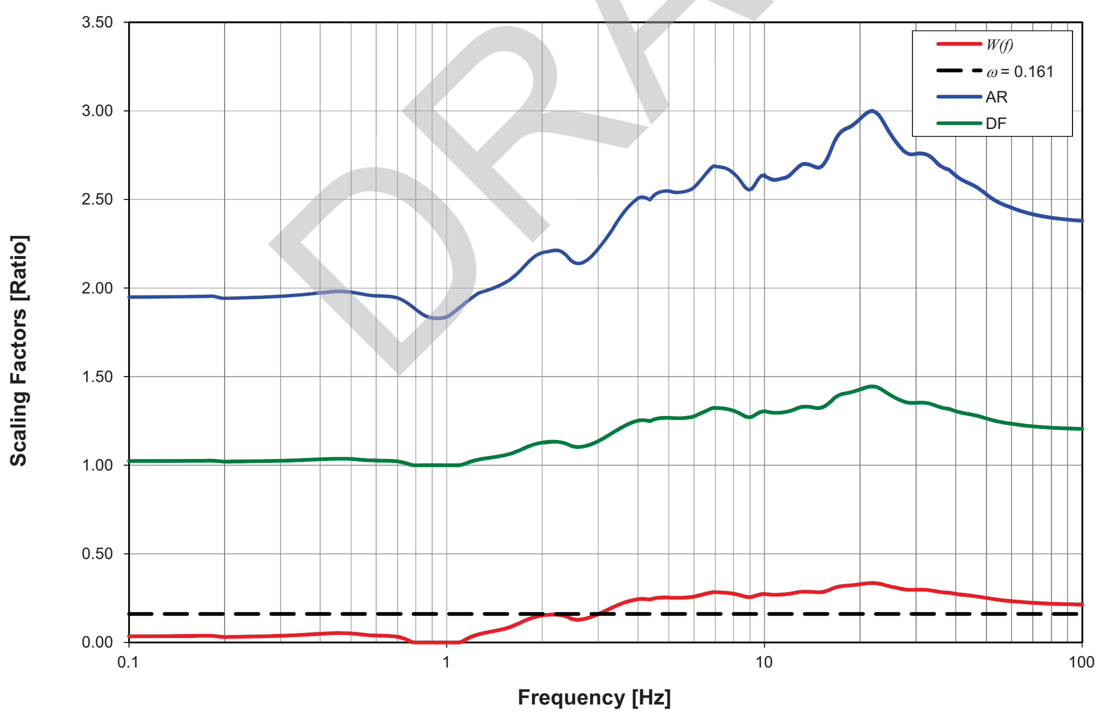

Figure 2

Part c:

The weighting factor w applied to the stress ratios $SR_{10\text{-}4}$ and $SR_{10\text{-}5}$ for the determination of SR_{DRS} is based on the average of the weighting factor W(f). Justify using an average value of W(f) over all frequencies, and describe how this is a conservative approach.

The average weighting factor ϖ is larger than the calculated frequency-dependent weighting factors below a frequency of 3 Hz, but smaller at frequencies larger than 3 Hz. To justify the adequacy of the adopted average weighting factor for the purpose of calculating conservative stress ratios for use in liquefaction analysis, the following was performed.

The site response analysis runs, using P-SHAKE, are repeated for the FAR soil column with an imposed cutoff frequency of analysis of 3 Hz, as opposed to the cutoff frequency of 100 Hz in the case of the original analysis. The analysis was performed for all 60 simulated profiles subjected to the low frequency (LF) and high frequency (HF) rock motions at the 1E-4 and 1E-5 hazard levels. The resulting stress ratios, which as in the case of the original analysis are the envelope of LF and HF results, are compared in Figure 3 at the 1E-4 and 1E-5 hazard levels. Noting that the top-most submerged granular soil layer, analyzed for potential liquefaction, is at a depth of around 145 ft (Elevation -120 ft), it follows that the contribution of the motion, with frequency content removed above 3Hz, is more than 90% of the total stress ratios for all layers.

