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Dominion®

February 8, 2012

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

Serial No. NA3-11-025RB
Docket No. 52-017
COL/MWH

**DOMINION VIRGINIA POWER**
**NORTH ANNA UNIT 3 COMBINED LICENSE APPLICATION**
**SRP 02.05.02: RESPONSE TO RAI LETTER 68**

On June 5, 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 Request for Additional Information (RAI) 5693 Question 02.05.02-3 was provided in Dominion letter NA3-11-025R dated August 25, 2011 (ML11241A058).

During a subsequent conference call on January 5, 2012, NRC staff requested supplemental information to support the review of the response to RAI 5693 Question 02.05.02-3. The requested supplemental information is provided in Enclosure 1.

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

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
V. Graizer, 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 8th day of February, 2012
My registration number is 112536 and my
Commission expires: April 30, 2015

Jane M. Mack
Notary Public

Enclosure:

1. Response to NRC RAI Letter 68, RAI 5693 Question 02.05.02-3 Supplemental Information

Commitments made by this letter:

Revise COL application as described in the letter.

**ENCLOSURE 1**

**Response to NRC RAI Letter 68**

**RAI 5693 Question 02.05.02-3
Supplemental Information**

## RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

**North Anna Unit 3**

**Dominion**

**Docket No. 52-017**

**RAI NO.: 5693 (RAI Letter 68)**

**SRP SECTION: 02.05.02 – VIBRATORY GROUND MOTION**

**QUESTIONS for Geosciences and Geotechnical Engineering Branch 2 (RGS2)**

**DATE OF RAI ISSUE: 5/5/2011**

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**QUESTION NO.: 02.05.02-3**

The response to RAI 5199 (02.05.02-2) states that a 1-D analysis is justified for calculation of the GMRS due to the similarity of the bed rock and RQD values. However, shear wave velocity measurements show considerable variation at equivalent elevation levels, indicating that weathered rock zones III and III-IV are of variable thickness. For example, the BE profile shown on Figure 2.5-202b (Rev. 4) is a result of combining shear wave velocity measurements from borings B-901, B-907 and B-909, and represents the log mean of the Profiles 1 and 2 shown on Figure 2.5-241a (Rev. 3). Values shown on Figure 2.5-241a indicate that shear wave velocities vary up to 100% from approximately elevation 184 ft to 250 ft. These considerable horizontal variations in shear wave velocity impedance contrasts indicate that a 1-D analysis may not be sufficient to describe the multi-dimensionality of the subsurface, and the use of the BE profile instead of enveloping site amplifications from Profiles 1 and 2 may result in an underestimation of the site amplification functions, and, ultimately the GMRS.

In accordance with 10 CFR 100.23(c) and RG-1.208, the staff requests that the applicant justify that the 1-D site response analysis utilizing only vertically propagating shear waves is appropriate for the underlying complex velocity structure and the results of the 1-D analysis produce a GMRS that adequately characterizes the local subsurface conditions.

Please provide a table of layer thicknesses, shear-wave velocities, and densities, and identify the type of shear modulus and damping curves used for all site amplification calculations. Also explain how the average shear wave velocity Profiles 1 and 2 displayed in Figure 2.5-241a were developed.

SUPPLEMENTAL INFORMATION REQUEST (by conference call on January 5, 2012):

The NRC requested, in a conference call with Dominion held on January 5, 2012, clarification of how the information provided in FSAR Table 2.5-208 (elevation, depth, and thicknesses of the subsurface zones) and FSAR Table 2.5-212 (engineering properties of subsurface materials) were used in the idealized shear wave velocity ($V_s$) profiles developed for the sensitivity study provided in the response to RAI 5693, question 02.05.02-3. The NRC also requested that the idealized $V_s$ profiles for boreholes B-901, B-907, and B-909, as well as boreholes M10 and M30, should be considered for addition to the FSAR.

