
2.5.3 Surface Faulting

NAPS COL 2.0-28-A The information needed to address DCD COL Item 2.0-28-A is included in [SSAR Section 2.5.3](#), which is incorporated by reference with the following variance and supplements.

NAPS ESP VAR 2.0-4 The last sentence of the first paragraph of this SSAR section is replaced as follows.

Information contained in these sections is consistent with RG 1.208 ([Reference 2.5-226](#)), and is intended to satisfy 10 CFR 100.23, “Geologic and Seismic Siting Criteria” ([SSAR Reference 4](#)).

2.5.3.2.1 Chopawamsic and Spotsylvania Thrust Faults

The following is added to the end of the second paragraph of this SSAR section as follows.

Recent geologic mapping by Hughes and Hibbard ([Reference 2.5-246](#)) at a scale of 1:24,000 in the northern half of the Ferncliff, VA 7.5' quadrangle indicates that the Chopawamsic fault may not be folded as had been interpreted by previous investigators ([SSAR References 66 and 105](#)). Mapping by Hughes and Hibbard ([Reference 2.5-246](#)) also indicates that the Ellisville pluton appears to cross-cut the Chopawamsic fault suggesting it is younger than the sinistral motion and thrust faulting that occurred approximately 471-440 million years ago. This mapping simplifies the geometry of the Chopawamsic fault and has the effect of moving the surface trace of the fault further to the northwest ([Reference 2.5-246](#)) than had been mapped by previous investigators ([SSAR References 66 and 105](#)).

2.5.3.2.4 Other Faults Within the Chopawamsic Terrane

Two sentences are added to the end of the fourth paragraph of this SSAR section as follows.

The mainshock and several deep aftershock epicenters associated with the Mineral earthquake are located near the mapped Long Branch fault ([Figure 2.5.1-205](#)). However, the up-dip projection of the aftershock rupture surface places the location of potential surface rupture several miles west of the Long Branch fault ([Figures 2.5.1-203 and 2.5.1-205](#)).

NAPS COL 2.0-28-A

2.5.3.2.5 Unit 3 Subsurface Investigation

Borehole data, from the supplemental subsurface investigation described in [Section 2.5.4.3](#), were reviewed for evidence of major shear zones. No such evidence was exhibited by the borehole data.

2.5.3.4 Ages of Most Recent Deformations

Two paragraphs are added to the end of this SSAR section as follows.

The subsurface structure defined by aftershocks resulting from the Mineral earthquake is located beyond the 5-mile radius site area ([Figures 2.5.1-204](#) and [2.5.1-205](#)).

- No evidence of surface rupture, surface fault features, or geomorphic expression of surface rupture or coseismic surface tectonic deformation exists for the Mineral earthquake. Based on reconnaissance performed by other researchers immediately following the earthquake and the field reconnaissance of the epicentral region on April 19-21, 2012, the **M** 5.8 earthquake did not produce any discernible rupture or deformation at the ground surface.
- The Mineral earthquake does not appear to have occurred on a previously mapped fault.
- It has been suggested that the contact between the Chopawamsic Formation and the Ellisville pluton is faulted, based on offset dikes mapped in the region and is a possible candidate for the causative fault for the Mineral earthquake. Alternatively, the causative fault may represent slip on other geologic contacts or intra-formational structures in the epicentral area.

The earthquake epicenter, the surface projection of the aftershock rupture and any known potentially related faults are all located outside the 5-mile radius site area ([Figures 2.5.1-204](#), [2.5.1-205](#), and [2.5.1-206](#)).

2.5.3.8 Potential for Tectonic or Non-Tectonic Deformation at the Site

Two paragraphs are added to the end of this SSAR section as follows.

The potential for tectonic deformation at the Unit 3 site is negligible. Since the original site studies in the early 1970s no new information has

been reported to suggest the existence of any Quaternary surface faults or capable tectonic sources within the site area. The occurrence of the Mineral, Virginia earthquake on August 23, 2011 stimulated new research efforts in the epicentral region by government agencies and academic institutions, including detailed geologic mapping at scales of 1:24,000 compared with previous mapping at scales of 1:100,000. Dominion performed additional analyses following the event to characterize the earthquake and determine whether surface faulting had occurred. Preliminary results of analyses by Dominion and others are summarized in [Section 2.5.1.1.7](#). [Section 2.5.1.1.7](#) provides more detail of Dominion's analysis of lidar data and the results of a field reconnaissance following the earthquake. [Section 2.5.1.1.7](#) also includes the results of geomorphic desktop studies to assess the shapes of both stream and topographic profiles using lidar and NED elevations where lidar data were not available. In [Section 2.5.1.1.7](#), Dominion explicitly acknowledges that the Mineral earthquake occurred in a previously unmapped subsurface fault zone. In [Section 2.5.2.2](#) explanation is provided on why this fault zone is not a Repeated Large Magnitude Event (RLME) seismic source and how the Mineral earthquake is incorporated into the updated PSHA.

Continental glaciation during the Quaternary age, particularly the Wisconsinan ice sheet, extended no closer to the site than central Pennsylvania and so there is little likelihood for ice-shove faulting in the site area. Dominion is not aware of any reported tepee folds or other stress relief features in the site vicinity that might be the result of glacial rebound and there is little likelihood that such features can affect the site. Carbonate or evaporite rocks are not present in the metamorphic/igneous terrane in the site vicinity so the potential for collapse structures due to rock dissolution and karst formation is unlikely. As stated in the regional and site physiography sections ([SSAR Sections 2.5.1.1](#) and [2.5.1.2](#), respectively), the site is located in the Piedmont Physiographic Province. Therefore, the geologic setting is not conducive to the development of growth faults unlike the subsiding Gulf Coastal Plain province, for example, where such syndepositional features are common. The stratigraphic and geologic history investigations indicate that evaporite deposition has not occurred in the site vicinity; therefore salt migration does not present a hazard at the site. There are no Quaternary age volcanic centers in the 200-mile radius site region indicating little chance for a volcanic intrusion during the plant's operating period. No oil or gas resources are expected to be found in the site area's

metamorphic/igneous terrane. This would preclude subsidence due to the withdrawal of underground fluids or seismicity induced by such activities. No mines with commercial value exist within or adjacent to the site. This would preclude surface deformation due to the collapse of underground mines or passageways.

2.5.4 Stability of Subsurface Materials and Foundations

NAPS COL 2.0-29-A

The information needed to address DCD COL Item 2.0-29-A is included in [SSAR Section 2.5.4](#), which is incorporated by reference with the following supplements.

[SSAR Section 2.5.4](#) has been supplemented by integrating information on the additional Unit 3 borings into a single section with the same numbering as the SSAR.

2.5.4.1 Geologic Features

[SSAR Section 2.5.1.1](#) describes the regional geology, including regional physiography and geomorphology, regional geologic history, regional stratigraphy, and the regional tectonic setting. [SSAR Section 2.5.1.2](#) addresses site-specific geology and structural geology, including site physiography and geomorphology, site geologic history, site stratigraphy, site structural geology, and a site geologic hazard evaluation.

2.5.4.2 Properties of Subsurface Materials

2.5.4.2.1 Introduction

This section describes the static and dynamic engineering properties of the Unit 3 site subsurface materials. An overview of the subsurface profile and materials is given in [Section 2.5.4.2.2](#). The field investigations are described in [Section 2.5.4.2.3](#). The laboratory tests on soil and rock samples from the investigation and their results are presented in [Section 2.5.4.2.4](#). The engineering properties of the subsurface materials are given in [Section 2.5.4.2.5](#).

2.5.4.2.2 Description of Subsurface Materials

The following is a brief description of the subsurface materials, giving the soil and rock constituents, and their range of thicknesses encountered at the Unit 3 site. The information was taken from the 93 borings made at the site (outlined in [Section 2.5.4.2.3](#)). For reference, the existing site ground surface elevations in the areas explored range from about

Elevation 250 ft to Elevation 335 ft, with a median of about Elevation 296 ft. The design plant grade elevation for Unit 3 is Elevation 290 ft. Vertical datum is with reference to NAVD88 throughout Section 2.5.4, unless stated otherwise. Also, directions are with respect to Plant North, unless stated otherwise.

a. Zone III-IV and Zone IV Bedrock

The Unit 3 subsurface investigation ([Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)) describes the bedrock underlying the power block area mostly as quartz gneiss, biotite quartz gneiss, quartz biotite gneiss, or biotite gneiss. A detailed description of the bedrock is provided in [Section 2.5.1.2.3](#).

The top of Zone IV bedrock encountered in the borings made for Unit 3 ranges from about Elevation 171 ft to Elevation 278 ft. Top of Zone IV rock contours beneath the Unit 3 power block area are shown on [Figure 2.5.4-201](#). The top of Zone III-IV bedrock ranges from about Elevation 187 ft to Elevation 292 ft. Top of Zone III-IV rock contours beneath the Unit 3 power block area are shown on [Figure 2.5.4-202](#).

b. Zone III Weathered Rock

The top of Zone III bedrock encountered in the borings made for Unit 3 ranges from about Elevation 206 ft to Elevation 292 ft. The maximum thickness measured is about 77 ft. Top of Zone III rock contours beneath the Unit 3 power block area are shown on [Figure 2.5.4-203](#).

c. Zone IIA and IIB Saprolites

Distribution of Zone IIA and IIB saprolites varies throughout the Unit 3 site. The Zone IIB saprolites represent about 25 percent of the saprolites on site and are typically very dense silty sands with from 10 to 50 percent core stone. The thickest Zone IIB deposit encountered in the Unit 3 borings was 43 ft while the median thickness was about 8 ft. The top of Zone IIB saprolite encountered ranges from about Elevation 215 ft to Elevation 302 ft. Top of Zone IIB saprolite contours beneath the Unit 3 power block area are shown on [Figure 2.5.4-204](#).

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 2.5.4-205](#).

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

[Figures 2.5.4-207](#) through [2.5.4-216](#) illustrate typical subsurface profiles across the Unit 3 power block area. The locations of these profiles are shown in [Figure 2.5.4-206](#). 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 cone penetrometer tests (CPTs) from the Unit 3 site exploration program are summarized in [Tables 2.5.4-201](#), [2.5.4-202](#), and [2.5.4-203](#), respectively. The elevations, depths and thicknesses of the subsurface zones observed from the individual borings are shown in [Table 2.5.4-204](#). 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 2.5.4-217](#). Selected borings from previous exploration programs

are also shown. Exploration points outside the power block area are shown on [Figure 2.5.4-218](#).

The scope of work and the special methods used to collect field data are listed below:

- 93 exploratory borings (MACTEC Engineering and Consulting, Raleigh, North Carolina) including 55 borings in the 900-series, 10 W-series borings, 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 including 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 quality assurance program 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 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 stored in an on-site 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 Reactor Building (RB) 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](#)) 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. Undisturbed 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 2.5.4-201](#). 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 2.5.4-204](#). 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 shear wave velocity (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.

Each well was developed by pumping. Two or three standing well volumes of water were purged initially by pumping, cycling the pump on and off to create a surging effect. The well was considered developed when the pH and conductivity stabilized and the pumped water was reasonably free of suspended sediment.

Permeability tests were performed in each of the three wells screened in soil/weathered rock, and in one of the wells screened in rock (OW-949) in

accordance with ASTM D 4044, Section 8 ([SSAR Reference 157](#)) using a procedure that is commonly termed the slug test method. Slug testing involves establishing a static water level, lowering a solid cylinder (slug) into the well to cause an increase in water level in the well, and monitoring the time rate for the well water to return to the pre-test static level. The slug is then rapidly removed to lower the water level in the well, and the time rate for the water to recover to the pre-test static level is again measured. Electronic transducers and data loggers were used to measure the water levels and times during the test.

Permeability testing by the packer method was conducted in the borings adjacent to the four wells screened in rock. Test procedures used are described in ASTM D 4630 ([Reference 2.5-207](#)), as modified by U.S. Army Corps of Engineers in their Rock Testing Handbook ([Reference 2.5-208](#)) to use a manually read flowmeter rather than a digitally recorded one. The packer testing method, known as the constant head injection test, involved establishing and maintaining a constant pressure in the test length, measured by an electronic transducer, to determine the rate of inflow associated with maintaining the pressure.

[Appendix 2.5.4AA](#) contains the boring logs for the observation wells, the well installation records, the well development records, and the well permeability and packer test results. Observation well locations, depths, etc., are summarized in [Table 2.5.4-202](#).

c. Cone Penetrometer Tests

The 23 CPTs were advanced using a track-mounted 20 ton self-contained cone rig. Each CPT was advanced to refusal, to depths ranging from about 3 ft to 60 ft. Tip resistance, sleeve friction and porewater pressure were measured. The CPTs were performed in accordance with ASTM D 5778 ([SSAR Reference 158](#)). The pore pressure filter was located immediately behind the cone tip.

Down-hole seismic testing was performed at approximately 3 ft intervals in four of the CPTs (C-902, C-916, C-921 and C-923, see [Section 2.5.4.4](#)). One pore pressure dissipation test was performed in each of four CPTs (C-902, C-904b, C-911 and C-917) at depths ranging from about 13 ft to 29 ft.

The CPT logs, shear wave time of arrival records, and pore pressure versus time plots are contained in [Appendix 2.5.4AA](#). CPT locations, depths, etc., are summarized in [Table 2.5.4-203](#).

d. Test Pits

Six test pits were excavated to depths ranging from about 2 ft to 4.5 ft to obtain bulk samples of site soils to test for suitability as backfill. A rubber-tired backhoe was used to excavate the test pits. Bulk samples were collected in new 5 gallon plastic buckets. Small portions of the samples were placed in glass jars and sealed for moisture retention.

2.5.4.2.4 Laboratory Testing

Numerous laboratory tests of soil and rock samples were performed for Unit 3. The types and numbers of these tests are shown in [Table 2.5.4-205](#).

The laboratory testing investigation was performed in accordance with the guidance presented in RG 1.138 ([SSAR Reference 148](#)). The laboratory work was performed under an approved quality assurance program with work procedures developed specifically for the Unit 3 project. Soil and rock samples were shipped under chain-of-custody protection from the storage area (described in [Section 2.5.4.2.3](#)) to the testing laboratory. When required, samples sent to the testing laboratory were divided and/or shipped to an appropriate testing laboratory under chain-of-custody rules. Laboratory testing of soil and rock samples, except for chemical tests and resonant column torsional shear (RCTS) tests, was performed at the MACTEC laboratories in Charlotte and Raleigh, North Carolina and Atlanta, Georgia. Chemical testing for pH, sulfates and chlorides in selected soil samples was conducted by Severn Trent Laboratories in Earth City, Missouri. RCTS testing of selected soil samples was performed by Fugro Inc. in Houston, Texas, under the technical direction of Dr. K. H. Stokoe of the University of Texas in Austin.

Since the Unit 3 power block area is approximately 1500 ft southwest of the center of the Unit 2 Containment Building, the tests focused on verifying that the properties of the soil and rock beneath the Unit 3 power block area were similar to those beneath Units 1 and 2 as determined 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 [Tables 2.5.4-206a](#), [2.5.4-206b](#), and [2.5.4-206c](#). [Table 2.5.4-207](#) gives the results of the unconfined compression tests on the rock cores. The results of the RCTS tests are shown in [Figure 2.5.4-219](#).

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 2.5.4-208](#) 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 2.5.4-208](#) under the subheadings: a) rock properties, including concrete fill; b) soil properties, including structural fill; c) RCTS results; and d) chemical properties.

a. Rock and Concrete Fill Properties

Rock

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 2.5.4-220](#). 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 55 percent, 90 percent, and 98 percent, respectively.

The unconfined compressive strengths and unit weights in [Table 2.5.4-208](#) are based on the rock strength test results shown in [Table 2.5.4-207](#). The elastic modulus values are also based on the values shown in [Table 2.5.4-207](#). The shear modulus values are derived from the elastic modulus values using the Poisson's ratio values tabulated in [Table 2.5.4-208](#). 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 of V_S and compression wave velocity V_P in [Table 2.5.4-208](#) 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

All of the saprolitic soils will be excavated from within the temporary wall that will be constructed around the power block area. As stated in [Section 2.5.4.10](#), if Zone III rock is encountered at foundation subgrade level of the RB/FB, CB, or FWSC, it will be removed and replaced with concrete fill. The concrete fill around and below the RB/FB, CB, and FWSC will be lean and unreinforced, and will have a mean compressive strength of at least 2,500 psi, with a unit weight of 145 pcf and Poisson's ratio of 0.15. The bearing capacity of concrete fill is addressed in [Section 2.5.4.10.1](#).

The top of Zone III-IV rock contours superimposed on the structure footprints in [Figure 2.5.4-202](#) are used to estimate the thickness of Zone III rock that will be removed from beneath the RB/FB, CB or FWSC foundations and replaced with concrete fill. Up to 10 ft of concrete fill will be required under the northern edge of the RB, and up to 15 ft will be required under the southeast corner of the FB. Up to 30 ft of concrete fill

will be required under the southeast portion of the CB while over 50 ft of concrete fill will be needed beneath most of the FWSC. Typical thicknesses of concrete fill are illustrated on most of the subsurface profiles in [Figures 2.5.4-207](#) through [2.5.4-216](#). Analysis indicates that if the top 25 ft of rock beneath the RB/FB foundation is replaced with concrete fill, the seismic response at foundation level decreases with increasing V_s of the concrete fill. Based on the calculated log-mean V_s value at and below the RB/FB foundation (shown in [Figure 2.5.4-242](#)), the V_s of the in-situ rock at 25 ft below the RB/FB 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 7000 ft/sec.

Construction, QA, 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.

Construction and placement of the lean concrete fill meet the applicable provisions of ACI 207.1R ([Reference 2.5-410](#)) and associated requirements in the Unit 3 Quality Assurance Program Description (QAPD), Part III, Section 1 ([Appendix 17AA](#)). Testing of materials and lean concrete fill during construction will verify that the as-placed concrete properties are consistent with the properties established during the mix design. At a minimum, the construction verification and testing will include:

- Verification of manufacturer's certifications for cementitious materials
- Fresh concrete testing of slump and placement temperature
- Hardened concrete testing of compressive strength
- Aggregate testing for gradation, moisture content, and specific gravity
- Aggregate testing for deleterious expansive alkali-silica reaction in accordance with ASTM C1260 ([Reference 2.5-405](#)) and ASTM C1293 ([Reference 2.5-406](#)).

For bonded horizontal construction lift joints in the concrete fill, the bonding process will follow the requirements of ACI 207.1R. Horizontal lift joint surfaces will be prepared between lifts to ensure appropriate bonding between lifts, as described in ACI 207.1R. Before placement of the next lift, surface film and contamination will be removed to expose a fresh, clean mortar and aggregate surface.

Measures will be taken to cope with the generation of heat from hydration of the cement and attendant volume change to minimize cracking. A thermal control plan will be developed during detailed design to minimize the thermal cracking of the concrete fill based on a thermal study performed in accordance with ACI 207.1R and its referenced ACI publications and ASTM specifications. Selected measures regarding temperature control will be used during construction, as outlined in ACI 207.1R and include, but are not limited to, controls on cementitious material content, precooling (if necessary), and construction management.

By selecting an appropriate mix design, performing verification testing during construction, and following industry guidance for construction and placement methods, the mechanical properties of the lean concrete fill, including bonded horizontal construction joints for the FWSC monolithic block, are consistent with the properties used for the design analyses.

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 2.5.4-221](#) 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 (PI) 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 2.5.4-222](#). 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 friction angles (ϕ') ranging from 23 to 33 degrees, with a median of 30.8 degrees. The average effective cohesive (c') component from the Appendix 2C tests was 0.275 ksf. A series of C-U tests performed for the Unit 3 program gave effective internal friction angles ranging from about 31 to 36 degrees, with a median of 33 degrees, and very little effective cohesion. The values of $\phi' = 33$ degrees and $c' = 0.125$ ksf were adopted for the Zone IIA saprolite. This compares with $\phi' = 30$ degrees and $c' = 0.25$ ksf used in the SSAR.

A large amount of testing was performed after low unit weights were measured in the Zone IIA saprolites in the Units 1 and 2 Service Water Reservoir area. The testing details and results are given in UFSAR Appendix 3E, Attachment 4 ([SSAR Reference 5](#)). It was concluded that there are isolated lower densities, but these are not typical. UFSAR Table 3.8-13 ([SSAR Reference 5](#)) identifies 125 pcf as a design total unit weight. A value of 125 pcf is shown in [Table 2.5.4-208](#).

The V_S versus depth measured in the soil by suspension P-S velocity logging and CPT seismic testing during the Unit 3 field investigation are shown in [Figure 2.5.4-223](#). The average V_S ranges from about 500 ft/sec to 1200 ft/sec in the upper 40 ft, with a best estimate of about 850 ft/sec. This is presented in more detail in [Section 2.5.4.4](#) and [Section 2.5.4.7](#).

The high strain (i.e., in the range of 0.25 to 0.5 percent) elastic modulus value has been derived using the relationship with SPT N-value given in [SSAR Reference 151](#). The shear modulus value has been obtained from the elastic modulus values using the relationship between elastic modulus, shear modulus and Poisson's ratio ([SSAR Reference 150](#)). The best estimate low strain (i.e., 10^{-4} percent) shear modulus has been derived from the V_S of 850 ft/sec. The low strain elastic modulus value has been obtained from this shear modulus value using the relationship between elastic modulus, shear modulus, and Poisson's ratio ([SSAR Reference 150](#)).

In [Table 2.5.4-208](#), the value of unit coefficient of subgrade reaction is based on the value for medium dense sand provided by Terzaghi ([SSAR Reference 152](#)), while the earth pressure coefficients are Rankine values, assuming level backfill and a zero friction angle between the soil and the wall (see also [Section 2.5.4.10.3](#)).

All of the bulk samples obtained from the test pits were Zone IIA saprolite, since the test pits only sampled near-surface soils. Details of the results of the modified Proctor compaction tests and the California Bearing Ratio (CBR) tests run on these samples are provided in [Appendix 2.5.4AA](#). The maximum dry density ranged from about 100 pcf to 126 pcf, with a median value of 116 pcf. The corresponding optimum moisture content ranged from 9 to 22 percent, with a median value of 13 percent. A plot of molded dry density versus CBR (soaked samples) is given in [Figure 2.5.4-224](#).

Zone IIB Saprolite

Grain size curves from 19 sieve analyses of Zone IIB silty sand samples are shown in [Appendices 2.5.4AA](#) and [2.5.4CC](#). The samples had fines contents ranging from about 15 to 27 percent. These fines contents are shown versus depth in [Figure 2.5.4-221](#). The Zone IIB USCS classification is SM.

The measured SPT N-values from 206 tests ranged from 24 to refusal (defined as >100 blows/ft), with a median value of 75 blows/ft. These are plotted versus depth on [Figure 2.5.4-222](#). The N_{60} median value adjusted for individual hammer energy is 100 blows/ft. The effective angle of internal friction of a very dense sand ($N = 100$ blows/ft) would typically be taken as over 40 degrees ([SSAR Reference 150](#)). However, with the moderately high silt content, ϕ' has been limited to 40 degrees with $c' = 0$. The unit weight of 130 pcf reflects the very dense nature of the Zone IIB saprolite.

The shear wave velocities measured in the soil by suspension P-S velocity logging and CPT seismic testing during the Unit 3 field investigation are shown in [Figure 2.5.4-223](#). The average shear wave velocity ranges from about 1200 ft/sec to 2500 ft/sec with a best estimate of about 1600 ft/sec. This is presented in more detail in [Section 2.5.4.4](#) and [Section 2.5.4.7](#).

The high strain (i.e., in the range of 0.25 to 0.5 percent) elastic modulus value has been derived using the relationship with SPT N-value given in

[SSAR Reference 151](#). The shear modulus value has been obtained from the elastic modulus values using the relationship between elastic modulus, shear modulus and Poisson's ratio ([SSAR Reference 150](#)). The low strain (i.e., 10^{-4} percent) shear modulus has been derived from the best estimate shear wave velocity of 1600 ft/sec.

In [Table 2.5.4-208](#), the value of unit coefficient of subgrade reaction is based on the value for dense sand provided by Terzaghi ([SSAR Reference 152](#)). The earth pressure coefficients are Rankine values, assuming level backfill and a zero friction angle between the soil and the wall (see also [Section 2.5.4.10.3](#)).

Structural Fill

Structural fill for placing around major power block structures is obtained from crushing the sound rock removed from the deep excavation for some of these structures, including the RB/FB, CB, and Radwaste Building. If there is an insufficient quantity of on-site sources, similar material will be obtained from offsite. The rock is crushed down to well-graded, angular or sub-angular sand and gravel-sized particles conforming to the gradation of Size No. 21A in the Virginia Department of Transportation (VDOT) Road and Bridge Specifications ([SSAR Reference 166](#)). It is compacted with heavy equipment in thin lifts to a dry density that is at least 95 percent of the maximum dry density obtained from ASTM D 1557 ([SSAR Reference 165](#)) (see also [Section 2.5.4.5](#)). Based on this, $N_{60} = 50$ blows/ft and $\phi' = 40$ degrees were selected as reasonable and conservative.

Zone IIB saprolite material may be used as secondary structural fill as described in [Section 2.5.4.5.3](#).

Additional details about the structural fill and the laboratory and field testing programs proposed for the backfill are presented in [Section 2.5.4.5.3](#).

c. RCTS Testing

The results of the three RCTS tests are presented in [Appendix 2.5.4AAS1](#) and illustrated in [Figure 2.5.4-219](#). Two of the tests were on Zone IIA saprolites (each an SM sample, obtained using a Shelby tube) and one test was on a sample of Zone IIB saprolite (also SM, obtained using a rotary Pitcher barrel sampler). The test results on [Figure 2.5.4-219](#) show normalized shear modulus ($G/G_{\max}$) and material damping ratio, D , versus shear strain, for both the resonant column and

torsional shear modes. The results are shown for a confining pressure approximately equal to the in-situ confining pressure.

Comparison of the RCTS results with the generic curves used in the seismic soil column analyses is illustrated and discussed in [Section 2.5.4.7](#).

d. Electrical Resistivity and Chemical Properties

When assessing the corrosion potential of soils, electrical resistivity and selected chemical testing results are typically used in combination. Field electrical resistivity and laboratory chemical tests were performed on the Zone IIA and Zone IIB saprolites during the Unit 3 subsurface investigation, and the results of the tests are given in [Appendix 2.5.4AA](#). The results of the chemical tests are also shown in Table 2.5.4-206. The results are described in the following paragraphs.

Zone IIA Saprolite

The electrical resistivity measured in two arrays ranges from over 100 ohm-m close to the surface to around 500 ohm-m at 30 ft depth. The chloride content of the soil, measured in 15 tests, ranges from about ≤ 2 to 210 parts per million (ppm), with a median value of about 6 ppm. These results suggest very low corrosion potential. The pH, measured in 15 tests, ranges from 4.8 to 7, with a median of 5.8. These pH results indicate a higher corrosion potential than the resistivity or chloride results. The sulfate content measured in 15 tests ranges from about ≤ 3 to 11 ppm, indicating that no special sulfate resisting cement is required.

Zone IIB Saprolite

The electrical resistivity measured in two arrays was about 450 ohm-m at 50 ft depth. The chloride content, measured in 5 tests, is less than 10 ppm, while the pH ranges from 6.7 to 7.4. These results suggest very low corrosion potential. The sulfate content measured in 5 tests ranges from ≤ 2 to 9 ppm, indicating that no special sulfate resisting cement is required.

2.5.4.3 Foundation Interfaces

NAPS ESP COL 2.5-2

The locations of site exploration points for the Unit 3 subsurface investigation, including borings, observation wells, CPTs, electrical resistivity tests, and test pits made in the power block area are shown on [Figure 2.5.4-217](#). Borings from previous exploration programs are also

shown. Exploration points outside the power block area are shown on [Figure 2.5.4-218](#).

NAPS ESP COL 2.5-3

[Figure 2.5.4-206](#) shows the excavation plan for the safety-related and other major facilities. Also shown in [Figure 2.5.4-206](#) are the locations of the 10 subsurface profiles shown on [Figures 2.5.4-207](#) through [2.5.4-216](#). The cross sections of the structure foundations and the proposed excavation and backfilling limits are superimposed on [Figures 2.5.4-207](#) through [2.5.4-216](#) to produce [Figures 2.5.4-225](#) through [2.5.4-234](#).

NAPS COL 2.0-29-A

Logs of the core borings are in [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#), and logs of observation wells, CPTs and test pits are in [Appendix 2.5.4AA](#).

2.5.4.4 Geophysical Surveys

The geophysical testing for Unit 3 consisted of field electrical resistivity testing, geophysical down-hole testing, and seismic CPTs.

2.5.4.4.1 Field Electrical Resistivity Testing

Field electrical resistivity testing was conducted along two crossing lines located as shown on [Figure 2.5.4-217](#). The Wenner four-electrode method was used to perform the tests in accordance with ASTM G 57 ([Reference 2.5-210](#)). In this method, four electrodes, two for current and two for voltage, are spaced an equal distance apart and inserted about 1 ft into the ground. A current is sent through the two outer electrodes and voltage is measured at the two inner electrodes. Electrode spacing ("A" spacing) ranged from 3 ft to 100 ft. The results of the testing are given in [Appendix 2.5.4AA](#) and are described relative to corrosion potential in [Section 2.5.4.2.5.d](#).

2.5.4.4.2 Geophysical Down-Hole Testing

This suite of tests was performed in borings B-901 (300.0 ft depth), B-907 (200.5 ft depth), B-909 (201.9 ft depth), M-10 (201.9 ft depth), and M-30 (201.7 ft depth). The tests conducted were natural gamma, three arm caliper, resistivity, spontaneous potential, borehole acoustic televiewer logging, boring deviation, and suspension P-S velocity logging. The results of all of these tests and detailed descriptions of the test methods are in [Appendices 2.5.4AA](#) and [2.5.4CC](#). Plots of the V_S and V_P results versus depth are presented in [Section 2.5.4.4.4](#). The descriptions below

are summarized from the more detailed descriptions in [Appendices 2.5.4AA](#) and [2.5.4CC](#).

For all of the tests, all five borings were logged as partially-cased borings, filled with clear water or polymer-based drilling mud, with a 4-in diameter PVC or steel casing placed in the top 40 ft (B-901 and B-907), 44 ft (M-30), 80 ft (B-909), or 90 ft (M-10) of soil above bedrock contact during the measurements in the lower rock portions of the borings. The casing was then removed and measurements were performed in the upper soil portion of the borings. The instrument probe receives control signals from, and sends the digitized receiver signals to, instrumentation on the surface via an armored 4-conductor cable. The cable is wound onto the drum of a winch and is used to support the probe.

a. Natural Gamma and 3-Arm Caliper

Natural gamma and caliper data were collected using a Model 3ACS 3-leg caliper probe, manufactured by Robertson Geologging, Ltd. With this tool, caliper measurements were collected concurrently with the measurement of natural gamma emission from the borehole wall. The probe is 6.82 ft long and 1.5 in. in diameter and can:

- Measure boring diameter and volume
- Locate hard and soft formations
- Locate fissures, caving, pinching and casing damage
- Identify bed boundaries
- Correlate strata between borings
- Provide natural gamma measurements

Natural gamma measurements rely upon small quantities of radioactive material contained in all rocks to emit gamma radiation as they decay. The measurement is useful because the radioactive elements are concentrated in certain rock types, e.g., clay or shales, and depleted in others, e.g., sandstone or coal.

For testing, the probe was lowered to the bottom of the boring where the caliper legs were opened, and data collection was begun. The probe was returned to the surface at a rate of 10 ft/minute, collecting data continuously at 0.05 ft spacing.

b. Resistivity, Spontaneous Potential and Natural Gamma

Resistivity, spontaneous potential, and natural gamma data were collected using a Model ELXG electric log probe, manufactured by Robertson Geologging, Ltd. The probe, which is 8.2 ft long and 1.73 in. in diameter, measures single point resistance, short and long normal resistivity, spontaneous potential, and natural gamma, and can:

- Identify bed boundaries
- Correlate strata between borings
- Identify strata geometry (shale indication)
- Provide natural gamma measurements

For testing, the probe was lowered to the bottom of the boring, and data collection was begun. The probe was returned to the surface at a rate of 10 ft/minute, collecting data continuously at 0.05 ft spacing.

c. Acoustic Televiwer and Borehole Deviation Measurement

Acoustic image and boring deviation data were collected using a High Resolution Acoustic Televiwer probe, manufactured by Robertson Geologging, Ltd. The probe, which is 7.58 ft long and 1.9 in. in diameter, is fitted with upper and lower four-band centralizers, and can:

- Measure boring inclination and deviation from vertical
- Determine need to correct soil and geophysical log depths to true vertical depths
- Provide acoustic imaging of the borehole to identify fractures, dikes, and weathered zones, and determine dip and azimuth of these features

This system produces images of the borehole wall based on the amplitude and travel time of an ultrasonic beam reflected from the formation wall. The strength of the reflected signal from the formation wall depends primarily upon the impedance contrast between the clear water or drilling fluid and the wall. In the North Anna rock borings, the contrast between the fluid and the rock formation generally provided high contrast. The acoustic wave propagates along the axis of the probe and is then reflected perpendicular to this axis by a reflector that focuses the beam to a 0.1 in. diameter spot about 2 in. from the central axis of the probe. This reflector is able to rotate. During the survey, data were collected at 360 samples per revolution.

The probe contains a fluxgate magnetometer to monitor magnetic north, and all raw televiewer data are referenced to magnetic north. In addition, a 3-axis accelerometer is enclosed in the probe, and boring deviation data are recorded during the logging runs, to permit correction of structure dip angle from apparent dip to true dip in non-vertical borings.

For testing, the probe was lowered to the bottom of the boring, and data collection was begun. The probe was returned to the surface at a rate of 3 ft/minute, collecting data continuously at 0.008 ft intervals. The data were presented on a computer screen for operator review during the logging run, and stored on hard disk for later processing.

d. Suspension P-S Logger

Suspension soil and rock velocity measurements were performed using the Robertson Geologging USB Micrologger II digital recorder with a digital OYO Suspension P-S Logging Probe. This system directly determines the average in-situ horizontal V_S and V_P measurements of a 3.3 ft high segment of the soil and rock column surrounding the borehole by measuring the elapsed time between arrivals of a wave propagating upwards through the soil and rock column.

Suspension P-S velocity logging uses a 23-ft long cable suspended probe containing a source near the bottom, and two geophone receivers spaced 3.3 ft apart. The probe is lowered into the borehole to a specified depth where the source generates a pressure wave in the borehole fluid (drilling mud). The pressure wave is converted to seismic waves (P-wave and S-wave) at the borehole wall. At each receiver location, the P- and S-waves are converted to pressure waves in the fluid and received by the geophones mounted in the probe, which in turn send the data to a recorder on the surface. At each measurement depth, two opposite horizontal records and one vertical record are obtained. This procedure is 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 quality assurance, 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 2.5.4-223](#). The corresponding measurements of V_P from the suspension P-S logging are shown in [Figure 2.5.4-235](#). 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 2.5.4-236](#). 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-10, 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 shear wave velocity generally increases with depth from around 500 ft/sec at the ground surface to 1200 ft/sec as it transitions to Zone IIB saprolite. The median value within the layer is about 850 ft/sec. This compares with a median of about 950 ft/sec noted in the SSAR. The results of the compression wave tests in Zone IIA saprolite are fairly consistent at around 1800 ft/sec, 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 ft/sec to 2500 ft/sec as it transitions to Zone III. The median value within the layer is about 1600 ft/sec which is the same as noted in the SSAR. The results of the compression wave tests in Zone IIB saprolite in [Figure 2.5.4-235](#) reflect the V_P of water. The V_P from [SSAR Table 2.5-45](#) of 3500 ft/sec was used, with a low strain Poisson's ratio of 0.37.

b. Rock

[Figure 2.5.4-237](#) 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 2.5.4-238](#) shows the corresponding measurements of V_P , while [Figure 2.5.4-239](#) shows Poisson's ratio versus elevation derived from V_S and V_P . These measurements were taken in the power block area at the

R/B/FB (B-901 and B-907), and CB (B-909) and to the south of the power block area (M-10 and M-30).

Based on a review of the V_S versus elevation information in [Figure 2.5.4-237](#), and the RQD data in [Figure 2.5.4-220](#) as described in [Section 2.5.4.2.5.a](#), it was concluded that the overall V_S of the rock as defined by the three rock zones (III, III-IV and IV) is 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 ft/sec to 4000 ft/sec, with a best estimate value of 3000 ft/sec. For Zone III-IV partially weathered rock, the range of V_S is approximately 4000 ft/sec to 8000 ft/sec, with a best estimate value of 6000 ft/sec. For Zone IV fresh rock, the range of V_S is approximately 8000 ft/sec to 11000 ft/sec, with a best estimate value of 9000 ft/sec. These best estimate V_S values are provided in [Table 2.5.4-208](#) 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 proximity of the V_S measurements to the structure.

In [Figure 2.5.4-237](#), Zone IV bedrock extends consistently up to around Elevation 184 ft, although the V_S 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 borings to the south of the power block – 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 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 velocities in the Zone III rock can be high enough (e.g., in B-907) that, in some instances, Zone III can be included in the “competent rock” description. The V_S profiles also demonstrate, along with the RQD profile in [Figure 2.5.4-220](#), that above about Elevation 184 ft, weathered/fractured zones can be encountered; however, there is no pattern to where these zones occur, indicating the randomized process of weathering.

2.5.4.5 Excavation and Backfill

NAPS ESP COL 2.5-3

This section describes the following topics:

- The extent (horizontally and vertically) of Seismic Category I excavations, fills and slopes
- Excavation methods and stability
- Backfill sources, quantities, compaction specifications and quality control (QC)

2.5.4.5.1 Extent of Excavations, Fills and Slopes

[Figure 2.5.4-206](#), the excavation plan, shows the extent of excavations, fills and slopes for Unit 3. These are shown in cross-section in [Figures 2.5.4-225](#) through [2.5.4-234](#). To obtain the finish ground level grade of Elevation 289.5 ft, considerable quantities of soil will be excavated. The location of original ground surface is shown in the cross-sections. There are some lower areas to the northeast that will be backfilled. The total estimated cut within the power block area is about 625,380 cubic yards, while the amount of backfilling with compacted structural fill is about 241,750 cubic yards and concrete fill is about 109,620 cubic yards. Benched 3-horizontal to 1-vertical (3H:1V) slopes extend up from finish ground level grade around the southern and eastern perimeter of the area.

To the northeast of the Turbine Building, going towards the existing Units 1 and 2, ground surface elevation reduces at an approximately 5 percent slope down to Elevation 280 ft at the Service Water Building. Stormwater management pond #1 is between these structures and the boundary of Units 1 and 2 with yard grade at Elevation 270 ft. As existing grade falls off from the power block area northeast towards Units 1 and 2 some backfilling will be required to achieve final grade. As much as 30 ft of backfill will be needed to bring grade up to the planned ground surface area of the originally planned Units 3 and 4, where ground level is presently at around Elevation 250 ft.

[Figures 2.5.4-206](#) and [2.5.4-217](#) show the outline of the power block foundations. As shown in [Figures 2.5.4-225](#) through [2.5.4-234](#), temporary excavation for Unit 3 construction will be supported by a wall system. This wall will be supported by tie-backs or other means.

The excavation will be backfilled first with concrete fill up to the surface of Zone III rock and after that with structural fill up to the finished grade. The

south and the west supports of the excavation will be located at least 10.27 ft away from the RB/FB foundation. The lateral distance between the CB foundation and the north support of the excavation will be at least 12.24 ft. The east support of the excavation will be located at least 19.65 ft from the FWSC foundation.

2.5.4.5.2 Excavation Methods and Stability

a. Excavation in Soil

Excavation in the soils (Zones IIA and IIB) and any existing fills is achieved with conventional excavating equipment. Excavation of less than 20 ft in height will adhere to OSHA regulations ([SSAR Reference 162](#)). As noted in the previous section, a temporary vertical wall system will be used to support the power block excavation. The slopes around the perimeter of the power block area are no steeper than 3H to 1V, with a bench at approximately 25 ft height. Since the saprolitic soils can be highly erosive, even temporary slopes cut into the saprolite are sealed and protected.

b. Excavation in Rock

Excavation in the Zone III moderately to severely weathered rock is achieved using conventional earthmoving equipment. A temporary vertical wall system will be used to support the excavation, where necessary.

Excavation made for the originally planned Units 3 and 4 in the slightly to moderately weathered rock (Zone III-IV) and fresh to slightly weathered rock (Zone IV) is documented in [SSAR Reference 163](#). Techniques employed were similar to those used for Units 1 and 2 ([SSAR Reference 164](#)) but with “lessons learned” applied. The methods of rock excavation outlined below for Unit 3 are based, in part, on the methods that worked successfully for Units 1 and 2 and the originally planned Units 3 and 4. Unit 3 is approximately 1500 ft from the center of the Unit 2 R/B, whereas the originally planned Unit 3 R/B was only about 300 ft from the Unit 2 R/B. Thus, the following techniques to reduce

vibrations that worked for the originally planned Unit 3 will be used and will be effective for the new Unit 3:

- Controlled blasting techniques, including cushion blasting, pre-splitting and line drilling may be used, with appropriately dimensioned bench lifts. The blasted faces are vertical except where the foliation dip is into the excavation. There, the excavation may be parallel to the foliation dip (typically about 1-H to 1-V).
- Any blasting is strictly controlled to preserve the integrity of the rock outside the excavations and to prevent damage to existing structures, equipment, and freshly poured concrete. Peak particle velocity is measured and kept within specified limits that are a function of distance from the blast.
- The rock is reinforced to ensure adequate support and safety. Reinforcing includes installation of rock bolts in finished rock faces (typically at around 5 ft centers), and the use of welded wire mesh. Necessary measures are taken when weathered or fractured zones are encountered. Instrumentation such as slope indicators and extensometers are installed to monitor rock movements, especially on the foliation dip slopes.
- 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.

NAPS ESP PC 3.E(6)

NAPS ESP PC 3.E(5)

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 concrete fill or a sound, well-graded granular material. The anticipated extent of the concrete and granular fill is shown on the foundation cross-sections on [Figures 2.5.4-225 through 2.5.4-234](#). The concrete fill is used to replace any moderately to severely weathered rock (Zone III) exposed at the bottom of the excavations for the Seismic Category I RB/FB, CB, and FWSC foundation mats. As noted in

[Section 2.5.4.2.5.a](#), the concrete fill will have a minimum strength of 2500 psi, a best estimate shear wave velocity of 7000 ft/sec, 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 as well as around and beneath Seismic Category 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 to specifications for use as structural fill. The onsite source will consist of crushed Zone III, Zone III-IV, and/or Zone IV rock. 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 VDOT Road and Bridge Specifications ([SSAR Reference 166](#)). This gradation is shown in [Figure 2.5.4-240](#). The soundness of the aggregate will be confirmed using sulfate soundness and Los Angeles abrasion tests. This structural fill will be placed in lifts not exceeding 12 inches 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 content. Compaction will be performed with a heavy steel-drummed vibratory roller, except within 5 ft of a structure wall, where smaller compaction equipment will be used in conjunction with reduced lift thickness to minimize excess pressures against the wall. As noted in [Section 2.5.4.2.5.b](#), based on the type of material and its degree of compaction, $N_{60} = 50$ blows/ft and $\phi' = 40$ degrees were considered as reasonable and conservative for this structural fill.

Although proposed structural fill material from the site is not presently available, bulk samples of similar material will be obtained from a quarry in the site vicinity that crushes the native rock (sound gneiss or schist) to the VDOT Size 21A gradation. Laboratory tests will be used to confirm the properties of the structural fill, and will include:

- Confirmatory gradation tests
- Modified Proctor compaction tests to provide values of maximum density and optimum moisture content

- Consolidated-undrained (CU) triaxial compression tests, with porepressure measurements, on compacted samples at different confining pressures to verify the angle of internal friction

Since the gradation of the fill material falls within a relatively narrow band, the test results should be consistent, and so the number of laboratory tests can be limited. Two each of the modified Proctor and C-U triaxial tests should provide sufficient data.

As an alternative or supplement to the onsite crushed rock, dense-graded aggregate can be used as structural fill material. Dense-graded aggregate will conform to VDOT Size 21A ([SSAR Reference 166](#)) as noted in the previous paragraph.

Zone IIB saprolite material may be used as secondary structural fill under and around structures (excluding Seismic Category I and II structures) and under roads. This material may be used either on its own or mixed with excess finer materials from crushing Zone III, Zone III-IV, and Zone IV material.

Fill placement and compaction control procedures will be addressed in a technical specification that includes requirements for suitable fill, sufficient testing to address potential material variations, and in-place density testing frequency. Compacted structural fill placement and testing will follow the guidelines of ASME NQA-1 ([Reference 2.5-221](#)). At least one field density test will be performed per lift and for no more than every 250 cubic yards of fill placed. The technical specification also includes requirements for an on-site testing laboratory for QC (gradation, moisture-density, placement, compaction, etc.) and requirements to ensure that the fill operations conform to the earthwork specification. The soil testing firm is required to be independent of the earthwork contractor and to have an approved Quality Assurance Program. Sufficient laboratory compaction tests (modified Proctor) and grain size distribution tests will be performed to ensure that variations in the fill material are accounted for. (Variations in the crushed and blended rock are expected to be minimal.)

A test fill program is also included for the purposes of determining an optimum size of roller, number of passes, lift thickness, and other relevant data for achievement of the specified compaction.

2.5.4.5.4 Control of Groundwater During Excavation

Construction dewatering is presented in [Section 2.5.4.6.2](#). Since the saprolitic soils can be highly erosive, sumps and ditches constructed for dewatering are lined. The tops of excavations are sloped back to prevent runoff down the excavated slopes during heavy rainfall.

NAPS COL 2.0-29-A

2.5.4.6 Groundwater Conditions

2.5.4.6.1 Groundwater Measurements and Elevations

Groundwater is present in unconfined conditions in both the surficial sediments and underlying bedrock at the Unit 3 site. Seven observation wells installed for the Unit 3 investigation (along with nine wells installed at the site as part of the ESP subsurface investigation program) have exhibited groundwater levels ranging from about Elevation 238 ft to Elevation 314 ft between December 2002 and August 2007. (The groundwater generally occurs at depths ranging from about 18 ft to 25 ft below the present-day ground surface in the main Unit 3 power block area.)

The logs and details of these seven wells, and tests in the wells, are given in [Appendix 2.5.4AA](#). Details of measured groundwater levels and their fluctuations are given in [Section 2.4.12](#). Hydraulic conductivity values for the saprolite based on slug tests performed in eleven of the observation wells range from 0.25 ft/day to 9.9 ft/day, with a geometric mean value of 1.74 ft/day. The hydraulic conductivity of the underlying shallow bedrock as determined from slug tests performed in two of the wells and packer tests performed in one of the wells is estimated to range from about 0.5 ft/day to 6.3 ft/day, with a geometric mean value of 2.05 ft/day. Groundwater movement at the site is generally to the north and east, towards Lake Anna. A detailed description of groundwater conditions is provided in [Section 2.4.12](#).

Groundwater levels at the site require temporary dewatering of foundation excavations extending below the water table during construction of Unit 3. This construction dewatering is performed in a manner that minimizes drawdown effects on the surrounding environment. Drawdown effects are expected to be limited to the NAPS site. The relatively low permeability of the saprolite and underlying rock means that sumps and pumps should be sufficient for successful construction dewatering, as presented in [Section 2.5.4.6.2](#).

The maximum allowable ground water level for operation of the power block area of Unit 3 is Elevation 288 ft which is at 2 ft below design plant grade at Elevation 290 ft. [Section 2.4.12.4](#) indicates that the maximum predicted groundwater level in the power block area of Unit 3 increases from north to south, ranging from about Elevation 271 ft at the north end of the Turbine Building to about Elevation 282.5 ft at the south end of the RB/FB.

2.5.4.6.2 Construction Dewatering and Seepage

Dewatering for all major excavations is achieved by gravity-type systems.

a. Soils

Due to the relatively impermeable nature of even the coarse-grained saprolite, sump-pumping of ditches is adequate to dewater the soil. These ditches are advanced below the progressing excavation grade.

During the construction of Units 1 and 2 and originally planned Units 3 and 4, plant excavation and dewatering was significant in causing local groundwater levels to decline. However, the extent of the area of influence of the construction dewatering was estimated to be a radius of less than 500 ft due to the low permeability of the materials being dewatered ([SSAR Reference 164](#)).

b. Rock

Sump-pumping is used to collect water from relief drains that are installed in the major rock excavation walls to prevent hydrostatic pressure buildup behind the walls. Such relief wells were spaced on 20 ft centers around the perimeters of the originally planned Units 3 and 4 containment excavations.

Although an approximately 40 ft head existed between excavation grade and the North Anna Reservoir during the final stages of excavation for the originally planned Units 3 and 4, no dewatering difficulties were encountered, due to the tight nature of the joints in the rock below about Elevation 240 ft. The excavation for Unit 3 is at least 1000 ft from Lake Anna, and so negligible seepage effects from the lake are anticipated.

2.5.4.6.3 Effect of Groundwater Conditions on Foundation Stability

NAPS ESP COL 2.5-4

Maximum allowable groundwater level is at least 2 ft below design plant grade, i.e., Elevation 288 ft. A water level at Elevation 290 ft was conservatively used in bearing capacity and settlement analyses and in

computing hydrostatic pressures on the buried structure walls ([Section 2.5.4.10](#)). As described in [Section 2.5.4.10](#), there are no buoyancy issues with deep buried structures because of the appreciable dead loads imposed by these structures. Large diameter buried piping such as the circulating water pipes are designed to resist buoyancy when empty.

No permanent dewatering system is required for Unit 3.

NAPS COL 2.0-29-A

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

The RB/FB common 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. A similar scheme is followed for the CB and FWSC foundations. The foregoing foundation subgrades are illustrated on [Figures 2.5.4-225](#) through [2.5.4-234](#).

The seismic acceleration at the hard bedrock ($V_S > 9200$ ft/sec) level is amplified or attenuated up through the rock column, or the 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 fill, the Zone III weathered rock, the concrete fill, the Zone III-IV slightly to moderately weathered rock, and the Zone IV slightly weathered to fresh rock.

V_S of the bedrock at the RB/FB basemat (B-901 and B-907) and the CB (B-909) as well as the two profiles south of the power block area (M-10 and M-30) are shown versus elevation in [Figure 2.5.4-237](#). Borings M-10 and M-30 are not considered further here because they are a significant distance from the power block structures.

The V_S profiles from B-901, B-907, and B-909 were used to develop the best estimate V_S profiles beneath the Seismic Category I RB/FB, CB,

and FWSC structures. The bottom of foundation elevation for these structures is Elevation 224 ft, Elevation 241 ft, and Elevation 282 ft, respectively.

The Unit 3 site is well characterized and extensively investigated with abundant high-quality data which reduces epistemic uncertainty in the site properties. These data also provide information to characterize the aleatory variation in layer thickness and shear-wave velocity across the site. These variations will be included in considerations of aleatory uncertainties for the best estimate profile during the randomization process. No alternate profiles are considered because of the relative insignificance of epistemic uncertainty with respect to the aleatory variability for this site.

a. Reactor Building/Fuel Building

B-901 and B-907 are beneath the RB/FB footprint and B-909 is under the footprint of the adjacent CB. The V_S measurements from all three of these borings were used in developing the best estimate V_S profile for the RB/FB. [Figure 2.5.4-241](#) 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 (Profile 1) and the other the mostly unweathered and unfractured profile (Profile 2). The log mean of Profile 1 and Profile 2 is used as the best estimate profile for the RB/FB up to the average top of rock at about Elevation 273 ft. The best estimate in-situ saprolite V_S profile that extends from Elevation 273 ft to grade at about Elevation 290 ft is the average profile measured in the saprolite at the Unit 3 site and reflects the fact that the original ground surface was significantly higher in most places than Elevation 290 ft. The best estimate profile used for the RB/FB is shown in [Figure 2.5.4-242](#).

b. Control Building

Since boring B-909 is directly below the footprint of the CB, the V_S profile from that boring is used as the best estimate V_S profile for the CB, and is shown in [Figure 2.5.4-242](#). As can be seen from the figure, the V_S values are generally averaged over 10-ft intervals.

c. Firewater Service Complex

The FWSC is counted as a surface foundation, with the bottom of foundation approximately 8 ft below grade, at Elevation 282 ft. The soil above this elevation is ignored when developing the V_S profile. Concrete

fill will be used between the bottom of the foundation and the top of Zone III-IV material. This concrete fill is of limited lateral extent and is not used in developing the V_S . The free-field profile in the soil surrounding the structure is used above rock.

Since boring B-909 is located less than 50 ft from the edge of the FWSC, the V_S profile from that boring is used as the basis for the best estimate V_S profile for the FWSC. The average thicknesses of the various zones beneath the FWSC based on borings beneath and adjacent to the FWSC footprint differ somewhat from the zone thicknesses encountered in B-909. The V_S values from B-909 were used for the FWSC but the thickness of the zones in the FWSC profile reflect the average zone thickness beneath the structure. Thus, the best estimate V_S profile shown in [Figure 2.5.4-243](#) for the FWSC is not identical to the V_S profile for the CB in [Figure 2.5.4-242](#).

The FWSC is surrounded by approximately 20 to 50 ft of structural fill, as shown in the Section D-D Profile ([Figure 2.5.4-228](#)). The best estimate V_S profile for the backfill is included in [Figure 2.5.4-243](#). The V_S of the backfill is similar to the V_S of the saprolite used in the best estimate profile.

d. Profile for Slope

In addition to the V_S profiles considered for the Seismic Category I structures, a V_S profile beyond the excavation for the power block is developed. This profile is employed to obtain peak ground accelerations in the free-field for use in slope stability ([Section 2.5.5](#)) and liquefaction analysis ([Section 2.5.4.8](#)). The profile is in the vicinity of boring B-947 and CPT C-916, on the planned 3H:1V slope to the east of the power block area. The ground elevation for the profile is Elevation 315 ft. The borings show the Zone II saprolite is thick in this area, with the combined thickness of the Zone IIA and Zone IIB materials being about 60 ft. The V_S profile developed for the saprolite takes into account V_S measured in C-916, the N-values measured in B-947, and the average V_S values derived from V_S measurements in saprolite sitewide. The V_S profile is shown in [Figure 2.5.4-244](#). The V_S profile for the rock below 60 ft depth is the log mean profile derived from Profile 1 and Profile 2 (for the RB/FB from borings B-901, B-907 and B-909), adjusted to take into account the lower elevation of the top of rock in B-947, and maintaining the transition to hard rock at Elevation 184 ft.

e. Profile for Structural Fill

A V_S profile is also developed for the structural fill that will be used as backfill around the Seismic Category I structures. As noted in [Section 2.5.4.5.3](#), the primary source of structural fill is crushed rock obtained from the power block excavation.

For this structural fill, there are no measured V_S values, since the backfill will be crushed rock obtained from the new plant excavation. To obtain a V_S profile range, the SPT N-value selected in [Section 2.5.4.2.5.b](#) for the structural fill, i.e., $N_{60} = 50$ blows/ft, is used. Relationships between N-value (adjusted for overburden pressure) and V_S developed by Seed, et al. ([Reference 2.5-212](#)) and Imai and Tonoucci ([Reference 2.5-213](#)) are used to obtain a profile of V_S versus depth, as shown in [Figure 2.5.4-245](#). This profile is averaged vertically in 5-ft intervals to obtain the average VS profile shown in [Figure 2.5.4-246](#) for P-SHAKE input. The upper and lower bound values shown in this figure are 1.414 and 0.707 times the mean value of shear wave velocity, respectively, which correspond to 2.0 and 0.5 times the shear modulus.

[Table 2.0-201](#) provides an evaluation of DCD site parameter values and corresponding Unit 3 site characteristic values for shear wave velocity.

NAPS COL 2.0-29-A

2.5.4.7.2 **Variation of Shear Modulus and Damping with Strain**

a. Shear Modulus

The shear modulus reduction curve for the Zone IIA saprolite is the same as used for the Zone IIA saprolite in the SSAR, i.e., Curve 1 in [SSAR Figure 2.5-63](#). This curve is reproduced here in [Figure 2.5.4-247](#), labeled “Recommended for Natural Soil.” A series of grain size tests on the Zone IIB saprolite indicated that all of the samples tested were sands, with no appreciable gravel content. Thus, Curve 1 in [SSAR Figure 2.5-63](#) was also used for the Zone IIB saprolite, and labeled “Recommended for Natural Soil” in [Figure 2.5.4-247](#). The typical thickness of the saprolite is about 35 ft. Curve 1 is almost identical to the average of the EPRI curves ([SSAR Reference 170](#)) for depths 0 to 20 ft, and 20 to 50 ft.

The results of the RCTS tests (normalized shear modulus (G/G_{max}) versus shear strain) from [Figure 2.5.4-219](#) are superimposed on Curve 1 in [Figure 2.5.4-248](#). These results show good agreement with Curve 1, and so no additional SHAKE runs were made using the RCTS shear modulus reduction curves. Note that the median thickness of the Zone IIA saprolite encountered in the Unit 3 borings was about 25 ft, and

approximately 80 percent of the material was classified as silty sand (SM). The two silty sand samples of Zone IIA saprolite tested in RCTS are thus considered sufficient and representative. Similarly, the median thickness of the Zone IIB saprolite encountered in the Unit 3 borings was about 9 ft, and all of this material was classified as silty sand (SM). Thus the sample of Zone IIB silty sand tested in RCTS is considered sufficient and representative.

As noted in [Section 2.5.4.2.5.b](#), the primary source of structural fill is bedrock excavated to construct the Unit 3 power block, crushed down to well-graded, angular sand and gravel-sized particles. Curve 2 in [SSAR Figure 2.5-63](#), which was derived for a gravel-type material, was selected as the shear modulus reduction curve for this structural fill and is included in [Figure 2.5.4-248](#). Curve 3 in [SSAR Figure 2.5-63](#) was used for the Zone III weathered rock. The shear modulus of the Zone IV and Zone III-IV weathered rock was considered non-strain dependent.

b. Damping

The typical thickness of the saprolite and the structural fill is about 35 ft. For the granular materials (Zone IIA and Zone IIB saprolite, and the structural fill), the average of the EPRI curves ([SSAR Reference 170](#)) for depths 0 to 20 ft, and 20 to 50 ft was selected. This curve is shown on [Figure 2.5.4-249](#). Curve 3 in [SSAR Figure 2.5-64](#) is used for the Zone III weathered rock. This curve is also shown on [Figure 2.5.4-249](#).

[Figure 2.5.4-248](#) shows the results of the RCTS tests from [Figure 2.5.4-219](#) for material damping ratio D versus shear strain superimposed on the granular soils curve from [Figure 2.5.4-249](#). These results show reasonable agreement, and so no additional SHAKE runs were made using the RCTS damping ratio reduction curves.

There is no variation of damping ratio of the Zone III-IV or Zone IV rock with cyclic shear strain. However, this rock has some intrinsic damping properties. A value of 1 percent was selected for the damping ratio with a variation of about ± 0.5 percent.

2.5.4.7.3 Site Specific Acceleration-Time Histories

[Section 2.5.4.7.1](#) used the P-SHAKE computer program ([Reference 2.5-222](#)) which does not use acceleration-time histories for input.

