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Dominion®

July 14, 2008

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

Serial No. NA3-08-059R
Docket No. 52-017
COL/RSH

DOMINION VIRGINIA POWER
NORTH ANNA UNIT 3 COMBINED LICENSE APPLICATION
RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION LETTER NO. 010

On June 17, 2008, the NRC requested additional information to support the review of certain portions of the North Anna Unit 3 Combined License Application (COLA). The responses to the following RAIs are provided as Enclosures 1 through 12:

- RAI Question 02.05.04-01 Effective Cohesion Value for Zone IIA Soil
- RAI Question 02.05.04-02 Shear Wave Velocity Values in FSAR versus SSAR
- RAI Question 02.05.04-03 Material and Engineering Properties of Backfill
- RAI Question 02.05.04-04 Backfill Properties to Meet or Exceed DCD Values
- RAI Question 02.05.04-05 Shear Wave Velocity Difference
- RAI Question 02.05.04-06 Allowable Dynamic Bearing Capacity Difference
- RAI Question 02.05.04-07 FWSC Foundation Stability Analyses
- RAI Question 02.05.04-08 Coefficient of Friction for Foundation Sliding
- RAI Question 02.05.04-09 Seismic Lateral Earth Pressure
- RAI Question 02.05.05-01 Different Types of Soil in the Same Boring
- RAI Question 02.05.05-02 Bishop's Method for Analysis
- RAI Question 02.05.05-03 Seismic Stability Analysis

This information will be incorporated into a future submission of the North Anna Unit 3 COLA, as described in the Enclosures.

Please contact Regina Borsh at (804) 273-2247 (regina.borsh@dom.com) if you have questions.

Very truly yours,

Eugene S. Grecheck

D079
NRO

Enclosures:

1. Response to RAI Letter Number 010, RAI Question 02.05.04-1
2. Response to RAI Letter Number 010, RAI Question 02.05.04-2
3. Response to RAI Letter Number 010, RAI Question 02.05.04-3
4. Response to RAI Letter Number 010, RAI Question 02.05.04-4
5. Response to RAI Letter Number 010, RAI Question 02.05.04-5
6. Response to RAI Letter Number 010, RAI Question 02.05.04-6
7. Response to RAI Letter Number 010, RAI Question 02.05.04-7
8. Response to RAI Letter Number 010, RAI Question 02.05.04-8
9. Response to RAI Letter Number 010, RAI Question 02.05.04-9
10. Response to RAI Letter Number 010, RAI Question 02.05.05-1
11. Response to RAI Letter Number 010, RAI Question 02.05.05-2
12. Response to RAI Letter Number 010, RAI Question 02.05.05-3

Commitments made by this letter:

1. Incorporate proposed changes for RAI Questions 02.05.04-3 and 02.05.04-6 in a future COLA submission.
2. Provide a response to RAI Question 02.05.04-8 within 30 days after GEH submits response to DCD RAI 3.8-96 S03.

COMMONWEALTH OF VIRGINIA

COUNTY OF HENRICO

The foregoing document was acknowledged before me, in and for the County and Commonwealth aforesaid, today by Eugene S. Grecheck, who is Vice President-Nuclear Development of Virginia Electric and Power Company (Dominion Virginia Power). He has affirmed before me that he is duly authorized to execute and file the foregoing document on behalf of the Company, and that the statements in the document are true to the best of his knowledge and belief.

Acknowledged before me this 14th day of July, 2008

My registration number is 7173057 and my

Commission expires: August 31, 2012

Wanda K. Marshall

Notary Public

WANDA K. MARSHALL
Notary Public
Commonwealth of Virginia
7173057
My Commission Expires Aug 31, 2012

cc: U. S. Nuclear Regulatory Commission, Region II
T. A. Kevern, NRC
J. T. Reece, NRC
J. J. Debiec, ODEC
G. A. Zinke, NuStart/Entergy
T. L. Williamson, Entergy
R. Kingston, GEH
K. Ainger, Exelon
P. Smith, DTE

ENCLOSURE 1

Response to NRC RAI Letter 010

RAI Question 02.05.04-1

NRC RAI 02.05.04-1

FSAR section 2.5.4.2.5, "Engineering Properties," subsection b, "Soil Properties," states that for Zone IIA saprolite soil, the SPT tests average N-values indicated a corresponding internal friction angle, phi', about 35 degrees; the C-U tests gave internal friction angles of 31 to 36 degrees with a median of 33 degrees and with very little effective cohesion, c'. The adopted parameters for Zone IIA soil were phi' = 33 degrees and c' = 6.0 kPa (0.125 ksf) [FSAR page 2-224]. Please justify why an effective cohesion value of 6.0 kPa was used for Zone IIA soil.

Dominion Response

The effective cohesion value of 6.0 kPa used for Zone IIA soil in the FSAR is derived from various data sources.

The Zone IIA saprolite is a residual soil, i.e., it is the product of in-place weathering and has not been transported. As a result, although grain size curves indicate it to be mostly a silty sand, the material texture shows angular interlocking grains with lack of void network, and a strongly anisotropic fabric. From tests performed for Units 1 and 2, the mineralogy, which reflects the parent rock, consists of significant amounts of clay minerals and mica. The fabric, texture and mineralogy of the saprolite explain the "soft" feel the sand has, compared with an alluvial sand. In short, given its mineralogy, texture and fabric, it would be anticipated that the Zone IIA saprolite would exhibit some effective cohesion.

Four consolidated-undrained triaxial (C-U) tests were run on samples of Zone IIA silty sand for the Units 1 and 2 investigation, as reported in the North Anna Units 1 and 2 Updated Finial Safety Analysis Report. The average c' value was about 13 kPa (0.275 ksf), with an average phi' value of about 29 degrees. Six C-U tests were run on Zone IIA saprolite for the COL subsurface investigation. The average c' value was about 4 kPa (0.08 ksf) and the average phi' value was just over 33 degrees. Based on the results of the 10 C-U tests, and the mineralogy, texture and fabric of the material, an effective cohesion of 6 kPa was selected for the Zone IIA saprolite.

