Exelon. Generation

NP-11-0029
July 8, 2011

10 CFR 52, Subpart A

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555-0001

Subject: Exelon Nuclear Texas Holdings, LLC
Victoria County Station Early Site Permit Application
Response to Request for Additional Information Letter No. 10
NRC Docket No. 52-042

Attached are responses to NRC staff questions included in Request for Additional Information (RAI) Letter No. 10, dated May 24, 2011, related to Early Site Permit Application (ESPA), Part 2, Sections 02.05.04, 02.05.05, 11.02 and 11.03. NRC RAI Letter No. 10 contained thirty-six (36) Questions. This submittal comprises a partial response to RAI Letter No. 10, and includes responses to the following twelve (12) Questions:

02.05.04-2	02.05.04-8	02.05.05-4
02.05.04-4	02.05.04-9	02.05.05-7
02.05.04-6	02.05.04-12	02.05.05-13
02.05.04-7	02.05.04-16	02.05.05-14

When a change to the ESPA is indicated by a Question response, the change will be incorporated into the next routine revision of the ESPA, planned for no later than March 31, 2012.

Of the remaining twenty-four (24) RAIs associated with RAI Letter No. 10, responses to eight (8) Questions were submitted to the NRC in Exelon Letter NP-11-0026, dated June 23, 2011. The response to RAI Questions 02.05.04-5, 02.05.04-10, 02.05.04-15, 02.05.04-17, 02.05.05-1, 02.05.05-6, 02.05.05-9, 02.05.05-12, 02.05.05-15, 02.05.05-16, and 02.05.05-17 will be provided by July 22, 2011. The response to RAI Questions 02.05.04-13, 02.05.04-14, 02.05.05-2, 02.05.05-3, and 02.05.05-8 will be provided by August 5, 2011. These response times are consistent with the response times described in NRC RAI Letter No. 10, dated May 24, 2011.

Regulatory commitments established in this submittal are identified in Attachment 13.

If any additional information is needed, please contact David J. Distel at (610) 765-5517 or Joshua Trembley at (610) 765-5345.

I declare under penalty of perjury that the foregoing is true and correct. Executed on the 8th day of July, 2011.

Respectfully,

Marilyn C Kray

Marilyn C. Kray
Vice President, Nuclear Project Development

Attachments:
1. Question 02.05.04-2
2. Question 02.05.04-4
3. Question 02.05.04-6
4. Question 02.05.04-7
5. Question 02.05.04-8
6. Question 02.05.04-9
7. Question 02.05.04-12
8. Question 02.05.04-16
9. Question 02.05.05-4
10. Question 02.05.05-7
11. Question 02.05.05-13
12. Question 02.05.05-14
13. Summary of Regulatory Commitments

cc: USNRC, Director, Office of New Reactors/NRLPO (w/Attachments)
USNRC, Project Manager, VCS, Division of New Reactor Licensing (w/Attachments)
USNRC Region IV, Regional Administrator (w/Attachments)

RAI 02.05.04-2:

Question:

In accordance with 10 CFR 100.23(d)(4), staff requests that the applicant provide the following:

a) SSAR Table 2.5.4-32 presents the design values for several geotechnical parameters including friction angle. Table 2.5.4-18 shows measured values recorded from direct shear testing and these values do not match the design values in Table 2.5.4-32. Please provide the basis for the friction angle values presented in Table 2.5.4-32 for each of the soil layers.

b) SSAR Table 2.5.4-32 presents an estimated SPT $(N_1)_{60}$ value of 30 for structural fill. Please explain how this value was developed.

c) SSAR Subsection 2.5.4.2.1.3.11 states that the friction angle for all clay strata is 20°. Please explain the basis for this value and why this value was not included in Table 2.5.4-32.

Response:

Part (a)

Although direct shear testing on undisturbed samples would be expected to provide a realistic value of effective internal friction angle (ϕ'), industry experience has been that it produces results that do not necessarily reflect the expected range of values based on other parameters that influence ϕ', such as relative density, grain shape and grain size distribution. Direct shear test results are considered along with ϕ' values derived by other means to obtain a best-estimate friction angle value.

The ϕ' value, in degrees, is derived from SPT values based on the following equation (Das, 2002):

$$\phi' = 27.1 + 0.3\ N_{60} - 0.00054\ N_{60}^2$$

where N_{60} is the SPT N-value normalized to 60% efficiency.

The ϕ' value in degrees is derived from CPT values based on the following equation (Conetec, 2004):

$$\phi' = \arctan([\log(q_c / \sigma_{v0}') + 0.29] \times (1 / 2.68))$$

where q_c is the measured CPT tip resistance, and σ_{v0}' is the effective overburden pressure.

Table 1 shows, for each sand layer, the average ϕ' derived from SPT N-values and CPT q_c values, as well as the average values from the direct shear (DS) testing. Also shown are the average fines contents for each layer from SSAR Table 2.5.4-A-32. The average tabulated ϕ' value is the arithmetic mean of the SPT, CPT and DS results. Note that in all cases, the average values derived from the SPT and CPT results are higher than the average DS results; however, a significant reduction in ϕ' is assumed based on fines content greater than 15 percent. As a result, the adopted ϕ' value is generally closer to the DS value than to the value derived from the SPT. In addition, the adopted ϕ' value is either equal to or less than the average calculated value.

Table 1: Summary of Average ϕ'-Values (in degrees)

Stratum	Fines, %	SPT	DS	CPT	Average ϕ', (degrees)	Use ϕ', (degrees)
Sand 1	40	36	--	40	38	33
Sand 2	40	35	--	38	36	33
Sand 4	25	43	34	39	39	37
Sand 5	25	38	--	--	38	36
Sand 6	15	45	33	--	39	39
Sand 8	30	47	35	--	41	36
Sand 10	25	48	33	--	41	38
Sand 12	25	43	--	--	43	36
Sand 14	30	45	--	--	45	36
Sand 16	25	45	--	--	45	38
Sand 18	25	50	--	--	50	40

Part (b)

SSAR Section 2.5.4.2.1.4.1 indicates that, although selection of structural fill has not been finalized, it is expected to be similar to the well-graded gravel and sand with trace amounts of fines produced by a local supplier in Victoria, Texas. Tests on this material showed it to be 50 percent gravel, 43 percent sand and 7 percent fines. Compaction in accordance with the modified Proctor test (ASTM D 1557) gave a maximum dry density of 138 pcf at an optimum moisture content of 5.5 percent. Direct shear testing gave a ϕ' value of 42 degrees. For conservatism, the recommended ϕ' value is 39 degrees, which is the lowest measured value from other similarly sampled and tested materials described in the response to RAI 02.05.04-5.

Although no direct correlation exists between compaction, expressed in terms of modified Proctor dry density, and compaction, expressed in terms of relative density, when a high degree of compaction is required for granular fill in the field, at least 95 percent modified Proctor dry density and at least 70 percent relative density are specified. Thus, the actual densities obtained using these two criteria will be similar. Table 2 (from USACE, 1992) shows approximate correlations between relative density and (1) angle of internal friction and (2) N_{60} for sands. For a sandy gravel (coarse-grained in the table) with a relative density of 70 percent, the ϕ' value from Table 2(a) is 42 to 43.5 degrees, depending on the degree of uniformity. This agrees well with the 42 degrees measured in the direct shear testing. Table 2(b) shows that at 70 percent relative density, the average N_{60} value expected is 40 bpf. It is expected that the value for coarser-grained, well-graded sands will be more than 40 bpf. Thus the N_{60} value of 30 bpf assumed in the SSAR is conservative.

