Dominion Nuclear North Anna, LLC
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Dominion®

August 19, 2004

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
Washington, D.C. 20555

Serial No. 04-347A
ESP/JDH
Docket No. 52-008

DOMINION NUCLEAR NORTH ANNA, LLC
NORTH ANNA EARLY SITE PERMIT APPLICATION
SUPPLEMENTAL RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION NO. 5

In its June 1, 2004 letter titled "Request for Additional Information Letter No. 5," the NRC requested additional information regarding certain aspects of Dominion Nuclear North Anna, LLC's (Dominion) Early Site Permit application. This letter contains our responses to the following requests for additional information (RAIs):

2.5.4-8(c), 2.5.4-9, 2.5.4-10, 2.5.5-1

Also included in this letter are corrections to several tables in Site Safety Analysis Report Section 2.5 regarding the calculation of controlling earthquake distances.

It is our intent to update the North Anna ESP application to reflect our responses to these and other RAIs to support issuance of the NRC staff's draft safety and environmental evaluations scheduled for later this year. Planned changes to the application are identified following the response to each RAI.

If you have any questions or require additional information, please contact Mr. Joseph D. Hegner at 804-273-2770.

Very truly yours,

Eugene S. Grecheck
Vice President-Nuclear Support Services

Enclosures:
1. Supplemental Response to RAI Letter No. 5 and Correction of Controlling Earthquake Distances
2. Sample Liquefaction Analysis for Zone IIA Saprolite in Response to RAI 2.5.4-10, Part b)

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Commitments made in this letter:

1. Revise North Anna ESP application to reflect RAI responses.

cc: U. S. Nuclear Regulatory Commission, Region II
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Mr. Michael Scott
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Washington, D.C. 20555

Mr. M. T. Widmann
NRC Senior Resident Inspector
North Anna Power Station

COMMONWEALTH OF VIRGINIA

COUNTY OF HENRICO

The foregoing document was acknowledged before me, in and for the County and Commonwealth aforesaid, today by Eugene S. Grecheck, who is Vice President, Nuclear Support Services, of Dominion Nuclear North Anna, LLC. He has affirmed before me that he is duly authorized to execute and file the foregoing document on behalf of Dominion Nuclear North Anna, LLC, and that the statements in the document are true to the best of his knowledge and belief.

Acknowledged before me this 19TH day of August, 2004

My Commission expires: May 31, 2006

Vicki L. Hull
Notary Public

(SEAL)

Enclosure 1

Supplemental Response to RAI Letter No. 5 and Correction of Controlling Earthquake Distances

RAI 2.5.4-8 (6/1/04 NRC Letter)

SSAR Subsection 2.5.4.7.2 (Variation of Shear Modulus and Damping with Strain) describes the shear modulus and damping ratio curves for Zone IIA saprolite (improved and unimproved), Zone IIB saprolite, and Zone III rock. With regard to this subsection:

a) Please provide the basis for the selected modulus reduction curves for Zone IIA saprolite, Zone IIB saprolite, and Zone III weathered rock.

b) Please explain the basis for the selected damping ratio curves for Zone IIA saprolite, Zone IIB saprolite and Zone III weathered rock.

c) Please explain the use of a damping ratio of 2% for the Zone III-IV rock.

<u>Response</u>

a) The response to Part a) was provided in Reference 1.

b) The response to Part b) was provided in Reference 1.

c) The damping ratio for rock varies widely from site to site depending on various factors, including the mineral composition of the rock, the integrity and fissuring of the rock mass, the level of shear deformation in the rock formation, etc. A range of damping ratios from 0.4% to 4.6% has been reported for rock (Schnabel et al 1972), covering a wide range of shear strain levels from 0.0001% to 1%. Based on engineering judgment and past experience of similar sites, the Zone III-IV rock for the North Anna ESP site was specified to be 2% damping for the SHAKE analyses described in SSAR Section 2.5.4.7.4. It was considered reasonable for the site since the soil layer above the rock bed is relatively thin, and a certain degree of weathering was observed in the rock.

The rock damping value has minimal effect on the results of the soil column analysis for the North Anna ESP site. To demonstrate this, additional parametric SHAKE runs were performed using 5%, 1%, and 0.5% damping ratios for the Zone III-IV rock. The runs used Profile 1 in SSAR Table 2.5-46. The SHAKE analysis used the V_s values listed for Profile 1 in SSAR Table 2.5-46 and SSAR Figure 2.5-63 for variation of normalized shear modulus with cycle shear strain.

Table 1 shows the numerical results (maximum acceleration (or ZPA) versus depth) from the additional SHAKE runs compared to the results using a 2% rock damping ratio. As shown in the table, the differences are negligible. The maximum acceleration data are plotted for comparison in Figure 1(a) and Figure 1(b), for low and high frequency response spectra respectively. (Table 1 and Figures 1(a), 1(b), 2(a), and 2(b) are located at the end of this RAI response.)

The calculated response spectra at the ground surface using different rock damping ratios are compared in Figure 2(a) and 2(b). Again, the differences shown are negligible.

Based on this sensitivity study, it is concluded that the effect of rock material damping on the design motion in the context of soil column analysis is negligible, and there is no need to develop a site-specific rock damping value for the purpose of soil column analysis.

References

1. August 5, 2004 Letter from Eugene S. Grecheck, Vice President-Nuclear Support Services, Dominion, to U.S. Nuclear Regulatory Commission, Document Control Desk, "Dominion Nuclear North Anna, LLC, North Anna Early Site Permit Application, Response to Request for Additional Information No. 5, Serial No. 04-347."

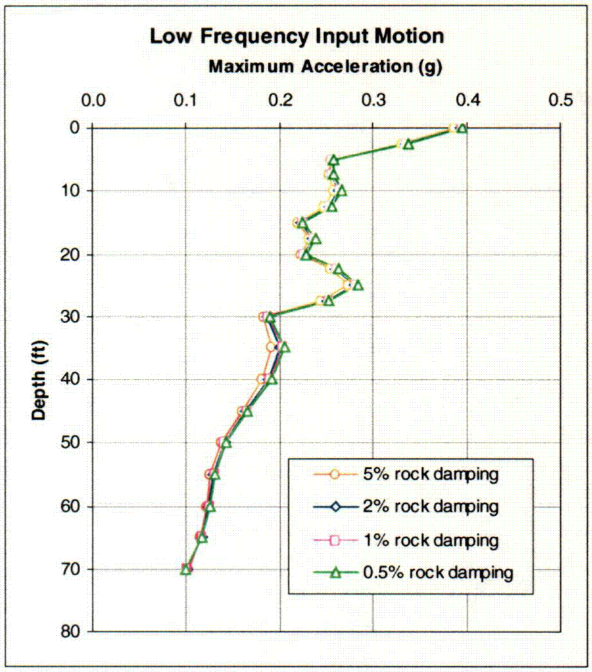
2. Schnabel, P.B., J. Lysmer, and H.B. Seed. "SHAKE – A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites," Report No. UCB/EERC-72/12, Earthquake Engineering Research Center, University of California, Berkeley, December, 1972.

Application Revision

None.

Table 1. Sensitivity of Maximum Accelerations to Rock Damping Assumption

Layer	Depth, ft	Maximum Acceleration, g, for rock damping: Low Frequency Time History 5%	2%	1%	0.5%	High Frequency Time History 5%	2%	1%	0.5%
SURFACE	0.0	0.39	0.39	0.39	0.40	0.79	0.80	0.80	0.80
WITHIN	2.5	0.33	0.33	0.34	0.34	0.72	0.73	0.73	0.74
WITHIN	5.0	0.25	0.26	0.26	0.26	0.49	0.50	0.50	0.50
WITHIN	7.5	0.25	0.26	0.26	0.26	0.47	0.48	0.49	0.49
WITHIN	10.0	0.26	0.26	0.26	0.27	0.49	0.50	0.51	0.51
WITHIN	12.5	0.25	0.25	0.25	0.26	0.45	0.46	0.46	0.46
WITHIN	15.0	0.22	0.22	0.22	0.22	0.50	0.51	0.51	0.51
WITHIN	17.5	0.23	0.24	0.24	0.24	0.48	0.49	0.49	0.49
WITHIN	20.0	0.22	0.23	0.23	0.23	0.51	0.51	0.51	0.51
WITHIN	22.5	0.25	0.26	0.26	0.26	0.54	0.55	0.56	0.56
WITHIN	25.0	0.27	0.28	0.28	0.28	0.58	0.59	0.59	0.60
WITHIN	27.5	0.24	0.25	0.25	0.25	0.57	0.58	0.58	0.58
WITHIN	30.0	0.18	0.19	0.19	0.19	0.49	0.50	0.50	0.50
WITHIN	35.0	0.19	0.20	0.20	0.21	0.49	0.51	0.51	0.51
WITHIN	40.0	0.18	0.19	0.19	0.19	0.50	0.51	0.51	0.51
WITHIN	45.0	0.16	0.16	0.17	0.17	0.44	0.45	0.45	0.45
WITHIN	50.0	0.14	0.14	0.14	0.14	0.41	0.42	0.43	0.43
WITHIN	55.0	0.13	0.13	0.13	0.13	0.32	0.32	0.32	0.33
WITHIN	60.0	0.12	0.12	0.12	0.12	0.33	0.34	0.34	0.34
WITHIN	65.0	0.12	0.12	0.12	0.12	0.33	0.33	0.33	0.33
WITHIN	70.0	0.10	0.10	0.10	0.10	0.28	0.28	0.28	0.28
OUTCROP	70.0	0.15	0.15	0.15	0.15	0.39	0.39	0.39	0.39



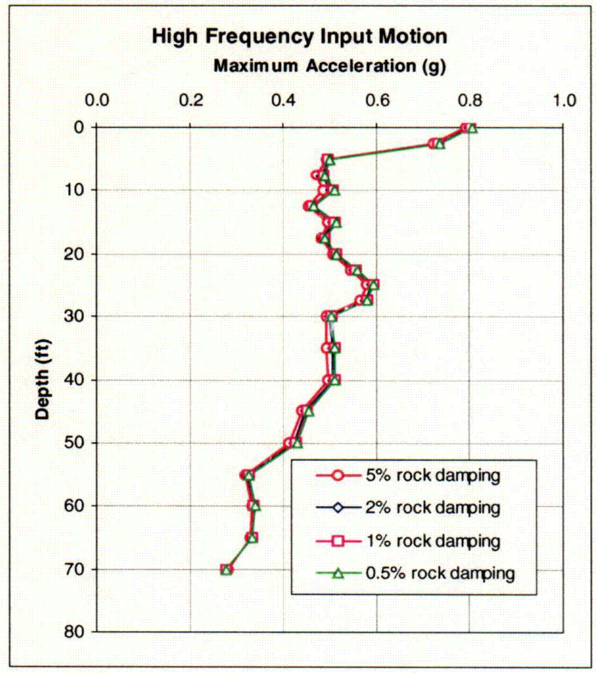


Figure 1(a) and Figure 1(b), for low and high frequency response spectra, respectively.

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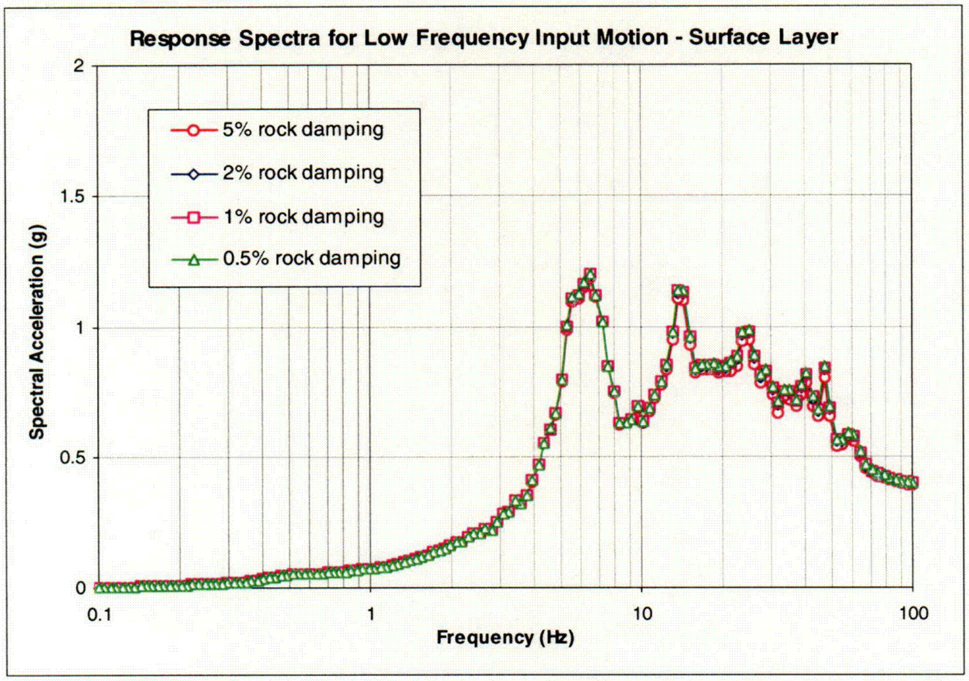

Figure 2(a). Response spectra comparison for the low frequency time history

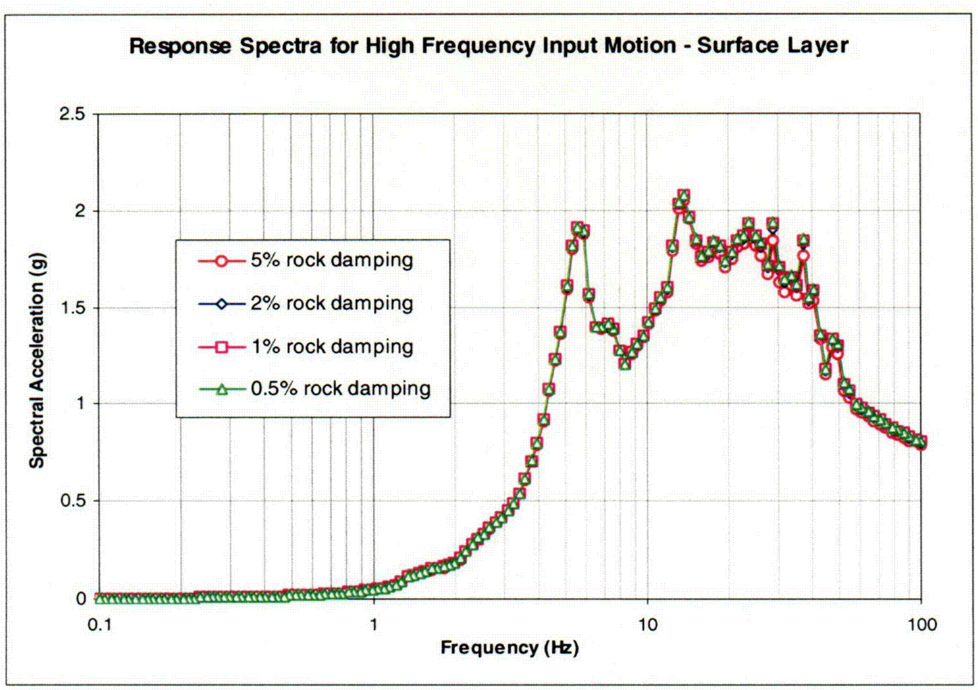

Figure 2(b). Response spectra comparison for the high frequency time history.
RAI 2.5.4-9 (6/1/04 NRC Letter)

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Please elaborate further on the method used for the development of the site-specific acceleration time histories which are briefly described in SSAR Subsection 2.5.4.7.3 and 2.5.4.7.4. Also, please provide a description of the subsurface model, showing layer thickness and geotechnical properties for each layer. Please describe how the variability in each of these engineering properties was accounted for in the development of the site-specific ground motion. Finally, please justify the use of the mean 10^{-4} Uniform Hazard Spectrum (UHS) ground motion as the input rock motion.

Response

1. Method Used For the Development of Site-Specific Acceleration Time Histories Described in SSAR Section 2.5.4.7.3

Two horizontal-component acceleration time histories were developed to be spectrum-compatible for use in the soil column amplification analysis. The spectral matching computer program that was used was written by Abrahamson (1993) and is based on the methodology developed by Lilhanand and Tseng (1988). This is a time-domain based procedure that takes a given input acceleration time history and makes it compatible with a given target acceleration response spectrum. The modification of a time history can be performed with a variety of different modification models. In doing so, the long period non-stationary phasing of the original time history is preserved.

Two target spectra were used in the analysis. Spectra developed to represent the 5-to-10 Hz high frequency and 1-to-2.5 Hz low frequency $5x10^{-5}$ mean hazard level ground motions were used along with the SSAR performance-based spectrum to develop hybrid high- and low-frequency spectra, the envelope of which replicates the performance-based spectrum itself. These horizontal acceleration target response spectra (5% spectral damping) are defined for the frequency range of 100 Hz to 0.1 Hz.

The selection of the two initial seed input time histories with the correct magnitude and distance for the spectral matching procedure was based on the deaggregation information from the probabilistic seismic hazard analysis. These selected seed input time histories were taken from the time history database of Central and Eastern United States (CEUS) time histories provided as an appendix to NUREG/CR-6728 (McGuire et al 2001). For the high frequency case, the 180-degree horizontal component from the San Ramon-Kodak station from the 1980 Livermore, California, earthquake was selected as the seed input time history. This earthquake had a magnitude 5.4 and was recorded at a rupture distance of 17.6 km. These magnitude and distance values are in good agreement with the North Anna ESP 5-10 Hz (i.e., high frequency range) deaggregation values of M = 5.4 and D = 20 km, respectively. For the low frequency range, the selected initial seed input time history was the longitudinal component from the Kashmar station from the 1978 Tabas, Iran earthquake. This earthquake had a magnitude 7.4 and the Kashmar station was at a rupture distance of 199.1 km. The deaggregation information for the 1-2.5 Hz (i.e., low frequency range) from the PSHA provides magnitude and distance values of 7.2 and 308 km, respectively. The magnitude value of the selected initial time history is similar to the deaggregation magnitude value of 7.2; however, the distance of 199.1 km is smaller than the deaggregation distance of 308 km. This limitation in distance was based on the limited distance of available candidate seed time histories from the NUREG/CR-6728 database. Time histories are only provided for the distance range of 100 – 200 km.