Proposed COLA Revision

None.

ENCLOSURE 2

Response to NRC RAI Letter 010

RAI Question 02.05.04-2

NRC RAI 02.05.04-2

FSAR 2.5.4.4.4, "Results of Shear and Compression Wave Velocity Tests" states that "the overall shear wave velocities of the rock as defined by the three rock zones (III, III-IV and IV) are somewhat higher at the Unit 3 plant location than described in the SSAR." The comparison of the median shear wave velocities revealed that the shear wave velocity values presented in FSAR are about 36 to 50 percent higher than that listed in the SSAR. Please explain why such different values were obtained for the same site.

Dominion Response

There are two primary reasons why the median shear wave velocity (V_s) values in the FSAR and SSAR are different.

(1) The SSAR V_s values were obtained from a variety of locations, both within and outside the ESP plant parameter envelope, with locations more than 305 m (1,000 ft) apart. All of the FSAR V_s values were obtained from three borings within 61 m (200 ft) of each other, beneath the proposed locations of the North Anna Unit 3 Reactor, Control and Fuel Buildings. The Unit 3 reactor location is 229 m (750 ft) to 533 m (1,750 ft) away from most of the SSAR sources, and is generally at a higher elevation. Top of rock in the SSAR was taken at El. 76.2 m (250 ft), whereas it was measured at an average of El. 83.2 m (273 ft) at Unit 3. In summary, the FSAR V_s measurements in rock were taken from relatively closely spaced borings and reflect conditions at the specific locations of Unit 3 seismic Category 1 structures. The SSAR V_s measurements were taken at more widely spaced locations throughout the site, and do not reflect the conditions at the specific Unit 3 location.

(2) The FSAR V_s measurements were taken with P-S Suspension logging equipment, which provided V_s values at 0.5-m (1.6-ft) depth intervals to a maximum depth of about 87.5 m (287 ft). The V_s values measured in the deeper portions of the three borings (i.e., below the more variable weathered and fractured zones) were extremely consistent, indicating repeatable high quality data. For the SSAR, limited V_s data were obtained during the ESP investigation. Results were obtained in rock from cross-hole testing to 13.7 m (45 ft) and from down-hole testing to 19.8 m (65 ft) depth. The remaining SSAR data came from shear wave velocity testing for Units 1 and 2 down to a maximum of 39 m (128 ft) depth. The Unit 1 and 2 data was obtained about 40 years ago with less sophisticated equipment than is used today.

In summary, the FSAR data reflect V_s at specific site locations of Category 1 structures and were taken with state-of-the-practice equipment. The SSAR readings were spread over a wider area of the site, and some of the data reflects less sophisticated measurements.

Proposed COLA Revision

None.

ENCLOSURE 3

Response to NRC RAI Letter 010

RAI Question 02.05.04-3

NRC RAI 02.05.04-3

FSAR section 2.5.4.5.3, "Backfill Sources, Compaction and Quality Control" (also in section 2.5.4.10.1 "Bearing Capacity"), states that concrete fill would be used to replace any moderately to severely weathered rock exposed at the bottom of excavation for Seismic Category I building foundation mats. Please provide material and engineering properties of the concrete fill.

Dominion Response

The properties of the concrete fill have not been determined. The concrete mix will be designed to result in a shear wave velocity in the same range as that of the Zone III-IV rock. Note that concrete typically has a higher shear wave velocity than bedrock of the same strength. Thus, although the concrete placed on the Zone III-IV rock may have a lower strength than the rock, it can still have a similar shear wave velocity.

Proposed COLA Revision

FSAR Section 2.5.4.5.3 will be revised as indicated in the attached markup.

Markup of North Anna COLA

The attached markup represents Dominion's good faith effort to show how the COLA will be revised in a future COLA submittal in response to the subject RAI. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be somewhat different than as presented herein.

NAPS ESP PC 7

- The excavation for safety-related structures will be geologically mapped and photographed by experienced geologists. Unforeseen geologic features that are encountered will be evaluated. The NRC will be notified no later than 30 days before any excavations for safety-related structures are open to allow for NRC staff examination and evaluation.
- There is no measurable rebound or heave of the sound rock subgrade, and monitoring is not needed.

2.5.4.5.3 Backfill Sources, Compaction and Quality Control

Although a large amount of saprolitic soil will be excavated for Unit 3, this material will not be used as structural fill to support or back fill Seismic Category I or II structures, or other major structures.

NAPS ESP COL 2.5-3 [RAI 02.05.04-3 (Draft 07/07/08]

Structural fill is either lean concrete or a sound, well-graded granular material. The anticipated extent of the concrete and granular fill is shown on the foundation cross-sections on Figure 2.5-229 through Figure 2.5-234. The concrete fill is used to replace any moderately to severely weathered rock (Zone III) exposed at the bottom of the excavations for the Seismic Category I RB/FB and Control Building foundation mats. The concrete fill will be designed to result in a shear wave velocity in the same range as that of the Zone III-IV rock.