Table 2: Effective Angle of Internal Friction of Sands

a. Relative Density and Gradation (Data from Schmertmann 1978)

Relative Density D_r, Percent	Fine-Grained		Medium-Grained		Coarse-Grained	
	Uniform	Well-Graded	Uniform	Well-Graded	Uniform	Well-Graded
40	34	36	36	38	38	41
60	36	38	38	41	41	43
80	39	41	41	43	43	44
100	42	43	43	44	44	46

b. Relative Density and In Situ Soil Tests

Soil Type	Relative Density D_r, Percent	Standard Penetration Resistance N_{60} (Terzaghi and Peck 1967)	Cone Penetration Resistance q_c, ksf (Meyerhof 1974)	Friction Angle ϕ', deg		
				Meyerhof (1974)	Peck, Hanson and Thornburn (1974)	Meyerhof (1974)
Very Loose	< 20	< 4	—	< 30	< 29	< 30
Loose	20–40	4–10	0–100	30–35	29–30	30–35
Medium	40–60	10–30	100–300	35–38	30–36	35–40
Dense	60–80	30–50	300–500	38–41	36–41	40–45
Very Dense	> 80	> 50	500–800	41–44	> 41	> 45

Part (c)

As noted in the question, SSAR Section 2.5.4.2.1.3.11 states, "For all clay strata, ϕ' is taken as 20 degrees." Section 2.5.4.2.1.3.11 discusses Static Earth Pressure Coefficients, and thus the assumption of $\phi' = 20$ degrees is for computing static earth pressure coefficients. Accordingly it is not necessarily recommended as a design value for use in situations other than computing static earth pressure coefficients, and is not included in Table 2.5.4-32. As static earth pressure coefficients for clay were only computed for Clay 1, Clay 3 and Clay 5, the $\phi' = 20$ degrees is only applied to these three layers.

Values of ϕ' obtained from consolidated undrained triaxial tests with porepressure measurements on the overconsolidated CH and CL clays in south and south central Texas tend to vary widely. This type of testing, in support of the STP 3 & 4 COLA, on similar types of soils gave ϕ' values ranging from zero to 32 degrees. For computing static earth pressure coefficients, $\phi' = 20$ degrees was used in the STP 3 & 4 COLA. This value gives reasonable results for computing K_0 values for Clay 1, Clay 3 and Clay 5, as demonstrated in the response to RAI 02.05.04-6, as shown next.

The table below is reproduced from the response to RAI 02.05.04-6.

Table 3: K_0 Values

	$K_{0,OCR}$		
	Alpan	**Wroth**	**Average**
Clay 1	0.86	0.64	0.75
Clay 3	0.72	0.60	0.66
Clay 5	0.60	0.57	0.59

The table shows the K_0 values for Clay 1, Clay 3 and Clay 5 using two different approaches that do not involve ϕ'. Using the simplified Jaky equation with $\phi' = 20$ degrees gives:

$K_0 = 1 - \sin \phi' = 0.66$ which compares reasonably with the "Average" tabulated values.

Response References:

ConeTec, Inc. and Greg InSitu, Inc. (2004). "Cone Penetration Testing Geotechnical Applications Guide," 4th edition, Toronto, Ontario.

Das, B.M. (2002). Principles of Geotechnical Engineering, 5th Edition, Wadsworth Group, California.

Meyerhof, G.G.(1974). "Ultimate Bearing Capacity of Footings on Sand Overlying Clay," *Canadian Geotechnical Journal,* Volume 11, pp 223-229.

Peck, R.B., Hanson, W.E., and T.H.Thornburn (1974). Foundation Engineering, John Wiley and Sons Ltd., New York.

Schmertmann, J.H. (1978). Guidelines for Cone Penetration Test Performance and Design, Report No. FHWA-TS-78-209., FHWA. McLean, VA.

Terzaghi, K., and R.B. Peck (1967). Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, ltd, New York.

USACE (1992). U.S. Army Corps of Engineers. Bearing Capacity of Soils, EM-1110-1-1905, Washington, DC.

Associated ESPA Revisions:

The second paragraph of SSAR Section 2.5.4.2.1.3.11 will be revised in a future revision of the ESPA, as follows:

Calculated static earth pressure coefficients are given in Table 2.5.4-32 for the power block area and in Table 2.5.4-33 for the cooling basin. Because foundations are unlikely to be constructed deeper than Stratum Sand 5, earth pressure coefficients are not calculated below this stratum. For each sand stratum, ϕ' is taken as the value recommended for use in Table 2.5.4-32 for the power block area and in Table 2.5.4-33 for the cooling basin. For all clay strata Clay 1, Clay 3, and Clay 5, ϕ' is taken as 20 degrees to compute static earth pressure coefficients.

RAI 02.05.04-4:

Question:

SSAR Figures 2.5.4-56 through 2.5.4-59 plot OCRs estimated from CPT results and laboratory testing using Equation 2.5.4-7, which applies to a plastic limit (PI) of 40%. However, none of the PI values presented in Tables 2.5.4-16 and 2.5.4-17 are equal to 40% except for Clays 5, 13, and 15. In accordance with 10 CFR 100.23(d)(4), explain the appropriateness of Equation 2.5.4-7, as several of the PI values are less than 40%.

Response:

SSAR Equation 2.5.4-7 represents a curve in Figure 5.12 of Lunne et al (1997). The curve is a plot of the ratio of undrained shear strength (s_u) to effective overburden pressure (σ_v') (y-axis), against overconsolidation ratio (OCR) (x-axis). As noted in the question, this curve is for soil with PI = 40 percent. This plot also contains curves for PI = 30, 20 and 10. It is noted that the curves for PI = 40 and PI = 30 are very close to each other.

Equation 2.5.4-7 was used for Clay 1 and Clay 3, because this equation is based on undrained shear strength derived from CPT results. From SSAR Table 2.5.4-A-32 the PI values for Clay 1 and Clay 3 are 35 and 30, respectively.

From SSAR Table 2.5.4-A-26, the average OCR in the power block area, derived from Equation 2.5.4-7, is 4.9 for Clay 1 and 1.9 for Clay 3. From Lunne et al (1997), Figure 5.12, for Clay 1, using the PI = 40 curve, the s_u/σ_v' value for OCR = 4.9 is approximately 1.4. For s_u/σ_v' = 1.4, using the PI = 30 curve, OCR is approximately 5.7. Interpolating, OCR for PI = 35 is about 5.3. For Clay 3, using the PI = 40 curve, the s_u/σ_v' value for OCR = 1.9 is approximately 0.6. For s_u/σ_v' = 0.6, OCR using the PI = 30 curve is approximately 2.0.

Thus, if the interpolated PI value of 35 is used from Lunne et al (1997) Figure 5.12, the OCR for Clay 1 (top) increases by about 8 percent and the OCR for Clay 3 increases by about 5 percent. Typically, lower OCR values are considered to be more conservative, and thus the OCR values obtained from the PI = 40 curves for Clay 1 and Clay 3 would be considered slightly conservative. It is also noted that the average OCR values estimated from laboratory consolidation tests in SSAR Table 2.5.4-A-26 are 2.7 for Clay 1 and 2.0 for Clay 3. For Clay 1, the laboratory test result differs more from the CPT derived values than the difference obtained using different PI curves.

In conclusion, using the PI = 40 curve for the Clay 1 and Clay 3 soils is slightly conservative, and is reasonable.

Response Reference

Lunne, T., Robertson, P.K., and J.J.M. Powell, Cone Penetration Testing in Geotechnical Practice, Blackie Academic and Professional, London, 1997.

Associated ESPA Revision:

No ESPA revision is required as a result of this response.

RAI 02.05.04-6:

Question:

SSAR Subsection 2.5.4.2.1.4.1 presents the lateral earth-pressure coefficients. In accordance with 10 CFR 100.23 (d)(4), explain why Ko=1-sin (φ') equation was used since Das (2010)* only recommends use of this equation for coarse-grained soils. Please justify why this equation for Ko is representative for the site.

*Das, Braja M. (2010). Principles of Geotechnical Engineering, Seventh Edition, Cengage Learning.

Response:

Although the original simplified Jaky equation $K_0 = 1 - \sin \varphi'$ was derived for granular soils, using the effective angle of friction for overconsolidated clay soils has been shown to provide a reasonable estimate of K_0. As noted in SSAR Section 2.5.4.2.1.3.11, the φ' value for computing K_0 for the shallower clay soils (Clay 1, Clay 3 and Clay 5) was taken as 20 degrees. This gives $K_0 = 0.66$, which was rounded up to 0.7. This is a typical value for K_0 for lightly overconsolidated clays (overconsolidation ratio (OCR) < 5).

