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Dominion®

October 20, 2011

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

Serial No. NA3-11-049R
Docket No. 52-017
COL/MWH

DOMINION VIRGINIA POWER
NORTH ANNA UNIT 3 COMBINED LICENSE APPLICATION
SRP 02.05.04: RESPONSE TO RAI LETTER 82

On August 15, 2011, the NRC requested additional information to support the review of certain portions of the North Anna Unit 3 Combined License Application (COLA), which consisted of one question. The response to the following Request for Additional Information (RAI) Question is provided in Enclosure 1:

- RAI 5941 Question 02.05.04-25 Concrete Fill Shear Wave Velocity

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

Very truly yours,

Eugene S. Grecheck

cc: U. S. Nuclear Regulatory Commission, Region II
C. P. Patel, NRC
T. S. Dozier, NRC
G. J. Kolcum, NRC

D089
NRO

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 20th day of October, 2011

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

Enclosure:

1. Response to NRC RAI Letter 82, RAI 5941 Question 02.05.04-25

Commitments made by this letter:

Revise COL application as described in the letter.

ENCLOSURE 1

Response to NRC RAI Letter 82

RAI 5941 Question 02.05.04-25

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

North Anna Unit 3

Dominion

Docket No. 52-017

RAI NO.: 5941 (RAI Letter 82)

SRP SECTION: 02.05.04 – STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS2)

DATE OF RAI ISSUE: 8/18/2011

QUESTION NO.: 02.05.04-25

FSAR Sections 2.5.4.7.1, 3.7.2.1 and Appendix 3OO.1.1 state that the fill concrete has a minimum design compressive strength of 2,500 psi and a best estimate shear wave velocity of 7,000 ft/s.

Based on ACI-318, concrete with a compressive strength of 2500 psi will result in a shear wave velocity of approximately 6200 ft/s. Therefore, please describe and justify how you will assure that the fill concrete will attain the shear wave velocity used in the FIRS calculations of at least 7,000 ft/s.

Dominion Response

A range of concrete fill shear wave velocity (V_s) values was used in calculating the foundation input response spectra (FIRS). This range of values was determined based on the best estimate (BE) V_s value of 7,000 ft/sec. This range of values was used as input to the soil profile randomization process used in developing the FIRS as further described in FSAR Appendix 3OO.1.1.

The determination of the BE V_s value was previously documented in the response to RAI 02.05.04-12 (Dominion letter NA3-09-013RB dated August 20, 2009). The BE value of 7,000 ft/sec is the rounded average between the V_s of 7,786 ft/sec using the Boone (2005) approach and the V_s of 6,295 ft/sec using the American Concrete Institute (ACI)-318-2008 approach.

The correlation developed by Boone (2005) is based on the V_s of a concrete test pad, measured directly using spectral analysis of surface waves (SASW) and cross-hole seismic techniques. The pad was poured in three foot lifts using a 2,500 psi concrete mix designed to produce a low heat of hydration by using flyash and the minimum amount of cement required to achieve the design strength. This mix and its placement method are typical for concrete fill applications and similar to the concrete fill design expected to be used for North Anna Unit 3. The Boone (2005) correlation was determined to provide reasonable V_s results because it is based on actual data from mass concrete testing of a pad design using concrete properties similar to the concrete fill properties expected at North Anna Unit 3.

The ACI approach is based on work by Pauw (1960), who correlated concrete strength with elastic modulus. The strength can then be related to V_s using relationships between elastic modulus and shear modulus, and shear modulus and V_s. These relationships assume elastic behavior, and thus any non-linear behavior of the concrete would not be accounted for in the ACI approach. Since the Boone approach measures V_s directly, any inaccuracies due to non-linearity are avoided. The V_s of concrete is a function not only of strength but also of the concrete constituents, notably the aggregate used (e.g., a harder aggregate will result in a higher V_s), which may have contributed to the difference in V_s results between the two methods.

Based on the above, the development of the BE V_s value of 7,000 ft/sec using an average of both ACI-318 and Boone results is considered to be reasonable. The use of the BE V_s value as input to the soil profile randomization process, used in developing the FIRS, is considered to produce representative results for seismic response analysis. FSAR Section 2.5 will be revised to be consistent with the FIRS calculations and to appropriately designate and clarify that 7,000 ft/sec is a BE V_s value of the minimum specified 2500 psi concrete fill.

References

ACI-318-2008. Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary. American Concrete Institute, Farmington Hills, MI.

Boone, S. D. (2005). "A Comparison between the Compressive Strength and the Dynamic Properties of Concrete as a Function of Time," MS Thesis, University of Tennessee, Knoxville.

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

Pauw, A. (1960). "Static Modulus of Elasticity of Concrete as Affected by Density," ACI Journal, Proceedings, Vol. 57, No. 6, December.