The granular structural fill material does not exist naturally on site. However, given the large amount of rock that will need to be excavated for Unit 3, it will be economical to set up a crushing and blending plant onsite to produce crushed aggregate to the required gradation specifications for use as structural fill. The rock will be crushed down to well-graded, angular or sub-angular gravel-sized particles, with less than 5 percent passing the number 200 sieve. The soundness of the aggregate will be confirmed using sulfate soundness and Los Angeles abrasion tests. This structural fill will be placed in thin lifts and compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557 (SSAR Reference 165), and to within 3 percent of its optimum moisture content. Compaction will be performed with a heavy steel-drummed vibratory roller, except within 1.5 m (5 ft) of a structure wall, where smaller compaction equipment will be used in conjunction with reduced lift thickness to minimize excess pressures against the wall. As noted in Section 2.5.4.2.5.b, based on the type of material and its

ENCLOSURE 4

Response to NRC RAI Letter 010

RAI Question 02.05.04-4

NRC RAI 02.05.04-4

FSAR Table 2.4.1-2, "ITAAC For Backfill Under Category I Structures," (1) does not specify the inspections, tests, or analyses that will be used to ensure that the properties of the selected backfill meet the ESBWR design control document (DCD) Tier I requirements, (2) commits to meeting minimum density values, and (3) does not provide specific acceptance criteria. 10 CFR 100.23 (d) (4) requires that "Each applicant shall evaluate all siting factors and potential causes of failure, such as the physical properties of the materials underlying the site ..." and Regulatory Guide 1.206 section C.I.2.5.4.5, "Excavations and Backfill" states that the applicant should discuss "sources and quantities of backfill and borrow, including a description of exploration and laboratory studies and the static and dynamic engineering properties of these materials." Since the only engineering property of the backfill soil was the assumed internal friction angle f' = 40 degrees (FSAR section 2.5.4.5.3, "Backfill Sources, Compaction and Quality Control"), please describe how you will ensure that static and dynamic properties of the backfill soil will meet or exceed: (1) the requirements of the ESBWR DCD, e.g., minimum shear wave velocity of 300 m/s (1000 ft/s) as listed in Tier I document; and (2) parameter values used for the site seismic response and liquefaction potential analyses, bearing capacity, settlement and earth pressure estimates.

Dominion Response

Response to Question (1)

The Fire Water Service Complex (FWSC) is the only seismic Category I structure that will be founded on backfill. The remaining seismic Category I structures will be on bedrock.

Table 5.1-1 of ESBWR DCD Tier 1 indicates that the minimum soil shear wave velocity for seismic Category 1 structures is 300 m/s (1000 ft/s). Table 2.0-1 of ESBWR DCD Tier 2 contains the same statement about minimum shear wave velocity, and Note 8 of Table 2.0-1 describes this minimum shear wave velocity (V_{eq}) as follows:

> This is the equivalent uniform shear wave velocity (V_{eq}) over the entire soil column at seismic strain, which is a lower bound value after taking into account uncertainties. V_{eq} is calculated to achieve the same wave traveling time over the depth equal to the embedment depth plus 2 times the largest foundation plan dimension below the foundation as follows:
>
> $$V_{eq} = \Sigma d_i/\Sigma(d_i/Vi)$$
>
> where di and Vi are the depth and shear wave velocity, respectively, of the ith layer. The ratio of the largest to the smallest shear wave velocity over the mat foundation width at the foundation level does not exceed 1.7.

The depth of embedment of the FWSC is about 2.4 m (8 ft), and the largest foundation plan dimension is 52 m (171 ft) (ignoring diagonal dimensions). Thus, V_{eq} is computed using the weighted averaging formula above, down to a depth of 2 x 52 = 104 m (342 ft) below the base of the FWSC foundation. Of this 104 m (342 ft), on the lower bound soil and rock profile, 13.7 m (45 ft) are compacted fill, underlain by 3 m (10 ft) of Zone III weathered rock, then 24.1 m (79 ft) of fractured Zone III-IV rock, then 63.4 m (208 ft) of sound Zone IV rock.

The estimated lower bound shear wave velocity for about 9.1 m (30 ft) below the FWSC foundation is below 305 m/s (1,000 ft/s). The underlying 4.6 m (15 ft) of fill has estimated lower bound velocities between 305 m/s (1,000 ft/s) and 335 m/s (1,100 ft/s). The 27.1 m (89 ft) of weathered and fractured rock has shear wave velocity values between 914 m/s (3,000 ft/s) and 1,372 m/s (4,500 ft/s). The 63.4 m (208 ft) of Zone 4 rock has a lower bound shear wave velocity value of over 2,440 m/s (8,000 ft/s). Using the above values in the formula for V_{eq} over the 107 m (350 ft) soil column gives a V_{eq} value of over 914 m/s (3,000 ft/sec).

For V_{eq} to drop below 305 m/s (1,000 ft/s), the shear wave velocity in the fill would have to be less than 61 m/s (200 ft/s), which is the level of shear wave velocity expected for a very loose sand. The backfill below the FWSC will be a highly compacted ($\geq$95% modified Proctor compaction), well graded, angular or sub-angular crushed rock. (Note that the lowest shear wave velocity recorded during the COLA site investigation was 131 m/s (430 ft/s) in a loose to very loose surficial deposit (N = 5 blows/ft at 0.9 m (3 ft) depth) of Zone IIA saprolite.)

In summary, the FWSC is the only seismic Category I structure that will be founded on backfill. The minimum shear wave velocity of 305 m/s (1,000 ft/s) for the FWSC, as defined in the ESBWR DCD, will be achieved because the backfill makes up only about 15% of the total thickness of the soil column considered in computing the minimum shear wave velocity, with the other 85% being rock with shear wave velocities much greater than 305 m/s (1,000 ft/s).

Response to Question (2)

(a) Site seismic response: As discussed in the response to Question (1), the minimum shear wave velocity below the FWSC will exceed the 305 m/s (1,000 ft/s) requirement using the definition of minimum shear wave velocity contained in Note 8 of Table 2.0-1 of ESBWR DCD Tier 2.

(b) through (e): The structural backfill below the FWSC will be a highly compacted ($\geq$95% modified Proctor compaction), well graded, angular or sub-angular gravel-sized crushed rock, compacted in thin lifts with a heavy vibratory steel-drummed roller. This material will be rock taken from the powerblock excavation, and crushed and blended to a gradation that will achieve maximum density when compacted. As noted in FSAR Section 2.5.4.5.3, fill placement and compaction control procedures will be addressed in an installation specification that includes requirements for suitable fill, sufficient testing to address potential material variations, and in-place density testing frequency, i.e., a minimum of one test per 930 m^2 (10,000 ft^2) per lift of fill placed. It also includes requirements for an on-site testing laboratory for quality control (gradation, moisture-density, placement, compaction, etc.) and requirements that the fill operations conform to the earthwork specification. In summary, the high quality of the compacted structural backfill will be verified by selecting premium structural fill material and then ensuring the required properties are achieved through application of an installation specification that specifies the necessary onsite testing and quality control.