The spectral matching criteria presented in NUREG/CR-6728 were followed in the development of the time histories for the frequency range of 100 Hz to 0.5 Hz. These final spectrum-compatible time histories were used for the site response analysis.

Comparison plots of the target and matched time history spectra are provided in Figures 1 and 2 for the high frequency and low frequency cases, respectively. (Figures are located at the end of this RAI response.)

2. <u>Method Used For the Development of the Soil Column Amplification Analysis Described in SSAR Section 2.5.7.4</u>

The SHAKE2000 computer program was used to compute the site dynamic responses for the soil and rock profiles described in SSAR Section 2.5.4.7.1. The computation was performed in the frequency domain using the complex response method. The analysis used the acceleration-time histories described in SSAR Section 2.5.4.7.3 and Section 1 of this response. Two earthquakes were modeled: the low frequency case with a moment magnitude of 7.2 and an acceleration at bedrock level of 0.15g; and the high frequency case with a moment magnitude of 5.4 and an acceleration at bedrock level of 0.39g. The top of bedrock was at 70 feet depth.

SHAKE2000 uses an equivalent linear procedure to account for the non-linearity of the soil and weathered rock by employing an iterative procedure to obtain values for shear modulus and damping that are compatible with the equivalent uniform strain induced in each sublayer. At the outset of the analysis, a set of properties (based on the values of shear modulus and damping presented in SSAR Section 2.5.4.7.1, and total unit weight) was assigned to each sublayer of the soil and rock profile. The analysis was conducted using these properties, and the shear strain induced in each sublayer was calculated. The shear modulus and damping ratio for each sublayer was then modified based on the shear modulus and damping ratio versus strain relationships presented in SSAR Section 2.5.4.7.2. The analysis was repeated until strain-compatible modulus and damping values were achieved.

2.1 Input Soil Parameters

The four soil profiles used in the analysis are characterized in SSAR Table 2.5-46. The input dynamic properties (shear modulus, damping, and total unit weight) are developed first. Table 1 shows the current (unimproved) soil profile for the project (Profile 1 in SSAR Table 2.5-46). Table 2 shows the soil profile with improved Zone IIA dynamic properties due to ground improvement (Profile 4 in SSAR Table 2.5-46). Profile 2 in SSAR Table 2.5-46 is Profile 1 with the top 30 feet removed, i.e., top layer is the Zone IIB saprolite. Profile 3 in SSAR Table 2.5-46 is Profile 1 with the top 40 feet removed, i.e., top layer is the Zone III weathered rock. All of the profiles are considered to be free-field.

The groundwater table is assumed to be located at a depth of 10 feet in Profiles 1 and 4 and at ground surface in Profiles 2 and 3.

Table 1. Unimproved Soil (Profile 1)

Zone	Depth, Ft	Design Parameters		
		Unit Weight, pcf	V_s, ft/sec	G_{max}, ksf
IIA	0-10	125	700	1,900
IIA	10-20	125	950	3,500
IIA	20-30	125	1,200	5,600
IIB	30-40	130	1,600	10,000
III	40-55	145	2,000	18,000
III-IV	55-70	163	3,300	54,000
IV	Below 70	163	6,300	201,000

Table 2. Improved Soil (Profile 2)

Zone	Depth, Ft	Design Parameters		
		Unit Weight, pcf	V_s, ft/sec	G_{max}, ksf
IIA	0-10	130	1,275	6,600
IIA	10-20	130	1,380	7,700
IIA	20-30	130	1,500	9,000
IIB	30-40	130	1,600	10,000
III	40-55	145	2,000	18,000
III-IV	55-70	163	3,300	54,000
IV	Below 70	163	6,300	201,000

The input motion in the SHAKE analysis is placed as an output motion at the top of the Zone IV bedrock at 70 feet depth.

The modulus reduction and damping ratio versus shear strain design curves used in the SHAKE analyses are discussed at length in the response to RAI 2.5.4-8, and are given in SSAR Figures 2.5-63 and 2.5-64, respectively.

The Zone III-IV rock below 55 feet depth is assumed to be elastic for the SHAKE analysis. The damping ratio for the rock is estimated to be approximately 2% for the range of shear strains, and has minimal effect on the analysis, as discussed in the response to RAI 2.5.4-8 Part c).

2.2 Input Object Motion

The input object motion described in detail in Section 1 of this response was assigned as outcropping at 70 feet depth from the original ground surface for each profile. A maximum cut-off frequency of 100 Hz was specified.

2.3 SHAKE Run Control

Eight iterations to compute strain-compatible modulus and damping values were specified in the SHAKE analyses. Convergence to strain-compatible properties is achieved in 5 to 8 iterations for most soil profiles. For the analysis, the equivalent uniform strain divided by the maximum strain is estimated to be 0.65. The simplified stress calculation approach is specified.

2.4 Acceleration and Response Spectra Outputs

The maximum accelerations and response spectra were computed for each SHAKE analysis case. The horizontal acceleration versus depth results for both the high frequency and low frequency cases are tabulated for each profile on SSAR Table 2.5-46. Horizontal acceleration response spectra (ARS) for 5% damping were reported for the outcropping layer and the base rock of each soil profile at 140 equally spaced points at periods ranging from 0.01 to 10 seconds (0.1 to 100 Hz). The horizontal response spectra were calculated using a constant time-step in the acceleration time history.

As can be seen in SSAR Table 2.5-46, the high frequency time history gives much larger accelerations than the low frequency case. For Profile 1, Figure 3 shows the zero period acceleration (ZPA) variation with depth, and Figure 4 shows the horizontal ARS, both obtained with the SHAKE runs using the V_s values given on SSAR Table 2.5-46, and using the high frequency time history. The relevant curves in Figures 3 and 4 are the solid lines marked BE(G_{max}). BE(G_{max}) stands for best estimate of G_{max}, the low strain shear modulus derived from V_s. The other curves in Figures 3 and 4 are described in Section 4 of this response.

3. Description of Subsurface Model Showing Layer Thickness and Geotechnical Properties For Each Layer

3.1 Variation in Stratum Thickness and Depth to Bedrock

The strata at the North Anna site (defined as Zones I through IV in SSAR Section 2.5.4.2.2) are consistent throughout the site, i.e., there are no strata other than these zones (except for fill materials) and the sequence typically occurs as Zone I at the top down to Zone IV at the bottom. However, the thickness of the zones throughout the site varies considerably due to natural depositional or erosional variations, or previous excavation activities. For example, within the ESP plant parameter envelope, the range of stratum thickness measured in the 5 ESP borings was:

Thickness of Strata in ESP Plant Envelope, ESP Borings

Zone	Range, feet	Median, feet	Average, feet
I	0 to 2	0	0.4
IIA	0 to 31	21	16
IIB	0 to 5	0	1
III	1 to 18	11	9
III-IV	2 to 37	2	11

The depths and elevations at which the Zone IV bedrock occurred are:

Depth and Elevation of Zone IV Bedrock, ESP Borings

Zone IV	Range, feet	Median, feet	Average, feet
Depth	20 to 76	36	41
Elevation	195 to 284	243	239

If the 32 borings from Units 1 & 2 and abandoned Units 3 & 4 that fall within the ESP plant parameter envelope are considered, then the thickness range becomes even larger, i.e.:

Thickness of Strata in ESP Plant Envelope, non-ESP Borings

Zone	Range, feet	Median, feet	Average, feet
I	0 to 10	0	1
IIA	0 to 70	24	24
IIB	0 to 10	0	3
III	0 to 39	8	11
III-IV	0 to 74	7	12

The depths and elevations at which the Zone IV bedrock occurred in the 32 borings from Units 1 & 2 and abandoned Units 3 & 4 that fall within the ESP plant parameter envelope are:

Depth and Elevation of Zone IV Bedrock, non-ESP Borings

Zone IV	Range, feet	Median, feet	Average, feet
Depth	10 to 82	56	41
Elevation	190 to 273	234	232

Thus, throughout the ESP plant parameter envelope, there could be many combinations of strata thickness. The objective of the soil column amplification/attenuation analysis for the ESP was to select a profile that falls within the range of each stratum and which will provide conservative results. A conservative result in this situation is interpreted as a higher amplification. Experience indicates that higher amplifications are generally achieved with a thicker soil column. The total thickness of the soil column (including weathered rock) was chosen as 70 feet. From the above tables, this is close to the maximum depth to rock in the 37 borings.

3.2 Soil Profiles Used in Soil Column Amplification/Attenuation Analyses

The new reactor buildings would be founded on Zone III-IV or Zone IV bedrock, along with the majority of the other major safety-related structures. However, it is anticipated that some safety-related structures (diesel generator building, certain pump structures, tanks, etc.), would be founded on strata above the bedrock, i.e., on the Zone III weathered rock, or the Zone IIA or Zone IIB saprolite. Note that the Zone IIA saprolite would be improved prior to any safety-related structures being founded on that stratum.

Once the locations of structures to be founded on improved Zone IIA saprolite, Zone IIB saprolite, and Zone III weathered rock are known during detailed engineering, structure-specific subsurface investigations would be performed to determine actual strata thickness at each location, and confirm the material properties. Soil column amplification/attenuation analyses would be performed for the structure-specific locations and described in the COL application.

For the SSAR, analyses were conducted for four subsurface profiles:

- Profile 1 is the full-depth soil profile (70 feet) with no improvement to the Zone IIA saprolite.
- Profile 2 has the Zone IIA saprolite removed, i.e., this is the profile for structures founded on the Zone IIB saprolite.

- Profile 3 has both the Zone IIA and Zone IIB saprolite removed, i.e., this is the profile for structures founded on the Zone III weathered rock.
- Profile 4 is the same as Profile 1 except the Zone IIA saprolite properties reflect soil improvement. This is the profile for safety-related structures founded on the improved Zone IIA saprolite.

These four profiles, tabulated below, represent a conservative assessment of expected conditions at the ESP site based on existing information.

The groundwater table was assumed to be located at a depth of 10 feet in Profiles 1 and 4 and at ground surface in Profiles 2 and 3. The at-rest coefficient of lateral earth pressure, K_o, equals 0.45.

In the following tables:

V_s = shear wave velocity
G_{max} = low strain shear modulus

Profile 1					
			Design Parameters		
Zone	Material Type	Depth, Ft	Unit Weight, pcf	V_s, ft/sec	G_{max}, ksf
IIA	Sand	0-10	125	700	1,900
IIA	Sand	10-20	125	950	3,500
IIA	Sand	20-30	125	1,200	5,600
IIB	Gravel	30-40	130	1,600	10,000
III	Weathered Rock	40-55	145	2,000	18,000
III-IV	Rock	55-70	163	3,300	54,000
IV	Rock	Below 70	163	6,300	201,000

Profile 2					
			Design Parameters		
Zone	Material Type	Depth, Ft	Unit Weight, pcf	V_s, ft/sec	G_{max}, ksf
IIB	Gravel	0-10	130	1,600	10,000
III	Weathered Rock	10-25	145	2,000	18,000
III-IV	Rock	25-40	163	3,300	54,000
IV	Rock	Below 40	163	6,300	201,000

Profile 3					
			Design Parameters		
Zone	Material Type	Depth, Ft	Unit Weight, pcf	V_s, ft/sec	G_{max}, ksf
III	Weathered Rock	0-15	145	2,000	18,000
III-IV	Rock	15-30	163	3,300	54,000
IV	Rock	Below 30	163	6,300	201,000

Profile 4					
			Design Parameters		
Zone	Material Type	Depth, Ft	Unit Weight, pcf	V_s, ft/sec	G_{max}, ksf
IIA	Sand	0-10	130	1,275	6,600
IIA	Sand	10-20	130	1,380	7,700
IIA	Sand	20-30	130	1,500	9,000
IIB	Gravel	30-40	130	1,600	10,000
III	Weathered Rock	40-55	145	2,000	18,000
III-IV	Rock	55-70	163	3,300	54,000
IV	Rock	Below 70	163	6,300	201,000

In all of the profiles, the modulus reduction curves used were:

- Curve 1 in the response to RAI 2.5.4-8 Part a) for the Zone IIA saprolite, both unimproved and improved.
- Curve 2 in the response to RAI 2.5.4-8 Part a) for the Zone IIB saprolite.
- Curve 3 in the response to RAI 2.5.4-8 Part a) for the Zone III weathered rock.

In all of the profiles, the damping ratio curves used were:

- Curve 1 in the response to RAI 2.5.4-8 Part b) for the Zone IIA saprolite, both unimproved and improved.
- Curve 2 in the response to RAI 2.5.4-8 Part b) for the Zone IIB saprolite.
- Curve 3 in the response to RAI 2.5.4-8 Part b) for the Zone III weathered rock.

As noted above, structure-specific subsurface investigations would be performed during detailed engineering to determine actual strata thickness at each location, and soil column amplification/attenuation analyses would be run for the structure-specific locations. Thus, Profiles 2, 3, and 4 would be modified and re-run during detailed engineering. As noted in SSAR Section 2.5.4.8, the unimproved Zone IIA saprolite is the only onsite soil with liquefaction potential. Since Profile 1 is the profile that contains unimproved Zone IIA saprolite, it is the profile that was used to develop the peak ground acceleration for the ESP liquefaction analysis (SSAR Section 2.5.4.9), and the acceleration versus depth profile used in ESP slope stability analysis (SSAR Section 2.5.5). Thus Profile 1 was looked at in more detail when varying soil parameters, as described in Section 4 of this response.

4. Description of How the Variability of Engineering Properties was Accounted for in the Development of the Site-Specific Ground Motion

The engineering property that has the most impact on the amplification/attenuation analysis is the shear wave velocity, V_s, (or the low strain shear modulus, G_{max}, that is derived from V_s). After the initial SHAKE runs were made using the soil and rock parameters tabulated in Section 3 of this response, the values of G_{max} were varied to determine the impact on the acceleration response spectrum (ARS) and the maximum acceleration (or zero period acceleration, ZPA).

The original G_{max} for each layer tabulated in Profile 1 was multiplied by 1.5 (150% G_{max}) and divided by 1.5 (67% G_{max}) to account for uncertainty in the soil parameters. This was based on the guidelines in ASCE (2000) for a site where sufficient and adequate soil investigation data are available. Figure 3 shows the zero period acceleration (ZPA) variation with depth, and Figure 4 shows the horizontal ARS for Profile 1 for 150% G_{max} and 67% G_{max}, using the high frequency earthquake motion. The plots on Figures 3 and 4 include the results using the original G_{max} values for comparison. Figure 4 also includes the bedrock ARS. The maximum acceleration at the ground surface is 0.89g, obtained using 150% G_{max}.

The unit weight values and the modulus and damping versus strain curves were not varied in the analyses.

5. Justification of Use of the Mean 10^{-4} Uniform Hazard Spectrum (UHS) Ground Motion as Input Rock Motion

Initial calculations of liquefaction potential and slope stability were performed using a time history whose response spectrum matched a target spectrum with a mean annual probability of exceedance of 10^{-4}.

These initial calculations have been supplemented by calculations (see the responses to RAIs 2.5.4-10 and 2.5.5-1) that use the two time histories discussed in Section 1 of this response. These two time histories, in composite, conservatively match and/or exceed the site-specific performance-based spectrum and/or the envelope of the site-specific low and high frequency $5x10^{-5}$ mean hazard spectra.

References

McGuire, R. K., W. J. Silva, and C. J. Constantino. Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard and Risk-Consistent Ground Motion Spectra Guidelines, NUREG/CR6728, October, 2001 (Reference 171 of SSAR Section 2.5).

Abrahamson, N. A. "Computer Program for the Time Domain Development of Spectrum Compatible Time Histories," FORTRAN Computer Program, 1993.

Lilhanand, K. and W. S. Tseng. "Development and Application of Realistic Earthquake Time Histories Compatible with Multiple-Damping Design Spectra," Proc. 9th World Conference on Earthquake Engineering, Tokyo-Kyoto Japan, Vol. (II), 1988.

ASCE. "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," ASCE 4-98, Reston, VA, May 2000.