From Figure 2, note that for frequencies below 3 Hz the calculated frequency dependent weighting factor is much smaller than the adopted weighting factor of 0.161 used in the stress ratio calculation, with a computed average weighting factor of 0.054 for frequencies below 3 Hz. Similarly, in the case of the near NI soil column, and for frequencies below 3 Hz, the average weighting factor is 0.060, which is smaller than the adopted weighting factor of 0.154 used in the stress ratio calculation.

It is therefore concluded that the adopted (larger) weighting factor is conservative and adequately compensates for any small contribution to the weighting factor from frequencies above 3 Hz, confirming the adequacy of the calculated stress ratios used in liquefaction analysis.

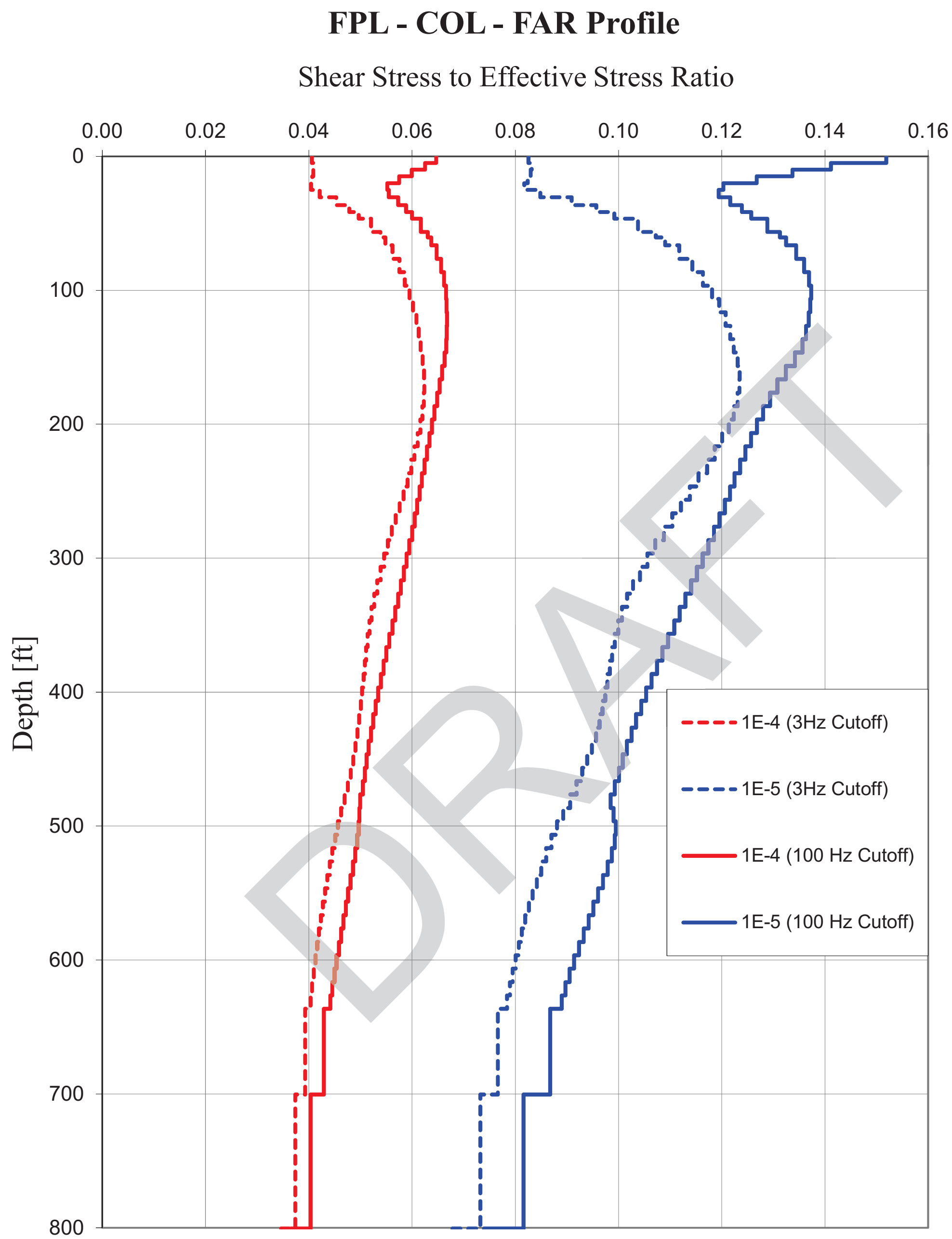
FPL - COL - FAR Profile
Shear Stress to Effective Stress Ratio
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
Depth [ft]
0
100
200
300
400
500
600
700
800
1E-4 (3Hz Cutoff)
1E-5 (3Hz Cutoff)
1E-4 (100 Hz Cutoff)
1E-5 (100 Hz Cutoff)
DRAFT