As noted in FSAR Section 2.5.4.2.5b, based on the type of material and its degree of compaction, N = 50 blows/ft and $\varphi' = 40°$ is reasonable and conservative for the structural fill. When the high quality of the compacted fill below the FWSC has been verified, as described above, then the performance of the fill based on these properties with regard to liquefaction, bearing capacity, settlement and earth pressure is as follows:

- Liquefaction – Liquefaction does not occur in dense granular soils (N > 30 blows/ft).

- Bearing Capacity –The bearing capacity (c) of the compacted structural fill below the FWSC is estimated to be over 11,500 kPa (240 ksf) while the demand (d) is 165 kPa (3.45 ksf) and 671 kPa (14 ksf) for static and dynamic loading, respectively (FSAR Table 2.5-213). This gives static c/d > 65 and dynamic c/d >15.

- Settlement – The maximum settlement of the FWSC is estimated to be less than 25 mm (1 in.) (FSAR Table 2.5-216). Settlement of the FWSC will be measured by a settlement monitoring program.

- Earth Pressure – There will be earth pressures against the buried structures due to the compacted structural fill. Estimated at-rest earth pressure against the reactor building due to the compacted structural fill is shown in FSAR Figure 2.5-254.

Proposed COLA Revision

None.

ENCLOSURE 5

Response to NRC RAI Letter 010

RAI Question 02.05.04-5

NRC RAI 02.05.04-5

FSAR Figure 2.5-244, Estimated Shear Wave Velocity versus Depth for Structural Fill. Please provide clarification on the difference between the values of shear wave velocity of the backfill plotted in FSAR Figure 2.5-244 (estimated shear wave velocity ranged from 152 to 724 m/s (500 to 1400 f/s)) and listed in FSAR Table 2.0-201 "Evaluation of Site/Design Parameters and Characteristics" (the minimum shear wave velocity of 1073 m/s (3520 ft/s) underneath the FWSC building).

Dominion Response

FSAR Figure 2.5-244 shows the estimated shear wave velocity versus depth for structural backfill; whereas, the value of minimum shear wave velocity beneath the FWSC given in FSAR Table 2.0-201 is a weighted average of not only the backfill shear wave velocities, but also the velocities of a considerable thickness of bedrock extending below the backfill.

Note 8 of Table 2.0-1 of ESBWR DCD, Tier 2, describes this minimum shear wave velocity (V_{eq}) as follows:

> This is the equivalent uniform shear wave velocity (V_{eq}) over the entire soil column at seismic strain, which is a lower bound value after taking into account uncertainties. V_{eq} is calculated to achieve the same wave traveling time over the depth equal to the embedment depth plus 2 times the largest foundation plan dimension below the foundation as follows:
>
> $$V_{eq} = \Sigma d_i / \Sigma (d_i / V_i)$$
>
> where d_i and V_i are the depth and shear wave velocity, respectively, of the ith layer. The ratio of the largest to the smallest shear wave velocity over the mat foundation width at the foundation level does not exceed 1.7.

The depth of embedment of the FWSC is about 2.4 m (8 ft), and the largest foundation plan dimension is 52 m (171 ft) (ignoring diagonal dimensions). Thus, V_{eq} is computed using the weighted averaging formula above, down to a depth of 2 x 52 = 104 m (342 ft) below the base of the FWSC foundation. Of this 104 m (342 ft), on the lower bound soil and rock profile, 13.7 m (45 ft) are compacted fill, underlain by 3 m (10 ft) of Zone III weathered rock, then 24.1 m (79 ft) of fractured Zone III-IV rock, then 63.4 m (208 ft) of sound Zone IV rock.

Although the estimated lower bound shear wave velocity for about 9.1 m (30 ft) below the FWSC foundation is below 305 m/s (1,000 ft/s), and the underlying 4.6 m (15 ft) of fill has estimated lower bound velocities between 305 m/s (1,000 ft/s) and 335 m/s (1,100 ft/s), the 27.1 m (89 ft)of weathered and fractured rock has shear wave velocity values between 914 m/s (3,000 ft/s) and 1,372 m/s (4,500 ft/s), and the 63.4 m (208 ft) of Zone 4 rock has a lower bound shear wave velocity value of over 2,440 m/s (8,000 ft/s). Using the above values in the formula for V_{eq} over the 107 m (350 ft) soil column gives the V_{eq} value of 1,073 m/s (3,520 ft/s) in FSAR Table 2.0-201.

Proposed COLA Revision

None.

ENCLOSURE 6

Response to NRC RAI Letter 010

RAI Question 02.05.04-6

NRC RAI 02.05.04-6

FSAR Table 2.5-215, Summary of Allowable Bearing Capacities for the Major Structures, and Table 2.0-201, Evaluation of Site/Design Parameters and Characteristics. Please provide clarification on the difference between the values of allowable dynamic bearing capacity for the Reactor/Fuel building listed in FSAR Table 2.5-215 (214 ksf) and FSAR Table 2.0-201 (12,401 kPa (259 ksf)).

Dominion Response

The dynamic bearing capacity value of 214 ksf in FSAR Table 2.5-215 is the computed value for concrete. The dynamic bearing capacity value of 12,401 kPa (259 ksf) in FSAR Table 2.0-201 is the computed value for Zone III-IV bedrock. Since the value for concrete is lower, the value in FSAR Table 2.0-201 will be changed to the value for concrete, i.e., 10,250 kPa (214,000 lbf/ft^2).

