PMTurkeyCOLPEm Resource

From: Comar, Manny
Sent: Thursday, November 01, 2012 5:05 PM
To: TurkeyCOL Resource
Subject: FW: DRAFT RAI Responses FPL Turkey Point 6 & 7 for eRAIs 6006 Stability of Subsurface Materials and Foundations
Attachments: Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-6 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-12 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-14 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-8 (eRAI 6006).pdf; Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-19 (eRAI 6006).pdf

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From: Franzone, Steve [mailto:Steve.Franzone@fpl.com]
Sent: Sunday, October 07, 2012 5:25 PM
To: Comar, Manny
Cc: Burski, Raymond; Maher, William; Franzone, Steve
Subject: DRAFT RAI Responses FPL Turkey Point 6 & 7 for eRAIs 6006 Stability of Subsurface Materials and Foundations

Manny,

To support a future public meeting, FPL is providing draft revised responses for eRAIs 6006 (RAI questions 02.05.04-6, 02.05.04-8, 02.05.04-12, 02.05.04-14, 02.05.04-19) in the attached files.

If you have any questions, please contact me.
Thanks
Steve Franzone
NNP Licensing Manager - COLA
"When you blame others, you give up your power to change." Dr. Robert Anthony
561.694.3209 (office)
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Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-12 (eRAI 6006).pdf	268815	
Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-14 (eRAI 6006).pdf	702299	
Draft Revised Response for NRC RAI Letter No. 040, RAI 02.05.04-8 (eRAI 6006).pdf	540125	
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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: 02.05.04-6 (eRAI 6006)

FSAR Section 2.5.4.2.3 "Laboratory Testing" states "due to the fragility of the rock and the porosity of the limestone, attaching strain gages for determination of stress-strain characteristics is not possible for most samples". In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," and Regulatory Guide (RG) 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," please justify why only two rock core samples were tested for stress-strain characteristics and why this is sufficient to characterize the Fort Thompson and Key Largo rock formations, especially since the Key Largo will be the bearing layer. Also, please explain how you validated the assumption in FSAR 2.5.4.2.1.3.11 that for rocks the elastic and shear modulus values generally remain constant at both small and large strains.

FPL RESPONSE:

Introduction

Numerous symbols are used in this response. They are tabulated at the end of the response.

FSAR Subsection 2.5.1.2.2 describes the Pleistocene Age Key Largo Limestone as a coralline limestone characterized by the presence of vuggy porosity with a high degree of interconnectivity and the Fort Thompson Formation as a sandy limestone with zones of uncemented sand interbeds, some vugs, and zones of moldic porosity. FSAR Subsection 2.5.4.2.1.2.3 characterizes the hardness and strength of the Key Largo Limestone as medium hard. FSAR Subsection 2.5.4.2.1.2.4 characterizes the Fort Thompson Formation as medium hard to hard above approximately El –60 feet (NAVD 88) and as medium hard to soft below approximately El –60 feet.

As indicated in FSAR Table 2.5.4-207, 31 samples of the Key Largo Limestone and 46 samples of the Fort Thompson Formation were tested for unconfined compressive strength. As reported in FSAR Subsection 2.5.4 Reference 257, due to the fragility and porosity of the limestone, only two of these samples were found to be acceptable for strain gage attachment for determination of the elastic modulus (E). The results from these two tests support the contention that the elastic properties of these limestone formations are not strain dependent, as discussed in this response.

Soil stiffness is strain dependent. Resonant column and torsional shear (RCTS) tests on the Tamiami and Peace River Formations established well-behaved relationships between shear modulus (G) and shear strain. This strain dependency occurs because soil is made up of discrete particles that start to move apart or slide against each other at higher strains, reducing the soil stiffness. Rock is cemented, and so the particles do not separate under typical foundation loading. There is a point at which a rock becomes so soft/weak that there is some movement of particles at higher strains. For instance, the Miami Limestone has an estimated unconfined compressive strength of 200 psi compared with the 1500 psi and

2000 psi of the Key Largo and Fort Thompson rock formations, respectively. It was assumed that the Miami Limestone at Turkey Point is strain dependent. The relationship of G to shear strain for the Miami Limestone is shown in FSAR Figure 2.5.4-233. The relationship was taken from the literature (FSAR Subsection 2.5.4 Reference 259) from tests on a soft mudstone.

In addition to the significant difference in strength, the structure of the Miami Limestone is different than that of the Key Largo and Fort Thompson rock formations. The Miami Limestone is an oolitic limestone, made up of small spheres of calcium carbonate weakly cemented together. The Key Largo and Fort Thompson Limestones are made up of a hard, brittle calcium carbonate material. The Key Largo Limestone contains numerous fossils and shells forming an open porous structure. The Fort Thompson Formation is a characteristically vuggy material, i.e., containing numerous small holes. The basic structure of the Key Largo and Fort Thompson Limestones is strong. Failure in unconfined compression tests on small diameter samples (3 inches or less) is typically sudden when the applied stresses collapse the voids (created by fossil shape or due to the vuggy structure). There is no mechanism for strain dependency.

It may be noted that, in cases where these materials contain vugs or small fossils such as shells, their behavior from stress applied in the micro environment may be different from that in the macro environment. A compression test on a small laboratory sample of the rock will be influenced by the voids in the sample that will eventually precipitate failure. These same small voids will not have the same effect under a large rigid foundation where the rigidity and confinement will not allow local collapse of such voids under the level of stress applied. Thus, it is expected that the behavior of these limestones beneath rigid foundations will be more robust than in laboratory testing.

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Strain Dependency

The RAI notes that only two tests were performed on the Key Largo and Fort Thompson Formations to show their lack of strain dependency. Strain dependency is a function of rock strength. As noted earlier, rock is cemented, and so the particles do not separate under typical foundation loading. There is a point at which a rock becomes so soft/weak that there is some movement of particles at higher strains, i.e., there is an element of strain dependency at higher strains. If the assumption is made that the moduli values of the Key Largo and Fort Thompson Formations are strain dependent, they would be less strain dependent than the much softer Miami Limestone. In other words, their degradation curve would be to the right of the Miami Limestone curve in Figure 2.5.4-233. In the next paragraph, the settlement of the Key Largo and Fort Thompson Formations is examined based on the assumption that the rock is strain dependent. For this it is conservatively assumed that the degradation curve for the Key Largo and Fort Thompson Formations is the same as for the Miami Limestone. Note that the degradation curve in the FSAR is the degradation of G in terms of shear strain. When considering settlement, E is used rather than G and axial strain replaces shear strain. In this situation, the maximum principal strain is the vertical strain (i.e., $\varepsilon_1 = \varepsilon_v$), while the minimum principal strain is assumed to be zero (i.e., $\varepsilon_3 = 0$). Because the maximum shear strain $\gamma_{max} = \varepsilon_1 - \varepsilon_3 = \varepsilon_1$ (FSAR Subsection 2.5.4 Reference 275) and $E/E_{max} = G/G_{max}$, the elastic modulus reduction curves with respect to vertical strain should be the same as the shear modulus reduction curves with respect to the shear strain.

The settlement analysis of the NI (8.9 ksf bearing pressure) showed that, using the low strain rock modulus values, the settlement of the combined Key Largo and Fort Thompson limestone formations was about 0.03 inches, much smaller than can typically be measured with conventional equipment. For soils immediately under large foundations, elastic axial strain is typically 0.25 to 0.5 percent. If it is assumed that the axial strain for the Miami Limestone degradation curve is 0.375 percent, the G/G_{MAX} (= E/E_{MAX}) value would be about 0.6. The Miami Limestone degradation curve at 0.375 percent, gives a G/G_{MAX} (= E/E_{MAX}) value of about 0.6. This means that at this level of strain, E is about 0.6 of Emax (the low strain value). Thus the settlement in the rock would increase by 1/0.6 = 1.67, i.e., it would be about 0.05 inches, still too small to be measured.

The thickness of the combined Key Largo and Fort Thompson strata below the NI (actually below the concrete fill) is about 80 feet or 960 inches. Thus, the axial strain due to 0.05 inch settlement is 100 percent x 0.05/960 = 0.005 percent. Looking at the Miami Limestone modulus reduction curve, there is no reduction at 0.005 percent strain. Thus, in reality, even if it is assumed that the E is strain dependent using a conservative curve for a much softer rock, there is no strain dependency at the strain levels produced by the NI loading on the rock. The only place in the FSAR where E is used is in the Settlement section (FSAR Subsection 2.5.4.10.3).

G of the Key Largo and Fort Thompson Formations is used in the SHAKE analysis of the soil and rock column. FSAR Figure 2.5.2-248 shows strain levels in these strata to be well below 0.005 percent. The Miami Limestone G degradation curve in FSAR Figure 2.5.4-233 shows that there is no degradation of G below 0.01 percent strain. Thus, even if the shear moduli of these formations were strain dependent, the strains generated would be too small to cause any modulus reduction.