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## Dominion Response to Supplemental Request

The shear wave velocity ($V_s$) values provided in FSAR Table 2.5-212 for different soil/rock subsurface zones are best estimate values based on data from all five of the $V_s$ borings (B-901, B-907, B-909, M-10 and M-30) and provide a general geotechnical characterization of each zone of material. These best estimate $V_s$ values were not used as direct input to the sensitivity analysis or the site response analysis (SRA) and are not consistent with the idealized rock $V_s$ profiles developed for the sensitivity study provided in the response to RAI 5693, question 02.05.02-3 (Dominion letter NA3-11-025R dated August 25, 2011). The idealized $V_s$ profiles for borings B-901, B-907, and B-909 provided in the response to question 02.05.02-3 were developed based on averaging the $V_s$ measurements from each respective borehole. These idealized $V_s$ profiles with lean concrete replacing Zone IIB and Zone III material (concrete fill) were used only in the sensitivity study provided in the question 02.05.02-3 response and were not used in the SRA for the Reactor Building Complex. Rather, the $V_s$ profiles shown in FSAR Figure 2.5-241a were developed for this structure as described in the response to question 02.05.02-3. The "Profile 1 and 2 Log-mean" $V_s$ profile shown in this figure was used as input to the SRA. Therefore, it would not be appropriate to include these idealized $V_s$ profiles in the FSAR.

Top of zone elevation information of borings B-901, B-907, and B-909 from FSAR Table 2.5-208 was used in the sensitivity analysis to determine the thickness of Zone III rock that is replaced with concrete fill, as indicated in Figure 8(b) in the response to question 02.05.02-3. The information provided in FSAR Table 2.5-208 (i.e., top of zone elevations (depths) and zone thicknesses) is primarily based on standard penetration test (SPT) N-values for the soil zones (I, IIA, and IIB) and on rock recovery and rock quality data (RQD) for the rock zones (III, III-IV, and IV) from all 93 borings, except that this information related to the five $V_s$ borings (B-901, B-907, B-909, M-10 and M-30) also considered $V_s$ measurement data.

The FSAR will be clarified to indicate that the $V_s$ values in FSAR Table 2.5-212 are best estimate values, and to describe the use of the information in FSAR Tables 2.5-208 and 2.5-212 in the development of $V_s$ profiles for seismic Category I structures.

## Proposed COLA Revision

FSAR Sections 2.5.4.2.3.a, 2.5.4.2.5.a, 2.5.4.4.4.b, 2.5.4.7.1.a, and Table 2.5-212 will be revised as indicated on the attached markups.

## 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.

Elevation 302 ft. Top of Zone IIB saprolite contours beneath the Unit 3 power block area are shown on Figure 2.5-212.

The overlying Zone IIA saprolites comprise, at the Unit 3 site, about 75 percent of the saprolitic materials on site. About 80 percent of the Zone IIA saprolites are classified as coarse grained (sands, silty sands), while the remainder are fine grained (clayey sands, sandy and clayey silts, and clays). The thickest Zone IIA deposit encountered in the Unit 3 borings was 94 ft while the median thickness was about 30 ft. The top of Zone IIA saprolite ranges from about Elevation 232 ft to Elevation 335 ft. Top of Zone IIA saprolite contours beneath the Unit 3 power block area are shown on Figure 2.5-213.

d. **Zone I and Fill**

For Unit 3 foundations, Zone I soils and existing fills will be excavated. Thus, they are not considered further here.

e. **Subsurface Profiles**

Figure 2.5-215 through Figure 2.5-220 illustrate typical subsurface profiles across the Unit 3 power block area. The locations of these profiles are shown in Figures 2.5-214 and 2.5-221. These profiles, with structure cross-sections added, are presented to illustrate foundation interfaces in Section 2.5.4.3. They also are used to illustrate the Unit 3 excavation in Section 2.5.4.5, and for bearing capacity considerations in Section 2.5.4.10.

### 2.5.4.2.3 Field Investigations

The borings, observation wells, and CPTs from the Unit 3 site exploration program are summarized in Table 2.5-205, Table 2.5-206, and Table 2.5-207, respectively. The elevations, depths and thicknesses of the subsurface zones observed from the individual borings are shown in Table 2.5-208. Geophysical surveys are described in Section 2.5.4.4.

