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: 02.05.04-17 (eRAI 6006)

The calculation for "Site Response and Strain Compatible Properties Calculation" Rev. 001 describes the procedure used to calculate stresses in the liquefaction analysis. In accordance with NUREG-0800, Standard Review Plan, Chapter 2.5.4, "Stability of Subsurface Materials and Foundations," and Regulatory Guide (RG) 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites" please clarify the following regarding the methodology used to calculate the CSR (cyclic stress ratio):

a. Clarify how the method used for determining SR_{DRS} meets the ground motion level requirements for liquefaction analysis per 10 CFR 50, Appendix S. The *GRMS* initially resulted in a *PGA* of less than 0.1g and was scaled upwards per RG 1.208. Since the method used for determining the SR_{DRS} is the same as the *GRMS*, describe how this method provides stress ratio values that are comparable to those calculated using a *PGA* value of at least 0.1g.

b. Describe how the amplitude ratio $A_R(f)$, defined by ($ARS_{10\text{-}5}$)/($ARS_{10\text{-}4}$) and used in the determination of the weighting factor w, correlates to the ratio of the in-situ *stress ratios* resulting from site response analysis using the $ARS_{10\text{-}5}$ and $ARS_{10\text{-}4}$ as input spectrums.

c. The weighting factor *w* applied to the stress ratios $SR_{10\text{-}4}$ and $SR_{10\text{-}5}$ for the determination of SR_{DRS} is based on the average of the weighting factor *W(f).* Justify using an average value of *W(f)* over all frequencies, and describe how this is a conservative approach.

d. Describe how $ARS_{10\text{-}5}$ and $ARS_{10\text{-}4}$ are used as input to the *RVT* for site response, and how this approach correctly accounts for duration effects as compared to time series inputs for the determination of Cyclic Stress Ratio (*CSR)*. Please justify and provide the technical basis of this approach, including any assumptions.

e. Justify use of equations (77) and (78) from Idriss and Boulanger (2008) for determining q_{c1Ncs} values, and how the resulting values are conservative compared to the methods outlined in RG 1.198 using your calculated I_c values.

FPL RESPONSE:

Part a:

Clarify how the method used for determining SR_{DRS} meets the ground motion level requirements for liquefaction analysis per 10 CFR 50, Appendix S. The GRMS initially resulted in a PGA of less than 0.1g and was scaled upwards per RG 1.208. Since the method used for determining the SR_{DRS} is the same as the GRMS, describe how this method provides stress ratio values that are comparable to those calculated using a PGA value of at least 0.1g.

The factor of safety (FOS) against liquefaction is computed by dividing the strength (capacity) of the soil available to resist liquefaction (cyclic resistance ratio or CRR) by the stresses (demand) in the soil caused by the earthquake (cyclic stress ratio, CSR, or simply SR as used in the question). For Units 6 & 7, the evaluation of the soil strength was primarily based on cone penetration test (CPT) results. The methodology that was applied to the computation of FOS values in FSAR Subsection 2.5.4.8 utilized the CSR values obtained from the site-specific ground response (P-Shake) analysis. This RAI response re-evaluates the FOS against liquefaction by directly computing the CSR values using the Seed simplified equation (FSAR Subsection 2.5.4 Reference 219) with a peak ground acceleration (PGA) of 0.1g.

FSAR Figure 2.5.4-238, which is reproduced as Figure 1 in this response, is a compilation of the four CPT data sets and presents the FOS values that are based on the CSR values from P-Shake analysis as a function of elevation. The three lowest FOS values that correspond to the CPT measurements at three different elevations are selected from this figure for comparison purposes. The FOS of 1.92 at El. -137.7 ft, 2.11 at El. -231.5 ft, and 2.16 at El. -252.3 ft (NAVD 88) are tabulated below, along with their corresponding CSR values of 0.047, 0.043, and 0.043.

