FPL

L-2011-552
10 CFR 52.3

December 16, 2011

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. 040 (eRAI 6006)
SRP Section - 02.05.04 Stability of Subsurface Materials and Foundations

Reference:

1. NRC Letter to FPL dated October 18, 2011, Request for Additional Information Letter No.040 Related to SRP Section 02.05.04 - Stability of Subsurface Materials and Foundations for the Turkey Point Nuclear Plant Units 6 and 7 Combined License Application
2. FPL Letter to NRC dated November 16, 2011, Response and Response Schedule to NRC Request for Additional Information Letter No. 040 (eRAI 6006) SRP Section - 02.05.04 Stability of Subsurface Materials and Foundations

Florida Power & Light Company (FPL) provides, as attachments to this letter, its responses to the Nuclear Regulatory Commission's (NRC) Request for Additional Information (RAI) 02.05.04 -9, RAI 02.05.04-10, RAI 02.05.04-12, RAI 02.05.04-14, RAI 02.05.04-16, RAI 02.05.04-19, and RAI 02.05.04-22 provided in Reference 1. FPL provided a schedule for the responses to RAI 02.05.04 -9, RAI 02.05.04-10, RAI 02.05.04-12, RAI 02.05.04-14, RAI 02.05.04-16, RAI 02.05.04-19, and RAI 02.05.04-22 in Reference 2. The attachments identifies 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.

D097
NRO

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

Executed on December 16, 2011

Sincerely,

William Maher
Senior Licensing Director – New Nuclear Projects

WDM/RFB

Attachment 1: FPL Response to NRC RAI No. 02.05.04-9 (eRAI 6006)
Attachment 2: FPL Response to NRC RAI No. 02.05.04-10 (eRAI 6006)
Attachment 3: FPL Response to NRC RAI No. 02.05.04-12 (eRAI 6006)
Attachment 4: FPL Response to NRC RAI No. 02.05.04-14 (eRAI 6006)
Attachment 5: FPL Response to NRC RAI No. 02.05.04-16 (eRAI 6006)
Attachment 6: FPL Response to NRC RAI No. 02.05.04-19 (eRAI 6006)
Attachment 7: FPL Response to NRC RAI No. 02.05.04-22 (eRAI 6006)

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-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-9 (eRAI 6006)

In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please describe how shear and compressive wave velocity values are selected for design in Table 2.5.4-209. Provide correlations for shear and compressive wave velocity between Table 2.5.4-209 and Table 2.5.4-215 to indicate what percentile is used for recommended values. Given the large deviations, especially on Key Largo and Fort Thompson Formations (see Figures 2.5.4-218 and 2.5.4-219 and Table 2.5.4-215), explain how the selected single value for each stratum statistically reflects the entire layer.

FPL RESPONSE:

The measured shear wave velocity (V_S) and compression wave velocity (V_P) values from each of 10 suspension P-S velocity logging tests are plotted against depth in FSAR Figures 2.5.4-218 and 2.5.4-219, respectively. Using all of these test results, V_S values were averaged over selected vertical depth intervals for the power block area, and a mean V_S profile with low/high end boundaries (mean $\pm$ standard deviation) was obtained, as shown in the FSAR Table 2.5.4-215. The same approach was applied to the V_P measurements and the resulting mean profile with low/high end boundaries is also included in FSAR Table 2.5.4-215. This methodology was followed for the Miami Limestone between depths of 0 and 30 ft, for the Key Largo Limestone between depths of 20 and 50 ft, for the Fort Thompson Formation between depths of 50 and 120 ft, for the Tamiami Formation between depths of 120 and 210 ft, and for the Peace River Formation between depths of 210 and 450 ft. Note that the interval from 20 to 30 ft depth was included in the analyses for both the Miami Limestone and the Key Largo Limestone as explained in the next paragraph. As an example, a summary of the V_S model for the Key Largo Limestone is presented as follows.

Depth (ft)	V_s (ft/sec) Mean	V_s (ft/sec) Mean - Std. Dev.	V_s (ft/sec) Mean + Std. Dev.
20 to 30	4,691	2,580	6,455
30 to 40	5,871	4,785	7,675
40 to 50	6,834	5,302	8,194
Average	**5,799**	4,222	7,441

V_S readings were taken at 1.64-ft intervals in each boring, i.e., there were 6 readings per 10 ft depth. The depth interval between 30 and 40 ft in the above table is used here as an example to explain how the mean and standard deviations in the above table were derived. All 10 V_S borings covered this depth interval. Using the 10 V_S readings, a mean V_S value with plus/minus standard deviation was computed and was assigned at each 1.64-ft interval.

By averaging the 6 mean V_S values that fall in the 10-ft increment from 30 to 40 ft depth, the mean V_S of 5,871 ft/sec was obtained. The minimum of the 6 mean minus standard deviation V_S values was assigned as the low end V_S boundary (4,785 ft/sec), whereas the maximum of the 6 mean plus standard deviation V_S values is assigned as the high end V_S boundary (7,675 ft/sec). The mean V_s of 5,799 ft/sec for the Key Largo Limestone was computed by taking the average of these three mean V_S values, each corresponding to a 10-ft vertical depth interval. A best-estimate V_S of 5,800 ft/sec is recommended as shown in FSAR Table 2.5.4-209. In deeper strata, when only one boring remained (as in part of the Peace River Formation and all of the Arcadia Formation) the standard deviation was not computed because there was only a single V_S value measured at each 1.64-ft interval. Note also that FSAR Table 2.5.4-201 indicates the top of the Key Largo Limestone as El. -27 ft (North American Vertical Datum 1988, NAVD 88), which corresponds to a depth of approximately 27 ft, i.e., above 27 ft depth the readings reflect the softer Miami Limestone. Therefore, including the mean V_S obtained between depths of 20 and 30 ft in the analysis reduces the average value somewhat.

Also, for the overlying Miami Limestone, the V_S measurements were averaged starting from the ground surface over 5-ft vertical intervals to 10 ft depth, below which 10-ft vertical intervals were used. For all other formations, the V_S measurements were averaged over 10-ft vertical intervals.

As mentioned earlier, for the Tamiami Formation, the V_S measurements were averaged between 120 and 210 ft depth. FSAR Table 2.5.4-201 indicates that the top of Tamiami Formation is at El. -115 ft and the bottom is at El. -215 ft, corresponding to the depths of approximately 115 and 215 ft, respectively. The mean V_S increases from 1769 ft/sec between 200 and 210 ft depth to 2235 ft/sec between 210 and 220 ft depth. The reason for such an increase is the early presence of the Peace River Formation between 210 and 220 ft depth. For design purposes, it is reasonable to exclude the V_S measured between the depths of 210 and 220 ft for obtaining best-estimate values. In general, the Upper Tamiami, which extends to 160 ft depth, has a lower V_S than the Lower Tamiami. The best-estimate V_S values of 1,400 and 1,600 ft/sec are recommended for the Upper and Lower Tamiami Formations.

The Poisson's ratios (μ) derived from the 10 suspension P-S velocity logs were analyzed with a methodology similar to that used in the V_S assessment. As an example, the μ model (mean with low/high end boundaries) is presented for the Key Largo Limestone as follows.

Depth (ft)	μ Mean	μ Mean - Std. Dev.	μ Mean + Std. Dev.
20 to 30	0.33	0.24	0.41
30 to 40	0.30	0.23	0.35
40 to 50	0.29	0.23	0.35
Average	**0.31**	0.23	0.37

Table 4-11 of FSAR Reference 2.5.4-217 presents the values of μ in the range of 0.24 and 0.45 for limestone rock. Compared to these published values, it seems reasonable to use a μ of 0.31 for the Key Largo Limestone.

Using the average V_S (5,799 ft/sec) and the Poisson's ratio (0.31) in FSAR Eq. 2.5.4-1, which defines the relationship between V_S, V_P and μ, the average compression wave velocity is computed as 11,051 ft/sec. A best-estimate V_P of 11,000 ft/sec is recommended for the Key Largo Formation as shown in FSAR Table 2.5.4-209. Following the same methodology, the best estimate values for V_S, V_P and μ are recommended for other formations, and are presented in FSAR Table 2.5.4-209.

In summary, this response describes how the V_S, V_P and μ values given in FSAR Table 2.5.4-209 were derived. The V_S and V_P are best-estimate values and are not meant to reflect the variation of these parameters within each layer. The statistical variation in V_S and V_P with increasing depth is demonstrated in FSAR Table 2.5.4-215. As discussed in FSAR Subsection 2.5.2.5.2, FSAR Table 2.5.4-215 is the source from which the randomized V_S profiles were developed for the site response analyses.

This response is PLANT SPECIFIC.

References:

None.

ASSOCIATED COLA REVISIONS:

None

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-10 (eRAI 6006)

FSAR Figures 203 through 209 indicate one boring for each of the two Units extending to a depth of about 450'. Most other borings taken at the site extend to depths of only about 150'. Figure 2.5.4-220 presents information on shear wave velocity, including best estimate (BE) and upper/lower bound (UB/LB) values down to a depth of about 600'. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please indicate how you estimated variations in shear wave velocity based on only two readings over the deeper portion of the profile.

FPL RESPONSE:

The soil formations are present between the depths of about 115 and 450 ft. The Upper Tamiami Formation extends from 115 to 160 ft depth and consists of dense to very dense silty sand. The Lower Tamiami Formation is present from 160 to 215 ft and consists of very stiff to hard sandy silt with minor amounts of silty clay. The Peace River Formation (part of Hawthorn Group), which is underlain by the Arcadia Formation (rock), extends from 215 to 450 ft depth and is a very dense silty sand.

