FPL.

L-2015-130
10 CFR 52.3

April 27, 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

Reference:

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

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) 02.05.04-33 and 02.05.04-35, provided in the Reference. 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).

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 27, 2015

Sincerely,

William Maher
Senior Licensing Director – New Nuclear Projects

WDM/RFB

Attachment: 1 FPL Response to NRC RAI No. 02.05.04-33 (eRAI 7811)
Attachment: 2 FPL Response to NRC RAI No. 02.05.04-35 (eRAI 7811)

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NRO

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: 02.05.04-33 (eRAI 7811)

FSAR Section 2.5.4.5.1.2 and response to RAI 0.2.05.04-12 indicated that a 20 foot thick layer concrete fill will be placed beneath the nuclear islands. 10 CFR 100.23 (d) (4) requires that "Each applicant shall evaluate all siting factors and potential causes of failure, such as the physical properties of the materials underlying the site ...," and 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 100.23 (d) (4) and Regulatory Guide 1.206 section C.I.2.5.4.5, please provide the Inspections, Tests, and Analyses and Acceptance Criteria (ITAAC) that will be used to ensure that the concrete fill placed underneath any Category I structures to a thickness greater than 5 feet, meet the design, construction and testing of applicable ACI standards.

FPL RESPONSE:

As discussed in FSAR Subsection 2.5.4.5.1.2, a 19 foot thick layer of lean concrete fill will be placed beneath the nuclear islands from El. –35 feet to El. –16 feet. The design and construction of the lean concrete fill will follow American Concrete Institute (ACI) 207, Guide to Mass Concrete (FSAR Subsection 2.5.4, Reference 281). The Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) that will be used to ensure that the lean concrete fill placed underneath Seismic Category I structures to a thickness greater than 5 feet, meet the specifications in ACI 207 are provided in Table 1, below. The ITAAC that will be used to ensure that the first lift of concrete fill meets the ACI 201.2R-08 durability requirements are provided in the Response to RAI 02.05.04-31. Additional details regarding the design and construction approach for the lean concrete fill will be provided in the Response to RAI 03.08.05-03.

Table 1
ITAAC for Fill Concrete Under Seismic Category I Structures

Design Commitment	Inspections, Tests, and Analyses	Acceptance Criteria
Fill concrete placed under Seismic Category I Structures to a thickness greater than 5 feet is designed and tested as specified in ACI 207 and in FSAR Subsection 2.5.4.5.1.2.	Testing will be performed to determine the mean compressive strength for the fill concrete.	The mean 28-day compressive strength of the fill concrete is equal to, or greater than 1500 psi.

This response is PLANT SPECIFIC.

REFERENCES:

1. American Concrete Institute, *Guide to Mass Concrete* (ACI 207), Detroit, Michigan, 2006.

ASSOCIATED COLA REVISIONS:

A second paragraph will be added to the new FSAR Subsection 14.3.3.5 in a future COLA revision as follows:

14.3.3.5 Fill Concrete ITAAC

Additionally, the ITAAC have been developed to ensure that the static and dynamic properties of the material will be the same as, or better than the design parameters. In general, by testing the mean 28-day compressive strength of cementitious construction material, this ITAAC provides a method to confirm that the properties (static and dynamic) of said material are met prior to the construction of the Seismic Category 1 structure.

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

Table 3.8-5
Fill Concrete

Design Commitment	Inspections, Tests, and Analyses	Acceptance Criteria
Fill concrete placed under Seismic Category I Structures to a thickness greater than 5 feet is designed and tested as specified in ACI 207 and in FSAR Subsection 2.5.4.5.1.2.	**Testing will be performed to determine the mean compressive strength for the fill concrete.**	**The mean 28-day compressive strength of the fill concrete is equal to, or greater than 1500 psi.**

Note: This ITAAC will be added to the Fill Concrete ITAAC presented in the Response to RAI 02.05.04-31

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: 02.05.04-35 (eRAI 7811)