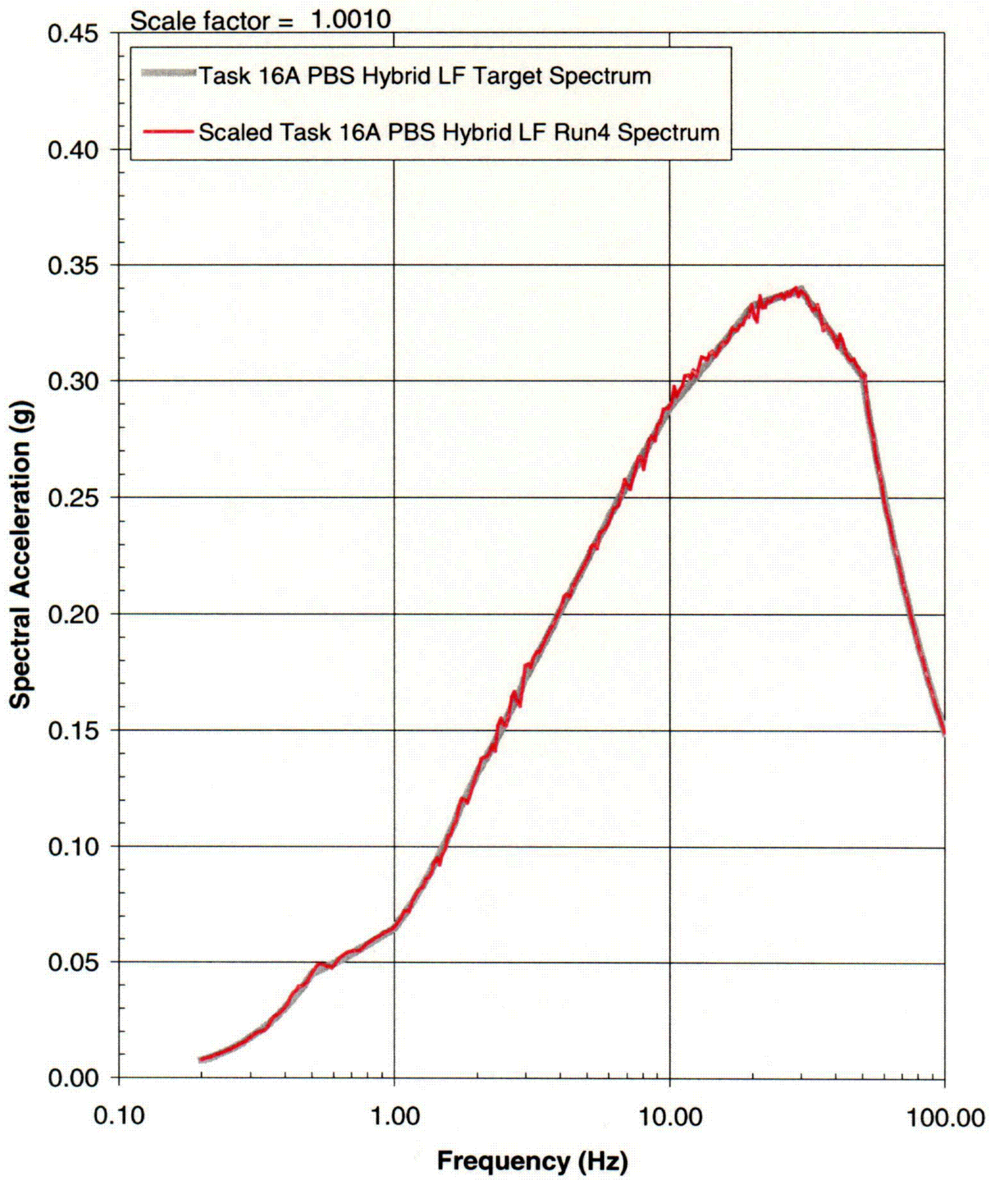

Figure 1. Comparison between the low frequency time history and the scaled Hybrid-LF target spectra

C03

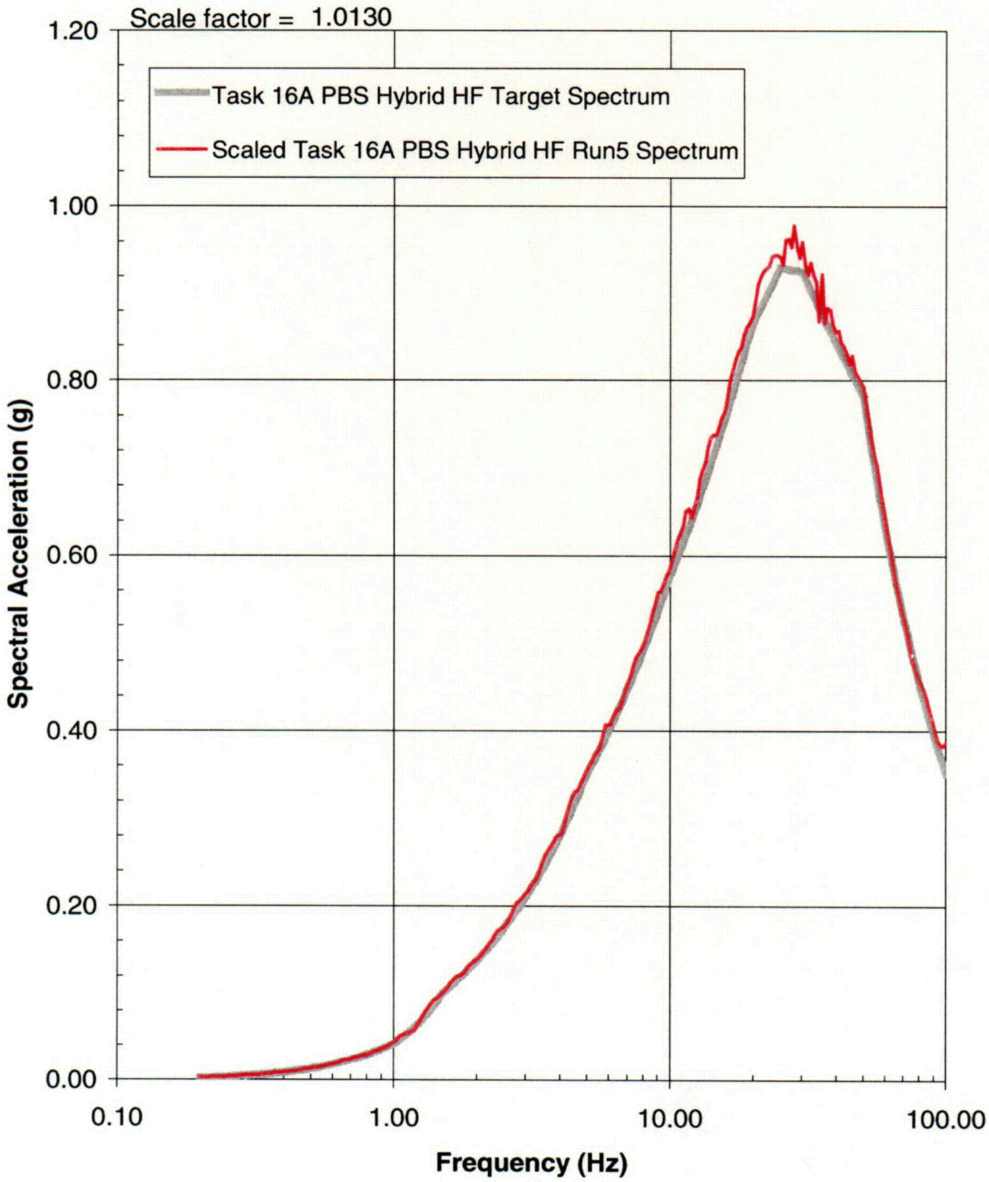

Figure 2. Comparison between the high frequency time history and the scaled Hybrid-HF target spectra

C04

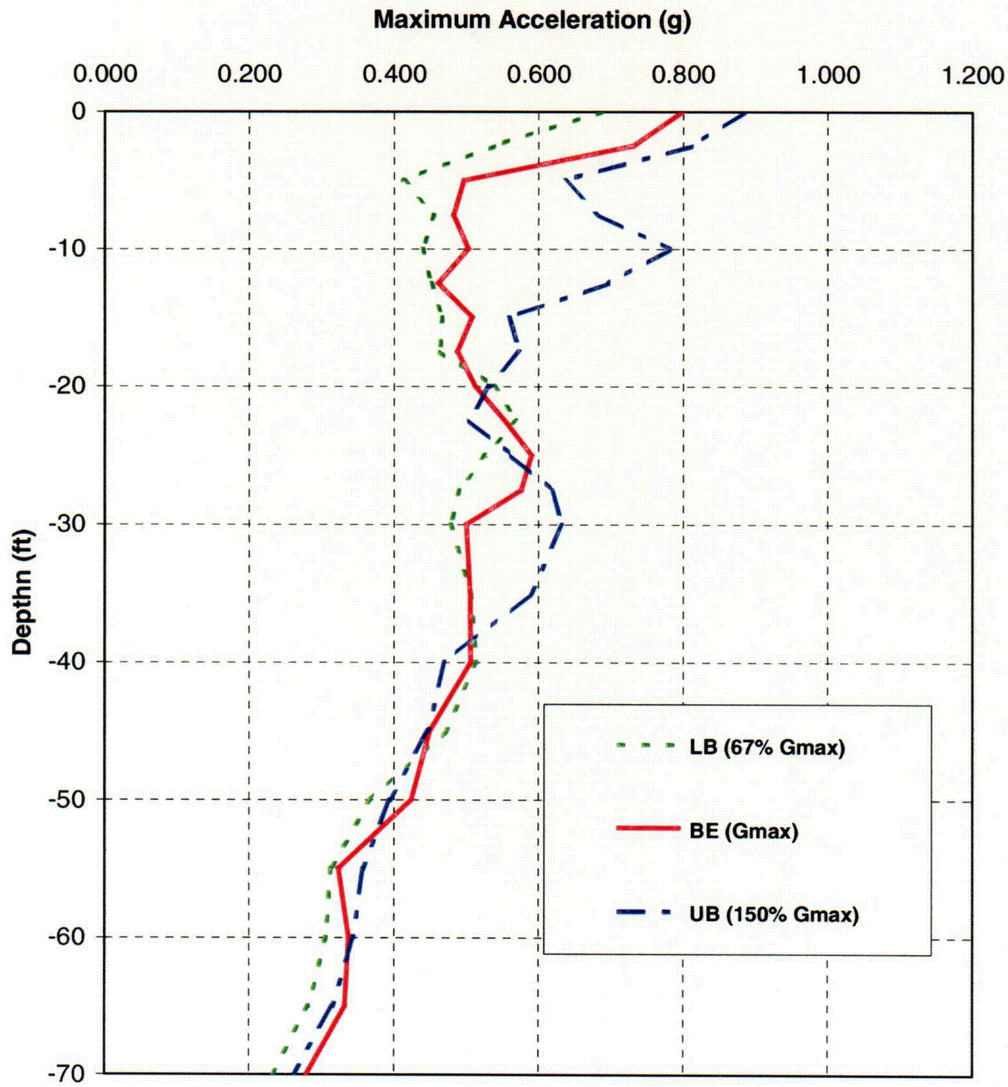

Figure 3. ZPA Variation with Depth for Profile 1

C05

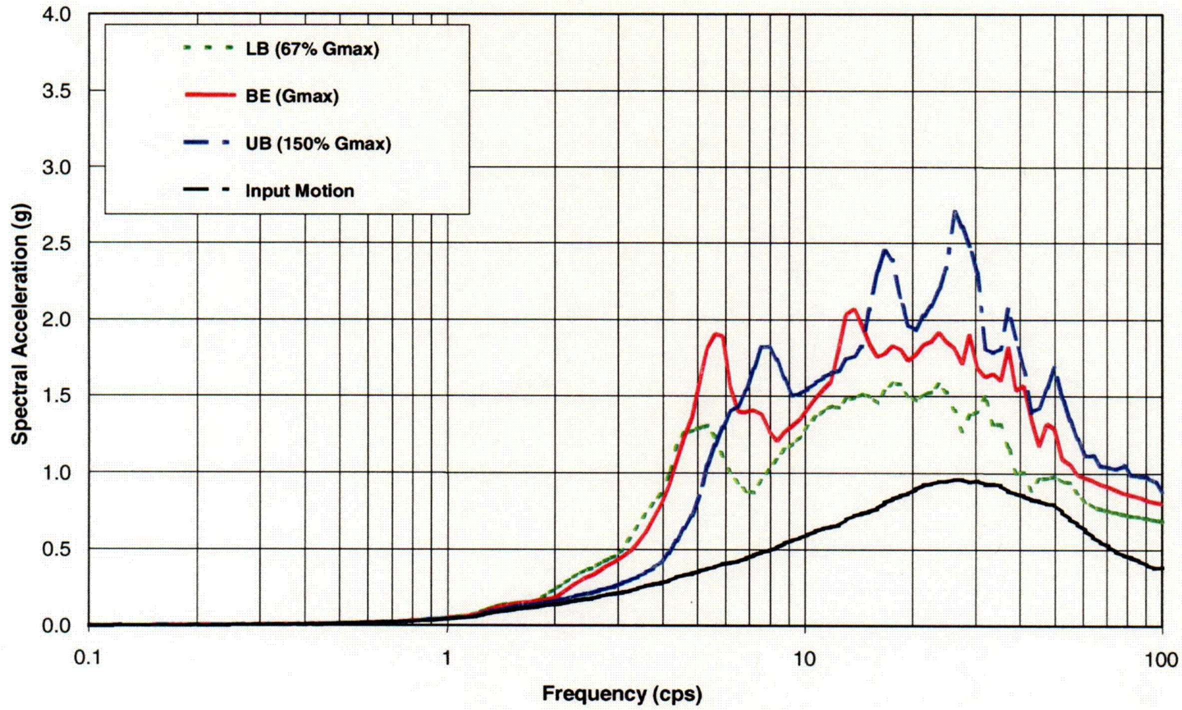

Figure 4. Horizontal ARS (5% Damping) for Profile 1

C06

Application Revision

SSAR Sections 2.5.4.7.3 and 2.5.4.7.4 will be revised to read as follows:

2.5.4.7.3 Site Specific Acceleration-Time Histories

Two single horizontal-component acceleration time histories were developed to be spectrum-compatible for use in the soil column amplification analysis described in Section 2.5.4.7.4. This is a time-domain based procedure which takes a given input acceleration time history and makes it compatible with a given target acceleration response spectrum.

Two target spectra were used in the analysis. Spectra developed to represent the 5-to-10 Hz high frequency and 1-to-2.5 Hz low frequency $5x10^{-5}$ mean hazard level ground motions were used along with the performance-based spectrum to develop hybrid high- and low-frequency spectra, the envelope of which replicates the performance-based spectrum itself. These horizontal acceleration target response spectra (5% spectral damping) are defined for the frequency range of 100 Hz to 0.1 Hz.

The spectral compatible matching criteria presented in NUREG/CR-6728 (Reference 171) were followed in the development of the spectrum-compatible time histories for the frequency range of 100 Hz to 0.5 Hz. These final spectrum-compatible time histories were used for the site response analysis.

2.5.4.7.4 Soil Column Amplification/Attenuation Analysis

The SHAKE2000 computer program was used to compute the site dynamic responses for the soil and rock profiles described in Section 2.5.4.7.1. The computation was performed in the frequency domain using the complex response method. The analysis used the acceleration-time histories described in Section 2.4.5.7.3. For the low frequency case, an earthquake with moment magnitude of 7.2 and an acceleration of 0.15g was used in the SHAKE2000 analysis, while for the high frequency case, an earthquake with moment magnitude of 5.4 and an acceleration at bedrock level of 0.39g was used.

SHAKE2000 uses an equivalent linear procedure to account for the non-linearity of the soil and weathered rock by employing an iterative procedure to obtain values for shear modulus and damping that are compatible with the equivalent uniform strain induced in each sublayer. At the outset of the analysis, a set of properties (based on the values of shear modulus and damping presented in Section 2.5.4.7.1, and total unit weight) was assigned to each sublayer of the soil and rock profile. The analysis was conducted using these properties and the shear strain induced in each sublayer was calculated. The shear modulus and damping ratio for each sublayer was then modified based on the shear modulus and damping ratio versus strain relationships presented in Section 2.5.4.7.2. The analysis was repeated until strain-compatible modulus and damping values were achieved.

The zero period acceleration (ZPA) results for the SHAKE2000 analysis for the four soil profiles listed at the end of Section 2.5.4.7.1 are shown in Table 2.5-46 for both the low frequency and high frequency cases, with V_s values based on the design shear wave velocity values given in Table 2.5-45. Values of G_{max} (proportional to the square of V_s) were varied in the SHAKE analysis to determine the impact on the ZPA, using G_{max} values that were 67% and 150% of the design G_{max} values derived from the V_s values in Table 2.5-46. For Profile 1, which is used in the liquefaction and slope stability analysis, the ZPA at the ground surface increased from 0.39g in Table 2.5-46 for the low frequency case to 0.46g using 150% G_{max}. For the high frequency case, the ZPA at the ground surface increased from 0.80g in Table 2.5-46 to 0.89g using 150% G_{max}. The ZPA results for Profile 1 using 150% G_{max} are also shown in Table 2.5-46. The 0.46g and 0.89g values were used for the peak ground acceleration in the liquefaction and slope stability analyses.

SSAR Table 2.5-46 will be revised to read as follows:

Table 2.5-46 ZPA Results from SHAKE Analysis

Depth, ft	V_s, ft/sec	Profile 1 G_{max}	Profile 1 150% G_{max}	Profile 2	Profile 3	V_s, ft/sec	Profile 4
Low Frequency Case							
0.0	700	0.393g	0.455g	-[(a)]	-	1275	0.338g
2.5	700	0.335g	0.402g	-	-	1275	0.321g
5.0	700	0.256g	0.275g	-	-	1275	0.271g
7.5	700	0.255g	0.274g	-	-	1275	0.200g
10.0	700/950	0.263g	0.246g	-	-	1275/1380	0.212g
12.5	950	0.253g	0.221g	-	-	1380	0.215g
15.0	950	0.223g	0.221g	-	-	1380	0.206g
17.5	950	0.236g	0.204g	-	-	1380	0.186g
20.0	950/1200	0.226g	0.204g	-	-	1380/1500	0.175g
22.5	1200	0.260g	0.209g	-	-	1500	0.184g
25.0	1200	0.281g	0.206g	-	-	1500	0.181g
27.5	1200	0.250g	0.194g	-	-	1500	0.167g
30.0	1200/1600	0.187g	0.219g	0.300g	-	1500/1600	0.208g
35.0	1600	0.201g	0.217g	0.249g	-	1600	0.214g
40.0	1600/2000	0.188g	0.160g	0.264g	0.275g	1600/2000	0.224g
45.0	2000	0.164g	0.144g	0.229g	0.248g	2000	0.220g
50.0	2000	0.141g	0.126g	0.199g	0.176g	2000	0.168g
55.0	2000/3300	0.129g	0.129g	0.152g	0.175g	2000/3300	0.130g

Table 2.5-46 ZPA Results from SHAKE Analysis

Depth, ft	V_s, ft/sec	Profile 1 G_{max}	Profile 1 150% G_{max}	Profile 2	Profile 3	V_s, ft/sec	Profile 4
65.0	3300	0.116g	0.134g	0.131g	0.150g	3300	0.135g
70.0	3300	0.101g	0.120g	0.118g	0.132g	3300	0.126g
Outcrop	6300	0.149g	0.149g	0.149g	0.149g	6300	0.149g
High Frequency Case							
0.0	700	0.800g	0.885g	-[a]	-	1275	0.651g
2.5	700	0.731g	0.811g	-	-	1275	0.634g
5.0	700	0.497g	0.636g	-	-	1275	0.579g
7.5	700	0.483g	0.684g	-	-	1275	0.481g
10.0	700/950	0.502g	0.781g	-	-	1275/1380	0.431g
12.5	950	0.461g	0.696g	-	-	1380	0.442g
15.0	950	0.508g	0.559g	-	-	1380	0.438g
17.5	950	0.487g	0.574g	-	-	1380	0.435g
20.0	950/1200	0.512g	0.531g	-	-	1380/1500	0.480g
22.5	1200	0.553g	0.504g	-	-	1500	0.520g
25.0	1200	0.590g	0.562g	-	-	1500	0.498g
27.5	1200	0.576g	0.618g	-	-	1500	0.488g
30.0	1200/1600	0.500g	0.633g	1.065g	-	1500/1600	0.458g
35.0	1600	0.505g	0.590g	1.037g	-	1600	0.523g
40.0	1600/2000	0.506g	0.470g	0.574g	0.770g	1600/2000	0.520g
45.0	2000	0.449g	0.447g	0.436g	0.783g	2000	0.477g
50.0	2000	0.424g	0.394g	0.382g	0.699g	2000	0.446g
55.0	2000/3300	0.323g	0.357g	0.295g	0.371g	2000/3300	0.345g
60.0	3300	0.337g	0.343g	0.296g	0.350g	3300	0.355g
65.0	3300	0.332g	0.315g	0.291g	0.326g	3300	0.353g
70.0	3300	0.279	0.261g	0.263g	0.270g	3300	0.305g
Outcrop	6300	0.386g	0.386g	0.386g	0.386g	6300	0.386g

a. Dash denotes soil not present.