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Rock Mass Quality

Rock mass quality has been comprehensively described in the supplemental response to RAI 02.05.04-25. The general concept of rock mass quality is that the rock mass behaves more poorly than samples of the rock tested in the laboratory because the mass is fractured, jointed, and weathered while the sample being tested is not. As described in the supplemental response to RAI 02.05.04-25, the Key Largo and Fort Thompson Limestone Formations are typically not fractured or jointed. Normal weathering patterns, where there is a progression from weak weathered rock to stronger, less weathered rock with depth, were not observed. As stated earlier in this response, the strength of the laboratory samples of these rocks can be affected by vugs whereas vugs will not impact the behavior of the rock mass under a large rigid foundation. Rock mass quality for these formations was not addressed because it was not considered relevant.

Results of Laboratory Tests to Evaluate Elastic and Shear Moduli

These results were reported in Appendix E.2 (Laboratory Test Results on Rock Cores) in Volume 3 of FSAR Reference 257. The samples were both obtained from around 50-foot depth at the boundary between the Key Largo and Fort Thompson Limestone Formations. The results were E = 3700 ksi and 2900 ksi for the two samples where E from the laboratory results represents a tangent modulus value corresponding to 40 to 60 percent of ultimate strength. The stress strain curves for both tests showed reasonable linearity all the way to failure at strains of approximately 6.5 and 8.3 x 10^{-2} percent, respectively. The corresponding unconfined compressive strengths (U) for the two samples were 2038 psi and 2487 psi, respectively. The strengths of these samples were above the best estimate strengths for the Key Largo and Fort Thompson Limestone Formations (1500 and 2000 psi, respectively).

Given the inability to obtain acceptable samples for testing in the laboratory, alternative evaluations were made based on the in situ shear wave velocity (V_s) measurements. FSAR Subsection 2.5.4.2.1.3.11 indicates that sound rock and even moderately weathered rock typically exhibits an elastic response to loading and that E and G remain relatively constant at both small and large strains. An estimate of low-strain G can be made using a relationship with V_s, provided in FSAR Equation 2.5.4-7:

$G_L = \gamma/g \cdot (V_s)^2$ (Equation 1)

where G_L= low strain G, typically at 10^{-4} percent strain

γ = unit weight

g = acceleration due to gravity

The relationship between the low strain shear and elastic moduli can be made using FSAR Equation 2.5.4-6.

$E_L = 2 \cdot G_L\ (1+\mu)$ for a low strain value, and μ is Poisson's ratio (Equation 2)

Based on the above for the first laboratory test sample, the following results were obtained:

- Laboratory test on sample CS-04 (depth 49.9 to 50.7 feet) from boring B-610 gave E = 3700 ksi. From Volume 2 of FSAR Reference 257, measured V_s in B-610 at 49.2 feet depth is 6540 fps and at 50.9 feet depth is 5050 fps, with an average of 5795 fps.
- Geotechnical Coring Log in Volume 1 of FSAR Reference 257 indicates a rock quality designation (RQD) value of 96 percent for the depth interval 46 to 51 feet in boring B-610 from which sample CS-04 was taken.
- Substituting in Equation 1, $G_L = (\gamma/g)V_s^{\ 2} = (0.139/32.2) \times 5795^2/144 = 1007$ ksi.
- From Equation 2, $E_L = 2 \times (1+0.28) \times 1007 = 2578$ ksi (where $\mu = 0.28$ from measured V_s and P-wave velocity values at the sample depth).
- 2578 ksi < 3700 ksi. If the V_s of 6540 fps at 49.2 feet depth is used, E_L = 3,212 ksi which is closer to the 3700 ksi measured in the laboratory test.

Similarly, for the second laboratory test sample, the following results were obtained:

- Laboratory test on sample CS-03 (depth 51.2 to 52 feet) from boring B-620 gave E = 2900 ksi. From Volume 2 of FSAR Reference 257, measured V_s in B-620 at 50.9 feet depth is 6940 fps and at 52.5 feet depth is 5460 fps, with an average of 6200 fps.

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- Geotechnical Coring Log in Volume 1 of FSAR Reference 257 indicates an RQD value of 72 percent for the depth interval 50.5 to 55.5 feet in boring B-620 from which sample CS-03 was taken.
- Substituting in Equation 1, $G_L = (\gamma/g)V_s^{\ 2} = (0.139/32.2) \times 6200^2/144 = 1152$ ksi.
- From Equation 2, $E_L = 2 \times (1+0.30) \times 1152 = 2995$ ksi (where $\mu = 0.30$ from measured V_s and P-wave velocity values at the sample depth).
- 2995 ksi > 2900 ksi, but very close.

The results from the two laboratory tests to determine E indicated that the E values were higher than (B-610 sample) or very close to (B-620 sample) the values derived from shear wave velocity measurements. These results demonstrate the linearity of the rock stiffness within the range of strains measured, i.e., for both rock formations, $E_L = E_H$ and $G_L = G_H$.

Summary

Based on the structure and strength of the Key Largo and Fort Thompson Formations, no strain dependency of elastic or shear moduli is expected. The two laboratory tests performed on samples of these materials confirm this. Additionally, if the strain-dependent modulus relationship of the much softer Miami Limestone is conservatively assumed for the Key Largo and Fort Thompson Formations, the strains generated in both the settlement and SHAKE analyses are too small to cause any reduction in modulus.

Symbols used in this Response

E = elastic modulus of material

$E_L = E_{MAX}$ = low strain (typically taken as 10^{-4} percent) elastic modulus of material

E_H = low strain elastic modulus of material

G = shear modulus of material

$G_L = G_{MAX}$ = low strain (typically taken as 10^{-4} percent) shear modulus of material

G_H = high strain elastic modulus of material

U = unconfined compressive strength

V_s = shear wave velocity

γ = total unit weight of material

g = acceleration due to gravity

μ = Poisson's ratio

RQD = rock quality designation

This response is PLANT SPECIFIC.

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References:

None

ASSOCIATED COLA REVISIONS:

Change the title of FSAR Subsection 2.5.4.2.1.3.11 as follows:

2.5.4.2.1.3.11 Elastic Modulus and Shear Modulus ~~(High Strain)~~

Change the last paragraph of FSAR Subsection 2.5.4.2.1.3.11 as follows:

~~Note that the results of l~~**L**aboratory elastic modulus **(E)** testing **was** performed on ~~one~~ **two** sample**s that were both at around 50 feet depth, at the boundary of** the Key Largo Limestone and ~~one sample of~~ the Fort Thompson **Limestone**~~Formation~~**. The results were E = 3,700 ksi and 2,900 ksi for the two samples, where E from the laboratory results represents a tangent modulus value corresponding to 40 to 60 percent of ultimate strength. The stress strain curves for both tests showed reasonable linearity all the way to failure, at strains of approximately 6.5 and 8.3 x 10^{-2} percent, respectively. The results from these two tests indicate elastic modulus values higher than or very close to the corresponding values derived from shear wave velocity measurements at the same depths. These results demonstrate the linearity of the rock stiffness within the range of strains measured.** ~~are compared to the E and G values derived based on the average shear wave velocities measured, and they indicate E = 2700 kips per square inch (ksi) for the Key Largo Limestone and E = 2900 ksi for the Fort Thompson sample. The shear and elastic modulus values based on shear wave velocity are considered more representative because the laboratory results are derived from samples with higher than average RQD.~~

Add the following after the last paragraph of Subsection 2.5.4.2.1.3.11:

Rock Modulus Strain Dependency

Rock modulus strain dependency is a function of rock strength. Rock is cemented, and so the particles do not separate under typical foundation loading. There is a point at which a rock becomes so soft/weak that there is some movement of particles at higher strains, i.e., there is an element of strain dependency at higher strains. If the assumption is made that the shear modulus values of the Key Largo and Fort Thompson Formations are strain dependent, they would be less strain dependent than the much softer Miami Limestone. In other words, their shear modulus degradation curves would be to the right of the Miami Limestone curve in Figure 2.5.4-233. Shear modulus of the Key Largo and Fort Thompson Formations is used in the SHAKE analysis of the soil and rock column. Figure 2.5.2-248 shows strain levels in these strata to be well below 0.005 percent. The Miami Limestone shear modulus degradation curve in Figure 2.5.4-233 shows that there is no degradation of G below 0.01 percent strain. Thus, even if the shear moduli of these formations were strain dependent, the strains generated would be too small to cause any modulus reduction.

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ASSOCIATED ENCLOSURES:

None

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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. Define “Lean Concrete” and clarify if CLSM is used. Also specify which ACI standard(s) will be followed.

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. In the remainder of the response, the term “lean concrete fill” will be shortened to “concrete fill”. The American Concrete Institute (ACI) standard that will be followed is ACI 207, “Guide to Mass Concrete” prepared by ACI Committee 207 (FSAR Subsection 2.5.4 Reference 281). Controlled Low Strength Material (CLSM) will not be used for fill beneath the nuclear island.

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.

FSAR Subsection 2.5.4.12 indicates the concrete fill will have an estimated compressive strength of 1500 psi. The design bearing capacity of this strength of concrete is over 100 ksf. According to the AP 1000 Design Control Document, the maximum applied bearing pressure (from the Reactor Building) is 8.9 ksf, less than 9 percent of the bearing capacity

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of the concrete. Thus, cracking of the concrete due to loading/overstressing is not expected.