FSAR Figure 2.5.4-220 presents the mean shear wave velocity (V_S) profile with low/high end boundaries (mean $\pm$ standard deviation) as a function of depth, and is a compilation of the 10 suspension P-S velocity data sets, from B-601(DH), B-604(DH), B-608(DH), B-610(DH), B-620(DH) in Unit 6, and B-701(DH), B-704G(DH), B-708(DH), B-710G(DH), B-720G(DH) in Unit 7. The response to RAI 02.05.04-9 describes how the statistical variation in the V_S profile was derived. The V_S measurements were taken in two borings [B-604(DH), B-704G(DH)] to a depth of 150 ft, in two borings [B-620(DH), B-720G(DH)] to a depth of 200 ft, and in four borings [B-608(DH), B-610(DH), B-708(DH), B-710G(DH)] to a depth of 250 ft. V_S boring B-601(DH) extended to a depth of 400 ft, while B-701(DH)] extended to 600 ft. Thus, the V_S measurements of the Upper Tamiami Formation were obtained in the entire thickness of the layer in eight borings and down to 150 ft depth in two borings. In the Lower Tamiami Formation, the V_S measurements were taken in the entire thickness of the layer in six borings and down to 200 ft depth in two borings. In the Peace River Formation, the V_S measurements were obtained in the entire thickness of the layer in one boring and down to 400 ft depth in another boring. V_S measurements were made in one boring in the Arcadia Formation. As noted in the response to RAI 02.05.04-9, when only one boring remained, the standard deviation was not computed. Thus, there is no variation presented below 400 ft depth in FSAR Figure 2.5.4-220 (or FSAR Table 2.5.4-215).

To account for variations in the V_S profile across the site, 60 randomized profiles were generated using a stochastic model as discussed in the FSAR Subsection 2.5.2.5.2. The average V_S profile in FSAR Figure 2.5.4-220 was utilized as the base-case of the site response analyses. The standard deviation of $\ln(V_S)$ (the natural logarithm) as a function of depth was used to define the variation in the randomization process. The input profiles of the median and plus/minus one standard deviation of the V_S are shown in the FSAR Figure 2.5.2-236.

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

None

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-12 (eRAI 6006)

Section 2.5.4.5.2 "Extent of Excavations, Fills, and Slopes", states that the TPNPP Units 6 and 7 nuclear islands will be founded directly on a 20 ft thick lean-concrete layer above a competent rock stratum (Key Largo Formation). In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please address the following:

a. Define "Lean Concrete" and clarify if CLSM is used. Also specify which ACI standard(s) will be followed.
b. Given the load path, how is the potential for cracking of the lean concrete evaluated? Also discuss your plan to control thermal cracking of the fill materials.
c. Describe the load transfer mechanism between the base of the NI structures and the lean fill concrete as well as the load transfer between the lean concrete and the surrounding supporting soils.
d. Your chemical tests of soil and rock indicated that that the chemistry of soil and rock is considered to be aggressive towards cementitious materials. Please provide test results on groundwater chemistry including pH, chlorides, and sulfates. Evaluate the potential aging effects and address the concrete durability for lean concrete backfill and subfoundation due to aggressive soil and groundwater conditions. Also provide a description on how potential settlement and differential settlement due to erosion of cement from porous lean concrete backfill will be addressed.

FPL RESPONSE:

a. Lean concrete is unreinforced concrete with a smaller ratio of cement to aggregate than structural concrete. It is used for filling and not structural duties. The American Concrete Institute (ACI) standard that will be followed is ACI 207.1R-05, "Guide to Mass Concrete" prepared by ACI Committee 207. Controlled Low Strength Material (CLSM) will not be used.

b. FSAR Section 2.5.4.12 indicates the lean concrete will have a minimum compressive strength of 1500 psi. According to FSAR Reference 2.5.4-273, the design bearing capacity of this strength of concrete is 128 ksf. According to the AP 1000 Design Control Document, the maximum applied bearing pressure (from the Reactor Building) is 8.9 ksf, less than 7 percent of the bearing capacity of the concrete. Thus, cracking of the concrete due to loading/overstressing is not an issue.

ACI (2007) 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 approximately 20-foot thick layer of concrete fill qualifies as mass concrete. As such, ACI (2007) guidelines for preventing thermal cracking in concrete will be followed in preparing a thermal control plan during the detailed design. A thermal control plan can include some or all of the following elements:

- Use a well-graded aggregate and Type I and/or Type II cement in the concrete mix.
- The low strength of the concrete fill will require relatively less cement and thus reduce the level of the heat of hydration found in stronger mixes. To reduce the heat of hydration further, use of Portland cement substitutes such as Class F flyash to replace a portion of the cement. Flyash has a slower pozzolanic reaction than cement, and thus less heat of hydration. Uncontrolled heat of hydration is the cause of thermal cracking and thus minimizing the heat of hydration will greatly reduce the possibility of thermal cracking.
- Even with the heat of hydration in the design mix minimized, it may still require the concrete fill to be placed in relatively thin lifts to avoid cracking. Thus, maximum thickness of each concrete fill lift will be set at around 3 feet.
- When another lift is required on top of an existing lift, the new lift will be poured only after the underlying lift has enough time to properly cool down.
- Concrete design and placement will be tailored to minimize the maximum temperature inside the concrete pour and to minimize the maximum temperature difference between the hottest spot and the surface of the concrete pour. The exposed surfaces will be insulated as required to limit the temperature differential in the concrete mass to 20°C maximum. This will necessitate that thermocouples be embedded within and on the concrete mass, which will effectively eliminate the potential for thermal cracking. Concrete placement temperature will be controlled as necessary by the use of ice, chilled water, shading aggregate piles, spraying coarse aggregate for evaporative cooling, and scheduling placements to take advantage of coolest temperatures (such as at night).

c. As described in Response b, the maximum applied bearing pressure to the concrete fill (from the Reactor Building) is less than 7 percent of the bearing capacity of the concrete fill. The rock beneath the concrete fill (Key Largo Formation) has the same compressive strength as the concrete fill (1,500 psi) and the rock beneath the Key Largo (Fort Thompson Formation) has a slightly higher strength of 2,000 psi (FSAR Table 2.5.4-209). Thus, during vertical load transfer from the foundation to the concrete fill and from the concrete fill to the underlying rock, stress levels will remain low in these materials and well within the elastic range. Consequently, there will be elastic stress distribution.

For transfer of lateral loading, FSAR Table 2.5.4-209 shows coefficient of friction against sliding between mass concrete and the Key Largo Formation to be 0.7. This coefficient value applies to resistance to sliding of the concrete fill bearing on the Key Largo Formation. The 0.7 value also applies to the resistance to sliding of the concrete foundation basemat bearing on the mudmat and the mudmat bearing on the concrete fill. As noted in FSAR Subsection 3.8.5.1, a sheet type HDPE waterproofing material will be used for both the horizontal and vertical surfaces under Seismic Category I structures. The material will be qualified by test, with commercial grade dedication and laboratory testing, to achieve a minimum coefficient of friction against sliding of 0.55, as shown in FSAR Subsection 3.8.5.1 (provided in COLA Revision 3). This waterproof membrane is sandwiched within the mudmat.

Because of the low seismic forces (and hence lateral loading) at the Turkey Point site, the friction between the foundation and the mudmat, the friction within the mudmat (waterproofing material), the friction between the mudmat and the concrete fill, and the friction between the concrete fill and the underlying rock will be sufficient to prevent any sliding movement. Thus the surrounding backfill and in-situ soils and rock will not be required to resist lateral loading from the building.

d. The measured values of chemical tests on groundwater samples from observation wells on the site are presented in FSAR Tables 2.4.12-210 (pH) and 2.4.12-211 (chloride and sulfate). The pH values measured from 24 water samples ranged from 6.65 to 7.29, resulting in a median of 7.06, i.e., essentially neutral. The chloride values measured from 24 water samples ranged from 16,300 to 37,500 ppm, resulting in a median value of about 29,000 ppm. The sulfate values measured from 24 water samples ranged from 2,280 to 4,400 ppm, resulting in a median value of about 3,800 ppm, or close to 0.4 percent by weight. This classifies the concrete exposure to sulfate attack as severe, according to the ACI Manual of Concrete Practice, Part 1. FSAR Tables 2.4.12-210 and 2.4.12-211 contain other parameters measured from chemical tests; these are considered inapplicable to the evaluation because they are not corrosion agents.

The approximate plan dimensions of the 20-foot thick mass of concrete fill are 240 feet x 290 feet, including 30-foot width of fill extending beyond the perimeter of the nuclear island. The fill will be placed on top of Key Largo Limestone that will have been extensively grouted to enable dewatering. The fill will be placed against the perimeter concrete diaphragm wall that extends down to El. -60 feet, as shown in FSAR Figure 2.5.4-222 (provided in COLA Revision 3). The majority of the surface of the concrete fill will be covered by the nuclear island, and the remainder will be covered by structural fill. Thus, there will be limited exposure of the concrete fill to aggressive groundwater and soil. On the perimeter, there is a 30-foot wide buffer of concrete fill placed against a concrete diaphragm wall, and on the surface, most of the concrete fill is covered by structures. The only plausible potential for exposure is on the base of the fill. One (of several) potential solutions to this situation would be to make the first lift of concrete fill from sulfate resisting cement. The high chloride content is not of concern, since the concrete is unreinforced.

Under the conditions described above, there is no mechanism to cause erosion of cement from the concrete fill, and thus there will be no impact on total or differential settlement.

This response is PLANT SPECIFIC.

References:

American Concrete Institute (ACI). *Report on Thermal and Volume Change Effects on Cracking of Mass Concrete, ACI 207.2R-07,* Reported by ACI Committee 207, Farmington Hills, MI, 2007.

ASSOCIATED COLA REVISIONS:

None

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-14 (eRAI 6006)

FSAR Figure 2.5.4-222 shows a general conceptual excavation cross-section. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please describe the procedures that will be followed during site excavation and construction activity to ensure that the appropriate strata for the proposed foundation locations are confirmed through objective measures and the exposed foundation laying surface is uniform. Also, please provide the vertical and horizontal extent of all seismic categories I excavations, fills, and slopes, including the locations and limits of excavations, fills, and backfills on plot plans and geologic sections and profiles.

FPL RESPONSE:

Excavation to support construction of the Nuclear Island safety-related foundations will involve the removal of the top layer of organic material or "muck", approximately four to six feet in thickness, followed by removal of all the underlying Miami Limestone down to approximate Elevation -35 feet NAVD 88. Removal of the Miami Limestone will expose the top of the Key Largo Limestone layer which will be left undisturbed.