Figure 3

Part d:

Describe how ARS $_{10-5}$ and ARS $_{10-4}$ are used as input to the RVT for site response, and how this approach correctly accounts for duration effects as compared to time series inputs for the determination of Cyclic Stress Ratio (CSR). Please justify and provide the technical basis of this approach, including any assumptions.

The Random Vibration Theory (RVT) has been implemented in the Bechtel computer program P-SHAKE which is the new enhanced version of the program SHAKE2000. The program follows the same methodology and inherits the same assumptions used in the computer program SHAKE and seeks for the solution in frequency domain using the equivalent linear method to consider the soil nonlinear effects. Once the iteration on soil properties in each layer has converged, the final solution is obtained. The methodology has been checked and verified against the computer program SHAKE for a large suite of soil columns and input time histories. With respect to RVT implementation, the major steps used in P-SHAKE are as follows:

1. In RVT approach the input motion is provided in terms of acceleration response spectrum and its associated spectral damping. From the acceleration response spectrum, the acceleration power spectral density function is computed using the peak factor.
2. From the frequency domain solution of the soil profile (following SHAKE approach), the transfer function for strain in each layer is obtained and convolved with the power spectral density (PSD) of input motion to get the peak factor and the maximum strain in each layer. The equivalent uniform strain is obtained from the maximum strain and is used to obtain the new soil properties (soil shear modulus and damping) for the next iteration.
3. The iterations are repeated until convergence is reached in all layers to the convergence limit set by the user.
4. Once the final frequency domain solution is obtained, the acceleration response spectrum for each horizon can be computed from the solution using an inverse process of obtaining PSD from the acceleration response spectrum.

As discussed in FSAR Section 2.5.2.5.3, the duration of the input motion is specified as a parameter in P-SHAKE and is provided for different rock input motions in FSAR Table 2.5.2-226.

In addition, similar to the approach used in SHAKE, a ratio of 0.65 is used in P-SHAKE to calculate equivalent uniform strain, starting from maximum strain, which translates into the same ratio for the corresponding stress.

A technical paper describing the RVT approach used and the methodology for obtaining the peak factors can be found in Reference 1.

Part e:

Justify use of equations (77) and (78) from Idriss and Boulanger (2008) for determining q_{c1Ncs} values, and how the resulting values are conservative compared to the methods outlined in RG 1.198 using your calculated I_c values.

For Units 6 & 7, factor of safety (FOS) against liquefaction was based on primarily cone penetration test (CPT) results, which are less susceptible to soil disturbance (for example, hydraulic gradients) than standard penetration test results. The FOS against liquefaction is computed by dividing the strength (capacity) of the soil available to resist liquefaction (cyclic resistance ratio or CRR) by the stresses (demand) in the soil caused by the earthquake (cyclic stress ratio or CSR). The method by Youd et al. (FSAR Subsection 2.5.4 Reference 219) for performing liquefaction analysis referred to in RG 1.198 utilizes the “soil behavior type index (I_c)”, which is a function of the tip resistance (q_c) and sleeve friction ratio (R_f), to account for the effect of fines content on the estimate of CRR values. A recent variation of this method is suggested by Idriss and Boulanger (FSAR Subsection 2.5.4 Reference 268). Equations (77) and (78) from Idriss and Boulanger take into account the actual fines content of the soil based on laboratory measurements from recovered samples. Utilizing actual measured fines content in lieu of estimates based on I_c is considered more appropriate and therefore this modification was incorporated into the Youd et al. method (equations (77) and (78) from Idriss and Boulanger) to compute normalized cone penetration resistance (q_{c1Ncs}). The analysis took into account the best estimate measured fines content for the Upper and Lower Tamiami, and the Peace River Formations. The resulting values were used in the computation of CRR. Although the method with equations (77) and (78) from Idriss and Boulanger may not generate more conservative FOS values compared to those using the Youd et al method, the Idriss and Boulanger approach reflects the actual site conditions better as it accounts for the effects of measured fines content.