Soil/Rock Columns

1. Profile from 0 to 70 feet, with 30 feet of unimproved Zone IIA saprolite, 10 feet of Zone IIB saprolite, 15 feet of Zone III rock, and 15 feet of Zone III-IV rock.
2. Profile from 30 to 70 feet depth for foundation sitting on 10 feet of Zone IIB saprolite, 15 feet of Zone III weathered rock, and 15 feet of Zone III-IV rock.
3. Profile from 40 to 70 feet depth for foundation sitting on 15 feet of Zone III weathered rock and 15 feet of Zone III-IV rock.
4. Profile from 0 to 70 feet, with 30 feet of improved Zone IIA saprolite, 10 feet of Zone IIB saprolite, 55 feet of Zone III weathered rock, and 15 feet of Zone III-IV rock.

RAI 2.5.4-10 (6/1/04 NRC Letter)

SSAR Section 2.5.4.8 describes the analyses to determine the potential for soil liquefaction at the ESP site.

<u>RAI 2.5.4-10 Part a)</u>

a) For each of the different methods used, please provide the results of any parametric evaluations of the liquefaction potential by varying the input of significant soil properties and seismic parameters.

<u>Response to Part a)</u>

1. <u>Variation of Significant Soil Parameters</u>

The liquefaction analysis is based on the current state-of-the-art paper by Youd et al (2001) and the evolution of the "simplified procedure" over the past 25 years. In the liquefaction analysis of the results of the ESP subsurface investigation, three different sets of subsurface data were used:

<u>Analysis Using SPT N-Values</u>. From the 7 sample borings, each sample of potentially liquefiable material (i.e., non-cohesive soil below the water table) was analyzed, using its N-value and fines content as the primary soil property inputs, with sample depth also a significant parameter. The factor of safety against liquefaction was computed for 17 samples at depths ranging from 2 to 30 feet, with N-values ranging from 6 to 44 blows/foot, measured fines contents ranging from 18.5 to 42.7 percent, and assumed fines contents ranging from 15 to 50 percent.

<u>Analysis Using CPT Values.</u> Cone penetrometer test (CPT) readings were interpreted at 0.5-foot intervals in the 8 CPTs. Liquefaction analysis was performed at each depth interval on potentially liquefiable soils, i.e., non-cohesive soil below the water table. Four of the CPTs did not reach below the water table. In the remaining 4 CPTs, analyses were performed at 105 depth intervals, ranging from 9 to 57 feet depth. The primary soil parameters used in the analyses were cone tip resistance and sleeve friction. The cone tip resistance ranged from 39 to 514 tsf, while the sleeve friction ranged from 0.4 to 9.7 tsf.

<u>Analysis Using Shear Wave Velocities</u>. In the analysis, the average shear wave velocities used for the Zone IIA saprolite ranged from 700 to 1,200 feet/second, at computed effective overburden pressures that ranged from 1.25 to 2.5 ksf. The lower bound shear wave velocities (approximately 63% of the average values) were also considered in the analysis.

2. Variation of G_{max} Values to Obtain Maximum Ground Acceleration

The value of the low strain shear modulus G_{max} was varied in the soil column amplification/attenuation analysis (SHAKE analysis) to obtain the appropriate value of peak ground acceleration to use in the liquefaction analysis. The values of G_{max} used in the initial SHAKE analysis were derived from the average shear wave velocity values given in SSAR Table 2.5-45. Additional SHAKE analyses were then run using G_{max} values that were 67% and 150% of G_{max} used in the initial SHAKE analysis. (This variation of G_{max} is discussed in Section 4 of the response to RAI 2.5.4-9. The maximum ground accelerations of 0.46g (low frequency case) and 0.89g (high frequency case) were obtained with 150% G_{max}. These values were used as peak ground accelerations in the liquefaction analyses.

3. Variation of Seismic Parameters

The two key seismic components of the liquefaction analysis are the peak earthquake acceleration and the earthquake magnitude.

Peak Earthquake Acceleration. The soil column amplification/attenuation analyses described in the response to RAI 2.5.4-9 noted that four different subsurface profiles were used in the analyses. The highest peak acceleration was obtained primarily using Profile 1, i.e., the profile that included 30 feet of unimproved Zone IIA saprolite. The peak ground accelerations from the Profile 1 analyses using both low frequency and high frequency acceleration-time histories were used in the liquefaction analysis. As noted above, the peak ground accelerations used were 0.46g for the low frequency case, and 0.89g for the high frequency case, both obtained from SHAKE analyses that used 150% G_{max}.

Earthquake Magnitude. An earthquake with moment magnitude of 7.2 was used in the liquefaction analysis for the low frequency case, while an earthquake with moment magnitude of 5.4 was used for the high frequency case.

RAI 2.5.4-10 Part b)

b) Please provide a copy of a sample liquefaction analysis for Zone IIA that shows the least factor of safety, stating and justifying all the assumptions made in the analysis.

Response to Part b)

A sample liquefaction analysis based primarily on Youd et al (2001) is provided in Enclosure 2 to this letter. This sample analysis shows the computations used to obtain the least factors of safety using N-values, CPT values, and shear wave velocity values.

References

Youd, T. L. et al. Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction of Soils, ASCE Journal of Geotechnical and Environmental Engineering, Volume 127, No. 10, October 2001 (Reference 178 of SSAR Section 2.5).

Application Revision

The 4th, 5th, and 6th paragraphs of SSAR Section 2.5.4.8.1 will be revised to read as follows:

> As discussed in Section 2.5.4.10, the Zone IIA saprolite has relatively high resistance to bearing failure but can produce excessive settlements under certain conditions. Where this soil forms the foundation material for safety-related structures, it would be improved (as discussed in Section 2.5.4.12) to decrease potential settlement to acceptable values. This improvement would be designed to ensure that the improved soil had a factor of safety against liquefaction equal to or greater than 1.1 (Section 2.4.8.2), at the safe shutdown earthquake ground motion.
>
> Despite its apparent low potential for liquefaction, the Zone IIA saprolite at the NAPS site has been the subject of several liquefaction analyses. These analyses are examined in Section 2.5.4.8.3 in light of the accelerations being assumed for the ESP. In addition, state-of-the-art liquefaction analysis is performed on potentially liquefiable samples obtained from the recent ESP exploration program, and is presented in Section 2.5.4.8.4.
>
> In Sections 2.5.4.8.1 through 2.5.4.8.4, Draft RG DG-1105 (Reference 172) is used as a guide.

The 2nd paragraph of SSAR Section 2.5.4.8.2 will be revised to read as follows:

> The safe shutdown earthquake (SSE) at rock for the existing units has a maximum acceleration of 0.12g. This was amplified to 0.18g in the soil. The seismic margin maximum acceleration in soil (Reference 174) was 0.30g. The maximum ESP acceleration (using the high frequency earthquake) at hard bedrock is 0.39g, amplified at the unimproved soil surface to 0.89g, as discussed in Section 2.5.4.7.4 and shown in Table 2.5-46.

SSAR Sections 2.5.4.8.4 and 2.5.4.8.5 will be revised to read as follows:

2.5.4.8.4 Liquefaction Analyses Performed for ESP

a. Magnitude and Acceleration Values for ESP Liquefaction Analyses

As noted in Section 2.5.4.7.3, two earthquakes were used in the liquefaction analysis. The low frequency earthquake had a magnitude of 7.2 and an acceleration at bedrock level of 0.15g. The high frequency earthquake had a magnitude of 5.4 and an acceleration at bedrock level of 0.39g.

Table 2.5-46 shows the zero period acceleration values for the four soil/rock profiles described in Section 2.5.4.7.1. Since the Zone IIB saprolite and the Zone III weathered rock are non-liquefiable, Profiles 2 and 3 in Table 2.5-46 are not considered in the liquefaction analysis. In Profile 4, the Zone IIA saprolite is improved, i.e., this would be the profile for any safety-related structures founded on the Zone IIA saprolite. The soil would be improved sufficiently to ensure that the improved soil had a factor of safety against liquefaction equal to or greater than 1.1 (Section 2.4.8.2), at the safe shutdown earthquake ground motion. In Profile 1, the Zone IIA saprolite (upper 30 feet) is not improved. Thus, Profile 1 is the only profile that is considered in the liquefaction analysis. As noted in Section 2.5.4.7.4, the ZPA at the ground surface increased from 0.39g to 0.46g for the low frequency case, and 0.80g to 0.89g for the high frequency case using 150% G_{max} (Table 2.5-46). The 0.46g and 0.89g values are used for the peak ground acceleration for the liquefaction analyses described in the following paragraphs.

b. Updated Seismic Margin Assessment

The seismic margin assessment described in Section 2.5.4.8.3 for the main plant area was modified in the ESP evaluation, maintaining the same assumptions as used in the original study but substituting the ESP design accelerations and moment magnitudes in soil of 0.46g and 7.2 (low frequency), and 0.89g and 5.4 (high frequency). Magnitude scaling factors of 1.13 and 2.5 were used in the analysis for the low and high frequency earthquakes, respectively. The resulting FS values ranged from about 0.7 to 1.8, with average values close to but lower than 1.1.

c. Analysis of ESP Samples and CPT Results

Liquefaction analysis of each sample of Zone IIA saprolite obtained by SPT sampling during the ESP subsurface investigation was performed to determine the FS against liquefaction. The CPT results were also analyzed. The analyses conservatively ignored the age, overconsolidation, and mineralogy/fabric effects of the saprolite. Cohesive samples and/or samples above the groundwater table were considered non-susceptible to liquefaction.

The analysis followed the method proposed by Youd et. al. (Reference 178). This state-of-the-art liquefaction methodology is based on the evolution of the Seed and Idriss "Simplified Procedure" over the past 25 years and updates DG-1105 (Reference 172). Magnitude scaling factors of 1.13 and 2.5 were used in the analysis for the moment magnitude 7.2 (low frequency) and 5.4 (high frequency) earthquakes, respectively. The K_σ factor for high overburden pressures was incorporated into the analysis, using a relative density of 60 percent.

Using the peak ground accelerations and magnitude scaling factors for the low and high frequency earthquakes described above, the analysis of the SPT results gave FS values against liquefaction greater than 1.1 for those samples that were liquefiable, except in one case. For the eight CPTs performed, the liquefaction analysis showed 4.5-foot thick zones in two CPTs and a 21-foot thick zone in another CPT where the FS against liquefaction was less than 1.1.

d. **Liquefaction Analysis Using Shear Wave Velocity Criteria**

The design values of shear wave velocity shown in Figure 2.5-62 and tabulated on Table 2.5-46 were corrected for overburden pressure using the method outlined in Youd, et al (Reference 178). The resulting values all fell into the "No Liquefaction" zone on Figure 9 of Reference 178. When the lower-bound values of shear wave velocity shown in Table 2.5-45 were used in the liquefaction analysis, most of the top 20 feet of the profile fell into the "Liquefaction" zone on Figure 9 of Reference 178.

e. **Dynamic Settlement**

Using the method outlined in Tokimatsu and Seed (Reference 179), the maximum estimated dynamic settlement of the Zone IIA saprolite due to earthquake shaking was about 5 inches.

2.5.4.8.5 **Conclusions About Liquefaction**

The conclusions from the foregoing sections on the analysis of liquefaction potential are as follows:

- No historical signs of liquefaction have been observed at the North Anna Site.
- Only the Zone IIA saprolites fall into the gradation and relative density categories where liquefaction would be considered possible.

- The age, structure, fabric, and mineralogy of these saprolites lower the potential for liquefaction very substantially.

- For a conventional liquefaction analysis, a FS ≥ 1.1 is adequate, based on the conservative estimate of the ESP design seismic acceleration.

- A seismic margin liquefaction analysis of the main plant area, modified to use the ESP seismic parameters (M = 7.2 with 0.46g peak ground acceleration for low frequency and M = 5.4 with 0.89g peak ground acceleration for high frequency), and that ignored structure, fabric, and mineralogy effects, gave average FS values that were close to but lower than 1.1.

- A state-of-the-art liquefaction analysis of the ESP SPT samples using the low and high frequency ESP seismic parameters gave FS values greater than 1.1 for all except one SPT result analyzed.

- A state-of-the-art liquefaction analysis of the ESP CPT measurements using the low and high frequency ESP seismic parameters indicated an approximately 21-foot thick zone and two 4.5-foot thick zones where the FS against liquefaction was less than 1.1.

- A state-of-the-art liquefaction analysis of the shear wave velocity profile, using shear wave velocity values corrected for overburden pressure, indicated no liquefaction when the design shear wave velocity values were used but indicated liquefaction of most of the top 20 feet when the lower bound shear wave velocity values were used.

- Estimated maximum dynamic settlements due to earthquake shaking are about 5 inches.

Based on the above analysis results, it can be concluded that some of the Zone IIA saprolitic soils have a potential for liquefaction based on the low and high frequency ESP seismic parameters. The liquefaction analysis did not take into account the beneficial effects of age, structure, fabric, and mineralogy. If safety-related structures are founded on the Zone IIA saprolitic soils, these soils would be improved to reduce potential settlements to within acceptable tolerances, as outlined in Sections 2.5.4.10 and 2.5.4.12. This improvement would be designed to ensure that the improved soil had a factor of safety against liquefaction equal to or greater than 1.1 (Section 2.4.8.2), at the safe shutdown earthquake ground motion.

RAI 2.5.5-1 (6/1/04 NRC Letter)

SSAR Section 2.5.5.2 presents an analysis of the stability of the existing slope to the north of the SWR. In view of the results of the liquefaction analysis (SSAR 2.5.4.8), which demonstrated the possibility of isolated zones of liquefaction in unimproved Zone IIA saprolite, please provide the basis for concluding that the existing slope has a "low susceptibility" to liquefaction, and therefore concluding that a horizontal acceleration of 0.1g is suitable for the pseudo-static analysis. In addition, please provide the rationale for concluding that the pseudo-static analysis adequately demonstrates that the existing slope would remain stable under SSE conditions.

Response

1. Basis For Conclusion that Existing Slope Has a Low Susceptibility to Liquefaction

Based on the revised peak ground accelerations, the liquefaction analysis described in SSAR Section 2.5.4.8 concludes that there could be liquefaction of some of the unimproved Zone IIA saprolitic soils at the ESP site. This liquefaction analysis is considered to be conservative since it does not take into account the age, fabric, structure and mineralogy of the saprolite, and uses significantly higher peak ground accelerations than previously applied at the North Anna site. As can be seen from Figure 3 in the response to RAI 2.5.4-9, the very high accelerations for the high frequency case are in the top 5 feet of the soil – the average acceleration in the soil is closer to 0.55g below the top 5 feet. Similarly for the low frequency case, the average acceleration in the soil is closer to 0.19g below the top 5 feet. Also, SSAR Figure 2.5-68 shows the estimated phreatic surface (depth to groundwater) beneath the existing slope to the north of the SWR to be more than 50 feet below ground surface under the majority of the existing area shown on that figure. SSAR Section 2.5.5.1.3 notes, "The depth of this phreatic surface precludes any potential for liquefaction of the near-surface soils in the slope". Thus, the assumption that the existing slope materials have a low susceptibility to liquefaction is reasonable. Nevertheless, in recognition of the high near-surface accelerations and the results of the liquefaction analysis, the SSAR will be revised to indicate measures that would be taken to ensure the safety of the slope and of the structures that may be located close to the bottom of the slope.

2. Rationale for Concluding That Pseudo-Static Analysis Adequately Demonstrates That Existing Slope Would Remain Stable Under SSE Conditions

The rationale is based on the arguments presented by Seed (1979) using work done by Newmark (1965). These authors recognized that, provided embankments did not liquefy or lose a significant amount of strength during a seismic event, they would displace at the crest but typically not fail in the conventional sense. This theory was backed up by observation of dams in California that had survived major earthquakes, sustaining displacement at the crest but not failure of the embankment. The displacements were within acceptable limits for crest accelerations up to about 0.75g. These dams were initially designed to withstand an inertia force of 0.1 or 0.15g. The pseudo-static analysis models such an inertia force. Seed indicates that an inertia force of 0.1g is appropriate up to a magnitude of 6.5, while 0.15g is appropriate for magnitudes between 6.5 and 8.25.

Although the 0.89g computed peak ground acceleration from the high frequency earthquake at North Anna is greater than the 0.75g referenced by Seed, Figure 3 in the response to RAI 2.5.4-9 indicates that the very high accelerations are in the top 5 feet of the soil – the average acceleration in the soil is closer to 0.55g below the top 5 feet. In addition, the design high frequency earthquake has a relatively low energy (magnitude 5.4), which is significant when estimating its potential impact on slope stability. Thus, at North Anna, a pseudo-static design using an inertia force of 0.1g is adequate for the high frequency earthquake. The low frequency earthquake gives a peak acceleration of 0.46g and a magnitude of 7.2. According to Seed, 0.15g acceleration is appropriate in the pseudo-static analysis for this level of magnitude. The pseudo-static slope stability analysis run with 0.1g and 0.15g both gave FS values greater than 1.1. Nevertheless, in recognition of the high near-surface accelerations and the results of the liquefaction analysis, the SSAR will be revised to indicate measures that would be taken to ensure the safety of the slope and of the structures that may be located close to the bottom of the slope.