Proposed COLA Revision

FSAR Section 2.5.4.2.5, 2.5.4.5.3, 2.5.4.7.1 and Table 2.5-212 will be revised as indicated on 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 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.

2.5.4.2.5 **Engineering Properties**

The engineering properties for Zones IIA, IIB, III, III-IV, and IV derived from the Unit 3 field exploration and laboratory testing programs are provided in Table 2.5-212 and described in the following paragraphs. These engineering properties are similar to those obtained from the previous field and laboratory testing programs (as shown in SSAR Table 2.5-45), with some differences. Where there are differences, the impact from an engineering standpoint is usually either the same or more favorable.

The following paragraphs discuss selected properties shown in Table 2.5-212 under the subheadings: a) rock properties, including concrete fill; b) soil properties, including structural backfill; c) RCTS results; and d) chemical properties.

a. **Rock and Concrete Fill Properties**

In general, the rock strength and stiffness values, derived from the field and laboratory testing of the Unit 3 rock, are higher than given in the SSAR. This could reflect less fractured or weathered rock beneath the Unit 3 area, and/or better rock coring equipment and techniques that produced better quality cores.

The Recovery and RQD are based on the results presented for each core in the boring logs in Appendices 2.5.4AA, 2.5.4BB, and 2.5.4CC. The RQDs from the borings for Strata III, III-IV and IV are plotted versus elevation in Figure 2.5-224. For Stratum III, RQD generally ranges from zero to around 50 percent, with some higher values. The average value is about 20 percent. For Stratum III-IV, RQD generally ranges from around 50 to 90 percent. The average value is about 65 percent (compared to 50 percent in the SSAR). For Stratum IV, RQD is generally above 80 percent and mostly above 90 percent. The average value is about 95 percent. The average recovery values for Zone III, III-IV and IV are about 50 percent, 90 percent, and 98 percent, respectively.

The unconfined compressive strengths and unit weights in Table 2.5-212 are based on the rock strength test results shown in Table 2.5-211. The elastic modulus values are also based on the values shown in Table 2.5-211. The shear modulus values are derived from the elastic modulus values using the Poisson's ratio values tabulated in Table 2.5-212. These higher strain shear modulus values agree well with the low strain values derived from the geophysical tests performed for the

Unit 3 exploration program described in Section 2.5.4.4. These high and low strain shear modulus values are essentially the same for high strength rock, certainly for the Zone IV and Zone III-IV rock. Some strain softening has been allowed in the case of the Zone III rock, as described in Section 2.5.4.7. Low strain is defined here as 10^{-4} percent while high strain is taken as 0.25 to 0.5 percent, the amount of strain frequently associated with settlement of structures on soil.

The shear and compression wave velocities in Table 2.5-212 are based on suspension P-S velocity logging performed as part of the Unit 3 exploration program (Appendices 2.5.4AA and 2.5.4CC). These results are summarized in Section 2.5.4.4.4.

Concrete Fill

As stated in Section 2.5.4.10, if Zone II saprolitic soils and/or Zone III weathered rock is encountered at foundation subgrade level of the R/B, Power Source Buildings (PS/Bs), and Power Source Fuel Storage Vaults (PSFSVs), they will be removed and replaced with concrete fill. Concrete fill will also replace Zone II saprolitic soils beneath the remaining seismic category I structures, i.e., Ultimate Heat Sink Related Structures (UHSRS), UHSRS pipe chase, and Essential Service Water Pipe Tunnel (ESWPT). The concrete fill will have a minimum strength of 2500 psi, with a unit weight and Poisson's ratio of 145 pcf and 0.15, respectively. The bearing capacity of concrete fill is addressed in Section 2.5.4.10.1.

Figures 2.5-229, 2.5-230, and 2.5-233 show weathered rock will be removed and replaced by concrete fill from up to about 38 ft depth below the base of the R/B foundation, with an average thickness of about 15 ft. Figure 2.5-233 shows weathered rock will be removed and replaced by concrete fill from up to 33 ft depth below the base of the West PS/B foundation, with an average thickness of about 23 ft. Figures 2.5-231 and 2.5-232 show as much as 60 ft of Zone II and Zone III materials being replaced by concrete fill beneath portions of the East and West PSFSVs.

Analysis indicates that if the top 25 ft of Zone III rock beneath the R/B foundation is replaced with concrete, the seismic response at foundation level decreases with increasing shear wave velocity (V_S) of the concrete. Based on the calculated log-mean V_S values at and below the R/B foundation (shown in Figure 2.5-241a), the V_S of the in-situ rock at 25 ft below the R/B foundation base is approximately 5000 ft/sec. Therefore, the V_S of the concrete fill should be equal to or greater than 5000 ft/sec to ensure that the seismic response of the column that includes the concrete fill is equal to or less than the response from the original analysis of the in-situ rock. Further analysis indicates that concrete with strength of 2500 psi has a best estimate V_s of ~~at least 6295~~ 7,000 ft/sec.