Bowles (1982) provides alternative methods from the literature for computing K_0 for overconsolidated clays. Alpan (1967) gives $K_{0,NC}$ for normally consolidated clays as:

$K_{0,NC} = 0.19 + 0.233 \log PI$, where PI is the plasticity index of the clay.

$K_{0,OCR}$ is derived as:

$$K_{0,OCR} = K_{0,NC} \times (OCR)^n$$

Where $n = 0.54 \times 10^{-PI/281}$

Wroth (1975) gives the following:

$$K_{0,OCR} = OCR \times K_{0,NC} - (u'/1-u')(OCR-1)$$

Where $K_{0,NC}$ is as defined in the equation by Alpan, and u' is Poisson's ratio in terms of effective stress or $u' = 0.23 + 0.003PI$

From SSAR Table 2.5.4-A-26, the OCR values for Clay 1, Clay 3 and Clay 5 are 3.0, 2.0 and 1.2, respectively. From SSAR Table 2.5.4-A-32, the corresponding PI values are 35, 30 and 40, respectively. Using these values in the equations proposed by Alpan and Wroth gives the following $K_{0,OCR}$ values.

Table 1: $K_{0,OCR}$

	$K_{0,OCR}$		
	Alpan	**Wroth**	**Average**
Clay 1	0.86	0.64	0.75
Clay 3	0.72	0.60	0.66
Clay 5	0.60	0.57	0.59

Given that K_0 is extremely difficult to measure in situ (Bowles, 1982) and all of the approaches outlined above are empirical, a value of $K_0 = 0.66$ rounded up to 0.7 based on $K_0 = 1 - \sin\varphi'$ with $\varphi' = 20$ degrees is a reasonable value and is supported by the values derived using the Alpan and Wroth equations.

Response References:

Alpan, I. (1967). "The Empirical Evaluation of the Coefficient K_0 and K_{0R}," *Soil and Foundation*, Tokyo, Vol. 7, No.1.

Bowles, J.E. (1982). Foundation Analysis and Design, 3rd Edition, McGraw-Hill, New York.

Wroth, C.P. (1975). "In-Situ Measurement of Initial Stresses and Deformation Characteristics," *Proceedings, Special Conference on In Situ Measurement of Soil Properties*, ASCE.

Associated ESPA Revision:

No ESPA revision is required as a result of this response.

RAI 02.05.04-7:

Question:

In accordance with 10 CFR 100.23(d)(4), the staff request that the applicant provide the following information regarding soil sampling and borings:

a) SSAR Subsection 2.5.4.2.2.1 states that the sampling intervals for the subsurface investigation borings vary from guidance provided in RG 1.132. Please quantify and explain these variations.

b) The staff was not able to locate C-2106, C-2204SA, and C-2206, B-2162A, B-2176A, B-2182A, and B-2282A in Figure 2.5.4-1. Please clarify where these borings are located.

Response:

Part (a)

Regulatory Guide (RG) 1.132, Section 4.3.2.2 states, "Also, one or more borings for each major structure should be continuously sampled." Boring B-2177, centrally located in the western portion of the power block area, and boring B-2277, centrally located in the eastern portion of the power block area, were continuously sampled. No other borings in the power block area were continuously sampled for their complete length. However, the results from these other borings with respect to stratum material type and thickness matched those from the continuously sampled borings.

RG 1.132, Section 4.3.2.2 also states, "For coarse-grained soils, samples should be taken at intervals no greater than 5 ft (1.5 m). Beyond a depth of 15 m (50 ft) below foundation level, the depth interval for sampling may be increased to 3 m (10 ft)." For the VCS subsurface investigation, except for B-2177 and B-2277, sampling was continuous to about 15 ft depth, and at 5-ft intervals from 15 ft to 100 ft depth. The sampling interval was increased to 10 ft from 100 to 200 ft depth. From 200 ft to the maximum depth of 600 ft below ground surface, the samples were taken at an interval of 20 ft. However, in selected borings the sampling interval was decreased to 10 ft to ensure all strata were sampled. Sampling intervals were made in accordance with the technical specification for the investigation and are described in detail in the second paragraph of SSAR Section 2.5.4.2.2.1. As noted in that paragraph, the sampling intervals are reasonable for characterizing site subsurface conditions. As such, they meet the intent of RG 1.132.

Part (b)

C-2106 and C-2106S (CPT with seismic readings) were performed within 6 ft of each other. Since the inverted triangle CPT symbol on SSAR Figure 2.5.4-1 is about 20 ft wide, only the C-2106S symbol and number were put on the drawing to preserve visual clarity.

C-2204S was planned as a 100 ft deep seismic CPT. It refused at a depth of 55 ft. C-2204SA, located about 7 ft from C-2204S, was a second attempt to reach 100 ft depth. It refused at a depth of 91 ft. C-2204SB, located approximately 11 ft from C-2204S, was a third attempt to reach 100 ft depth. It refused at a depth of 90 ft. Since the inverted triangle CPT symbol on Figure 2.5.4-1 is about 20 ft wide, only the C-2204S symbol and number were put on the drawing to preserve visual clarity.

C-2206 and C-2106S (CPT with seismic readings) were performed within 16 ft of each other. Since the inverted triangle CPT symbol on Figure 2.5.4-1 is about 20 ft wide, only the C-2206 symbol and number were put on the drawing to preserve visual clarity.

B-2162A is mislabeled as B-2162 on Figure 2.5.4-1. The label on the Figure 2.5.4-1 will be changed to B-2162A.

B-2176A is mislabeled as B-2176 on Figure 2.5.4-1. The label on the Figure 2.5.4-1 will be changed to B-2176A.

B-2182A is shown on Figure 2.5.4-1, just south of the Unit 1 Fuel Building.

B-2282A is mislabeled as B-2282 on Figure 2.5.4-1. The label on the Figure 2.5.4-1 will be changed to B-2282A.

Associated ESPA Revisions:

SSAR Figure 2.5.4-1 will be modified in a future revision of the ESPA to change the labels B-2162, B-2176 and B-2282 to B-2162A, B-2176A and B-2282A, respectively, as indicated.