2.5.4.7.4 Rock and Soil Column Amplification/Attenuation Analysis

NAPS ESP COL 2.5-5

The ARS derived from the P-SHAKE analyses for the Seismic Category I structures described in [Section 2.5.4.7.1](#) are presented in [Section 3.7.1](#).

The P-SHAKE program is used to obtain peak ground accelerations in the free-field for use in slope stability ([Section 2.5.5](#)) and liquefaction analysis ([Section 2.5.4.8](#)) using the V_S profile described in [Section 2.5.4.7.1.d](#). [Figure 2.5.4-250](#) shows the maximum acceleration versus depth profiles obtained from P-SHAKE for the high and low frequency earthquakes. The peak ground acceleration occur at about 42 ft depth and are about 0.56g for the high frequency earthquake and about 0.31g for the low frequency earthquake.

The accelerations shown in [Figure 2.5.4-250](#) were obtained using input rock motions consistent with the EPRI (2004, 2006) ([References 2.5-224, 2.5-225](#)) ground motion model (GMM). These input rock motions were subsequently updated based on the EPRI (2013) ([Reference 2.5-407](#)) GMM for use in developing the GMRS and FIRS for all Seismic Category I buildings, as discussed in [Sections 2.5.2.5](#) and [2.5.2.6](#). The updated input rock motions were not used to update the liquefaction, seismic lateral earth pressure, or pseudo-static slope stability analyses presented in [Sections 2.5.4.8, 2.5.4.10, and 2.5.5.2.3.b](#), respectively. These updated rock motions are lower than those from the EPRI (2004, 2006) GMM and result in lower site ground motion response spectra, and correspondingly lower peak ground accelerations for low frequency and high frequency input motions. Therefore, [Figure 2.5.4-250](#) is not updated because the accelerations shown on the figure are conservative.

NAPS COL 2.0-29-A

2.5.4.8 Liquefaction Potential

The Zone IIB saprolitic soils are extremely dense and the Zone III weathered rock has over 50 percent core stone and has typically been sampled by rock coring. Neither of these materials has liquefaction potential. The primary source of structural fill is bedrock excavated for the Unit 3 power block. This consists of crushed angular or sub-angular sand and gravel-sized particles compacted in thin lifts with a heavy vibratory steel-drummed roller. This fill is not liquefiable. The only material analyzed here for liquefaction is the Zone IIA saprolitic soil.

NAPS ESP PC 3.E(7)

All of the Seismic Category I structures are founded on rock or on concrete fill on rock at the Unit 3 site. Thus, even if the Zone IIA saprolite

is liquefiable, such liquefaction does not impact the stability of any Seismic Category I structure. As described in [Section 2.5.4.10](#), the Zone IIA saprolite has relatively high resistance to bearing failure but can produce excessive settlements under certain conditions. Thus, the Zone IIA saprolite is not used to support Seismic Category I structures, whether it is potentially liquefiable or not.

The peak ground accelerations obtained from the Unit 3 P-SHAKE analyses through the natural soil profile are less than those reported in the SSAR, due to some slightly different rock and soil profiles, and the randomization process applied to these profiles. The previous liquefaction analyses are in [Section 2.5.4.8.1](#). [Section 2.5.4.8.2](#) presents the results of summarized liquefaction analyses performed on Zone IIA saprolites on slopes outside the power block area, based on borings and CPTs performed for Unit 3 in these areas, i.e., analyses of soils that will generally not be excavated.

Any failure of slopes due to liquefaction could impact adjacent safety-related structures. Locations having this potential are identified, and a liquefaction analysis of the slope soils is performed.

2.5.4.8.1 Previous Liquefaction Analyses Performed

The SSAR describes liquefaction analyses performed previously for the North Anna site. The primary analysis was performed for a seismic margin assessment. This included, for the main Units 1 and 2 plant area, an analysis using a version of the simplified procedure and a threshold strain analysis. For the Units 1 and 2 Service Water Reservoir, an analysis was performed based on the results of 15 stress-controlled cyclic triaxial tests. An updated seismic margin assessment was performed for the SSAR maintaining the same assumptions as used in the original study but substituting the ESP design accelerations and moment magnitudes. As noted previously, these accelerations were higher than those being used for the present analysis ([Section 2.5.4.8.2](#)). The SSAR also describes liquefaction analysis performed using subsurface data gathered for the ESP investigation.

2.5.4.8.2 Liquefaction Analyses Performed for Unit 3

This section was developed in accordance with, and conforms to guidance in RG 1.198 ([Reference 2.5-214](#)).

As noted earlier, at the locations of the majority of the borings and CPTs in the power block area that contains the Seismic Category I structures,

the Zone IIA saprolite will be excavated. Thus, analyzing the liquefaction potential of these soils prior to excavation is of little relevance.

The liquefaction analysis focuses on slopes whose failure could impact safety-related structures. The excavation plan ([Figure 2.5.4-206](#)) shows the cut slopes to the east and south of the power block are in the 30 to 35-ft high range. The bottom of the slope to the east of the FWSC is approximately 120 ft from the structures. This is the nearest point of the bottom of any slope to a Seismic Category I structure. The Seismic Category II Ancillary Diesel Building is slightly farther from the south slope. The liquefaction potential of the east and south slope soils was computed, as described below. Note that even if these slopes failed due to liquefaction (or other mechanisms) it is extremely unlikely the failed 35-ft high slopes would have any impact on structures 120 ft or more from the slope.

Six SPT sample borings were conducted on the east and south slopes (B-930, B-947, M-6, M-9, M-10 and M-11) along with two CPTs (C-909 and C-916). V_S measurements were taken in M-10 and C-916. The results from these explorations were analyzed to determine the liquefaction potential of the slope soils.

The analysis conservatively ignored mineralogy/fabric effects of the saprolite. The saprolite is estimated to be between 0.8 and 1.6 million years old, according to [SSAR Reference 176](#). A conservative age factor of 1.4 was used in the analysis. Cohesive samples and/or samples above the groundwater table were considered non-susceptible to liquefaction.

The analysis followed the methods proposed by Youd, et al. ([SSAR Reference 178](#)). This state-of-the-art liquefaction methodology is based on the evolution of the Seed and Idriss "Simplified Procedure" since the early 1980s. Magnitude scaling factors of 1.05 and 1.93 were used in the analysis for the moment magnitude 7.4 (low frequency) and 6.0 (high frequency) earthquakes, respectively. The K_σ factor for high overburden pressures was incorporated into the analysis, using a relative density of 60 percent.

The analysis of the SPT results from the six borings gave factor of safety values against liquefaction greater than 1.1 for the approximately 80 Zone IIA saprolite samples that were potentially liquefiable, except for eight samples. For the two CPTs, the liquefaction analysis showed no measurement where the factor of safety against liquefaction was less

than 1.1. For the two sets of V_S measurements, one measurement in C-916 indicated a factor of safety of less than 1.1 against liquefaction. In M-10, seven measurements indicated a factor of safety of less than 1.1 against liquefaction. Youd et al. ([SSAR Reference 178](#)) express concerns about the applicability of V_S measurements to liquefaction potential evaluation because V_S is a small strain measurement whereas pore-water pressure buildup and the onset of liquefaction are medium- to high-strain phenomena.

The liquefaction analyses described above for Unit 3 used accelerations obtained from input rock motions consistent with the EPRI (2004, 2006) GMM. As noted in [Section 2.5.4.7.4](#), these input motions were subsequently updated based on the EPRI (2013) GMM for use in developing the GMRS and FIRS for Seismic Category I buildings. These updated rock motions are lower than those from the EPRI (2004, 2006) GMM and result in lower site ground motion response spectra, and correspondingly lower peak ground accelerations for low frequency and high frequency input motions. Furthermore, the controlling earthquake magnitudes for low frequency and high frequency input motions are either reduced or remain unchanged in the EPRI (2013) GMM rock motions. Thus, the results of the liquefaction analyses described here, which are based on the EPRI (2004, 2006) GMM, can be conservatively used in place of the EPRI (2013) GMM, and are therefore not updated.

Using the method outlined in Tokimatsu and Seed ([SSAR Reference 179](#)), the maximum estimated dynamic settlement of the Zone IIA saprolite due to earthquake shaking was less than the 5 in. estimated based on soil encountered in one of the CPTs performed for the ESP investigation using the same computation method. This value of 5 in. is conservatively adopted as the maximum dynamic settlement that could occur in the saprolite due to the design seismic event.

2.5.4.8.3 Conclusions about Liquefaction

Only the Zone IIA saprolites fall into the gradation and relative density categories where liquefaction would be considered possible.

Any liquefaction of the Zone IIA saprolite will not impact the stability of any Seismic Category I structure.

The conclusions from the foregoing sections on the analysis of liquefaction potential of Zone IIA saprolite are as follows. These conclusions apply to measurements taken on or close to the slopes to the

east and south of the power block area. Note that all of the analysis neglected the beneficial effects against liquefaction of structure, fabric, and mineralogy. A conservative factor was adopted for age effects.

- A state-of-the-art liquefaction analysis of the SPT measurements gave factor of safety values against liquefaction greater than 1.1 for those samples that were potentially liquefiable, except for eight samples
- A state-of-the-art liquefaction analysis of the CPT measurements showed the factor of safety against liquefaction was greater than 1.1 for all measurements
- A state-of-the-art liquefaction analysis of the V_S measurements gave factor of safety values against liquefaction greater than 1.1 for all except eight measurements
- Maximum dynamic settlement of the Zone IIA saprolite due to earthquake shaking is conservatively estimated as about 5 in. based on the maximum value obtained from the ESP investigation. This settlement will be outside the zone of loading influence of any of the Seismic Category I structures

Based on the above analysis results, it can be concluded that a very small percentage of the Zone IIA saprolitic soils have a potential for liquefaction based on the low and high frequency Unit 3 seismic characteristics. The liquefaction analysis did not take into account the beneficial effects of structure, fabric, and mineralogy, and adopted a conservative factor for age; thus the chances of any liquefaction occurring are extremely low. Any liquefaction of the Zone IIA saprolite that does occur will not impact the stability of any Unit 3 Seismic Category I structure.

2.5.4.9 Earthquake Design Basis

See [Sections 2.5.2](#) and [3.7.1](#) for the GMRS and FIRS, respectively.

2.5.4.10 Static Stability

NAPS ESP COL 2.5-6

As with the Units 1 and 2, and the originally planned Units 3 and 4, the Unit 3 RB/FB is founded on Zone III-IV or Zone IV bedrock. If Zone III weathered rock or fractured rock is encountered at foundation subgrade level, then it will be removed and replaced with concrete fill. The subgrade of the other Seismic Category I structures and the Seismic Category II structures depends on their elevation and location.

[Table 2.5.4-209](#) shows the bottom of foundation elevations and depths for the Seismic Category I structures (RB/FB, CB, FWSC), the Seismic Category II structures (Turbine Building, Service Building and Ancillary Diesel Building), and the Radwaste Building. The foundation dimensions are also included in [Table 2.5.4-209](#). The cross-sections in [Figures 2.5.4-225](#) through [2.5.4-234](#) show the materials supporting these structures (except for the Ancillary Diesel Building). The average subsurface profiles beneath the Seismic Category I structures used for bearing capacity and settlement analyses are shown on [Figure 2.5.4-251](#). The corresponding profiles beneath the Seismic Category II structures and the Radwaste Building are shown on [Figure 2.5.4-252](#). There may be several materials immediately beneath the foundations of the larger structures (e.g., the Turbine Building) because of the variable stratigraphy and the different depths of the parts of the building, and because any Zone IIA saprolite beneath the shallow Seismic Category I or II structures (and the Radwaste Building) is removed and replaced with structural fill. [Figure 2.5.4-209](#) also shows the static and dynamic design loads (demand) for these structures.

2.5.4.10.1 Bearing Capacity

a. Allowable Bearing Capacity of Rock and Concrete Fill

The allowable static bearing capacity values for each bedrock zone and concrete fill are given in [Table 2.5.4-210](#). The Zone III allowable static bearing capacity of 20 ksf is less than the value of 20 percent of the ultimate crushing strength (or unconfined compressive strength) given in several building codes ([SSAR Reference 181](#)). The ultimate crushing strength is given as 1.0 kips per square inch (ksi) (144 ksf) in [Table 2.5.4-208](#). The 20 ksf value is the same value given for weathered rock in Table 2.5-2 of the Units 1 and 2 UFSAR ([SSAR Reference 5](#)). For dynamic loading, 20 percent of the ultimate crushing strength can be used. Although the 20 ksf allowable static bearing capacity is greater than the maximum static bearing pressure from the RB/FB basemat, as noted earlier the RB/FB foundation will not be founded directly on the Zone III weathered rock. If excavation during construction for this foundation reveals any weathered or fractured zones at foundation level, such zones will be over-excavated and replaced with concrete fill.

The Zone III-IV and Zone IV bedrock have design unconfined compressive strengths of 9 ksi (1296 ksf) and 17 ksi (2448 ksf), respectively ([Table 2.5.4-208](#)). The allowable static values of the bearing

capacity of 80 ksf and 160 ksf for Zone III-IV and Zone IV rock, respectively, are presumptive values based on various building codes for moderately weathered to fresh foliated rock ([SSAR Reference 181](#)). For dynamic loading, 20 percent of the ultimate crushing strength can be used, i.e., 259 ksf for Zone III-IV, and 490 ksf for Zone IV. For 2500 psi concrete fill, the computed allowable bearing capacity is 199 ksf ([Reference 2.5-215](#)) for both static and dynamic loading.

b. Allowable Bearing Capacity of Soil

For granular soils like the Zone IIB saprolite and the engineered structural fill, bearing capacity is based on Terzaghi's bearing capacity equations modified by Vesic ([SSAR Reference 180](#)). The ultimate (gross) bearing capacity of a footing, q_{ult} , supported on homogeneous soils can be estimated by ([SSAR Reference 180](#)):

$$q_{ult} = cN_c\zeta_c + \gamma'D_fN_q\zeta_q + 0.5\gamma'BN_\gamma\zeta_\gamma$$

where:

c = undrained shear strength for clay (c_u) or cohesion intercept for (c, ϕ) soil

$\gamma'D_f$ = effective overburden pressure at base of foundation

γ' = effective unit weight of soil

D_f = depth from ground surface to base of foundation

B = width of foundation

N_c , N_q , and N_γ are bearing capacity factors (defined in [SSAR Reference 180](#)), and

ζ_c , ζ_q , and ζ_γ are shape factors (defined in [SSAR Reference 180](#))

These equations use the effective unit weight of the soil, the width and depth of the foundation, and bearing capacity and shape factors that are a function of the angle of internal friction of the soil. Consequently, each foundation has a different bearing capacity, depending on the foundation dimensions. For large foundations that are founded at large depths below grade, these equations can give very large bearing capacity values, even when a factor of safety of 3 is included for the allowable bearing value. In such situations, settlement, discussed in [Section 2.5.4.10.2](#), normally governs.

c. Allowable Bearing Capacity for Structures

[Table 2.5.4-211](#) gives the estimated allowable bearing capacity for the Seismic Category I and II structures, and the Radwaste Building based on the materials underlying the structures shown in [Figures 2.5.4-251](#) and [2.5.4-252](#). Review of [Figure 2.5.4-251](#) for the Seismic Category I structures shows that there is concrete fill beneath each structure underlain by Zone III-IV bedrock. [Table 2.5.4-210](#) indicates that, for the static case, the Zone III-IV bearing capacity is less than that of concrete fill, while for the dynamic case, the concrete fill bearing capacity is less than that of Zone III-IV. The smaller bearing capacity is conservatively assumed in each case. Where the major structure bears on soil (Zone IIB saprolite or structural fill), the theoretical allowable capacities of the soil are very large, for the reasons explained above. The design static bearing capacity given in [Table 2.5.4-211](#) is the minimum value for any layer beneath the structure. For structures on soil, settlement estimates are needed to determine what value of bearing pressure can be realistically applied.

[Table 2.5.4-211](#) also contains values of allowable bearing capacity under dynamic or transient loading conditions. For bedrock subgrade, as noted earlier, these values are equivalent to 20 percent of the ultimate crushing strength. The allowable static and dynamic bearing capacity values in [Table 2.5.4-211](#) for the Seismic Category I RB/FB, CB and FWSC foundations exceed the maximum static and dynamic bearing demand values given in [Table 2.5.4-209](#).

The Zone IIA saprolite can be used to support relatively lightly-loaded, non-settlement sensitive structures that are not classified as Seismic Category I or II. The allowable bearing capacity value is limited to 4 ksf because of settlement considerations. As noted in [Section 2.5.4.10.2](#), settlement considerations usually dominate when this material is used for supporting foundations, and the actual allowable bearing capacity may be less than 4 ksf, especially for larger foundations.

d. Buoyancy Effects

NAPS ESP COL 2.5-4

As noted in [Section 2.5.4.6.1](#), [Section 2.4.12.4](#) indicates that the maximum predicted groundwater level in the power block area of Unit 3 increases from north to south, ranging from about Elevation 271 ft at the north end of the Turbine Building to about Elevation 282.5 ft at the south end of the RB/FB. Thus, there can be a hydrostatic uplift force on many of

the structures founded below grade. All of the below-ground structures shown in [Table 2.5.4-209](#) have sufficient applied foundation loads ([Tables 2.5.4-209](#) and [2.5.4-211](#)) that there are no net uplift forces even at the DCD site parameter for maximum groundwater level. However, such forces can be significant in the design of buried piping, particularly when the pipe is empty. In such a situation, the weight and strength of the backfill above the pipe is analyzed to confirm satisfactory resistance to the uplift forces. The normal factor of safety of 3 against soil failure is used in this analysis.

NAPS ESP COL 2.5-6

2.5.4.10.2 Settlement Analysis

The pseudo-elastic method of analysis was used for settlement estimates. This approach is suitable for the granular soils and bedrock at the site. The analysis is based on a stress-strain model that computes settlement of discrete layers:

$$\delta = \sum (\Delta p_i \times \Delta h_i) / E_i$$

where:

δ = settlement

i = 1 to n , where n is the number of soil layers

Δp_i = vertical applied pressure at center of layer i

Δh_i = thickness of layer i

E_i = elastic modulus of layer i

The stress distribution below rectangular foundations is based on a Boussinesq-type distribution for flexible foundations ([Reference 2.5-216](#)). The computation extends to a depth where the increase in vertical stress (Δp) due to the applied load is equal to or less than 10 percent of the applied foundation pressure. The Boussinesq-type vertical pressure under a rectangular footing, σ_z , is as follows ([Reference 2.5-216](#)):

$$\sigma_z = (p/2\pi)(\tan^{-1}(lb/(zR_3)) + (lbz/R_3)(1/R_1^2 + 1/R_2^2))$$

where:

l = length of footing

b = width of footing

z = depth below footing at which pressure is computed

$$R_1 = (l^2 + z^2)^{0.5}$$

$$R_2 = (b^2 + z^2)^{0.5}$$

$$R_3 = (l^2 + b^2 + z^2)^{0.5}$$

Settlement estimates were made using the preceding relationships and the soil and rock properties given in [Table 2.5.4-208](#). These estimates were made for each Seismic Category I and II structure, and the Radwaste Building, and are presented in [Table 2.5.4-212](#). The applied pressures from the foundations are also shown on [Table 2.5.4-212](#).

As would be anticipated, the settlement of the structures founded on Zone III-IV or Zone IV bedrock is negligible. Similarly, settlements of structures sitting on the dense to very dense structural fill or Zone IIB saprolite overlying rock are modest in light of the large applied pressures.

The total and differential settlements under the RB/FB, CB and FWSC are well within the limits stated in [Table 2.0-201](#). Based on the computed settlement values in [Table 2.5.4-212](#), the average settlement of these structures is 0.1 in. or less.

2.5.4.10.3 Earth Pressures

Static and seismic lateral earth pressures are addressed for plant below-ground walls. Both active and at-rest cases are included. Active earth pressure is used for temporary retaining wall installed to facilitate construction. For these, the earth pressure coefficients are Rankine values, assuming level backfill and a zero friction angle between the soil and the wall. Hydrostatic pressures are conservatively based on the groundwater table being at grade. An area wide surcharge pressure of 500 psf is used. Lateral pressures due to compaction are not included; these pressures are controlled by compacting backfill with light equipment near structures. The soil properties used in the calculation of lateral earth pressures are from [Table 2.5.4-208](#).

For the active lateral earth pressure case, earthquake-induced horizontal ground accelerations are addressed by the application of $k_h \cdot g$ applying the Mononobe-Okabe method. Vertical ground accelerations ($k_v \cdot g$) are considered negligible and were ignored ([Reference 2.5-217](#)). The peak low frequency acceleration of 0.31g was used for developing the seismic active earth pressure diagram. Use of the peak high frequency acceleration was considered overly conservative given the low magnitude (energy) of this earthquake.

The method described in ASCE 4-98 Section 3.5.3.2 ([Reference 2.5-218](#)) can be used to estimate the dynamic component of seismic at-rest lateral earth pressure for the below-grade walls of the power block structures. [Reference 2.5-218](#) provides an elastic solution

that is demonstrated by a nomograph. In the nomograph, a dimensionless normalized in-situ lateral stress at 1.0g horizontal earthquake acceleration is developed for a normalized depth at a given Poisson's ratio. The appropriate site-specific at-rest pressure is calculated from the nomograph at various depth intervals using the site-specific acceleration and Poisson's ratio.

Figures 2.5.4-225 through 2.5.4-234 show structural fill between some below-ground structures, e.g., between the FWSC and CB in Figure 2.5.4-228 (Section D-D), and between structures and the temporary retaining wall installed to facilitate construction. Figure 2.5.4-225 (Section A-A) shows approximately 50 ft of excavation is required from the existing ground surface to the bottom of excavation for the Turbine Building. This wall will hold back the natural in-situ soil (Zone IIA and IIB saprolites) which will be in an active condition. This case is used as an example of active earth pressure for the lateral earth pressure diagram in Figure 2.5.4-253. This figure includes active earth pressure along with the groundwater, surcharge and seismic components of lateral pressure against the temporary wall.

The same section shows approximately 60 ft depth of structural fill against the Turbine Building wall. This case is used as an example of at-rest earth pressure against an unyielding wall for the lateral earth pressure diagram in Figure 2.5.4-254. This figure includes at-rest earth pressure along with the groundwater, surcharge and seismic components of lateral pressure against the permanent wall.

The 0.31g peak low frequency acceleration used as input to the active and at-rest seismic (dynamic) lateral earth pressure computations was obtained using input rock motions consistent with the EPRI (2004, 2006) GMM. As noted in Section 2.5.4.7.4, these input motions were subsequently updated based on the EPRI (2013) GMM for use in developing the GMRS and FIRS for Seismic Category I buildings. These updated rock motions are lower than those from the EPRI (2004, 2006) GMM and result in lower site ground motion response spectra, and correspondingly lower peak ground acceleration for the low frequency input motions. Thus, the seismic lateral earth pressures shown in Figures 2.5.4-253 and 2.5.4-254, which are based on the EPRI (2004, 2006) GMM, can be conservatively used in place of the EPRI (2013) GMM, and are therefore not updated.

The lateral pressures in [Figures 2.5.4-253](#) and [2.5.4-254](#) are best estimate pressures with a factor of safety of 1. Appropriate safety factors need to be incorporated into the wall structural design. The factor of safety against a gravity wall or structure foundation sliding is normally taken as 1.1 when seismic pressures are included. The same factor of safety is applied against a wall overturning.

NAPS COL 2.0-29-A

2.5.4.11 **Design Criteria**

NAPS ESP COL 2.5-7

Applicable design criteria are covered in various sections. The criteria summarized below are geotechnical criteria and also geotechnical-related criteria that pertain to structural design.

[Section 2.5.4.8](#) specifies that the acceptable factor of safety against liquefaction of site soils is ≥ 1.1 .

Bearing capacity and settlement criteria are presented in [Section 2.5.4.10](#). [Table 2.5.4-211](#) provides allowable bearing capacity values for the Seismic Category I and II structures and the Radwaste Building. A minimum factor of safety of 3 is used when applying bearing capacity equations. This factor of safety is also applied against breakout failure due to uplift forces on buried piping.

[Section 2.5.4.10](#) also discusses factors of safety related to lateral earth pressures. The lateral pressures shown in [Figures 2.5.4-253](#) and [2.5.4-254](#) have a factor of safety of 1. A factor of safety of 1.1 should be used in the analyses of sliding and overturning due to these lateral loads when the seismic component is included.

[Section 2.5.5.2](#) specifies that the minimum acceptable long-term static factor of safety against slope stability failure is 1.5. [Section 2.5.5.3](#) specifies that the minimum acceptable long-term seismic factor of safety against slope stability failure is 1.1.

NAPS COL 2.0-29-A

2.5.4.12 **Techniques to Improve Subsurface Conditions**

NAPS ESP COL 2.5-8

For Unit 3, any Zone IIA and Zone IIB saprolite beneath or adjacent to Seismic Category I or II structures is removed and replaced with concrete fill. Improvement of the Zone IIA saprolite as described [SSAR Section 2.5.4.12](#) is suitable for non-Seismic Category I and II structures.

Zones of weathered or fractured rock encountered immediately beneath the RB/FB, CB, and FWSC footprints are removed and replaced with concrete fill.

Appendix 2.5.4AA MACTEC Geotechnical Data Report, Rev. 1; September 28, 2007

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NAPS COL 2.0-29A Table 2.5.4-201 Borehole Information

Boring Number	Coordinates (ft)		Ground Surface Elevation (ft)	Penetration Depth (ft)
	Northing	Easting		
B-901	3,909,777.72	11,685,928.59	309.42	300.0
B-902	3,909,874.19	11,685,884.28	302.20	201.7
B-903	3,909,812.10	11,686,028.80	301.59	151.0
B-904	3,909,692.47	11,685,970.43	316.75	151.7
B-905	3,909,732.86	11,685,821.97	306.75	150.4
B-906	3,909,670.03	11,685,795.34	311.72	150.5
B-907	3,909,607.90	11,685,938.35	322.71	200.5
B-908	3,909,716.65	11,686,060.89	307.71	151.4
B-909	3,909,695.46	11,686,107.40	304.90	201.9
B-910	3,909,667.63	11,685,883.11	316.54	148.4
B-911	3,909,919.91	11,685,992.68	299.79	101.0
B-911A	3,909,916.04	11,686,000.53	299.91	21.7
B-912	3,910,021.70	11,686,051.36	275.10	151.8
B-913	3,910,148.50	11,686,114.71	273.37	100.9
B-914	3,909,939.55	11,685,922.35	297.45	200.5
B-915	3,909,877.48	11,686,088.55	301.79	112.8
B-916	3,910,049.54	11,686,008.70	276.24	100.3
B-917	3,910,160.68	11,686,029.45	274.85	150.8
B-918	3,910,115.28	11,686,194.05	272.13	150.1
B-919	3,909,575.39	11,685,764.67	317.79	76.2
B-920	3,909,545.07	11,685,980.20	327.17	150.7
B-921	3,909,680.19	11,686,162.71	307.96	73.9
B-921A	3,909,686.89	11,686,161.68	307.39	40.4
B-922	3,909,943.65	11,686,232.99	271.30	26.0
B-922A	3,909,949.30	11,686,244.02	271.33	76.5
B-923	3,910,076.97	11,686,309.48	272.00	75.4
B-924	3,909,969.53	11,686,475.40	271.52	75.6
B-925	3,910,036.67	11,686,576.27	270.01	75.8

NAPS COL 2.0-29A **Table 2.5.4-201 Borehole Information** *(continued)*

Boring Number	Coordinates (ft)		Ground Surface Elevation (ft)	Penetration Depth (ft)
	Northing	Easting		
B-926	3,910,043.20	11,685,709.26	289.03	155.5
B-927	3,909,966.07	11,685,878.59	292.51	100.4
B-928	3,910,222.75	11,686,159.07	272.17	75.2
B-928A	3,910,220.39	11,686,165.35	271.82	37.5
B-929	3,909,214.44	11,685,654.82	329.02	74.0
B-929A	3,909,214.15	11,685,665.51	329.03	52.5
B-930	3,909,275.95	11,685,842.87	326.12	123.6
B-931	3,910,152.94	11,685,921.54	278.52	74.0
B-932	3,910,444.31	11,686,415.70	249.88	35.1
B-933	3,909,827.41	11,685,790.97	296.48	100.3
B-933A	3,909,826.28	11,685,802.01	296.58	27.5
B-934	3,909,860.37	11,685,686.09	294.80	101.6
B-936	3,910,745.87	11,685,929.15	286.56	100.7
B-937	3,910,688.52	11,686,672.12	270.25	55.3
B-939	3,911,317.60	11,686,605.91	254.03	76.1
B-940	3,910,266.77	11,688,901.02	268.32	76.1
B-941	3,910,403.63	11,688,912.87	267.19	75.8
B-942	3,909,614.69	11,684,326.45	291.85	100.8
B-943	3,909,355.39	11,683,892.47	300.40	101.9
B-944	3,908,772.38	11,684,127.62	334.69	86.4
B-945	3,910,135.55	11,683,779.79	281.51	100.6
B-946	3,908,787.24	11,683,810.59	333.36	100.7
B-947	3,909,574.53	11,686,367.21	312.48	88.8
B-948	3,909,619.26	11,685,565.69	310.41	100.6
B-949	3,909,018.09	11,685,157.27	334.82	106.4
B-950	3,910,835.82	11,686,282.11	282.50	100.8
B-951	3,910,548.26	11,686,821.80	249.93	101.0
W-1	3,909,853.0	11,685,959.0	306.2	154.0

NAPS COL 2.0-29A

Table 2.5.4-201 **Borehole Information** *(continued)*

Boring Number	Coordinates (ft)		Ground Surface Elevation (ft)	Penetration Depth (ft)
	Northing	Easting		
W-2	3,909,822.0	11,685,864.0	298.4	143.3
W-3	3,909,714.5	11,685,899.0	312.3	150.7
W-4	3,909,749.0	11,686,002.0	311.9	150.6
W-5	3,909,978.5	11,686,141.5	273.0	100.4
W-6	3,909,830.5	11,686,205.5	301.9	123.9
W-7	3,909,730.5	11,686,146.5	303.1	151.2
W-8	3,909,767.5	11,686,273.0	307.8	150.6
W-9	3,909,600.5	11,686,022.0	320.2	151.0
W-10	3,909,598.5	11,685,820.0	319.1	153.0
M-1	3,909,611.0	11,685,483.5	314.1	151.1
M-2	3,909,531.0	11,685,586.0	315.3	153.4
M-3	3,909,538.5	11,685,678.5	313.9	152.6
M-4	3,909,456.0	11,685,694.5	321.8	154.0
M-6	3,909,401.0	11,685,759.5	327.8	150.4
M-7	3,909,504.0	11,685,835.5	326.0	151.5
M-8	3,909,413.5	11,685,847.0	329.3	150.6
M-9	3,909,333.5	11,685,946.0	327.3	153.6
M-10	3,909,243.5	11,685,962.0	323.6	201.9
M-11	3,909,351.5	11,686,038.5	325.9	148.7
M-12	3,909,723.0	11,685,560.0	307.0	151.2
M-13	3,909,519.5	11,686,025.0	326.8	151.6
M-14	3,909,451.5	11,686,111.0	323.8	60.3
M-15	3,909,531.0	11,686,166.0	311.3	60.0
M-16	3,909,989.5	11,685,801.5	284.6	61.9
M-17	3,909,775.0	11,686,213.5	306.2	151.9
M-18	3,909,608.0	11,686,213.5	304.2	60.4
M-19	3,910,052.5	11,685,855.5	280.4	151.4
M-20	3,909,793.5	11,686,067.5	302.6	151.0

NAPS COL 2.0-29A Table 2.5.4-201 **Borehole Information** *(continued)*

Boring Number	Coordinates (ft)		Ground Surface Elevation (ft)	Penetration Depth (ft)
	Northing	Easting		
M-21	3,909,811.0	11,686,269.5	303.9	151.8
M-27	3,909,426.0	11,685,937.5	330.2	151.4
M-28	3,909,635.5	11,685,672.0	308.2	150.0
M-29	3,909,710.5	11,685,460.0	309.3	151.2
M-30	3,909,695.0	11,685,381.5	313.3	201.7
M-31	3,909,799.0	11,685,459.5	306.9	151.5
M-32	3,909,875.5	11,685,526.5	313.2	62.2
M-33	3,909,983.5	11,685,614.5	303.8	64.9
M-34	3,910,122.0	11,685,736.0	280.9	63.0

NAPS COL 2.0-29-A Table 2.5.4-202 **Observation Well Information**

Well Number	Coordinates (ft)		Surface Elev. (ft)	Depth (ft)	Elev. of Top of Screen (ft)	Screen Length (ft)
	Northing	Easting				
OW-901	3,909,772	11,685,917	309.6	108.0	214.6	10
OW-945	3,910,136	11,683,793	281.6	54.5	240.1	10
OW-946	3,908,788	11,683,823	334.0	43.4	303.6	10
OW-947	3,909,580	11,686,372	313.3	58.0	268.3	10
OW-949	3,909,025	11,685,153	335.7	105.0	243.2	0
OW-950	3,910,842	11,686,285	283.0	92.0	203.0	10
OW-951	3,910,521	11,686,786	249.7	67.0	194.6	10

NAPS COL 2.0-29-A Table 2.5.4-203 Information on the CPTs Performed

CPT Number	Coordinates (ft)		Ground Surface Elevation (ft)	Depth (ft)
	Northing	Easting		
C-901	3,909,627.77	11,686,012.67	318.56	20.0
C-902	3,909,552.59	11,685,842.21	323.66	29.0
C-903	3,909,719.02	11,685,775.66	306.84	29.0
C-904	3,910,026.29	11,685,793.52	283.92	35.5
C-905	3,910,137.61	11,685,857.21	279.29	45.6
C-906	3,910,013.77	11,686,269.94	270.75	2.6
C-907	3,910,174.67	11,686,277.14	271.66	13.1
C-908	3,910,326.76	11,686,187.39	271.91	28.1
C-909	3,909,346.74	11,685,717.77	330.26	60.0
C-910	3,909,154.43	11,685,782.42	326.99	25.1
C-911	3,910,716.79	11,685,941.76	286.69	15.3
C-912	3,909,959.42	11,686,349.77	271.16	2.8
C-913	3,910,999.95	11,686,812.54	268.65	20.0
C-914	3,910,360.20	11,688,917.62	267.86	31.0
C-915	3,909,784.60	11,686,794.40	320.92	54.0
C-916	3,909,584.68	11,686,372.70	312.91	49.1
C-917	3,909,337.29	11,686,293.79	320.37	49.2
C-918	3,909,151.49	11,685,509.11	329.55	25.1
C-919	3,909,154.30	11,685,255.41	338.06	25.1
C-920	3,909,071.70	11,685,870.40	324.73	25.1
C-921	3,910,112.20	11,685,717.17	281.10	30.0
C-922	3,909,889.28	11,684,055.95	311.73	20.3
C-923	3,910,107.49	11,683,828.42	283.03	22.2

Table 2.5.4-204 Elevation, Depth, and Thickness of the Subsurface Zones

Boring Number	Top Elevation of Zones (ft)						Top Depth of Zones (ft)						Thickness of Zones (ft)				
	I	IIA	IIB	III	III-IV	IV	I	IIA	IIB	III	III-IV	IV	I	IIA	IIB	III	III-IV
B-901	309.4	309.4	279.9	269.5	229.4	174.4	0.0	0.0	29.5	39.9	80.0	135.0	0.0	29.5	10.4	40.1	55.0
B-902	302.2	302.2	283.0	283.0	-	278.4	0.0	0.0	19.2	19.2	-	23.8	0.0	19.2	0.0	4.6	-
B-903	301.6	301.6	281.9	279.0	220.8	185.6	0.0	0.0	19.7	22.6	80.8	116.0	0.0	19.7	2.9	58.2	35.2
B-904	316.8	316.8	288.3	270.0	235.1	195.1	0.0	0.0	28.5	46.8	81.7	121.7	0.0	28.5	18.3	34.9	40.0
B-905	306.7	306.7	286.8	282.9	271.4	176.2	0.0	0.0	19.9	23.8	35.3	130.5	0.0	19.9	3.9	11.5	95.2
B-906	311.7	311.7	282.8	276.8	262.0	176.2	0.0	0.0	28.9	34.9	49.7	135.5	0.0	28.9	6.0	14.8	85.8
B-907	322.7	322.7	287.7	283.7	207.2	177.2	0.0	0.0	35.0	39.0	115.5	145.5	0.0	35.0	4.0	76.5	30.0
B-908	307.7	307.7	280.7	245.0	-	241.3	0.0	0.0	27.0	62.7	-	66.4	0.0	27.0	35.7	3.7	-
B-909	304.9	304.9	275.9	258.0	225.0	195.0	0.0	0.0	29.0	46.9	79.9	109.9	0.0	29.0	17.9	33.0	30.0
B-910	316.5	316.5	294.5	274.5	226.1	-	0.0	0.0	22.0	42.0	90.4	-	0.0	22.0	20.0	48.4	-
B-911	299.8	299.8	282.8	278.8	268.7	233.8	0.0	0.0	17.0	21.0	31.1	66.0	0.0	17.0	4.0	10.1	34.9
B-911A	299.9	299.9	282.9	278.8	268.7	233.8	0.0	0.0	17.0	21.1	31.2	66.1	0.0	17.0	4.1	10.1	34.9
B-912	275.1	275.1	255.5	251.0	-	238.3	0.0	0.0	19.6	24.1	-	36.8	0.0	19.6	4.5	12.7	-
B-913	273.4	273.4	223.4	217.9	-	215.5	0.0	0.0	50.0	55.5	-	57.9	0.0	50.0	5.5	2.4	-
B-914	297.4	297.4	275.4	275.4	236.9	202.1	0.0	0.0	22.0	22.0	60.5	95.3	0.0	22.0	0.0	38.5	34.8
B-915	301.8	301.8	288.3	284.8	279.4	-	0.0	0.0	13.5	17.0	22.4	-	0.0	13.5	3.5	5.4	-
B-916	276.2	276.2	251.1	-	-	250.6	0.0	0.0	25.1	-	-	25.6	0.0	25.1	0.5	-	-
B-917	274.9	274.9	217.9	206.4	187.1	178.8	0.0	0.0	57.0	68.5	87.8	96.1	0.0	57.0	11.5	19.3	8.3
B-918	272.1	271.1	267.0	245.8	-	239.8	0.0	1.0	5.1	26.3	-	32.3	1.0	4.1	21.2	6.0	-
B-919	317.8	317.8	294.8	279.9	264.7	-	0.0	0.0	23.0	37.9	53.1	-	0.0	23.0	14.9	15.2	-

Table 2.5.4-204 Elevation, Depth, and Thickness of the Subsurface Zones (continued)

Boring Number	Top Elevation of Zones (ft)						Top Depth of Zones (ft)						Thickness of Zones (ft)				
	I	IIA	IIB	III	III-IV	IV	I	IIA	IIB	III	III-IV	IV	I	IIA	IIB	III	III-IV
B-920	327.2	324.8	289.2	274.2	-	221.5	0.0	2.4	38.0	53.0	-	105.7	2.4	35.6	15.0	52.7	-
B-921	308.0	308.0	260.0	236.2	-	-	0.0	0.0	48.0	71.8	-	-	0.0	48.0	23.8	-	-
B-921A	307.4	307.4	259.4	236.2	-	-	0.0	0.0	48.0	71.2	-	-	0.0	48.0	23.2	-	-
B-922	271.3	271.3	265.0	262.5	257.3	-	0.0	0.0	6.3	8.8	14.0	-	0.0	6.3	2.5	5.2	-
B-922A	271.3	271.3	271.3	263.1	254.8	209.8	0.0	0.0	0.0	8.2	16.5	61.5	0.0	0.0	8.2	8.3	45.0
B-923	272.0	269.2	266.8	-	266.8	260.3	0.0	2.8	5.2	-	5.2	11.7	2.8	2.4	0.0	-	6.5
B-924	271.5	271.1	265.0	265.0	252.9	227.9	0.0	0.4	6.5	6.5	18.6	43.6	0.4	6.1	0.0	12.1	25.0
B-925	270.0	270.0	253.0	-	249.6	213.7	0.0	0.0	17.0	-	20.4	56.3	0.0	17.0	3.4	-	35.9
B-926	289.0	289.0	235.0	235.0	225.2	179.5	0.0	0.0	54.0	54.0	63.8	109.5	0.0	54.0	0.0	9.8	45.7
B-927	292.5	292.5	268.5	-	252.7	217.9	0.0	0.0	24.0	-	39.8	74.6	0.0	24.0	15.8	-	34.8
B-928	272.2	272.2	244.2	235.1	220.1	212.0	0.0	0.0	28.0	37.1	52.1	60.2	0.0	28.0	9.1	15.0	8.1
B-928A	271.8	271.8	243.8	235.1	220.1	212.0	0.0	0.0	28.0	36.7	51.7	59.8	0.0	28.0	8.7	15.0	8.1
B-929	329.0	329.0	283.0	265.0	-	-	0.0	0.0	46.0	64.0	-	-	0.0	46.0	18.0	-	-
B-929A	329.0	329.0	283.0	265.0	-	-	0.0	0.0	46.0	64.0	-	-	0.0	46.0	18.0	-	-
B-930	326.1	323.7	265.1	244.1	-	-	0.0	2.4	61.0	82.0	-	-	2.4	58.6	21.0	-	-
B-931	278.5	278.5	228.7	221.5	-	-	0.0	0.0	49.8	57.0	-	-	0.0	49.8	7.2	-	-
B-932	249.9	231.9	221.9	-	-	-	0.0	18.0	28.0	-	-	-	18.0	10.0	-	-	-
B-933	296.5	291.0	274.5	269.5	248.3	239.6	0.0	5.5	22.0	27.0	48.2	56.9	5.5	16.5	5.0	21.2	8.7
B-933A	296.6	291.1	274.6	269.5	248.3	239.6	0.0	5.5	22.0	27.1	48.3	57.0	5.5	16.5	5.1	21.2	8.7
B-934	294.8	294.8	252.8	252.8	-	246.4	0.0	0.0	42.0	42.0	-	48.4	0.0	42.0	0.0	6.4	-

Table 2.5.4-204 Elevation, Depth, and Thickness of the Subsurface Zones (continued)

Boring Number	Top Elevation of Zones (ft)						Top Depth of Zones (ft)						Thickness of Zones (ft)				
	I	IIA	IIB	III	III-IV	IV	I	IIA	IIB	III	III-IV	IV	I	IIA	IIB	III	III-IV
B-936	286.6	286.6	266.3	253.1	190.6	-	0.0	0.0	20.3	33.5	96.0	-	0.0	20.3	13.2	62.5	-
B-937	270.3	270.3	245.3	237.0	220.0	-	0.0	0.0	25.0	33.3	50.3	-	0.0	25.0	8.3	17.0	-
B-939	254.0	254.0	215.2	-	-	-	0.0	0.0	38.8	-	-	-	0.0	38.8	-	-	-
B-940	268.3	268.3	249.8	249.8	212.1	-	0.0	0.0	18.5	18.5	56.2	-	0.0	18.5	0.0	37.7	-
B-941	267.2	267.2	258.7	219.3	-	205.8	0.0	0.0	8.5	47.9	-	61.4	0.0	8.5	39.4	13.5	-
B-942	291.8	291.8	285.8	-	-	263.0	0.0	0.0	6.0	-	-	28.8	0.0	6.0	22.8	-	-
B-943	300.4	300.4	283.9	278.0	268.5	220.1	0.0	0.0	16.5	22.4	31.9	80.3	0.0	16.5	5.9	9.5	48.4
B-944	334.7	334.7	299.7	-	281.2	-	0.0	0.0	35.0	-	53.5	-	0.0	35.0	18.5	-	-
B-945	281.5	281.5	228.1	-	221.0	210.6	0.0	0.0	53.4	-	60.5	70.9	0.0	53.4	7.1	-	10.4
B-946	333.4	333.4	301.2	-	291.6	-	0.0	0.0	32.2	-	41.8	-	0.0	32.2	9.6	-	-
B-947	312.5	312.5	260.8	248.8	-	-	0.0	0.0	51.7	63.7	-	-	0.0	51.7	12.0	-	-
B-948	310.4	310.4	288.4	281.9	274.7	-	0.0	0.0	22.0	28.5	35.7	-	0.0	22.0	6.5	7.2	-
B-949	334.8	334.8	281.9	-	258.4	-	0.0	0.0	52.9	-	76.4	-	0.0	52.9	23.5	-	-
B-950	282.5	282.5	261.8	-	232.2	218.7	0.0	0.0	20.7	-	50.3	63.8	0.0	20.7	29.6	-	13.5
B-951	249.9	249.9	230.4	209.3	-	179.9	0.0	0.0	19.5	40.6	-	70.0	0.0	19.5	21.1	29.4	-
W-1	306.2	306.2	284.2	277.9	211.2	-	0.0	0.0	22.0	28.3	95.0	-	0.0	22.0	6.3	66.7	-
W-2	298.4	298.4	279.9	275.6	275.6	212.3	0.0	0.0	18.5	22.8	22.8	86.1	0.0	18.5	4.3	0.0	63.3
W-3	312.3	312.3	283.7	278.7	241.6	231.6	0.0	0.0	28.6	33.6	70.7	80.7	0.0	28.6	5.0	37.1	10.0
W-4	311.9	311.9	275.9	273.0	241.3	201.3	0.0	0.0	36.0	38.9	70.6	110.6	0.0	36.0	2.9	31.7	40.0
W-5	273.0	273.0	273.0	269.1	268.8	228.8	0.0	0.0	0.0	3.9	4.2	44.2	0.0	0.0	3.9	0.3	40.0

Table 2.5.4-204 Elevation, Depth, and Thickness of the Subsurface Zones (continued)

Boring Number	Top Elevation of Zones (ft)						Top Depth of Zones (ft)						Thickness of Zones (ft)				
	I	IIA	IIB	III	III-IV	IV	I	IIA	IIB	III	III-IV	IV	I	IIA	IIB	III	III-IV
W-6	301.9	301.9	263.2	248.2	208.0	-	0.0	0.0	38.7	53.7	93.9	-	0.0	38.7	15.0	40.2	-
W-7	303.1	303.1	253.8	243.8	226.9	201.9	0.0	0.0	49.3	59.3	76.2	101.2	0.0	49.3	10.0	16.9	25.0
W-8	307.8	307.8	253.5	233.5	227.8	216.4	0.0	0.0	54.3	74.3	80.0	91.4	0.0	54.3	20.0	5.7	11.4
W-9	320.2	320.2	291.9	251.9	187.2	-	0.0	0.0	28.3	68.3	133.0	-	0.0	28.3	40.0	64.7	-
W-10	319.1	319.1	276.1	275.8	226.1	-	0.0	0.0	43.0	43.3	93.0	-	0.0	43.0	0.3	49.7	-
M-1	314.1	314.1	280.9	273.1	260.6	255.6	0.0	0.0	33.2	41.0	53.5	58.5	0.0	33.2	7.8	12.5	5.0
M-2	315.3	315.3	282.5	272.5	251.4	241.9	0.0	0.0	32.8	42.8	63.9	73.4	0.0	32.8	10.0	21.1	9.5
M-3	313.9	313.9	299.1	289.6	274.8	171.3	0.0	0.0	14.8	24.3	39.1	142.6	0.0	14.8	9.5	14.8	103.5
M-4	321.8	321.8	292.9	287.9	252.8	207.8	0.0	0.0	28.9	33.9	69.0	114.0	0.0	28.9	5.0	35.1	45.0
M-6	327.8	327.8	263.7	253.7	231.4	211.4	0.0	0.0	64.1	74.1	96.4	116.4	0.0	64.1	10.0	22.3	20.0
M-7	326.0	326.0	302.0	291.9	219.7	184.7	0.0	0.0	24.0	34.1	106.3	141.3	0.0	24.0	10.1	72.2	35.0
M-8	329.3	329.3	275.6	250.6	242.9	-	0.0	0.0	53.7	78.7	86.4	-	0.0	53.7	25.0	7.7	-
M-9	327.3	327.3	264.6	244.6	-	203.7	0.0	0.0	62.7	82.7	-	123.6	0.0	62.7	20.0	40.9	-
M-10	323.6	323.6	264.5	221.5	212.8	172.0	0.0	0.0	59.1	102.1	110.8	151.6	0.0	59.1	43.0	8.7	40.8
M-11	325.9	325.9	232.3	212.3	-	-	0.0	0.0	93.6	113.6	-	-	0.0	93.6	20.0	-	-
M-12	307.0	305.0	269.0	263.5	243.1	240.8	0.0	2.0	38.0	43.5	63.9	66.2	2.0	36.0	5.5	20.4	2.3
M-13	326.8	326.8	-	274.8	255.8	182.7	0.0	0.0	-	52.0	71.0	144.1	0.0	52.0	-	19.0	73.1
M-14	323.8	323.8	265.0	-	-	-	0.0	0.0	58.8	-	-	-	0.0	58.8	-	-	-
M-15	311.3	311.3	267.8	-	-	-	0.0	0.0	43.5	-	-	-	0.0	43.5	-	-	-
M-16	284.6	283.8	235.3	230.3	-	224.4	0.0	0.8	49.3	54.3	-	60.2	0.8	48.5	5.0	5.9	-

Table 2.5.4-204 Elevation, Depth, and Thickness of the Subsurface Zones *(continued)*

Boring Number	Top Elevation of Zones (ft)						Top Depth of Zones (ft)						Thickness of Zones (ft)				
	I	IIA	IIB	III	III-IV	IV	I	IIA	IIB	III	III-IV	IV	I	IIA	IIB	III	III-IV
M-17	306.2	305.7	-	252.1	214.3	204.3	0.0	0.5	-	54.1	91.9	101.9	0.5	53.6	-	37.8	10.0
M-18	304.2	304.2	245.7	244.0	-	-	0.0	0.0	58.5	60.2	-	-	0.0	58.5	1.7	-	-
M-19	280.4	280.4	222.1	-	213.8	206.7	0.0	0.0	58.3	-	66.6	73.7	0.0	58.3	8.3	-	7.1
M-20	302.6	302.1	-	265.6	246.6	193.6	0.0	0.5	-	37.0	56.0	109.0	0.5	36.5	-	19.0	53.0
M-21	303.9	303.9	260.4	255.4	222.1	207.1	0.0	0.0	43.5	48.5	81.8	96.8	0.0	43.5	5.0	33.3	15.0
M-27	330.2	330.2	286.5	261.5	215.2	208.8	0.0	0.0	43.7	68.7	115.0	121.4	0.0	43.7	25.0	46.3	6.4
M-28	308.2	308.2	292.6	284.4	265.2	183.2	0.0	0.0	15.6	23.8	43.0	125.0	0.0	15.6	8.2	19.2	82.0
M-29	309.3	309.3	277.3	-	268.3	265.1	0.0	0.0	32.0	-	41.0	44.2	0.0	32.0	9.0	-	3.2
M-30	313.3	313.3	279.6	274.6	269.6	266.6	0.0	0.0	33.7	38.7	43.7	46.7	0.0	33.7	5.0	5.0	3.0
M-31	306.9	306.9	-	-	247.9	245.4	0.0	0.0	-	-	59.0	61.5	0.0	59.0	-	-	2.5
M-32	313.2	313.2	264.6	259.6	256.0	-	0.0	0.0	48.6	53.6	57.2	-	0.0	48.6	5.0	3.6	-
M-33	303.8	303.8	259.7	-	-	-	0.0	0.0	44.1	-	-	-	0.0	44.1	-	-	-
M-34	280.9	280.9	252.1	237.0	232.9	220.9	0.0	0.0	28.8	43.9	48.0	60.0	0.0	28.8	15.1	4.1	12.0

NAPS COL 2.0-29-A Table 2.5.4-205 Type and Number of Laboratory Tests Performed

Material	Test	Number
Soil	Natural moisture content	122
	Specific gravity	6
	Sieve and hydrometer analysis	55
	Grain size analysis with no. 200 wash	72
	Atterberg limits	23
	Chemical analysis (pH, chloride, sulfate)	19
	Triaxial consolidated-undrained compression	6
	Resonant column torsional shear	5
	California bearing ratio	5
	Moisture density (modified Proctor)	9
Rock	Unit weight	92
	Unconfined compression	65
	Unconfined compression with stress-strain measurements	27

NAPS COL 2.0-29-A Table 2.5.4-206a Results of Laboratory Tests on Soil Samples*

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel ⁽¹⁾ (%)	Sand ⁽¹⁾ (%)	Fines ⁽²⁾ (%)	Silt ⁽¹⁾ (%)	0.005 mm Clay ⁽¹⁾ (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G _s	pH ⁽³⁾	Chloride (mg/kg) ^{(3), (6), (7)}	Sulfate (mg/kg) ^{(3), (6), (7)}
B-901	B-901-2	3.5-5.0	SPT	0.0	53.6	46.4	10.8	35.6	(8)	21.5	(8)	(8)	(8)	(8)	(8)	(8)
B-901	B-901-4	11.5-13.0	SPT	0.0	76.6	23.4	16.0	7.4		10.2				5.8	ND ⁽⁵⁾	ND ⁽⁵⁾
B-901	B-901-6	22.2-23.7	SPT	0.0	76.8	23.2				16.4						
B-901	B-901-9	37.2-38.7	SPT	0.7	71.9	22.5	15.2	7.3		16.4						
B-901	UD-2	9.5-11.5 ⁽⁴⁾	UD	0.0	78.0	22.0	12.6	9.4		15.0						
B-902	B-902-2	3.5-5.0	SPT	0.0	86.1	13.9				5.6						
B-902	B-902-4	8.5-10.0	SPT	1.3	71.0	29.0	13.4	15.6	SM	23.9	33	7				
B-902	B-902-6	13.5-15.0	SPT	0.0	80.0	20.0				14.0						
B-907	B-907-2	3.5-5.0	SPT	0.0	67.0	33.0	17.7	15.3	SM	14.0	33	8				
B-907	B-907-3	5.5-7.0	SPT	0.0	74.9	25.1				16.4				4.8	51.1 ^J	ND ⁽⁵⁾
B-907	B-907-5	11.0-12.5	SPT	0.0	76.0	24.0				20.2						
B-907	B-907-7	17.5-19.0	SPT	0.0	80.9	19.1	11.7	7.4		12.3						
B-907	B-907-9	27.5-29.0	SPT	0.0	73.9	26.1										
B-907	B-907-10	32.5-34.0	SPT	0.0	66.6	23.4										
B-908	B-908-3	6.0-7.5	SPT	2.0	72.6	25.4	11.6	13.8		12.3			2.62			
B-908	B-908-6	13.5-15.0	SPT	0.0	76.6	23.4							2.69			
B-908	B-908-8	23.7-25.2	SPT	0.0	68.1	31.9										
B-908	B-908-13	47.1-48.6	SPT	0.0	76.0	24.0	18.9	5.1		14.5						

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A Table 2.5.4-206a Results of Laboratory Tests on Soil Samples* (continued)

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel ⁽¹⁾ (%)	Sand ⁽¹⁾ (%)	Fines ⁽²⁾ (%)	Silt ⁽¹⁾ (%)	0.005 mm Clay ⁽¹⁾ (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G _s	pH ⁽³⁾	Chloride (mg/kg) ^{(3), (6), (7)}	Sulfate (mg/kg) ^{(3), (6), (7)}
B-909	B-909-3	6.0-7.5	SPT	0.0	66.9	33.1	19.3	13.8	SM	25.9	57	12				
B-909	B-909-5	11.0-12.5	SPT	0.0	77.6	22.4				31.4				5.4	137 ^J	6.7
B-909	B-909-7	18.5-20.0	SPT	0.0	63.7	36.3	29.0	7.3	SM	25.1	30	4				
B-909	B-909-8	23.5-25.0	SPT	1.7	56.1	42.2				35.4						
B-909	B-909-12	41.9-43.4	SPT	0.0	75.3	24.7				17.6						
B-910	B-910-2	3.5-5.0	SPT	4.0	31.9	64.1	12.1	52.0		27.7						
B-910	B-910-5	11.0-12.5	SPT							30.5	45	13		5.8	3.6 ^J	5.1 ^B
B-910	B-910-7	18.5-20.0	SPT	0.0	46.4	53.6	43.1	10.5		33.1						
B-910	B-910-9	25.9-27.4	SPT	2.3	76.3	21.4				14.6				6.7	5.2 ^J	4.2 ^B
B-911	B-911-2	3.5-5.0	SPT	0.3	59.1	40.6				12.8						
B-911	B-911-4	8.0-9.5	SPT	0.0	70.6	29.4	13.6	15.8		19.6						
B-911	B-911-5	11.0-12.5	SPT	0.0	78.3	21.7								5.6	3.4 ^J	ND ⁽⁵⁾
B-911	B-911-7	18.5-20.0	SPT	0.1	80.0	19.9				11.1						
B-912	B-912-1	9.1-10.6	SPT	0.0	73.7	26.3	20.8	5.5		24.0						
B-912	B-912-3	14.1-15.6	SPT	0.0	72.6	27.4				15.2						
B-912	B-912-4	19.1-19.9	SPT	14.5	84.9	0.6				15.7						
B-913	B-913-8	43.5-48.5	SPT	0.0	72.3	27.7										
B-914	B-914-2	3.5-5.0	SPT	0.1	52.9	47.0	21.0	26.0	SC	16.6	27	10				

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A Table 2.5.4-206a Results of Laboratory Tests on Soil Samples* (continued)

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel ⁽¹⁾ (%)	Sand ⁽¹⁾ (%)	Fines ⁽²⁾ (%)	Silt ⁽¹⁾ (%)	0.005 mm Clay ⁽¹⁾ (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G _s	pH ⁽³⁾	Chloride (mg/kg) ^{(3), (6), (7)}	Sulfate (mg/kg) ^{(3), (6), (7)}
B-914	B-914-3	6.0-7.5	SPT	4.0	63.0	33.0										
B-914	B-914-5	11.0-13.5	SPT	2.1	78.0	19.9										
B-914	B-914-7	19.0-20.5	SPT	27.8	61.0	11.2	8.6	2.6		20.8						
B-914	B-914-9	35.6-37.1	SPT	5.7	70.1	24.2								6.8	8.4 ^J	ND ⁽⁵⁾
B-914	B-914-10	40.6-42.1	SPT	0.1	74.4	25.5	19.5	6.0		20.5						
B-917	B-917-13	48.5-53.5	SPT	0.0	81.9	18.1	15.0	3.1								
B-918	B-918-2	1.8-3.2	SPT	1.2	85.7	13.1	7.3	5.8		15.8			2.68			
B-918	B-918-3	5.1-6.6	SPT	0.0	85.0	15.0				13.3				6.9	8.0 ^J	9.4
B-918	B-918-4	9.3-10.8	SPT	0.0	80.6	19.4	13.4	6.0		13.7						
B-918	B-918-6	13.2-14.7	SPT	0.0	77.7	22.3				13.9						
B-918	B-918-8	22.4-23.9	SPT	1.4	79.4	19.2				17.8						
B-919	B-919-1	1.5-3.0	SPT							18.6	32	11				
B-919	B-919-3	5.9-7.4	SPT	2.5	80.9	16.6				11.1						
B-919	B-919-5	11.0-12.5	SPT	0.6	80.4	19.0				11.2						
B-919	B-919-7	18.9-19.4	SPT	3.7	75.5	20.8	10.8	10.0		13.8						
B-919	B-919-13	51.3-52.8	SPT	0.0	65.9	34.1	26.0	8.1		17.9						
B-920	B-920-1	2.0-3.5	SPT							25.2						
B-920	B-920-2	3.8-5.3	SPT											5.9	1.5 ^{B J}	7.5