For re-evaluation purposes, the CSR values for the same data points are recalculated using a PGA of 0.1g. Using Equations 1, 2a and 2b of FSAR Subsection 2.5.4 Reference 219, the corresponding CSR values are computed as 0.060, 0.064, and 0.065, respectively. Thus, the CSR values based on a PGA of 0.1g are increased compared to those directly obtained from the site-specific P-Shake analysis. As a result, because there is an inversely proportional relationship between the CSR and FOS values, substituting the CSR values from 0.1g will reduce the corresponding FOS values. Thus, the FOS values computed using a PGA of 0.1g at El. -137.7 ft, -231.5 ft, and -252.3 ft are 1.50, 1.42, and 1.43, respectively. As indicated in FSAR Subsection 2.5.4.11, the minimum allowable FOS of 1.25 was conservatively selected as the trigger value for the liquefaction analysis of site soils. Given that RG 1.198 considers soils with a FOS value less than 1.1 as liquefiable, there is about 14% conservatism employed in the analysis (1.1 versus 1.25). Nonetheless, the FOS values of 1.50, 1.42, and 1.43 exceed the minimum allowable FOS of 1.25.

CPT	El. (ft)	P-Shake		PGA of 0.1g						
		CSR	FOS	Current depth (ft)	Finished depth (ft)	Total overburden pressure (ksf)	Effective overburden pressure (ksf)	Stress reduction coefficient r_d	CSR $a=0.1g$	FOS $a=0.1g$
C-601	-137.7	0.047	1.92	137.6	163.2	21.93	13.34	0.56	0.060	1.50
C-701	-231.5	0.043	2.11	230.1	257.0	33.21	18.76	0.56	0.064	1.42
C-701	-252.3	0.043	2.16	250.9	277.8	35.70	19.96	0.56	0.065	1.43

In summary, using a PGA of 0.1g in calculation of CSR values will reduce the FOS values against liquefaction. However, as observed above, the latter FOS values exceed the minimum allowable FOS of 1.25. Thus, no modifications are proposed to the approach presented in the FSAR.

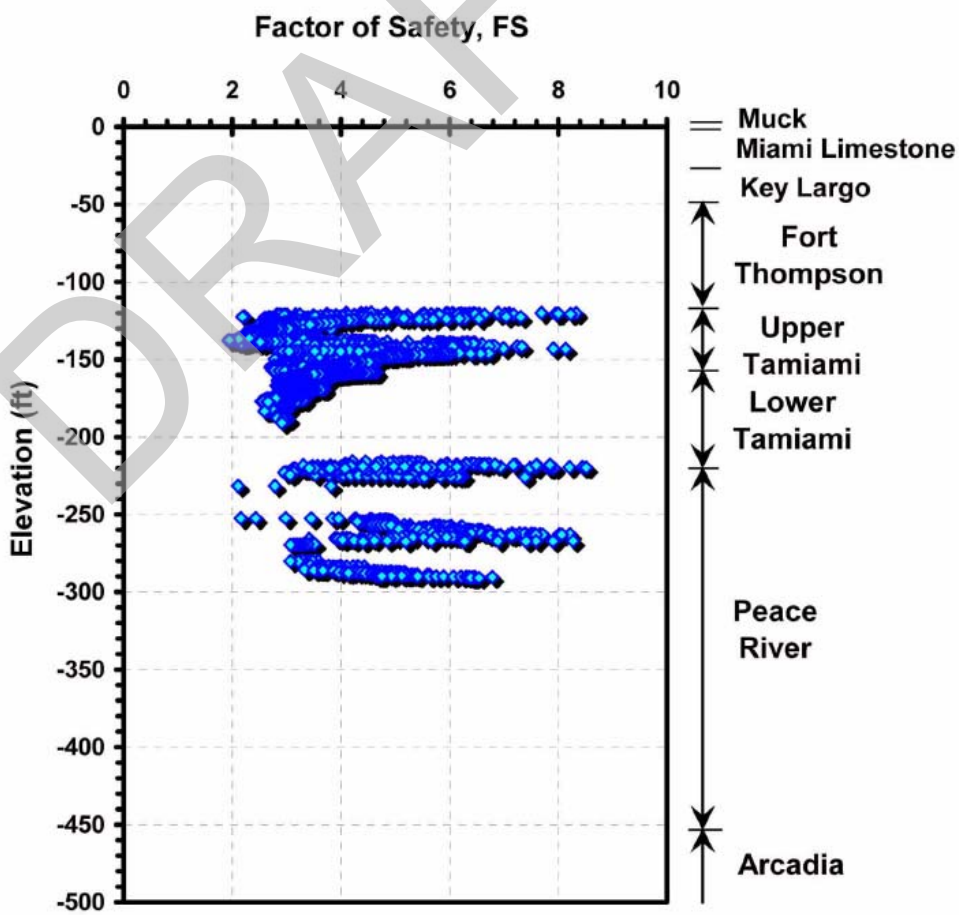

Figure 1
Factor of safety against liquefaction based on CPT values
(reproduced from FSAR Figure 2.5.4-238)

Part b:

Describe how the amplitude ratio $A_R(f)$, defined by (ARS $_{10-5}$)/(ARS $_{10-4}$) and used in the determination of the weighting factor w, correlates to the ratio of the in-situ stress ratios resulting from site response analysis using the ARS $_{10-5}$ and ARS $_{10-4}$ as input spectrums.