The allowable dynamic bearing capacity beneath the Reactor/Fuel building was conservatively taken as the least value of allowable bearing capacity of any of the strata underlying the building foundation, regardless of the thickness of the stratum. Weathered rock will be removed from under the Reactor/Fuel building foundation. Thus, the only strata that will immediately underlie the foundation are the Zone III-IV and Zone IV bedrock strata. However, if Zone III rock is encountered beneath the foundation it will be replaced with concrete. Therefore, although the concrete will form only a thin layer in comparison with the zone of influence of the foundation, it is included as a stratum in the estimate of dynamic bearing capacity.

Proposed COLA Revision

FSAR Table 2.0-201 will be revised as indicated in the attached markup.

Markup of North Anna COLA

The attached markup represents Dominion's good faith effort to show how the COLA will be revised in a future COLA submittal in response to the subject RAI. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be somewhat different than as presented herein.

Table 2.0-201 Evaluation of Site/Design Parameters and Characteristics

RAI 02.05.04-6 (Draft 07/03/08)

Subject (16)	DCD Site Parameter Value (1)(16)	Site Characteristic	Evaluation
Soil Properties (continued)			
Minimum Dynamic Bearing Capacity (continued)			
Reactor/Fuel Building			
Soft	2700 kPa (56,400 lbf/ft^2)	**ESP** No values provided	
Medium Hard	7300 kPa (152,500 lbf/ft^2) 5400 kPa (112,800 lbf/ft^2)	**Unit 3** ~~12,401~~ 10,250 kPa (~~259,000~~ 214,000 lbf/ft^2)	The Unit 3 site characteristic value for minimum dynamic bearing capacity for the RB/FB structure is from Table 2.5-215 and falls within (is greater than) the DCD site parameter minimum value for any type of soil: hard, medium, or soft. Based on the equivalent uniform shear wave velocity identified below, the materials beneath the RB/FB structure are classified as hard in accordance with Note (7).
Control Building			
Soft	2800 kPa (58,500 lbf/ft^2)	**ESP** No values provided	
Medium Hard	2500 kPa (52,300 lbf/ft^2) 2400 kPa (50,200 lbf/ft^2)	**Unit 3** 6895 kPa (144,000 lbf/ft^2)	The Unit 3 site characteristic value for minimum dynamic bearing capacity for the CB structure is from Table 2.5-215 and falls within (is greater than) the DCD site parameter minimum value for any type of soil: hard, medium, or soft. Based on the equivalent uniform shear wave velocity identified below, the materials beneath the CB structure are classified as hard in accordance with Note (7).

ENCLOSURE 7

Response to NRC RAI Letter 010

RAI Question 02.05.04-7

NRC RAI 02.05.04-7

FSAR section 2.5.4.10, FSAR Figure 2.5-232, and Table 2.0-201. Since the Seismic Category I structure Fire Water Service Complex (FWSC) building is planned to be built on about 45 feet of backfill soil (FSAR Figure 2.5-232), please provide clarifications and justifications for related foundation stability analyses:

(a) The backfill soil was designed to be granular material and may be saturated as the maximum ground water level was determined to be 2.1 m (7 ft), and the maximum flood level was determined as 0.85 m (2.8ft) below design plant grade (FSAR Table 2.0-201), therefore the backfill soil may be liquefiable under seismic loadings. Regulatory Guide 1.206 section C.I.2.5.4.8, "Liquefaction Potential" states that "If the foundation materials at the site adjacent to and under safety-related structures are saturated soils or soils that have a potential to become saturated and the water table is above bedrock, the applicant should provide an appropriate state-of-the-art analysis of the potential for liquefaction occurring at the site. The applicant should indicate the extent to which the guidance provided in RG 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites," was followed." Please provide the analysis or justify why liquefaction potential analysis was not performed for the backfill soil.

(b) In FSAR section 2.5.4.10.1, "Bearing Capacity," subsection b, "Soil" it stated that "bearing capacity is based on Terzaghi's bearing capacity equations modified by Vesic." Although this method is commonly used in foundation bearing capacity analysis under the normal shear failure assumption, for nuclear power plant structures, especially for the nuclear island, bearing capacity should be estimated "particularly due to overturning forces," as stated in ESBWR DCD Tier 2, section 3.8.5.4 "Design and Analysis Procedures." Please clarify if the overturning forces were considered in the foundation allowable bearing capacity analysis.

(c) In FSAR section 2.5.4.10.2, "Settlement Analysis," the settlement was estimated using a formula where the layer elastic modulus E was involved. Please clarify and justify what type of E values - corresponding to small or large strains, were used in the settlement calculation.

(d) In FSAR section 2.5.4.10.2, "Settlement Analysis," the differential settlement for the FWSC was estimated excluding the weight of the base mat. Please justify why the weight of the base mat was not included in the settlement calculation.

(e) Please explain why the seismic settlement of the FWSC foundation was not considered.

Dominion Response

Responses to the five items identified by the subject RAI are as follows:

(a) Analyses have shown that the backfill soil for North Anna Unit 3 is non-liquefiable. In particular, certain materials are not subject to liquefaction, even though they are saturated and are subject to significant peak ground acceleration. Liquefaction results from the buildup of pore-water pressure in the soil due to the seismic motion, and subsequent loss of effective shear strength of the soil. In dense granular soils, pore-water pressure buildup can lead to increased cyclic shear strains, but dilation of the soil inhibits major strength loss. In gravel-size soils, pore-water pressure cannot build up sufficiently to cause loss of effective shear strength due to the high permeability of the soil. The fill beneath the FWSC at North Anna will be both dense and consist of gravel-size particles, and thus is non-liquefiable. (As noted in FSAR Section 2.5.4.5.3, "...given the large amount of rock that will need to be excavated for Unit 3, it will be economical to set up a crushing and blending plant onsite to produce crushed aggregate to the required gradation specifications for use as structural fill. The rock will be crushed down to well-graded, angular or sub-angular gravel-size particles, with less than 5 percent passing the Number 200 sieve. The soundness of the aggregate will be confirmed using sulfate soundness and Los Angeles abrasion tests. This structural fill will be placed in thin lifts and compacted to at least 95% of the maximum dry density as determined by ASTM D 1557, and to within 3 percent of its optimum moisture content.")

