FPL.

L-2012-297
10 CFR 52.3

July 30, 2012

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. 61 (eRAI 6432)
Related to SRP Section 03.07.01 - Seismic Design Parameters

Reference:

1. NRC Letter to FPL dated May 17, 2012, Request for Additional Information Letter No. 61 Related to SRP Section 03.07.01 Seismic Design Parameters for the Turkey Point Units 6 and 7 Combined License Application

2. FPL Letter to NRC dated June 29, 2012, Schedule for Response to NRC Request for Additional Information Letter No. 61 (eRAI 6432) Related to SRP Section 03.07.01 - Seismic Design Parameters

Florida Power & Light Company (FPL) provides, as attachments to this letter, its responses to the Nuclear Regulatory Commission's (NRC) requests for additional information (RAI) 03.07.01-17, RAI 03.07.01-19, and RAI 03.07.01-20 provided in the referenced letter. 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).

Reference 2 provided the schedule for responding to RAI 03.07.01-14, RAI 03.07.01-16, and RAI 03.07.01-18.

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

Florida Power & Light Company

700 Universe Boulevard, Juno Beach, FL 33408

D097
NRO

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

Executed on July 30, 2012.

Sincerely,

William Maher
Senior Licensing Director – New Nuclear Projects

WDM/ETC

Attachment 1: FPL Response to NRC RAI No. 03.07.01-17 (eRAI 6432)
Attachment 2: FPL Response to NRC RAI No. 03.07.01-19 (eRAI 6432)
Attachment 3: FPL Response to NRC RAI No. 03.07.01-20 (eRAI 6432)

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-061 Dated May 17, 2012

SRP Section: 03.07.01 – Seismic Design Parameters

Questions from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-17 (eRAI 6432)

In Revision 3 of the applicant's FSAR, (aka. TPG-1000-S2R-802, "Turkey Point Site-Specific Seismic Evaluation Report") the second paragraph in Section 3JJ.4, "Spectral Matching of Acceleration Time Histories," indicates that the required value of the zero-lag cross correlation should be 0.3. Section 3.7.1 of the SRP requires that each pair of time histories considered to be statistically independent if the absolute value of their correlation coefficient does not exceed 0.16. The applicant is requested to reconcile this difference in requirements or, alternatively, provide the basis for the value of 0.3.

FPL RESPONSE:

For the development of the spectrum compatible time histories used in the SSI analysis the spectral matching criteria given in NUREG/CR-6728 (Risk Engineering Inc., 2001, FSAR Subsection 2.5.2.7, Reference 308) were followed. As part of the spectral matching criteria, a zero-lag cross correlation value of 0.3 or less is required.

As noted in the RAI, however, Section 3.7.1 of NUREG-0800 Rev 3 (3.7.1) requires that each pair of time histories have a zero-lag cross correlation of 0.16 or less. The absolute values of the zero-lag cross correlations are given in Table 1 and it can be verified that these time histories are statistically independent based on their correlation values being less than 0.16 as required in Section 3.7.1 of the SRP.

The FSAR text will be changed to indicate this acceptance based on the value of 0.16 and the values listed below in Table 1 will be included in the FSAR change.

Table 1. Absolute value of the zero-lag cross correlations between components for the spectrum compatible time histories developed for the SSI analysis.

Components	Cross Correlation
H1 – H2	0.013
H1 – UP	0.105
H2 – UP	0.015

This response is PLANT SPECIFIC.

References:

1. Risk Engineering, Inc., *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-Consistent Ground Motion Spectra Guidelines*, NUREG/CR-6728, U.S. Nuclear Regulatory Commission, Washington, D.C., 2001.

2. NRC, *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition*, NUREG-0800, Section 3.7.1, Seismic Design Parameters**,** Revision 3, March 2007.

ASSOCIATED COLA REVISIONS:

The text for the last two paragraphs of FSAR Appendix 3JJ, Section 3JJ.4 will be changed in a future COLA revision as shown below:

