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Date: 8/5/04 2:38PM
Subject: Outgoing Dominion Correspondence to NRC (Second part) - North Anna ESPResponse to RAI No. 5 (SN 04-347)

(See attached file: 04-347_Letter&Attonly_Part2of2.pdf)

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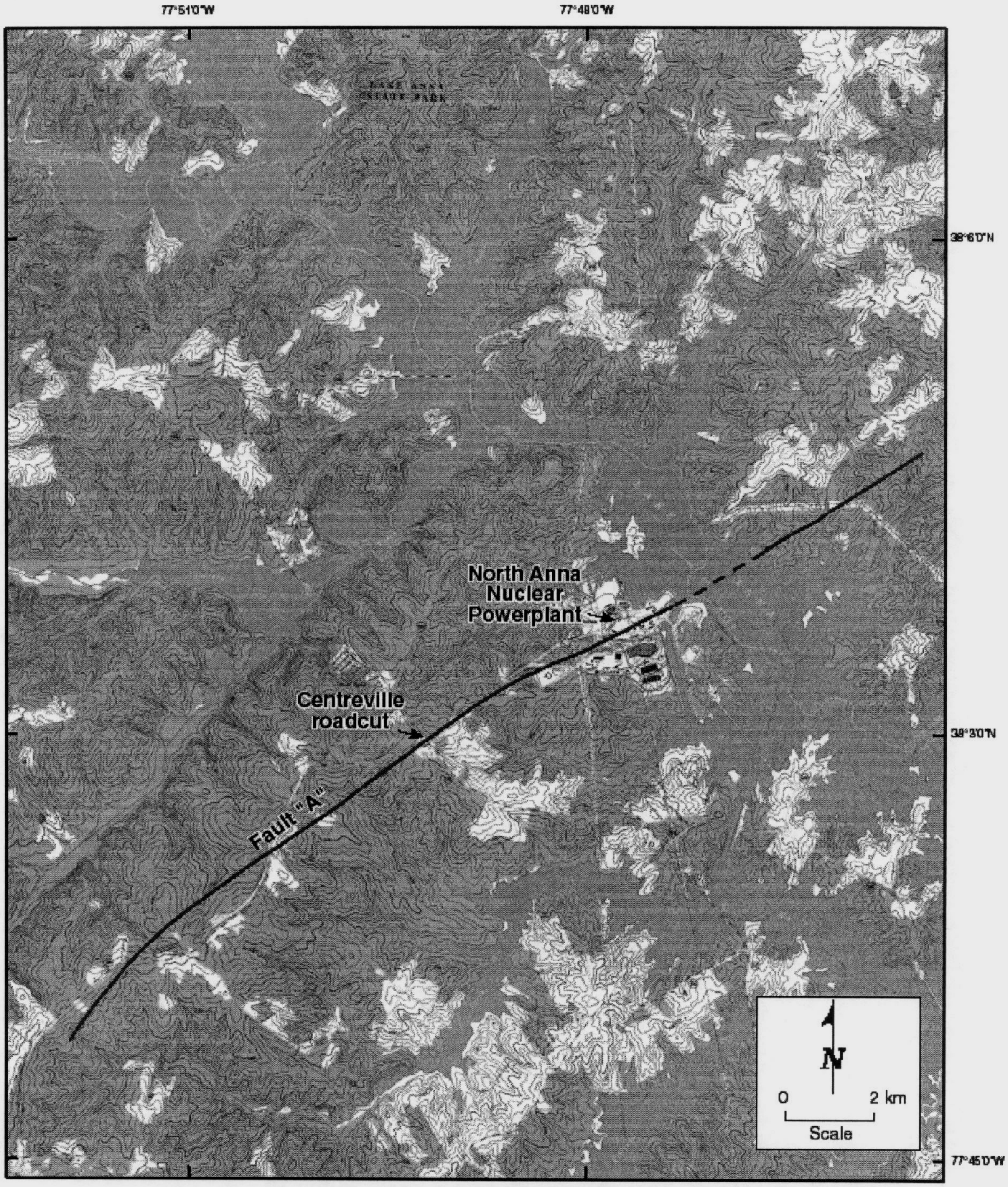

Figure 1. Topographic map of the site location with Fault "A" as located by Mixon et al. 2000. Lake Anna West 7.5-minute USGS quadrangle base map reduced to 1:60,000 scale, contour interval is 10 feet.

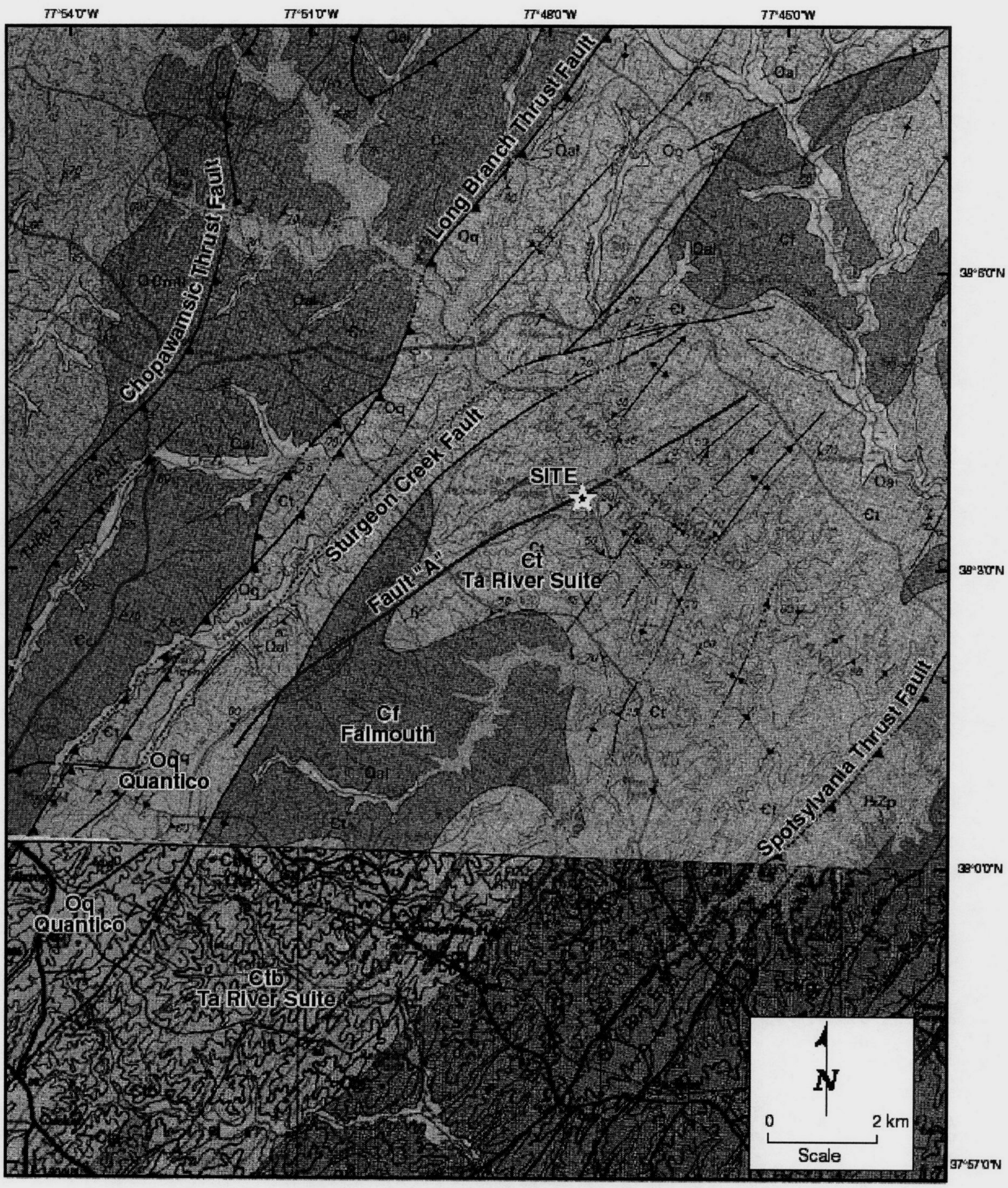

Figure 2. Geologic map of the site area with Fault "A" highlighted. Geology taken from Mixon et al. (2000) Fredericksburg 100k quadrangle and Marr (2002) western portion of Richmond 100k quadrangle.

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

SSAR Section 2.5.4 states that additional structure-specific exploration and testing would be performed during detailed engineering and would be described in the combined license (COL) application. Regulatory Guide 1.132 recommends borings at 100 ft spacings for major structures. Please provide the basis (especially given the documented presence of severely weathered, fractured and jointed intervals in the Zone III-IV and Zone IV rock) for concluding that the subsurface conditions in the southwest part of the ESP footprint (an area roughly 1000 ft by 500 ft, in which there have apparently been no borings) do not materially differ from conditions in the adjacent areas where borings have been drilled.

<u>Response</u>

The North Anna site is underlain by a consistent geologic profile (bedrock of the Ta River Metamorphic Suite), as described in SSAR Section 2.5.1.2.3 and illustrated in SSAR Figures 2.5.4-11 (plan view) and 2.5.4-17 (section). This rock extends to a depth of several thousand feet. The soils that overlie this bedrock are the results of in-situ weathering of the rock, and range from saprolites with up to 50 percent of the core stone remaining, to a veneer of residual soils with all structure of the parent rock lost. This profile is described in SSAR Section 2.5.4.2.2, that is:

Zone I	Residual clays and clay silts – all structures of parent rock are lost.
Zone IIA	Saprolite – core stone less than 10 percent of volume of overall mass.
Zone IIB	Saprolite – core stone 10 to 50 percent of volume of the overall mass.
Zone III	Weathered rock – core stone more than 50 percent of volume of the overall mass.
Zone IV	Parent rock – slightly weathered to fresh rock below zone of isolated core stones.

The materials overlying the parent Zone IV rock represent a continuously more pronounced form of in-place weathering. An additional zone, termed Zone III-IV, has been adopted to represent this slightly to moderately weathered rock.

The 145 borings performed throughout the North Anna site (including 7 for the ESP subsurface investigation) indicated a consistent overall subsurface profile, with expected variations in the thickness of the various strata.

The anticipated soil and rock profile in the roughly 1,000 feet x 500 feet area referenced in the RAI is discussed in the next paragraphs.

Soil Profile

As noted previously, all of the natural soils onsite are residual materials derived from in-situ weathering of the underlying bedrock. These soils consist almost entirely of the Zone II saprolites – less than 1 percent of the soils encountered in the borings were Zone I residual soils. The existing topography in the 1,000 feet by 500 feet area is gently rolling, ranging from about Elevation 300 to 330 feet, very typical of the site topography. Excavation to the proposed plant elevation at Elevation 271 feet would be through the Zone II saprolites, although bedrock could be encountered at some locations above Elevation 271 feet, as described below.

Bedrock Profile

The bedrock in the 1,000 feet by 500 feet area is described in SSAR Figure 2.5-18 as an interbedded hornblende gneiss, biotite granite gneiss and granite gneiss. The top of bedrock at the site is generally gently sloping, as shown on the two subsurface profiles on SSAR Figures 2.5-57 and 2.5-58, with steepest slopes in the 12 to 15% range. (Vertical exaggeration on these figures is approximately 5 and 2.5, respectively.)

The 1,000 feet by 500 feet area has borings on all sides, all showing similar and consistent conditions. Table 1 summarizes these borings, with their direction relative to the 1,000 feet by 500 feet area. In addition to the tabulated information, two observation wells were drilled for the ESP investigation, close to the referenced area, with OW-842 on the western edge of the area, and OW-847 about 350 feet south of the area. These wells were terminated in dense or very stiff Zone IIA soils at Elevations 284 feet and 266 feet, respectively.

Table 1. Borings Adjacent to 1,000 feet x 500 feet area

		Zone III-IV or IV Bedrock		
		Number of Borings To Rock	**Top of Rock Elevation (feet)**	
Investigation	**Direction**		**Range**	**Median**
Units 1 and 2	Northeast	44	201-298	236
Units 3 and 4 *	Northeast	38	190-266	234
SWR	East	7	216-234	221
ISFSI	South	0	**	**
ESP, B-801, 802 & 805	Northeast	3	229-263	232
ESP, B-803 & 804	North	2	244-287	266
ESP, B-806 & 807	Northwest	2	254-288	271

* Most of the abandoned Units 3 and 4 borings were drilled from plant grade at Elevation 271 feet, and so the median values are lower since bedrock had already been excavated at some of the boring locations.
** Borings only advanced into Zone III weathered rock.

From the information given in Table 1, it is reasonable to expect that the top of rock elevations in the referenced 1000 feet by 500 feet area will fall within the tabulated top of rock ranges for the surrounding borings. The overlying soils will be the residual materials found universally throughout the site. If any weathered, fractured, and/or jointed intervals are found in the rock directly beneath safety related structures in this area, they would be removed or treated, as described in the response to RAI 2.5.4-2.

Application Revision

None.

RAI 2.5.4-2 (NRC 6/1/04 Letter)

SSAR Subsection 2.5.4.1 (Geologic Features) references SSAR Section 2.5.1.2.3 (Site Area Stratigraphy), which states that borings drilled for the ESP application revealed severely weathered, fractured and jointed intervals in the Zone III-IV and Zone IV rock. Section 2.5.1.2.3 further states that these severely weathered fracture zones were encountered in four of the seven borings drilled for the ESP application.

RAI 2.5.4-2 Part a)

a) Please describe the extent of similar severely weathered fracture zones, if any, that were observed during the site investigation performed for the abandoned Units 3 and 4.

Response to Part a)

Table 1 summarizes the zones where very poor quality rock, defined as having a Rock Quality Designation (RQD) of 0 – 25% according to Peck et al. (1974), were cored in the investigation for abandoned Units 3 and 4. This level of RQD can be anticipated in zones that are weathered, fractured, and jointed. Table 1 divides the RQD into 0-10% and 11-25% columns. The 0-10% column reflects the severely fractured zones.

