FPL.

L-2014-354
10 CFR 52.3

December 11, 2014

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
Voluntary Revised Response to NRC Request for Additional Information Letter No. 007 (eRAI 4975) SRP Section 03.07.01 – Seismic Design Parameters

References:

1. NRC Letter to FPL dated September 23, 2010, Request for Additional Information Letter No.007 Related to SRP Section 3.07.01 – Seismic Design Parameters for the Turkey Point Nuclear Plant Units 6 and 7 Combined License Application
2. FPL Letter L-2010-241 to NRC dated October 27, 2010, Response to NRC Request for Additional Information Letter No. 007 (eRAI 4975) Standard Review Plan Section 3.07.01 – Seismic Design Parameters
3. FPL Letter L-2014-314 to NRC dated October 29, 2014, Submittal of the Annual Update of the COL Application – Revision 6 and the Semi-Annual Update of the Departures Report

Florida Power & Light Company (FPL) provides, as an attachment to this letter, its revised responses to the Nuclear Regulatory Commission's (NRC) requests for additional information (RAI) 03.07.01-2, 03.07.01-3, 03.07.01-4, 03.07.01-5, and 03.07.01-10 provided in Reference 1. These revisions resulted from the supplemental site investigation testing and analyses performed to support the revised responses for FSAR Subsection 2.5.4 submitted in Reference 2.

The voluntary revised responses do not provide any associated COLA changes as any COLA revisions resulting from the revised responses were incorporated in Revision 6 of the Turkey Point 6 & 7 COL Application submitted in Reference 3.

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

D097
NRO

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

Executed on December 11, 2014

Sincerely,

William Maher
Senior Licensing Director – New Nuclear Projects

WDM/RFB

Attachment 1: FPL Revised Response to NRC RAI No. 03.07.01-2 (eRAI 4975)
Attachment 2: FPL Revised Response to NRC RAI No. 03.07.01-3 (eRAI 4975)
Attachment 3: FPL Revised Response to NRC RAI No. 03.07.01-4 (eRAI 4975)
Attachment 4: FPL Revised Response to NRC RAI No. 03.07.01-5 (eRAI 4975)
Attachment 5: FPL Revised Response to NRC RAI No. 03.07.01-10 (eRAI 4975)

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-007

SRP Section: 03.07.01 – Seismic Design Parameters

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-2 (e-RAI 4975)

Section 2.2 of Appendix 3KK also indicates that the fill to be added to the site will be composed of crushed limestone. Please describe the fill to be placed above the concrete supporting the foundation in Figure 2.7 of Appendix 3KK. If it will also be crushed limestone, please provide information on whether any long-term serviceability issues should be considered when judging adequacy of this material for use in providing support to the NPP. Please provide information on the data used to develop BE, LB and UB S- and P-wave velocity values comprised for the NEAR and FAR site soil profiles.

FPL RESPONSE:

FPL submitted a response to this Request for Additional Information in Reference 2. The FPL response remains unchanged, with the exception of specific references cited to section/figure numbers in Appendices 3JJ and 3KK that have been renumbered in subsequent revisions to the Turkey Point Units 6 & 7 COL Application.

This submittal revises the initial response to update the section/figure number references, as follows:

Question 03.07.01-2 has three parts. Part [1] requests describing the fill to be placed above the concrete foundation, as shown in Figure 3.1-2 of Appendix 3KK. Part [2] requests addressing any long-term serviceability issues associated with this fill. Part [3] requests providing information on the development of the BE, LB, and UB shear and compression wave velocities for the NEAR and FAR site soils given in Appendix 3KK. Response to each of these three parts is presented below.

Part [1]: Description of Fill Placed Above Concrete Foundation

Response: The fill material is comprised of limerock. Limerock is a terminology used in the Florida aggregate industry referring to crushed limestone that has particle gradations from small to large and is used as a compacted base layer beneath roads and buildings as well as in other structural earth fill applications (Reference 1).

As shown in Figure 3.1-2 of Appendix 3KK, approximately 19 ft of concrete fill is placed up to El. -16 ft for support of the foundation of the Nuclear Island. Above the concrete fill, Category I engineered fill, consisting of limerock, is placed and compacted up to the ground level of about El. 25 ft. The limerock fill is placed around (not below) the Nuclear Island structures.