> The spectral matching procedure is a time domain procedure and emphasis was placed on maintaining the phase characteristics of the initial time history in the final modified spectrum-compatible time history. In addition, emphasis was placed on maintaining the characteristic of the normalized Arias intensities (the integral of the square of the acceleration-time history, a ground motion parameter that captures the potential destructiveness of an earthquake) of the initial and final modified spectrum-compatible time histories. These time histories were modified to be spectrum-compatible to the FIRS target spectra following the spectral matching criteria given in ~~NUREG/CR-6728~~ **NUREG-0800 Rev 3 (3.7.1)**. In most cases, as additional constant scale factor was applied after the spectral matching procedure to comply with the spectral matching criteria given in ~~NUREG/CR-6728~~ **NUREG-0800 Rev 3 (3.7.1)**. Scale factors of 1.02, 1.022, and 1.01 were applied for the two horizontal directions (H1, H2), and vertical direction (UP) components, respectively.
>
> The modified spectrum-compatible acceleration, velocity, and displacement time histories prior to the application of the noted constant scale factors are plotted in Figure 3JJ-222a for the H1 component. Figure 3JJ-222c shows target horizontal FIRS spectrum, 1.3*FIRS target spectrum, 0.9*FIRS target spectrum and the modified time history response spectrum including the 1.02 constant scale factor. The normalized Arias intensities for the first horizontal (H1) component initial and modified spectrum-compatible time histories are plotted in Figure 3JJ-222b. The results for the second horizontal (H2) component and the UP component are shown in Figures 3JJ-223a, 3JJ-223b, and 3JJ-223c and Figures 3JJ-224a, 3JJ-224b, and 3JJ-224c, respectively. The zero-lag cross correlation ~~was~~ **values were** computed for **the** combinations between the three spectrum-compatible acceleration time histories and ~~they all fall below the required value of 0.30~~ **are listed in Table 3JJ-208. These values are all less than the required value of 0.16**.

Table 3JJ-208 will be added to FSAR Appendix 3JJ in a future COLA revision.

Table 3JJ-208

Absolute value of the zero-lag cross correlations between components for the spectrum compatible time histories developed for the SSI analysis.

Components	Cross Correlation
H1 – H2	0.013
H1 – UP	0.105
H2 – UP	0.015

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-061 Dated May 17, 2012

SRP Section: 03.07.01 – Seismic Design Parameters

Questions from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-19 (eRAI 6432)

In Revision 3 of the applicant's FSAR, (aka. TPG-1000-S2R-802, "Turkey Point Site-Specific Seismic Evaluation Report") the third paragraph in Section 2.5.4.7.3.3, "Shear modulus and Damping for Rock," states 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 ½% was used in the analyses. The applicant is requested to provide clarification as to the actual level of damping used in the analyses, including the SSI analyses, and provide basis for selection considering the large variability in RQD shown in Figure 2.5.4-215.

FPL RESPONSE:

This RAI is very similar to RAI 02.05.04-16 and thus the response given here follows closely the response to RAI 02.05.04-16 with some additional information. Proposed changes to the FSAR are given in the response to RAI 02.05.04-16.

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 ft of structural fill (with surface at El. +25.5 ft NAVD 88) above approximately 25 ft of Miami Limestone, overlying about 90 ft 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 ft depth. The Arcadia Formation, consisting of very weak rock mixed with some soil extends to about 640 ft 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, 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 = 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 1,500 and 2,000 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-ft 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.

This response specifies the level of damping used in the generation of simulated (randomized) soil profiles, which are in turn used in seismic site response analysis. The SSI analyses use the strain-compatible damping ratios that result from the site response analysis of the simulated (randomized) profiles. FSAR Table 3.2-1 in Appendix 3KK provides the damping ratio values used in the SSI for the best estimate profile.