Table 1. Zones of Rock Quality Designation 0-25% for Abandoned Units 3 and 4

Boring	RQD (0-10% range)	Depth, feet/Elevation, feet	RQD (11-25% range)	Depth, feet/Elevation, feet
B-602	9	22-30/255-247	25	36-39/241-238
B-607	0	43-45/227-225		
B-615			16	43-45/227-225
B-616	0	54-59/217-212		
	0	61-63/210-208		
B-618	0	34-36/236-234		
B-624	0	12-15/259-256	25	83-93/188-178
	0	114-119/157-152		
	0	120-122/151-149	25	136-140/135-131
B-626	8	8-13/264-259		
	0	44-48/228-224		
	0	62-70/210-202	15	70-74/202-198
	0	74-83/198-189		
B-627			23	49-51/222-220
	0	76-79/195-192		
B-628	0	29-31/242-240		

Table 1. Zones of Rock Quality Designation 0-25% for Abandoned Units 3 and 4

Boring	RQD (0-10% range)	Depth, feet/Elevation, feet	RQD (11-25% range)	Depth, feet/Elevation, feet
B-633	0	57-59/227-225		
B-635	6	50-51/225-224		
B-637			23	56-66/215-205
B-638	9	40-50/228-218		
B-639	0	56-57/218-217		
	0	60-61/214-213		
B-643			21	65-69/205-201
B-644	1	26-50/245-221		
B-645	0	5-6/266-265	17	20-27/251-244

Note that the rock thicknesses for many of the RQD = 0 intervals in Table 1 are in the 1 to 2-foot thick range. This is similar to the situation noted in the 4 ESP borings referenced in the RAI, where the fracture zones range in thickness from 0.5 to 1 feet.

RAI 2.5.4-2 Part b)

b) Please describe the impact of the existence of the severely weathered fracture zones on the suitability of the site to host safety-related structures.

Response to Part b)

SSAR Section 2.5.1.2.3 states:

> Severely weathered fracture zones were encountered in Zone III-IV rock at varying depths, ranging from about 11 feet (El. 260) to 81 feet (El. 211) below the ground surface. These fracture zones were encountered in four of the borings (B-802, B-803, B-805, and B-806) and ranged in thickness from about 0.5 to 1-foot thick.

SSAR Section 2.5.4.10.1 states:

> The Zone III-IV and Zone IV bedrock have design unconfined compressive strengths of 4 ksi (576 ksf) and 12 ksi (1728 ksf), respectively (SSAR Table 2.5-45). Allowable bearing capacities of these materials are much higher than any applied structure bearing pressure. If excavation during construction reveals any weathered or fractured zones at foundation level, such zones would be overexcavated and replaced with lean concrete. The allowable values of the bearing capacity of 80 ksf and 160 ksf for Zone III-IV and Zone IV rock, respectively, are presumptive values based on various building codes for moderately weathered to fresh foliated rock.

As noted in these SSAR sections, any weathered or fractured zone encountered at foundation level would be excavated and replaced with lean concrete. If such zones exist below sound rock beneath the foundation, they would have no impact on the stability of the foundation, since these zones are typically only 0.5 to 1-foot thick, and are confined within an unfractured rock mass with strengths of 4,000 to 12,000 psi (compared to the maximum foundation pressure of just over 100 psi). The foundation itself would consist of a large, thick, highly-reinforced concrete mat that is so stiff that it cannot locally yield.

Multiple borings would be performed at each structure location once the building locations are chosen as part of detailed engineering. These borings would identify whether there are any thicker fracture zones beneath the foundation than those encountered in the ESP borings and in the abandoned Units 3 and 4 borings. If any thicker zones are found, analysis would be performed to identify their impact on foundation stability. If they are close enough to the foundation to potentially impact stability, they would be excavated and replaced with lean concrete.

References

Peck, R. B., W. E. Hanson, and T. H. Thornburn. *Foundation Engineering,* Second Edition, John Wiley and Sons, Inc., New York, 1974 (Reference 182 of SSAR Section 2.5).

Application Revision

None.

RAI 2.5.4-3 (NRC 6/1/04 Letter)

SSAR Section 2.5.4.2 (Properties of Subsurface Materials) provides the results of the extensive field and laboratory tests that were performed earlier for the abandoned Units 3 and 4, the service water reservoir (SWR), and the independent spent fuel storage installation (ISFSI) facilities at North Anna Power Station. Please discuss how the results of the site investigations for the SWR and the ISFSI, which are located away from the abandoned Units 3 and 4, were integrated with those of the ESP borings in characterizing the subsurface materials at the ESP site.

Response

The results of the site investigations for the SWR and the ISFSI were integrated into the site characterization of the ESP area in the following manner:

- As noted in the response to RAI 2.5.4-1, some of the SWR borings are closer to the southeast portion of the 500 feet by 1,000 feet area referred to in that RAI than any of the other borings. Similarly, some of the ISFSI borings are as close to the southwest portion of the 500 feet by 1,000 feet area as any of the other borings. Thus, the SWR and ISFSI borings can reasonably be used to help characterize the ESP area.

- All of the borings that were performed at the North Anna site prior to the ESP borings showed the same general subsurface profile, with consistent geology, i.e., Zones I through IV as described in SSAR Section 2.5.4.2.2 and the response to RAI 2.5.4-1. This included the SWR and the ISFSI borings. As expected, the ESP borings also fit into the general subsurface profile. This was one reason for including ESP borings B-806 and B-807. Although these borings were performed for non-safety related structures (i.e., the plant cooling towers), they illustrate that the same general subsurface profile extends well to the west of any previous exploration points.

In summary, the North Anna site has a consistent geology and has displayed a very consistent subsurface profile, with expected variations in the thickness of the various strata overlying bedrock. The SWR and ISFSI borings, although located away from abandoned Units 3 and 4, are closer or as close to the ESP area as any other borings, and disclosed the same profile, thus adding to the overall confidence level in the subsurface consistency.

Application Revision

None.

RAI 2.5.4-4 (NRC 6/1/04 Letter)

Table 2.5-29 in SSAR Section 2.5.4 compares the total thicknesses of the soil layers sampled at the locations of Units 1 and 2, abandoned Units 3 and 4, the ISFSI, the SWR, and the ESP site. Table 2.5-29 shows that the total thickness of all the soil layers sampled at the ESP site is only 105 ft, whereas the total thicknesses of soil layers sampled at the other sites mentioned range from 451 ft for the ISFSI to 2204 ft for Units 1 and 2. Please explain how the total thickness of soil layers sampled at the ESP site is sufficient to characterize the soil conditions there.

Response

The soils at the North Anna site have been very well characterized by the 138 borings previously performed. The in-situ soils in all of the borings showed the same general subsurface soil profile, i.e., Zones I, IIA and IIB as described in SSAR Section 2.5.4.2.2 and the response to RAI 2.5.4-1. Subsurface profiles shown on SSAR Figures 2.5-57 and 2.5-58 demonstrate these zones as typically found at the ESP site. One of the primary purposes of the 7 ESP borings was to show that the soil (and rock) profiles in each of the borings fit within the general subsurface profile. The results of the borings did indeed demonstrate this. The cone penetrometer tests and geophysical tests performed for the ESP also gave the same conclusion.

The 105 feet referred to in SSAR Table 2.5-29 is the total thickness of Zone IIA saprolite sampled in the 7 borings. The Zone IIA saprolite is the dominant soil type at the North Anna site. The thickness per boring ranged from 0 to 31 feet. In some cases, the small thickness of Zone IIA saprolite is the result of excavation for the existing or abandoned units at the site, e.g., the B-802 location (3 feet of Zone IIA saprolite) had about 40 feet of soil excavated for the original construction, and B-801 (zero Zone IIA saprolite) is at the location of the abandoned Unit 3 excavation. SSAR Table 2.5-29 shows that the constituents of the Zone IIA saprolite are in line with the constituents found in the previous borings.

As noted in SSAR Section 2.5.4: "The additional field and laboratory investigations performed for the ESP were intended to confirm the already large volume of geotechnical data developed for the existing units and the abandoned Units 3 and 4 within the ESP site area. Additional structure-specific exploration and testing would be performed during detailed engineering and would be described in the COL application." The main purpose of the structure-specific borings from the soils aspect would be to verify the thickness of the soil strata at the structure location.

Application Revision

None.

RAI 2.5.4-5 (NRC 6/1/04 Letter)

With regard to Table 2.5-45 (Summary of Geotechnical Engineering Properties) in SSAR Section 2.5.4:

RAI 2.5.4-5 Part a)

a) Please explain why no shear wave velocities are given for Zone IIB saprolite and for Zones III and III-IV weathered rock.

Response to Part a)

SSAR Table 2.5-45 gives average shear wave velocity values for Zones IIB, III, and III-IV, but does not provide a range of values. There are both average values and a range of values provided for Zone IIA and Zone IV. The reason the values were presented this way in SSAR Table 2.5-45 was that there were a range of measured and computed values of shear wave velocity for Zones IIA and IV, but there were much fewer values for Zones IIB, III, and III-IV, as explained in the following paragraphs. Additions, however, have been made in the revision to SSAR Table 2.5-45 at the end of this RAI response.

SSAR Figure 2.5-62 (a) illustrates the 600 to 1,350 feet/second range of shear wave velocity values for Zone IIA saprolite – values from a cross-hole seismic test and two CPT down-hole seismic tests performed as part of the ESP investigation, and average shear wave velocity values from the investigation for Units 1 and 2.

SSAR Section 2.5.4.4.1 describes a 4,000 to 8,000 feet/second range of shear wave velocity values for Zone IV bedrock. For the Units 1 and 2 investigation, shear wave velocities were measured with a Birdwell 3-D velocity recorder and from cross-hole seismic tests. Cross-hole seismic and down-hole seismic tests were performed in the Zone IV bedrock as part of the ESP investigation.

Zone IIB saprolite occurs much less frequently than the Zone IIA saprolite, and there are correspondingly less shear wave velocity measurements in the Zone IIB saprolite. No shear wave velocity values were attributed to the Zone IIB saprolite in the Units 1&2 investigations. For the ESP investigation, the CPT-825 down-hole seismic test was interpreted as penetrating about 10 feet of Zone IIB saprolite, and gave a shear wave velocity measurement of about 1,650 feet/second. Using a different approach, the Zone IIB shear wave velocity was computed from the high strain modulus values given in SSAR Table 2.5-45 using the relationship between high and low strain modulus given in SSAR Figure 2.5-63. The resulting computed shear wave velocity value was 1,574 feet/second. The average shear wave velocity value of 1,600 feet/second given in SSAR Table 2.5-45 was selected based on the 1,650 and 1,574 feet/second values. The 1,574 to 1,650 feet/second range was not included in SSAR Table 2.5-45 since it was considered that including such a narrow range would provide an optimistic assessment of the actual range. A note has been added in the revision to SSAR Table 2.5-45 stating that there is no range of values available for the Zone IIB saprolite.

As with the Zone IIB saprolite, the CPT-825 down-hole seismic test provides the only field measurement of shear wave velocity in the Zone III weathered rock. As shown in SSAR Figure 2.5-62 (a), shear wave velocities of 1,650 and 2,440 feet/second were measured in the upper and lower portions of the Zone III stratum. Using the same approach adopted for Zone IIB, the Zone III shear wave velocity was also computed from the high strain modulus values given in SSAR Table 2.5-45 using the relationship between high and low strain modulus given in SSAR Figure 2.5-63. The resulting computed shear wave velocity value was 2,000 feet/second. Since this was close to the median of the field shear wave velocity measurements, it was adopted as the average shear wave velocity value in SSAR Table 2.5-45. A range of 1,500 to 2,500 feet/second is added for the Zone III weathered rock in the revision to SSAR Table 2.5-45.

There were several measured shear wave velocities for Zone III-IV. In boring B-104 performed for Units 1 and 2, measured shear wave velocity in Zone III-IV ranged from 3,000 to 4,500 feet/second (Birdwell 3-D velocity recorder). In the ESP down-hole seismic test, the measured shear wave velocities in Zone III-IV near boring B-802 were 1,482, 3,435 and 5,278 feet/second. The shear wave velocity computed from the shear modulus derived from the rock strength was 3,366 feet/second. These values are the basis for the average value of 3,300 feet/second in SSAR Table 2.5-45. A range of 2,500 to 4,500 feet/second is added for the Zone III-IV rock in the revision to SSAR Table 2.5-45.

<u>RAI 2.5.4-5 Part b)</u>

b) Please provide the range of standard penetration test (SPT) values separately for coarse-grained and fine-grained soil zone IIA, along with the depths of the soils at which the N-values were obtained.

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

Table 1 (located at the end of this RAI response) provides the standard penetration test (SPT) values for coarse-grained and fine-grained Zone IIA saprolites from all the site borings. Note that Dames and Moore used their "Dames and Moore Sampler" on many occasions, especially in the Units 1 and 2 investigation. Since the blowcount from the Dames and Moore sampler cannot be directly correlated with the SPT blowcount, the Dames and Moore blowcounts are not included in Table 1.

Coarse-grained soils in the table are poorly graded gravels (GP), poorly graded sands (SP), silty sands (SM), and clayey sands (SC). Fine-grained soils are low and high plasticity silts (ML and MH) and low and high plasticity clays (CL and CH).

SPT N-values were obtained for 397 samples of coarse-grained Zone IIA saprolites. Range was 4 to 1,260 blows/foot. (1,260 blows/foot is extrapolated from 210 blows for 2 inches.) The median N-value was 33 blows/foot.

SPT N-values were obtained for 200 samples of fine-grained Zone IIA saprolites. Range was 6 to 171 blows/foot. (171 is extrapolated from 50 blows for 3.5 inches.) The median N-value was 19 blows/foot.