RAI 02.05.04-25

RAI 02.05.04-22

Construction, quality assurance, and engineering properties of the concrete fill, including strength and durability, are controlled through project specifications. The project specifications provide controls on the construction process (including placement techniques), material properties (including mix design and concrete properties during placement; for example slump, air content, and mix temperature) and other variables. The concrete fill is required by project specifications to conform to the pertinent provisions of ACI 349 (Reference 2.5-215) including provisions contained in ASTM standards and ACI Committee 201 and 207 publications referenced within ACI 349.

b. **Soil Properties**

Zone IIA Saprolite

Grain size curves from sieve analyses of Zone IIA silty and clayey sand, and sandy silt samples are shown in Appendices 2.5.4AA and 2.5.4CC. The tests were run mainly on the silty sand samples with more than 90 percent having fines contents of less than 50 percent. Figure 2.5-225a shows fines content versus depth from these tests. The median fines content for the Zone IIA saprolite is about 24 percent, with the majority of samples having a Unified Soil Classification System (USCS) classification (Reference 2.5-209) of SM.

The median natural moisture content from 100 tests performed is 19 percent. For the relatively small percentage of samples that exhibited plasticity, the median liquid limit was 38 percent while the plasticity index was 11 percent.

The measured SPT N-values from 656 tests ranged from 2 to refusal (defined as >100 blows/ft), with a median value of 15 blows/ft. These are plotted versus depth on Figure 2.5-226. The N_{60} median value adjusted for hammer energy is 20 blows/ft. The effective angle of internal friction of a medium dense coarse-grained saprolite (N =20 blows/ft) would typically be taken as around 35 degrees (SSAR Reference 150). However, the relatively high silt content and the presence of low plasticity clay minerals reduce this angle. Consolidated-undrained (C-U) triaxial tests reported in UFSAR Appendices 2C and 3E (SSAR Reference 5) produced internal

NAPS ESP PC 3.E(6)

- 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 **Structural Fill Sources, Compaction and Quality Control**

Although a large amount of Zone IIA soil will be excavated for Unit 3, this material will not be used as structural fill to support seismic category I or II structures. 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. If Zone III weathered rock or fractured rock is encountered at foundation subgrade level of the R/B, PS/Bs or PSFSVs, it will be removed and replaced with concrete fill. Concrete fill will also replace Zone IIA and Zone IIB saprolitic soils beneath the remaining seismic category I structures.In short, all structural fill beneath seismic category I or II structures will be concrete fill. As noted in Section 2.5.4.2.5.a, the concrete fill will have a minimum strength of 2500 psi, a ~~minimum~~ best estimate shear wave velocity of ~~6295~~ 7000 fps, and a unit weight and Poisson's ratio of 145 pcf and 0.15, respectively.

The granular structural fill material that will be used as backfill around seismic category I and II structures 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 backfill. The rock will be crushed down to well-graded, angular or sub-angular sand and gravel-sized particles conforming to the gradation of Size No. 21A specified by the Virginia Department of Transportation (DOT) Road and Bridge Specifications (SSAR Reference 166). This gradation is shown in Figure 2.5-225b. The soundness of the aggregate will be confirmed using sulfate soundness and Los Angeles abrasion tests. This structural backfill will be placed in lifts not exceeding 12 in. loose thickness and compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557 (SSAR Reference 165) to within 3 percent of its optimum moisture

2.5.4.7.1 **Shear Wave Velocity Profile**

NAPS ESP COL 2.5-9

Various measurements were made at the Unit 3 site to obtain estimates of the shear wave velocity in the soil and rock. These are summarized in Section 2.5.4.4. The materials of interest here are the Zone IIA and Zone IIB saprolitic soils, the structural backfill, the Zone III weathered rock, the Zone III-IV slightly to moderately weathered rock, and the Zone IV slightly weathered to fresh rock. The V_s profiles described under a. Bedrock below are the profiles developed specifically for the seismic category I structures supported on rock or on concrete fill on rock. The V_s profiles described under b. Soil below are the profiles developed (1) using the in-situ soil for slope stability analysis (Section 2.5.5) and liquefaction analysis (Section 2.5.4.8) and (2) using the structural backfill profile above the foundation level.

a. **Bedrock**

V_s profiles of the bedrock measured in the five Suspension P-S Logging boreholes are shown on Figure 2.5-237. One or more of the five V_s profiles is used as the input V_s for the analysis to develop input motions for each of the various seismic category I and II structures. Since in most cases the V_s profile was not directly beneath the footprint of the structure, the V_s profile or combination of V_s profiles used was based on the proximity of the V_s measurement to the structure, and/or the similarity of the average subsurface profile (in terms of Zone III, Zone III-IV, etc.) beneath the structure to the subsurface profile in the V_s borehole.