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A Table 2.5.4-206a Results of Laboratory Tests on Soil Samples* (continued)

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel ⁽¹⁾ (%)	Sand ⁽¹⁾ (%)	Fines ⁽²⁾ (%)	Silt ⁽¹⁾ (%)	0.005 mm Clay ⁽¹⁾ (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G _s	pH ⁽³⁾	Chloride (mg/kg) ^{(3), (6), (7)}	Sulfate (mg/kg) ^{(3), (6), (7)}
B-920	B-920-3	6.0-7.5	SPT	0.3	58.9	40.8				24.1						
B-920	B-920-6	13.8-15.3	SPT							15.7				6.5	63.0 ^J	7.5
B-920	B-920-7	18.8-20.3	SPT	0.0	72.3	27.7	21.3	6.4		15.4						
B-920	B-920-9	27.3-28.8	SPT	0.0	79.9	20.1				19.5						
B-920	B-920-12	43.5-44.7	SPT							12.9				6.9	1.4 ^{B J}	2.3 ^B
B-921	B-921-1	1.5-3.0	SPT	11.5	52.1	36.4				12.0						
B-921	B-921-3	6.0-7.5	SPT	0.0	41.3	58.7	29.2	29.5	CL	24.8	34	14				
B-921	B-921-4	8.5-10.0	SPT	0.0	53.5	46.5	37.3	9.2		28.0				7.0	4.4 ^J	10.8
B-921	B-921-6	13.5-15.0	SPT	0.0	74.2	25.8	16.1	9.7		26.0						
B-921	B-921-8	23.8-25.3	SPT							32.1	38	NP				
B-921	B-921-10	33.8-35.3	SPT	0.0	75.5	24.5				20.4						
B-921	B-921-11	38.8-40.3	SPT	0.0	81.3	18.7				15.8						
B-921	B-921-16	63.8-65.3	SPT	0.0	75.1	24.9	18.2	6.7		8.5						
B-923	B-923-2	3.3-4.8	SPT	10.9	55.5	33.6	16.7	16.9	SC	22.5	33	10				
B-924	B-924-2	3.5-5.0	SPT	23.2	65.8	11.0	7.9	3.1		2.1						
B-924	B-924-3	6.0-7.5	SPT	11.1	74.5	14.4				4.8						
B-927	B-927-1	1.5-3.0	SPT	0.0	61.4	38.6	12.6	26.0	SC	14.1	28	10				
B-927	B-927-2	3.5-5.0	SPT	0.0	75.8	24.2				11.7						

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A Table 2.5.4-206a Results of Laboratory Tests on Soil Samples* (continued)

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel ⁽¹⁾ (%)	Sand ⁽¹⁾ (%)	Fines ⁽²⁾ (%)	Silt ⁽¹⁾ (%)	0.005 mm Clay ⁽¹⁾ (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G _s	pH ⁽³⁾	Chloride (mg/kg) ^{(3), (6), (7)}	Sulfate (mg/kg) ^{(3), (6), (7)}
B-927	B-927-3	6.0-7.5	SPT	0.0	73.2	26.8	17.1	9.7		12.2						
B-927	B-927-4	8.5-10.0	SPT	0.0	83.3	16.7				6.8				5.8	2.8 ^J	4.3 ^B
B-927	B-927-6	13.5-15.0	SPT	0.0	81.2	18.8				11.2						
B-927	B-927-7	18.5-20.0	SPT	0.0	76.2	23.8				11.4						
B-927	B-927-8	23.5-25.0	SPT	0.0	79.7	20.3				15.7				7.4	5.6 ^J	3.4 ^B
B-928	B-928-2	3.5-5.0	SPT	0.0	78.4	21.6				17.9						
B-928	B-928-4	8.3-9.8	SPT	0.0	73.4	26.6				18.5				6.8	120.0 ^J	4.9 ^B
B-928	B-928-6	14.0-15.5	SPT	0.0	77.0	23.0	17.8	5.2		24.5						
B-928	B-928-8	22.1-23.6	SPT	0.0	78.7	21.3				17.0						
B-928	B-928-9	27.1-28.6	SPT	0.0	74.7	25.3	19.2	6.1		16.4						
B-928 A	UD-3	20-22 ⁽⁴⁾	UD	0.0	82.0	18.0	13.2	4.8								
B-929	B-929-1	1.5-3.0	SPT	12.2	43.7	44.1	16.6	27.5	SC	14.5	36	17				
B-929	B-929-2	3.5-5.0	SPT								54	16				
B-929	B-929-4	8.7-10.2	SPT	0.0	65.5	34.5				18.9				5.9	2.8 ^J	2.7 ^B
B-929	B-929-5	13.5-15.0	SPT	0.0	73.8	26.2				19.6						
B-929	B-929-7	23.0-24.5	SPT	0.0	76.9	23.1	17.0	6.1		18.8						
B-929	B-929-9	33.0-34.5	SPT	0.0	82.7	17.3				16.9						
B-929	B-929-11	43.0-44.5	SPT	0.7	81.4	17.9				17.2						

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A Table 2.5.4-206a Results of Laboratory Tests on Soil Samples* (continued)

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel ⁽¹⁾ (%)	Sand ⁽¹⁾ (%)	Fines ⁽²⁾ (%)	Silt ⁽¹⁾ (%)	0.005 mm Clay ⁽¹⁾ (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G _s	pH ⁽³⁾	Chloride (mg/kg) ^{(3), (6), (7)}	Sulfate (mg/kg) ^{(3), (6), (7)}
B-929	B-929-13	53.0-54.5	SPT	0.0	80.0	20.0				13.8						
B-929A	UD-1	15.0-16.8 ⁽⁴⁾	UD	0.0	78.6	21.4	15.1	6.3		13.1						
B 929A	UD-6	40-41.8 ⁽⁴⁾	UD	0.0	83.3	16.7	11.7	5.0		16.9						
B-931	B-931-10	47.3-48.8	SPT	0.0	78.5	21.5	15.9	5.6								
B-932	B-932-5	19.0-20.5	SPT	0.0	77.7	22.3	15.7	6.6		21.5						
B-933	B-933-3	6.0-7.5	SPT	0.0	62.3	37.7	22.6	15.1	SM	24.2	28	3				
B-933	B-933-5	11.2-12.7	SPT	0.0	58.8	41.2				25.9				5.4	210 ^J	3.0 ^B
B-933	B-933-7	19.5-21.0	SPT	0.0	76.6	23.4				26.7						
B-933	B-933-8	24.5-25.0	SPT	0.0	80.5	19.5				18.7						
B-945	B-945-1	1.5-3.0	SPT	0.0	82.0	18.0				14.5						
B-945	B-945-3	4.7-6.2	SPT	0.0	75.5	24.5	16.2	8.3		15.9						
B-945	B-945-5	11.3-12.8	SPT	0.0	84.2	15.8				21.6				6.4	6.9 ^J	3.1 ^B
B-945	B-945-7	19.4-20.9	SPT	0.0	84.8	15.2				27.6			2.58			
B-945	B-945-9	27.8-29.4	SPT	0.0	82.9	17.1	10.2	6.9		24.1						
B-945	B-945-11	39.4-40.9	SPT	0.0	90.1	9.9				20.4						
B-945	B-945-13	49.4-50.9	SPT	0.0	90.3	9.7				15.6						
B-947	B-947-1	1.5-3.0	SPT							16.7	55	25	2.60			
B-947	B-947-3	4.5-6.0	SPT	0.0	38.3	61.7	23.5	38.2	MH	36.0	56	19				

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A Table 2.5.4-206a Results of Laboratory Tests on Soil Samples* (continued)

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel ⁽¹⁾ (%)	Sand ⁽¹⁾ (%)	Fines ⁽²⁾ (%)	Silt ⁽¹⁾ (%)	0.005 mm Clay ⁽¹⁾ (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G _s	pH ⁽³⁾	Chloride (mg/kg) ^{(3), (6), (7)}	Sulfate (mg/kg) ^{(3), (6), (7)}
B-947	B-947-4	8.5-10.0	SPT	0.0	60.0	40.0			SM	20.7	38	9				
B-947	B-947-5	9.5-11.0	SPT	1.6	55.9	42.5	21.1	21.4		28.2			2.78			
B-947	B-947-6	13.5-15.0	SPT	0.0	30.5	69.5				22.5						
B-947	B-947-7	17.2-18.7	SPT	0.0	75.8	24.2				21.1				6.4	21.4 ^J	6.4
B-947	B-947-8	22.2-23.7	SPT	0.6	79.4	20.0	10.7	9.3		24.3						
B-947	B-947-9	28.7-30.2	SPT	0.0	66.6	33.4				28.8	33	NP				
B-947	B-947-10	33.7-35.2	SPT	0.0	81.3	18.7				20.2						
B-947	B-947-11	38.7-40.2	SPT	0.0	85.8	14.2				16.9						
B-947	B-947-12	42.2-43.7	SPT	0.0	79.7	20.3	13.4	6.9		20.5						
B-948	B-948-1	1.5-3.0	SPT	0.0	54.7	45.3				83.7						
B-948	B-948-3	6.0-7.5	SPT	0.0	51.1	48.9				16.2				5.7	3.8 ^J	ND ⁽⁵⁾
B-948	B-948-5	9.5-11.0	SPT	0.0	31.0	69.0	61.9	7.1		13.7						
B-948	B-948-7	18.5-20.0	SPT	0.0	35.9	64.1				15.2						
B-948	B-948-8	23.5-24.4	SPT	0.0	77.7	22.3				13.6						
B-951	B-951-8	23.0-24.5	SPT	0.2	82.9	16.9	10.5	6.4		13.9						
M-10 (DH)	SS-2	11.7-13.2		0	57.5	42.5	30.2	12.3	SM	48.5	59	9				
M-10 (DH)	SS-4	19.2-20.7		0	61.9	38.1	29.4	8.7	SM	35.9	54	6				

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A Table 2.5.4-206a Results of Laboratory Tests on Soil Samples* (continued)

Boring Number	Sample Number	Depth (ft)	Sample Type	Gravel ⁽¹⁾ (%)	Sand ⁽¹⁾ (%)	Fines ⁽²⁾ (%)	Silt ⁽¹⁾ (%)	0.005 mm Clay ⁽¹⁾ (%)	USCS Symbol	Natural Moisture (%)	LL	PI	G _s	pH ⁽³⁾	Chloride (mg/kg) ^{(3), (6), (7)}	Sulfate (mg/kg) ^{(3), (6), (7)}
M-10 (DH)	SS-5	24.2-25.7		0	61.3	38.7	28.4	10.3	SM	53.7	59	12				
M-10 (DH)	SS-6	29.2-30.7		0	56.6	43.4	31.9	11.5	SM	66.7	51	7				
M-10 (DH)	SS-8	39.1-40.6		0	53.5	46.5	42.4	4.1	SM	30.6	42	6				
M-10 (DH)	SS-10	49.1-50.6		0	79.9	20.1			SM ⁽⁹⁾	16.4						
M-10 (DH)	SS-12	59.1-60.6		0.7	77.2	22.1			SM ⁽⁹⁾	15.1						
M-10 (DH)	SS-15	74.1-75.6		0	72.6	27.4			SM ⁽⁹⁾	29.9						
M-10 (DH)	SS-17	84.1-85.6		0	79	21			SM ⁽⁹⁾	15.1						
M-30 (DH)	SS-1	8.7-10.2		0.6	72.3	27.1			SM ⁽⁹⁾	17						
M-30 (DH)	SS-3	13.7-15.2		0	64	36			SM ⁽⁹⁾	19.8						
M-30 (DH)	SS-5	23.7-25.2		0	82	18			SM ⁽⁹⁾	18.5						
M-30 (DH)	SS-7	33.7-35.2		0	77.1	22.9			SM ⁽⁹⁾	14.8						

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A **Table 2.5.4-206a Results of Laboratory Tests on Soil Samples* (continued)**

- (1) Due to computer roundoff, particle size fractions may total 100 ±1. Fines include silt plus clay.
- (2) Fines include silt plus clay.
- (3) Tests performed by STL - St. Louis, MO
- (4) Depth interval shown reflects total pushed depth of UD tube.
- (5) ND indicates analyte not detected at or above the Method Detection Limit
- (6) B = Estimated Result. Analyte detected above the Method Detection Limit but not above the Reporting Limit.
- (7) J = Method blank contamination. The associated method blank contains the target analyte at a reportable level
- (8) Shaded cells indicate that information not obtained.
- (9) Classification is based on quantitative and qualitative (visual inspection) information. LL= Liquid Limit, PI = Plasticity Index, Gs = Specific Gravity

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

Table 2.5.4-206b Results of Laboratory Tests on Soil Samples; Consolidated-Undrained Triaxial Tests*

Source of Sample	Sample No.	Sample Depth ⁽¹⁾ (ft)	Sample Type	Test Type	C' (psf)	Φ' (degree)	C (psf)	Φ' (degree)	Comment
B-901	UD-2	9.5-11.5	UD Tube	CU	0.0	33.6	0.0	37.5	
B-928 A	UD-3	20-22	UD Tube	CU	423.4	31.4	103.7	41.2	
B-929 A	UD-1	15-16.75	UD Tube	CU	5.4	32.4	178.6	35.8	Only 2 points tested due to limited sample
B-929 A	UD-4	30-31.5	UD Tube	CU	0.0	33.0	0.0	33.0	Only 2 points tested due to limited sample
B-929 A	UD-6	40-41.5	UD Tube	CU	0.0	36.1	318.2	36.4	
B-933 A	UD-2	15-16.25	UD Tube	CU	55.0	32.6	479.5	30.5	Only 2 points tested due to limited sample
(1) Sample depth shown reflects the depth of start of push plus the length of the recovered sample									

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

**NAPS COL 2.0-29-A Table 2.5.4-206c Results of Laboratory Tests on Soil Samples
Moisture-Density and CBR Tests***

Source of Sample	Sample No.	Moisture/Density Results ^A			CBR Results ^B			
		Natural Moisture (%)	Maximum Dry Density (pcf)	Optimum Moisture (%)	Molded Density (pcf)	Molded Moisture (%)	Soaked CBR (0.10") (%)	Soaked CBR (0.20") (%)
Test Pit 1	TP-1-1	23.4	108.7	17.6		Not Tested		
Test Pit 1	TP-1-2	22.6	108.8	17.1	90.3	17.0	1.2	1.6
					94.4	17.0	6.3	5.5
					105.3	17.2	14.7	15.6
Test Pit 2	TP-2	22.6	100.4	22.3	83.0	22.8	1.1	1.1
					89.1	22.0	1.3	1.2
					101.0	22.0	6.2	6.5
Test Pit 3	TP 3-1	16.1	124.9	9.5		Not Tested		
Test Pit 3	TP 3-2	12.4	124.5	10.9	117.5	10.7	5.9	6.0
					122.9	10.6	3.2	5.0
					125.6	10.5	4.2	8.4
Test Pit 4	TP 4-1	30.2	108.6	17.1		Not Tested		
Test Pit 4 ^C	TP 4-2	15.2	125.5	10.8	119.4	11.0	4.9	7.3
					121.5	10.6	8.8	11.9
Test Pit 5	TP 5	9.4	126	9.2		Not Tested		
Test Pit 6	TP 6	18.2	116.1	13.2	110.3	12.3	6.9	8.0
					111.7	12.7	6.4	9.5
					115.1	12.3	12.1	13.8

A Proctor Test results, ASTM D 1557-02 Method A Modified

B California Bearing Ratio Test results, ASTM D 1883-05 (Section 7.12)

C Insufficient Material for three tests

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A Table 2.5.4-207 Results of Unconfined Compression Tests on Rock*

Boring No.	Run Number	Sample Top Depth (ft)	Sample Length (L) (Inches)	Sample Diameter (D) (inches)	L/D Ratio	Unit Weight (pcf)	Type of Break ⁽¹⁾	Unconfined Compressive Strength (psi) ⁽²⁾	Young's Modulus (psi)	Poisson's Ratio
B-901	5	54.0	5.27	2.49	2.1	160	Shear	4,375	(ND) ³	(ND)
B-901	7	60.3	5.27	2.49	2.1	162	Columnar	15,425	3,970,000	* ⁽⁴⁾
B-901	14	97.9	5.34	2.50	2.1	162	C&S	12,629	(ND)	(ND)
B-901	25	129.5	5.35	2.49	2.1	164	C&S	14,171	(ND)	(ND)
B-901	34	170.5	5.33	2.40	2.2	168	Shear	10,865	5,360,000	0.31
B-901	42	208.5	5.32	2.40	2.2	163	Shear	12,777	(ND)	(ND)
B-901	51	240.5	5.35	2.39	2.2	165	C&S	23,619	(ND)	(ND)
B-901	59	280.5	5.36	2.39	2.2	164	C&S	25,335	8,320,000	0.39
B-902	3	27.3	5.29	2.38	2.2	162	C&S	14,947	4,090,000	* ⁽⁴⁾
B-902	9	47.4	5.35	2.40	2.2	163	Shear	21,007	(ND)	(ND)
B-902	14	72.3	5.34	2.40	2.2	164	C&S	25,100	(ND)	(ND)
B-902	18	92.8	5.32	2.40	2.2	164	Shear	6,030	1,840,000	0.42
B-902	28	141.9	5.31	2.40	2.2	170	Shear	6,982	(ND)	(ND)
B-902	38	184.6	5.36	2.40	2.2	163	C&S	27,303	(ND)	(ND)
B-907	3	51.9	5.29	2.45	2.2	152	Shear	957	(ND)	(ND)
B-907	12	90.0	5.23	2.46	2.1	155	Shear	751	(ND)	(ND)
B-907	24	116.8	5.27	2.47	2.1	173	Shear	4,599	(ND)	(ND)
B-907	27	131.8	5.32	2.48	2.1	173	C&S	8,519	(ND)	(ND)

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A Table 2.5.4-207 Results of Unconfined Compression Tests on Rock* (continued)

Boring No.	Run Number	Sample Top Depth (ft)	Sample Length (L) (Inches)	Sample Diameter (D) (inches)	L/D Ratio	Unit Weight (pcf)	Type of Break ⁽¹⁾	Unconfined Compressive Strength (psi) ⁽²⁾	Young's Modulus (psi)	Poisson's Ratio
B-907	33	160.8	5.32	2.50	2.1	163	Columnar	19,333	7,700,000	0.30
B-907	40	200.0	5.35	2.50	2.1	165	C&S	20,166	(ND)	(ND)
B-908	2	67.5	5.32	2.38	2.2	163	Shear	5,476	(ND) 3	(ND)
B-908	4	79.4	5.25	2.39	2.2	164	C&S	14,695	3,400,000	0.41
B-908	7	96.0	5.31	2.39	2.2	163	Shear	17,164	(ND)	(ND)
B-908	11	112.7	5.32	2.38	2.2	178	Shear	15,284	(ND)	(ND)
B-908	17	135.7	5.28	2.38	2.2	187	Shear	5,670	3,180,000	0.21
B-908	20	146.8	5.31	2.38	2.2	173	Shear	7,687	(ND)	(ND)
B-909	11	82.4	5.32	2.39	2.2	176	C&S	9,464	3,520,000	* (4)
B-909	14	96.5	5.28	2.39	2.2	190	Shear	5,897	(ND)	(ND)
B-909	17	107.4	5.35	2.39	2.2	179	Shear	3,938	(ND)	(ND)
B-909	21	127.4	5.35	2.39	2.2	174	Shear	8,167	(ND)	(ND)
B-909	26	152.3	5.27	2.38	2.2	184	C&S	6,467	4,600,000	0.39
B-909	33	187.3	5.32	2.39	2.2	175	Shear	9,305	(ND)	(ND)
B-910	5	53.1	5.27	2.15	2.2	159	Shear	6,935	(ND)	(ND)
B-910	13	91.1	5.24	2.15	2.2	159	Shear	4,821	670,000	* (4)
B-910	20	120.9	5.27	2.40	2.2	163	Columnar	9,395	(ND)	(ND)
B-910	24	142.1	5.35	2.40	2.2	168	C&S	28,834	(ND)	(ND)

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

Table 2.5.4-207 Results of Unconfined Compression Tests on Rock* (continued)

Boring No.	Run Number	Sample Top Depth (ft)	Sample Length (L) (Inches)	Sample Diameter (D) (inches)	L/D Ratio	Unit Weight (pcf)	Type of Break ⁽¹⁾	Unconfined Compressive Strength (psi) ⁽²⁾	Young's Modulus (psi)	Poisson's Ratio
B-911	3	34.3	5.27	2.37	2.2	161	Shear	5,558	1,230,000	* (4)
B-911	5	44.3	5.28	2.38	2.2	162	Cone	10,209	(ND)	(ND)
B-911	10	66.5	5.35	2.39	2.2	164	Cone	24,646	(ND)	(ND)
B-911	13	82.1	5.36	2.40	2.2	164	C&S	20,431	5,730,000	0.40
B-911	16	97.6	5.36	2.40	2.2	163	Shear	6,561	(ND) ³	(ND)
B-912	3	37.1	5.32	2.39	2.2	170	C&S	3,524	2,570,000	(ND)
B-912	5	48.9	5.26	2.40	2.2	163	C&S	12,992	(ND)	(ND)
B-912	8	62.2	5.26	2.40	2.2	164	C&S	32,680	(ND)	(ND)
B-912	12	82.4	5.25	2.40	2.2	163	Shear	27,356	(ND)	(ND)
B-912	17	111.4	5.32	2.40	2.2	163	Shear	16,702	8,220,000	0.31
B-912	24	143.9	5.26	2.40	2.2	161	Columnar	15,996	(ND)	(ND)
B-914	8	63.8	5.34	2.40	2.2	169	Cone	17,866	(ND)	(ND)
B-914	10	75.3	5.32	2.40	2.2	164	C&S	36,600	(ND)	(ND)
B-914	15	95.8	5.37	2.40	2.2	164	C&S	29,776	8,980,000	0.31
B-914	20	120.6	5.32	2.39	2.2	169	C&S	17,942	(ND)	(ND)
B-914	26	151.4	5.31	2.40	2.2	166	C&S	16,517	8,930,000	0.32
B-914	34	192.7	5.32	2.40	2.2	163	Cone	30,162	(ND)	(ND)
B-918	2	31.7	5.29	2.39	2.2	164	Shear	19,038	(ND)	(ND)

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

NAPS COL 2.0-29-A Table 2.5.4-207 Results of Unconfined Compression Tests on Rock* (continued)

Boring No.	Run Number	Sample Top Depth (ft)	Sample Length (L) (Inches)	Sample Diameter (D) (inches)	L/D Ratio	Unit Weight (pcf)	Type of Break ⁽¹⁾	Unconfined Compressive Strength (psi) ⁽²⁾	Young's Modulus (psi)	Poisson's Ratio
B-918	4	37.1	5.32	2.40	2.2	164	C&S	29,636	9,530,000	0.35
B-918	7	51.6	5.29	2.40	2.2	165	Cone	15,409	(ND)	(ND)
B-918	9	60.7	5.32	2.40	2.2	164	Columnar	21,064	(ND)	(ND)
B-918	15	88.1	5.28	2.40	2.2	165	Shear	21,944	7,850,000	0.24
B-918	22	122.0	5.25	2.40	2.2	166	C&S	33,610	(ND)	(ND)
B-920	7	90.2	5.28	2.39	2.2	160	Shear	1,021	(ND)	(ND)
B-920	11	107.7	5.32	2.39	2.2	163	Cone	29,621	8,500,000	0.34
B-920	13	119.1	5.33	2.39	2.2	181	Shear	9,456	(ND)	(ND)
B-920	18	141.1	5.35	2.40	2.2	166	Cone	18,040	5,970,000	* ⁽⁴⁾
B-923	6	20.0	5.35	2.39	2.2	164	C&S	28,911	8,510,000	0.28
B-923	9	30.8	5.35	2.39	2.2	162	C&S	26,779	(ND)	(ND)
B-923	12	45.7	5.33	2.39	2.2	163	Shear	13,477	(ND)	(ND)
B-923	16	65.7	5.35	2.39	2.2	164	Cone	21,069	7,150,000	0.29
B-924	1	21.7	5.33	2.39	2.2	162	Shear	10,588	(ND) ³	(ND)
B-924	3	30.2	5.35	2.39	2.2	163	C&S	15,110	(ND)	(ND)
B-924	6	44.0	5.33	2.39	2.2	174	Shear	6,384	2,620,000	* ⁽⁴⁾
B-924	12	75.1	5.33	2.40	2.2	179	C&S	5,681	(ND)	(ND)
B-927	2	43.0	5.35	2.39	2.2	163	C&S	19,288	(ND)	(ND)

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

Table 2.5.4-207 Results of Unconfined Compression Tests on Rock* (continued)

Boring No.	Run Number	Sample Top Depth (ft)	Sample Length (L) (Inches)	Sample Diameter (D) (inches)	L/D Ratio	Unit Weight (pcf)	Type of Break ⁽¹⁾	Unconfined Compressive Strength (psi) ⁽²⁾	Young's Modulus (psi)	Poisson's Ratio
B-927	6	51.6	5.35	2.39	2.2	163	C&S	27,239	6,550,000	0.49
B-927	13	74.9	5.33	2.39	2.2	164	Cone	30,297	(ND)	(ND)
B-927	18	96.3	5.35	2.39	2.2	164	C&S	28,266	(ND)	(ND)
B-928	2	52.6	5.33	2.39	2.2	153	Shear	1,318	(ND)	(ND)
B-928	6	74.7	5.35	2.39	2.2	162	Cone	20,333	5,070,000	0.35
B-933	3	50.5	5.33	2.39	2.2	163	Cone	19,395	(ND)	(ND)
B-933	7	66.6	5.34	2.38	2.2	162	Columnar	15,764	8,600,000	* ⁽⁴⁾
B-933	11	90.1	5.32	2.39	2.2	164	Cone	30,993	(ND)	(ND)
B-948	6	56.8	5.28	2.39	2.2	162	C&S	17,089	(ND)	(ND)
B-948	10	76.1	5.25	2.40	2.2	167	C&S	22,435	(ND)	(ND)
M-10 (DH)	R-7	117.45	5.15	2.41	2.1	160.1	S	7960	(ND)	(ND)
M-10 (DH)	R-10	133.75	5.09	2.41	2.1	161.9	S	19640 ⁽⁵⁾	(ND)	(ND)
M-10 (DH)	R-15	153.7	5.08	2.41	2.1	163.5	C	33830 ⁽⁵⁾	(ND)	(ND)
M-10 (DH)	R-20	177.6	5.14	2.39	2.2	163.3	S	20880 ⁽⁵⁾	(ND)	(ND)
M-10 (DH) ⁽⁶⁾	R-24	196.7	5.18	2.39	2.2	163.7	C	30780	(ND)	(ND)
M-30 (DH) ⁽⁶⁾	R-4	57	5.18	2.4	2.2	162.8	C	28650	(ND)	(ND)
M-30 (DH)	R-18	95.4	5.06	2.39	2.1	162.7	C	23700	(ND)	(ND)

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

Table 2.5.4-207 Results of Unconfined Compression Tests on Rock* (continued)

Boring No.	Run Number	Sample Top Depth (ft)	Sample Length (L) (Inches)	Sample Diameter (D) (inches)	L/D Ratio	Unit Weight (pcf)	Type of Break ⁽¹⁾	Unconfined Compressive Strength (psi) ⁽²⁾	Young's Modulus (psi)	Poisson's Ratio
M-30 (DH) ⁽⁶⁾	R-26	134.9	5.26	2.39	2.2	163.7	S	26200	(ND)	(ND)
M-30 (DH)	R-34	166.9	5.06	2.4	2.1	164.6	C/S	24820	(ND)	(ND)
M-30 (DH)	R-40	197.05	5.16	2.4	2.2	162.6	C	33040	(ND)	(ND)

(1) Type of Breaks: Columnar; Cone (C); Shear (S); Cone & Shear (C&S)

(2) Unconfined compressive strength corrected for L/D Ratio

Compressive strength testing was performed in general accordance with ASTM D7012-04.

(3) (ND) indicates that information was not determined

(4) Value of Poisson's ratio is greater than 0.5 which indicates inelastic behavior probably due to presence of fractures or discontinuities affecting lateral strain.

(5) Test duration exceeded 15 minute maximum time as indicated by ASTM D 7012-07e1.

(6) Core samples did not meet the dimensional tolerances for straightness or perpendicularity per ASTM D 4543-08.

* From [Appendices 2.5.4AA](#), [2.5.4BB](#), and [2.5.4CC](#)

Table 2.5.4-208 Best Estimate 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
USCS symbol	GW	-	SM, SC	SM	-	-	-
Total unit weight, g (pcf)	130	145	125	130	150	163	164
Fines Content (%)	6-12	-	25	20	-	-	-
Natural water content, w_N (%)	-	-	19	15	-	-	-
Atterberg limits		-					
Liquid limit, LL	-	-	-	-	-	-	-
Plastic limit, PL	-	-	-	-	-	-	-
Plasticity index, PI	-	-	-	-	-	-	-
Measured SPT N-value (blows/ft)	-	-	15	75	Ref	-	-
Adjusted SPT N_{60} -value (blows/ft)	50	-	20	100	Ref	-	-
Undrained properties							
Undrained shear strength, s_u (ksf)	-	-	-	-	-	-	-
Unconfined compressive strength, q_u (ksi)	-	2.5	-	-	1.0	9.0	17.0

Table 2.5.4-208 **Best Estimate Engineering Properties of Subsurface Materials** (*continued*)

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, ϕ' (degrees)	40	-	33	40	-	-	-
Shear wave velocity, V_s (ft/sec)	1,100	7,000	850	1,600	3,000	6,000	9,000
Compression wave velocity, V_p (ft/sec)	2,100	10,900	1,800	3,500	7,300	12,000	16,000
Poisson's ratio, ν (high strain)	0.3	0.15	0.35	0.3	0.4	0.33	0.27
Poisson's ratio, ν (low strain)	0.3	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
Consolidation characteristics							
Compression ratio, CR		-			-	-	-
Recompression ratio, RR		-			-	-	-

Table 2.5.4-208 **Best Estimate Engineering Properties of Subsurface Materials** (*continued*)

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
Coefficient of subgrade reaction, k_1 (kcf)	2,000	-	260	2,000	-	-	-
Coefficient of sliding	0.55	0.7	0.35	0.45	0.6	0.65	0.7
Static earth pressure coefficients							
Active, K_a	0.22	-	0.30	0.22	-	-	-
Passive, K_p	4.60	-	3.40	4.60	-	-	-
At-rest, K_0	0.36	-	0.50	0.36	-	-	-
Optimum moisture content, w_{opt} (%)	-	-	14	-	-	-	-
Maximum dry unit weight, g_{max} (pcf)	-	-	116	-	-	-	-
Rock Quality Designation, RQD (%)	-	-	-	-	20	65	95

NAPS ESP COL 2.5-6 Table 2.5.4-209 Size, Depth and Loading of Structures

Structure	Seismic Category ⁽²⁾	Approximate Dimensions (ft)	Bottom of Foundation Elevation ⁽¹⁾ (ft)	Embedment Depth (ft)	Design Load (ksf)	
					Static	Dynam-ic
Reactor/Fuel Building	I	161 x 230	223.9	65.6	14.6	23.0
Control Building	I	78 x 99	240.6	48.9	6.1	8.8
Fire Water Service Complex	I	66 x 171	281.8	7.7	3.45	25.1
Turbine Building	II	194 x 377	263.6	25.9	6	—
Radwaste Building	NS	108 x 213	237.5	52.0	6	—
Service Building	II	111 x 163	274.1	15.4	4	—
Ancillary Diesel Building	II	61 x 71	286.2	3.3	4	—

Note: (1) The bottom of foundation is derived from the finished ground level grade at Elevation 289.5 ft.

(2) See [DCD Table 3.2-1](#).

NAPS ESP COL 2.5-6 Table 2.5.4-210 Allowable Bearing Capacities of Rock and Concrete

Rock Type	Unconfined Compressive Strength, q_u (ksi)	$q_a = 0.2 q_u$ (ksf)	Recommended q_a (ksf)
Zone III	1	29	20
Zone III-IV	9	259	80
Zone IV	17	490	160
Concrete	2.5	40	199

NAPS ESP COL 2.5-6 Table 2.5.4-211 **Bearing Demand and Capacity**

Structure	Calculated Allowable Static (Dynamic) Bearing Capacity, q_a (ksf)					Selected q_a (ksf)	
	Structural Fill ⁽¹⁾	Concrete Fill	Zone III	Zone III-IV	Zone IV	Static	Dynamic
Reactor/Fuel Building	—	199 (199)	—	80 (259)	160 (490)	80	199
Control Building	—	199 (199)	—	80 (259)	160 (490)	80	199
Fire Water Service Complex	—	199 (199)	—	80 (259)	160 (490)	80	199
Turbine Building	242 (323)	—	20 (29)	80 (259)	160 (490)	20	29
Radwaste Building	214 (285)	—	20 (29)	80 (259)	160 (490)	20	29
Service Building	135 (180)	—	20 (29)	80 (259)	160 (490)	20	29
Ancillary Diesel Building	58 (78)	—	20 (29)	80 (259)	160 (490)	20	29

Note:

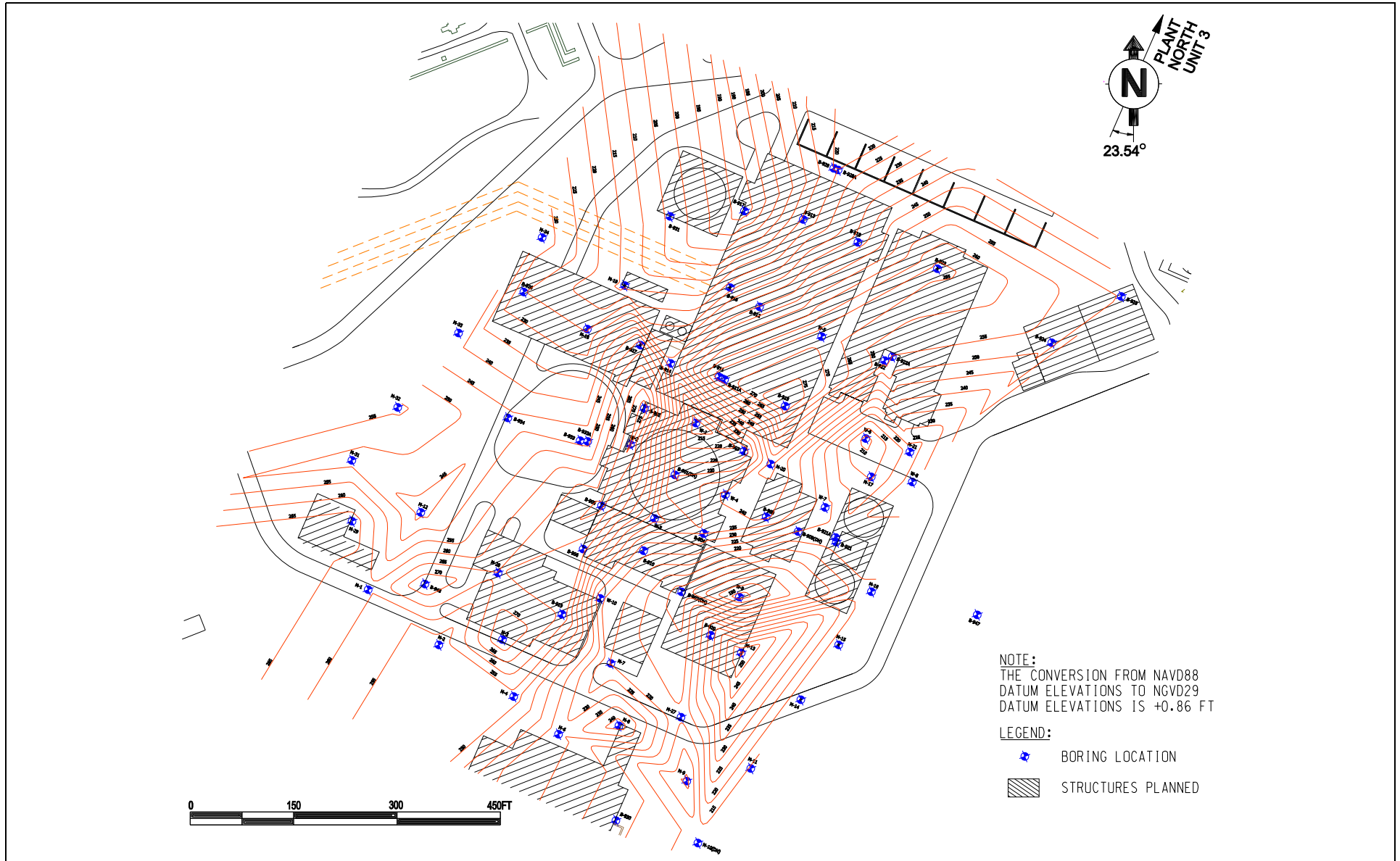
(1) This value also applies to structures on Zone IIB, where applicable.

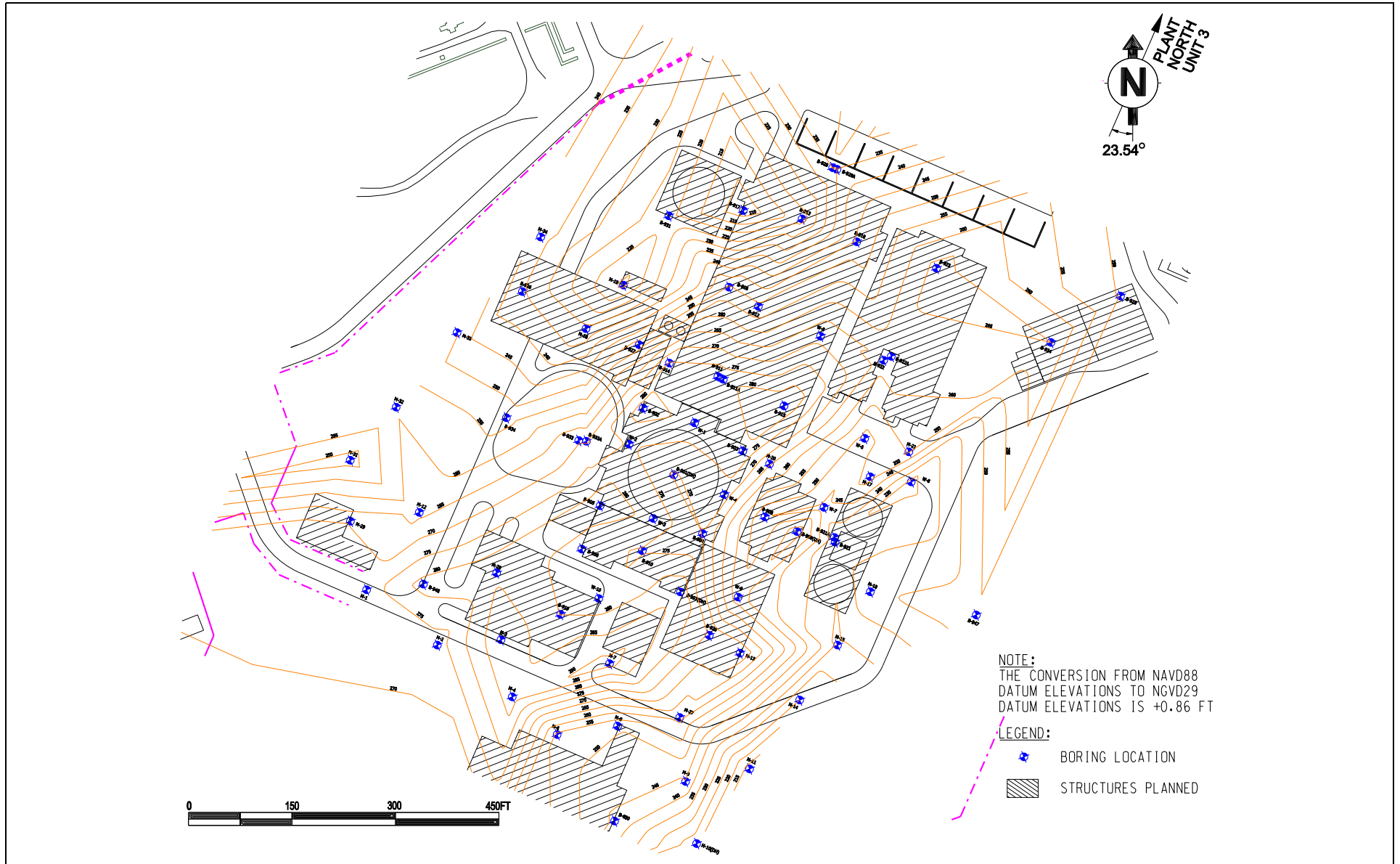
NAPS ESP COL 2.5-6 Table 2.5.4-212 Estimated Settlements

Structure	Applied Load (ksf)	Calculated Settlement (in.)			
		Center	Edge	Average⁽¹⁾	Corner
Reactor/Fuel Building	14.6	0.12	0.075	0.10	0.05
Control Building	6.1	0.022	0.014	0.018	0.010
Fire Water Service Complex	3.45	0.011	0.008	0.010	0.005
Turbine Building	6	2.23	1.14	1.69	0.58
Radwaste Building	6	0.75	0.38	0.57	0.27
Service Building	4	0.68	0.35	0.52	0.27
Ancillary Diesel Building	4	0.13	0.065	0.10	0.034

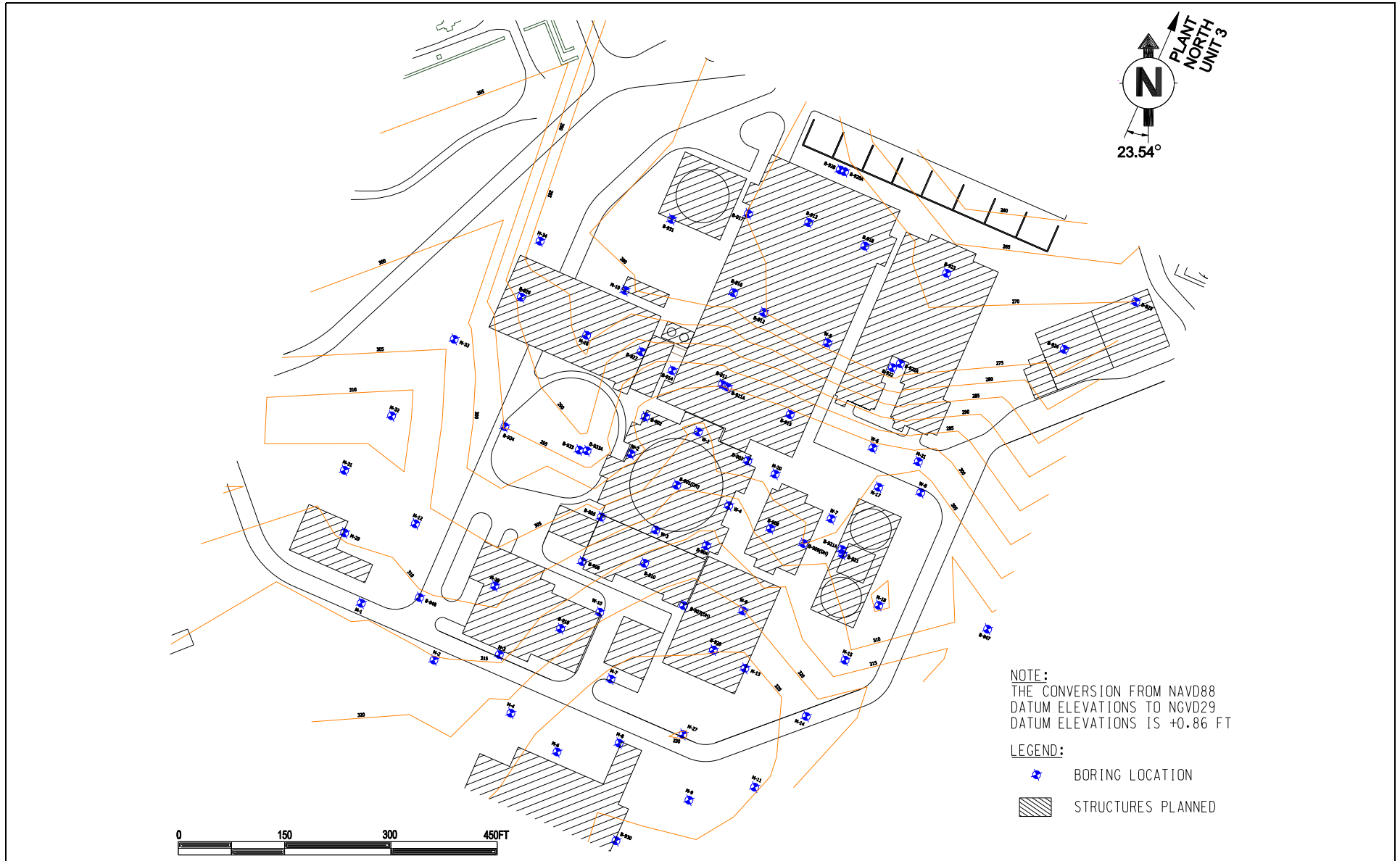
Note: (1) Average is average of center and edge settlements







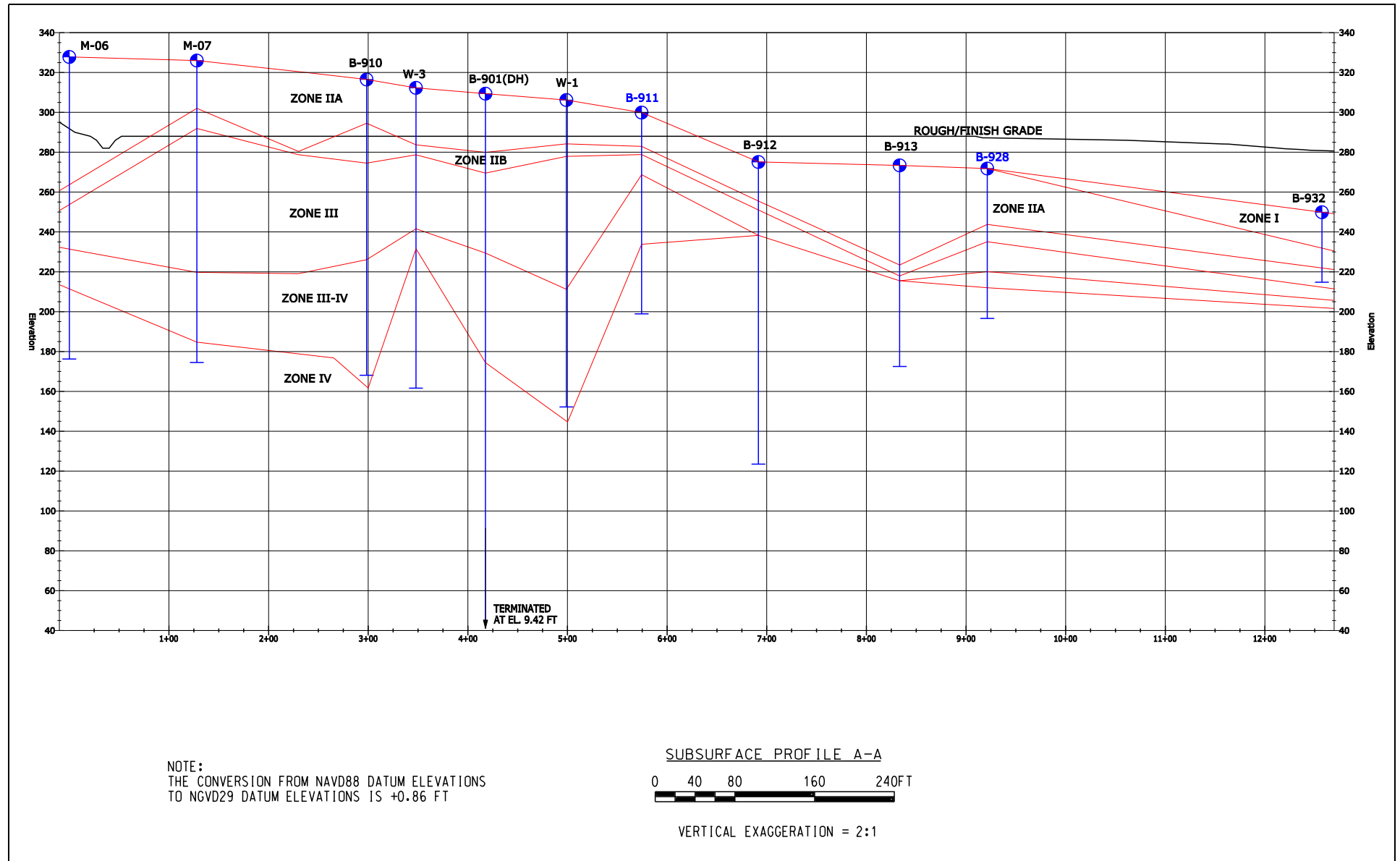




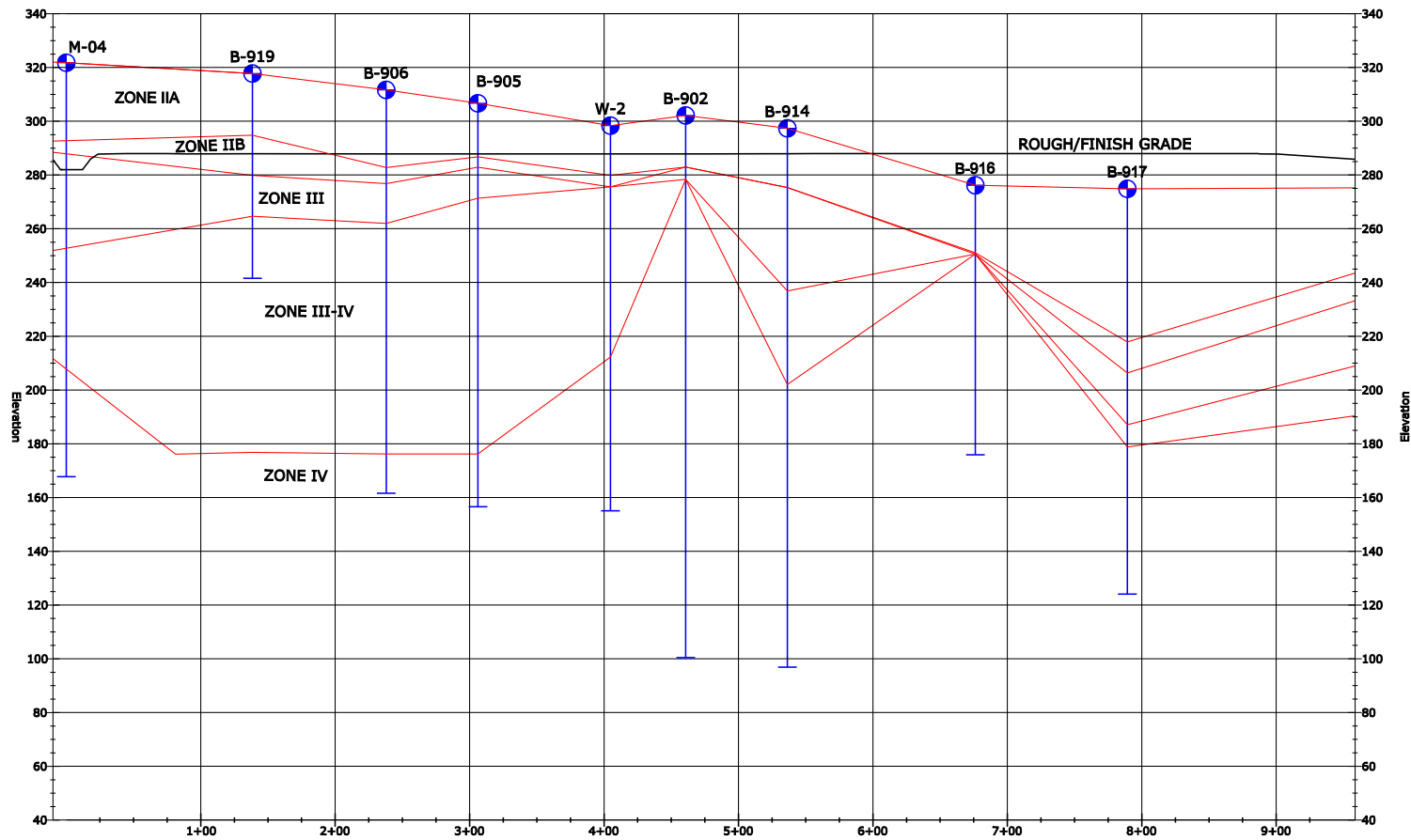
NAPS COL 2.0-29-A Figure 2.5.4-206 Excavation Plan and Locations of Profiles



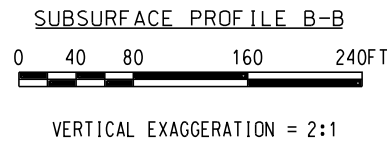
NAPS COL 2.0-29-A Figure 2.5.4-207 Subsurface Profile A-A



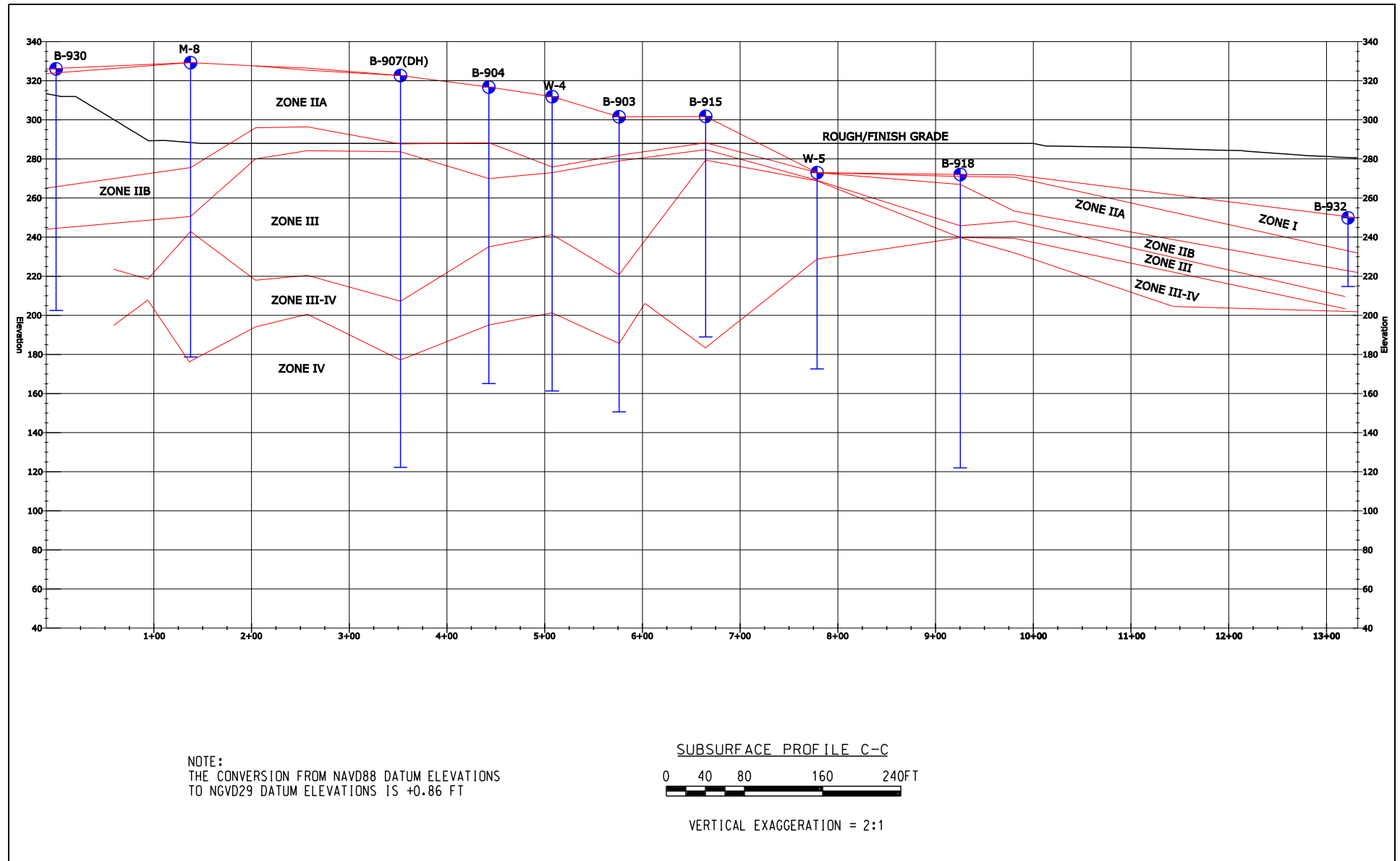
NAPS COL 2.0-29-A Figure 2.5.4-208 Subsurface Profile B-B



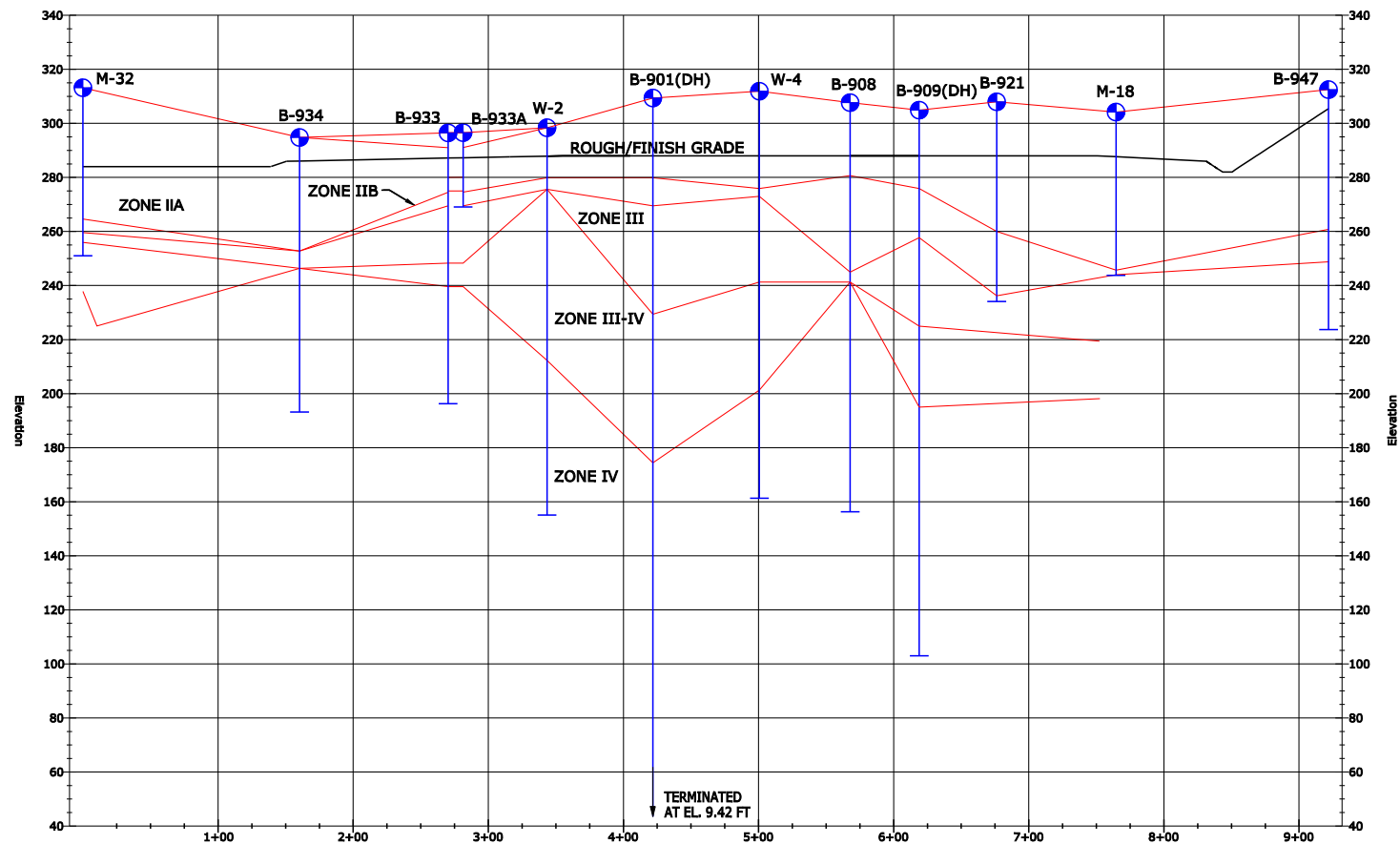
NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT



NAPS COL 2.0-29-A Figure 2.5.4-209 Subsurface Profile C-C



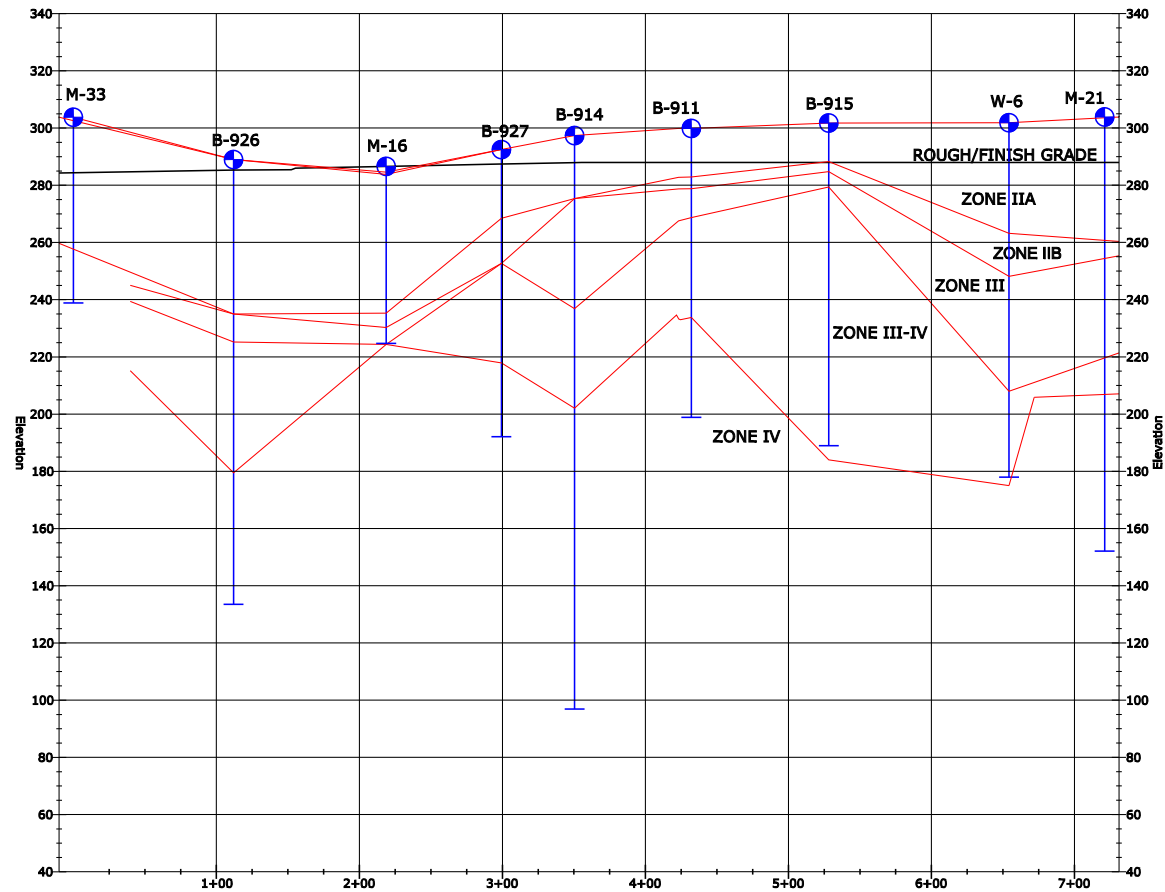
NAPS COL 2.0-29-A Figure 2.5.4-210 Subsurface Profile D-D



NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT

SUBSURFACE PROFILE D-D
0 40 80 160 240 FT
VERTICAL EXAGGERATION = 2:1

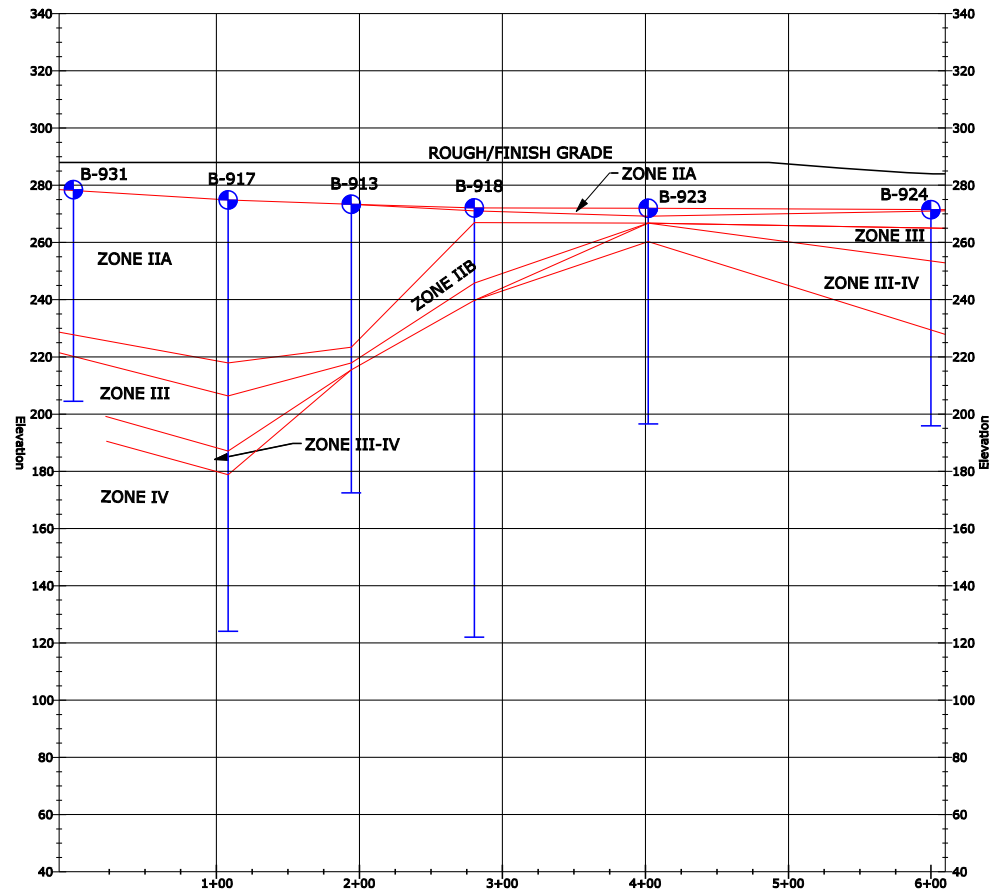
NAPS COL 2.0-29-A Figure 2.5.4-211 Subsurface Profile E-E



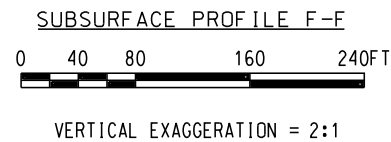
NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT

SUBSURFACE PROFILE E-E
0 40 80 160 240FT
VERTICAL EXAGGERATION = 2:1

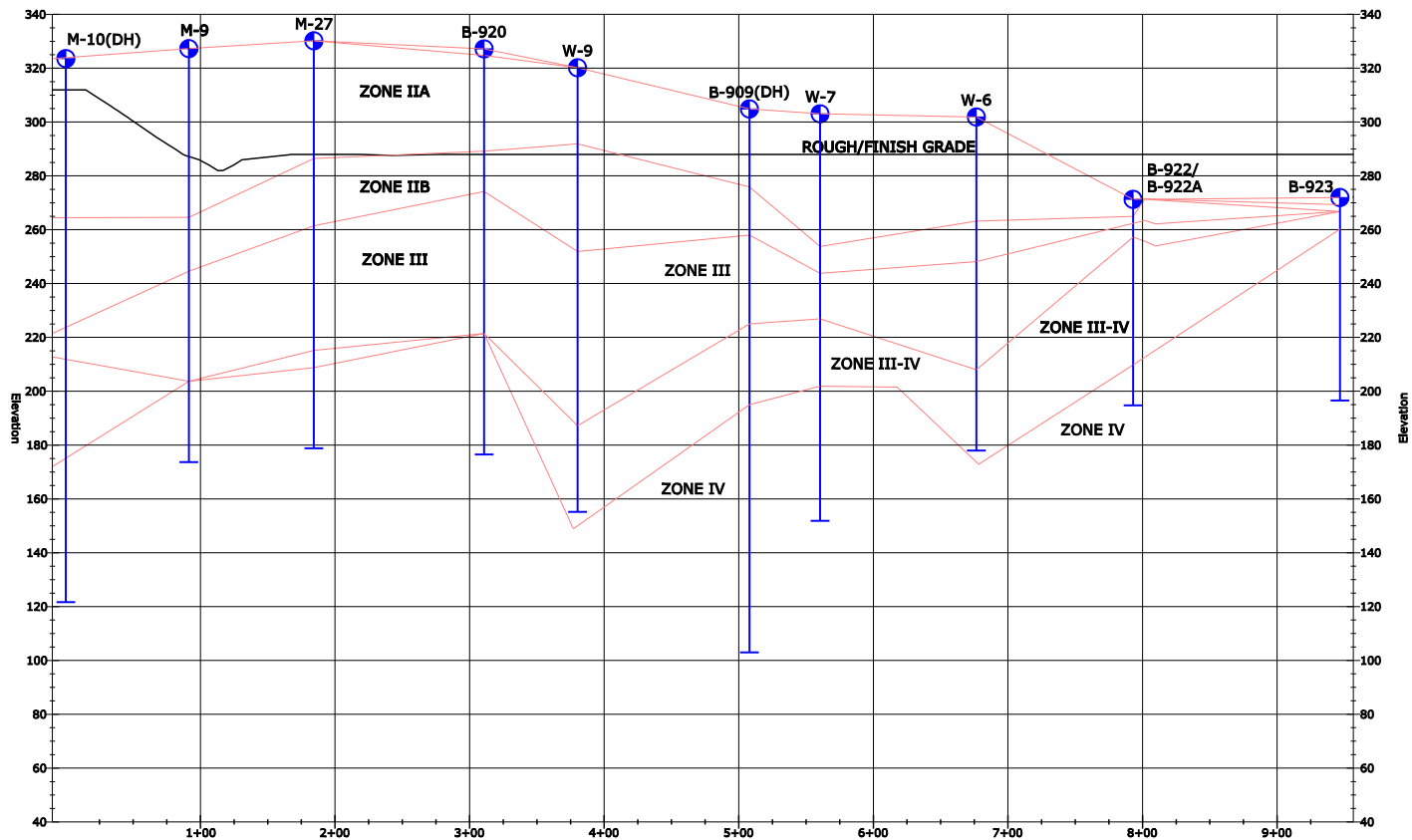
NAPS COL 2.0-29-A Figure 2.5.4-212 Subsurface Profile F-F



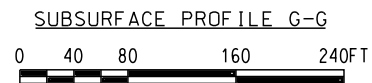
NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT



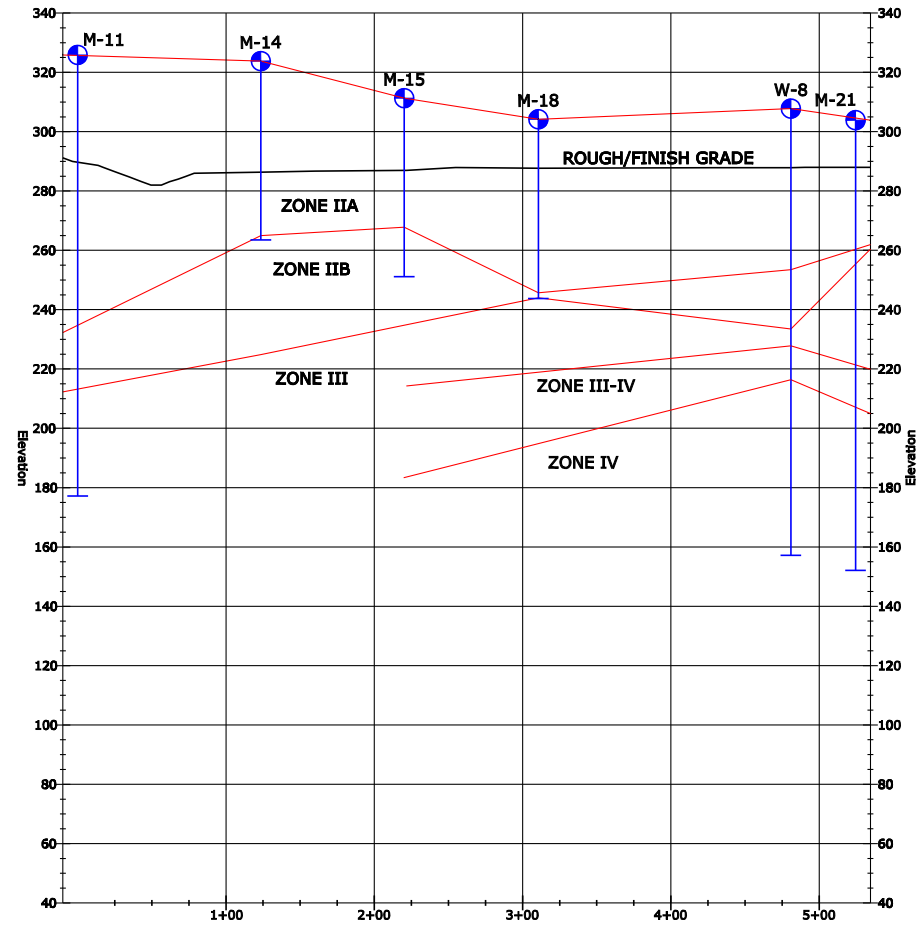
NAPS COL 2.0-29-A Figure 2.5.4-213 Subsurface Profile G-G



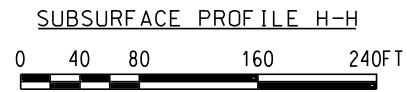
NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT



VERTICAL EXAGGERATION = 2:1

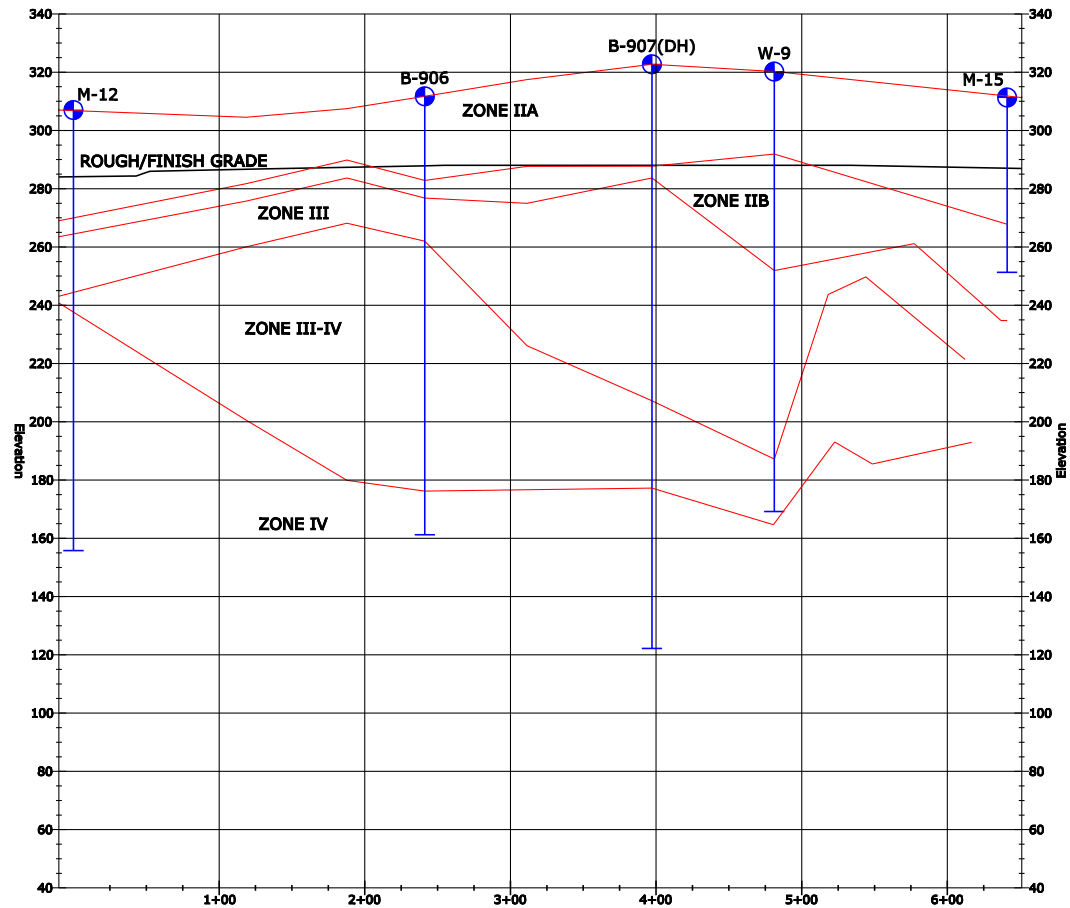


NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT

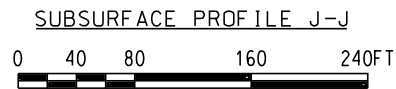


VERTICAL EXAGGERATION = 2:1

NAPS COL 2.0-29-A Figure 2.5.4-215 Subsurface Profile J-J

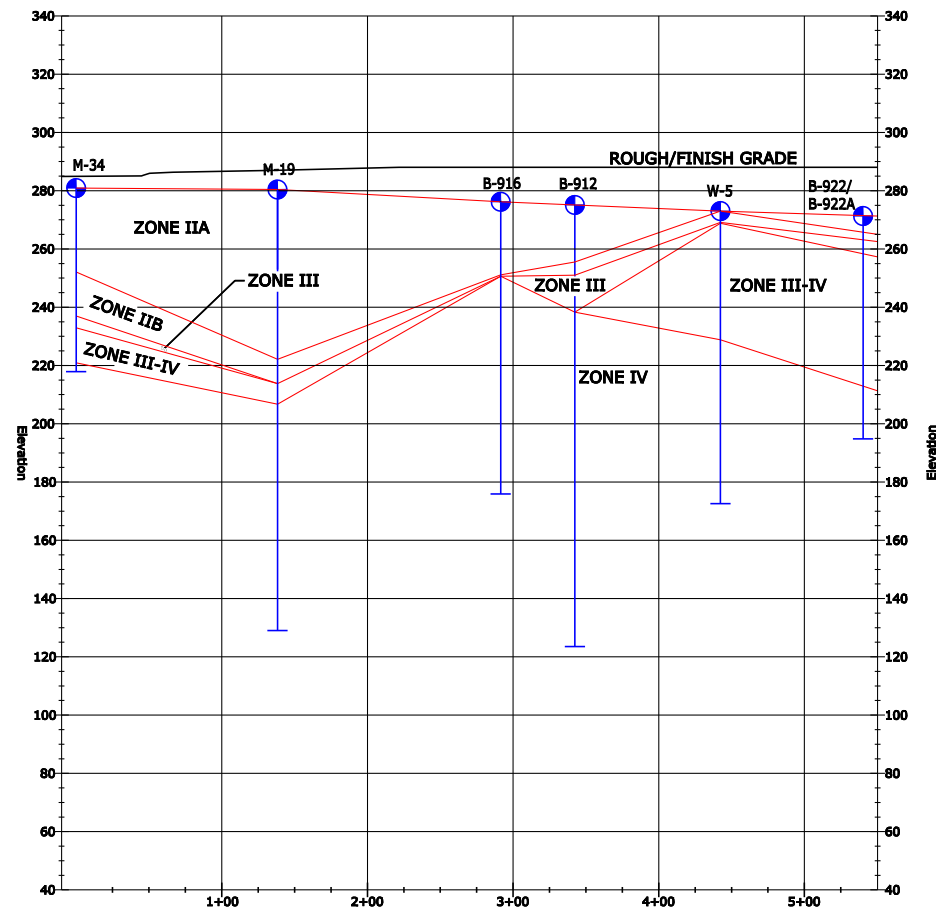


NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT



VERTICAL EXAGGERATION = 2:1

NAPS COL 2.0-29-A Figure 2.5.4-216 Subsurface Profile K-K



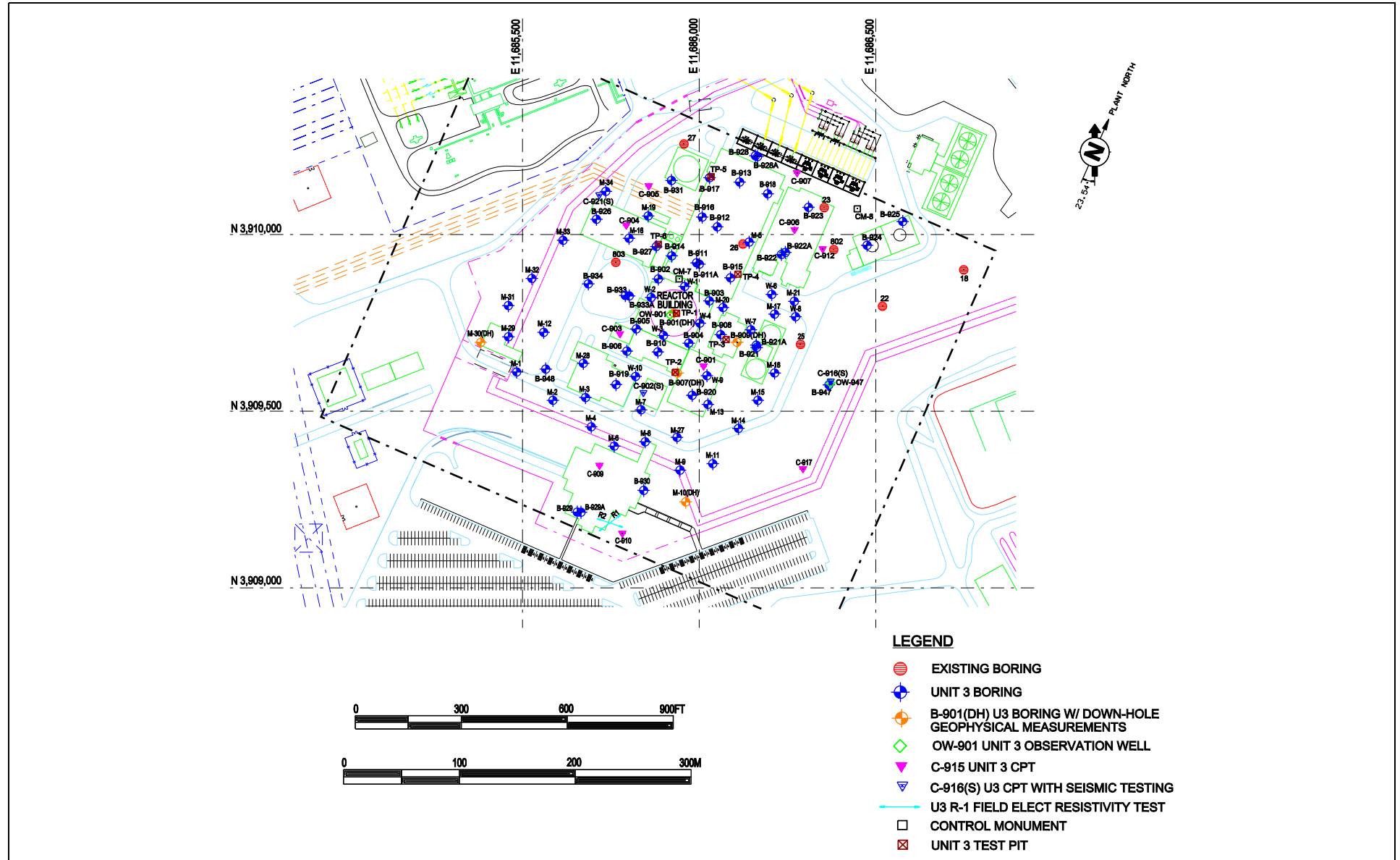
NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT

SUBSURFACE PROFILE K-K

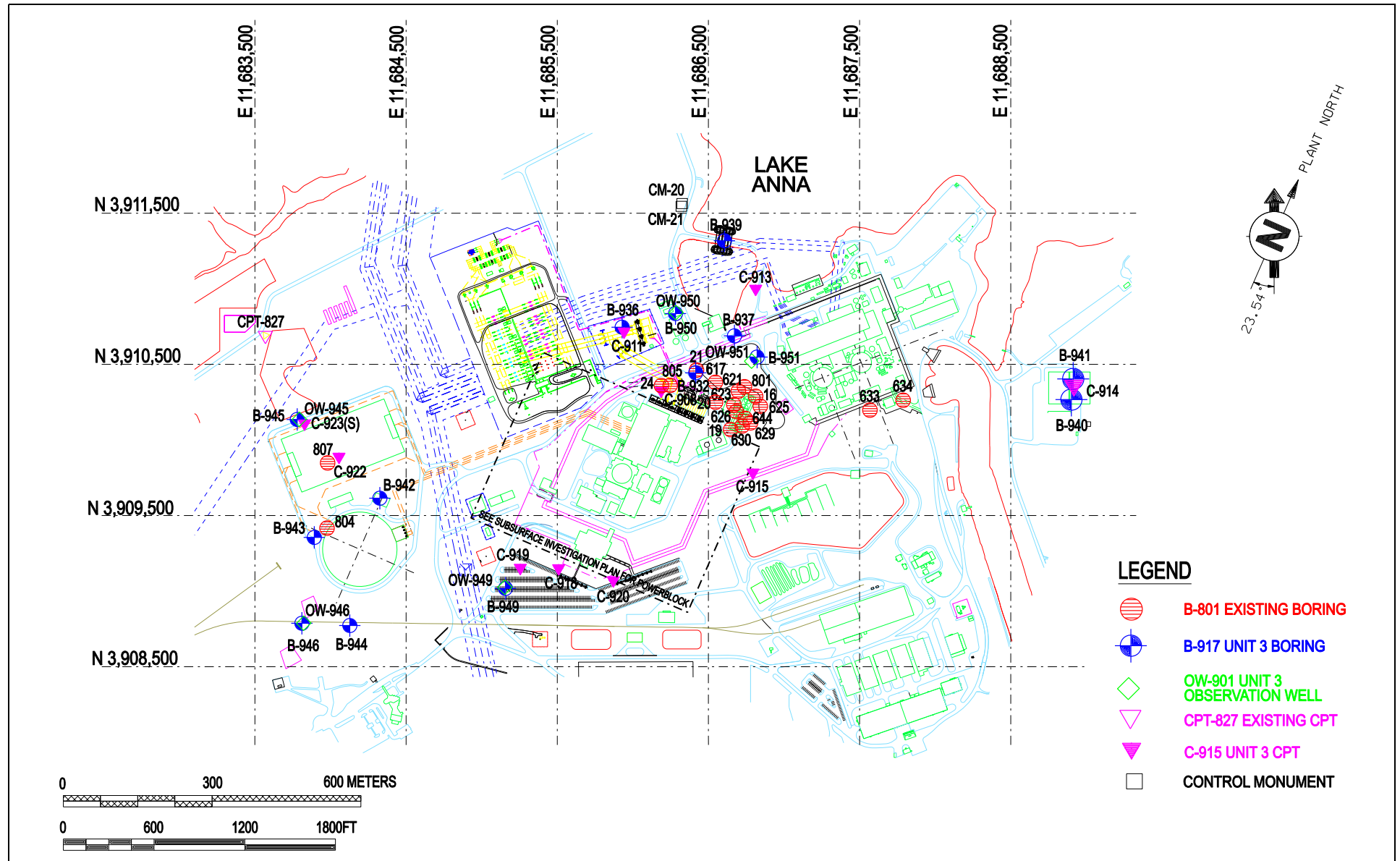


VERTICAL EXAGGERATION = 2:1

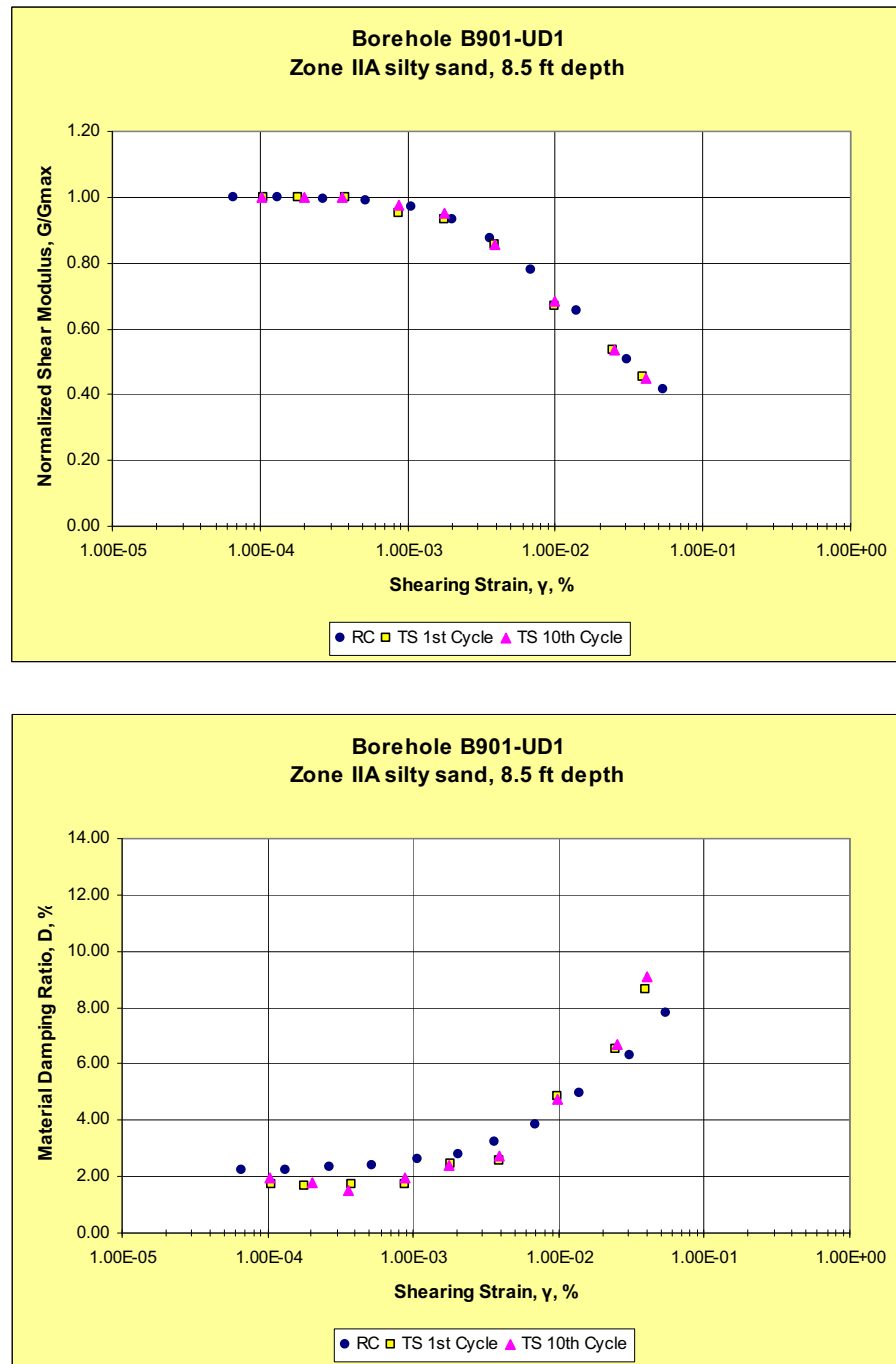
NAPS COL 2.0-29-A Figure 2.5.4-217 Unit 3 Boring Locations – Power Block



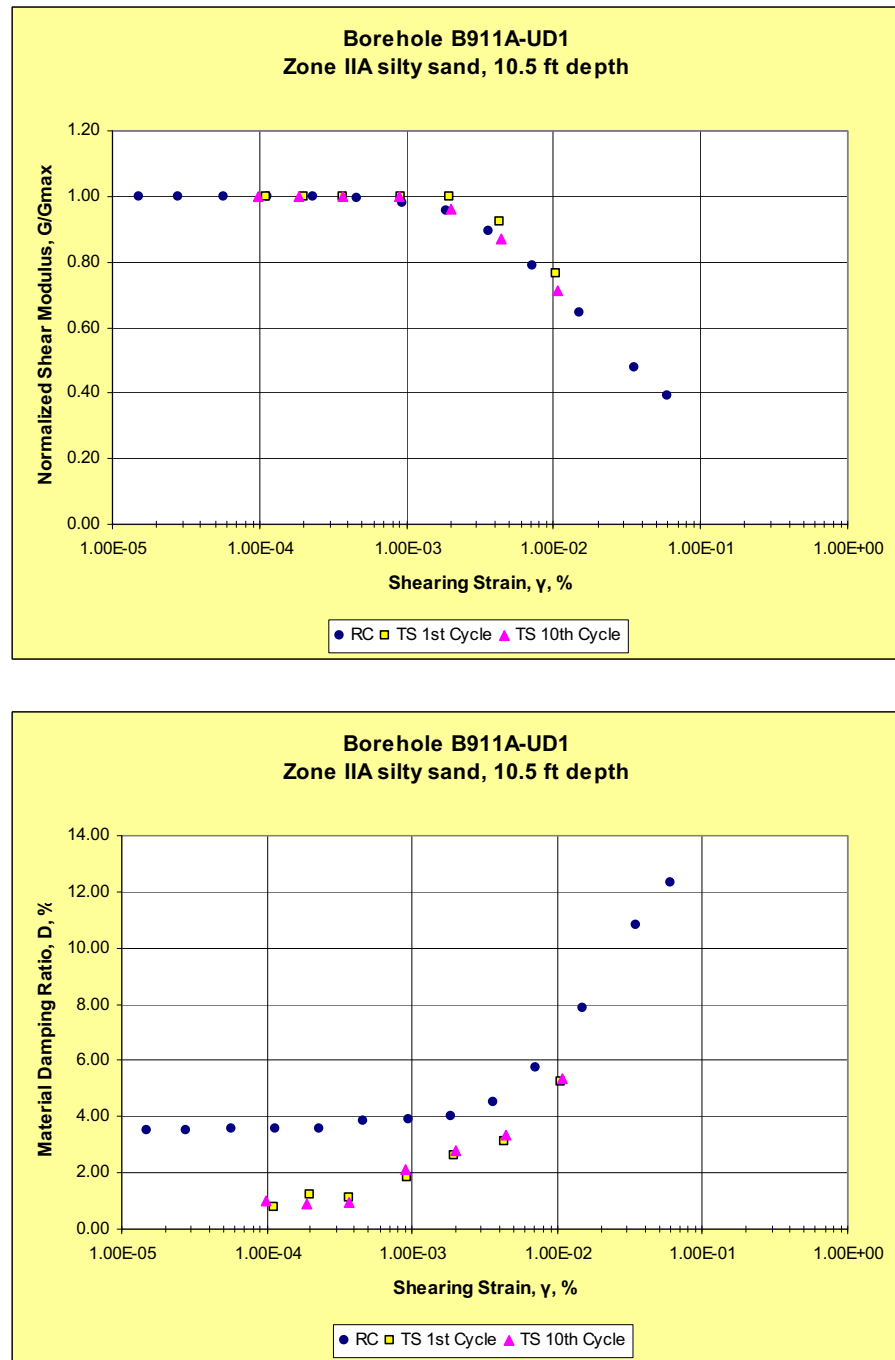
NAPS COL 2.0-29-A Figure 2.5.4-218 Unit 3 Boring Locations – Site



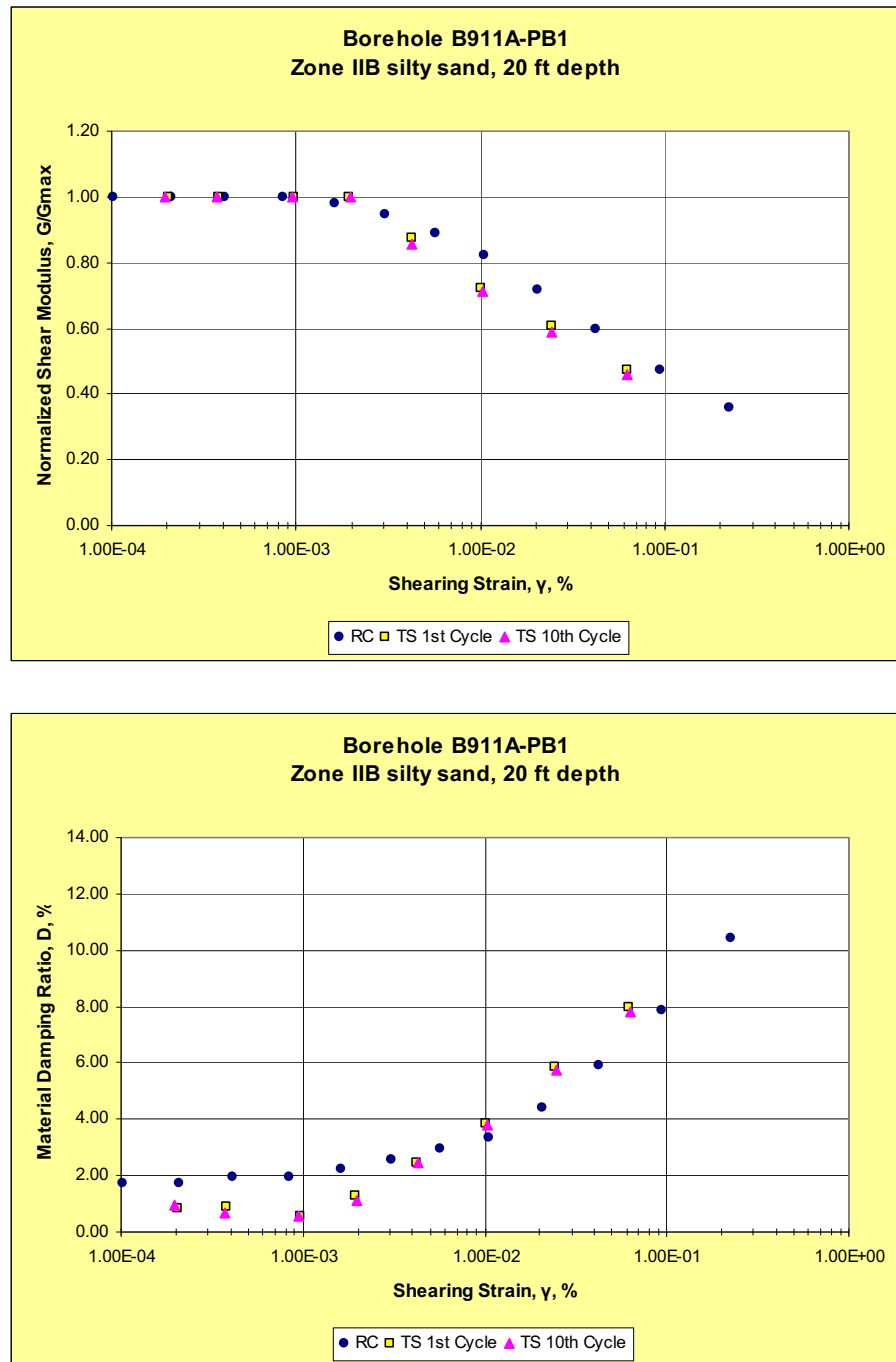
NAPS COL 2.0-29-A Figure 2.5.4-219 **RCTS Test Results (Sheet 1 of 3)**
 G/G_{max} and D vs. Strain, B-901 UD-1: 4.3 psi
Confining Pressure



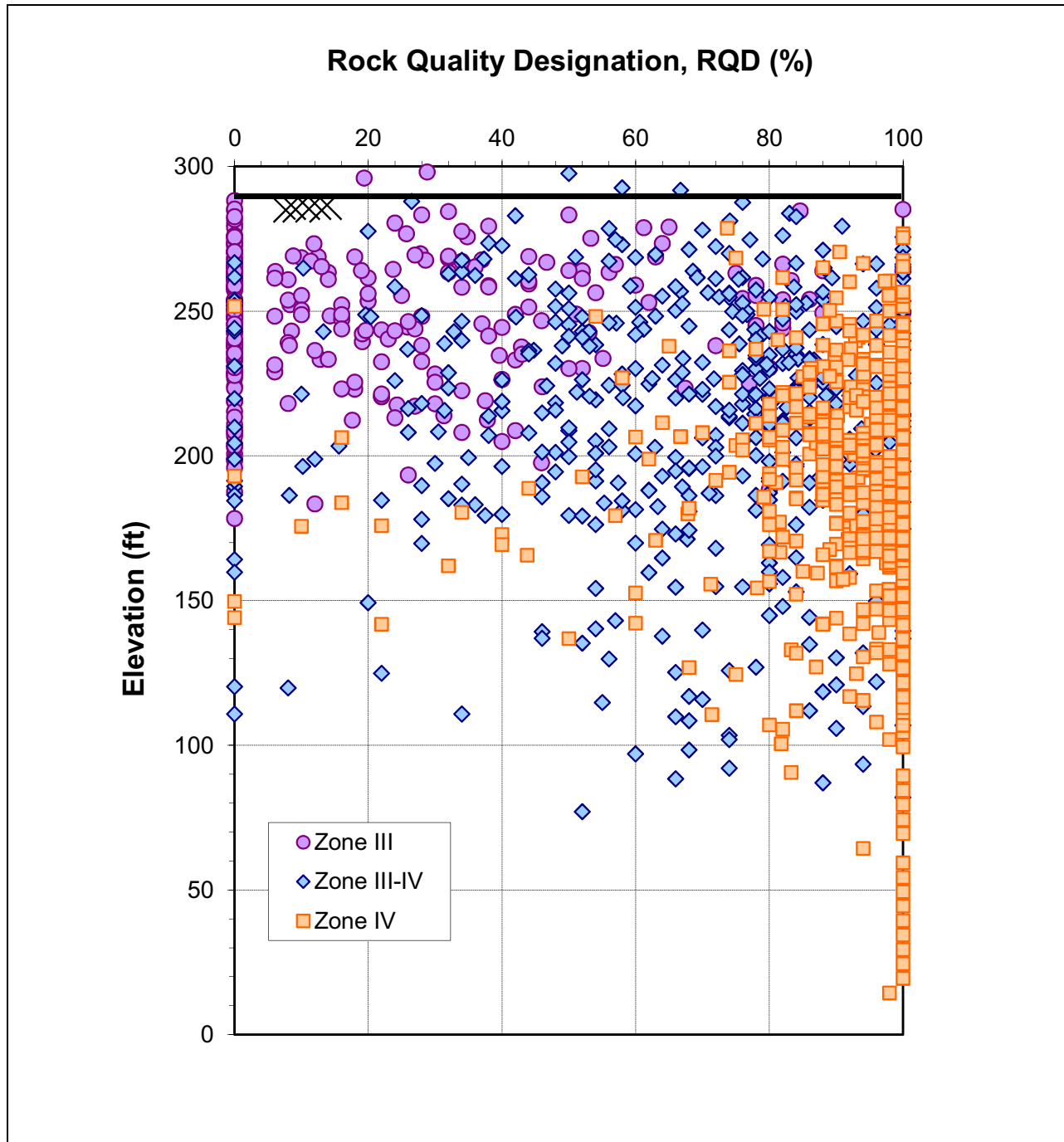
NAPS COL 2.0-29-A Figure 2.5.4-219 **RCTS Test Results (Sheet 2 of 3)**
 G/G_{max} and D vs. Strain, B-911A UD-1: 5.6 psi
Confining Pressure



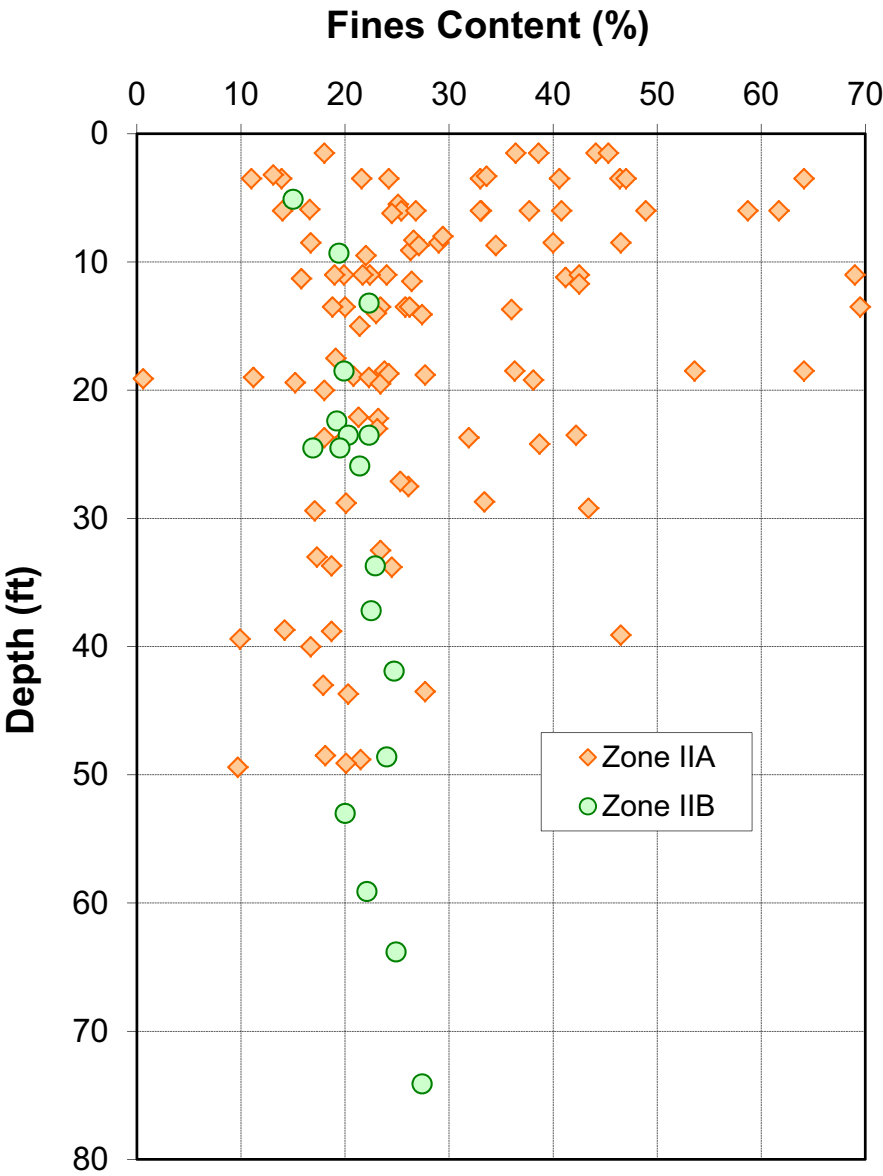
NAPS COL 2.0-29-A Figure 2.5.4-219 **RCTS Test Results (Sheet 3 of 3)**
 $G/G_{\max}$ and D vs. Strain, B-911A PB-1: 11.4 psi
Confining Pressure



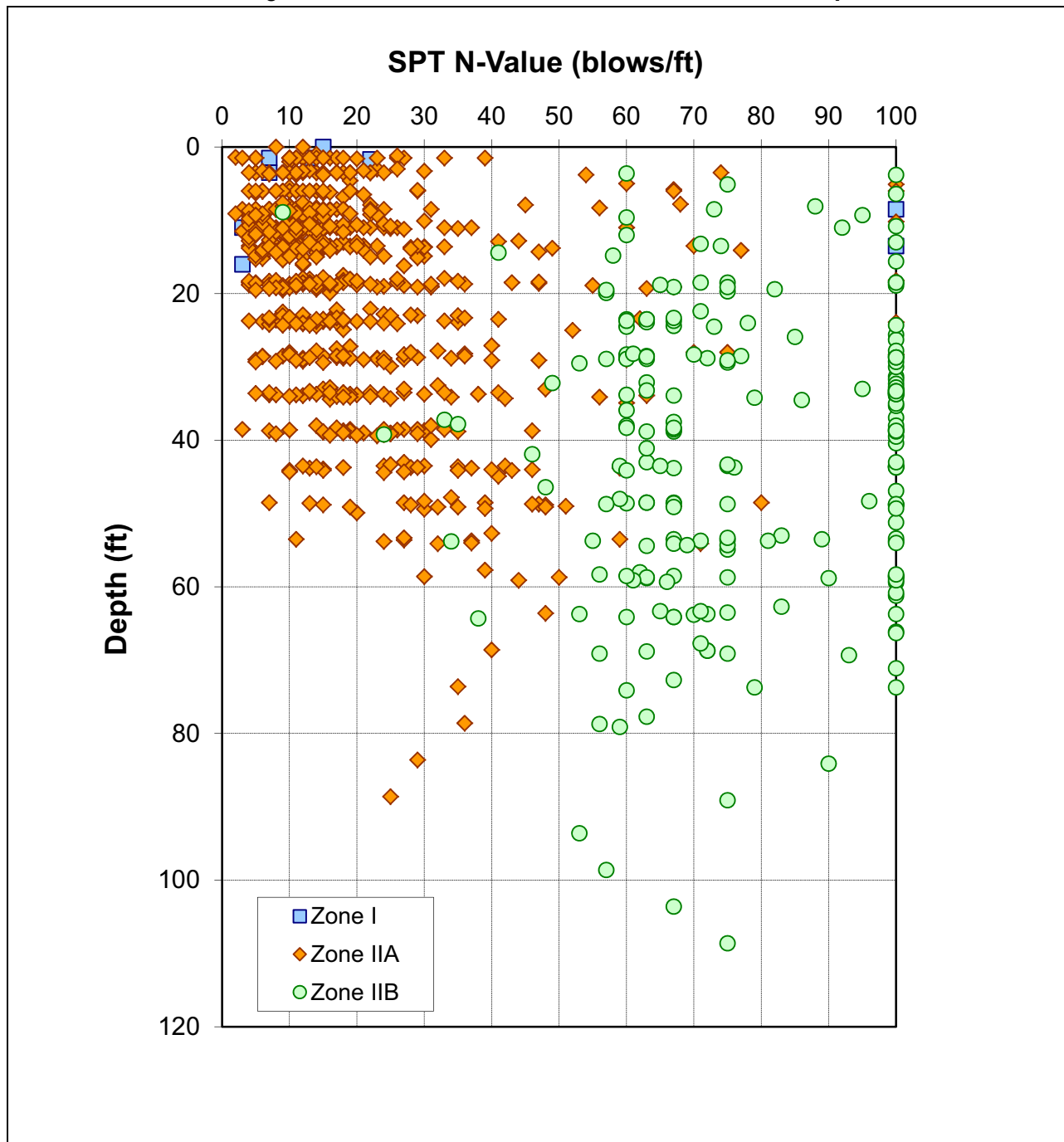
NAPS COL 2.0-29-A Figure 2.5.4-220 Rock Quality Designation versus Elevation



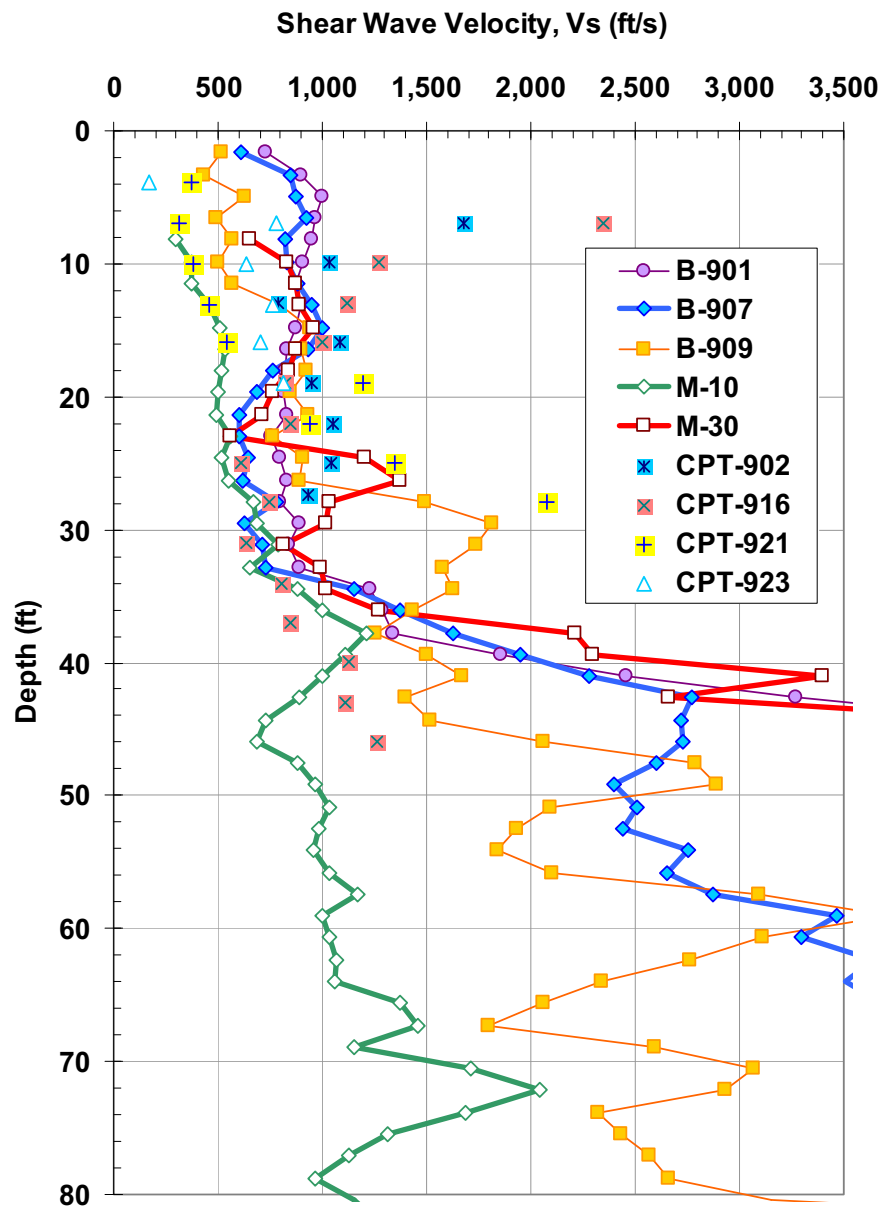
NAPS COL 2.0-29-A Figure 2.5.4-221 **Fines Content of Saprolite versus Depth**



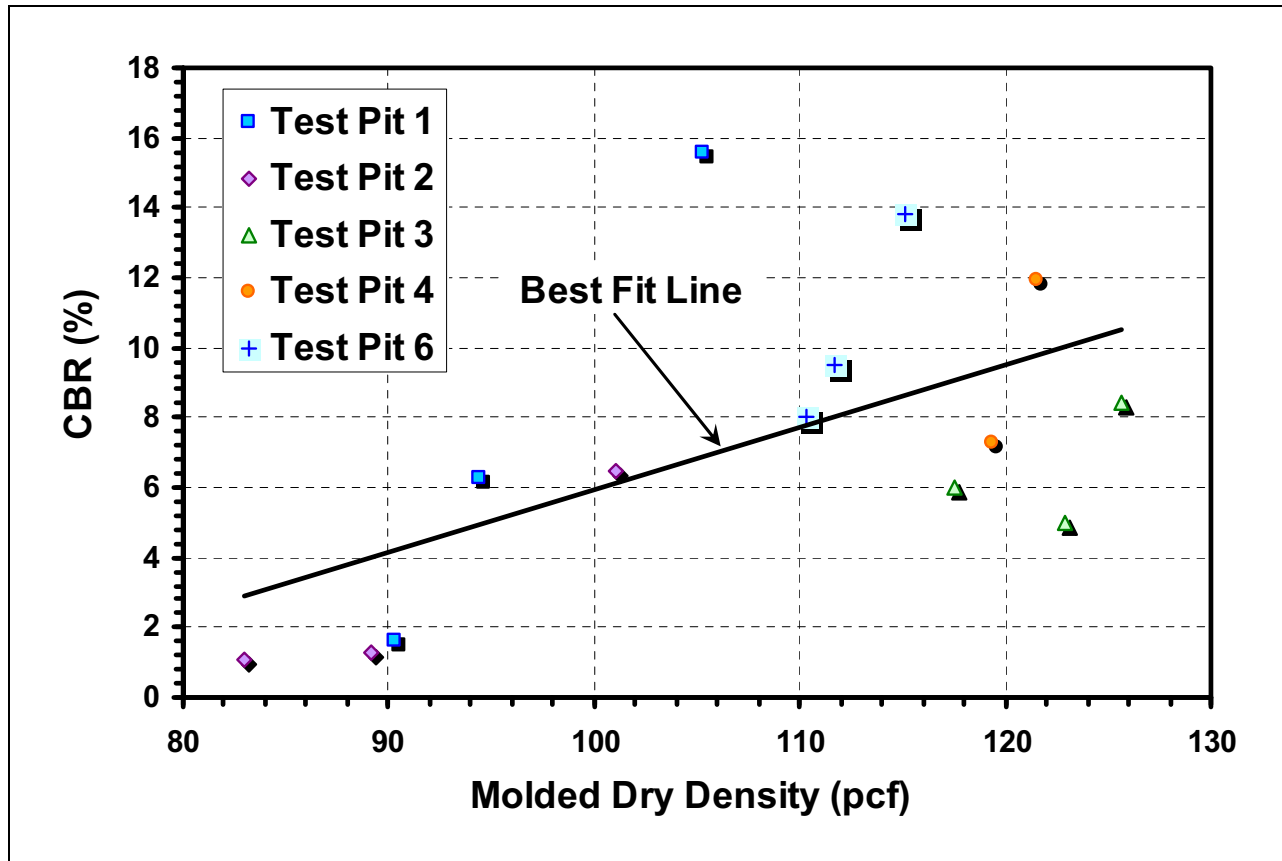
NAPS COL 2.0-29-A Figure 2.5.4-222 Measured SPT N-Value versus Depth