The weighting factor approach results in stress ratios that are compatible with the design response spectrum (DRS), that is larger than stress ratios compatible with 1E-4 seismic motion, and lower than stress ratios compatible with 1E-5 seismic motion. The amplitude ratio $A_R(f)$ correlates positively to the weighting factor $W(f)$ as shown in Figure 2. Stress ratios are calculated using the following equation:

$$SR_{DRS} = (SR_{10-4})^{1-\omega}(SR_{10-5})^{\omega}$$

Where ω is the average $W(f)$ over the entire frequency range. Note that the larger the value of ω the smaller the contribution of SR_{10-4} and the larger the contribution of SR_{10-5}. Therefore, the amplitude ratio A_R also correlates positively to the resulting stress ratios SR_{DRS}.

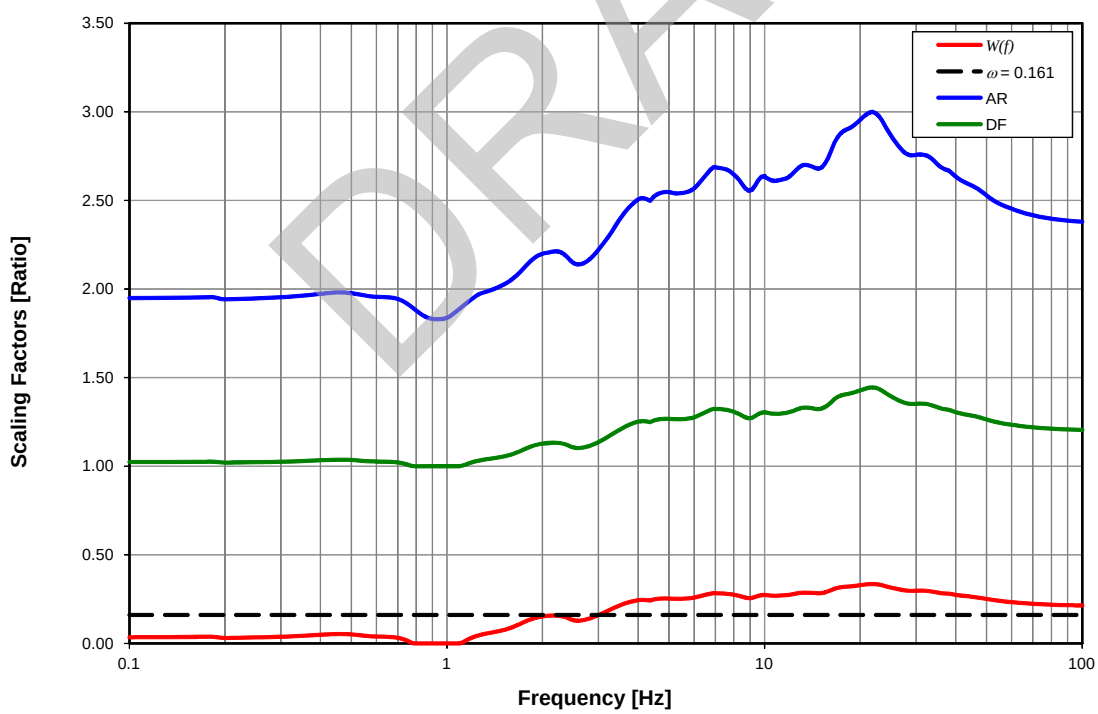

Figure 2

Part c:

The weighting factor w applied to the stress ratios SR_{10-4} and SR_{10-5} for the determination of SR_{DRS} is based on the average of the weighting factor W(f). Justify using an average value of W(f) over all frequencies, and describe how this is a conservative approach.

