Mark D. Mitchell
Vice President - Generation Construction
Dominion Generation

An operating segment of
Dominion Resources, Inc.
5000 Dominion Boulevard, Glen Allen, VA 23060

dom.com

Dominion

August 11, 2014

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

Serial No. NA3-14-030B
Docket No. 52-017
COL/RGM

DOMINION VIRGINIA POWER
NORTH ANNA UNIT 3 COMBINED LICENSE APPLICATION
SRPs 03.07.02 AND 03.08.05:
RESPONSE TO RAI LETTERS 123 AND 125

On June 5, 2014, 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 Requests for Additional Information (RAI) Questions are provided in the enclosures:

- RAI 7536, Question 03.07.02-11 — SSI Input Control Motions
- RAI 7536, Question 03.07.02-19 — Enveloping Horizontal Accelerations
- RAI 7536, Question 03.07.02-24 — SSI Analyses- Non-Seismic Category I
- RAI 7538, Question 03.08.05-7 — Maximum Dynamic Bearing Pressures

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,

Mark D Mitchell

Mark D. Mitchell

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 Mark D. Mitchell, who is Vice President–Generation Construction 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 7 day of August, 2014
My registration number is 337029 and my
Commission expires: January 31, 2015

Donna A. Port
Notary Public

Embossed Hereon Is My
Commonwealth Of Virginia Notary Public Seal
My Commission Expires January 31, 2015
DONNA A. PORT

DONNA A. PORT
NOTARY PUBLIC
MY COMMISSION NUMBER 337029
COMMONWEALTH OF VIRGINIA

Enclosures:

1. Response to NRC RAI Letter No. 123, RAI 7536, Question 03.07.02-11
2. Response to NRC RAI Letter No. 123, RAI 7536, Question 03.07.02-19
3. Response to NRC RAI Letter No. 123, RAI 7536, Question 03.07.02-24
4. Response to NRC RAI Letter No. 125, RAI 7538, Question 03.08.05-7

Commitments made by this letter:

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

cc: U. S. Nuclear Regulatory Commission, Region II
C. P. Patel, NRC
T. S. Dozier, NRC
G. J. Kolcum, NRC
D. Paylor, VDEQ
W. T. Lough, SCC
P. W. Smith, DTE
M. K. Brandon, DTE
R. J. Bell, NEI

ENCLOSURE 1

Response to NRC RAI Letter No. 123

RAI No. 7536, Question 03.07.02-11

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

North Anna Unit 3

Dominion

Docket No. 52-017

RAI NO.: 7536 (RAI LETTER 123)

SRP SECTION: 3.7.2 - SEISMIC SYSTEM ANALYSIS

DATE OF RAI ISSUE: 06/05/2014

QUESTION NO.: 03.07.02-11

FSAR 3.7.2.4.1 indicates that SSI input control motions for the CB are defined at the bottom of the CB foundation basemat, which is consistent with the development of the SSI inputs described in FSAR 3.7.1. However, FSAR Section 3.7.2.4.1.1 indicates that the concrete fill below the CB basemat is an integral part of the structural model used in the SSI analysis (FSAR Figures 3.7.2-205 and 206). Since the combined CB-concrete fill is modeled as an integral structure, the control motions for SSI analysis should be defined at the bottom of the concrete fill to be consistent with the guidance in ISG-17, which the applicant has committed to. The staff also notes that the difference in elevation between the bottom of the basemat and the bottom of the concrete fill is approximately 5 m. Therefore, the applicant is requested to provide the technical justification for defining the SSI input control motions for the CB at the bottom of the basemat and not at the bottom of the concrete fill.

<u>Dominion Response</u>

Defining the SSI input control motion for the Control Building (CB) at its basemat bottom elevation is acceptable because it is consistent with the DCD and with NRC and NEI guidance. Additionally, the noted difference in elevations between the basemat bottom elevation and the bottom of the concrete fill has only a small effect on the seismic response and does not impact the adequacy of the site-specific seismic evaluation or the design of the North Anna Unit 3 (NA3) CB. The information supporting this conclusion is provided below.

The configuration and the dynamic properties of the NA3 CB structure and basemat foundation are identical to the ESBWR DCD standard design. Prior to the CB construction at NA3, the in-situ materials located above the top of Zone III-IV rock located at the nominal elevation of 225 ft NAVD88 will be excavated.

A layer of concrete fill will be placed on top of the Zone III-IV rock to support the 3 m thick CB reinforced concrete basemat. This layer of concrete fill that will be placed in the excavation before the construction of the CB basemat is considered a competent material supporting the foundation and is not an integral part of the CB structure. The horizontal extent of the concrete fill is limited to the perimeter of the excavation. In order to account for the limited horizontal extent, the concrete fill is represented in the SASSI2010 structural (house) model by a block of solid elements embedded in layers of in-situ Zone III rock with shear wave velocities ranging from 750 m/sec (2,461 ft/sec) to 810 m/sec (2,657 ft/sec). The 16 foot deep block of concrete fill is resting on the surface of the Zone III-IV rock and extends up to the CB basemat bottom.

