2.5.4 Stability and Uniformity of Subsurface Materials and Foundations

This section presents information on the stability of subsurface materials and foundations at the site of VCSNS Units 2 and 3. The information has been developed in accordance with NUREG-0800, “Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants,” Subsection 2.5.4 (Reference 235), following the guidance presented in Regulatory Guide 1.206, Subsection 2.5.4, and the regulatory guides identified in the subsections that follow. Information presented in this section was developed from the results of a subsurface investigation program implemented at the Units 2 and 3 site. The data are contained in Reference 232. The geological, geophysical, and geotechnical information obtained is used as a basis to evaluate the stability of subsurface materials and foundations at the site.

2.5.4.1 Geologic Features

subsection 2.5.1.1 addresses the regional geology, including regional physiography and geomorphology, regional geologic history, regional stratigraphy, regional tectonic and non-tectonic conditions, and geologic hazards, as well as maps, cross sections, and references. subsection 2.5.1.2 describes the 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.

The Units 2 and 3 site is located within the Piedmont physiographic province of central South Carolina, bounded on the southeast and northwest by the Coastal Plain and Blue Ridge physiographic provinces, respectively. The site topography is characteristic of the region, consisting of gently to moderately rolling hills and generally well-drained mature valleys. Within a 5-mile radius of the site, ground surface elevations range from about elevation (El.) 220 to 520 feet. (All elevations in this section are with respect to NAVD88.) Steep gullies, resulting from differential weathering of the rock, exist within the site area.

The geologic profile consists of residuum and saprolitic soils underlain by partially and moderately weathered rock, grading downward into sound rock. The combined thickness of residual soil and saprolite ranges from about 25 feet to 70 feet at the Units 2 and 3 site. Granodiorite and quartz diorite are the most commonly encountered rocks in the site area. Amphibolite-grade metaigneous and metasedimentary rocks of the Carolina Zone encountered within the site area include biotite and hornblende gneiss and amphibolite schist. Migmatites are the least commonly encountered of the principal rock types found at the site area based on field reconnaissance data, geologic mapping, and core from foundation borings.

2.5.4.2 Properties of Subsurface Materials

Soil structure interaction and foundation design are a function of the uniformity of the soil or rock below foundation. Although the design and analysis of the AP1000 is based on soil or rock conditions with uniform properties within horizontal layers, it includes provisions and design margins to accommodate many non-uniform sites. This subsection identifies the requirements for site investigation that may be used to demonstrate that:

- A site is “uniform” based on the criteria outlined in subsection 2.5.4.5.3. If the site can be demonstrated to be “uniform,” no further site specific analysis is required to qualify the site for the AP1000.
- A “non-uniform” site is acceptable to locate the AP1000 based on the criteria for acceptability outlined in subsection 2.5.4.5.3. Non-uniform sites may be shown to be acceptable as described in subsection 2.5.4.5.3.1 using site-specific evaluation as part of the Combined License application.

Considerations with respect to the materials underlying the nuclear island are the type of site, such as rock or soil, and whether the site can be considered uniform. If the site is non-uniform, the non-uniform soil characteristics, such as the location and profiles of soft and hard spots, should be considered. These considerations can be assessed with the information developed in response to Regulatory Guides 1.132 and 1.138. The geological investigations of Subsections 2.5.1 and 2.5.4.6.1 provide information on the uniformity of the site, whether it may be geologically impacted, and whether the bedrock may be sloping or undulatory.

Site Foundation Material Evaluation Criteria

The AP1000 is designed for application at a site where the foundation conditions do not have extreme variation within the nuclear island footprint. This subsection provides criteria for evaluation of soil variability. The subsurface may consist of layers and these layers may dip with respect to the horizontal. If the dip is less than 20 degrees, the generic analysis using horizontal layers is applicable as described in NUREG/CR-0693 (Reference 2). The physical properties of the foundation medium may or may not vary systematically across a horizontal plane. The recommended methodology for checking uniformity is to calculate from the boring logs a series of “best-estimate” planes beneath the nuclear island footprint that define the top (and bottom) of each layer. The planes could represent stratigraphic boundaries, lithologic changes, and unconformities, but most important, they should represent boundaries between layers having different shear wave velocities. Shear wave velocity is the primary property used for defining uniformity of a site.

The distribution of bearing reactions under the basemat is a function of the subgrade modulus, which in turn is a function of the soil properties. The Combined License applicant shall demonstrate that the variation of subgrade modulus across the footprint is within the range considered for design of the nuclear island basemat. The farther that the non-uniform layer is located below the foundation, the less influence it has on the bearing pressures at the basemat. Lateral variability of the shear wave velocity at depths greater than 120 feet below grade (80 feet below the foundation) do not significantly affect the subgrade modulus.

Subsurface conditions should be evaluated by the Combined License applicant based on the geologic investigation in accordance with Regulatory Guide 1.132. Subsurface conditions should be evaluated within the nuclear island footprint and 40 feet5 beyond the boundaries of the nuclear island footprint at depths less than 120 feet below grade. Subsurface conditions may be considered uniform if the geologic and stratigraphic features can be correlated from one boring or sounding location to the next with relatively smooth variations in thicknesses or properties of the geologic units. An occasional anomaly or a limited number of unexpected lateral variations may occur. If a site can be classified as uniform, it qualifies for the AP1000 based on analyses and evaluations performed to support design certification without additional site-specific analyses.

As an example of sites that are considered uniform, the variation of soil properties in the material below the foundation to a depth of 120 feet below finished grade within the nuclear island footprint and 40 feet beyond the boundaries of the nuclear island footprint meets the criteria outlined below:

- The depth to a given layer indicated on each boring log may not fall precisely on the postulated “best-estimate” plane. The deviation of the observed layers from the “best-estimate” planes should not exceed 5 percent of the observed depths from the ground surface to the plane. If the deviation is greater than 5 percent, additional planes may be appropriate or additional borings may be required. This thereby diminishes the spacing.

- For a layer with a low strain shear wave velocity greater than or equal to 2500 feet per second, the layer should have approximately uniform thickness and should have a dip no greater than 20 degrees, and the shear wave velocity at any location within any layer should not vary from the average velocity within the layer by more than 20 percent.

Site-Specific Subsurface Uniformity Design Basis

Many sites that do not meet the above criteria for a uniform site are acceptable for the AP1000. The key attribute for acceptability of the site for an AP1000 is the bearing pressure on the underside of the basemat. A site having local soft or hard spots within a layer or layers does not meet the criteria for a uniform site. Non-uniform soil conditions may also require evaluation of the AP1000 seismic response as described in subsection 2.5.2.3.

As described in subsection 3.8.5, the nuclear island foundation is designed specifically for bearing pressures of 120 percent of those of the uniform soil properties case. Evaluation criteria are defined to evaluate sites that do not satisfy the site parameters directly. The design basis provided below is included to provide a clear specification of the design commitment and evaluation criteria required to demonstrate that a site-specific application satisfies AP1000 requirements. Application of the AP1000 to sites using this site-specific evaluation is not approved as part of the AP1000 design certification and the evaluation should be provided and reviewed as part of the Combined License application.

Rigid Basemat Evaluation

A site with nonuniform soil properties may be demonstrated to be acceptable by evaluation of the bearing pressures on the underside of a rigid rectangular basemat equivalent to the nuclear island. The soils identified in the site investigation may be included in a finite element model of the soil to analyze the effect of the lateral variability. When the variability identified at the site can be modeled in two dimensions (there is not significant variability in one horizontal direction), 2D analyses may be used. Where the variability occurs in both horizontal directions, a 3D analysis should be performed. Bearing pressures are calculated in a linear analysis for unit vertical load and overturning moments. For the site to be acceptable, the bearing pressures from this analysis need to be less than or equal to 120 percent of the bearing pressures calculated in similar analyses for a site having uniform soil properties.

Alternatively, the safe shutdown earthquake loads may be determined from a site-specific seismic analysis of the nuclear island using site-specific inputs as described in Subsections 2.5.2.1 or 2.5.2.3. For the site to be acceptable, the bearing pressures from the site-specific analyses (with site-specific response and site-specific soil properties) need to be less than or equal to 120 percent of the bearing pressures calculated in similar rigid basemat analyses using the AP1000 design ground motion at a site having uniform soil properties.

Flexible Basemat Evaluation

For sites having bedrock close to the foundation level, the assumption of a rigid basemat may be overly conservative because local deformation of the basemat will reduce the effect of local soil variability. For such sites, a site-specific analysis may be performed using the AP1000 basemat model and methodology described in subsection 3.8.5. The soils may be represented by soil springs or by a finite element model of the soil depending on the type of variability identified at the site. The safe shutdown earthquake loads are those from the AP1000 design soil case representative of the site-specific soil. Alternatively, bearing pressures may be determined from a site-specific soil structure interaction analysis using site-specific inputs as described in subsection 2.5.2.3. For the site to be acceptable, the bearing pressures from the site-specific analyses, including static and dynamic loads, need to be less than the design bearing strength of each portion of the basemat.

The Unit 1 UFSAR Subsection 2.5.4.6 (Reference 249) contains geotechnical information from previous subsurface investigations and subsequent analyses, and from the excavation for Unit 1. Units 2 and 3 are located approximately 1 mile southwest of Unit 1. In general, because of the distance between Unit 1 and Units 2 and 3, and because of the comprehensive nature of the subsurface investigation for Units 2 and 3, comparisons between the Unit 1 UFSAR data and the

Units 2 and 3 geotechnical information presented here were not made, except where considered relevant.

2.5.4.2.1 Introduction

This section describes the static and dynamic engineering properties of the Units 2 and 3 site subsurface materials. An overview of the subsurface profile and materials is given in subsection 2.5.4.2.2. The field investigations are presented in subsection 2.5.4.2.3. The geophysical investigations are described in detail in subsection 2.5.4.4. Laboratory testing performed for the investigation is summarized in subsection 2.5.4.2.4. The engineering properties of the natural soil and rock and compacted fill are presented in subsection 2.5.4.2.5.

2.5.4.2.2 Description of Subsurface Materials

The subsurface profile consists of shallow residual/saprolitic soils underlain by bedrock, which continues approximately 50 feet below the existing ground surface in the power block area (PBA). The profile can be divided into five layers, with the following descriptions:

I. Residuum — silts and silty sands with variable clay content.

II. Saprolite — completely weathered rock but with preserved relict rock structure.

III. Partially weathered rock (PWR) — decomposed rock matrix mixed with semi-hard rock fragments.

IV. Moderately weathered rock (MWR) — more than 50% by volume of sound rock interspersed with decomposed layers.

V. Sound rock — hard fresh to slightly discolored igneous rock with numerous metamorphic inclusions. Rock consists of granodiorite, quartz diorite, gneiss, migmatite, etc. as discussed in subsection 2.5.1.2.

The natural ground surface elevations at the time of the exploration showed variations within the PBA. The ground surface in the vicinity of Unit 2 ranged from approximately El. 374 feet to 428 feet, with an average elevation of 418 feet. In the vicinity of Unit 3, the ground surface was between El. 353 feet and 426 feet, with an average of El. 415 feet. These values are based on the elevations of the 200-series (Unit 2) and the 300-series (Unit 3) borings. The locations of the borings inside and outside the Unit 2 and Unit 3 PBAs are shown on Figure 2.5.4-208 and Figure 2.5.4-201, respectively.

Design plant grade is at approximately El. 400 feet. For each unit, the soil beneath the seismic Category I nuclear island is excavated down to sound rock, and the nuclear island basemat is founded at El. 360.5 feet on sound rock or on concrete placed on top of sound rock. The soil underneath the seismic Category II portions of the annex building and turbine building is excavated all the way to the rock formation (Layers III, IV, or V) and replaced with compacted granular structural fill up to El. 400 feet. The site grade is shown on the site grade plan in Figure 2.5.4-245. Consequently, the Layer I and II (residuum/saprolite) soils have no direct impact on the foundation performance of an seismic Category I or II structure. Nonetheless, the engineering properties of each layer are provided in subsection 2.5.4.2.5 for completeness. The following is a description of the subsurface materials, giving the soil and rock constituents, and their range of thicknesses encountered at the Units 2 and 3 site. RN-14-051 RN-12-029 RN-12-029

2.5.4.2.2.1 Layer V: Sound Rock

The Units 2 and 3 subsurface investigation (Reference 232) describes the bedrock underlying the main plant area mostly as granodiorite, quartz diorite, gneiss or migmatite. A detailed description of the bedrock is contained in subsection 2.5.1.2.

The top of Layer V (sound rock) was estimated using a rock quality designation (RQD) of rock core samples from boring logs of at least 50%, but typically exceeding 70%. The top of Layer V encountered in the Unit 2 borings ranges from about El. 296 feet to 384 feet, with the corresponding range in the Unit 3 borings from El. 316 feet to 384 feet. Top of sound rock contours beneath the main Unit 2 and 3 plant areas are shown in Figure 2.5.4-202.

The top of Layer V was also defined using shear wave velocity (V_s) measurements, as detailed in subsection 2.5.4.4.4. For seismic analyses (subsection 2.5.4.7), El. 355 feet was adopted as top of sound rock beneath the nuclear islands of both Units 2 and 3.

Additional information on the top of Layer V at locations site-wide is presented in Table 2.5.4-201 using the RQD criteria.

2.5.4.2.2.2 Layers III and IV: Partially and Moderately Weathered Rock

Layer IV (MWR) typically has RQD values that range from 0% to 50%. Based on this, the top of MWR encountered in the borings at Unit 2 ranges from about El. 317 feet to 391 feet, and ranges from El. 327 feet to 390 feet at Unit 3. Using shear wave velocity (V_s) measurements as discussed in subsection 2.5.4.4.4, the top of MWR is estimated to be at El. 370 feet for seismic analyses for the Unit 2 nuclear island, and at El. 360 feet for the Unit 3 nuclear island.

Layer III (PWR) typically has zero RQD when cored, but has SPT N-values of greater than 100 blows per foot (bpf) as discussed in subsection 2.5.4.2.3. Based on this, the top of PWR encountered in the borings at Unit 2 ranges from about El. 331 feet to 396 feet, and ranges from El. 353 feet to 394 feet at Unit 3. Using V_s measurements as discussed in subsection 2.5.4.4.4, the top of PWR is estimated to be at El. 375 feet for seismic analyses for the Unit 2 nuclear island, and at El. 365 feet for the Unit 3 nuclear island. This gives an estimated thickness of 5 feet for PWR at the nuclear island of each unit.

Additional information on the top of Layers III and IV at locations site-wide is presented in Table 2.5.4-201 using the RQD and N-value criteria.

2.5.4.2.2.3 Layers I and II: Residiuum and Saprolite

Layer I (residual soils) consists primarily of red fine-grained silts with varying amounts of lean clay content (ML/MH in the Unified Soil Classification System, Reference 210) and coarse-grained silty sands (SM). Although Layer II (saprolitic soils) is completely weathered rock with some preserved relict rock structure, it also consists mostly of ML/MH and SM soils, with overall engineering properties similar to Layer I. The majority of the saprolite found at the site is classified as a brown silty sand. The distribution of the Layer I and II soils varies throughout the site. The subsurface profiles beneath and beyond both Unit 2 and Unit 3 areas show that Layers I and II consist of interbedded layers of fine-grained and coarse-grained soils. From the soil samples classified in Reference 232, 69% of the samples were silty sands and 29% of the samples were silts/clays.

2.5.4.2.2.4 Subsurface Profiles

Figures 2.5.4-204 through 2.5.4-207 illustrate typical subsurface profiles across the Units 2 and 3 main plant area in east-west and north-south directions, with the associated subsurface profile

legend in Figure 2.5.4-203. The locations of these profiles are shown on the power block boring location plan in Figure 2.5.4-209. The four profiles that are drawn through the centers of the reactors, with structure cross sections added, are presented to illustrate foundation interfaces in subsection 2.5.4.3. They are also used to illustrate excavation for the new units in subsection 2.5.4.5, and for bearing capacity and settlement considerations in subsection 2.5.4.10.

2.5.4.2.3 Field Investigations

NRC Regulatory Guide 1.132 addresses the site investigation for nuclear power plants, and discusses the objectives of the subsurface investigation for the design of foundations and associated critical structures. Because the subsurface investigation should be site specific, Regulatory Guide 1.132 recognizes the need for flexibility and adjustments in the overall program, and the exercise of sound engineering judgment, so that the program is tailored to the specific conditions of the site. This guidance was used to make adjustments to the subsurface investigation during field operations so that a more comprehensive subsurface description evolved. This included adjustments in field testing locations, and adjustments in the types, depths, and frequency of sampling.

The test location summary of standard penetration test (SPT) borings, observation wells, and cone penetrometer tests (CPTs) from the Units 2 and 3 site exploration program is provided in Reference 232, and tabulated in Table 2.5.4-202. Geophysical surveys are described in subsection 2.5.4.4.

The subsurface field investigation was performed during April through August 2006. Some borehole abandonment (grouting) activity occurred after August 2006. Surveying activities to locate as-built coordinates were completed by September 2006. Most of the investigation was conducted in the main plant area with the number and depth of investigation points conforming to the guidance provided in Regulatory Guide 1.132. Additional exploration points were located outside the main plant area, *i.e.*, at the general location of the cooling towers (B-400 series), makeup water intake structure location (B-500 series), and remaining out-of-PBAs (B-600 series). The Units 2 and 3 exploration point locations are shown in Figure 2.5.4-208 (power block) and Figure 2.5.4-201 (outside power block).

The scope of work and the methods used to collect field data, as summarized in Table 2.5.4-203, are listed below. The fieldwork was performed by MACTEC Engineering and Consulting of Charlotte, North Carolina, and various subcontractors and subconsultants to MACTEC, as described in Reference 232, and consisted of the following:

- 111 exploratory borings
- 31 observation wells
- 4 packer tests
- 36 CPTs plus 7 down-hole seismic cone tests, and pore pressure dissipation tests in 6 CPTs
- 8 sets of borehole geophysical logging and 8 sets of suspension primary-shear (P-S) velocity logging
- 6 sets of field soil electrical resistivity tests
- Survey of all exploration points
- 4 test pits

- 12 Standard Penetration Tests (SPTs) hammer energy measurements

The fieldwork was performed under an audited and approved quality assurance program and work procedures developed specifically for the Units 2 and 3 project. MACTEC Engineering and Consulting, contracted to Bechtel to perform the subsurface investigation, worked under MACTEC's Quality Assurance Plan that meets the requirements of Appendix B of 10 CFR 50. This Plan included meeting the requirements of Subpart 2.20 of ASME NQA-1 (Reference 245).

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 was also on site continuously during these operations. The draft boring and well logs were prepared in the field by MACTEC geologists.

Details and results of the exploration program are contained in Reference 232. The borings, observation wells, CPTs and test pits are described in the following paragraphs. The laboratory tests are summarized and the results are presented in subsection 2.5.4.2.4. The geophysical tests are summarized and the results are presented in subsection 2.5.4.4.

2.5.4.2.3.1 Borings and Samples/Cores

A total of 88 borings, ranging from 10 to 350 feet deep, were drilled in the PBAs of Units 2 and 3. A 350-foot-deep boring was drilled at the center of each containment, to about 300 feet depth into sound rock beneath the bottom of the basemat level. All of the borings were advanced in soil using hollow stem augers and/or mud rotary wash drilling techniques until SPT refusal (defined as 50 blows per 1 inch or less) occurred. Once refusal was encountered, a steel or PVC casing was set to rock, and the holes were advanced using wire-line rock coring equipment consisting of a 5-foot or 10-foot long "NQ" or "HQ" core barrel with a split inner barrel.

The soil was sampled using an SPT sampler at 2.5-foot vertical intervals to about 15 feet depth and at 5-foot intervals below 15 feet. The SPT was performed using an automatic hammer, and was conducted in accordance with ASTM D 1586-99 (Reference 206). The recovered soil samples were visually described and classified by the onsite geologists. 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 sample storage area. This storage area consisted of climate-controlled rooms within the secured office facility used for the SCE&G New Nuclear Development project, and located about 2 miles from the Units 2 and 3 site. Each sample was logged into an inventory system. Samples removed from the facility were noted in the inventory logbook. A chain-of-custody form was also completed for all samples removed from the facility. Material storage handling was in accordance with ASTM D 4220-95 (Reference 213).

Energy measurements were made on each of the automatic SPT hammers used by the 12 drill rigs that performed the borings. The energy measurements were made in accordance with ASTM D 4633-05 (Reference 215). The average energy transfer ratio (ETR) for the hammers ranged from 72% to 86.5% as shown in Table 2.5.4-205.

Undisturbed samples were obtained in accordance with ASTM D 1587-00 (Reference 207) using a Shelby tube sampler or a rotary Pitcher sampler. Upon sample retrieval, the disturbed portions at both ends of the tube were removed, and both ends were trimmed square to establish an effective seal. 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. Table 2.5.4-204 provides a summary of undisturbed sampling performed during the subsurface investigation. Undisturbed samples are also identified on the boring logs included in Reference 232.

Rock coring was performed in accordance with ASTM D 2113-06 (Reference 209). 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 core boxes were transported to the onsite sample storage facility.

The boring logs and the photographs of the rock cores are in Reference 232, along with details of the automatic hammer energy measurements. The location and depth of each borehole are summarized in Table 2.5.4-202. The elevations of the subsurface zones observed from the individual borings are summarized in Table 2.5.4-201.

2.5.4.2.3.2 Observation Wells

Twenty-two observation wells were screened in the soil/weathered rock zone, while nine were screened in rock. The wells were installed in separate borings made between about 5 and 20 feet from the geotechnical boring with the same number, with the exception of OW-227, OW-617, OW-622, and OW-625. In these cases, borings B-227, B-617, B-622, and B-625 were reamed out and/or deepened for installation of the observation wells.

After the designated depth of each well was reached, and the PVC screen and casing were 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. Each well was capped with a lockable steel cap and surrounded with a concrete pad.

Each well was developed by pumping and bailing. The development procedure involved bailing until the water showed minimal sediment, then pumping at least three standing well volumes of water, cycling the pump on and off to create a surging effect. The well was considered developed when the pumped water was reasonably free of suspended sediment.

Field permeability testing by slug test method was performed in each observation well (except OW-501 due to its proximity to Monticello Reservoir) in accordance with ASTM D 4044-96, Section 8 (Reference 212). 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 pretest 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 pretest static level is again measured. Electronic transducers and data loggers were used to measure the water levels and times during the test.

Field permeability testing by the packer method was conducted in borings B-201, B-205, B-305, and B-330. Test procedures used are described in ASTM D 4630-96 (Reference 214), as modified by the U.S. Army Corps of Engineers in their Rock Testing Handbook (Reference 248) 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. A test length of 10 feet was used in all the tested borings.

Reference 232 contains logs for the observation wells, the well installation records, the well development records, and the well permeability and packer test results. Observation well locations and depths are summarized in Table 2.5.4-202.

2.5.4.2.3.3 Cone Penetrometer Tests

The 36 CPTs were advanced using a track-mounted, 20-ton, self-contained cone rig. Each CPT was generally advanced to refusal, at depths ranging from about 20 to 76 feet. Tip resistance, sleeve friction, and pore water pressure were measured. The CPTs were performed in accordance with ASTM D 5778-95 (Reference 219). The pore pressure filter was located immediately behind the cone tip.

Seismic CPTs were performed at approximately 3-foot intervals in 7 of the 36 CPTs as described in subsection 2.5.4.4.3. Pore pressure dissipation tests were performed in 6 CPTs at depths ranging from about 20 to 69 feet.

The CPT logs, shear wave time of arrival records, and pore water pressure versus time plots are contained in Reference 232. CPT locations and depths are summarized in Table 2.5.4-202.

2.5.4.2.3.4 Test Pits

A rubber-tired backhoe was used to excavate four test pits to depths ranging from about 3 to 6 feet to obtain bulk samples of site soils to test for suitability as backfill. 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 the Units 2 and 3 subsurface investigation. The types and numbers of laboratory tests performed on the soil samples and rock cores are shown on Table 2.5.4-206.

The laboratory testing program was selected and performed in accordance with the guidance presented in Regulatory Guide 1.138. The laboratory work was conducted under an approved quality assurance program with work procedures developed specifically for the Units 2 and 3 project. Soil and rock samples were shipped under chain-of-custody rules from the storage area to the testing laboratory as described in subsection 2.5.4.2.3. Laboratory testing of soil and rock samples, except for chemical tests and resonant column torsional shear (RCTS) tests, was performed at MACTEC laboratories in Charlotte, North Carolina and Atlanta, Georgia.

Chemical testing for pH, chlorides and sulfates in selected soil samples (to test for corrosiveness toward buried steel and aggressiveness toward buried concrete) was conducted by Severn Trent Laboratories in Earth City, Missouri. RCTS testing was performed by Fugro Consultants in Houston, Texas, under the technical direction of Dr. K. H. Stokoe of the University of Texas in Austin. RCTS tests were run on selected saprolite and granular fill samples to determine shear modulus and damping ratio variation with cyclic strain as discussed in subsection 2.5.4.2.5.4.

The details and results of the laboratory testing are included in Reference 232, which also includes references to the industry standards used for each specific laboratory test. The results of the tests on soil samples (excluding RCTS and strength tests) are summarized in Table 2.5.4-207. Table 2.5.4-208 gives the results of the unconfined compression tests on the rock cores. The results of strength tests on soil are given in Table 2.5.4-212. The results of the RCTS tests are shown in Figure 2.5.4-218. The results of the tests on bulk samples from the test pits and stockpiles are given in Table 2.5.4-210.

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

2.5.4.2.5 Engineering Properties

The engineering properties of Layers I, II, III, IV, and V derived from the Units 2 and 3 field exploration and laboratory testing programs are provided in Table 2.5.4-209 and discussed in the following paragraphs. In most cases, the engineering properties of the materials below Units 2 and 3 were identical; any variations are noted on Table 2.5.4-209.

2.5.4.2.5.1 Layers III, IV, and V: PWR, MWR and Sound Rock

The RQD and recovery values of Layers IV and V in the area of each nuclear island, annex, and radwaste building were obtained from 30 borehole logs presented in Reference 232. The borehole logs of borings B-201, B-202, B-203, B-204, B-205, B-206, B-207, B-209, B-210, B-211, B-222, B-223, B-224, B-225, B-226, and the same 300-series borings, were selected. Average RQD values from these boreholes are presented versus elevation in Figure 2.5.4-210 and Figure 2.5.4-211, for Layer IV and Layer V, respectively. In each figure, average values (mean) over 5-foot intervals are presented at mid-depth of each interval. The RQD for Layer III (PWR) is not applicable.

Average RQD values of Layer IV (MWR) in Figure 2.5.4-210 range between 0% and 50% at Unit 2, and between 0% and 60% at Unit 3. The Layer V (sound rock) at Unit 2 is generally very hard and intact, with an average RQD in the range of 80% to 100%, as shown in Figure 2.5.4-211. Below about El. 300 feet in Unit 2, the degree of variation in the RQD becomes increasingly less intense, and the rock exhibits an average RQD between 95% and 100%. The Layer V (sound rock) at Unit 3 exhibits minimal weathering and fracturing (even less than at Unit 2) with an average RQD in the range of 90% to 100%. Below El. 300 feet at Unit 3, average RQD is almost constantly 100%. Based on ASTM D 6032-02 (Reference 220), the quality of sound rock in Unit 2 and 3 areas classify as “good to excellent.”

Average recovery values of Layer IV (MWR) range between 0% and 90% at Unit 2, and between 20% and 100% at Unit 3. The average recovery of Layer V (sound rock) at Unit 2 ranges between 90% and 100%. Below El. 300 feet, average recovery is constant at 100%. The sound rock at Unit 3 exhibits a recovery of 95% to 100% above El. 300 feet and 100% below El. 300 feet.

The unconfined compression test results of 95 rock cores, obtained from the vicinity of Units 2 and 3, are presented versus elevation in Figure 2.5.4-212. For design, an unconfined compressive strength (U) of 25 kips per square inch (ksi) is adopted for the Layer V (sound rock). An average unit weight was calculated for each depth where the samples were obtained and the results are shown versus elevation in Figure 2.5.4-213. A total unit weight of 182 pounds per cubic foot (pcf) is adopted for sound rock at Units 2 and 3. For MWR and PWR, total unit weights of 160 and 145 pcf, respectively, are recommended.

The elastic modulus of each layer is derived from the results of the suspension P-S velocity logging geophysical tests performed for the Units 2 and 3 exploration program given in subsection 2.5.4.4.4. For Layer V, these low strain values agree well with the higher strain elastic moduli obtained from the unconfined compression tests. Figure 2.5.4-214 shows the variation of the ratio of elastic modulus to unconfined compressive strength from these compression tests. The median ratio is about 340.

Shear modulus values are derived from the elastic modulus obtained from the compression tests using the Poisson’s ratio values of 0.33 for PWR and MWR, and 0.24 for sound rock described in subsection 2.5.4.4.4. These shear modulus values are very similar to those computed from the V_s measurements as discussed in subsection 2.5.4.4.4 confirming that low- and high-strain modulus values are essentially the same for high strength rock, certainly for Layer V (sound rock) and Layer IV (MWR). Some strain softening has been allowed for the Layer III (PWR), as discussed in subsection 2.5.4.7. Low strain is defined here as 10^{-4}% while high strain is taken as 0.25% to 0.5%, the amount of strain frequently associated with settlement of structures on soil. A summary of low-

and high-strain moduli of each layer is presented in Table 2.5.4-209.

The sliding coefficient is tangent δ, where δ is the friction angle between the rock and the material it is bearing against, i.e., concrete in this case. Based on Reference 234, tangent $\delta = 0.7$ is adopted for Layers III, IV, and V rock. Where concrete fill is placed on top of the rock beneath the nuclear island as discussed in subsection 2.5.4.5.3.1, the surface of the concrete fill is left in a roughened state prior to pouring the mat foundation for the nuclear island to ensure that a coefficient of sliding of at least 0.7 is achieved between the concrete surfaces.

2.5.4.2.5.2 Layers I and II: Residuum and Saprolite

Index tests for determination of engineering properties were performed on selected samples of Layer I and II soils. As noted earlier, of the soil samples classified in Reference 232, most were silty sand with 69%, with the percentage of silt/clay being 29%. The fines content results of 188 tests are presented versus elevation in Figure 2.5.4-215. Layer I and II soils in the PBAs are generally characterized as nonplastic with an average fines content (materials passing No. 200 Sieve) of 37% and a median of 32% below El. 400 feet.

The Unified Soil Classification System (USCS) designations are silty sand (SM) for coarse-grained soils and mostly low to high plasticity silt (ML/MH) for fine-grained soils. While MH soils show some plastic characteristics, the ML soils have no plasticity at all as shown on Table 2.5.4-211. Similarly, almost none of the coarse-grained soils, silty sand (SM), show any plastic characteristics. For the relatively small percentage of samples that exhibited plasticity, assessed from Table 2.5.4-211, the median liquid limit was 63% while the plasticity index was 19%. The remaining 62 out of the 74 samples tested for Atterberg limits were nonplastic. The water content adopted for the overall site soils is 25%.

The measured SPT N-values ranged from 0 to refusal (defined as >100 bpf). Twelve drill rigs were used as part of the Units 2 and 3 exploration program, and ETR of each hammer was measured. The N_{60} values were adjusted by a factor of 1.20 to 1.44 depending on the measured ETR of the specific equipment used. The range of N_{60} values versus elevation is presented for soil type at each unit in Figure 2.5.4-216 and Figure 2.5.4-217. For engineering design purposes, an N_{60} value of 20 bpf was adopted for Layers I and II soils below El. 400 feet at both unit areas.

The effective angle of internal friction of a medium dense saprolite (N_{60}=20 bpf) would typically be taken as around 33° (Reference 251). However, the relatively high silt content and the presence of low plasticity clay minerals reduce this angle. The effective friction angle (ϕ') and effective cohesive component (c') of Layers I and II soils were evaluated based on the results of laboratory testing, notably a series of consolidated isotropically undrained triaxial tests and direct shear tests performed on undisturbed samples in accordance with ASTM D 4767-04 (Reference 216) and ASTM D 3080-04 (Reference 211), respectively. Table 2.5.4-212 summarizes the test results.

The consolidated isotropically undrained tests performed on silty sand (SM) soils produced a median ϕ' of 27.1°, while the direct shear test results gave a median ϕ' of 30.8°. The median c' was 0.33 kips per square foot (ksf) for consolidated isotropically undrained tests. Similarly, the consolidated isotropically undrained tests of silt (ML/MH) samples produced an average ϕ' of 28.5° and a median ϕ' of 30°. The median c' was 0.22 ksf. This high-friction angle indicates that silt/clay soils show characteristics of granular soils rather than cohesive soils. Also, as stated earlier, silt/clay soils are mostly nonplastic. Therefore, silt/clay and silty sand soils have essentially the same effective strength parameter values. Since most of the soils in Layers I and II are nonplastic, ϕ' of 30° and c' of 0.25 ksf were adopted for engineering design purposes.

Consolidation properties and stress history of Layers I and II soils were evaluated via laboratory testing. A summary of the laboratory consolidation test results is presented in Table 2.5.4-213,

including the derived compression ratio and recompression ratio values of the PBA soils. Although most of the samples were very silty sands, the fines content (and possibly the mica content) provided consolidation characteristics. Results indicate that, on average, Layers I and II soils have a compression ratio of 0.160 and a recompression ratio of 0.030. Reference 226 provides a classification for compressibility of saturated normally consolidated and overconsolidated sandy soils at various densities. For normally consolidated SM soils, compression ratio ranges between 0.017 and 0.003; for saturated overconsolidated soils, recompression ratio is typically about one-third of the values for compression ratio. The high compressibility of the samples tested is most likely due to the silt and mica content in the soil.

The unit weights of undisturbed soil samples prepared for consolidated isotropically undrained, direct shear, and consolidation tests were measured before each test. There were isolated lower densities, but these are not considered typical. A design total unit weight of 110 pcf was adopted.

The specific gravity (G_s) results of 16 undisturbed samples are reported in Reference 232. For design purposes, a G_s of 2.75 was adopted for Layers I and II soils at Units 2 and 3.