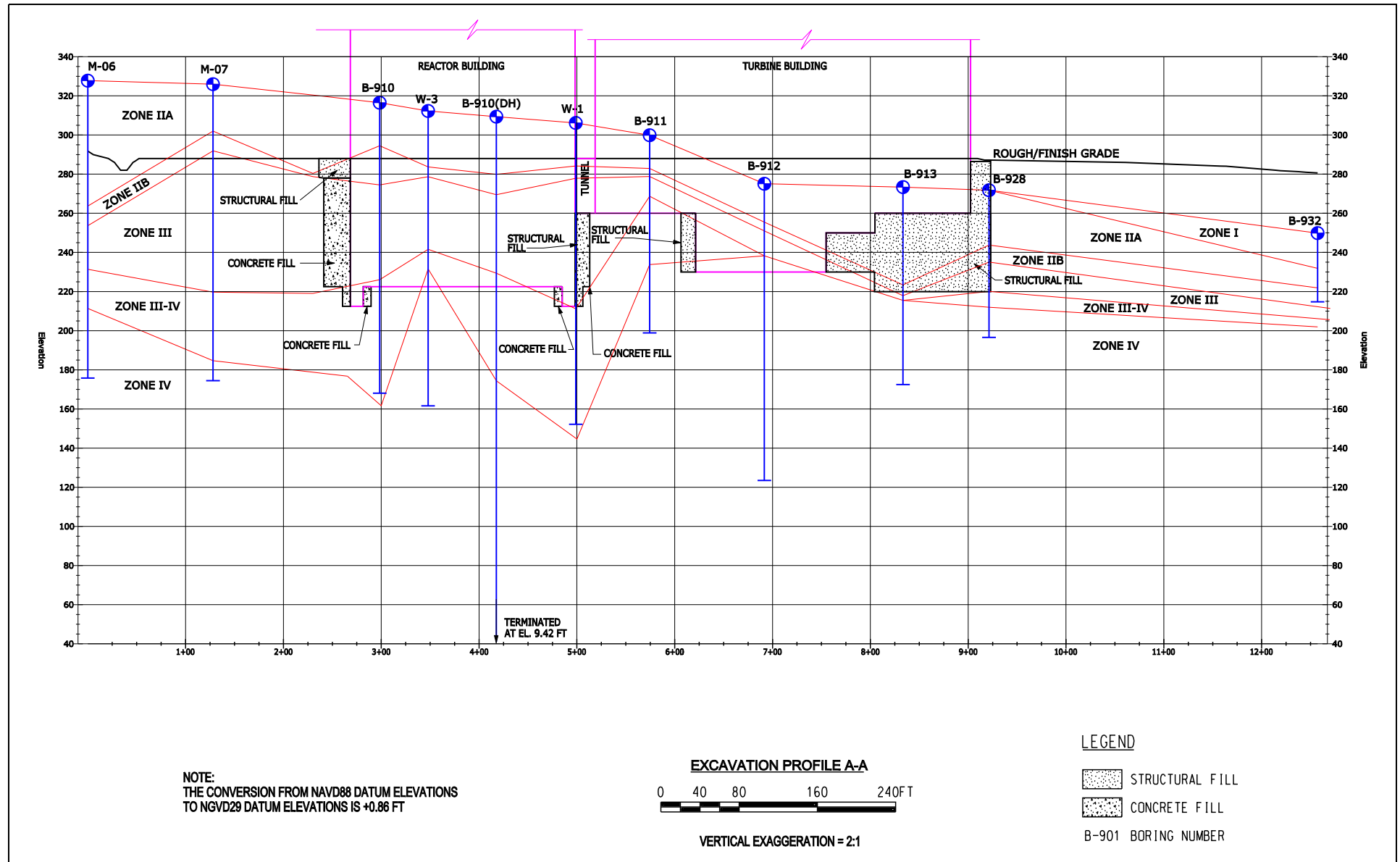
NAPS COL 2.0-29-A Figure 2.5.4-223 Measured Soil Shear Wave Velocity versus Depth



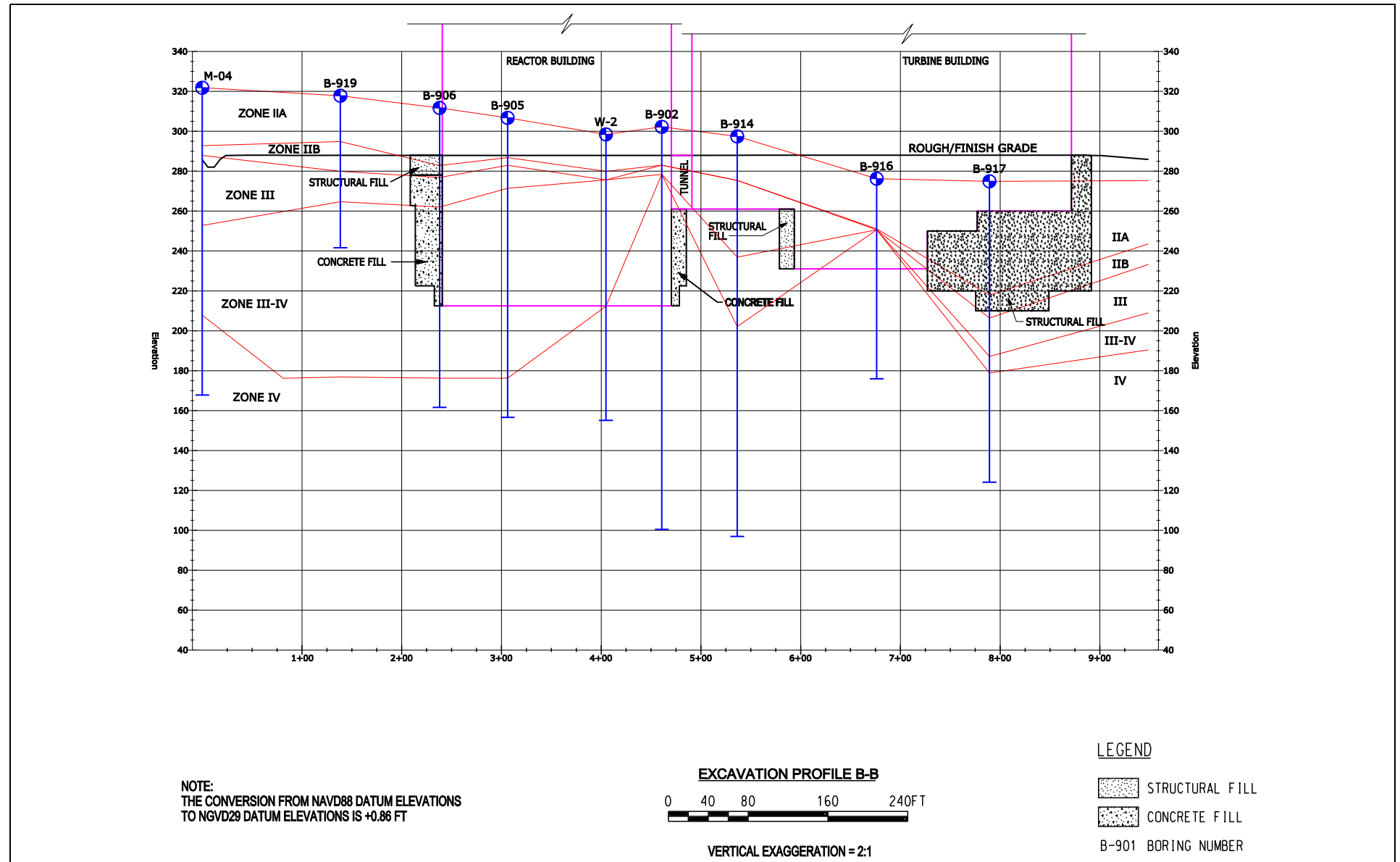
NAPS COL 2.0-29-A Figure 2.5.4-224 Relationship between CBR and Molded Dry Density



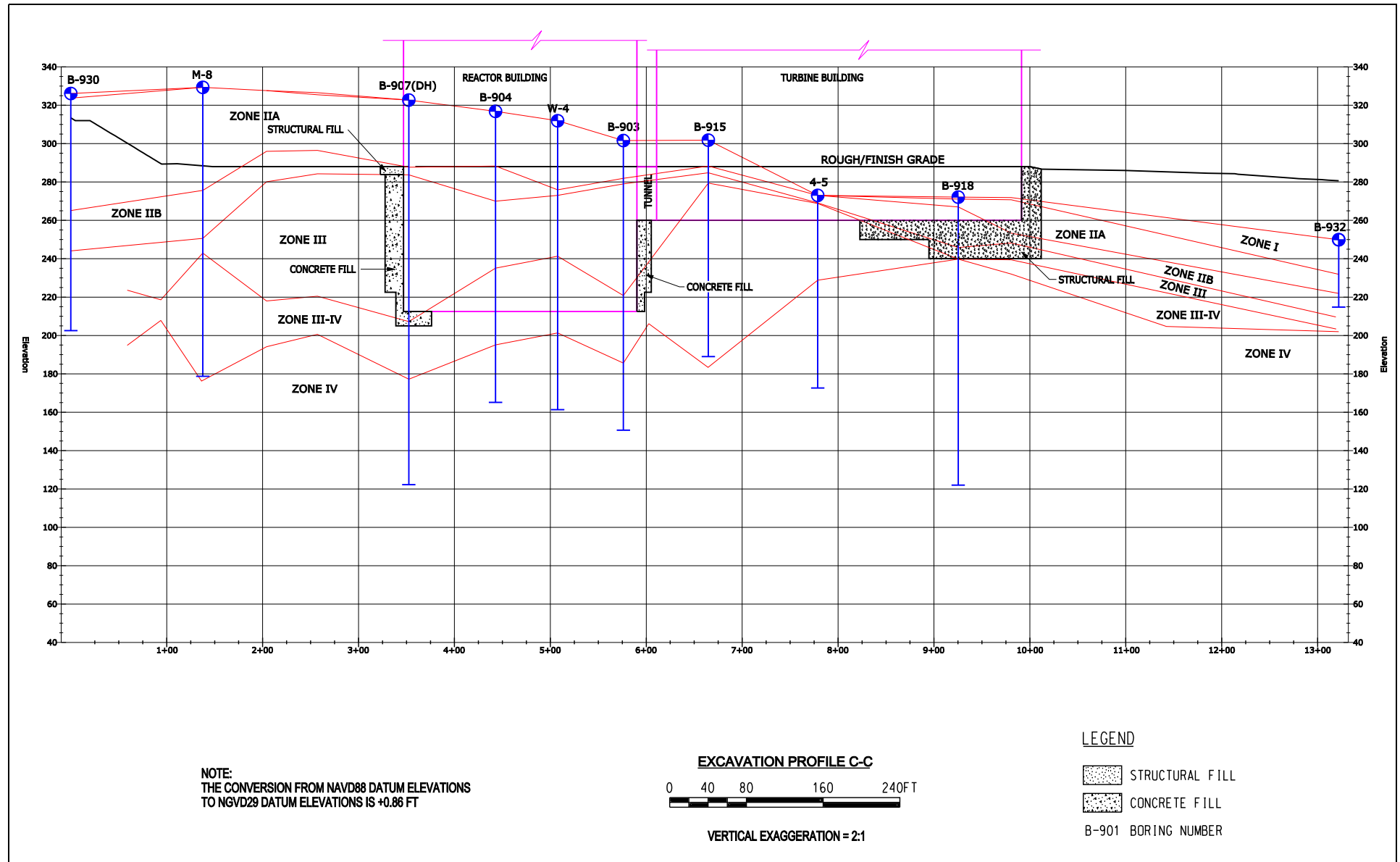
NAPS ESP COL 2.5-3 Figure 2.5.4-225 Subsurface Profile A-A with Foundation Outline



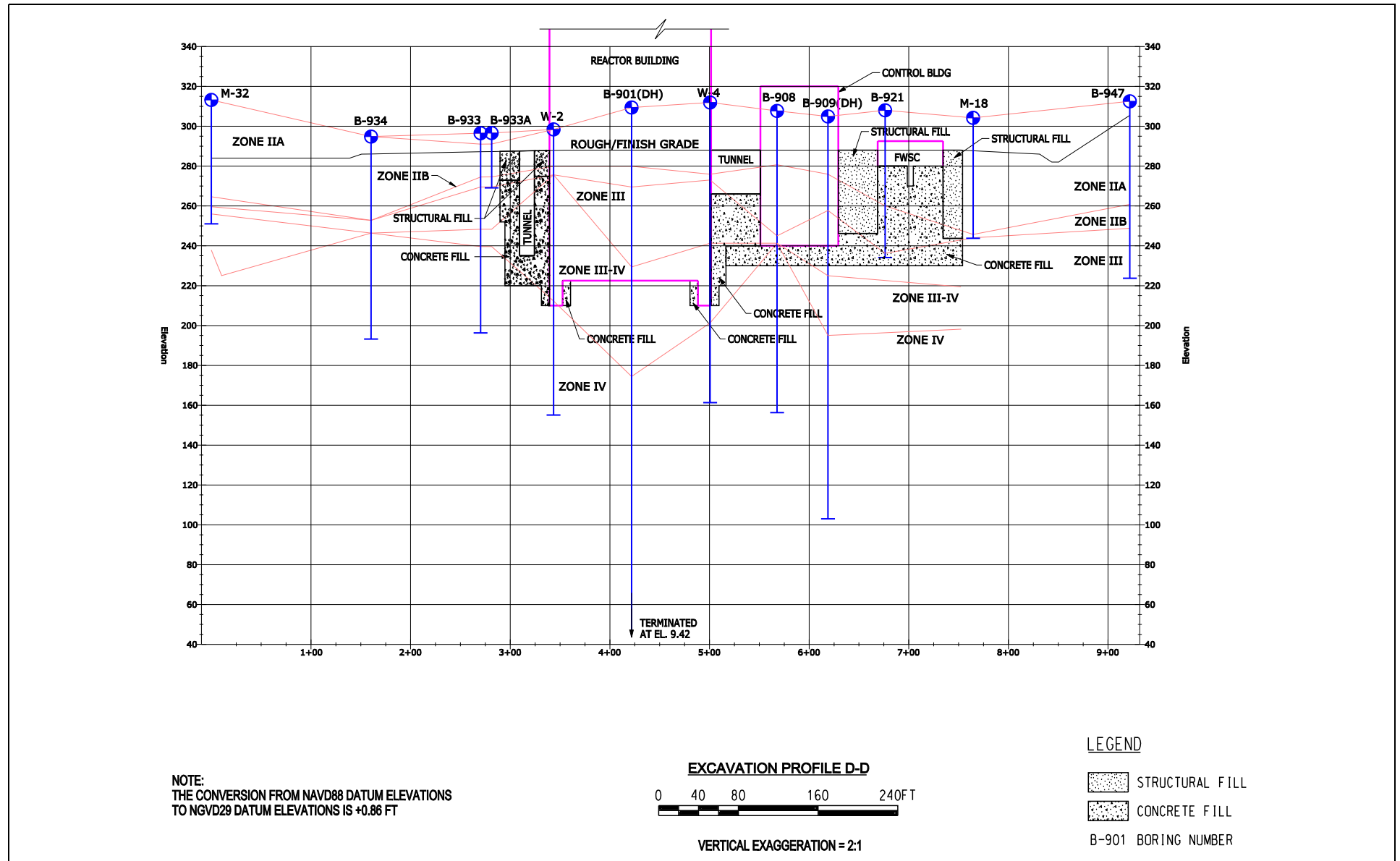
NAPS ESP COL 2.5-3 Figure 2.5.4-226 Subsurface Profile B-B with Foundation Outline



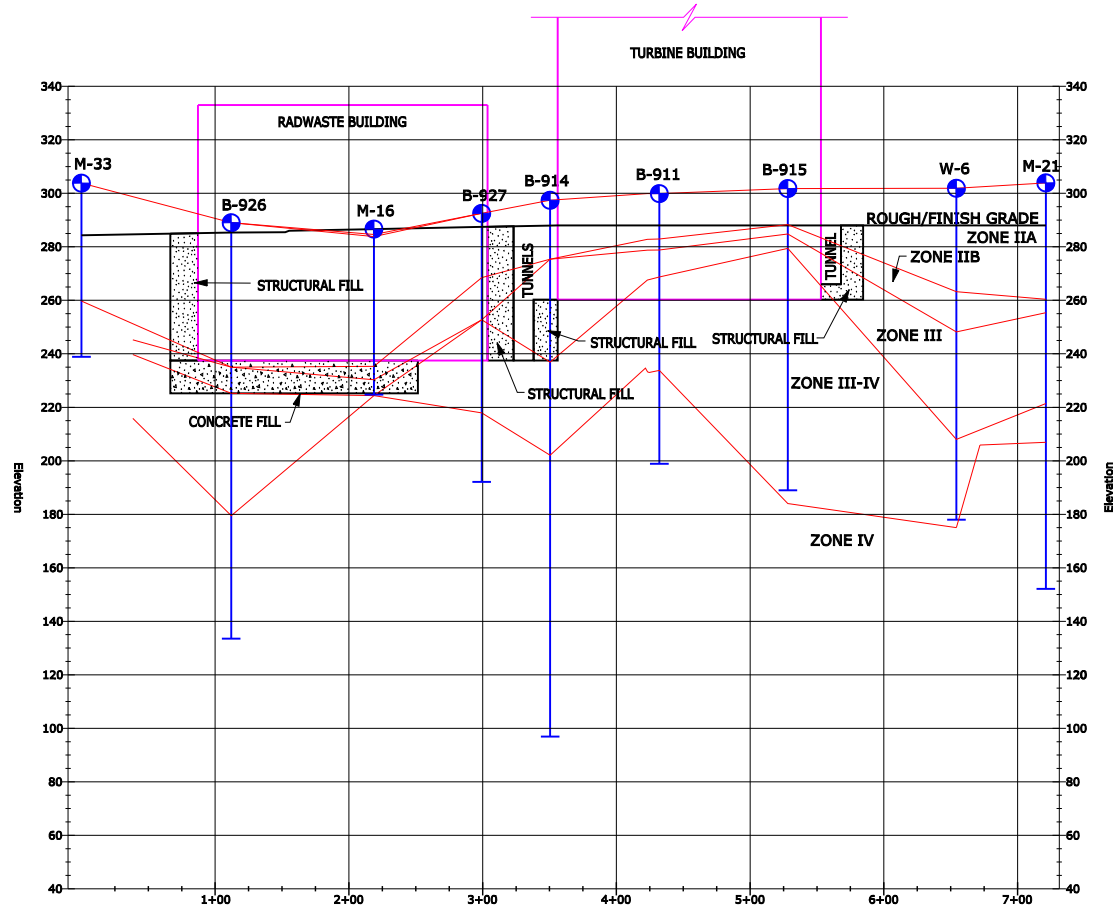
NAPS ESP COL 2.5-3 Figure 2.5.4-227 Subsurface Profile C-C with Foundation Outline



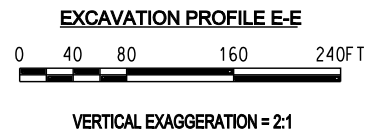
NAPS ESP COL 2.5-3 Figure 2.5.4-228 Subsurface Profile D-D with Foundation Outline



NAPS ESP COL 2.5-3 Figure 2.5.4-229 Subsurface Profile E-E with Foundation Outline



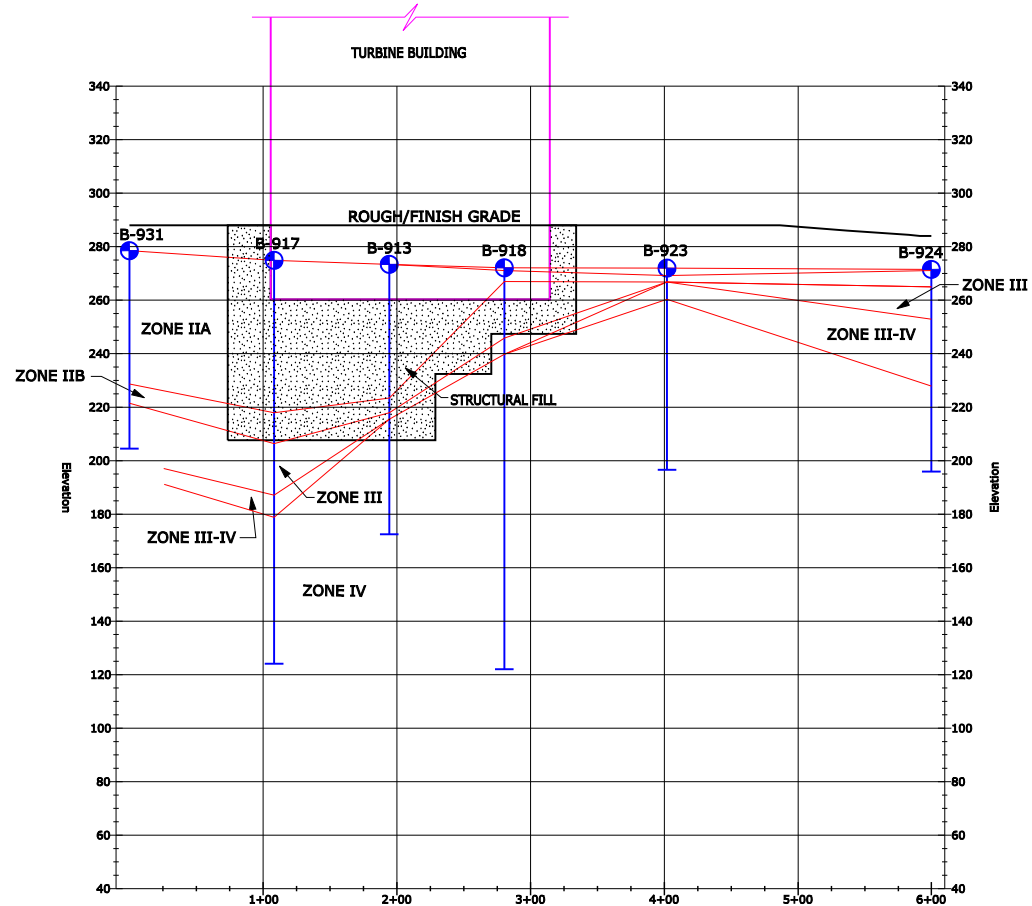
NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT



LEGEND

- STRUCTURAL FILL
- CONCRETE FILL
- B-901 BORING NUMBER

NAPS ESP COL 2.5-3 Figure 2.5.4-230 Subsurface Profile F-F with Foundation Outline



NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT

EXCAVATION PROFILE F-F

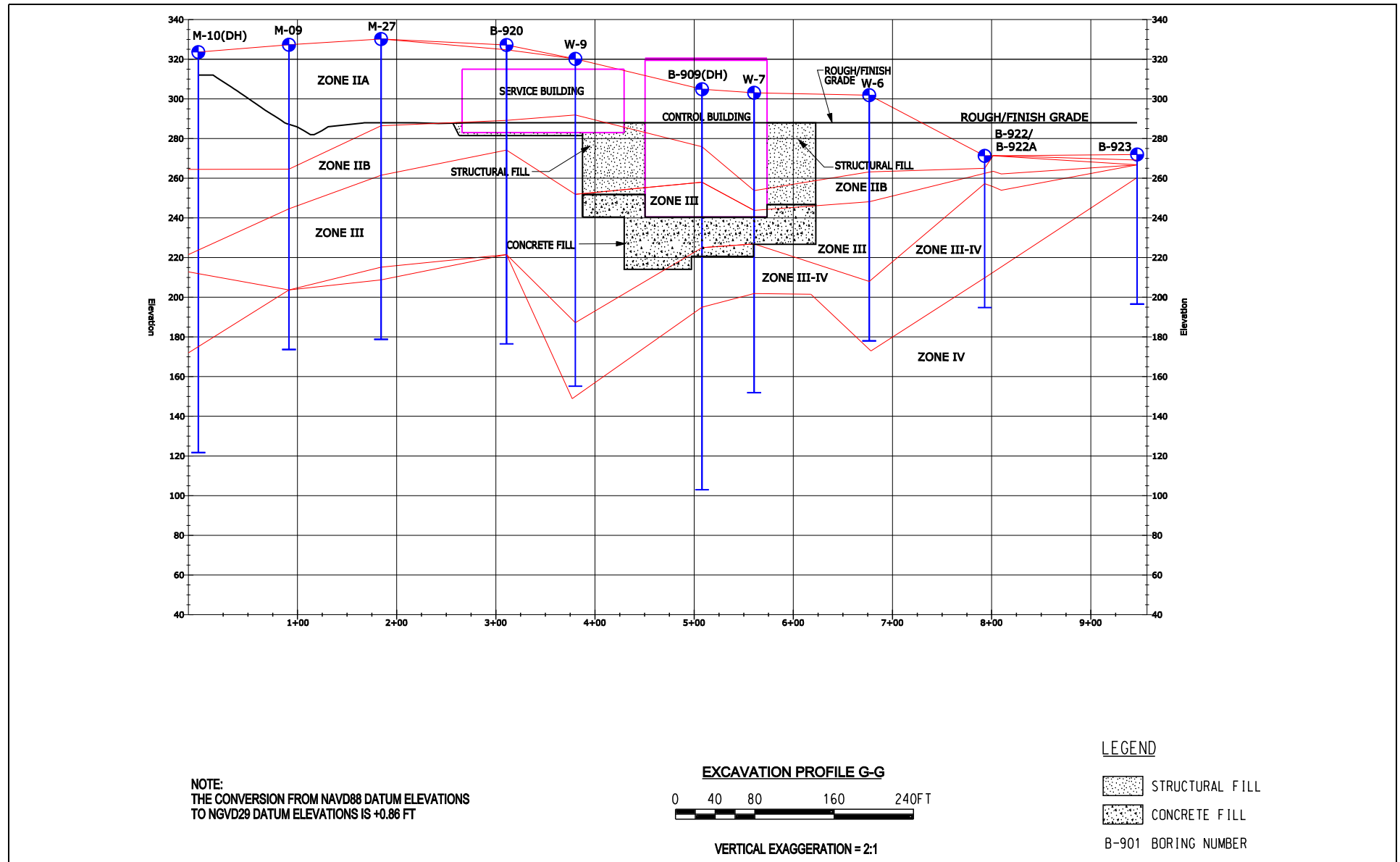


VERTICAL EXAGGERATION = 2:1

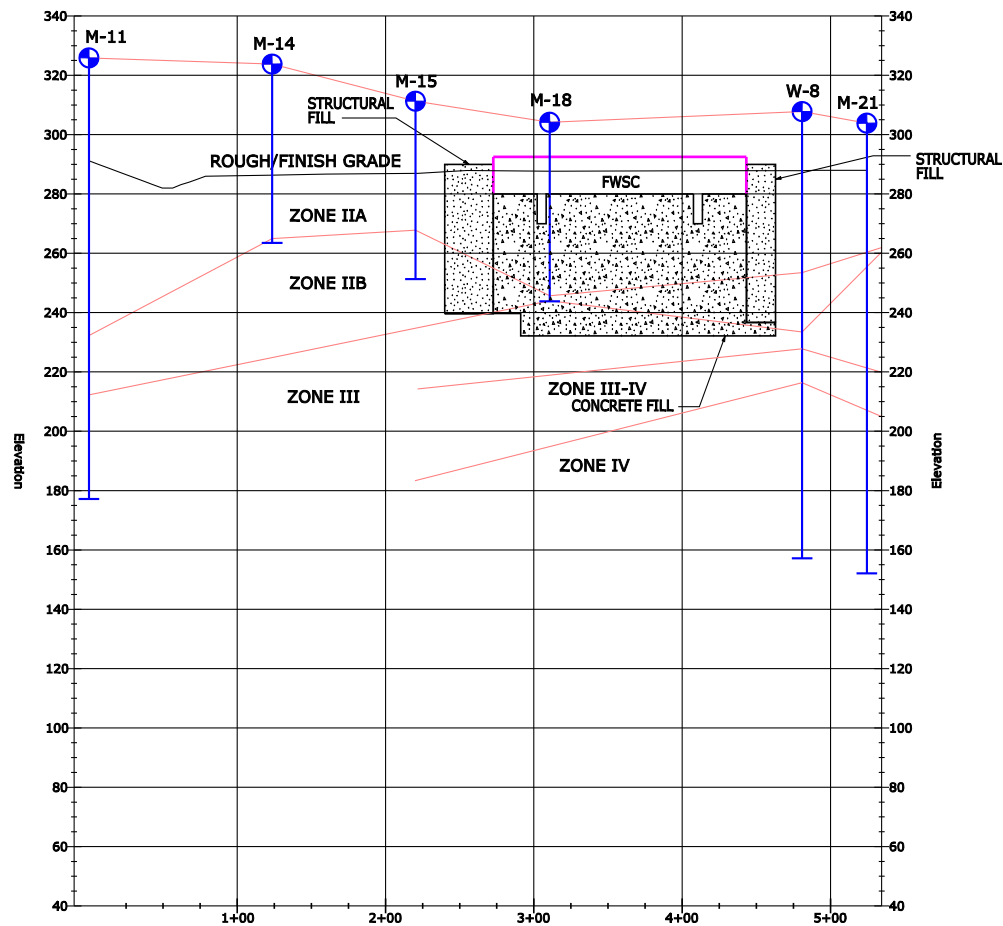
LEGEND

-  STRUCTURAL FILL
-  CONCRETE FILL
- B-901 BORING NUMBER

NAPS ESP COL 2.5-3 Figure 2.5.4-231 Subsurface Profile G-G with Foundation Outline



NAPS ESP COL 2.5-3 Figure 2.5.4-232 Subsurface Profile H-H with Foundation Outline



NOTE:
THE CONVERSION FROM NAVD88 DATUM ELEVATIONS
TO NGVD29 DATUM ELEVATIONS IS +0.86 FT

EXCAVATION PROFILE H-H

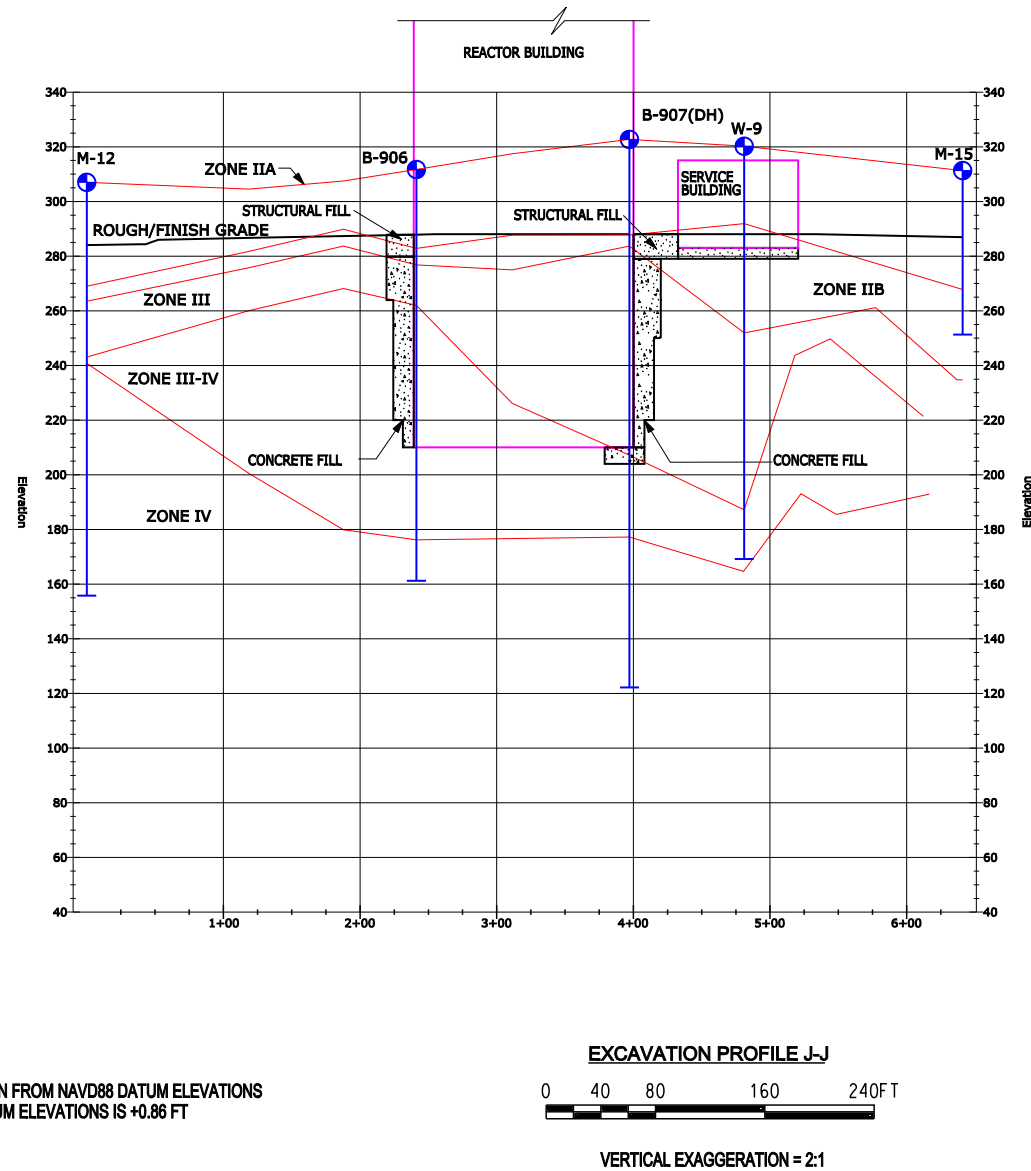


VERTICAL EXAGGERATION = 2:1

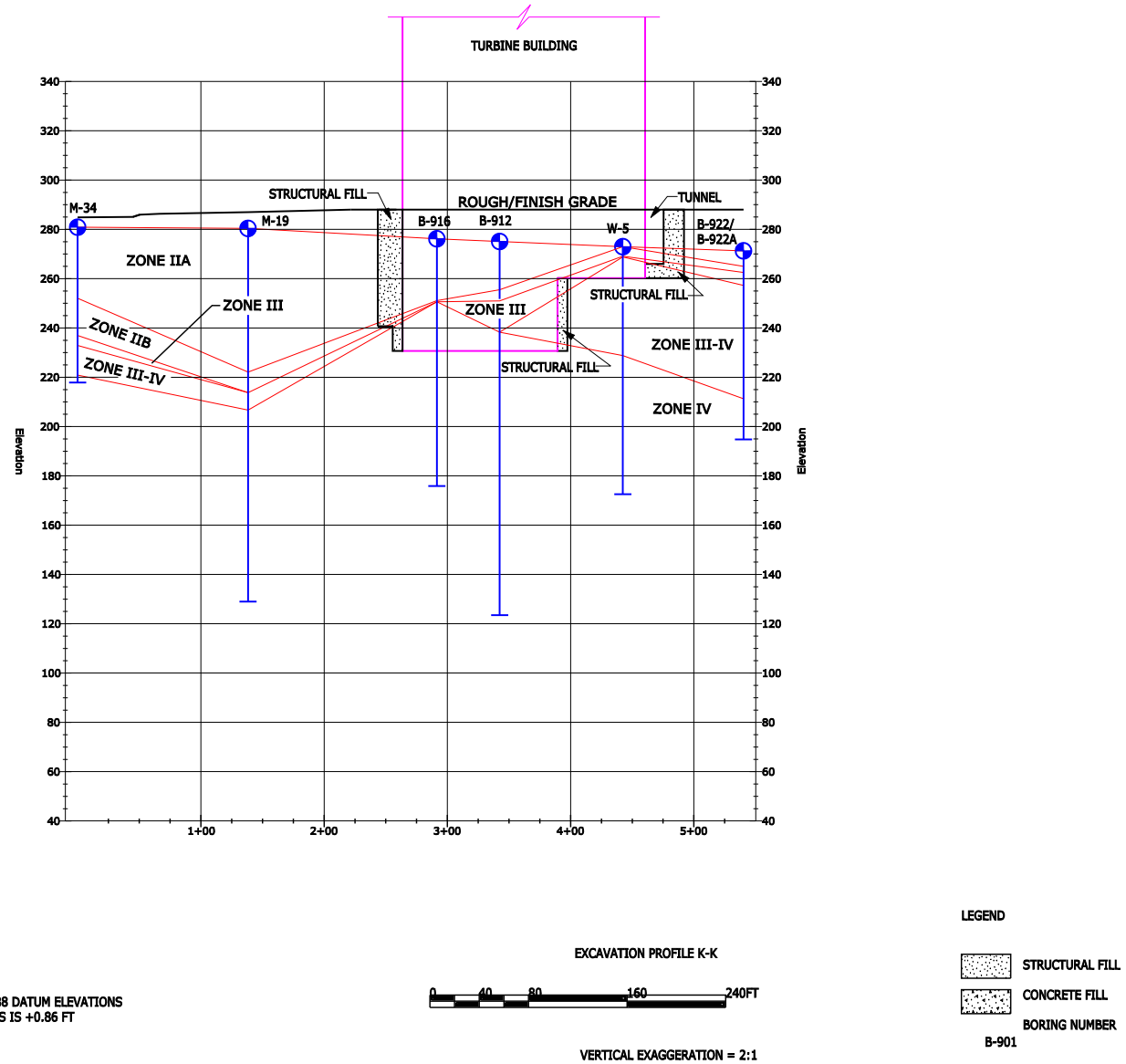
LEGEND

-  STRUCTURAL FILL
-  CONCRETE FILL
- B-901 BORING NUMBER

NAPS ESP COL 2.5-3 Figure 2.5.4-233 Subsurface Profile J-J with Foundation Outline

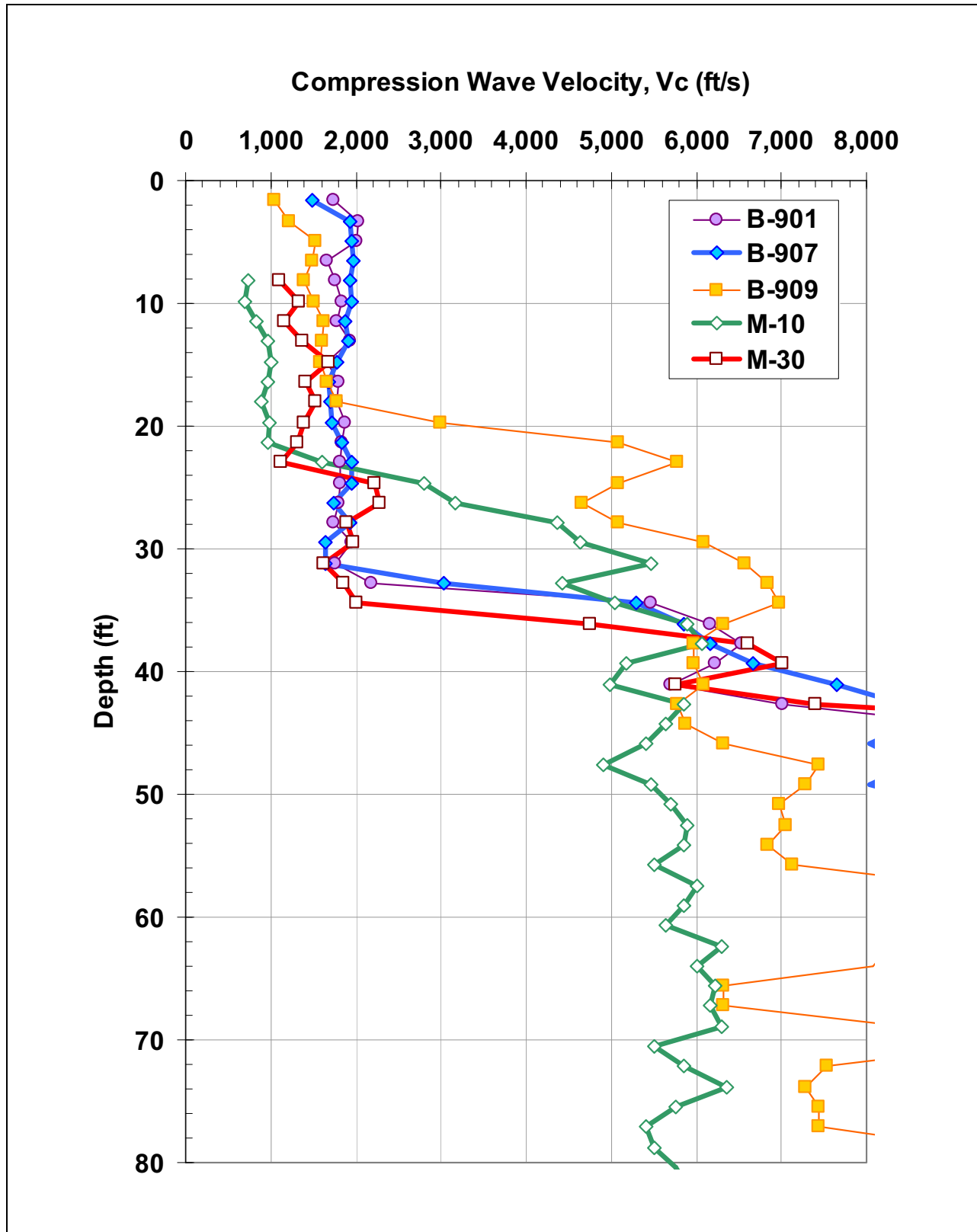


NAPS ESP COL 2.5-3 Figure 2.5.4-234 Subsurface Profile K-K with Foundation Outline



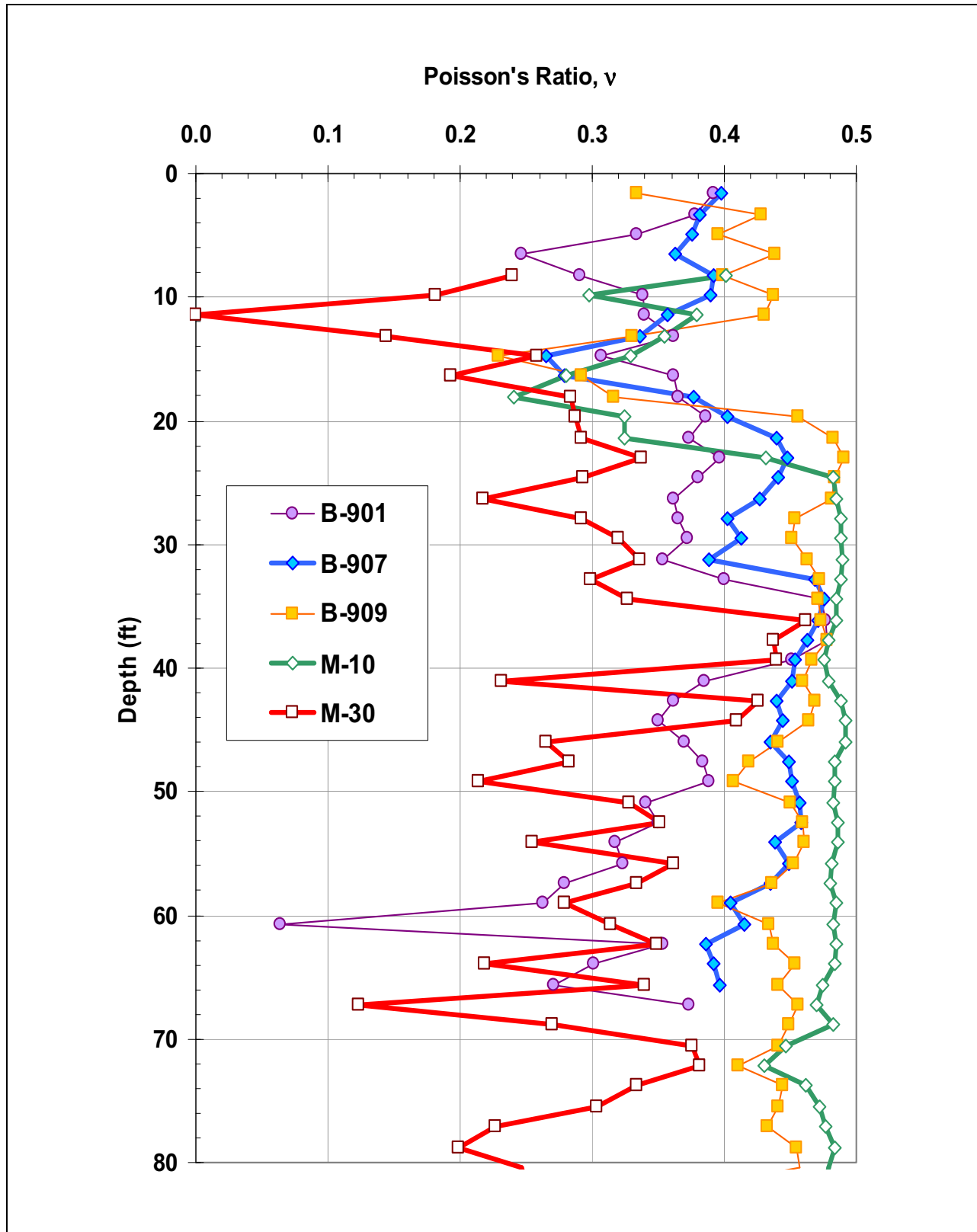
NAPS 2.0-29-A

Figure 2.5.4-235 Measured Compression Wave Velocity versus Depth



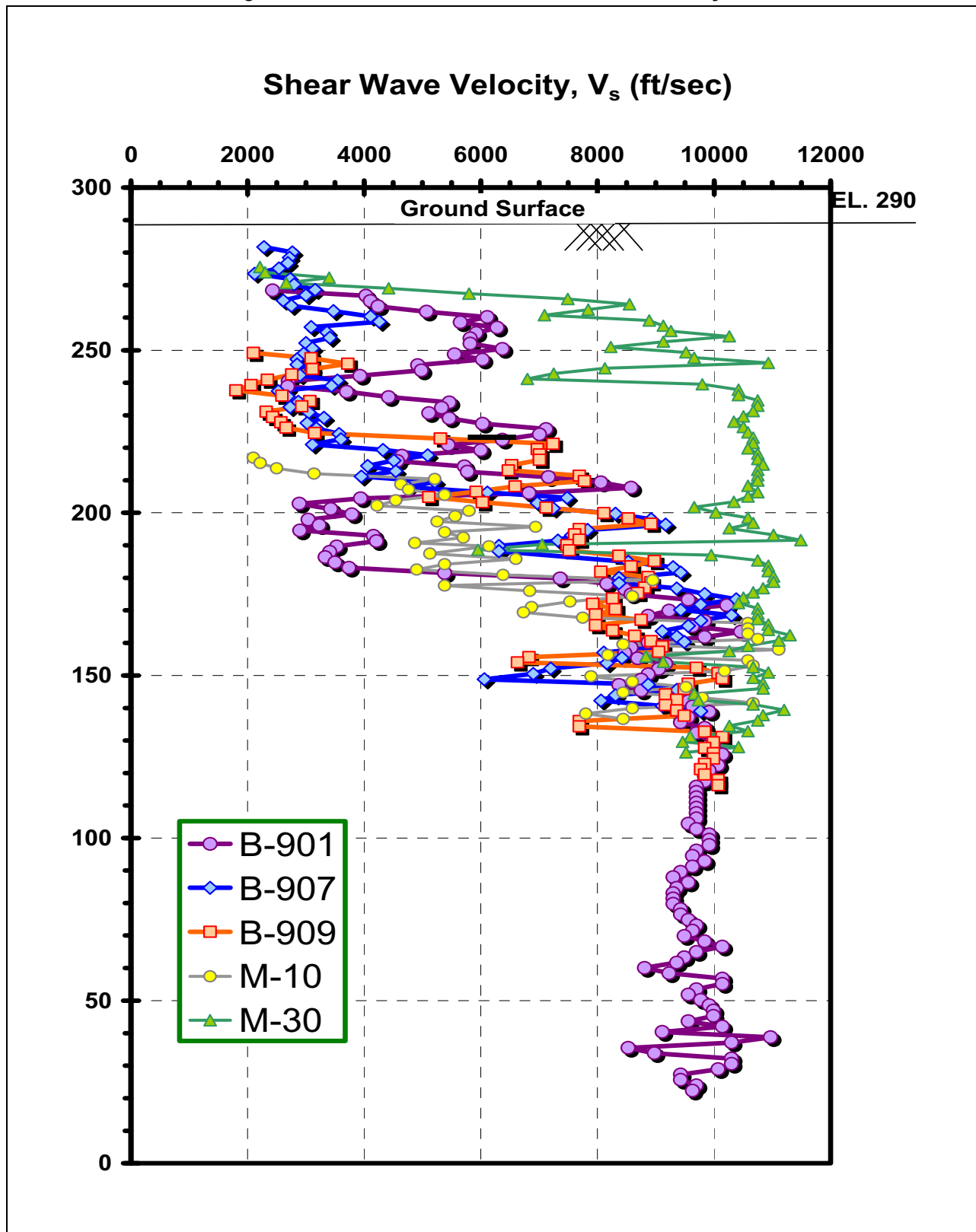
NAPS 2.0-29-A

Figure 2.5.4-236 Soil Poisson's Ratio versus Depth

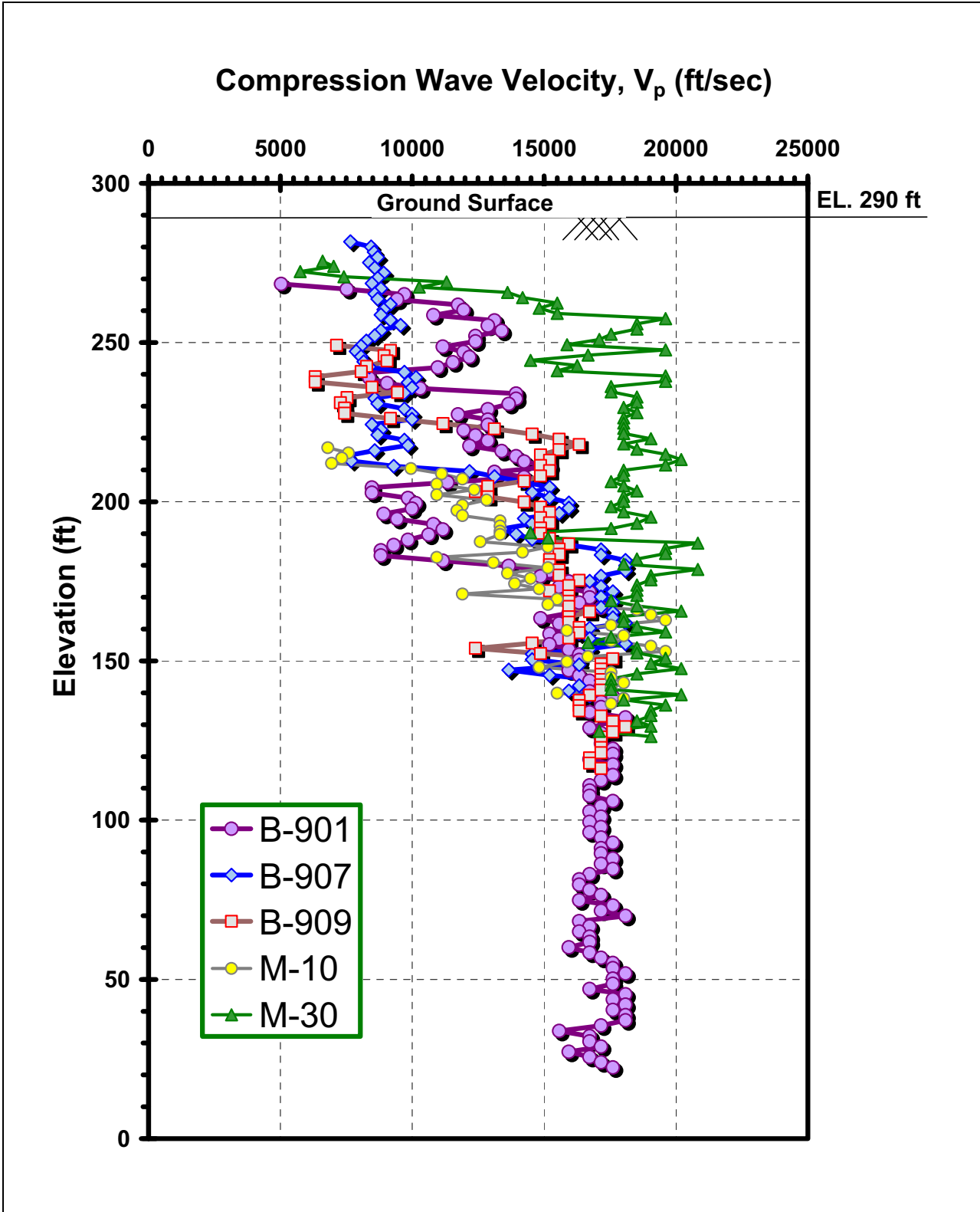


NAPS 2.0-29-A

Figure 2.5.4-237 Bedrock Shear Wave Velocity versus Elevation

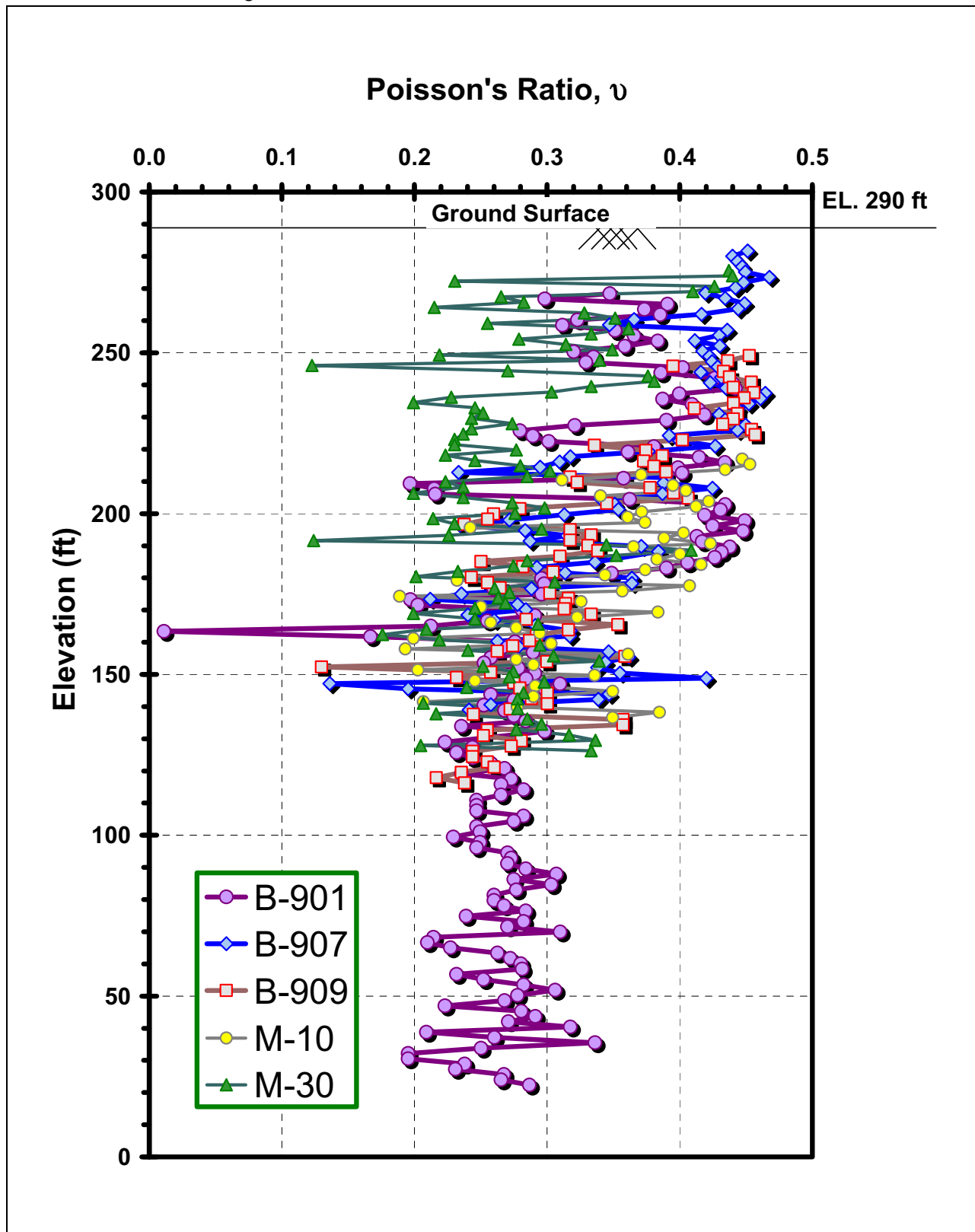


NAPS 2.0-29-A Figure 2.5.4-238 **Bedrock Compression Wave Velocity versus Elevation**