The limerock material intended for Turkey Point Units 6 & 7 is granular in nature. It is derived from the excavated portions of the onsite Miami Limestone and/or obtained from offsite sources. FSAR Section 2.5.4.5 provides further details on backfill materials. Laboratory compaction tests performed on materials derived from two exploratory test pits excavated in Miami Limestone show that they are gravel-sand mixtures (SP-SM and SM) with fines content of 12 to 17 percent. The offsite sources of backfill are also expected to be granular, as noted in FSAR Section 2.5.4.5.1.1. For Nuclear Island structures, engineered fill is compacted to a minimum of 95 percent modified Proctor maximum dry density, as noted in FSAR Section 2.5.4.5.3. Limerock fill compacted to these requirements is expected to attain dense conditions. FSAR Table 2.5.4-209 shows the following properties for such fill: an adjusted Standard Penetration Test (SPT) N-value of 30 blows/ft, an effective friction angle of 33 degrees, an average design shear wave velocity of 860 ft/sec, and a total unit weight of 130 pcf.

Part [2]: Long-Term Serviceability of Fill

Response: Construction fill serviceability, or lack thereof, can stem from major factors such as suitability of material aggregates and minerals, control during material production, and/or placement and compaction control. These factors are each discussed below.

Aggregates and Minerals: The limerock material intended for Turkey Point Units 6 & 7 is derived from the excavated portions of the onsite Miami Limestone and/or obtained from offsite sources. The following description on aggregate suitability and minerals is based on information from Reference 1.

The Miami Limestone and the Fort Thompson Limestone in the Lake Belt Region of Miami-Dade County are extensively mined. The Lake Belt Region (area between the Everglades and the urbanized Miami-Dade County) produces a highly desirable grade of crushed stone from the Miami Limestone. Approximately 55 million tons of rock from this area is processed into aggregate products each year and is a major supplier for the construction industry. The Florida Department of Transportation (FDOT) is the largest single consumer of this material in the state. FDOT establishes requirements for hardness, durability, and chemical content. Portions of the Miami Limestone contain 10 to 20 percent chemical by redeposited silica, which enhances the engineering properties of the rock.

Control During Production: In Florida, crushed stone materials that do not meet specifications for use as concrete aggregate are used as limerock fill. The characteristics of the Miami Limestone at the Turkey Point Units 6 & 7 site indicate that this material will generally be more suitable for use as limerock fill than as crushed aggregate. This in no way infers that limerock is an inferior material for its intended use. As noted earlier, the Miami Limestone has been extensively exploited in the Miami-Dade area because of its high quality minerals. Nevertheless, quality control is an integral part of material verification and acceptance at its source in the Lake Belt Region of Miami-Dade County, and its practice is well-developed and regulated by the FDOT for state-owned projects.

The FDOT is the quality control and standard setting organization in the State of Florida for construction aggregates (Reference 1). Mining companies must meet the engineering tests prescribed by FDOT for materials used on state projects. Aggregate producers selling materials for state-owned projects must participate with the FDOT State Materials Office to verify the quality and consistency of mined materials. Therefore, only certified materials are used for construction projects. Tests recognized by the FDOT State Materials Office ensure that aggregates meet engineering standards and are performed by the mining companies at the production facilities. Ultimately, approval of aggregates for use on FDOT projects is through the specification regulated by the State Materials Office.

For limerock materials intended for Turkey Point Units 6 & 7, whether produced onsite or obtained from off-site sources, the FDOT specifications for limerock application as structural fill will be referenced in project earthwork specifications developed during detailed design. This results in the necessary control of material production, consistent with FDOT applications for state-owned projects.

Placement and Compaction Control: Limerock fill is placed and compacted based on dedicated specifications that will be developed for Turkey Point Units 6 & 7. Limerock has been used for construction of existing units at the Turkey Point site. These specifications and the experience gained in such applications are relied upon in controlling the materials for Units 6 & 7. The control includes specification of regionally-accepted materials for use as engineered fill, qualified processes and sources for material preparation, experienced technicians and engineers overseeing the process, and employment of observation and testing techniques for qualification of the process and adherence to project specifications. As noted previously, for Nuclear Island structures, engineered fill is compacted to a minimum of 95 percent modified Proctor maximum dry density, as noted in FSAR Section 2.5.4.5.3. Materials compacted to such densities are expected to attain dense, stable conditions, designed to meet all of the project performance requirements. In the specific case of Units 6 & 7, as shown in Figure 3.1-2 of Appendix 3KK, the limerock fill is placed around (not below) the Nuclear Island structures, such that it is confined in all directions; it is confined by the rigid, concrete diaphragm wall and the Nuclear Island structures on the sides, by the 19-ft thick concrete fill from the bottom, and by the weight of the overlying fill from the top.