The site-specific SSI analyses of the CB were performed using free field in-layer input control motions, described in FSAR Section 3.7.1.1.5.1.2, that are compatible to the SSI input response spectra defining the outcrop site-specific design motion at the bottom of the CB basemat. As noted in FSAR Section 3.7.1.1.4.2.2, the CB SSI response spectra is defined as the envelope of the minimum design earthquake, defined by RG 1.60 spectrum anchored at peak ground acceleration of 0.1 g, and the CB Foundation Input Response Spectra (FIRS). The definition of the FIRS at the bottom of the CB basemat follows the recommendations of DC/COL-ISG-017 and the NEI White Paper (Reference 1) and is consistent with the ESBWR DCD definition of the input motion used for the SSI analysis of the CB as part of the certified design. Consistently, the control point corresponding to the application of the FIRS in the NA3 site-specific SSI analysis is defined at the bottom of the CB basemat at elevation 241 ft NAVD88.

Two sets of spectra were developed for the CB SSI analyses of the partial column profiles (the profiles that exclude the in-situ materials above Zone III rock) and the full column profiles (the profiles that include the in-situ materials above Zone III rock). The FIRS, defining the NA3 site-specific design motion, were developed at the bottom of the Seismic Category I building basemats from the results of probabilistic site response analysis, presented in FSAR Section 2.5.2.5.3, of different partial column and full column profiles representative of the subgrade conditions at the locations of the NA3 Seismic Category I buildings. Based on the results of the probabilistic site response analysis of the profile representative of the subgrade conditions at the NA3 CB, Design Response Spectra (DRS) were developed at the following two horizons:

1. RB/FB basemat bottom elevation of 224 ft NAVD88 which is 1 ft below the bottom elevation of the concrete fill block below the CB foundation (the horizontal full column DRS and partial column DRS at this elevation are presented in FSAR Figures 2.5.2-300 and 2.5.2-303, respectively)
2. CB basemat bottom elevation of 241 ft NAVD88 (the horizontal full column DRS and partial column DRS that were adopted as CB Horizontal FIRS are presented in FSAR Figures 2.5.2-301 and 2.5.2-304, respectively)

As described in FSAR Section 3.7.1.1.4.1.2, the probabilistic site response analysis of this profile also provided the strain compatible dynamic properties of the in-situ subgrade materials used for the development of the subgrade profiles for the site-specific SSI analysis of the CB.

To evaluate the impact of defining the SSI input control motions for the CB at the bottom of the basemat and not at the bottom of the concrete fill, a comparison of the DRS was performed. Figure 1 presents a comparison of the horizontal full column DRS for the CB full column profile at these two different horizon elevations as described above. Figure 2 presents a comparison of the horizontal partial column DRS for the CB partial column profile at the two different horizon elevations. The ratio of the DRS at elevation 241 ft NAVD88 and the DRS at elevation 224 ft NAVD88 is also presented in each figure with a dashed line for ease of comparison.

The comparison of the DRS at elevation 241 ft NAVD88 and the DRS at elevation 224 ft NAVD88 shows that upward propagation of the seismic waves through the rock from bottom of the concrete fill to the bottom of the CB foundation does not result in shifts of the frequency content of the input motions towards lower frequencies or reduction of the high frequency amplitudes. Therefore, the input control motion used for the site-specific SSI analyses of the CB defined at basemat bottom elevation is acceptable and does not affect the results of the site-specific seismic evaluation or the design of the CB.

