FPL

L-2015-127
10 CFR 52.3

April 17, 2015

U.S. Nuclear Regulatory Commission
Attn: Document Control Desk
Washington, D.C. 20555-0001

Re: Florida Power & Light Company
Proposed Turkey Point Units 6 and 7
Docket Nos. 52-040 and 52-041
Response to NRC Request for Additional Information Letter No. 082 (eRAI 7811)
SRP Section 02.05.04 – Stability of Subsurface Materials and Foundations

References:

1. NRC Letter to FPL dated February 18, 2015, Request for Additional Information Letter No. 082 Related to SRP Section 02.05.04 – Stability of Subsurface Materials and Foundations for the Turkey Point Nuclear Plant Units 6 and 7 Combined License Application
2. FPL Letter L-2015-123 to NRC dated April 10, 2015, Updated Schedule for FSAR Chapter 2 RAI Responses and FSAR Chapter 19

Florida Power & Light Company (FPL) provides, as attachments to this letter, its responses to the Nuclear Regulatory Commission's (NRC) requests for additional information (RAIs) 02.05.04-29, 02.05.04-30, and 02.05.04-32 provided in Reference 1. The attachments identify changes that will be made in a future revision of the Turkey Point Units 6 and 7 Combined License Application (if applicable).

In addition, RAI 02.05.04-35 target submittal was incorrectly identified as April 17, 2015 in Reference 2 and should have been identified as April 27, 2015.

If you have any questions, or need additional information, please contact me at 561-691-7490.

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

Executed on April 17, 2015.

Sincerely,

William Maher
Senior Licensing Director – New Nuclear Projects

WDM/RFB

D097
NRO

Attachment 1: FPL Response to NRC RAI No. 02.05.04-29 (eRAI 7811)
Attachment 2: FPL Response to NRC RAI No. 02.05.04-30 (eRAI 7811)
Attachment 2: FPL Response to NRC RAI No. 02.05.04-32 (eRAI 7811)

cc:
PTN 6 & 7 Project Manager, AP1000 Projects Branch 1, USNRC DNRL/NRO
Regional Administrator, Region II, USNRC
Senior Resident Inspector, USNRC, Turkey Point Plant 3 & 4

NRC RAI Letter No. PTN-RAI-LTR-082

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-29 (eRAI 7811)

In response to RAI 02.05.04-5, the applicant stated in Note 1 of Table 3 that the SPT based stiffness for Lower Tamiami Formation is too low compared to values obtained from other methods, thus, it is disregarded in the overall average. However, the applicant also indicated in its response to RAI 02.05.04-2 and FSAR Subsection 2.5.4.2.1.3.2.1 that the overall pool of "SPT N" values from both the initial and supplemental investigations are used in development of engineering parameters or engineering analyses. In accordance with 10 CFR 100.23, please provide the following to assist NRC staff's further review in this area:

1. Calculations of the SPT based stiffness for Lower Tamiami Formation obtained from both the overall (combined) data pool and the supplemental test data.
2. Basis on why the SPT based stiffness for Lower Tamiami Formation can be selected to ignore, while the SPT based stiffness is used in the overall average for Upper Tamiami Formation and Peace River Formation, given the facts that the stiffness for Upper Tamiami Formation by STP method is reasonably agreed with the values by other methods and the stiffness for Peace River Formation by STP method is relatively higher than the values by other methods.
3. Clarification of engineering parameters or engineering analyses that are developed based on and/or not based on the overall pool of "SPT N" values.

FPL RESPONSE:

Different methodologies, i.e., standard penetration test (SPT), cone penetration test (CPT), pressuremeter, CU triaxial, and shear wave velocity (through a proper reduction factor), are used to obtain the design large strain stiffness, hereafter referred to as stiffness, for each stratum.

In the response to RAI 02.05.04-28, the effect of disturbance due to overwashing and stress relief was discussed in relation to the reduced blow counts. This effect was most significant when discontinuous SPT sampling was conducted.

Low blow counts from the initial investigation were included while developing parameters such as stiffness and friction angle, which is conservative. However, SPT blow counts from initial investigation were not used in the liquefaction analysis, as the liquefaction analysis based on SPT is a point wise evaluation. The result would not be a conservative estimate of a performance parameter, such as stiffness, but a wrong conclusion that the soils may liquefy at certain depths.

