FPL

L-2010-241
10 CFR 52.3

October 27, 2010

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. 007 (eRAI 4975)
Standard Review Plan Section 3.07.01 – 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

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) 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).

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 October 27, 2010

Sincerely,

William Maher
Senior Licensing Director – New Nuclear Projects

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

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Attachment Number	Attachment Title
1	FPL Response to NRC RAI No. 3.07.01-1 (eRAI 4975)
2	FPL Response to NRC RAI No. 3.07.01-2 (eRAI 4975)
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13	FPL Response to NRC RAI No. 3.07.01-13 (eRAI 4975)

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: 03.07.01-1 (e-RAI 4975)

Figure 2.7 in Section 2.2 of Appendix 3KK (a.k.a. TPG-1000-S2R-802) indicates an East-West cross-section of the plant. It indicates that the plant is to be located on approximately 19 feet of concrete. The concrete extends to the side of the NI approximately 40 feet on each side. Fill is apparently placed atop the lean concrete to the sides of the NI. Section 7.0 of Appendix 3KK indicates that the 3D SSI analyses were performed using the generic NI20 FEM constructed by Westinghouse to determine seismic loads to the plant. This generic NI20 FEM model includes assumed uniform site conditions in and around the plant, rather than the relatively complex site-specific soil profile. Please justify why the FEM used to perform the site-specific SSI analyses did not include the NI20 model amplified by the supporting concrete/side fills as part of the complete structural model to determine the potential impact of this modified configuration on the computed responses.

FPL RESPONSE:

The FEM analysis was a 3D SASSI (System for Analysis of Soil Structure Interaction) analysis using the standard NI20 model and NEAR (soil column close to Nuclear Island) field soil profile (under the Nuclear Island) with infinite extension of lean concrete and backfill soil properties. The comparison of results between the Turkey Point site and the CSDRS (Certified Seismic Design Response Spectra) for the six key locations show a significant margin between the site response and the CSDRS envelope. Further, 2D SASSI parametric sensitivity analyses of the site-specific geological configuration and backfill condition were performed to assess the relatively complex site-specific soil profile. The comparison of results between the AP 2D model and bathtub model for the six key locations are shown in Figure 6-1 through Figure 6-6. The results for the AP1000 stick AP 2D model are similar to those for the bathtub model for most of the frequency ranges. Based on the comparison results, the effects of the lean concrete and backfill soil are minimal.

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-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:

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 2-7 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

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 2-7 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

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 2.7 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.3 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:

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.

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-007

SRP Section: 03.07.01 – **Seismic Design**

Question from Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

NRC RAI Number: 03.07.01-3 (e-RAI 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. 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 3.9.1.7. 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. ACI 207, "Mass Concrete," 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 ACI 207 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.

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.

Reference:

American Concrete Institute, *Guide to Mass Concrete (ACI 207)*, Detroit, MI, 2006

ASSOCIATED COLA REVISIONS:

In response to RAI 03.07.01-3, the following will be updated in a future revision to the COLA.

Add the following to end of second paragraph in FSAR Subsection 2.5.4.5.4:

> Power block excavations are primarily open cuts, with temporary ground support provided by a reinforced concrete diaphragm wall surrounding each power block excavation area. Excavation is performed with standard excavation equipment, but may be supplemented with other methods. The reinforced diaphragm walls resist lateral earth and hydrostatic pressures while providing a barrier to groundwater flow. The reinforced diaphragm walls are seated at about El. -65 feet, just below the most competent portion of the Fort Thompson Formation, to provide a hydraulic seal at the base of the wall. The wall design may include tiebacks and/or other reinforcing methods or it may be circularly shaped in plan view to resist lateral forces. Tiebacks to provide resistance to the lateral earth and hydraulic pressures are installed as necessary based on the final design that includes embedment, spacing and other details, as applicable. **The completed reinforced diaphragm walls will effectively impede any overturning or sliding from the lean concrete fill, provided as a sub-basemat for Category I seismic structures, confined within the walls.**

Add a paragraph between the second and third paragraph in FSAR Subsection 2.5.4.12 as follows:

> **According to ACI 207 (Reference 281), the lean concrete fill under the Nuclear Island is defined as mass concrete. A thermal control plan considering the geometry of the fill concrete, the proposed 1,500 psi strength, total volume of fill concrete placement, and rate of concrete production, will be prepared to minimize thermal cracking in accordance with ACI 207 guidelines.**

Add the following reference to end of FSAR Subsection 2.5.4.13 as follows:

> **281. American Concrete Institute, *Guide to Mass Concrete (ACI 207)*, Detroit, MI, 2006**

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:

With regards to the two statements, the one statement addresses the site which has an area of 0.6 mile radius. 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 specific physiographic region, the Atlantic Coastal Plains, within which the site is located. The ground surface in the physiographic 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 -2.4 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:

FSAR Appendix 3KK, Section 2.0, third paragraph, the first two sentences will be revised in a future revision of the FSAR.

- The site is located within the ~~Gulf~~ **Atlantic** Coastal Plains physiographic ~~province~~ **region**. Elevation of the ground surface in the **physiographic** ~~site~~ **region** ~~varies~~ **ranges** from 3 feet below sea level to ~~400~~ **345** feet above sea level.

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: 03.07.01-5 (e-RAI 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 the Turkey Point Units 6 & 7 COL Application, Revision 1, FSAR Section 2.5.4.8. Please refer to this section for a detailed evaluation of the topic.

Stratum 1 (muck) that currently makes up the surface soils, and is on average about 3 ft thick, will be removed in its entirety prior to commencing major earthwork and replaced with compacted fill. This is addressed in the Turkey Point Units 6 & 7 COL Application, Revision 1, 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 210 m/s (690 ft/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 223 to 236 m/s, exceeding the 210 m/s value. Also, this conservatively ignores any fines contribution. Additionally, the fill average design shear wave velocity of 860 ft/sec 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.

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-007

SRP Section: 03.07.01 – Seismic Design Parameters

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-6 (e-RAI 4975)

Section 5.1 of Appendix 3KK, SASSI Stick Model describes the 2D model of NI used to perform sensitivity analyses of the site. Please describe the details of the bathtub model used in the calculations, such as whether the 19-foot-thick concrete foundation mat is included in the SSI model and whether the configuration of the fill to the side of the NI is included. Please provide information as well on the free-field soil columns and ground motions used in the 2D calculations.

FPL RESPONSE:

The 19 foot thick concrete foundation mat and the fill to the side of NI is shown in Figure 5-2 and detailed in Figure 2-7. The soil and backfill material properties are shown in Table 3-1 and Table 3-2. Table 3-1 is the free-field soil columns property below the bathtub and on the side of the bathtub. Table 3-2 is the lean concrete and backfill soil inside the bathtub. In Figure 2-7, EF-1 and EF-2 are the backfill soil material as shown in Table 3-2 layer 1-8; LC is the lean concrete as shown in Table 3-2 layer 9-12. Lean concrete is the 19 foot thick concrete foundation mat in the bathtub model. The ground motions used in the 2D calculations are shown in Figure 4-4 through Figure 4-6.

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-007

SRP Section: 03.07.01 – Seismic Design Parameters

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-7 (e-RAI 4975)

Section 6.0 of Appendix 3 KK, 2D SASSI Analysis Results presents a comparison of computed 2D SASSI results with the 3D generic design spectra developed for the six key locations of the NI. Given the potential differences in radiation damping between 2D and 3D calculations, please provide justification on why such comparisons shown in Figures 6-1 through 6-6 are appropriate to make.

FPL RESPONSE:

The appropriate comparison is made in Section 6.0 between the 2D site-specific results and AP1000 2D generic results. Figure 6-1 through Figure 6-6 presents the comparisons of the 2D bathtub model, shown in Figure 5-2 (TP_2D) and the generic 2D model AP2D, shown in Figure 5-1 (TP_AP). The ssienv_ap2d is the 2D generic envelope of the six soil cases. The comparison of the two 2D models and the 2D generic envelope is shown in these figures. The responses of the six key locations between the two models are significantly lower than the 2D generic envelope. The generic 3D NI20 envelope (ssienv) was not compared with the 2D response spectra.