FSAR Reference 281 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 19-foot thick layer of concrete fill qualifies as mass concrete. As such, FSAR Reference 281 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 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. Typically, maximum thickness of each concrete fill lift is 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; effective monitoring of the thermocouples should 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. 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.

The rock beneath the concrete fill (i.e., the Key Largo Formation) has the same compressive strength as the concrete fill (1,500 psi) and the rock beneath the Key Largo Formation (i.e., the 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

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distribution. A stress distribution below the foundation can be conservatively taken as no steeper than 2V:1H, but most likely closer to 1V:1H.

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 base of the concrete foundation mat of the nuclear island bearing on the mudmat and the mudmat bearing on the concrete fill. The mudmat provides a working surface for the construction of the concrete foundation mat; the mudmat has a minimum thickness of 12 inches of un-reinforced concrete. 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 structural backfill and in-situ soils and rock will not be required to resist lateral loading from the building.

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.

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 approximately 19-foot thick mass of concrete fill are 240 feet x 290 feet, including 30-foot width of concrete fill extending beyond the perimeter of the nuclear island. The concrete fill will be placed on top of Key Largo Limestone that will have been extensively grouted to enable dewatering. The concrete fill

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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 concrete 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 that can cause steel corrosion is not of concern, since the concrete is unreinforced.

Based on the conditions described above and the potential solution for combating the effects of high sulfate content, there are little or no mechanisms that could 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:

None

ASSOCIATED COLA REVISIONS:

The second paragraph of FSAR Subsection 2.5.4.5.1 will be revised in a future FSAR revision as follows:

The deepest excavation is **to** approximately El. -35 feet. Structural fill is placed around but not below the power block structures extending to as deep as El. -14 feet. Lean concrete fill is placed between **the bottom of the mudmat that is below** El. -14 feet and the bottom of the excavation. **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 final grade is shown on Figure 2.5.4-201. The grade in profile is shown in Figure 2.5.4-221.

The third paragraph of FSAR Subsection 2.5.4.5.1.2 will be revised in a future FSAR revision as follows:

Structural fill consisting of excavated fill material is placed around but not below any nuclear island structure. Replacement material below the nuclear islands consists of lean concrete fill. The selection of lean concrete mix design is made at project detailed design. The compressive strength of 1.5 ksi is estimated for lean concrete fill. **The approximately 19-foot thick layer of lean concrete fill qualifies as mass concrete. As such, Reference 281 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.**

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- **The low strength of the lean 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 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 lean concrete fill to be placed in relatively thin lifts to avoid cracking. Typically, maximum thickness of each lean concrete fill lift is 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; effective monitoring of the thermocouples should 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).

ASSOCIATED ENCLOSURES:

None

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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 El. -35 feet to expose the Key Largo Limestone. In some areas, the Key Largo Limestone occurs above El. -35 feet. This Key Largo Limestone will be excavated down the El. -35 feet.

The profiles from the power block subsurface investigation (refer to FSAR Figures 2.5.4-203 through 2.5.4-208, copies of which are attached) 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 and testing of actual subsurface materials encountered, and their suitability for foundation support.

Once subgrade suitability at the proposed bearing stratum is confirmed, Nuclear Island excavations are backfilled with concrete fill up to the foundation level of the structures. Adequate top surface uniformity and integrity of the 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.

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- 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. A more detailed description of the structure and content of the Miami Limestone is given in FSAR Subsection 2.5.1.1.1.2.1.1.
- 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. A more detailed description of the structure and content of the Key Largo Limestone is given in FSAR Subsection 2.5.1.1.1.2.1.1.

The Miami Limestone is considerably softer than, and has a quite different structure, from the coralline Key Largo Limestone, as described above. 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 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 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, concrete fill 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.