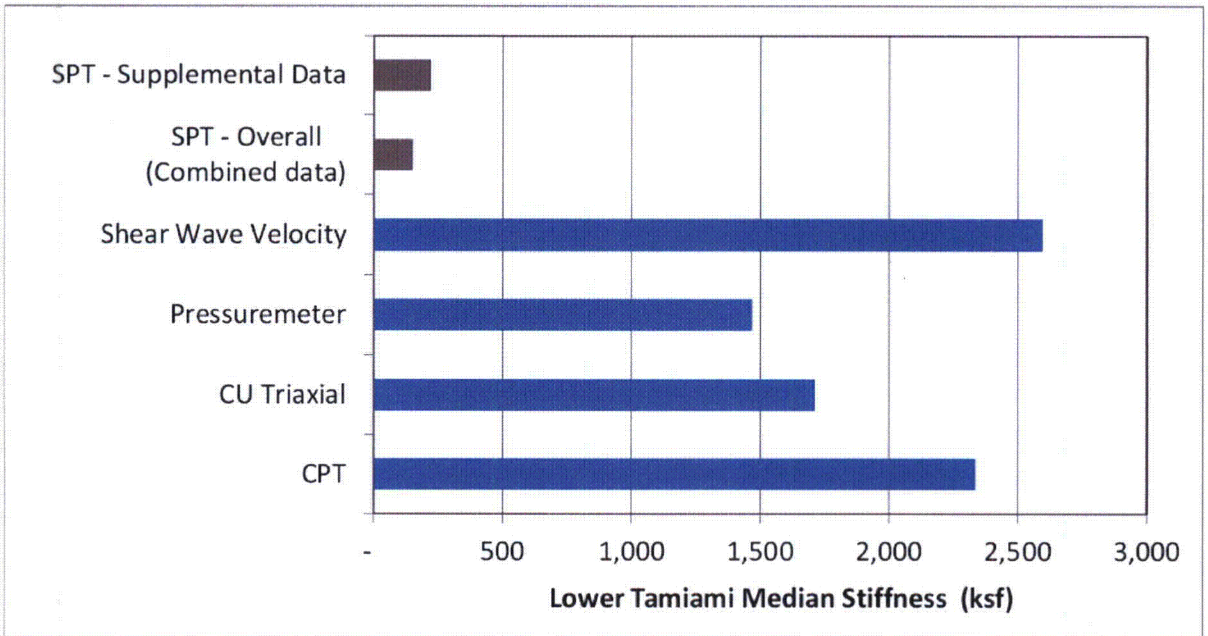
1. SPT based stiffness for the lower Tamiami Formation (overall data pool and supplemental test data) is included in Table 1 and Figure 1, which also includes stiffness as obtained from other methodologies for comparison.

It is evident from Table 1 and Figure 1 that SPT based stiffness is approximately an order of magnitude less than the average stiffness obtained considering other methodologies.

Table 1
Stiffness for Lower Tamiami Stratum

Statistical Parameters	Methodology					
					SPT	
	CPT	**CU Triaxial**	**Pressuremeter**	**Shear Wave Velocity**	**Overall (Combined) Data**	**Supplemental Data**
E_{median} (ksf)	2,337	1,713	1,468	2,595	151	221
E_{avg} (ksf)	2,363	1,624	1,696	2,768	175	242
E_{min} (ksf)	1,302	816	214	1,737	49	135
E_{max} (ksf)	4,917	2,196	5,628	6,150	572	362

Figure 1 Median Stiffness for Lower Tamiami Stratum



2. The correlations used to obtain the stiffness based on SPT data are from Reference 1. Reference 1 suggests SPT based stiffness correlations for different soil types according to the following general equation (Equation 1):

Es = C1 (N+C2) Equation 1

Where: N corresponds to N55. N55 is obtained from the corrected SPT N values (N60). Es units are kiloPascal (kPa).

From the suggested correlations in Reference 1 for different soil types, the correlation in Equation 2 is used for the silty soil (lower Tamiami Formation) and the correlation in Equation 3 is used for the sandy soil (upper Tamiami and Peace River formations).

Es = 300 (N+6) Equation 2

Es = 2600 (N) Equation 3

Reference 1 recommends correlations in Equations 2 and 3 to be checked based on site specific data, since there is considerable uncertainty associated with these correlations. Reference 3 also recommends the use of site specific correlations, such as pressuremeter-SPT to increase accuracy of the SPT correlations.

SPT based stiffness for silty soils, i.e., the lower Tamiami Formation considering the overall data pool is only about 10% to 15% of the average obtained from other methodologies. All methodologies except SPT produce consistent results for the lower Tamiami Formation. Therefore, the recommended silty soil correlation presented in Reference 1 (Equation 2) is not considered applicable for the site specific conditions.