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-007

SRP Section: 03.07.01 – Seismic Design Parameters

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-8 (e-RAI 4975)

Please provide information on frequency cut-offs used in the various 2D and 3D SSI calculations, including the effects of the frequency transmission capability of the site profiles used along with the NI20 FEM in the SASSI calculations.

FPL RESPONSE:

The analyzed frequency cut-off is 50 Hz for both the 2D and 3D SASSI calculation. For a maximum analysis frequency of 50 Hz, there are an adequate number of nodes per wavelength to transmit the high frequency through the finite elements in the NI20 model.

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-007

SRP Section: 03.07.01 – Seismic Design Parameters

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-9 (e-RAI 4975)

Section 7.0 of Appendix 3KK, 3D SASSI Analysis, indicates that the original NI20 model was used for the various 3D SSI calculations. During the past year, errors identified in this model led to recent model modifications by Westinghouse and new SSI calculations being performed for the generic sites. In addition, modifications to the Shield Building have been made that have been incorporated into these revised calculations. Please provide information on whether these modifications have been incorporated into these site-specific SSI results.

FPL RESPONSE:

Appendix 3KK was completed in June 2009 prior to the modifications in the model. Therefore, the modifications to the model are not incorporated into the site-specific SSI results.

Appendix 3KK will be updated in a future revision of the FSAR with the results of the revised analysis.

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

Appendix 3KK will be updated in a future revision of the FSAR with the results of the revised analysis.

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 (e-RAI 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 stiff soil layers in the top 600 ft of the soil column, are expected to behave linearly, 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 (Costantino, 1996). In the case of the deeper soil 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:

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 COLA changes have been identified as a result of this 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: 03.07.01-11 (e-RAI 4975)

SASSI calculations generally assume an infinite extent of soil layers. Provide a rationale for the assumptions used in the FAR and NEAR soil profiles with respect to the horizontal extent of soils.

FPL RESPONSE:

The rationale for the assumptions used was to treat the NEAR soil profile as structured soil properties, and the FAR soil profile as soil layers that extend to infinity. The bathtub 2D model was used to perform the SASSI analyses. In this manner, the proper representation of the site soil conditions could be reflected in the analyses performed.

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-007

SRP Section: 03.07.01 – Seismic Design Parameters

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-12 (e-RAI 4975)

Section 3JJ.2 on Page 3JJ-4 of Part 2 - FSAR, Rev. 0 indicates that vertical spectra were obtained by applying V/H ratios obtained from RG 1.60. Please explain why these ratios were selected as opposed to V/H ratios from the generic data base for the characteristic events associated with the site-specific PSHA. Please explain why RG 1.6 V/H ratios were used as opposed to the recommendation available from the full data base that can then be reconciled with the site seismic hazard.

FPL RESPONSE:

NUREG/CR-6728 (Risk Engineering, Inc., 2001) presents ratios of vertical-to-horizontal spectral ground motions [V/H] for both western US [WUS] and central and eastern US [CEUS] rock sites that may be used to scale horizontal response spectra to obtain corresponding vertical rock response spectra. Figure 1 shows the WUS ratios. Figure 2 shows the CEUS ratios. For comparison and subsequent discussion in this RAI response, Figure 3 shows the V/H from Reg. Guide 1.60 (NRC, 1973).

In Chapter 4 of the NUREG it is noted that previous guidance on V/H - such as a simple constant of 2/3, or even the NRC Reg. Guide 1.60 V/H - ignores the dependence of V/H on specific earthquake parameterization. As is apparent from the large WUS empirical earthquake database now available, V/H varies by magnitude, distance, and/or ground motion intensity. The NUREG considers WUS ground motion attenuation relations, based on the empirical data, to recommend peak rock acceleration as an independent variable for V/H, where the peak rock acceleration is argued as an appropriate proxy for V/H dependency on magnitude and distance. The recommended WUS rock V/H ratios as a function of peak rock acceleration are shown in Figure 1.