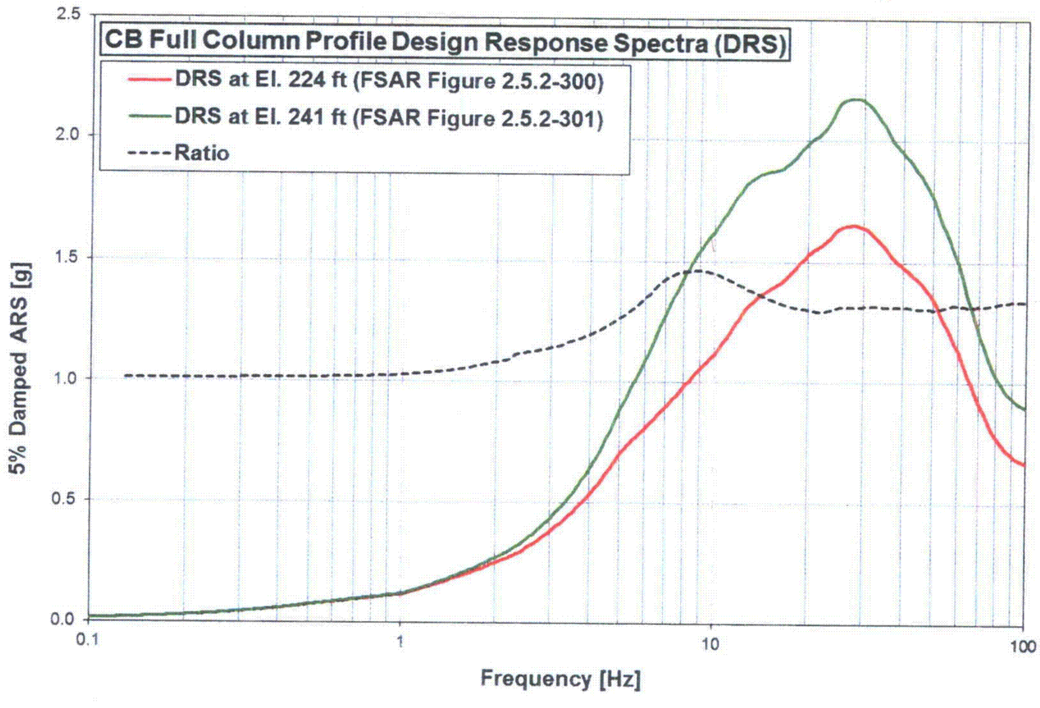

Figure 1: Comparison of Horizontal Full Column DRS for the CB Full Column Profile at Two Different Elevations

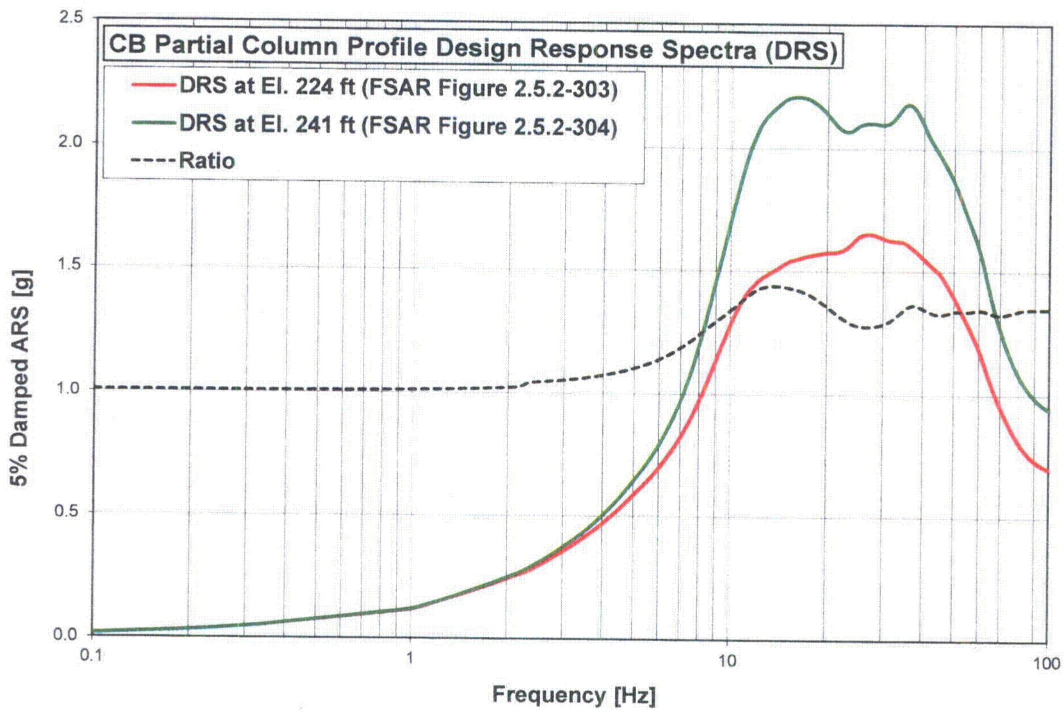

Figure 2: Comparison of Horizontal Partial Column DRS for the CB Partial Column Profile at Two Different Elevations

References

1. NEI White Paper, "Consistent Site-Response/Soil-Structure Interaction Analysis and Evaluation," NEI, June 12, 2009. (ADAMS Accession No. ML091680715).

Proposed COLA Revision

None

ENCLOSURE 2

Response to NRC RAI Letter No. 123

RAI No. 7536, Question 03.07.02-19

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

North Anna Unit 3

Dominion

Docket No. 52-017

RAI NO.: 7536 (RAI LETTER 123)

SRP SECTION: 3.7.2 - SEISMIC SYSTEM ANALYSIS

DATE OF RAI ISSUE: 06/05/2014

QUESTION NO.: 03.07.02-19

FSAR Section 3.7.2.4.1.6.1 indicates that the site-specific enveloping horizontal accelerations of flexible walls (i.e., SDOF oscillators) in the RB/FB exceed the corresponding standard design enveloping horizontal accelerations by as much as 84 %. However, the FSAR does not provide documentation of stress check results for flexible walls in the RB/FB. Instead, a brief description of the method used to perform the stress checks is provided in FSAR pgs. 3-127 (last paragraph) and 3-128 (first paragraph). In order to review the adequacy of the methodology used for performing the stress checks for flexible walls, additional information is needed. Therefore, the applicant is requested to describe the method in detail and explain whether the method used for the site-specific stress evaluation is different from the approach used in the ESBWR DCD and, if so, provide additional information of the method in the FSAR as well as a technical validation of the method with that of the DCD or other acceptable approaches.