The initial subsurface field investigation (900-series borings, observation wells, etc.) was performed during August through November 2006. Two supplemental subsurface investigations were performed later, one in September and early October 2009 (M-series borings) and the other in October 2009 (W-series borings). The W-series borings were labeled as Investigation Supplement No. 1 and the M-series borings were labeled as Investigation Supplement No. 2. Most of the initial investigation and all of the supplemental investigations were conducted in the power block area with the number and depth of investigation points conforming to the guidance provided in RG 1.132 (SSAR Reference 153). Additional exploration points were located outside the power block area, e.g., at the proposed locations for the cooling towers.

The Unit 3 exploration point locations in the power block area are shown in Figure 2.5-221. Borings from previous exploration programs are also shown. Exploration points outside the power block area are shown on Figure 2.5-222.

The scope of work and the special methods used to collect field data are listed below. The work was performed during the initial (900-series) investigation except as noted:

- 93 exploratory borings (MACTEC Engineering and Consulting, Raleigh, North Carolina) including 55 borings in the 900-series, and 10 W-series and 28 M-series borings
- 7 observation wells with permeability (slug) tests in 4 wells (MACTEC Engineering and Consulting, Raleigh, North Carolina, and Bedford Well Drilling, Bedford, Virginia)
- 4 packer tests (Miller Well Drilling, Hayesville, North Carolina, under MACTEC supervision)
- 23 CPTs plus 4 down-hole seismic cone tests and pore pressure dissipation tests in 4 CPTs (Gregg InSitu, Inc., Columbia, South Carolina)
- 6 test pits (MACTEC Engineering and Consulting, Raleigh, North Carolina)
- 5 sets of borehole geophysical logging and 5 sets of suspension P-S velocity logging (GEOVision, Corona, California) including 3 sets in the 900-series and 2 M-series sets
- 2 sets of electrical resistivity tests (MACTEC Engineering and Consulting, Raleigh, North Carolina)
- Survey of exploration points (McKim and Creed, Virginia Beach, Virginia) for all the investigations

The exploration program was performed using the guidance in RG 1.132 (SSAR Reference 153). The fieldwork was performed under an audited and approved QAP and work procedures developed specifically for the Unit 3 project. MACTEC Engineering and Consulting, contracted to Dominion to perform the subsurface investigation, worked under

MACTEC's Quality Assurance Plan that met the requirements of 10 CFR 50, Appendix B. This Plan included meeting the requirements of Subpart 2.20 of ASME NQA-1, 1994 edition (Reference 2.5-204).

The subsurface investigation and sample/core collection was directed by the MACTEC site manager who was on site at all times during the field operations. A Bechtel geotechnical engineer or geologist, along with a Dominion representative, was also on site continuously during these operations. MACTEC's QA/QC engineer was on site part of the time. The draft boring and well logs were prepared in the field by MACTEC geologists.

Sample and core storage and handling were in accordance with ASTM D 4220 (Reference 2.5-205). For the initial subsurface investigation, an on-site storage facility for soil samples and rock cores was established before the fieldwork began. This facility was in the limited access and climate controlled "A" Level area of the Units 1 and 2 warehouse. Samples and cores were stored either within a 12-ft square area surrounded by a 6-ft high chain link fence, or in an adjacent secured area. For the supplemental subsurface investigations, samples were sorted in an onsite lockable, climate controlled 20 ft by 8 ft trailer, with a high security door system and security bars over each window. Each sample and core in each storage area was logged into an inventory control system. Samples removed from the facility were noted in the sample inventory logbook. A chain-of-custody form was also completed for samples removed from the facility.

Details and results of the exploration program are contained in Appendices 2.5.4AA (900-series), 2.5.4BB (W-series), and 2.5.4CC (M-series). The borings, observation wells, CPTs and test pits are summarized below. The laboratory tests are summarized and the results presented in Section 2.5.4.2.4. The geophysical tests are summarized and the results presented in Section 2.5.4.4.

a. **Borings and Samples/Cores**

The 93 borings drilled ranged from 22 ft to 300 ft in depth. The 300-ft deep boring was drilled at the center of the R/B location, to about 215 ft depth in sound rock beneath the bottom of the basemat level. The borings were advanced in soil using rotary wash drilling techniques until standard penetration test (SPT) refusal (defined as 50 blows per 1 in. or less for start of rock coring) occurred. Steel casing was then set into the rock, and the holes were advanced using wireline rock coring equipment consisting of a 5-ft long "HQ" or "NQ" core barrel with a split inner barrel.