The FPL Turkey Point site is located in CEUS, which has notably different crustal attenuation characteristics than the WUS. Therefore, different V/H functions are expected. As discussed in the NUREG, there are insufficient data in CEUS, however, to derive CEUS rock V/H ratios directly from empirical records. Similar to the development of horizontal CEUS ground motion attenuation relations (e.g., EPRI, 1993), earthquake source and crustal modeling is used to develop WUS-to-CEUS transfer functions to estimate simplified CEUS V/H ratios from the empirically-based WUS V/H. The recommended CEUS rock V/H ratios as a function of peak rock acceleration are shown in Figure 2.

The FPL Turkey Point site is a soil site where "CEUS rock", characterized by a shear-wave velocity of 9,200 ft/sec, is located at a depth of several thousand feet. The V/H distinction of rock vs. soil adds complexity to defining V/H ratios already distinct for WUS vs. CEUS strong ground motions. The NUREG does not discuss soil V/H in Chapter 4, even for WUS sites, but directs the reader to its Appendix J, where the characteristics of vertical ground motions are discussed and a general outline of a procedure to estimate V/H for specific sites is presented. The procedure, as it is, is based on using the significant WUS database and developing WUS-to-CEUS and rock-

to-soil transfer functions, but is not unambiguous or prescriptive, leaving some room for interpretation as to how to apply this procedure. The discussion of vertical-to-horizontal ground motion ratios, however, is very useful, and includes the following points:

- V/H, regardless of WUS or CEUS, rock or soil, tends to be higher for higher spectral frequencies than for lower spectral frequencies. For high frequencies V/H can be greater than 1.0, and for low frequencies V/H can be less than 2/3.
- Particularly in the higher frequencies, V/H tends to increase with increasing magnitude and/or decreasing distance; the NUREG concludes that a proxy of V/H tending to increase with increasing peak rock acceleration is valid for capturing the primary character of magnitude and distance dependence. Soil tends to enhance these dependences compared to rock.
- CEUS V/H generally peaks at higher frequencies than the corresponding V/H peak for WUS.
- The high frequency peak of V/H tends to be broader for soils sites than for rock sites, particularly for CEUS.

For ESP/COL sites similar to Turkey Point – that is, CEUS deep soil sites with low peak rock acceleration [< 0.1g]; e.g. STP (STPNOC, 2009) and Victoria County Station (Exelon Generation, 2010) – application of the NUREG/CR-6728 Appendix J approach, along with the characteristics of V/H, discussed above and shown in the low-acceleration panels of Figures 1 and 2, resulted in a recommendation of Reg. Guide 1.60 as adequately and simply defining V/H for these sites. The same recommendation is made for the FPL Turkey Point site.