This material is shown to be non-liquefiable using analysis methods based on SPT, CPT and shear wave velocity given in Youd, et al (SSAR Reference 178), as follows. FSAR Table 2.5-212 indicates the structural fill to have an adjusted N-value of 50 blows/ft, an effective friction angle of 40 degrees, an average design shear wave velocity of 1,100 ft/sec, and a total unit weight of 130 pcf.

Analysis based on SPT: Youd, et al, Figure 2 shows "No Liquefaction" for clean sands with an adjusted N-value greater than about 30 blows/ft.

Analysis based on CPT: A cone penetrometer will be unable to penetrate any significant depth into a highly compacted, well graded crushed rock. Since liquefaction potential can be equated to cone tip resistance in a similar way to SPT N-value, cone tip refusal indicates a tip value much higher than any limiting value used to indicate "No Liquefaction", (Youd, et al, Figure 4).

Analysis based on shear wave velocity: Youd, et al, Figure 10 shows "No Liquefaction" for overburden stress-corrected shear wave velocity greater than about 210 m/s (690 ft/s). (The overburden stress-correction factor increases the measured shear wave velocity at shallow depths and reduces it at larger depths. The correction factor is 1 at an effective overburden pressure of about 100 kPa (about 2 ksf). Thus, referring to the relationship between estimated shear wave velocity and depth of structural fill shown on FSAR Figure 2.5-244, the depth to a correction factor of 1 assuming a water table at 0.85 m (2.8 ft) depth, would be about 8.2 m (27 ft), i.e., shear wave velocity values above 8.2 m (27 ft) depth would be increased due to the overburden stress-correction factor).

It should also be noted that the NRC previously reviewed and found acceptable that the structural fill is non-liquefiable per ESP Application SER Sections 2.5.4.1.8 and 2.5.4.3.8.

(b) Overturning forces were considered in the foundation allowable bearing capacity analysis. ESBWR DCD, Tier 2, Table 2.0-1, indicates minimum static and dynamic bearing capacity values for the FWSC and for the Reactor/Fuel and Control buildings on soft, medium and hard foundation materials. The dynamic values are significantly higher than the static values, by factors ranging from about 2.7 to 10.4. These much higher dynamic values are due to transient forces (such as seismic and wind loading) acting on the structure and causing temporary high bearing pressures on the foundation. These include the forces due to overturning moments caused by the transient loading.

The calculated allowable bearing capacities of the major structures are given in FSAR Table 2.5-215. In all cases, the minimum dynamic values exceed the ESBWR DCD Tier 2 Table 2.0-1 minimum dynamic values that included overturning moment effects. For the seismic

Category I structures, only the FWSC is on soil and was analyzed using Terzaghi's bearing capacity equation modified by Vesic. As shown in Table 2.5-215, the computed allowable bearing capacity in this soil (including a factor of safety of 3) is 83.4 ksf, compared to the minimum required values in ESBWR DCD Tier 2 Table 2.0-1 of 540 kPa (11.3 ksf) for medium soil. Dynamic forces are typically not evenly distributed across the base of the foundation. In cases where the foundation consists of sets of individual footings or a thin element (such as the steel plate beneath a conventional tank), then local failure of the foundation soil is possible, and the width of foundation assumed in the analysis should reflect this local failure condition. However, local failure will not occur in the 2.5-m (8.2-ft) thick reinforced concrete mat foundation of the FWSC, and so the inclusion of the complete foundation mat width in the calculation of allowable bearing capacity is justified.

In summary, the minimum dynamic bearing capacity values in the ESBWR DCD include the effects of overturning moments. The ultimate bearing capacity of the soil below the FWSC was computed as 3 x 3,995 kPa, i.e., 11,985 kPa (250 ksf), compared to the minimum required ESBWR DCD value of 540 kPa (11.3 ksf). The 11,985 kPa (250 ksf) value was computed by applying Terzaghi's bearing capacity equation modified by Vesic and using the complete foundation width of the mat. This is justified since the high strength of the mat precludes local bearing failure.

(c) Large strain E values were used in the settlement analysis. For the Zone III-IV and Zone IV rock, the E values are not strain dependent and so the large and small strain E values are the same. For the Zone III weathered rock and the structural fill, the E values are strain dependent, and the large strain E values are lower and thus give correspondingly higher estimated settlements.

(d) The weight of the basemat was excluded from the settlement analysis in FSAR Section 2.5.4.10.2 because Note 15 of Table 2.0-1 of the ESBWR DCD, Tier 2, says, "Settlement values are long-term (post-construction) values except for differential settlement within the foundation mat. The design of the foundation mat accommodates immediate and long-term (post-construction) differential settlements after the installation of the basemat." FSAR Table 2.5-216 includes an estimate of FWSC settlement excluding the weight of the basemat in order to compare the computed settlements to those given in Table 2.0-1, since the Table 2.0-1 values are after basemat installation, i.e., they exclude the weight of the basemat. Note that FSAR Table 2.5-216 also provides estimated settlements including the basemat.

(e) As discussed in (a), the structural fill will consist of well-graded, highly compacted, angular to sub-angular gravel-size particles of crushed rock. The high degree of compaction (and relative density) will be achieved using a heavy vibratory steel-drummed roller. The well-graded particles will be densely packed. There will be some small settlement of the fill under the FWSC due to the tank loading, as shown in FSAR Table 2.5-216. This is due to the elastic compression of the compacted structural fill. (Even the bedrock under the Reactor/Fuel and Control buildings has some very small elastic settlement, as shown in Table 2.5-216.) However, given the high relative density of the fill, no significant additional densification and hence no significant settlement is expected during the design seismic event.