Application Revision

For the Description "Shear wave velocity range, ft/sec," revise SSAR Table 2.5-45 for Stratums IIB, III, and III-IV to read as follows:

Description	IIB	III	III-IV
Shear and compression wave velocity			
Shear wave velocity range, ft/sec	No range available	1,500 to 2,500	2,500 to 4,500

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites					
		Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
Study	Boring Number	Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
Units 1&2	B-1	2	27		
		5	24		
		11	50		
		15	138		
		20	194/6 in.		
		25	225/6 in.		
		30	250/5 in.		
Units 1&2	B-10	2	40		
		5	17		
		11	62		
		15	151		
		21	207		
		25	210/2 in.		
		31	205/8 in.		
Units 1&2	B-27	2.5	17		
		11	16		
		21	55		
		31	107		
Units 1&2	B-43	5	89		
		11	140		
		16	106		
		21	69		
		26	81		
		31	87		
Units 1&2	B-50	5	4		
		11	4		
		21	4		
		25	7		
		31	9		
		36	10		
		41	10		
		46	17		
		51	65		
Units 1&2	B-103	11	29		
		21	28		
		31	22		
		41	52		

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites					
		Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
Study	Boring Number	Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
		51	60		
		61	119		
		71	277		
Units 1&2	B-105	11	6		
		21	7		
Units 3&4	B-601	6	16		
		21	25/3 in.		
Units 3&4	B-603	26	105		
		31	175		
Units 3&4	B-604			7	40
Units 3&4	B-605	15	35		
		20	54		
		25	123		
Units 3&4	B-606	5	18		
		10	26		
		15	70		
		20	70/6 in.		
Units 3&4	B-607	5	13		
		10	23		
		15	32		
		20	50/2 in.		
		25	50/3 in.		
Units 3&4	B-608	5	31		
		25	146		
		30	143		
Units 3&4	B-609	15	21	5	13
		25	17/3 in.	10	18
		30	70/6 in.		
Units 3&4	B-610	5	25		
		10	22		
		15	28		
		25	52		
		45	79		
		50	77		
		55	45/2 in.		
		60	78		
Units 3&4	B-611	7	20		
		21	15		
		34	33		

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites					
		Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
Study	Boring Number	Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
		41	59		
		46	100/5 in.		
Units 3&4	B-612	10	13		
Units 3&4	B-613	5	34		
		10	15		
		15	21		
		20	25		
		25	34		
		30	30		
		40	90		
Units 3&4	B-614	5	23		
		10	23		
		15	20		
		20	18		
		30	33		
Units 3&4	B-615	5	12		
		10	17		
		15	40		
		20	44		
Units 3&4	B-616	5	22	10	9
		15	31	20	24
				25	45
Units 3&4	B-617	5	26		
		10	28		
		15	94		
		20	64		
		25	108		
		30	57/6 in.		
		35	68/6 in.		
Units 3&4	B-618	5	14		
		10	24		
		20	40		
		25	32		
		30	44		
Units 3&4	B-619	5	65		
		10	110		
Units 3&4	B-620	5	40		
Units 3&4	B-622	5	41		
		10	210		

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites

Study	Boring Number	Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
		Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
		15	120/4 in.		
Units 3&4	B-623	5	170/4 in.		
		10	49		
Units 3&4	B-624	5	49		
		7	150		
Units 3&4	B-625	4	6		
Units 3&4	B-626	5	119		
Units 3&4	B-631	25	46		
		40	19		
		50	30		
		60	51		
		65	59		
		70	240		
		75	262		
Units 3&4	B-632	5	44		
		10	56		
		15	58/6 in.		
Units 3&4	B-634	5	25	10	23
		25	100	15	48
		30	65/5 in.	20	65
Units 3&4	B-636	5	15		
		10	25		
		15	70/2 in.		
		20	15/1.5 in.		
		25	100/6 in.		
Units 3&4	B-637	10	14		
		25	42		
		30	50/3 in.		
Units 3&4	B-638	5	116		
Units 3&4	B-639	25	40/3 in.		
		30	75/7 in.		
Units 3&4	B-640	5	22		
		10	41		
		15	29		
		20	22		
		25	59		
		30	156		
		35	101/5 in.		
		40	100/5 in.		

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites

Study	Boring Number	Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
		Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
Units 3&4	B-641	5	19		
		10	24		
		15	25		
		20	16		
		25	22		
		30	31		
		35	45		
		40	63		
		45	70/3 in.		
		50	50/2 in.		
Units 3&4	B-642	5	19		
		10	25		
		15	26		
		20	21		
		40	47/6 in.		
		45	80		
		50	34		
Units 3&4	B-643	5	18		
		10	20		
		15	59		
		20	51		
		25	149		
		30	100/3 in.		
Units 3&4	B-646	10	25	15	25
		20	57		
		25	79		
		30	118		
		35	162		
		40	100/5 in.		
		45	20		
Units 3&4	B-647	5	13		
		10	23		
		15	44		
		20	69		
		25	50/3 in.		
SWR	P-10	3	24		
		10	43		
		15	142		
		20	20		

Table 1.	Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites				
		Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
Study	Boring Number	Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
	P-11	21	23	16	16
		35	21	25	16
		40	13	30	16
		45	17		
SWR	P-12	11	18	5	17
		15	25		
		20	18		
SWR	P-16	40	19	35	18
		45	19		
		50	28		
		55	39		
		60	107		
		65	62		
SWR	P-17	65	45	35	18
				40	17
				45	17
				50	17
				55	22
				60	23
SWR	S1-1	45	17	15	26
		50	19	20	18
		55	27	25	22
		60	25	30	31
		65	24		
		70	56		
		75	80		
		80	100		
SWR	S1-2	7	15		
		10	18		
		15	33		
		20	94		
		25	33		
		30	100		
		35	100		
SWR	S1-3	10	47	5	31
		15	57		
		20	92		
		25	63		
		30	50		

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites

Study	Boring Number	Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
		Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
		35	134		
		40	32		
		47	155		
SWR	SWR-1			1	15
				2.5	21
				4	21
				5.5	20
				7	14
				8.5	13
				10	18
				11.5	17
				13	17
				14.5	20
				16	16
				18	17
				19.5	20
				21	15
				24	21
				24.5	14
				26	12
				28	9
				31.5	16
				33	15
				34.5	22
				36	15
				37.5	15
				39	16
				40.5	20
				42	22
				43.5	24
SWR	SWR-2			1.5	16
				3	18
				4.5	24
				6	18
				7.5	13
				9	12
				10.5	16
				12	20
				13.5	17

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites

Study	Boring Number	Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
		Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
				15	16
				16.5	16
				18	17
				19.5	18
				21	19
				22.5	20
				24	18
				25.5	22
				27	21
				28.5	11
				30	12
				31.5	11
				33	12
				34.5	13
				36	13
				37.5	14
				39	19
				40.5	20
				42	45
				43.5	55
				45	74
				46.5	84
				48	38
				49.5	46
SWR	SWR-3	7	15		
		10	16		
		15	13		
		20	55		
		25	12		
		30	25		
		35	17		
		40	29		
		45	33		
		50	45		
		55	41		
		60	51		
		65	45		
		70	91		
		75	75		

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites					
		Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
Study	Boring Number	Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
		80	131		
		85	100		
		90	142		
		95	138		
SWR	SWR-4	5	39		
		10	16	15	16
		20	17		
		25	19		
		30	27		
		35	18		
		40	29		
		45	19		
		50	24		
		55	38		
		60	31	65	40
		70	57		
		75	27		
		80	52		
		85	37		
		90	100		
		95	100		
		100	400		
SWR	SWR-5	25	15		
		30	22	35	16
		40	21		
		45	21		
		50	32		
		55	31		
		60	28		
		65	25		
		70	37		
		75	100		
		80	39		
		90	226		
SWR	SWR-6	35	21	15	21
		40	21	20	27
		45	26	25	22
		55	16	30	23
		65	19	50	22

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites

Study	Boring Number	Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
		Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
		70	48	60	21
		75	100+		
		80	100+		
		85	100+		
		90	100+		
		95	100+		
		100	400		
SWR	SWR-7	35	16	15	17
		40	9	25	19
		50	15	30	24
		55	17	45	8
		60	19	75	26
		65	23		
		70	32	80	37
SWR	SWR-8	30	16	10	24
		35	19	15	15
		40	35	20	9
		45	25	25	10
		50	41		
		55	50		
		60	109		
		65	98		
		70	81		
SWR	SWR-9	20	10	15	12
		30	17	25	8
		35	17		
		40	60		
		45	68		
		50	274		
		55	50		
		60	75		
		65	163		
SWR	SWR-10	33	14	45	24
		35	21	47.5	37
		37.5	18	50	19
		40	16	52.5	26
		42.5	14	55	14
				57.5	25
				60	30

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites					
		Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
Study	Boring Number	Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
				62.5	36
SWR	SWR-11	33	300	17.5	17
		36	300	22.5	19
				28	48
SWR	SWR-13	40	21	30	22
				35	19
				45	25
				50	14
				55	13
				60	24
				65	31
				70	62
ISFSI	F-2	15	14	1	13
		20	18	4	20
		25	18	8	18
		30	22	10	18
		35	14		
		40	17		
		45	18		
		50	43		
		55	54		
		60	78		
ISFSI	F-4	10	15	1	25
		15	21	3	29
		20	16	4.5	19
		25	23	7	19
		29.5	50/5 in.		
ISFSI	F-5			1	25
				3	25
				4.5	18
				7	21
				10	9
				15	13
				20	9
				25	12
				30	14
				35	26
				40	31
				45	27

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites

Study	Boring Number	Coarse –Grained Zone IIA Depth, Ft	Coarse –Grained Zone IIA SPT N-Value, Blows/ft	Fine-Grained Zone IIA Depth, Ft	Fine-Grained Zone IIA SPT N-Value, Blows/ft
				50	44
				55	27
				60	40
ISFSI	F-6	30	17	1	19
		35	23	3	26
		40	80/9 in.	4.5	26
				7	26
				10	19
				15	14
				20	13
				25	13
ISFSI	F-7	65	54	1	19
		70	71	3	41
		75	50/3 in.	4.5	36
				7	27
				10	15
				15	10
				20	10
				25	10
				30	15
				35	15
				40	14
				45	17
				50	22
				55	36
				60	38
ISFSI	F-8	1	18	3	33
				4.5	29
				7	36
				10	17
				15	18
				20	25
				25	24
				30	16
ISFSI	F-9	1	17		
		3	25		
		4.5	23		
		7	24		
		10	16		

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites

Study	Boring Number	Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
		Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
		15	14		
		20	12		
		25	7		
		30	14		
		35	21		
		40	26		
		45	52		
		50	56		
ISFSI	F-10	1	23		
		3	30		
		4.5	28		
		7	27		
		10	20		
		15	24		
		20	32		
		25	22		
		30	48		
		35	61		
		40	80		
		45	26		
ISFSI	F-11	25	61	1	43
				3	62
				4.5	41
				7	32
				10	38
				15	50/3.5 in.
				20	39
ESP	B-802	5	44		
ESP	B-803	1	12		
		5	31		
		7	14		
		10	22		
		12	13		
		15	23		
		20	18		
		25	31		
		30	30		
ESP	B-804	7	6	1	13
		9.5	5	2.5	13

Table 1. Standard penetration test values for coarse-grained and fine-grained Zone IIA saprolites					
		Coarse –Grained Zone IIA		Fine-Grained Zone IIA	
Study	Boring Number	Depth, Ft	SPT N-Value, Blows/ft	Depth, Ft	SPT N-Value, Blows/ft
		12	5	4.5	6
		14.5	9		
		19.5	24		
ESP	B-805	8.5	17	1	12
		11.5	25	3.5	20
		14.5	38	6	14
		19.5	34		
		24	100/8.5 in.		
		28.5	100/1 in.		
ESP	B-806	4	22		
		6.5	18		
ESP	B-807	23	22	1	12
		27.5	100	3	17
		32.5	80	5.5	15
		36.5	100/2.5 in.	8	12
				11	13
				13	13
				16	21

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

With regard to Table 2.5-44 (Summary of ESP Test Rest Results - Rock) in SSAR Section 2.5.4:

RAI 2.5.4-6 Part a)

a) Please explain why test results were not provided for the materials at several depths, for example, between depths 25 ft and 48 ft in boring B-801, between depths 21 ft and 44 ft, 46 ft to 66 ft, and 67 ft to 85 ft in boring B-802, and several depths in borings B-803 and B-806.

Response to Part a)

The containment (reactor) buildings for the new units would be founded on the Zone III-IV and/or Zone IV metamorphic gneiss bedrock at the North Anna site. Rock coring and testing performed on 23 cores from the Units 1 and 2 investigation gave unconfined compression strengths of the Zone III-IV and Zone IV rock ranging from 1.0 to 16.3 ksi with a median strength of 6.8 ksi, that is, rock strengths that were typical for this type of rock, and more than sufficient to support the maximum containment (reactor) building loads of about 0.1 ksi.

During logging of the rock cores in the field for the ESP investigation, it was apparent that the metamorphic rock was a strong material. (See, for example, the photos of the recovered cores from boring B-801 in SSAR Section 2.5.4 Appendix B, page 2.5.4B-31.) Sufficient tests were performed on the ESP cores to verify that the rock strengths were similar to or higher than those cores tested for Units 1 and 2. Rock coring and testing performed on 18 cores from the ESP investigation gave unconfined compression strengths of the Zone III-IV and Zone IV rock ranging from 2.7 to 28.4 ksi with a median strength of 18.4 ksi, generally higher than the Units 1 and 2 strengths.

In this situation where there are moderately strong or strong rocks, more important parameters from a structure stability standpoint are the recovery and the rock quality designation (RQD). (The RQDs are given as percentages for each core in the detailed rock coring logs in SSAR Section 2.5.4 Appendix B.) These parameters indicate the degree of recovery and fracturing of the core run. For bearing capacity on rock, it is more desirable to have a lower strength rock with high recovery and RQD than a strong rock with low recovery and RQD since the low strength rock has adequate strength to support the loads, whereas high strength rock with many fractures may be subject to local differential settlement.

The recovery and RQD values for the ESP site cores were typically higher than for the Units 1 and 2 investigation, although this could have been due to better coring equipment in the recent investigation.

The 18 ESP rock core tests were assigned on representative cores recovered from the borings. For example, all six 5-foot long core runs in boring B-801 had 100 percent recovery and 100 percent RQD, with similar descriptions. Strength tests were made on sections of core taken from the top and bottom core runs, and gave very consistent results.

Similarly, the four tests on sections of core from B-802 were representative of the core recoveries and RQDs of the cores in that boring, and were taken at representative depths – 90% recovery and 72% RQD at 20.7 feet depth, 100% and 96% at 45.3 feet depth, 100% and 80% at 66.4 feet depth, and 100% and 92% at 85.6 feet depth. The median recovery and RQD in B-802 were 100% and 80%, respectively.

The same approach was applied to core testing in B-803 and B-806. Thus, although testing was not conducted within certain depth intervals, the field characterization coupled with the laboratory test results demonstrate the quality and consistency of the rock.

RAI 2.5.4-6 Part b)

b) Please explain why no test results were provided for boring B-807.