References

Seed, H. B. Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams, Geotechnique, Volume 29, No. 3, 1979 (Reference 186 of SSAR Section 2.5).

Newmark, N. M. Effects of Earthquakes on Dams and Embankments, Geotechnique, Volume 15, No. 2, 1965 (Reference 187 of SSAR Section 2.5).

Application Revision

The last paragraph of SSAR Section 2.5.5.1.2 will be revised to read as follows:

> The liquefaction characteristics of all of the Zone IIA saprolite are thoroughly examined in Section 2.5.4.8. That section concludes that the results of the liquefaction analysis indicate that some of the Zone IIA saprolitic soils have a potential for liquefaction based on the ESP seismic parameters. The liquefaction analysis did not take into account the beneficial effects of age, structure, fabric, and mineralogy.

The title of SSAR Section 2.5.5.2.3 will be revised to read as follows:

2.5.5.2.3 **Analysis of Existing Slope**

SSAR Section 2.5.5.2.3.b will be revised to read as follows:

b. **Seismic Slope Stability Analysis**

The pseudo-static approach is used as a first approximation for the seismic analysis of slopes. In this approach, the horizontal and vertical seismic forces are assumed to act on the slope in a static manner; that is, as a constant static force. This is an obviously conservative approach, since the actual seismic event occurs for only a short period of time, and during that time, the forces alternate their direction at a relatively high frequency. Also, the pseudo-static analysis tends to be run using the peak seismic acceleration; the mean acceleration during the design seismic event is significantly less than the peak value. A pseudo-static analysis using peak acceleration values can be a useful tool in a limit analysis where the peak acceleration is relatively low. In such analyses, the computed factor of safety may well exceed the minimum of 1.1, thus requiring no further analysis. However, where the peak seismic acceleration values are high, the pseudo-static analysis produces unreasonably low safety factor values.

The pseudo-static analysis was run using SLOPE/W. For the high frequency earthquake, the peak horizontal acceleration used was 0.57g. This is the average peak acceleration in the top 55 feet of unimproved soil shown in Table 2.5-46 for 150% G_{max}. The vertical acceleration used was 0.285g. The computed factor of safety was significantly less than the required 1.1. For the low frequency earthquake, the equivalent peak horizontal acceleration used was 0.21g with a vertical acceleration of 0.105g. The computed factor of safety was slightly greater than 1.1.

Seed (Reference 186), in the 19th Rankine Lecture, addressed the over-conservatism intrinsic in the pseudo-static analysis. He looked at the more rational approach proposed by Newmark (Reference 187), where the effective acceleration time-history is integrated to determine velocities and displacements of the slope. He also examined dams in California that had been subjected to seismic forces, including several dams that survived the 1906 San Francisco earthquake. Based on his studies, he concluded that for embankments that consist of materials that do not tend to build up large pore pressures or lose significant percentages of their shear strength during seismic shaking, seismic coefficients of only 0.15g are adequate to ensure acceptable embankment performance for earthquakes up to magnitude M = 8.25 with peak ground accelerations of 0.75g. For earthquakes in the range of M = 6.5, Seed recommends a horizontal seismic coefficient of only 0.1g with a vertical seismic coefficient of zero.

recommends a horizontal seismic coefficient of only 0.1g with a vertical seismic coefficient of zero.

Although the liquefaction analysis of the Zone IIA saprolite indicated some of the material has a potential for liquefaction, its age, fabric and interlocking angular grain structure, along with the significant portion of low plasticity clay minerals present in the material, have been demonstrated to give the grain structure a low susceptibility to pore pressure build-up or liquefaction (Section 2.5.4.8). This material would not lose a significant proportion of its shear strength during shaking. Thus, for the low frequency earthquake, with a design Magnitude M = 7.2, the pseudo-static analysis should be limited to a horizontal acceleration of only 0.15g. For the high frequency earthquake, with a design magnitude M =5.4, the pseudo-static analysis should be limited to a horizontal acceleration of only 0.1g. The pseudo-static analysis was again run using SLOPE/W. This time the horizontal accelerations used were 0.1g and 0.15g, with zero vertical acceleration. The computed factors of safety were greater than 1.1. The input to the analysis and the results for the 0.1g case are shown in Figure 2.5-70.

Other researchers have also recommended substantially reducing the peak acceleration when applying the pseudo-static analysis. Kramer (Reference 188) recommends using an acceleration of 50 percent of the peak acceleration. Using the average peak acceleration for the high frequency earthquake in the top 55 feet of 0.57g, the horizontal input using Kramer's recommendation would be 0.285g and the vertical input would be 0.1425g. This level of input provides a factor of safety against slope failure of just below 1.0. Although this is slightly less than the required factor of safety of 1.1, it is considered marginal based on the high level of seismic acceleration being applied and the relatively low energy level of the design earthquake. For the low frequency earthquake, where the average peak acceleration in the top 55 feet is about 0.21g, the horizontal input using Kramer's recommendations would be 0.105g and the vertical input would be about 0.05g. This results in a factor of safety of greater than the required 1.1.

Based on the possibility of some liquefaction in the slope area and the marginal results obtained using Kramer's method, measures would be taken to ensure the safety of the slope and of the structures that may be located close to the bottom of the slope. These measures are outlined in Section 2.5.5.6.

The last paragraph of SSAR Section 2.5.5.5 will be revised to read as follows:

If the selected design for the new units requires that the new slope be constructed, and it is deemed that any failure of the slope could impact the new units, then investigation and analysis of the slope would be performed as part of detailed engineering and described in the COL application. If the analysis, based on the subsurface investigation results, showed an inadequate factor of safety against slope failure, then the design would be modified to eliminate any risk of slope failure. Such modifications are outlined in Section 2.5.5.6.

SSAR Section 2.5.5.6 will be revised to read as follows:

2.5.5.6 **Conclusions**

Existing slopes and embankments that are not impacted by the new units (such as the SWR embankments) are not analyzed. New slopes of the non-safety-related, deepened intake channel, which would be used for the normal cooling water system supply of the new units, would be analyzed during detailed design, if required. Such analysis is not part of the ESP SSAR.

The only existing slope whose failure could adversely affect the safety of the new units because of its proximity to the ESP site is a 55-foot high, 2h:1v slope that descends from north of the SWR down to south of the existing excavation made for abandoned Units 3 and 4. The slope is made almost entirely in cut material. Static long-term analyses of the existing slope using the computer program SLOPE/W gave values of factor of safety in excess of the minimum 1.5 required. Pseudo-static analyses using ESP design values of horizontal and vertical seismic acceleration gave safety factor values less than the minimum acceptable value of 1.1 for the high frequency earthquake. However, when the seismic input was modified to conform to the reductions given by Seed (Reference 186), the computed safety factors against slope failure were in excess of 1.1. The Seed reductions are considered reasonable and valid. When the Kramer recommendations were applied, the computed factor of safety against seismic slope failure was considered satisfactory for the low frequency earthquake and marginal for the high frequency earthquake. Based on the possibility of some liquefaction in the slope area and the marginal results obtained using Kramer's method, measures would be taken to ensure the safety of the slope and of the structures that may be located close to the bottom of the slope. These measures could include reducing the slope steepness, removing and replacing materials that could lose significant strength during the design earthquake, ground improvement measures such as soil nailing, moving structures further from the toe of the slope, and/or providing walls/barriers to protect those structures.

A new slope may be excavated to the west of the SWR to accommodate UHSs for the new units. The new slope would be approximately the same height, would have the same 2h:1v slope, and would have the same soil and rock characteristics as the existing slope that was analyzed. If analysis during the design stage of this slope indicates unacceptable factors of safety against slope failure, modifications such as those proposed for the existing slope in the previous paragraph would be employed.

Correction of Controlling Earthquake Distance Calculations

Risk Engineering, Inc. identified an error in their software CALCRG which performs deaggregation of seismic hazard, deaggregating hazard results into 35 magnitude-distance bins and combining hazard results from two structural frequencies (typically 1 and 2.5 Hz, or 5 and 10 Hz). The procedures followed by software CALCRG (the definitions of the magnitude-distance bins, the procedure for combining two structural frequencies, and the rule for deciding when to deaggregate the hazard for distant (>100 km) sources separately) are as described in Regulatory Guide 1.165.

The procedure used in CALCRG to determine the distance of the controlling earthquake was to calculate the mean distance using the centroid of each distance bin:

$$[\sum_{D}(D_C * \sum_{M} DEAGG_{D,M})] / TOTAL$$

where,

D_C = centroid distance of a distance bin.

$$TOTAL = \sum_{D}\sum_{M} DEAGG_{DM}$$

and, $DEAGG_{D,M}$ is the median frequency of exceedance for each D,M bin.

Regulatory Guide 1.165 indicates a more complicated calculation, however:

$$\exp[\sum_{D}(\ln D_C * \sum_{M} DEAGG_{D,M})] / TOTAL$$

The original calculation in CALCRG is more conservative for long period motions because it results in a longer average distance and more low frequency energy. For short distances that dominate the high frequencies, the two procedures give similar distances.

The error in CALCRG and the related software has been corrected resulting in the following changes to tables in the SSAR:

SSAR Table	Title	Description	Old Value	New value
2.5-20	Controlling Earthquake Magnitude and Distances 1989 EPRI Sources and Ground Motion Models	Low frequencies	r_{epi}=31 km	r_{epi}=25 km
			r_{CD}=29 km	r_{CD}=23 km
		High frequencies	r_{epi}=21 km	r_{epi}=18 km
			r_{CD}=20 km	r_{CD}=17 km
2.5-23	Controlling Earthquake Magnitude and Distances, Updated Models	Low frequencies	r_{epi}=28 km*	r_{epi}=20 km
			r_{CD}=27 km*	r_{CD}=19 km
		High frequencies	r_{epi}=17 km	r_{epi}=15 km
			r_{CD}=17 km	r_{CD}=15 km
2.5-26	Controlling Earthquake Magnitude and Distances Corresponding to Mean 5×10^{-5} Annual Frequency	Low frequencies	r_{CD}=37 km*	r_{CD}=24 km
		High frequencies	r_{CD}=23 km	r_{CD}=19 km

* Old value included in revised SSAR table provided in Reference 1.

These changes are minor and do not impact the final conclusions of SSAR Section 2.5.2.

References

1. June 11, 2004 Letter from Eugene S. Grecheck, Vice President-Nuclear Support Services, Dominion, to U. S. Nuclear Regulatory Commission, Document Control Desk, "Dominion Nuclear North Anna, LLC, North Anna Early Site Permit Application, Response to Request for Additional Information No. 2 and Corrected Seismic Hazard Deaggregation Results."

Application Revision

SSAR Tables 2.5-20, 2.5-23, and 2.5-26 will be revised to read as follows:

Table 2.5-20 Controlling Earthquake Magnitude and Distances 1989 EPRI Sources and Ground Motion Models

	m_b	M[a]	r_{epi}, km	R_{cd}[b], km
Low frequency (1 and 2.5 Hz)	6.2	5.9	25	23
High frequency (5 and 10 Hz)	5.9	5.5	18	17

a. M converted from m_b as described in Section 2.5.2.2.1.

b. r_{CD} converted from r_{epi} as given in Reference 116, model F3.

Table 2.5-23 Controlling Earthquake Magnitude and Distances, Updated Models

	m_b	M[a]	r_{epi}, km	R_{cd}[b], km
Low frequency (1 and 2.5 Hz)	5.9	5.6	20	19
High frequency (5 and 10 Hz)	5.7	5.3	15	15

a. M converted from m_b as described in Section 2.5.2.2.1.

b. r_{CD} converted from r_{epi} as given in Reference 116, model F3.

Table 2.5-26 Controlling Earthquake Magnitudes and Distances Corresponding to Mean 5×10^{-5} Annual Frequency

Frequencies	M	r_{CD}, km
Low (1 and 2.5 Hz)	5.6	24
High (5 and 10 Hz)	5.3	19

Enclosure 2

**Sample Liquefaction Analysis for Zone IIA Saprolite
In Response to RAI 2.5.4-10 Part b)**

Sample Liquefaction Analysis for Zone IIA Saprolite
In Response to RAI 2.5.4-10 Part b)

1.0 Purpose

The purpose of this sample liquefaction analysis is to analyze the site soils to determine the least factor of safety against liquefaction based on the results of the recently completed subsurface investigation and laboratory testing program (Ref. 1) and on existing subsurface information (Refs. 2 - 13) at the site.

2.0 References

1. MACTEC Engineering and Consulting. "Results of Geotechnical Exploration and Testing, North Anna ESP Project, Louisa County, Virginia," for Bechtel Power Corporation, February 2003.

2. North Anna Power Station UFSAR, Revision 38, October 2002.

3. Dames & Moore. "Report, Site Environmental Studies, Proposed North Anna Power Station, Louisa County, Virginia," for Virginia Electric Power Company, January 1969.

4. Dames & Moore. "Report, Site Environmental Studies, North Anna Nuclear Power Station, Proposed Units 3& 4, Louisa County, Virginia," for Virginia Electric Power Company, August 1971.

5. NAPS UFSAR, Appendix 3E. "Geotechnical Investigations and Soil Sample Testing for the Service Water Reservoir," Revision 38, October 2002.

6. NAPS UFSAR, Attachment 4 to Appendix 3E. "Investigations of Loose Saprolite," Revision 38, October 2002.

7. North Anna ISFSI Safety Analysis Report, Revision 3, June 2002.

8. NAPS UFSAR Revision 38, Appendix 2G. "Seismic Survey for the North Anna Power Station," for Stone and Webster Corporation by Weston Geophysical Engineers, Inc., report dated February, 1969.

9. Weston Geophysical Research, Inc. "Velocity Measurements, North Anna Power Station, Virginia Electric and Power Company," for Stone and Webster Corporation, January 1970. (Also contained as an addendum to the reply to Question 2.8 in the North Anna Units 1 & 2 PSAR Supplement Volumes).

10. NAPS UFSAR Revision 38, Attachment 1 to Appendix 3E. "Cyclic Triaxial Tests on Soil Samples, Service Water Reservoir, North Anna Power Station," for Stone and Webster Corporation by Geotechnical Engineers, Inc., report dated December 1975.

11. NAPS UFSAR, Attachment 2 to Appendix 3E. "Cyclic Triaxial Tests on Soil Samples from the Service Water Reservoir, North Anna Power Station," Revision 38, October 2002.

12. NAPS UFSAR, Attachment 3 to Appendix 3E. "Report on Laboratory Testing, Service Water Reservoir, Virginia Electric Power Company," Revision 38, October 2002.

13. NAPS UFSAR, Appendix 2C. "Report on Foundation Studies for the Proposed North Anna Power Station in Louisa County, Virginia, Prepared by Dames and Moore," Revision 38, October 2002.

14. Bechtel Calculation 24830-G-003. "Geotechnical Engineering Properties," North Anna ESP Project, March 2003.

15. Bechtel Calculation 24830-G-004. "Bearing Capacity and Settlement Analysis," North Anna ESP Project, March 2003.

16. ASCE. "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," ASCE 4-98, Reston, VA, May 2000.

17. Bechtel Calculation 24830-G-029. "SHAKE Analysis," North Anna ESP Project, August 2004.

18. Bechtel Calculation 24830-G-010. "Properties for SHAKE Analysis," North Anna ESP Project, August 2003

19. Martin J.R. and Fragaszy, R.J. "Soil Failure/Liquefaction Susceptibility Analysis for North Anna Power Station (NAPS) – Seismic Margin Assessment," Report by Geotechnics for Virginia Power Company, December 1994.

20. Seed, H.B., and I.M. Idriss. "Ground Motions and Soil Liquefaction During Earthquakes," Earthquake Engineering Research Center, Berkeley, CA, 1982.

21. Bierschwale, J.G., and K.H. Stokoe. "Analytical Evaluation of Liquefaction Potential of Sands Subjected to the 1981 Westmoreland Earthquake," Geotechnical Engineering Report GR-84-15, Civil Engineering Department, University of Texas, 1984.

22. Pavich, M.J., L. Brown, J.N. Valette-Silver, J. Klein, and R. Middleton. "^{10}Be Analysis of a Quaternary Weathering Profile in the Virginia Piedmont," Geology, Vol. 13, January 1985.

23. Lewis, M.R., I. Arango, J.K. Kimball, and T.E. Ross. "Liquefaction Resistance of Old Sand Deposits," Proceedings of XI Pan-American Conference on Soil Mechanics and Geotechnical Engineering, Foz do Iguassu, Brazil, August 1999.

24. Seed, H.B. "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes," ASCE Journal of Geotechnical Engineering Division, Vol. 105, No. GT2, February 1979.

25. Committee of Earthquake Engineering. "Liquefaction of Soils During Earthquakes," Commission on Engineering and Technical Systems, National Research Council, Published by the National Academy Press, Washington, DC, 1985.

26. Youd et al. "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction of Soils," ASCE Journal of Geotechnical and Environmental Engineering, Vol. 127, No. 10, October 2001.

27. Stark, T.D., and S.M. Olson. "Liquefaction Resistance Using CPT and Field Case Histories," ASCE Journal of Geotechnical and Environmental Engineering, Vol. 121, No. 12, December 1995.

28. Robertson, P.K., and R.G. Campanella. "Guidelines for Geotechnical Design Using CPT and CPTU," *Soil Mechanics Series No. 20*, University of British Columbia, 1988.

29. U.S. Nuclear Regulatory Commission. Draft Regulatory Guide DG-1105, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Plant Sites," March 2001.

3.0 Subsurface Conditions

3.1 Subsurface Conditions

Ref. 14 divides the site soils into five zone categories:

IIA Saprolite – core stone less than 10% of volume of overall mass.