It is noted that the levels of strain expected in the elevations of the lower Tamiami Formation due to static loads are substantially lower than the strains experienced by the soil during testing. The recommended stiffness is consistent with large strains, and is considered to yield a conservative representation of the stiffness behavior. Considering the correlation (Equation 2) for silty soils would lead to extreme conservatism. Instead of calibrating the equation for site specific conditions, the preferred approach is to exclude the results based on Equation 2 from the database in favor of the more reliable sources for stiffness characterization. The availability of four methodologies other than SPT, namely CPT, pressuremeter, shear wave velocity, and CU triaxial is considered adequate to estimate the stiffness for the lower Tamiami Formation.

Comparisons of stiffness as obtained from different methodologies for the other soil layers, i.e., sandy soils, are provided in Tables 2 and 3 and shown graphically on Figures 2 and 3. SPT based results for sandy soils, i.e., upper Tamiami and Peace River formations were found to be within the range or slightly higher compared with the other methodologies and therefore they were considered to obtain the design stiffness for the upper Tamiami and Peace River formations.

Table 2
Stiffness for Upper Tamiami Stratum

Statistical Parameters	Methodology				
	CPT	CU Triaxial	Pressuremeter	Shear Wave Velocity	SPT Overall (Combined data)
E_{median} (ksf)	2,294	622	530	2,277	1,172
E_{avg} (ksf)	2,228	622	724	2,400	1,335
E_{min} (ksf)	354	622	228	542	135
E_{max} (ksf)	3,666	622	2,879	14,097	4,507

Figure 2 Median Stiffness for Upper Tamiami Stratum

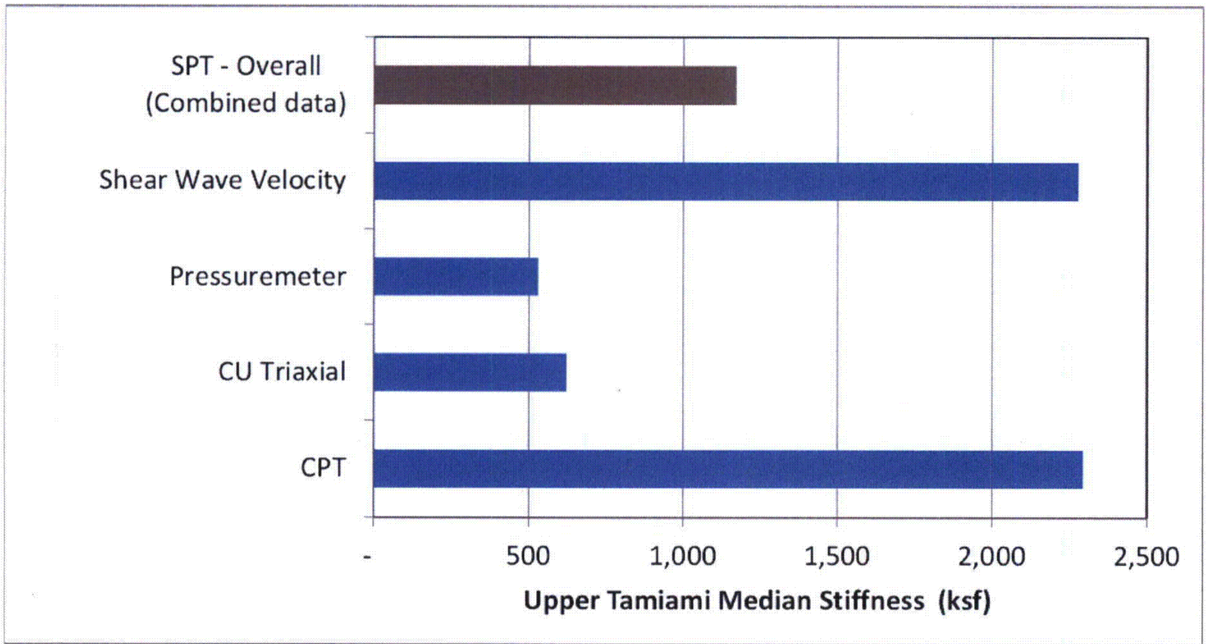
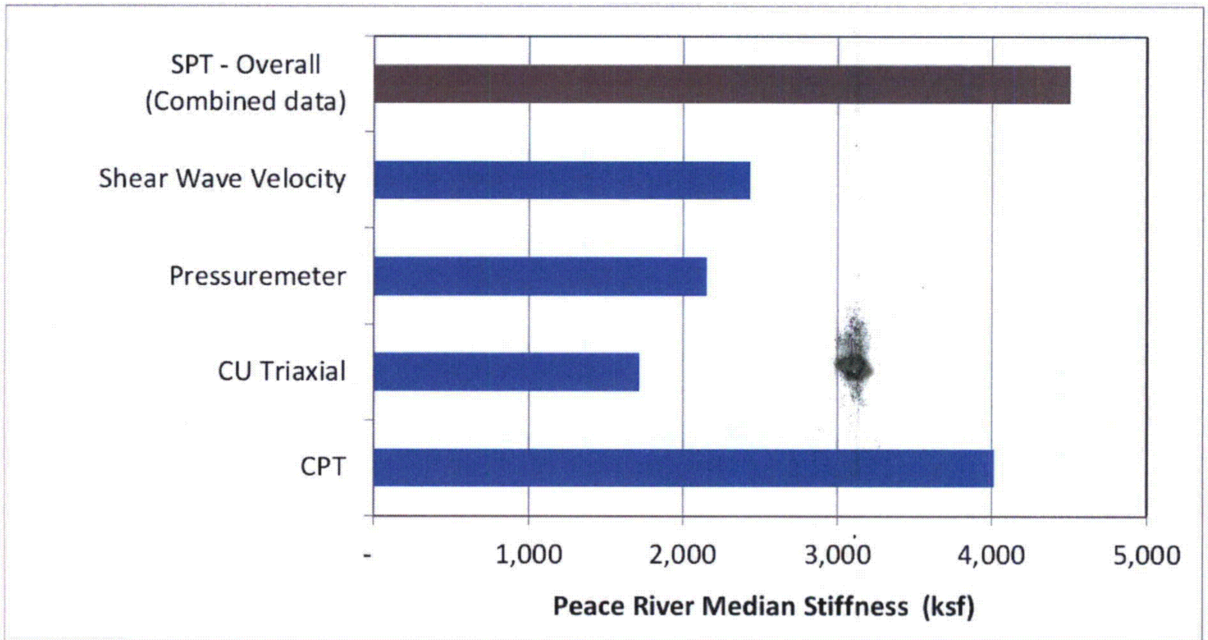