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

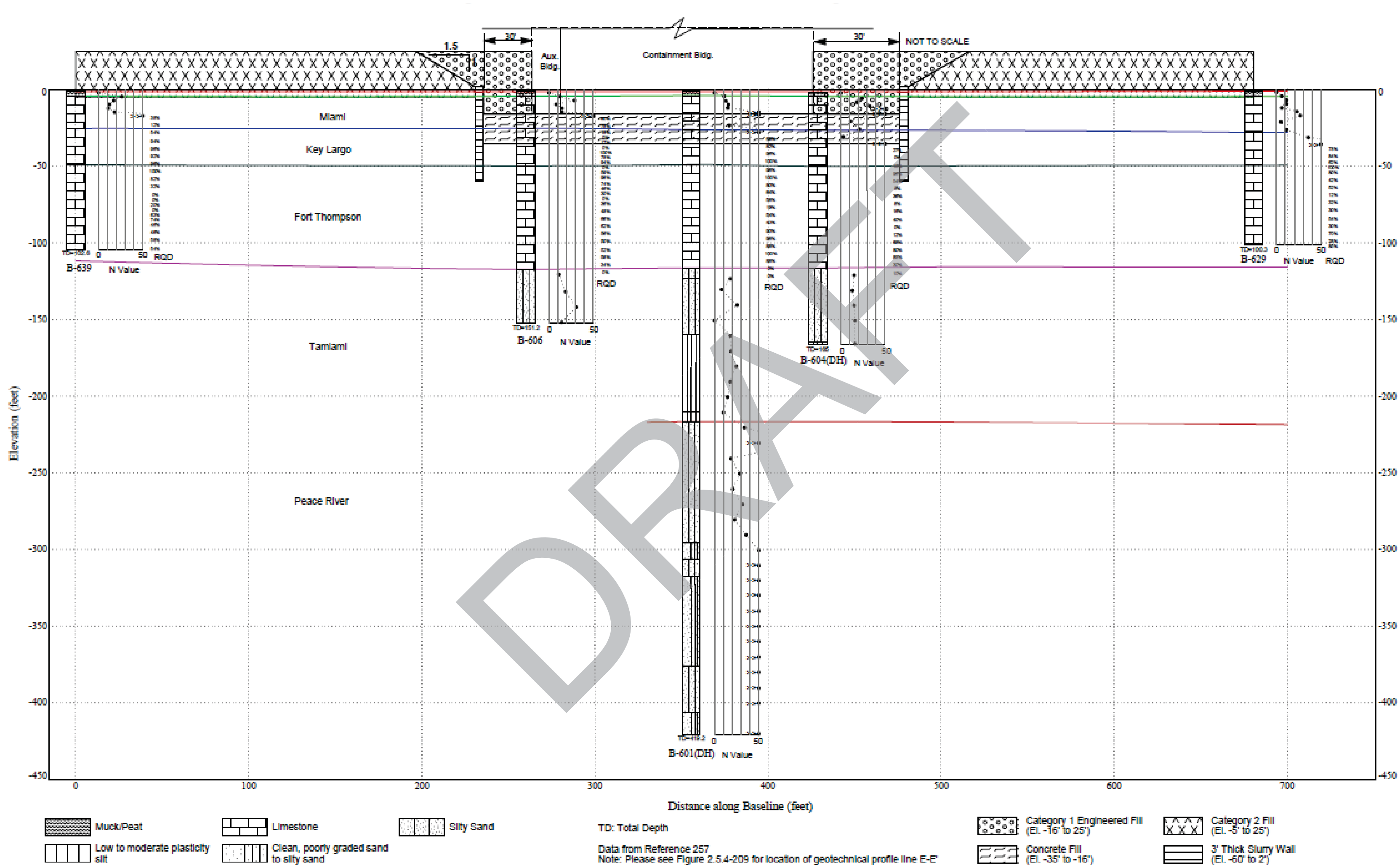


Fig 2.5.4-204 Geotechnical Cross Section E-E' Through Unit 6 Power Block

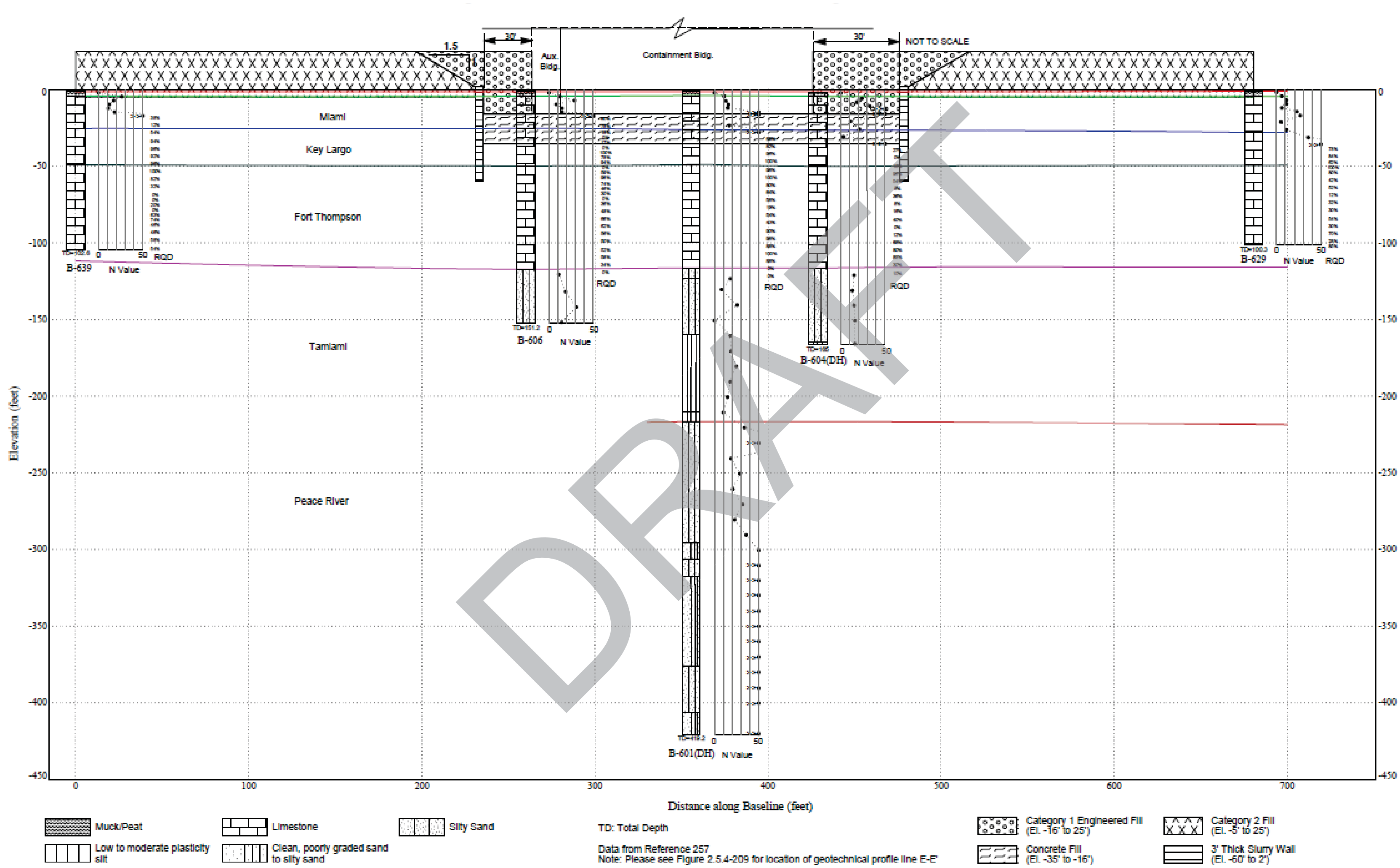


Fig 2.5.4-205 Geotechnical Cross Section F-F’ Through Unit 6 Power Block

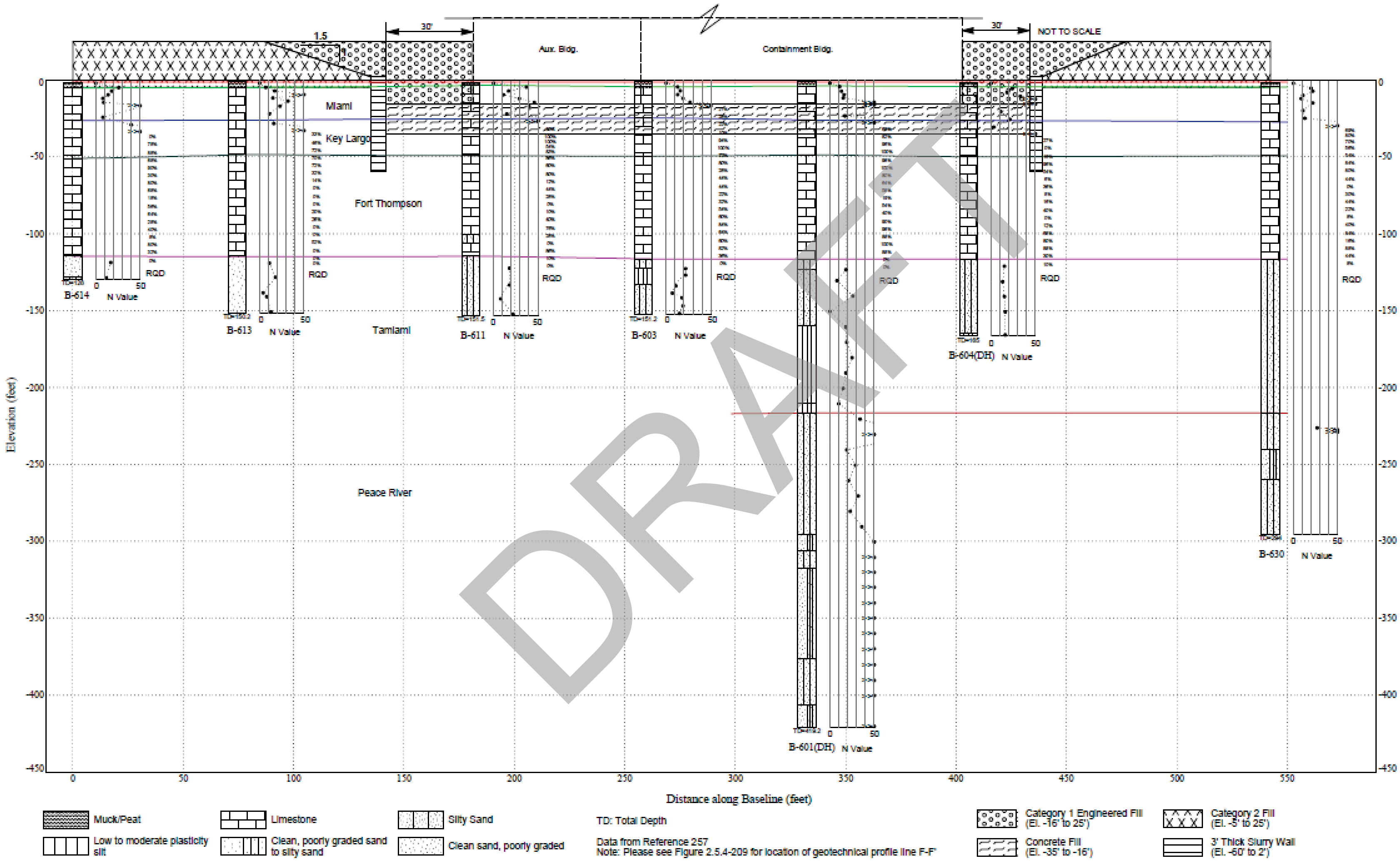


Fig 2.5.4-206 Geotechnical Cross Section A-A' Through Unit 7 Power Block