In order to demonstrate that the site soils essentially have no liquefaction potential, the liquefaction evaluation was also performed based on the field measurements of shear wave velocity (V_S) using the approach by Youd et al. As indicated in FSAR Subsection 2.5.4.8.3, the V_S measurements, taken generally at 1.6 to 1.7 foot depth intervals, were used for the computation of FOS against liquefaction with a total of 878 points considered. According to the liquefaction resistance criteria suggested by FSAR Subsection 2.5.4 Reference 219, soils with V_S higher than the 200-215 m/s (656-705 ft/s) range (range based on the fines content) are considered non-liquefiable. FSAR Figure 2.5.4-218 shows that all of the measured V_S to depths of 400 and 600 ft at Units 6 & 7, respectively, exceed 705 ft/s, with only a few values below 1,000 ft/s. Based on these measurements, the site soils are expected to have no liquefaction potential. As FSAR Table 2.5.4-218 demonstrates, the FOS computed based on V_S exceeds the minimum allowable FOS of 1.25, which was conservatively selected as the trigger value for the liquefaction analysis of site soils (see Part a of this response).

In addition, liquefaction resistance increases markedly with geologic age. Youd et al. indicate that pre-Pleistocene sediments (sediments older than 1.6 million years) are generally immune to liquefaction. The Tamiami Formation is Pliocene (1.6 to 5.3 million years old) and the Peace River Formation is Pliocene-Miocene (1.6 to 23.7 million years old). FSAR Subsection 2.5.4 Reference 269 proposes an age correction factor, C_A, that accounts for the low probability of liquefaction of older deposits. Although this factor was not applied in the liquefaction analysis, it would be approximately 2 to 2.5; therefore, use of this factor would increase the calculated factors of safety against liquefaction by a factor of 2 to 2.5. Thus, no modifications are proposed to the methodology presented in the FSAR. The factor of safety values tabulated in Part a are considered to be conservative.

This response is PLANT SPECIFIC.

References:

1. Nan Deng and Farhang Ostadan, “Random Vibration Theory Based Seismic Site Response Analysis,” The 14th World Conference on Earthquake Engineering, October 12-17, 2008, Beijing, China, Paper 04-02-0024.

ASSOCIATED COLA REVISIONS:

The second paragraph of FSAR Subsection 2.5.4.8.3 will be revised as follows in a future FSAR revision.

Table 2.5.4-218 is a summary of the results of the calculations. The native soils that indicate the lowest FOS values are those in the upper Tamiami Formation. However, the FOS values calculated indicate adequate resistance to liquefaction based on published criteria (FOS > 1.25). The FOS as a function of **elevation** ~~depth~~ for the CPT-based calculations is presented in Figure 2.5.4-238. As described above, even if liquefaction occurs, the thickness and stiffness of the overlying rock, lean concrete fill, and compacted limerock fill precludes the effects of liquefaction from reaching near the ground surface.

The footnote to FSAR Figure 2.5.4-238 will be deleted as follows in a future FSAR revision.

~~Data from Reference 257~~

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: 02.05.04-10 (eRAI 6006)

FSAR Figures 203 through 209 indicate one boring for each of the two Units extending to a depth of about 450'. Most other borings taken at the site extend to depths of only about 150'. Figure 2.5.4-220 presents information on shear wave velocity, including best estimate (BE) and upper/lower bound (UB/LB) values down to a depth of about 600'. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please indicate how you estimated variations in shear wave velocity based on only two readings over the deeper portion of the profile.

FPL RESPONSE:

The soil formations described in the FSAR are present between the depths of about 115 and 450 feet. The Upper Tamiami Formation extends from 115 to 160 feet depth and consists of dense to very dense silty sand. The Lower Tamiami Formation is present from 160 to 215 feet and consists of very stiff to hard sandy silt with minor amounts of silty clay. The Peace River Formation (part of the Hawthorn Group), which is underlain by the Arcadia Formation (rock), extends from 215 to 450 feet depth and is a very dense silty sand.