Table 3
Stiffness for Peace River Stratum

Statistical Parameters	Methodology				
	CPT	**CU Triaxial**	**Pressuremeter**	**Shear Wave Velocity**	**SPT Overall (Combined data)**
E_{median} (ksf)	4,016	1,568	2,153	2,432	4,507
E_{avg} (ksf)	3,926	1,716	3,863	2,911	3,689
E_{min} (ksf)	362	1,144	963	1,412	1,037
E_{max} (ksf)	6,528	2,572	13,422	10,974	4,507

Figure 3 Median Stiffness for Peace River Stratum



3. The engineering parameters or engineering analyses that use SPT results are shown in Tables 4 and 5.

Table 4
Engineering Parameters based on SPT results

Parameters	Formation	Summary	Stiffness based on SPT Correlation
Friction Angle	Upper Tamiami	Shear strength parameters (cohesion and friction angle) for soils were obtained based on CU Triaxial tests for the lower Tamiami (five samples) and Peace River (15 samples) Formations. Since only one triaxial test was performed for the upper Tamiami Formation due to the difficulty to obtain samples for testing, an average friction angle was obtained from the CU Triaxial test and correlations from CPT and SPT results (overall pool results).	Reference 2
Static Elastic Moduli	Upper Tamiami Peace River	The design stiffness was obtained as the geometric mean value of the median stiffness values from each of the following tests: Pressuremeter, CU Triaxial, CPT, P-S Suspension loggings, and SPT (overall pool results).	Reference 1

Table 5
Engineering Analyses based on SPT results

Analyses	Summary
Liquefaction based on SPT	SPT based liquefaction analysis considered both blow counts, 2+3 and 3+4, of the supplemental investigation.
Bearing Capacity	Friction angle (see Table 4) for the upper Tamiami Formation (overall pool results) is used in this analysis.
Settlement	Design stiffness is obtained including stiffness based on SPT for the upper Tamiami and Peace River formations (overall pool results).
Lateral Variability	The lateral uniformity of subsurface conditions included the following soil parameters obtained based on SPT results: friction angle and static elastic modulus.

This response is PLANT SPECIFIC.

References:

1. Bowles, J., Foundation Analysis and Design, 5th ed., McGraw-Hill Companies, Inc., 1996.
2. Kulhawy, F.H., and P.W. Mayne, Manual on Estimating Soil Properties for Foundation Design, Electric Power Research Institute, Palo Alto, CA, August 1990.
3. Phoon, K.K. and F.H. Kulhawy, Evaluation of Geotechnical Property Variability, Canadian Geotechnical Journal, 36: pp. 625-639, 1999.

ASSOCIATED COLA REVISIONS:

None

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-082

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-30 (eRAI 7811)

In response to RAI 02.05.04-9, the applicant conducted sensitivity study on bearing pressure uniformity. For the laterally variable case, the applicant assumed a scenario, in which, half of the foundation rests on the softer soil/rock column with lower bound (LB) parameters and half of the foundation rests on the stiffer soil/rock column with the best estimated (BE) parameters. In accordance with 10 CFR 100.23, please justify why upper bound (UB) parameters is not considered for the stiffer soil/rock column in the evaluation for the worst-case scenario.