Proposed COLA Revision

None.

ENCLOSURE 8

Response to NRC RAI Letter 010

RAI Question 02.05.04-8

NRC RAI 02.05.04-8

FSAR section 2.5.4.10.3 "Earth Pressures," states that "[t]he factor of safety against a gravity wall or structure foundation sliding is normally taken as 1.1 when seismic pressures are included." The staff notes that the ESBWR DCD (Section 3.8.5.4 "Design and Analysis Procedures") states that "selected waterproofing material for the bottom of the basemat is a chemical crystalline powder that is added to the mud mat mixture forming a water proof barrier." Please clarify and justify the coefficient of friction at the interface of the basemat and underlying material used in the calculation of the factor of safety against foundation sliding.

Dominion Response

Dominion's response to this RAI relies upon standard plant design information that is currently being developed by General Electric-Hitachi (GEH). Therefore, Dominion will provide the information requested after GEH completes its work.

The coefficient of friction at the interface of the basemat and its underlying material is dependent on the composition of the mud mat. The composition of the mud mat is design information that is within the scope of the ESBWR DCD, which is the responsibility of GEH.

The NRC has issued DCD RAI 3.8-96 S03 requesting that GEH provide information related to "the use of crystalline powder in the foundation mud mat". In order to fully respond to the COLA RAI, Dominion must use the information that GEH will provide in its response to DCD RAI 3.8-96 S03.

By letter dated July 7, 2008 (MFN 08-566) GEH informed NRC that it was committed to respond to DCD RAI 3.8-96 SO3 by August 18, 2008. Accordingly, Dominion will provide a response to COLA RAI 02.05.04-8 within 30 days after GEH submits the response to DCD RAI 3.8-96-S03.

Proposed COLA Revision

None.

ENCLOSURE 9

Response to NRC RAI Letter 010

RAI Question 02.05.04-9

NRC RAI 02.05.04-9

As indicated in FSAR section 2.5.4.10.3 "Earth Pressures," the Ostadan method was used to estimate the seismic lateral at-rest pressures against the buried structure walls and active earth pressures due to the Zone IIA and IIB saprolites are included when applicable. Please provide detailed information on the analysis when both at-rest and active seismic lateral earth pressure were involved.

Dominion Response

FSAR Figure 2.5-232 shows structural fill between the vertical excavation support wall and the below-ground wall of the Reactor Building. This fill is in an at-rest condition and generates at-rest lateral pressure, since the Reactor Building wall is a non-yielding wall. As indicated in FSAR Section 2.5.4.10.3, the Zone IIA and IIB saprolites on the outside of the vertical excavation support wall would have been in an active condition after excavation within the wall, since this wall is considered to be a yielding wall. This active lateral earth pressure against the outside of the vertical support wall will have some influence on the lateral pressure applied to the structural fill and hence to the structure wall. Therefore the active earth pressure in the saprolites needs to be considered in addition to the at-rest pressure in the structural fill.

Seismic At-Rest Lateral Earth Pressure

For the at-rest earth seismic lateral earth pressure computation in the structural fill, the method developed by Ostadan and White (Reference 2.5-218) was adopted. In this method the following steps are taken:

(a) Perform a free-field soil column analysis and obtain the ground response motion at the depth corresponding to the base of the wall in the free-field.

(b) Compute the total mass, m for a single degree of freedom system using the Poisson's ratio and mass density of the soil.

$$m = 0.5\rho H^2\Psi_v$$
$$\Psi_v = 2/[(1-\nu)(2-\nu)]^{0.5}$$

where ρ is the mass density of the soil, H is height of the wall, and ν is Poisson's ratio.

(c) Obtain the lateral seismic force from the product of the total mass obtained in Step (b) and the acceleration spectral value of the free-field response at the soil column frequency obtained at the depth of the bottom of the wall (Step (a)).

FSAR Figure 2.5-254 shows the seismic component of the at-rest lateral earth pressure using the Ostadan and White method.

Seismic Active Lateral Earth Pressure

For the active earth pressure of the saprolites against the outside vertical excavation support wall, the method first developed by Mononobe-Okabe was adopted. This method is explained and discussed in Reference 2.5-217.

In the Mononobe-Okabe method, a Coulomb wedge is acted upon by horizontal and vertical pseudostatic forces whose magnitudes are related to the mass of the wedge by the pseudostatic accelerations $a_h = k_h g$ and $a_v = k_v g$. The total earth pressure exerted on the wall can therefore be obtained from force equilibrium on the soil wedge. Assuming the back of the wall is vertical and the ground surface is horizontal behind the wall, the active seismic force, P_{AE}, can be calculated as follows using the Mononobe-Okabe method:

$$P_{AE} = \frac{1}{2}\gamma H^2 (1-k_V) K_{AE}$$

Where $K_{AE} = \dfrac{\cos^2(\phi-\theta)}{\cos\theta\cos(\delta+\theta)\left(1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\theta)}{\cos(\delta+\theta)}}\right)^2}$

$\theta = \tan^{-1}(\dfrac{k_h}{1-k_v})$,

ϕ is the friction angle of soil, δ is the angle of wall friction, and H is the height of the wall.

The total active seismic thrust, P_{AE}, can be divided into a static component, P_A, and a dynamic component, ΔP_E, which is termed the dynamic force of soil, as follows:

$$P_{AE} = P_A + \Delta P_E$$

Reference 2.5-217 suggested expressing the coefficient of seismic active earth pressure, K_{AE}, as follows:

$$K_{AE} = K_a + \Delta K_{AE}$$

And thus the dynamic active earth force, ΔP_E, becomes:

$$\Delta P_E = \frac{1}{2}\gamma H^2 \cdot \Delta K_{AE}$$

where γ is unit weight of the soil, H is the height of the retaining wall.