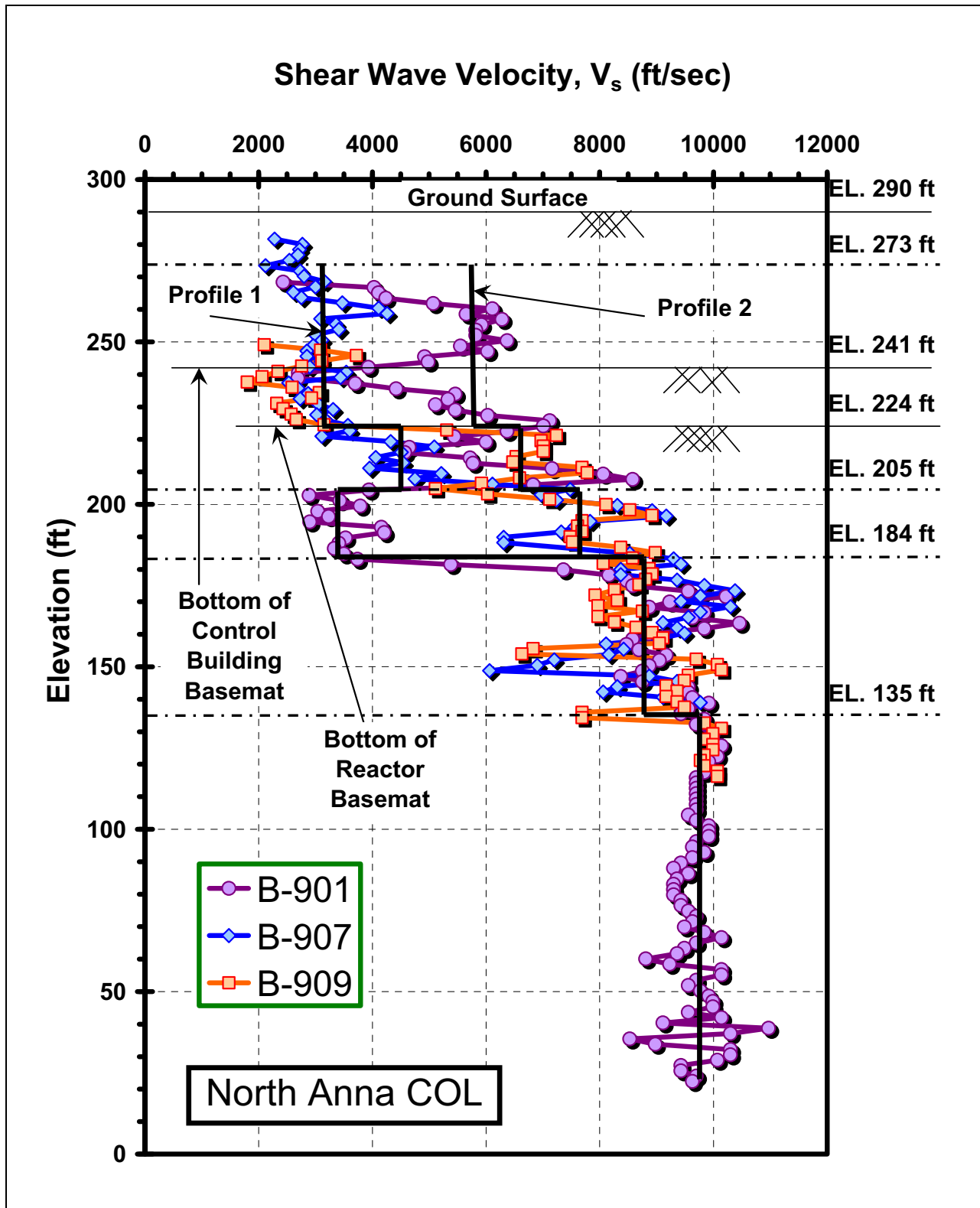


NAPS 2.0-29-A

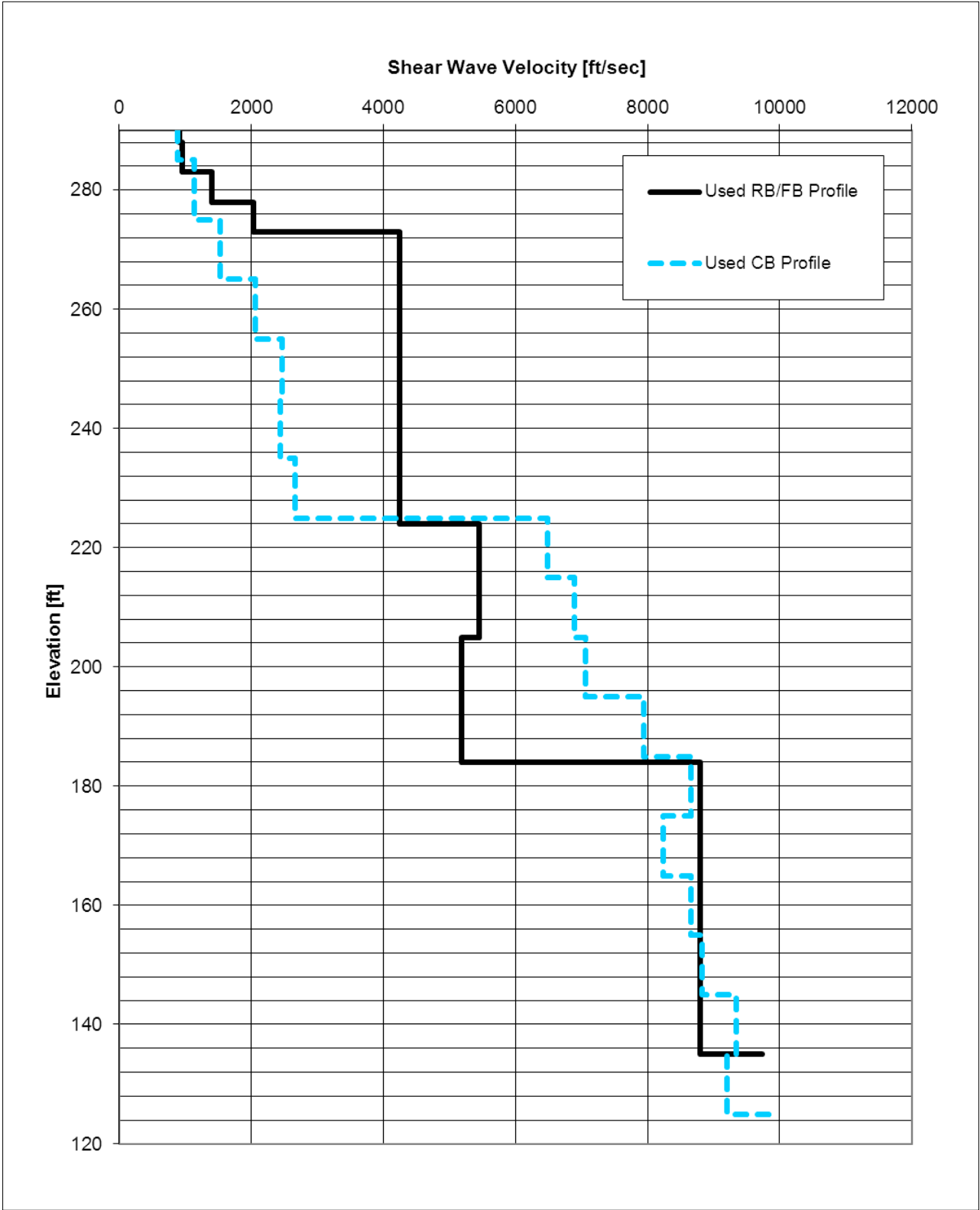
Figure 2.5.4-239 Bedrock Poisson's Ratio versus Elevation



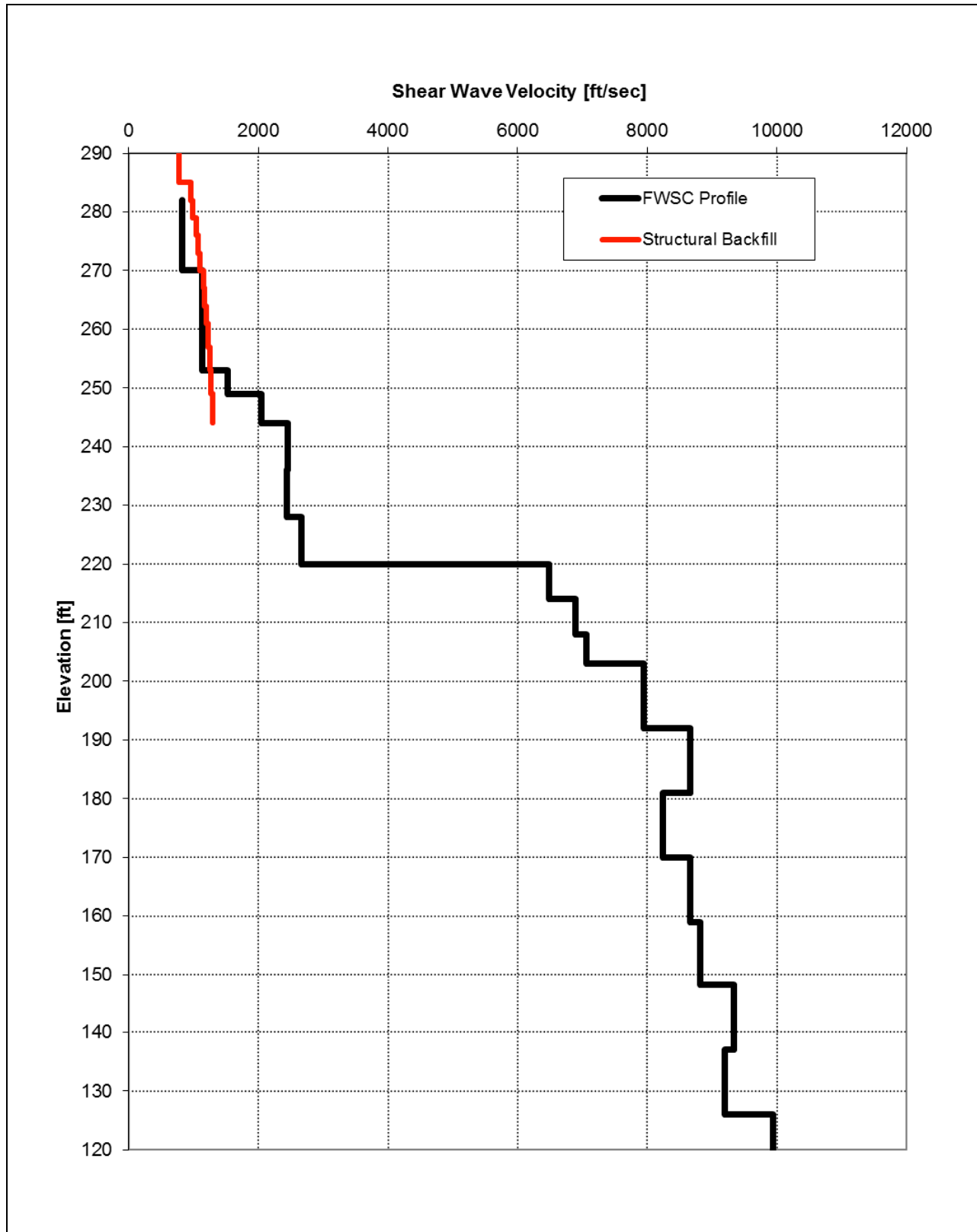
NAPS ESP COL 2.5-9 Figure 2.5.4-241 **Bedrock Shear Wave Velocity Profiles 1 & 2 versus Elevation**



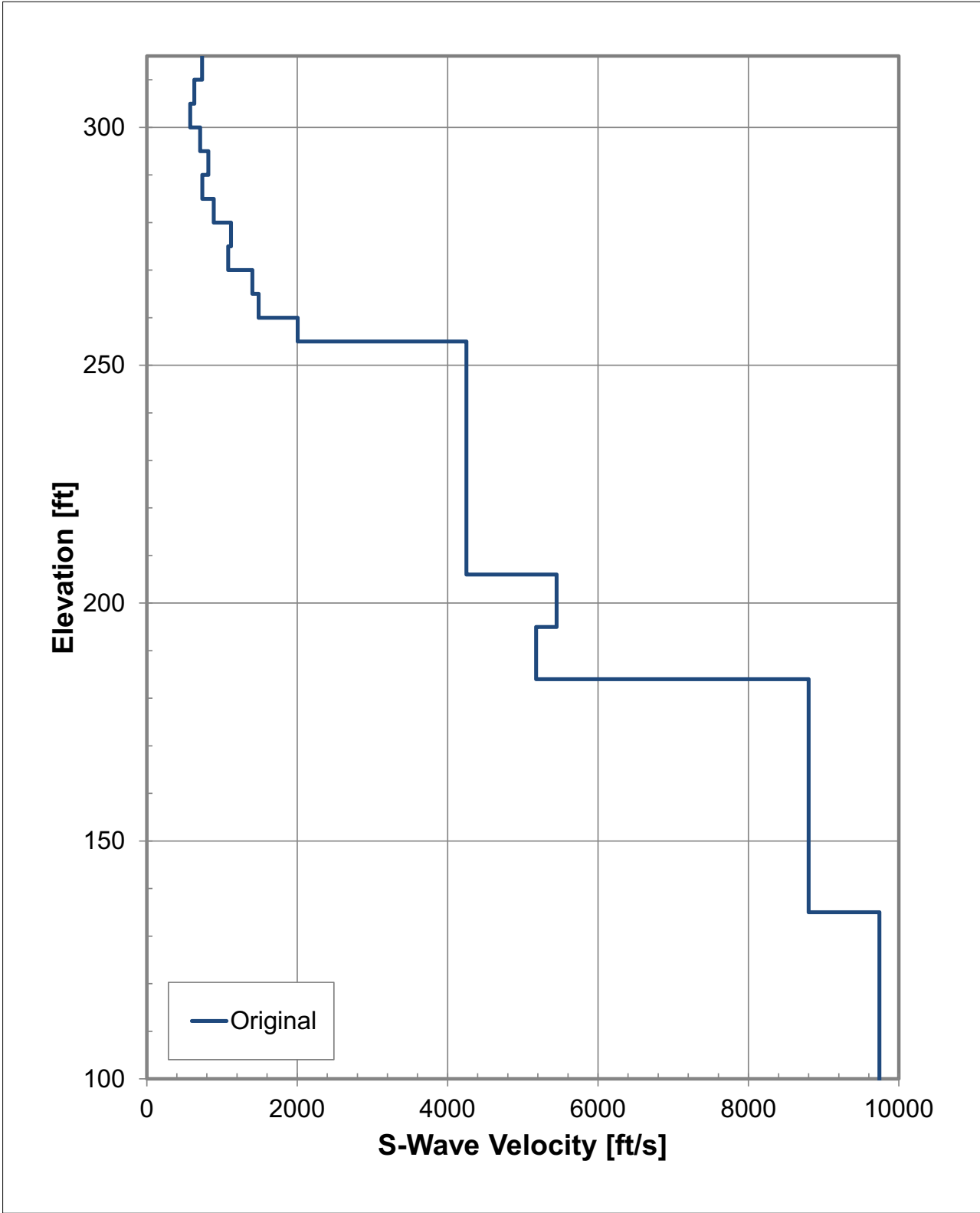
NAPS ESP COL 2.5-9 Figure 2.5.4-242 **Best Estimate Shear Wave Velocity Profiles for RB/FB and CB**



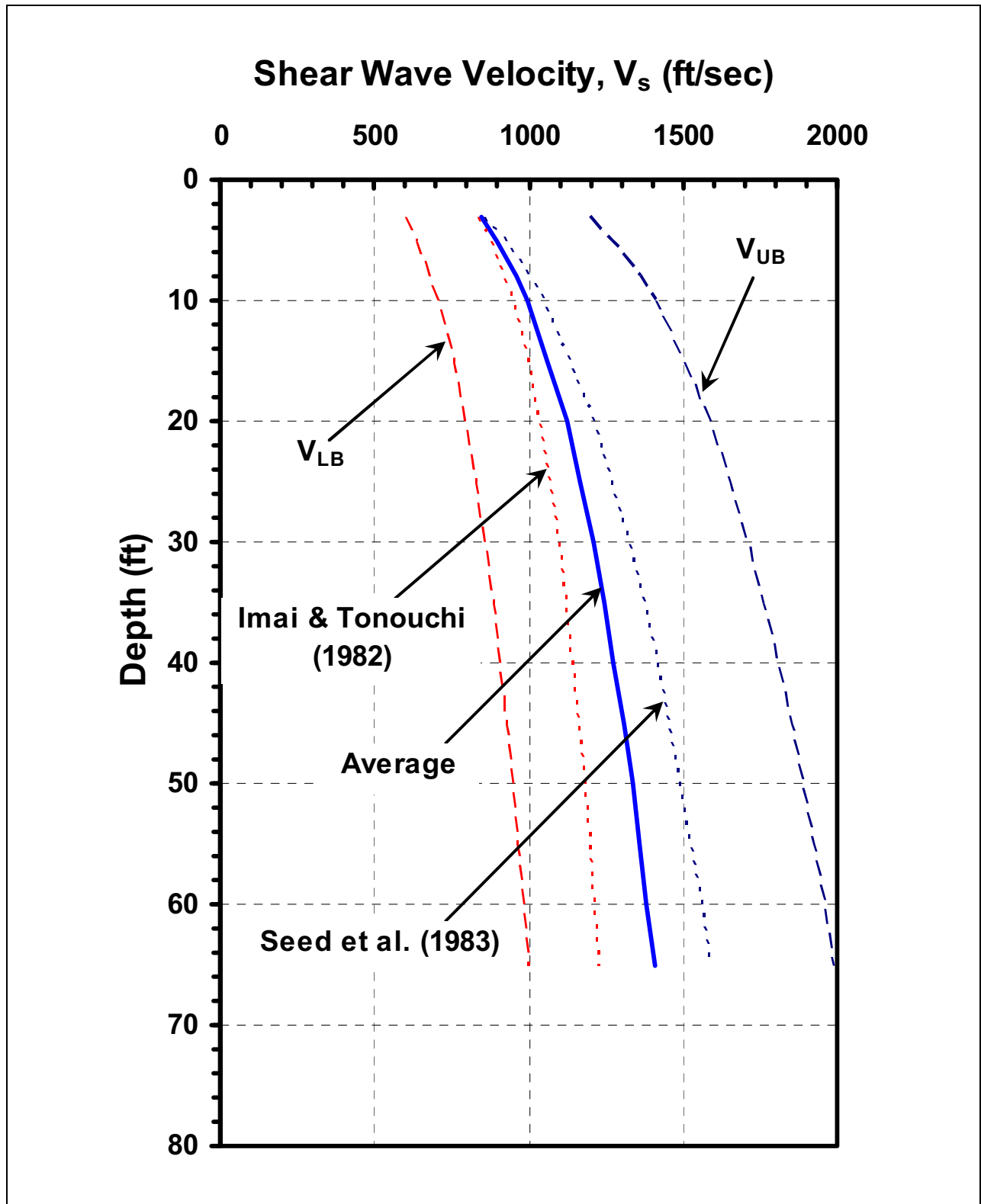
NAPS ESP COL 2.5-9 Figure 2.5.4-243 Best Estimate Shear Wave Velocity Profile for FWSC



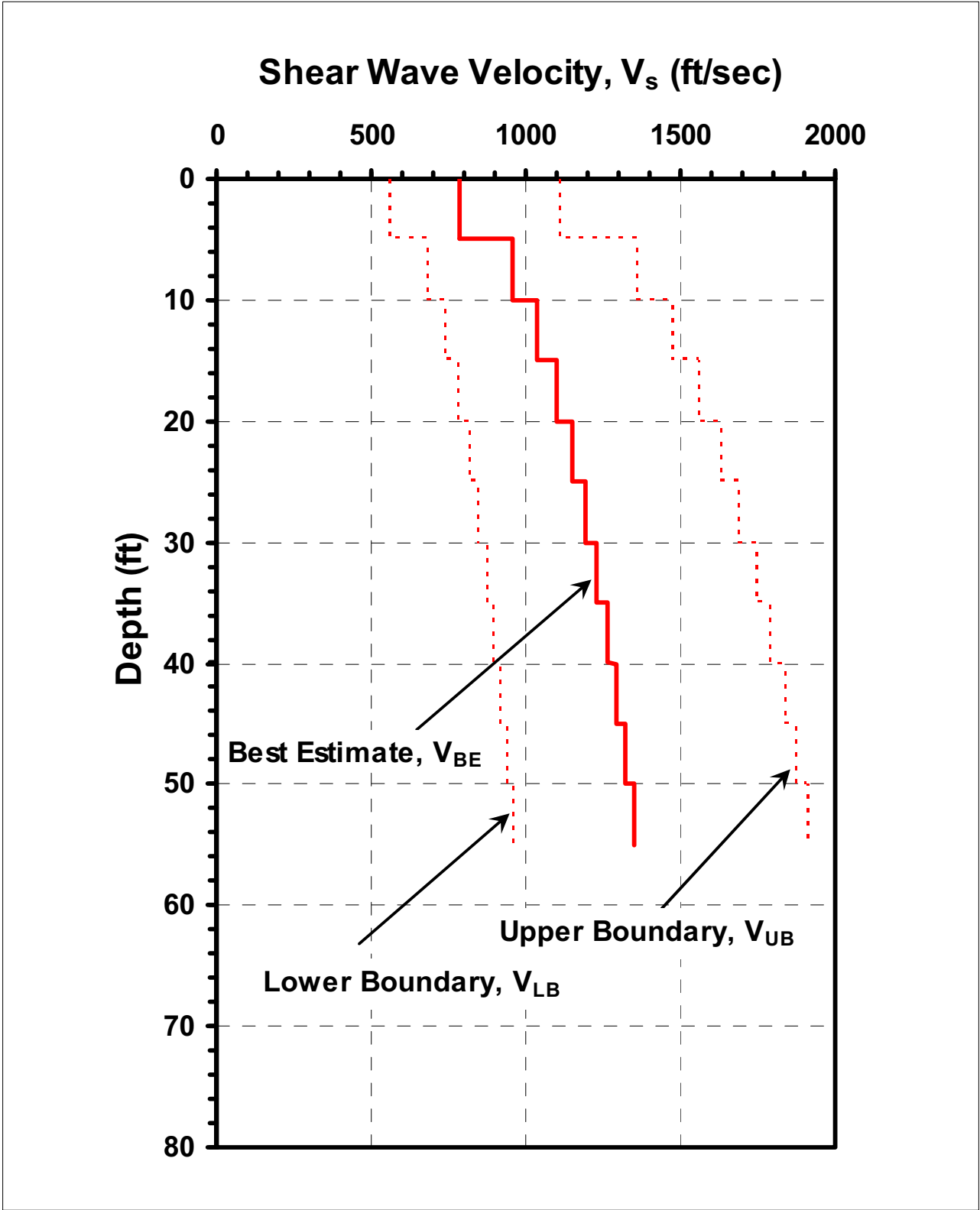
NAPS ESP COL 2.5-9 Figure 2.5.4-244 **Best Estimate Shear Wave Velocity Profile for Free-Field Slope**



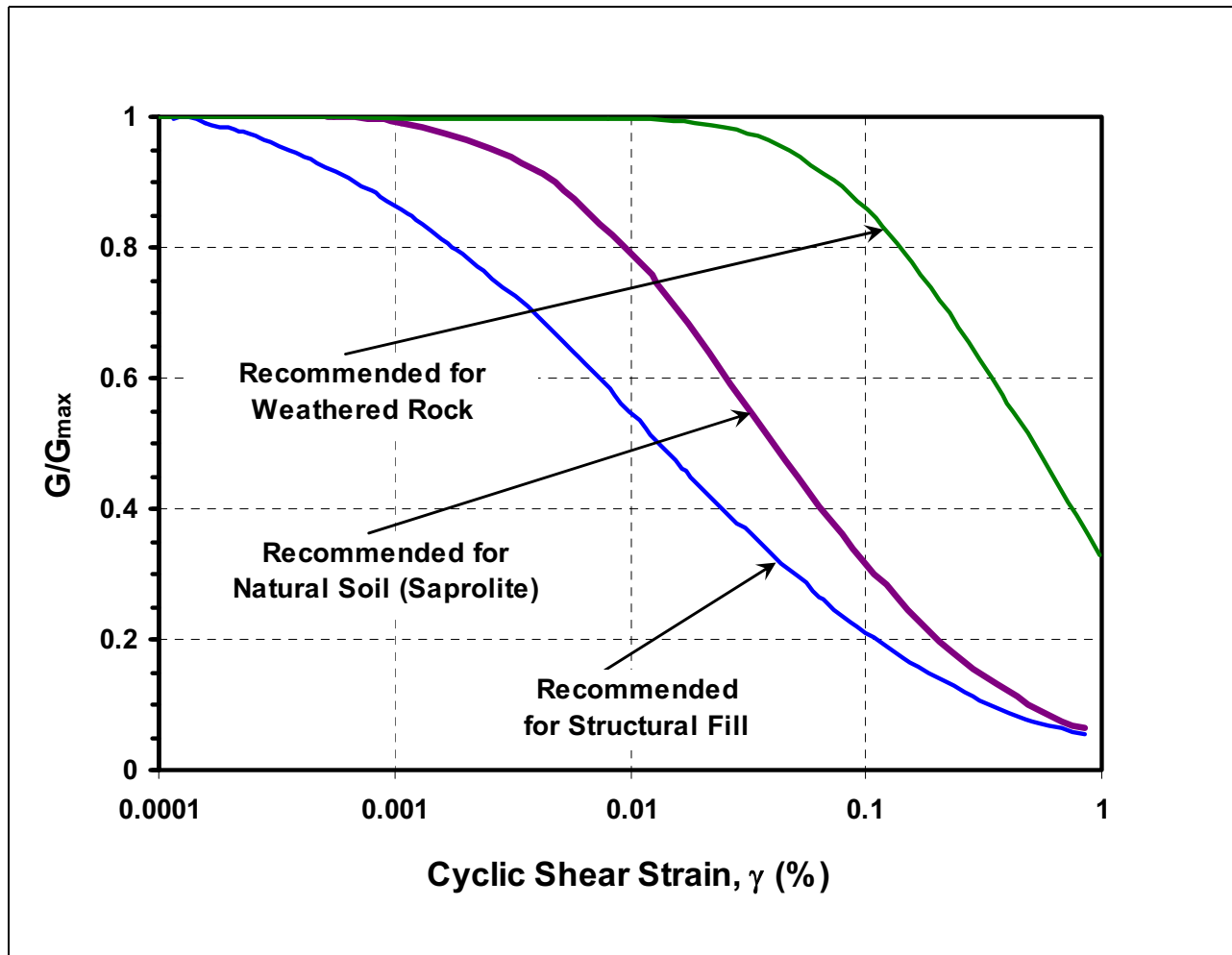
NAPS ESP COL 2.5-9 Figure 2.5.4-245 **Best Estimate Shear Wave Velocity Profile for Structural Fill**



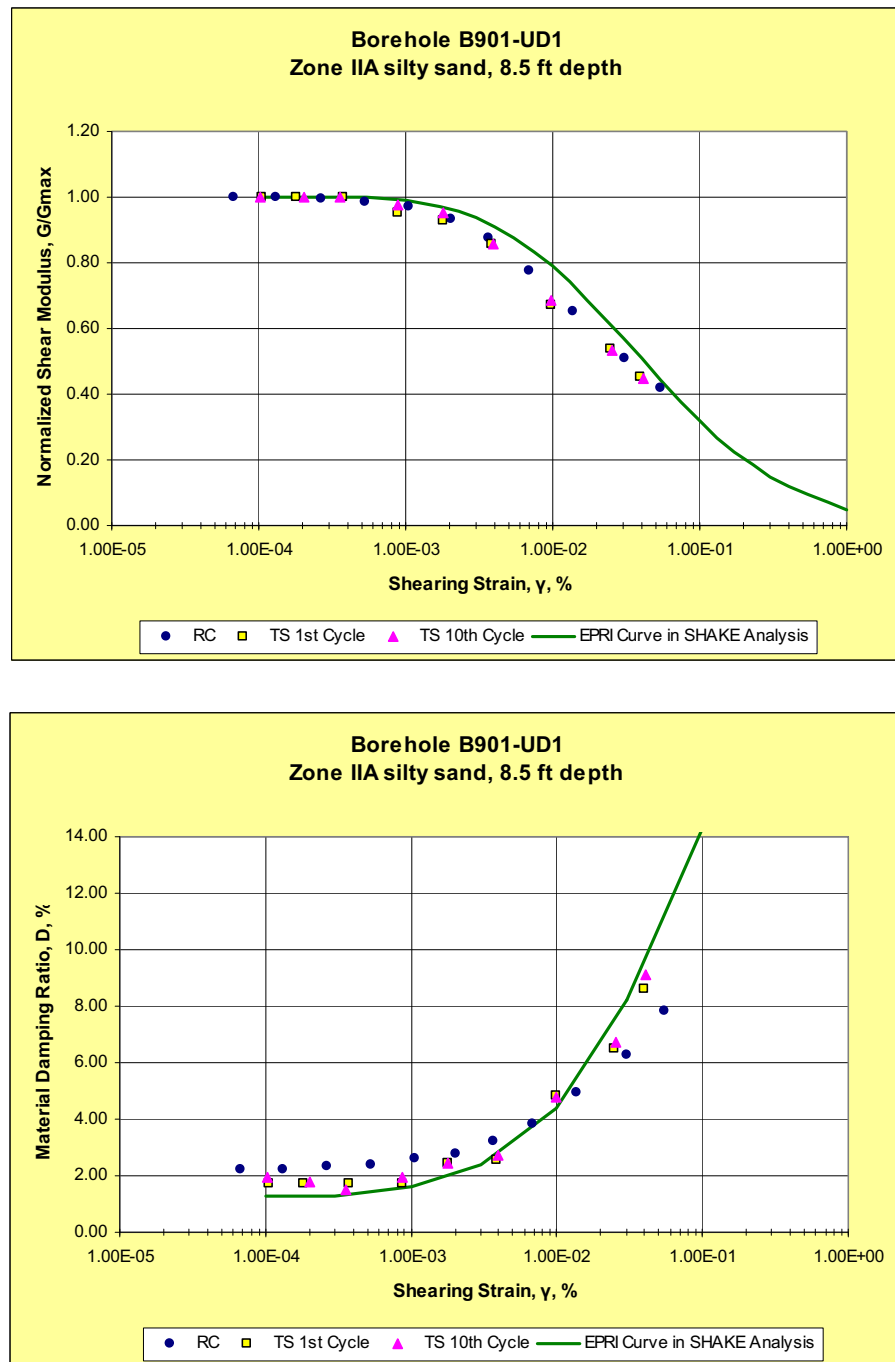
NAPS ESP COL 2.5-9 Figure 2.5.4-246 **Best Estimate Shear Wave Velocity Profile for Structural Fill in 5-Foot Intervals**



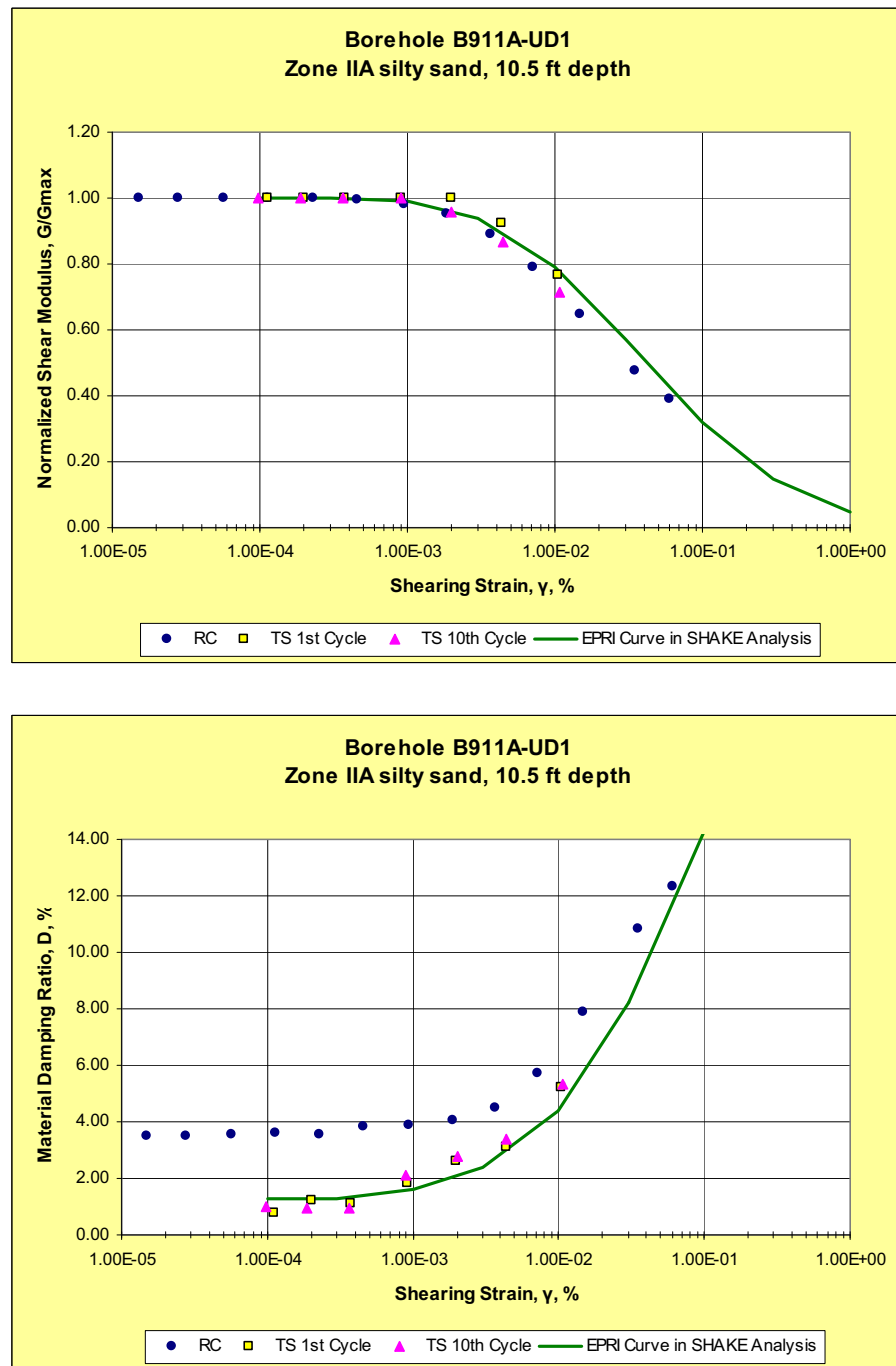
NAPS COL 2.0-29-A Figure 2.5.4-247 **Shear Modulus Reduction Design Curves**



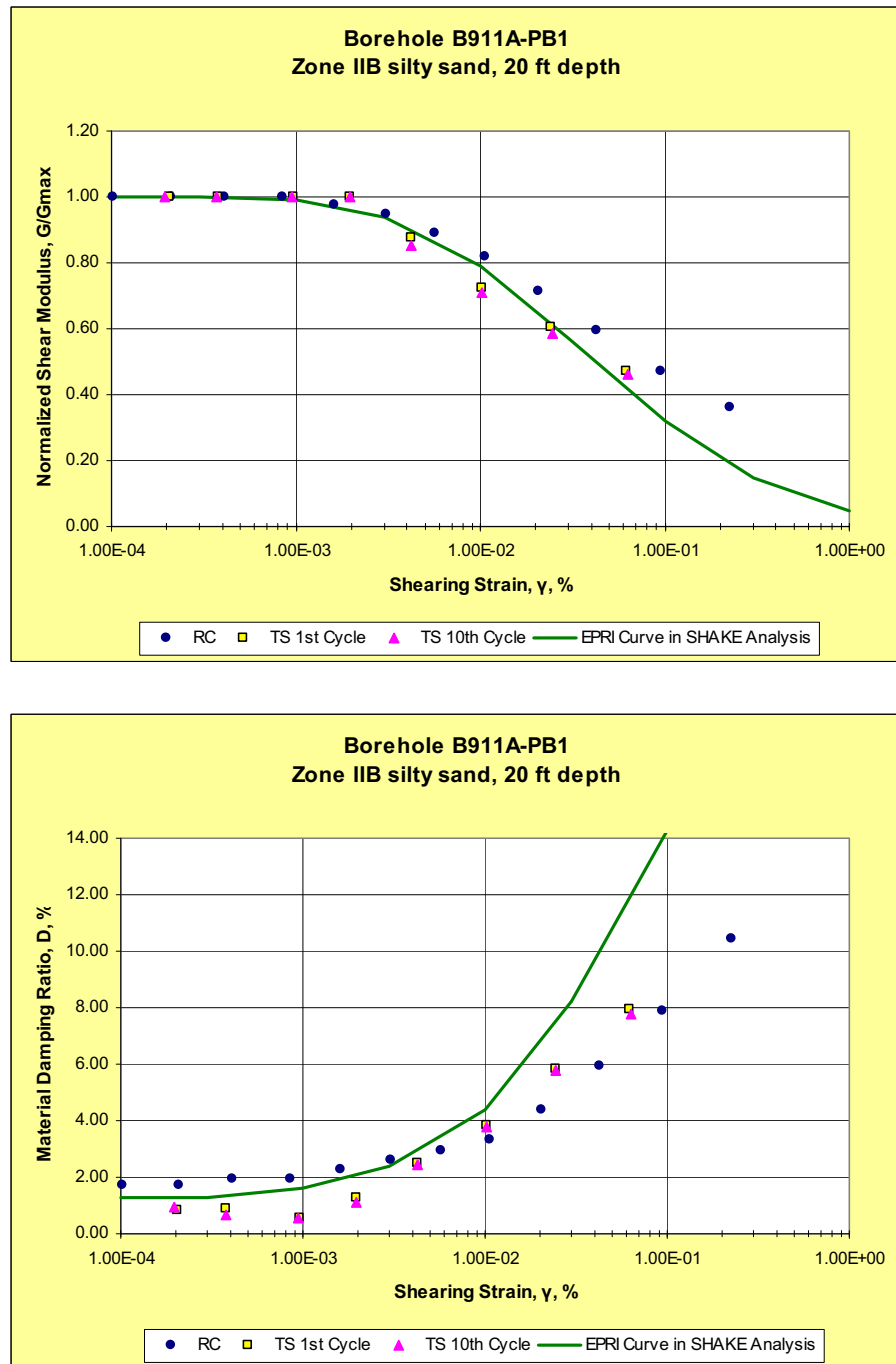
NAPS COL 2.0-29-A Figure 2.5.4-248 **RCTS Results with $G/G_{\max}$ and D Curve $G/G_{\max}$ vs. Strain, B-901 UD-1: 4.3 psi Confining Pressure (Sheet 1 of 3)**



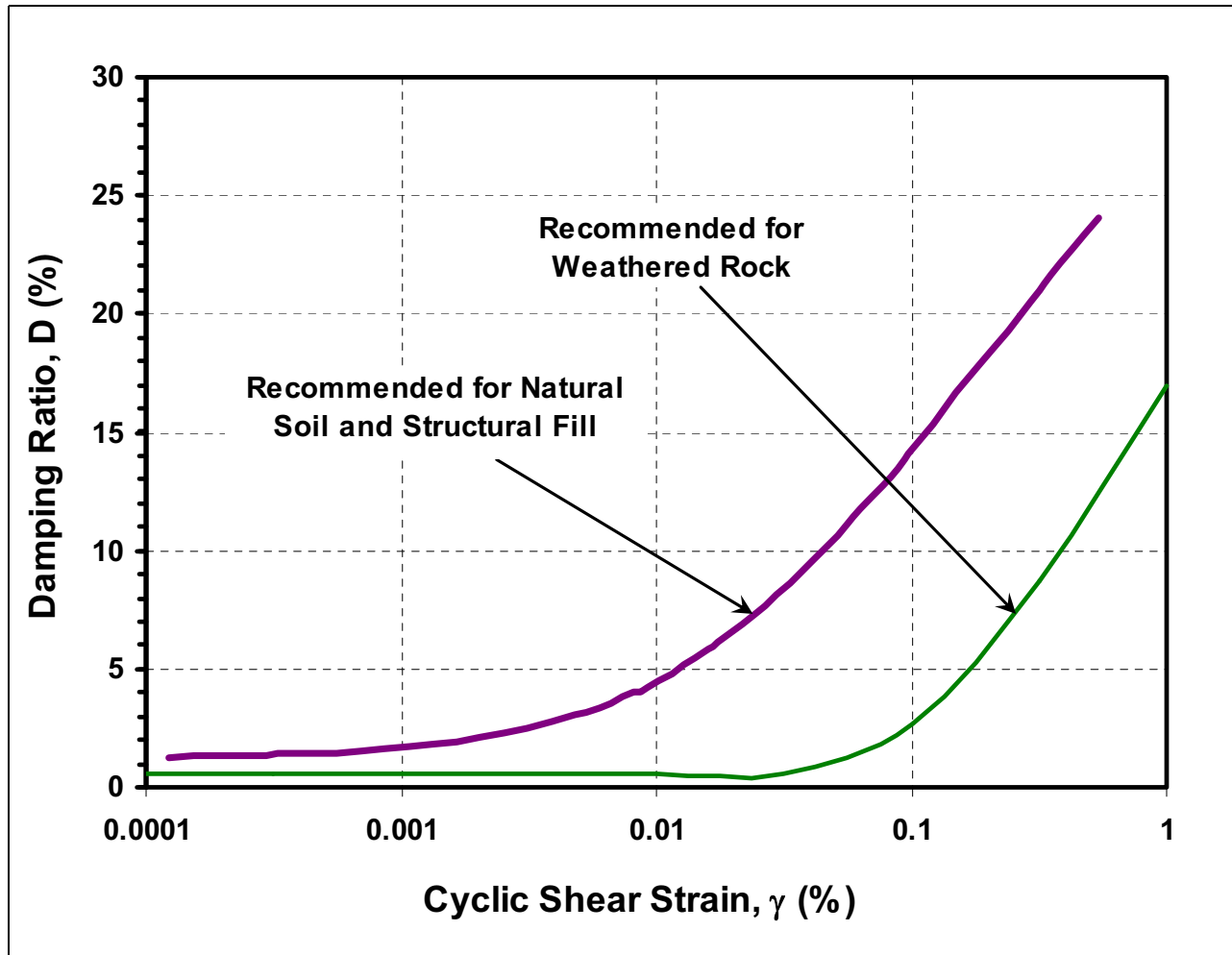
NAPS COL 2.0-29-A Figure 2.5.4-248 RCTS Results with $G/G_{\max}$ and D Curve $G/G_{\max}$ vs. Strain, B-911A UD-1: 5.6 psi Confining Pressure (Sheet 2 of 3)



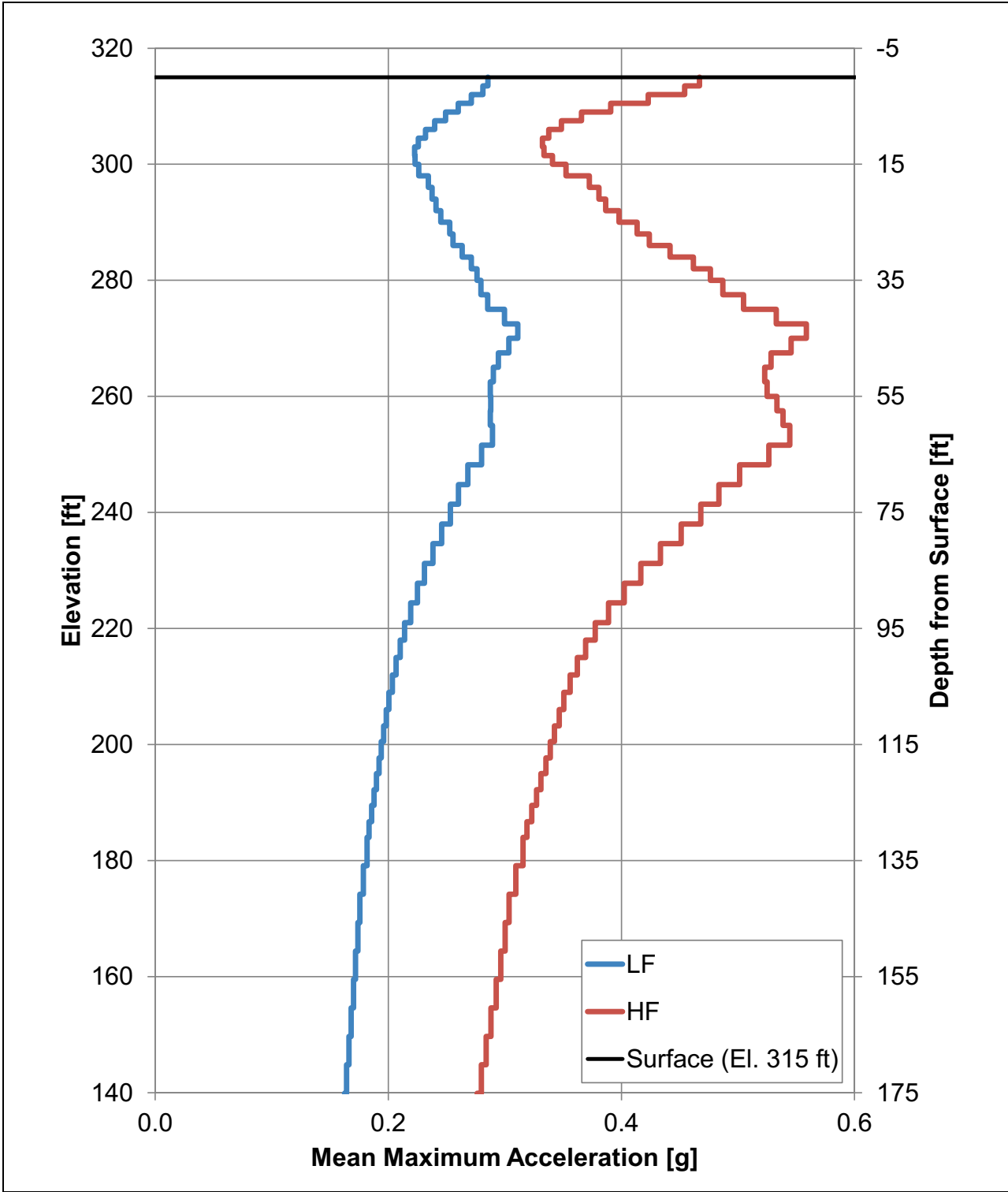
NAPS COL 2.0-29-A Figure 2.5.4-248 **RCTS Results with G/G_{max} and D Curve G/G_{max} vs. Strain, B-911A PB-1: 11.4 psi Confining Pressure (Sheet 3 of 3)**



NAPS COL 2.0-29-A Figure 2.5.4-249 Damping Ratio versus Cyclic Shear Strain

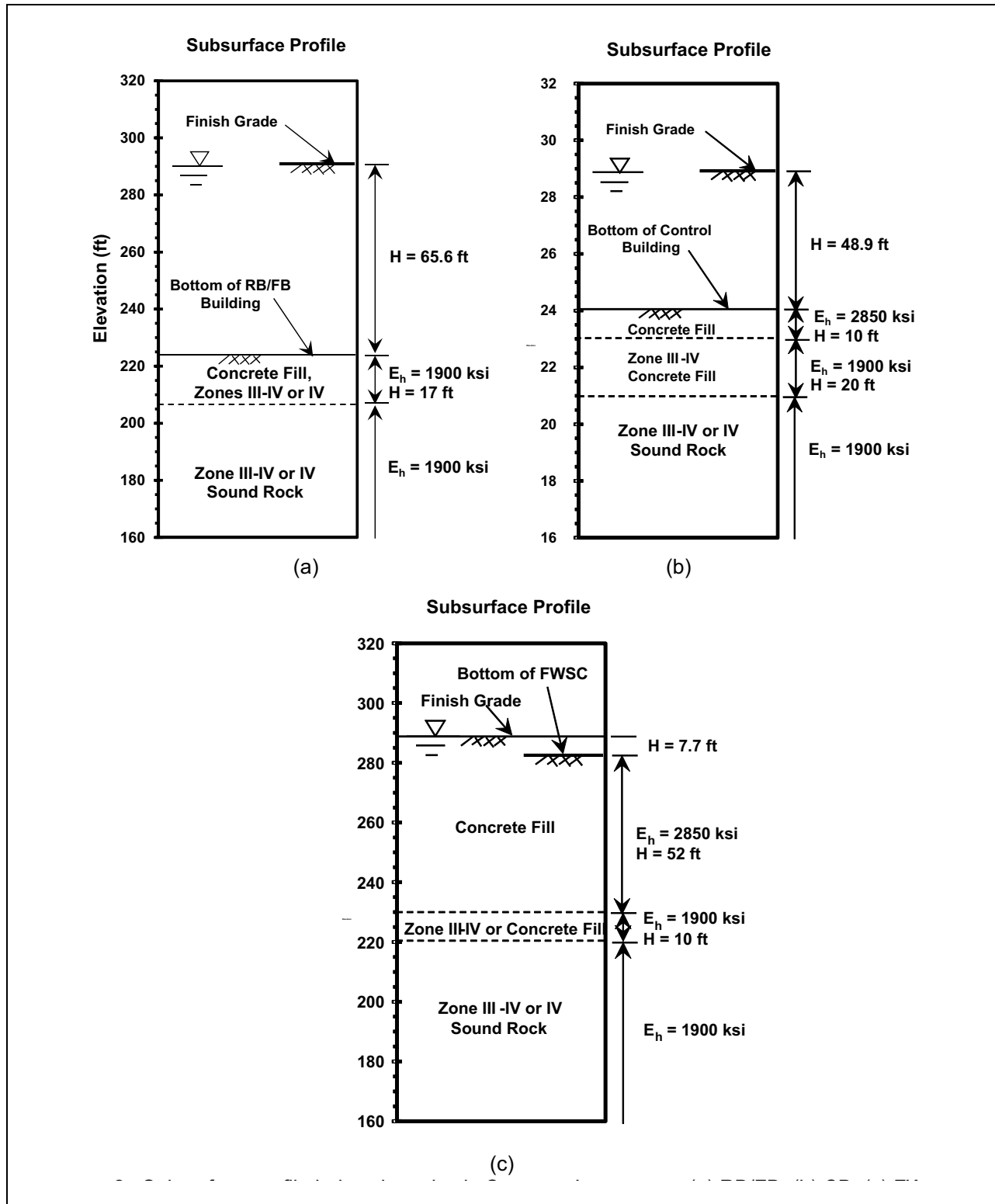


NAPS ESP COL 2.5-5 Figure 2.5.4-250 **Maximum Acceleration versus Depth for Free-Field Slope**

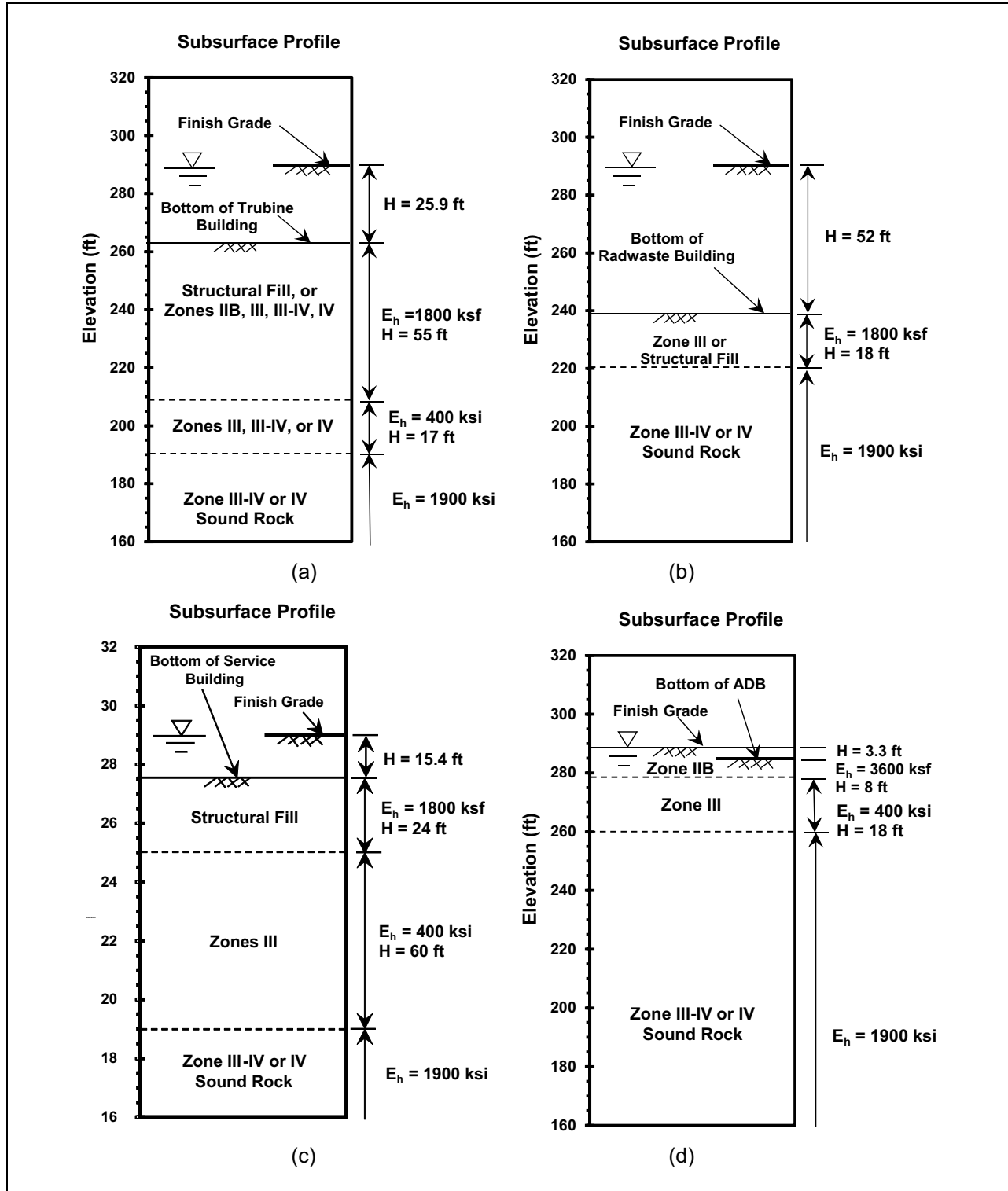


Note: The accelerations shown in this figure are obtained using input rock motions consistent with the EPRI (2004, 2006) GMM.

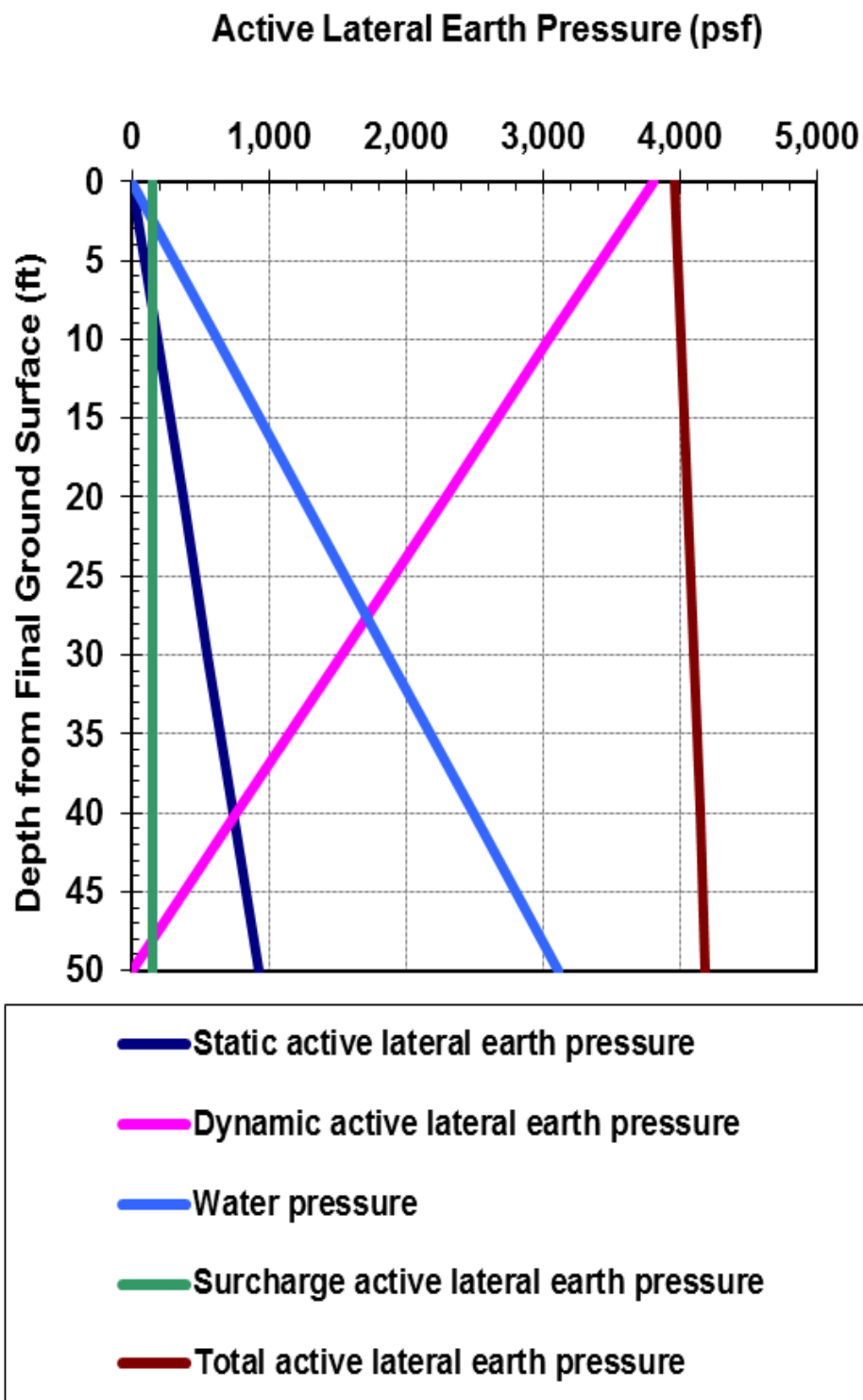
NAPS ESP COL 2.5-6 Figure 2.5.4-251 Subsurface Profiles Below the Seismic Category I Structures: (a) Reactor/Fuel Building; (b) Control Building; (c) FWSC



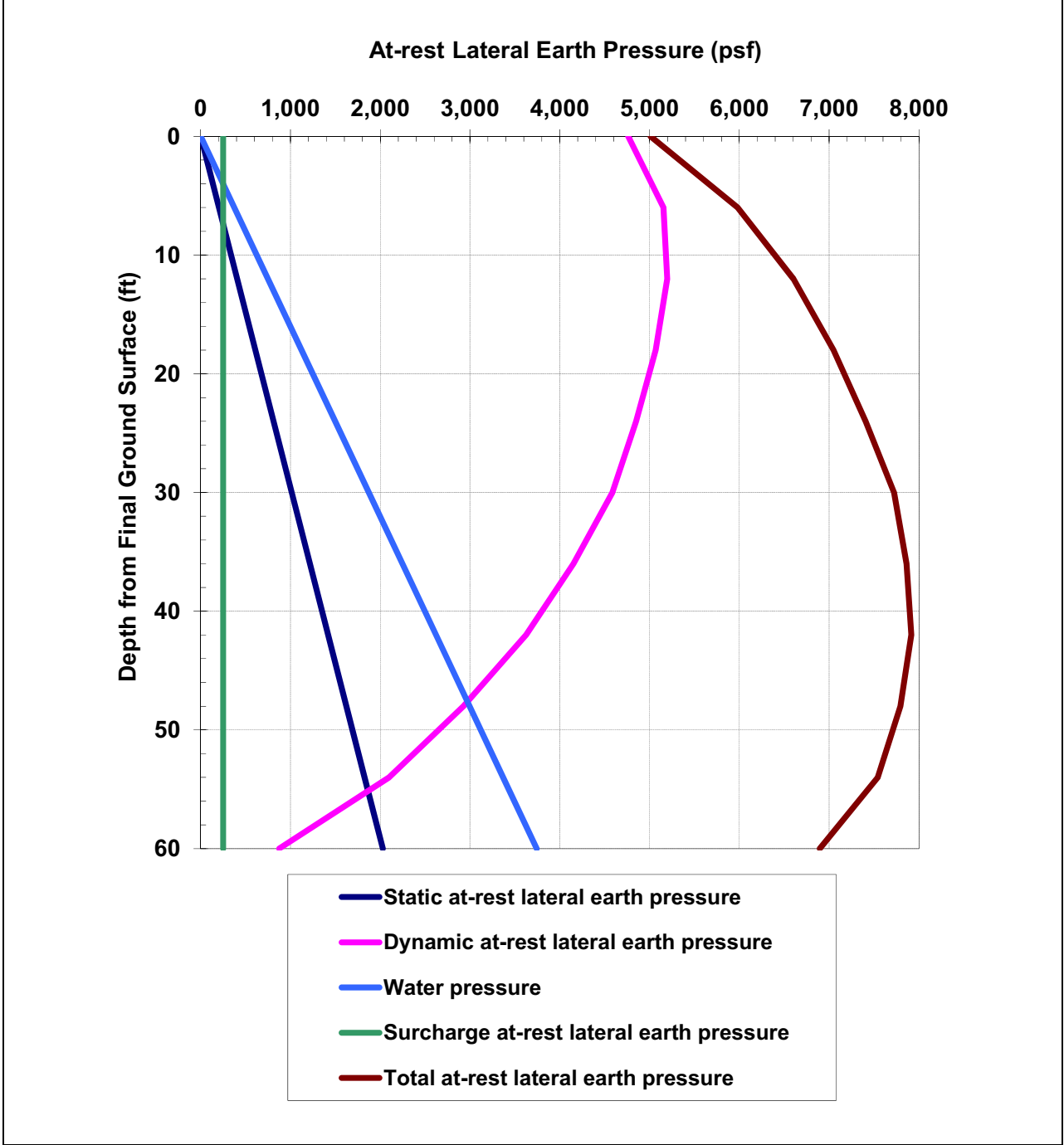
NAPS ESP COL 2.5-6 Figure 2.5.4-252 Subsurface Profiles Below Non-Seismic Category I Structures: (a) Turbine Building; (b) Radwaste Building; (c) Service Building; (d) Ancillary Diesel Building



NAPS ESP COL 2.5-6 Figure 2.5.4-253 Lateral Pressure on Yielding Wall (Active Case)



NAPS ESP COL 2.5-6 Figure 2.5.4-254 Lateral Pressure on Non-Yielding Wall (At-Rest Case)



2.5.5 Stability of Slopes

NAPS COL 2.0-30-A The information needed to address DCD COL Item 2.0-30-A is included in the following sections.