Dominion Response

In the current North Anna Unit 3 (NA3) FSAR, the ESBWR DCD methods were used to perform the site-specific stress evaluations of the RB/FB walls, with the exception of a scaling factor approach that was used to determine the NA3 combined stress demands and to perform stress checks against the code-allowable values.

However, based on discussions with the NRC during a public meeting held on June 19, 2014, Dominion has decided to replace the scaling factor approach with the method that was used in the ESBWR DCD. The stress checks will be

performed in a direct manner (i.e., without scaling the DCD stress checks), using the computed site-specific seismic loads, and will include the contribution of all other load cases to the total stresses and all applicable load combinations. A comparison will be made between the resulting site-specific total stress demands and the code-allowable stresses. As a result, the information related to stress checks performed using the scaling factor approach will be deleted from FSAR Section 3.7.2.4.1.6.1 and the related tables, and will be replaced with the results from the direct approach as a part of the response to RAI 03.07.02-17.

The direct stress check methodology for the flexible walls in the RB/FB is described as follows:

The site-specific seismic load demands on the RB/FB flexible walls in terms of equivalent out-of-plane acceleration loads are determined following the same weighted-average methodology used in the standard design. This methodology was used for the current FSAR. These site-specific enveloping horizontal accelerations are determined from the site-specific soil-structure interaction (SSI) analysis results for maximum accelerations of single-degree-of-freedom (SDOF) flexible wall oscillators and the horizontal accelerations of the floor lumped masses located above and below the wall.

The RB/FB lumped mass SSI model includes several SDOF oscillators to capture the out-of-plane responses of walls in flexible modes of vibration with frequencies less than 50 Hz. The 84% exceedance of site-specific enveloping horizontal accelerations of SDOF oscillators in the RB/FB represents exceedance of the response only in the flexible modes of vibration. The equivalent seismic acceleration loads used for the stress checks of these walls include contributions of flexible and rigid modes of vibration of the walls and are developed following the methodology identical to that used for the standard design.

These site-specific equivalent out-of-plane acceleration loads will be used as input for the finite element analysis, which will provide site-specific seismic stress demands on the RB/FB flexible walls. This stress analysis will be performed using the same RB/FB finite element model and methodology as those used for the standard design of the RB/FB structures.

The computed site-specific seismic stress demands will then be combined with stress demands from other applicable load cases in the corresponding seismic load combinations to determine the total site-specific stress demands. The stress demands from non-seismic design loads will be obtained from the results of the standard design finite element analysis of the RB/FB structures.

Finally, stress checks of the flexible walls will be performed by comparing the total site-specific out-of-plane stress demands with the allowable out-of-plane

bending and shear stresses, which will be computed considering the effect of the site-specific axial loads on the strength of the reinforced concrete members. Stress checks will be performed using the same methodology and the same critical sections as those used for the standard design.

In summary, upon completion of these steps, the site-specific stress evaluation methodology, which includes evaluation of the out-of-plane seismic load demands on the RB/FB flexible walls, will be the same as the approach used in the DCD. The COLA will be revised to provide the additional information regarding the methodology and results when the stress evaluation and stress checks are completed as a part of the response to RAI 03.07.02-17.

Proposed COLA Revision

None

ENCLOSURE 3

Response to NRC RAI Letter No. 123

RAI No. 7536, Question 03.07.02-24

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

North Anna Unit 3

Dominion

Docket No. 52-017

RAI NO.: 7536 (RAI LETTER 123)

SRP SECTION: 3.7.2 - SEISMIC SYSTEM ANALYSIS

DATE OF RAI ISSUE: 06/05/2014

QUESTION NO.: 03.07.02-24

FSAR Section 3.7.2.8 indicates that site-specific SSI analyses for the Non-Seismic Category I TB, RW, SB, and ADB structures are performed following the methodology for Seismic Category I structures described in FSAR Section 3.7.2.4. Acceptance criteria for these site-specific SSI analyses are provided in NA3 ITAAC Tables 2.4.15-1, 2.4.16-1, 2.4.17-1 and 2.4.18-1 for the TB, RW, SB, and ADB structures respectively. However, the staff's review of the site cross sections shown in FSAR Figures 2.5.4-225 through 234 indicates that the foundations for the TB, RW, SB, and ADB structures may be supported on granular fill in some cases (e.g., FSAR Figure 2.5.4-230 for the TB) and not always on rock or concrete fill. It is not clear to the staff that the granular fill meets the requirements in FSAR Table 2.0-201 for minimum shear wave velocity for supporting soil (i.e., 1000 ft/sec). Therefore, it is not clear to the staff how the methodology for Seismic Category I structures, described in FSAR Section 3.7.2.4, is applicable to structures that are not supported on soil material that meets the requirements in FSAR Table 2.0-201. In order to ensure that the seismic analysis of the NA3 non-Seismic Category I structures are acceptable, the applicant is requested to address this inconsistency.