The profiles from the power block subsurface investigation (refer to FSAR Figures 2.5.4-203 through 2.5.4-208) show that the subsurface strata to support foundations are relatively horizontal. However, it should be noted that the extent of excavation to final subgrade and/or to final over-excavation level is determined during construction. This determination is based on observation of actual subsurface conditions encountered, and their suitability for foundation support.

Once subgrade suitability at the proposed bearing stratum is confirmed, Nuclear Island excavations are backfilled with lean concrete fill up to the foundation level of the structures. Adequate top surface uniformity and integrity of the lean concrete fill will be verified prior to placement of mudmat(s) and waterproofing membrane in accordance with good construction practices. Structural fill used as backfill against the Nuclear Island is controlled and placed in accordance with a quality program per Appendix B of 10 CFR Part 50.

From FSAR Subsection 2.5.4.2.1.2 "Description of Soil and Rock Strata", the following is a description of each soil and near-surface rock stratum encountered in the subsurface investigation for the power block areas and what is expected to be observed during the excavation process. The stratum thickness indicated in each description for the power block is the calculated average within the two power block units (Unit 6 and Unit 7) because the subsurface conditions encountered in the subsurface boring program are relatively uniform.

- **Muck** consists primarily of light gray to black silty clay with varying amounts of sand and peat. Typically, this stratum contains trace organics near the surface. This stratum has a very soft to medium-stiff consistency. The thickness of Stratum 1 ranges from 2 to 7 feet, with an average of 3.4 feet. The top of this layer is typically at El. –1.2 feet. The average base elevation of this stratum is –4.6 feet.

- **Miami Limestone** (or the Miami Oolite, as it is referred to in some publications) is encountered at elevations ranging from –3.3 to –8.3 feet. The range of thickness for the Miami Limestone varies from 17.2 to 30.3 feet with an average of 22.6 feet. This stratum consists of pale yellow, light brownish gray, and white limestone. It has a porous, sometimes fossiliferous texture, comprising oolite grains with varying carbonate cementation. Observed fossils include mollusks, bryozoans, and corals. This stratum has very weak to weak consistency, depending on the degree of cementation.

- The top of **Key Largo Limestone** is encountered between El. –24.1 and El. –35.3 feet, at an average of El. –27.2 feet. The thickness varies between 13.5 and 28.0 feet in the borings, with an average thickness of 22.3 feet. The Key Largo Limestone is a coralline, porous formation with recrystallized calcite infill visible in core samples. The color varies between white, pale yellow, light brownish gray, and gray.

The Miami Limestone is considerably softer than, and has a quite different structure, from the coralline Key Largo Limestone. During the excavation process, onsite geotechnical engineers and geologists will validate complete removal of Miami Limestone to the Key Largo Limestone layer by visual inspection and by hardness testing.

An approximate 19-foot thick layer of lean concrete will be poured from exposed Key Largo Limestone layer at Elevation -35 ft to approximate Elevation -16 ft. As noted earlier, adequate top surface uniformity and integrity of the lean concrete fill will be verified in accordance with good construction and quality-control practices.

As noted in FSAR Subsection 2.5.4.5.3, fill placement and compaction control procedures are addressed in a technical specification prepared at project detailed design. The specification includes requirements for suitability of the various required fill materials and sufficient testing to address potential material variations. The specification also includes requirements for an onsite testing firm for quality control, especially to ensure specified material gradation and plasticity characteristics, the achievement of specified moisture-density criteria, earthwork equipment, maximum lift thickness, and other requirements to ensure that fill operations conform to a high standard of practice. The onsite testing firm is required to be independent of the earthwork contractor and to have an approved quality assurance/quality control program. A sufficient number of laboratory tests are required to ensure that any variations in the various required fill materials are accounted for. A materials-testing laboratory is established onsite to exclusively serve the project site work.

All excavations, lean concrete and backfills for safety-related structures will be controlled and placed in accordance with a quality program per Appendix B of 10 CFR Part 50.

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

Revised FSAR Figures 2.5.4-203 through 2.5.4-209 will be included in a future FSAR revision to show vertical and horizontal extent of excavations, fill locations and slopes. Note that dimensions and slopes on figures as shown are preliminary pending final design