In response to RAI No. 02.05.04-17 the applicant revised the potential for soil liquefaction at the Turkey Point site using RG 1.198. The applicant evaluated the liquefaction potential using CPT, Vs, and SPT state-of-the art methodologies. For the CPT method, Figure 2 presents many factors of safety within the range of 1.1 and 1.4 for the upper Tamiami and lower Tamiami. The applicant only used supplemental SPT test data when conducting its liquefaction assessment using SPT methodologies. In accordance with 10 CFR 100.23 and given the use of limited data for SPT liquefaction evaluation and having many values from CPT liquefaction evaluation within the range for further justification, please provide further evaluation on the potential for soil liquefaction.

FPL RESPONSE:

The assessment of liquefaction potential is based on the most up to date guidance available, provided in Reference 1. In the SPT approach of the liquefaction assessment, the exclusive use of the supplemental SPT test data is based on disturbance considerations as discussed in the response to RAI Nos. 02.05.04-17 and 02.05.04-28.

To provide further justification/evaluation of the liquefaction susceptibility assessment, it is important to note that reported factors of safety are considered to have a significant safety margin. The main factors that provide conservatism in the approach are the following:

- The liquefaction assessment uses a scaled up peak ground acceleration (PGA) instead of the actual calculated PGA.
- The liquefaction assessment ignores the influence of age of deposit on cyclic shear strength.

Below is a discussion on the scaled up PGA and the ignored effect of age on the cyclic shear strength of soils on site.

<u>Scaled up PGA</u>

The PGA used in the liquefaction assessment is 0.1g. This is a scaled up value which corresponds to the minimum PGA required for the safe shutdown earthquake (SSE) per RG 1.208. The PGA for the ground motion response spectrum (GMRS) is 0.0579g as given in FSAR Table 2.5.2-228 for 100 Hz. The PGA used in the analysis is approximately 1.73 times higher than the calculated PGA at the site.

Influence of Age of Deposit

Reference 1 provides the following information regarding the liquefaction/no-liquefaction cases that form the database of the simplified procedure:

> "The simplified procedure was developed from empirical evaluations of field observations and field and laboratory test data. Field evidence of liquefaction generally consisted of surficial observations of sand boils, ground fissures, or lateral spreads. Data were collected mostly from sites on level to gently sloping terrain, underlain by Holocene alluvial or fluvial sediment at shallow depths (<15 m)."

Reference 1 states that sediments deposited within the past few thousand years are generally much more susceptible to liquefaction than older Holocene sediments; it states that Pleistocene sediments are even more resistant, and regards pre-Pleistocene sediments as generally immune to liquefaction. The upper Tamiami and lower Tamiami are Pliocene Formations, and the Peace River is a Miocene-Pliocene Formation (FSAR Subsection 2.5.1.1.1.2.1.1). These deposits are thus substantially older than the range of ages considered in Reference 1 as being susceptible to liquefaction.

Adherence to the direct use of the charts recommended by Reference 1 without consideration of the effects of aging results in a conservative approach. For a given magnitude of field penetration resistance (i.e., $[N_1]_{60}$ or q_{C1N}), the cyclic shear strength obtained from the simplified procedure is strictly applicable to Holocene formations consistent with the database. The liquefaction assessment for Turkey Point Units 6 & 7 is therefore considered to be conservative by ignoring corrections for the effects of aging and treating the in-situ soils essentially as Holocene deposits.

In relation to the field parameters used in the simplified approach, Reference 2 states that penetration resistance does not fully capture the effect of aging of soils:

> "The inability of the SPT and CPT tests to capture the effect of aging is attributed to the nature of these tests which are not sufficiently sensitive to detect minor changes in soil fabric that can increase the liquefaction resistance of the soil. The effect of disturbance during the tests can be significant thereby destroying the macrostructure of the soil and consequently the aging factors that are present. Likewise, but to a much smaller degree, the small-strain shear wave velocity measurements are anticipated to disturb the existent aging factor."