For the conditions described above, which are specific for Turkey Point Units 6 & 7, the engineering properties of limerock fill are greatly enhanced, leading to maintenance of their condition as placed and compacted, without being adversely affected by, or affecting, the structures they support. In summary, there are no long term serviceability issues.

Part [3]: Shear and Compression Velocities, NEAR and FAR site

The BE properties are based on the average strain-compatible properties provided as a result of the site response analysis. The development of the BE, LB and UB soil profiles for the NI (NEAR) and FAR site conditions is described in Section 3JJ.4 of Appendix 3JJ. Note that the fill occupies the top 41.5 ft and 30.5 ft of the NI and FAR soil columns, respectively.

This response is PLANT SPECIFIC.

References:

1. Florida Department of Transportation, Strategic Aggregate Study: Sources, Constraints, and Economic Value of Limestone and Sand in Florida, Part I and II, Final Report, March 12, 2007.
2. FPL Letter L-2010-241 to the NRC dated October 27, 2010, Response to NRC Request for Additional Information Letter No. 007 (eRAI 4975) Standard Review Plan Section 3.07.01 - Seismic Design Parameters.

ASSOCIATED COLA REVISIONS:

No additional changes to COLA Revision 6 have been identified as a result of this revised response.

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-007

SRP Section: 03.07.01 – Seismic Design

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-3 (eRAI 4975)

Please provide material property information on concrete cracking criteria to be used to judge acceptability of the placement of the supporting concrete pad under the NI, and the impact of such cracking on sliding and overturning stability of the concrete needed to support the NI.

FPL RESPONSE:

As noted in FSAR Section 2.5.4.5.2, Category I seismic structures will bear on lean concrete fill placed over Key Largo limestone. As discussed in the revised response to RAI 02.05.04-12 (Reference 1), lean concrete is unreinforced concrete with lower cement to aggregate ratio than typical structural or reinforced concrete. Controlled Low Strength Material (CLSM) will not be used. The revised response to RAI 02.05.04-12 (Reference 1) shows that cracking due to overstressing is not expected.

The concrete fill will be placed from Elevation -35 feet NAVD 88 to Elevation -16 feet NAVD 88 resulting in an approximate 19-foot thick concrete sub-basemat as described in ER Section 3.9.2.1. The selection of lean concrete mix for design is made at the time of project detailed design. The compressive strength of 1,500 psi is estimated for the lean concrete fill. The fill concrete under the Nuclear Island of Turkey Point Units 6 & 7 is a mass concrete.

American Concrete Institute (ACI) defines mass concrete as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking." The definition is intentionally vague because many factors, including the concrete mix design, the dimensions, the type of the placement, and the curing methods, affect whether or not cracking will occur. Reference 2, prepared by ACI Committee 207, governs the design and construction of mass concrete. There are two design considerations: (1) the maximum temperature inside a concrete pour and (2) the maximum temperature difference between the hottest spot and the surface of a concrete pour. Specifications of mass concrete typically limit the maximum temperature difference between the interior and the surface to 20° Celsius, so that early-age thermal cracks in mass concrete will be minimized. It is a common practice to limit the least dimension of each concrete pour so that the temperature and temperature differences of the pour can stay within their respective limits.

Following the Reference 2 guidelines, a thermal control plan considering the geometry of Units 6 & 7 fill concrete, the proposed 1,500 psi (minimum) strength, total volume of fill concrete placement, and rate of concrete production, will be prepared to make sure that the recommended temperature limits will not be exceeded. Additional details on the thermal control plan that will be developed are provided in the revised response to RAI 02.05.04-12 (Reference 1).

In response to the question on impact of cracking on sliding and overturning capability, the concrete sub-basemat will be completely confined within the boundaries of a reinforced concrete diaphragm wall as described in FSAR Section 2.5.4 and depicted in FSAR Figure 2.5.4-222. The diaphragm wall will be constructed prior to placement of the lean concrete fill. The combined resistance of the diaphragm wall, the compacted fill, and the surrounding ground will effectively impede any possibility of overturning or sliding of the mass concrete confined within.