Figure 2.5.4-203 Geotechnical Cross Section D-D' Through Unit 6 Power Block

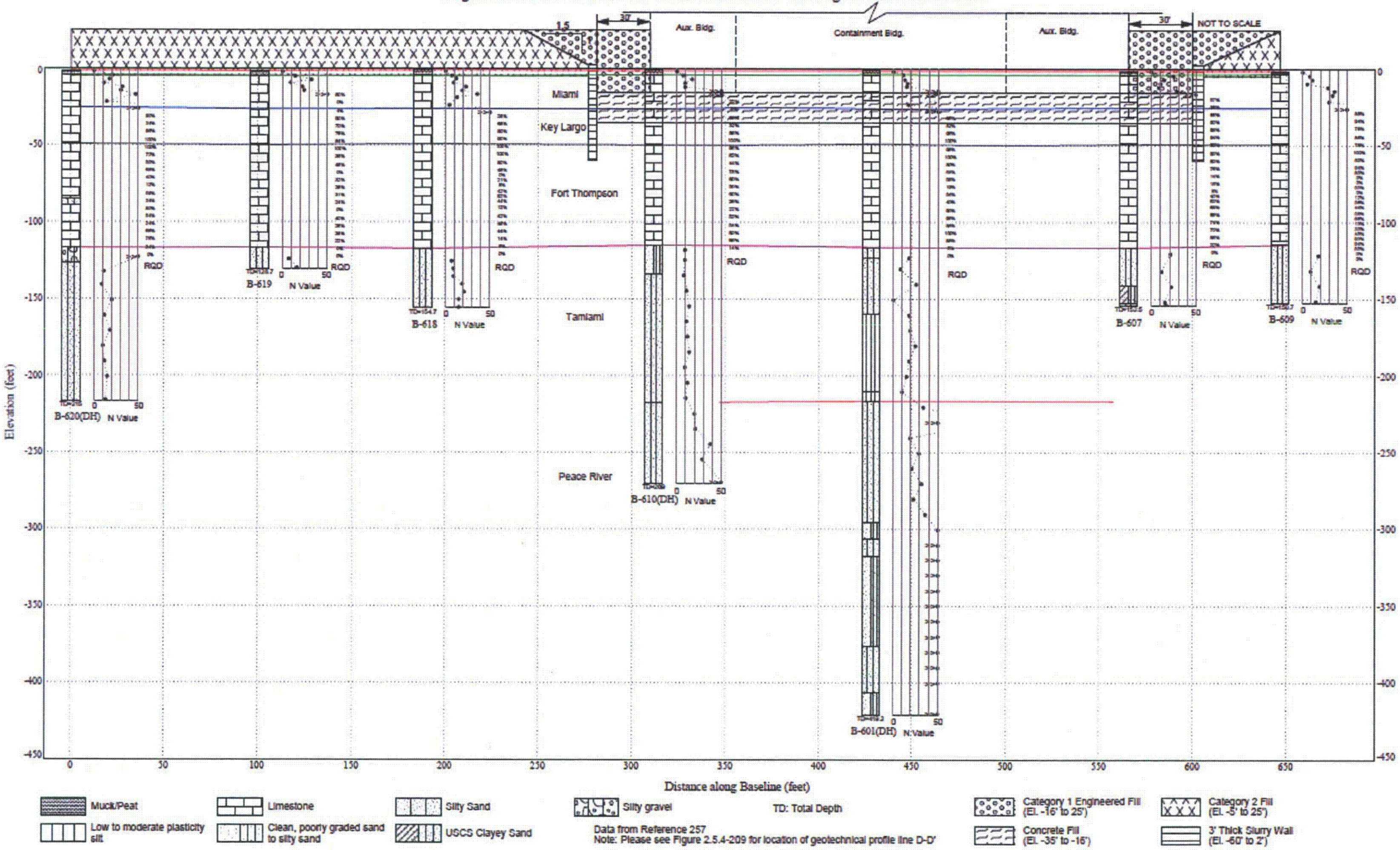


Figure 2.5.4-204 Geotechnical Cross Section E-E' Through Unit 6 Power Block

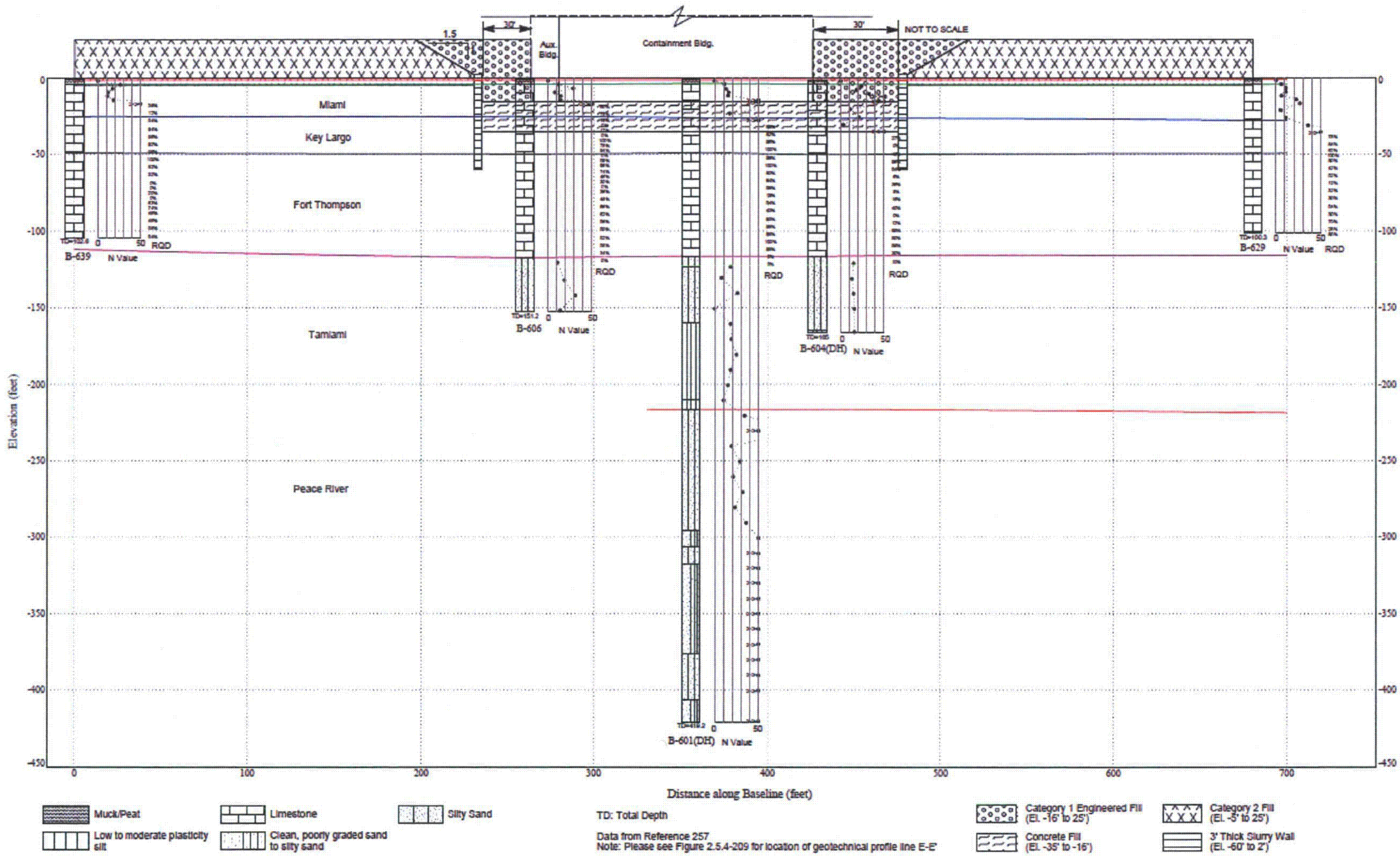


Figure 2.5.4-205 Geotechnical Cross Section F-F' Through Unit 6 Power Block

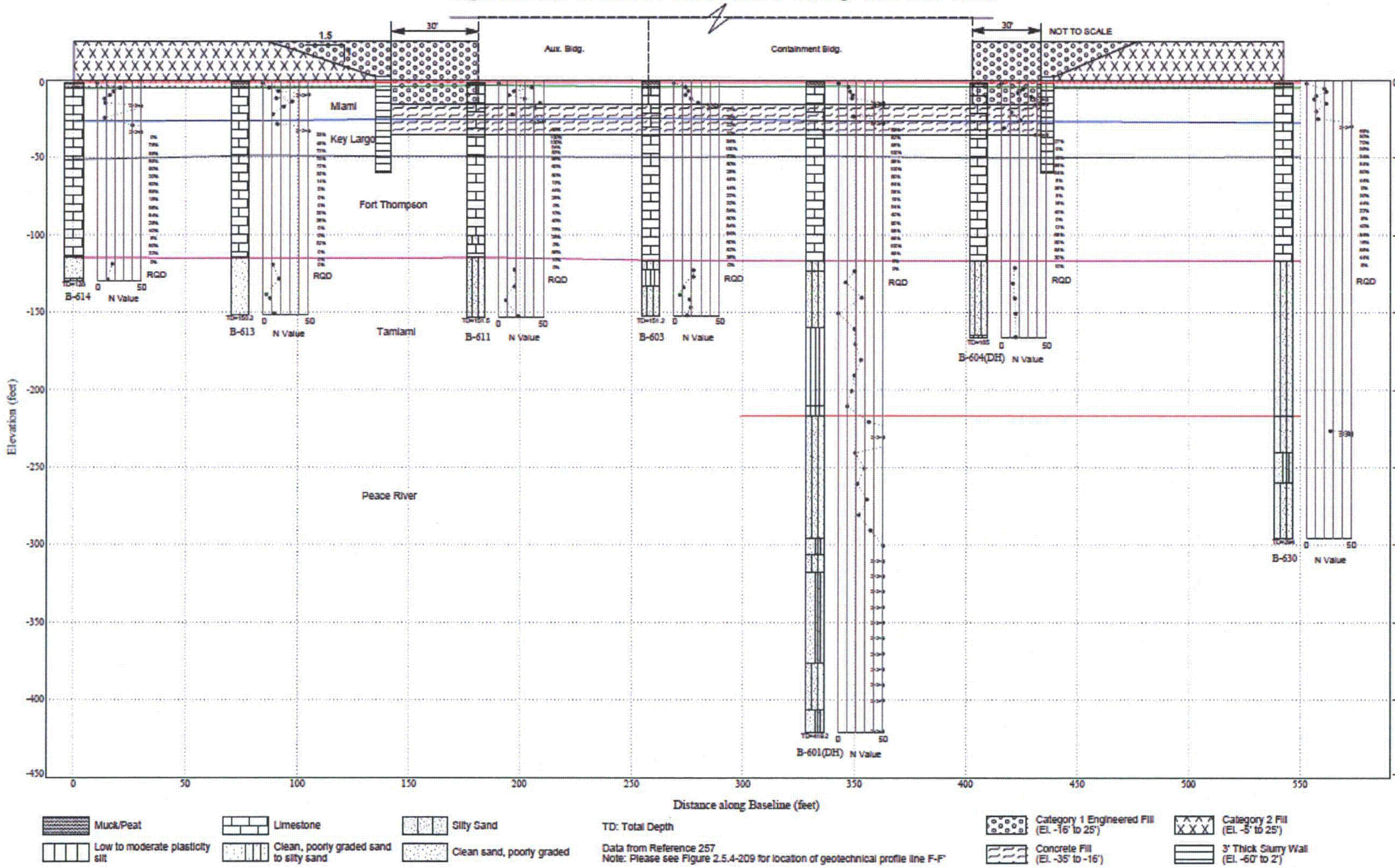