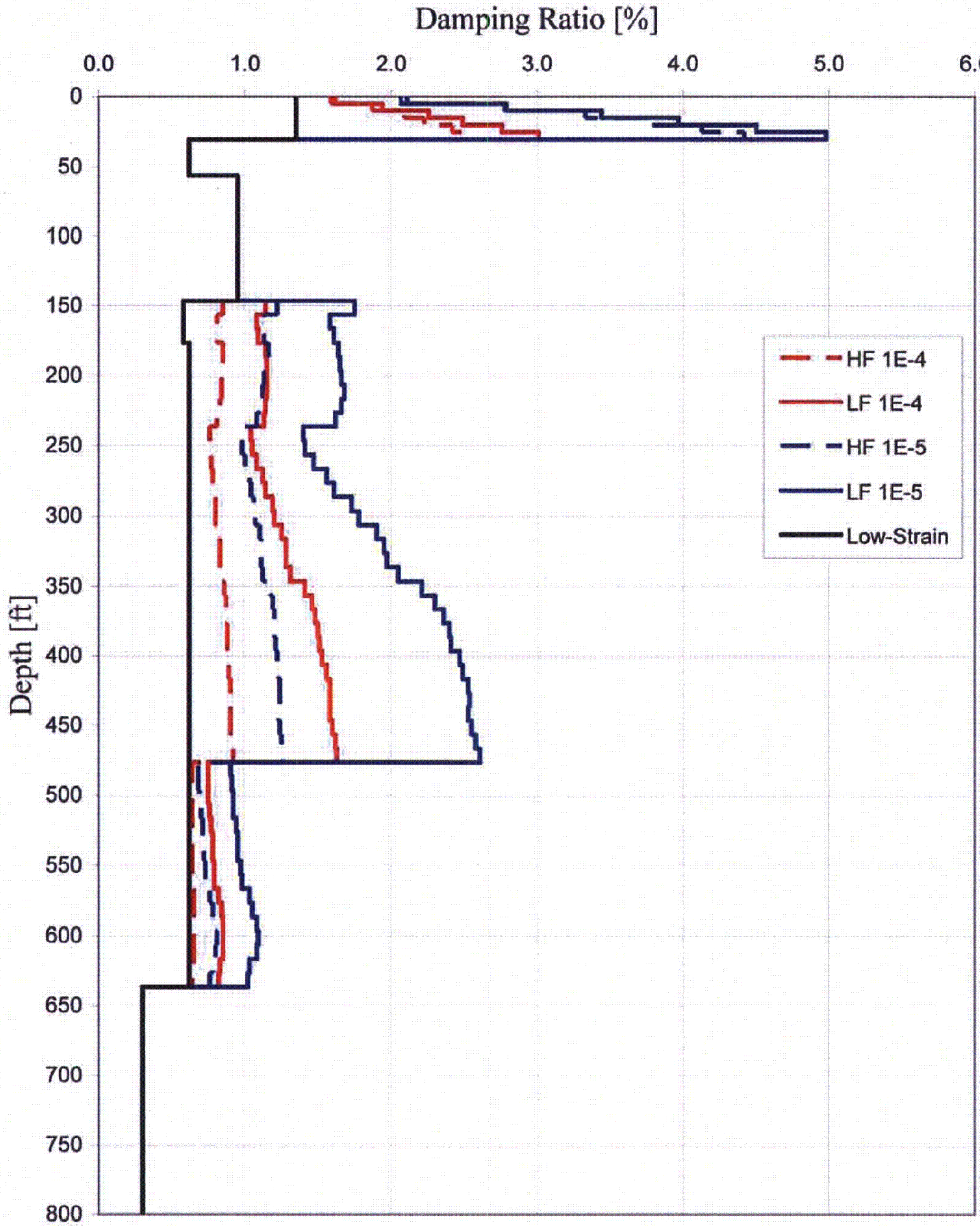

Figure 2.5.2-249 Median Profiles of Strain-Compatible Soil Damping (Upper 800 feet) (Sheet 2 of 2)

(FSAR Revision 3)

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

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

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-061 Dated May 17, 2012

SRP Section: 03.07.01 – Seismic Design Parameters

Questions from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-20 (eRAI 6432)

In Revision 3 of the applicant's FSAR, (aka. TPG-1000-S2R-802, "Turkey Point Site-Specific Seismic Evaluation Report") the second paragraph in Section 2.5.4.7.3.4, "Dynamic Properties of Structural Fill," states that a large coefficient of variation (COV) of 1.5 was used for the fill to accommodate uncertainties associated with the fill material. However, in Section 2.5.2.5.1 of the FSAR, "Base Case Site-specific Column and Uncertainties," it is stated that a COV of 0.5 was used to provide upper and lower bounds on the dynamic properties of the site-specific soil column. Section 3.7.1.1.1.2, "Strain-Compatible Soil Property Profiles," indicates that the results of the soil amplification analysis were used to develop strain-compatible properties used in Appendix 3JJ inferring that the COV used in the SSI analyses is consistent with the 0.5 value used in developing the soil amplification functions.

Given the uncertainties associated with the fill, for which a COV of 1.5 was recommended in Section 2.5.4.7.3.4, a justification is needed for the use of a COV of 0.5 for both the development of soil amplification functions and for the SSI analyses. The staff requests that the applicant provide an evaluation of the significance of the larger uncertainty on the computed soil amplification functions and on the responses computed in the SSI analyses, including the effect on estimated relative displacements between the NI and adjacent structures.

FPL RESPONSE:

A COV of 1.5 applied to the shear modulus was maintained as a minimum variation for structural fill layers throughout the different steps of analysis including SSI.

For the purpose of calculating soil profiles for SSI analysis, the median strain-compatible shear wave velocities are calculated, as well as their corresponding strain-compatible shear modulus, and considered as the Best Estimate (BE) profile. Per Section 3.3.1.7 in ASCE 4-98, the COV is applied to the BE shear modulus (G) to calculate the corresponding Lower Bound (LB) and Upper Bound (UB) shear moduli, as follows:

$$G_{LB} = \frac{G_{BE}}{1+COV} \text{ and } G_{UB} = G_{BE} \times (1+COV) \qquad \text{Equation 1}$$