Response to Part b)

Boring B-807 is located in the ESP site cooling tower area. The cooling towers that would be located in this area would not serve a safety-related function. The cooling towers are relatively lightly-loaded structures (1 to 2 ksf or 0.007 to 0.014 ksi loading). They would be founded at plant elevation (Elevation 271 feet) or above on improved Zone IIA saprolite or Zone IIB saprolite, 15 to 20 feet above the underlying gneiss in B-807. The gneiss would not impact the performance of the cooling tower foundations. During detailed engineering, once the actual cooling tower locations are established, borings would be made to confirm the soil properties at that location and the depth and quality of the bedrock.

RAI 2.6.5-6 Part c)

c) Please discuss the significance of the relatively low value (4.43 ksi) of the unconfined compressive strength of the Zone IV rock in Boring B-805, as compared to the values for the Zone IV rock strengths in Borings B-802, 803, and 806 at similar depths, which are much higher (by a factor 2 to 6).

Response to Part c)

There is no significance from a foundation stability standpoint – 4.43 ksi puts the rock in the moderately strong classification, and is around the compressive strengths typical of reinforced concrete foundations (4,000 to 5,000 psi).

This core had 100% recovery and 92% RQD, with the RQD only slightly below the median value for Zone IV cores recovered in the ESP investigation. However, reference to the tested core (SSAR Section 2.5.4 Appendix B, page 2.5.4B-324) shows the core failed in a clean diagonal break, not along the foliation plane but fairly close to it. The core is described as strongly foliated. The pictures of failed high strength cores (e.g., B-801, 24.1 to 24.8 feet depth, 27.21 ksi strength, SSAR Section 2.5.4 Appendix B, page 2.5.4B-325) show a failure along multiple planes, and these cores are generally described as weakly foliated. Other cores that are described as strongly foliated tend to have lower strengths than the very high strength weakly foliated materials. Thus, the lower strength is probably linked to strong foliation. It should be noted that failure along a predetermined plane can occur in an unconfined compression test, but not in the rock mass itself.

Application Revision

None.

RAI 2.5.4-7 (NRC 6/1/04 Letter)

SSAR Subsection 2.5.4.7.1 (Shear Wave Velocity Profile) states (on page 2.2-291) that some safety-related structures (excluding the reactors) may be founded on the Zone III weathered rock, Zone IIB saprolite, or Zone IIA saprolite. However subsection 2.5.1.2.6 (Site Engineering Geology Evaluation) of the SSAR states (on page 2.2-222) that Zone III is not a suitable material for safety-related plant structures. Please reconcile these two statements.

Response

The statement in SSAR Section 2.5.4.7.1 is correct—some safety-related structures (excluding the reactor containment building) may be founded on the Zone III weathered rock, Zone IIB saprolite, or improved Zone IIA saprolite. Note that SSAR Section 2.5.4.10.2 states that Zone IIA saprolite is unsuitable for the support of any safety-related structure without ground improvement. Ground improvement is discussed in SSAR Section 2.5.4.12.

The statement in SSAR Section 2.5.1.2.6 will be deleted because Zone III weathered rock is suitable under certain conditions. (See response to RAI 2.5.4-11.)

Application Revision

The 2nd paragraph under the heading "Rock" in SSAR Section 2.5.1.2.6 will be revised to delete the last sentence which reads: "These results indicate that Zone III is not a suitable bearing surface for the safety-related plant structures."

RAI 2.5.4-8 (NRC 6/1/04 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:

RAI 2.5.4-8 Part a)

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

Response to Part a)

1. Introduction

EPRI (1993) comprehensively reviews much of the published literature on the topic of shear modulus reduction curves, including the work of Seed et al at the University of California, Berkeley, in the 1970s and 1980s. The SSAR design curves for shear modulus reduction with strain are based on the EPRI (1993) recommendations, wherever applicable.

EPRI (1993) indicates that the property most affecting the shape of the shear modulus versus strain curves is grain size. Exhibit 1 (from EPRI (1993)) shows typical ranges for different grain sizes. The coarser grained soils show greater reduction with increasing strain than the finer grained soils. At North Anna, the Zone IIA soils are classified as sands. However, the Zone IIA soils are also looked at as clays for comparison, since these soils do have some cohesive characteristics.

Although the Zone IIB saprolite contains relict structure of the parent rock, it does not appear to exhibit any of the cohesive characteristics noted in the Zone IIA saprolite. In fact, with up to 50 percent of core stone remaining intact, the Zone IIB saprolite required rock coring in some instances. It can be argued that the Zone IIB saprolite will behave more like a gravel or crushed stone than a sand.

Solid rock does not exhibit the strain softening characteristics of soil. Like steel and concrete, sound rock has essentially the same modulus (shear and elastic) throughout the strain range. The elastic modulus values computed from the stress-strain measurements (relatively high strain) on samples of sound rock core obtained during the ESP subsurface investigation were similar to those calculated from the ultra low strain cross-hole seismic tests. However, at some stage of weathering, rock becomes sufficiently decomposed to exhibit modulus attenuation. The Zone III moderately to severely weathered rock is considered to fall into this sufficiently weathered state. Unlike soils, relatively little research has been performed on weathered rock. Sun et al (1988) developed a shear modulus versus strain relationship for mudstone (a soft rock) with a shear wave velocity of 1,500 feet/sec. As would be expected, the attenuation at the highest measured strain (about 0.5 percent) is only about 50 percent, compared to about 90 percent for sand, gravel and clay at that strain.

2. Zone IIA Saprolite

This saprolite is treated as a sand, but the sand curves are also compared with clay curves.

As noted above, EPRI (1993) indicates that the property most affecting the shape of the shear modulus versus strain curves is grain size. For sands, the second most influential property is the confining pressure. For clays, plasticity index plays a major role in determining the shape of the curves.

EPRI (1993) summarizes its recommendations for sands and clays in a series of 5 figures. These figures are included here as Exhibit 2. Each of these figures is reviewed below to see how it relates to the North Anna situation.

a. Page 1 of Exhibit 2. This shows modulus reduction as a function of reference strain. The "reference strain" is defined as τ_{max}/G_{max}, where τ_{max} is the "shear strength" of the soil. For sands, EPRI (1993) notes that the reference strain is typically about 0.1. Thus, the 0.1 reference strain curve is used as a starting point for the North Anna curve. This is plotted on Figure 1 as curve 1.

b. Pages 2 and 3 of Exhibit 2. These show the shear strain reduction curves as a function of vertical effective stress for dry and saturated sands, respectively. Groundwater table at the North Anna site generally varies from about 6 feet to 58 feet below ground surface. Assume groundwater level is at (1) 6 feet depth and (2) 30 feet depth, to see what difference is made to the shear modulus reduction curve.

For the Zone IIA saprolite, this zone is assumed to be 30 feet thick for computation purposes. Unit weight of soil is 125 pcf.

With water table at 6 feet, effective vertical stress at mid layer (15 feet) is:
$(6 \times 125) + (9 \times (125 - 62.4)) = 1{,}313$ psf.

With water table at 30 feet, effective vertical stress at mid layer (15 feet) is:
$15 \times 125 = 1{,}875$ psf.

The curves on Pages 2 and 3 of Exhibit 2 are spaced proportionally to the log of the effective vertical pressure. The effective vertical pressures of 1,313 psf and 1,875 psf for the dry sands were interpolated from the Exhibit 2, page 2 curves, and are plotted as curves 4 and 2, respectively, on Figure 1. There is minimal difference between the dry and submerged sand curves, and so the curves for the submerged sands are not plotted.

c. Page 4 of Exhibit 2. These curves are suitable for generic site response studies in Eastern North America, and cover the range from gravelly sands to low plasticity clays. A curve was interpolated for 10 to 40 feet depth for the North Anna sands. This curve is closer to the 20 to 50 feet curve than the 0 to 20 feet curve. It is plotted as curve 3 on Figure 1.

d. Page 5 of Exhibit 2. This shows modulus reduction curves for clays, demonstrating the variation with plasticity index. The cohesive portions of the Zone IIA saprolite are generally low plasticity. Curve 6 on Figure 1 is for a PI of 10. For PI = 30, the curve is close to curve 1 on Figure 1.

Curve 5 on Figure 1 is the average Seed and Idriss (1970) curve for sands that has been used in many SHAKE analyses.

The curves in Figure 1 fall into a fairly close group. The average curve is close to curves 3 and 4. This curve is plotted as curve 1 on SSAR Figure 2.5-63 and is the design curve for the Zone IIA saprolite. The curve is tabulated below.

Curve 1: Modulus Reduction Design Curve for Zone IIA Saprolite									
Cyclic Shear Strain, Percent	.0001	.000316	.001	.00316	.01	.0316	.1	.316	1
G/G_{max}	1	1	0.98	0.93	0.79	0.57	0.32	0.15	0.05

3. Zone IIB Saprolite

As noted in the introductory section, it can be argued that the Zone IIB saprolite will behave more like a gravel or crushed stone than a sand. EPRI (1993) points out that the reduction curves show greater reduction for coarser grained materials (Exhibit 1). However, EPRI (1993) does not contain specific recommendations for a gravel curve.

Seed et al (1984) provide a modulus reduction curve (included as Exhibit 3) that can be used for gravels, based on tests of four different gravels and crushed stone samples. This curve is tabulated below. The curve is plotted on SSAR Figure 2.5-63 as curve 2 and is the design curve for the Zone IIB saprolite. This curve has a somewhat higher reduction than the curve proposed by Rollins et al (1998).

Curve 2: Modulus Reduction Design Curve for Zone IIB Saprolite									
Cyclic Shear Strain, Percent	.0001	.000316	.001	.00316	.01	.0316	.1	.316	1
G/G_{max}	1	0.96	0.86	0.72	0.54	0.36	0.20	0.10	0.05

4. Zone III Weathered Rock

As noted in the introductory section, Sun et al (1988) developed a shear modulus versus strain for mudstone (a soft rock) with a shear wave velocity of 1,500 feet/sec (included as Exhibit 4). As would be expected, the attenuation at the highest measured strain (about 0.5 percent) is only about 50 percent, compared to about 90 percent for sand, gravel and clay at that strain. The Zone III weathered rock has a shear wave velocity of 2,000 feet/sec. Thus, the SSAR Reference 169 mudstone curve is used for shear modulus input in the soil/rock column amplification/attenuation analysis for the Zone III weathered rock. This curve is tabulated below. The curve is plotted on SSAR Figure 2.5-63 as curve 3 and is the design curve for the Zone III weathered rock. (Note that this curve is close to the highly plastic clay curve in Exhibit 2, page 5, with PI = 70).

Curve 3: Modulus Reduction Design Curve for Zone III Weathered Rock									
Cyclic Shear Strain, Percent	.0001	.000316	.001	.00316	.01	.0316	.1	.316	1
G/G_{max}	1	1	1	1	1	0.97	0.85	0.61	0.32

Note that Sun et al (1988) gives only upper and lower bounds for the curve (see Exhibit 4). The tabulated curve plotted on SSAR Figure 2.5-63 is the average of the upper and lower bounds. Also, the Sun et al (1988) curve is only plotted as far as a strain of about 0.3%. The curve has been extrapolated to 1% shear strain in the table above, and on SSAR Figure 2.5-63.

<u>RAI 2.5.4-8 Part b)</u>

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

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

1. <u>Introduction</u>

For sands and clays, EPRI (1993) follows a similar course for damping as it did for shear modulus. Thus a similar process is followed for the Zone IIA saprolite damping as for the shear modulus. EPRI (1993) does not specifically address damping for gravel and soft rock. Sound rock will display some damping characteristics. However, this damping will not be dependent on the shear strain, i.e., it will exhibit a constant damping ratio.

2. Zone IIA Saprolite

EPRI (1993) summarizes its recommendations for sands and clays in a series of 5 figures. These figures are included here as Exhibit 5. Each of these figures is reviewed to see how it relates to the North Anna situation.

a. Page 1 of Exhibit 5. This shows damping ratio as a function of reference strain. The "reference strain" is defined as τ_{max}/G_{max}, where τ_{max} is the "shear strength" of the soil. For sands, SSAR Reference 170 notes that the reference strain is typically about 0.1. Thus, the 0.1 reference strain curve is used as a starting point for the North Anna curve. This is plotted on Figure 2 as curve 1.

b. Pages 2 and 3 of Exhibit 5. These show the damping ratio versus shear strain curves as a function of vertical effective stress for dry and saturated sands, respectively. Groundwater table generally varies from about 6 feet to 58 feet below ground surface. As in the shear modulus reduction curve computation, assume groundwater level is at (1) 6 feet depth and (2) 30 feet depth. This gives effective vertical pressures of 1,313 psf and 1,875 psf, respectively.

   The curves on Pages 2 and 3 of Exhibit 5 are spaced proportionally to the log of the effective vertical pressure. These were interpolated for effective vertical pressures of 1,313 psf and 1,875 psf for the dry sands and plotted as curves 2 and 3, respectively, on Figure 2. For the saturated sands, the curves are plotted as curves 4 and 5, respectively, on Figure 2.

c. Page 4 of Exhibit 5. These curves are suitable for generic site response studies in Eastern North America, and cover the range from gravelly sands to low plasticity clays. A curve was interpolated for 10 to 40 feet depth for the North Anna sands. This curve is closer to the 20 to 50 feet curve than the 0 to 20 feet curve. It is plotted as curve 6 on Figure 2.

d. Page 5 of Exhibit 5. This shows damping ratio versus shear strain curves for clays, demonstrating the variation with plasticity index. The cohesive portions of the Zone IIA saprolite are generally low plasticity. Curve 7 on Figure 2 is for a PI of 10.

   Curve 8 on Figure 2 is the average Seed et al (1970) curve for sands that has been used in many SHAKE analyses.

   The curves in Figure 2 fall into a fairly close group. The average curve is closest to curve 2. This average curve is plotted as curve 1 on SSAR Figure 2.5-64 and is the design curve for the Zone IIA saprolite. The curve is tabulated below.