IIB Saprolite – core stone 10% to 50% of soil mass.

III Weathered rock – Moderately to severely weathered gneiss.

III-IV Partly weathered rock – Slightly to moderately weathered gneiss.

IV Parent rock – slightly weathered to fresh gneiss.

These zones are based on the Dames & Moore zones listed in Refs. 3 and 4. As noted in Ref. 14, the original Zone I (residual soil) is almost non-existent at the site, and so it has been combined with Zone IIA. The summary table of geotechnical properties from Ref. 14 for the 5 zones is included as Table 1 (at the end of the text). Reference to Table1 shows that the Zone IIB saprolite has a design SPT N-value of 100 blows/ft. Reference to Fig. 2 of Ref. 26 shows that materials with N values over about 30 will typically not liquefy. Thus, this material will not liquefy.

The only material that we have to analyze is the Zone IIA saprolite. Ref. 14 shows, for those soils classified as coarse-grained, the amount passing the No. 200 sieve ranges from 15% to 45%. Ref. 14 shows the Zone IIA soils tested at the site consist of 61.1% SM, 14.7% MH/CL/CH, 10.1% ML, 3.6% SC, and 10.5% SP. Now only the SP and SM materials are typically considered liquefiable. The SP soils generally have significantly higher N values than the SM soils. Thus, we will concentrate the analysis on the SM soils.

The following is taken from Ref. 14 concerning the mineralogy and fabric of the Zone IIA sands:

"The fabric, texture, and mineralogy of the samples were examined (Ref. 6, Section 3). As would be expected with these residual soils, the fabric is that of the parent rock, a biotitic quartz gneiss. There is strong foliation in the saprolite, dipping at angles of about 50 degrees to the horizontal. The fabric is strongly anisotropic. The texture shows angular geometrically interlocking grains with a lack of void network. The mineralogy also reflected the parent rock, with 30-40% quartz, 20 to 30% microline, 25 to 40% clay minerals, and 5 to 20% biotite (mica). The major clay mineral is halloysite (a hydrated form of kaolinite), with lesser amounts of illite and montmorillonite.

Apparently, the relative inactivity of the halloysite allows the sand to be classified as non plastic.

The fabric, texture and mineralogy of the saprolite explains the "soft" feel the sand has, compared with an alluvial sand. The foliation, along with the clay minerals and significant mica content, was apparently the reason for the excessive settlements when the material became saturated. We will use lower bound values for the geotechnical properties of this soil, based on its poor foundation performance at the site."

Ref. 6, Section 3 adds: "The fabric of the saprolite contrasts strongly with that of a sand. The sand shows no foliation and no interlocking of grains, even though the grains are quite angular. The sand thin section also shows a well-developed void network unlike that of saprolite. The fabric of saprolite is therefore not one of a transported soil but one of the parent rock material. The fabric is anisotropic; that is, it has strongly directional properties.

The most striking feature of the saprolite is the angularity and interlocking nature of the grains....<u>The geometric interlocking of the grains and the lack of a void network that would allow reorientation of grains indicates that the saprolite could not liquefy</u>." (underlining added for emphasis)

3.2 Groundwater Conditions

Groundwater is present in unconfined conditions in both the surficial sediments and underlying bedrock at the North Anna site. The groundwater generally occurs at depths ranging from about 6 to 58 ft below the present day ground surface. The exception to this is the area of the abandoned Units 3 and 4 excavation that was partially backfilled and where groundwater is within about 2 ft of the ground surface.

For the ESP boring/CPT specific liquefaction analysis in this calculation, the depth of water table will be taken as the highest level measured in the boring or closest observation wells.

3.3 Seismic Conditions

The time histories used in the SHAKE analysis (Ref. 17) are based on the performance-based spectrum (PBS) used for the SSE in the ESP. This is slightly higher than the 5 x 10^{-5} uniform hazard response spectrum, and is thus appropriately conservative. Two time histories were developed, the low frequency scaled spectrum and the high frequency scaled spectrum. The SHAKE analysis gave the following results using the 2 spectra. The V_s values tabulated below are the shear wave velocity values derived in Ref. 18. The SHAKE analyses were run using the G_{max} (low strain shear stiffness) derived from V_s, and also G_{max} x 1.5 and $G_{max}/1.5$ based on recommendations from Ref.

16 on uncertainties in the analysis. Ref. 16 indicates that G_{max} shall be multiplied and divided by 1 + C_v, where C_v is the coefficient of variation. If we have sufficient, adequate soil investigation data available, then we can use a minimum value of C_v of 0.5. Based on the large amount of geotechnical data available from the North Anna site, C_v of 0.5 was used.

ZPA Results from SHAKE Analysis: Accelerations Obtained using 1.5 x G_{max} High Frequency Scaled Spectrum

Depth, ft	V_s, ft/sec	G_{max} ksf	Profile 1	Profile 2	Profile 3	V_s, ft/sec	Profile 4
0.0	700	1904	0.885g			1275	0.713g
2.5	700	1904	0.811g			1275	0.705g
5.0	700	1904	0.636g			1275	0.673g
7.5	700	1904	0.684g			1275	0.611g
10.0	700/950	1904/3507	0.781g			1275/1380	0.547g
12.5	950	3507	0.696g			1380	0.486g
15.0	950	3507	0.559g			1380	0.501g
17.5	950	3507	0.574g			1380	0.525g
20.0	950/1200	3507/5595	0.531g			1380/1500	0.579g
22.5	1200	5595	0.504g			1500	0.622g
25.0	1200	5595	0.562g			1500	0.648g
27.5	1200	5595	0.618g			1500	0.646g
30.0	1200/1600	5595/10345	0.633g	1.004g		1500/1600	0.609g
35.0	1600	10345	0.590g	0.969g		1600	0.554g
40.0	1600/2000	10345/18029	0.470g	0.591g	0.792g	1600/2000	0.475g
45.0	2000	18029	0.447g	0.514g	0.635g	2000	0.456g
50.0	2000	18029	0.394g	0.419g	0.499g	2000	0.404g
55.0	2000/3300	18029/55178	0.357g	0.292g	0.364g	2000/3300	0.365g
60.0	3300	55178	0.343g	0.282g	0.327g	3300	0.351g
65.0	3300	55178	0.315g	0.279g	0.287g	3300	0.322g
70.0	3300	55178	0.261g	0.283g	0.281g	3300	0.270g
Base	-	-	0.386g	0.386g	0.386g	-	0.386g

ZPA Results from SHAKE Analysis: Accelerations Obtained using 1.5 x G_{max} Low Frequency Scaled Spectrum

Depth, ft	V_s, ft/sec	G_{max} ksf	Profile 1	Profile 2	Profile 3	V_s, ft/sec	Profile 4
0.0	700	1904	0.455g			1275	0.435g
2.5	700	1904	0.402g			1275	0.421g
5.0	700	1904	0.275g			1275	0.382g
7.5	700	1904	0.274g			1275	0.322g

ZPA Results from SHAKE Analysis: Accelerations Obtained using 1.5 x G_{max} Low Frequency Scaled Spectrum							
Depth, ft	**V_s, ft/sec**	**G_{max} ksf**	**Profile 1**	**Profile 2**	**Profile 3**	**V_s, ft/sec**	**Profile 4**
10.0	700/950	1904/3507	0.246g			1275/1380	0.252g
12.5	950	3507	0.221g			1380	0.233g
15.0	950	3507	0.221g			1380	0.215g
17.5	950	3507	0.204g			1380	0.206g
20.0	950/1200	3507/5595	0.204g			1380/1500	0.199g
22.5	1200	5595	0.209g			1500	0.195g
25.0	1200	5595	0.206g			1500	0.198g
27.5	1200	5595	0.194g			1500	0.204g
30.0	1200/1600	5595/10345	0.219g	0.305g		1500/1600	0.199g
35.0	1600	10345	0.217g	0.263g		1600	0.193g
40.0	1600/2000	10345/18029	0.160g	0.202g	0.238g	1600/2000	0.203g
45.0	2000	18029	0.144g	0.196g	0.218g	2000	0.184g
50.0	2000	18029	0.126g	0.177g	0.175g	2000	0.145g
55.0	2000/3300	18029/55178	0.129g	0.148g	0.138g	2000/3300	0.145g
60.0	3300	55178	0.137g	0.138g	0.128g	3300	0.145g
65.0	3300	55178	0.134g	0.117g	0.123g	3300	0.134g
70.0	3300	55178	0.120g	0.113g	0.124g	3300	0.120g
Base	-	-	0.149g	0.149g	0.149g	-	0.149g

Soil Columns (Profiles)

1. Profile from 0 to 70 feet, with 30 feet of unimproved Zone IIA saprolite, 10 feet of Zone IIB saprolite, 15 feet of Zone III rock, and 15 feet of Zone III-IV rock.

2. Profile from 30 to 70 feet depth for foundation sitting on 10 feet of Zone IIB saprolite, 15 feet of Zone III weathered rock, and 15 feet of Zone III-IV rock.

3. Profile from 40 to 70 feet depth for foundation sitting on 15 feet of Zone III weathered rock and 15 feet of Zone III-IV rock.

4. Profile from 0 to 70 feet, with 30 feet of improved Zone IIA saprolite, 10 feet of Zone IIB saprolite, 15 feet of Zone III weathered rock, and 15 feet of Zone III-IV rock.

Soil columns 2 and 3 are not considered further since these soils will not liquefy, as noted earlier.

For new Category I foundations, the Zone IIA saprolite will be improved with stone columns. This profile is Soil Profile 4. The soil will be improved to the extent that liquefaction will not be an issue.

Where there are no new Category I foundations (e.g., in the vicinity of the SWR), the Zone IIA saprolite will not be improved. This is Soil Profile 1. In this calculation, only the accelerations obtained for Soil Profile I will be used in the analysis.

From the tables on the previous pages showing the accelerations obtained using 1.5 x G_{max}, we see that the ground acceleration is 0.89g for Profile 1 using the high frequency spectrum and 0.46g using the low frequency spectrum. Now, the high frequency spectrum has a magnitude of 5.4 while the low frequency spectrum has a magnitude of 7.2. Since the liquefaction analysis has a magnitude scaling factor included in the analysis, it is not clear until we look at the magnitude scaling factors for the 2 spectra which will give the lower factor of safety (FS) values.

4.0 Previous Liquefaction Studies

There have been several liquefaction studies of the Zone IIA saprolitic sand. Bear in mind that the state-of-the-art has developed considerably since the original studies.

4.1 1976 Analyses

In 1976, liquefaction analyses were presented for the soils beneath the service water reservoir (SWR) facilities, and the SWR embankment. These analyses used the soil SSE of 0.18g and the results of 15 cyclic triaxial tests on the saprolite.

The initial write-up for the SWR facilities is in Section 3E.6 of Ref. 5. Analyses using cyclic triaxial tests from samples from 7 borings gave factors of safety (FS) ranging from 2.52 to 3.31 with an average of 3.0. For 2-dimensional shaking, Seed has suggested the FS be reduced by about 10%, i.e., to 2.27 to 2.98 with an average of 2.7.

The analysis for the SWR reservoir is presented in Ref. 6. The computed FS against liquefaction range from 1.51 to 6.02. For 2-dimensional shaking, the range of FS is from 1.36 to about 5.4.

4.2 Geotechnics Analyses

In December 1994, James Martin and Richard Fragaszy of Geotechnics performed a thorough liquefaction analysis of the North Anna site soils for a seismic margin assessment (Ref. 19). They used a maximum acceleration of 0.3g and a magnitude of 6.8.

Ref. 19 uses three approaches to liquefaction assessment at the North Anna plant:

- For the main plant area, a modified version of the Seed and Idriss (Ref. 20) Simplified Procedure based on SPTs was used. (Ref. 20 procedure is the basis for the Youd et al (Ref. 26) procedure used in Section 5.4).
- For the main plant area, a threshold shear strain analysis was applied (Ref. 21).
- For the SWR, the results of cyclic triaxial tests were used as the basis of the analysis.

Ref. 19 points out that the Seed and Idriss procedure needs to be modified because the saprolitic sand is quite different from recently-deposited alluvial type sands, on which the Seed and Idriss procedure is based, in 4 ways.

(1) As described in Section 3.1, i.e., the closely interlocking angular particles of the weathered rock structure and the lack of voids increase the FS against liquefaction significantly. This increase is not taken into account in the Ref. 19 analysis (conservative).

(2) Ref. 19 notes that the FS against liquefaction also increases as the age of the deposit increases. Ref. 19 notes that the age of the saprolitic sands has been estimated between 1 and 10 million years. Ref. 22 suggests the sands are between 800,000 and 1.6 million years old. The FS is increased in the analysis by a factor of 2 to account for the age effects (factors ranging from 2 to 3 are suggested by Ref. 23).

(3) The saprolitic sands are overconsolidated. Correction factors used in Ref. 19 were only half of those suggested by Seed (Ref. 24).

(4) The penetration resistance of some "soft" saprolitic zones could be artificially low due to the heavy mica content in some places in the saprolite. No correction factor was applied.

Thus, correction factors were only applied for (2) and (3) above, and even then, with smaller values than noted in the literature. Thus, the modified analysis can be considered conservative. The liquefaction analysis in the main plant (intake structure, quench spray and main steam valve house, service building, auxiliary building, turbine building) gave FS that ranged from 1.54 to 3.51.

With the threshold strain approach, using an average shear wave velocity in the saprolite of 950 ft/sec (same as used in Ref. 14), Ref. 21 would indicate a FS of nearly

3.0 for the plant soils against liquefaction, for an earthquake magnitude of 6.5 (not 6.8) and a maximum acceleration of 0.30g.

Using the results of the 15 cyclic triaxial tests performed on undisturbed samples of borings in the SWR area (tests summarized in Ref. 14), Ref. 19 computed FS of safety against liquefaction. The values ranged from 1.51 to 1.99 for the SWR facilities (pump house, valve house, tie-in vault, service water lines). Analysis under the SWR embankment gave FS values ranging from 0.91 to 3.61, with an average of more than 1.5. Of the few locations with FS < 1, Ref. 19 notes, "..these factors of safety would not appear to indicate a significant stability problem in that they are only slightly below 1.0 and occur in a localized zone. The overall factors of safety across the embankment are well within acceptable limits, and there is no consistent pattern of low safety factors across the foundation that would indicate that significant movements of the embankment would occur."

5.0 Present Liquefaction Analysis

5.1 Introduction

Soil liquefaction is a process by which loose, saturated, granular deposits lose a significant portion of their shear strength due to porewater pressure buildup resulting from cyclic loading, such as that caused by an earthquake. Soil liquefaction can occur, leading to foundation bearing failures and excessive settlements when:

- the design ground acceleration is high
- the soil is saturated, i.e., below the water table, and
- the site soils are sands or silty sands in a loose or medium dense condition.

The present liquefaction analysis will consist of two parts:

- The methods and results of the previous seismic margin liquefaction study (Ref. 19) will be assessed with respect to the recently developed earthquake magnitude and site design acceleration.
- The state-of-the-art method described by Youd et al (Ref. 26) will be applied to the potentially liquefiable soils recorded in the recent ESP subsurface investigation (Ref. 1) and described in Ref. 14.

5.2 Acceptable Factor of Safety

Ref. 25, p.96, states, "There is no general agreement on the appropriate margin (factor) of safety, primarily because the degree of conservatism thought desirable at this point depends upon the extent of the conservatism already introduced in assigning the design earthquake. If the design earthquake ground motion is regarded as reasonable, a safety factor of 1.33 to 1.35...is suggested as adequate. However, when the design ground motion is excessively conservative, engineers are content with a safety factor only slightly in excess of unity."

Now, the original SSE on rock had a maximum acceleration of 0.12g. This was amplified to 0.18g in the soil (Ref. 2). The seismic margin maximum acceleration in soil (Ref. 19) was 0.30. Now we are using a maximum acceleration on rock of 0.39g, and have amplified it in the soil to a maximum acceleration of 0.89g based on using the maximum accelerations from a range of G_{max} values, and the high frequency earthquake. We can assume that our present design motion is conservative. Assume an allowable FS = 1.1.

5.3 Seismic Margin Study

The seismic margin study was a very thorough study, using state-of-the-art methods. In the ESP analysis, the seismic acceleration and magnitude are different, i.e., instead of the magnitude 6.8, peak acceleration 0.3g assumed in the seismic margin study, the ESP Project has a magnitude 5.4 and a peak acceleration of 0.89g (high frequency) and a magnitude 7.2 and a peak acceleration of 0.46g (low frequency).

The magnitude scaling factor (MSF – see Section 6.4.1 below) used in the seismic margin analysis was 1.60 for M = 6.8 in the modified version of the Seed and Idriss Simplified Procedure used in the main plant area. As noted in Section 5.4.1 below, for the low frequency spectrum, we use MSF = 1.13, and for the high frequency spectrum we use MSF = 2.5.

Thus, the ratio of FS against liquefaction for the seismic margin study in the main plant area to that for the ESP Project is the following:

Low Frequency:	(1.6 x 0.46)/(1.13 x 0.30) = 2.17
High Frequency:	(1.6 x 0.89)/(2.5 x 0.30) = 1.90

Thus, we can divide the FS values computed for the seismic margin study in the main plant area by 2.17 (low frequency) and 1.90 (high frequency) to obtain the FS for the ESP Project.

For the main plant area (i.e., Units 1&2 main plant), from Table 1 of Ref. 19:

	FS Seismic Margin Table 1 of Ref. 19		FS ESP Project			
			Low Frequency		High Frequency	
Location	Range	Average	Range	Average	Range	Average
Intake structure	1.63 to 2.56	1.98	0.75 to 1.18	0.91	0.86 to 1.35	1.04
Spray and valve house	1.57 to 3.42	2.13	0.72 to 1.58	0.98	0.83 to 1.80	1.12
Service building	1.54 to 3.51	2.24	0.71 to 1.62	1.03	0.81 to 1.85	1.18
Auxiliary building	1.58 to 3.51	2.15	0.73 to 1.62	0.99	0.83 to 1.85	1.13
Turbine building	1.56 to 3.42	2.05	0.72 to 1.58	0.94	0.82 to 1.80	1.08

It is not apparent from Ref. 19 that the liquefaction analysis of the SWR using the results of the cyclic triaxial tests employed any magnitude scaling factors. Thus, these results are not re-analyzed here.