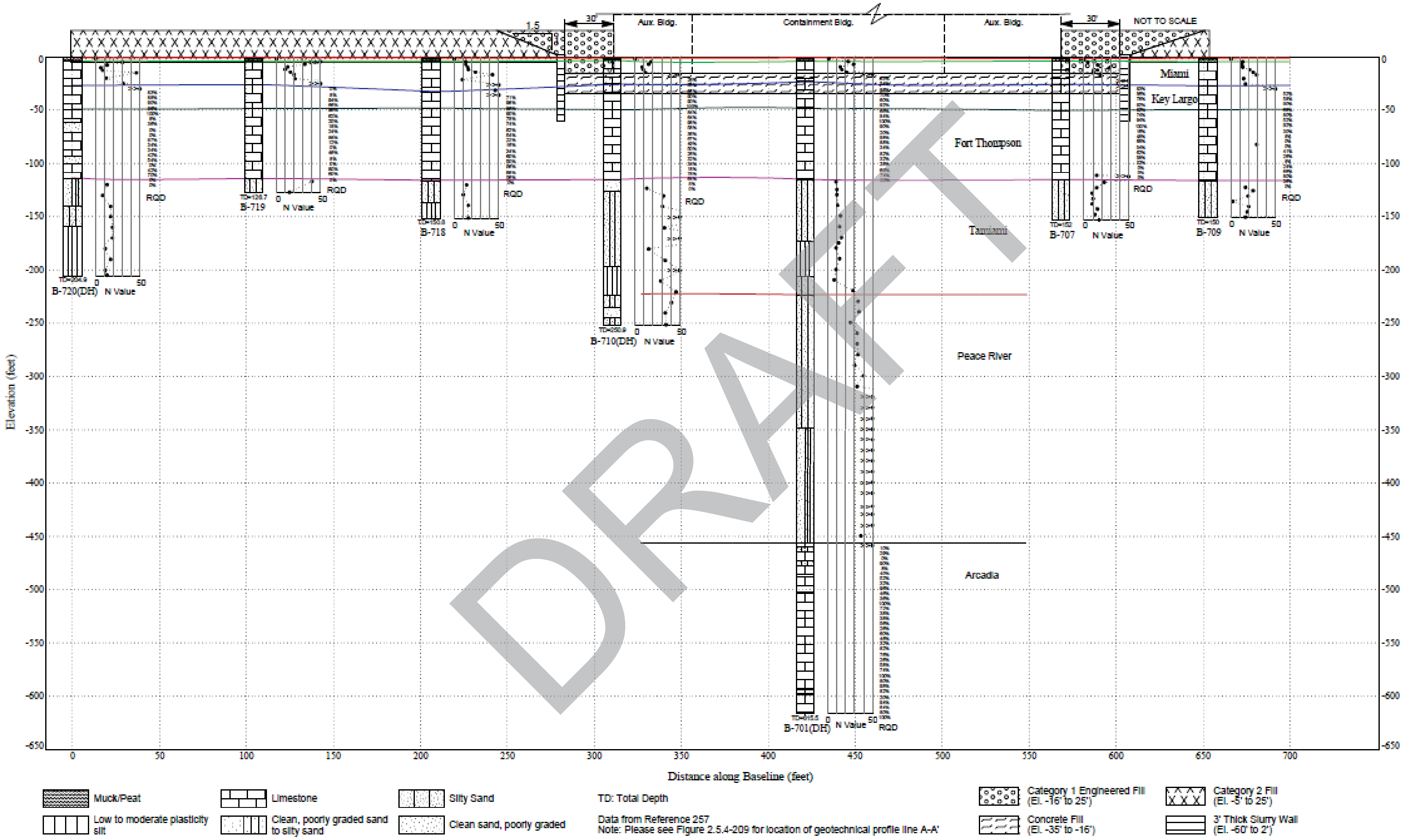


Fig 2.5.4-207 Geotechnical Cross Section B-B' Through Unit 7 Power Block

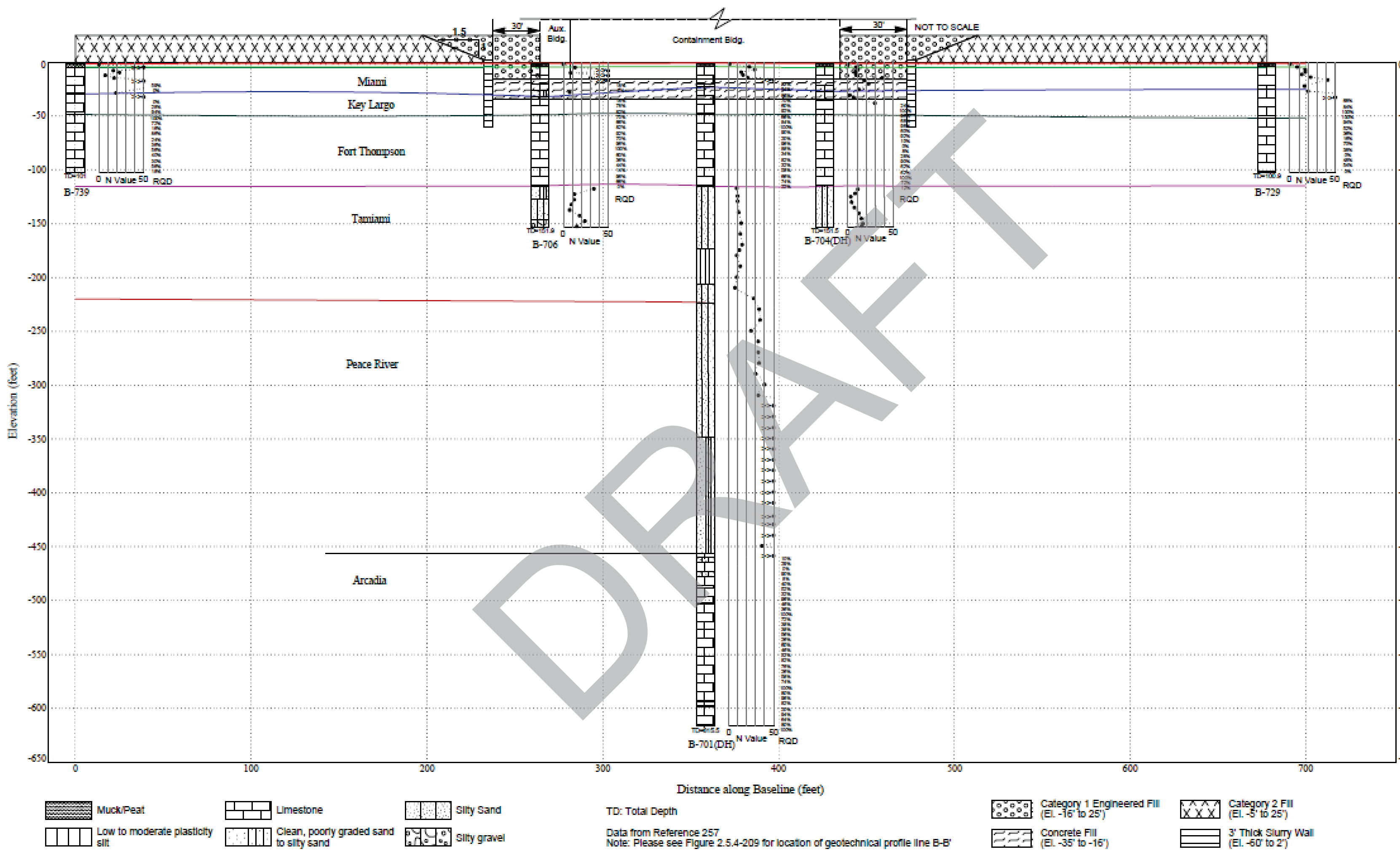
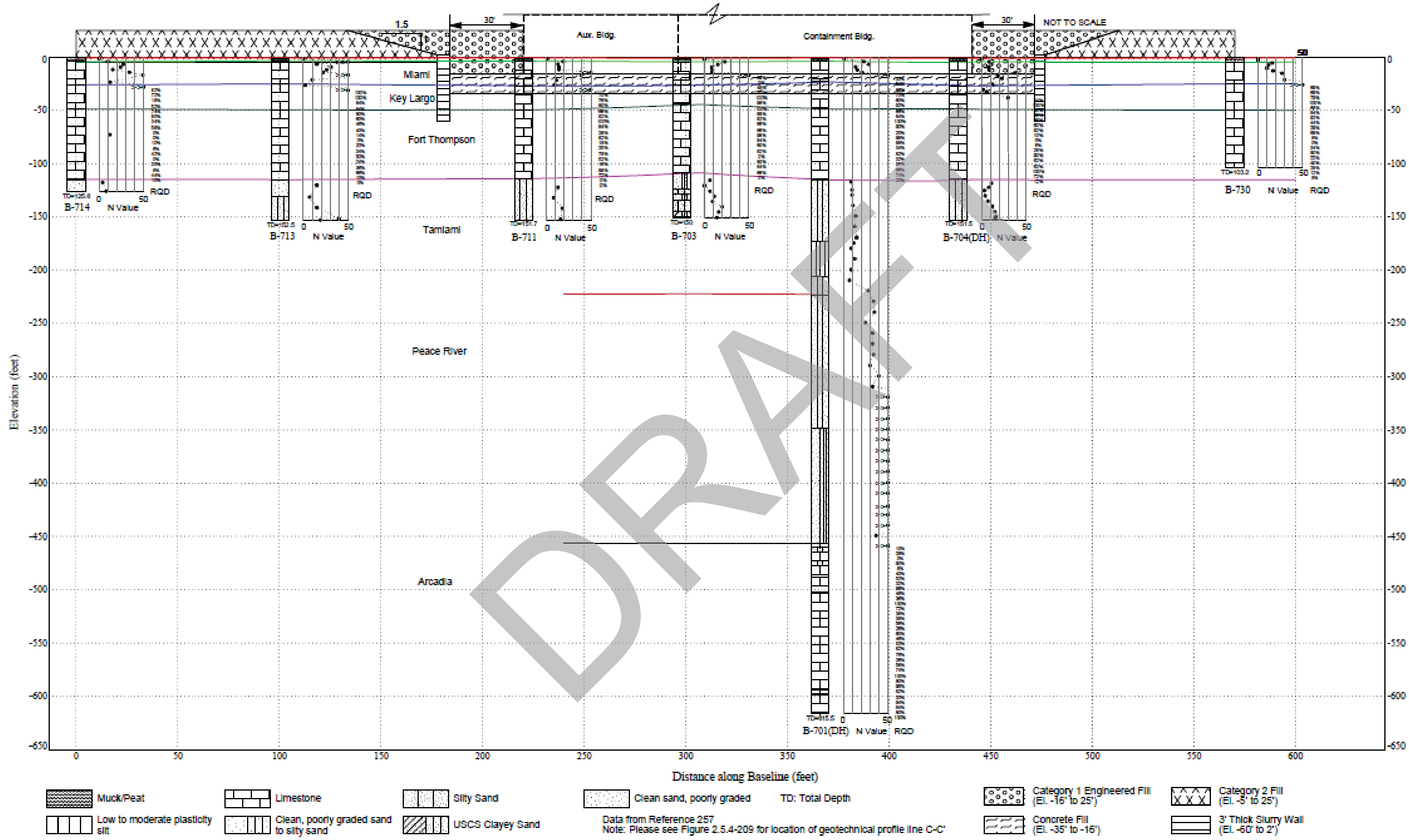


Fig 2.5.4-208 Geotechnical Cross Section C-C' Through Unit 7 Power Block



This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

Revised FSAR Figures 2.5.4-203 through 2.5.4-208 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.