The soil was sampled using an SPT sampler at 2.5-ft intervals to about 15 ft depth and at 5-ft intervals below 15 ft. The SPT was performed using an automatic hammer, and was conducted in accordance with ASTM D 1586 (SSAR Reference 155). The recovered soil samples were visually described and classified by the onsite geologist. A selected portion of the soil sample was placed in a glass sample jar with a moisture-proof lid. The sample jars were labeled, placed in boxes, and transported to the on-site storage area.

A set of energy measurements was made on each of the automatic SPT hammers used by the drill rigs that performed the borings. The nine sets of energy measurements were made in accordance with ASTM D 4633 (Reference 2.5-206). The average energy transfer ratio (ETR) for each rig ranged from 75.2 percent to 87.4 percent, with an overall average of 82.1 percent. The N-values shown on the boring logs (Appendices 2.5.4AA, 2.5.4BB, and 2.5.4CC) and on the subsurface profiles (Figure 2.5-215 through Figure 2.5-220) are not adjusted for hammer energy. N-values used in engineering analysis (e.g., liquefaction analysis) are adjusted for hammer energy, i.e., $N_{60}$ was used in these situations.

Intact samples were obtained in accordance with ASTM D 1587 (Reference 2.5-220) using a Shelby tube sampler or a rotary Pitcher sampler. Upon sample retrieval, the disturbed portions at both ends of the tube were removed, both ends were trimmed square to establish an effective seal, and pocket penetrometer (PP) tests were performed on the trimmed lower end of the samples. Both ends of the sample were then sealed with hot wax, covered with plastic caps, and sealed once again using electrician tape and wax. The tubes were labeled and transported to the sample storage area. Intact samples are identified on the boring logs included in Appendix 2.5.4AA.

Rock coring was performed in accordance with ASTM D 2113 (SSAR Reference 156). After removal from the split inner barrel, the recovered rock was carefully placed in wooden core boxes. The onsite geologist visually described the core, noting the presence of joints and fractures, and distinguishing natural breaks from mechanical breaks. The geologist also computed the percentage recovery and the RQD.

Photographs of the cores were taken in the field. Filled and labeled core boxes were transported to the on-site sample storage facility.

The boring logs and the photographs of the rock cores are provided in Appendices 2.5.4AA, 2.5.4BB, and 2.5.4CC, along with details of the automatic hammer energy measurements. Borehole locations, depths, etc. are summarized in Table 2.5-205. The soil and rock materials encountered in the Unit 3 borings were similar to those found in the previous sets of borings conducted at the NAPS site. The elevations, depths and thicknesses of the subsurface zones observed from the individual borings are shown in Table 2.5-208. These values are based on SPT N-values, rock recovery values, and RQD depending on the zone for all 93 borings, except that the five $V_S$ borings (B-901, B-907, B-909, M-10, and M-30) also consider $V_S$ measurement data. Section 2.5.4.4.4 presents the results of $V_S$ measurements and summarizes the range of $V_S$ values for each zone of rock. Section 2.5.4.7.1 describes the use of $V_S$ ranges to determine top of zone elevations and thicknesses of subsurface zones.

b. **Observation Wells**

Each of the seven observation wells was installed adjacent to a sample boring. Three of the wells were screened in the soil/weathered rock zone, while four were screened in rock. Each well depth was selected in the field after a review of the borehole record. For the wells screened in rock, the screen depth was also based on the rock core description and packer test results. Boreholes for the wells in soil/weathered rock were advanced with hollow stem augers while the boreholes for all but one of the wells in rock were advanced using air-rotary drilling techniques. The borehole for the fourth well in rock (OW-951) was advanced with hollow stem augers until auger refusal, and was completed in rock using an “HQ” core barrel with a split inner barrel. This was after repeated cave-ins during attempts to advance the hole with air-rotary drilling.