This response is PLANT SPECIFIC.

References:

1. FPL Letter L-2014-285 to NRC dated, October 3, 2014, Voluntary Revised Response to NRC Request for Additional Information Letter No. 040 (eRAI 6006) – Standard Review Plan Section 02.05.04 – Stability of Subsurface Materials and Foundations
2. American Concrete Institute, *Guide to Mass Concrete (ACI 207)*, Detroit, MI, 2006

ASSOCIATED COLA REVISIONS:

No additional changes to COLA Revision 6 have been identified as a result of this revised response.

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-007

SRP Section: 03.07.01 – Seismic Design Parameters

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-4 (e-RAI 4975)

Section 2.0 of Appendix 3KK indicates that the site is at or near sea level with a natural relief of about 3 feet across the site. The subsequent paragraph indicates that the elevation of the site region varies from 3 feet below to 400 feet above sea level. Please explain how these two statements are consistent with one another and indicate whether the elevation change of about 400 feet across the site has any potential impact on SSI analyses that typically assume uniform horizontal soil layers. State if the meaning of the site region is much larger than the site, In addition, the question really has to do with evaluating the potential impact of site topography on SSI.

FPL RESPONSE:

In response to an NRC Letter to FPL dated September 23, 2010, Request for Additional Information Letter No. 007 Related to SRP Section 3.07.01 - Seismic Design Parameters for the Turkey Point Nuclear Plant Units 6 and 7 Combined License Application, FPL submitted a response to eRAI 4975 in a letter, L-2010-241, to the NRC dated October 27, 2010, "Response to NRC Request for Additional Information Letter No. 007 (eRAI 4975) Standard Review Plan Section 3.07.01 - Seismic Design Parameters". Subsequent to that submittal, supplemental subsurface investigations were conducted at the Turkey Point Units 6 & 7 site that resulted in clarifications needed for the existing site elevation.

This submittal revises the initial response to provide this clarification, as follows:

With regards to the two statements, the one statement addresses the site which covers the area within a 0.6 mile radius of the site. The site's natural relief is approximately 3 feet from its northern to southern boundary and approximately 0.5 feet of relief from its western to eastern boundary.

The second statement addresses the site region, which covers an area within a 200-mile radius of the site, with the site located in the Atlantic Coastal Plain physiographic region. The ground surface in the site region ranges from about 3 feet below sea level to 345 feet above.

Based on the original site being at or near sea level with an existing elevation of -3.2 to 0.8 feet and generally flat throughout, there is no expected potential impact on the SSI analyses.

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

No additional changes to COLA Revision 6 have been identified as a result of this revised RAI response.

ASSOCIATED ENCLOSURES:

None

SRP Section: 03.07.01 – Seismic Design Parameters

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-5 (eRAI 4975)

In light of the expected site ground water elevation, please provide information on potential for liquefaction of in-situ and fill soils when the generic plant design presumes that no potential for liquefaction exists at the site.

FPL RESPONSE:

Question 03.07.01-5 has two parts. Part [1] is related to liquefaction of "in-situ" soils, i.e., soils currently present at the site. Part [2] is related to liquefaction of "fill" soils, i.e., soils which will be used as backfill for construction. Response to each of these two parts is presented below.

Part [1]: Liquefaction of in-situ soils

Liquefaction of in-situ soils, i.e., soils currently present at the site, is addressed in FSAR Section 2.5.4.8 and the revised response to RAI 02.05.04-17 (Reference 1). Please refer to FSAR Section 2.5.4.8 for a detailed evaluation of the topic.

Stratum 1 (muck/peat) that currently makes up the surface soils, and is on average about 3.6 feet thick, will be removed in its entirety prior to commencing major earthwork and replaced with compacted fill. This is addressed in FSAR Section 2.5.4.5.1.1. Please refer to this section for further details on the topic.

Part [2]: Liquefaction of fill soils

As background information, the power block area is raised by placing about 26 ft of compacted structural fill (FSAR Figure 2.5.4-221). The bottom of the fill is at about El. -16 ft, extending from the top of concrete fill to the ground surface. This fill is only placed around (not below) the Nuclear Island structures. Nonsafety-related (Category II) structural fill (also termed general fill) is used below nonsafety-related structures. Please refer to Figure 2.5.4-222 of the FSAR for details which show the B/FOOTING (bottom of footing) as El. -14 ft; the bottom of the fill, however, is at about El. -16 ft. The 2-ft space is for mudmat plus waterproofing membrane(s) between top of concrete fill and the underside of base slab.