The average weighting factor ω is larger than the calculated frequency-dependent weighting factors below a frequency of 3 Hz, but smaller at frequencies larger than 3 Hz. To justify the adequacy of the adopted average weighting factor for the purpose of calculating conservative stress ratios for use in liquefaction analysis, the following was performed.

The site response analysis runs, using P-SHAKE, are repeated for the FAR soil column with an imposed cutoff frequency of analysis of 3 Hz, as opposed to the cutoff frequency of 100 Hz in the case of the original analysis. The analysis was performed for all 60 simulated profiles subjected to the low frequency (LF) and high frequency (HF) rock motions at the 1E-4 and 1E-5 hazard levels. The resulting stress ratios, which as in the case of the original analysis are the envelope of LF and HF results, are compared in Figure 3 at the 1E-4 and 1E-5 hazard levels. Noting that the top-most submerged granular soil layer, analyzed for potential liquefaction, is at a depth of around 145 ft (Elevation -120 ft), it follows that the contribution of the motion, with frequency content removed above 3Hz, is more than 90% of the total stress ratios for all layers.

From Figure 2, note that for frequencies below 3 Hz the calculated frequency dependent weighting factor is much smaller than the adopted weighting factor of 0.161 used in the stress ratio calculation, with a computed average weighting factor of 0.054 for frequencies below 3 Hz. Similarly, in the case of the near NI soil column, and for frequencies below 3 Hz, the average weighting factor is 0.060, which is smaller than the adopted weighting factor of 0.154 used in the stress ratio calculation.

It is therefore concluded that the adopted (larger) weighting factor is conservative and adequately compensates for any small contribution to the weighting factor from frequencies above 3 Hz, confirming the adequacy of the calculated stress ratios used in liquefaction analysis.

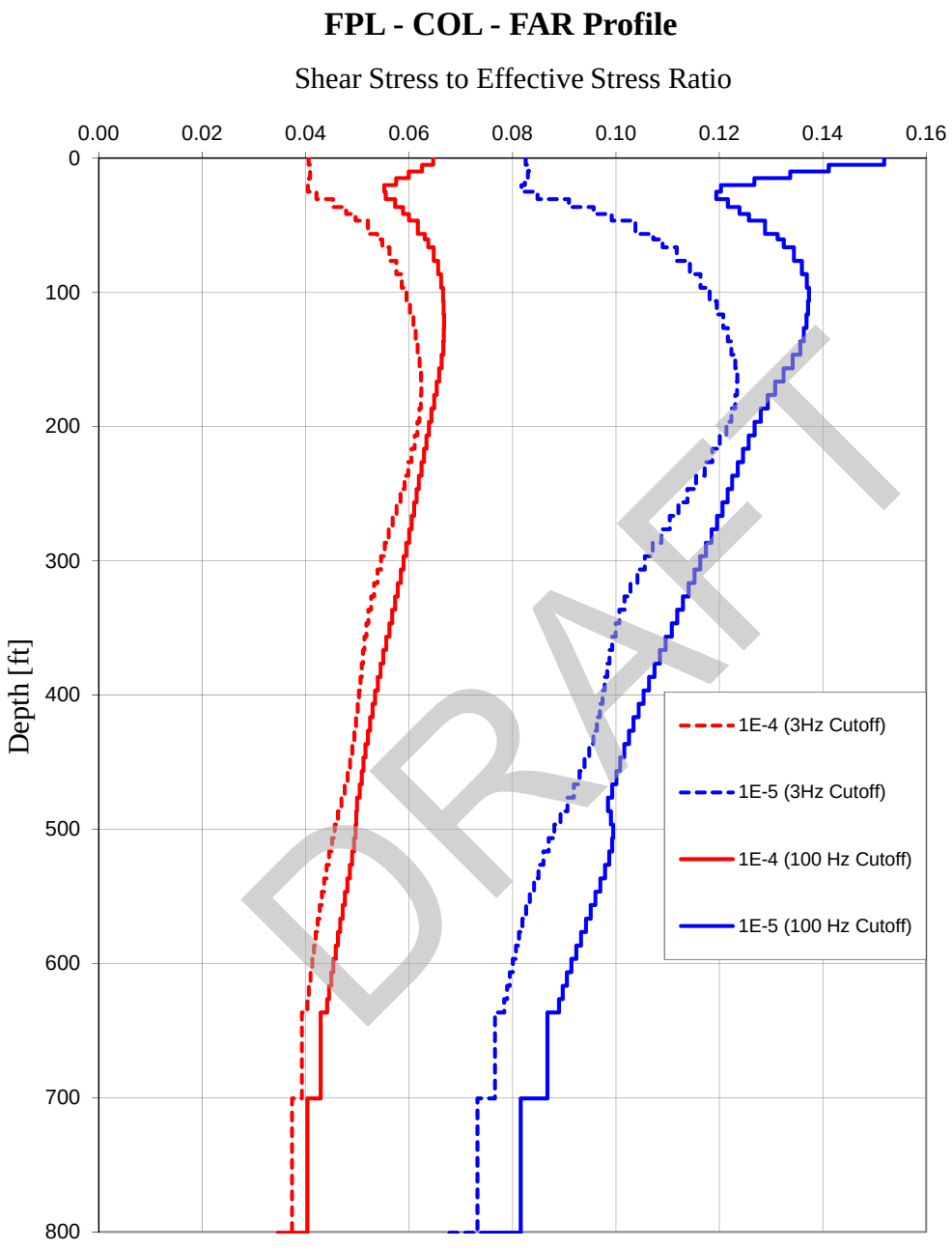
FPL - COL - FAR Profile
Shear Stress to Effective Stress Ratio
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
0
100
200
300
400
500
600
700
800
Depth [ft]
1E-4 (3Hz Cutoff)
1E-5 (3Hz Cutoff)
1E-4 (100 Hz Cutoff)
1E-5 (100 Hz Cutoff)
DRAFT