Given that $V_S = \sqrt{G/\gamma}$, where G is the shear modulus, V_S is the shear wave velocity, and γ is the unit weight, the resulting LB and UB shear wave velocities are calculated as:

$$(V_S)_{LB} = \frac{(V_S)_{BE}}{\sqrt{1+COV}} \text{ and } (V_S)_{UB} = (V_S)_{BE} \times \sqrt{1+COV} \qquad \text{Equation 2}$$

Given a COV of 1.5 (applied to *G*) for structural fill:

$(V_S)_{LB} = \frac{(V_S)_{BE}}{1.58}$ and $(V_S)_{UB} = 1.58 \times (V_S)_{BE}$ Equation 3

Appendix 3KK provides the BE, LB and UB strain-compatible shear wave velocity profiles used in the SSI analyses of Turkey Point Units 6 & 7 in Table 3.2-1, 3.2-2 and 3.2-3, respectively. These tables show that the LB and UB shear wave velocities for the structural fill layers were calculated maintaining a minimum COV of 1.5 applied to the strain-compatible BE shear modulus. The LB and UB values in FSAR tables 3.2-2 and 3.3-3 are calculated using the median and standard deviation of the strain-compatible shear wave velocity of each layer, by subtracting and adding one standard deviation to the median, respectively. If the calculated COV using the strain-compatible shear modulus (associated with the V_S values in Table 3.2-2 and 3.2-3 of Appendix 3KK) from the set of 60 simulated profiles exceeds 1.5, the calculated values are used. Otherwise the values calculated using Equation 3 (i.e. using COV of 1.5 applied to the shear modulus) are used.

As an example, the top most fill layer in Table 3.2-1 has a $(V_S)_{BE}$ of 679.1 ft/sec. Applying Equation 3 above to $(V_S)_{BE}$ of 679.1 results in $(V_S)_{LB}$ of 429.5 ft/sec and $(V_S)_{UB}$ of 1073.8 ft/sec. Table 3.2-2 and Table 3.2-3 provide the values of 411.8 ft/sec (smaller than 429.5) for LB V_S, and 1119.9 ft/sec (larger than 1073.8) for UB V_S, respectively. The values in the tables translate into a COV of 1.72 (larger than 1.5).

For the purpose of soil profile simulation of the low-strain shear wave velocity profiles, a coefficient of variation of 0.5, as stated in Section 2.5.2.5.1, is applied to shear wave velocity of backfill, which is equivalent to a COV of 1.5 applied to the shear modulus.

Noting that the simulation of soil profile properties assumes a natural logarithmic standard distribution for shear wave velocity, the natural logarithmic standard deviation for shear wave velocity, $(\sigma_{ln})_{V_S}$, is calculated using Equation 4:

$(\sigma_{ln})_{V_S} = ln(\sqrt{1 + COV}) = ln(\sqrt{1 + 1.5}) = 0.46$ Equation 4

Where COV in Equation 4 is as defined in Equation 1 and applied to the shear modulus. To calculate the equivalent coefficient of variation applied to shear wave velocity, $(CV)_{V_s}$ (defined as arithmetic standard deviation divided by the arithmetic mean), Equation 5 (Haldar and Mahadevan, 2000) is used:

$(CV)_{V_s} = \sqrt{exp\left[\left((\sigma_{ln})_{V_S}\right)^2\right] - 1} = \sqrt{exp[(0.46)^2] - 1} = 0.49 \cong 0.5$ Equation 5

The FSAR text in Section 2.5.2.5.1 will be modified to specify that a COV of 1.5 is applied to initial backfill soil shear modulus. Similarly, the FSAR text in Subsection 2.5.4.7.3.4 will be modified to specify that a COV of 1.5 is applied to shear modulus.

This response is PLANT SPECIFIC.

References:

1. Seismic Analysis of Safety-Related Nuclear Structures and Commentary (ASCE 4 - 98), American Society of Civil Engineers, 1999.
2. Haldar, A. and Mahadevan, S. (2000), "Reliability Assessment Using Stochastic Finite Element Analysis", John Wiley & Sons, Inc., ISBN 0-471-36961-6.

ASSOCIATED COLA REVISIONS:

The first sentence of the second paragraph in FSAR Subsection 2.5.4.7.3.4 will be updated in a future COLA revision, as shown below:

"Estimated shear wave velocity for structural limerock fill with upper and lower boundary estimates using a coefficient of variation (COV) of 1.5, **applied to the shear modulus**, is shown on Figure 2.5.4-236."

and

The first sentence of the fourth paragraph in FSAR Subsection 2.5.2.5.1 will be updated in a future COLA revision, as shown below:

"In order to capture the uncertainty in this estimate, a coefficient of variation **(COV)** of ~~0.5~~ **1.5, applied to the shear modulus**, was used to provide upper and lower bounds."

ASSOCIATED ENCLOSURES:

None