FSAR Figure 2.5.4-220 presents the mean shear wave velocity (V_S) profile with low/high end boundaries (mean plus/minus one standard deviation) as a function of depth and is a compilation of the 10 suspension P-S velocity data sets, comprising 5 data sets in Unit 6 (B-600 (DH) borehole series) and 5 data sets in Unit 7 (B-700 (DH) borehole series), where DH stands for Down-Hole. The response to RAI 02.05.04-9 describes how the statistical variation in the V_S profile was derived. The V_S measurements were taken in two borings that extended to a depth of 150 feet (B-604 and B-704), in two borings that extended to a depth of 200 feet (B-620 and B-720), and in four borings that extended to a depth of 250 feet (B-608, B-610, B-708, and B-710). V_S boring B-601 extended to a depth of about 400 feet, while B-701 extended to about 600 feet. Thus, the V_S measurements of the Upper Tamiami Formation were obtained in the entire thickness of the layer (i.e., to about 160 feet depth) in eight borings and down to 150 feet depth in two borings. In the Lower Tamiami Formation, the V_S measurements were taken in the entire thickness of the layer in six borings (i.e., to about 215 feet depth) and down to 200 feet depth in two borings. In the Peace River Formation, the V_S measurements were obtained in the entire thickness of the layer in one boring (i.e., to about 450 feet depth) and down to 400 feet depth in another boring. V_S measurements were made in one boring in the Arcadia Formation to about 600 feet depth. The response to RAI 02.05.04-9 describes that when only one boring remained, the standard deviation was not computed. Thus, there is no variation presented below 400 feet depth in FSAR Figure 2.5.4-220 (or FSAR Table 2.5.4-215). As noted in the next paragraph, 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.

To account for variations in the V_S profile across the site, 60 randomized profiles were generated using a stochastic model as discussed in FSAR Subsection 2.5.2.5.2. The average V_S profile in FSAR Figure 2.5.4-220 was used as the base case of the site response analyses. 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.

Amount of Testing Data

Testing data obtained at four sites will be compared.

Levy County Units 1 & 2, Turkey Point Units 6 & 7, Vogtle Units 3 & 4, and VC Summer Units 2 & 3 will each use AP1000 units.

Levy County has extensive moderately weak to moderately strong karstic limestone underlying up to 100 feet of soil.

Turkey Point can be considered a predominantly rock site (moderately weak to moderately strong) from a stability standpoint. The limestone rock is underlain by about 335 feet of dense coarse-grained and very stiff to hard fine-grained soil.

Vogtle is a soil site with about 90 feet of Upper Sand (removed during construction), underlain by about 65 feet of Blue Bluff Marl (hard clay, closer to mudstone), underlain by about 900 feet of the Lower Sand (very dense sand).

VC Summer is a hard rock site overlain by about 15 feet of partially and moderately weathered rock and about 45 feet of saprolite.

The field and laboratory testing at the Turkey Point site was performed under an approved QA program that meets the Code of Federal Regulations 10 CFR 50, Appendix B, and conforms to applicable sections and elements of ANSI/ASME NQA-1-1983 Edition with 1a-1983 addenda.

In-Situ Testing

<u>Standard Penetration Test (SPT) Borings</u>

Table 1 summarizes the borings performed beneath the footprints of the nuclear islands (NIs) at each site. The NI contains the only safety-related structures in the AP1000 units. The depths of the borings are dependent on subsurface conditions.

Table 1. Summary of Borings

Site	No. of Borings	Total Depth Drilled, Ft	Depth Drilled, Ft			No. of Down-hole Geophysical Borings
			Avg.	Min.	Max.	
Levy County	41	9720	237	87	500	12
Turkey Pt [a]	22	4860	221	150	615	8
Vogtle [b]	12	2774	231	150	420	6
VC Summer [c]	13	2563	197	150	351	4

Notes:

[a]Observation well clusters installed next to B-606 and B-607 not included here. B-710 (DH) R to 15-ft depth not included.
[b]B-3004, B-3005, and B-4005 included here are 12 to 20 ft outside NI footprint.
CPTs performed next to B-3002 (DH) and B-4002 (DH) not included here.
[c]Observation well clusters installed next to B-205 and B-305 not included here. UD sample borings B-201 (UDP) & B-305 (UDP) in soil above rock not included.