Figure 3

Part d:

Describe how ARS $_{10\text{-}5}$ and ARS $_{10\text{-}4}$ are used as input to the RVT for site response, and how this approach correctly accounts for duration effects as compared to time series inputs for the determination of Cyclic Stress Ratio (CSR). Please justify and provide the technical basis of this approach, including any assumptions.

The Random Vibration Theory (RVT) has been implemented in the Bechtel computer program P-SHAKE which is the new enhanced version of the program SHAKE2000. The program follows the same methodology and inherits the same assumptions used in the computer program SHAKE and seeks for the solution in frequency domain using the equivalent linear method to consider the soil nonlinear effects. Once the iteration on soil properties in each layer has converged, the final solution is obtained. The methodology has been checked and verified against the computer program SHAKE for a large suite of soil columns and input time histories. With respect to RVT implementation, the major steps used in P-SHAKE are as follows:

1. In RVT approach the input motion is provided in terms of acceleration response spectrum and its associated spectral damping. From the acceleration response spectrum, the acceleration power spectral density function is computed using the peak factor.
2. From the frequency domain solution of the soil profile (following SHAKE approach), the transfer function for strain in each layer is obtained and convolved with the power spectral density (PSD) of input motion to get the peak factor and the maximum strain in each layer. The equivalent uniform strain is obtained from the maximum strain and is used to obtain the new soil properties (soil shear modulus and damping) for the next iteration.
3. The iterations are repeated until convergence is reached in all layers to the convergence limit set by the user.
4. Once the final frequency domain solution is obtained, the acceleration response spectrum for each horizon can be computed from the solution using an inverse process of obtaining PSD from the acceleration response spectrum.

As discussed in FSAR Section 2.5.2.5.3, the duration of the input motion is specified as a parameter in P-SHAKE and is provided for different rock input motions in FSAR Table 2.5.2-226.

In addition, similar to the approach used in SHAKE, a ratio of 0.65 is used in P-SHAKE to calculate equivalent uniform strain, starting from maximum strain, which translates into the same ratio for the corresponding stress.

A technical paper describing the RVT approach used and the methodology for obtaining the peak factors can be found in Reference 1.

Part e:

Justify use of equations (77) and (78) from Idriss and Boulanger (2008) for determining q_{c1Ncs} values, and how the resulting values are conservative compared to the methods outlined in RG 1.198 using your calculated I_c values.