Figure 2.5.4-206 Geotechnical Cross Section A-A' Through Unit 7 Power Block

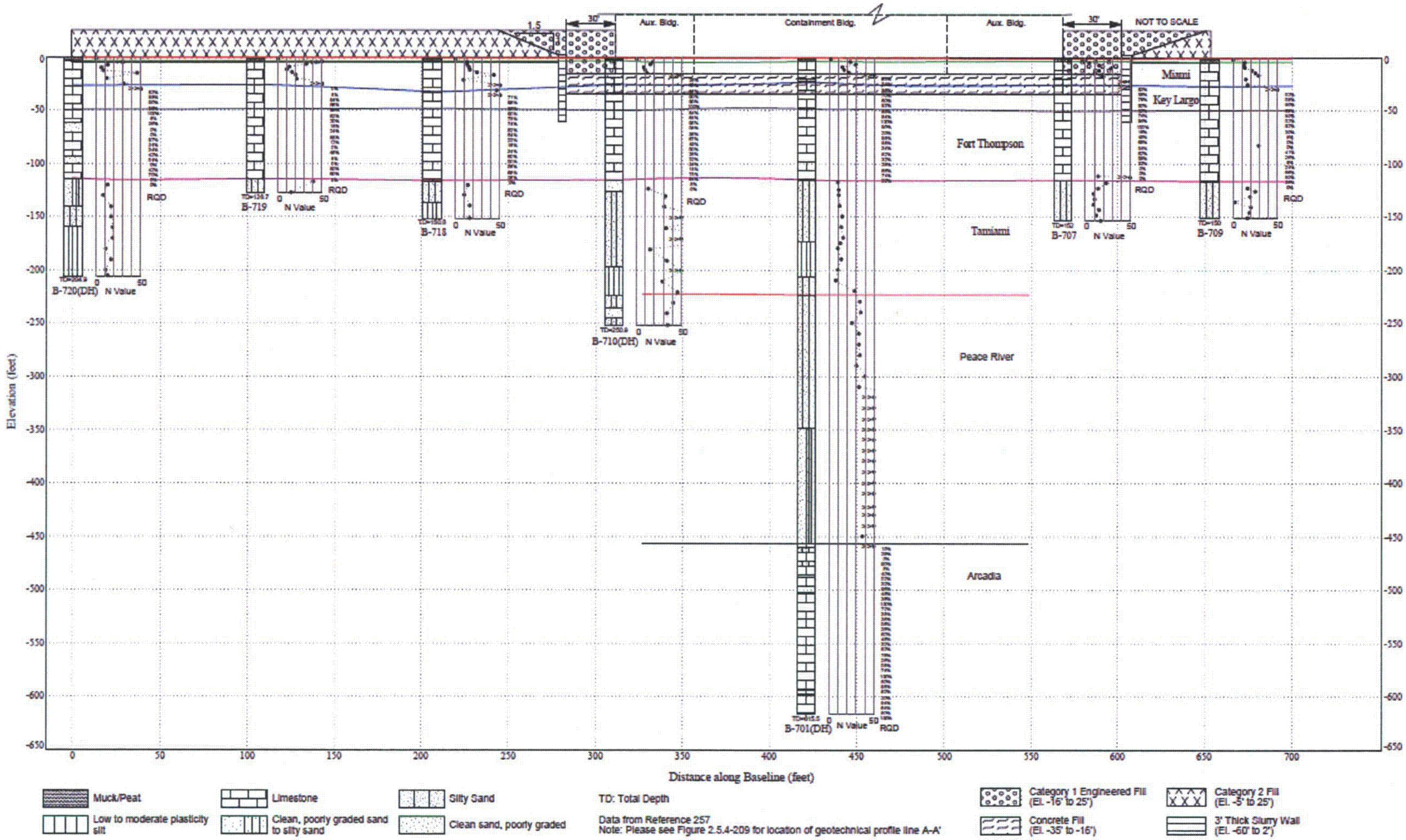


Figure 2.5.4-207 Geotechnical Cross Section B-B' Through Unit 7 Power Block

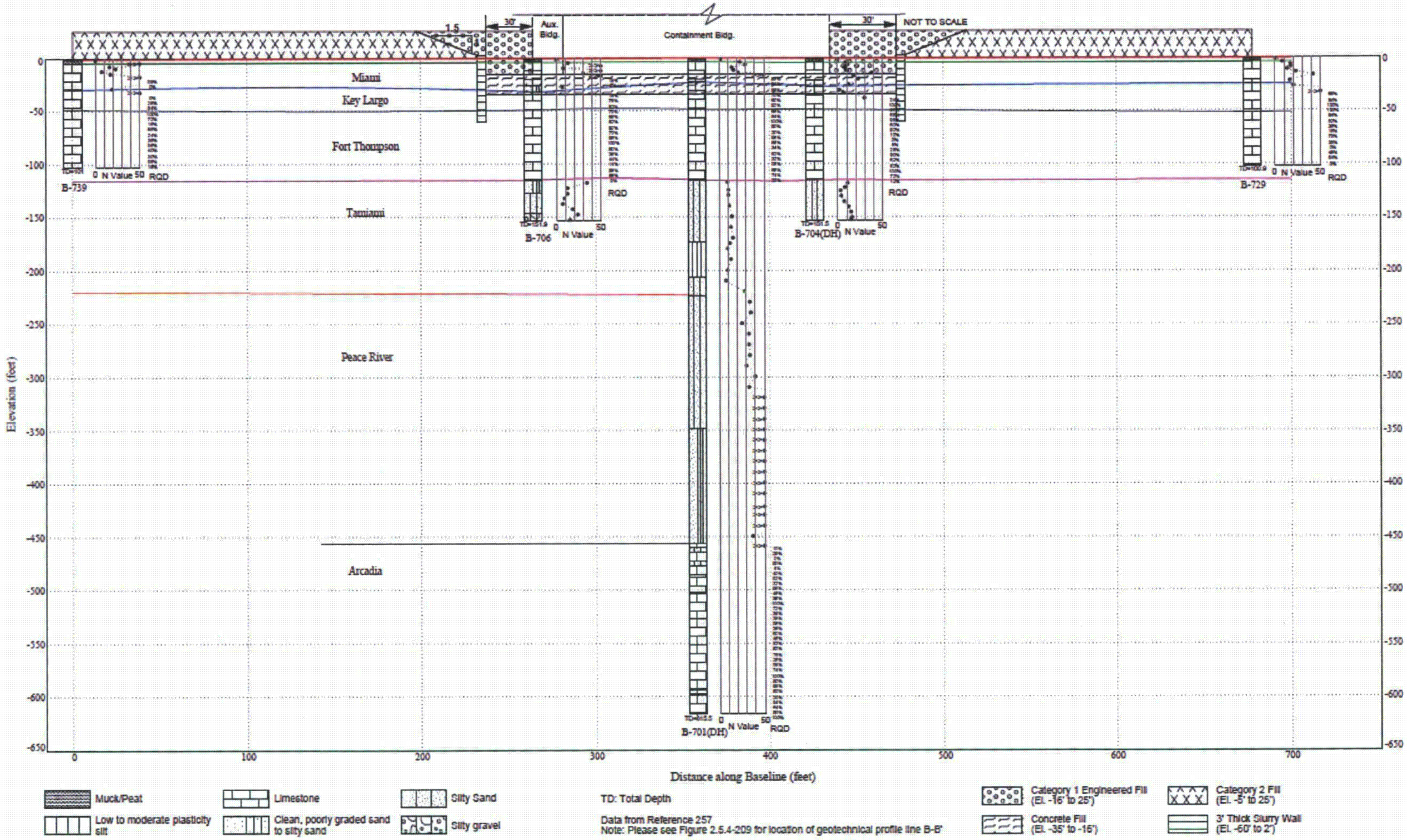


Figure 2.5.4-208 Geotechnical Cross Section C-C' Through Unit 7 Power Block

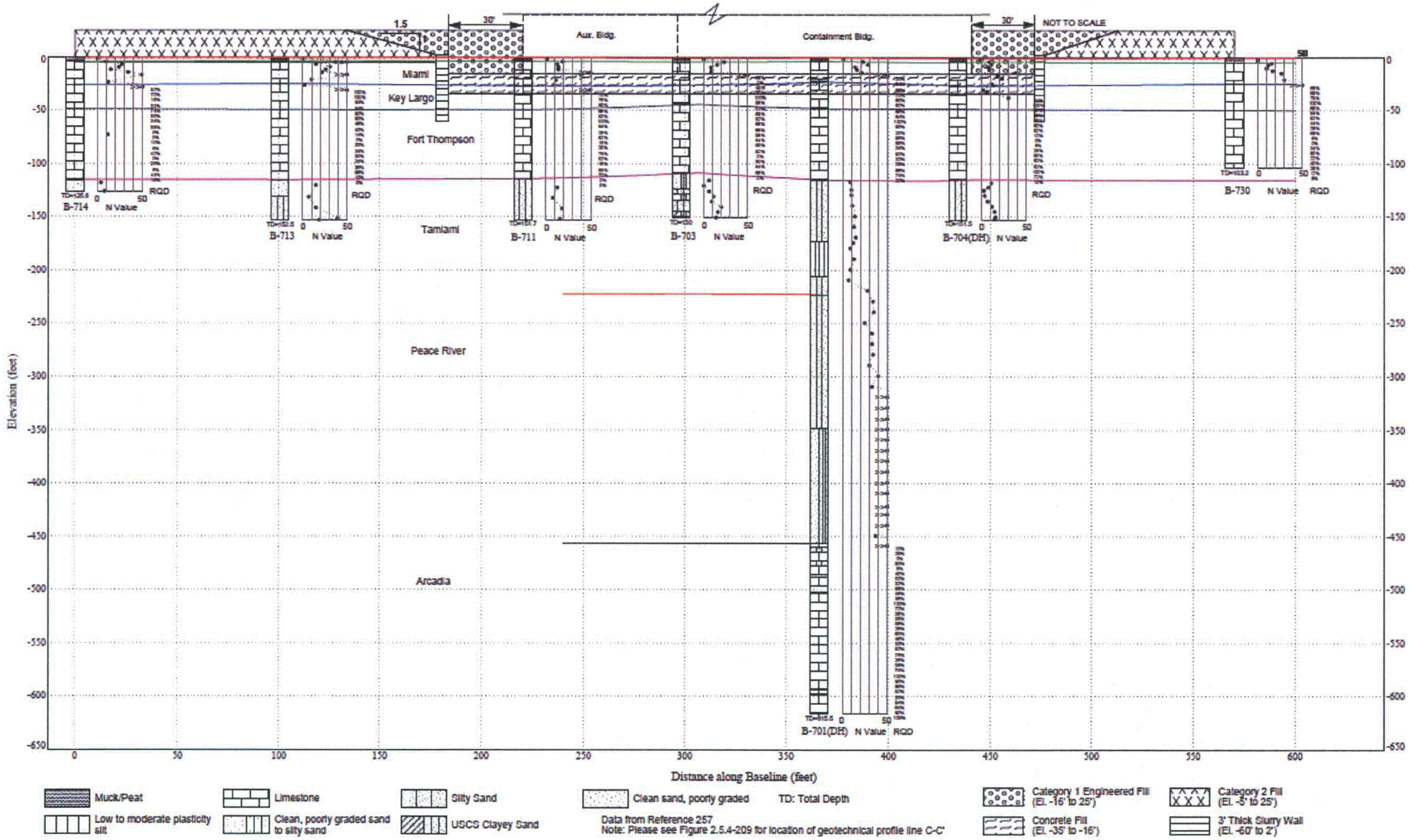
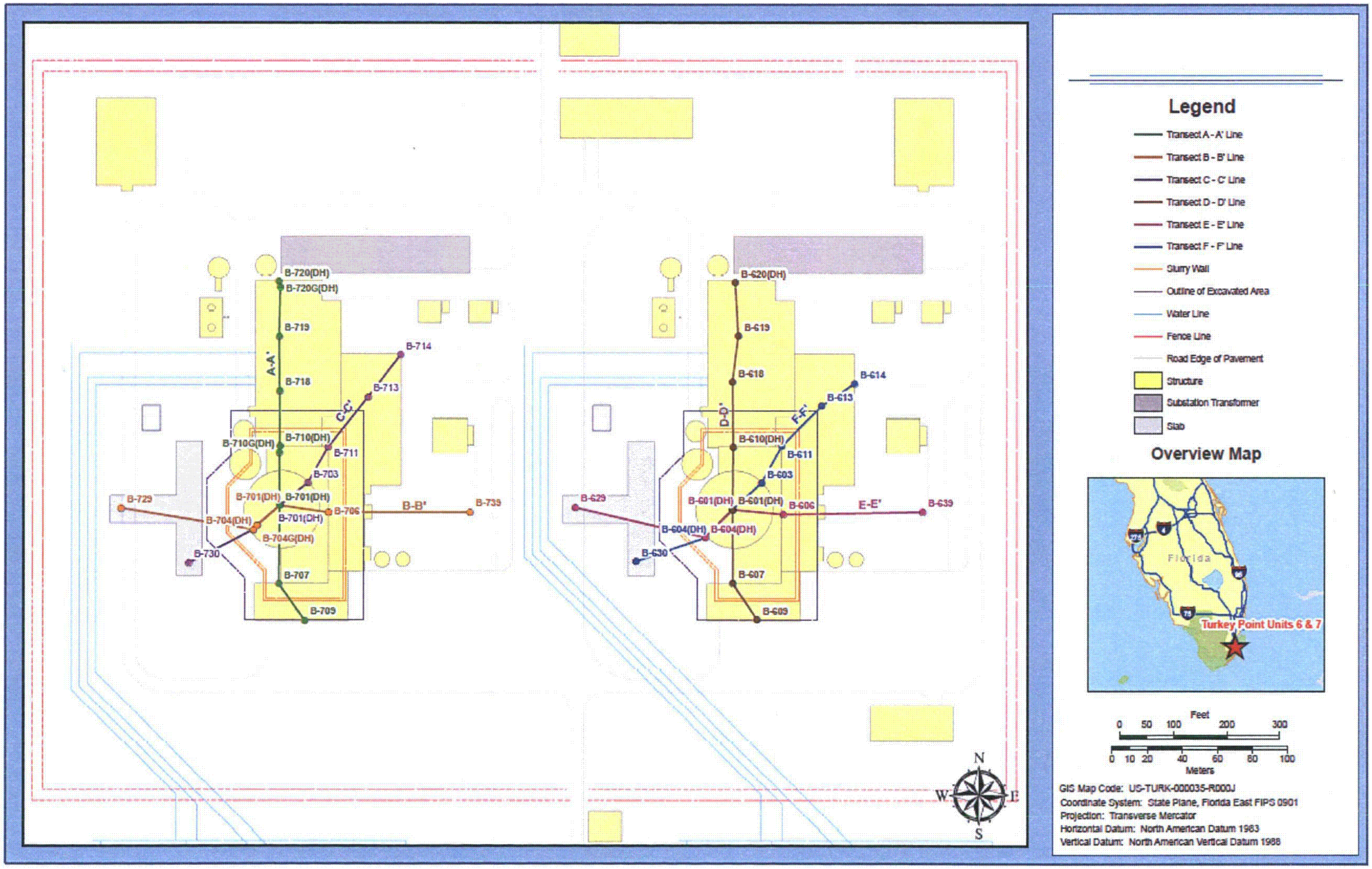