The high-strain elastic modulus (E_H) value is derived using the relationship with SPT N-value given in Reference 228. The high-strain modulus is typically taken as the modulus at a strain between 0.25% and 0.5%, *i.e.*, 0.375% (Reference 243). The shear modulus (G_H) value is obtained using the relationship between elastic modulus, shear modulus, and Poisson's ratio (Reference 224). For engineering design purposes, an E_H of 720 ksf and a G_H of 270 ksf were adopted for Layers I and II soils at Units 2 and 3 below El. 400 feet. Values of E_H and G_H are shown in Table 2.5.4-209.

The shear and compression wave velocities measured in the soil by suspension P-S velocity logging are shown in Figure 2.5.4-224 and Figure 2.5.4-225, respectively. The average V_s ranges from about 500 to 1,000 fps with increasing depth in Layers I and II. Below El. 400 feet, a best estimate of 900 fps is selected beneath each unit. This is presented in more detail in Subsections 2.5.4.4 and 2.5.4.7. The best estimate low-strain (*i.e.*, 10^{-4}) shear modulus (G_L) is derived from the V_s of 900 fps. The low-strain elastic modulus (E_L) value is obtained using the relationship between elastic modulus, shear modulus, and Poisson's ratio (Reference 224). For engineering design purposes, G_L of 2,750 ksf and an E_L of 7,350 ksf were adopted for Layers I and II soils at Units 2 and 3 below El. 400 feet. Values of G_H and E_L are shown in Table 2.5.4-209.

The unit coefficient of subgrade reaction (k_1) is based on the value for medium dense sand provided by Terzaghi (Reference 247). Based on material characterization of Layers I and II soils, a k_1 of 240 kips per cubic feet (kcf) was estimated and adopted for engineering design purposes.

The earth pressure coefficients are estimated based on Rankine's Theory, assuming level backfill and a zero friction angle between the soil and the wall as discussed in subsection 2.5.4.10. Substituting previously adopted $\phi'=30°$ for Layers I and II soils, the following earth pressure coefficients were estimated and adopted: $K_a=0.33$, $K_p=3.0$, $K_0=0.50$.

The sliding coefficient is tangent δ, where δ is the friction angle between the soil and the material it is bearing against, *i.e.*, concrete in this case. Based on Reference 234, tangent $\delta=0.35$ was adopted for Layers I and II soils.

All of the material properties designated for engineering purposes for Layer I and II soils, as well as other relevant information, are summarized in Table 2.5.4-209.

2.5.4.2.5.3 Compacted Fill

The soil underneath the seismic Category II portions of the annex building and turbine building (at both units) is replaced with well-graded sandy structural fill (SW or SW-SM), extending from rock RN-12-029

(Layers III, IV, or V) up to approximately El. 400 feet as discussed in subsection 2.5.4.5.3. It is compacted with heavy equipment in thin lifts to a dry density that is at least 95% of the maximum dry density obtained from ASTM D 1557-02 (Reference 205) as discussed in subsection 2.5.4.5. Based on this, N_{60} = 30 bpf, ϕ' = 36°, and a total unit weight of 125 pcf were selected as reasonable and conservative. RN-12-029

As an aid to construction operations, relatively small amounts of graded aggregate base (GAB), clean washed stone, and controlled low strength material (CLSM) may be used in localized areas of the power block excavations, outside the footprint of the nuclear island, as alternates to the compacted structural fill described above and in subsection 2.5.4.5.3.1. The alternate materials have engineering properties that are generally similar to, or better than, the more widely used structural fill. These fill materials are described in subsection 2.5.4.5.3.1. Their use will be limited to relatively small, localized applications within the power block excavations, as described in subsection 2.5.4.5.3.1, such that there will be no degradation in the performance of Seismic Category II or other power block structure foundations (Reference 259). The engineering properties of the alternate backfill materials are summarized in Table 2.5.4-209. RN-12-031

2.5.4.2.5.4 RCTS Tests

The results of the five RCTS tests are presented in Figure 2.5.4-218. Three of the tests were on saprolite and two tests were on samples of compacted fill. The test results on Figure 2.5.4-218 show normalized shear modulus (G/G_{max}) and damping ratio (D) versus shear strain for both the resonant column and torsional shear modes. The results are shown for a confining pressure equal to the in situ confining pressure.

Comparison of the RCTS results with the generic curves used in the seismic soil column analyses are discussed in subsection 2.5.4.7.

2.5.4.2.5.5 Chemical Properties of Layers I and II

Three criteria—electrical resistivity, pH, and chloride content—were used to evaluate the corrosion potential of the foundation soils in Layers I and II. In addition, the sulfate content was used as an indicator of the soil aggressiveness towards concrete. Twenty-two sets of chemical tests were conducted on Layer I and II soils between 6 and 53.5 feet depth. As noted in subsection 2.5.4.5, the nuclear island is supported on sound rock or on concrete placed on sound rock, and is surrounded by compacted structural fill. Buried piping, duct banks, etc. are founded in structural fill placed and compacted in the power block excavation. Thus, the chemical properties of the in-situ Layer I and II soils discussed in the following paragraphs do not impact the nuclear island nor buried utilities in the power block area. As described in subsection 2.5.4.4.1, six field electrical resistivity tests were performed using the Wenner 4-electrode array, at locations shown in Figures 2.5.4-208 and 2.5.4-201. Typically, the equivalent depth for each measurement is taken as half of the electrode spacing (Reference 229). Guidelines to assess the corrosiveness and aggressiveness of the soil are provided in Table 2.5.4-214, based on various references (References 202, 244, and 201).

<u>Attack on Steel (Corrosiveness)</u>

The electrical resistivity test results in Reference 232 indicate that the natural soils are essentially noncorrosive. In addition, the chloride contents, tabulated in Table 2.5.4-215, vary from about 1.8 ppm to 8.5 ppm, which indicate soil with little corrosive potential. However, the pH values ranging from 4.9 to 6.0 indicate the soil to be mildly corrosive to corrosive. Based on the pH results, all Layer I and II soils at the site should be considered at least moderately corrosive to metals at this stage, requiring protection if metal is placed within them.

Attack on Concrete (Aggressiveness)

The sulfate content, tabulated in Table 2.5.4-215, varies from 0.0003% to 0.0017%. Based on the Table 2.5.4-214 guidelines, no special sulfate resisting cement is required for non-safety related structures that are in contact with these in-situ materials.

2.5.4.3 Foundation Interfaces

The locations of all site exploration points for the Units 2 and 3 subsurface investigation, including borings, observation wells, CPTs, electrical resistivity tests, and test pits are shown on Figure 2.5.4-201 and Figure 2.5.4-208. The locations of the subsurface profiles on Figures 2.5.4-204 through 2.5.4-207 are shown on Figure 2.5.4-209.

Figure 2.5.4-219 shows the excavation geometry for the safety-related and other major facilities. The cross sections of the structure foundations and the proposed excavation and backfilling limits are superimposed on Figures 2.5.4-204 through 2.5.4-207 to produce Figures 2.5.4-220 through 2.5.4-223.

Logs of all the core borings and test pits are contained in Reference 232.

2.5.4.4 Geophysical Surveys

The geophysical testing for Units 2 and 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 at the six locations shown in Figures 2.5.4-208 and 2.5.4-201. The Wenner four-electrode method was used in accordance with ASTM G 57-06 (Reference 223). In this method, four electrodes, two for current and two for voltage, are spaced an equal distance apart and inserted about 12 inches 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 to 300 feet. The results of the testing are given in Reference 232 and are discussed relative to corrosion potential in subsection 2.5.4.2.5.

2.5.4.4.2 Geophysical Down-Hole Testing

Geophysical down-hole tests were performed in eight borings in the PBA. Four tests—B-201 (350 feet depth), B-206 (215 feet depth), B-207 (175 feet depth), and B-211/211A (175 feet depth)—were carried out in the Unit 2 area. The other four tests—B-301 (350 feet depth), B-306 (215 feet depth), B-307/307A (175 feet depth), and B-311 (175 feet depth)—were conducted in the Unit 3 area. The tests performed were natural gamma, three-arm caliper, long and short normal 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 contained in Reference 232. Plots of the shear and compression wave velocity results versus elevation are presented in subsection 2.5.4.4.4. The descriptions below are summarized from the more detailed description in Reference 232.

For most of the tests, the eight borings were logged as partially-cased borings, filled with clear water or polymer-based drilling mud, with a 4-inch PVC or steel casing placed in the top 40 to 60 feet of softer soil above bedrock contact during the measurements in the lower rock portions of the borings. In some cases, acceptable results were obtained from the suspension P-S logger in the PVC-cased soil hole, provided the casing was well grouted into the soil. Where lack of coupling occurred between the casing and the soil leading to poor quality velocity measurements, a separate uncased hole was drilled in the soil about 10 feet from the original hole, and P-S suspension velocity readings

were taken in the uncased hole. The instrument probe receives control signals from, and sends the digitized receiver signals to, instrumentation on the surface via an armored four-conductor cable. The cable is wound onto the drum of a winch and is used to support the probe.

2.5.4.4.2.1 Natural Gamma and Three-Arm Caliper

Caliper and natural gamma data were collected using a Model 3ACS three-arm caliper probe, manufactured by Robertson Geologging, Ltd, in accordance with ASTM D 6167-97 (Reference 221) and ASTM D 6274-98 (Reference 222). With this tool, caliper measurements were collected concurrently with the measurement of natural gamma emission from the borehole wall. The probe is 6.82 feet long and 1.5 inches 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 that emit gamma radiation as they decay. The measurement is useful because the radioactive elements are concentrated in certain rock types, *e.g.*, clay or shale, 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 9.8 feet/minute, collecting data continuously at 0.05-foot spacing.

2.5.4.4.2.2 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, in accordance with ASTM D 5753-05 (Reference 218). The probe, which is 8.2 feet long and 1.73 inches 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 feet/minute, collecting data continuously at 0.05 foot spacing.

2.5.4.4.2.3 Acoustic Televiewer and Borehole Deviation Measurement

Acoustic image and boring deviation data were collected using a high-resolution acoustic televiewer probe, manufactured by Robertson Geologging, Ltd. The probe, which is 7.58 feet long and 1.9 inches 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. The changes in contrast between native rock and dikes provide imaging of fracture filling. 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-inch diameter spot about 2 inches from the central axis of the probe. The reflector has the ability to rotate, and data were collected at 360 samples per revolution during the survey.

The probe contains a fluxgate magnetometer to monitor magnetic north, and all raw televiewer data are referenced to magnetic north. In addition, a three-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 feet/minute, collecting data continuously at 0.008-foot intervals. The data were presented on a computer screen for operator review during the logging run, and stored on hard disk for later processing.

2.5.4.4.2.4 Suspension P-S Velocity Logger

Soil velocity measurements were performed using a digital OYO Model 170 suspension P-S logging recorder and probe. This system directly determines the average in situ horizontal shear and compressional wave velocity measurements of a 3.3-foot high segment of the soil or rock column surrounding the borehole by measuring the elapsed time between arrivals of a wave propagating upwards through the soil or rock column.

Suspension P-S velocity logging uses a 19-foot-long probe containing a source near the bottom and a receiver pair centered 12.1 feet above the bottom end of the probe. The average wave velocity is determined from the travel time between the two receivers, which are 3.3 feet apart. For quality assurance, analysis is also performed on source-to-receiver data. The entire probe is suspended in the boring by the cable. 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 was repeated at 1.6-foot intervals.

2.5.4.4.3 Seismic Tests with Cone Penetrometer

Seven seismic CPTs were performed at approximate 3-foot vertical intervals in Layer I and II soils. Three tests—C-202, C-207, and C-209—were carried out in the Unit 2 area with a depth range of 36 to 51 feet. Three tests—C-302c (repeat of C-302), C-307, and C-309—were carried out in the Unit 3 area with a depth range of 45 to 48 feet. One test—C-602b—was performed in the general area of the cooling towers, which is on the southeast side of the power block, to a depth of 58 feet.

Shear waves were generated by striking a heavy beam adjacent to the CPT location. Compression waves were not 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 Reference 232. Plots of the CPT V_s results versus elevation are presented in subsection 2.5.4.4.4.

2.5.4.4.4 Results of Shear and Compression Wave Velocity Tests

2.5.4.4.4.1 Layer V

Based on the RQD definition of sound rock (Layer V) in subsection 2.5.4.2.2, the elevation of the top of Layer V is interpreted using the rock samples cored in the PBA (*i.e.*, borehole logs of B-200 and B-300 series). The average and median elevation interpretations for the overall PBA are tabulated in Table 2.5.4-201, and the top of sound rock is computed to be at El. 350 feet and El. 360 feet in the vicinity of Units 2 and 3, respectively. This gives an average of El. 355 feet for both units. The elevation of top of sound rock can also be defined based on a V_s of 6,500 fps. The 6,500 fps value is selected based on rock that is non-rippable with a very large ripper (Reference 225). The elevations of top of sound rock at boreholes where suspension P-S logging tests were performed (*i.e.*, B-201, B-206, B-211, B-301, B-306, B-307 and B-311), are selected based on the bedrock elevations, where V_s is at least 6,500 fps and continually stays above 6,500 fps as the depth increases. For the four boreholes with suspension P-S logging at each unit, the elevations of top of sound rock based on the V_s criterion is about El. 355 feet. Thus the top of sound rock based on RQD definitions and based on the V_s approach is consistent. Consequently, El. 355 feet is adopted as the best estimate elevation of top of Layer V in the Units 2 and 3 nuclear island areas.

Figure 2.5.4-224 shows the measurements of V_s from suspension P-S logging—four tests at each unit—in Layer I through Layer V versus elevation. Figure 2.5.4-225 shows the corresponding measurements of compression wave velocity (V_p). These measurements were taken in the PBA of each unit (*i.e.*, at the reactor, turbine, auxiliary/radwaste buildings, and [plant] west of the reactor). In Figure 2.5.4-226, V_s values of Layer V are averaged over 5-foot vertical intervals for each unit. The average value (mean) and the low/high ends (mean $\pm$ standard deviation) are illustrated as vertical bars along each 5-foot-long interval. A best estimate V_s of 10,000 fps is adopted for Layer V in the PBA below El. 355 feet.

The values of low strain Poisson's ratio (μ) are determined from a relationship between V_s and compression wave velocity. The average Poisson's ratio values derived from 4 suspension P-S loggings for each unit are shown in Figure 2.5.4-227. In these plots, Poisson's ratio values are averaged over 5-foot vertical intervals. The average value (mean) and the low/high ends (mean $\pm$ standard deviation) are illustrated as vertical bars. The plots show an average μ of 0.23–0.25 for sound rock under each unit. A best estimate Poisson's ratio of 0.24 is adopted for Layer V in the power block below El. 355 feet. The Poisson's ratios obtained from unconfined compression tests of rock as shown in Table 2.5.4-208 are somewhat higher than the seismic test results: for Unit 2, the average μ is 0.30 with a median of 0.31, and for Unit 3, the average μ is 0.32 with a median of 0.30. These were obtained from the readings from lateral and vertical strain gauges that were attached to the rock specimen. These differences are attributed to the difference in measurement method.

The average V_p values are determined from the same relationship between V_s and low strain Poisson's ratio (μ). Therefore, using the previously established best estimate V_s of 10,000 fps and a Poisson's ratio of 0.24, gives a value of V_p of just over 17,000 fps for Layer V. Based on this and the very consistent values shown in Figure 2.5.4-225, a best estimate value of 17,500 fps was selected.

2.5.4.4.4.2 Layers I, II, III, and IV

The measurements of V_s from suspension P-S logging tests and seismic CPTs in Layers I through IV (and the top of Layer V) are shown versus elevation in Figure 2.5.4-228 (Sheets 1 and 2) for Units 2 and 3, respectively. In both figures, the shear wave velocities in Layers I and II show an increase from approximately 500 fps to 1,000 fps with increasing depth. In Figure 2.5.4-229 (Sheets 1 and 2), V_s values of Layers I and II are averaged over 5-feet vertical intervals. The average value (mean) and the low/high ends (mean $\pm$ standard deviation) are shown as a vertical bar along each 5-foot long interval. A best estimate V_s of 900 fps is adopted for Layers I and II in the PBA below the site grade (*i.e.*, El. 400 feet) down to top of PWR/MWR (*i.e.*, El. 375 feet at Unit 2 and El. 365 feet at Unit 3).

Based on the RQD definitions listed in subsection 2.5.4.2.2, the elevations of top of Layers III and IV (PWR and MWR) are interpreted using the rock samples cored in the PBA (*i.e.*, borehole logs of B-200 and B-300 series). The elevations of top of each layer are summarized in Table 2.5.4-201 for Units 2 and 3 with average/median values. Given that PWR/MWR is a transition zone from soil to rock, the elevation of the top of Layer III (PWR) is also defined based on a V_s of 2,500 fps, given in Reference 231 as the transition velocity between strong soil and soft rock. The elevations of top of Layer III in eight boreholes, where suspension P-S logging tests were performed, are selected based on the bedrock elevations where V_s is at least 2,500 fps and continually stays above 2,500 fps as the depth increases. Accordingly, El. 375 feet and El. 365 feet are adopted as the top of Layer III in the Unit 2 and 3 nuclear island areas, respectively.

The values of V_s increase very quickly with increasing elevation through the transition zone, and so the average thickness of Layer III is selected as 5 feet, and thus El. 370 feet and El. 360 feet are determined as top of Layer IV in the Unit 2 and 3 nuclear island areas, respectively. Given that El. 355 feet is top of Layer V as described in subsection 2.5.4.4.4, the results indicate minimal thickness of Layer III and relatively thin layers of Layer IV. In Figure 2.5.4-230 the V_s values of Layers III and IV are presented averaged over 5-foot vertical intervals, as well as the low/high ends (mean $\pm$ standard deviation). The best estimate for V_s of 3,000 fps and 6,000 fps for Layer III and Layer IV, respectively, are adopted in the PBA, respectively.

The values of low-strain Poisson's ratios (μ) are determined from a relationship between V_s and V_p. The measurements of V_p from suspension P-S logging—four tests at each unit—in Layers I and II and Layers III and IV (above El. 355 feet) are shown versus elevation in Figure 2.5.4-231 (Sheets 1 and 2). The average Poisson's ratio values of Layers I, II, III and IV derived from 4 suspension P-S velocity logging tests at each unit are shown in Figure 2.5.4-232. In these plots, Poisson's ratio values are averaged over 5-feet vertical intervals, and the average value (mean) and the low/high ends (mean $\pm$ standard deviation) are illustrated as vertical bars. These plots show a range between 0.3 and 0.4 for ML-MH-SM type of soil (Layers I and II). A best estimate Poisson's ratio (μ) of 0.33 is adopted for Layers I and II in the PBA below the site grade (*i.e.*, El. 400 feet) down to top of PWR/MWR (*i.e.*, El. 375 feet at Unit 2 and El. 365 feet at Unit 3). Compared to published values of 0.3 for granular soils and silts, and 0.4 for cohesive soils (Reference 224), the calculated values are consistent. In a similar manner, a best estimate Poisson's ratio (μ) of 0.33 is adopted for Layers III and IV in the PBA.

The average V_p values are determined from the relationship between V_s and low-strain Poisson's ratio (μ). Therefore, using the previously established V_s and the Poisson's ratios, a best estimate V_p

of 1,800 fps is adopted for Layers I and II. Similarly, compression wave velocities of 6,000 fps and 12,000 fps are adopted for Layers III and IV, respectively.

2.5.4.5 Excavation and Backfill

This section describes the following topics:

- The extent (horizontally and vertically) of anticipated excavations, fills, and slopes required for construction of the nuclear island and major power block structures for each unit. RN-12-029
- Excavation methods and stability.
- Backfill sources, quantities, compaction specifications, and quality control.
- Construction dewatering impacts.

2.5.4.5.1 Extent of Excavations, Fills and Slopes

Figure 2.5.4-219 shows the location of the excavation cross-sections and temporary slopes for Units 2 and 3. The site grade plan Figure 2.5.4-245 shows the extent of backfill and permanent outer slopes. The bottoms of foundations and backfill locations are shown in cross sections in Figures 2.5.4-220 through 2.5.4-223. The topography of the original ground surface with boring locations is shown in Figure 2.5.4-233.

To obtain plant grade of about El. 400 feet, the natural ground surface is leveled by excavating up to 28 feet of residuum and saprolite. The excavation for each unit is sized to include the footprints of non-seismic major power block structures that are located in proximity to the nuclear island and the seismic Category II structures. Temporary retaining walls (soldier pile and lagging with tiebacks) are used to support the near-vertical power block excavations (as depicted on Figures 2.5.4-219 through 2.5.4-223). Temporary construction slopes and benches are used in some limited areas within and beyond the retaining walls where the excavation is deeper than about 40-50 feet and for the access ramps and circulating water pipe trenches. Temporary construction slopes in soil are typically 2-horizontal to 1-vertical (2H:1V), and benched about every 20 vertical feet. Temporary slopes steeper than 2H:1V may be required for backfill operations as described in subsection 2.5.4.5.3.1. The residuum, saprolite and partially weathered rock is removed using conventional excavating equipment. Further excavation in MWR and sound rock is by controlled blasting as needed to achieve final design grade as described in subsection 2.5.4.5.2.2. RN-12-029 RN-14-134

As shown in Figure 2.5.4-245, the PBA and cooling tower areas have a finished grade ranging from just below El. 400 feet to El. 390 feet, descending downward beyond the perimeter of the plant at approximately a 3H:1V slope. The largest slope descends from around El. 390 feet to El. 315 feet beyond the (plant) western perimeter. There are limited areas where existing ground rises at the perimeter to the (plant) north of Unit 3. This is a (typical) 3H:1V slope, with a maximum height of about 25 feet. The stability of temporary and permanent slopes is addressed in subsection 2.5.5.

2.5.4.5.2 Excavation Methods and Stability

Excavation

Excavation for the nuclear island structures below grade may use either a sloping excavation or a vertical face as described in subsequent paragraphs. If sloping excavations are to be used on a soil site, the Combined License applicants must evaluate the 3D effects on the site response and perform site-specific SSI analyses using either or a combination of 2D or 3D SASSI models that reflect the sloping excavations. If backfill is to be placed adjacent to the exterior walls of the nuclear island, the Combined License applicant will provide information on the properties of backfill and its compaction

requirements as described in subsection 2.5.4.6.3 and will evaluate its properties against those used in the seismic analyses described in subsection 3.7.2.

For the vertical face alternative, excavation in soil for the nuclear island structures below grade will establish a vertical face with lateral support of the adjoining undisturbed soil or rock. This vertical face will be covered by a waterproof membrane as described in subsection 3.4.1.1.1.1 and is used as the outside form for the exterior walls below grade of the nuclear island. Alternative methods include a soil nailing and mechanically stabilized earth (MSE) walls.

Vertical Face Using Soil Nails

Soil nailing is a method of retaining earth in-situ. As the nuclear island excavation progresses vertically downward, holes are drilled horizontally into the adjoining undisturbed soil, a metal rod is inserted into the hole, and grout is pumped into each hole to fill the hole and to anchor the “nail” rod.

As each increment of the nuclear island excavation is completed, nominal eight to ten inch diameter holes are drilled horizontally through the vertical face of the excavation into adjacent undisturbed soil. These “nail” holes, spaced horizontally and vertically on five to six feet centers, are drilled slightly downward to the horizontal. A “nail”, normally a metal bar/rod, is center located for the full length of the hole. The nominal length of soil nails is 60 percent to 70 percent of the wall height, depending upon soil conditions. The hole is filled with grout to anchor the rod to the soil. A metal face plate is installed on the exposed end of the rod at the excavated wall vertical surface. Welded wire mesh is hung on the wall surface for wall reinforcement and secured to the soil nail face plates for anchorage. A 4,000 psi to 5,000 psi non-expansive pea gravel shotcrete mix is blown onto the wire mesh to form a nominal four to six inch thick soil retaining wall. Installation of the soil retaining wall closely follows the progress of the excavation and is from the top down, with each wire mesh-reinforced, shotcreted wall section being supported by the soil “nails” and the preceding elevations of soil nailed wall placements.

Soil nailing as a method of soil retention has been successfully used on excavations up to 55 feet deep on projects in the U.S. Soils have been retained for up to 90 feet in Europe. The state of California CALTRANS uses soil nailing extensively for excavations and soil retention installations. Soil nailing design and installation has a successful history of application which is evidenced by its excellent safety record.

The soil nailing method produces a vertical surface down to the bottom of the excavation and is used as the outside forms for the exterior walls below grade of the nuclear island. Concrete is placed directly against the vertical concrete surface of the excavation.

For methods of soil retention other than soil nailing, such as for excavation in rock, four to six inches of shotcrete are blown on to the vertical surface. The concrete for the exterior walls is placed against the shotcrete.

Vertical Face Using Mechanically Stabilized Earth Walls

Mechanically stabilized earth walls (MSE) are flexible retaining wall systems that use strip, grid, or sheet type of tensile reinforcements in the soil mass, and a discrete modular pre-cast concrete, which is vertical. MSE walls function like, and are generally more economical than, conventional retaining walls. The tensile strength of the reinforcements and the slip at the interface of the reinforcement and the soil provide great internal stability to MSE walls. These walls may be used where the side soils have to be removed or the grade elevation needs to be raised. The walls and backfill are placed prior to construction of the nuclear island.

Mudmat

The mudmat provides a working surface prior to initiating the placement of reinforcement for the foundation mat structural concrete. The lower and upper mudmats are as follows:

- Lower mudmat – (minimum 6 inches thick) of un-reinforced concrete, with a minimum compressive strength of 2,500 psi. The lower mudmat will be used as the final dental concrete layer on the underlying foundation media.

- Upper mudmat – (minimum 6 inches thick) of un-reinforced concrete with a minimum compressive strength of 2,500 psi. This upper mudmat will support the chairs that, in turn, support the reinforcing steel.

The waterproofing system is described in subsection 3.8.5.1.

2.5.4.5.2.1 Excavation in Soil

Excavation in the soils (Layers I and II) and any existing fills is achieved with conventional excavating equipment. Excavation will adhere to OSHA regulations (Reference 236) when less than 20 feet high. As noted in subsection 2.5.4.5.1, temporary tied-back retaining walls are used to support the near-vertical excavations. Temporary construction slopes are used in some limited areas within and RN-12-029 beyond the retaining wall where the excavation is deeper than about 40 to 50 feet and for the access ramps and circulating water pipe trenches. Since the saprolitic soils can be highly erosive, temporary RN-12-029 slopes cut into the saprolite are sealed and protected, as needed to prevent excessive erosion.

2.5.4.5.2.2 Excavation in Rock

Excavation in Layer III (PWR) rock is achieved using conventional earthmoving equipment. Temporary retaining walls are used to support the near- vertical excavations.

In subsection 2.5.4.4.4, it was noted that the top of sound rock for both units is taken at El. 355 feet, based on consideration of RQD and Vs. This is the top of rock used in the seismic analysis described in subsection 2.5.2 and subsection 2.5.4.7. However, El. 355 feet is the average top of sound rock. Beneath the nuclear island, sound (non-rippable) rock extends as high as El. 374 feet in Unit 2, *i.e.*, 14 feet above the bottom of the nuclear island basemat. The top of sound rock extends only about 3 RN-12-029 feet above the bottom of the basemat in Unit 3. For both Units limited hard rock excavation techniques are needed.

Hard rock excavation in Layers IV and V (MWR and sound rock) is performed with “lessons learned” RN-12-029 application from previous projects. The following methods of rock excavation employ techniques to reduce vibrations.

- Controlled blasting techniques, including cushion blasting, pre-splitting and line drilling may be used, with appropriately dimensioned bench lifts. The blasted faces are vertical.

- 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 is a function of distance from the blast.

- The rock is reinforced, if necessary, to ensure adequate support and safety.

- The excavation is mapped and photographed by experienced geologists. Appropriate measures are taken if weathered or fractured zones are encountered.

2.5.4.5.3 Backfill Sources, Compaction, and Quality Control

2.5.4.5.3.1 Structural Fill

Although a large amount of residual and saprolitic soil is excavated for the units, this material is not used as structural fill to support or back fill structures, but is used as common fill. The structural fill is a compacted well-graded granular material. The anticipated extent of concrete fill and structural fill is shown on the foundation cross sections on Figures 2.5.4-220 through 2.5.4-223. The concrete fill is used mainly to replace any partially or moderately weathered rock exposed at the bottom of the excavations for the seismic Category I nuclear island foundation mat. Concrete fill is also used beneath the turbine building basemat and locally as a leveling layer on top of rock. RN-12-012 RN-12-029

The granular structural fill material does not exist naturally on site. Therefore, this material is imported to the site. A source of suitable structural fill is located about 20 miles from the site, at Martin Marietta Aggregate's North Columbia Quarry. The material is granitic sand from the quarry's rock crushing operation. There are hundreds of thousands of tons of the sand stockpiled, with an estimated 30 year's future supply. Particle size distribution curves from samples of the material are shown on Figure 2.5.4-234. The sand is classified as SW or SW-SM. Modified Proctor compaction test (ASTM D 1557-02) (Reference 205) results shown in Figure 2.5.4-235 indicate a maximum dry density in the 123 to 125 pcf range, with an optimum moisture content between about 8% and 11%. RCTS tests were performed on two samples of this material, and the results are shown on Figure 2.5.4-218 and discussed in subsection 2.5.4.7.2. RN-12-029

This structural fill is placed in thin lifts and compacted to at least 95% of the maximum dry density as determined by ASTM D 1557-02 (Reference 205), with moisture conditioning as needed to achieve this degree of compaction. Compaction is performed with a heavy steel-drummed vibratory roller, except within 5 feet of a structure wall, where smaller compaction equipment is used to minimize excess pressures against the wall. As noted in subsection 2.5.4.2.5, based on the type of material and its degree of compaction, a minimum N_{60} value of 30 bpf and an effective friction angle (φ') of 36° were adopted as reasonable and conservative for this structural fill. RN-12-058

During placement of backfill within power block excavations, temporary slopes may be constructed in structural fill as aids to construction operations. Slopes steeper than 2-horizontal to 1-vertical (2H:1V) may be required. Reinforcement for temporary slopes will utilize geogrids and a geotextile-wrapped face, with or without wire mesh baskets. The reinforced slopes will be abandoned in place when no longer needed by subsequent placements of structural fill against the face. Temporary slopes will not be used beneath the nuclear islands. RN-14-134

Structural fill comprise more than 90% of the material used to fill the power block excavations around the nuclear island and beneath foundations for Seismic Category II and non-seismic structures. The following materials will be used in limited, local applications as substitutes for the Structural fill, as aids to construction operations. The volume of alternate fill materials is less than 10% of the total volume of backfill used for each of the power block excavations. The use of alternate fill material installed in the Power Block excavations will be documented. Engineering properties of the alternate materials are summarized in Table 2.5.4-209. Because of the similarities of these engineering properties to those of structural fill and the relatively small quantities of alternate fill materials used, there is no effect on the dynamic loading response, liquefaction potential, or static foundation analyses described in subsection 2.5.4.7, 2.5.4.8, and 2.5.4.10. The alternate backfill materials are not used beneath the nuclear island. RN-12-031

A. Graded Aggregate Base (GAB) RN-12-031

Graded Aggregate Base is obtained from similar source quarries as the Structural Fill and therefore has similar granitic origins. Like Structural Fill, it is a well graded granular material, but it includes a significant amount of coarser gravel particles and is generally classified as GW or GW-GM. The

specified GAB is a widely used base course material consisting of SCOOT Macadam Base Course (SCOOT Specification Sec 305.2.5.5) (Reference 258).

GAB will be used primarily to construct haul roads and pads for cranes or other heavy construction equipment within the Power Block excavations. However, since it is a well graded granular material with excellent compaction prope1ties, it may be substituted for Structural Fill locally within the Power Block excavations where a denser and coarser grained material is deemed useful to support construction operations. Similar to Structural Fill, GAB is placed in maximum 12 inch loose lifts and compacted to at least 95% of the modified Proctor maximum dry density determined in accordance with ASTM D 1557-02 (Reference 205). RN-12-031 RN-12-031

B. Clean Washed Stone

Clean Washed Stone is a processed open-graded material classified as GP. It is specified as SCDOT #57 Stone (SCDOT Specification Appendix A-4) (Reference 258) and shall be obtained from similar source quarries as the Structural Fill and GAB. It therefore has the same granitic origins as the Structural Fill. RN-12-031

Within the power block excavations Clean Washed Stone will only be used locally for backfill of sumps and ditches where it is difficult to meet the specified compaction requirements for Structural Fill or GAB. Because it is open graded and lacks fines, Clean Washed Stone is not placed to a 95% of the modified Proctor compaction criteria, but rather placement is controlled by compaction effort within a confined area. The material is placed in maximum 12-inch lifts only within small depressions, sumps, and ditches such that the stone is laterally confined. It is compacted by tamping with a backhoe or worked with other heavy equipment until no significant settlement or deformation is evident. Clean Washed stone is used sparingly in the power block excavations and only in locations where it is capped by a layer of CLSM and covered by more than about 10 ft of Structural Fill, so that loads from overlying foundations are not concentrated on the stone. Under these limited conditions, the stone does not affect the settlement or stability of overlying structure foundations. RN-12-031

C Controlled Low Strength Material (CLSM)

CLSM is a cementitious material commonly referred to as flowable fill. It is intended to be self consolidating and, when strength is kept below about 200 psi, excavatable. CLSM is used locally within the power block excavations (1) as a leveling layer, generally less than 3-4 ft thick, to fill depressions on irregular rock surfaces in order to provide an improved base for placement and compaction of the first lift of Structural Fill, (2) as a cap over Clean Washed Stone used to fill wet sumps and ditches, and (3) in confined areas where access for compaction of Structural Fill is difficult. CLSM has 28-day strength in the range of approximately 50 to 200 psi. RN-12-031

Fill placement and compaction control procedures are addressed in a technical specification. It includes requirements for suitable fill and sufficient testing to address potential material variations, in-place density testing frequency, and compressive strength (CLSM). The compacted structural fill and GAB placement and testing follow the guidelines of ASME NQA-1-1994 (Reference 253), with one field density test being performed per lift and per shift, and for no more than every 250 cubic yards of fill placed (Table 5.6 of Reference 253). The specification also includes requirements for an onsite testing laboratory for quality control (e.g., gradation, moisture density, placement, and compaction) 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 program. Sufficient laboratory compaction (modified Proctor) and grain size distribution tests are performed to ensure that variations in the fill material are accounted for. 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. RN-12-031

2.5.4.5.3.2 Common Fill

The residual and saprolitic soils excavated from the site can be used for common fill placed and compacted outside the structural fill as shown on Figures 2.5.4-220 through 2.5.4-223. Most of these soils are silty sands, which are suitable for common fill, although the sandy silt and silty clay saprolite can also be used for common fill provided the liquid limit is less than 50%. Modified Proctor compaction tests results (Reference 232) indicate a maximum dry density in the 106 to 109 pcf range, with an optimum moisture content between about 15% and 18%.