[SSAR Section 2.5.5](#) is incorporated by reference with the following variances and/or supplements.

NAPS ESP VAR 2.5-1 [SSAR Section 2.5.5](#) addressed the stability of slopes at the North Anna ESP site. However, the information presented in this FSAR section replaces the analyses presented in [SSAR Section 2.5.5](#) because the slopes being considered have changed, and, for the seismic slope stability analysis, the peak ground acceleration (PGA) being applied is different. The method of analysis remains essentially the same. In summary, the slopes considered herein are lower, less steep, and have a smaller applied seismic acceleration than the slopes analyzed in [SSAR Section 2.5.5](#). As a result, the slopes addressed in this section have a higher computed factor of safety against failure, and are stable under both long-term static and short-term seismic conditions.

This section presents information on the stability of permanent slopes at the Unit 3 site. The information was developed from a review of reports prepared for the existing units and the originally planned Units 3 and 4, geotechnical literature, the ESP subsurface investigation, and the Unit 3 subsurface investigation. The review included the site-specific reports from the UFSAR ([SSAR Reference 5](#)), and reports prepared by Dames and Moore regarding the design and construction of the existing units ([SSAR Reference 7](#)) and the originally planned Units 3 and 4 ([SSAR Reference 8](#)).

a. Description of Slopes

The grading plan for Unit 3 is shown in [Figure 2.5.5-201](#). The design plant grade for the power block area is at Elevation 290 ft with elevations around the perimeter of this area ranging from about Elevation 289 ft to 284 ft to allow for adequate surface drainage. To the northeast of the power block area, going towards the existing Units 1 and 2, ground surface elevation reduces by an approximately 5 percent slope down to Elevation 280 ft at the Service Water Building. (Directions in this section are with respect to plant north.) Stormwater management pond #1 is between these structures and the boundary of Units 1 and 2 with yard grade at Elevation 270 ft. As existing grade falls off from the power block

area northeast towards Units 1 and 2 some backfilling will be required to achieve final grade. As much as 30 ft of backfill will be needed to bring grade up to the planned ground surface area of the originally planned Units 3 and 4, where ground level is presently at around Elevation 250 ft.

All of the Seismic Category I structures at Unit 3 are founded directly on Zone III-IV or Zone IV rock or on concrete fill placed on rock. Foundation support for Seismic Category II structures can also include structural fill and Zone III rock. This is illustrated in the cross sections shown in [Figures 2.5.4-225 through 2.5.4-234](#). The locations of these cross sections are shown in [Figure 2.5.4-206](#). The concrete fill has a minimum strength of 2500 psi. Thus, there are no slopes that contribute to the support of any Unit 3 Seismic Category I and II structure or any other major power block structure.

The only slopes that could impact Unit 3 are cut slopes that surround and ascend from the southern and eastern edges of the plant, comprising existing and new cut slopes. As discussed in [Section 2.5.5b](#), material from sloughing or collapse of certain of these slopes could impact adjacent facilities.

The existing slopes (see Slope ES in [Figure 2.5.5-201](#) and refer to [Section 2.5.5.1.1](#)) run in a southwest-northeast direction, to the east of the originally planned Units 3 and 4 and existing Units 1 and 2. These slopes were excavated during construction of Units 1 and 2. Based on previous topographic maps, this slope was described in the SSAR as a 2h:1v slope, 55 ft high. A more recent topographic survey performed for Unit 3 shows that the slope is actually about 2.4h:1v with a maximum height of 52 ft (from Elevation 271 ft to Elevation 323 ft). Based on the final grade for Unit 3, the maximum height of this existing slope within the vicinity of any new structures is about 45 ft (from Elevation 278 ft to Elevation 323 ft).

The new cut slopes are east and south of the power block (see for example Slope DD in [Figure 2.5.5-201](#) and refer to [Section 2.5.5.1.2](#)), and as the slopes turn to the northeast, they merge into the existing slopes. These new slopes are cut at a 3-horizontal to 1-vertical (3h:1v) slope into the existing natural ground surrounding the plant, with a 15-ft wide bench constructed at about 10 ft height below the top of the slope. These slopes reach a maximum height of 39 ft (from about Elevation 281 ft up to Elevation 320 ft) east of the power block. A typical

maximum-height cross-section through this slope is shown as Slope DD in [Figure 2.5.5-201](#).

The maximum depth of the stormwater management pond to the northeast of the main plant area is 20 ft (from Elevation 280 ft down to Elevation 260 ft). This pond is cut at a 3h:1v slope.

[SSAR Section 2.5.5](#) refers to slopes resulting from the nonsafety-related deepened intake channel. In fact, the intake channel for Unit 3 will not be deepened, and thus there will be no new slopes associated with the intake channel.

As shown in [Figures 2.5.4-225](#) through [2.5.4-234](#), temporary excavation for Unit 3 construction will be performed using vertical walls.

b. Impact of Slope Instability

Instability of the stormwater management pond sides does not impact the safety of the plant, nor any of the other plant structures, and therefore these slopes are not addressed further here. Failure of any temporary slope or excavation created for construction of the plant cannot adversely affect the safety of the nuclear power plant facilities, and likewise are not addressed further here.

The existing 2.4h:1v slope (Slope ES) was excavated during construction of the Units 1 and 2, and is almost entirely in cut material. The top of this slope is about 200 ft from the top of the existing service water reservoir (SWR) embankment, and thus any potential instability of the slope will have no impact on the stability of the SWR embankment. However, material from sloughing or collapse of these slopes could possibly impact the new diesel tanks and/or service water cooling tower. However, the toe of the slope is over 100 ft from the new diesel tanks and even farther from the closest point on the service water cooling tower. Slope ES is a representative section along the approximately 700 ft length of the existing slope.

Instability of the new 3h:1v slope to the east of the FWSC (Slope DD) does not impact the foundation stability of this Seismic Category I facility since the facility is founded on concrete fill on top of Zone III-IV bedrock. However, material from sloughing or collapse of this slope could possibly impact the facility, because the base of this new 39 ft high slope is about 55 ft from the FWSC. As can be seen from [Figure 2.5.5-201](#), the new slopes extend to the south and then southwest of Slope DD and then west past the administration building, which is built into the slope.

Although these slopes are of similar height to Slope DD, they are much farther away from the Seismic Category I structures, and sloughing or collapse of these slopes would not impact any of the Seismic Category I structures. Thus, Slope DD is considered the critical slope in the area.

The stability of the existing slope closest to the new service water cooling tower and diesel tanks (Slope ES), and the stability of the new slope closest to the FWSC (Slope DD) are addressed in the following subsections.

2.5.5.1 Slope characteristics

2.5.5.1.1 Existing Slope Characteristics

The location and direction of the existing 45 ft high, 2.4h:1v slope to the north of the Units 1 and 2 SWR (Slope ES) is shown in plan view in [Figure 2.5.5-201](#); the location is also shown in the photograph in [SSAR Figure 2.5-66](#). The photograph in [SSAR Figure 2.5-67](#) shows the existing slope clearly, descending from the SWR to close to the excavation for the originally planned Unit 3 and 4 containment buildings. The structure behind the slope on the SWR embankment is the Units 1 and 2 valve house, which was initially designed to be the originally planned Units 3 and 4 pump house. An approximate cross-section through the existing slope is shown in [Figure 2.5.5-202](#).

As shown in a section view in [Figure 2.5.5-202](#), a boring (B-18) was drilled close to the toe of the slope. This boring, made for the Units 1 and 2 investigation, is located in plan view in [Figure 2.5.4-217](#). During the Unit 3 subsurface investigation, CPT C-915 (located in plan view in [Figure 2.5.4-218](#)) was performed near to the top of the slope. Also during the Unit 3 investigation, boring B-947 was drilled to the southwest of C-915, but at a similar elevation within the same original terrain as C-915. CPT C-916 and observation well OW-947 were located adjacent to B-947. The locations of boring B-18 and CPT C-915 are included in [Figure 2.5.5-202](#), along with the groundwater levels extrapolated from the values measured in OW-947. The boring and CPT logs are presented in [Section 2.5.5.3](#).

2.5.5.1.2 New Slope Characteristics

NAPS ESP COL 2.5-11

The location of the new 39 ft high, 3h:1v slope to the east of the FWSC (Slope DD) is shown in plan view in [Figure 2.5.5-201](#). An approximate cross-section through the new slope is shown in [Figure 2.5.5-203](#). As

shown in [Figure 2.5.4-217](#), boring B-947 was drilled relatively close to the final location of the top of the slope during the Unit 3 subsurface investigation. CPT C-916 and observation well OW-947 were located adjacent to B-947. The boring and CPT logs are presented in [Section 2.5.5.3](#). The stability analysis performed for Slope DD ([Section 2.5.5.2.4](#)) conservatively neglected the 15-ft wide bench in the slope. For consistency, this bench is not shown in [Figure 2.5.5-203](#).

NAPS COL 2.0-30-A

2.5.5.1.3 Slope Subsurface Conditions

The site soils and bedrock are described in detail in [Section 2.5.4.2.2](#). As can be seen from [Figures 2.5.5-202](#) and [2.5.5-203](#), the materials in the existing and new slopes, respectively, consist mostly of Zone IIA saprolites. Saprolites are a further stage of weathering beyond weathered rock. They have been derived by in-place disintegration and decomposition and have not been transported. Saprolites are classified as soils but still contain the relict structure of the parent rock, and they also typically still contain some core stone of the parent rock. The North Anna saprolites in many instances maintain the foliation (banded texture) characteristics of the parent rock. The majority of the saprolites in the Unit 3 area are classified as silty sands, although there are also sands, clayey sands, sandy silts, clayey silts and clays, depending very much on their degree of weathering. The fabric is strongly anisotropic. The texture shows angular geometrically interlocking grains with a lack of void network, very unlike the well-pronounced voids found in marine or alluvial sands and silts. The Zone IIA saprolites comprise a large majority of the saprolitic materials onsite. Most of the saprolites obtained from the borings in the slope area are medium dense to dense silty sands. The underlying Zone IIB saprolites are generally very dense silty sands.

Boring B-18 in [Figure 2.5.5-202](#) indicates top of bedrock levels rising significantly towards the toe of the existing slope, with top of weathered rock close to the slope surface at the B-18 location at around Elevation 290 ft. This is consistent with the top of bedrock levels shown in [Figure 2.5-2](#), from [SSAR Reference 5](#). For the new slope shown in [Figure 2.5.5-203](#), the top of weathered rock is closer to Elevation 250 ft. The bedrock at North Anna ranges from moderately to severely weathered (Zone III) as encountered in B-18, to fresh to slightly weathered (Zone IV) at greater depths. The bedrock throughout the North Anna site is classified as a gneiss, which is a metamorphic rock that exhibits foliation in which light and dark bands alternate. It is

composed of feldspar, quartz, and one or more other minerals such as mica and hornblende.

The engineering properties of the site soils and bedrock are described in [Section 2.5.4.2.5](#) and are tabulated in [Table 2.5.4-208](#). These properties are based on extensive field and laboratory testing described in [Section 2.5.4.2](#).

The liquefaction characteristics of all of the Zone IIA saprolites are thoroughly examined in [Section 2.5.4.8](#). That section concludes that the results of the liquefaction analysis indicate that only a very limited amount of the Zone IIA saprolitic soil has a potential for liquefaction based on the Unit 3 seismic parameters. The liquefaction analysis did not take into account the beneficial effects of structure, fabric, and mineralogy of the saprolitic soils.

Details of the soils encountered in the new and existing slopes are outlined in the following paragraphs.

New Slope Subsurface Conditions

Boring B-947, close to the top of the new slope, indicates a predominantly silty sand profile, alternating with layers of silt in the top 15 ft (boring and CPT logs are presented in [Section 2.5.5.3](#)). Grain size analyses performed on 10 samples ranging in depth from 5 ft to 43 ft (see [Section 2.5.5.3](#)) showed fines contents varying from about 14 to 70 percent, with a median of about 29 percent. The bottom 10 ft of soil has an adjusted SPT N-value of over 50 blows/ft, which is characteristic of Zone IIB saprolite. The overlying soils are Zone IIA saprolites. Interpretation of CPT C-916 (performed adjacent to B-947) based on friction ratio, indicates mainly silty clays and clays, and thus, for these saprolites, this interpretation indicates a less granular composition than shown in the grain size tests.

For stability analyses of the new slope presented in [Section 2.5.5.2](#), based on the results of B-947 and C-916, the new slope has the properties of Zone IIA silty sand saprolite given in [Table 2.5.4-208](#) down to about 35 to 50 ft below existing ground level. The bottom 10 ft of saprolite above weathered rock has the Zone IIB saprolite properties given in [Table 2.5.4-208](#).

The log of CPT-915 (located close to the top of the existing slope) is very similar to the log of CPT-916 in the top 30 ft. CPT-915 continues in a similar pattern below 30 ft down to 50 ft where it shows significantly

increased tip resistance. Below 30 ft, C-916 shows higher tip resistance values than C-915 down to 50 ft depth.

Existing Slope Subsurface Conditions

Boring B-18 provides information on the subsurface materials at the toe of the slope, namely, weathered rock. There are no borings at or near the top of the slope, only CPT C-915. However, as noted in [Section 2.5.5.1.1](#), boring B-947 was drilled to the southwest of C-915, but at a similar elevation within the same original terrain as C-915. B-947 shows a predominantly silty sand profile. Grain size analysis conducted on 10 samples ranging from 5 ft depth to 43 ft depth from B-947 showed fines contents with a median of about 29 percent. The log of CPT C-915 is similar to the log of CPT C-916 (immediately adjacent to B-947) and indicates a mainly silty clay and clay profile, in contrast with the actual silty sand samples obtained from B-947. CPTs provide valuable information on the soil by measuring cone tip resistance, sleeve friction, and pore-water pressure at very closely-spaced depth intervals throughout the soil profile. However, no samples were obtained with the CPT and so there is no direct evidence of the type of soil being measured. Instead, the soil type is selected based mainly on the friction ratio, which is the ratio, expressed as a percentage, of the measured sleeve friction to the tip resistance. The interpretation of soil type from friction ratio is empirical, based on historical correlations between ratio and soil type identified from adjacent borings. However, like most empirical correlations in geotechnical engineering, it is not exact for all soil types, and this was the case in C-916 (and presumably C-915). Thus, the silty sand profile in B-947 was the profile used in the slope stability analysis of the existing slope.

For the stability analysis of the existing slope presented in [Section 2.5.5.2](#), as noted above, the existing slope has the properties of Zone IIA silty sand saprolite given in [Table 2.5.4-208](#). This material extends down to about 53 ft below the ground level beyond the top of the slope. The thickness of Zone IIB saprolite below the Zone IIA material becomes less towards the toe of the slope and this layer eventually pinches out as the top of weathered rock rises, as shown in [Figure 2.5.5-202](#). The Zone IIB saprolite and the weathered rock have the properties given in [Table 2.5.4-208](#).

2.5.5.1.4 Slope Phreatic Surface

The phreatic surfaces shown in [Figure 2.5.5-202](#) (existing slope ES) and [Figure 2.5.5-203](#) (new slope DD) have been developed from the water table levels measured in OW-947 and derived in [Section 2.4.12](#). The depth of this phreatic surface precludes any potential for liquefaction of the near-surface soils in the slope.

2.5.5.2 Design Criteria and Analyses

2.5.5.2.1 Required Factor of Safety

Minimum required factors of safety for stability of slopes under long-term static (non-seismic) loading and for dynamic (seismic) loading are 1.5 and 1.1, respectively.

2.5.5.2.2 Stability of Existing Slope

The photograph in [SSAR Figure 2.5-67](#) of the existing 2.4h:1v slope to the north of the SWR was taken over 20 years ago. The condition of the slope is essentially the same today. It was thoroughly inspected during the ESP site investigation. The slope shows no signs of distress.

2.5.5.2.3 Analysis of Existing Slope

The static and dynamic stability of the existing slope to the north of the SWR was analyzed using the computer program SLOPE/W ([Reference 2.5-219](#)).

a. Long-Term Static Analysis

The SLOPE/W Program used the Bishop method of slices ([SSAR Reference 185](#)) for analysis of the long-term static condition. As noted in [Section 2.5.5.1.3](#), the analysis assumed the saprolite was predominantly coarse grained. The effective strength parameters given in [Table 2.5.4-208](#) are an angle of internal friction $\phi' = 33$ degrees and effective cohesion $c' = 0.125$ ksf for the Zone IIA saprolite and $\phi' = 40$ degrees and effective cohesion $c' = 0$ ksf for the Zone IIB saprolite. The underlying weathered rock used $c = 70$ ksf, approximately half of the value for unconfined compressive strength given in [Table 2.5.4-208](#).

The input to the analysis and the results are shown in [Figure 2.5.5-204](#). The computed factor of safety is 2.29. This value is above the minimum 1.5 factor of safety required.

b. Seismic Slope Stability Analysis

NAPS ESP COL 2.5-10

The pseudo-static approach is used as a first approximation for the seismic analysis of slopes. In this approach, the horizontal and vertical seismic forces are assumed to act on the slope in a static manner, that is, as a constant static force. This is an obviously conservative approach, since the actual seismic event occurs for only a short period of time, and during that time, the forces alternate their direction at a relatively high frequency. Also, the pseudo-static analysis tends to be run using the peak seismic acceleration; the mean acceleration during the design seismic event can be significantly less than the peak value. A pseudo-static analysis using peak acceleration values can be a useful tool in a limit analysis where the peak acceleration is relatively low. In such analyses, the computed factor of safety may well exceed the minimum of 1.1, thus requiring no further analysis. However, where the peak seismic acceleration values are high, the pseudo-static analysis produces unreasonably low safety factor values.

The pseudo-static analysis was run on the existing 45 ft high slope (Slope ES) using SLOPE/W with the Bishop method of slices. For the low frequency earthquake, the peak horizontal acceleration used was about 0.26g. This is the average peak acceleration in the top 45 ft of soil shown in [Table 2.5.5-201](#). (The maximum horizontal acceleration is 0.31g at about 42 ft depth.) The vertical acceleration used was about 0.13g. The computed factor of safety was 1.30, more than the minimum 1.1 required. For the high frequency earthquake, the equivalent peak horizontal acceleration used was 0.42g with a vertical acceleration of 0.21g. (The maximum horizontal acceleration is 0.56g at about 42 ft depth.) The computed factor of safety was about 1.04, less than the minimum 1.1 required. The input to the analysis, and the results, are shown in [Figure 2.5.5-205](#) for the low frequency earthquake and [Figure 2.5.5-206](#) for the high frequency earthquake.

The low frequency and high frequency acceleration values noted in the previous paragraph were obtained using input rock motions consistent with the EPRI (2004, 2006) ([References 2.5-224, 2.5-225](#)) GMM. As noted in [Section 2.5.4.7.4](#), these input motions were subsequently updated based on the EPRI (2013) ([Reference 2.5-407](#)) GMM for use in developing the GMRS and FIRS for Seismic Category I buildings. These updated rock motions are lower than those from the EPRI (2004, 2006) GMM and result in lower site ground motion response spectra, and

correspondingly lower ground accelerations for the low frequency and high frequency input motions. Thus, the factors of safety against slope failure shown in [Figures 2.5.5-205](#) and [2.5.5-206](#), and in [Figures 2.5.5-209](#) and [2.5.5-210](#) for Kramer's approach (below), which are based on the EPRI (2004, 2006) GMM, can be conservatively used in place of the EPRI (2013) GMM. These figures are therefore not updated.]

Seed ([SSAR Reference 186](#)), in the 19th Rankine Lecture, addressed the over-conservatism intrinsic in the pseudo-static analysis. He looked at the more rational approach proposed by Newmark ([SSAR Reference 187](#)), where the effective acceleration time-history is integrated to determine velocities and displacements of the slope. He also examined dams in California that had been subjected to seismic forces, including several dams that survived the 1906 San Francisco earthquake. Based on his studies, he concluded that for embankments that consist of materials that do not tend to build up large pore pressures or lose significant percentages of their shear strength during seismic shaking, seismic coefficients of only 0.15g are adequate to ensure acceptable embankment performance for earthquakes up to Magnitude $M = 8.25$ with PGAs of 0.75g. For earthquakes in the range of $M = 6.5$, Seed recommends a horizontal seismic coefficient of only 0.1g with a vertical seismic coefficient of zero. Note that it is the magnitude of the earthquake that determines the acceleration to be used here; magnitude is not part of the input to the pseudo-static analysis.

The liquefaction analysis of the Zone IIA saprolite indicated that only a limited amount of the material has a potential for liquefaction. Also, because of its age, fabric and interlocking angular grain structure, this material does not lose a significant proportion of its shear strength during shaking. Thus, for the low frequency earthquake, with a design Magnitude $M = 7.2$, the pseudo-static analysis should be limited to a horizontal acceleration of 0.15g. A pseudo-static design using an inertia force of 0.1g is adequate for the high frequency earthquake.

The pseudo-static analysis was run again using SLOPE/W. This time the horizontal accelerations used were 0.1g and 0.15g, with zero vertical acceleration. The computed factors of safety were 1.76 and 1.57, respectively, greater than the minimum 1.1 required. The input to the analysis, and the results, for the 0.1g and 0.15g cases are shown in [Figures 2.5.5-207](#) and [2.5.5-208](#), respectively.

Other researchers have also recommended substantially reducing the peak acceleration when applying the pseudo-static analysis. Kramer ([SSAR Reference 188](#)) recommends using an acceleration of 50 percent of the peak acceleration. For the low frequency earthquake, where the average peak acceleration in the top 45 ft is about 0.26g, the horizontal input using Kramer's recommendations was about 0.13g and the vertical input was about 0.065g. This results in a factor of safety of 1.65. Using the average peak acceleration for the high frequency earthquake in the top 45 ft of 0.42g, the horizontal input using Kramer's recommendation was 0.21g and the vertical input was 0.105g. This level of input provides a factor of safety against slope failure of 1.41. Thus the low and high frequency inputs give factors of safety above the minimum 1.1 required. The input to the analysis, and the results, for the low frequency and high frequency cases are shown in [Figures 2.5.5-209](#) and [2.5.5-210](#), respectively.

In the preceding analyses (both long-term static, and seismic), the only case that gave a factor of safety lower than the required minimum was the pseudo-static analysis using the high frequency peak acceleration. As noted above, the pseudo-static analysis does not take into account the frequency of the motion nor the magnitude of the earthquake. For high frequency, low magnitude earthquakes, (as is the case at North Anna) the pseudo-static analysis is particularly conservative. Thus, it is concluded that the existing 2.4h:1v slope to the north of the SWR remains stable under long-term static and design seismic conditions.

NAPS COL 2.0-30-A

2.5.5.2.4 Analysis of New Slope

The static and dynamic stability of the new 39-ft high 3h:1v slope (Slope DD) to the east of the FWSC was analyzed using the computer program SLOPE/W ([Reference 2.5-219](#)).

a. Long-Term Static Analysis

The SLOPE/W Program used the Bishop method of slices ([SSAR Reference 185](#)) for analysis of the long-term static condition. As noted in [Section 2.5.5.1.3](#), the properties assumed for the Zone IIA and Zone IIB saprolite were the same as those for the existing slope that was analyzed.

The input to the analysis and the results are shown in [Figure 2.5.5-211](#). The computed factor of safety is 2.27. This value is above the minimum 1.5 factor of safety required.

b. Seismic Slope Stability Analysis

NAPS ESP COL 2.5-10

The pseudo-static analysis was run on the new 39 ft high slope using SLOPE/W with the Bishop method of slices. For the low frequency earthquake, the average peak horizontal acceleration in the top 39 ft used was about 0.25g with a vertical acceleration of about 0.125g. (The maximum horizontal acceleration is 0.31g at about 42 ft depth.) The computed factor of safety was 1.24, greater than the minimum 1.1 required. For the high frequency earthquake, the peak horizontal acceleration used was about 0.41g. This is the average peak acceleration in the top 39 ft of soil shown in [Table 2.5.5-201](#). (The maximum horizontal acceleration is 0.56g at about 42 ft depth.) The vertical acceleration used was about 0.205g. The computed factor of safety was 1.00, less than the minimum 1.1 required. The input to the analysis, and the results, for the low frequency and high frequency cases are shown in [Figures 2.5.5-212](#) and [2.5.5-213](#), respectively.

As noted for the existing slope in [2.5.5.2.3.b](#), the factors of safety against slope failure for the new slope shown in [Figures 2.5.5-212](#) and [2.5.5-213](#), and in [Figures 2.5.5-216](#) and [2.5.5-217](#) for Kramer's approach (below) that are based on the EPRI (2004, 2006) GMM can be conservatively used for the EPRI (2013) GMM. These figures are therefore not updated.

The pseudo-static analysis was run again using SLOPE/W and Seed's ([SSAR Reference 186](#)) approach described in [Section 2.5.5.2.3](#). Again the horizontal accelerations used were 0.1g and 0.15g for the high and low frequency cases, respectively, with zero vertical acceleration. The computed factors of safety were 1.64 and 1.43, respectively, greater than the minimum 1.1. The input to the analysis, and the results, for the 0.1g and 0.15g cases are shown in [Figures 2.5.5-214](#) and [2.5.5-215](#), respectively.

The pseudo-static analysis was then run using SLOPE/W and Kramer's ([SSAR Reference 188](#)) recommendations described in [Section 2.5.5.2.3](#). For the low frequency earthquake, where the average peak acceleration in the top 39 ft is about 0.25g, the horizontal input using Kramer's recommendations was about 0.125g and the vertical input was about 0.063g. Using the average peak acceleration for the high frequency earthquake in the top 39 ft of about 0.41g, the horizontal input using Kramer's recommendation was 0.205g and the vertical input was 0.103g. These levels of input provide a factor of safety against slope failure of 1.59 and 1.34 for the low and high frequency cases, respectively, greater

than the minimum 1.1 required. The input to the analysis, and the results, for the low frequency and high frequency cases are shown in [Figures 2.5.5-216](#) and [2.5.5-217](#), respectively.

The results of the stability analyses for the new slope are almost identical to those for the existing slope, and the conclusion about stability is the same, i.e., the new 3h:1v slope to the east of the FWSC remains stable under long-term static and design seismic conditions.

NAPS COL 2.0-30-A

2.5.5.3 **Logs of Borings**

2.5.5.3.1 **Boring Logs**

As noted in [Section 2.5.5.1](#), boring B-18 was drilled close to the toe of the existing 2.4h:1v slope to the north of the SWR and boring B-947 was drilled near the top of the proposed new 3h:1v slope east of the FWSC. The logs of borings B-18 and B-947 are reproduced in [Figures 2.5.5-218](#) and [2.5.5-219](#), respectively.

2.5.5.3.2 **CPT Logs**

As noted in [Section 2.5.5.1](#), CPT C-915 was drilled close to the top of the existing 2.4h:1v slope to the north of the SWR and CPT C-916 was drilled adjacent to B-947 near the top of the new 3h:1v slope east of the FWSC. The logs of CPTs C-915 and C-916 are reproduced in [Figures 2.5.5-220](#) and [2.5.5-221](#), respectively.

2.5.5.3.3 **Observation Wells**

As noted in [Section 2.5.5.1](#), observation well OW-947 was installed adjacent to boring B-947 near the top of the new 3h:1v slope east of the FWSC. The log of OW-947 is reproduced in [Figure 2.5.5-222](#). Water levels measured in this well over a 12-month period are shown in [Table 2.5.5-202](#).

2.5.5.3.4 **Laboratory Test Results**

The grain size tests results for the saprolites in boring B-947 and noted in [Section 2.5.5.1](#) are provided in [Table 2.5.5-203](#). Details of these test results are provided in [Appendix 2.5.4AA](#).

2.5.5.4 **Compacted Fill**

The existing 2.4h:1v slope described and analyzed in the previous sections is a cut slope and does not contain fill materials in any significant quantity.

As shown in [Figure 2.5.5-203](#), the grading plan results in the top approximately 8 ft of the new 3h:1v slope east of the FWSC being new fill. This is not structural fill since it is used only for site grading and consists of re-compacted saprolitic soils obtained from plant excavations. These are described in [Section 2.5.4.5](#). For slope stability analysis, this fill has been given the same properties as the in-situ Zone IIA saprolite.

2.5.5.5 **Conclusions**

Existing slopes and embankments that are not impacted by Unit 3 (such as the SWR embankments) do not require analysis for Unit 3 and are not addressed here. New slopes, such as in the stormwater management pond #1 that will not impact the safety of the plant or any other structure if they fail also do not require analysis and are not addressed here. Failure of any temporary slope or excavation created for construction of Unit 3 cannot adversely affect the safety of Unit 3, consequently, this is not addressed further here.

The only existing slope which, by its failure, could adversely affect the safety of Unit 3, because of its proximity, is the 45 ft high, 2.4h:1v slope that descends from north of the SWR down to southeast of the existing excavation made for the originally planned Units 3 and 4. The slope is made almost entirely in cut material. The stability of this existing slope was analyzed using the computer program SLOPE/W. The only case that gave a factor of safety lower than the required minimum was the pseudo-static analysis using the high frequency peak ground acceleration. This analysis does not take into account the frequency of the motion or the magnitude of the earthquake. For high frequency, low magnitude earthquakes, (as is the case at North Anna) the pseudo-static analysis is particularly conservative. Thus, based on less conservative analysis, it is concluded that this slope remains stable under long-term static and design seismic conditions.

The results of the stability analyses for the new 3h:1v slope to the east of the FWSC are almost identical to those for the existing slope described above, and the conclusion about stability is the same, i.e., the new slope remains stable under long-term static and design seismic conditions.

NAPS ESP COL 2.5-10 Table 2.5.5-201 Maximum Acceleration Results

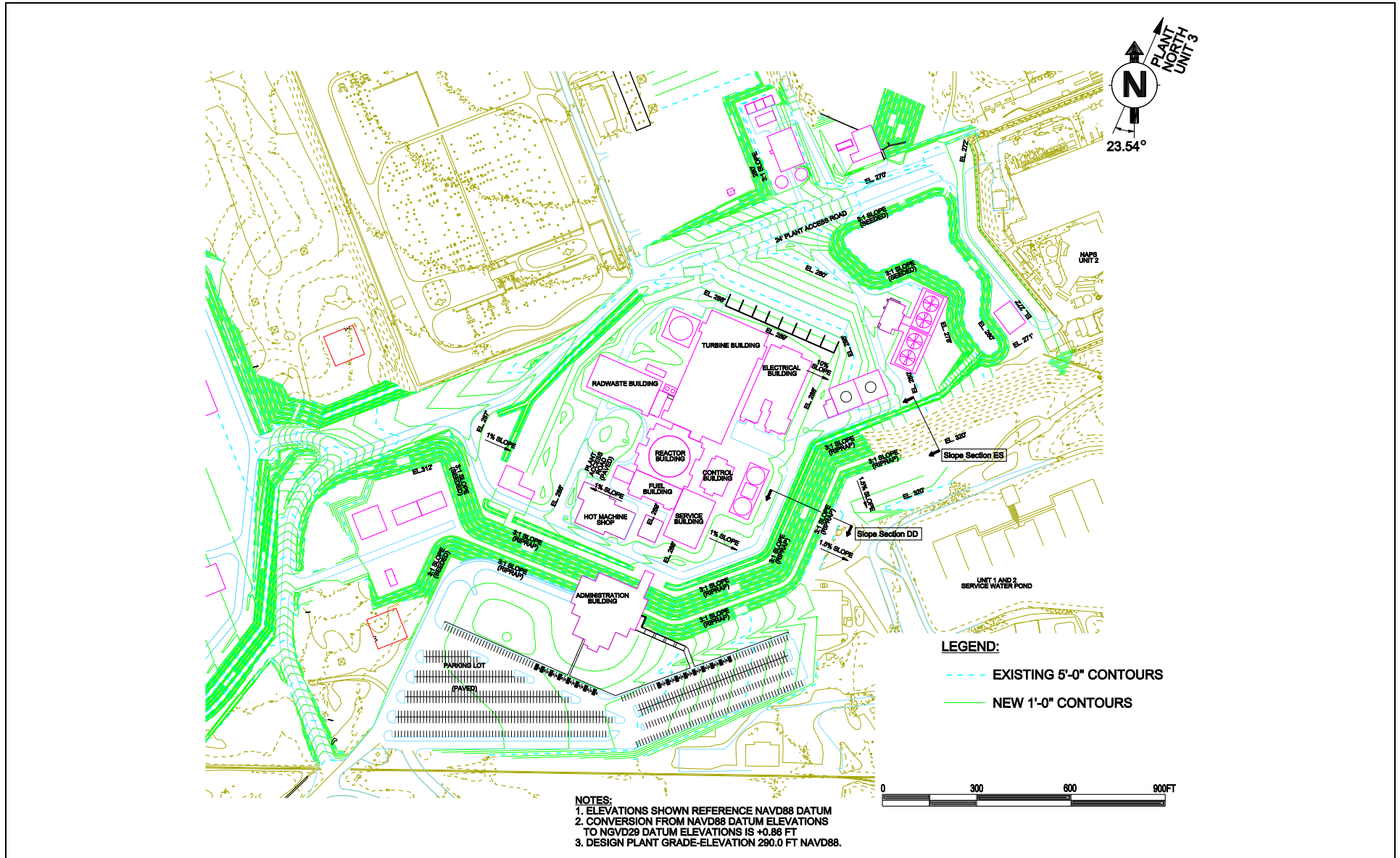
Depth (ft)	Low Frequency Max. Acc. (g)	High Frequency Max. Acc. (g)
0.0	0.285	0.467
1.5	0.281	0.454
3.0	0.271	0.423
4.5	0.260	0.391
6.0	0.249	0.366
7.5	0.240	0.349
9.0	0.232	0.338
10.5	0.226	0.332
12.0	0.223	0.334
13.5	0.223	0.341
15.0	0.226	0.352
17.0	0.234	0.372
19.0	0.237	0.381
21.0	0.241	0.386
23.0	0.245	0.398
25.0	0.253	0.413
27.0	0.255	0.424
29.0	0.263	0.442
31.0	0.271	0.462
33.0	0.276	0.476
35.0	0.280	0.487
37.5	0.285	0.505
40.0	0.300	0.533
42.5	0.311	0.559
45.0	0.303	0.546
47.5	0.294	0.528
50.0	0.290	0.523
52.5	0.288	0.525
55.0	0.288	0.533

NAPS COL 2.0-30-A Table 2.5.5-202 Water Level Measurements for Well OW-947

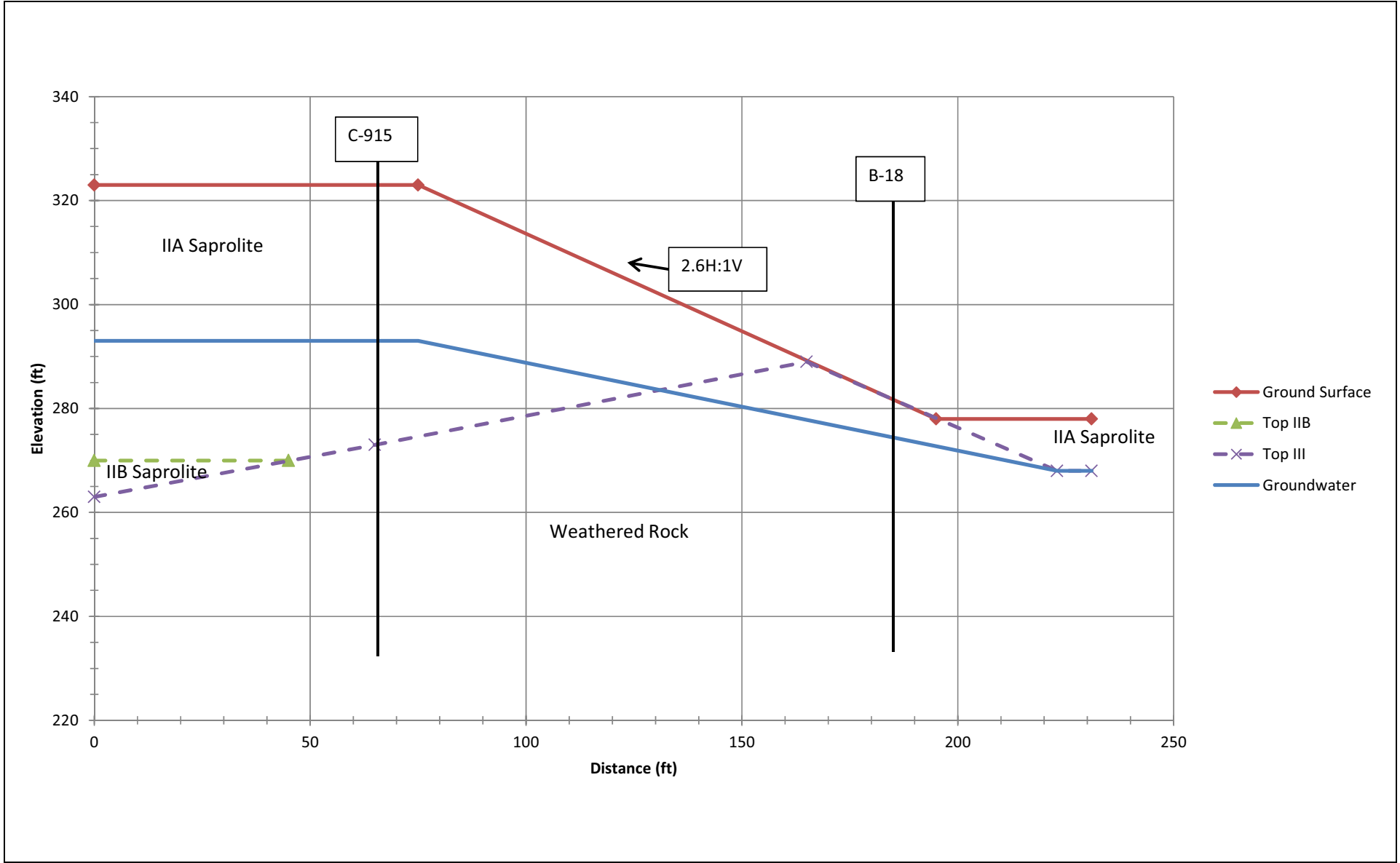
Date	Groundwater Elevation, Ft
11/29/2006	297.61
2/28/2007	297.81
5/30/2007	297.92
8/29/2007	296.00

NAPS COL 2.0-30-A Table 2.5.5-203 Grain-Size Test Results for Boring B-947

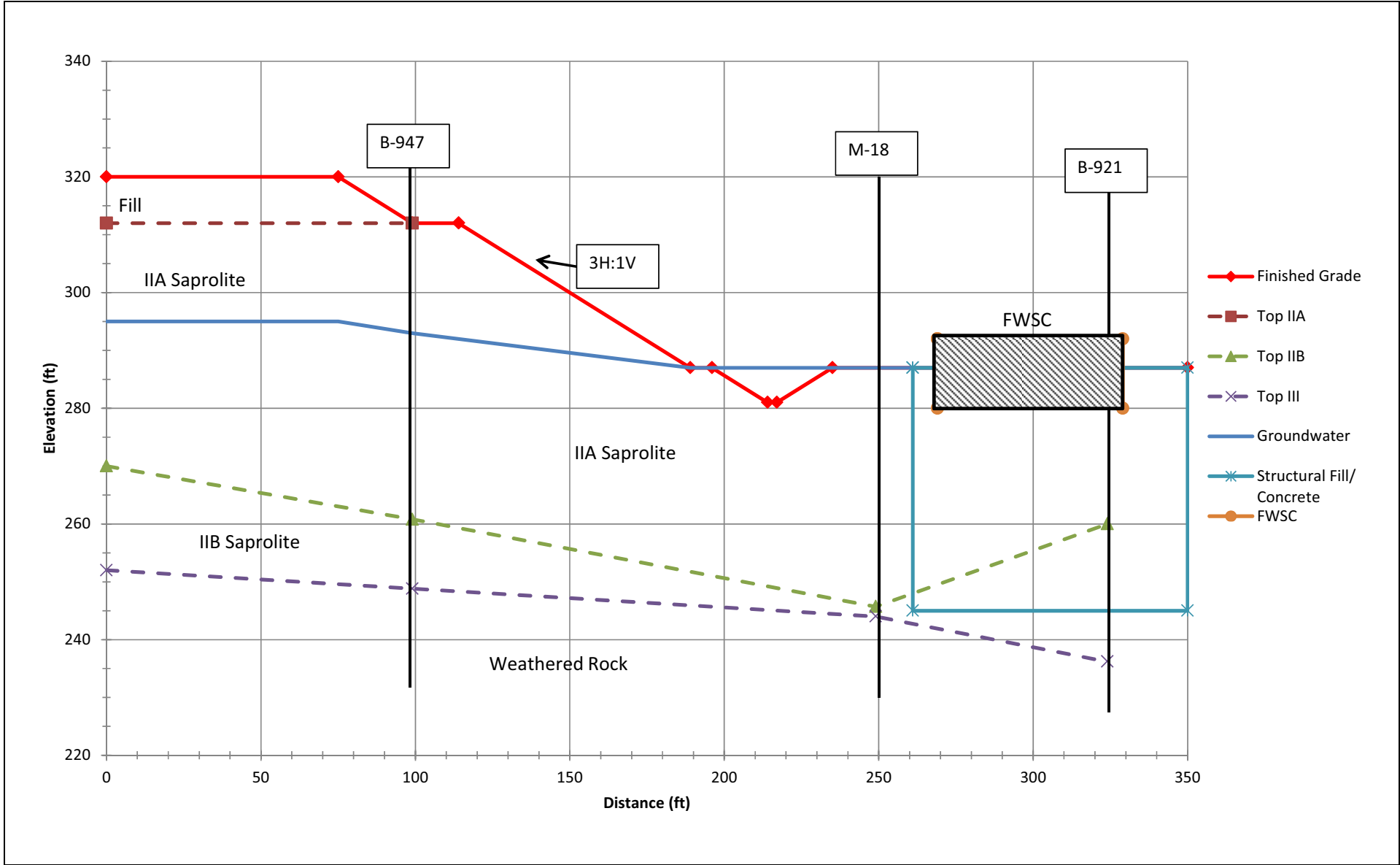
Sample No.	Depth (Ft)	Gravel (%)	Sand (%)	Fines (%)	Silt (%)	Clay (%)
B-947-3	4.5–6.0	0.0	38.3	61.7	23.5	38.2
B-947-4	8.5–10.0	0.0	60.0	40.0	-	-
B-947-5	9.5–11.0	1.6	55.9	42.5	21.1	21.4
B-947-6	13.5–15.0	0.0	30.5	69.5	-	-
B-947-7	17.2–18.7	0.0	75.8	24.2	-	-
B-947-8	22.2–23.7	0.6	79.4	20.0	10.7	9.3
B-947-9	28.7–30.2	0.0	66.6	33.4	-	-
B-947-10	33.7–35.2	0.0	81.3	18.7	-	-
B-947-11	38.7–40.2	0.0	85.8	14.2	-	-
B-947-12	42.2–43.7	0.0	79.7	20.3	13.4	6.9

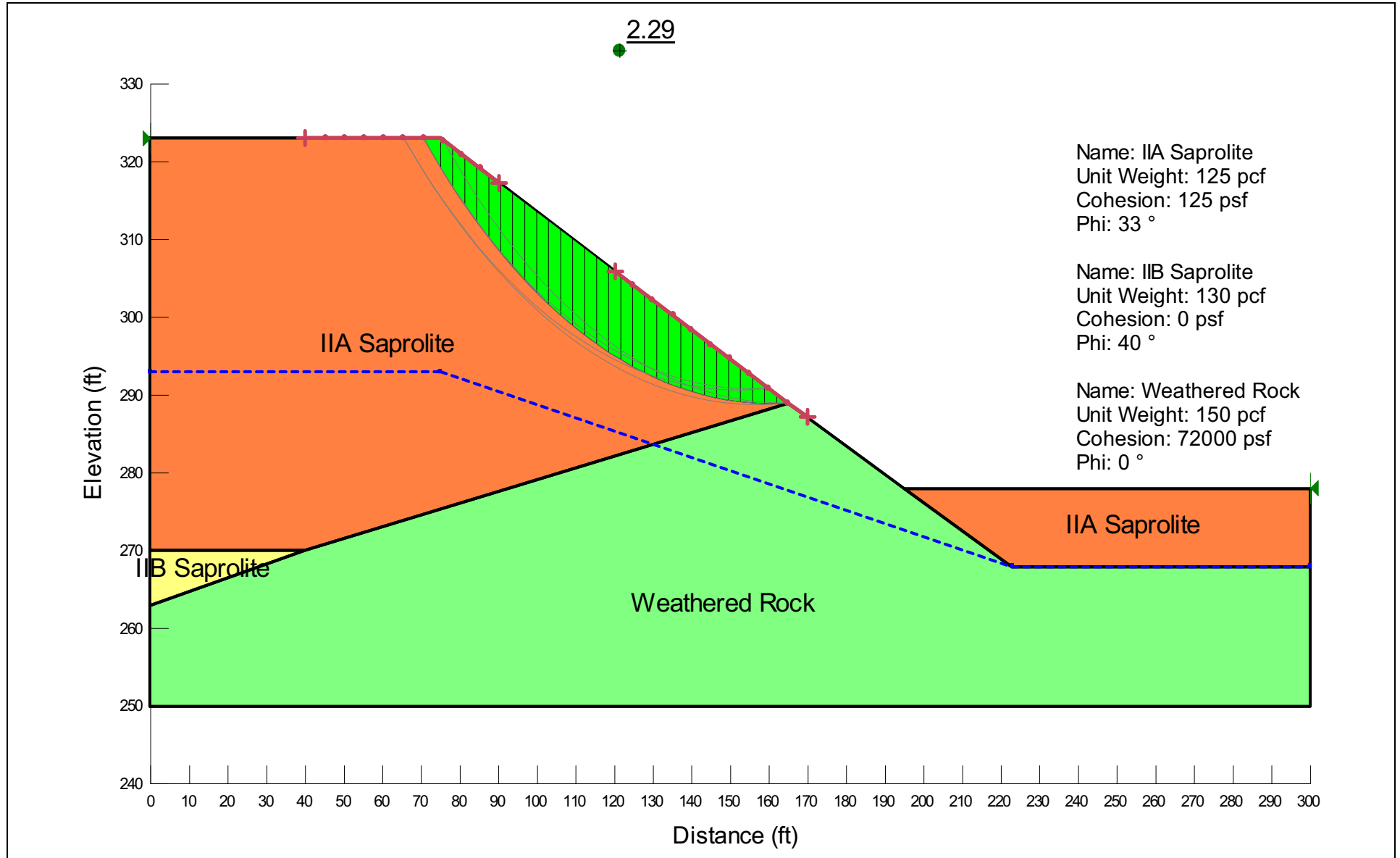


NAPS COL 2.0-30-A Figure 2.5.5-202 Cross-Section of Existing Slope (ES)

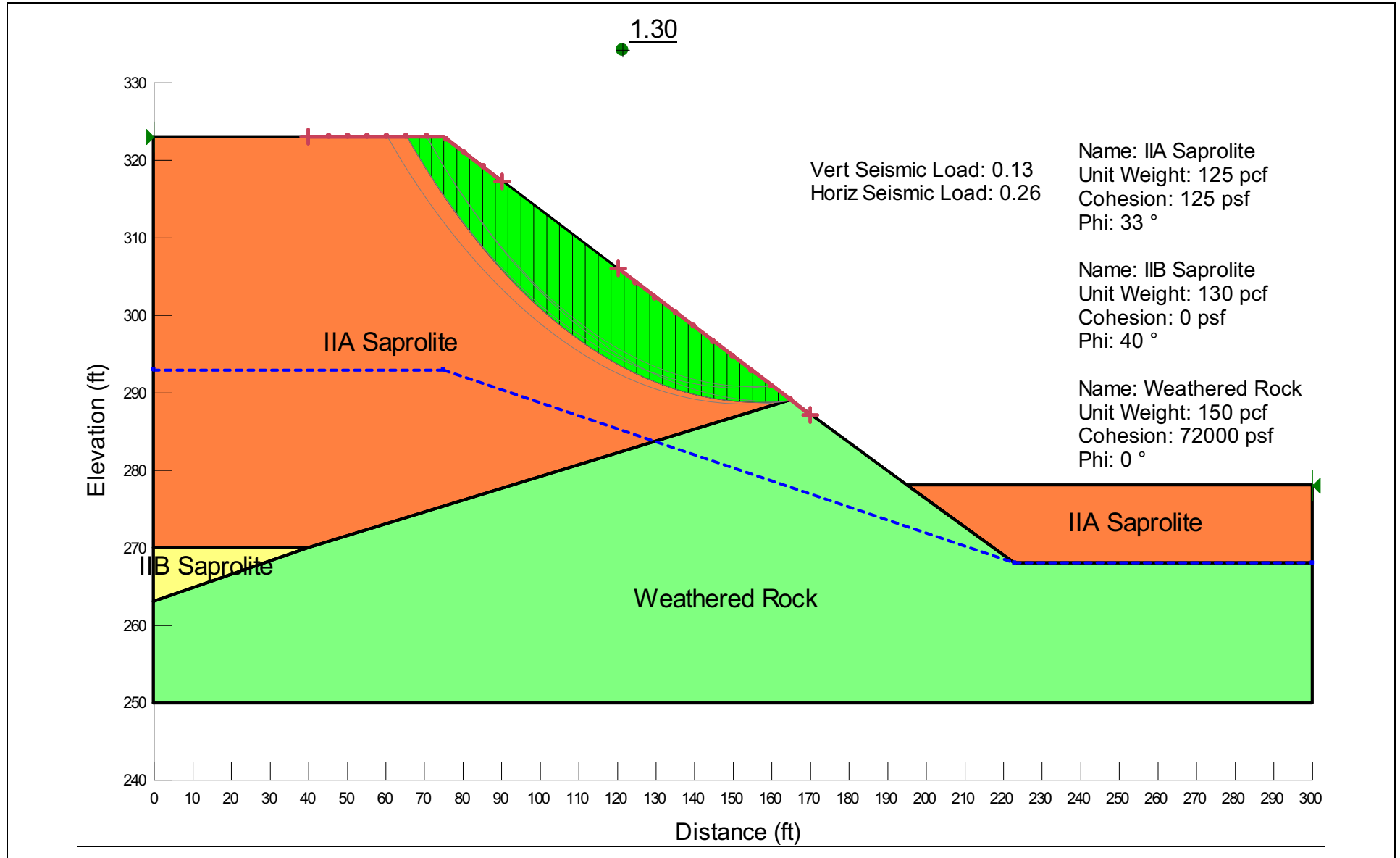


NAPS ESP COL 2.5-11 Figure 2.5.5-203 Cross-Section of Slope DD

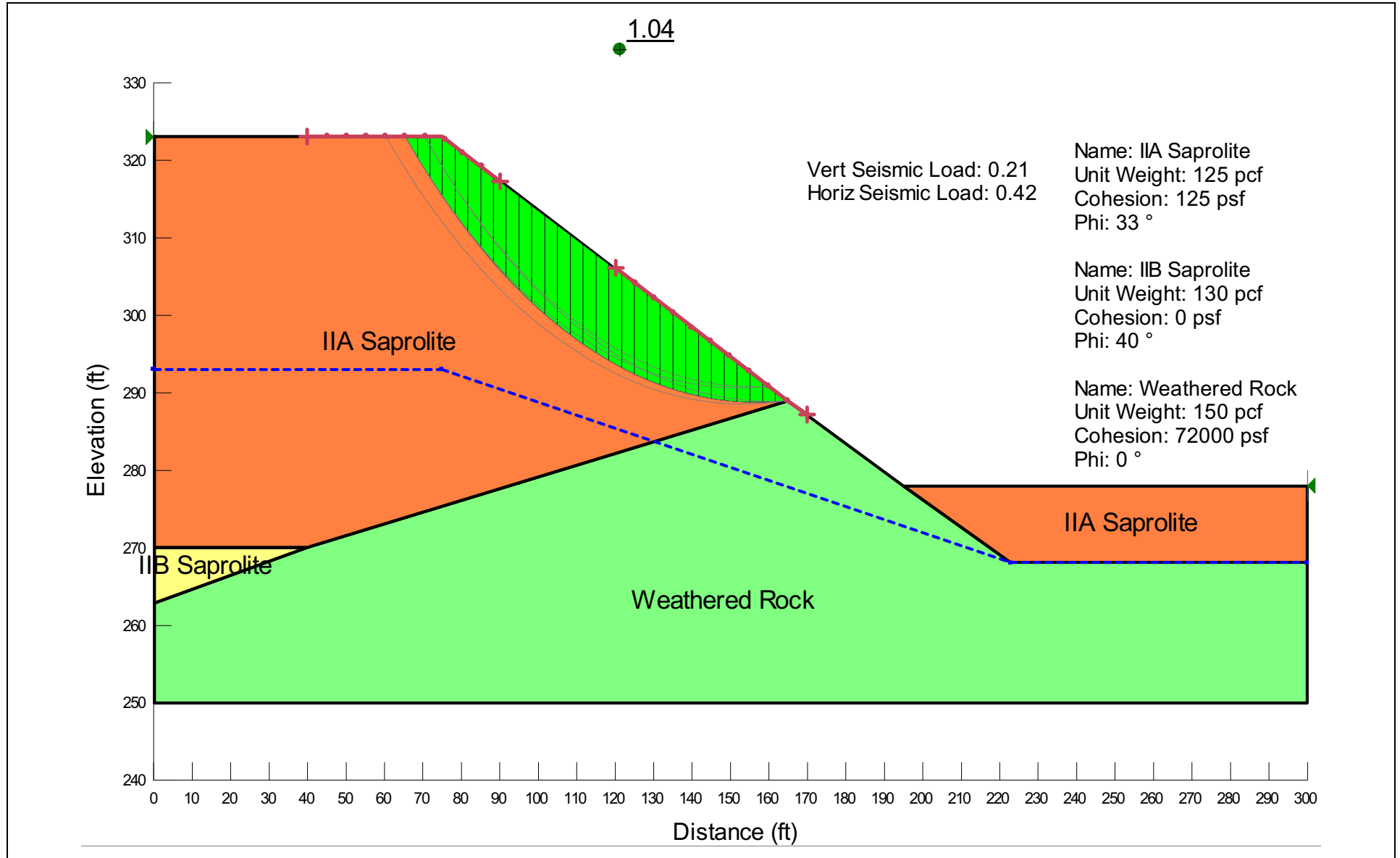




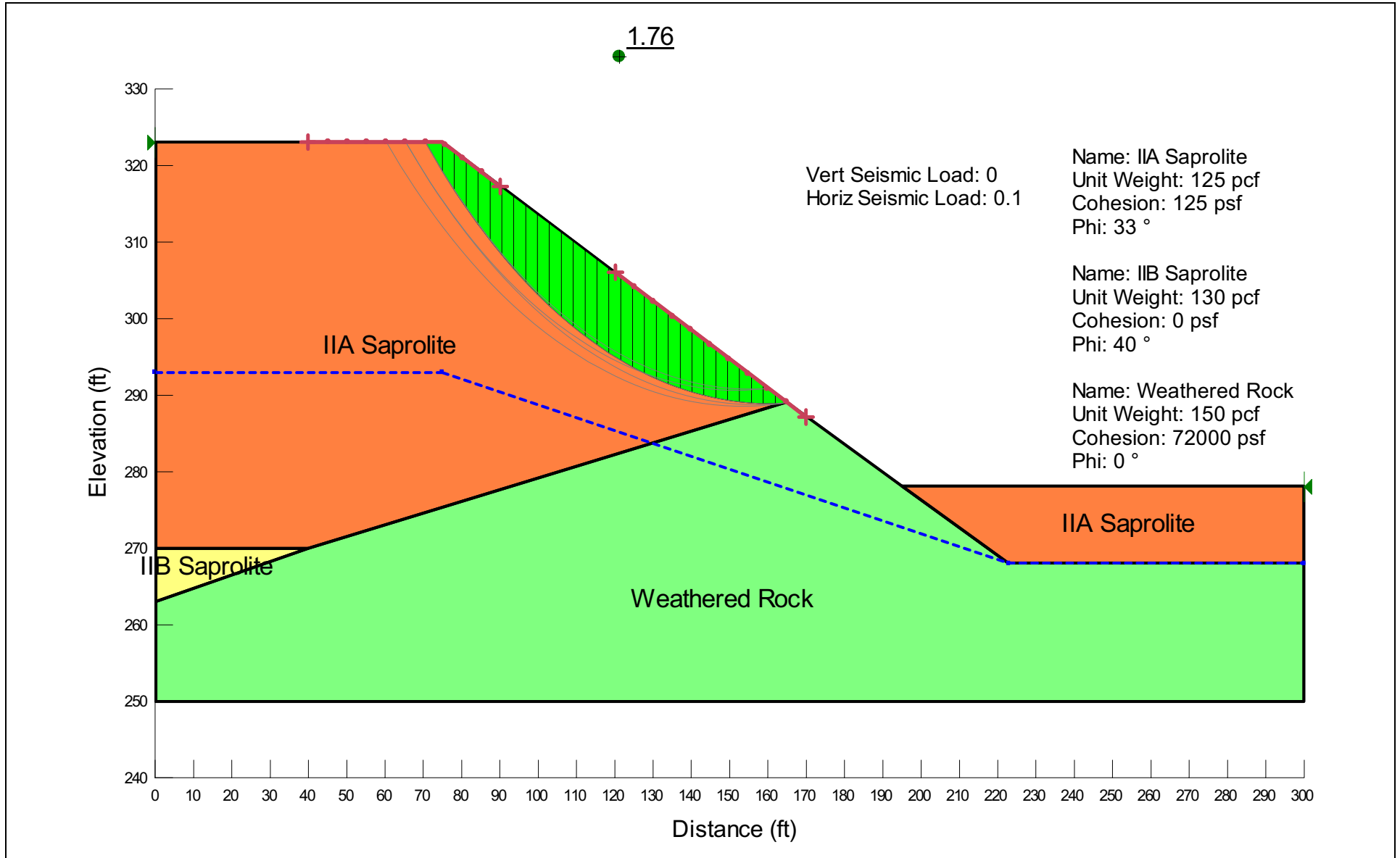
NAPS ESP COL 2.5-10 Figure 2.5.5-205 Slope Stability Analysis; Existing Slope; Pseudo-Static; LF