R/B, East PS/B and East PSFSV

For these structures, all three of the V_s profiles in the main power block complex (B-901, B-907 and B-909) were combined, and are shown in Figure 2.5-240. Below about Elevation 135 ft, the shear wave velocity is fairly constant at between approximately 9000 fps and 10,000 fps. The figure shows Zone IV bedrock extending up to around Elevation 184 ft. Above this elevation, two distinct V_s profiles are identified, with one representing the more weathered and fractured rock profile, and the other the mostly unweathered and unfractured profile. These profiles (Profiles 1 and 2) are also shown on Figure 2.5-241a along with the log mean values derived from Profiles 1 and 2 and from the measured V_s values. The boring log mean plot indicates that V_s = 9200 fps is reached at about Elevation 145 ft.

West PS/B and West PSFSV

V_S boring B-909 is relatively close to the West PS/B and West PSFSV and has a fairly similar subsurface profile to the average profile beneath and in the immediate vicinity of these structures. The shear wave velocity profile used for the West PS/B and West PSFSV analyses is thus based on the B-909 V_S profile, and is shown in Figure 2.5-241b. The V_S values are averaged over 10-ft intervals. Since readings are taken every 1.6 ft, there are 6 readings per 10-ft interval. The minimum and maximum readings shown on Figure 2.5-241b are the minimum and maximum readings with the 10-ft interval. Note the 26 ft of concrete fill has a ~~designated~~ best estimate V_S of 7000 fps with minimum and maximum values of 6000 and 8000 fps.

UHSRS

The four UHSRS are labeled UHSRS A through UHSRS D, with UHSRS A at the eastern end and UHSRS D at the western end. The UHSRS Pipe Chase runs between UHSRS B and UHSRS C. A minimum of 3 to 4 ft of concrete fill is placed immediately below the foundation of each UHSRS.

UHSRS A & B

V_S boring M-30 is the closest to UHSRS A and UHSRS B and has a relatively similar subsurface profile to the average profiles beneath UHSRS A and B. The shear wave velocity profile used for the UHSRS A and B analyses is thus based on the M-30 V_S profile, and is shown in Figure 2.5-241c.

UHSRS C

V_S boring B-907 is the closest to UHSRS C and has a relatively similar subsurface profile to the average profile beneath UHSRS C. The shear wave velocity profile used for the UHSRS C analysis is thus based on the B-907 V_S profile, and is shown in Figure 2.5-241d.

UHSRS D

V_S borings B-907 and M-10 are the closest to UHSRS D and have a relatively similar subsurface profile to the average profile beneath UHSRS D. The shear wave velocity profile used for the UHSRS D analysis is thus based on the combined B-907 and M-10 V_S profiles, and is shown in Figure 2.5-241e.

NAPS COL 2.5(1)

Table 2.5-212 Engineering Properties of Subsurface Materials

Stratum	Structural Fill	Concrete Fill	Zone IIA	Zone IIB	Zone III	Zone III-IV	Zone IV
Description	Gravelly materials derived from crushing rock material		Saprolite – core stone less than 10% of volume of overall mass	Saprolite – core stone 10% to 50% of volume of overall mass	Weathered rock – core stone more than 50% of volume of overall mass	Moderately weathered to slightly weathered rock	Parent rock – slightly weathered to fresh rock
Drained properties							
Effective cohesion, c' (ksf)	0	-	0.125	0	-	-	-
Effective friction angle, ' (degrees)	40	-	33	40	-	-	-
Shear wave velocity, V_s (ft/sec)	1,100	~~6,295~~ 7,000*	850	1,600	3,000	4,500	9,000
Compression wave velocity, V_p (ft/sec)	2,400	~~9,810~~ 10,900*	1,800	3,500	7,300	9,000	16,000
Poisson's ratio, u (high strain)	0.3	0.15	0.35	0.3	0.4	0.33	0.27
Poisson's ratio, u (low strain)	0.37	0.15	0.35	0.37	0.4	0.33	0.27
Elastic modulus (high strain), E_h	1,800 ksf	2,850 ksi	720 ksf	3,600 ksf	400 ksi	1,900 ksi	7,250 ksi
Elastic modulus (low strain), E_l	13,000 ksf	2,850 ksi	7,500 ksf	28,000 ksf	800 ksi	1,900 ksi	7,250 ksi
Shear modulus (high strain), G_h	700 ksf	1,240 ksi	270 ksf	1,400 ksf	150 ksi	700 ksi	2,900 ksi
Shear modulus (low strain), G_l	5,000 ksf	1,240 ksi	2,800 ksf	10,000 ksf	300 ksi	700 ksi	2,900 ksi

* Best Estimate Value