This common fill is placed in relatively thin lifts and compacted to at least 90% of the maximum dry density as determined by ASTM D 1557-02 (Reference 205), and to within 3% of its optimum moisture content.

2.5.4.5.4 Control of Groundwater During Excavation

Construction dewatering is presented in subsection 2.5.4.6.2. Since the saprolitic soils can be highly erosive, sumps and ditches constructed for dewatering these soils are lined as needed to prevent excessive erosion. The tops of excavations are sloped back to prevent runoff down the excavated slopes during heavy rainfall. RN-12-029

2.5.4.6 Groundwater Conditions

2.5.4.6.1 Groundwater Measurements and Elevations

Thirty-one observation wells were installed at the site as part of the subsurface investigation plan. Twenty-two of the wells were completed in the saprolite/ shallow bedrock zone and nine were completed in the deep bedrock zone. Figure 2.5.4-236 shows the locations of the shallow wells in the vicinity of the PBA. The groundwater level measurements in the observation wells were taken between June 2006 and June 2007 on a monthly basis. These levels are shown for each well in Figure 2.4-235.

Groundwater is present in unconfined conditions in both the saprolitic soils and in the underlying bedrock at the Units 2 and 3 site. The piezometric levels in shallow wells range between El. 351 feet and El. 366 feet in the area of Unit 2, and between El. 359 feet and El. 374 feet in the area of Unit 3. Five sets of groundwater contours given in subsection 2.4.12 present quarterly levels based on the monthly measured data. Figure 2.5.4-237 is included as a representative piezometric level contour map for the shallow wells and shows the contours for the March 2007 period. For Units 2 and 3, the maximum groundwater level for the main plant area is projected at El. 380 feet in subsection 2.4.12.

The existing ground surface is reduced to approximately El. 400 feet during construction as explained in subsection 2.5.4.2.2, resulting in removal of around 20 feet of soil in the PBA. This reduces the groundwater levels to some extent; however, the existing groundwater contours can be conservatively used where suitable for design purposes. Further details of measured groundwater levels and their fluctuations are given in subsection 2.4.12. Logs and details of the 31 wells, and tests performed in the wells, are provided in Reference 232.

The hydraulic conductivity values for the saprolite/shallow bedrock, based on the results of 16 slug tests, range from 0.0017 feet/day to 18 feet/day, with a geometric mean value of 0.60 feet/day. The hydraulic conductivity of the underlying deep bedrock (*i.e.*, sound rock), as determined from the results of five slug tests, range from 0.0088 feet/day to 0.38 feet/day, with a geometric mean value of 0.07 feet/day. The results of packer tests conducted in selected geotechnical borings in deep bedrock provided a hydraulic conductivity varying between 0 feet/day and 1.14 feet/day, with a geometric mean value of 0.166 feet/day. The differences in values measured by the two test methods are

interpreted as a result of the depths at which the tests were conducted. A detailed description of hydraulic conductivity values is provided in subsection 2.4.12.

The need for a permanent groundwater dewatering system is not anticipated for Units 2 and 3. However, localized temporary dewatering is expected to be required during plant foundation excavation and construction. 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 immediate Units 2 and 3 area as discussed in subsection 2.4.12.5. The relatively low permeability of the saprolite and underlying rock means that temporary sumps and pumps should be sufficient for successful dewatering during construction of the units, as presented in subsection 2.5.4.6.2.

2.5.4.6.2 Construction Dewatering and Seepage

Dewatering for all major excavations can be achieved by gravity-type systems using sumps and pumps.

2.5.4.6.2.1 Soils

Sump-pumping of ditches is adequate to dewater the soil because of the relatively impermeable nature of saprolite. For unsupported excavations, these ditches are advanced below the progressing excavation grade. As noted earlier, since the saprolitic soils can be highly erosive, sumps and ditches constructed for dewatering these soils are lined as needed to prevent excessive erosion. Generally excavations supported by a retaining wall do not require dewatering from ditches and sumps.

RN-12-029

RN-12-029

2.5.4.6.2.2 Rock

Sump-pumping is used to collect water from ditches that are installed below the progressing excavation grade. During construction of Unit 1, groundwater entered the excavation in sufficient quantity to require such dewatering of the rock in only three areas.

2.5.4.6.3 Effect of Groundwater Conditions on Foundation Stability

The highest anticipated groundwater level is assumed to be at El. 380 feet as noted in subsection 2.5.4.6.1. Given that the existing ground surface is reduced to approximately El. 400 feet, groundwater level is expected to drop down to some extent. Nevertheless, this water level was used in computing hydrostatic pressures on the buried structure walls as discussed in subsection 2.5.4.10.

There are no buoyancy issues with deep buried structures because of the appreciable dead loads imposed by these structures as discussed in subsection 2.5.4.10. Large diameter buried piping such as the circulating water pipes are designed to resist buoyancy when empty.

No permanent dewatering system is required for the PBA of Units 2 and 3 as noted in subsection 2.5.4.6.1.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

The basemat for the nuclear island for each of the units is founded on Layer V (sound rock) or on concrete placed on sound rock. The seismic Category II portions of the annex building and turbine building are founded on concrete or compacted structural fill placed on top of Layers III and IV and/or Layer V. The proposed foundation cross sections are illustrated on Figures 2.5.4-220 through 2.5.4-223.

RN-12-029
RN-12-012

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

- V_s profiles of the rock and the overlying soil
- 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 Profiles

Various measurements were made at the Units 2 and 3 site to obtain estimates of the V_s in the soil and rock. These are summarized in subsection 2.5.4.4.4. All of the subsurface layers are of interest here, *i.e.*, Layers I and II (residuum/saprolitic soils), Layers III and IV (PWR and MWR), Layer V (sound rock), and structural fill. Since the bedrock supports the seismic Category I structures, it is considered first.

2.5.4.7.1.1 Bedrock

Shear wave velocity (V_s) of the bedrock measured at the nuclear island of each unit (B-201/ B-301), and the surrounding major power block structures (B-206/B-306, B-207/B-307, B-211/B-311) is shown versus elevation in Figure 2.5.4-224. Figure 2.5.4-226 shows best-fit design values applied to these measured V_s profiles. In these plots, V_s values of Layer V are averaged over 5-feet vertical intervals for each unit. Each average V_s (mean) with corresponding low and high boundaries (mean $\pm$ standard deviation) is illustrated as a vertical bar. In the vicinity of the Unit 2 nuclear island, V_s shows some scattering in the upper 90 feet or so of the sound rock (between El. 360 feet and El. 250 feet) before it reaches an almost constant value below El. 250 feet. This scattering seems to be relatively localized since the variation in V_s in sound rock of the Unit 3 nuclear island area is much smaller. The average mean value over the measured range in these plots is more than 10,000 fps at each unit.

2.5.4.7.1.2 Soil and Weathered Rock

The PWR/MWR layer is a transition zone from soil to sound rock. Reference 231 defines very dense soil and soft rock with V_s between 1,200 fps and 2,500 fps, and rock with V_s higher than 2,500 fps. Thus, V_s of 2,500 fps can be defined as the lower bound value for PWR. Figure 2.5.4-228 shows V_s for PWR/MWR layers above El. 355 feet. Although Figure 2.5.4-228 indicates the presence of PWR up to about El. 380 feet under the Unit 2 reactor based on the 2,500 fps criterion, the average top of PWR/MWR for Unit 2 is around El. 375 feet. The corresponding top of PWR/MWR can be taken as El. 365 feet for Unit 3. Figure 2.5.4-230 shows best-fit design values applied to the measured PWR/ MWR V_s profiles in Figure 2.5.4-228.

For the natural soil profile (Layers I and II), the measured V_s profiles in Figure 2.5.4-228 were averaged vertically in 5 feet intervals to obtain the average, low, and high boundary profiles shown in Figure 2.5.4-229.

For the structural fill beneath the seismic Category II structures, there is no measured V_s, since the fill has not yet been constructed. To obtain a V_s profile range for the fill, the SPT N-value selected in subsection 2.5.4.2.5 for the fill (*i.e.*, N_{60} = 30 bpf) was used. Using the relationship between N_{60} and V_s developed by Seed & Idriss (Reference 241) a profile of V_s versus depth was obtained, as shown in Figure 2.5.4-238. The velocity values were adjusted for overburden pressure plus limited surcharge loading from locked-in stresses from compaction, and stresses from the structure itself. This profile was averaged vertically in 5-foot intervals to obtain the average V_s profile, also shown in Figure 2.5.4-238. The upper and lower bounds shown in this figure are 1.225 and 0.775 times the RN-12-029

mean value of V_s, respectively, which correspond to 1.5 and 0.60 times the shear modulus.

2.5.4.7.2 Variation of Shear Modulus and Damping with Strain

2.5.4.7.2.1 Shear Modulus

As noted in subsection 2.5.4.2.5, RCTS testing was performed on three representative samples of the saprolite and two samples of compacted structural fill. Shear modulus reduction curves (ratio of shear modulus to maximum shear modulus versus cyclic shear strain) were selected to run in the PSHAKE (Reference 240) analysis as discussed in subsection 2.5.4.7.3. These curves were then compared with the RCTS curves.

The shear modulus reduction curve for the Layer I and II soils (residuum and saprolite) was selected as the EPRI (Reference 230) curve for granular soils and low plasticity clays in the 20- to 50-foot depth range. This curve is illustrated on Figure 2.5.4-239. The results of the RCTS tests (normalized shear modulus (G/G_{max}) versus shear strain) from Figure 2.5.4-218 (Sheets 1 through 3) are superimposed on this curve in Figure 2.5.4-240 (Sheets 1 through 3). The RCTS results for the two silty sand (SM) saprolite samples (B-309 UD-2, and B-325 UD-4) show reasonable agreement with the EPRI curve for granular soils and low plasticity clays referenced above and used in the PSHAKE analysis (Figure 2.5.4-240, Sheets 1 and 2). The RCTS results for the elastic silt (MH) saprolite sample (B-208 UD-3) show less shear modulus degradation than the EPRI granular and low plasticity clay curve used in the PSHAKE analysis, but indicate close agreement with the EPRI curve for a plasticity index of 50 (Figure 2.5.4-240, Sheet 3). About 70% of the saprolite in the powerblock area consists of silty sand, with the remaining 30% made up of more plastic materials. Since plasticity of the saprolite is a function of the degree of in-place weathering of the soil, the silty sands and the more plastic materials are not found in well-defined layers, but occur in random zones throughout the saprolite layer. Thus, for the 25 feet of saprolite included in the PSHAKE analysis for Unit 2 (35 feet for Unit 3) it was considered reasonable to use the EPRI granular and low plasticity clay curve as representative of the layer.

The shear modulus reduction curve for the granular structural fill was also selected as the EPRI curve for granular soils and low plasticity clays in the 20- to 50-foot depth range, as shown on Figure 2.5.4-239. The results of the RCTS tests (normalized shear modulus (G/G_{max}) versus shear strain) from Figure 2.5.4-218 (Sheets 4 and 5) are superimposed on this curve in Figure 2.5.4-240 (Sheets 4 and 5). These results show good agreement with the EPRI curve, and so no additional PSHAKE runs were made using the RCTS shear modulus reduction curves.

The shear modulus values of the Layer IV (MWR) and Layer V (sound rock) are considered non-strain dependent. However, at some stage of weathering, rock becomes sufficiently decomposed to exhibit modulus reduction. The PWR layer is considered to fall into this sufficiently weathered state. Reference 246 developed a shear modulus versus strain curve for a soft rock material. This curve was selected for the PWR, and is shown on Figure 2.5.4-239. RN-12-029

2.5.4.7.2.2 Damping Ratio

Damping ratio versus cyclic shear strain curves were selected to run in the PSHAKE analysis as discussed in subsection 2.5.4.7.3. These curves were then compared with the RCTS curves once the test results were available.

The damping ratio versus shear strain curve for the Layer I and II soils (residuum and saprolite) was selected as the EPRI (Reference 230) curve for granular soils and low plasticity clays in the 20- to 50-foot depth range. This curve is illustrated on Figure 2.5.4-241. The results of the RCTS tests for damping ratio from Figure 2.5.4-218 (Sheets 1 through 3) are superimposed on this curve in Figure 2.5.4-240 (Sheets 1 through 3). The RCTS results for the two silty sand (SM) saprolite

samples (B-309 UD-2, and B-325 UD-4) show reasonable agreement with the EPRI curve for granular soils and low plasticity clays referenced above and used in the PSHAKE analysis (Figure 2.5.4-240, Sheets 1 and 2). The RCTS results for the elastic silt (MH) saprolite sample (B-208 UD-3) show a smaller damping ratio with increasing shear strain than the EPRI granular and low plasticity clay curve used in the PSHAKE analysis, but indicate reasonable agreement with the EPRI curve for a plasticity index of 50 (Figure 2.5.4-240, Sheet 3). About 70% of the saprolite in the powerblock area consists of silty sand, with the remaining 30% made up of more plastic materials. Since plasticity of the saprolite is a function of the degree of in-place weathering of the soil, the silty sands and the more plastic materials are not found in well-defined layers, but occur in random zones throughout the saprolite layer. Thus, for the 25 feet of saprolite included in the PSHAKE analysis for Unit 2 (35 feet for Unit 3) it was considered reasonable to use the EPRI granular and low plasticity clay curve as representative of the layer.

The damping ratio versus shear strain curve for the granular structural fill was also selected as the EPRI curve for granular soils and low plasticity clays in the 20- to 50-foot depth range, as shown on Figure 2.5.4-241. The results of the RCTS tests for damping ratio from Figure 2.5.4-218 (Sheets 4 and 5) are superimposed on this curve in Figure 2.5.4-240 (Sheets 4 and 5). These results show good agreement with the EPRI curve, and so no additional PSHAKE runs were made using the RCTS damping ratio versus shear strain curves.

The Layer IV (MWR) and Layer V (sound rock) are considered to have a damping ratio, but this ratio is non-strain dependent. A damping ratio of 1% was used for these materials. As with shear modulus, the damping ratio of PWR is considered to be strain dependent. Reference 246 developed a damping ratio versus strain curve for soft rock material. This curve was selected for the PWR, and is shown on Figure 2.5.4-241.

Note that damping ratios versus cyclic shear strains are frequently cut off at 15% damping ratio. The curves in Figure 2.5.4-241 are cut off at 15% when the damping ratio is limited to 15%.

2.5.4.7.3 Rock and Soil Column Amplification/Attenuation Analysis

The PSHAKE computer program (Reference 240) was used to compute the site dynamic responses for the soil profiles described in subsection 2.5.4.7.1. The analysis used the sound rock response spectrum presented in Figure 2.5.2-226. Although this site is considered a hard rock site with V_s of 9,200 fps directly beneath the nuclear island of each unit, Figure 2.5.4-226 shows minor variations in the V_s below the average top of sound rock elevation of 355 feet, especially in Unit 2. Thus, the sound rock response spectrum was input at various depths above and below El. 355 feet for the 60 randomized soil and rock profiles used in PSHAKE for each unit. For Unit 2, this ranged from about 15 feet above to about 45 feet below El. 355 feet, with the corresponding Unit 3 variation of about 15 feet above and 10 feet below El. 355 feet.

As described in subsection 2.5.2.5, the 1993 EPRI study, in addressing the variation in several crustal models considered for the CEUS (Reference 230), as well as uncertainty in Poisson's Ratio—used for converting the original compressional-wave velocity-based crustal models to shear-wave velocity models—suggests at least an uncertainty of several hundred feet/sec in the specification of the best estimate of 9,200 ft/s. Further, the 1993 EPRI study concluded that this variability in shear-wave velocity was not significant in ground motion modelling compared to other modeling factors.

The natural soil profile described in subsection 2.5.4.7.1 and shown in Figure 2.5.4-228 was randomized along with the shear modulus and damping ratio relationships with strain described in subsection 2.5.4.7.2, and used as input to PSHAKE. Figure 2.5.4-242 shows the acceleration versus depth profiles obtained from PSHAKE for both units. This acceleration at El. 400 feet is about 0.55g for Unit 2 and 0.42g for Unit 3. The maximum mean peak ground acceleration is used as input into the liquefaction analysis for the Units 2 and 3 site soils, described in subsection 2.5.4.8.

For the structural fill profile, the randomized profile described in subsection 2.5.4.7.1 along with the shear modulus and damping ratio relationships with strain described in subsection 2.5.4.7.2 were input into the PSHAKE analysis.

2.5.4.8 Liquefaction Potential

The AP1000 design has not been evaluated for a site where there is a liquefaction potential of the soil below the nuclear island.

Regulatory Guide 1.198 is used to address liquefaction.

Soil liquefaction is a process by which loose, saturated, granular deposits lose a significant portion of their shear strength due to pore pressure buildup resulting from cyclic loading, such as that caused by an earthquake. Soil liquefaction can occur, leading to foundation bearing failures and excessive settlements, when all of the following criteria are met:

- Design ground acceleration is high
- Soil is saturated (*i.e.*, close to or below the water table)
- Site soils are sands or silty sands in a loose or medium dense condition.

At the Units 2 and 3 site, the peak ground acceleration is high and portions of the soil are saturated where they are below the ground water table. However, much of the soil/rock at the site is not in a loose or medium dense condition. The PWR is a very dense decomposed rock matrix mixed with semi-hard rock fragments while the MWR has more than 50% by volume of sound rock interspersed with decomposed layers. Neither the PWR nor MWR has the potential to liquefy. The engineered structural fill is a dense well-graded sand compacted to at least 95% of the maximum dry density from the modified Proctor test (Reference 205). This fill does not have the potential to liquefy. The only site materials that need to be analyzed to determine their potential to liquefy under the design earthquake are the Layers I and II (residuum and saprolite) soils that are close to or below the ground water table.

The seismic Category I nuclear island is to be founded on sound rock or on concrete placed on sound rock. The seismic Category II portions of the annex building and turbine buildings are founded on structural fill on top of rock. As shown in Figures 2.5.4-220 and 2.5.4-223, the structural fill beneath the annex building and turbine building extends laterally well beyond the bottom of the structure so that the zone of loading influence from the foundation is entirely within the structural fill. Thus, even if the residuum and saprolite were to liquefy at the Units 2 and 3 site, such liquefaction would have no impact on the stability of the seismic Category I and II structures. In fact, referring to Figures 2.5.4-220 through 2.5.4-223, the residuum and saprolite are to be removed from below all of the structures adjacent to the nuclear island and replaced with concrete or structural fill, and thus liquefaction of the residuum and saprolite does not effect the stability of any of these structures.

RN-12-029
RN-12-029
RN-12-029
RN-12-012

Even though liquefaction of the residuum and saprolite do not impact the stability of the nuclear island or any of the surrounding structures, for completeness, this section examines the potential for these materials to liquefy. For the liquefaction analysis, the following information is needed:

- The locations of samples to be analyzed
- The material that makes up the residuum and saprolite to be analyzed
- The peak ground acceleration and corresponding earthquake moment magnitude

- The acceptable factor of safety against liquefaction.

2.5.4.8.1 Locations of Samples to be Analyzed

The residuum and saprolite is removed down to rock (Layers III, IV or V) at Units 2 and 3 as noted in subsection 2.5.4.5.1 and shown in the foundation excavation geometry in Figures 2.5.4-220 through 2.5.4-223. There is little relevance in analyzing for liquefaction potential these soils that are removed. However, 17 borings were identified that were located outside but in the near vicinity of the proposed power block excavations. Two similarly located CPTs were also identified. These borings and CPTs are listed in Table 2.5.4-216. Soils in these borings are analyzed for liquefaction. RN-12-029 RN-12-029

Liquefaction occurs due to pore pressure buildup between the soil particles, and thus is limited to soils that are close to or below the ground water table. The measured groundwater level contours are presented and discussed in subsection 2.4.12 with a typical set of contours given in Figure 2.5.4-237. The groundwater level at each of the borings and CPT locations in Table 2.5.4-216 was estimated from the groundwater contours, and the groundwater level used to determine whether the soil in the boring or CPT was saturated (and thus potentially liquefiable) was taken as 5 feet above this level. In the remainder of this section, this is referred to as the liquefaction groundwater level. The liquefaction groundwater level for each boring and CPT is shown in Table 2.5.4-216.

2.5.4.8.2 Material to be Analyzed

Saturated saprolite was encountered below the liquefaction groundwater table in only 4 of the 17 borings listed in Table 2.5.4-216–only fine-grained saprolite (more than 50% fines), PWR/MWR or sound rock was found below the liquefaction groundwater table in the remaining 13 borings. (Note, since the residuum is typically found above the saprolite, very limited residuum was identified below the liquefaction groundwater table in the 4 borings, and so only the term saprolite is used for the analysis.) The granular saprolite was silty sand, with generally more than 35% fines. The saprolitic silty sand in these borings is analyzed for liquefaction potential.

It should be noted that the fabric of saprolitic sand contrasts strongly with that of alluvial or marine deposited sand. The saprolitic sand can retain the foliation of the original rock and has interlocking of grains. Such foliation and interlocking is absent in alluvial or marine sand deposits, even though the grains can be quite angular. The fabric of saprolite is, therefore, not one of a transported soil but one of the parent rock material. The fabric is anisotropic, *i.e.*, it has strongly directional properties. The geometric interlocking of the grains and the lack of a void network that would allow reorientation of grains indicates that the saprolite should not typically liquefy.

Almost all of the materials identified by the 2 CPTs in Table 2.5.4-216 were clays or silts. Also, the liquefaction groundwater table was below the bottom of the CPT in one of the soundings. The equivalent N-value was above 25 bpf everywhere below the top 5 feet. Thus, for these 2 CPTs, there are no liquefiable soils.

2.5.4.8.3 Ground Acceleration and Earthquake Magnitude

The peak ground acceleration obtained from the PSHAKE analyses described in subsection 2.5.4.7.3 is 0.55g for Unit 2 and 0.42g for Unit 3. Only the 0.55g Unit 2 value was used for the liquefaction analysis. The corresponding earthquake magnitude is 7.2, as interpreted from Table 2.5.2-218.

2.5.4.8.4 Acceptable Factor of Safety Against Liquefaction

Regulatory Guide 1.198 suggests that factors of safety ≤1.1 against liquefaction are considered low, factors of safety between 1.1 to 1.4 are considered moderate, and factors of safety > 1.4 are

considered high. The Committee on Earthquake Engineering (Reference 233) states, "There is no general agreement on the appropriate margin (factor) of safety, primarily because the degree of conservatism thought desirable at this point depends upon the extent of the conservatism already introduced in assigning the design earthquake. If the design earthquake ground motion is regarded as reasonable, a safety factor of 1.33 to 1.35 is suggested as adequate. However, when the design ground motion is excessively conservative, engineers are content with a safety factor only slightly in excess of unity."

Based on the above opinions, a factor of safety of 1.25 is considered adequate for the saprolitic sands at the Units 2 and 3 site.

2.5.4.8.5 Liquefaction Analysis

The present state-of-the-practice considers an evaluation of data from SPT, CPT, and V_s measurements, with the method employing SPT measurements being the most well-developed, and well-recognized. Initially, a measure of the stress imparted to the soils by the ground motion is calculated, referred to as the cyclic stress ratio (CSR). Then, a measure of the resistance of soils to the ground motion is calculated, referred to as the cyclic resistance ratio (CRR). The factor of safety (FS) against liquefaction is then calculated as the ratio of the resisting stress, CRR to the driving stress, CSR. Details of the liquefaction methodology and the relationships for calculating CSR, CRR, FS, and other intermediate parameters such as the stress reduction coefficient (r_d), the magnitude scaling factor (MSF), the $K\sigma$ correction factor accounting for liquefaction resistance with increasing confining pressure, and other correction factors, can be found in Reference 252. Note that a MSF of 1.11 was used in the analyses, based on the magnitude 7.2 earthquake. A review of the results of liquefaction potential analyses using the available SPT, CPT, and V_s data discussed earlier follows.

2.5.4.8.5.1 Liquefaction Analysis Using SPT Measurements

Liquefaction analysis of each sample of saprolitic silty sand obtained by SPT sampling in the 17 borings in Table 2.5.4-216 at or below the liquefaction groundwater table was performed to determine the factor of safety against liquefaction. The analysis conservatively ignored the age and mineralogy/ fabric effects of the saprolite. Fine-grained samples and/or samples above the groundwater table were considered non-susceptible to liquefaction.

The analysis followed the method proposed by Youd et al. (Reference 252), based on the evolution of the Seed and Idriss "Simplified Procedure" over the past 25 years. Overburden pressure and hammer ETR corrections were applied to the measured N-values. The CRR computed from the corrected N-values used the 35% fines curve. The $K\sigma$ factor for high overburden pressures was incorporated into the analysis, using a relative density of 40% to 80%.

Using the peak ground acceleration, the analysis of the SPT results gave factor of safety values against liquefaction greater than 1.25 for those samples that were liquefiable, except for three samples, where the computed factors of safety were less than 1.25.

2.5.4.8.5.2 Liquefaction Analysis Using CPT Measurements

No liquefiable soils were identified in the selected CPTs, as noted in subsection 2.5.4.8.2.

2.5.4.8.5.3 Liquefaction Analysis using Shear Wave Velocity

No V_s measurements were made in the borings and CPTs in Table 2.5.4-216. To use V_s measurements in the analysis, the average values of V_s shown in Figure 2.5.4-229 (which include all the V_s measurements performed in the residuum and saprolite in the PBA) were analyzed. The measured V_s values were corrected for overburden pressure using the method outlined in Youd et al.

(Reference 252). The corrected values all fell into the "No Liquefaction" zone on Figure 9 of Reference 252.

2.5.4.8.6 Conclusions About Liquefaction

Only the saprolitic sand present onsite falls into the gradation and relative density categories where liquefaction is considered possible.

Any liquefaction of the saprolitic sand will not impact the stability of any Units 2 and 3 seismic Category I and II structures since the zone of loading influence of these structures does not reach the saprolitic sands.

The conclusions from the foregoing sections on the analysis of liquefaction potential of the saprolitic sand are as follows:

- The liquefaction analysis of the SPT measurements in 17 borings along and close to the perimeter of the area to be excavated gave factor of safety values against liquefaction greater than 1.25 for those samples that were liquefiable, except for three samples, where the factor of safety was less than 1.25. None of the soils in the two CPTs in these areas was potentially liquefiable.
- The liquefaction analysis of the average Unit 2 and Unit 3 V_s measurements indicated the soil to be non-liquefiable.
- The analysis conservatively ignored the age and mineralogy/fabric effects of the saprolite.

Based on the above analysis results, it can be concluded that a small percentage of the saprolitic sands has a possible potential for liquefaction based on the design seismic parameters. The liquefaction analysis did not take into account the beneficial effects of age, fabric, and mineralogy. However, any liquefaction of the saprolitic sands will not impact the stability of any seismic Category I or II structure, or any of the other structures that surround the nuclear island since the zone of influence of these structures does not reach the saprolitic sands.

2.5.4.9 Earthquake Design Basis

The horizontal ground motion response spectrum (GMRS) was developed from the horizontal uniform hazard response spectrum (UHRS) using the approach described in ASCE/SEI Standard 43-05 and Regulatory Guide 1.208. The vertical GMRS was developed from the vertical UHRS.

The ASCE/SEI Standard 43-05 approach defines the GMRS using the site-specific UHRS, which is defined for Seismic Design Category SDC-5 at a mean 10^{-4} annual frequency of exceedance.

The GMRS is derived, and presented in detail, in subsection 2.5.2.6.

2.5.4.10 Static Stability

The seismic Category I nuclear island (on a common basemat) for each unit is directly founded on top of Layer V (sound rock). If Layer III (PWR) and/or Layer IV (MWR) are encountered at foundation subgrade level, they are removed. Concrete is placed on the sound rock where required to bring subgrade up to the bottom of the foundation. The seismic Category II portions of the annex building and the turbine building for each unit are supported on structural fill above rock (Layers III, IV, or V). The other major structures that surround the nuclear island (turbine and radwaste buildings) are also supported on concrete or structural fill above rock as shown on Figures 2.5.4-220 through 2.5.4-223. RN-12-029 RN-12-012

Figures 2.5.4-220 through 2.5.4-223 show the subsurface profiles in the east-west and north-south directions along with the cross-sections of the major power block structures. Note that (1) Layer I and II (residuum/saprolite) soils in the overall PBA are removed and replaced with structural or concrete fill, and (2) the thickness of structural fill and concrete fill material beneath the foundation of the turbine building varies due to the different depths of the parts of the buildings as shown on Figures 2.5.4-221 and 2.5.4-223. Table 2.5.4-217 shows the bottom of foundation elevations for the seismic Category I and II structures, along with the non-seismic portions of the turbine and radwaste building. Since the plan dimensions of some of the buildings are irregular, two cases, reflecting the effects of minimum and maximum dimensions, are considered in the bearing capacity and settlement analyses, as shown in Table 2.5.4-217. RN-12-012 RN-12-029

2.5.4.10.1 Bearing Capacity

The maximum bearing reaction determined from the 3D SASSI analyses described in Appendix 3G is less than 35,000 lb/ft^2 under all combined loads, including the safe shutdown earthquake. The maximum dynamic bearing demand of 35 ksf occurs under the west edge of the shield building and is primarily due to the response to the east-west component of the earthquake. The east edge of the nuclear island lifts off the soil. The Combined License applicant will verify that the site-specific allowable soil bearing capacities for static and dynamic loads at the site will exceed the static and dynamic bearing demand given in Table 2.0-201.

The evaluation of the allowable capacity of the soil is based on the properties of the underlying materials, including appropriate laboratory test data to evaluate strength, and considering local site effects, such as fracture spacing, variability in properties, and evidence of shear zones. The allowable bearing capacity should provide a factor of safety appropriate for the design load combination, including safe shutdown earthquake loads.

If the shear wave velocity or the allowable bearing capacity is outside the range evaluated for AP1000 design certification, a site-specific evaluation can be performed using the AP1000 basemat model and methodology described in subsection 3.8.5. The safe shutdown earthquake loads are those from the AP1000 analyses described therein. Alternatively, bearing pressures may be determined from a site-specific analysis using site-specific inputs as described in subsection 2.5.2.3. For the site to be acceptable, the bearing pressures from the site-specific analyses, including static and dynamic loads, need to be less than the capacity of each portion of the basemat.

2.5.4.10.1.1 Bearing Capacity of Rock

The allowable bearing capacity values for each bedrock layer (III, IV and V) are given in Table 2.5.4-218. These values are the same as in Table 2.5-22 of the Unit 1 UFSAR which recommends an allowable rock bearing capacity of 200 ksf for Layer V (sound rock), 100 ksf for Layer IV (MWR) and 40 ksf for Layer III (PWR) (Reference 249). It should be noted that although the 40 ksf allowable bearing capacity for PWR is greater than the maximum static bearing pressure from the nuclear island basemat, the nuclear island is not founded directly on the PWR or MWR. If excavation for this foundation reveals any weathered or fractured zones at foundation level, such zones are overexcavated and replaced with concrete above sound rock.

Several building codes in Reference 227 give an allowable bearing capacity of rock of not more than 20% of its ultimate crushing strength (compressive strength). In that case, for Layer V (sound rock), 20% of 25 ksi (compressive strength) gives 5 ksi (=720 ksf). However, the concrete placed on sound rock, if any, is expected to have a compressive strength of 5 ksi. Then, 20% of 5 ksi gives 1 ksi (=144 ksf). Note that using 20% of ultimate crushing strength for concrete is very conservative due to the uniform properties and homogeneous nature of concrete. Between the recommended allowable sound rock bearing capacity of 200 ksf and a conservatively assumed allowable bearing capacity of

144 ksf for concrete, it is reasonable and conservative to use an allowable bearing capacity of 160 ksf for the nuclear island at Units 2 and 3.

2.5.4.10.1.2 Bearing Capacity of Soil

For granular soils such as Layers I and II (residuum/saprolite) and the engineered structural fill, bearing capacity is based on Terzaghi's bearing capacity equations modified by Vesic (Reference 250). The ultimate (gross) bearing capacity of a footing (q_{ult}) supported on homogeneous soils can be estimated by (Reference 250):

$$q_{ult} = cN_c\zeta_c + \gamma' D_f N_q \zeta_q + 0.5\gamma' B N_\gamma \zeta_\gamma$$

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

$\gamma' D_f$ = 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 as defined in Reference 250, and

ζ_c, ζ_q, and ζ_γ are shape factors as defined in Reference 250.

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 depth below grade, these equations can give very large bearing capacity values, even when a factor of safety of 3.0 is included for the allowable bearing value. In such situations, settlement, discussed in subsection 2.5.4.10.2, normally governs.

2.5.4.10.1.3 Allowable Bearing Capacity of Structures

Table 2.5.4-219 gives the estimated allowable bearing capacity for the seismic Category I nuclear island, seismic Category II portions of the annex building and turbine building, and major nonseismic structures (turbine and radwaste buildings), based on the materials underlying the structures shown in Figures 2.5.4-220 through 2.5.4-223. Because of the irregular shape of these structures, minimum and maximum dimensions are considered in the theoretical allowable bearing capacity analyses. For the nuclear island, the value on Table 2.5.4-219 exceeds the required allowable static and dynamic bearing capacities given in Table 2-1 of the AP1000 DCD.

RN-12-029

RN-13-024

Layers I and II (residuum/saprolite) can be used to support relatively lightly loaded, nonsettlement sensitive structures that are not classified as seismic Category I or II (*e.g.*, switchyard or cooling tower structures). For these buildings, a generic analysis was performed for various footing sizes, with a minimum width of at least 5 feet and a length of 5 to 50 feet. The allowable bearing capacity value is limited to 4.0 ksf because of settlement considerations. As noted in subsection 2.5.4.10.2, settlement considerations usually dominate when the saprolite is used for supporting foundations, and the actual allowable bearing capacity may be less than 4.0 ksf, especially for larger foundations.