Dominion Response

The structural fill surrounding and under the Turbine Building, Service Building, and Ancillary Diesel Building structures (Seismic Category II structures) and the Radwaste Building will not meet the shear wave velocity requirement in FSAR Table 2.0-201 at all depths. Site-specific Soil-Structure Interaction (SSI) analyses will be performed to address the effect of the subgrade and seismic conditions at Unit 3, including the effects of the structural fill on the seismic response of these structures. These SSI analyses will follow the same SASSI

methodology as the one used for the seismic response analyses of Seismic Category I Structures described in FSAR Section 3.7.2, which is appropriate for analyses of foundations supported by competent materials as the Unit 3 structural fill. A detailed explanation is provided below.

The purpose of the site-specific SSI analyses specified for the Unit 3 Seismic Category II structures and Radwaste Building in FSAR Section 3.7.2.8 is to confirm the seismic design adequacy of the structures at the Unit 3 site and to address:

1. Departure NAPS DEP 3.7-1 CSDRS ground motion exceedance.
2. The impact of using Unit 3 structural fill material, which does not meet the standard design soil parameter requirement for a minimum shear wave velocity of 1000 ft/sec at all depths, on the seismic response of these structures.

The shear wave velocity of granular materials such as the Unit 3 structural fill is a function of both the material properties and the confining pressure of the overlying soils and/or structures. As described in FSAR Section 2.5.4.7.1(e) and shown in the plots of best estimate small-strain shear wave velocity profiles in FSAR Figures 2.5.4-245 and 2.5.4-246, the required shear wave velocity of 1000 ft/sec will not be achievable around and immediately below the foundations of the Seismic Category II structures or the Radwaste Building because of the lack of confining pressure. However, even with a shear wave velocity less than the standard design soil parameter, the Unit 3 structural fill is a competent material, as described in FSAR Sections 2.5.4.2.5, 2.5.4.5.3, and 2.5.4.8, with an effective friction angle of 40 degrees and with no potential for liquefaction, as shown in FSAR Table 2.5.4-208.

Use of the SASSI methodology is appropriate for SSI analyses of foundations supported by Unit 3 structural fill because this is a competent material with high-strength properties and which is not liquefiable. The analyses will be performed for a set of subgrade profiles of subgrade strain-compatible properties to address the effects of soil variability on the seismic response of the structures. The strain-compatible properties of the structural fill material will be obtained from the results of probabilistic site response analyses following the methodology described in FSAR Section 3.7.1.1.4.1, using as input randomized small-strain structural fill properties and shear modulus degradation curves. The results of these Unit 3 site-specific analyses for seismic structural load demands of these structures will be compared to the seismic loads from the standard design SSI analyses. If the Unit 3 seismic load demands on any particular structure exceed the corresponding standard design seismic loads, an explicit site-specific stress check will be performed for this structure to evaluate its adequacy for the Unit 3 site, using the same methodology as the one used for the structural evaluations of Seismic Category I structures.

As described above, the site-specific SSI analyses for the Seismic Category II structures and Radwaste Building will account for both the site-specific ground motion exceedances of the CSDRS and the Unit 3 structural fill material that does not meet the standard design soil parameter requirement for a minimum shear wave velocity of 1000 ft/sec. The Unit 3 ITAAC will be modified to more clearly specify that the resulting seismic load demands obtained from the site-specific SSI analyses are bounded by the seismic allowable loads associated with the standard design of each structure or, if the site-specific SSI analyses yield seismic load demands which are larger than the seismic loads used for the standard design, explicit stress checks will be performed to ensure these structures are adequate to withstand the Unit 3 seismic loads. This is consistent with the approach used for the site-specific evaluation of Seismic Category I structures and the associated site-specific ITAAC are modified to reflect this approach, as shown on the attached COLA markups.

In summary, the methodology used for the Seismic Category I structures described in FSAR Section 3.7.2 is appropriate for the Seismic Category II structures and the Radwaste Building because the structural fill surrounding and under these structures is competent material, even though the material will not meet the 1000 ft/sec shear wave velocity standard design requirement. The analyses will use a set of subgrade profiles of subgrade strain-compatible properties to address the effects of using structural fill with variable properties on the seismic response of these structures. The site-specific seismic load demands will be obtained through SSI analyses and, if the seismic loads exceed the seismic loads of the standard design, specific structural evaluations will be performed to demonstrate that the stresses are bounded by Code allowable limits.

Proposed COLA Revision

COLA Part 10, Sections 2.4.15, 2.4.16, 2.4.17, and 2.4.18, 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.

2.4.9 Communications Systems (Emergency Notification System)

Addressed in Table 2.3-1, 3.0 Emergency Communications

2.4.10 Makeup Water System

No ITAAC are required for this system.

2.4.11 (Deleted)

2.4.12 (Deleted)

2.4.13 Hydrogen Water Chemistry System

No ITAAC are required for this system.

2.4.14 Meteorological Monitoring System

No ITAAC are required for this system.