FPL RESPONSE:

In the Response to RAI 02.05.04-09, the following excerpt was included from the AP1000 DCD regarding the "Lateral Variability":

> *Soils supporting the nuclear island should not have extreme variations in subgrade stiffness. This may be demonstrated by one of the following:*
>
> 1. *Soils supporting the nuclear island are uniform in accordance with Regulatory Guide 1.132 if the geologic and stratigraphic features at depths less than 120 feet below grade can be correlated from one boring or sounding location to the next with relatively smooth variations in thicknesses or properties of the geologic units, or*
> 2. *Site-specific assessment of subsurface conditions demonstrates that the bearing pressures below the nuclear island do not exceed 120% of those from the generic analyses of the nuclear island at a uniform site, or*
> 3. *Site-specific analysis of the nuclear island basemat demonstrates that the site-specific demand is within the capacity of the basemat.*

In the Response to RAI 02.05.04-09, it was shown that the first requirement regarding stratigraphic uniformity was met. Although the DCD states that lateral variability can be demonstrated by meeting one of the requirements, an additional analysis was conducted to show that second requirement about bearing pressures was also met.

As part of the bearing pressure analysis provided in the Response to RAI 02.05.04-09, an unlikely laterally variable case was assumed where half of the foundation rested on a softer soil/rock column with lower bound properties, and half of the foundation rested on a stiffer soil/rock column with best estimate properties (BE/LB case). This case was considered to be conservative.

After review of the Response to RAI 02.05.04-09, the staff requested an additional bearing pressure analysis for the laterally variable case in which half of the foundation rests on a softer soil/rock column with lower bound properties, and half of the foundation rests on a stiffer soil/rock column with upper bound properties (UB/LB case). The likelihood of realization of this case is considered to be extremely unlikely, and this case

is considered to be too conservative. However, another bearing pressure sensitivity analysis is conducted to consider this scenario.

The upper bound properties are developed using a similar methodology as the lower bound properties (as discussed in the Response to RAI 02.05.04-09 and FSAR Subsection 2.5.4.2.1.3.19).

The upper bound and lower bound properties are provided in Tables 1 and 2, below. Transition zone properties are taken as the average of the upper bound and lower bound properties.

Table 1
Lower Bound Properties

Strata		Unit Weight (kcf)	Cohesion c' (ksf)	Friction Angle (degree)	E (ksf)
Unfractured (FD1)	Miami Limestone	0.119	1.15	55	12,343
	Key Largo	0.125	8.78	55	47,918
	Fort Thompson	0.125	3.02	50	20,407
Fractured (FD4)	Miami Limestone	0.119	0.40	54	7,667
	Key Largo	0.125	2.32	55	16,723
	Fort Thompson	0.125	1.96	49	14,062
Upper Tamiami		0.116	0.00	29	702
Lower Tamiami		0.115	0.25	24	1,280
Peace River		0.117	0.00	27	1,685
Arcadia		0.125	0.00	27	107,650

Table 2
Upper Bound Properties

Strata		Unit Weight (kcf)	Cohesion c' (ksf)	Friction Angle (degree)	E (ksf)
Unfractured (FD1)	Miami Limestone	0.131	9.55	58	57,630
	Key Largo	0.149	26.32	57	117,593
	Fort Thompson	0.149	26.32	55	77,832
Fractured (FD4)	Miami Limestone	0.131	2.23	56	16,043
	Key Largo	0.149	6.03	57	44,451
	Fort Thompson	0.149	7.59	54	36,705
Upper Tamiami		0.124	0.00	39	1,811
Lower Tamiami		0.118	1.35	37	2,748
Peace River		0.124	3.73	41	4,472
Arcadia		0.133	3.73	41	190,732

The results of the UB/LB case are shown on Figures 1 and 2, where results from the BE/LB case (as presented in the Response to RAI 02.05.04-09) are also provided for reference. Figure 1 provides the bearing pressure distributions, while Figure 2 provides the percent bearing pressure difference with respect to the laterally uniform case. Case 1 represents the laterally uniform case, while Cases 2, 3, and 4 vary the width of the transition zone as one-third, one-fifth, and one-sixteenth of the foundation width, respectively. For both the UB/LB and BE/LB cases the maximum difference in bearing pressures is less than 20 percent, which meets the DCD criterion despite the extremely unlikely nature of the scenario and high level of conservatism involved in the analysis.