Groundwater table is conservatively assumed to be at El. 400 feet in these calculations. There are hydrostatic uplift forces on buried structures. All of the underground seismic Category I and II structures have applied foundation loads well in excess of hydrostatic uplift pressures, and so there are no net uplift forces. However, such forces could 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 would be analyzed to confirm satisfactory resistance to the uplift forces. The normal factor of safety of 3 against soil failure is used in this analysis.

2.5.4.10.2 Settlement Analysis

The AP1000 does not rely on structures, systems, or components located outside the nuclear island to provide safety-related functions. Differential settlement between the nuclear island foundation and the foundations of adjacent buildings does not have an adverse effect on the safety-related functions of structures, systems, and components. Differential settlement under the nuclear island foundation could cause the basemat and buildings to tilt. Much of this settlement occurs during civil construction prior to final installation of the equipment. Differential settlement of a few inches across the width of the nuclear island would not have an adverse effect on the safety-related functions of structures, systems, and components. Table 2.5-1 provides guidance to the Combined License applicant on predictions of absolute and differential settlement that are acceptable without further evaluation. The predicted settlements will cover the periods before construction begins through the construction phase, and for the subsequent plant operating period or otherwise justified. The predicted settlements will be based on conservative assumptions of soil properties. If the predicted settlements exceed the limits of Table 2.5-1, a detailed evaluation and construction plan will be described by the Combined License applicant. During construction and plant operation at a soil site, settlements would be measured and compared to the predicted settlement values and any exceedances would require additional investigation.

Alternatives for the additional evaluations are provided as follows:

1. Evaluate the impact of the elevated estimated settlement values on the critical components of the AP1000 including, but not limited to, piping spanning between the nuclear island and the adjacent structures, the equipment support pads, the construction gap between the nuclear island and adjacent buildings, and the stresses on the basemat (along with influences to the underlying soil).

2. Submit a construction sequence to control the predicted settlement behavior. A revised sequence should follow the specific schedule to distribute construction loads as necessary in order to obtain acceptable values. Depending on soil conditions, a significant amount of the settlement could occur during construction and can be controlled through the construction sequence.

3. Provide a uniform excavation and engineered backfill to manage static building rotation and differential settlement between the nuclear island and adjacent structures.

4. Implement an active settlement monitoring system throughout the entire construction sequence as well as a long-term (plant operation) plan. By monitoring the settlement throughout construction, the Combined License applicant will be able to modify the construction sequences of adjacent buildings to conform to the site's settlement characteristics and minimize differential settlement. For soil sites, the potential heave or rebound of the excavation bottom, the effect of dewatering, and the effect of foundation loading during construction should be monitored by the Combined License applicant. The monitoring system shall consist of three primary elements as follows:

    - Piezometers to measure pore pressures in a soil layer prone to consolidation type settlement. Vibrating wire piezometers are preferred for this purpose because they are adequately sensitive and responsive and easily record positive and negative changes on a real time basis.

    - Settlement monuments placed directly on concrete, preferably on the mudmat for early construction monitoring and on the corners of the structures at grade once the mudmat monuments have been covered by backfill to be used for long-term monitoring. Monuments at grade are to be accessible with conventional surveying equipment.

- Settlement telltales if monuments are not practical or if fills are used over consolidation type soils and it is necessary to monitor settlement of the consolidation type in-situ soils independent of the consolidation of the engineered fill soil. Most soil sites will not need this particular form of monitoring.

Develop graphs and plots of the field measurements to:

- Show movement (settlement or heave) versus time.
- Estimate construction loads versus time.
- Measure ground water levels from the dewatering activities versus time.

This data should be maintained during construction and post-construction as needed depending on the field measurement results.

For the large mat foundations that support the major power plant structures, general considerations based on geotechnical experience indicate that if total foundation settlement is limited to 2 inches, with differential settlement limited to 3/4 inch (Reference 238), the performance of the structure should not be impacted. For individual footings that support smaller plant components, the corresponding value of total settlement is 1 inch, while the differential settlement is 1/2 inch.

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:

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

where,

δ = settlement

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

p_i = vertical applied pressure at center of layer i

h_i = thickness of layer i

E_i = elastic modulus of layer i (high strain value used)

The stress distribution below rectangular foundations is based on a Boussinesq-type distribution for flexible foundations (Reference 239). 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% of the applied foundation pressure. The Boussinesq-type vertical pressure under a rectangular footing (σ_z) is as follows (Reference 239):

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

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 following the preceding relationships and using soil and rock properties given in Table 2.5.4-209. These estimates were made for the seismic Category I nuclear island, seismic Category II annex building, and major nonseismic structures (turbine and radwaste buildings), and are presented in Table 2.5.4-220. The applied pressure used in the settlement computation for the nuclear island foundation is from Table 2-1 of the AP1000 DCD. The 6 ksf applied pressure for the other major structures is a best estimate, and is expected to be conservative.

As would be expected, the anticipated settlements under the nuclear islands are negligible since they are supported on Layer V (sound rock). Similarly, settlements of structures sitting on the dense to very dense structural or concrete fill underlain by rock formation are modest in light of the large applied pressures. The anticipated average settlements under the annex, and radwaste buildings supported on structural fill are on the order of 1.5 to 2.5 inches. The anticipated average settlement for the turbine building supported partially on structural fill and partially on concrete fill is less than 1 inch. Note that these settlements mainly occur during construction. Differential settlements within the structure should be less than 50% of the total settlement, except for the turbine building where the basement of the structure is founded on concrete over bedrock and other parts are on relatively thick structural fill as shown on Figures 2.5.4-221 and 2.5.4-223. In such a case, the differential settlement within the structure can approach the total settlement value. Since the turbine building is such a large structure, the angular distortion is within acceptable limits.

RN-12-029
RN-13-024
RN-12-012
RN-12-029

2.5.4.10.3 Earth Pressures

Static and seismic lateral earth pressures are addressed for plant underground walls with a height of 40 feet (the typical underground wall height of the nuclear island in the AP1000 design). Both active and at-rest cases are included for the structural fill case. The earth pressure coefficients are Rankine values, assuming level backfill and a zero friction angle between the soil and the wall. Hydrostatic pressures are based on assuming the groundwater table is at El. 380 feet, which is the anticipated maximum level. The area-wide surcharge pressures of 500 pounds per square foot (psf) and 2,500 psf are conservatively used under active and at-rest conditions, respectively. Lateral pressures due to compaction are conservatively included in the pressure diagrams. Note that subsection 2.5.4.5.3.1 states that compaction is performed with a heavy steel-drummed vibratory roller, except within 5 ft of a structure wall, where smaller compaction equipment is used to minimize excess pressures against the wall. Fill placement and compaction control procedures are addressed in a technical specification.

For the active lateral earth pressure case, earthquake-induced horizontal ground accelerations are addressed by the application of $k_h \cdot g$. Vertical ground accelerations ($k_v \cdot g$) are considered negligible and are ignored (Reference 242). The peak horizontal ground acceleration of 0.55g (obtained for in-situ soils) is conservatively used for developing the seismic active earth pressure diagrams (i.e., k_h=0.55), even though the peak horizontal ground acceleration obtained for the structural fill is 0.38g. As recommended in Reference 257 (ASCE 4-98, Section 3.5.3.3), the Mononobe-Okabe method (Reference 242) is used to establish seismic lateral active earth pressures, provided that wall displacements required to develop the active earth pressure are tolerated without loss of wall function.

Recognizing the limitation of the Reference 242 method for design of building walls, Ostadan (Reference 237) developed a method to compute seismic soil pressure that focuses on building walls rather than soil retaining walls. This method specifically considers the following: (1) the movement of the walls is limited due to the presence of the floor diaphragms and the walls are considered non-yielding; (2) the frequency content of the design motion is fully considered; and (3) appropriate soil properties, in terms of soil V_s and damping, are included in the analysis. The method is flexible to allow for consideration of soil nonlinear effect where soil nonlinearity is expected to be significant.

Another approach to estimating the seismic lateral earth pressure against buried, non-yielding walls is the elastic solution recommended in ASCE 4-98 (Reference 257). This solution contains a nomograph in which a dimensionless normal stress diagram at 1.0g horizontal earthquake acceleration is displayed for a normalized depth at a given Poisson's ratio. Following the recommendation in ASCE 4-98, this elastic solution is used to estimate the seismic lateral at-rest pressures against buried structure walls. The peak horizontal ground acceleration of 0.38g (obtained for the structural fill) is used for developing the seismic at-rest earth pressure diagram. This is the seismic lateral earth pressure shown in Figure 2.5.4-244.

For the compaction-induced pressures under at-rest conditions, the methodology described in References 254 and 255 is used. To be on the conservative side, the highest available line load (q = 800 lb/in) is selected, which produces the highest lateral pressure. This is the compaction-induced pressure under at-rest conditions shown in Figure 2.5.4-244. As noted earlier, this is conservative since subsection 2.5.4.5.3.1 states that compaction is performed with a heavy steel-drummed vibratory roller, except within 5 ft of a structure wall, where smaller compaction equipment is used to minimize excess pressures against the wall.

Reference 256 contains a procedure to evaluate the compaction-induced lateral earth pressures for active conditions; i.e., lateral earth pressures against walls that are allowed to rotate away from the backfill. However, in the proposed nuclear island area, no permanent retaining wall type structures are planned; therefore, the compaction-induced lateral earth pressures for active conditions are not presented here.

Figures 2.5.4-220 through 2.5.4-223 show structural or concrete fill below and around the major structures. In all cases, lateral pressures are from the structural fill; the concrete fill, in-situ saprolite and saprolite common fill have no impact on the lateral earth pressures. The structural fill properties used in the calculation of lateral earth pressures are from Table 2.5.4-209. RN-12-029

Lateral earth pressure diagrams for the active and at-rest cases are given in Figures 2.5.4-243 and 2.5.4-244, respectively. Note that these lateral pressures are best-estimate pressures with a factor of safety of 1.0. Appropriate safety factors are 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.

2.5.4.11 Design Criteria

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.

subsection 2.5.4.8 specifies that the acceptable factor of safety against liquefaction of site soils should be ≥ 1.25.

Bearing capacity and settlement criteria are presented in subsection 2.5.4.10. Table 2.5.4-219 provides allowable bearing capacity values for the seismic Category I and II structures, and other major structures. 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. For soils, this factor of safety can be reduced to 2.25 when dynamic or transient loading conditions apply. Table 2.5.4-220 shows estimated structure settlements under assumed foundation loads. Generally, if total and differential settlements are limited to 2 inches and 3/4 inches, respectively, for mat foundations, and 1 inch and 1/2 inch, respectively, for footings, settlement will not impact foundation performance.

subsection 2.5.4.10 also discusses factors of safety related to lateral earth pressures. The lateral pressures shown in Figures 2.5.4-243 and 2.5.4-244 are best estimate values and thus have a factor of safety of 1.0. 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.

No pile or pier foundations are planned for the seismic Category I and II structures for Units 2 and 3. There may be situations where such foundations are used for other PBA structures. For axial pile and pier design capacity, a factor of safety of 3 is used for the end bearing component, and a factor of safety of 2 is used for skin friction. For lateral loading, the maximum allowable lateral load is taken as half of the load that produces 1 inch of lateral movement on the head of the pile.

Subsection 2.5.5 concluded that there are no slopes that could impact plant safety if they failed. Thus, required factors of safety against failure were not specified.

2.5.4.12 Techniques to Improve Subsurface Conditions

For Units 2 and 3, any residuum or saprolite beneath or within the zone of influence of seismic Category I or II structures is removed and replaced with compacted structural fill.

Zones of weathered or fractured rock encountered immediately beneath the nuclear island basemat are removed and replaced with concrete.

To establish the foundation bearing level for the Nuclear Island (NI), fill concrete will be used beneath the footprint of the NI basemat, and extending a minimum of one foot outward as necessary to mitigate edge stresses. To serve as a construction aid, this fill concrete is typically extended two or more feet outward. The excavation around the NI and beneath other major power block structures will be backfilled with compacted granular structural fill or concrete fill. The relative concrete and structural fill locations are shown on Figures 2.5.4-220 through 2.5.4-223. Concrete fill will be used between the bottom of the NI foundation and the finish grade on sound rock. Based on the top of Layer V (Sound Rock) contours (Figure 2.5.4-202), the top of sound rock was estimated at El. 360 +/- 5 ft beneath the NI at Units 2 and 3, but was interpreted to be approximately 17 ft lower (El. 343 ft) at the northeast corner of the Unit 2 NI and approximately 12 ft higher (El. 372 ft) beneath the southern part of the Unit 2 NI. The NI areas will be excavated in sound rock to approximately El. 357 ft, where required, to allow a minimum 3 ft thickness of fill concrete and mud mat beneath the NI basemats.The excavation for the Unit 2 NI was extended to approximately El. 333 ft in a localized area to reach the top of sound rock near the northeast corner of the NI. RN-12-037 RN-12-012 RN-12-037 RN-12-029 RN-12-029

American Concrete Institute (ACI) defines mass concrete as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking." The definition is intentionally vague because many factors, including the concrete mix design, the dimensions, the type of the placement, and the curing methods, affect whether or not cracking will occur. ACI 207, "Mass Concrete," prepared by ACI Committee 207, is a guide for the design and construction of mass concrete. Typically, there are two common concerns associated with thermal cracks in mass concrete. They are: (1) the maximum temperature inside a concrete pour and (2) the maximum temperature difference between the hottest spot and the surface of a concrete pour. It is a common practice to limit the least dimension of each concrete pour so that the temperature and temperature difference of the pour can stay within their respective limits. RN-12-045 RN-12-045

The northeastern corner under the Unit 2 basemat will require up to approximately 27 feet of fill concrete. According to the definition of mass concrete in ACI 207, "Mass Concrete", the fill concrete under the NI of Unit 2 is a mass concrete. A thermal control plan considering the geometry of Unit 2 fill concrete, the proposed 5,000 psi strength, total volume of fill concrete placement, and rate of concrete production, will be prepared. The thermal control plan, based on the ACI 207 guidelines for preventing thermal cracking in concrete, will have the following elements: RN-12-029 RN-12-045

- Use well-graded aggregate and Type I and/or II cement in the concrete mix.
- Because of its relatively high strength specification, the fill concrete will likely have a high content of Portland cement substitutes, such as Class F flyash and/or slag, to minimize the heat of hydration.
- In anticipation of variations in elevation in sound rock surface, the minimum thickness of fill concrete will be set at 3 feet, which includes the 6-inch layer of mud mat.

- Even with the heat of hydration in the design mix minimized, it may still require the concrete to be placed in relatively thin lifts to avoid cracking. Thus, the maximum thickness of each concrete lift will be set at about five feet.
- Concrete will be placed using a step technique to minimize the live face of concrete, thus minimizing the chance for cold joints.
- Exposed surfaces of each concrete lift will be insulated, if required.
- When another lift is required on top of an existing lift, the top lift will be placed only when acceptable within the thermal control plan. RN-12-045
- Concrete placing temperature will be controlled as necessary by use of ice, chilled water, shading aggregate piles, spraying coarse aggregate for evaporative cooling, and scheduling placements (such as at night) to take advantage of coolest temperatures.
- Planned vertical joints in each concrete lift will be properly treated.
- Planned horizontal joints between two concrete lifts will be properly treated.

2.5.4.13 Subsurface Instrumentation

Since the nuclear island will be founded on sound bedrock, or on concrete placed on sound bedrock, no settlement monitoring of the nuclear island is required. There will be settlement monitoring of nonsafety-related structures that are not supported on bedrock, or on concrete placed on bedrock.

2.5.4.14 Waterproofing System

The waterproofing system used for the foundation mat (mudmat) and below grade exterior walls exposed to flood and groundwater under seismic Category I structures is addressed in Subsections 3.4.1.1.1.1 and 3.8.5.1.

2.5.4.15 References

201. American Concrete Institute, *Manual of Concrete Practice*, Part 1, Materials and General Properties of Concrete, 1994.

202. American Petroleum Institute, *Cathodic Protection of Aboveground Petroleum Storage Tanks,* API Recommended Practice No. 651, Washington, D.C., 1991.

203. ASTM C 150-07, ASTM International, *Standard Specification for Portland Cement*, West Conshohocken, Pennsylvania, 2007.

204. ASTM C 595-07, ASTM International, *Standard Specifications for Blended Hydraulic Cements*, West Conshohocken, Pennsylvania, 2007.

205. ASTM D 1557-02, ASTM International, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3))*, West Conshohocken, Pennsylvania, 2003.

206. ASTM D 1586-99, ASTM International, *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soil*, West Conshohocken, Pennsylvania, 1999.

207. ASTM D 1587-00, ASTM International, *Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes*, West Conshohocken, Pennsylvania, 2000.

208. ASTM D 2488-06, ASTM International, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*, West Conshohocken, Pennsylvania, 2006.

209. ASTM D 2113-06, ASTM International, *Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation*, West Conshohocken, Pennsylvania, 2006.

210. ASTM D 2487-06, ASTM International, *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)*, West Conshohocken, Pennsylvania, 2006.

211. ASTM D 3080-04, ASTM International, *Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions*, West Conshohocken, Pennsylvania, 2004.

212. ASTM D 4044-96, ASTM International, *Standard Test Method (Field Procedure) for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers*, West Conshohocken, Pennsylvania, 2002.

213. ASTM D 4220-95, ASTM International, *Standard Practice for Preserving and Transporting Soil Samples*, West Conshohocken, Pennsylvania, 1995.

214. ASTM D 4630-96, ASTM International, *Standard Test Method for Determining Transmissivity and Storage Coefficient of Low-Permeability Rocks by In Situ Measurements Using the Constant Head Injection Test*, West Conshohocken, Pennsylvania, 2002.

215. ASTM D 4633-05, ASTM International, *Standard Test Method for Energy Measurement for Dynamic Penetrometers*, West Conshohocken, Pennsylvania, 2005.

216. ASTM D 4767-04, ASTM International, *Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils*, West Conshohocken, Pennsylvania, 2004.

217. Not used.

218. ASTM D 5753-05, ASTM International, *Standard Guide for Planning and Conducting Borehole Geophysical Logging*, West Conshohocken, Pennsylvania, 2005.

219. ASTM D 5778-95, ASTM International, *Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils*, West Conshohocken, Pennsylvania, 2000.

220. ASTM D 6032-02, ASTM International, *Standard Test Method for Determining Rock Quality Designation (RQD) of Rock Core*, West Conshohocken, Pennsylvania, 2006.

221. ASTM D 6167-97, ASTM International, *Standard Guide for Conducting Borehole Geophysical Logging: Mechanical Caliper*, West Conshohocken, Pennsylvania, 2004.

222. ASTM D 6274-98, ASTM International, *Standard Guide for Conducting Borehole Geophysical Logging—Gamma*, West Conshohocken, Pennsylvania, 2004.

223. ASTM G 57-06, ASTM International, *Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method*, West Conshohocken, Pennsylvania, 2006.

224. Bowles, J. E., *Foundation Analysis and Design,* 5th Edition, McGraw-Hill, NY, 1997.

225. Caterpillar Inc., *Caterpillar Performance Handbook,* 24th Edition, Peoria, Illinois, October 1993.

226. Coduto, D. P., *Foundation Design: Principles and Practices,* 2nd Edition, Prentice-Hall, New Jersey, 2001.

227. D'Appolonia, E., Ellison, R. D., and D'Appolonia, D. J., *Drilled Piers*, in Foundation Engineering Handbook, Winterkorn, H. F. and Fang, H-Y, editors, Van Nostrand Reinhold Co., New York, 1975.

228. Davie, J. R. and Lewis, M. R., *Settlement of Two Tall Chimney Foundations,* Proc. 2nd International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, pp. 1309-1313, 1988.

229. Davie, J. R., Young, L. W., and Stashik, G. A., *Soil Electrical Resistivity, an Essential Parameter for Power Plant Design,* Proceedings of the American Power Conference, Chicago, Illinois, April 1996.

230. Electric Power Research Institute (EPRI), *Guidelines for Determining Design Basis Ground Motions,* EPRI Report No. TR-102293, Volumes 1-5, Appendix 7.A, *Modeling of Dynamic Soil Properties,* Palo Alto, California, 1993.

231. International Code Council, *International Building Code*, 2003.

232. MACTEC, *SCE&G COL Geotechnical Data Report, Results of Geotechnical Exploration and Testing,* Rev. 2, V.C. Summer Nuclear Plant, for Bechtel, February 2007.

233. National Research Council, *Liquefaction of Soils During Earthquakes*, Committee on Earthquake Engineering, National Academy Press, Washington, D.C., 1985.

234. Naval Facilities Engineering Command, *Foundations and Earth Structures*, Design Manual 7.02, Alexandria, Virginia, 1986.

235. NUREG-0800, *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants,* Section 2.5.4, Revision 3, March 2007.

236. Occupational Safety and Health Administration (OSHA), 29 CFR Part 1926, Safety and Health Regulations for Construction, 2000.

237. Ostadan, F., *Seismic Soil Pressure for Building Walls—An Updated Approach,* Proceedings, 11th International Conference on Soil Dynamics and Earthquake Engineering and 3rd International Conference on Earthquake Geotechnical Engineering, University of California, Berkeley, 2004.

238. Peck, R. B., Hanson, W. E., and Thornburn, T. H., *Foundation Engineering*, Second Edition, John Wiley & Sons, Inc., New York, 1974.

239. Poulos, H. G. and Davis, E. H., *Elastic Solutions for Soil and Rock Mechanics*, John Wiley, New York, 1974.

240. PSHAKE; *A Computer Program for Conducting Equivalent Linear Seismic Response Analyses of Horizontally Layered Soil Deposits Based on Random Vibration Theory*, Bechtel Standard Computer Program GE315, Version 1.0, December 2006.

241. Seed, H. B. and Idriss, I. M., *Soil Moduli and Damping Factors for Dynamic Response Analyses,* Report No. EERC 70-10, University of California, Berkeley, December 1970.

242. Seed, H. B. and Whitman, R. V., *Design of Earth Retaining Structures for Dynamic Loads,* Proc. Specialty Conference on Lateral Stresses in the Ground and Design of Earth-Retaining Structures, ASCE, NY, pp. 103-147, 1990.

243. Senapathy, H., Clemente, J. L. M., and Davie, J. R., *Estimating Dynamic Shear Modulus in Cohesive Soils,* 15th International Conference on Soil Mechanics and Geotechnical Engineering, Istanbul, Turkey, August 2001.

244. STS Consultants, Inc., *Reinforced Soil Structures, Vol. 1, Design and Construction Guidelines,* FHWA Report No. FHWA-RD-89-043, McLean, Virginia, 1990.

245. Subpart 2.20 of ASME NQA-1,*Quality Assurance Program for Nuclear Facilities,* 1994.

246. Sun, J. I., Golesorkhi, R., and Seed, H. B., *Dynamic Moduli and Damping Ratios for Cohesive Soils,* Report No. UCB/ EERC-88/15, University of California, Berkeley, August 1988.

247. Terzaghi, K., *Evaluation of Coefficients of Subgrade Reactions,* Geotechnique, Volume 5, 1955.

248. U.S. Army Corps of Engineers, *Rock Testing Handbook*, USAEWES, 1993.

249. V. C. Summer Unit 1 UFSAR, accessed 2006.

250. Vesic, A. S., *Bearing Capacity of Shallow Foundations,* in Foundation Engineering Handbook, H. F. Winterkorn and Fang, H-Y, editors, Van Nostrand Reinhold Co., New York, 1975.

251. Winterkorn, H. F. and Fang, H. Y., *Soil Technology and Engineering Properties of Soils,* in Foundation Engineering Handbook, Winterkorn, H. F. and Fang, H.Y., editors, Van Nostrand Reinhold Co., New York, New York, 1975.

252. Youd et al., *Liquefaction Resistance of Soils: Summary of Report from the 1996 NCEER and 1998 NCEER.NSF Workshops on Evaluation of Liquefaction of Soils,* ASCE Journal of Geotechnical and Environmental Engineering, Volume 127, No. 10, October 2001.

253. American Society of Mechanical Engineers, ASME NQA-1-1994, *Quality Assurance Requirements for Nuclear Facility Applications*, New York, NY, 1994.

254. Duncan, J.M., G.W. Williams, A.L. Sehn and R.B. Seed. “Estimation of Earth Pressures due to Compaction,” *Journal of Geotechnical Engineering*, ASCE, New York, NY, 117(12):1833-1847, 1991.

255. Duncan, J.M., G.W. Williams. A.L. Sehn, and R.B. Seed. “Closure of Estimation of Earth Pressures due to Compaction,” *Journal of Geotechnical Engineering*, ASCE, New York, NY, 119(7):1172-1177, 1993.

256. Department of Navy, *Foundations and Earth Structures, Design Manual 7.2,* Alexandria, VA, pp. 7.2-76-77, 1982.

257. ASCE 4-98. *Seismic Analysis of Safety-Related Nuclear Structures and Commentary*, ASCE, Reston, VA, 2000.

258. South Carolina Department of Transportation (SCOOT), 2007, Standard Specifications for Highway Construction. RN-12-031

259. Shaw Nuclear, 2012, Calculation VSG-G1-SC1-016 Revision 1, Evaluation of Alternative Materials for Power Block Fill.

Table 2.5.4-201 (Sheet 1 of 2)
Termination Elevations of Soil Strata

	Elevation of Top of					
BH-200s	**Ground Surface**	**Residuum**	**Saprolite**	**PWR**	**MWR**	**Sound Rock**
Maximum:	428.4	428.4	421.6	396.1	391.2	383.5
Minimum:	374.4	374.7	355.9	330.9	316.9	296.4
Average:	417.5	418.4	403.3	369.2	364.5	351.4
Median:	423.0	423.0	405.4	372.7	369.1	349.4
Best estimate:	418	418	403	369	365	350
[(a)]Use:	—	—	—	375	370	355
BH-300s	**Ground Surface**	**Residuum**	**Saprolite**	**PWR**	**MWR**	**Sound Rock**
Maximum:	426.3	426.3	422.8	393.7	390.2	383.5
Minimum:	352.8	352.8	342.3	353.0	327.1	316.1
Average:	414.8	413.3	401.8	372.5	366.7	359.4
Median:	417.9	417.7	406.5	371.9	365.3	360.3
Best estimate:	415	413	402	373	367	360
[(a)]Use:	—	—	—	365	360	355
BH-400s	**Ground Surface**	**Residuum**	**Saprolite**	**PWR**	**MWR**	**Sound Rock**
Maximum:	411.8	411.8	408.3	357.2	356.2	352.9
Minimum:	384.7	384.7	363.8	332.5	298.3	310.2
Average:	399.6	399.7	389.8	345.7	335.3	334.6
Median:	400.7	401.3	395.0	343.4	340.4	336.7
Best estimate:	400	400	389	346	335	335
BH-500s	**Ground Surface**	**Residuum**	**Saprolite**	**PWR**	**MWR**	**Sound Rock**
Maximum:	430.0	403.5	388.0	367.5	NE[(b)]	NE
Minimum:	428.8	386.8	376.8	350.3	NE	NE
Average:	429.6	395.2	382.4	358.9	NE	NE
Median:	430.0	395.2	382.4	358.9	NE	NE
Best estimate:	430	395	382	359	NE	NE

Table 2.5.4-201 (Sheet 2 of 2)
Termination Elevations of Soil Strata

	Elevation of Top of					
BH-600s	**Ground Surface**	**Residuum**	**Saprolite**	**PWR**	**MWR**	**Sound Rock**
Maximum:	450.1	450.1	421.2	403.2	392.7	381.3
Minimum:	308.2	308.2	289.7	280.3	268.8	264.8
Average:	406.4	410.0	389.8	348.5	345.4	350.2
Median:	339.1	412.8	401.6	348.7	352.6	358.2
Best estimate:	406	410	390	349	345	350

(a) Suggested elevations using the V_s measurements.
(b) NE = not encountered

Table 2.5.4-202 (Sheet 1 of 9)
Field Testing Locations and Depths

Test Location	Location/Remarks	Northing[(d)]	Easting[(d)]	Elevation (ft)	Total Depth (ft, bgs)	Bottom of Hole Elevation (ft)	Top of Fill (ft)	Top of Alluvial Soil (ft)	Top of Residual Soil (ft)[(e)]	Top of Saprolite (ft)[(e)]	Top of PWR (ft)[(e)]	Top of Rock (ft)[(g)]	Top of Sound Rock (ft)[(d)]
B-201(DH)	Nuclear Island/Down-hole Geophysical	892740.9	1903285.1	423.7	350.0	73.7	NE	NE	423.7	405.2	384.9	373.4	361.5
B-201UDP[(a)]	Nuclear Island/Down-hole Geophysical	892737.8	1903293.2	423.8	47.0	376.8	NE	NE	NE	NE	NE	NE	NE
B-202	Nuclear Island	892792.7	1903302.6	423.9	175.5	248.4	NE	NE	423.9	405.4	NE	377.9	359.9
B-203	Nuclear Island	892696.1	1903268.9	423.5	151.5	272.0	423.5	NE	422.5	401.5	NE	372.0	372.0
B-204	Nuclear Island	892754.6	1903400.2	424.5	150.0	274.5	NE	NE	424.5	401.0	377.5	377.5	345.5
B-205	Nuclear Island	892840.4	1903199.2	423.1	175.0	248.1	NE	NE	423.1	381.6	369.1	369.1	358.1
B-206(DH)	Nuclear Island/Down-hole Geophysical	892683.5	1903416.2	424.3	214.8	209.5	NE	NE	424.3	405.3	NE	352.3	348.3
B-207(DH)	Power Block/Down-hole Geophysical	892824.8	1902949.7	423.9	175.0	248.9	423.9	NE	423.4	415.4	385.4	380.9	373.9
B-208	Power Block	892989.8	1902925.3	422.0	10.5	411.5	NE	NE	422.0	NE	NE	NE	NE
B-208A[(f)]	Power Block	892990.7	1902928.9	421.7	115.0	306.7	NE	NE	421.7	399.7	388.2	387.2	330.2
B-209	Power Block	893015.1	1903210.9	407.9	150.0	257.9	NE	NE	407.9	384.4	354.4	353.9	324.9
B-210	Power Block	892842.5	1903457.4	416.5	115.0	301.5	NE	NE	416.5	403.0	356.8	350.5	340.5
B-211(DH)	Adjacent to Power Block/Down-hole Geophysical	892570.0	1903213.8	422.2	176.0	246.2	NE	NE	422.2	416.7	NE	380.2	377.2
B-211A[(b)]	Adjacent to Power Block/Down-hole Geophysical	892568.4	1903205.5	421.8	39.0	382.8	NE	NE	NE	NE	NE	NE	NE
B-212	Adjacent to Power Block	893100.7	1903027.4	397.2	68.5	328.7	NE	NE	397.2	378.7	346.7	328.7	NE
B-212A[(b)]	Adjacent to Power Block	893099.4	1903031.8	397.8	115.4	282.5	NE	NE	NE	NE	342.8	341.8	314.6
B-213	Adjacent to Power Block	892986.5	1903458.5	401.5	150.0	251.5	NE	NE	401.5	388.0	343.5	342.3	331.7
B-214	Power Block	892735.7	1903158.7	423.4	115.0	308.4	NE	NE	423.4	415.4	389.1	384.9	369.1
B-215	Power Block	892789.9	1903053.3	423.4	175.0	248.4	NE	NE	423.4	419.9	374.9	374.9	369.6
B-216	Power Block	892871.6	1902884.1	423.1	85.0	338.1	NE	NE	423.1	416.6	396.1	375.1	368.4
B-216UDP[(a)]	Power Block	892863.6	1902876.4	423.1	40.5	382.6	NE	NE	NE	NE	NE	NE	NE
B-217	Power Block	892933.8	1902898.3	423.3	175.0	248.3	NE	NE	423.3	417.3	373.5	372.3	349.3
B-218	Power Block	892898.9	1902973.4	423.0	115.0	308.0	NE	NE	423.0	409.5	382.0	364.5	343.0
B-218UDP[(a)]	Power Block	892909.2	1902978.1	422.8	50.5	372.3	NE	NE	NE	NE	NE	NE	NE
B-219	Power Block	892859.6	1903080.5	423.0	86.0	337.0	423.0	NE	422.5	409.5	375.5	371.0	347.0
B-220	Power Block	892976.3	1903010.5	421.5	105.0	316.5	421.5	NE	421.0	399.5	359.5	347.7	336.5