ASSOCIATED ENCLOSURES:

None

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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: 02.05.04-8 (eRAI 6006)

Section 2.5.4.2.1.3.8 discusses the computation of effective (drained) friction angle in each sand stratum from corrected SPT, CPT and laboratory direct shear test results. However, previous discussion in the FSAR Section 2.5.4.2.1.3.2.1 indicates that the SPT data is suspect due to anomalies of sampling, only 4 CPT profiles are available for use and no direct shear testing results were provided. Correlations of these data to generate typical soil properties are expected to have a high degree of uncertainty. Laboratory tests of these material samples can be expected to be extremely disturbed considering the depth and behavior of materials. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," please justify the adequacy of the friction angle. Also, provide detailed information regarding your laboratory direct-shear test program and test results.

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FPL RESPONSE:

The information regarding the derivation of the corrected SPT N-values (N_{60}) from the SPT and CPT results is provided in the supplemental response to RAI 02.05.04-2 response. This RAI 02.05.04-8 supplemental response discusses the methodology used to estimate the effective friction angle (ϕ') of the Upper Tamiami and Peace River formations using the SPT and CPT results, and of the Lower Tamiami formation using the consolidated undrained triaxial testing results. Also in this response, information regarding the consolidated undrained triaxial testing is provided; no direct shear tests were performed. As noted in the supplemental response to RAI 02.05.04-2, the high relative density of the Upper Tamiami and the hardness of the Lower Tamiami are reflected in the high shear wave velocity measurements obtained in these strata, namely average values of 1400 ft/sec and 1600 ft/sec, respectively.

<u>Upper Tamiami Formation</u>

In FSAR Table 2.5.4-209, a best estimate N_{60}-value of 40 blows/ft is given for the Upper Tamiami formation, and 32 blows/ft for the Lower Tamiami formation.

For an N_{60}-value range of 30 to 50 blows/ft, the value of ϕ' can vary from 36^o to 41^o on the low end and 40^o to 45^o on the high end for cohesionless soils (FSAR Subsection 2.5.4 Reference 221, Table 2.43). Considering the moderate fines content (28% in FSAR Table 2.5.4-209) observed in the Upper Tamiami formation, for an N_{60}-value of 40 blows/ft, an effective friction angle of 35^o is conservatively recommended. Note that the average N_{60}-value from all of the measured N-values in the Upper Tamiami was 27 blows/ft, including those values considered unreasonably low due to an upward vertical hydraulic gradient in the soil (FSAR Subsection 2.5.4.8.2). Thus, considering this conservative average N_{60}-value, FSAR Subsection 2.5.4 Reference 221, Table 2.43 shows for an N_{60}-value range of 10 to 30 blows/ft that the value of ϕ' can vary from 30^o to 36^o on the low end and 35^o to 40^o on the high end for cohesionless soils. Based on these values, ϕ' equal to 35^o would have been a reasonable choice for N_{60} equal to 27 blows/ft.

In addition, two empirical correlations proposed for CPT were utilized to further evaluate the effective friction angle of the Upper Tamiami formation:

(i) Based on the calibration chamber test data from several quartz sands, Reference 1 (based on FSAR Subsection 2.5.4 Reference 222) recommends the use of Eq. 1 for uncemented, unaged quartz sands. Considering that the average calcite content for the Tamiami formation is 20% (FSAR Table 2.5.4-205), the correlation proposed for quartz sand is reasonable to use.

$$\phi' = \arctan[0.1 + 0.38 \cdot \log(q_T / \sigma_{v0}')] \qquad \text{(Eq. 1)}$$

Where, ϕ' is the peak friction angle, q_T is the CPT corrected tip resistance, and σ_{v0}' is the effective overburden stress.

(ii) Based on the statistical analyses of CPT calibration chamber data corrected for boundary effects, FSAR Subsection 2.5.4 Reference 282 suggests that in clean quartzitic sandy soils, the peak friction angle ϕ' can be approximated by Eq. 2.

$$\phi' = 17.6^o + 11.0 \log(q_{T1}) \qquad \text{(Eq. 2)}$$

where ϕ' is in degrees, $q_{T1} = (q_T / \sigma_{atm})/(\sigma_{vo}' / \sigma_{atm})^{0.5}$, and σ_{atm} is atmospheric pressure ($\sigma_{atm} \approx$ 1 tsf).

The above correlations proposed in Reference 1 and FSAR Subsection 2.5.4 Reference 282 for cone penetration testing were used to derive the friction angles, and the results are presented in Figure 1 as a function of elevation. It is observed that the friction angles derived by these two correlations are generally in the range between 33^o and 40^o for the Upper Tamiami formation, which is consistent with that of 35^o derived from the N_{60}-value. Thus, it is reasonable to use an effective friction angle of 35° for the Upper Tamiami formation, as listed in FSAR Table 2.5.4-209.

In summary, the N_{60}-values from SPT and CPT measurements and the derived friction angles using the suggested correlations should be similar if the tests are not affected adversely. Due to the adverse effect of upward hydraulic gradient on SPT measurements, the results, as expected, showed considerably smaller N_{60}-values than would normally be recorded for granular materials at such depth. The subsequent measurements of CPTs provided more distinct patterns with depth. More reliable equivalent N_{60}-values were obtained from the CPT results based on various empirical correlations. Thus, using the correlations for SPT and CPT, the CPT measurements are considered adequate and suitable for deriving friction angle.

Lower Tamiami Formation

As stated in FSAR Section 2.5.2.1.3.9, only one primarily fine-grained stratum (i.e., Lower Tamiami formation) was encountered in the subsurface investigation. Considering the high fines content (62% in FSAR Table 2.5.4-209) observed in the Lower Tamiami Formation layer, the undrained shear strength was derived using the correlation in Eq. 3 (derived from Chapter 1, Table 4 of Reference 2).

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$s_u = N_{60}/8$ (in kips per square foot [ksf]) (Eq. 3)

For an N_{60}-value of 32 blows/ft, the undrained shear strength is estimated as 4 ksf for the Lower Tamiami formation.

In addition, consolidated undrained triaxial shear testing was performed on a sample (UD-12 from boring B-630) of the Lower Tamiami Formation, in accordance with ASTM D 4767-04. Three UD-12 specimens were isotropically consolidated under effective stresses of 5.08, 10.03 and 15.01 ksf, and were subsequently sheared under undrained conditions with confinement pressures of 34.7, 69.4 and 104.2 psi, respectively. Based on the results from these tests, an effective friction angle (ϕ') of 20°, and an effective cohesion (c') of 1.7 ksf were estimated for the Lower Tamiami Formation. According to Reference 3, the average ϕ' ranges from around 20° for normally consolidated highly plastic clays up to 30° or more for silty and sandy clays. Thus, an effective friction angle of 20° for the Lower Tamiami formation consisting of sandy silt with minor amounts of silty clay is reasonable and conservative. As noted in the question, these material samples can be expected to be extremely disturbed considering the depth and behavior of materials. Thus, the laboratory testing of “undisturbed” samples of these materials was purposely limited.

Peace River Formation

For the Peace River formation, an N_{60}-value of 75 blows/ft is recommended for use (FSAR Table 2.5.4-209). For an N_{60}-value range of 30 to 50 blows/ft, the value of ϕ' can be in the range of 36° to 41° on the low end and 40° to 45° on the high end for cohesionless soils (FSAR Subsection 2.5.4 Reference 221, Table 2.43). Considering the relatively low fines content (16% in Table 2.5.4-209) observed in Peace River, and the fact that the recommended N_{60}-value is higher than the upper limit of the correlation range, it is conservative and reasonable to use an effective friction angle of 40°.