Figure 1 Bearing Pressure Distributions for Cases 1 through 4

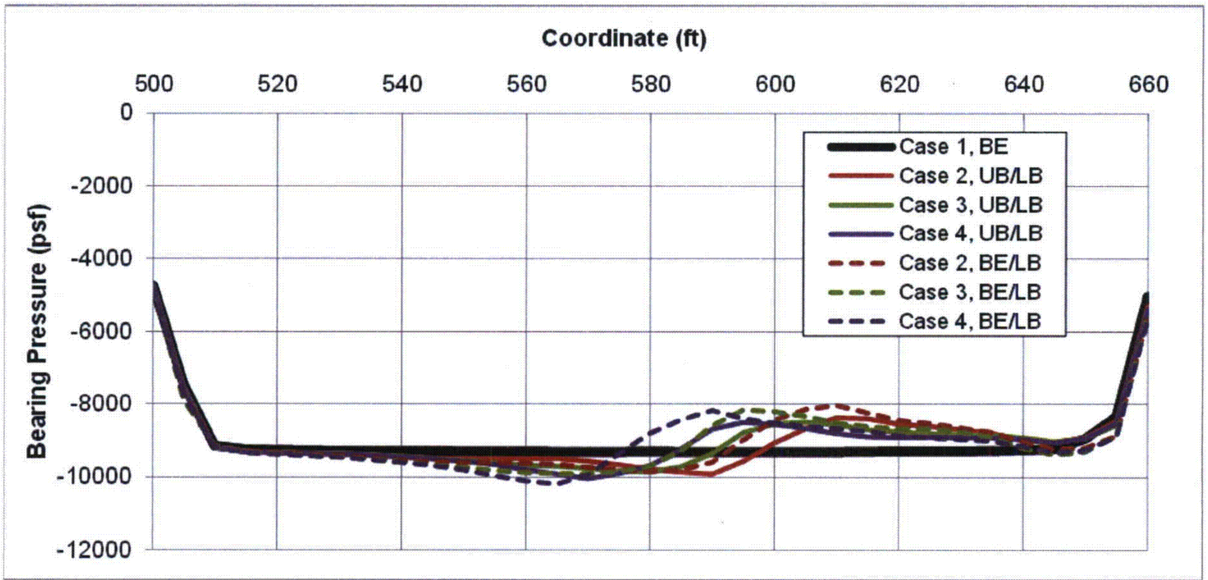
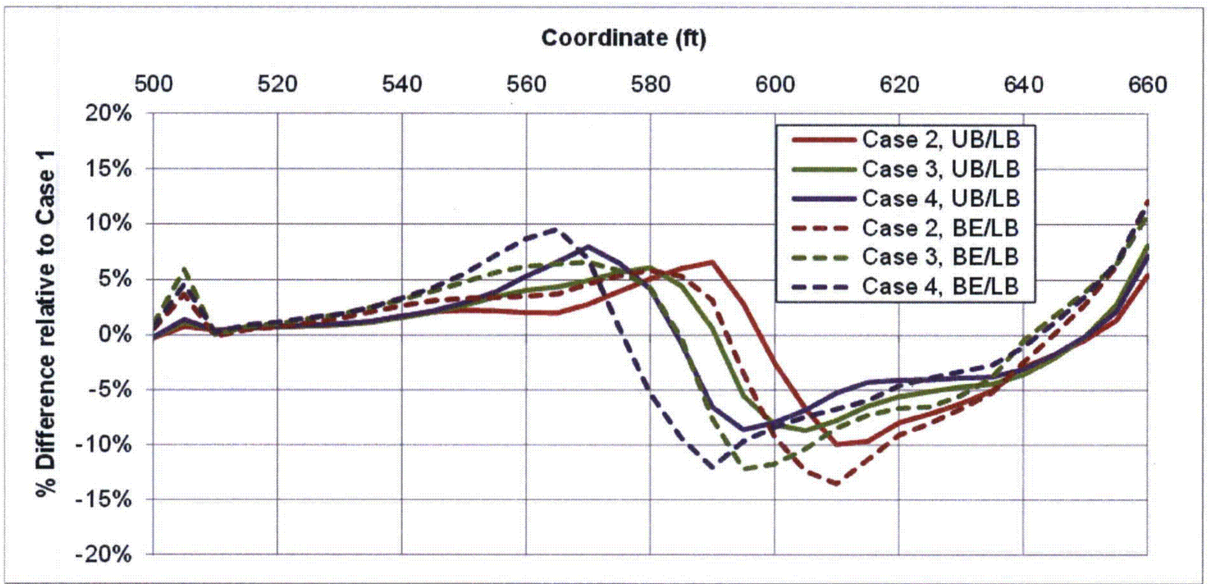