Curve 1: Damping Ratio Versus Shear Strain Design Curve for Zone IIA Saprolite									
Cyclic Shear Strain, Percent	.0001	.000316	.001	.00316	.01	.0316	.1	.316	1
Damping Ratio	0.6	0.8	1.1	1.8	3.5	7.1	12.2	18.9	23.7

3. Zone IIB Saprolite

EPRI (1993) indicates that at intermediate and high strains, coarser cohesionless soils show greater values of damping with increasing strain than fine-grained, cohesive soils. At low strains, the picture is not clear. Seed et al (1984), on the other hand, concludes that damping ratios for gravels are very similar to those for sands. Referring to Figure 2, it can be seen that the Seed et al (1984) curve (curve 8) for sands has generally higher damping ratios than any of the EPRI (1993) curves. Thus, if the Seed et al (1984) sand curve is selected for the coarser grained Zone IIB saprolite, it will satisfy the Seed et al (1984) conclusion that the sand and gravel damping curves are similar (with the curves as defined in Seed et al (1984)), and also the EPRI (1993) observation that, at intermediate and higher strains, the coarser grained soils have higher damping values (comparing the Seed et al (1984) curve with the selected design curve for sands on SSAR Figure 2.5-64). At low strains of 0.0001% and 0.000316%, assume the Zone IIB saprolite design curve is the same as the Zone IIA saprolite design curve. This curve is plotted as curve 2 on SSAR Figure 2.5-64, and is the design curve for the Zone IIB saprolite. The curve is tabulated below. This curve is very similar to the curve proposed by Rollins et al (1998) at lower strains, and gives a higher damping ratio at higher strains.

Curve 2: Damping Ratio Versus Shear Strain Design Curve for Zone IIB Saprolite									
Cyclic Shear Strain, Percent	.0001	.000316	.001	.00316	.01	.0316	.1	.316	1
Damping Ratio	0.6	0.8	1.6	3.3	5.8	10.0	15.5	21.0	24.6

4. Zone III Weathered Rock

As noted in the introductory section, sound rock will have a constant damping ratio with strain, possibly in the 1 to 2 percent range. The Zone III weathered rock will display some variation with strain, but not to the extent of the saprolites discussed above. For the modulus reduction curve for the Zone III weathered rock, the mudstone curve in SSAR Reference 169 was used. Unfortunately, Sun et al (1988) makes no mention of an equivalent damping ratio versus strain curve. However, the relationship between the gravel, sand and weathered rock modulus reduction curves in SSAR Figure 2.5-63 can be used as a basis for deriving the damping ratio versus strain curves for the weathered rock.

SSAR Figure 2.5-63 shows no modulus reduction down to 0.01% strain. It is known that there is some damping at low strains for all materials. Assume that the weathered rock has 0.6 damping ratio at 0.001% strain (same as the sand and gravel). Assume this damping ratio remains constant to 0.01% strain.

At 0.0316% strain, the weathered rock shear modulus had reduced by (1 – 0.97) = 0.03, while the sand (Zone IIA) had reduced by (1 – 0.57) = 0.43. The sand damping ratio is 7.1 at 0.0316% strain. Thus it could be assumed that the weathered rock damping ratio was (0.03/0.43) x 7.1 = 0.5. Since the damping ratio will not decrease with increasing strain, assume the 0.6 damping ratio adopted for the lower strain values.

At 0.1% strain, the weathered rock shear modulus had reduced by (1 – 0.85) = 0.15, while the sand (Zone IIA) had reduced by (1 – 0.32) = 0.68. The sand damping ratio is 12.2 at 0.1% strain. Thus it can be assumed that the weathered rock damping ratio was (0.15/0.68) x 12.2 = 2.7.

At 0.316% strain, the weathered rock shear modulus had reduced by (1 – 0.61) = 0.39, while the sand (Zone IIA) had reduced by (1 – 0.15) = 0.85. The sand damping ratio is 18.9 at 0.1% strain. Thus it can be assumed that the weathered rock damping ratio was (0.39/0.85) x 18.9 = 8.7.

At 1.0% strain, the weathered rock shear modulus had reduced by (1 – 0.32) = 0.68, while the sand (Zone IIA) had reduced by (1 – 0.05) = 0.95. The sand damping ratio is 23.7 at 0.1% strain. Thus it can be assumed that the weathered rock damping ratio was (0.68/0.95) x 23.7 = 17.0.

This curve is plotted as curve 3 on SSAR Figure 2.5-64, and is the design curve for the Zone III weathered rock. The curve is tabulated below. Note that this curve is fairly similar to the highly plastic clay (LL = 70) in Exhibit 5.

Curve 3: Damping Ratio Versus Shear Strain Design Curve for Zone III Weathered Rock									
Cyclic Shear Strain, Percent	.0001	.000316	.001	.00316	.01	.0316	.1	.316	1
Damping Ratio	0.6	0.6	0.6	0.6	0.6	0.6	2.7	8.7	17

RAI 2.5.4-8 Part c)

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

Response to Part c)

The response to Part c) will be provided by separate correspondence.

References

Guidelines for Determining Design Basis Ground Motions, Electric Power Research Institute (EPRI), Volumes 1-5, EPRI TR-102293, Palo Alto, CA, 1993 (Reference 170 of SSAR Section 2.5).

Sun, J. I., R. Golesorkhi, and H. B. Seed. Dynamic Moduli and Damping Ratios for Cohesive Soils, Report No. UCB/EERC-88/15, University of California, Berkeley, August 1988 (Reference 169 of SSAR Section 2.5).

Seed, H. B., and I. M. Idriss. Soil Moduli and Damping Factors for Dynamic Response Analyses, Report No. UCB/EERC-70/10, University of California, Berkeley, December 1970 (Reference 167 of SSAR Section 2.5).

Seed, H. B., R. T. Wong, I. M. Idriss, and K. Tokimatsu. Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils, Report No. UCB/EERC-84/14, University of California, Berkeley, September 1984 (Reference 168 of SSAR Section 2.5).

Rollins, K.M., M.D. Evans, N.D. Diehl, and W.D. Daily. "Shear Modulus and Damping Relationships for Gravels," ASCE Journal of Geotechnical and Environmental Engineering, Vol. 124, No. 5, May 1998.

Application Revision

None.