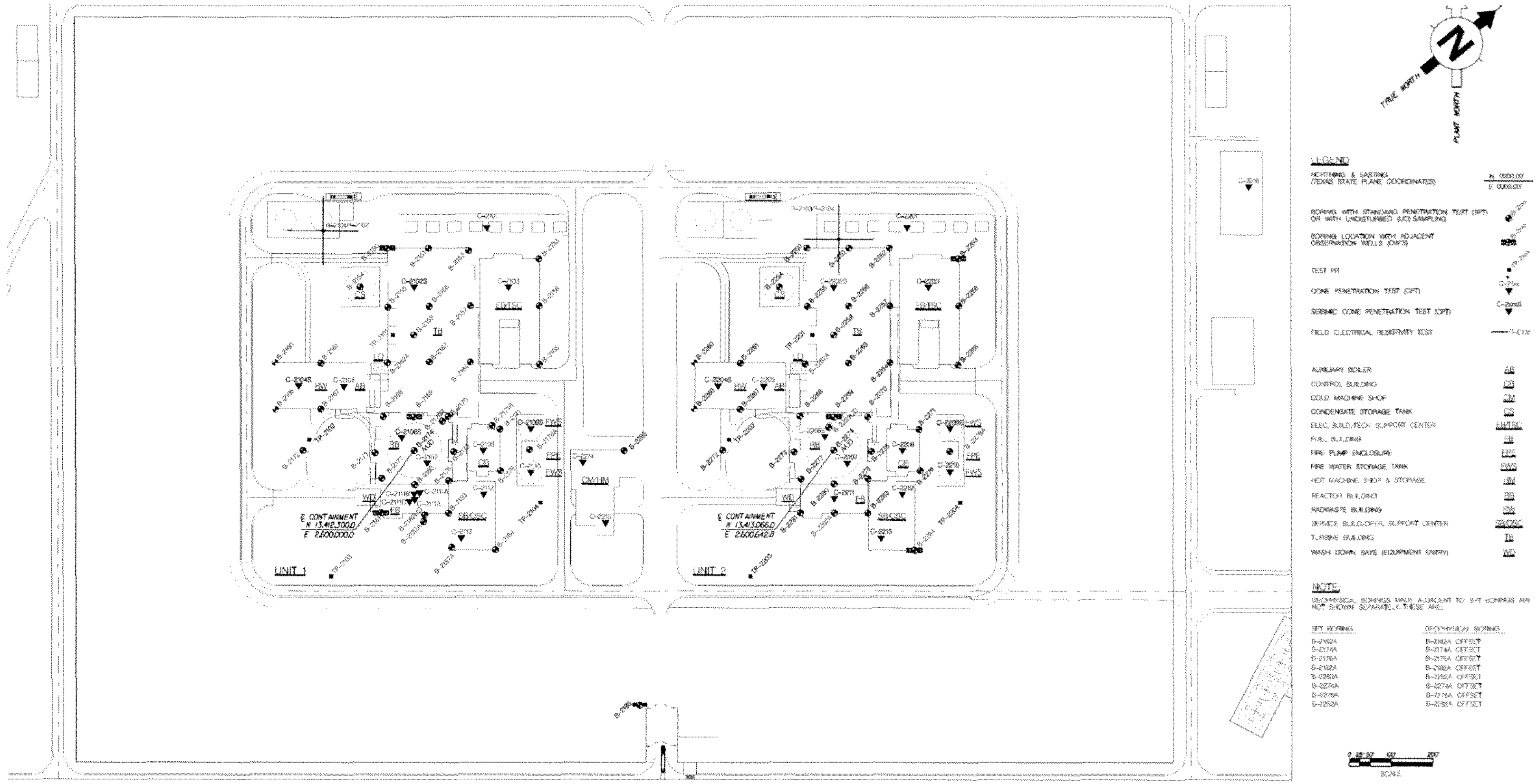

Figure 2.5.4-1 Subsurface Investigation Location Plan (Power Block Area) (Representative Dual Unit ESBWR Layout Shown)

RAI 02.05.04-8:

Question:

In accordance with 10 CFR 100.23(d)(4), the staff request that the applicant provide the following information regarding shear wave velocity:

a) SSAR Subsection 2.5.4.4.5 provides equivalent shear wave (Veq) velocities using the equation from ESBWR DCD Rev 4. Justify use of this equation in this application.

b) SSAR Table 2.5.4-51 presents shear wave velocities. Please explain how the minimum, maximum, and average velocities were determined, and provide an example for Sand 4.

Response:

Part (a)

V_{eq} was computed using the equation from the ESBWR DCD Rev. 4 because it represents a typical LWR and the constraints linked to the shear wave velocity values used in the equation result in a conservative estimate of V_{eq}.

$$V_{eq} = \Sigma d_i / (\Sigma d_i / V_i)$$

where d_i = thickness of soil stratum i
V_i = lower bound V_s of soil stratum, reduced for seismic strain, in stratum i
V_s = shear wave velocity

The equation for V_{eq} above is the harmonic mean of the shear wave velocity in a layered system. This is an appropriate mean to use in this situation since it is the total thickness of the system divided by the total time the shear wave took to travel through the system. The harmonic mean will give lower values than the arithmetic mean, except where all of the strata have the same shear wave velocities. In this case, all the means are the same.

According to ESBWR DCD Rev. 4, the "lower bound V_s value" should be used for each layer. The lower bound value for each stratum is the average V_s value minus one standard deviation. The V_s values measured using P-S suspension logging or seismic CPTs are considered to be measured at low strains (typically taken as 10^{-4} percent or less). Although there is considerable variation, average levels of strain in the soil column during a SHAKE analysis are on the order of 10^{-2} percent. Thus, to account for these seismic strain levels, the measured shear wave velocity values are reduced to levels at about 10^{-2} percent strain, rather than at the measured 10^{-4} percent strain. SSAR Table 2.5.4-54, reproduced here as Table 1, shows the reduction in shear modulus at 10^{-2} percent is 15 percent for Sand 1, 5 percent for the remaining sand layers, and 1 percent for all of the clay layers. Since V_s is a function of the square root of shear modulus, the reduction in V_s at 10^{-2} percent strain will be less than the reduction in shear modulus. As noted in SSAR Section 2.5.4.4.5, an overall reduction in V_s of 10 percent was taken for seismic strain. Based on the reductions noted above, this is a conservative value.

In summary, the V_{eq} approach used by ESBWR DCD, Rev. 4, was selected because it represents a typical LWR and provides a conservative, but reasonable value. The V_s values used were the measured values less 10 percent less one standard deviation. As a result, the V_{eq} tabulated in SSAR Section 2.5.4.4.5 ranges from 757 to 915 ft/sec for the various structures considered. These representative site-specific values fall beneath the minimum value of 1,000 ft/sec required by most, if not all, DCDs. As indicated in SSAR Subsection 2.5.4.4.5, site-specific soil-structure interaction modeling will be performed, if required, in the COL application, when a reactor technology has been selected.

Part (b)

In order to explain how the minimum, maximum, and average shear wave velocities were determined, an example is set for Sand 4 beneath VCS Unit 1 (i.e., representing the western portion of the power block area). This example uses the data points presented in Table 2.5.4-A-51, which includes supplemental data points in addition to the ones provided in Table 2.5.4-51. The methodology used to determine the minimum, maximum, and average shear wave velocities for the values presented in Table 2.5.4-A-51 is identical to that used in Table 2.5.4-51. The statistical analysis of the shear wave velocities determined for the various strata beneath Unit 1 is presented in Table 1.

Note that the strata thickness and depth ranges are determined from the average of all borings and CPTs completed in Unit 1, Unit 2 (i.e., the eastern portion of the power block area) and Units 1 & 2 combined. Based on these values, the top and base elevations are determined for each stratum. The elevations are rounded up/down to the nearest 0.5 ft as presented in Table 1.

The shear wave velocity analysis is based on the site measurements of shear wave velocity from P-S suspension logging (6 sets at each unit), and seismic CPTs (4 sets at each unit), as shown in Tables 2 and 3.

Borings B-2301A and B-2307A are excluded from the list as these two were located outside of the power block.

For the example of Sand 4 stratum beneath Unit 1, the measured shear wave velocity and the corresponding P-S suspension logging borehole or seismic CPT are provided in Table 4. Note that all shear wave velocity data points for Sand 4 stratum are considered in the statistical analysis. Thus, some of the shear wave velocity data points in Table 4 correspond to elevations outside the range given in Table 1. Eighty-seven shear wave velocity measurements are analyzed for the values of maximum, minimum, median, average, and standard deviation. The computed values are summarized at the bottom of Table 4, as well as in Table 1 next to the Sand 4 row. The resulting values of average shear wave velocity and the corresponding thickness are further used to define the thickness and lower bound V_s of soil strata in the equation of V_{eq}.

Table 1: Shear wave velocity beneath Unit 1

Table 2.5.4-A-51 (Sheet 1 of 3)
S-Wave Velocity Profile Numerical Values; Upper Approximately 600 Feet of Site Soils (Power Block Area)