Figure 2.5.4-209 Plan Showing Geotechnical Cross Section Locations



ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-16 (eRAI 6006)

Section 2.5.4.7.3.3 “Shear modulus and Damping for Rock”, indicates 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 0.5% was used in the analyses. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please provide clarification as to the actual level of damping used in the analyses and provide a basis for its selection considering the large variability in RQD shown in Figure 2.5.4-215.

FPL RESPONSE:

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) 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 470 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. 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 a strain-dependent modulus and damping values. FSAR Table 2.5.4-216 shows the damping ratio (D%) versus shear strain values. D remains constant at 0.6% from 0.0001% to 0.03% strain. In the SHAKE analysis, shear strain did not exceed 0.03%, and so D is constant at 0.6% in FSAR Figure 2.5.2-249. D = 1% 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%. 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, is 0.32% based on the median value of kappa and associated uncertainty.

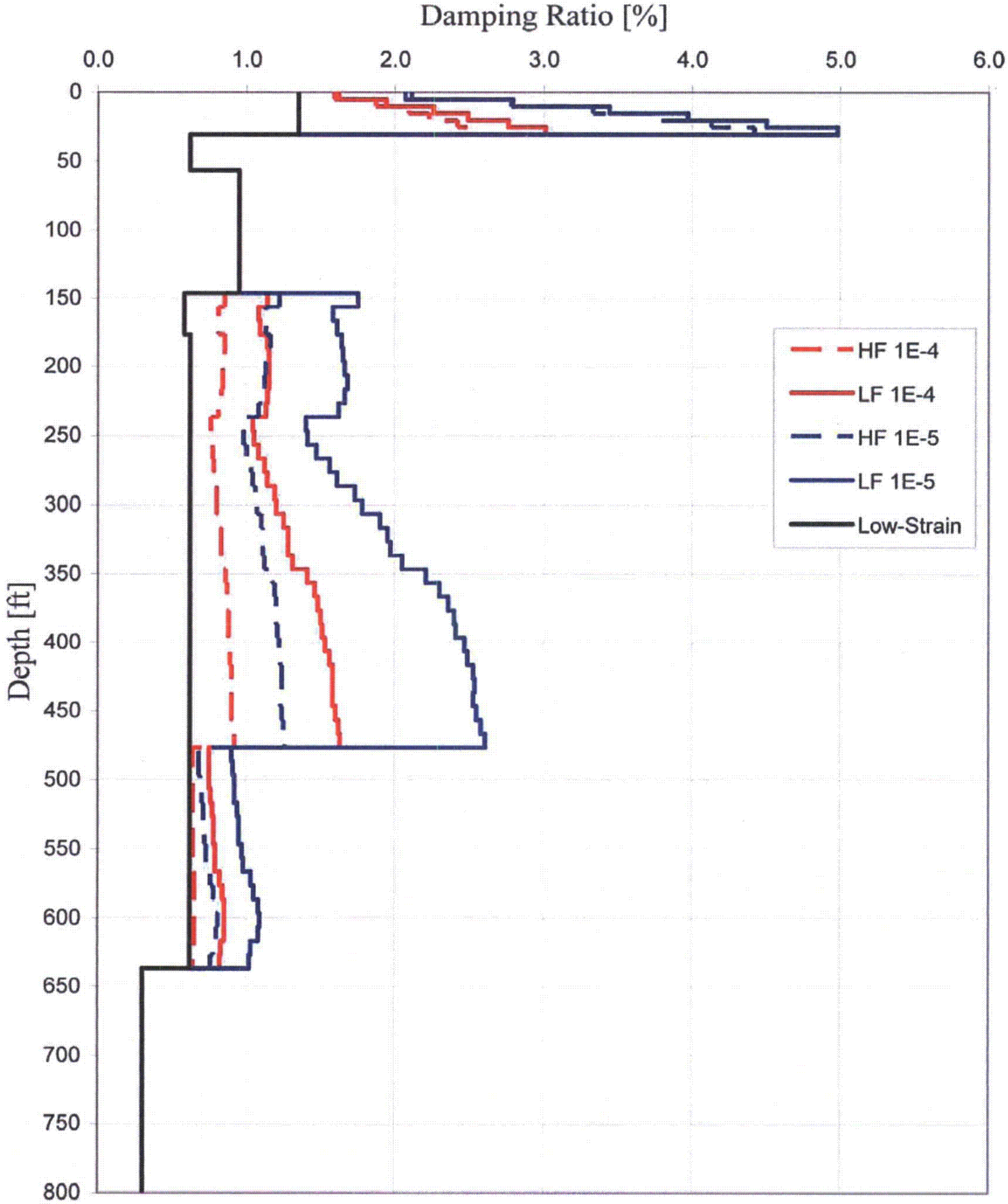


Figure 2.5.2-249 Median Profiles of Strain-Compatible Soil Damping (Upper 800 feet) (Sheet 2 of 2)

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

The following changes will be made in a future FSAR revision.

FSAR Subsection 2.5.4.7.3.1, Fifth and sixth paragraphs:

Due to the similarity of the grain size distribution and the materials, the recommended shear modulus degradation of Stratum 7 is the same as for Stratum 6, i.e., natural soil deeper than 159 ft depth in Figure 2.5.4-233. **This modulus degradation curve is also selected for Stratum 8 which consists of very weak rock and is part of the same geological formation (Hawthorn Group) as Stratum 7.**

Rock Strata 3~~, 4~~ and ~~8~~**4** are considered not subject to modulus degradation, as described in Subsection 2.5.4.7.3.3.

FSAR Subsection 2.5.4.7.3.2, Last sentence:

Figure 2.5.4-235 shows the selected values of D versus shear strain for tested Strat~~um~~**a** 5, 6, and 7, i.e., the natural soil curve used for all three strata. **This D versus shear strain curve is also selected for Stratum 8.**

FSAR Subsection 2.5.4.7.3.3, First paragraph, second sentence:

For Strata 3~~, 4~~ and ~~8~~**4**, the shear modulus is considered non-strain dependent based upon the competency of the rock.

FSAR Subsection 2.5.4.7.3.3, Third paragraph, second sentence:

For site-specific work, damping of 1 percent is adopted for Strata 3~~, 4~~ and ~~8~~**4**, and bedrock shear modulus is considered to remain constant (i.e., no degradation) in the strain range of 10^{-4} percent and 1 percent.

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-19 (eRAI 6006)

AP 1000 DCD, Revision 17, Table 2.5-1 provides the total- and differential-settlement limits. The table states that the total settlement limit for the nuclear island foundation mat is 3 inches and the differential settlement limit across the nuclear island foundation mat is 0.5 inch in 50 ft. Rev.18 revised Table 2.5-1, to state that the total settlement for the nuclear island foundation mat is limited to 6 inches; however, the differential settlement limit across the nuclear island foundation mat remained 0.5 inch in 50 ft . In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations,":

a. Please update the settlement calculations based on the DCD Rev.18 applied contact pressure for Reactor Building of 8.9 ksf instead of the 8.6 ksf stated in FSAR Rev. 2.

b. Provide additional information describing the differential settlement calculations across the nuclear island foundation mat since values appears to exceed the acceptable limits in DCD Table 2.5-1.

c. Provide a description of the monitoring program that will implemented to ensure that the actual settlements and differential settlements of the structures relative to the nuclear island do not exceed the DCD settlement criteria.

d. Provide additional explanation on why and how a dynamic shear modulus degradation curve was used to compute static unidirectional settlements.

FPL RESPONSE:

a. The settlement calculation has been updated to reflect the 8.9 ksf applied pressure for the Reactor Building. Based on this increase in applied pressure, two of the estimated settlement values for the Reactor Building (and Auxiliary Building) in FSAR Table 2.5.4-219 changed. The Case II center settlement changes from 3.3 in. to 3.4 in. and the Case II mean settlement changes from 2.6 in. to 2.7 in. The remaining estimated settlements for these buildings do not change when expressed to the nearest tenth of an inch.

b. The AP1000 unit has a highly reinforced concrete foundation basemat that is a minimum of 6 ft thick. For Units 6 & 7, this foundation sits on about 20 ft of concrete fill which is immediately underlain by about 80 ft of the Key Largo and Fort Thompson rock formations. Together these material layers form an exceedingly rigid base which will have no measurable bending under the maximum applied 8.9 ksf static pressure. The combined system can be considered as a loaded reinforced concrete basemat sitting on a 100-ft thick concrete/rock beam. The AP1000 setting described above is the basis of the projected value of <0.1 inch in 50 ft that is given for differential settlement across the nuclear island (NI) foundation mat in FSAR Table 2.0-201.