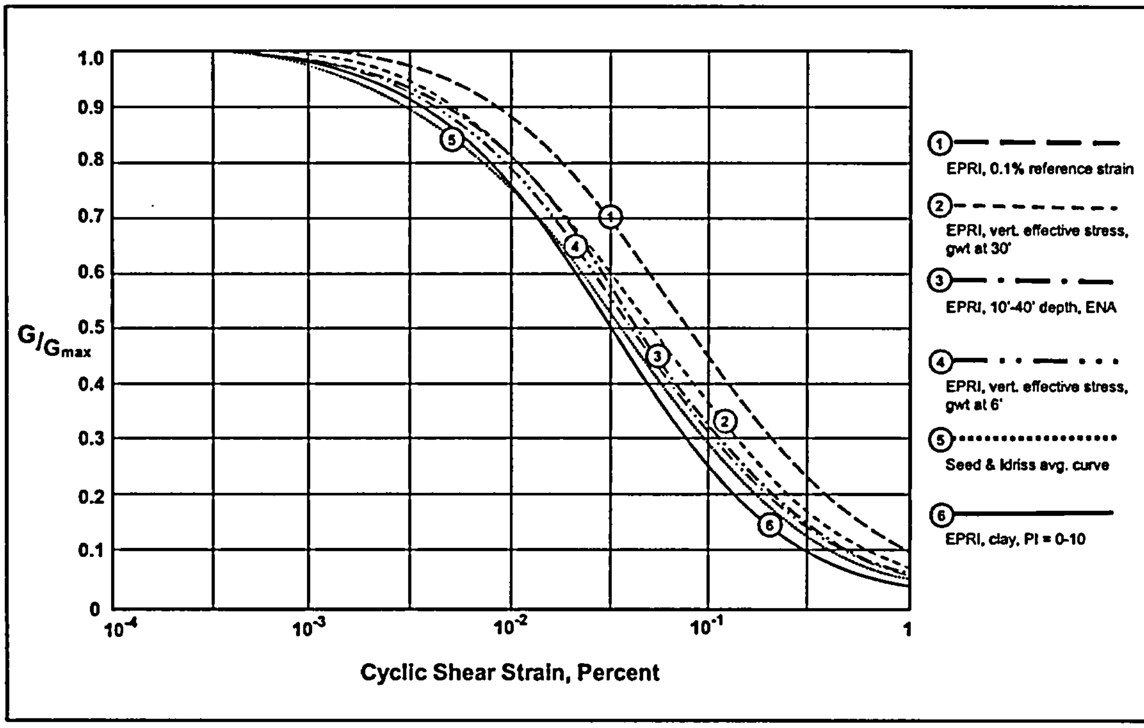

Figure 1. G/G_{max} versus Shear Strain Curves for Sands and Clay

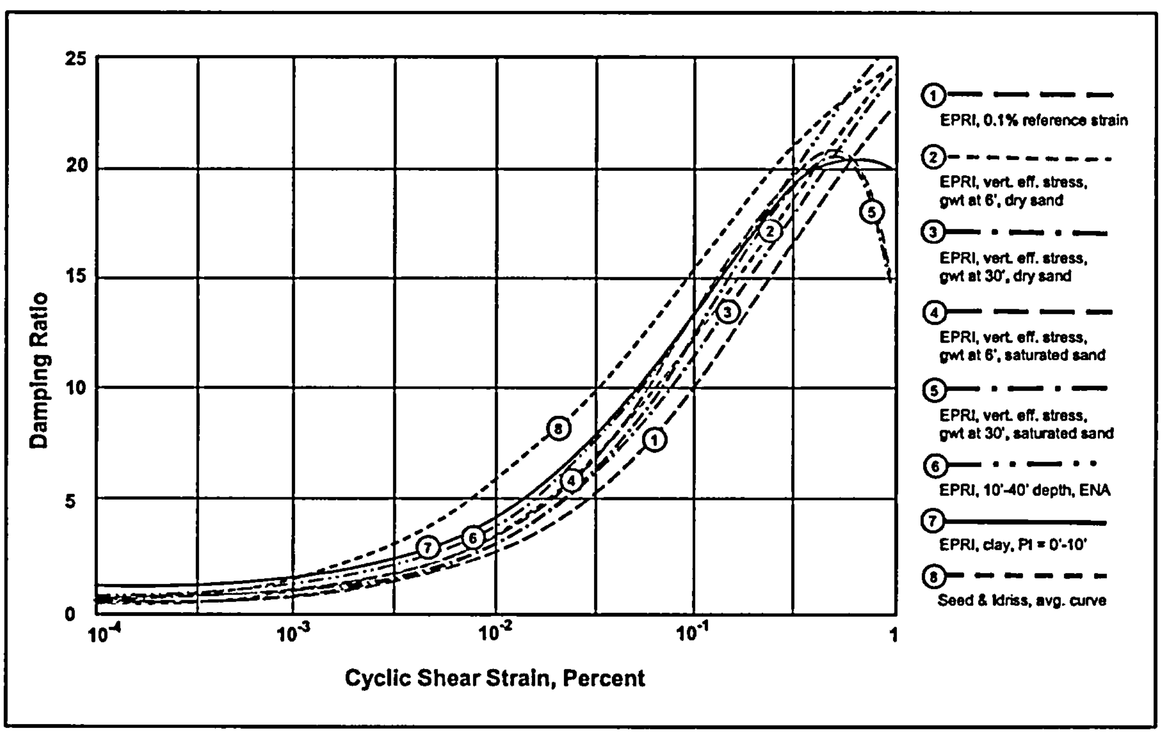

Figure 2. Damping Ratio versus Shear Strain Curves for Sands and Clay

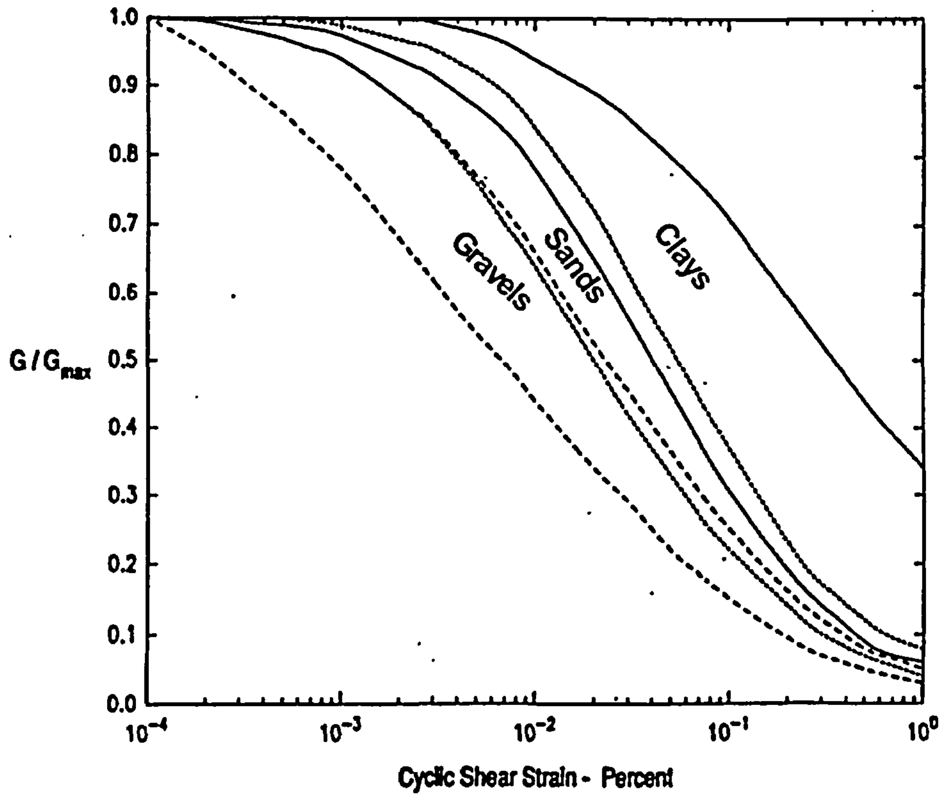

Exhibit 1. Typical Ranges for Modulus Reduction Curves

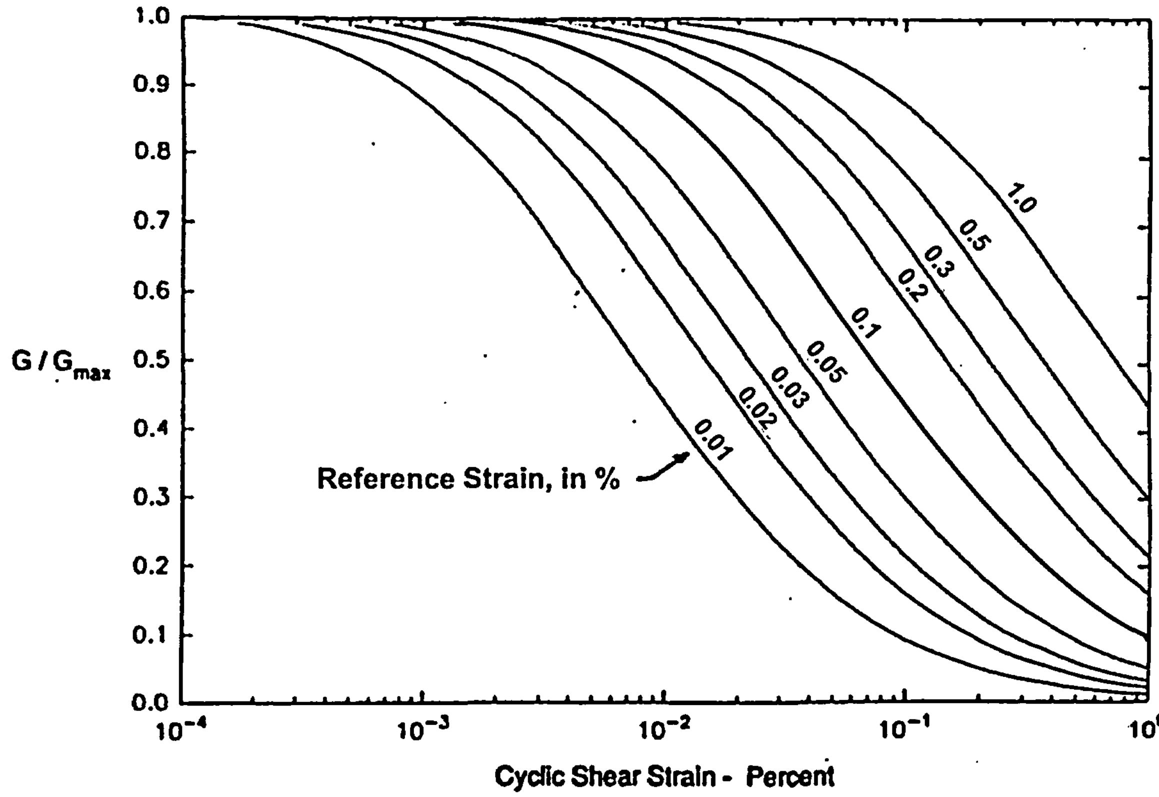

Exhibit 2: Recommended Modulus Reduction Curves
Page 1: Modulus Reduction as a Function of Reference Strain

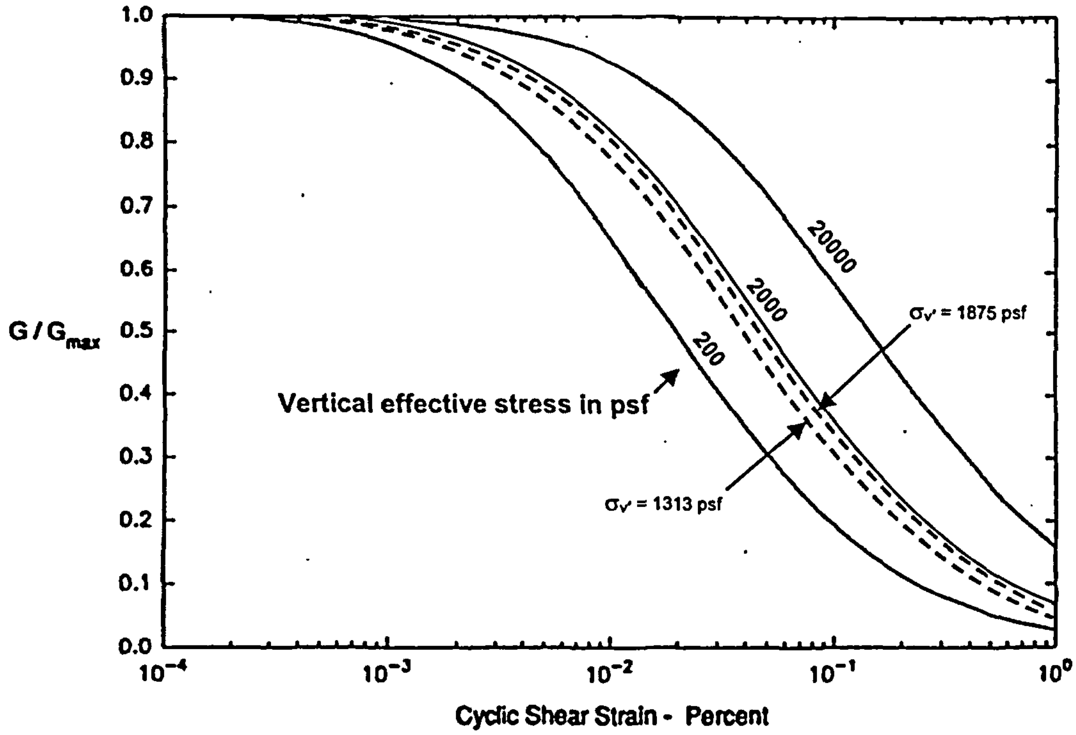

Exhibit 2: Recommended Modulus Reduction Curves
Page 2: Modulus Reduction Curves for Dry Sands

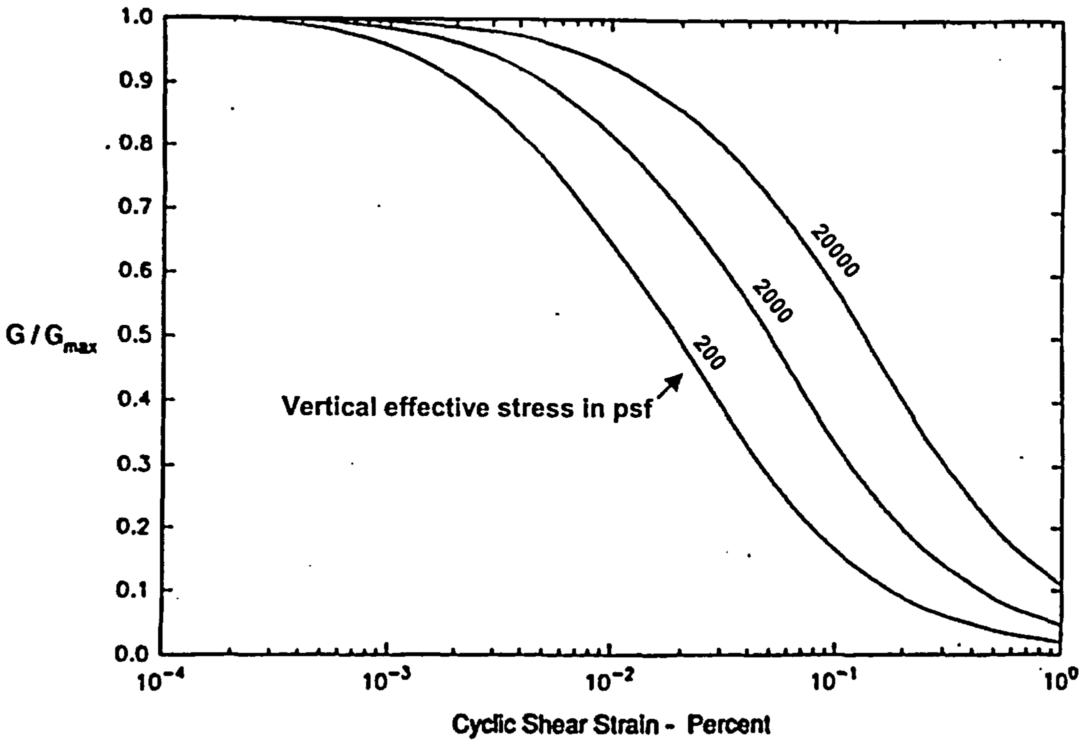

Exhibit 2: Recommended Modulus Reduction Curves
Page 3: Modulus Reduction Curves for Saturated Sands

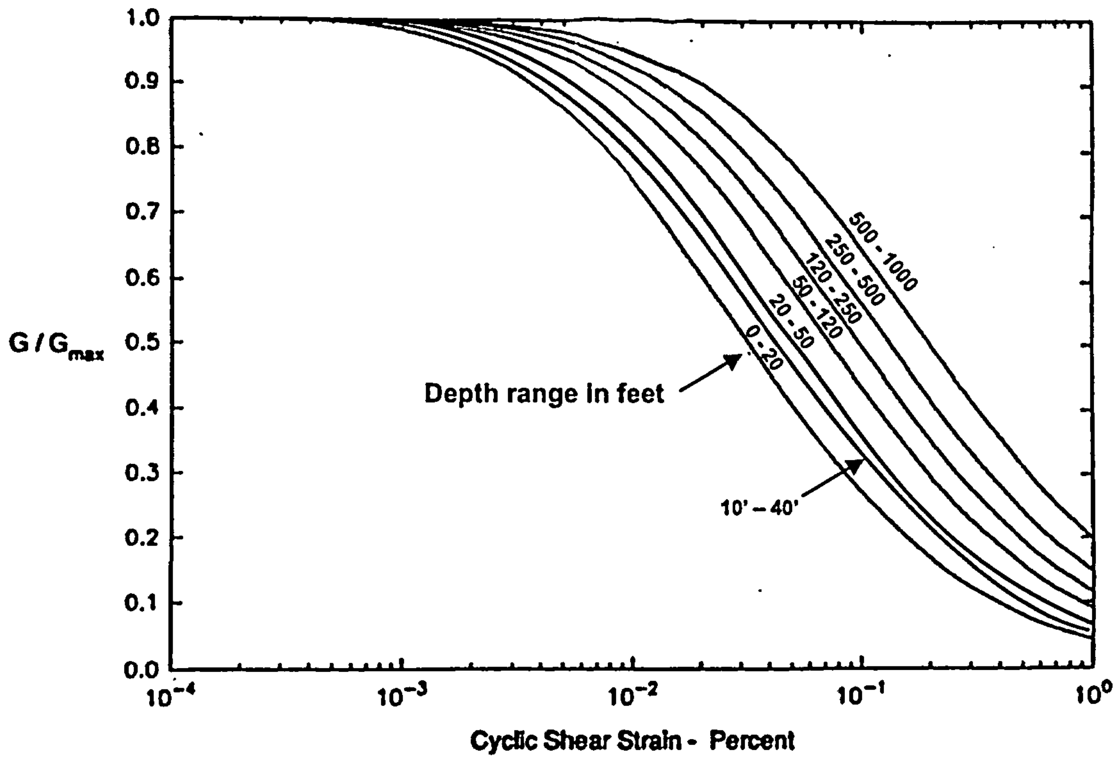

Exhibit 2: Recommended Modulus Reduction Curves
Page 4: Modulus Reduction Curves for Generic ENA Sites

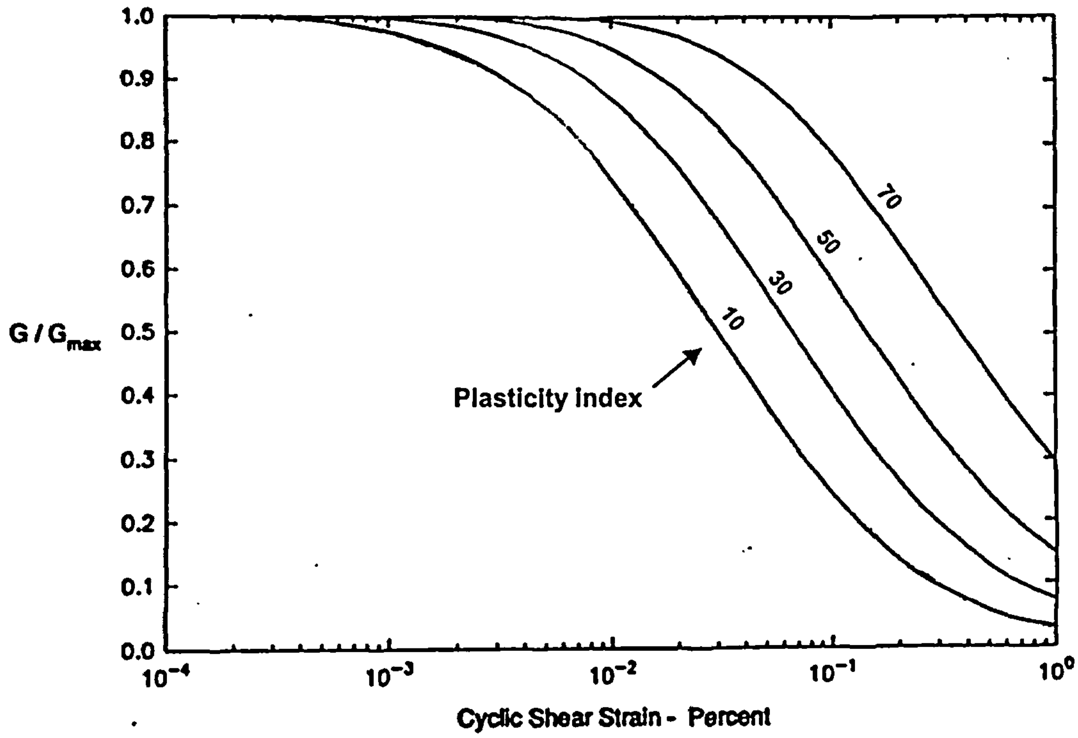

Exhibit 2: Recommended Modulus Reduction Curves
Page 5: Modulus Reduction Curves for Clays