Table 2.5.4-202 (Sheet 2 of 9)
Field Testing Locations and Depths

Test Location	Location/Remarks	Northing(d)	Easting(d)	Elevation (ft)	Total Depth (ft, bgs)	Bottom of Hole Elevation (ft)	Top of Fill (ft)	Top of Alluvial Soil (ft)	Top of Residual Soil (ft)(e)	Top of Saprolite (ft)(e)	Top of PWR (ft)(e)	Top of Rock (ft)(g)	Top of Sound Rock (ft)(d)
B-221	Power Block	892928.8	1903108.9	421.7	69.5	352.2	421.7	NE	420.2	410.7	363.7	352.2	NE
B-221A(b)	Power Block	892934.9	1903109.9	421.6	91.2	330.4	NE	NE	NE	NE	NE	361.6	335.4
B-222	Power Block	892879.6	1903150.9	423.2	115.0	308.2	NE	NE	423.2	399.7	364.7	364.7	363.7
B-223	Power Block	892961.9	1903324.3	410.5	85.3	325.2	NE	NE	410.5	393.7	372.0	369.8	343.8
B-224	Power Block	892895.9	1903344.4	419.2	116.2	303.0	NE	NE	419.2	400.7	365.7	349.7	349.5
B-224UDP(a)	Power Block	892889.9	1903354.9	419.0	55.5	363.5	NE	NE	NE	NE	NE	NE	NE
B-225	Power Block	892926.5	1903216.4	425.2	85.0	340.2	NE	NE	425.2	409.2	391.7	391.2	359.6
B-226	Power Block	892723.8	1903532.7	422.3	112.5	309.8	NE	NE	422.3	411.3	NE	351.3	342.8
B-227	Adjacent to Power Block	892494.0	1903408.0	425.1	54.5	370.6	NE	NE	425.1	421.6	373.6	NE	NE
B-228	Adjacent to Power Block	892304.0	1903395.0	419.2	85.1	334.1	NE	NE	418.9	406.2	NE	363.7	347.2
B-229	Adjacent to Power Block	892394.7	1903147.6	423.2	85.7	337.5	NE	NE	423.2	410.2	368.2	368.0	364.4
B-230	Adjacent to Power Block	892658.4	1903033.9	424.5	85.3	339.2	NE	NE	424.5	419.0	388.0	385.2	382.5
B-231	Adjacent to Power Block	892519.0	1902844.2	428.4	115.0	313.4	NE	NE	428.4	419.9	377.4	374.0	374.0
B-232	Adjacent to Power Block	892767.1	1902865.1	424.0	55.4	368.6	NE	NE	424.0	405.5	NE	388.6	383.5
B-233	Adjacent to Power Block	892784.5	1902686.9	426.1	75.0	351.1	NE	NE	426.1	417.6	NE	388.7	379.6
B-234	Adjacent to Power Block	893072.0	1902801.4	421.1	55.0	366.1	NE	NE	421.1	399.6	NE	NE	NE
B-235	Adjacent to Power Block	893192.6	1902941.0	379.4	85.5	293.9	NE	NE	379.4	355.9	330.9	316.9	303.4
B-236	Adjacent to Power Block	893133.1	1903296.0	374.7	27.3	347.4	NE	NE	374.7	366.2	NE	347.4	NE
B-236A(b)	Adjacent to Power Block	893140.6	1903298.7	374.4	115.1	259.3	NE	NE	NE	NE	342.4	335.9	296.4
B-301(DH)	Nuclear Island/Down-hole Geophysical	891906.9	1902949.2	417.1	129.8	287.3	NE	NE	417.1	404.6	NE	359.1	357.1
B-301A(b)	Nuclear Island/Down-hole Geophysical	891895.0	1902945.0	416.2	350.9	65.3	NE	NE	NE	NE	NE	361.7	357.7
B-302	Nuclear Island	891954.5	1902970.9	417.2	175.8	241.5	NE	NE	417.2	407.2	NE	363.7	357.2
B-303	Nuclear Island	891861.4	1902923.5	415.1	150.0	265.1	NE	NE	415.1	393.6	NE	363.6	359.1
B-304	Nuclear Island	891921.7	1903063.2	415.3	150.0	265.3	NE	NE	415.3	402.3	NE	362.6	359.8
B-305	Nuclear Island	892004.9	1902859.1	423.9	175.0	248.9	NE	NE	423.9	391.9	372.4	366.4	363.3
B-305UDP(a)	Nuclear Island	891997.6	1902844.2	424.0	55.5	368.5	NE	NE	NE	NE	NE	NE	NE
B-306(DH)	Nuclear Island/Down-hole Geophysical	891854.8	1903077.2	413.4	215.0	198.4	NE	NE	413.4	400.4	369.9	368.9	362.6
B-307(DH)	Power Block/Down-hole Geophysical	891989.1	1902613.3	402.6	176.0	226.6	NE	NE	402.6	394.6	364.1	362.6	362.6

RN-13-017

Table 2.5.4-202 (Sheet 3 of 9)
Field Testing Locations and Depths

Test Location	Location/Remarks	Northing[d]	Easting[d]	Elevation (ft)	Total Depth (ft, bgs)	Bottom of Hole Elevation (ft)	Top of Fill (ft)	Top of Alluvial Soil (ft)	Top of Residual Soil (ft)[e]	Top of Saprolite (ft)[e]	Top of PWR (ft)[e]	Top of Rock (ft)[g]	Top of Sound Rock (ft)[d]
B-307A[b]	Power Block/Down-hole Geophysical	891982.7	1902610.6	402.4	40.0	362.4	NE	NE	NE	NE	NE	NE	NE
B-308	Power Block	892154.5	1902587.6	418.3	115.0	303.3	NE	NE	418.3	412.3	NE	378.8	371.9
B-309	Power Block	892160.7	1902842.7	422.6	150.0	272.6	NE	NE	422.6	414.1	NE	376.1	365.1
B-310	Power Block	892010.5	1903114.7	417.0	105.8	311.2	NE	NE	417.0	413.5	NE	363.7	362.0
B-311(DH)	Adjacent to Power Block/Down-hole Geophysical	891747.1	1902871.4	419.5	175.0	244.5	NE	NE	419.5	381.0	367.5	363.0	347.5
B-312	Adjacent to Power Block	892269.4	1902694.0	425.2	115.0	310.2	NE	NE	425.2	408.7	393.7	390.2	383.5
B-313	Adjacent to Power Block	892151.4	1903120.7	420.5	150.0	270.5	NE	NE	420.5	397.0	374.0	359.5	359.5
B-313A[b]	Adjacent to Power Block	892138.9	1903121.8	420.1	35.0	385.1	NE	NE	NE	NE	NE	NE	NE
B-314	Power Block	891905.1	1902819.6	417.8	115.0	302.8	NE	NE	NE	NE	NE	358.8	357.8
B-314A[b]	Power Block	891905.0	1902814.2	417.9	59.0	358.9	417.9	NE	408.4	396.9	NE	NE	NE
B-315	Power Block	891945.4	1902714.1	413.4	175.0	238.4	413.4	404.4	NE	394.9	370.4	370.4	353.4
B-316	Power Block	892005.9	1902534.6	401.2	85.0	316.2	401.2	NE	392.7	377.7	NE	364.2	353.6
B-317	Power Block	892095.2	1902571.1	415.5	175.3	240.2	NE	NE	415.5	406.5	390.5	388.5	366.3
B-317A[b]	Power Block	892095.9	1902567.2	415.3	27.0	388.3	NE	NE	NE	NE	NE	NE	NE
B-318	Power Block	892066.6	1902642.7	420.2	115.2	305.0	NE	NE	420.2	409.7	371.7	371.2	364.5
B-319	Power Block	892046.7	1902720.5	420.5	85.5	335.0	NE	NE	420.5	407.0	372.0	372.0	360.7
B-320	Power Block	892140.4	1902674.8	422.5	115.0	307.5	NE	NE	422.5	414.5	NE	372.0	372.0
B-321	Power Block	892101.3	1902773.3	422.8	85.1	337.7	NE	NE	422.8	417.3	368.0	367.8	367.3
B-322	Power Block	892048.6	1902812.5	425.3	115.5	309.8	NE	NE	425.3	411.8	376.8	376.8	359.9
B-323	Power Block	892134.3	1902992.0	420.1	84.9	335.2	NE	NE	420.1	411.6	375.1	373.4	372.1
B-324	Power Block	892054.4	1903009.4	419.4	115.2	304.2	NE	NE	NE	419.4	NE	367.4	361.4
B-325	Power Block	892084.9	1902905.1	420.3	85.0	335.3	NE	NE	420.3	411.3	370.8	365.3	361.8
B-325UDP[a]	Power Block	892088.1	1902912.0	420.0	48.5	371.5	NE	NE	NE	NE	NE	NE	NE
B-326	Power Block	891942.1	1903185.1	412.7	115.0	297.7	NE	NE	412.7	391.2	NE	357.7	348.2
B-327	Adjacent to Power Block	891669.1	1903076.7	410.8	59.3	351.5	NE	NE	410.8	399.1	362.3	361.5	NE
B-328	Adjacent to Power Block	891465.0	1903044.6	424.6	85.0	339.6	NE	NE	424.6	421.6	NE	349.1	348.8
B-329	Adjacent to Power Block	891561.8	1902808.3	410.0	85.0	325.0	NE	NE	410.0	399.5	353.0	350.0	336.8
B-330	Adjacent to Power Block	891818.9	1902689.4	401.6	86.0	315.6	NE	NE	401.6	NE	353.1	352.8	346.6
B-331	Adjacent to Power Block	891714.2	1902465.4	352.8	116.3	236.5	NE	NE	352.8	342.3	NE	327.1	316.1
B-332	Adjacent to Power Block	891931.5	1902530.0	398.4	58.8	339.6	NE	398.4	381.9	376.9	354.9	354.6	344.6

Table 2.5.4-202 (Sheet 4 of 9)
Field Testing Locations and Depths

Test Location	Location/Remarks	Northing[(d)]	Easting[(d)]	Elevation (ft)	Total Depth (ft, bgs)	Bottom of Hole Elevation (ft)	Top of Fill (ft)	Top of Alluvial Soil (ft)	Top of Residual Soil (ft)[(e)]	Top of Saprolite (ft)[(e)]	Top of PWR (ft)[(e)]	Top of Rock (ft)[(g)]	Top of Sound Rock (ft)[(d)]
B-333	Adjacent to Power Block	891946.5	1902319.8	394.4	86.0	308.4	NE	NE	394.4	383.4	372.4	368.3	346.4
B-334	Adjacent to Power Block	892235.2	1902463.7	418.7	55.5	363.2	NE	NE	418.7	410.2	386.7	384.4	383.2
B-335	Adjacent to Power Block	892354.8	1902604.2	426.3	85.0	341.3	NE	NE	426.3	422.8	387.8	386.3	382.8
B-336	Adjacent to Power Block	892359.6	1903068.4	424.3	115.0	309.3	NE	NE	424.3	417.8	387.3	387.3	366.8
B-401	Cooling Tower	891028.4	1903589.1	404.0	120.0	284.0	404.0	NE	402.0	396.0	342.5	340.0	335.0
B-402	Cooling Tower	891102.4	1903999.8	403.9	61.5	342.4	NE	NE	403.9	395.4	NE	356.2	352.9
B-403	Cooling Tower	890640.6	1903819.7	400.7	82.0	318.7	NE	NE	400.7	394.7	357.2	344.7	328.7
B-404	Cooling Tower	890206.7	1904139.7	410.9	112.6	298.3	NE	NE	410.9	397.4	343.4	298.3	NE
B-405	Cooling Tower	890180.1	1903635.0	392.0	52.6	339.4	NE	NE	392.0	386.5	353.5	344.8	NE
B-406	Cooling Tower	890109.4	1903182.2	384.7	64.7	320.0	NE	NE	384.7	371.2	342.2	331.2	331.2
B-421	Cooling Tower	891447.2	1902586.2	396.0	78.0	318.0	NE	NE	396.0	387.5	332.5	NE	NE
B-421A[(b)]	Cooling Tower	891444.9	1902585.1	396.2	95.0	301.2	NE	NE	NE	NE	NE	317.3	310.2
B-422	Cooling Tower	891422.1	1902840.4	411.8	83.5	328.3	NE	NE	411.8	408.3	348.3	340.8	339.8
B-423	Cooling Tower	892033.8	1903520.8	408.0	77.4	330.6	NE	NE	408.0	397.0	NE	341.0	341.0
B-424	Cooling Tower	891283.9	1903783.6	387.3	60.7	326.6	NE	NE	387.3	363.8	NE	338.4	338.4
B-501	Makeup Water Structure	897815.3	1903693.7	430.0	80.0	350.0	430.0	NE	403.5	388.0	367.5	NE	NE
B-501A[(b)]	Makeup Water Structure	897814.0	1903688.9	430.0	10.0	420.0	NE	NE	NE	NE	NE	NE	NE
B-502	Makeup Water Structure	897841.4	1903750.9	428.8	80.0	348.8	428.8	NE	386.8	376.8	350.3	NE	NE
B-601	Switchyard	892885.4	1902148.3	418.8	85.0	333.8	NE	NE	418.8	395.3	370.3	363.6	355.3
B-602	Switchyard	892808.5	1902336.0	438.4	115.8	322.6	NE	NE	438.4	409.9	377.4	374.5	374.4
B-603	Switchyard	892736.6	1902523.0	429.3	55.3	374.0	NE	NE	429.3	417.8	NE	381.1	381.3
B-604	Switchyard	892508.3	1902001.9	414.6	86.0	328.6	414.6	NE	411.6	391.1	346.1	345.0	345.0
B-605	Switchyard	892437.8	1902187.0	432.2	55.0	377.2	NE	NE	432.2	421.2	NE	NE	NE
B-606	Switchyard	892343.2	1902368.3	424.2	85.0	339.2	NE	NE	424.2	405.7	377.2	374.1	374.2
B-607	Switchyard	892137.3	1901852.6	432.0	55.0	377.0	NE	NE	432.0	418.5	NE	NE	NE
B-608	Switchyard	892054.3	1902009.8	411.8	46.0	365.8	NE	NE	411.8	405.8	368.3	365.8	NE
B-608A[(b)]	Switchyard	892053.8	1902007.6	412.1	85.8	326.3	NE	NE	NE	NE	NE	381.1	361.1
B-609	Switchyard	891984.6	1902227.2	406.1	53.5	352.6	NE	NE	406.1	NE	NE	352.6	NE
B-610	Relocated Access Road	893456.0	1904107.8	422.5	40.0	382.5	NE	NE	422.5	409.0	NE	NE	NE
B-611	Relocated Access Road	892895.3	1904453.2	405.4	40.0	365.4	NE	NE	405.4	403.9	NE	NE	NE
B-612	Relocated Access Road	892396.1	1904222.2	405.0	62.0	343.0	NE	NE	405.0	391.5	346.5	NE	NE

Table 2.5.4-202 (Sheet 5 of 9)
Field Testing Locations and Depths

Test Location	Location/Remarks	Northing[d]	Easting[d]	Elevation (ft)	Total Depth (ft, bgs)	Bottom of Hole Elevation (ft)	Top of Fill (ft)	Top of Alluvial Soil (ft)	Top of Residual Soil (ft)[e]	Top of Saprolite (ft)[e]	Top of PWR (ft)[e]	Top of Rock (ft)[g]	Top of Sound Rock (ft)[d]
B-613	Relocated Access Road	892503.1	1903763.1	412.8	40.0	372.8	NE	NE	412.8	399.3	NE	NE	NE
B-614	Relocated Access Road	891686.1	1903545.0	375.0	34.8	340.2	NE	NE	375.0	356.5	341.5	340.2	NE
B-615	Relocated Access Road	890997.3	1902873.2	387.9	40.0	347.9	NE	NE	NE	387.9	349.4	NE	NE
B-616	Relocated Access Road	890514.7	1902642.5	400.3	40.0	360.3	NE	NE	400.3	388.8	NE	NE	NE
B-617	Existing Access Road	889886.3	1902373.7	450.1	105.0	345.1	NE	NE	450.1	416.1	351.6	345.1	NE
B-618	South of Switchyard	890962.5	1901499.0	308.2	32.6	275.6	NE	NE	308.2	289.7	NE	275.6	NE
B-619	West of Southern Nuclear Island	892586.7	1901845.3	405.1	66.0	339.1	NE	NE	405.1	366.6	341.1	339.1	NE
B-620	North of Northern Nuclear Island	893600.9	1903011.1	381.7	100.0	281.7	NE	NE	381.7	380.2	315.2	NE	NE
B-621	North of Northern Nuclear Island	893742.3	1903670.2	421.5	101.0	320.5	NE	NE	421.5	415.0	370.0	363.0	345.5
B-622	North of Northern Nuclear Island	894292.4	1904134.3	437.7	45.0	392.7	NE	NE	437.7	409.2	403.2	392.7	NE
B-623	Northeast of Nuclear Island	893814.0	1904949.3	439.6	73.9	365.7	NE	NE	439.6	411.1	371.1	365.7	NE
B-624	East of Nuclear Island	891608.9	1904614.0	359.0	31.6	327.4	NE	NE	359.0	357.5	348.0	327.4	NE
B-625	East of Cooling Tower Area	889889.7	1904938.0	404.2	110.0	294.2	NE	NE	404.2	370.7	295.7	294.2	NE
B-626	Existing Access Road	893200.4	1904143.7	417.2	103.6	313.6	NE	NE	417.2	406.7	319.7	313.6	NE
B-627	South of Nuclear Island	891226.4	1902128.7	326.3	101.5	224.8	326.3	324.8	NE	309.8	280.3	268.8	264.8
C-201	Power Block	892773.0	1903149.7	423.4	33.8	389.6							
C-202(S)	Power Block/Seismic Cone	892888.5	1903062.6	422.5	49.4	373.1							
C-203	Power Block	892915.3	1902940.3	422.8	39.5	383.3							
C-204	Power Block	892848.9	1903329.6	428.3	50.7	377.6							
C-205	Power Block	892713.8	1903499.0	423.1	37.9	385.2							
C-206	Outside Power Block	893044.5	1902877.5	420.5	76.1	344.4							
C-207(S)	Outside Power Block/Seismic Cone	892903.1	1903451.5	413.0	50.5	362.5							
C-208	Outside Power Block	892800.9	1902817.8	423.4	30.0	393.4							
C-209(S)	Outside Power Block/Seismic Cone	892471.8	1902958.6	427.0	36.4	390.6							
C-210	Outside Power Block	893241.2	1903128.5	367.7	20.1	347.6							
C-301	Power Block	891941.9	1902811.3	421.0	54.8	366.2							
C-302(S)	Power Block/Seismic Cone	892052.1	1902726.6	421.3	47.2	374.1							
C-303	Power Block	892040.7	1902622.5	415.9	42.7	373.2							

Table 2.5.4-202 (Sheet 6 of 9)
Field Testing Locations and Depths

Test Location	Location/Remarks	Northing(d)	Easting(d)	Elevation (ft)	Total Depth (ft, bgs)	Bottom of Hole Elevation (ft)	Top of Fill (ft)	Top of Alluvial Soil (ft)	Top of Residual Soil (ft)(e)	Top of Saprolite (ft)(e)	Top of PWR (ft)(e)	Top of Rock (ft)(g)	Top of Sound Rock (ft)(d)
C-304	Power Block	892013.7	1902992.9	418.1	51.3	366.8							
C-305	Power Block	891841.4	1903149.5	413.0	47.0	366.0							
C-306	Outside Power Block	892210.3	1902541.3	417.4	29.7	387.7							
C-307(S)	Outside Power Block/Seismic Cone	892076.1	1903116.6	418.7	52.0	366.7							
C-308	Outside Power Block	891967.1	1902484.2	398.9	37.6	361.3							
C-309(S)	Outside Power Block/Seismic Cone	891638.6	1902622.3	397.2	47.6	349.6							
C-310	Outside Power Block	892406.1	1902791.8	427.6	31.8	395.8							
C-401	Cooling Tower	890975.6	1904482.2	407.9	66.0	341.9							
C-402(S)	Cooling Tower/Seismic Cone	890576.8	1903321.4	399.7	58.0	341.7							
C-403	Cooling Tower	889805.7	1903955.5	401.0	57.0	344.0							
C-407	Cooling Tower	891688.6	1903553.1	374.0	24.0	350.0							
C-409	Cooling Tower	891306.3	1903124.9	390.5	36.0	354.5							
C-501	Makeup Water Structure	897785.5	1903807.9	427.9	64.0	363.9							
C-601	Switchyard	892737.0	1902205.3	433.6	60.0	373.6							
C-602	Switchyard	892669.2	1902376.0	433.5	59.0	374.5							
C-603	Switchyard	892262.8	1902038.0	422.2	67.0	355.2							
C-604	Switchyard	892193.1	1902215.5	424.2	60.0	364.2							
C-605	Relocated Access Road	893092.5	1904069.5	415.3	40.0	375.3							
C-606	Relocated Access Road	893211.6	1904476.0	412.0	40.0	372.0							
C-607	Relocated Access Road	892575.2	1904318.2	407.2	40.0	367.2							
C-608	Relocated Access Road	892406.2	1904230.0	405.8	40.0	365.8							
C-609	Relocated Access Road	891462.6	1903410.2	397.2	40.0	357.2							
C-610	Relocated Access Road	890608.9	1902714.0	393.4	40.0	353.4							
TP-201	Nuclear Island Down-hole Geophysical	892745.5	1903290.3	423.6	6.0	417.6	423.6	NE	422.6	NE	NE	NE	NE
TP-201	Nuclear Island Down-hole Geophysical	892753.7	1903291.9	423.4	6.0	417.4	423.4	NE	422.4	NE	NE	NE	NE
TP-201	Nuclear Island Down-hole Geophysical	892752.2	1903296.1	423.4	6.0	417.4	423.4	NE	422.4	NE	NE	NE	NE
TP-201	Nuclear Island Down-hole Geophysical	892745.3	1903296.4	423.7	6.0	417.7	423.7	NE	422.7	NE	NE	NE	NE
TP-227	Adjacent to Power Block	892489.8	1903411.7	422.6	5.0	417.6	NE	NE	422.6	419.6	NE	NE	NE

Table 2.5.4-202 (Sheet 7 of 9)
Field Testing Locations and Depths

Test Location	Location/Remarks	Northing(d)	Easting(d)	Elevation (ft)	Total Depth (ft, bgs)	Bottom of Hole Elevation (ft)	Top of Fill (ft)	Top of Alluvial Soil (ft)	Top of Residual Soil (ft)(e)	Top of Saprolite (ft)(e)	Top of PWR (ft)(e)	Top of Rock (ft)(g)	Top of Sound Rock (ft)(d)
TP-227	Adjacent to Power Block	892487.3	1903418.9	422.4	5.0	417.4	NE	NE	422.4	419.4	NE	NE	NE
TP-227	Adjacent to Power Block	892484.3	1903417.3	422.4	5.0	417.4	NE	NE	422.4	419.4	NE	NE	NE
TP-227	Adjacent to Power Block	892487.3	1903410.2	422.7	5.0	417.7	NE	NE	422.7	419.7	NE	NE	NE
TP-301	Nuclear Island Down-hole Geophysical	891900.2	1902968.9	415.6	3.0	412.6	NE	NE	415.6	NE	NE	NE	NE
TP-301	Nuclear Island Down-hole Geophysical	891893.0	1902972.7	415.4	3.0	412.4	NE	NE	415.4	NE	NE	NE	NE
TP-301	Nuclear Island Down-hole Geophysical	891890.0	1902969.3	415.4	3.0	412.4	NE	NE	415.4	NE	NE	NE	NE
TP-301	Nuclear Island Down-hole Geophysical	891898.0	1902964.7	415.6	3.0	412.6	NE	NE	415.6	NE	NE	NE	NE
TP-405	Cooling Tower	890185.9	1903648.7	392.4	4.0	388.4	NE	NE	NE	392.4	NE	NE	NE
TP-405	Cooling Tower	890193.9	1903649.0	392.5	4.0	388.5	NE	NE	NE	392.5	NE	NE	NE
TP-405	Cooling Tower	890191.8	1903640.0	392.2	4.0	388.2	NE	NE	NE	392.2	NE	NE	NE
TP-405	Cooling Tower	890185.0	1903639.9	392.3	4.0	388.3	NE	NE	NE	392.3	NE	NE	NE
OW-205a(c)	Nuclear Island	892829.3	1903189.8	425.9	110.0								
OW-205b	Nuclear Island	892842.4	1903192.5	425.0	60.0								
OW-212	Adjacent to Power Block	893105.1	1903036.8	399.3	68.0								
OW-213	Adjacent to Power Block	892975.6	1903457.3	404.5	55.3								
OW-227	Adjacent to Power Block	892494.0	1903408.0	425.1	84.3								
OW-233	Adjacent to Power Block	892786.5	1902693.4	428.3	120.0								
OW-305a	Nuclear Island	892008.7	1902841.2	427.8	141.0								
OW-305b	Nuclear Island	891996.7	1902857.5	426.3	66.5								
OW-312	Adjacent to Power Block	892256.5	1902709.6	427.1	36.5								
OW-313	Adjacent to Power Block	892167.6	1903132.5	423.8	59.0								
OW-327	Adjacent to Power Block	891669.2	1903084.1	413.4	66.0								
OW-333	Adjacent to Power Block	891954.4	1902319.6	397.1	71.0								
OW-401a	Cooling Tower	891017.8	1903595.5	406.3	92.5								
OW-401b	Cooling Tower	891013.1	1903585.0	406.8	66.0								
OW-405	Cooling Tower	890180.4	1903650.2	395.4	58.5								
OW-501	Makeup Water Structure	897817.4	1903702.3	431.9	32.0								
OW-612	Relocated Access Road	892415.5	1904227.3	409.4	62.0								
OW-614	Relocated Access Road	891671.1	1903536.1	379.1	33.0								

Table 2.5.4-202 (Sheet 8 of 9)
Field Testing Locations and Depths

Test Location	Location/Remarks	Northing[d]	Easting[d]	Elevation (ft)	Total Depth (ft, bgs)	Bottom of Hole Elevation (ft)	Top of Fill (ft)	Top of Alluvial Soil (ft)	Top of Residual Soil (ft)[e]	Top of Saprolite (ft)[e]	Top of PWR (ft)[e]	Top of Rock (ft)[g]	Top of Sound Rock (ft)[d]
OW-617	Existing Access Road	889886.3	1902373.7	450.1	108.0								
OW-618	South of Switchyard	890955.6	1901480.1	310.5	32.5								
OW-619	West of Southern Nuclear Island	892594.0	1901843.9	407.7	104.0								
OW-620	North of Northern Nuclear Island	893593.8	1903017.2	385.0	91.0								
OW-621a	North of Northern Nuclear Island	893732.7	1903676.2	423.5	97.0								
OW-621b	North of Northern Nuclear Island	893742.6	1903677.8	423.6	71.0								
OW-622	North of Northern Nuclear Island	894292.2	1904118.1	440.7	62.0								
OW-623	Northeast of Nuclear Island	893819.9	1904946.1	441.8	90.0								
OW-624	East of Nuclear Island	891595.7	1904623.8	361.6	62.0								
OW-625	East of Cooling Tower Area	889895.0	1904957.3	405.9	108.0								
OW-626	Existing Access Road	893202.4	1904129.9	418.8	85.0								
OW-627a	South of Nuclear Island	891239.9	1902130.4	330.3	86.0								
OW-627b	South of Nuclear Island	891231.6	1902129.7	329.5	56.0								
R-1/end north	Electrical Resistivity Test	892725.8	1902531.4	429.2	—								
R-1/end south	Electrical Resistivity Test	892081.0	1902042.5	408.4	—								
R-1/R-2 center	Electrical Resistivity Test	892448.2	1902299.7	429.5	—								
R-2/end north	Electrical Resistivity Test	892803.5	1902028.0	416.7	—								
R-2/end south	Electrical Resistivity Test	892101.2	1902584.0	415.9	—								
R-3/end east	Electrical Resistivity Test	892369.5	1902956.3	425.5	—								
R-3/end west	Electrical Resistivity Test	892209.0	1902090.7	426.4	—								
R-4/end north	Electrical Resistivity Test	892619.8	1902297.1	436.6	—								
R-4/end south	Electrical Resistivity Test	891993.0	1902781.1	421.9	—								
R-3/R-4 center	Electrical Resistivity Test/ U3 Transformers	892273.6	1902534.8	422.6	—								
R-5/center	Electrical Resistivity Test /U2 Crane Area	892658.0	1902916.1	425.5	—								
R-5/end north	Electrical Resistivity Test	893101.9	1902871.1	412.8	—								
R-5/end south	Electrical Resistivity Test	892212.5	1902960.9	426.2	—								
R-6 center	Electrical Resistivity Test/ U2 Turbine Building	892816.9	1903101.3	423.2	—								

Table 2.5.4-202 (Sheet 9 of 9)
Field Testing Locations and Depths

Test Location	Location/Remarks	Northing[(d)]	Easting[(d)]	Elevation (ft)	Total Depth (ft, bgs)	Bottom of Hole Elevation (ft)	Top of Fill (ft)	Top of Alluvial Soil (ft)	Top of Residual Soil (ft)[(e)]	Top of Saprolite (ft)[(e)]	Top of PWR (ft)[(e)]	Top of Rock (ft)[(g)]	Top of Sound Rock (ft)[(d)]
R-6/east end	Electrical Resistivity Test	892599.0	1903498.8	423.4	—								
R-6/end west	Electrical Resistivity Test	893086.3	1902702.0	407.3	—								

(a) Borings with the suffix “UDP” were drilled as directed by Bechtel to obtain undisturbed samples. Refer to the original boring for geologic layer information.

(b) Borings with the suffix “A” were drilled adjacent to the original location due to either difficulties encountered during drilling in the original location; for SPT energy measurements; or for geophysical logging purposes. Refer to original boring for geologic layering information.

(c) Coordinates and elevations shown for observation wells are for the PVC casing. Refer to Reference 232 for coordinates and elevations of concrete pad and ground surface adjacent to the pad.

(d) From Reference 232

(e) The elevations shown are the elevations at which residual soil, saprolite, and PWR were first encountered in the boring. In some isolated cases, multiple layers of either residual soil, saprolite or PWR where encountered in an interlayered manner.

(f) “Top of rock” tabulated above is the elevation at which diamond coring techniques began to advance the borehole. If no diamond coring was performed, then the elevation shown is the elevation of soil boring refusal.

(g) “Top of sound rock” is defined as generally hard, slightly discolored to fresh (bright mineral surfaces) rock with slight alteration/staining localized along joints and shears in the rock mass. RQD typically exceeds about 70%. May be underlain by zones of RQD <70% but that are composed of mostly slightly weathered to fresh rock. **Special Note:** Top of sound rock depths are MACTEC's interpretation and are generally based on the definition of sound rock described above and in the data report. Alternate interpretations of depth to top of sound rock could be made by Bechtel for some of the borings, including but not limited to the following: B-205: Highly weathered seam 82.5 - 85.0 feet; alternate top of sound rock deeper = 85.0, B-206: Highly weathered seams 76.5-77.2, 80.0-80.5, and 81.6-82.5; alternate top of sound rock deeper = 82.5, B-217: Low RQD (32%) due to moderate weathering and jointing 79.0–84.0, weathered seam 88.8-91.0; alternate top of sound rock deeper = 91.0, B-219: Lower RQ208A6.0-71.0 (57%) and 71.0-76.0 (60%); alternate top of sound rock shallower = 52.0, B-333: Highly weathered seams 52.2-53.5, 59.8-60.5, 63.0-65.4,and 67.8-68.2; alternate top of sound rock = 68.2.

NE = Not Encountered
PWR= Partially Weathered Rock
bgs = Belowground surface
— = Not Applicable

Table 2.5.4-203
Field Testing Quantities

Description	Quantity
Test Borings and Samples/Cores	111
Observation Wells	31
CPT Soundings	36
Suspension P-S Velocity Logging	8
Test Pits	4
Field Electrical Resistivity Arrays	6
SPT Hammer Energy Measurements	12

Table 2.5.4-204 (Sheet 1 of 2) Details of Undisturbed Samples

Source of Sample	Sample No.	Depth (ft)	Gravel[a] (%)	Sand[a] (%)	Fines[a] (%)	Silt[a] (%)	0.005 mm Clay[a] (%)	USCS Note [b]	Natural Moisture Content (%) at e_o: Triaxial or Direct Shear 1	Natural Moisture Content (%) at e_o: Triaxial or Direct Shear 2	Natural Moisture Content (%) at e_o: Triaxial or Direct Shear 3	Natural Moisture Content (%) at e_o: Cons.[c]	Natural Moisture Content (%) at e_o: Avg.	LL	PI	G_s	Dry Density (pcf) at e_o: Triaxial or Direct Shear 1	Dry Density (pcf) at e_o: Triaxial or Direct Shear 2	Dry Density (pcf) at e_o: Triaxial or Direct Shear 3	Dry Density (pcf) at e_o: Cons.	Dry Density (pcf) at e_o: Avg.	Wet Density Avg. (pcf)
B-204	UD-2	18.5						ML	17.30			18.2	17.8	NV	NP	2.87	91.14			98.99	95.07	112
B-204	UD-3	28.5						ML				24.1	24.1	NV	NP	2.95				87.44	87.44	109
B-208	UD-1	8.5	0	16	84	21	63	CH	22.30	25.00			23.7	59	31		97.66	90.15			93.91	116
B-208	UD-3	28.5	(b)	(b)	(b)	(b)	(b)	(b)						(b)	(b)							
B-209	UD-1	8.5						MH	42.90			42.9	42.9	56	11	2.81	71.22			69.95	70.59	101
B-209	UD-2	18.5	2	55	43	30	13	SM	56.90	45.50	43.70		48.7	55	12		59.71	64.90	68.52		64.38	96
B-209	UD-4	38.5						ML	29.60			30.7	30.2	NV	NP	2.86	85.87			88.77	87.32	114
B-210	UD-1	8.5						ML	21.90			22.7	22.3	NV	NP	2.75	88.55			88.57	88.56	108
B-210	UD-3	28.5						ML	26.00			20.7	23.4	NV	NP	2.73	91.87			99.83	95.85	118
B-210	UD-4	38.5						ML				27.1	27.1	NV	NP	2.78				84.91	84.91	108
B-215	UD-1	8.5						SM	32.50			28.4	30.5	NV	NP	2.78	84.01			87.93	85.97	112
B-215	UD-2	18.5						SM	23.80			24.6	24.2	NV	NP	2.82	90.34			92.00	91.17	113
B-215	UD-3	28.5	0	70	30			SM	24.20				24.2				86.70				86.70	108
B-216	UD-1	6.5	0	5	95	70	25	ML	35.80	35.80			35.8	NV	NP		64.72	63.38			64.05	87
B-216	UD-2	13.5	0.5	17	83	66	17	ML	37.60	27.60			32.6	NV	NP		74.62	87.76			81.19	108
B-216	UD-3	23.5	0	15	84	63	21	ML	35.00	35.40	35.80		35.4	NV	NP		72.86	80.86	90.94		81.55	110
B-217	UD-1	8.5	0	65	35	25	10	SM	29.00	26.50			27.8	NV	NP		86.37	89.48			87.93	112
B-222	UD-1	8.5						ML				26.7	26.7	NV	NP	2.71				90.49	90.49	115
B-222	UD-2	18.5						ML	23.80			20.8	22.3	NV	NP	2.84	86.95			92.61	89.78	110
B-222	UD-3	28.5	0	64	36			SM	20.30				20.3				87.10				87.10	105
B-309	UD-1	8.5	0	65	36	26	10	SM	32.3	12.4			22.4	NV	NP		83.65	90.72			87.19	107
B-309	UD-2	18.5	(b)	(b)	(b)			SM						(b)	(b)							
B-309	UD-3	28.5	0	30	70	48	22	ML	28.6	26.8			27.7	NV	NP		77.83	85.07			81.45	104
B-309	UD-4	38.5	0	51	49			SM	21.7				21.7				88.60				88.60	108
B-319	UD-2	18.5	1	71	28			SM	19.50				19.5				91.60				91.60	109
B-319	UD-3	28.5						ML	22.90			26.8	24.9	NV	NP	2.75	89.36			94.34	91.85	115
B-319	UD-4	38.5						ML				19.6	19.6	NV	NP	2.75				102.80	102.8	123
B-321	UD-2	18.5	0	66	34	25	9	SM	19.90	19.40			19.7	NV	NP		88.67	92.90			90.79	109
B-321	UD-3	28.5						SM				16.7	16.7	NV	NP	2.83				102.60	102.6	120
B-322	UD-2	18.5	0	71	29	20	9	SM	16.90	13.90	14.90		15.2	NV	NP		85.96	95.15	83.74		88.28	102

Table 2.5.4-204 (Sheet 2 of 2)
Details of Undisturbed Samples

									Natural Moisture Content (%) at e_o								Dry Density (pcf) at e_o					
									Triaxial or Direct Shear								Triaxial or Direct Shear					Wet Density Avg. (pcf)
Source of Sample	Sample No.	Depth (ft)	Gravel[a] (%)	Sand[a] (%)	Fines[a] (%)	Silt[a] (%)	0.005 mm Clay[a] (%)	USCS Note[b]	1	2	3	Cons.[c]	Avg.	LL	PI	G_s	1	2	3	Cons.	Avg.	
B-325	UD-1	3.5	0	44	57			ML	38.00				38.0				78.20				78.2	108
B-325	UD-3	13.5						SM	30.70			20.9	25.8	NV	NP	2.77	74.67			91.14	82.91	104
B-325	UD-4	18.5	(b)	(b)	(b)			(b)						(b)	(b)							
B-325	UD-8	38.5						SM	23.50			18.5	21.0	NV	NP	2.69	93.47			101.30	97.39	118

(a) Due to computer roundoff, particle size fractions may total 100 ±1. Fines include silt plus clay.
(b) USCS symbol is based on visual-manual method (Reference 208) where incomplete classification testing was performed.
(c) Cons. = Consolidation.