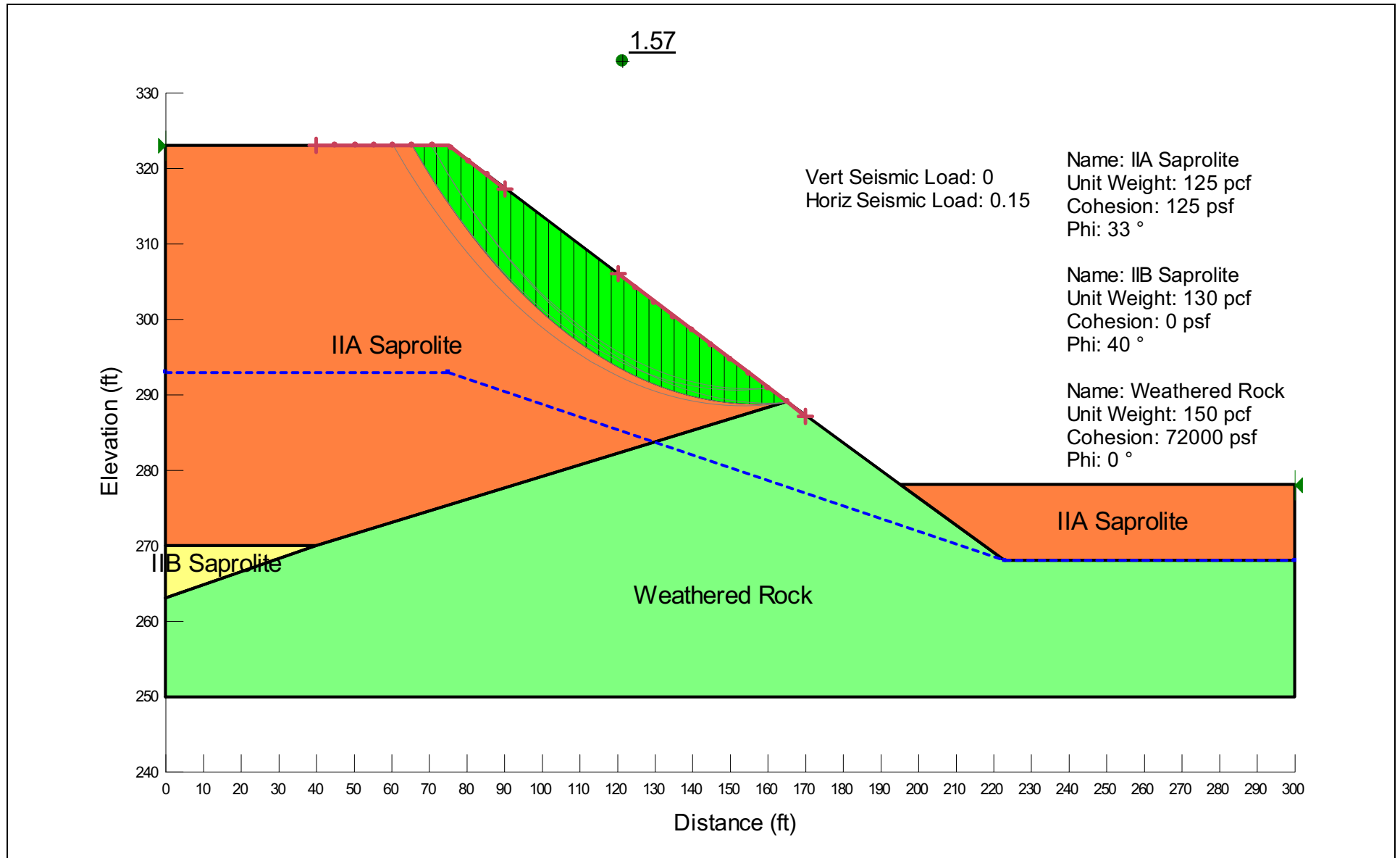
NAPS ESP COL 2.5-10 Figure 2.5.5-206 Slope Stability Analysis; Existing Slope; Pseudo-Static; HF



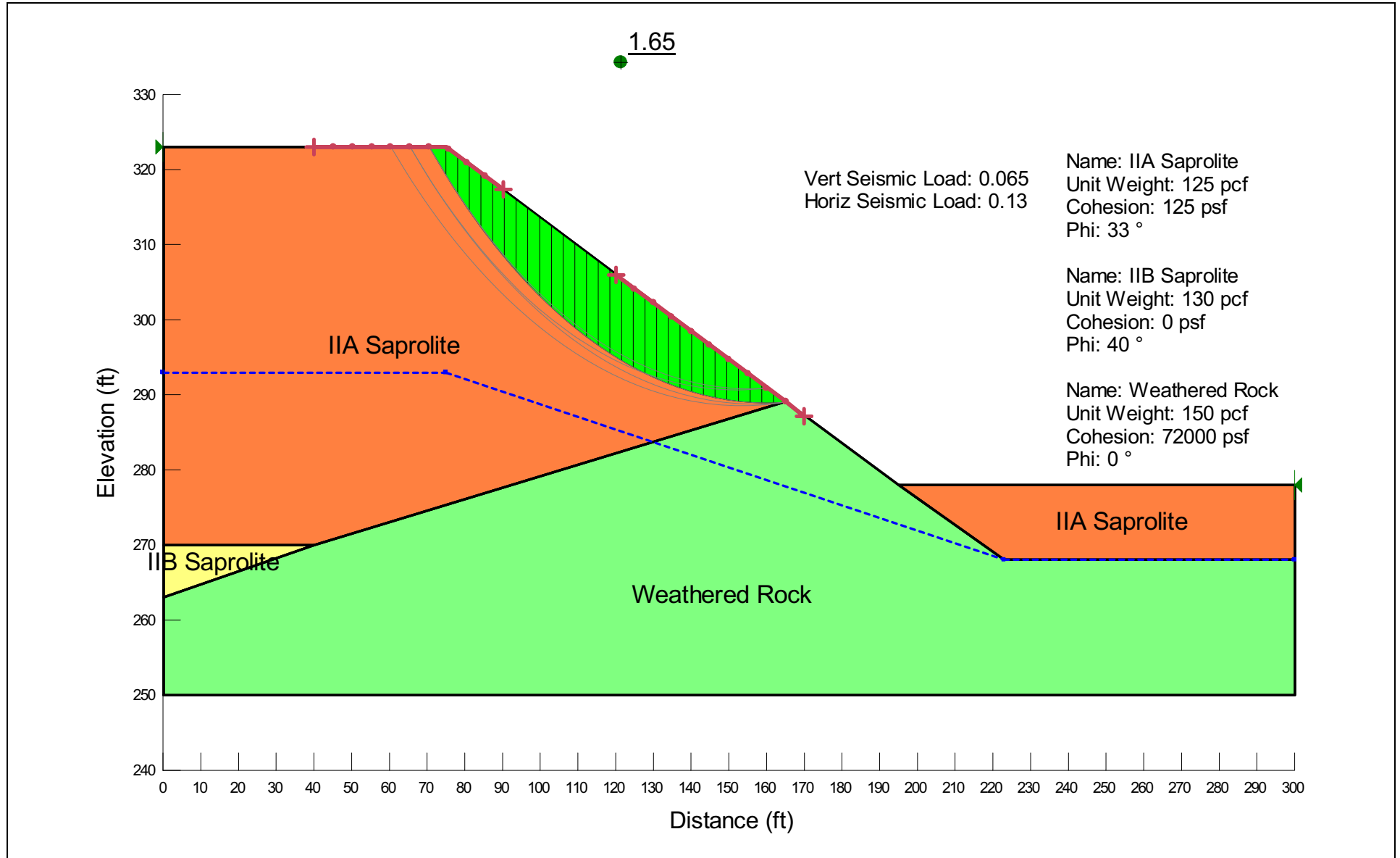
NAPS ESP COL 2.5-10 Figure 2.5.5-207 Slope Stability Analysis; Existing Slope; Seed Approach; Acceleration of 0.1g



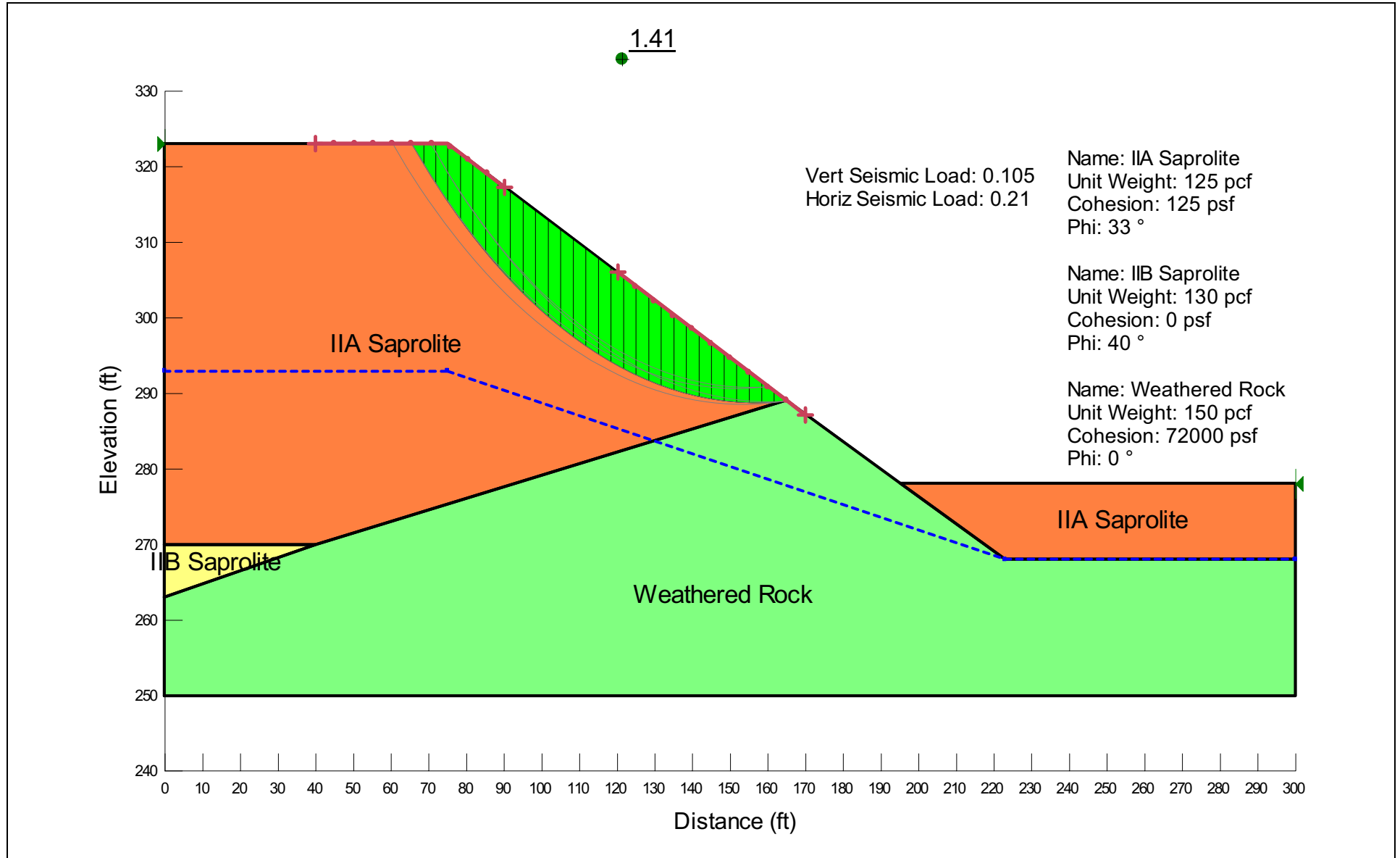
NAPS ESP COL 2.5-10 Figure 2.5.5-208 Slope Stability Analysis; Existing Slope; Seed Approach; Acceleration of 0.15g

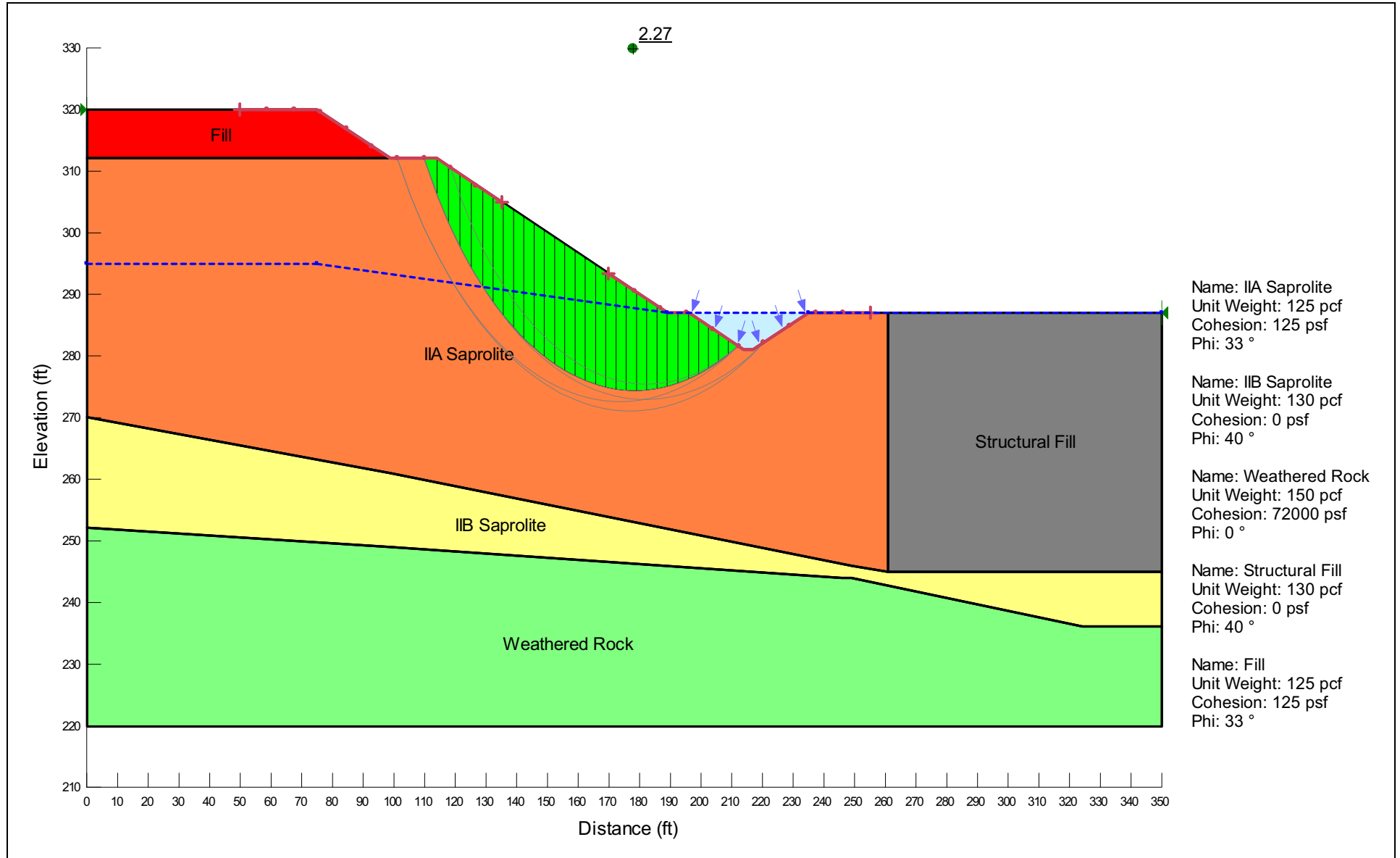


NAPS ESP COL 2.5-10 Figure 2.5.5-209 Slope Stability Analysis; Existing Slope; Kramer Approach; LF

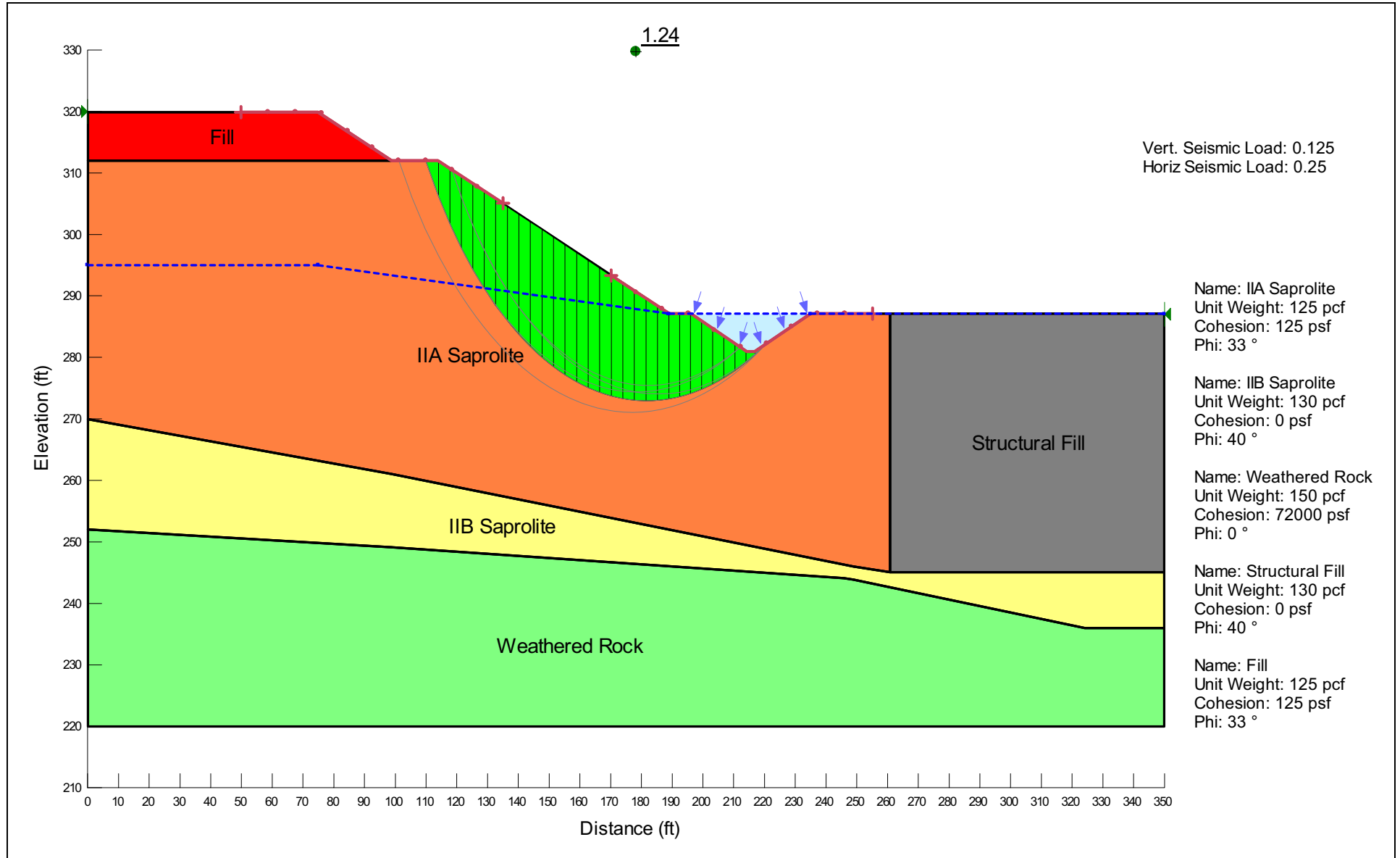


NAPS ESP COL 2.5-10 Figure 2.5.5-210 Slope Stability Analysis; Existing Slope; Kramer Approach; HF

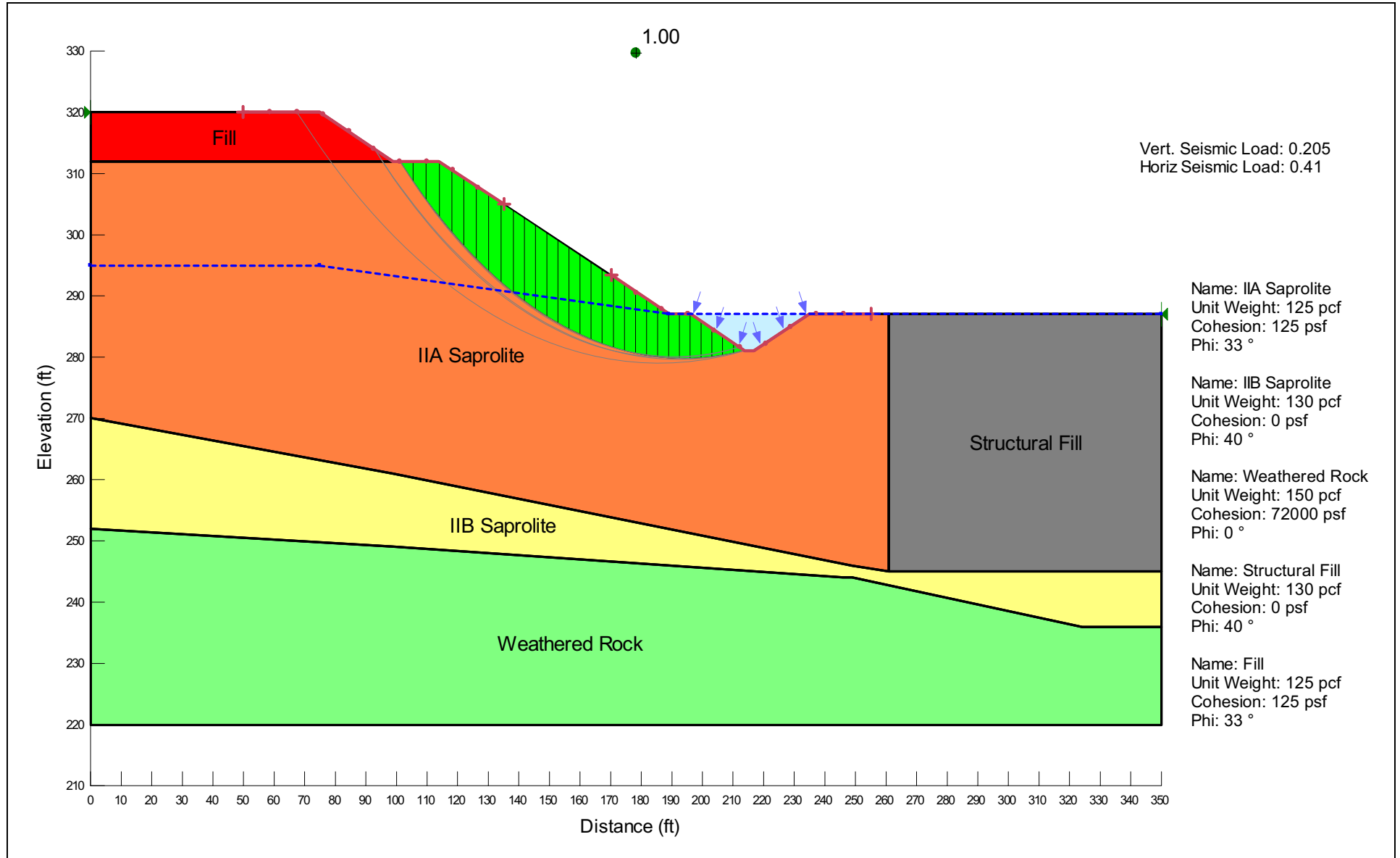




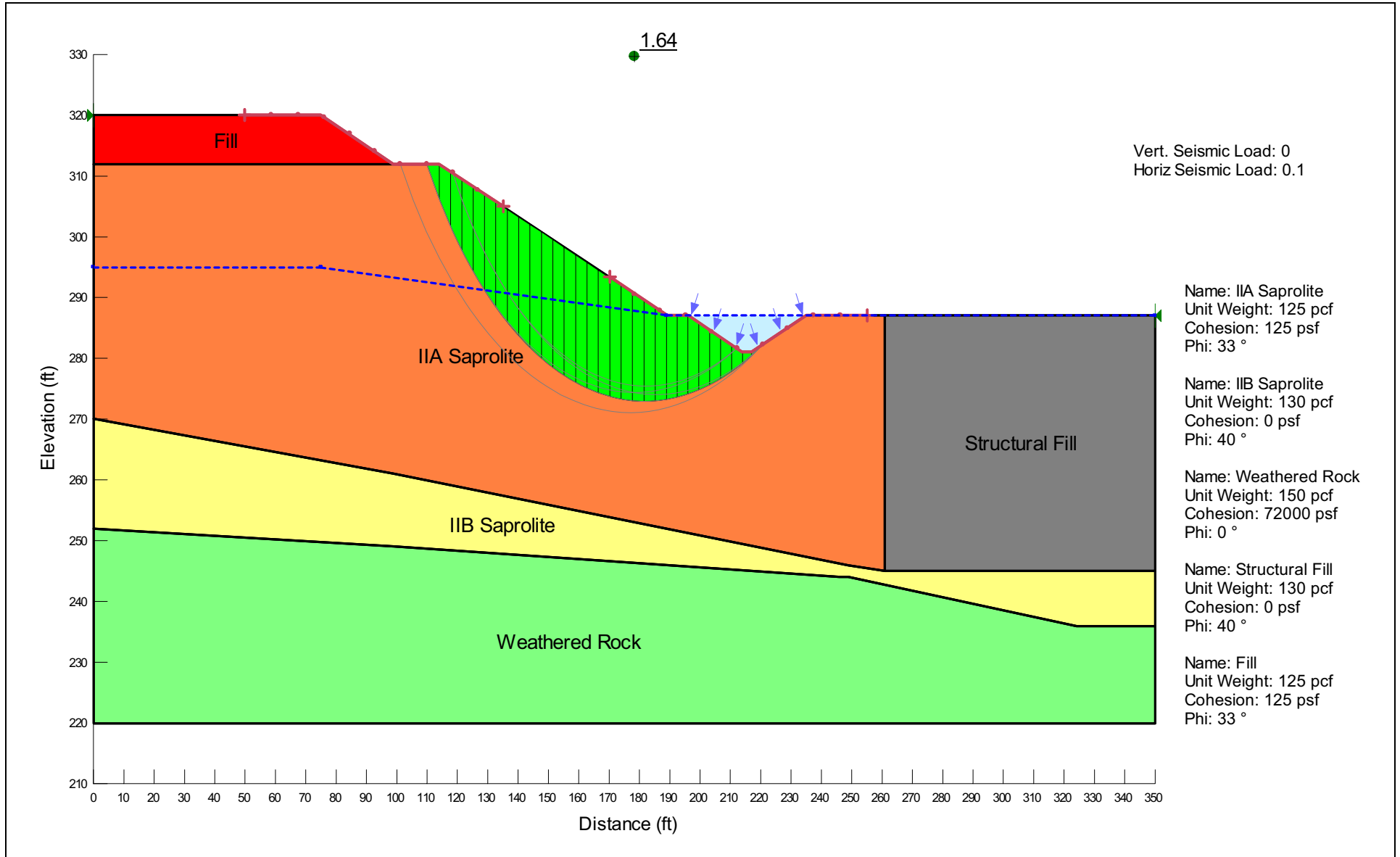
NAPS ESP COL 2.5-10 Figure 2.5.5-212 Slope Stability Analysis; Slope DD; Pseudo-Static; LF



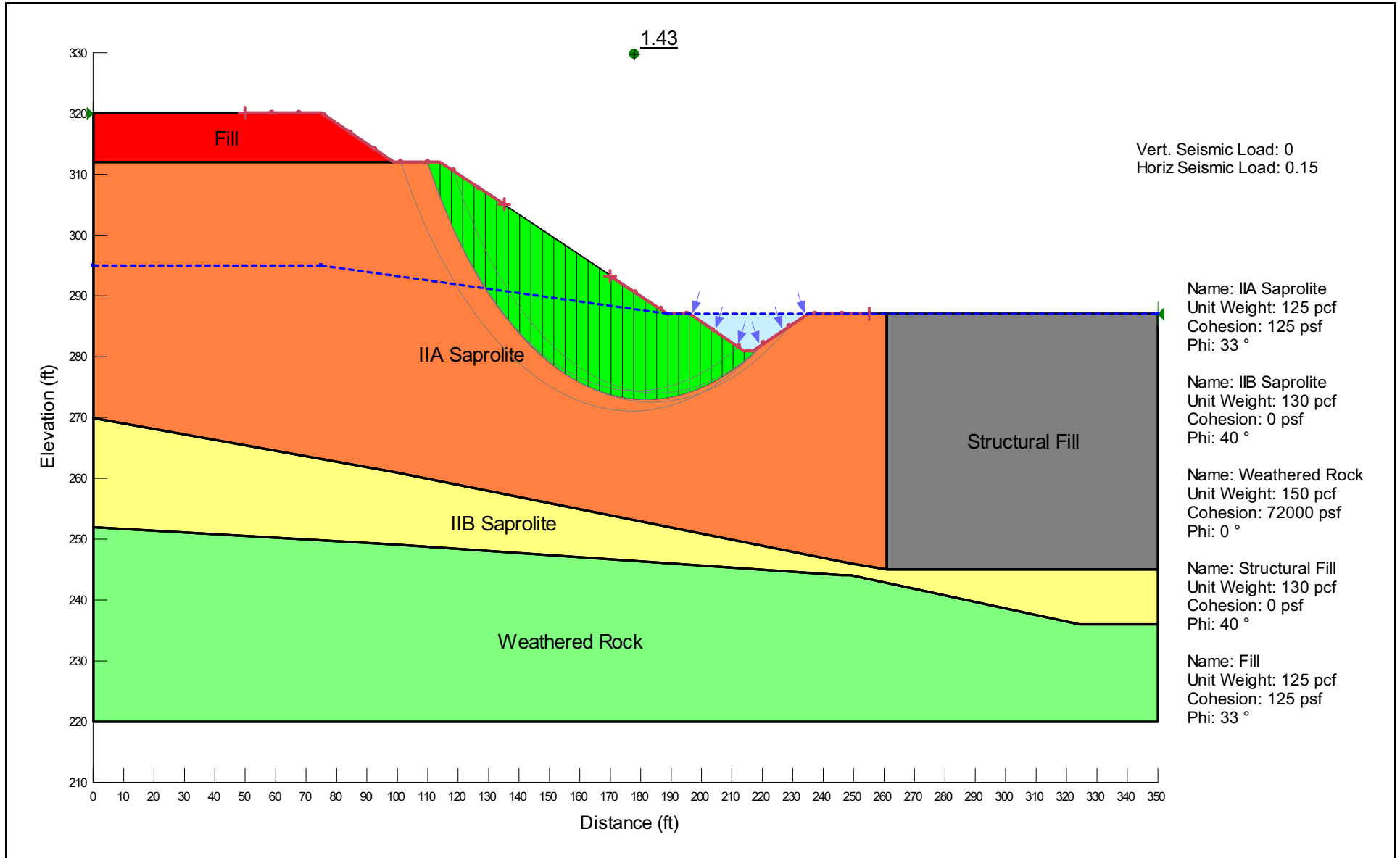
NAPS ESP COL 2.5-10 Figure 2.5.5-213 Slope Stability Analysis; Slope DD; Pseudo-Static; HF



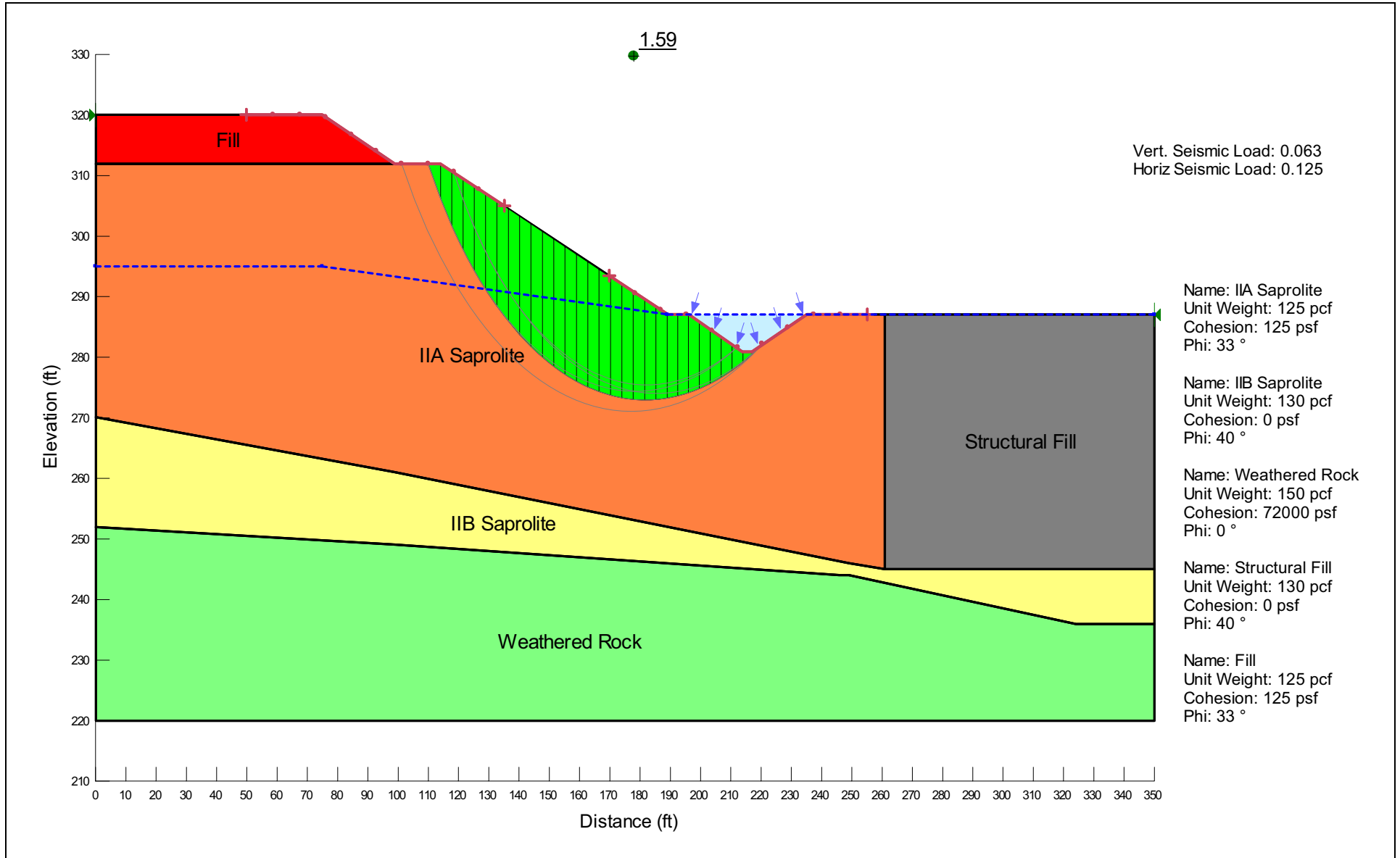
NAPS ESP COL 2.5-10 Figure 2.5.5-214 Slope Stability Analysis; Slope DD; Seed Approach; Acceleration of 0.1g



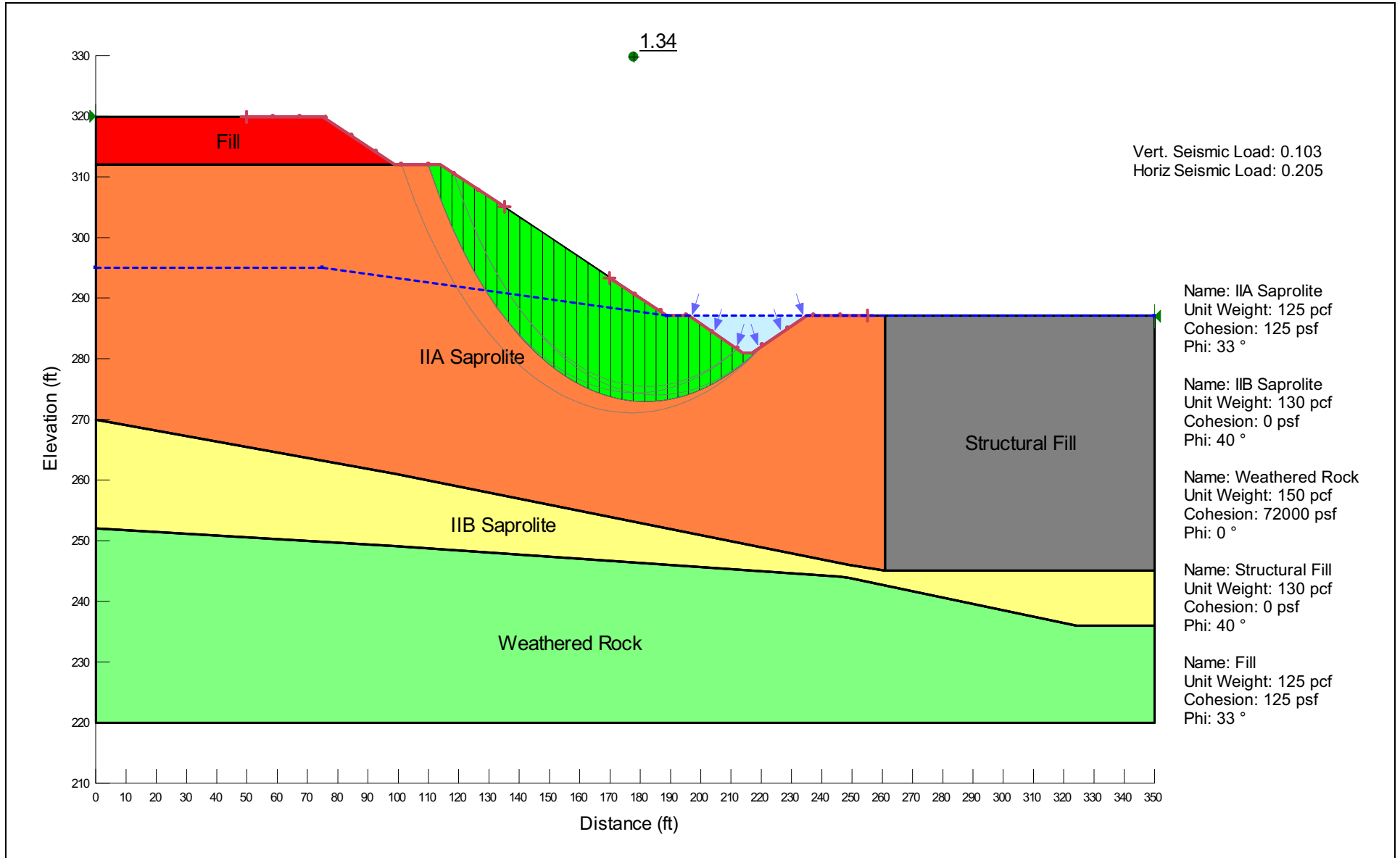
NAPS ESP COL 2.5-10 Figure 2.5.5-215 Slope Stability Analysis; Slope DD; Seed Approach; Acceleration of 0.15g



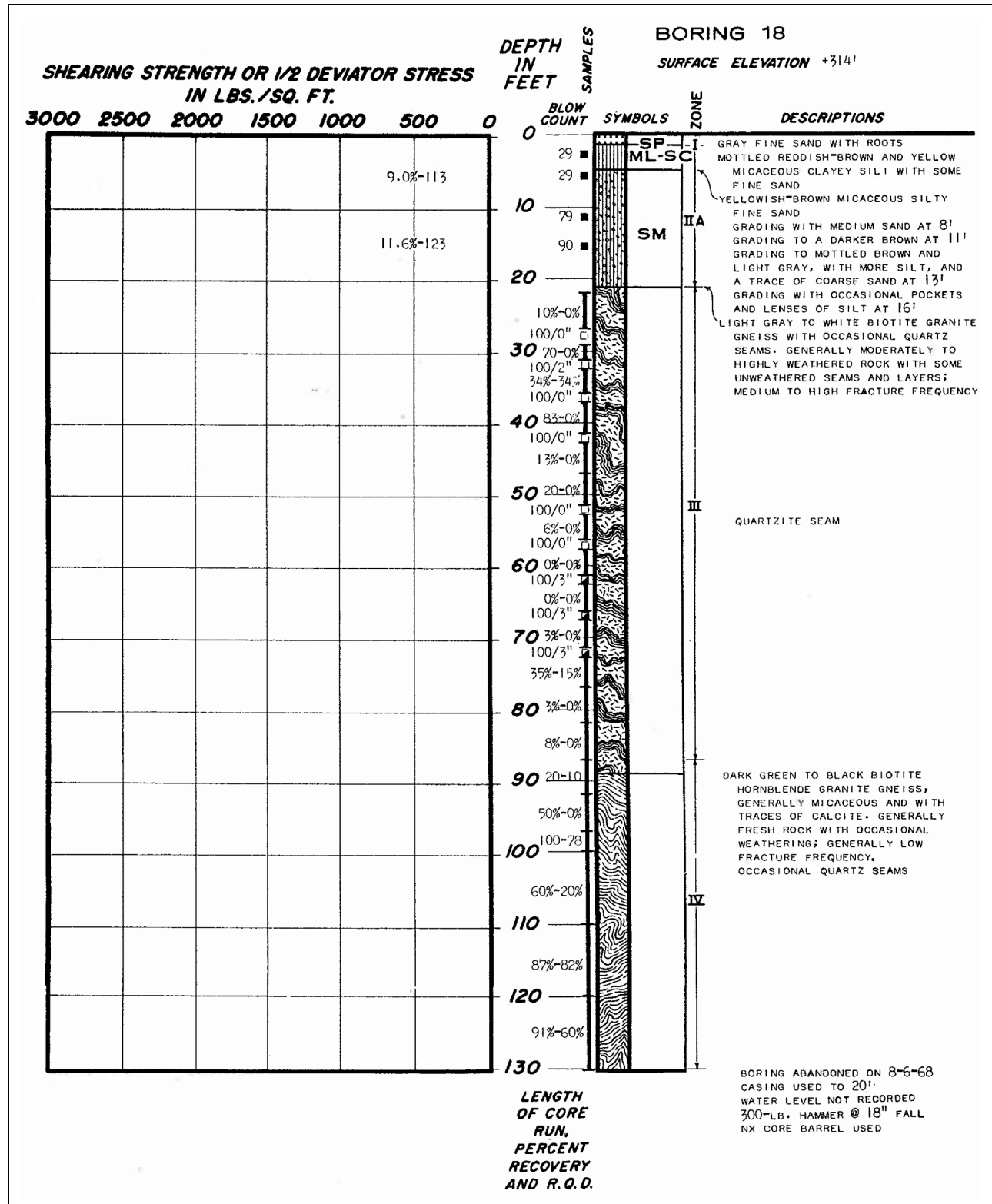
NAPS ESP COL 2.5-10 Figure 2.5.5-216 Slope Stability Analysis; Slope DD; Kramer Approach; LF

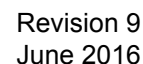


NAPS ESP COL 2.5-10 Figure 2.5.5-217 Slope Stability Analysis; Slope DD; Kramer Approach; HF



NAPS COL 2.0-30-A Figure 2.5.5-218 Log of Boring B-18





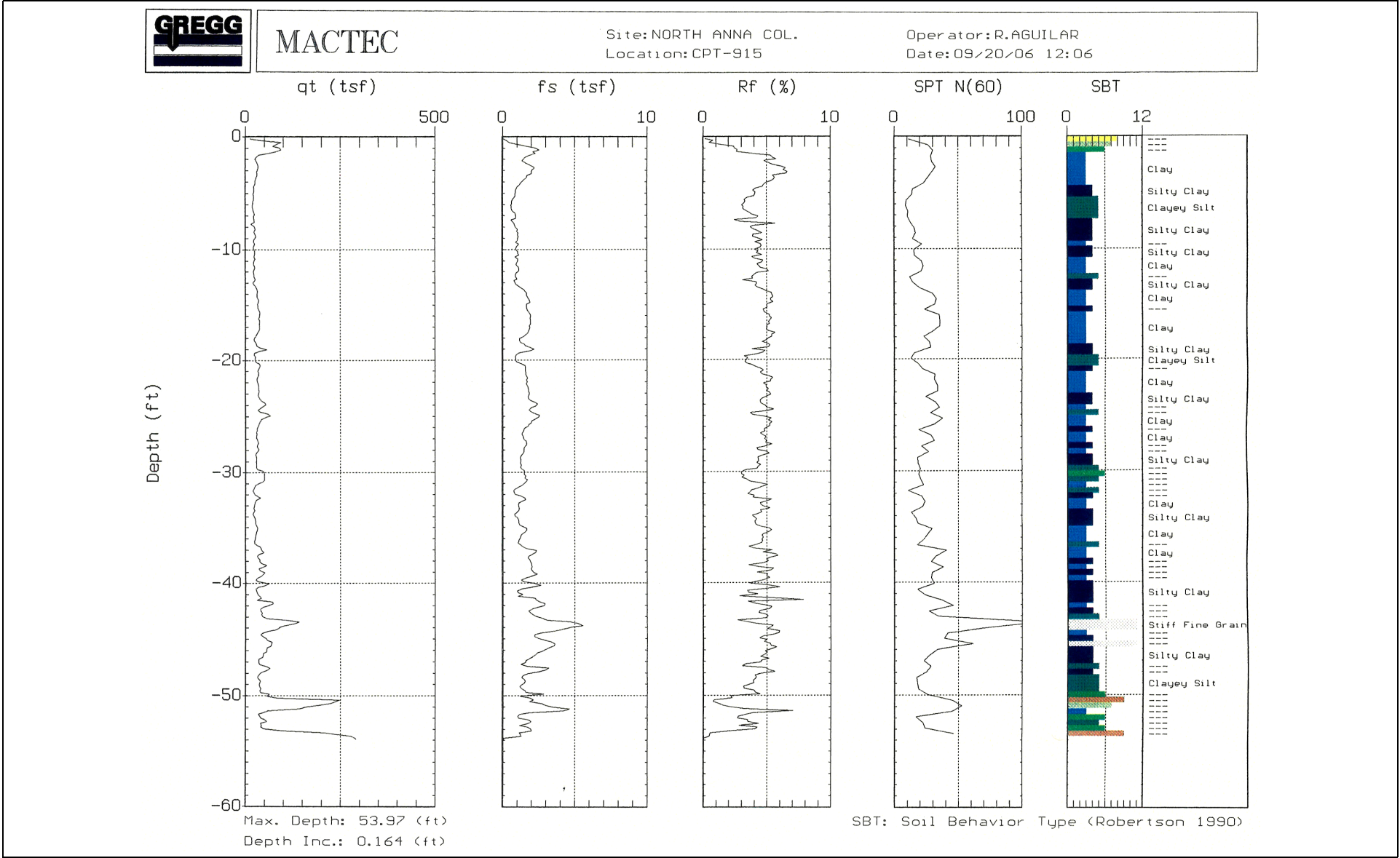
NAPS COL 2.0-30-A Figure 2.5.5-219 Log of Boring B-947 (continued)

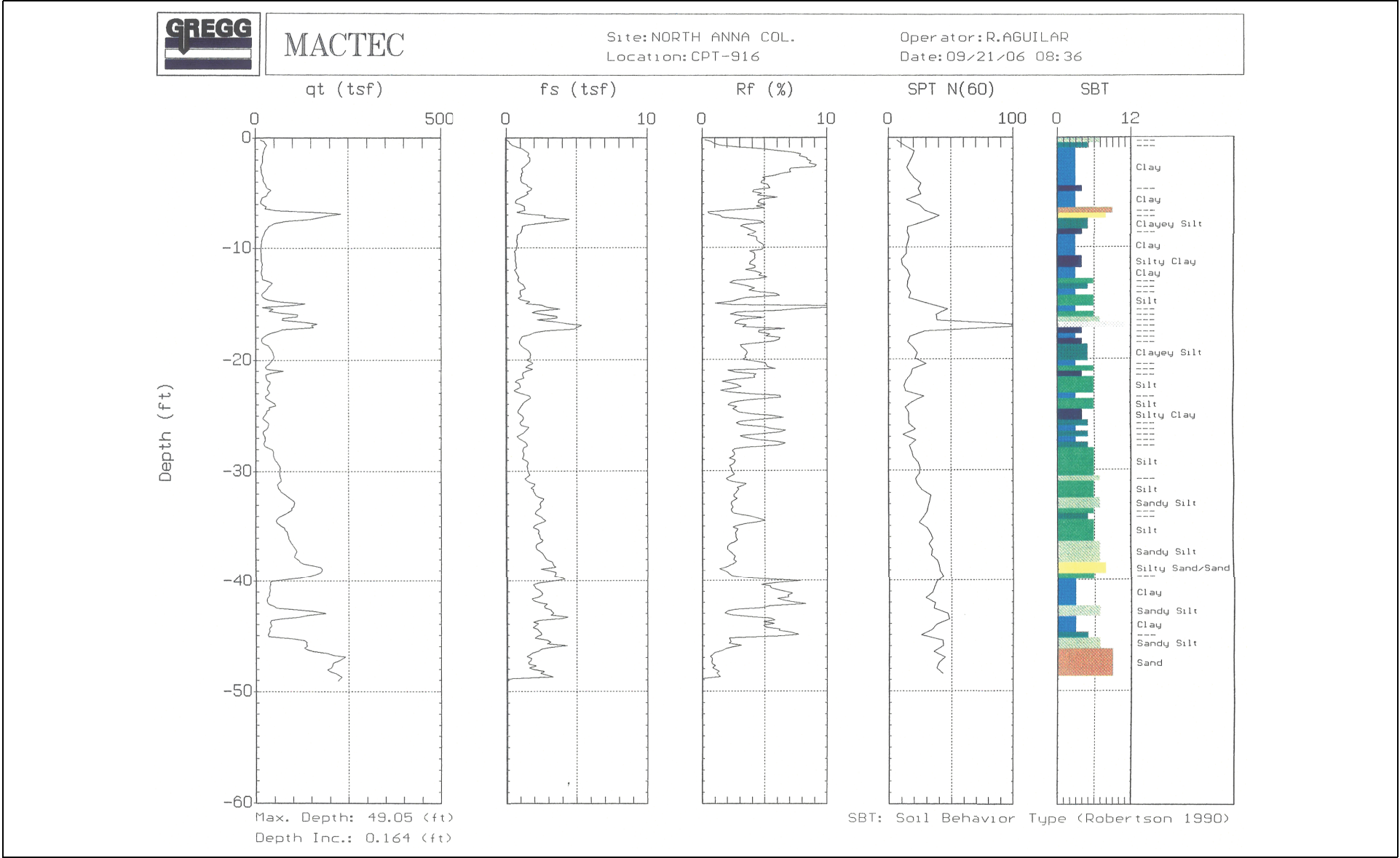
MACTEC PROJECT NO.: 6468-06-1472					COUNTY Louisa, VA					GEOLOGIST S.Lehman					
SITE DESCRIPTION NORTH ANNA COL										GROUND WATER (ft)					
BORING NO. B-947		DRILL METHOD: Mud Rotary					SAMPLE METHODS: SPT					0 HR. ND			
COLLAR ELEV. 312.5 ft (NAVD88)		NORTHING 3,909,575		US ft (NAD83)		EASTING 11,686,367		US ft (NAD83)		24 HR. 19.5					
TOTAL DEPTH 88.8 ft		DRILL MACHINE CME 55LC Track					DRILLER: D. White					HAMMER TYPE 140 lbs Auto			
DATE STARTED 9/27/06			COMPLETED 9/28/06			CORE BARREL TYPE: NA									
ELEV. (ft)	DEPTH (ft)	BLOW COUNT			BLOWS PER FOOT						SAMP. NO.	LOG	SOIL AND ROCK DESCRIPTION		
		0.5ft	0.5ft	0.5ft	0	20	40	60	80	100	MOI				
237.7					Continued from previous page										
233.8	78.7	50/0.2									947-19		Weathered Rock: Light yellowish brown (2.5Y 6/4) and light brownish gray (2.5Y 6/2), moist, very dense, severely weathered, soft, QUARTZ GNEISS (continued)		
228.8	83.7	50/0.1									947-20				
223.8	88.7	50/0.1									947-21				
													223.7	88.8	Boring terminated at 88.8 ft in Weathered Rock: Very dense, severely weathered, soft, QUARTZ GNEISS

NORTH ANNA COL. CORE NORTH ANNA COL DATA REPORT REV0.GPJ NORTH ANNA COL.GDT 1/19/07

NORTH ANNA COL CORE NORTH ANNA COL DATA REPORT REV0.GPJ NORTH ANNA COL GDT 1/19/07

NAPS COL 2.0-30-A Figure 2.5.5-220 Log of CPT C-915





NAPS COL 2.0-30-A Figure 2.5.5-222 Log of Well OW-947

OBSERVATION WELL INSTALLATION RECORD	
JOB NAME NORTH ANNA COL	JOB NUMBER 6468-06-1472
WELL NUMBER OW-947	INSTALLATION DATE 11-06-06
LOCATION (NAD83) N = 3,909,579.58 E = 11,686,371.84	
GROUND SURFACE ELEVATION* (NAVD88) 313.30	REFERENCE POINT ELEVATION** (NAVD88) 315.08
GRANULAR BACKFILL MATERIAL Southern Silica #1 & #3 Sand*	SLOT SIZE .010
SCREEN MATERIAL PVC Schd. 40-Standard	SCREEN DIAMETER 2 in.
RISER MATERIAL PVC Schd. 40-Standard	RISER DIAMETER 2 in.
DRILLING TECHNIQUE Hollow-stem auger 4.25" I.D.	DRILLING CONTRACTOR MACTEC
BOREHOLE DIAMETER Approximately 8"	MACTEC FIELD REPRESENTATIVE Kim Charles-Smith
LOCK BRAND Master	SIZE/MODEL N/A
KEY CODE/COMBINATION #3206	
* Both #1 and #3 sand met the technical specifications for use as granular backfill material. MACTEC used #3 sand to backfill the sump portion of observation well OW-947, and #1 sand for the remaining portion of the well boring.	

NOT TO SCALE

STICKUP **2.0'**

GROUND SURFACE

LENGTH OF SOLID SECTION **45'**

DEPTH TO TOP OF BENTONITE SEAL **36'**

DEPTH TO TOP OF GRANULAR MATERIAL **41'**

TOTAL DEPTH OF WELL **58'**

STABILIZED WATER LEVEL AFTER DEVELOPMENT **18.24** FEET BELOW TOP OF INNER CASING MEASURED ON **11-09-06**

LENGTH OF SLOTTED SECTION **10' (45-55')**

LENGTH OF TAIL PIPE **3' (55-58')**

* Reference point is a marked location on the concrete well pad
** Reference point is the top of PVC casing

NORTH ANNA POWER STATION MINERAL, VIRGINIA COL PROJECT Dominion Purchase Order 7015798	 MACTEC MACTEC Engineering and Consulting, Inc. 3301 Atlantic Avenue Raleigh, North Carolina 27604	OBSERVATION WELL INSTALLATION RECORD <i>002 11-08-07</i>
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2.5.6 Embankments and Dams

SSAR Section 2.5.6 is incorporated by reference with the following supplement.

This SSAR section is supplemented as follows with a new paragraph on Unit 3 embankments and dams.

NAPS COL 2.0-30-A

Because Lake Anna is only used as a source of makeup water for Unit 3, the North Anna Dam, which is designed and constructed to meet requirements for a Seismic Category I structure in support of the existing Units 1 and 2, was not re-analyzed as part of this FSAR. Construction of Unit 3 does not adversely affect the slopes of the SWR for Units 1 and 2. There is an existing slope to the north of the SWR and a new slope to the east of the FWSC. These slopes are described and analyzed in Section 2.5.5.

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Appendix 2A ARCON96 Source/Receptor Inputs

This section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

2A.2.1 Meteorological Data

Add the following as the last sentence of this section.

NAPS COL 2A.2-1-A

Instrumentation heights used in the analysis are described in [SSAR Section 2.3.3.1.3](#). Meteorological data from 1996 to 1998 as described in [SSAR Section 2.3](#) is used in the analysis.

2A.2.3 ARCON96 ESBWR Inputs

Replace the last sentence of the first paragraph with the following.

NAPS COL 2A.2-1-A

These directions are adjusted by the difference in angle (approximately 23.5 degrees east of true North); Unit 3 receptor to source directions are shown in [Table 2A-4R](#).

2A.2.4 Confirmation of the ESBWR χ/Q Values

Replace this section with the following.

NAPS COL 2A.2-1-A

[DCD Figure 2A-1](#) shows the locations of the sources and receptors for ESBWR control room determinations, also used in the Unit 3 evaluations. The dimensions of the diffuse source planes provided in [DCD Table 2A-3](#) are determined as directed by RG 1.194, Regulatory Position 3.2.4.5, for the nearest receptor locations. ARCON96 calculations are performed for source/receptor pairs compared in [Table 2.0-201](#). The calculations used design inputs listed in [DCD Table 2A-3](#), directions in [Table 2A-4R](#), and site-specific meteorological data. Results of the site-specific analysis are provided in [Tables 2.3-201 through 2.3-204](#), and [Table 2.3-206](#).

2A.2.5 Confirmation of the Reactor Building χ/Q Values

Replace this section with the following.

NAPS COL 2A.2-2-A

During movement of irradiated fuel, those doors and personnel air locks on the plant east sides of the Reactor Building or Fuel Building that could act as a point source could result in control room χ/Q values that are higher than the ESBWR χ/Q values for a release in the Reactor Building or Fuel Building ([DCD Table 15.4-4](#)). Therefore, those doors and

personnel air locks on the plant east sides of the Reactor Building or Fuel Building that could act as a point source are administratively controlled to remain closed during movement of irradiated fuel.

2A.3 COL Information

2A.2-1-A Confirmation of the ESBWR χ/Q Values

NAPS COL 2A.2-1-A

This COL item is addressed in [Section 2.3.4.3](#) and in [Section 2A.2.4](#).

2A.2-2-A Confirmation of the Reactor Building χ/Q Values

NAPS COL 2A.2-2-A

This COL item is addressed in [Section 2A.2.5](#).

NAPS ESP COL 2.3-2
NAPS COL 2A.2-1-A

Table 2A-4R ARCON96 Direction Design Inputs Used for the Determination of On-Site χ/Q Values

Source/Receptor	Direction (degrees from true North)
RB to CBL	294
RB to EN	284
RB to ES	304
RB to N	308
RB to TSCB	236
RB to TSCA	240
PCCS to CBL	333
PCCS to EN	309
PCCS to ES	328
PCCS to N	332
PCCS to TSCB	238
PCCS to TSCA	241
TB to CBL	7
TB to EN	348
TB to ES	355
TB to N	360
TB to TSCB	256
TB to TSCA	260
TB-TD to CBL	5
TB-TD to EN	355
TB-TD to TSCB	301
FB to CBL	252
FB to EN	258
FB to ES	272
FB to N	276
RW to N	328
RB-VS to CBL	271
RB-VS to ES	285
RB-VS to N	286
TB-VS to CBL	20

NAPS ESP COL 2.3-2
NAPS COL 2A.2-1-A

**Table 2A-4R ARCON96 Direction Design Inputs Used for the
Determination of On-Site χ/Q Values (continued)**

Source/Receptor	Direction (degrees from true North)
TB-VS to EN	5
TB-VS to N	12
RW-VS to CBL	326
RW-VS to EN	314
RW-VS to N	328
BPN to CBL	346
BPN to EN	309
BPN to ES	330
BPN to N	339
BPS to CBL	243
BPS to EN	253
BPS to ES	279
BPS to N	283

Appendix 2B Ventilation Stack Pathway Information for Long-Term χ/Q Values

This section of the referenced DCD is incorporated by reference with no departures or supplements.