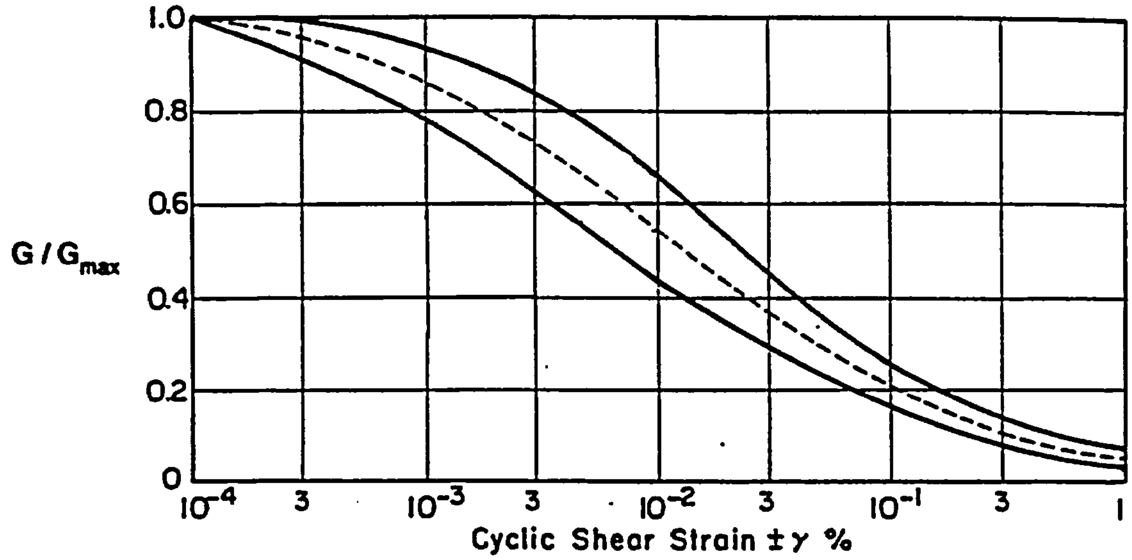

Exhibit 3: Modulus Reduction Curve for Gravel

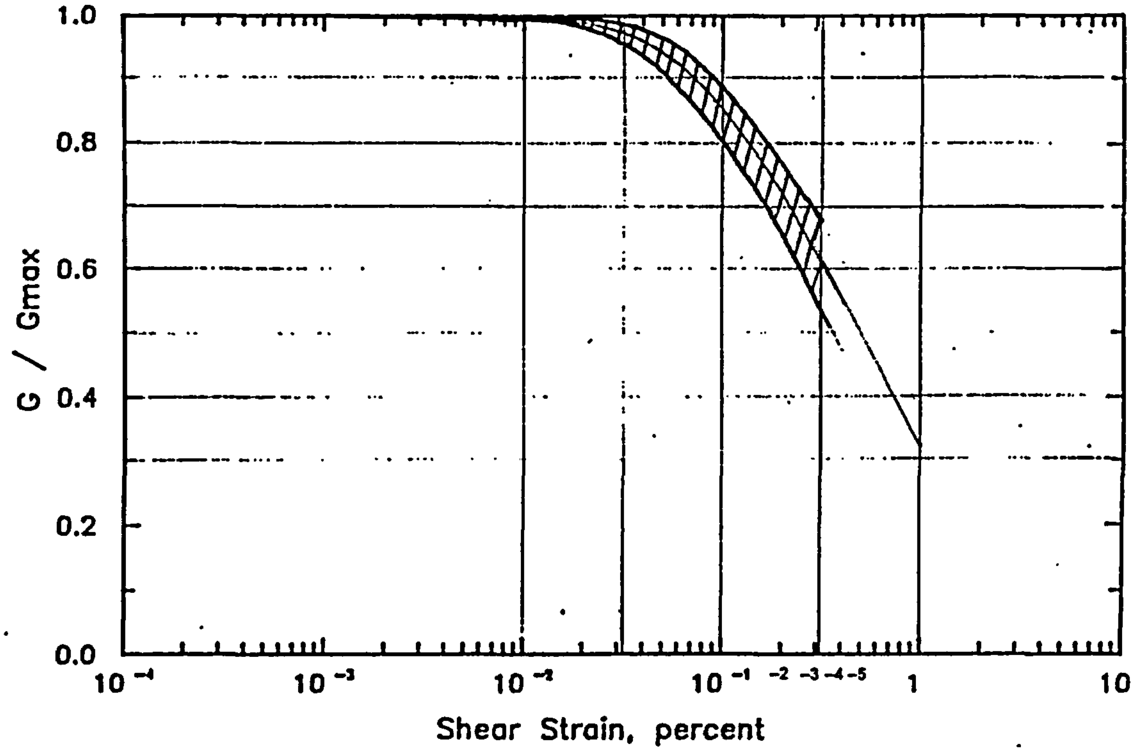

Exhibit 4: Modulus Reduction Curve for Mudstone

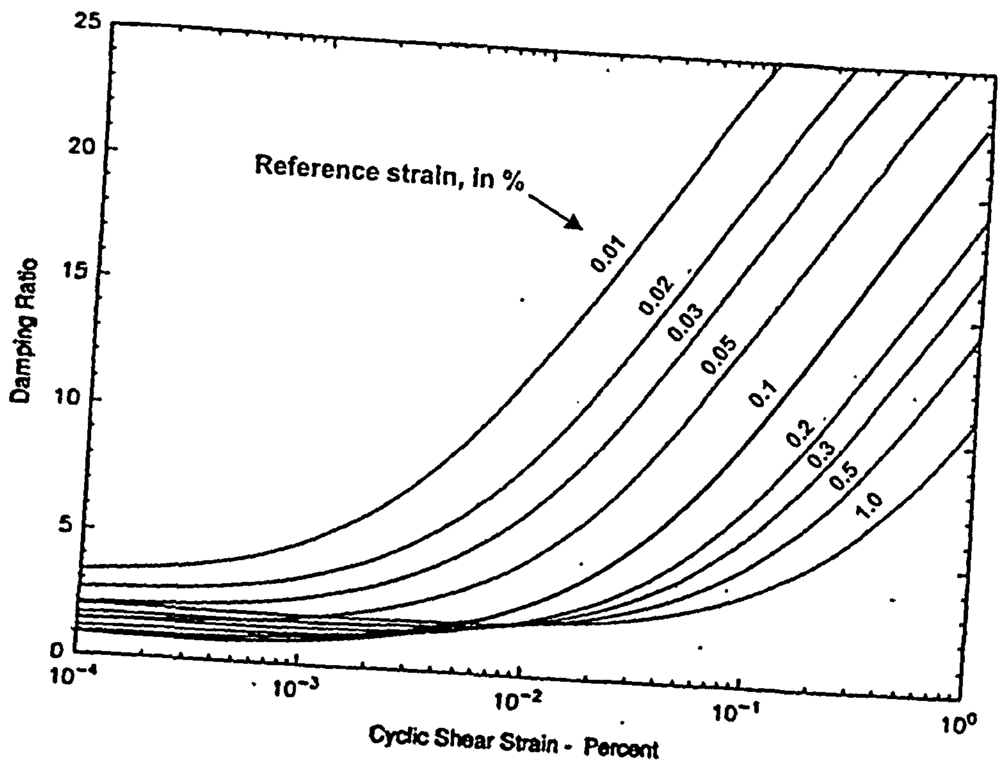

Exhibit 5: Recommended Damping Versus Strain Curves
Page 1: Damping Curves as a Function of Reference Strain

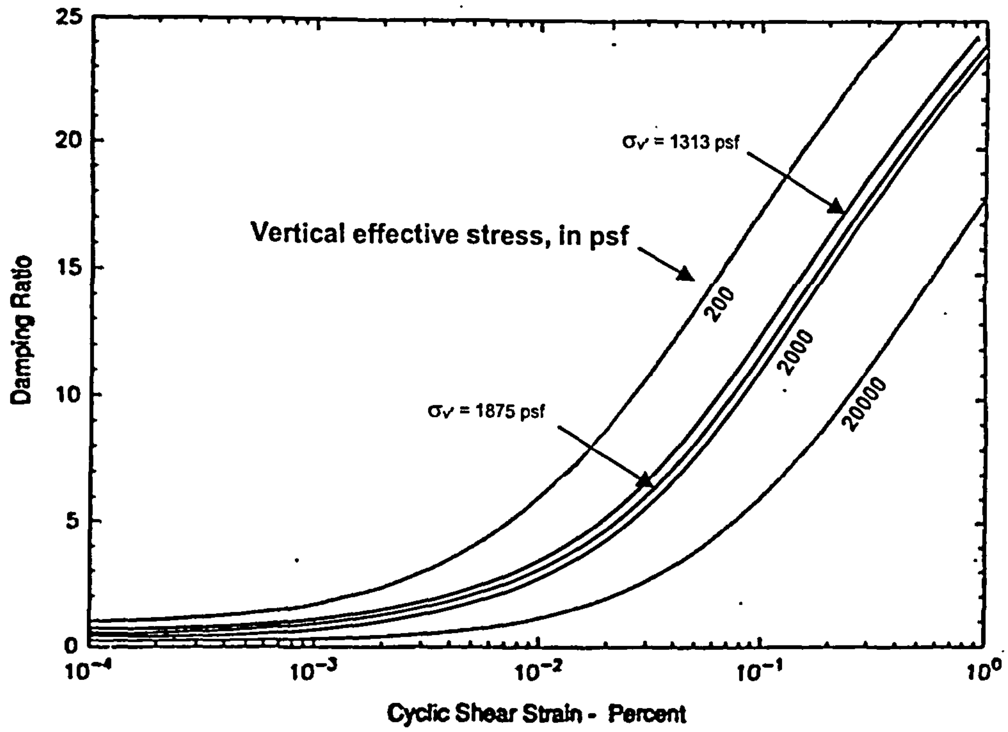

Exhibit 5: Recommended Damping Versus Strain Curves
Page 2: Damping Curves for Dry Sands

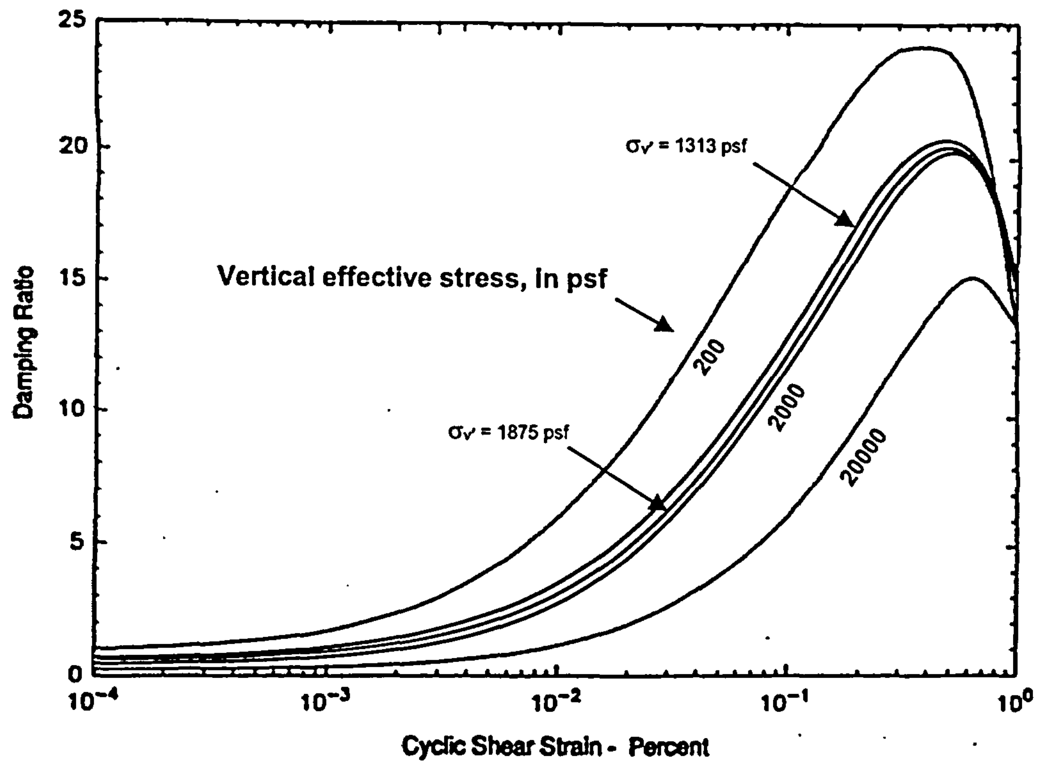

Exhibit 5: Recommended Damping Versus Strain Curves
Page 3: Damping Curves for Saturated Sands

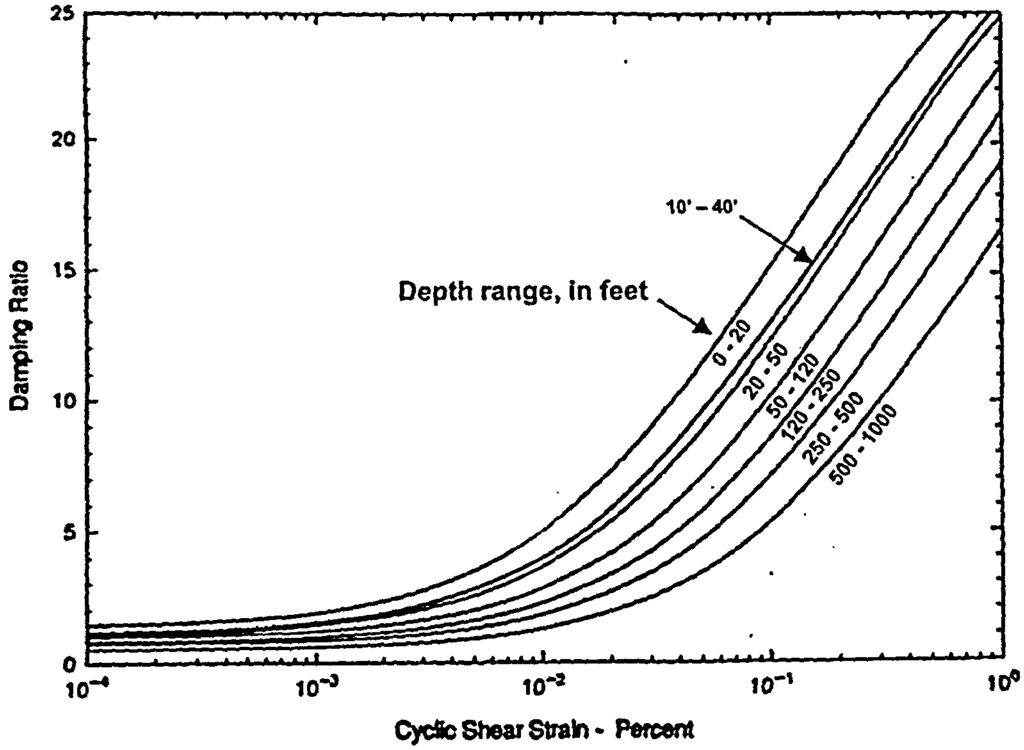

Exhibit 5: Recommended Damping Versus Strain Curves
Page 4: Damping Curves for Generic ENA Sites