Note that all four of the sites satisfy the RG 1.132 guidance for minimum number of borings beneath safety-related structures and the maximum boring depth. The maximum boring depth for the Turkey Point site based on the RG 1.132 guidelines is 285 feet.

The conclusion from the numbers in Table 1 is that there is no typical amount of exploration—the amount depends on the site conditions.

<u>Down-Hole Geophysical</u>

This testing was performed by GeoVision at all four sites, and the same suite of tests was used, including measurement of shear and compression wave velocity.

Table 2 summarizes the down-hole geophysical tests performed beneath the footprints of the NIs at each site.

Table 2. Summary of Down-Hole Geophysical Tests

Site	No. of Borings	Total Depth[a] Drilled, Ft	Depth Drilled[a], Ft		
			Avg.	Min.	Max.
Levy County	12	4219	352	265	500
Turkey Pt.	8	2415	302	164	615
Vogtle[b]	6	1570	262	250	420
VC Summer	4	1131	283	215	351

Notes:
[a]Depth tested for shear and compression wave velocities is about 15 ft less than depth drilled because of length of measuring equipment.
[b]A boring with down-hole V_s measurements was performed at the Vogtle site beyond the NI area to a depth of more than 1000 ft in conjunction with a fault study.

The conclusion from the numbers in Table 2 is that there is no typical amount of exploration— the amount depends on the site conditions.

Field Testing in Addition to Borings

The number and depth of these exploration points are dependent on subsurface conditions.

CPTs

No cone penetrometer tests (CPTs) were performed at the Levy County site. Pressuremeter tests in rock were performed in two borings, to a maximum depth of 129 feet.

Four CPTs were performed in or close to the power block area at the Turkey Point site. (Their locations were defined mostly by accessibility. The CPT rig is very heavy and had to stay on the primary access road constructed through the site. Because the site stratification is so uniform, it was not envisioned that the lateral differences in the locations of the CPT would affect the CPT readings. The consistency of the readings confirmed this.) These CPTs were run specifically to obtain data in the Tamiami and Peace River soil formations below the bottom of the limestone at about 115 feet. Three of the tests penetrated 100 to 110 feet below the bottom of rock to the bottom of the Lower Tamiami. The fourth test penetrated 170 feet below the bottom of the Fort Thompson Limestone, about 70 to 75 feet into the Peace River Formation. Part of the upper portion of the Peace River Formation had to be drilled through because the CPT refused. Note that the CPTs at the Turkey Point site were added to the exploration program because of the unsatisfactory results obtained from some of the SPTs as explained in the response to RAI 02.05.04-6.

Several CPTs, both static and seismic, were performed in or close to the NI at both the Vogtle and VC Summer sites. In all cases, these CPTs were terminated at or above the bearing strata. The average depth of CPTs in the power block areas at Vogtle and VC Summer are about 75 feet and 46 feet, respectively. All of the material penetrated by these CPTs in the power block area will be removed during plant construction. Thus, the data obtained are useful only for general subsurface classification purposes. The seismic cones produce V_s measurements in the soils that provide confirmation of the V_s values obtained from the down-hole suspension logging. The V_s values are used in developing free-field response spectra.

Observation Wells

At the Levy County site, 16 monitoring wells and 7 observation wells were installed. Depths ranged from about 32 to 154 feet.

At the Turkey Point site, 22 observation wells were installed, consisting of eight 2-well clusters and two 3-well clusters. Depths ranged from 25 to 136 feet.

At the Vogtle site, 15 single observation wells were installed to depths ranging from 90 to 247 feet.

At the VC Summer, 31 observation wells were installed, consisting of five 2-well clusters, with the remainder being single wells. Depths ranged from 32 to 141 feet.