Stratum	Top El. (feet)[a]	Base El. (feet)[a]	Max. V_s (ft/sec)	Min. V_s (ft/sec)	Median V_s (ft/sec)	Avg. V_s (ft/sec)	Std. Dev. (ft/sec)	No. of Tests
Power Block	**(Unit 1)**							
Fill I	95.0	90.0	—	—	—	597	176	—
Fill II	90.0	85.0	—	—	—	708	209	—
Fill III	85.0	80.0	—	—	—	783	232	—
Clay 1 (Top)	80.0	51.5	1100	276	715	722	161	94
Sand 1	—	—	—	—	—	—	—	—
Clay 1 (Btm)	51.5	30.5	1560	470	830	863	228	93
Sand 2	30.5	18.5	1283	570	940	979	165	30
Clay 3	18.5	-4.5	1760	710	1030	1054	188	120
Sand 4	-4.5	-27.0	5380	687	1650	1853	863	87
Clay 5 (Top)	-27.0	-49.0	1650	700	1040	1045	194	62
Sand 5	-49.0	-66.0	1540	870	1135	1137	153	46
Clay 5 (Btm)	-66.0	-77.5	1650	850	1245	1229	250	58
Sand 6	-77.5	-128.5	5600	920	1420	1566	548	199
Clay 7	-128.5	-167.5	2060	800	1480	1434	331	63
Sand 8	-167.5	-202.0	5130	980	1530	1656	588	99
Clay 9	-202.0	-245.0	1660	990	1285	1281	433	72
Sand 10	-245.0	-268.5	2220	1160	1650	1667	278	23
Clay 11	-268.5	-328.0	1750	820	1160	1173	192	59
Sand 12	-328.0	-346.0	2030	1580	1820	1821	127	11
Clay 13	-346.0	-421.5	2310	1020	1325	1410	263	46
Sand 14	-421.5	-454.5	2040	1540	1795	1780	136	20
Clay 15	-454.5	-465.5	1800	1100	1600	1523	245	7
Sand 16	-465.5	-480.5	1980	1720	1740	1814	109	9
Clay 17	-480.5	-507.0	2030	1180	1605	1623	318	16
Sand 18	-507.0	-520.0	2380	1830	1970	2022	190	9
Power Block	**(Unit 2)**							
Fill I	95.0	90.0	—	—	—	597	176	—
Fill II	90.0	85.0	—	—	—	708	209	—
Fill III	85.0	80.0	—	—	—	783	232	—
Clay 1 (Top)	80.0	51.0	1160	167	630	646	170	98
Sand 1	51.0	43.5	1350	738	1079	1129	154	32
Clay 1 (Btm)	43.5	26.5	1300	550	950	944	196	79
Sand 2	26.5	16.5	1470	750	1115	1129	163	40
Clay 3	16.5	-8.5	1670	490	980	999	276	134
Sand 4	-8.5	-36.5	1850	900	1300	1347	221	77
Clay 5 (Top)	-36.5	-52.5	2870	790	1035	1151	342	66
Sand 5	-52.5	-67.5	2490	1140	1540	1530	229	43

Note:
(a) Elevations are referenced to NAVD 88

Table 2: P-S Suspension Logging

P-S suspension logging in boreholes	Note
B-2162A	Unit 1
B-2174A	Unit 1
B-2176A	Unit 1
B-2182A	Unit 1
B-3170A	Unit 1
B-3185A	Unit 1
B-2262A	Unit 2
B-2274A	Unit 2
B-2276A	Unit 2
B-2282A	Unit 2
B-3270A	Unit 2
B-3285A	Unit 2

Table 3: Seismic CPTs

Seismic CPT	Note
C-2102s	Unit 1
C-2104s	Unit 1
C-2106s	Unit 1
C-2109s	Unit 1
C-2202s	Unit 2
C-2204s	Unit 2
C-2206s	Unit 2
C-2209s	Unit 2

Table 4: Statistical analysis of shear wave velocity for the example of Sand 4 stratum

Location	Depth (feet)	Elevation (NAVD 88) (feet)	Vs (feet/sec)
C-2102s	81.50	-1.3	1165
C-2102s	86.50	-6.3	881
C-2102s	89.80	-9.6	687
C-2104s	not encountered		---
C-2106s	76.20	3.3	1325
C-2109s	81.50	-1.6	1284
C-2109s	86.50	-6.6	1532
B-2162A	72.18	7.87	1080
B-2162A	73.82	6.23	1060
B-2162A	75.46	4.59	1170
B-2162A	77.1	2.95	1550
B-2162A	78.74	1.31	1790
B-2162A	80.38	-0.33	1980
B-2162A	82.02	-1.97	1560
B-2162A	83.66	-3.61	2540
B-2162A	85.3	-5.25	2100
B-2162A	86.94	-6.89	2000
B-2162A	88.58	-8.53	2210
B-2162A	90.22	-10.17	2430
B-2162A	91.86	-11.81	2580
B-2162A	93.5	-13.45	3510
B-2162A	95.14	-15.09	5050
B-2162A	96.78	-16.73	5130
B-2162A	98.43	-18.38	5380
B-2162A	100.07	-20.02	3750
B-2162A	101.71	-21.66	2490
B-2162A	103.35	-23.3	1700
B-2162A	104.99	-24.94	1440
B-2162A	106.63	-26.58	1100
B-2162A	108.27	-28.22	850
B-2162A	109.91	-29.86	790
B-2174A	85.63	-6.35	1320
B-2174A	86.94	-7.66	1020
B-2174A	88.58	-9.3	1220
B-2174A	90.22	-10.94	1370
B-2174A	91.86	-12.58	1640
B-2174A	93.5	-14.22	1540
B-2174A	95.14	-15.86	1650

Location	Depth (feet)	Elevation (NAVD 88) (feet)	Vs (feet/sec)
B-2174A	96.78	-17.5	2120
B-2174A	98.43	-19.15	2140
B-2174A	100.07	-20.79	2080
B-2174A	101.71	-22.43	2250
B-2176A	91.86	-11.87	1500
B-2176A	93.5	-13.51	1540
B-2176A	95.14	-15.15	1590
B-2176A	96.78	-16.79	2070
B-2176A	98.43	-18.44	2400
B-2176A	100.07	-20.08	2490
B-2176A	101.71	-21.72	1980
B-2176A	103.35	-23.36	2060
B-2176A	104.99	-25	2160
B-2176A	106.63	-26.64	1960
B-2176A	108.27	-28.28	1250
B-2182A	82.02	-2.32	1490
B-2182A	83.01	-3.31	1320
B-2182A	85.3	-5.6	970
B-2182A	86.94	-7.24	1190
B-2182A	88.58	-8.88	1590
B-2182A	90.22	-10.52	1810
B-2182A	91.86	-12.16	1800
B-2182A	93.5	-13.8	1600
B-2182A	95.14	-15.44	1480
B-2182A	96.78	-17.08	1110
B-2182A	98.43	-18.73	860
B-2182A	100.07	-20.37	1100
B-2182A	101.71	-22.01	1930
B-2182A	103.35	-23.65	1950
B-2182A	104.99	-25.29	1580
B-2182A	106.63	-26.93	940
B-3170A	93.5	-13.38	1520
B-3170A	95.14	-15.02	1430
B-3170A	96.78	-16.66	1230
B-3170A	98.43	-18.31	1170
B-3170A	100.07	-19.95	1610
B-3170A	101.71	-21.59	1750
B-3185A	82.02	-2.44	1460
B-3185A	83.66	-4.08	1720
B-3185A	85.3	-5.72	1760
B-3185A	87.27	-7.69	1750
B-3185A	88.58	-9	2380

Location	Depth (feet)	Elevation (NAVD 88) (feet)	Vs (feet/sec)
B-3185A	90.22	-10.64	2150
B-3185A	91.86	-12.28	2530
B-3185A	93.5	-13.92	2130
B-3185A	95.14	-15.56	2280
B-3185A	97.11	-17.53	3210
B-3185A	98.43	-18.85	2820
B-3185A	100.07	-20.49	2310
B-3185A	101.71	-22.13	1850
	UNIT 1 SAND 4	**max**	5380
		min	687
		median	1650
		average	1853
		std dev	863
		count	87

Associated ESPA Revisions:

No ESPA revision is required as a result of this response.

RAI 02.05.04-9:

Question:

SSAR Subsection 2.5.4.5.1.1.1 states that the upper soils of Clay 1 through Clay 3 will be excavated. Figure 2.5.4-131 shows Clay 3 under structural fill. In accordance with 10 CFR 100.23(d)(4), explain this discrepancy.

Response:

The first sentence of SSAR Section 2.5.4.5.1.1.1 indicates a typical dual-unit LWR with an independent UHS is the bounding configuration, when considering excavation and backfill quantities. The "typical" design from which these excavation and backfill quantities are derived is the ABWR plant. SSAR Figure 2.5.4-131 shows the subsurface profile for the reactor/fuel building for a typical LWR with an integral UHS. The "typical" design from a bearing capacity settlement standpoint is the ESBWR plant.