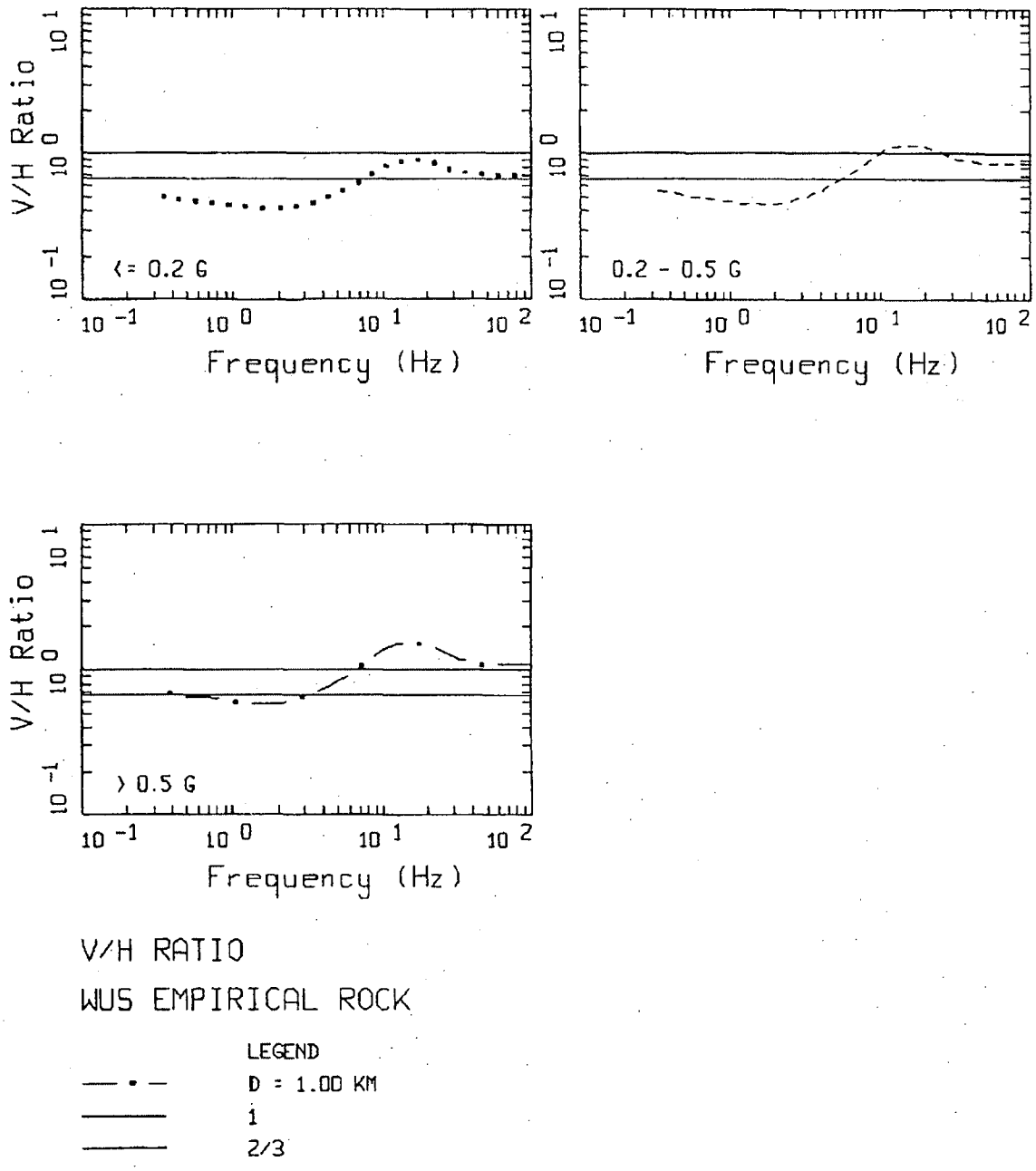

Figure 1. V/H ratios for WUS rock sites, as given in NUREG/CR-6728 (Risk Engineering, Inc, 2001).