The calculation used to estimate the settlement values beneath the NI reported in FSAR Table 2.5.4-219 makes several simplifying and conservative assumptions, namely (a) the structure is sitting on a perfectly flexible base, (b) the concrete and rock allow settlement of the underlying soil to occur, and (c) the applied stresses from the NI are distributed through the concrete and rock in a Boussinesq distribution. These assumptions are discussed below.

(a) In the situation where the foundation system is almost perfectly rigid instead of perfectly flexible, the mean of the center and edge settlement provides a reasonable estimate of the settlement that can be expected beneath the whole foundation (i.e., both the center and edge of the foundation). Thus the mean values given in FSAR Table 2.5.4-219 for the NI (1.4 in. for Case I and 2.7 in. for Case II) reflect the anticipated settlements beneath the whole foundation.
(b) The settlement analysis shows that less than 0.04 in. of settlement is due to compression of the 100 ft of concrete fill and underlying rock due to the 8.9 ksf loading. The remainder of the settlement noted in (a) is due to the compression of the soil beneath the rock. This soil can only settle if the rock settles with it. This can only happen if the rock mass bends or shears, neither of which will occur.
(c) This assumption is linked to the discussion in (b). Where there is a difference in the elastic modulus (E) between the upper (E_1) and lower (E_2) layers, the stress distribution through the upper layer will vary from a pure Boussinesq distribution, as shown in Figure 4.30 of Winterkorn & Fang (1975) (FSAR Reference 221). The greater the elastic modulus contrast (E_1/E_2), the greater the variation. The average elastic modulus values of the concrete and rock are in the range of 100 times greater than those of the underlying soil. With $E_1/E_2 = 100$, the distributed stress at the base of the rock (or top of the soil) beneath the NI is 3.7 to 8.0 times less than the stress obtained using the Boussinesq distribution, interpreted from Table 6.1 of Poulos & Davis (1974) (FSAR Reference 275). Since settlement in these overconsolidated soils is proportional to the applied stress, the estimated settlement in the soils beneath the rock will conservatively only be about 25% of the settlement computed using the Boussinesq distribution. This is the same principle that makes a crane mat work over a soft soil – the stresses keep getting distributed out across the mat until equilibrium is reached.

To recap, because the NI has a thick highly-reinforced foundation sitting on about 100 ft of concrete fill and rock, there will be negligible differential settlement (<0.1 inch in 50 ft) across the foundation. The mean settlements shown for the NI in FSAR Table 2.5.4-219 reflect the settlements under the rigid foundation, and are almost entirely due to the settlement of the soil below the rock. This settlement is overestimated by about a factor of 4 because it is computed using a Boussinesq distribution. Removing this overestimation would effectively reduce the mean settlement values in FSAR Table 2.5.4-219 to less than 1 inch. Even then, this settlement requires bending or shearing of the overlying rock to occur, and this will not happen.

For a sensitivity check, the level of differential settlement that would be estimated per 50 ft of the NI foundation was examined to determine the effects if (a) the rigidity of the foundation and underlying concrete and rock were neglected (i.e., the system was assumed to be flexible), (b) the factor of 4 reduction in settlement due to E1/E2 = 100 was also neglected, and (c) it was assumed that the rock would bend or shear to allow the soil below to settle. The values for the NI in FSAR Table 2.5.4-219 reflect these assumptions. For the Case II foundation, the distance from the foundation center to the longer edge is 159/2 = 79.5 ft. The difference in settlement is 3.4 -1.9 inches = 1.5 inches. This is equivalent to 0.9 inches over 50 ft, which is somewhat greater than the 0.5 in. limit stipulated in DCD Table 2.5-1. For Case I, the differential settlement is less than 0.5 inches per 50 ft.

As described above, differential settlement across the NI foundation will be negligible. For sites where differential settlement can occur, differential settlement that occurs during construction is generally of less concern than differential settlement that occurs after construction. This is because adjustments are made for any such settlement as construction progresses. Most major equipment is placed after a significant portion of the final loading is already in place. These adjustments are recognized in Section 2.5.4.3 of Revision 19 of the DCD, which states, "Much of this [differential] settlement occurs during civil construction prior to final installation of the equipment. Differential settlement of a few inches across the width of the nuclear island would not have an adverse effect on the safety-related functions of structures, systems and components." Because of the overconsolidated nature of the deep soils that comprise the Tamiami and Peace River Formations, no measurable settlement will occur in these deposits after the end of construction.

In summary, there will be negligible differential settlement (<0.1 inch in 50 ft) across the foundation mat of the NI because of the rigidity of the foundation mat and the underlying concrete fill and rock formations as identified in FSAR Table 2.0-201. Mean settlements are estimated to be less than 1.0 in. and will occur during construction.

c. A settlement monitoring program is given as one of the alternatives for the additional evaluation of settlement in DCD Section 2.5.4.3. Based on the estimated settlements of the NI, no additional evaluation is anticipated. If additional evaluation is deemed necessary, and if the settlement monitoring alternative is selected, then the program will follow the guidelines provided in the DCD regarding settlement monuments, i.e., "Settlement monuments placed directly on concrete, preferably on the mudmat for early construction monitoring and on the corners of structures at grade once the mudmat monuments have been covered by backfill to be used for long-term monitoring. Monuments at grade are to be accessible with conventional surveying equipment." The DCD also notes that there should be piezometers to measure pore pressures in a soil layer prone to consolidation type settlement. Since the soils at the Turkey Point site are not prone to consolidation type settlement, piezometers will not be used.

d. The shear modulus degradation curve referred to as "dynamic" is more accurately termed "strain-dependent". Shear modulus values at very small strains (typically 10^{-4}

percent) are values associated with strains caused by wave motion through the soil. Somewhat larger strains (although still considered very small) are associated with vibrations due to equipment operating at high frequencies. Both of these (wave motion and high frequency equipment) are considered "dynamic".

FSAR Section 2.5.4.10.3 explains how the shear modulus ratio G/G_{max} plotted against shear strain (dynamic shear modulus degradation curve) is the same curve as the elastic modulus ratio E/E_{max} plotted against axial strain.

At the higher strain end of the elastic modulus degradation curve are modulus values associated with typical static loading and strain. For soils directly beneath a large heavily-loaded foundation, axial strain can be as high as 1 percent. However, for very stiff and very dense soils, particularly at greater depths where the applied loading has been distributed and significantly reduced, strains will be much less (as low as 2.5×10^{-2} percent in the Peace River Formation settlement calculation).

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

The seventh paragraph in FSAR Section 2.5.4.10.3 will be revised in a future FSAR revision as follows:

The settlements under the nuclear island foundation with plan dimensions of 88 feet by 254 feet and 159 feet by 254 feet are calculated with an applied pressure of ~~8.6~~ **8.9** ksf. The estimated total settlements at the center and at midpoints of the sides are largely impacted by the large foundation size and loading, and by the elastic modulus values of the soil strata. The preconsolidated soils of the Tamiami and Peace River Formations are confined below an 80-foot thick stratum of rock, and thus a relatively low settlement estimate is expected from these dense granular and stiff fine-grained layers. Settlements at the center of the mat foundations are evaluated using the strain compatible elastic moduli of the Tamiami and Peace River Formations with corresponding axial strains. The strain compatible evaluation is performed only for the soil strata, i.e., the Tamiami and Peace River Formations where there is a difference between the high and low strain moduli. In order to apply the elastic moduli in Equation 2.5.4-21, their values need to be equated with strain level. In this case, the modulus degradation with increasing strain is based on the recommended curves in Figure 2.5.4-233 after converting shear strain to axial strain.

The ninth paragraph in FSAR Section 2.5.4.10.3 will be revised in a future FSAR revision as follows:

A trial process is followed for soil Strata 5, 6, and 7 using the degradation curves to arrive at a compatible axial strain, such that the strain for the adopted modulus and the calculated strain converge. For rock and concrete strata, the settlements computed are based on a constant (not strain dependent) elastic modulus. The results of the settlement analysis on Table 2.5.4-219 show the computed settlements at the center and edge of the nuclear island foundations with dimensions of 88 feet by 254 feet and 159 feet by 254 feet, under loading of ~~8.6~~ **8.9** ksf. (Two sets of plan dimensions are used because of the irregular shape of the foundation.) Similar settlement calculations are made for the turbine, annex and radwaste buildings, and the results are presented in Table 2.5.4-219. As with the nuclear island, settlements of the annex building are analyzed for two sets of foundation dimensions.

FSAR Table 2.5.4-219 will be revised in a future FSAR revision as follows.

Table 2.5.4-219 Estimated Foundation Settlements

					Low Strain Anticipated Settlement (in.)					
	Contact Pressure (ksf)		BxL (ftxft)		Center		Mid of Side		Mean	
Structure		**Subsurface**	**Case I**	**Case II**	**I**	**II**	**I**	**II**	**I**	**II**
Reactor & Auxiliary	~~8.6~~ **8.9**	Lean Concrete Fill on Rock	88x254	159x254	1.6	~~3.3~~ **3.4**	1.2	1.9	1.4	~~2.6~~ **2.7**
Turbine	6.0	Compacted Fill	156x309	—	3.6	—	2.2	—	2.9	—
Annex	6.0	Compacted Fill	66x405	145x405	3.0	3.9	1.8	2.4	2.4	3.15
Radwaste	6.0	Compacted Fill	66x175	—	2.8	—	1.6	—	2.2	—

~~Data from Reference 257.~~

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-040

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

QUESTIONS from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-22 (eRAI 6006)

The lateral earth pressure diagram shown in Figure 2.5.4-240 shows a plot corresponding to the dynamic lateral earth pressure. The shape of this plot appears to be consistent with the shape for dynamic pressure considering a rigid structural wall (see ASCE 4). In Section 2.5.4.10.4.2 "Seismic Lateral Earth Pressures", the active seismic pressure was computed using the Mononobe-Okabe equation. The last sentence of the section indicates that at-rest pressure as a function of depth for below-grade walls is developed consistent with Reference 277 (ASCE-4) using the design ground motion. It is noted that the pressure developed using the ASCE-4 methodology uses the zpa value from the input motion.