Table 2.5.4-205
Hammer-Rod Energy Measurements

Drill Rig Serial Number	Boring No.	Average ETR[(a)](%)	Energy Adjustment (ETR[(a)]%/60%)
90117	B-305	86.5	1.44
100	B-220	73.5	1.22
285584	B-326	72.0	1.20
219907	B-317A	77.4	1.29
331145	B-304	82.8	1.38
233517	B-313A	81.5	1.36
211797	B-403	76.8	1.28
311025	B-301	82.4	1.37
209195	B-323	75.2	1.25
190742	(b)	82.1	1.37
337153	(b)	78.2	1.30
212393	(b)	76.6	1.28

(a) ETR= Percentage of theoretical hammer energy measured in the field.
(b) Hammer-rod energy measurements made at site other than V. C. Summer.

Table 2.5.4-206
Laboratory Tests and Quantities

Test Type, Standard	Quantity
Moisture content of soil, ASTM D 2216-05	237
Atterberg limits, ASTM D 4318-05	74
Sieve and hydrometer analysis, ASTM D 422-63(2002) and ASTM D 6913-04	188 and 95
Specific gravity of soil, ASTM D 854-06	16
Chemical analysis (pH, chloride, sulfate) of soil	22
Unit weight of soil, ASTM D 5084-03	55
Consolidation tests, ASTM D 2435-04	16
Unconsolidated-undrained triaxial compression, ASTM D 2850-03	11
Consolidated-undrained triaxial compression, ASTM D 4767-04	10
Direct shear, ASTM D 3080-04	5
Resonant column torsional shear	5
Moisture-density, ASTM D 1557-02	6
California Bearing Ration testing, ASTM D 1883-05	6
Compressive strength and elastic moduli of rock cores, ASTM D 7012-04	95 and 33

Table 2.5.4-207 (Sheet 1 of 8)
Summary of Laboratory Tests on Soil Samples

Source of Sample	Samp. No.	Depth (ft)	Samp. Type	Gravel[a] (%)	Sand[a] (%)	Fines[a] (%)	Silt[a] (%)	0.005 mm Clay[a] (%)	USCS Note[c]	Natural Moisture Content (%) at e_o						LL	PI	G_s	Dry Density (pcf) at e_o					Wet Density Avg. (pcf)	pH	Chloride (mg/kg)	Sulfate (mg/kg)
											Triaxial or Direct Shear			Cons.[d]	Avg.				Triaxial or Direct Shear			Cons.	Avg.				
										SPT	1	2	3						1	2	1						
B-201	1	0	SPT						CL-ML	10.8																	
B-201	2	1.5	SPT	0	57	43	20	23	SM	19.3						NV	NP										
B-201	4	6	SPT	0	57	42	34	8	SM	18.4						NV	NP								5.4	4.1	5.1[e]
B-201	6	11	SPT	0	67	33			SM	18.8																	
B-201	7	13.5	SPT	0	63	37	28	9	SM	20.1						NV	NP										
B-201	8	18.5	SPT	0	68	32			SM	24.9																	
B-201	9	23.5	SPT						SM	24.5															5.6	3.9	6.0[e]
B-201	10	28.5	SPT	0	62	39	34	5	SM	28.6						NV	NP										
B-201	11	33.5	SPT	4	87	8			SW-SM	9.1															6.0	1.9[e]	7.5
B-201	13	43.5	SPT	0	79	20	19	1	SM	15.9																	
B-201	14	48.5	SPT	1	77	23			SM	16.0																	
B-203	2	1.5	SPT	0	74	26			SM	15.4																	
B-203	4	6	SPT	0	70	30	26	4	SM	20.5																	
B-203	6	11	SPT	0	67	24			SM	24.0																	
B-203	8	18.5	SPT	0	68	32			SM	23.3																	
B-203	9	23.5	SPT	0	63	37	31	6	SM	31.1																	
B-203	11	33.5	SPT	0	70	31			SM	29.1																	
B-203	13	43.5	SPT	0	58	42	37	5	SM	32.3																	
B-203	14	48.5	SPT	0	71	29			SM	24.6																	
B-204	UD-2	18.5	UD						ML		17.30			18.2	17.8	NV	NP	2.87		91.14			98.99	95.07	112		
B-204	UD-3	28.5	UD						ML					24.1	24.1	NV	NP	2.95					87.44	87.44	109		
B-205	2	1.5	SPT	0	23	78	42	36	ML	34.6						NV	NP										
B-205	4	6	SPT	0	29	71			ML	22.9															5.3	4.5	5.6[e]
B-205	6	11	SPT	0	35	65			ML	31.7																	
B-205	8	18.5	SPT	0	38	62	51	11	ML	30.8						NV	NP										
B-205	9	23.5	SPT	0	60	40			SM	31.6																	
B-205	10	28.5	SPT	15	51	34			SM	34.0																	

Table 2.5.4-207 (Sheet 2 of 8)
Summary of Laboratory Tests on Soil Samples

Source of Sample	Samp. No.	Depth (ft)	Samp. Type	Gravel[(a)] (%)	Sand[(a)] (%)	Fines[(a)] (%)	Silt[(a)] (%)	0.005 mm Clay[(a)] (%)	USCS Note[(c)]	Natural Moisture Content (%) at e_o						LL	PI	G_s	Dry Density (pcf) at e_o					Wet Density Avg. (pcf)	pH	Chloride (mg/kg)	Sulfate (mg/kg)
										SPT	Triaxial or Direct Shear 1	Triaxial or Direct Shear 2	Triaxial or Direct Shear 3	Cons.[(d)]	Avg.				Triaxial or Direct Shear 1	Triaxial or Direct Shear 2	1	Cons.	Avg.				
B-205	11	33.5	SPT	64	30	7			GW-GM	13.5																	
B-205	13	43.5	SPT	4	58	38	34	4	SM	21.5						NV	NP										
B-205	14	48.5	SPT	11	45	44	40	4	SM	7.8						NV	NP										
B-206	2	1.5	SPT						ML	22.8																	
B-206	4	6	SPT	0	63	37			SM	29.8																	
B-206	6	11	SPT	0	61	39	32	7	SM	30.7																	
B-206	8	18.5	SPT	0	74	26			SM	13.4																	
B-206	10	28.5	SPT	0	68	32	27	5	SM	30.8																	
B-206	12	38.5	SPT	0	64	36	31	5	SM	27.9						NV	NP										
B-206	14	48.5	SPT	0	70	31			SM	26.4																	
B-206	16	58.5	SPT	0	72	28			SM	24.1																	
B-206	18	68.5	SPT	0	78	22	21	1	SM	21.5																	
B-207	1	0	SPT	0	70	30	24	6	SM	9.4						NV	NP										
B-207	5	8.5	SPT	0	81	19			SM	20.8															5.4	5.8	15.4
B-207	6	11	SPT	0	75	25	23	2	SM	19.2						NV	NP										
B-207	7	13.5	SPT	0	79	21			SM	17.8																	
B-207	8	18.5	SPT	0	77	23			SM	21.4																	
B-207	9	23.5	SPT	0	79	22	20	2	SM	32.8																	
B-207	10	28.5	SPT	0	76	24			SM	29.9																	
B-207	11	33.5	SPT	0	64	35	29	6	SM	23.1						NV	NP										
B-207	12	38.5	SPT	6	78	16			SM	16.4																	
B-208	UD-1	8.5	UD	0	16	84	21	63	CH		22.30	25.00			23.7	59	31		97.66		90.15			93.91	116		
B-208	UD-3	28.5	UD	(b)	(b)	(b)	(b)	(b)	(b)							(b)	(b)										
B-209	UD-1	8.5	UD						MH		42.90			42.9	42.9	56	11	2.81		71.22			69.95	70.59	101		
B-209	UD-2	18.5	UD	2	55	43	30	13	SM		56.90	45.50	43.70		48.7	55	12		59.71		64.90	68.52		64.38	96		

Table 2.5.4-207 (Sheet 3 of 8)
Summary of Laboratory Tests on Soil Samples

Source of Sample	Samp. No.	Depth (ft)	Samp. Type	Gravel[a] (%)	Sand[a] (%)	Fines[a] (%)	Silt[a] (%)	0.005 mm Clay[a] (%)	USCS Note[c]	Natural Moisture Content (%) at e_o						LL	PI	G_s	Dry Density (pcf) at e_o					Wet Density Avg. (pcf)	pH	Chloride (mg/kg)	Sulfate (mg/kg)
											Triaxial or Direct Shear			Cons.[d]					Triaxial or Direct Shear			Cons.					
										SPT	1	2	3		Avg.				1	2	1		Avg.				
B-209	UD-4	38.5	UD						ML		29.60			30.7	30.2	NV	NP	2.86		85.87			88.77	87.32	114		
B-210	UD-1	8.5	UD						ML		21.90			22.7	22.3	NV	NP	2.75		88.55			88.57	88.56	108		
B-210	UD-3	28.5	UD						ML		26.00			20.7	23.4	NV	NP	2.73		91.87			99.83	95.85	118		
B-210	UD-4	38.5	UD						ML					27.1	27.1	NV	NP	2.78					84.91	84.91	108		
B-211	2	1.5	SPT	0	65	35	31	4	SM	14.8						NV	NP										
B-211	3	3.5	SPT	0	44	56			ML	20.6																	
B-211	4	6	SPT	0	70	30	22	8	SM	28.2						NV	NP										
B-211	5	8.5	SPT	0	71	30			SM	35.4																	
B-211	6	11	SPT	0	62	38	34	4	SM	17.9						NV	NP										
B-211	7	13.5	SPT	0	63	37			SM	26.7																	
B-211	8	18.5	SPT	0	56	44	38	6	SM	22.6						NV	NP										
B-211	9	23.5	SPT	0	70	30			SM	26.7															5.7	3.3[f]	3.5[e]
B-211	10	28.5	SPT	0	72	28	24	4	SM	26.0						NV	NP										
B-211	11	33.5	SPT	0	72	28			SM	23.4																	
B-211	12	38.5	SPT	0	69	31	29	2	SM	31.3						NV	NP										
B-215	2	1.5	SPT						ML	21.1																	
B-215	3	3.5	SPT	0	59	41			SM	28.8																	
B-215	4	6	SPT	0	64	36	30	6	SM	34.1																	
B-215	UD-1	8.5	UD						SM		32.50			28.4	30.5	NV	NP	2.78		84.01			87.93	85.97	112		
B-215	5	11	SPT	0	71	29	23	6	SM	25.7						NV	NP										
B-215	6	13.5	SPT	0	75	25			SM	25.6																	
B-215	UD-2	18.5	UD						SM		23.80			24.6	24.2	NV	NP	.82		90.34			92.00	91.17	113		
B-215	7	23.5	SPT	0	68	32	28	4	SM	22.7																	
B-215	UD-3	28.5	UD	0	70	30			SM		24.20				24.2				86.70					86.70	108		

Table 2.5.4-207 (Sheet 4 of 8)
Summary of Laboratory Tests on Soil Samples

Source of Sample	Samp. No.	Depth (ft)	Samp. Type	Gravel[a] (%)	Sand[a] (%)	Fines[a] (%)	Silt[a] (%)	0.005 mm Clay[a] (%)	USCS Note[c]	Natural Moisture Content (%) at e_o SPT	Natural Moisture Content: Triaxial or Direct Shear 1	Triaxial or Direct Shear 2	Triaxial or Direct Shear 3	Cons.[d]	Avg.	LL	PI	G_s	Dry Density (pcf) at e_o: Triaxial or Direct Shear 1	Triaxial or Direct Shear 2	1	Cons.	Avg.	Wet Density Avg. (pcf)	pH	Chloride (mg/kg)	Sulfate (mg/kg)
B-215	8	33.5	SPT	0	68	32			SM	24.6															5.6	1.9[e][f]	3.0
B-215	9	43.5	SPT	0	59	41	36	5	SM	27.1																	
B-216	2	1.5	SPT	0	57	43	35	8	SM	20.6						NV	NP										
B-216	3	3.5	SPT	0	56	43			SM	22.4																	
B-216	UD-1	6.5	UD	0	5	95	70	25	ML		35.80	35.80			35.8	NV	NP		64.72		63.38			64.05	87		
B-216	4	8.5	SPT	0	17	83	60	23	ML	41.1																	
B-216	5	11	SPT	0	26	74			ML	38.1																	
B-216	UD-2	13.5	UD	0.5	17	83	66	17	ML		37.60	27.60			32.6	NV	NP		74.62		87.76			81.19	108		
B-216	6	18.5	SPT	1	32	68	53	15	ML	49.3						NV	NP										
B-216	UD-3	23.5	UD	0	15	84	63	21	ML		35.00	35.40	35.80		35.4	NV	NP		72.86		80.86	90.94		81.55	110		
B-216	7	28.5	SPT	34	34	33			GM	24.5															6.0	1.8[e]	4.6[e]
B-216	8	32	SPT	76	12	12			GM	10.1																	
B-216	9	38.5	SPT	0	27	72	65	7	ML	28.6																	
B-216	10	43.5	SPT	14	50	36			SM	24.7																	
B-217	2	1.5	SPT	2	31	67	26	41	ML	26.6						NV	NP										
B-217	3	3.5	SPT	0	57	43			SM	28.9																	
B-217	4	6	SPT	0	56	43	23	20	SM	23.7																	
B-217	UD-1	8.5	UD	0	65	35	25	10	SM		29.00	26.50			27.8	NV	NP		86.37		89.48			87.93	112		
B-217	5	10.5	SPT	0	56	44	35	9	SM	21.3						NV	NP								5.4	5.9	3.3[e]
B-217	6	13.5	SPT	0	71	29			SM	27.9																	
B-217	7	23.5	SPT	0	70	30	24	6	SM	26.5																	
B-217	8	33.5	SPT	0	60	40			SM	45.8																	
B-217	9	43.5	SPT	6	69	25			SM	19.0																	
B-217	10	48.5	SPT	44	39	17	15	2	SM	13.3																	
B-220	2	1.5	SPT	0	32	68	26	42	MH	20.5																	
B-220	3	3.5	SPT	0	26	74			MH	25.4																	

Table 2.5.4-207 (Sheet 5 of 8)
Summary of Laboratory Tests on Soil Samples

Source of Sample	Samp. No.	Depth (ft)	Samp. Type	Gravel[a] (%)	Sand[a] (%)	Fines[a] (%)	Silt[a] (%)	0.005 mm Clay[a] (%)	USCS Note[c]	Natural Moisture Content (%) at e_o						LL	PI	G_s	Dry Density (pcf) at e_o					Wet Density Avg. (pcf)	pH	Chloride (mg/kg)	Sulfate (mg/kg)
										SPT	Triaxial or Direct Shear 1	Triaxial or Direct Shear 2	Triaxial or Direct Shear 3	Cons.[d]	Avg.				Triaxial or Direct Shear 1	Triaxial or Direct Shear 2	Triaxial or Direct Shear 1	Cons.	Avg.				
B-220	4	6	SPT						MH	25.1						73	20										
B-220	5	8.5	SPT	0	36	64	22	42	MH	23.7															5.5	3.4	3.7[e]
B-220	6	11	SPT	0	76	24			SM	23.0																	
B-220	7	13.5	SPT	0	58	42	27	15	SM	21.3																	
B-220	8	18.5	SPT	0	59	41			SM	25.2																	
B-220	9	23.5	SPT	0	62	39			SM	22.8																	
B-220	11	33.5	SPT	0	70	30	26	4	SM	20.6																	
B-220	12A	41	SPT	0	75	25			SM	19.3																	
B-220	14	48.5	SPT	6	55	39	35	4	SM	27.9																	
B-220	16	58.5	SPT	3	56	42	38	4	SM	22.6																	
B-222	UD-1	8.5	UD						ML					26.7	26.7	NV	NP	2.71					90.49	90.49	115		
B-222	UD-2	18.5	UD						ML		23.80			20.8	22.3	NV	NP	2.84		86.95			92.61	89.78	110		
B-222	UD-3	28.5	UD	0	64	36			SM		20.30				20.3				87.10					87.10	105		
B-301	2	1.5	SPT	1	72	27			SM	12.9																	
B-301	3A	3.5	SPT						SM	12.6						NV	NP										
B-301	3B	3.5	SPT						CH	62.6																	
B-301	4	6	SPT	0	65	35	27	8	SM	18.9															5.7	4.7	12.0
B-301	6	11	SPT	0	75	25			SM	15.1																	
B-301	7	13.5	SPT	0	71	29	26	3	SM	15.9						NV	NP										
B-301	8	18.5	SPT	0	76	24			SM	15.7															5.3	3.2	4.0[e]
B-301	9	23.5	SPT	0	77	23			SM	14.7																	
B-301	10	28.5	SPT	0	76	24	22	2	SM	15.9						NV	NP										
B-301	11	33.5	SPT	0	74	26			SM	17.0																	
B-301	12	38.5	SPT	0	74	26			SM	19.6																	
B-301	13	43.5	SPT	0	64	36	33	3	SM	33.4																	
B-301	14	48.5	SPT	0	79	20	19	1	SM	18.4						NV	NP										
B-301	15	53.5	SPT	1	78	22			SM	20.9																	
B-305	2	1.5	SPT	0	68	32			SM	18.6																	

Table 2.5.4-207 (Sheet 6 of 8)
Summary of Laboratory Tests on Soil Samples

Source of Sample	Samp. No.	Depth (ft)	Samp. Type	Gravel[a] (%)	Sand[a] (%)	Fines[a] (%)	Silt[a] (%)	0.005 mm Clay[a] (%)	USCS Note[c]	Natural Moisture Content (%) at e_o						LL	PI	G_s	Dry Density (pcf) at e_o					Wet Density Avg. (pcf)	pH	Chloride (mg/kg)	Sulfate (mg/kg)
										SPT	Triaxial or Direct Shear 1	2	3	Cons. [d]	Avg.				Triaxial or Direct Shear 1	2	1	Cons .	Avg.				
B-305	3	3.5	SPT	0	54	46	22	24	SM	30.3						NV	NP										
B-305	5	8.5	SPT	0	71	29			SM	38.0															5.2	8.5	4.0[e]
B-305	7	13.5	SPT	0	69	31			SM	39.9																	
B-305	8	18.5	SPT	0	66	34	29	5	SM	26.4																	
B-305	10	28.5	SPT	0	75	25			SM	26.8																	
B-305	12	38.5	SPT	0	73	28	25	3	SM	29.5																	
B-305	14	48.5	SPT	0	76	24			SM	27.6																	
B-306	2	1.5	SPT	0	21	79	39	40	ML	29.9						NV	NP										
B-306	3	3.5	SPT	0	70	30			SM	34.2																	
B-306	4	6	SPT	0	45	55	40	15	ML	29.6																	
B-306	6	11	SPT	0	57	43			SM	29.9						NV	NP								5.2	7.0	5.4[e]
B-306	8	18.5	SPT	0	31	68	60	8	ML	29.6																	
B-306	9	23.5	SPT	1	23	77			MH	52.1						62	13										
B-306	11	33.5	SPT	0	71	30	27	3	SM	25.6																	
B-306	12	38.5	SPT	0	60	40			SM	31.6																	
B-307	1	0	SPT						MH	34.7						63	25										
B-307	2	1.5	SPT	0	8	93	34	59	MH	29.3						76	19										
B-307	3	3.5	SPT	0	16	84			MH	27.9																	
B-307	4	6	SPT	0	17	83	47	36	MH	27.8															5.2	8.4	6.7
B-307	5	8.5	SPT	0	67	33			SM	11.0																	
B-307	6	11	SPT	0	61	38			SM	13.8																	
B-307	7A	16	SPT	0	44	56	30	26	ML	46.5						NV	NP										
B-307	9	23.5	SPT	2	38	60			ML	31.0																	
B-307	10	28.5	SPT	0	58	42	37	5	SM	22.5																	
B-307	11	33.5	SPT	10	67	24			SM	23.8																	
B-307	12	38.5	SPT	0	54	46	41	5	SM	36.3																	
B-309	UD-1	8.5	UD	0	65	36	26	10	SM		32.3	12.4			22.4	NV	NP		83.65		90.72			87.19	107		
B-309	UD-2	18.5	UD	(b)	(b)	(b)			SM							(b)	(b)										
B-309	UD-3	28.5	UD	0	30	70	48	22	ML		28.6	26.8			27.7	NV	NP		77.83		85.07			81.45	104		
B-309	UD-4	38.5	UD	0	51	49			SM		21.7				21.7				88.60					88.60	108		
B-311	1	0	SPT	0	11	88	33	55	MH	30.9						70	19										

Table 2.5.4-207 (Sheet 7 of 8)
Summary of Laboratory Tests on Soil Samples

Source of Sample	Samp. No.	Depth (ft)	Samp. Type	Gravel[a] (%)	Sand[a] (%)	Fines[a] (%)	Silt[a] (%)	0.005 mm Clay[a] (%)	USCS Note[c]	Natural Moisture Content (%) at e_o						LL	PI	G_s	Dry Density (pcf) at e_o					Wet Density Avg. (pcf)	pH	Chloride (mg/kg)	Sulfate (mg/kg)
										SPT	Triaxial or Direct Shear 1	2	3	Cons.[d]	Avg.				Triaxial or Direct Shear 1	2	1	Cons .	Avg.				
B-311	2	1.5	SPT	0	26	74			MH	35.0																	
B-311	3	3.5	SPT	0	36	64	26	38	MH	30.5						77	25										
B-311	4	6	SPT	0	30	70			ML	34.1																	
B-311	5	8.5	SPT	0	49	51	34	17	ML	29.1						NV	NP										
B-311	6	11	SPT	0	68	32			SM	26.5																	
B-311	7	13.5	SPT	0	76	24			SM	20.0															5.3	4.5	6.0
B-311	8	18.5	SPT	0	10	90	75	15	ML	28.8																	
B-311	9	23.5	SPT	0	57	44			SM	24.6																	
B-311	10	28.5	SPT	0	28	72	53	19	ML	34.0																	
B-311	11	33.5	SPT	0	40	60			ML	35.0																	
B-311	12	38.5	SPT	0	34	66	50	16	ML	39.7																	
B-311	13	43.5	SPT	0	56	45			SM	43.2																	
B-311	14	48.5	SPT	0	75	25	22	3	SM	21.1																	
B-311	15	53.5	SPT	18	60	21			SM	13.4															5.9	2.9	7.3
B-317	1	0	SPT						MH	28.5						64	27										
B-317	2	1.5	SPT	3	81	16			SM	24.6																	
B-317	3	3.5	SPT	0	38	62	33	29	MH	26.1						58	11										
B-317	4	6	SPT	0	29	72	31	41	MH	29.5																	
B-317	5	8.5	SPT	0	92	8			SW-SM	24.4															5.0	6.5	14.5
B-317	6	11	SPT	0	33	67	43	24	MH	26.4																	
B-317	7	13.5	SPT	0	37	63	35	28	MH	33.2						57	16										
B-317	8	18.5	SPT	0	29	71			ML	31.8																	
B-317	9	23.5	SPT	1	54	44			SM	32.4																	
B-319	UD-2	18.5	UD	1	71	28			SM		19.50				19.5				91.60					91.60	109		
B-319	UD-3	28.5	UD						ML		22.90			26.8	24.9	NV	NP	2.75		89.36			94.34	91.85	115		
B-319	UD-4	38.5	UD						ML					19.6	19.6	NV	NP	2.75					102.8	102.8	123		
B-320	2	1.5	SPT	0	35	65	39	26	ML	23.9						NV	NP										
B-320	3	3.5	SPT	0	70	30			SM	29.5						NV	NP										
B-320	4	6	SPT	0	61	39	26	13	SM	20.4																	
B-320	5	8.5	SPT	0	63	37			SM	25.3															4.9	6.4	6.1[e]
B-320	6	11	SPT	0	62	38	31	7	SM	33.4																	

Table 2.5.4-207 (Sheet 8 of 8)
Summary of Laboratory Tests on Soil Samples

Source of Sample	Samp. No.	Depth (ft)	Samp. Type	Gravel[a] (%)	Sand[a] (%)	Fines[a] (%)	Silt[a] (%)	0.005 mm Clay[a] (%)	USCS Note[c]	Natural Moisture Content (%) at e_o SPT	Triaxial or Direct Shear 1	Triaxial or Direct Shear 2	Triaxial or Direct Shear 3	Cons.[d]	Avg.	LL	PI	G_s	Dry Density (pcf) at e_o Triaxial or Direct Shear 1	Triaxial or Direct Shear 2	Triaxial or Direct Shear 1	Cons .	Avg.	Wet Density Avg. (pcf)	pH	Chloride (mg/kg)	Sulfate (mg/kg)
B-320	7	13.5	SPT	0	65	35			SM	23.3																	
B-320	8	18.5	SPT	0	58	42			SM	30.0																	
B-320	9	23.5	SPT	0	69	31	27	4	SM	27.5																	
B-320	10	28.5	SPT	0	69	31			SM	22.5																	
B-320	11	33.5	SPT	0	73	27	23	4	SM	17.2																	
B-320	12	38.5	SPT	1	73	26			SM	24.1															6.0	7.3	16.6
B-320	13	43.5	SPT	0	46	54	49	5	ML	44.2						NV	NP										
B-321	UD-2	18.5	UD	0	66	34	25	9	SM		19.90	19.40			19.7	NV	NP		88.67		92.90			90.79	109		
B-321	UD-3	28.5	UD						SM					16.7	16.7	NV	NP	2.83					102.6	102.6	120		
B-322	UD-2	18.5	UD	0	71	29	20	9	SM		16.90	13.90	14.90		15.2	NV	NP		85.96		95.15	83.74		88.28	102		
B-325	2	1.5	SPT	0	56	44			SM	29.0																	
B-325	UD-1	3.5	UD	0	44	57			ML		38.00				38.0				78.20					78.2	108		
B-325	3	6	SPT	1	51	48	36	12	SM	39.9						NV	NP										
B-325	4	11	SPT	0	58	42	32	10	SM	18.0																	
B-325	UD-3	13.5	UD						SM		30.70			20.9	25.8	NV	NP	2.77		74.67			91.14	82.91	104		
B-325	5	16	SPT	0	65	34	26	8	SM	22.3						NV	NP										
B-325	UD-4	18.5	UD	(b)	(b)	(b)			(b)							(b)	(b)										
B-325	6	21	SPT	0	71	29			SM	35.6															5.6	3.4	10.3
B-325	7	26	SPT	0	71	29	22	7	SM	16.6						NV	NP										
B-325	8	31	SPT	1	67	32			SM	19.9																	
B-325	9	36	SPT	0	70	31	26	5	SM	16.4						NV	NP										
B-325	UD-8	38.5	UD						SM		23.50			18.5	21.0	NV	NP	2.69		93.47			101.3	97.39	118		
B-325	10	41	SPT	0	55	45	39	6	SM	23.9																	
B-325	11	46	SPT	2	34	64			ML	24.1																	
B-325	13	53.5	SPT	No Recovery																							

(a) Due to computer roundoff, particle size fractions may total 100 ±1. Fines include silt plus clay.
(b) These results included with RCTS tests in Reference 232.
(c) USCS symbol is based on visual-manual method (Reference 208) where incomplete classification testing was performed.
(d) Cons. = Consolidation
(e) Estimated result. Result is less than STL laboratory reporting limit. Actual value will not exceed values shown.
(f) The associated method blank contains the target analyte at a reportable level. The actual value may be less than value shown.