5.4 Youd et al Study

In this analysis we will examine each sample of Zone IIA saprolite and determine the factor of safety against liquefaction based on the Youd et al (Ref. 26) paper. In this approach we will conservatively ignore the age, overconsolidation and mineralogy/fabric effects described in the Ref. 19 seismic margin study to decrease a soil's susceptibility to liquefaction. Samples that are cohesive and/or above the ground water table are considered non-susceptible to liquefaction.

As can be seen from the tables in Section 3.3, the zero period acceleration (ZPA) on rock for the low frequency case is about 0.15g. We will increase this to a maximum ground acceleration of 0.46g due to soil amplification effects, based on the results of the SHAKE analysis in Ref. 17, as discussed in Section 3.3. For the high frequency case, the ZPA on rock is about 0.39g, amplified to 0.89g at the ground surface.

As noted in Section 3.2, the groundwater level presently ranges from about 6 to 58 ft below existing ground level. The design groundwater level has been set at El. 265 ft to 270 ft in the powerblock for design of structures. For this analysis, we will use the highest water level measured in the respective boring or in an adjacent observation well as the basis for determining if a sample is above or below the water table.

Ref. 26 presents a state-of-the-art liquefaction methodology based on evolution of the "simplified procedure" over the past 25 years. This method will be used in the analysis. All of the equations in this section can be found in Ref. 26.

As stated in Ref. 26, "Calculation, or estimation, of 2 variables is required for evaluation of liquefaction resistance of soils: (1) the seismic demand on the soil layer, expressed in terms of CSR (cyclic stress ratio) and (2) the capacity of the soil to resist liquefaction, expressed in terms of CRR (cyclic resistance ratio)."

The factor of safety (FS) of a soil against liquefaction under known seismic conditions is CRR/CSR.

- <u>CSR</u>

$$CSR = (\tau_{av}/\sigma_{vo}') = 0.65\times(a_{max}/g)\times(\sigma_{vo}/\sigma_{vo}')\times r_d$$

where: τ_{av} = average shear stress induced in the soil by the earthquake
a_{max} = peak earthquake acceleration = 0.46g and 0.89g
σ_{vo} = total overburden pressure at depth being analyzed
σ_{vo}' = effective overburden pressure at depth being analyzed
r_d = reduction factor of induced shear stress with depth
g = acceleration of gravity

Ref. 26 indicates that: $r_d = 1 - 0.00765z$ for $z \leq 9.15$ m
$r_d = 1.174 - 0.0267z$ for $9.15 \text{ m} \leq z \leq 23$ m
z = depth below ground surface in m.

Note that Ref. 26 works in SI units.

- <u>CRR</u>

$$CRR = (\tau_{liq}/\sigma_{vo}')$$

τ_{liq} is the shear stress required to cause liquefaction of the soil and is a function of the strength of the soil. Three approaches (all empirical) are used in this analysis to account for the soil strength, namely the standard penetration test (SPT) N-value, cone penetration test (CPT) tip resistance, and shear wave velocity. Liquefaction analysis using N-values is presented in Section 5.4.1, analysis using CPT results is presented in Section 5.4.2, and analysis using shear wave velocity is presented in Section 5.4.3.

5.4.1 Analysis Using SPT Values

Fig. 2 from Ref. 26 shows the basis of determining CRR using N-values. The plot is for corrected N-value (see below) versus CSR or CRR. Each point represents CSR versus corrected N-value for a particular recorded case. The points represent cases where liquefaction has or has not occurred during an actual magnitude 7.5 earthquake. The solid points are where liquefaction occurred while the open points are where liquefaction did not occur. Three lines have been drawn to delineate between liquefaction and non-liquefaction situations. It has been observed that liquefaction of sands depends on the fines content of the sands. The higher the fines content, the less likely it is that the sand will liquefy. As shown on the plot, the 3 lines represent fines contents of ≥ 5%, 15% and 35%. Knowing the corrected N-value, the CRR for a magnitude 7.5 earthquake is the vertical axis reading where the corrected N-value intersects with the appropriate fines line. An example will be given after we have discussed the CRR curves, the corrected N-value, the magnitude scaling factor for magnitudes other than 7.5, and the K_σ effects for non-linear confining stress.

- CRR Curves

As noted above and shown on the plot on Fig. 2 of Ref. 26, there are 3 curves representing different fines contents to compute CRR. The ≥ 5% curve, known as the clean sand base curve, can be represented with reasonable accuracy for a magnitude 7.5 earthquake as follows:

$$CRR = (1/(34-N_1)) + (N_1/135) + (50/(10N_1 + 45)^2) - 0.005$$

Where N_1 = corrected N-value (see below). Equation is valid for $N_1<30$.

Although Ref. 26 gives modifications to equation 5 to take account of fines content, these are quite complex. For our liquefaction analysis, we will read off the CRR curves for fines contents >5%.

- Corrected N-Values

There are several corrections that need to be made to the N-value recorded in the boring so that it can become $(N_1)_{60}$ (or N_1) as we shall refer to it in this calculation and be used to determine CRR.

Thus, $(N_1)_{60} = N_1 = N_m C_N C_E C_B C_R C_S$

Where N_m is the SPT N-value measured in the field and the C factors are correction factors described below.

Overburden Pressure Factor C_N

This normalizes the N-value to a standard effective overburden pressure of about 100 kPa (1 atmosphere).

$$C_N = 2.2/(1.2 + \sigma_{vo}'/100)$$

where σ_{vo}' is the effective overburden pressure in kPa.

Energy Ratio Factor C_E

This factor is used when a method other than the rope and cathead method is employed. In the North Anna ESP borings, only the standard cathead and rope were used, thus $C_E = 1$.

Borehole Diameter Factor C_B

This factor is 1.0 for boreholes between 65 and 115 mm diameter (2.5 and 4.5 in.). The bit used in the rotary wash borings was about 3.5 in. diameter, and so $C_B = 1$.

Rod Length Factor C_R

This factor is 0.95 for rod lengths between 6 and 10 m and 1.0 for rod lengths between 10 and 30 m. Use $C_R = 1.0$.

Sampler Factor C_S

This factor is 1.0 for an SPT sampler with liners and 1.1 to 1.3 for an SPT sampler without liners. Conservative to assume $C_S = 1.0$ for the typical case of a sampler without liners.

Thus, for the North Anna ESP liquefaction analysis:

$$N_1 = NC_N$$

- <u>Magnitude Scaling Factor (MSF)</u>

As noted above, the CRR plot in Fig. 2 of Ref. 2 was derived for a magnitude 7.5 earthquake. For smaller magnitude earthquakes, the resistance to liquefaction (CRR) will be greater, and for larger magnitude earthquakes, the resistance to liquefaction (CRR) will be less. Thus CRR is multiplied by the MSF. For a magnitude M = 7.5 earthquake, the MSF = 1.

For the high frequency case, M = 5.4. Ref. 26, Table 3 shows various MSF values for M = 5.5. The recommended lower and upper bound MSF values are 2.2 and 2.8 in Ref. 26. Assume an average of these, i.e., MSF = 2.5 for M = 5.5. Since M = 5.5 is the lowest magnitude tabulated in Ref. 26, use MSF = 2.5 for the M = 5.4 earthquake.

For the low frequency earthquake, M = 7.2. Ref. 26, Table 3 shows various MSF values for M = 7.0. The recommended lower and upper bound MSF values for M = 7.0 are 1.19 and 1.25 in Ref. 26. Assume an average of these, i.e., MSF = 1.22 for M = 7.0. Now MSF = 1 for M = 7.5. Thus for M = 7.2, we can use MSF = 1 + (0.6 x 0.22) = 1.13.

When we include the MSF in the equation for factor of safety, we get:

$$FS = (CRR_{7.5}/CSR) \times MSF$$

where $CRR_{7.5}$ is the CRR for a 7.5 magnitude earthquake.

Now, as we have seen above, CSR is directly proportional to peak ground acceleration, a_{max}.

Thus, FS is directly proportional to MSF/ a_{max}.

For the high frequency spectrum, MSF/ a_{max} = 2.5/0.89 = 2.81
For the low frequency spectrum, MSF/ a_{max} = 1.13/0.46 = 2.46

Thus, the low frequency spectrum will give lower FS values than the high frequency spectrum.

- Correction for High Overburden Stresses and Age

Ref. 26 notes that Seed introduced a correction factor K_σ to account for non-linearity between CRR and effective overburden pressure. K_σ becomes less than 1 for overburden pressures greater than 2 ksf or 100 kPa (but is assumed as 1 for overburdens less than 2 ksf or 100 kPa).

When we include the K_σ, we get:

$$FS = (CRR_{7.5}/CSR) \times MSF \times K_\sigma$$

Ref. 26 indicates $K_\sigma = (\sigma_{vo}'/2\ \text{ksf})^{(f-1)}$ or $K_\sigma = (\sigma_{vo}'/100\ \text{kPa})^{(f-1)}$
For a relative density of about 60% (reasonable for the Zone IIA saprolite), f = 0.7, according to Ref. 26. Now at σ_{vo}' = 2 ksf, K_σ = 1. At, say, σ_{vo}' = 4 ksf, K_σ = 0.81.

Ref. 26 notes that "..some knowledgeable engineers have omitted application of the K_σ factor as partial compensation for the unquantified, but substantial increase of liquefaction resistance with age." As noted earlier, The Zone IIA saprolites are considerably older than those on which the analysis method was based. Thus, inclusion of the K_σ reduction factor here without making an allowance for the age of the deposit can be considered conservative.

- Example Showing Least Factor of Safety

Liquefaction analysis was run for samples taken in the recent ESP borings (Ref. 1). The analysis was limited to samples at or below the water table, and that consisted of potentially liquefiable. The FS values in the potentially liquefiable samples ranged from 0.91 to 1.35 for the low frequency earthquake, and 1.04 to 1.54 for the high frequency earthquake.

Boring B-803, SPT sample at 19.5 ft (5.94 m) depth gave the least FS of 0.91 using the low frequency earthquake.

- In the boring log, sample is described as an orange, whitish tan, and grayish white, firm to dense, micaceous silty, fine to coarse sand.
- USCS classification: SM
- No sieve analysis performed. Conservative to assume 15% passing #200 sieve.
- Water level in boring is about 21 ft. Water level in OW-846, about 90 ft from B-803, is about 18.7 ft below ground level (June 2003 reading). Water level taken as 19.5 ft depth (5.94 m) to include the sample at 19.5 ft depth.
- Peak ground acceleration = 0.46g with M = 7.2.
- N = 18 bpf (or blows/300 mm).
- Unit weight of 125 pcf (19.65 kN/m^3) (from Ref. 14).

$CSR = (\tau_{av}/\sigma_{vo}') = 0.65\times(a_{max}/g)\times(\sigma_{vo}/\sigma_{vo}')\times r_d$

$\sigma_{vo} = 5.94 \text{ x } 19.65 = 116.8 \text{ kPa}$
$\sigma_{vo}' = 116.8 \text{ kPa}$

$r_d = 1 - 0.00765z$
$= 1 - (0.00765 \text{ x } 5.94) = 0.955$

$CSR = (\tau_{av}/\sigma_{vo}') = 0.65 \text{ x } 0.46 \text{ x } (116.8/116.8) \text{ x } 0.955 = 0.286$

$N_1 = NC_N$
$C_N = 2.2/(1.2 + \sigma_{vo}'/100)$
$= 2.2/(1.2 + 116.8/100) = 0.93$

$N_1 = 18 \times 0.93 = 16.74$ bpf. Use $N_1 = 17$ bpf.

Look at CRR chart, with $N_1 = 17$ bpf, with 15% fines, CRR = 0.24.

Now, for M = 7.2, MSF = 1.13

Also, $K_\sigma = (\sigma_{vo}'/100)^{(f-1)}$
Assuming 60% relative density, f = 0.7, $K_\sigma = (116.8/100)^{(f-1)} = 0.955$

Thus, CRR = 1.13 x 0.955 x 0.24 = 0.259

- FS = CRR/CSR = 0.259/0.286 = 0.91

5.4.2 Analysis Using CPTs

In this analysis we will examine each CPT log from Ref. 1 and determine the factor of safety against liquefaction based on (1) the Youd et al (Ref. 26) paper, and (2) a paper by Stark and Olson (Ref. 27). The method of CPT interpretation is based on Ref. 28. In this approach we will conservatively ignore the age, overconsolidation and mineralogy/fabric effects described in the Ref. 19 seismic margin study to decrease a soil's susceptibility to liquefaction. Soils that are above the ground water table and/or are interpreted in the CPT as being cohesive are considered non-susceptible to liquefaction.

As in Section 5.4.1, we will the increase the 0.15g acceleration on rock for the low frequency case to a maximum ground acceleration of 0.46g due to soil amplification effects, based on the results of the SHAKE analysis in Ref. 17, as discussed in Section 3.3. For the high frequency case, the ZPA on rock is about 0.39g, amplified to 0.89g at the ground surface.

As noted in Section 3.2, the groundwater level is influenced by the water level in Lake Anna, with water depth varying from about 6 to 58 ft below ground surface. For this analysis, we use the highest water level measured in nearby borings or observation wells as the basis for determining if a sample is above or below the water table. The following table gives the depth of each CPT and the estimated water table depth.

CPT No.	Total Depth	Groundwater Depth	Nearest Boring/OW
821	4 ft	>4ft	B-802/OW-844
822A	22.6 ft	9 ft	B-805
823	32.4 ft	18 ft	B-803/OW-846
824	4 ft	>4 ft	B-803/OW-846
825	52.5 ft	28 ft	B-804
827	57.7 ft	Assume 25 ft	--

CPT No.	Total Depth	Groundwater Depth	Nearest Boring/OW
828	5 ft	>5 ft	--
830	15.8 ft	18 ft	B-803/OW-846

CPTs 821, 824, 828 and 830 did not reach below the ground water table.

For the low frequency case, with a_{max} = 0.46g and MSF = 1.13, there are locations where the computed FS is less than 1.1, as noted below. The CPT results were recorded at 0.5 ft intervals.

CPT-822A No location
CPT-823 Between 18.25 ft and 22.75 ft depth (FS = 0.43 to 0.91)
CPT-825 Between 28.25 ft and 49.25 ft (FS = 0.50 to 1.07)
CPT-827 Between 25.25 ft and 29.75 ft depth (FS = 0.38 to 0.65)
Between 53.25 ft and 53.75 ft depth (FS = 0.76 to 1.02)

For the high frequency case, with a_{max} = 0.89g and MSF = 2.5, there are locations where the computed FS is less than 1.1, as noted below. The CPT results were recorded at 0.5 ft intervals.

CPT-822A No location
CPT-823 Between 18.25 ft and 22.75 ft depth (FS = 0.49 to 1.04)
CPT-825 Between 28.25 ft and 46.75 ft (FS = 0.57 to 1.08)
CPT-827 Between 25.25 ft and 29.75 ft depth (FS = 0.43 to 0.74)
Between 53.25 ft and 53.75 ft depth (FS = 0.87 to 0.89)

CPT-827 at 29.75 gave the least FS of 0.38 using the Youd et al (Ref. 26) method. The Stark and Olson (Ref. 27) gave a FS = 0.43. We will look at both methods.

- <u>Youd et al (Ref. 26) Method</u>

The CRR computation is more complex for the CPT than the N-value approach described in the previous section.

For a magnitude 7.5 earthquake,

$CRR_{7.5} = 0.833 \times (q_{c1N})_{cs}/1{,}000 + 0.05$ for $(q_{c1N})_{cs} < 50$
$CRR_{7.5} = 93 \times ((q_{c1N})_{cs}/1{,}000)^3 + 0.08$ for $50 \leq (q_{c1N})_{cs} < 160$

where: q_{c1N} is the normalized cone penetration resistance,
and: $(q_{c1N})_{cs}$ is q_{c1N} for silty sands converted to an equivalent clean sand value.

$(q_{c1N})_{cs} = K_c q_{c1N}$

Where K_c is the correction factor for grain size characteristics (later).

Before defining K_c, we will define q_{c1N}, the normalized cone penetration resistance.

$q_{c1N} = C_Q(q_c/P_a)$
$C_Q \; = (P_a/\sigma'_{vo})^n$

where: C_Q = normalizing factor for cone penetration resistance
P_a = 1 atmosphere of pressure in same units as σ'_{vo}
σ'_{vo} = effective vertical overburden pressure at depth being analyzed
q_c = cone tip resistance
n = exponent that varies with soil type; varies from 0.5 to 1, with 1 characteristic of clays

Note that at shallow depths, C_Q becomes large because of low overburden pressure; however, values of > 1.7 should not be applied.

Now, K_c is a function of the soil behavior type index, I_c

where: $I_c = ((3.47 - \log Q)^2 + (1.22 + \log F)^2)^{0.5}$
and: $Q = ((q_c - \sigma_{vo})/ P_a) \times (P_a/\sigma'_{vo})^n$
$F = f_s/(q_c - \sigma_{vo}) \times 100\%$
f_s = cone sleeve resistance

For $I_c \leq 1.64$, $K_c = 1.0$
For $I_c \geq 1.64$, $K_c = -0.403I_c^4 + 5.581I_c^3 - 21.63I_c^2 + 33.75I_c - 17.88$.

- Example Showing Least Factor of Safety

We can demonstrate the application of the above equations by looking at the least FS achieved in any of the 7 CPT soundings performed for the ESP. This occurred at 29.75 ft depth in CPT-827.

- Tip resistance q_c = 39.11 tsf = 78.22 ksf
- Sleeve friction f_s = 0.562 tsf = 1.124 ksf
- Water level in CPT assumed as 25 ft
- Friction ratio $f_s/q_c \times 100\%$ = 1.437
- Tip resistance and friction ratio charts indicate soil is silty sand to sandy silt
- Peak ground acceleration = 0.46g with M = 7.2
- Unit weight of soil is assumed as 125 pcf

The computation of CSR is the same as in the N-value analysis. We can work in the ft-lb system here.