2.4.15 ITAAC for the Turbine Building

Design Description

The Turbine Building is a Seismic Category II building. The design and analysis of the Turbine Building will preclude any adverse interaction with Seismic Category I structures, considering the soil properties. The Unit 3 seismic design response spectra are based on 5 percent damping of the free-field outcrop spectra at the foundation level (bottom of the base slab): 1) the scaled CSDRS shown in DCD Figures 2.0-1 and 2.0-2; and 2) the FIRS for each individual structure. Foundation input response spectra will be developed for the Turbine Building at the foundation level. Site-specific soil structure interaction (SSI) analyses using the ~~two~~ Unit 3 seismic design response spectra and using site-specific soil properties will be performed for the Turbine ~~Buidling~~ Building following the same methodology used in FSAR Section 3.7.2 to determine SSI enveloping seismic loads and to develop in-structure response spectra.

Inspections, Test, Analyses and Acceptance Criteria

Table 2.4.15-1 provides a definition of the inspections, tests, and/or analyses, together with associated acceptance criteria for the Turbine Building.

Table 2.4.15-1 ITAAC for the Turbine Building

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
1. Site-specific soil structure interaction (SSI) analyses using the two Unit 3 seismic design response spectra and using site specific soil properties will be performed for the Turbine Buidling following the same methodology used in FSAR Section 3.7.2 to determine SSI enveloping seismic loads and to develop instructure response spectra.	Unit 3 soil properties will be determined. SSI analyses of the Turbine Buidling will be conducted. Structure-soil-structure interaction (SSSI) analyses of the Turbine Buidling will be conducted, if necessary.	The results of the site specific SSI analyses of the Turbine Buidling are compared with the Turbine Buidling seismic responses presented in DCD Tier 1 ITAAC Table 2.16.8-1 seismic analyses to confirm the Unit 3 SSI is adequate for the Turbine Buidling seismic design.
1. The Turbine Building structure seismic load demands are within acceptable limits to ensure that the structure is seismically adequate.	Perform site-specific SSI analysis, following the methodology specified for Seismic Category I structures in FSAR Section 3.7.2, to address ground motion exceedances and site-specific effects of subgrade properties, and (if necessary) perform a structural evaluation for the Turbine Building structure.	(1) Seismic load demands obtained from the site-specific SSI analysis for the Turbine Building structure are acceptable because the seismic loads are bounded by the standard design seismic loads used for the Turbine Building; or, (2) If the site-specific seismic loads exceed the standard design seismic loads, the Turbine Building structure is seismically adequate for the Unit 3 site if the results from the structural evaluation demonstrate that the total stresses are bounded by Code allowable stress limits.

2.4.16 ITAAC for the Radwaste Building

Design Description

The Radwaste Building is a Seismic Category NS building. The design and analysis of the Radwaste Building will preclude any adverse interaction with Seismic Category I structures, considering the soil properties. The Unit 3 seismic design response spectra are based on 5% damping of the free-field outcrop spectra at the foundation level (bottom of the base slab): 1) the scaled CSDRS shown in DCD Figures 2.0-1 and 2.0-2; and 2) the FIRS for each individual structure. Foundation input response spectra will be developed for the Radwaste Building at the foundation level. Site-specific soil structure interaction (SSI) analyses using the ~~two~~ Unit 3 seismic design response spectra and using site-specific soil properties will be performed for the Radwaste Building following the same methodology used in FSAR Section 3.7.2 to determine SSI enveloping seismic loads and to develop in-structure response spectra.

Inspections, Tests, Analyses, and Acceptance Criteria

Table 2.4.16-1 provides a definition of the inspections, tests, and/or analyses, together with associated acceptance criteria for the Radwaste Building.

Table 2.4.16-1 ITAAC for the Radwaste Building

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
~~1. Site-specific soil structure interaction (SSI) analyses using the two Unit 3 seismic design response spectra and using site-specific soil properties will be performed for the Radwaste Building following the same methodology used in FSAR Section 3.7.2 to determine SSI enveloping seismic loads and to develop in-structure response spectra.~~	~~Unit 3 soil properties will be determined. SSI analyses of the Radwaste Building will be conducted. Structure-soil-structure interaction (SSSI) analyses of the Radwaste Building will be conducted, if necessary.~~	~~The results of the site-specific SSI analyses of the Radwaste Building are compared with the Radwaste Building seismic responses presented in DCD Tier 1 ITAAC Table 2.16.9-1 seismic analyses to confirm the Unit 3 SSI is adequate for the Radwaste Building seismic design.~~
1. The Radwaste Building structure seismic load demands are within acceptable limits to ensure that the structure is seismically adequate.	Perform site-specific SSI analysis, following the methodology specified for Seismic Category I structures in FSAR Section 3.7.2, to address ground motion exceedances and site-specific effects of subgrade properties, and (if necessary) perform a structural evaluation for the Radwaste Building structure.	(1) Seismic load demands obtained from the site-specific SSI analysis for the Radwaste Building structure are acceptable because the seismic loads are bounded by the standard design seismic loads used for the Radwaste Building; or, (2) If the site-specific seismic loads exceed the standard design seismic loads, the Radwaste Building structure is seismically adequate for the Unit 3 site if the results from the structural evaluation demonstrate that the total stresses are bounded by Code allowable stress limits.