Table 2.5.4-208 (Sheet 1 of 4)
Summary of Unconfined Compression Tests on Rock Cores

Source of Sample	Depth (ft)	Rock Type	Length to Diameter Ratio	Unit Wt. (pcf)	Unconfined Compressive Strength (psi)	Unconfined Compressive Strength (psi) (L/D Correction)	Modulus (psi)	Poisson's Ratio	Type of Break	Maximum Mineral Grain Size > Diameter/10 (Y or N)
B-201	53.00	Granodiorite	2.18	171	22,918	23,134	—	—	Cone	Y
B-201	58.08	Granodiorite	2.21	171	23,056	23,298	7,830,000	0.35	Cone	Y
B-201	65.65	Granodiorite	2.22	170	9,361	9,464	—	—	Columnar	Y
B-201	70.70	Granodiorite	2.22	169	18,760	18,967	—	—	Columnar	Y
B-201	81.70	Granodiorite	2.21	170	24,258	24,512	8,080,000	0.35	Cone	Y
B-201	92.10	Granodiorite	2.22	168	23,593	23,858	—	—	Cone & Shear	Y
B-201	101.30	Quartz Diorite	2.19	181	28,396	28,675	—	—	Cone & Shear	N
B-201	109.73	Quartz Diorite	2.21	180	29,501	29,809	9,730,000	0.32	Cone & Shear	N
B-201	131.20	Quartz Diorite	2.21	184	23,027	23,269	—	—	Shear	N
B-201	151.53	Quartz Diorite	2.18	184	23,278	23,494	—	—	Shear	N
B-201	191.48	Quartz Diorite	2.23	185	19,005	19,222	9,390,000	0.30	Columnar	N
B-201	238.10	Quartz Diorite	2.19	183	25,081	25,325	—	—	Cone	N
B-201	271.23	Quartz Diorite	2.22	188	21,922	22,161	—	—	Columnar	N
B-201	311.90	Quartz Diorite	2.22	185	21,552	21,790	8,880,000	0.30	Shear	N
B-201	349.06	Biotite Gneiss	2.22	165	28,594	28,908	—	—	Shear	N
B-203	56.20	Quartz Diorite	2.00	185	28,367	28,372	9,190,000	0.32	Cone & Shear	N
B-203	61.45	Granodiorite	2.12	172	25,112	25,266	—	—	Cone	Y
B-203	63.10	Granodiorite	2.18	169	34,660	34,987	—	—	Cone & Shear	N
B-203	71.87	Granodiorite	2.12	182	29,052	29,231	10,110,000	0.30	Cone & Shear	N
B-203	83.13	Quartz Diorite to Migmatite	2.10	184	30,453	30,611	—	—	Cone	N
B-203	99.09	Quartz Diorite	2.13	184	22,418	22,566	—	—	Cone & Shear	N
B-203	114.55	Quartz Diorite	2.10	184	30,880	31,042	9,390,000	0.33	Cone & Shear	N
B-203	133.35	Quartz Diorite	2.10	184	24,139	24,264	—	—	Columnar	N
B-203	148.12	Quartz Diorite	2.18	183	22,777	22,991	—	—	Cone & Shear	N
B-205	68.50	Quartz Diorite	2.18	182	25,217	25,451	—	—	Columnar	Y
B-205	72.54	Quartz Diorite	2.24	181	24,074	24,360	9,990,000	0.30	Shear	N
B-205	91.40	Quartz Diorite	2.22	182	21,417	21,659	—	—	Cone & Shear	N
B-205	124.32	Quartz Diorite	2.20	184	29,753	30,056	—	—	Cone & Shear	N

Table 2.5.4-208 (Sheet 2 of 4)
Summary of Unconfined Compression Tests on Rock Cores

Source of Sample	Depth (ft)	Rock Type	Length to Diameter Ratio	Unit Wt. (pcf)	Unconfined Compressive Strength (psi)	Unconfined Compressive Strength (psi) (L/D Correction)	Modulus (psi)	Poisson's Ratio	Type of Break	Maximum Mineral Grain Size > Diameter/10 (Y or N)
B-205	155.50	Quartz Diorite	2.20	183	27,113	27,388	9,730,000	0.29	Cone & Shear	N
B-206	78.70	Quartz Diorite	2.11	181	25,164	25,310	9,030,000	0.34	Cone & Shear	N
B-206	79.55	Quartz Diorite	2.11	179	13,352	13,433	—	—	Shear	N
B-206	88.70	Granodiorite	2.12	170	24,578	24,729	—	—	Cone & Shear	Y
B-206	104.69	Quartz Diorite	2.11	180	25,308	25,450	6,830,000	0.21	Shear	N
B-206	125.02	Quartz Diorite	2.13	184	15,860	15,964	—	—	Cone & Shear	N
B-206	146.50	Quartz Diorite	2.14	186	22,782	22,954	—	—	Cone & Shear	Y
B-206	177.58	Quartzite	2.13	166	37,596	37,857	9,340,000	0.27	Columnar	N
B-206	212.50	Granodiorite	2.13	171	27,257	27,443	—	—	Cone & Shear	Y
B-207	52.00	Granodiorite	2.12	170	40,784	41,037	9,360,000	0.37	Columnar	Y
B-207	58.90	Granodiorite	2.11	169	34,459	34,654	—	—	Cone & Shear	N
B-207	80.63	Granodiorite	2.22	186	—[(a)]	—	—	—	NA	Y
B-207	121.30	Biotite Gneiss	2.11	167	37,211	37,435	9,500,000	0.31	Cone & Shear	N
B-207	159.15	Granodiorite	2.11	172	25,829	25,980	—	—	Cone & Shear	Y
B-215	54.25	Quartz Diorite	2.33	183	24,578	24,976	8,940,000	0.34	Cone & Shear	N
B-215	58.43	Quartz Diorite	2.33	182	18,644	18,942	—	—	Cone & Shear	N
B-215	66.45	Quartz Diorite	2.33	184	22,795	23,164	—	—	Cone & Shear	N
B-216	56.20	Biotite Amphibole Gneiss	2.22	184	15,322	15,495	—	—	Columnar	N
B-216	60.14	Biotite Amphibole Gneiss	2.22	192	25,838	26,126	8,520,000	0.20	Shear	N
B-217	76.05	Biotite Amphibole Gneiss	2.26	189	21,587	21,865	—	—	Cone	N
B-217	97.73	Biotite Amphibole Gneiss	2.24	179	33,847	34,262	10,970,000	0.34	Cone & Shear	N
B-217	104.85	Migmatite	2.31	180	32,087	32,577	—	—	Cone	Y
B-217	136.00	Quartz Diorite	2.31	182	20,760	21,069	—	—	Cone & Shear	Y
B-220	87.24	Hornblende Gneiss	2.25	193	20,133	20,385	—	—	Columnar	N
B-220	95.85	Hornblende Gneiss	2.28	191	20,711	20,997	12,310,000	0.23	Shear	N
B-301A	61.00	Granodiorite	2.20	188	31,666	31,991	—	—	Cone & Shear	N
B-301A	66.77	Granodiorite	2.20	171	24,115	24,364	8,110,000	0.31	Cone & Shear	Y
B-301A	76.72	Quartz Diorite	2.21	192	15,769	15,939	—	—	Columnar	N
B-301A	85.64	Quartz Diorite	2.19	191	25,084	25,322	—	—	Cone	N

Table 2.5.4-208 (Sheet 3 of 4)
Summary of Unconfined Compression Tests on Rock Cores

Source of Sample	Depth (ft)	Rock Type	Length to Diameter Ratio	Unit Wt. (pcf)	Unconfined Compressive Strength (psi)	Unconfined Compressive Strength (psi) (L/D Correction)	Modulus (psi)	Poisson's Ratio	Type of Break	Maximum Mineral Grain Size > Diameter/10 (Y or N)
B-301A	94.10	Quartz Diorite	2.20	190	22,789	23,026	9,130,000	0.29	Cone & Shear	N
B-301A	106.08	Quartz Diorite	2.21	182	24,938	25,206	—	—	Cone & Shear	N
B-301A	113.74	Quartz Diorite	2.21	184	27,770	28,068	—	—	Cone	N
B-301A	125.90	Migmatite	2.18	191	45,009	45,419	14,960,000	0.30	Crush	N
B-301A	156.23	Migmatite	2.19	171	22,941	23,168	—	—	Cone	Y
B-301A	195.18	Granodiorite	2.18	170	25,408	25,639	—	—	Cone & Shear	Y
B-301A	234.13	Quartz Diorite	2.19	179	23,704	23,940	8,200,000	0.28	Cone & Shear	N
B-301A	274.85	Quartz Diorite	2.19	183	29,359	29,639	—	—	Cone & Shear	N
B-301A	311.50	Migmatite/Quartz Diorite	2.19	167	27,306	27,573	—	—	Cone	Y
B-301A	349.10	Migmatite	2.20	168	28,813	29,102	7,570,000	0.35	Shear	N
B-305	61.00	Granodiorite	2.12	171	22,282	22,419	NA	NA	Cone & Shear	Y
B-305	62.90	Granodiorite	2.10	170	24,315	24,449	8,380,000	0.30	Cone & Shear	Y
B-305	73.50	Granodiorite Migmatite	2.11	189	41,021	41,252	—	—	Crush	N
B-305	95.23	Hornblende Gneiss	2.14	185	25,713	25,898	—	—	Cone & Shear	N
B-305	123.55	Amphibolite Schist	2.11	183	26,553	26,705	7,390,000	0.35	Columnar	N
B-305	165.15	Granodiorite	2.14	174	27,997	28,200	—	—	Cone & Shear	N
B-306	48.25	Granodiorite	2.10	172	22,091	22,210	—	—	Cone	Y
B-306	52.55	Quartz Diorite	2.11	188	31,079	31,257	9,370,000	0.28	Cone	Y
B-306	62.20	Hornblende Gneiss	2.11	191	37,616	37,833	—	—	Crush	N
B-306	76.43	Granodiorite	2.11	179	23,200	23,332	—	—	Cone & Shear	N
B-306	96.40	Quartz Diorite	2.12	188	26,164	26,324	—	—	Cone & Shear	N
B-306	123.47	Granodiorite	2.12	185	26,139	26,300	8,560,000	0.35	Cone & Shear	Y
B-306	152.19	Hornblende Gneiss	2.12	186	35,689	35,911	—	—	Cone	Y
B-306	187.60	Granodiorite	2.13	178	23,523	23,678	8,930,000	0.30	Cone & Shear	Y
B-307	41.08	Biotite Gneiss	2.11	167	26,350	26,505	—	—	Crush	N
B-307	49.10	Granodiorite	2.10	170	22,267	22,384	8,390,000	0.29	Shear	Y
B-307	69.32	Migmatite	2.12	186	29,760	29,944	—	—	Cone & Shear	N
B-307	99.05	Migmatite	2.06	181	22,227	22,297	—	—	Cone & Shear	N
B-307	134.45	Granodiorite Migmatite	2.10	172	21,305	21,415	9,020,000	0.35	Cone & Shear	Y

Table 2.5.4-208 (Sheet 4 of 4)
Summary of Unconfined Compression Tests on Rock Cores

Source of Sample	Depth (ft)	Rock Type	Length to Diameter Ratio	Unit Wt. (pcf)	Unconfined Compressive Strength (psi)	Unconfined Compressive Strength (psi) (L/D Correction)	Modulus (psi)	Poisson's Ratio	Type of Break	Maximum Mineral Grain Size > Diameter/10 (Y or N)
B-307	171.71	Granodiorite Migmatite	2.11	185	15,149	15,237	—	—	Cone & Shear/Split	N
B-317	50.75	Migmatite	2.24	186	55,506	56,169	—	—	Cone/Crush	N
B-317	71.48	Amphibole Schist	2.22	189	15,834	16,012	11,730,000	0.40	Cone	N
B-317	90.44	Migmatite Gneiss	2.22	167	33,255	33,622	—	—	Crush	Y
B-317	132.79	Migmatite	2.26	186	26,959	27,306	—	—	Cone & Shear	N
B-320	52.08	Migmatite	1.99	181	NA[b]	NA	—	—	NA	N
B-320	61.88	Migmatite	2.26	181	28,872	29,249	—	—	Cone & Shear	N
B-320	77.68	Migmatite	2.13	187	27,465	27,649	—	—	Cone & Shear	N
B-320	100.43	Granodiorite Migmatite	2.18	170	28,966	29,239	—	—	Columnar	N
B-325	60.31	Granodiorite	2.30	172	21,804	22,120	—	—	Cone & Shear	Y
B-325	67.58	Migmatite	2.27	176	24,286	24,615	9,110,000	0.30	Cone & Shear	N

(a) Specimen broke along mineral filled fracture during end preparation — specimen used for unit weight only.
(b) Specimen did not meet minimum length to diameter ratio for compressive strength - specimen used for unit weight only.

Table 2.5.4-209 (Sheet 1 of 2)
Summary of Engineering Properties — Units 2 and 3

Stratum[a]	I-II[e]	I-II[e]	I-II[e]	III	IV	V	Fill			
Description	**Silt/ Clay**	**Silty Sand**	**Residuum / Saprolite**	**PWR**	**MWR**	**Sound Rock**	**Structural Fill**	**CLSM**	**GAB**	**#57**
Elevation of top of layer (ft) — Unit 2	—	—	+418	+375	+370	+355	+395	(g)	(g)	(g)
Elevation of top of layer (ft) — Unit 3	—	—	+415	+365	+360	+355	+395	(g)	(g)	(g)
USCS symbol	ML-MH	SM	ML-MH-SM	—	—	—	SW	—	GW or GW/GM	GP
Total unit weight γ (pcf)		110	110	145	160	182	125	129	140	120
Natural water content, w, (%)	30	24	25	—	—	—	—	—	—	—
Fines content (%)	70	32	32	—	—	—	—	—	0-12	0-5
Atterberg limits[c]										
Liquid limit, LL	63-NV	—	NV	—	—	—	—	—	—	—
Plastic limit, PL	45-NP	—	NP	—	—	—	—	—	—	—
Plasticity index, PI	19-NP	—	NP	—	—	—	—	—	—	—
SPT N_{60}-value (blows/ft)	18	22	20	—	—	—	30	—	30	30
Undrained properties										
Undrained shear strength, s_u (ksf)	2.5[f]	—	—	—	—	—	—	—	—	—
Internal friction angle, ϕ, (deg)	—	—	—	—	—	—	—	—	—	—
Drained properties										
Effective cohesion, c′ (ksf)	0.25	0.25	0.25	—	—	—	—	—	—	—
Effective friction angle, ϕ' (deg)	30	30	30	—	—	—	36	—	38	37
Rock core recovery (%) [e]	—	—	—	—	0–90	90–100	—	—	—	—
RQD (%) [e]	—	—	—	—	0–50	80–100	—	—	—	—
Unconfined compressive strength, U (ksi)	—	—	—	2.2	10	25	—	133 psi	—	—
Elastic modulus (high strain), E_H	—	—	720 ksf	500 ksi	3,300 ksi	9,500 ksi	1,080 ksf	14,125 ksf	(h) 1,080 ksf	(h) 1,080 ksf
Elastic modulus (low strain), E_L	—	—	7,350 ksf	750 ksi	3,300 ksi	9,500 ksi	9,700 ksf	28,250 ksf	(h) 9,700 ksf	(h) 9,700 ksf
Shear modulus (high strain), G_H	—	—	270 ksf	185 ksi	1,250 ksi	3,800 ksi	400 ksf	5,650 ksf	(h) 400 ksf	(h) 400 ksf
Shear modulus (low strain), G_L	—	—	2,750 ksf	280 ksi	1,250 ksi	3,800 ksi	3,600 ksf	11,300 ksf	(h) 3,600 ksf	(h) 3,600 ksf

RN-12-031

RN-12-031

Table 2.5.4-209 (Sheet 2 of 2)
Summary of Engineering Properties — Units 2 and 3

Stratum[a]	I-II[e]	I-II[e]	I-II[e]	III	IV	V	Fill			
Description	**Silt/ Clay**	**Silty Sand**	**Residuum / Saprolite**	**PWR**	**MWR**	**Sound Rock**	**Structural Fill**	**CLSM**	**GAB**	**#57**
Shear wave velocity, V_s, (ft/sec)	—	—	900	3,000	6,000	10,000	960	1,680	(h) 960 ksf	(h) 960 ksf
Compression wave velocity, V_c, (ft/sec)	—	—	1,800	6,000	12,000	17,500	2,000	2,910	(h) 2,000 ksf	(h) 2,000 ksf
Consolidation characteristics										
Compression ratio, CR	—	—	0.190	—	—	—	—	—	—	—
Recompression ratio, RR	—	—	0.030	—	—	—	—	—	—	—
Coeff. of vertical subgrade reaction[f], k_1 (kcf)	—	—	240	—	—	—	600	—	(h) 600 ksf	(h) 600 ksf
Coefficient of sliding	—	—	0.35		0.70	0.70	0.50	—	(h) 0.50 ksf	(h) 0.50 ksf
Poisson's ratio, μ	—	—	0.33		0.33	0.24	0.35	0.25	0.35	0.35
Static earth pressure coefficients										
Active, K_a	—	—	0.33	—	—	—	0.26	—	(h) 0.26 ksf	(h) 0.26 ksf
Passive, K_p	—	—	3.0	—	—	—	3.85	—	(h) 3.85 ksf	(h) 3.85 ksf
At-rest, K_o	—	—	0.5	—	—	—	0.41	—	(h) 0.41 ksf	(h) 0.41 ksf

RN-12-031

(a) The values tabulated are for use as a design guideline only. Refer to specific boring logs, CPT logs, and laboratory test results for appropriate modifications at specific design locations.
(b) Values are for MH soils only. ML soils are nonviscous and nonplastic. NP: nonplastic, NV: nonviscous.
(c) Based on averaged values over 5-foot vertical intervals.
(d) Values are for 1-foot square plates or 1-foot diameter pipes. Adjustments are necessary to account for actual size of foundation or pipe.
(e) The parameters are provided for residuum/saprolite (including silt/clay and silty sand) below El. 400 ft.
(f) Undrained shear strength for silt/clay applies only to soils with measurable plasticity, which constitutes only a small portion of the material.
(g) Alternative fill materials will represent minor dispersed volumes within the fill mass. The fill mass will predominantly consist of Structural Fill. RN-12-031
(h) Value presented is identical to that for Structural Fill and thereby conservative. Due to limited extent of placements of alternate fill materials, use of the Structural Fill Value is recommended.

Table 2.5.4-210
Summary of Laboratory Test Results on Bulk Samples

Source of Sample	Depth	Material Description	Gravel (%)	Sand (%)	Fines (%)	Silt (%)	Clay (%)	USCS[a]	Natural Moisture (%)	LL	PL	PI	Max Dry Density (pcf)	Optimum Moisture (%)	CBR Soaked (at 0.1")	CBR Unsoaked (at 0.1")
Test Pit - TP-201	1'–6'	SAND, Silty (SM), Red, Micaceous	0	57	43	28	15	SM	23.4	NV	NP	NP	107.8	17.0	7.0	27.1
Test Pit - TP-227	3'–5'	SILT, Sandy (ML), Red, Micaceous	0	46	54	39	15	ML	27.8	NV	NP	NP	107.0	17.9	6.9	31.6
Test Pit - TP-301	0'–3'	SAND, Silty, (SM), Yellowish Brown, Micaceous	0	68	32	24	8	SM	21.1	NV	NP	NP	105.7	16.1	6.3	28.2
Test Pit - TP-405	0'–4'	SAND, Silty (SM), Dark Yellowish Brown, Micaceous	0	64	36	32	4	SM	27.3	NV	NP	NP	108.8	15.3	3.6	21.9
Test Pit - TP-MM1	n/a	SAND (SW), Dark Gray, Washed Granitic Screenings from Stockpile	2	95	3	-	-	SW[a]	5.0	—	—	—	122.9	10.7	21.9	32.4
Test Pit - TP-MM2	n/a	SAND (SW-SM) with Silt, Dark Gray, Unwashed Granitic Screenings from Stockpile	4	86	10	5	5	SW-SM[a]	1.7	—	—	—	125.2	8.2	25.8	29.2

(a) USCS symbol based on visual-manual examination (Reference 208) if no test performed for LL and PI.
See individual test reports for complete test results.

Table 2.5.4-211
Atterberg Limits — Units 2 and 3

BH No.	Depth (ft)	Fines (%)	USCS[a]	LL (%)	PI (%)	BH No.	Depth (ft)	Fines (%)	USCS[a]	LL (%)	PI (%)
B-201	1.5	43	SM	NV	NP	B-301	48.5	20	SM	NV	NP
B-201	6	42	SM	NV	NP	B-301	3.5	—	SM	NV	NP
B-201	13.5	37	SM	NV	NP	B-305	3.5	46	SM	NV	NP
B-201	28.5	39	SM	NV	NP	B-306	1.5	79	ML	NV	NP
B-204	18.5	—	ML	NV	NP	B-306	11	43	SM	NV	NP
B-204	28.5	—	ML	NV	NP	B-307	16	56	ML	NV	NP
B-205	1.5	78	ML	NV	NP	B-309	8.5	36	SM	NV	NP
B-205	18.5	62	ML	NV	NP	B-309	28.5	70	ML	NV	NP
B-205	43.5	38	SM	NV	NP	B-311	8.5	51	ML	NV	NP
B-205	48.5	44	SM	NV	NP	B-319	28.5	—	ML	NV	NP
B-206	38.5	36	SM	NV	NP	B-319	38.5	—	ML	NV	NP
B-207	0	30	SM	NV	NP	B-320	1.5	65	ML	NV	NP
B-207	11	25	SM	NV	NP	B-320	3.5	30	SM	NV	NP
B-207	33.5	35	SM	NV	NP	B-320	43.5	54	ML	NV	NP
B-209	38.5	—	ML	NV	NP	B-321	18.5	34	SM	NV	NP
B-210	8.5	—	ML	NV	NP	B-321	28.5	—	SM	NV	NP
B-210	28.5	—	ML	NV	NP	B-322	18.5	29	SM	NV	NP
B-210	38.5	—	ML	NV	NP	B-325	6	48	SM	NV	NP
B-211	1.5	35	SM	NV	NP	B-325	16	34	SM	NV	NP
B-211	6	30	SM	NV	NP	B-325	26	29	SM	NV	NP
B-211	11	38	SM	NV	NP	B-325	36	31	SM	NV	NP
B-211	18.5	44	SM	NV	NP	B-325	13.5	—	SM	NV	NP
B-211	28.5	28	SM	NV	NP	B-325	38.5	—	SM	NV	NP
B-211	38.5	31	SM	NV	NP	B-208	8.5	84	CH	59	31
B-215	11	29	SM	NV	NP	B-209	18.5	43	SM	55	12
B-215	8.5	—	SM	NV	NP	B-209	8.5	—	MH	56	11
B-215	18.5	—	SM	NV	NP	B-220	6	—	MH	73	20
B-216	1.5	43	SM	NV	NP	B-306	23.5	77	MH	62	13
B-216	18.5	68	ML	NV	NP	B-307	0	—	MH	63	25
B-216	6.5	95	ML	NV	NP	B-307	1.5	93	MH	76	19
B-216	13.5	83	ML	NV	NP	B-311	0	88	MH	70	19
B-216	23.5	84	ML	NV	NP	B-311	3.5	64	MH	77	25
B-217	1.5	67	ML	NV	NP	B-317	13.5	63	MH	57	16
B-217	10.5	44	SM	NV	NP	B-317	3.5	62	MH	58	11
B-217	8.5	35	SM	NV	NP	B-317	0	—	MH	64	27
B-222	8.5	—	ML	NV	NP				Max:	77	31
B-222	18.5	—	ML	NV	NP				Min:	55	11
B-301	13.5	29	SM	NV	NP				Average:	64	19
B-301	28.5	24	SM	NV	NP				Median:	63	19

(a) USCS symbol is based on visual/manual method (Reference 208) where incomplete classification testing was performed.
NP = nonplastic
NV = nonviscous
— = not tested

Table 2.5.4-212
Laboratory Strength Test Results — Units 2 and 3

						UU[a]	CU[a]		Direct Shear		
BH No.	Sample No.	Depth (ft)	USCS	Fines (%)	PI	s_u (ksf)	c' (ksf)	ϕ' (deg)	Normal stress (ksf)	Failure stress (ksf)	ϕ' (deg)
B-208	UD-1	8.5	CH	84	31		0.22	30.0			
B-209	UD-1	8.5	MH	—	11	2.8					
B-204	UD-2	18.5	ML	—	NP	3.4[b]					
B-209	UD-4	38.5	ML	—	11	4.1					
B-210	UD-1	8.5	ML	—	NP	3.2[b]					
B-210	UD-3	28.5	ML	—	NP	3.2[b]					
B-216	UD-1	6.5	ML	95	NP		0.45	17.3			
B-216	UD-2	13.5	ML	83	NP		0.00	37.1	‘		
B-216	UD-3	23.5	ML	84	NP		0.07	31.2			
B-222	UD-2	18.5	ML	—	NP	2.4[b]					
B-309	UD-3	28.5	ML	70	NP		0.66	26.8			
B-319	UD-3	28.5	ML	—	NP	4.0[b]					
B-325	UD-1	3.5	ML	57	—				0.7	0.8	48.7
				Min:		2.4	0.00	17.3	—	—	—
				Max:		4.1	0.66	37.1	—	—	—
				Average:		3.3	0.28	28.5	—	—	—
				Median:		3.2	0.22	30.0	—	—	—
B-209	UD-2	18.5	SM	43	12		0.00	30.5			
B-215	UD-1	8.5	SM	—	NP	2.5					
B-215	UD-2	18.5	SM	—	NP	1.2					
B-215	UD-3	28.5	SM	30	—				3.6	1.6	24.2
B-217	UD-1	8.5	SM	35	NP		0.52	23.6			
B-222	UD-3	28.5	SM	36	—				3.6	3.0	39.6
B-309	UD-1	8.5	SM	36	NP		0.33	27.1			
B-309	UD-4	38.5	SM	49	—				5.0	2.5	26.5
B-319	UD-2	18.5	SM	28	—				2.2	1.5	35.1
B-321	UD-2	18.5	SM	34	NP		0.27	30.8			
B-322	UD-2	18.5	SM	29	NP		0.64	24.6			
B-325	UD-3	13.5	SM	—	NP	3.1					
B-325	UD-8	38.5	SM	—	NP	3.8					
				Min:		1.2	0.00	23.6	—	—	24.2
				Max:		3.8	0.64	30.8	—	—	39.6
				Average:		2.6	0.35	27.3	—	—	31.4
				Median:		2.8	0.33	27.1	—	—	30.8

(a) UU: unconsolidated undrained triaxial test; CU: consolidated undrained triaxial test.
(b) Nonplastic soils should have no undrained shear strengths. For those, the values shown just represent half of the deviator stress.

Table 2.5.4-213
Consolidation Properties — Units 2 and 3

BH No.	Sample No.	Depth (ft)	Description	e_o	C_c	C_e	CR	RR	CR/RR	p'_o (ksf)	Specific Gravity
B-204	UD-2	18.5	silty sand	0.81	0.249	0.030	0.138	0.017	8.3	9.5	2.87
B-204	UD-3	28.5	silty sand	1.11	0.492	0.060	0.233	0.028	8.2	20.3	2.95
B-209	UD-1	8.5	sandy elastic silt	1.51	0.734	0.040	0.292	0.016	18.4	19.7	2.81
B-209	UD-4	38.5	silty sand	1.01	0.379	0.070	0.189	0.035	5.4	33.3	2.86
B-210	UD-1	8.5	silty sand	0.94	0.350	0.040	0.180	0.021	8.8	20.7	2.75
B-210	UD-3	28.5	silty sand	0.72	0.230	0.050	0.134	0.029	4.6	33.3	2.73
B-210	UD-4	38.5	silty sand	1.05	0.340	0.030	0.166	0.015	11.3	11.9	2.78
B-215	UD-1	8.5	silty sand	0.97	0.395	0.050	0.201	0.025	7.9	33.5	2.78
B-215	UD-2	18.5	silty sand	1.25	0.520	0.060	0.231	0.027	8.7	5.2	2.82
B-222	UD-1	8.5	silty sand	0.87	0.320	0.060	0.171	0.032	5.3	33.3	2.71
B-222	UD-2	18.5	silty sand	0.91	0.370	0.060	0.194	0.031	6.2	7.8	2.83
			Min:	**0.72**	**0.230**	**0.030**	**0.134**	**0.015**	4.600	5.2	**2.71**
			Max:	**1.51**	**0.734**	**0.070**	**0.292**	**0.035**	18.350	33.5	**2.95**
			Average:	**1.01**	**0.398**	**0.050**	**0.193**	**0.025**	8.456	20.8	**2.81**
			Median:	**0.97**	**0.370**	**0.050**	**0.189**	**0.027**	8.200	20.3	**2.81**
B-319	UD-3	28.5	silty sand	0.75	0.279	0.060	0.159	0.034	4.7	29.1	2.75
B-319	UD-4	38.5	silty sand	0.67	0.150	0.040	0.090	0.024	3.8	3.6	2.75
B-321	UD-3	28.5	silty sand	0.72	0.186	0.060	0.108	0.035	3.1	16.7	2.83
B-325	UD-3	13.5	silty sand	0.90	0.352	0.050	0.185	0.026	7.0	33.1	2.77
B-325	UD-8	38.5	silty sand	0.66	0.153	0.050	0.092	0.030	3.1	11.0	2.69
			Min:	**0.66**	**0.150**	**0.040**	**0.090**	**0.024**	3.060	3.6	**2.69**
			Max:	**0.90**	**0.352**	**0.060**	**0.185**	**0.035**	7.040	33.1	**2.83**
			Average:	**0.74**	**0.224**	**0.052**	**0.127**	**0.030**	4.320	18.7	**2.76**
			Median:	**0.72**	**0.186**	**0.050**	**0.108**	**0.030**	3.750	16.7	**2.75**

Notes:
C_c: compression index
C_e: recompression index
p'_o: preconsolidation pressure
e_o: initial void ratio
CR: compression ratio
RR: recompression ratio

Table 2.5.4-214
Guidelines for Soil Corrosiveness and Aggressiveness

Soil Property	Property Value Range for Steel Corrosiveness				
	Little Corrosive	Mildly Corrosive	Moderately Corrosive	Corrosive	Very Corrosive
Resistivity, ohm-m	>100[a],[b]	20-100[a] 50-100[b] >30[c]	10-20[a] 20-50[b]	5-10[a] 7-20[b]	<5[a] <7[b]
pH		>5.0 and <10[b]		5.0-6.5[a]	<5.0[a]
Chlorides, ppm		<200[b]		300-1000[a]	>1000[a]

Recommendations for Normal Weight Concrete Subject to Sulfate Attack[d]			
Concrete Exposure	Water Soluble Sulfate (SO_4) in Soil, Percent	Cement Type[e]	Water Cement Ratio (Maximum)
Mild	0.00-0.10	—	—
Moderate	0.10-0.20	II, IP(MS), IS(MS)	0.5
Severe	0.20-2.0	V[f]	0.45
Very Severe	Over 2.0	V with pozzolan	0.45

(a) From Reference 202
(b) From Reference 244
(c) From Reference 244, provided 5<pH<10, chlorides <200 ppm, and sulfates <1,000 ppm
(d) From Reference 201
(e) Per Reference 203 or Reference 204
(f) Or a blend of Type II cement and a ground granulated blast furnace slag or a possolan that gives equivalent sulfate resistance.

Table 2.5.4-215
Chemical Test Results — Units 2 and 3

Source of Sample	Sample No.	Depth (ft)	USCS Note[(a)]	pH	Chloride (mg/kg)	Sulfate (mg/kg)
B-201	4	6	SM	5.4	4.1	5.1[(b)]
B-201	9	23.5	SM	5.6	3.9	6.0[(b)]
B-201	11	33.5	SW-SM	6.0	1.9 [(b)]	7.5
B-205	4	6	ML	5.3	4.5	5.6[(b)]
B-206	8	18.5	SM	5.2	4.2	6.2
B-207	5	8.5	SM	5.4	5.8	15.4
B-211	9	23.5	SM	5.7	3.3 [(c)]	3.5[(b)]
B-215	8	33.5	SM	5.6	1.9[(b)(c)]	3.0
B-216	7	28.5	GM	6.0	1.8[(b)]	4.6[(b)]
B-217	5	10.5	SM	5.4	5.9	3.3[(b)]
B-220	5	8.5	MH	5.5	3.4	3.7[(b)]
B-301	4	6	SM	5.7	4.7	12.0
B-301	8	18.5	SM	5.3	3.2	4.0[(b)]
B-305	5	8.5	SM	5.2	8.5	4.0[(b)]
B-306	6	11	SM	5.2	7.0	5.4[(b)]
B-307	4	6	MH	5.2	8.4	6.7
B-311	7	13.5	SM	5.3	4.5	6.0
B-311	15	53.5	SM	5.9	2.9	7.3
B-317	5	8.5	SW-SM	5.0	6.5	14.5
B-320	5	8.5	SM	4.9	6.4	6.1[(b)]
B-320	12	38.5	SM	6.0	7.3	16.6
B-325	6	21	SM	5.6	3.4	10.3
			Min:	4.9	1.8	3.0
			Max:	6.0	8.5	16.6
			Average:	5.5	4.7	7.1
			Median:	5.4	4.4	6.0
				pH	ppm[(d)]	%[(e)]
1 mg/kg = 1 ppm 10,000 mg/kg = 1%			**Min:**	4.9	1.8	0.0003
			Max:	6.0	8.5	0.0017
			Average:	5.5	4.7	0.0007
			Median:	5.4	4.4	0.0006

(a) USCS symbol is based on visual-manual method (Reference 208) where incomplete classification testing was performed.
(b) Estimated result. Result is less than STL laboratory reporting limit. Actual value will not exceed values shown.
(c) The associated method blank contains the target analyte at a reportable level. The actual value may be less than value shown.
(d) 1 mg/kg = 1 ppm
(e) 10,000 mg/kg = 1%

Table 2.5.4-216
Borings and CPTs Referenced in Liquefaction Analysis

Boring/ CPT No.	Ground Elevation (ft)	Estimated Groundwater Elevation (ft)	Liquefaction Groundwater El. (ft)	Top of PWR El. (ft)	Number of Samples
B-212 (OW)	397.2	351.1	356.1	346.7	14
B-213 (OW)	401.5	359.1	364.1	343.5	15
B-233 (OW)	426.1	355.0	359.0	388.6	11
B-234	421.1	350.0	355.0	366.1	15
B-235	379.4	350.0	355.0	330.9	13
B-236/236A	374.7	357.0	362.0	342.7	10
B-311	419.5	360.0	365.0	366.0	14
B-327 (OW)	410.8	359.3	364.3	362.3	13
B-330	401.6	355.0	360.0	353.1	13
B-332	398.4	351.0	356.0	354.9	12
B-333 (OW)	394.4	345.0	350.0	372.4	8
B-334	418.7	353.0	358.0	386.7	19
B-423	408.0	358.0	363.0	346.0	16
B-603	429.3	353.0	358.0	381.3	13
B-606	424.2	350.0	355.0	377.2	13
B-609	406.1	345.0	350.0	—	14
B-613	412.8	362.0	367.0	—	12
C-210	367.7	353.0	358.0	—	—
C-308	398.9	351.0	356.0	—	—

Table 2.5.4-217
Major Structures — Units 2 and 3

Structure	Seismic Category	Subsurface	Elevation of Base of Foundation (ft)	Width	Length	Case I	Case II	
				B (ft)	L (ft)	BxL (ftxft)		
Nuclear Island	I	Rock	360.5	90 to 160	255	90x255	160x255	RN-14-051
Annex	II	Fill (SW)	395	65	285	65x285	—	RN-12-012 RN-12-029
Turbine	II and Non-seismic	Fill (SW) or Concrete	395–365	155	300	155x300	—	
Radwaste	Non-seismic	Fill (SW)	395	70	150	70x150	—	

Table 2.5.4-218
Allowable Bearing Capacity of Rock

Rock Layer)	Design Allowable Bearing Capacity (ksf)
Layer V	200
Layer IV	100
Layer III	40

Table 2.5.4-219
Allowable Bearing Capacity of Major Structures

Structure	Subsurface	BxL (ftxft)		q_{allow}[(b),(c)](ksf)		q_{allow}(ksf)
		Case I	Case II	Case I	Case II	Recommended
Nonsafety-related	Silty sand [(a)]	5 x 5	5 x 50	5.96	4.44	4
	Silt/clay[(a)]	5 x 5	5 x 50	5.16	4.41	4
Nuclear Island	Sound Rock/ Concrete	90 x 255	160 x 255	160	160	160
Turbine	Fill (SW) or Concrete	155 x 300	—	75.5	—	75
Annex	Fill (SW)	65 x 285	—	37.5	—	35
Radwaste	Fill (SW)	70 x 150	—	36.6	—	35

RN-12-012
RN-12-029

(a) The soil type reflects the composition of the residuum/saprolite layer beneath the non-safety related structure being considered. Silty sand soils constitute a major portion of the residuum/saprolite layer compared to silt/clay soils.
(b) Factor of safety of 3 is used in the analyses.
(c) Groundwater level is assumed to be at the ground surface.
.