Figure 2 Percent Difference in Bearing Pressure Distribution of Cases 2, 3, and 4 Relative to Case 1



This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

None

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-082

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-32 (eRAI 7811)

FSAR Subsection 2.5.4.5.1 indicated three offsite structural fill sources. The applicant stated that "Each of these sources, as well as onsite material excavated from the power block excavations, offers Miami Limestone (Stratum 2) material and other limestone derived materials in granular form. This material is locally known as limerock." The applicant further stated that "The results of laboratory index tests (natural moisture content, gradation), chemical tests (pH, sulphate content, chloride content), moisture-density relationship tests (modified Proctor compaction), and strength tests (LBR and CBR) for these materials are contained in Appendix E.1 of Reference 257." Regulatory Guide 1.206 Section C.I.2.5.4.5, "Excavations and Backfill", states that the applicant should discuss sources and quantities of backfill and borrow, including a description of exploration and laboratory studies and the static and dynamic engineering properties of these materials. In accordance with 10 CFR 100 and given that the materials from offsite sources are potential Category I engineered fill, please provide the following to assist NRC staff's further review in this area:

1. Clarification of any material tests conducted for offsite structural fill sources.
2. Similarities and dissimilarities of offsite "limerock" with onsite Miami Limestone, including a description of exploration and laboratory studies and the static and dynamic engineering properties of these materials.
3. Adequacy of fill material to borrow for Category I engineered fill.

FPL RESPONSE:

Part 1)

As discussed in FSAR Subsection 2.5.4.5.1.1, three potential sources for offsite structural fill have been identified. The onsite backfill sources correspond to the limestone material that will be excavated at the project site, i.e., Miami Limestone and Key Largo Limestone materials. The compaction properties of the Miami Limestone backfill source have been evaluated; the material was sampled via two test pits, and the results are provided in Appendix E.1 of FSAR Subsection 2.5.4 Reference 257, and are summarized in FSAR Table 2.5.4-214. Chemical tests for onsite sources have been performed and the results are summarized in FSAR Table 2.5.4-205. Based on onsite chemical test results presented in FSAR Table 2.5.4-205 and the guidelines for the evaluation of soil chemistry presented in FSAR Table 2.5.4-211, both the Miami Limestone and the Key Largo Limestone have a sulfate content that is classified as mild against concrete. Laboratory compaction tests will be performed for the Key Largo limestone and offsite structural fill sources and chemical tests will be performed for offsite structural fill sources prior to earthwork operations.

As stated in FSAR Subsection 2.5.4.5.3, borrow sources are qualified prior to earthwork operations by testing for index properties, chemical properties, and engineering properties, especially: grain size and plasticity; soil pH, sulfate content, chloride content and moisture-density relationships. Further, FSAR Subsection 2.5.4.5.3 states that the backfill around the nuclear island structures (Category I engineered fill) will be compacted to a minimum of 95 percent of modified Proctor maximum dry density. These requirements as well as fill placement and compaction control procedures will be included in a technical specification that will be prepared during the detailed project design phase.

The Inspection, Tests, Analyses, and Acceptance Criteria (ITAAC) to ensure that onsite and offsite (if necessary) Category I fill sources meet the required compaction criteria are provided in Table 1 in addition to the chemical requirements to protect the safety related concrete that is in contact with the backfill.

Table 1
ITAAC for Category I Engineered Fill

Design Commitment	Inspections, Tests, and Analyses	Acceptance Criteria
Category I backfill is compacted to meet a minimum of 95 percent of modified Proctor maximum dry density.	Required testing will be performed before and after placement of the backfill materials.	Category I backfill is compacted to a minimum of 95 percent of modified Proctor maximum dry density.
Category I backfill meets the soil chemistry evaluation guidelines for mild to moderate concrete exposure to sulfate as provided in FSAR Table 2.5.4-211.	Required testing will be performed before and after placement of the backfill materials.	The water soluble sulfate (SO_4) concentration in Category I backfill does not exceed 0.20 percent by weight.

Part 2)

Any offsite backfill material, or “limerock,” if necessary for Category I engineered fill will be obtained from local quarries. As discussed in FSAR Subsection 2.5.4.5.1.1, the identified offsite structural fill sources offer local “limerock,” or crushed limestone material, which can be graded into a variety of grain size distributions. The quarries identified as likely offsite structural fill sources are all located within Miami-Dade County. As shown in FSAR Figure 2.5.1-201, Miami Limestone material is present throughout Miami-Dade County; therefore the local quarries identified are expected to have Miami Limestone material available. Geologists or geotechnical engineers will verify from visual observation (i.e., texture, hardness, etc.) and laboratory testing, as indicated in

FSAR Subsection 2.5.4.5.3, that the offsite backfill material is acceptable for use. Further, the ITAAC presented in Part 1 of this response will confirm that offsite backfill meets required compaction and chemical criteria.