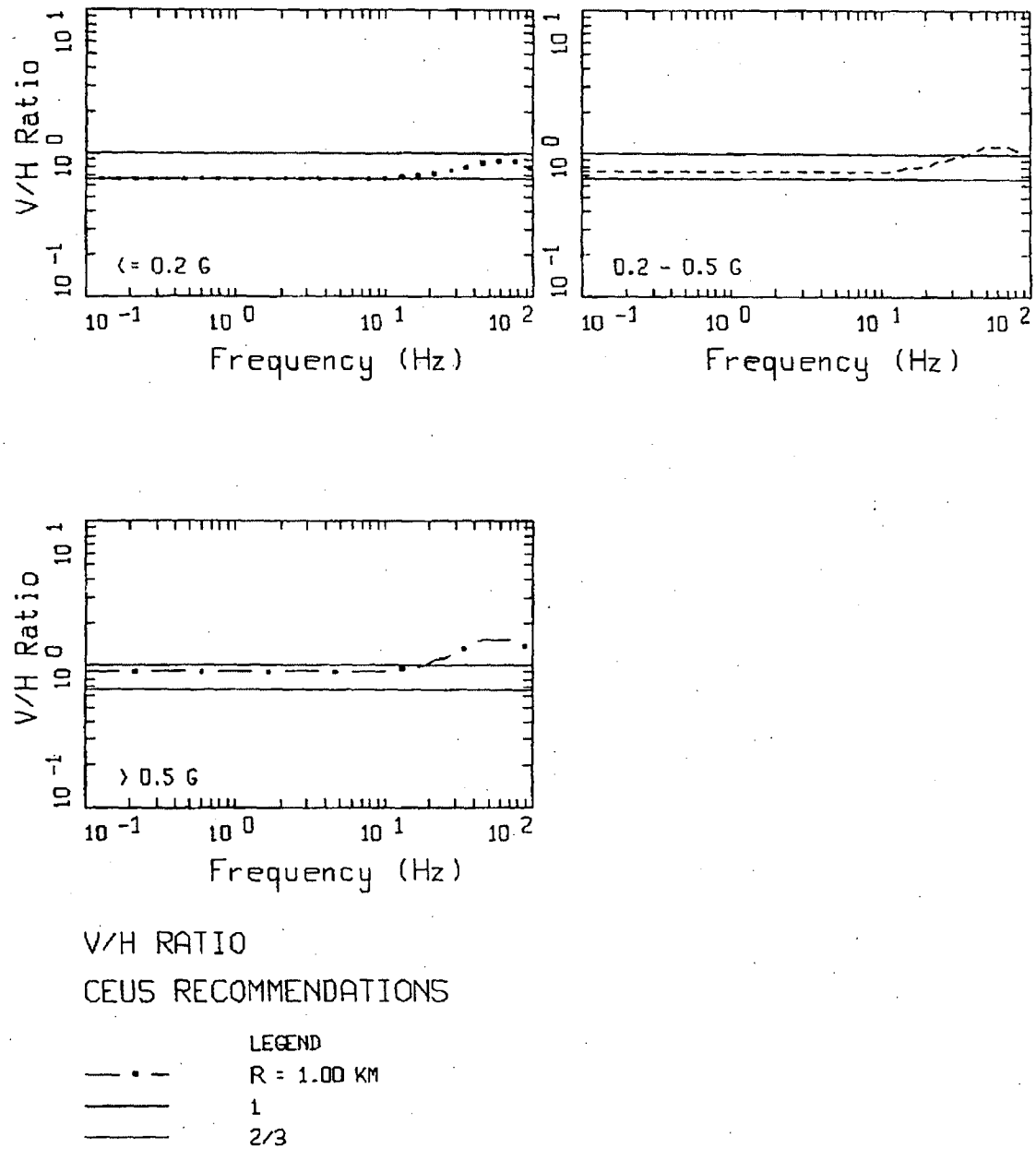

Figure 2. V/H ratios for CEUS rock sites, as given in NUREG/CR-6728 (Risk Engineering, Inc, 2001).