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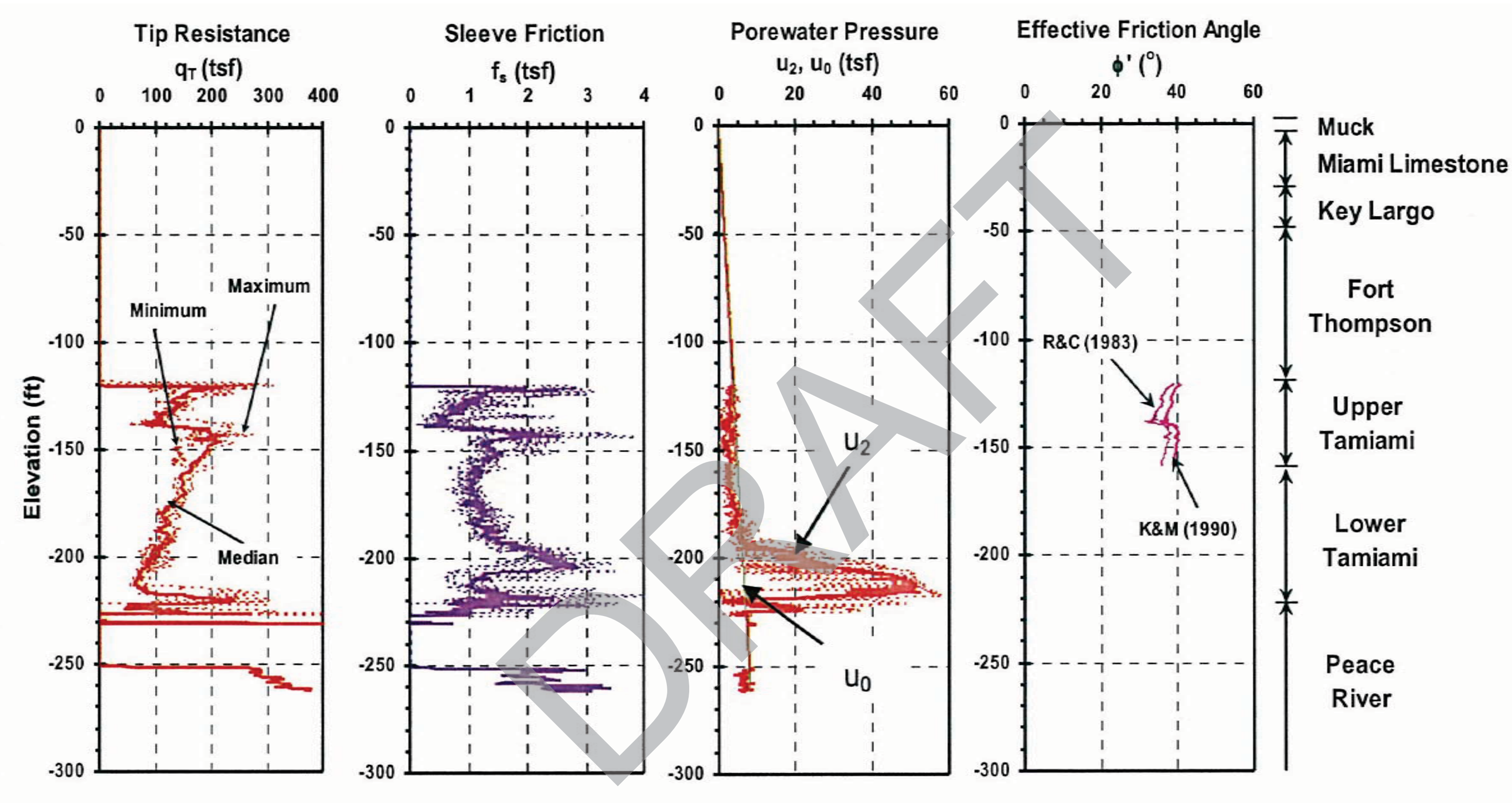
Figure 1. CPT Results and Derived Friction Angle for Upper Tamiami Formation



u_2 = Static porewater pressure
u_0 = Measured porewater pressure

This response is PLANT SPECIFIC

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References:

1. Robertson, P.K. *Cone Penetration Testing, Geotechnical Applications Guide,* Produced by ConeTec, Inc., and Gregg In Situ, Inc., 3rd Edition, November 2000.
2. Naval Facilities Engineering Command (NAVFAC), "Foundations & Earth Structures," *Design Manual 7.01*, Alexandria, VA, 1986.
3. Holtz, R.D. and Kovacs, W.D. *An Introduction to Geotechnical Engineering*, Prentice-Hall, NJ, 1981.

ASSOCIATED COLA REVISIONS:

The following changes to FSAR Subsection 2.5.4.2.1.3.8 will be made in a future COLA revision:

2.5.4.2.1.3.8 Angle of Internal Friction

The drained/effective angle of internal friction (ϕ') of each sand stratum is estimated **using the data** from corrected SPT N60-values, CPT tip resistances (q_t), and laboratory **consolidated undrained triaxial** ~~direct shear~~ testing results.

In Table 2.5.4-209, a best estimate N_{60}-value of 40 blows/ft is given for the Upper Tamiami formation, and 32 blows/ft for the Lower Tamiami formation. For an N_{60}-value range of 30 to 50 blows/ft, the value of ϕ' can vary from 36° to 41° on the low end and 40° to 45° on the high end for cohesionless soils (Reference 221, Table 2.43). Considering the moderate fines content (28% in Table 2.5.4-209) observed in the Upper Tamiami formation, an effective friction angle of 35° is conservatively recommended for an N_{60}-value of 40 blows/ft. Note that the average N_{60}-value from all of the measured N-values in the Upper Tamiami is 27 blows/ft, including those values considered unreasonably low due to an upward vertical hydraulic gradient in the soil (Subsection 2.5.4.8.2). Reference 221, Table 2.43 shows that, for an N_{60}-value range of 10 to 30 blows/ft the value of ϕ' can vary from 30° to 36° on the low end and 35° to 40° on the high end for cohesionless soils. Based on these values, ϕ' equal to 35° would have been a reasonable choice for N_{60} equal to 27 blows/ft.

In addition, two empirical correlations proposed for CPT were utilized to further evaluate the effective friction angle of the Upper Tamiami formation:

~~Using Reference 221, ϕ' is estimated from the average corrected N60-value.~~
(i) Based on the calibration chamber test data from several quartz sands, Reference 283 (based on Reference 222) recommends the use of Eq. 2.5.4-3 for uncemented, unaged quartz sands. Considering that the average calcite content for the Tamiami formation is 20% (Table 2.5.4-205), the correlation proposed for quartz sand is reasonable to use.

The empirical correlation used to obtain ϕ' from CPT tip resistance (Reference 222) is:

ϕ' = arctangent [0.1 + 0.38 · log(q_t/σ_v')] (in degrees) Equation 2.5.4-3

where,

q_t = the CPT **corrected** tip resistance
σ_v' = the effective overburden pressure at the depth of the CPT test interval

(ii) Based on the statistical analyses of CPT calibration chamber data corrected for boundary effects, Reference 282 suggests that in clean quartzitic sandy soils, the peak friction angle ϕ' can be approximated by Eq. 2.5.4-3a:

ϕ' = 17.6 + 11.0 log(q_{t1}) (in degrees) Equation 2.5.4-3a

where $q_{t1} = (q_t/\sigma_{atm})/(\sigma_v'/\sigma_{atm})^{0.5}$, and σ_{atm} is atmospheric pressure ($\sigma_{atm} \approx 1$ tsf).

The above correlations proposed in References 283 and 282 for cone penetration testing were used to derive the friction angles. It is observed that the friction angles derived by these two correlations are generally in the range between 33° and 40° for the Upper Tamiami formation, which is consistent with that of 35° derived from the N_{60}-value. Thus, it is reasonable to use an effective friction angle of 35° for the Upper Tamiami formation, as listed in Table 2.5.4-209.

Recommended values of ϕ' derived from the different correlations/test methods (i.e., from SPT correlation, CPT correlation, and laboratory **consolidated undrained triaxial** ~~direct shear~~ testing), and for each stratum, are shown in Table 2.5.4-209. An effective friction value (ϕ') of 20 degrees is measured in triaxial testing on one tube sample of the lower Tamiami Formation sandy silt (Stratum 6) as presented in Table 2.5.4-208.

Add the following to FSAR Subsection 2.5.4.13 References

283. Robertson, P.K., *Cone Penetration Testing, Geotechnical Applications Guide,* Produced by ConeTec, Inc., and Gregg In Situ, Inc., 3rd Edition, November 2000.

ASSOCIATED ENCLOSURES:

None

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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. 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.

The settlement calculation has been updated to reflect the 8.9 ksf applied contact pressure for the reactor building. This calculation has been updated to DCD Rev. 19. The loading and settlement requirements have not changed from DCD Rev. 18 to DCD Rev. 19. FSAR Table 2.5.4-219 will be revised accordingly.

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

The revised calculated settlement based on having approximately 19 ft of concrete fill beneath the foundation and the underlying 80 ft of Key Largo Limestone and Fort Thompson Limestone will be less than 0.05 in. due to the application of 8.9 ksf from the reactor building/nuclear island (NI). As shown in FSAR Figures 2.5.4-203 through 2.5.4-208, these rock formations are continuous across the site, and, as shown in FSAR Subsection 2.5.1, across the region. Thus, the NI is sitting on an approximately 100-ft thick rock plate extending for considerable distance in all directions. This 100 ft includes the 19 ft of concrete fill below and 30 ft beyond the NI foundation. There will be negligible flexure of the 100-ft thick rock plate under 8.9 ksf loading. Using the conservative analogy of a fixed beam with uniformly distributed loading, and neglecting the rigidity of the NI basemat itself, estimated settlement beneath the center of the rock due to bending is less than 0.1 in. If the rock plate does not bend, then there will be no settlement of the soils below the rock formations. Differential settlement across the NI will be negligible (< 0.1 in.). FSAR Table 2.5.4-219 will be revised to indicate total estimated settlement of the NI will be less than 0.15 in.

The annex, turbine and radwaste buildings are founded on compacted structural fill that extends from the bottom of foundation down to the top of the concrete fill where the buildings are between the diaphragm wall and the NI, and down to the top of the Miami Limestone where the buildings are outside the diaphragm wall. Essentially all of the settlement that these buildings will experience will be due to settlement of the structural fill. As with the NI, settlement of the rock and underlying materials will be less than 0.15 in. The annex building varies in width (approximately 66 ft to 145 ft) and the depth of the turbine building foundation varies (approximately El. +18.5 ft to El. +1.3 ft). These variations, combined with the different thickness of fill within and outside the diaphragm wall, result in different estimated settlements under different portions of the buildings. Nevertheless, the estimated maximum settlement of the structural fill beneath these buildings is about 2.5 in. under the center and about 1.3 in. under the edge, neglecting the structural stiffness of the buildings' foundations. FSAR Table 2.5.4-219 will be revised to indicate these settlements. These settlements are within the limits of acceptable settlement "without need for further evaluation" given in Table 2.5-1 of Revision 19 of the AP1000 DCD (and in FSAR Table 2.0-201). As stated in Section 2.5.4.3 of Revision 19 of the DCD, "Differential settlement between the nuclear island foundation and the foundations of adjacent buildings does not have an adverse effect on the safety-related functions of structures, systems, and components". It should be noted that almost all of the settlement of the structural fill will occur during construction of these buildings, since the fill is granular.