Fill materials intended for Turkey Point Units 6 & 7 are not subject to liquefaction, even though they may be saturated due to the site groundwater condition. There are several reasons for this lack of susceptibility to liquefy; they include density, gradation, earthquake intensity, and other factors, as described below.

Dense granular soils subject to shear straining have a tendency to dilate. This inhibits major strength loss during a seismic event. In coarser granular soils, porewater pressure cannot build up sufficiently to cause complete loss of effective shear strength due to their high permeability. The fill materials intended for Turkey Point Units 6 & 7 are dense and granular. These materials are referred to as limerock, and are derived from the on-site Miami Limestone and/or are limerock materials from offsite sources. Please refer to FSAR Section 2.5.4.5 for details on backfill materials. Laboratory compaction tests on materials derived from two test pits excavated in Miami Limestone show that they are gravel-sand mixtures (SP-SM and SM) with fines content of 12 to 17 percent. The offsite sources of backfill are also expected to be granular, as noted in FSAR Section 2.5.4.5.1.1. For safety-related applications, (structural) fill is compacted to a minimum of 95% modified Proctor maximum dry density; for non-safety-related applications, (general) fill is compacted to a minimum of 92% modified Proctor maximum dry density, as noted in FSAR Section 2.5.4.5.3. Materials compacted to such densities are shown to be non-liquefiable using analysis methods based on Standard Penetration Test (SPT), Cone Penetrometer Test (CPT) and shear wave velocity, as described below.

FSAR Table 2.5.4-209 shows the following properties for the fill: an adjusted SPT N-value of 30 blows/ft, an effective friction angle of 33 degrees, an average design shear wave velocity of 860 ft/sec, and a total unit weight of 130 pcf.

- Analysis based on SPT: According to Youd, et al, Figure 2 (FSAR Section 2.5.4.13, Reference 219), "No Liquefaction" for clean sands with an adjusted N-value greater than 30 blows/ft. This conclusion is also reinforced by the very low peak ground acceleration for the site of 0.1g (FSAR Section 2.5.4.8.1). Also, it conservatively ignores any fines contribution.
- Analysis based on CPT: A cone penetrometer will not be able to penetrate to any significant depth into a well-compacted limerock fill. Since liquefaction potential can be equated to cone tip resistance, similar to SPT N-value, high cone tip resistance values or tip refusals indicate "No Liquefaction" conditions (Youd, et al, Figure 4). This conclusion is also reinforced by the very low peak ground acceleration for the site of 0.1g (FSAR Section 2.5.4.8.1). Also, it conservatively ignores any fines contribution.
- Analysis based on shear wave velocity: Similar to SPT and CPT, shear wave velocity is a measure of soil stiffness and can be used to evaluate liquefaction susceptibility. An average design shear wave velocity for the fill is 860 ft/sec (262 m/s). Figure 9 of Youd, et al shows "No Liquefaction" for overburden stress-corrected shear wave velocity greater than about 690 ft/s (210 m/s). (The overburden stress-correction factor increases the measured shear wave velocity at shallow depths and reduces it at greater depths. The correction factor is 1 at an effective overburden pressure of about 1 ton/sq ft). Referring to FSAR Figure 2.5.4-222 and assuming saturated fill conditions below El. 0 ft, the overburden stress correction factor is about 0.85 to 0.9 for a point halfway between El. 0 and El. -15 ft, resulting in an overburden-corrected shear wave velocity of about 730 ft/s (223 m/s) to 774 ft/s (236 m/s, exceeding the 690 ft/s (210 m/s value). Also, this conservatively ignores any fines contribution. Additionally, the fill average design shear wave velocity of 860 ft/s (223 m/s) corresponds to depth interval of 5 to 10 ft below the ground surface, as shown in FSAR Figure 2.5.4-236. Further, this depth interval will be covered by roughly another 30 ft of compacted fill due to grading in the power block area, resulting in increased shear wave velocity due to weight of the overburden, with increased safety against liquefaction.