Part 3)

In order to determine the quantity of an available onsite backfill material for Category I engineered fill, a cut-fill calculation has been performed. Approximately 67,000 yd3 of limestone or limerock will be excavated for each unit, and 64,700 yd3 of Category I fill is required for each unit. Therefore, onsite limerock is likely adequate in terms of amount, however offsite sources may be necessary for Category I fill.

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

The second paragraph of FSAR Subsection 2.5.4.2.1.2.9 will be revised in a future COLA revision as follows:

2.5.4.2.1.2.9 Compacted Limerock Fill

The muck layer underneath the power block area is removed and replaced with compacted limerock fill from onsite excavated Miami Limestone, **Key Largo,** and offsite sources, with fill placement starting from approximately El. –5 feet and building up to El. +25.5 feet. Excavations and fill on other areas of the site as described in Subsections 2.5.4.3 and 2.5.4.5 are completed. All other non-Category I structures are supported on compacted limerock fill.

The first paragraph of FSAR Subsection 2.5.4.2.1.4.1 will be revised in a future COLA revision as follows:

2.5.4.2.1.4.1 Laboratory Chemical Testing and Evaluation

Twenty-three sets of chemical analysis, consisting of pH, chloride content, and sulfate content, are performed on samples from the power block areas. Depths range from ground surface to approximately 155 feet. Samples tested are from the muck/peat, Miami Limestone, Key Largo Limestone, Fort Thompson Formation, and upper Tamiami Formation. Test results are summarized in Table 4.6 of Reference 257 and Table 4 of Reference 290. As noted in Subsection 2.5.4.5.1, the nuclear island is supported on lean concrete backfill and surrounded by limerock structural fill. Buried piping, duct banks, etc. are founded in limerock structural fill placed from about El. –5 feet (bottom of excavated muck) to El. +25.5 feet (final plant grade). ~~Thus, the chemical properties of the in-situ soils discussed in the following paragraphs do not impact the nuclear island or buried utilities in the power block area.~~

The first paragraph of FSAR Subsection 2.5.4.5.1.1 will be revised in a future COLA revision as follows:

2.5.4.5.1.1 Replacement of Stratum 1 with Compacted Limerock Fill

Due to the poor soil properties of Stratum 1 (muck/peat), Stratum 1 is removed in its entirety prior to commencing the major earthwork and grading operations. After removing the muck/peat, the grade is raised to approximately El. +0 feet through placement and compaction of Miami Limestone **and Key Largo** fill material**s** and limerock material from other sources.

The first paragraph of FSAR Subsection 2.5.4.7.3.4 will be revised in a future COLA revision as follows:

2.5.4.7.3.4 Dynamic Properties of Structural Fill

The muck layer underneath the power block area at Units 6 & 7 is removed and replaced with compacted limerock fill from onsite excavated Miami Limestone**, Key Largo,** and offsite sources, with fill placement starting from El. –5 feet and building up to El. +25.5 feet. Non-Category I structures are supported on compacted structural limerock fill.

FSAR Subsection 14.3.3.6 will be added in a future COLA revision as follows:

14.3.3.6 Category I Engineered Fill

The ITAAC for Category I Engineered Fill have been developed to ensure that the static and dynamic properties of the in-place backfill material will be the same as, or better than the design parameters. By specifying the expected compaction specifications of backfill material, this ITAAC provides a method to confirm that the aforementioned static and dynamic properties of said backfill are met prior to the construction of the Seismic Category 1 structure. By specifying the expected chemistry specifications of backfill material, this ITAAC provides a method to confirm that the chemistry of the backfill material meets acceptable requirements for concrete durability.

The following ITAAC will be added to the COLA, Part 10, Appendix B:

Table 3.8-6
ITAAC for Category I Engineered Fill

Design Commitment	Inspections, Tests, and Analyses	Acceptance Criteria
Category I backfill is compacted to meet a minimum of 95 percent of modified Proctor maximum dry density.	Required testing will be performed before and after placement of the backfill materials.	Category I backfill is compacted to a minimum of 95 percent of modified Proctor maximum dry density.
Category I backfill meets the soil chemistry evaluation guidelines for mild to moderate concrete exposure to sulfate as provided in FSAR Table 2.5.4-211.	Required testing will be performed before and after placement of the backfill materials.	The water soluble sulfate (SO_4) concentration in Category I backfill does not exceed 0.20 percent by weight.

ASSOCIATED ENCLOSURES:

None