For Units 6 & 7, factor of safety (FOS) against liquefaction was based on primarily cone penetration test (CPT) results, which are less susceptible to soil disturbance (for example, hydraulic gradients) than standard penetration test results. The FOS against liquefaction is computed by dividing the strength (capacity) of the soil available to resist liquefaction (cyclic resistance ratio or CRR) by the stresses (demand) in the soil caused by the earthquake (cyclic stress ratio or CSR). The method by Youd et al. (FSAR Subsection 2.5.4 Reference 219) for performing liquefaction analysis referred to in RG 1.198 utilizes the "soil behavior type index (I_c)", which is a function of the tip resistance (q_c) and sleeve friction ratio (R_f), to account for the effect of fines content on the estimate of CRR values. A recent variation of this method is suggested by Idriss and Boulanger (FSAR Subsection 2.5.4 Reference 268). Equations (77) and (78) from Idriss and Boulanger take into account the actual fines content of the soil based on laboratory measurements from recovered samples. Utilizing actual measured fines content in lieu of estimates based on I_c is considered more appropriate and therefore this modification was incorporated into the Youd et al. method (equations (77) and (78) from Idriss and Boulanger) to compute normalized cone penetration resistance (q_{c1Ncs}). The analysis took into account the best estimate measured fines content for the Upper and Lower Tamiami, and the Peace River Formations. The resulting values were used in the computation of CRR. Although the method with equations (77) and (78) from Idriss and Boulanger may not generate more conservative FOS values compared to those using the Youd et al method, the Idriss and Boulanger approach reflects the actual site conditions better as it accounts for the effects of measured fines content.

In order to demonstrate that the site soils essentially have no liquefaction potential, the liquefaction evaluation was also performed based on the field measurements of shear wave velocity (V_S) using the approach by Youd et al. As indicated in FSAR Subsection 2.5.4.8.3, the V_S measurements, taken generally at 1.6 to 1.7 foot depth intervals, were used for the computation of FOS against liquefaction with a total of 878 points considered. According to the liquefaction resistance criteria suggested by FSAR Subsection 2.5.4 Reference 219, soils with V_S higher than the 200-215 m/s (656-705 ft/s) range (range based on the fines content) are considered non-liquefiable. FSAR Figure 2.5.4-218 shows that all of the measured V_S to depths of 400 and 600 ft at Units 6 & 7, respectively, exceed 705 ft/s, with only a few values below 1,000 ft/s. Based on these measurements, the site soils are expected to have no liquefaction potential. As FSAR Table 2.5.4-218 demonstrates, the FOS computed based on V_S exceeds the minimum allowable FOS of 1.25, which was conservatively selected as the trigger value for the liquefaction analysis of site soils (see Part a of this response).

In addition, liquefaction resistance increases markedly with geologic age. Youd et al. indicate that pre-Pleistocene sediments (sediments older than 1.6 million years) are generally immune to liquefaction. The Tamiami Formation is Pliocene (1.6 to 5.3 million years old) and the Peace River Formation is Pliocene-Miocene (1.6 to 23.7 million years old). FSAR Subsection 2.5.4 Reference 269 proposes an age correction factor, C_A, that accounts for the low probability of liquefaction of older deposits. Although this factor was not applied in the liquefaction analysis, it would be approximately 2 to 2.5; therefore, use of this factor would increase the calculated factors of safety against liquefaction by a factor of 2 to 2.5. Thus, no modifications are proposed to the methodology presented in the FSAR. The factor of safety values tabulated in Part a are considered to be conservative.

This response is PLANT SPECIFIC.

References:

1. Nan Deng and Farhang Ostadan, "Random Vibration Theory Based Seismic Site Response Analysis," The 14th World Conference on Earthquake Engineering, October 12-17, 2008, Beijing, China, Paper 04-02-0024.

ASSOCIATED COLA REVISIONS:

The second paragraph of FSAR Subsection 2.5.4.8.3 will be revised as follows in a future FSAR revision.

Table 2.5.4-218 is a summary of the results of the calculations. The native soils that indicate the lowest FOS values are those in the upper Tamiami Formation. However, the FOS values calculated indicate adequate resistance to liquefaction based on published criteria (FOS > 1.25). The FOS as a function of **elevation** ~~depth~~ for the CPT-based calculations is presented in Figure 2.5.4-238. As described above, even if liquefaction occurs, the thickness and stiffness of the overlying rock, lean concrete fill, and compacted limerock fill precludes the effects of liquefaction from reaching near the ground surface.

The footnote to FSAR Figure 2.5.4-238 will be deleted as follows in a future FSAR revision.

~~Data from Reference 257~~

ASSOCIATED ENCLOSURES:

None