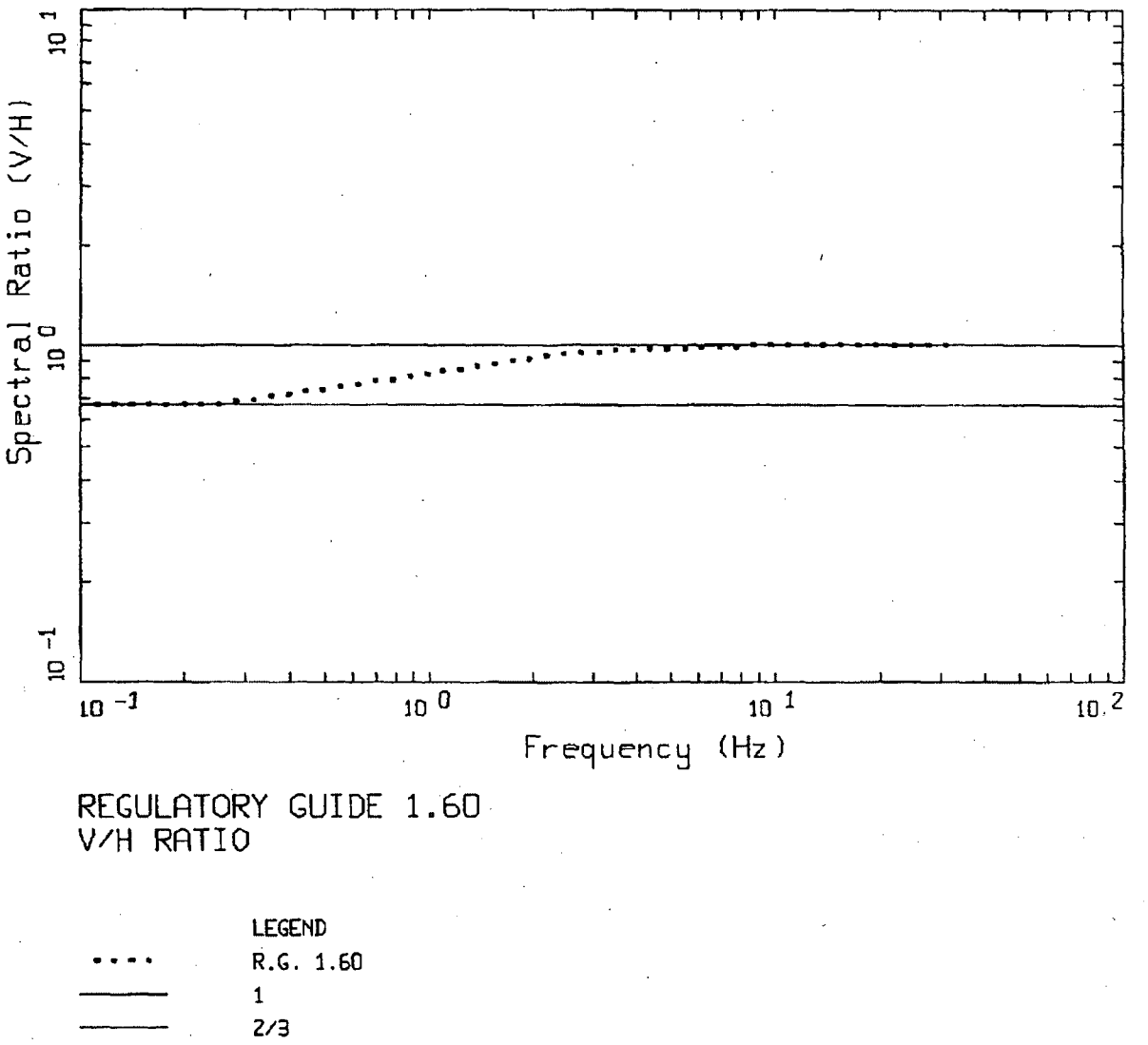

Figure 3. V/H ratios for Reg. Guide 1.60 (NRC, 1997), as plotted in Figure 4-31 of NUREG/CR-6728 (Risk Engineering, Inc, 2001).

This response is PLANT SPECIFIC.

References:

Electric Power Research Institute (1993). "Guidelines for Determining Design Basis Ground Motions," Volumes 1-5, EPRI TR-102293, Electric Power Research Institute, Palo Alto, California.

Exelon Generation (2010). Victoria County Station ESP, SSAR, Rev. 0 - SSAR Chapter 02 - Site Characteristics - Section 02.05.02 - Vibratory Ground Motion, Nuclear Regulatory Commission document ML101030933, March 2010.

Nuclear Regulatory Commission [NRC] (1973). "Regulatory Guide 1.60 – Design Response Spectra for Seismic Design of Nuclear Power Plants," U.S. Nuclear Regulatory Commission, Washington, DC, December 1973, Revision 1.

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

STPNOC (2009). South Texas Project Units 3 & 4 COLA (FSAR), Rev. 3 - Tier 2 - Chapter 02 - Site Characteristics - Section 02.05S.02 Vibratory Ground Motion, Nuclear Regulatory Commission document ML092931277, September 2009.

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-007

SRP Section: 03.07.01 – Seismic Design Parameters

Question from Structural Engineering Branch 1

NRC RAI Number: 03.07.01-13 (e-RAI 4975)

Section 3JJ.4, Spectral Matching of Acceleration Time Histories, indicates that seed records were selected from the database given in NUREG/CR-6728 for the LF de-aggregation results for magnitudes > 7 and distances > 500 km. However, the TAP024 record was selected from the Chi-Chi seismic event, which was indicated to be about 100 km from the recording station. Please indicate why this particular record was selected as opposed to another more distant event. In addition, please indicate whether these records have appropriate characteristics to match the damping associated with SSI and not just the 5% damped spectra.