The ABWR was selected as the bounding configuration from an excavation and backfill standpoint because the reactor building foundation basemat is 85 ft below grade, which is deeper than all of the other designs being considered except mPower. Because mPower is a small modular reactor (and has a smaller footprint than the other designs being considered), there is less excavation involved for an mPower unit than an ABWR. Figure 1 is the ABWR equivalent to SSAR Figure 2.5.4-131. Finish grade is El. 95 ft, and the bottom of the reactor foundation is El. 10 ft. In Unit 1 (i.e., the western portion of the power block area), Clay 3 starts at El. 20 ft and stops at El. -8 ft. All elevations are referenced to NAVD 88. As shown in Figure 1, the Clay 3 is removed and replaced with structural fill below the foundation base. In Unit 2 (i.e., the eastern portion of the power block), Clay 3 starts at El. 18 ft and stops at El. -15 ft. Similarly, the Clay 3 is removed and replaced with structural fill below the foundation base. Thus, the statement in SSAR Section 2.5.4.5.1.1.1 about removing Clay 3 from beneath the representative LWR (with an independent UHS), that is the ABWR, is correct for the purposes of determining bounding excavation and backfill quantities.

The ESBWR was selected to illustrate bearing capacity and settlement because, although it has similar plan dimensions and total applied bearing pressure to the ABWR, the base of the reactor/fuel building combined foundation mat is almost 20 ft shallower than the ABWR reactor building. Thus, the ESBWR experiences less buoyancy effects and is founded on soils that are nearer the surface, which generally implies they are less stiff or dense. Bearing capacity calculations show that if Sand 2 and Clay 3 below the base of the reactor/fuel building foundation mat, at about El. 29 ft, were excavated down to El. 8 ft (i.e., 21 ft of over-excavation) and replaced with structural fill, then the relatively thin layer of Clay 3 remaining below the structural fill would provide sufficient bearing capacity and would not contribute excess settlement.

In summary, SSAR Subsection 2.5.4.5.1.1.1 is correct in that the upper soils of Clay 1 through Clay 3 will be excavated based on an ABWR design. Figure 2.5.4-131 is correct in that for the ESBWR design all of Clay 3 is not excavated. This information will be updated, as required, at the COL stage when a specific reactor technology is selected.

Associated ESPA Revision:

No ESPA revision is required as a result of this response.

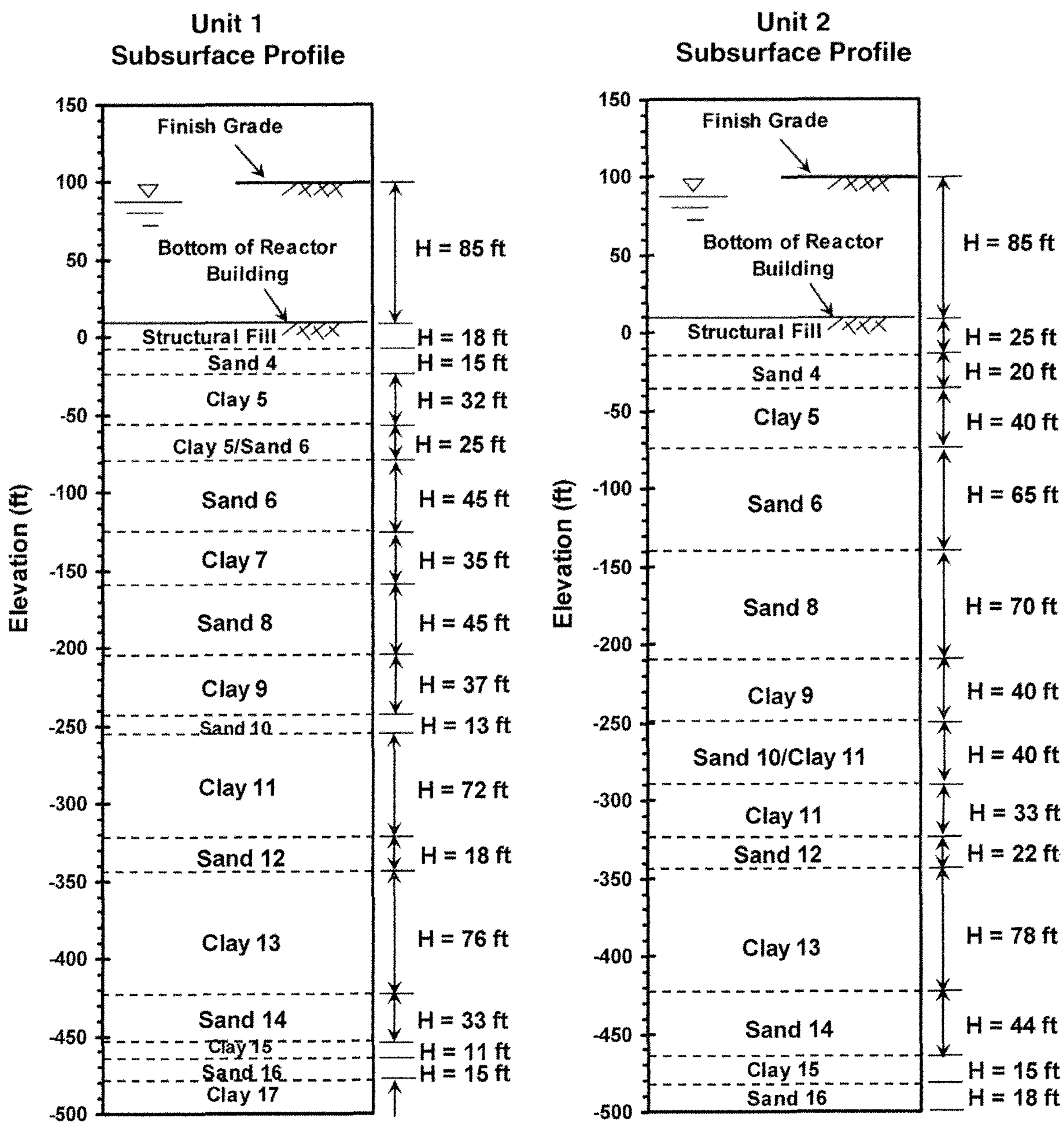

**Figure 1: Subsurface Profile (Typical LWR with an Independent UHS)
(Elevations in NAVD 88)**

RAI 02.05.04-12:

Question:

In accordance with 10 CFR 100.23(d)(4), the staff request that the applicant provide the following information regarding bearing capacity:

a) SSAR Table 2.0-1 lists a Dynamic Bearing Capacity of 56.4 ksf and minimum Static Bearing Capacity of 15.0 ksf. Provide the basis for these selected values and provide sample calculations, including all assumptions and a profile of the layers used in the calculations. This table also lists the ESBWR DCD as a reference for the dynamic bearing capacity. Please justify your reference to only this reactor design and not a bounded PPE value.

b) SSAR Subsection 2.5.4.10.2 states that the allowable and ultimate bearing capacities are higher than the required static design loads and dynamic design loads. SSAR Table 2.0-1 presents a minimum bearing capacity value of 15ksf. However, in Table 2.5.4-88 the allowable and ultimate bearing capacities for the FWSC structure are less than the design load values. Please explain how the values presented in Table 2.5.4-88 satisfy the minimum criteria presented in Table 2.0-1.

Response:

Part (a)

The values of allowable static and dynamic bearing capacities given in SSAR Table 2.0-1 are independent of the site subsurface conditions. They were selected after reviewing the values given in Table 2.0-1 of the DCDs of the various plant designs being considered in the ESP application. The DCD bearing capacity values reflect the design bearing demand (static and dynamic) that the vendors consider must be satisfied by the foundation bearing stratum.

The minimum allowable static bearing capacity of 15 ksf was selected since this was the maximum demand value given for any of the designs being considered. The ABWR and US-APWR both give 15 ksf, the ESBWR gives 14.6 ksf, and the AP1000 gives 8.9 ksf.

As indicated in the question, the 56.4 ksf dynamic bearing capacity is from Table 2.0-1 for the ESBWR. The ABWR DCD does not provide a dynamic bearing capacity value. Revision 17 of the AP1000 DCD gives a value of 35 ksf. Although 60 ksf provided in the US-APWR DCD is slightly higher than 56.4 ksf, the ESBWR DCD appears to have a much firmer basis for its dynamic bearing values. Table 2.01-1 for the ESBWR DCD gives separate values for soft, medium and hard bearing strata for three different structures, suggesting that the values are based on analyses that take into account the relative stiffnesses of the structural foundation and the bearing stratum. The 56.4 ksf value is the maximum of the nine values given. The US-APWR DCD provides a single value, and it is anticipated that this value could change as the US-APWR DCD analysis progresses.