Figure 2.5.2-252 shows the input motion (GMRS) developed for the site, the GMRS is located at Elevation 35 *[sic].* In this Figure the zpa is approximately 0.058g. However, the elevation of the GMRS is considerably lower than the surface of the soils adjacent to the basement walls that are to be evaluated for seismic lateral earth pressure. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please clarify on the definition of the design ground motion, and how that motion is consistent with Appendix S to 10CFR50.

FPL RESPONSE:

The Design Response Spectra (DRS) at 5% damping are calculated at the ground surface for the near Nuclear Island (NI) and far from NI soil sites using the envelope of low frequency (LF) and high frequency (HF) acceleration response spectra (ARS) at 10^{-4} and 10^{-5} annual probability of exceedance. These ARS envelopes and the DRS are plotted in Figures 1 and 2 for the near NI and far from NI soil sites, respectively. From Figures 1 and 2, the peak ground acceleration at the ground surface is equal to approximately 0.0824g and 0.0806g (DRS at 100 Hz) for the near NI and far from NI soil sites, respectively.

Regarding the computation of active seismic pressure using the Mononobe-Okabe equation, according to Whitman (1991), use of horizontal ground acceleration for design at the base level of the wall may result in underestimating the movements; it seems best to use the acceleration at the surface of the backfill, or an average between the surface and the base of the wall. Thus, an acceleration of 0.1g, rather than the peak ground acceleration of 0.0824g (near NI) or 0.0806g (far from NI), is conservatively used in the Mononobe-Okabe equation.

Similarly, for the computation of at-rest seismic pressure using the ASCE 4-98 method, an acceleration of 0.1g, rather than the peak ground acceleration of 0.0824g (near NI) or 0.0806g (far from NI), is conservatively used.

Figure 1. 5% Damping ARS at Ground Surface – Near NI, Envelope of LF and HF

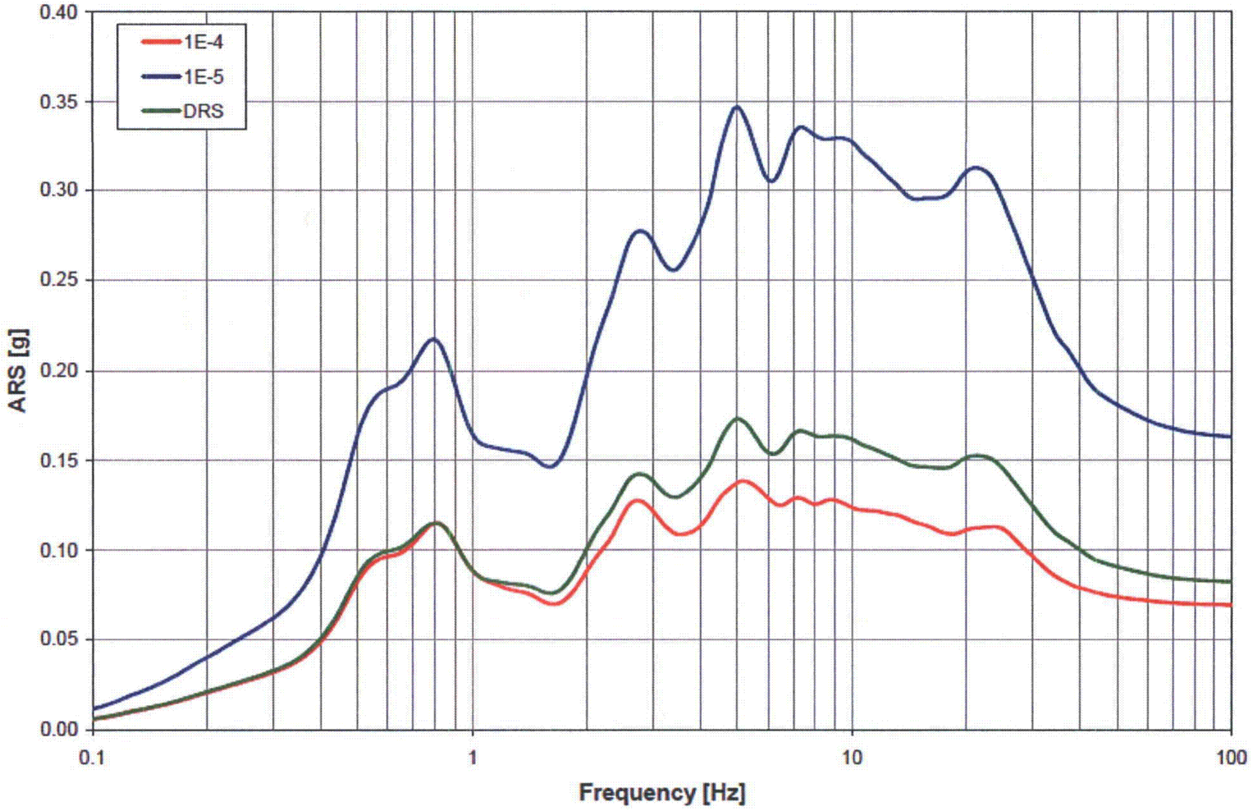
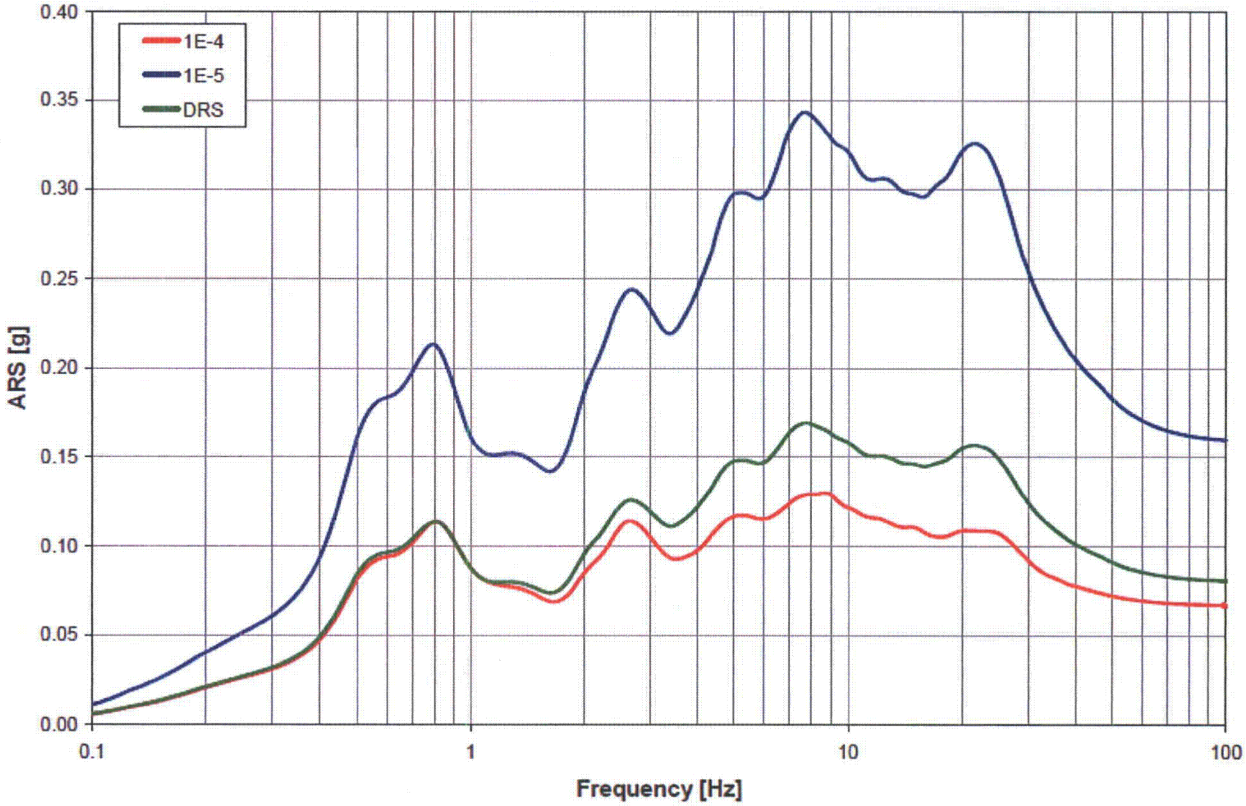


Figure 2. 5% Damping ARS at Ground Surface – Far from NI, Envelope of LF and HF



This response is PLANT SPECIFIC.

References:

Whitman, R.V. “Seismic Design of Earth Retaining Structures,” *Proc. 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, St. Louis, MO, pp. 1767-1778, 1991.

ASSOCIATED COLA REVISIONS:

FSAR Subsection 2.5.4.10.4 will be revised as follows in a future FSAR revision.

2.5.4.10.4 Earth Pressures

The static and seismic active and at-rest lateral earth pressures acting on underground structure below-grade walls are addressed in this subsection. The analysis of seismic earth pressure is addressed generically. Note that active earth pressures apply to yielding walls such as steel sheet pile walls, MSE walls, and, to a lesser extent, more rigid concrete slurry (diaphragm) walls, which are used primarily as temporary ground support in construction. At-rest earth pressures occur in the case of non-yielding walls, such as the rigid, below-grade walls of underground structures (e.g., for the containment/auxiliary buildings, control buildings, etc.).

Increases in lateral earth pressures resulting from compaction close-in to below grade structures are not considered here. These increases are controlled at the construction stage by limiting the size of compaction equipment and its proximity to below-grade walls. Note that the magnitude of compaction-induced earth pressure increases can only be assessed once a range of allowable equipment sizes and types are selected/specified.

For the seismic active and **at-rest** earth pressure cases, earthquake-induced horizontal ground accelerations are accounted for by employing the factor $k_h g$. Here, $k_h = 0.1$ is used. Vertical ground accelerations (k_v g) are considered negligible (Reference 276).

ASSOCIATED ENCLOSURES:

None