Electrical Resistivity Tests

Electrical resistivity of the borehole environment was measured at all four sites in the down-hole geophysical borings. Electrical resistivity measurements were not made at the Turkey Point site because the existing surficial muck will be removed across the site, and the site will be built up to final grade at around El. +25 feet, with approximately 10 million cubic yards of structural fill. In the NI area, excavation will be down to El. –35 feet.

Test Pits

Two test pits were dug at the Turkey Point site to obtain bulk samples of the Miami Oolite for testing for use as structural fill. Most of the 10 million cubic yards of structural fill will be obtained from the offsite sources cited in FSAR Subsection 2.5.4.5.1.1.

Additional Geophysical Testing

At the Turkey Point site, additional geophysical explorations were performed for possible dissolution features. These consisted of a microgravity survey (11 survey lines using a gravimeter), a seismic refraction survey (each seismic array was 230 feet with 10-foot spacing), and multi-channel analysis of surface waves (MASW) using 11 survey lines with 24 geophones spaced at 4-foot intervals. At the Vogtle site, four refraction microtremor tests were conducted to measure V_s to effective depths of about 100 feet. In addition, 12 seismic CPTs were performed to depths ranging from 68 to 100 feet. At the VC Summer site, seven seismic CPTs were performed to depths ranging from 36 to 58 feet.

<u>Summary of Field Testing</u>

A review of the field exploration program for the four sites indicates differences in scope, mainly depending on the conditions being explored. As detailed in the response to RAI 02.05.04-2, many of the SPT N-values obtained in the Tamiami and Peace River Formations were discounted because of disturbance from an upward hydraulic gradient. Excellent results were obtained from the CPTs that were added to the scope to supplement the SPT results.

Laboratory Testing

RG 1.138, Section B (Discussion) states, “The course of site and laboratory investigations will depend on actual site conditions, the nature of problems encountered or expected at the site, and design requirements for foundations and earthworks. Therefore, a program should be made flexible and tailored to each site and plant design as the site and laboratory investigations proceed... Specific testing requirements and details of testing procedures will depend on the nature of the soils and rocks encountered.”

<u>Testing of Rock Cores</u>

The laboratory testing program recognized that, from a foundation stability standpoint, the strength of the concrete fill, Key Largo Limestone, and Fort Thompson Limestone (approximately 100 feet total thickness) beneath the NI foundation is a key parameter. The amount of strength testing of the Key Largo and Fort Thomson Limestones was comparable to the testing of the bedrock at the Levy County and VC Summer sites, based on the thickness of the rock strata tested. Table 3 compares the rock laboratory testing at all three sites.

Table 3. Comparison of Rock Lab Testing
Levy County, Turkey Point and VC Summer

Site	Stratum	Thickness, Ft (approx.)	No. of Tests	
			Compression	Unit Weight
Levy County	Limestone	400	209	209
Turkey Point	Key Largo	23	31	32
	Ft Thompson	66	46	56
	Combined	89	77	88
VC Summer	Sound Rock	About 300	95	97

Note: No. of compression and unit weight tests within the top 89 ft of the VC Summer sound rock = 75 and 77, respectively.

The response and the supplemental response to RAI 02.05.04-6 explain in detail the derivation of elastic and shear modulus values of the rock and the effects of assuming strain dependency of (a) the elastic modulus on the predicted settlement of the rock (negligible) and (b) the shear modulus on the results of the SHAKE analysis of the soil and rock column (none).

<u>Testing of Soil Samples</u>

Intact Samples

RAI 02.05.04-8 notes that samples from the deep soils beneath the rock (about 115 to 450 feet) will be extremely disturbed. This disturbance is because of the unavoidable pressure relief on the sample when it is extracted. In the case of the Tamiami Formation, the disturbance is exacerbated by porepressure imbalance. The only intact samples taken from the Tamiami and Peace River Formations were in B-630, where 27 tube samples were extracted between depths of 115 and 294 feet.

Seven of the samples (between 129 and 294 feet) were used for RCTS testing, and these results are presented in FSAR Figures 2.5.4-232 and 2.5.4-234. At the VC Summer site, three RCTS tests were performed in the in-situ soils, while at Vogtle nine RCTS tests were performed in the in-situ soil. No RCTS tests were performed at the Levy County site.