2.4.17 ITAAC for the Service Building

Design Description

The Service Building is a Seismic Category II building. The design and analysis of the Service Building will preclude any adverse interaction with Seismic Category I structures, considering the soil properties. The Unit 3 seismic design response spectra are based on 5% damping of the free-field outcrop spectra at the foundation level (bottom of the base slab): 1) the scaled CSDRS shown in DCD Figures 2.0-1 and 2.0-2; and 2) the FIRS for each individual structure. Foundation input response spectra will be developed for the Service Building at the foundation level. Site-specific soil structure interaction (SSI) analyses using the ~~two~~ Unit 3 seismic design response spectra and using site-specific soil properties will be performed for the Service Building following the same methodology used in FSAR Section 3.7.2 to determine SSI enveloping seismic loads and to develop in-structure response spectra.

Inspections, Tests, Analyses, and Acceptance Criteria

Table 2.4.17-1 provides a definition of the inspections, tests, and/or analyses, together with associated acceptance criteria for the Service Building.

Table 2.4.17-1 ITAAC for the Service Building

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
1. Site-specific soil structure interaction (SSI) analyses using the two Unit 3 seismic design response spectra and using site-specific soil properties will be performed for the Service Building following the same methodology used in FSAR Section 3.7.2 to determine SSI enveloping seismic loads and to develop instructure response spectra.	Unit 3 soil properties will be determined. SSI analyses of the Service Building will be conducted. Structure-soil-structure interaction (SSSI) analyses of the Service Building will be conducted, if necessary.	The results of the site-specific SSI analyses of the Service Building are compared with the Service Building seismic responses presented in DCD Tier 1 ITAAC Table 2.16.10-1 seismic analyses to confirm the Unit 3 SSI is adequate for the Service Building seismic design.
1. The Service Building structure seismic load demands are within acceptable limits to ensure that the structure is seismically adequate.	Perform site-specific SSI analysis, following the methodology specified for Seismic Category I structures in FSAR Section 3.7.2, to address ground motion exceedances and site-specific effects of subgrade properties, and (if necessary) perform a structural evaluation for the Service Building structure.	(1) Seismic load demands obtained from the site-specific SSI analysis for the Service Building structure are acceptable because the seismic loads are bounded by the standard design seismic loads used for the Service Building; or, (2) If the site-specific seismic loads exceed the standard design seismic loads, the Service Building structure is seismically adequate for the Unit 3 site if the results from the structural evaluation demonstrate that the total stresses are bounded by Code allowable stress limits.

2.4.18 ITAAC for the Ancillary Diesel Building

Design Description

The Ancillary Diesel Building is a Seismic Category II building. The design and analysis of the Ancillary Diesel Building will preclude any adverse interaction with Seismic Category I structures, considering the soil properties. The Unit 3 seismic design response spectra are based on 5% damping of the free-field outcrop spectra at the foundation level (bottom of the base slab): 1) the scaled CSDRS shown in DCD Figures 2.0-1 and 2.0-2; and 2) the FIRS for each individual structure. Foundation input response spectra will be developed for the Ancillary Diesel Building at the foundation level. Site-specific soil structure interaction (SSI) analyses using the ~~two~~ Unit 3 seismic design response spectra and using site-specific soil properties will be performed for the Ancillary Diesel Building following the same methodology used in FSAR Section 3.7.2 to determine SSI enveloping seismic loads and to develop in-structure response spectra.

Inspections, Tests, Analyses, and Acceptance Criteria

Table 2.4.18-1 provides a definition of the inspections, tests, and/or analyses, together with associated acceptance criteria for the Ancillary Diesel Building.

Table 2.4.18-1 ITAAC for the Ancillary Diesel Building

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
~~1. Site-specific soil structure interaction (SSI) analyses using the two Unit 3 seismic design response spectra and using site-specific soil properties will be performed for the Ancillary Diesel Building following the same methodology used in FSAR Section 3.7.2 to determine SSI enveloping seismic loads and to develop instructure response spectra.~~	~~Unit 3 soil properties will be determined. SSI analyses of the Ancillary Diesel Building will be conducted. Structure soil structure interaction (SSSI) analyses of the Ancillary Diesel Building will be conducted, if necessary.~~	~~The results of the site-specific SSI analyses of the Ancillary Diesel Building are compared with the Ancillary Diesel Building seismic responses presented in DCD Tier 1 ITAAC Table 2.16.11-1 seismic analyses to confirm the Unit 3 SSI is adequate for the Ancillary Diesel Building seismic design.~~
1. The Ancillary Diesel Building structure seismic load demands are within acceptable limits to ensure that the structure is seismically adequate.	Perform site-specific SSI analysis, following the methodology specified for Seismic Category I structures in FSAR Section 3.7.2, to address ground motion exceedances and site-specific effects of subgrade properties, and (if necessary) perform a structural evaluation for the Ancillary Diesel Building structure.	(1) Seismic load demands obtained from the site-specific SSI analysis for the Ancillary Diesel Building structure are acceptable because the seismic loads are bounded by the standard design seismic loads used for the Ancillary Diesel Building; or, (2) If the site-specific seismic loads exceed the standard design seismic loads, the Ancillary Diesel Building structure is seismically adequate for the Unit 3 site if the results from the structural evaluation demonstrate that the total stresses are bounded by Code allowable stress limits.