After the designated depth of each well was reached, and the PVC screen and casing set, the sand pack and bentonite seal were placed, and then a grout plug was placed from the top of the bentonite seal to the ground surface. (In OW-951, a filter sock was placed over the screen, above which a formation packer and bentonite seal were set.) Each well was capped with a lockable steel cap and surrounded with a concrete pad.

during previous studies. In addition, chemical tests (for corrosiveness toward buried steel and aggressiveness toward buried concrete) and RCTS tests (for shear modulus and damping ratio variation with cyclic strain) were run on selected saprolite samples.

The details and results of the laboratory testing are included in Appendices 2.5.4AA and 2.5.4CC, except for the RCTS test results which are included in Appendix 2.5.4AAS1. Appendices 2.5.4AA and 2.5.4CC include references to the industry standards used for each specific laboratory test. The results of the tests on soil samples (excluding strength and RCTS tests) are summarized in Table 2.5-210. Table 2.5-211 gives the results of the unconfined compression tests on the rock cores. The results of the RCTS tests are shown in Figure 2.5-223.

The results of the laboratory tests as they relate to the engineering properties of the soil and rock are described in Section 2.5.4.2.5.

### 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.

Best estimate values for $V_s$ and $V_p$ 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, PS/Bs, and 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 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.

typically repeated every 1.65 ft or 3.3 ft as the probe is moved from the bottom of the borehole towards the ground. The elapsed time between arrivals of the waves at the geophone receivers is used to determine the average velocity of a 3.3 ft high column of soil or rock around the borehole. For QA, analysis is also performed on source-to-receiver data.

#### 2.5.4.4.3 Seismic Tests with Cone Penetrometer

The tests were performed at 5-ft intervals in C-902, C-916, C-921 and CPT-923. Shear waves were generated by striking a heavy beam adjacent to the CPT location. Only shear waves were generated. The wave arrival was recorded by a geophone attached near the bottom of the cone string. The results of these seismic CPTs are provided in Appendix 2.5.4AA, and discussed in Section 2.5.4.4.4.

#### 2.5.4.4.4 Results of Shear and Compression Wave Velocity Tests

a. **Soil**

The measurements of $V_s$ from suspension P-S logging and seismic CPT tests in the Zone IIA and Zone IIB saprolite (and top of Zone III weathered rock) are shown versus depth in Figure 2.5-227. The corresponding measurements of $V_p$, from the suspension P-S logging are shown in Figure 2.5-235. Low strain Poisson's ratio can be determined from a relationship between $V_s$ and $V_p$ (SSAR Reference 150). A plot of Poisson's ratio versus depth derived from the suspension P-S logging $V_s$ and $V_p$ measurements is shown in Figure 2.5-236. Note that on these plots, the Zone IIA saprolite extends to about 29 ft depth in boring B-909, to about 34 ft depth in boring M-30, to about 35 ft depth in borings B-901 and B-907, and to about 59 ft depth in boring M-10.

For the Zone IIA saprolite, the average $V_s$ generally increases with depth from around 500 fps at the ground surface to 1200 fps as it transitions to Zone IIB saprolite. The median value within the layer is about850 fps. This compares with a median of about 950 fps noted in the SSAR. The results of the compression wave tests in Zone IIA saprolite are fairly consistent at around1800 fps, while the low strain Poisson's ratio can be taken as 0.35.

For the Zone IIB saprolite, the average $V_s$ generally ranges from around 1200 fps to 2500 fps as it transitions to Zone III. The median value within the layer is about 1600 fps which is the same as noted in the SSAR. The results of the compression wave tests in Zone IIB saprolite in

Figure 2.5-235 reflect the $V_p$ of water. $V_p$ from SSAR Table 2.5-45 of 3500 fps was used, with a low strain Poisson's ratio of 0.37.

b. **Rock**

Figure 2.5-237 shows the measurements of $V_S$ from suspension P-S logging in the Zone III, Zone III-IV and Zone IV bedrock versus elevation. Figure 2.5-238 shows the corresponding measurements of $V_p$, while Figure 2.5-239 shows Poisson's ratio versus elevation derived from $V_S$ and $V_p$. These measurements were taken in the power block area, i.e., at the R/B (B-901), Auxiliary Building (A/B) (B-907 and B-909), at the east end of the UHSRS complex (M-30) and to the west of the UHSRS complex (M-10).