The above observation is consistent with the following excerpt from Reference 3:

> "For older sands... V_s is more sensitive to age and cementation than SPT blow count and CPT tip resistance."

The results of the liquefaction assessment for Turkey Point Units 6 & 7 based on shear wave velocity measurements are consistent with the above observation. The factors of safety based on shear wave velocity are in general considerably higher than the factors of safety based on SPT and CPT. Even if the SPT and CPT techniques do not fully capture the effects of aging, it is important to note that the factors of safety resulting from SPT and CPT approaches are satisfactory, especially in light of the conservative scaled up PGA.

In an alternative treatment of the effect of aging on liquefaction susceptibility assessment, Reference 1 states that the application of the overburden pressure correction factor (K) has been omitted by knowledgeable engineers as partial compensation for the unquantified, but substantial increase of liquefaction resistance for deeply buried sediments more than a few thousand years old. The factor K is a correction factor used to extrapolate the simplified procedure of liquefaction assessment to soils with overburden pressures above 100 kPa (Reference 1). At Turkey Point Units 6 & 7, the average value of K is 0.55 for the full elevation range covering the upper Tamiami, lower Tamiami, and Peace River Formations, with a maximum value of 0.66 at the top of upper Tamiami, and a minimum value of 0.48 towards the bottom of the Peace River Formation.

Alternatively, quantitative approaches that consider the effect of aging on strength are available in the literature. For example, Reference 4 recommends an average strength gain factor that reaches magnitudes above 2.3 for an age of 1 million years or older. The effect of accounting for the two factors, i.e., K and a strength gain factor, would be somewhat equivalent to the omission of K made by knowledgeable engineers as stated in Reference 1 in favor of a partial compensation for the effects of aging.

In spite of the strong qualitative statement that pre-Pleistocene formations are essentially immune to liquefaction (Reference 1) and the availability in the literature of quantitative approaches that consider the effect of age on cyclic shear strength (e.g., References 2, 3, and 4), no consideration of strength gain as a function of age is made in the liquefaction assessment at Turkey Point Units 6 & 7. Neither through the application of a strength gain factor, nor by ignoring factor K. The conservatism in the approach is thus maintained.

<u>Summary</u>

In summary, there are two main factors that build up considerable conservatism in the liquefaction assessment approach, namely 1) the scaled up PGA, and 2) ignoring the influence of age of deposit on cyclic shear strength in the assessment. The combined effect of these two factors provides considerable margin in the approach taken. It is therefore concluded that liquefaction at Turkey Point Units 6 & 7 is not of concern.

This response is PLANT SPECIFIC.

References:

1. Youd, T.L, I.M. Idriss, R.D. Andrus, I. Arango, G. Castro, J.T. Christian, R. Dobry, W.D.L. Finn, L.F. Harder, Jr., M.E. Hynes, K. Ishihara, J.P. Koester, S.S.C. Liao, W.F. Marcuson III, G.R. Martin, J.K. Mitchell, Y. Moriwaki, M.S. Power, P.K. Robertson, R.B. Seed, and K.H. Stokoe II, Liquefaction Resistance of Soils: Summary Report from the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) Workshops on Evaluation of Liquefaction Resistance of Soils, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, pp. 817–833, 2001.

2. Leon, E., S. Gassman, and P. Talwani, Accounting for Soil Aging when Assessing Liquefaction Potential, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 3, pp. 363–377, 2006.

3. Andrus, R.D., H. Hayati, and N.P. Mohanan, Correcting Liquefaction Resistance for Aged Sands Using Measured to Estimated Velocity Ratio, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 6, pp. 735–744, 2009.

4. Arango, I., M. R. Lewis, and C. Kramer, Updated Liquefaction Potential Analysis Eliminates Foundation Retrofitting of Two Critical Structures, Soil Dynamics and Earthquake Engineering, Vol. 20, pp. 17–25, 2000.

ASSOCIATED COLA REVISIONS:

None

ASSOCIATED ENCLOSURES:

None