FSAR Figure 2.5-253 shows the seismic component of the active lateral earth pressure using the Mononobe-Okabe method.

Proposed COLA Revision

None.

ENCLOSURE 10

Response to NRC RAI Letter 010

RAI Question 02.05.05-1

NRC RAI 02.05.05-1

FSAR section 2.5.5.1.3, "Slopes Subsurface Conditions," regarding the soil properties of the new slope, states that (page 2-265) "interpretation of CPT C-916 (performed adjacent to B-947) based on friction ratio, indicated mainly silty clays and clays" (underline added); but it also states, on the same page, that "based on the results of B-947 and C-916, the new slope has the properties of Zone IIA silty sand saprolite." Please clarify why two different types of soil were identified for the same boring and same CPT data.

Dominion Response

Cone penetrometer tests (CPTs) provide valuable information on the soil by measuring cone tip resistance, sleeve friction, and pore-water pressure at very closely-spaced depth intervals throughout the soil profile. However, no samples are obtained with the CPT and so there is no direct evidence of the type of soil being measured. Instead, the soil type is selected based mainly on the friction ratio, which is the ratio, expressed as a percentage, of the measured sleeve friction to the tip resistance. The interpretation of soil type from friction ratio is empirical, based on historical correlations between friction ratio and soil type identified from adjacent borings. However, like most empirical correlations in geotechnical engineering, it is not exact for all soil types, and this was the case in C-916. As noted in Section 2.5.5.1.3, the friction ratio correlations indicated the soil to be mainly silty clays and clays, whereas the soil was mainly silty sand, based on visual observation and grain size testing. The soil in the profile at B-947 and C-916 was mainly silty sand, and this was the profile used in the slope stability analysis.

Proposed COLA Revision

None.

ENCLOSURE 11

Response to NRC RAI Letter 010

RAI Question 02.05.05-2

NRC RAI 02.05.05-2

FSAR section 2.5.5.2, "Design Criteria and Analyses" indicates that the Bishop's method was used in all slope stability analyses. The Bishop's method only considers moment equilibrium and may not always be conservative depending on the geometry, the soil properties and the shape of the critical slip surface of the slopes. Regulatory Guide 1.206, section C.I.2.5.5, "Stability of Slopes" states that "the results of slope stability evaluations using classic and contemporary methods of analyses should be presented." Please explain why only analysis results based on Bishop's method are presented.

Dominion Response

There are various methods of computing slope stability commonly in use today that model the slope cross-section as a series of vertical slices. Their differences 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 the Bishop method, the only approximation used in the inter-slice forces is to assume:

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

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.

It is estimated that this assumption results in an error of about 1% whereas the error in neglecting the horizontal and vertical inter-slice forces (Ordinary Method) is closer to 15% (Reference 1). The Bishop method is accepted and commonly used because of this recognized high degree of accuracy.

It should also be noted that the NRC previously reviewed and found acceptable the Bishop method of analysis per ESP Application SER Sections 2.5.5.1.1 and 2.5.5.3.1.

Reference 1

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

Proposed COLA Revision

None.

ENCLOSURE 12

Response to NRC RAI Letter 010

RAI Question 02.05.05-3

NRC RAI 02.05.05-3

In FSAR section 2.5.5.2 "Design Criteria and Analyses," several assumptions were used in the seismic stability analysis. Those assumptions are 1) liquefaction was not considered, 2) average peak horizontal and vertical accelerations were used rather than peak accelerations at the ground surface, and 3) reduced peak accelerations were considered (0.1g at high frequency and 0.15g at low frequency for horizontal acceleration with zero vertical acceleration (FSAR page 2-238) as recommended by Seed in his 1979 study; and 50 percent of the peak acceleration as suggested by Kramer (1996)). Regulatory Guide 1.206, section C.I.2.5.5.2, "Design Criteria and Analyses" states that the applicant "should present valid static and dynamic analyses to demonstrate the reliable performance of these slopes throughout the lifetime of the plant. It should describe the methods used for static and dynamic analyses, and indicate the reasons for selecting them." Accordingly, please provide the following: (1) Describe the impact of the possible maximum dynamic settlement of the slope soil (refer to FSAR section 2.5.4.8.1) on the slope stability; and (2) Describe how the assumptions used by the pseudo-static methods (such as earth materials do not undergo significant strength loss upon cyclic loading (<15 percent) and large displacement is acceptable) were verified.

Dominion Response

Responses to each of the two items identified in the subject RAI are as follows:

(a) The possible maximum dynamic settlement predicted in FSAR Section 2.5.4.8.1 is about 41 mm (1.6 in). For the 10.7-m (35-ft) and 13.1-m (43-ft) high slopes analyzed, this represents a reduction in slope height of about 0.38% and 0.31%, respectively. This level of slope reduction will not impact the slope stability.

(b) The reason for a slope failure during a seismic event is not a slip failure of the slope due to a seismic lateral force acting on the slope (as modeled in the pseudo-static analysis); rather, it is because a significant portion of the slope loses strength due to liquefaction. Examination of the two slopes analyzed in Section 2.5.5 indicates that the top 8.5 m (28 ft) of both the existing slope (ES) and the new slope (A-A) are above the groundwater table. Thus a major portion of each slope will not liquefy, regardless of material properties. With respect to material properties, for in-situ soils (Zone IIA saprolite) at North Anna Unit 3, FSAR Section 2.5.4.8.2 states, "...it can be concluded that a very small percentage of the Zone IIA saprolitic soils have a potential for liquefaction based on the low and high frequency Unit 3 seismic characteristics. The liquefaction analysis did not take into account the beneficial effects of age, structure, fabric and mineralogy, and thus the chances of any liquefaction occurring are very low." Based on the foregoing arguments, a strength loss of even a small portion of either of the slopes due to liquefaction is remote, and thus the slopes will remain stable during the design earthquake.

Proposed COLA Revision

None.