As described in the supplemental response to RAI 02.05.04-2, given the depth of the soils and the inevitable disturbance to the intact samples obtained, reasonable and conservative strength and stiffness parameters for the Tamiami and Peace River strata were obtained using established empirical correlations derived for V_s measurements and, to a lesser extent, CPT measurements. Such measurements are much less impacted by soil disturbance than soil samples. The 10 V_s boreholes (8 under the NIs and 2 under the turbine buildings) provide very consistent results in the deep soils.

One C-U test with porepressure measurements was performed on an intact sample from the Lower Tamiami Formation. The response to RAI 02.05.04-8 states that, "... the laboratory testing of 'undisturbed' samples of these materials was purposely limited." That response notes that the results from the C-U test were in line with the values given in the literature.

Disturbed Samples

The remainder of the tests in the deep soils at the Turkey Point site were classification tests. The numbers of tests are tabulated below, along with those for the upper strata. These tests were all performed on SPT samples. Sufficient tests were performed to characterize the grain size distribution, plasticity, chemical content and percentage calcite of each stratum. The grain-size results are presented in FSAR Figure 2.5.4-216 and show a clear pattern with depth for both the Tamiami and Peace River Formations.

Note that chemical tests were not performed for the two deepest strata because there is no possible contact with structures or piping. Tests on the muck were minimal because all muck will be removed from the site.

<u>Summary of Laboratory Testing</u>

The main focus of the laboratory testing at the Turkey Point site was strength testing of the rock formations that form the support for the NI foundation. The number of tests performed were similar to those at the VC Summer site, a predominantly rock site. Apart from RCTS testing, laboratory testing of the deep underlying soils focused on classification of the materials. Because intact samples of these materials could not be obtained, strength and stiffness properties were mainly derived from field tests. Table 4 shows a summary of the number of soil laboratory tests on disturbed soil samples.

Table 4. Summary of Laboratory Testing in Soils

Test	Number of Tests					
	Total	**Muck**	**Miami Limestone**	**Upper Tamiami**	**Lower Tamiami**	**Peace River**
Sieve only	119	1	54	37	15	12
Sieve + hydrometer	62	-	7	37	11	7
Atterberg limits	24	-	-	5	13	6
Moisture content	8	-	-	1	4	3
Chemical	14	1	5	8	-	-
Calcite	40	-	15	17	5	3
Specific gravity	11	-	5	4	-	2

Site Characterization

FSAR Figures 2.5.4-203 through 208 show subsurface profiles through each of the power block areas. All of the profiles demonstrate the very small variation in stratum thickness across the site. The stratigraphy is extremely even and well established. The mode of formation and geologic age of each stratum is described in FSAR Subsection 2.5.1. FSAR Table 2.5.4-209 provides a complete set of reasonable and conservative properties relevant to each stratum except for the muck, which will be completely removed. Based on the above, the site has been accurately and completely characterized from a geotechnical standpoint, and the site and laboratory investigations are fully compliant with NRC requirements under RG 1.132 and RG 1.138.

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

Change the first paragraph of FSAR Subsection 2.5.4.2.3 as follows:

Soil laboratory testing is conducted on approximately 178 disturbed (split-spoon), ~~7~~ **8** intact (tube), and 2 bulk samples (from test pits) obtained during the subsurface investigation. In addition, 88 selected rock core samples are tested for unconfined compressive strength, and two of these are tested with stress-strain measurements. A summary of the testing performed is provided in Table 4.3 of Reference 257. **The following table lists the number of each type of test performed on the disturbed soil samples:**

Test	**Number of Tests**					
	Total	**Muck**	**Miami Limestone**	**Upper Tamiami**	**Lower Tamiami**	**Peace River**
Sieve only	119	1	54	37	15	12
Sieve + hydrometer	62	-	7	37	11	7
Atterberg limits	24	-	-	5	13	6
Moisture content	8	-	-	1	4	3
Chemical	14	1	5	8	-	-
Calcite	40	-	15	17	5	3
Specific gravity	11	-	5	4	-	2

The testing is performed in accordance with the current respective ASTM standards, other standards, or documented test procedures where applicable. Sampling, handling, and transportation of samples are further described in Reference 257.

ASSOCIATED ENCLOSURES:

None