- Other factors: As previously discussed, the bottom of the fill is at about El. -16 ft, however, it is only placed around (not below) the Nuclear Island structures. The Nuclear Island structures are surrounded by the concrete diaphragm wall. Therefore, the compacted fill is confined between the concrete diaphragm wall and the Nuclear Island structures. This confinement provides additional stability to the fill in case of liquefaction susceptibility, notwithstanding the discussions above on the lack of such potential.

This response is PLANT SPECIFIC.

Reference:

1. FPL Letter L-2014-285 to NRC dated, October 3, 2014, Voluntary Revised Response to NRC Request for Additional Information Letter No. 040 (eRAI 6006) – Standard Review Plan Section 02.05.04 – Stability of Subsurface Materials and Foundations

ASSOCIATED COLA REVISIONS:

No additional changes to COLA Revision 6 have been identified as a result of this revised response.

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-007

SRP Section: 03.07.01 – Seismic Design Parameters

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-10 (eRAI 4975)

Section 3JJ.1 of Appendix 3JJ, Development of Amplification Factors at FIRS Horizon, indicates that the procedure used in the randomization process to develop FIRS was the same as used in Section 2.5.2.5 in incorporating anelastic attenuation (the kappa value). It is typical, however, to remove the kappa effect from the layered soil profile description so as not to double count energy loss incorporated into the site damping moduli used in the SHAKE type calculations for the randomized profiles. Please provide details on how damping moduli were determined in these calculations to assure that the site response effects calculated in the FIRS calculations are consistent with the development of the UHRS developed in Section 2.5.2. Please provide information on selection of the BE and sigma variation of both shear wave velocity and hysteretic damping used in the randomization process for calculation of surface and FIRS spectra.

The explanation of the treatment of "kappa" in section 2.5.2 is adequate. Section 3.7 should merely refer back to section 2.5.2 to address this issue.

FPL RESPONSE:

Response to the first part of the RAI (pertaining to the treatment of "kappa"):

The treatment of "kappa" for the purpose of generating FIRS is identical to the one implemented for generating GMRS. As stated in Section 3JJ.1 of Appendix 3JJ (as part of Section 3.7.1), the procedure for developing GMRS, including the treatment of "kappa" is described in detail in Section 2.5.2.5.

Response to the second part of the RAI (pertaining to BE and sigma variation):

The base case shear wave velocity profile, described in Section 2.5.4, is used as the BE profile for the randomization process. For the top 600 ft of in-situ soils and rock, the base case profile and its associated sigma variation are developed using seismic velocity data generated from the site specific subsurface investigation. In the case of structural fill and concrete fill layers, the base case shear wave velocity profile and its sigma variation are not provided by direct measurements, instead dynamic properties were developed using other available geotechnical information. The variation of the dynamic properties for the fill layers are based on the recommendations of SRP Section 3.7.2. For deep soil layers, below 600 ft depth, sonic logs obtained from borings made in the vicinity of the site provide the necessary data.

In the case of in-situ soils in the top 600 feet, damping is modeled by strain-dependent damping curves based on RCTS data. Some rock layers in the top 600 ft are expected to behave linearly at the ground motion levels expected at the site, as discussed in Subsection 2.5.4.7.3.3 and the revised response to RAI 02.05.04-16 (Reference 1), and are assigned a strain-independent best estimate damping ratio of 1%. The uncertainty of the strain-independent and strain-dependent curves are obtained from literature

(Reference 2). In the case of the deeper layers, below a depth of 600 ft, strain-independent damping is calculated based on the estimated "kappa" value at the site.

This response is PLANT SPECIFIC.

References:

1. FPL Letter L-2014-285 to NRC dated, October 3, 2014, Voluntary Revised Response to NRC Request for Additional Information Letter No. 040 (eRAI 6006) – Standard Review Plan Section 02.05.04 – Stability of Subsurface Materials and Foundations

2 Costantino, C.J. (1996). Recommendations for Uncertainty Estimates in Shear Modulus Reduction and Hysteretic Damping Relationships. Published as an appendix in Silva, W.J., N. Abrahamson, G. Toro and C. Costantino. (1997). "Description and Validation of the Stochastic Ground Motion Model." Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc. Upton, New York 11973, Contract No. 770573.

ASSOCIATED COLA REVISIONS:

No additional changes to COLA Revision 6 have been identified as a result of this revised response.

ASSOCIATED ENCLOSURES:

None