ENCLOSURE 4

Response to NRC RAI Letter No. 125

RAI No. 7538, Question 03.08.05-7

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

North Anna Unit 3

Dominion

Docket No. 52-017

RAI NO.: 7538 (RAI LETTER 125)

SRP SECTION: 3.8.5 - FOUNDATIONS

DATE OF RAI ISSUE: 06/05/2014

QUESTION NO.: 03.08.05-7

The maximum dynamic bearing pressures for the RB/FB, CB, and FWSC basemats are reported in FSAR Tables 3.8.5-204 through 3.8.5-206 of the FSAR, for the BE, UB, and LB subsurface profiles. FSAR Table 3.8.5-204 further indicates that the maximum site-specific dynamic bearing pressure exerted by the RB/FB on the underlying rock is 1.17 MPa, which is slightly greater than the value of 1.10 MPa reported in ESBWB DCD Tables 2.0-1 and 3G.1-58 for the "hard" site condition. Similarly, FSAR Table 3.8.5-205 indicates that the maximum site-specific dynamic bearing pressure exerted by the CB on the underlying concrete fill is 0.52 MPa, which is greater than the value of 0.42 MPa reported in ESBWB DCD Tables 2.0-1 and 3G.2-27 for the "hard" site condition.

Since the site-specific maximum dynamic bearing pressures for the RB/FB and CB exceed the values reported in the DCD for the "hard" site condition, which is the DCD condition that most resembles the underlying rock or concrete fill materials at the NA3 site, the applicant is requested to evaluate whether the site-specific bending moments and shears induced in the RB/FB and CB basemats, due to load combinations that include seismic loads, are bounded by the bending moments and shears considered in the standard design. In addition confirm that the dynamic bearing pressures for the RB/FB, CB, and FWSC basemats are less than the allowable dynamic bearing capacities of the underlying rock or concrete fill material.

Dominion Response

The North Anna Unit 3 (NA3) site-specific demands for bending moments and shears induced in the RB/FB and CB basemats are bounded by the bending

moments and shears considered in the standard design. The dynamic bearing pressures (Safe Shutdown Earthquake (SSE) + Static) for the RB/FB, CB, and FWSC basemats are less than the allowable dynamic bearing capacities of the underlying rock or concrete fill material.

Bending Moment and Shear Forces

The average basemat thickness of the RB/FB is about 13 ft. The average basemat thickness of the CB is about 10 ft. Because the RB/FB and CB have thick basemats with several connecting walls and soil springs, a detailed stress analysis for each basemat was performed using a 3-D finite element model.

The evaluation of site-specific bending moments and shears induced in the RB/FB and CB basemats against the corresponding bending moments and shears considered in the standard design was performed using the indirect method. The indirect method compared the shear force, bending moment, and torsional moment outputs of those bottom elements in the SASSI stick models that connect the sticks to the basemat model. Based on the NA3 to DCD shear force and moment ratios in these bottom elements, an engineering evaluation was performed to determine if the site-specific bending moments and shears induced in the RB/FB and CB basemats, due to load combinations that include seismic loads, are bounded by the bending moments and shears considered in the standard design.

FSAR Tables 3.7.2-205, 3.7.2-206, 3.7.2-207 compare the DCD and NA3 seismic outputs in the RB/FB model. FSAR Table 3.7.2-217 compares the DCD and NA3 seismic outputs in the CB model. The comparisons of forces and moments in the bottom stick elements that connect the sticks to the basemats indicate that the site-specific seismic induced forces and moments induced in the RB/FB and CB are bounded by the seismic forces and moments considered in the standard design by a large margin.

Dynamic Bearing Pressures

ESBWR DCD Table 2.0-1, Envelope of ESBWR Standard Plant Site Parameters, lists the maximum dynamic bearing (SSE + Static) demands of the RB/FB, CB, and FWSC basemats for soft, medium, and hard site conditions. As presented in the following table, these maximum dynamic bearing pressures (Safe Shutdown Earthquake (SSE) + Static) are less than the corresponding NA3 allowable bearing capacities.

Maximum DCD Dynamic Bearing Demand (SSE + Static)				NA3 Dynamic Bearing Capacity	
	Soft Site	Medium Site	Hard Site	Fill Concrete	Rock
Reactor/Fuel Building (RB/FB)	1.10 MPa	2.70 MPa	1.10 MPa	9.53 MPa	12.40 MPa
Control Building (CB)	0.50 MPa	2.20 MPa	0.42 MPa	9.53 MPa	12.40 MPa
Firewater Service Complex (FWSC)	0.46 MPa	0.69 MPa	1.20 MPa	9.53 MPa	12.40 MPa

Proposed COLA Revision

None