At the COL stage, when the actual plant is selected, SSAR Table 2.0-1 will be modified, as necessary, to reflect the values of bearing capacity/demand of the reactor design selected. The bearing capacity and settlement analyses included in the COLA FSAR will confirm that the site soils can accommodate the design values with an adequate factor of safety against bearing failure, and with settlements within tolerable limits.

Part (b)

SSAR Table 2.0-1 states that the minimum static bearing capacity is greater than or equal to 15 ksf for the reactor building. No limiting values are given for structures other than the reactor building.

Associated ESPA Revision:

No ESPA revision is required as a result of this response.

RAI 02.05.04-16:

Question:

The staff noted that many tables in the SSAR were updated by removing any reference to an specific reactor design and referring instead to a "Typical LWR with and independent UHS" design. However, the numbers in these tables were not changed. Please explain if the values presented still represent the PPE bounding values.

Response:

In response to a request to remove the reference to the ESBWR as being an "example," during the ESPA acceptance review, the SSAR was updated by replacing the reference to the ESBWR reactor design with "Typical LWR (with an integral UHS)" and by replacing reference to the ABWR reactor design with "Typical LWR (with an independent UHS)." As the values in the SSAR form the basis of the PPE, they were not changed during the update. These values are bounding for the designs being considered in the ESP application (ESBWR, ABWR, APWR, AP1000, and mPower) and represent the PPE bounding values. At the COL stage, when a specific reactor design is selected, any values for the selected design that are not bounded by the PPE will be evaluated and addressed in the FSAR, as stated in SSAR Section 2.0.

Associated ESPA Revision:

No ESPA revision is required as a result of this response.

RAI 02.05.05-4:

Question:

In SSAR Subsection 2.5.5.2.1.1, the applicant provided conclusions based on case histories from different references. In accordance with 10 CFR 100.23(d)(4), please clarify if these conclusions apply specifically to the VCS site or are conclusions made in general based on information presented in the references.

Response:

SSAR Subsection 2.5.5.2.1.1 provides a review of the static performance of earthen dams, based on multiple case histories. The cited references discuss the various causes of earthen dam failures and unsatisfactory performance of earthen dams. While in general these earthen dams are similar to the cooling basin embankment dams at the VCS site, the conclusions provided from the cited references are not specific to the VCS site.

Associated ESPA Revisions:

SSAR Subsection 2.5.5.2.1.1 will be revised in a future revision to the ESPA, as indicated:

2.5.5.2.1.1 **Static Performance**

Reference 2.5.5-3 compiles a list of 35 earth dam failures occurring between 1879 and 1938. Causes of dam failures are listed as: foundation and/or embankment low shear strength, inboard drawdown, overtopping, and piping.

Reference 2.5.5-6 compiles a list of the unsatisfactory performance of 206 earth dams between the years 1901 and 1951. The author differentiates the reasons for poor performance into several categories including, among others: overtopping, foundation or embankment piping, outboard slope sliding, and inboard slope drawdown.

Reference 2.5.5-9 reviews the data compiled in Reference 2.5.5-6 and concludes that:

- Inboard slope slides caused by drawdown have not often threatened to cause complete failure of the dam because they usually happen when the reservoir has dropped below a dangerous level.
- Embankment or foundation slides during construction never threaten a catastrophic failure unless water is retained while the dam is built. Slope and crest erosion by waves, wind, and rain do not lead to danger of complete failure except in special circumstances.

<u>Note that the conclusions from the cited references are not specific to the VCS site but rather provide a general assessment of past performances of older earthen slopes not constructed to the standards that will be employed at VCS.</u>

RAI 02.05.05-7:

Question:

SSAR Subsection 2.5.5.2.5.3 indicates that over-excavation of the foundation clay (Stratum Clay 1 (Top)) and the foundation sand (Stratum Sand 1) is required along the north cooling basin dam. In addition Subsection 2.5.4.5.1.1.2 states that Clay 1 and Sand 1 will be excavated. In accordance with 10 CFR 100.23(d)(4):

1) Please clarify which layers will be excavated in the cooling basin.

Response:

As described in SSAR Subsections 2.5.5 and 2.5.4.2.1.1, the natural ground surface in the cooling basin area (as shown in SSAR Figure 2.5.4-2) at the time of the subsurface investigation was gently sloping downward from northwest to southeast, ranging from approximately elevation 80 feet to 42 feet NAVD 88, with an average elevation of 70 feet. An embankment having crest at elevation 102 feet NAVD 88 surrounds the cooling basin. The proposed base level of the cooling basin is 69 feet NAVD 88. SSAR Figure 2.5.5-1 showing the subsurface stratigraphy at elevation 69 feet NAVD 88 indicates the presence of primarily Strata Clay 1-Top and Sand 1, and to limited extent of Stratum Clay 1- Bottom. Consequently, SSAR Subsection 2.5.4.5.1.1.2 refers to the volume of soil that will be moved during earthwork to establish grade within the cooling basin and states that, during site grading, primarily the upper soils of Strata Clay 1-Top and Sand 1 will be excavated.

Over-excavation beneath the embankments will be conducted to a limited extent, as described in SSAR Subsection 2.5.5.2.5.3. The over-excavation of the foundation clay, Clay 1-Top, and the foundation sand, Sand 1, is required along the north cooling basin embankment, adjacent to the power block, in order to achieve the minimum slope stability factor-of-safety of 1.3 under the rapid drawdown case (refer to the embankment Profile A in SSAR Figure 2.5.4-81).

Associated ESPA Revisions:

SSAR Subsection 2.5.4.5.1.1.2 will be updated in a future revision to the ESPA, as indicated:

2.5.4.5.1.1.2 Cooling Basin

At the cooling basin, current estimates are that approximately 27 million cubic yards of material are moved during earthwork to establish site grades, comprised of 20 million cubic yards of clay to construct the embankment dam and dikes, 1 million cubic yards of sand (from offsite sources) for a sand drainage blanket at the outside toe of the embankment dam and 7 million cubic yards of topsoil that will be moved to a spoils area or throughout the site to reestablish vegetation in disturbed areas.

Cooling basin area materials excavated during site grading are primarily the upper soils of Strata Clay 1-Top and Sand 1, consisting of clays (Stratum Clay 1-Top) and clayey or silty fine sands (Stratum Sand 1). To evaluate the uppermost soil strata (Strata Clay 1-Top and Sand 1) for construction purposes, 12 test pits were excavated at the cooling basin, as shown on Figure 2.5.4-2 and summarized in Table 2.5.4-43. The maximum depth of test pits was 10 feet below ground surface. The results of laboratory testing of bulk samples collected from the test pits for moisture-density (modified Proctor compaction) and other index tests are summarized in Table 2.5.4-45, with details included in Reference 2.5.4-2. These tests show that Stratum Clay 1-Top soils are low plasticity, with an average fines content of 70 percent, and occur at natural moisture contents typically 2 percent to 6 percent above their optimum moisture contents. These tests also show that Stratum Sand 1 soils are clayey or silty, with an average fines content of 34 percent, and occur at natural moisture contents typically 2 percent below to 2 percent above their optimum moisture contents. Both the sand soils and the clay soils in their natural states are unsuitable for use as drainage materials, but are suitable for reuse as fill for embankment dams and for interior dikes. For proper reuse as embankment fill, clay materials and sand materials are separated during excavation, and are moisture conditioned, normally to between 2 percent and 6 percent above their optimum moisture contents, prior to placement in fill areas.

RAI 02.05.05-13:

Question:

In accordance with 10 CFR 100.23(d)(4), please explain how the potential for uplift due to blocked exits caused by a meandering stream was considered in the seepage and slope stability analysis.

Response:

The seepage analysis was performed based on the assumption that on the outboard side of the embankment, the water table is at the ground surface and there is a drainage blanket beneath the outboard toe. The drainage blanket was represented with zero pressure head. This implies that the permeability is high enough in the drainage blanket that there is no head loss in the drain relative to the amount of seepage that will come through the low permeability embankment and foundation soils.