FPL RESPONSE:

Guidance for the selection of the seed input time history for the spectral matching procedure was obtained from the deaggregation results of the probabilistic seismic hazard analysis (PSHA) and the availability of candidate strong ground motion time histories. Based on the low-frequency (i.e., 1-2.5 Hz) deaggregation results, the mean magnitude and distance values for annual exceedance frequencies of 10^{-4} to 10^{-6} are approximately a magnitude 7.25 and a distance between 550 – 600 km (Risk Engineering Inc., 2008). Note that these mean values are computed based on the contribution from seismic sources with distances greater than 100 km based on the percentage contribution to the total hazard of these more distant sources being greater than 5% of the total hazard (NRC, 2007).

The selection of the candidate seed input time history for any spectral matching procedure should be based on the controlling event for a given site. However, the lack of empirical strong ground motion time histories in the eastern US (CEUS) significantly limits the availability of candidate time histories for large magnitude earthquakes (i.e., M > 7) at large distances (i.e., > 500 km).

The time history database contained in NUREG/CR-6728 (Risk Engineering, Inc., 2001) was compiled to provide times histories that could be used as seed input time histories for either a spectral matching or spectral scaling analysis. This database of time histories was developed for both the western US (WUS) and eastern US (CEUS) regions based on defined magnitude and distance bins of engineering interest. The WUS magnitude-distance bins were populated from the database of empirically recorded strong ground motions time histories. For the CEUS magnitude-distance bins, the dearth of empirical strong ground motion time histories required the population of these bins to be based on modifying empirically recorded WUS time histories to have spectral shapes more appropriate for the CEUS (i.e., to increase their response spectral amplitudes in the 10 Hz to 100 Hz frequency range relative to the WUS spectra with the same PGA). The largest magnitude and distance bin in this database is one for events with magnitudes > 7 and distances between 100 – 200 km. Based on the deaggregation results, the candidate time histories were selected from this largest magnitude-distance bin.

The 15 sets of three component time histories that are given in the noted magnitude-distance bin span the magnitude and distance ranges of 7.3 – 7.6 and 100 – 165 km, respectively. An additional consideration that was implemented in the selection procedure was to select a seed input time history set that was not significantly bandpass filtered as part of the original data processing. The selection of a candidate seed input time history that is severely bandpass filtered for the spectral matching procedure can lead to a spectrum-compatible time history that has significantly different non-stationary characteristics arising from the need to increase the spectral amplitudes over a large frequency range, especially at the lower frequency range (e.g., less than 1 Hz).

Based on these considerations, the recording from the station TAP024 from the 1999 Chi-Chi earthquake was selected as the candidate seed input time history. This earthquake had a magnitude of 7.6 and the TAP024 recording station was located 100.2 km from the fault plane. The bandpass filter corners were 0.02 Hz and 50.0 Hz, which is the largest frequency range between the two filter corners of any of the 15 candidate seed time histories within this magnitude-distance bin. Although the distance of 100.2 km is the smallest within the database bin, the larger frequency range between filter corners and larger magnitude made the TAP024 recording an acceptable seed input time history for the spectral matching analysis.

These three-component seed input time histories were modified to be spectrum compatible to corresponding 5% spectral damped target spectra (i.e., horizontal and vertical). The spectral matching procedure followed the guidelines and requirements given in NUREG/CR-6728 all for a spectral damping level of 5%. The applicability of these spectrum compatible time histories for spectral damping levels other than 5% has not been, but could be, evaluated with the development of any required target spectra for other spectral damping levels.

This response is PLANT SPECIFIC.

References:

Risk Engineering Inc. (2001). "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.

Nuclear Regulatory Commission (2007). "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion," Reg. Guide 1.208, March, 2007.

Risk Engineering Inc. (2008). "Deaggregation of 10-4, 10-5, and 10-6 Rock seismic hazard for the Turkey Point Site (Task 9)," 0822-ACR-032, Rev. 3, December 2008.

ASSOCIATED COLA REVISIONS:

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

ASSOCIATED ENCLOSURES:

None