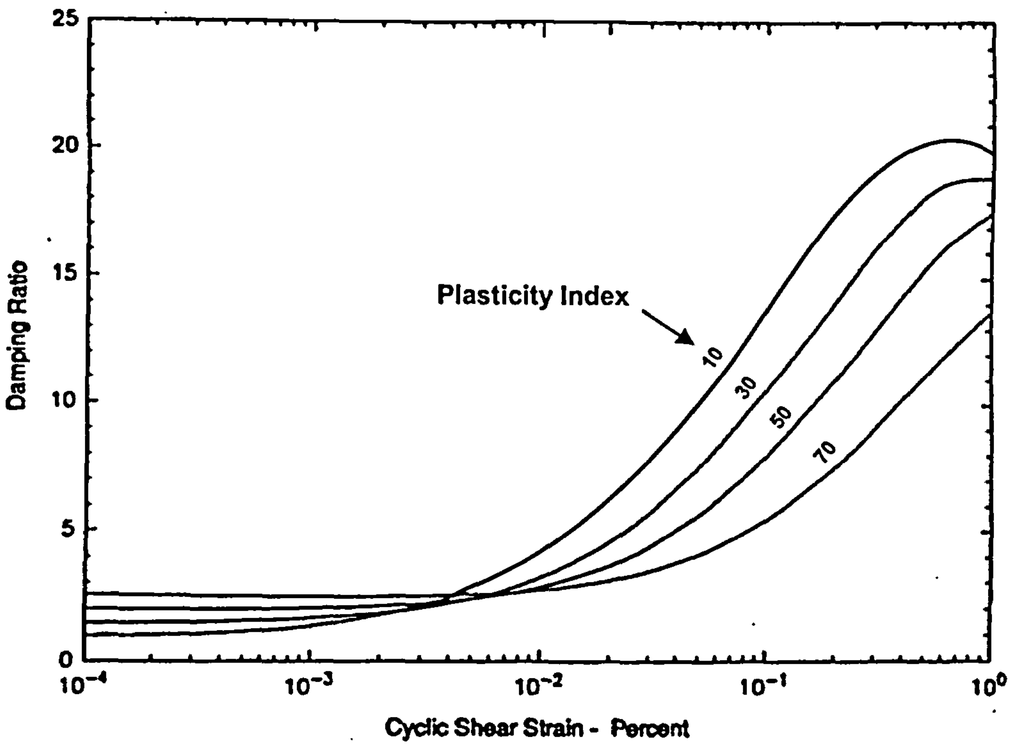

Exhibit 5: Recommended Damping Versus Strain Curves
Page 5: Damping Curves for Clays

RAI 2.5.4-11 (NRC 6/1/04 Letter)

Please provide a sample set of the calculations to substantiate the bearing capacities of soil and rock beneath major Category I structures, as shown in SSAR Table 2.5-47. Please indicate if and how the local site effects, such as the slope of the rock surface, fracture spacing, variability in properties, and evidence of shear zones, if any, were considered in determining the allowable bearing capacities of soil and rock for different structures.

Response

A copy of Bechtel Calculation 24830-G-004, "Bearing Capacity and Settlement Analysis," will be submitted by separate letter. The allowable bearing capacities given in SSAR Table 2.5-47 were derived in this calculation. These are reproduced below:

Table 1. Allowable Bearing Capacities

Zone	Allowable Bearing Capacity, q_a, ksf
IIA	4
IIB	8
III	16
III-IV	80
IV	160

Table 2 below illustrates the structural foundations that would be placed on the various type materials.

Table 2: Structure Foundations by Zone

Zone	Used for Reactor Containment Structure Foundation?	Used for Other Category 1 Structure Foundations?
IV	Yes	Yes
III-IV	Yes	Yes
III	No	Yes, as limited by bearing capacity. See Table 1.
II-B	No	Yes, as limited by bearing capacity. See Table 1.
Improved II-A(1)	No	Yes, as limited by bearing capacity. See Table 1.
II-A	No	No
I	No	No

(1) Improved Zone II-A saprolite is discussed in SSAR Section 2.5.4.12.

The maximum bearing pressure from the containment (reactor) building foundation is 15 ksf, which is only a fraction of the allowable bearing capacity of the bedrock. The allowable bedrock bearing capacity values given in the table are conservative presumptive values based mainly on building codes, and are themselves only a very small fraction of the theoretical ultimate bearing capacity of the rock. For example, the Zone III-IV rock has a design unconfined strength of 576 ksf, and the theoretical ultimate bearing capacity for mat foundations is several times the unconfined strength. The reason that the allowable bearing capacity is so much less than the theoretical ultimate capacity is because of the non-homogeneity of most rock masses, including variation in properties and the presence of fracture zones, referenced in the RAI. Thus the bearing capacity values for the Zone III-IV and Zone IV bedrock given in SSAR Table 2.5-47 take into account normal variations and fracturing in the rock. These bearing capacity values would only be affected if the bedrock were severely sloped, fractured, sheared, etc. This is discussed in the next paragraphs.

Sloped Rock Surface

As discussed in the response to RAI 2.5.4-1, bedrock level at the site is generally gently sloping, as shown on the two subsurface profiles in SSAR Figures 2.5-57 and 2.5-58, with steepest slopes in the 12 to 15% range. (Vertical exaggeration on these figures is approximately 5 and 2.5, respectively.) This is a normal rock surface slope.

Fracture Spacing

The recovery and rock quality designation (RQD) are typically good indicators of the degree of fracturing of cored rock. For the ESP borings, the recoveries for the Zone III-IV and Zone IV bedrock were around 90 and 100 percent, respectively, while the corresponding RQD values were around 50 and 95 percent. This puts the Zone III-IV rock into the "fair" category and the Zone IV rock into the "excellent" category according to Table 5.2 of Peck et al (1974). Some thin fracture zones found in the ESP borings are described in SSAR Section 2.5.1.2.3 and are discussed in the response to RAI 2.5.4-2. SSAR Section 2.5.1.2.6 concludes that, "The joints and fractures present in both zones (i.e., Zones III-IV and IV) are not considered to be of sufficient density or areal extent to affect the engineering behavior of the rock with respect to its foundation bearing capacity or integrity". Also, SSAR Section 2.5.4.10.1 states, "If excavation during construction reveals any weathered or fractured zones at foundation level, such zones would be overexcavated and replaced with lean concrete."

Variability of Properties

The variation in the strength of the bedrock was discussed in the response to RAI 2.5.4-6 Part c).

Shear Zones

As noted in SSAR Section 2.5.1.2.6 c:

> A shear zone was found in the Ta River Metamorphic Suite during the excavation for abandoned Units 3 and 4 at the North Anna Power Station. The shear zone was investigated by Dames and Moore (Reference 9) and the results presented to the U.S. Atomic Energy Commission. The results of the investigation concluded that movement occurred along the shear zone approximately 200 million years ago, and that movement has not occurred since, or at least not within the last one million years, given the relatively undisturbed thickness of residual soil that overlies the shear zone. The results of the investigation also concluded that the shear zone is of limited extent, and while it was traced through the Units 1 and 2 foundation area, no evidence of movement was observed along this section of the shear zone.
>
> The U.S. Atomic Energy Commission, following a review of the results of the above mentioned investigation, concluded that the shear zone at the site is not "capable", within the meaning of Section III (g) of 10 CFR 100, Appendix A (Reference 108).

No evidence of shear zones was found during the ESP subsurface investigation.

References

Peck, R. B., W. E. Hanson, and T. H. Thornburn. Foundation Engineering, Second Edition, John Wiley and Sons, Inc., New York, 1974 (Reference 182 of SSAR Section 2.5).

<u>Application Revision</u>

Table 2.5-47 will be revised as follows:

Table 2.5-47 Allowable Bearing Capacity Values

Zone	Allowable Bearing Capacity, ksf
IIB	8
III	16
III-IV	80(1)
IV	160(1)

(1) The new containment (reactor) buildings would be founded on Zone III-IV or Zone IV material.

Note: The above values include a factor of safety against bearing failure of at least 3. Minimum assumed foundation width is 5 feet. Minimum assumed foundation depth is 3 feet.

RAI 2.5.4-12 (NRC 6/1/04 Letter)

SSAR Section 2.5.4.11 (Design Criteria) states that geotechnical-related design criteria that pertain to structural design are not included in the application. Please provide the reasons for not providing the geotechnical-related design criteria that pertain to structural design (such as sliding, and overturning).

Response

SSAR Section 2.5 deals with site-related issues and not with specific structural design issues. The design criteria noted in SSAR Section 2.5.4.11 are either (a) not related to any specific structural design, e.g., factor of safety against liquefaction or slope stability failure, or (b) they are specific to the ESP site, e.g., allowable bearing capacity of the site soils and bedrock.

Structural criteria such as allowable wall rotation and factors of safety against structure sliding or overturning are not site specific and thus, for consistency, are not included in SSAR Section 2.5. These criteria would be established during detailed engineering and described in a design certification and/or COL application.

Application Revision

None.

RAI 17.1-2 (NRC 6/1/04 Letter)

Sections 8 and 9 of Dominion's Early Site Permit Application Development Quality Assurance Manual and Section 4 of Bechtel's Quality Assurance Program Plan state that the safety-related scope of the development of the ESP application would not involve the use of quality assurance measures for the identification and control of materials, parts, and components and for the control of special processes. Please describe why these quality assurance measures were not applicable to the development of the ESP application. Alternatively, if these quality assurance measures were applicable to the ESP application, please describe the quality assurance measures used by Dominion and the primary contractor (Bechtel) for these activities.

Response

1. Dominion ESP QA Manual

[Please note that Dominion's Early Site Permit Application Development Quality Assurance Manual addresses the quality requirements mentioned in the question in its Sections 9 and 10, not in Sections 8 and 9, as stated in the question.]

Under Dominion's overall direction, several companies were involved in the preparation of the North Anna ESP application. The quality requirements imposed on the various companies differed depending on their scope of work. For example, Section 9 of Dominion's own ESP QA Manual, "Identification and Control of Material, Parts and Components," was deemed not applicable to Dominion on the basis that no safety-related, materials, parts, or components were to be procured within Dominion's project scope. Section 10 of Dominion's ESP QA Manual, "Control of Special Processes," was also deemed not applicable to the ESP project because Dominion's project activities did not involve the use of special processes in the development of the ESP application.

More importantly, Section 5 of the Dominion ESP QA Manual, "Procurement Document Control," governs supporting company involvement to ensure that appropriate quality requirements are included in procurement documents defining the scope of the supporting companies' project involvement. In accordance with the Dominion QA Manual requirements, Dominion selected Bechtel as a primary contractor and awarded a contract for the development of the ESP application. The contract documents invoked 10 CFR 50 Appendix B and required that Bechtel prepare and submit a project-specific QA Program Plan for the project, meeting the Dominion ESP QA Manual requirements. Bechtel's Quality Assurance Program Plan (QAPP), which is based on the Bechtel Nuclear QA Manual (NQAM), was reviewed and approved by Dominion to verify that it met Dominion's ESP QA Manual requirements.

2. Bechtel Quality Assurance Program Plan

Bechtel's NQAM Policy Q-8.1, "Identification and Control of Materials, Parts, and Components," contains quality assurance measures for the identification and control of materials, parts, and components and Policy Q-9.1, "Control of Special Processes," contains requirements for the control of special processes. Both of these policies were not invoked in the ESP project-specific QAPP because the Bechtel scope of work for the ESP project does not include procurement and/or receipt of safety-related materials, parts, or components or other construction activities involving special processes. Dominion reviewed and approved the Bechtel QAPP.

However, to ensure that appropriate quality program requirements are specified in procurement documents for Bechtel suppliers/subcontractors, Bechtel NQAM Policy Q-4.1, "Preparation of Procurement Documents," governs. Policy Q-4.1 defines measures for determining and specifying appropriate quality program requirements for suppliers/subcontractors. Such quality program requirements receive a review and concurrence by Quality Assurance.

Bechtel used four subcontractors on the ESP project: MACTEC, Risk Engineering Incorporated (REI), Tetra-Tech NUS, Inc (TtNUS) and William Lettis & Associates (WLA). The activities of these subcontractors and the related quality program requirements are described below:

- MACTEC

  MACTEC provided surveying and sub-soil investigation services that included core drilling, sampling and laboratory testing of samples, etc. This included traceability of core samples. No special processes, such as, welding or non-destructive examination were included in MACTEC's scope of work.

  For the above services, Bechtel's specification defined the work scope and technical and quality requirements. The specification required that the subcontractor's QA program meet the requirements of 10 CFR 50 Appendix B. The specification also required labeling of core samples in accordance with the applicable ASTM D2113.

  MACTEC's QA program was satisfactorily audited by Bechtel to verify that appropriate program elements were covered and were being implemented. This included the 10 CFR 50 Appendix B, Criterion 8, "Identification and Control of Materials, Parts and Components."

- Risk Engineering

  REI provided computational and expert consulting services to prepare the SSAR Section 2.5 probabilistic seismic hazard sensitivity analyses. The work scope did not include any hardware requiring identification and control or special processes.

  Bechtel's service requisition defined the work scope and technical and quality program requirements. The service requisition required that the subcontractor's QA program meet the requirements of 10 CFR 50 Appendix B.

  REI's QA program was satisfactorily audited by Bechtel to verify that appropriate program elements were covered and were being implemented.

- Tetra-Tech NUS

  TtNUS provided services for preparing certain sections of the Environmental Report. These services included data collection, impact analysis, and document preparation. The work scope did not include any hardware requiring identification and control or special processes.

  The services of TtNUS are non-safety related. Bechtel's service requisition defined the work scope and technical and quality program requirements. The service requisition required that the subcontractor have a QA program that is compatible with the provisions and requirements of ISO 9000.

  TtNUS' QA program was satisfactorily audited by Bechtel to verify that appropriate program elements were covered and were being implemented.

- William Lettis & Associates

  WLA provided services related to geologic mapping and characterization of seismic sources for preparation of SSAR Section 2.5 and related ER sections. The work scope did not include any hardware requiring identification and control or special processes.

  Bechtel's service requisition defined the work scope and technical and quality program requirements. The service requisition required that the Subcontractor perform work in accordance with Bechtel's 10 CFR 50, Appendix B, QA program as described by the Bechtel QAPP and the implementing procedures contained in Bechtel's Project Engineering Procedures Manual.

Application Revision

None.