As explained in SSAR Subsection 2.4.1.2.4, there are intermittent or ephemeral streams traversing the VCS site, in the area of the cooling basin. Kuy Creek passes by the southwest corner of the site and discharges to the Guadalupe River. Dry Kuy Creek passes by the northwest corner of the site, flows southeast and discharges to Kuy Creek south of the site. However, based on the assumed permeability conditions explained above, the potential for uplift due to blocked exits caused by a meandering stream was not considered in the seepage and slope stability analyses. During the detailed design at the COL stage, the thickness and the permeability of the drainage blanket will be specified to ensure that the exit will not be blocked.

Associated ESPA Revisions:

SSAR Subsection 2.5.5.2.3 will be updated in a future revision of the ESPA, as indicated. Note that changes to SSAR Subsection 2.5.5.2.3 were previously made in response to RAI 02.05.5-5.

2.5.5.2.3 Analytical Slope Stability and Seepage Models

Slope stability studies are based on models that account for the stratification of the subsurface materials, take into account the pore water pressure distribution (effective stress analyses) or the variation of undrained shear strengths (total stress analyses). Pore water pressure distribution within the embankment dams for the steady-state condition is based on seepage flow nets.

Note that at this preliminary stage of design, subsurface and groundwater conditions along the alignment of the embankment dams are defined by investigations (e.g., borings, CPTs) on plan spacings ~~of~~ on the order of 1500 feet center to center. Subsurface and groundwater conditions at locations beyond the outboard toe of the embankment dams (particularly beyond the easternmost dam) would be defined by supplemental investigations. The preliminary engineering analyses reported on here conservatively assume that the groundwater level, to distances considerably beyond the outboard toe of the embankment dams, lies at the ground surface, an assumption which is unlikely to occur. Under these conservative conditions, the analyses show that zones of high hydraulic gradient develop at distances away from the toe of the embankment. If warranted, s~~S~~upplemental investigations, conducted at the COL stage, would provide the means to analyze this potential occurrence in more detail.

The seepage analysis was performed based on the assumption that on the outboard side of the embankment, the water table is at the ground surface and there is a drainage blanket beneath the outboard toe. The drainage blanket was represented with zero pressure head. This implies that the permeability is high enough in the drainage blanket that there is no head loss in the drain relative to the amount of seepage that will come through the low permeability embankment and foundation soils. During the detailed design at the COL stage, the thickness and the permeability of the drainage blanket will be specified to ensure that the exit will not be blocked.

RAI 02.05.05-14:

Question:

SSAR Subsection 2.5.5.2.5 indicates that the Bishop method was used for the slope stability analysis. In accordance with 10 CFR 100.23 (d)(4), justify the selection of Bishop Method for slope stabilization and explain if a block analysis was conducted to estimate the FOS against sliding, particularly where the shear strength of the embankment fill is greater that the foundation of soils.

Response:

There are various methods of computing slope stability, commonly in use, that model the slope cross-section as a series of vertical slices. In general, the differences between the available methods are mainly in the type and degree of simplifying assumptions made in the analysis model, with respect to the shape of the failure surface and to forces within and between these vertical slices. In most cases, the more accurately modeled slice forces give higher computed factors of safety against stability failure. Thus, obtaining a lower factor of safety does not indicate that a more conservative approach was used, only that a less accurate approach was used. The slope geometry and soil parameters are the same in all the methods.

In summary, the Bishop Method considers normal inter-slice forces, but ignores inter-slice shear forces, and satisfies overall moment equilibrium, but not overall horizontal force equilibrium (GEO-SLOPE, 2008). In the Bishop Method, the approximation used in the inter-slice forces assumes (Fang, 1975):

$$\Sigma(T_1\text{-}T_2) \tan \varphi' = 0$$

Where T_1 and T_2 are the vertical inter-slice forces on each side of the slice, and φ' is the soil effective internal friction angle.

Fang (1975) and Bishop (1955) estimated that this approximation results in an error of about 1 percent, whereas the error in neglecting the horizontal and vertical inter-slice forces (Ordinary Method) is approximately 15 percent. Duncan & Wright (2005) and Bishop (1955) showed that the procedure gives improved results over the Ordinary Method, especially when analyses are being performed using effective stresses and the pore water pressures are relatively high. Duncan & Wright (2005) state that Wright et al. (1973) have shown that the factor of safety calculated by the Bishop Method agrees favorably (within about 5 percent) with the factor of safety calculated using stresses computed independently using finite element procedures. The Bishop Method is accepted and commonly used because of this recognized high degree of accuracy.

Although the Bishop Method is restricted to circular slip surfaces, a block analysis was not conducted to estimate the factor-of-safety against sliding. As stated in GEO-SLOPE (2008), "the convergence difficulties with the Block Method can result in a large number of trial slip surfaces with an undefined safety factor. This is particularly problematic when the grid blocks get close to each other. The Block Method works the best and is the most suitable for case[s] where there is a significant distance between the two blocks." GEO-SLOPE (2008) acknowledges that "slip surfaces seldom, if ever, have sharp corners in reality, which is one of the assumptions made in the Block Method. This reality points to another weakness of this [Block] method with respect to forming trial slip surfaces." Thus, the use of the Bishop method is reasonable for this application.

Response References:

Bishop, A.W. "The Use of Slip Circles in the Stability Analysis of Earth Slopes," *Geotechnique*, 4(4), 148-152, 1955.

Duncan, J.M., and Wright, S.G. "Soil Strength and Slope Stability," John Wiley & Sons, Inc., NJ, 2005.

Fang, H-Y. "Stability of Earth Slopes," Foundation Engineering Handbook, Winterkorn, H.F. and Fang, H-Y, Editors, Van Nostrand Reinhold, New York, 1975.

GEO-SLOPE International Ltd. "Stability Modeling with SLOPE/W 2007 Version, An Engineering Methodology," Third Edition, March 2008.

Wright, S.G., Kulhawy, F.H., and Duncan, J.M. "Accuracy of Equilibrium Slope Stability Analyses," ASCE, *Journal of the Soil Mechanics and Foundation Division*, 99(10), pp. 783-791, 1973.

Associated ESPA Revision:

No ESPA revision is required as a result of this response.

ATTACHMENT 13

SUMMARY OF REGULATORY COMMITMENTS

(Exelon Letter to USNRC, NP-11-0029, dated July 8, 2011)

The following table identifies commitments made in this document. (Any other actions discussed in the submittal represent intended or planned actions. They are described to the NRC for the NRC's information and are not regulatory commitments.)

COMMITMENT	COMMITTED DATE	COMMITMENT TYPE	
		ONE-TIME ACTION (Yes/No)	Programmatic (Yes/No)
Exelon will revise VCS ESPA SSAR Subsection 2.5.4.2.1.3.11 to incorporate the changes shown in the enclosed response to the following NRC RAI: 02.05.04-2 (Attachment 1)	Revision 1 of the ESPA SSAR and ER planned for no later than March 31, 2012	Yes	No
Exelon will revise VCS ESPA SSAR Figure 2.5.4-1 to incorporate the changes shown in the enclosed response to the following NRC RAI: 02.05.04-7 (Attachment 4)	Revision 1 of the ESPA SSAR and ER planned for no later than March 31, 2012	Yes	No
Exelon will revise VCS ESPA SSAR Subsection 2.5.5.2.1.1 to incorporate the changes shown in the enclosed response to the following NRC RAI: 02.05.05-4 (Attachment 9)	Revision 1 of the ESPA SSAR and ER planned for no later than March 31, 2012	Yes	No
Exelon will revise VCS ESPA SSAR Subsection 2.5.4.5.1.1.2 to incorporate the changes shown in the enclosed response to the following NRC RAI: 02.05.05-7 (Attachment 10)	Revision 1 of the ESPA SSAR and ER planned for no later than March 31, 2012	Yes	No
Exelon will revise VCS ESPA SSAR Subsection 2.5.5.2.3 to incorporate the changes shown in the enclosed response to the following NRC RAI: 02.05.05-13 (Attachment 11)	Revision 1 of the ESPA SSAR and ER planned for no later than March 31, 2012	Yes	No