$CSR = (\tau_{av}/\sigma_{vo}') = 0.65\times(a_{max}/g)\times(\sigma_{vo}/\sigma_{vo}')\times r_d$

$\sigma_{vo} = (29.75 \times 125) = 3719$ psf $= 3.719$ ksf.
$\sigma_{vo}' = (25 \times 125) + 4.75 \times (125 - 62.4) = 3422$ psf $= 3.422$ ksf

$r_d = 1 - 0.00765z$
$= 1 - (0.00765 \times 29.75/3.281) = 0.93$

$CSR = (\tau_{av}/\sigma_{vo}') = 0.65 \times 0.46 \times (3.719/3.422) \times 0.93 = 0.302$

The parameters n and I_c are good starting points for the analysis of CRR.

Assume n = 1, making $Q = ((q_c - \sigma_{vo})/ P_a) \times (P_a/\sigma'_{vo}) = (q_c - \sigma_{vo})/\sigma'_{vo}$
$\sigma_{vo} = 3.719$ ksf.
$\sigma'_{vo} = 3.422$ ksf
$Q = (q_c - \sigma_{vo})/\sigma'_{vo} = (78.22 - 3.719)/3.422 = 21.77$
$F = f_s/(q_c - \sigma_{vo}) \times 100\% = 1.124/(78.22 - 3.719) \times 100\% = 1.51$
$I_c = ((3.47 - \log Q)^2 + (1.22 + \log F)^2)^{0.5}$
$= ((3.47 - \log 21.77)^2 + (1.22 + \log 1.51)^2)^{0.5}$
$= ((3.47 - 1.338)^2 + (1.22 + 0.179)^2)^{0.5} = (4.545 + 1.957)^{0.5} = 2.55$

Since $I_c < 2.6$, recalculate with n = 0.5

(Note that if $I_c > 2.6$, the soil is classified as clayey, and considered too clay-rich to liquefy. At $I_c = 2.6$, we are very close to that situation.)

$Q = ((q_c - \sigma_{vo})/ P_a) \times (P_a/\sigma'_{vo})^{0.5}$
$= ((78.22 - 3.719)/2.17 \times (2.17/3.422)^{0.5}$ using $P_a = 2.17$ ksf.
$= 34.33 \times 0.796 = 27.33$
$I_c = ((3.47 - \log Q)^2 + (1.22 + \log F)^2)^{0.5}$
$= ((3.47 - \log 27.33)^2 + (1.22 + \log 1.51)^2)^{0.5}$
$= ((3.47 - 1.437)^2 + (1.22 + 0.179)^2)^{0.5} = (4.133 + 1.957)^{0.5} = 2.468$

$K_c = -0.403I_c^4 + 5.581I_c^3 - 21.63I_c^2 + 33.75I_c - 17.88.$
$= (-0.403 \times 2.468^4) + (5.581 \times 2.468^3) - (21.63 \times 2.468^2) + (33.75 \times 2.468) - 17.88$
$= -14.952 + 83.897 - 131.749 + 83.295 - 17.88 = 2.611$

Now, $C_Q = (P_a/\sigma'_{vo})^n$
Using n = 0.5,
$C_Q = (2.17/3.422)^{0.5} = 0.796$

$q_{c1N} = C_Q(q_c/P_a)$

$= 0.796 \times 78.22/2.17 = 28.69$

$(q_{c1N})_{cs} = K_c q_{c1N}$
$= 2.611 \times 28.69 = 74.91$

$CRR_{7.5} = 93 \times ((q_{c1N})_{cs}/1{,}000)^3 + 0.08$
$= 93 \times ((74.91/1{,}000)^3 + 0.08$
$= 0.120$

Now, $K_\sigma = (\sigma_{vo}'/2)^{(f-1)}$ where σ_{vo}' is in ksf.

Assuming 60% relative density, f = 0.7, $K_\sigma = (3.422/2)^{(f-1)} = 0.85$

Taking the MSF and K_σ into account:
$FS = (CRR_{7.5}/CSR) \times MSF \times K_\sigma$
$= (0.119/0.302) \times 1.13 \times 0.85 = 0.38$

- Stark and Olson (Ref. 27)

The Stark and Olson method is considerably less complex than Youd et al, and follows more closely the N-value approach.

As with Youd et al, Stark and Olson use:

$CSR = (\tau_{av}/\sigma_{vo}') = 0.65 \times (a_{max}/g) \times (\sigma_{vo}/\sigma_{vo}') \times r_d$

All terms are as defined before except r_d:
$r_d = 1 - 0.012z$ where z is in meters.

For CRR, instead of using N_1, Stark and Olson use corrected CPT tip resistance, q_{c1} (MPa).
$q_{c1} = C_q q_c$

$C_q = 1.8/(0.8 + (\sigma'_{vo}/\sigma'_{ref})$

σ'_{ref} is the same as P_a in Youd et al, i.e., a reference stress of 1 atmosphere. Here we can approximate to 2 ksf.

Stark and Olson provide a series of plots of seismic shear stress ratio (equivalent to CRR for M = 7.5) versus q_{c1} for clean sand (< 5% fines), silty sand (5% < fines < 35%), and sandy silt (fines > 35%). Figure 9 of Stark and Olson (included here as Attachment 4) shows seismic shear stress ratio (or CRR) versus q_{c1} for clean sand, silty sand and sandy silt.

- Example : CPT-827 at 29.75 Ft

    - Tip resistance $q_c = 39.11$ tsf $= 78.22$ ksf $= 3.743$ MPa
    - Sleeve friction $f_s = 0.562$ tsf $= 1.124$ ksf
    - Water level in CPT assumed as 25 ft
    - Friction ratio $f_s/q_c \times 100\% = 1.437$
    - Tip resistance and friction ratio charts indicate soil is silty sand to sandy silt
    - Peak ground acceleration = 0.46g with M = 7.2
    - Unit weight of soil is assumed as 125 pcf

$CSR = (\tau_{av}/\sigma_{vo}') = 0.65\times(a_{max}/g)\times(\sigma_{vo}/\sigma_{vo}')\times r_d$

$\sigma_{vo} = (29.75 \times 125) = 3719$ psf $= 3.719$ ksf.
$\sigma_{vo}' = (25 \times 125) + 4.75 \times (125 - 62.4) = 3422$ psf $= 3.422$ ksf

$r_d = 1 - 0.012z$
$= 1 - (0.012 \times 29.75/3.281) = 0.89$

$CSR = (\tau_{av}/\sigma_{vo}') = 0.65 \times 0.46 \times (3.719/3.422) \times 0.89 = 0.289$

$C_q = 1.8/(0.8 + (\sigma'_{vo}/\sigma'_{ref})$
$= 1.8/(0.8 + (3.422/2) = 0.717$

$q_{c1} = C_q q_c$
$= 0.717 \times 3.743 = 2.68$ MPa.

From Figure 9 of Stark and Olson, for $q_{c1} = 2.68$ MPa, and using the sandy silt curve:

$CRR_{7.5} = 0.13$.

Taking the MSF and K_σ into account:
$FS = (CRR_{7.5}/CSR) \times MSF \times K_\sigma$
$= (0.13/0.289) \times 1.13 \times 0.85 = 0.43$

5.4.3 Analysis Using Shear Wave Velocity

Youd et al provide a plot of CRR versus overburden stress-corrected shear wave velocity (Fig. 9 of Ref. 26).

The shear wave velocity corrected for overburden V_{s1}, is

$$V_{s1} = V_s\,(P/\sigma_{vo}')^{0.25}$$

where P is one atmosphere, approximately 2 ksf.

The overburden pressure and V_{s1} values are tabulated below.

Water table was assumed at 10 ft depth.

Referring to Fig. 9 of Ref. 26, we see that, even for clean sands, the limit for liquefaction is about 210 m/s or 689 ft/sec. The lowest corrected value tabulated on the next page is 787 ft/sec. Thus, the average shear wave velocities indicate no liquefaction.

Note that the table of properties in Table1 shows a range of shear wave velocities from 600 to 1,350 ft/sec, with a design value of 950 ft/sec. The 600 ft/sec. is about 37 percent lower than the 950 ft/sec. If we lower the V_{s1} values by 37%, we see that most of the values down to 20 ft depth are below 689 ft/sec., i.e., liquefiable.

Depth, ft	**V_s, ft/sec**	**σ_{vo}', ksf**	**V_{s1}, ft/sec**
0.0	700	0	-
2.5	700	0.313	1122
5.0	700	0.625	936
7.5	700	0.938	846
10.0	700/950	1.25	787/1068
12.5	950	1.41	1037
15.0	950	1.56	1011
17.5	950	1.72	986
20.0	950/1200	1.88	965/1219
22.5	1200	2.03	1196
25.0	1200	2.19	1173
27.5	1200	2.35	1153
30.0	1200/1600	2.5	1135/1513
35.0	1600	2.81	1469
40.0	1600/2000	3.13	1431/1788
45.0	2000	3.44	1746
50.0	2000	3.75	1709

Depth, ft	V_s, ft/sec	σ_{vo}', ksf	V_{s1}, ft/sec
55.0	2000/3300	4.07	1675/2763
60.0	3300	4.38	2712
65.0	3300	4.69	2666
70.0	3300	5.01	2624
Base	-		

6.0 Conclusions

Soils that were classified as ML, CL, MH and CH were assumed to be non-liquefiable. Soils above the ground water table were assumed not to liquefy. Zone III weathered rock will not liquefy. Zone IIB sand, with an average N-value of at least 100, will not liquefy. Based on the fabric and mineralogy of the Zone IIA silty sands and sands, and their age (0.8 to 1.6 million years), their potential for liquefaction is low. The liquefaction analyses performed in this calculation were limited to the Zone IIA silty sands and sands.

Two sets of seismic parameters were used for the liquefaction analysis: low frequency with magnitude of 7.2 and a maximum ground acceleration of 0.46g; and high frequency with magnitude of 5.4 and a maximum ground acceleration of 0.89g. Taking magnitude scaling factor into account, the low frequency parameters give slightly lower FS values than the high frequency parameters. An acceptable factor of safety against liquefaction is 1.1 or more.

The 1994 Geotechnics comprehensive liquefaction analysis (Ref. 19) was revisited using the ESP low frequency seismic parameters. All of the areas analyzed within the 800 ft x 2,000 ft envelope for the new units had lower bound FS against liquefaction of <1.1 and upper bound values > 1.1. The average FS values were close to but lower than 1.1. The analysis was not re-run for the SWR area – it is anticipated there would be some areas with FS < 1.1 in that area.

A state-of-the-art liquefaction analysis (Ref. 26) was performed on all of the SPT silty sand and sand samples below groundwater from the recent ESP Project investigation. FS was lower than 1.1 in one sample.

State-of-the-art liquefaction analyses (Refs. 26 and 27) were performed on all of the CPTs below groundwater from the recent ESP Project investigation. In two of the CPTS, there were 4.5-ft thick zones with FS values <1.1. In one CPT, there was a 21-ft thick layer with FS values less than 1.1. Note that these analyses did not take into account the fabric and age of the soils.

A state-of-the-art liquefaction analysis (Ref. 26) was performed using shear wave velocity values corrected for overburden. The results indicated no liquefaction using the design shear wave velocity values, regardless of acceleration level. When the lower bound shear wave velocity values were applied for the Zone IIA saprolites, most of the soils in the top 20 ft were shown to be liquefiable.

In summary, the age, fabric, texture and mineralogy of the Zone IIA saprolite indicate that it has a low potential for liquefaction. These factors were not taken into account in the analyses that were performed on samples from the ESP investigation. The analyses based on SPT N-values indicated the possibility of isolated liquefaction. The CPT analysis indicated that liquefaction could occur in relatively thick layers. The analyses based on shear wave velocity values showed no liquefaction when the design shear wave velocities were used, but extensive liquefaction in the top 20 ft when the lower bound shear wave velocities were applied. In conclusion, the analysis shows liquefaction is likely to occur in areas of the Zone IIA saprolites under either a low frequency seismic event with M = 7.2 and a peak ground acceleration of 0.46g, or a high frequency seismic event with M = 5.4 and a peak ground acceleration of 0.89g; the analysis ignores the beneficial effects of the age, fabric, texture and mineralogy of the soil. (Note that, because of significant settlement potential of the Zone IIA saprolite under static loads, Ref. 15 recommends that, where significant structures (including any safety-related structure) are founded on the Zone IIA saprolite, the saprolite must be improved with stone columns or equivalent. This improvement would be designed to ensure that the improved soil had a factor of safety against liquefaction equal to or greater than 1.1 (SSAR Section 2.4.8.2 and DG-1105), at the safe shutdown earthquake ground motion.)

7.0 Draft Regulatory Guide DG-1105

Before and during the foregoing analyses, DG-1105 (Ref. 29) was consulted. The analysis conforms closely to the DG-1105 guidelines. It should be noted that DG-1105 was published before the publication of Youd et al (Ref. 26), which is considered to be the present state-of-the-art. (The "et al" in this case consists of 20 of the most prominent names in the liquefaction business.) Thus, at times, DG-1105 seems somewhat outdated, particularly with a lack of emphasis on in-situ methods.

Under "Screening Techniques for Evaluation of Liquefaction Potential", DG-1105 lists the most commonly observed liquefiable soils – fluvial-alluvial deposits, eolian sands and silts, beach sands, reclaimed land, and uncompacted hydraulic fills. The North Anna saprolite clearly does not come under any of these headings. In the same section, DG-1105 indicates that CL-ML and SM-SC soils can be considered potentially liquefiable. Our analysis treats SM-SC soils saprolites as potentially liquefiable. However, CL-ML (and ML) saprolites are considered non-liquefiable because of their mineralogy/texture. The same section confirms that potentially liquefiable soils that are currently above the groundwater table (gwt), are above the historic high gwt (more easy to guarantee since levels generally rose when Lake Anna was created), and cannot reasonably be expected to become saturated, pose no potential liquefaction hazard. Similarly, DG-1105 indicates that potentially liquefiable soils may not pose a liquefaction risk to the facility if they are insufficiently thick (e.g., 4.5-ft thickness in CPT-823 and 827) and of limited lateral extent.

Under "Procedures for Evaluating Liquefaction Susceptibility", DG-1105 lists CPTs, SPTs, and cyclic triaxial, and shear wave velocity tests as acceptable methods. All are used in our analyses.

Under "Factors of Safety Against Liquefaction", a FS of 1.1 appears to be the lowest value acceptable. This is in line with our analysis.

Table 1. Summary of Geotechnical Engineering Properties

Stratum	IIA		IIB	III	III-IV	IV
Description	**Coarse-grained Saprolite**	**Fine-grained Saprolite**	**Saprolite w/10 to 50% Core Stone**	**Moderately to Highly Weathered Quartz Gneiss w/Biotite**	**Slightly to Moderately Weathered Quartz Gneiss w/Biotite**	**Fresh to Slightly Weathered Quartz Gneiss w/Biotite**
Rock properties						
Recovery, %	—	—	—	60	90	100
RQD, %	—	—	—	20	50	95
Unconfined compressive strength, ksi	—		—	0.6	4	12
USCS symbol	SP, SM, SC	ML, CL, MH, CH	Mainly SM	—	—	—
Range of fines content, %	15 to 45	—	—	—	—	—
Natural moisture content, w, %	—	26	—	—	—	—
Undrained shear strength, c_u, ksf	—	2.0	—	—	—	—
Effective cohesion, c′, ksf	0.25	0.5	—	—	—	—
Effective friction angle, ϕ', degrees	30	25	40	—	—	—
Total unit weight, γ, pcf	125		130	145	163	163
SPT N-value, N_{60}, blows/ft	20		100	—	—	—
Shear and compression wave velocity						

Table 1. Summary of Geotechnical Engineering Properties

Stratum	IIA		IIB	III	III-IV	IV
Description	**Coarse-grained Saprolite**	**Fine-grained Saprolite**	**Saprolite w/10 to 50% Core Stone**	**Moderately to Highly Weathered Quartz Gneiss w/Biotite**	**Slightly to Moderately Weathered Quartz Gneiss w/Biotite**	**Fresh to Slightly Weathered Quartz Gneiss w/Biotite**
Shear wave velocity range, ft/sec	600 to 1350		No range available	1500 to 4500	2500 to 4500	4000 to 8000
Shear wave velocity average, ft/sec	950		1600	2000	3300	6300
Compression wave velocity average, ft/sec	2100		3500	4500	7400	14,000
Elastic and shear moduli						
Elastic modulus (high strain), E_{hs}	1200 ksf		3500 ksf	120 ksi	1000 ksi	3750 ksi
Elastic modulus (low strain), E_{ls}	9500 ksf		28,000 ksf	300 ksi	1000 ksi	3750 ksi
Shear modulus (high strain), G_{hs}	450 ksf		1300 ksf	50 ksi	375 ksi	1400 ksi
Shear modulus (low strain), G_{ls}	3500 ksf		10,000 ksf	125 ksi	375 ksi	1400 ksi
Consolidation characteristics						
Recompression ratio, RR	0.015		—	—	—	—
Coeff. of secondary compression, C_α	0.0008		—	—	—	—
Coeff. of subgrade reaction, k_1, kcf	230		1500	-	-	-

Table 1. Summary of Geotechnical Engineering Properties

Stratum	IIA		IIB	III	III-IV	IV
Description	**Coarse-grained Saprolite**	**Fine-grained Saprolite**	**Saprolite w/10 to 50% Core Stone**	**Moderately to Highly Weathered Quartz Gneiss w/Biotite**	**Slightly to Moderately Weathered Quartz Gneiss w/Biotite**	**Fresh to Slightly Weathered Quartz Gneiss w/Biotite**
Coefficient of sliding against concrete	0.35		0.45	0.6	0.65	0.7
Poisson's ratio, μ (high strain)	0.35		0.3	0.33	0.33	0.33
Static earth pressure coefficients						
Active, K_a	0.33		0.22	—	—	—
Passive, K_p	3.0		4.6	—	—	—
At-rest, K_o	0.5		0.36	—	—	—
Hydraulic conductivity, cm/sec	5×10^{-4}		—	—	—	—

Note:Dash denotes no design parameter given