As noted in the previous paragraph, there will be very limited differential settlement between the annex, turbine and radwaste buildings and the NI due to the settlement of the structural fill supporting these buildings adjacent to the NI. However, there will be no relative lateral movement between these buildings and the NI, or between the buildings themselves, under design earthquake conditions. The peak ground acceleration (pga) at the Turkey Point site is less than 0.1g, although lateral seismic earth pressures are based on a pga of 0.1g. Thus, if the coefficient of sliding resistance between the structure foundation and the supporting material is marginally greater than 0.1, there will be sufficient resistance to sliding. FSAR Table 2.5.4-209 shows the coefficient of sliding of a concrete foundation on the structural fill is 0.5, on the Miami Limestone is 0.6 and on the Key Largo Limestone is 0.7. The coefficient of sliding of a concrete foundation on concrete fill can also be taken as 0.7. Thus, neither the NI nor the surrounding buildings will slide under design earthquake conditions

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.

Since the predicted settlements are within the limits shown in FSAR Table 2.0-201, no further evaluation is needed based on the DCD criteria. If a settlement monitoring program is adopted for confirming the predicted settlement of the structural fill, the program will follow the

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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. Settlement would be monitored throughout the entire construction sequence as well as during plant operation.

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

The high-strain elastic modulus was used to estimate the settlement of the structural fill. As described in Part b of this response, no settlement of the other soil strata, i.e., the Tamiami and Peace River Formations, is anticipated and this is reflected in the proposed changes to the FSAR given at the end of this response. Thus, no modulus degradation curve was used.

This response is PLANT SPECIFIC.

References:

None

ASSOCIATED COLA REVISIONS:

FSAR Subsection 2.5.4.10.3 will be revised in a future COLA revision as follows:

The containment and auxiliary buildings (nuclear island) share the same mat foundation and are founded on lean concrete placed above rock of the Key Largo Limestone. Therefore, for settlement computations, the bottom of the foundation is taken at El. -14 feet on lean concrete. Settlement of the rock strata is computed using the elastic modulus values tabulated in Table 2.5.4-209. ~~Settlement of the soil strata are evaluated using the strain compatible elastic moduli of the Tamiami and Peace River Formations with corresponding axial strains, as discussed later in this section.~~ The elastic modulus for the lean concrete used for settlement estimates is derived as follows:

The thickest part of lean concrete is between El. -14 feet and El. -35 ft, i.e., 21 feet thick (see Figure 2.5.4-222). The elastic modulus of lean concrete with a unit weight of 145 pcf can be calculated using the following equation (Reference 274).

$$E_C = 1820 \cdot f'_C{}^{0.5} \text{ (ksi)} \qquad \text{Equation 2.5.4-23}$$

where,

f'_C = specified compressive strength of concrete (ksi)

The lean concrete placed on rock is expected to have a ~~minimum~~ compressive strength of 1.5 ksi.

DRAFT

$$f'_C = 1.5 \text{ ksi, then } E_C = 1820 \cdot 1.5^{0.5} = 2229 \text{ ksi} \approx \cancel{32{,}000}\ 321{,}000 \text{ ksf}$$

The estimated settlement of the foundations constructed on the concrete fill and the underlying approximately 80 feet of Key Largo Limestone and Fort Thompson Limestone is less than 0.05 inches due to the application of 8.9 ksf from the nuclear island. As shown in Figures 2.5.4-203 through 2.5.4-208, these rock formations are continuous across the site, and, as shown in Subsection 2.5.1, across the area. Thus, the nuclear island is sitting on an approximately 100-foot thick rock plate extending for considerable distance in all directions. There is negligible flexure of the 100-foot thick rock plate under 8.9 ksf loading. Using the conservative analogy of a fixed beam with uniformly distributed loading, and neglecting the rigidity of the nuclear island basemat itself, estimated settlement beneath the center of the rock due to bending is less than 0.1 inch. If the rock plate does not bend, then there can be no settlement of the soils below the rock formations. Differential settlement across the nuclear island is negligible (< 0.1 inch). The estimated settlement of the nuclear island is less than 0.15 inch, as shown in Table 2.5.4-219.

The annex, turbine and radwaste buildings are founded on compacted structural fill that extends from the bottom of foundation down to the top of the concrete fill where the buildings are between the diaphragm wall and the nuclear island, and down to the top of the Miami Limestone where the buildings are outside the diaphragm wall. Essentially all of the settlement that these buildings experience is due to settlement of the structural fill. As with the nuclear island, settlement of the rock and underlying materials is less than 0.15 inches. The width of the annex building varies, as does the depth of the turbine building foundation. These variations, combined with the different thickness of structural fill within and outside the diaphragm wall, result in different estimated settlements under different portions of the buildings. Nevertheless, the estimated maximum settlement of the structural fill beneath these buildings is about 2.5 inches under the center and about 1.3 inches under the edge, neglecting the structural stiffness of the buildings' foundations. The estimated settlements of these buildings are shown on Table 2.5.4-219. It should be noted that most of the settlement of the structural fill will occur during construction of these buildings, since the fill is granular.
~~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 8.9 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.~~

DRAFT

~~For calculating settlement of structures using the elastic method, the maximum principal strain is the vertical strain (i.e., $\varepsilon_1 = \varepsilon_v$), while the minimum principal strain is assumed to be zero (i.e., $\varepsilon_3 = 0$). Because the maximum shear strain $\gamma_{max} = \varepsilon_1 - \varepsilon_3 = \varepsilon_1$ (Reference 275) and $E/E_{max} = G/G_{max}$, the elastic modulus reduction curves with respect to vertical strain should be the same as the shear modulus reduction curves with respect to the shear strain. Therefore, the strain levels in degradation curves (Figure 2.5.4-233) are interchangeable between the shear strain and axial strain without any need of correction. Thus, the same degradation curve in Figure 2.5.4-233 with G/G_{max} on the vertical axis and percent shear strain on the horizontal axis can be used to determine appropriate values of E for settlement calculations. The ratio of E/E_{max} is equivalent to G/G_{max} and the computed percent axial strain corresponds to the percent shear strain using Figure 2.5.4-233.~~

~~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 2.5-219 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 and 254 feet, under loading of 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.~~

As noted earlier, Equation 2.5.4-22 computes the stress distribution beneath a flexible foundation, which accounts for the sometimes significant difference in computed settlement between the center of the foundation and the mid-point of the side of the foundation. In fact, the foundations of the structures on structural fill listed in Table 2.5.4-219 are ~~thick~~ reinforced concrete mats with appreciable structural stiffness. Thus the mean settlements listed in Table 2.5.4-219 more closely reflect the actual anticipated settlements across the whole foundation.

Table 2.0-201 lists the DCD limits of acceptable settlement without need of additional evaluation. Limits for the nuclear island are 6 inches total settlement and a differential settlement across the nuclear island foundation mat of one half inch in 50 feet. The Table 2.5.4-219 values **for the nuclear island** are within the**se** limits for total **and differential** settlement. ~~The values for differential settlement are within the limits for Case I and outside the limits for Case II. However, as noted above, the Table 2.5.4-219 values assume a flexible foundation, and the actual differential settlement across the thick reinforced mat foundation is negligible.~~ Table 2.0-201 also lists values of differential settlement between the nuclear island and surrounding structures as 3 inches. The difference between the estimated settlement of the nuclear island and the settlement of the surrounding structures in Table 2.5.4-219, i.e., the differential settlement, is within limits. As noted below, because of the nature of the soils and rock underlying the new units, post-construction settlement will be negligible.

Because the construction of each unit is over a period of greater than five years, the elastic settlement estimated in Table 2.5.4-219 is essentially complete prior to the start of operation of the unit. No time-dependent consolidation settlement is anticipated. Any additional settlement after completion is considered not significant.

DRAFT

Table 2.5.4-219 will be replaced in a future COLA revision with the following table:

Table 2.5.4-219

Estimated Foundation Settlements

Structure	Contact Pressure, (ksf)	Subsurface	B x L (ft)	Estimated Settlement (in.)		
				Center	Edge	Mean
Nuclear Island	8.9	Concrete Fill on Rock	(88 to 159) x 254	<0.15	<0.15	<0.15
Turbine	6.0	Compacted Fill	156 x 309	2.3	1.1	1.7
Annex	6.0	Compacted Fill	(66 to 145) x 405	2.5	1.3	1.9
Radwaste	6.0	Compacted Fill	66 x 175	2.3	1.3	1.8

DRAFT

ASSOCIATED ENCLOSURES:

None