Based on a review of the $V_S$ versus elevation information in Figure 2.5-237, and the RQD data in Figure 2.5-224 as described in Section 2.5.4.2.5.a, it was concluded 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. For Zone III weathered rock, the range of $V_S$ is approximately 2000 fps to 4000 fps, with a BE value of 3000 fps. For Zone III-IV partially weathered rock, the range of $V_S$ is approximately 3000 fps to 8000 fps, with a BE value of 4500 fps. For Zone IV fresh rock, the range of $V_S$ is approximately 8000 fps to 11,000 fps, with a BE value of 9000 fps. These BE $V_S$ values are provided in Table 2.5-212 and represent a best estimate for each rock zone based on all five $V_S$ borings. Section 2.5.4.7.1 describes the development of $V_S$ profiles for seismic Category I structures based on the five individual $V_S$ profiles as shown in Figure 2.5-237.

In Figure 2.5-237, Zone IV bedrock extends consistently up to around Elevation 184 ft, although the shear wave velocity values indicate that Zone IV extends above this elevation in some of the borings, and well above it in M-30. Conversely, B-901 shows Zone III rock extending from this elevation up to about Elevation 205 ft before grading to Zone III-IV rock. From Elevation 205 ft to about Elevation 225 ft, all the borings show Zone III-IV, except for the two UHSRS borings – M-10 indicates Zone III while M-30 indicates Zone IV. Above about Elevation 225 ft, B-907 and B-909 show mostly Zone III and lower end and Zone III-IV rock material, while B-901 shows Zone III-IV rock and M-30 indicates mostly Zone IV rock.These $V_S$ profiles demonstrate that, whereas previously the "top of competent rock" was the top of the Zone III-IV (SSAR), the shear wave

### 2.5.4.7 Response of Soil and Rock to Dynamic Loading

The R/B basemat at Unit 3 is founded on Zone III-IV or Zone IV bedrock or on concrete placed on Zone III-IV or Zone IV bedrock, after removing Zone III weathered rock. (Although the cross-sections in Figures 2.5-229, 2.5-230 and 2.5-233 through the R/B show no excavation of the Zone IV bedrock, the top of the Zone IV in B-902 on the eastern edge of the R/B is at around Elevation 278 ft; thus, appreciable excavation of the Zone IV rock at this location will be needed to reach foundation level at Elevation 251 ft). A similar scheme is followed for the PS/B and PSFSV foundations, with all Zone III material immediately beneath the foundation being removed and replaced with concrete fill. For the other seismic category I structures (UHSRS, UHSRS pipe chase, and ESWPTs) all Zone II soil beneath the foundations will be removed and replaced with concrete fill, but Zone III material beneath the foundation will not be removed. The aforementioned foundation subgrades are illustrated in Figures 2.5-229 through 2.5-234.

The seismic acceleration at the sound bedrock level is amplified or attenuated up through the weathered rock and soil column. To estimate this amplification or attenuation, the following data are required:

- Shear wave velocity profiles of the rock and soil overlying hard rock
- Variation with strain of the shear modulus and damping values of the weathered rock and soil
- Site-specific seismic acceleration-time histories

#### 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 (B-901, B-907, B-909, M-10, and M-30) 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 thicknesses and top of zone elevations of Zone III, Zone III-IV, etc. based on Table 2.5-208) beneath the structure relative to the subsurface profile in the $V_S$ borehole location(s) as determined by $V_S$ data. The top elevations and thicknesses of rock zones can be estimated from both $V_S$ data and the use of the $V_S$ ranges for rock zones that are defined in Section 2.5.4.4.4.b.

**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

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**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 | 7,000* | 850 | 1,600 | 3,000 | 4,500 | 9,000 |
| Compression wave velocity, $V_p$* (ft/sec) | 2,400 | 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 |

* ~~BE Value~~ Best Estimate Values