RN-12-012
RN-12-029

Table 2.5.4-220
Anticipated Settlement of Major Structures

Structure	Contact Pressure (ksf)	Subsurface	BxL (ftxft)		Anticipated Settlement (in.)		
			Case I	Case II	Center	Mid of side	Mean
Nonsafety- related	4.0	Residuum/ Saprolite	5 x 5	5 x 50	0.3—0.6	0.2–0.5	0.3–0.6
Nuclear Island	8.9	Sound Rock/ Concrete	90 x 255	160 x 255	0.02	0.01	0.015
Turbine	6.0	Fill (SW) or Concrete	155 X 300	—	<3.0	<1.5	<2.2
Annex	6.0	Fill (SW)	65 x 285	—	2.7–3.0	1.5	2.2
Radwaste	6.0	Fill (SW)	70 x 150	—	3.0	1.6	2.3

RN-12-012
RN-12-029

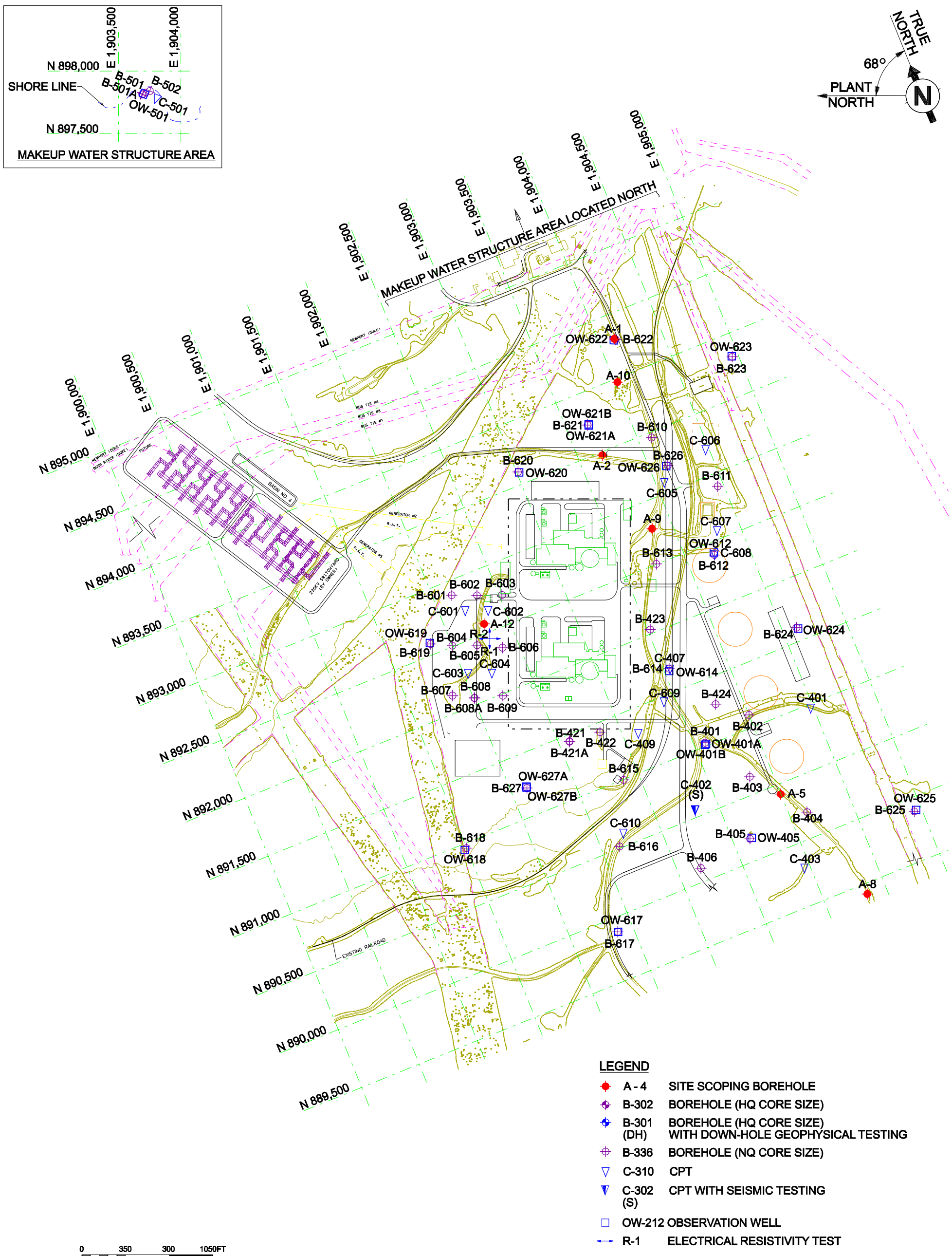


Figure 2.5.4-201 Boring Location Plan (Out of Power Block)

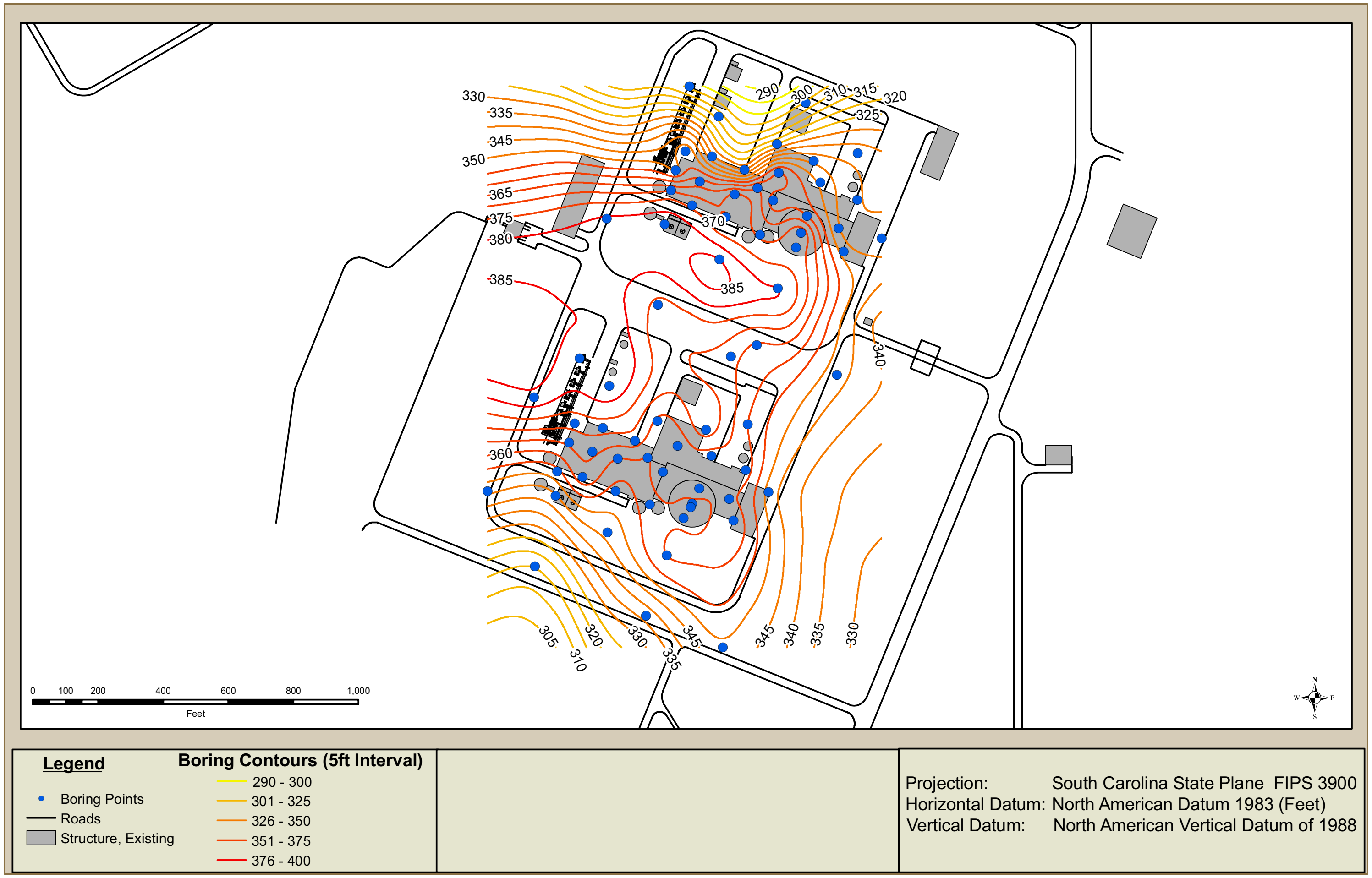


Figure 2.5.4-202 Top of Layer V (Sound Rock) Contour

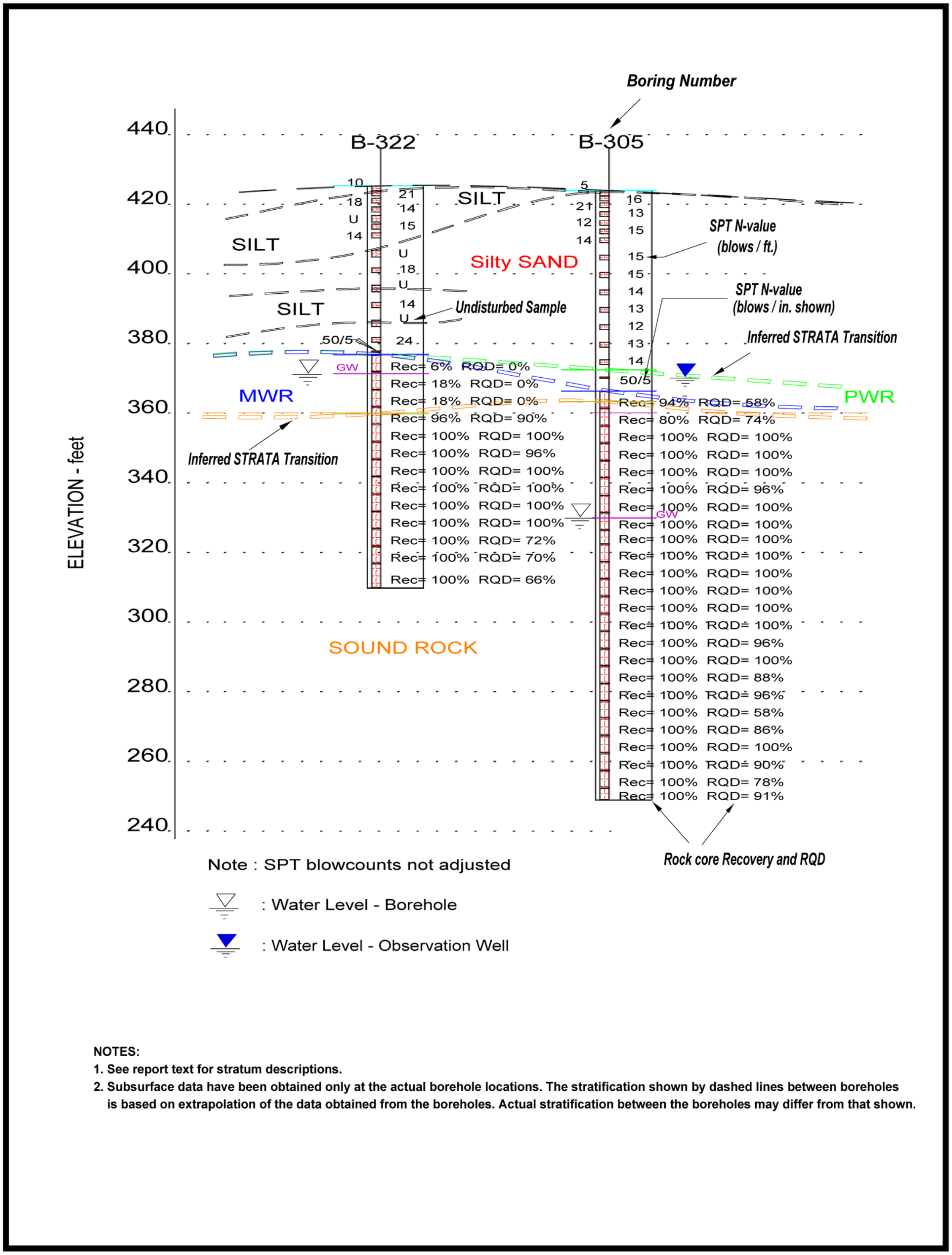


Figure 2.5.4-203 Subsurface Profile Legend

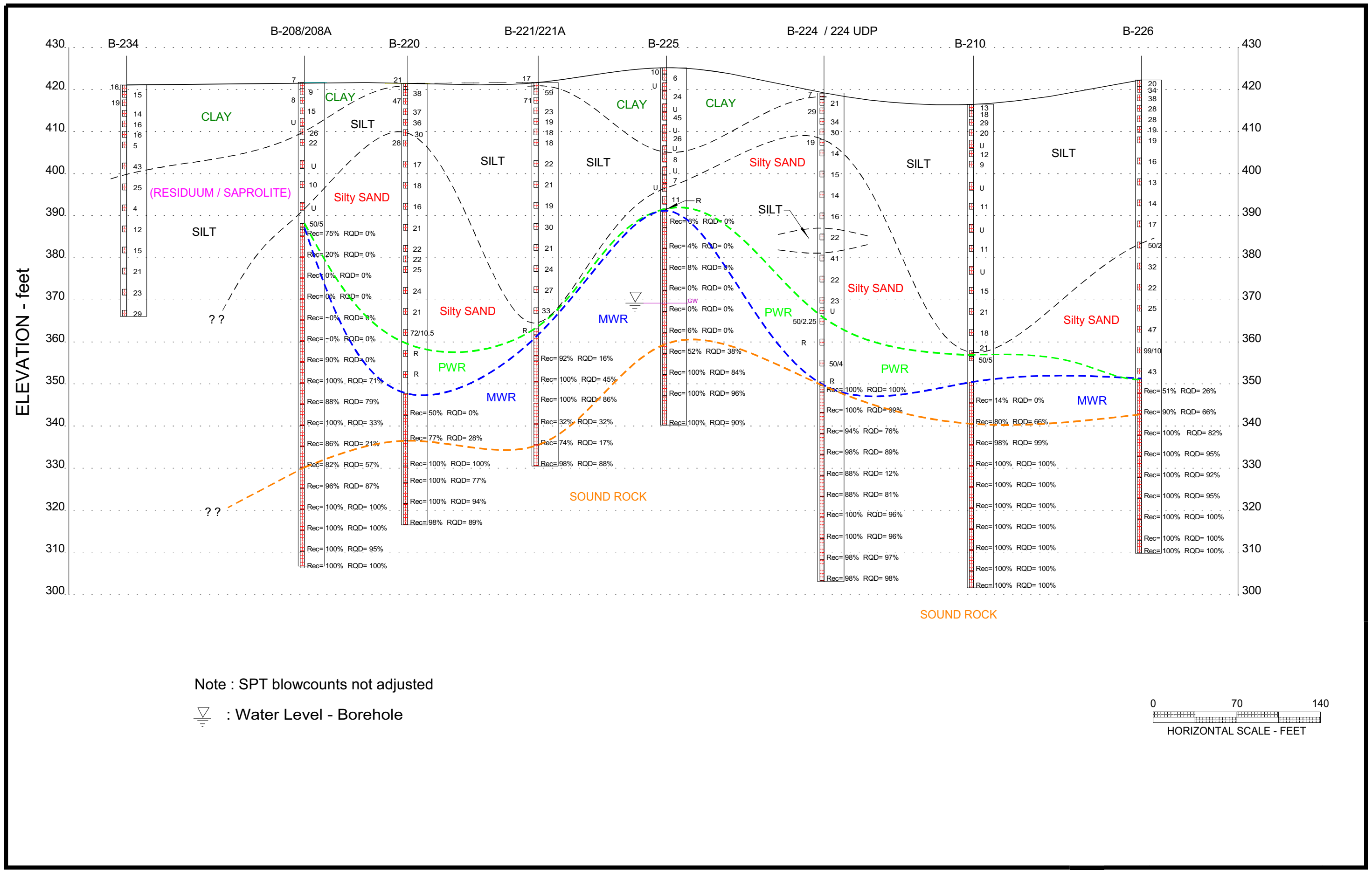


Figure 2.5.4-204 Inferred Subsurface Profiles Unit 2 East-West: A-A (Sheet 1 of 2)

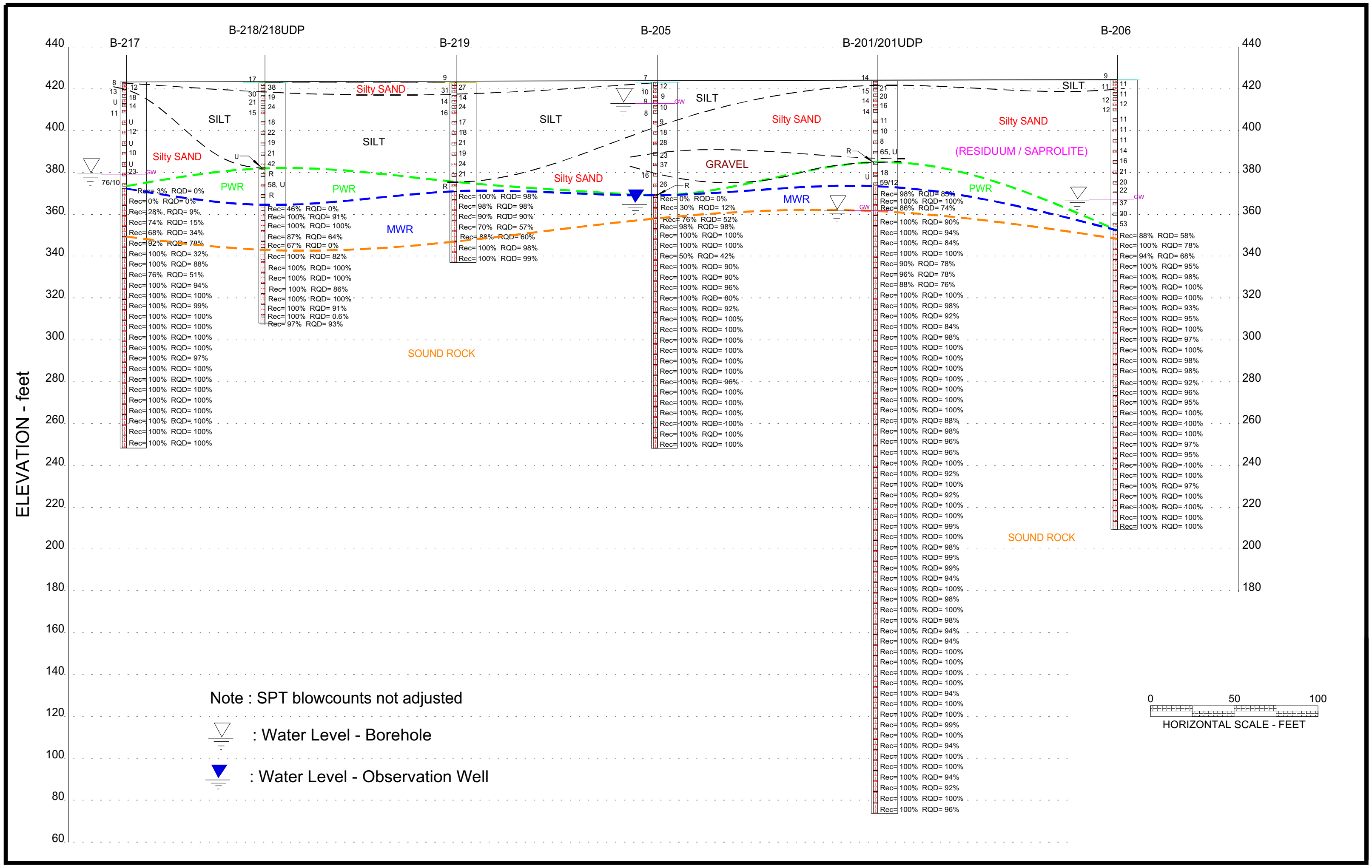


Figure 2.5.4-204 Inferred Subsurface Profiles Unit 2 East-West: B-B (Sheet 2 of 2)

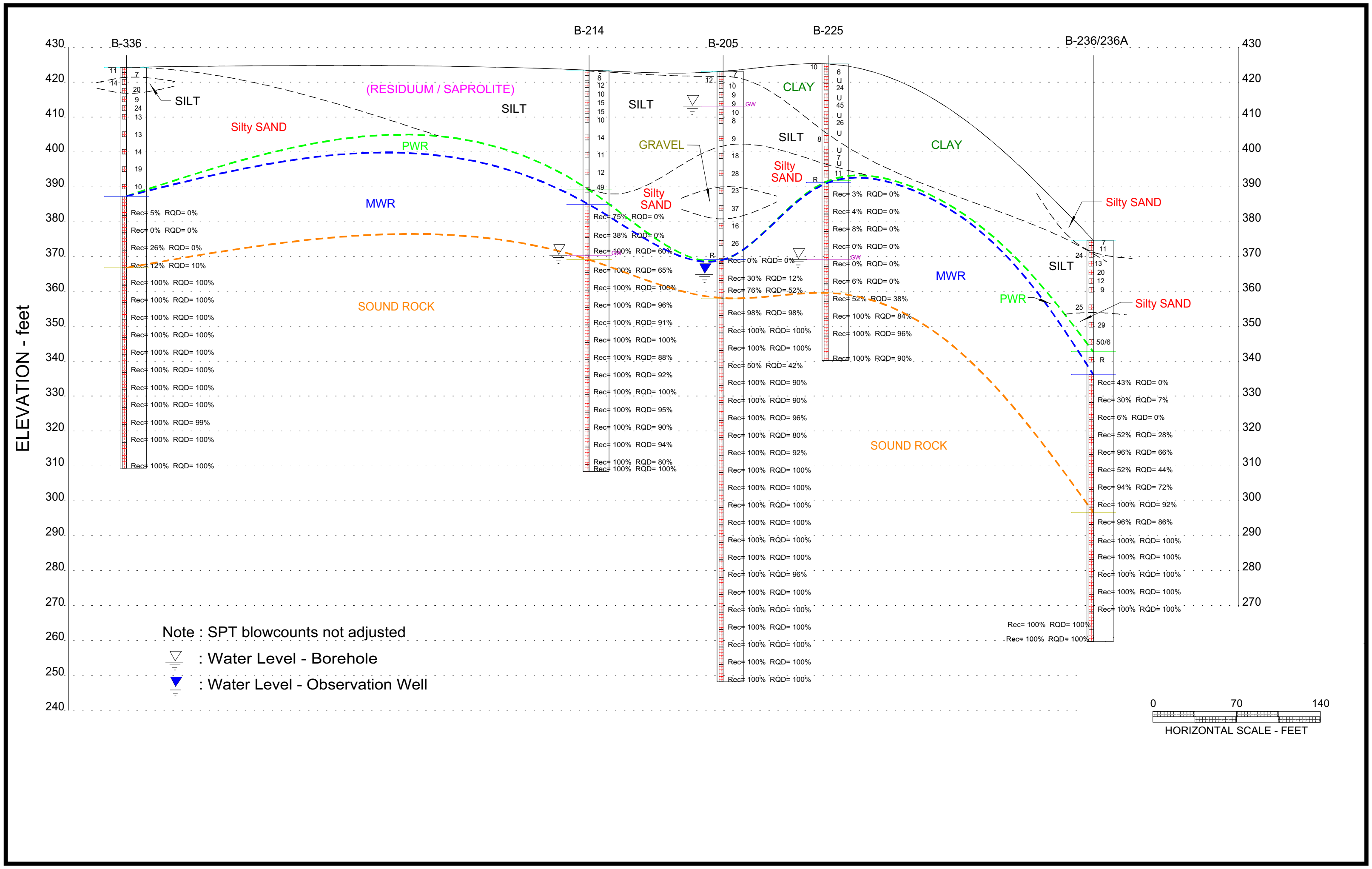


Figure 2.5.4-205 Inferred Subsurface Profiles Unit 2 North-South: E-E (Sheet 1 of 4)

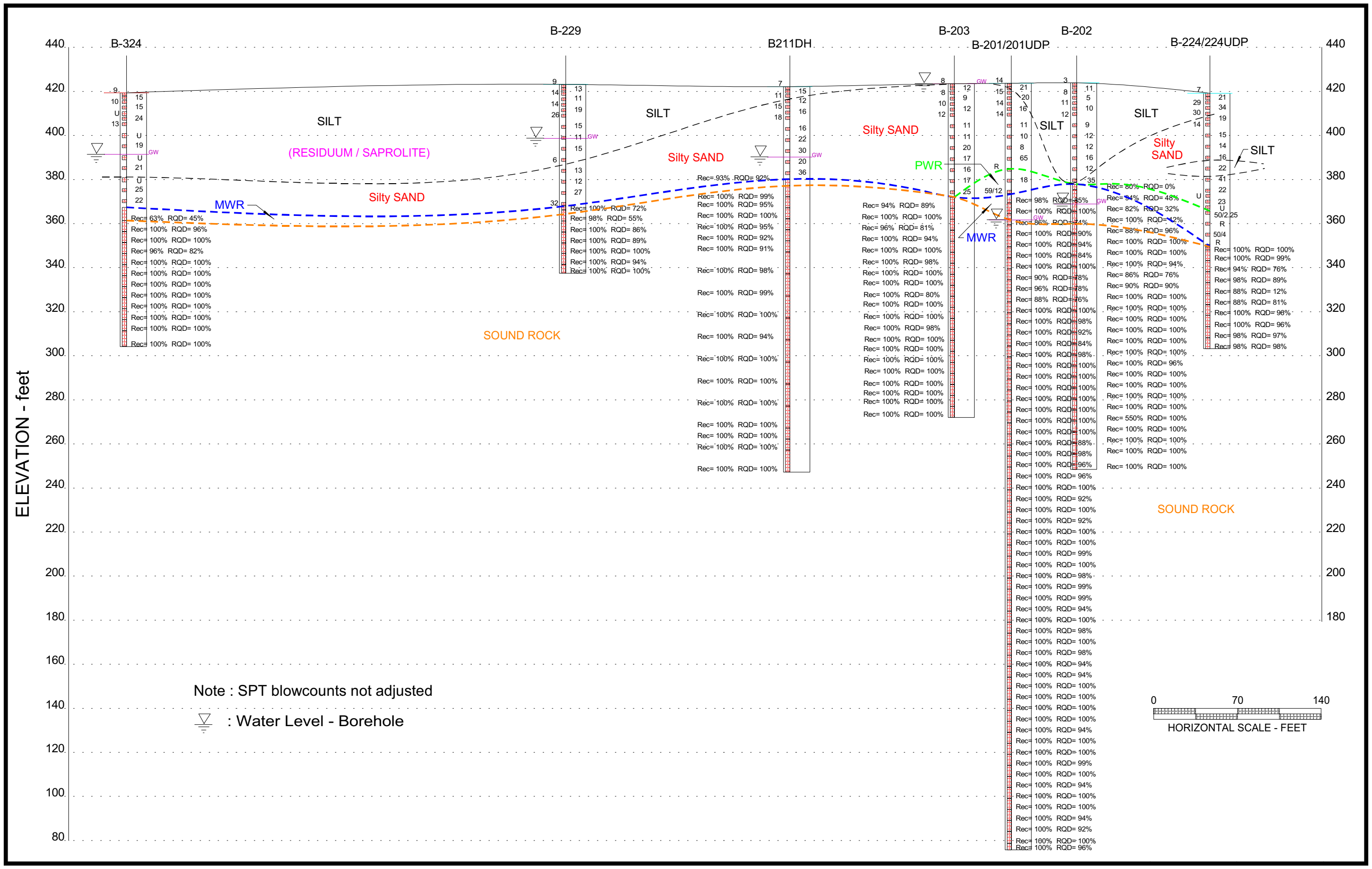


Figure 2.5.4-205 Inferred Subsurface Profiles Unit 2 North-South: F-F (Sheet 2 of 4)

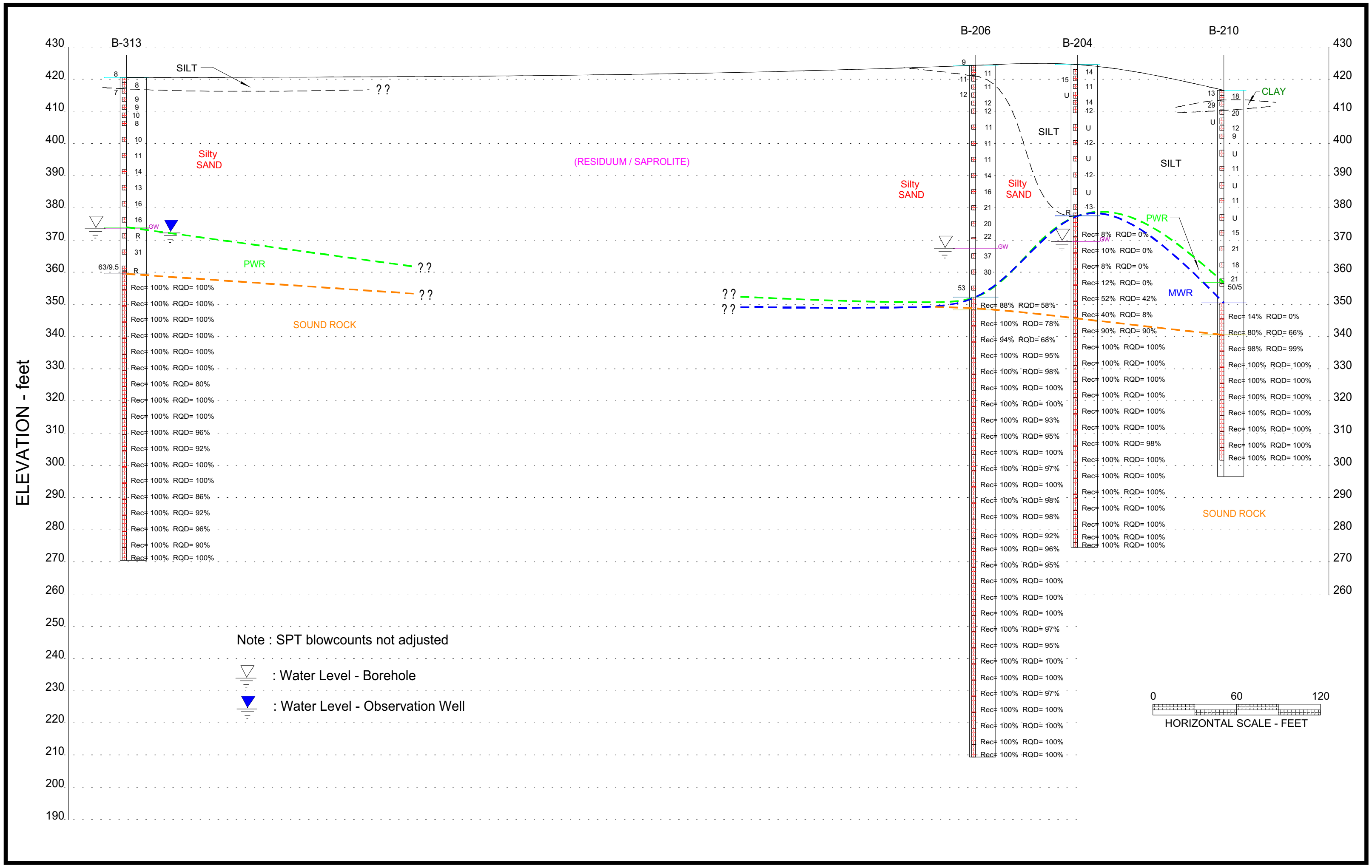


Figure 2.5.4-205 Inferred Subsurface Profiles Unit 2 North-South: G-G (Sheet 3 of 4)

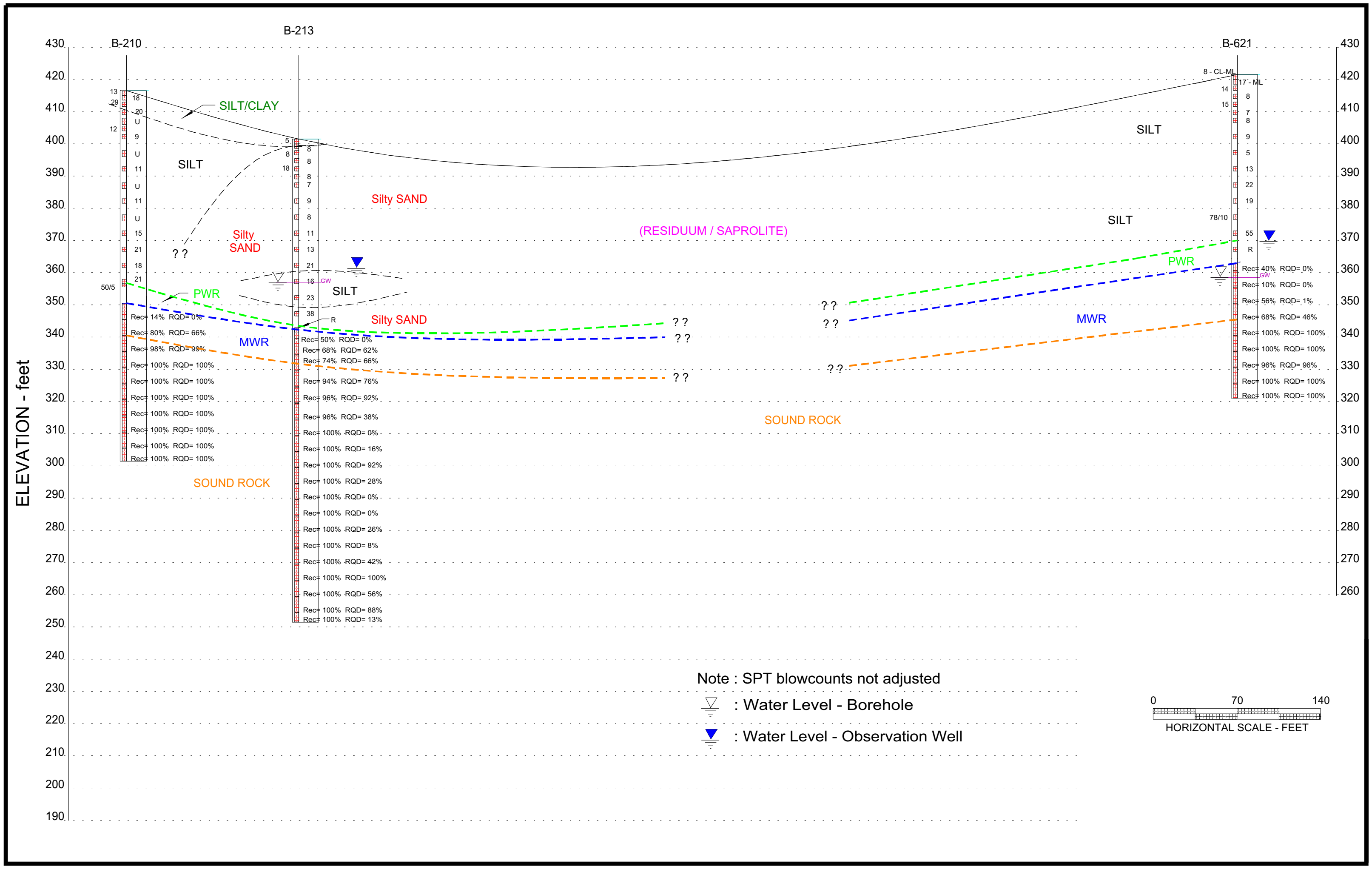


Figure 2.5.4-205 Inferred Subsurface Profiles Unit 2 North-South: H-H (Sheet 4 of 4)

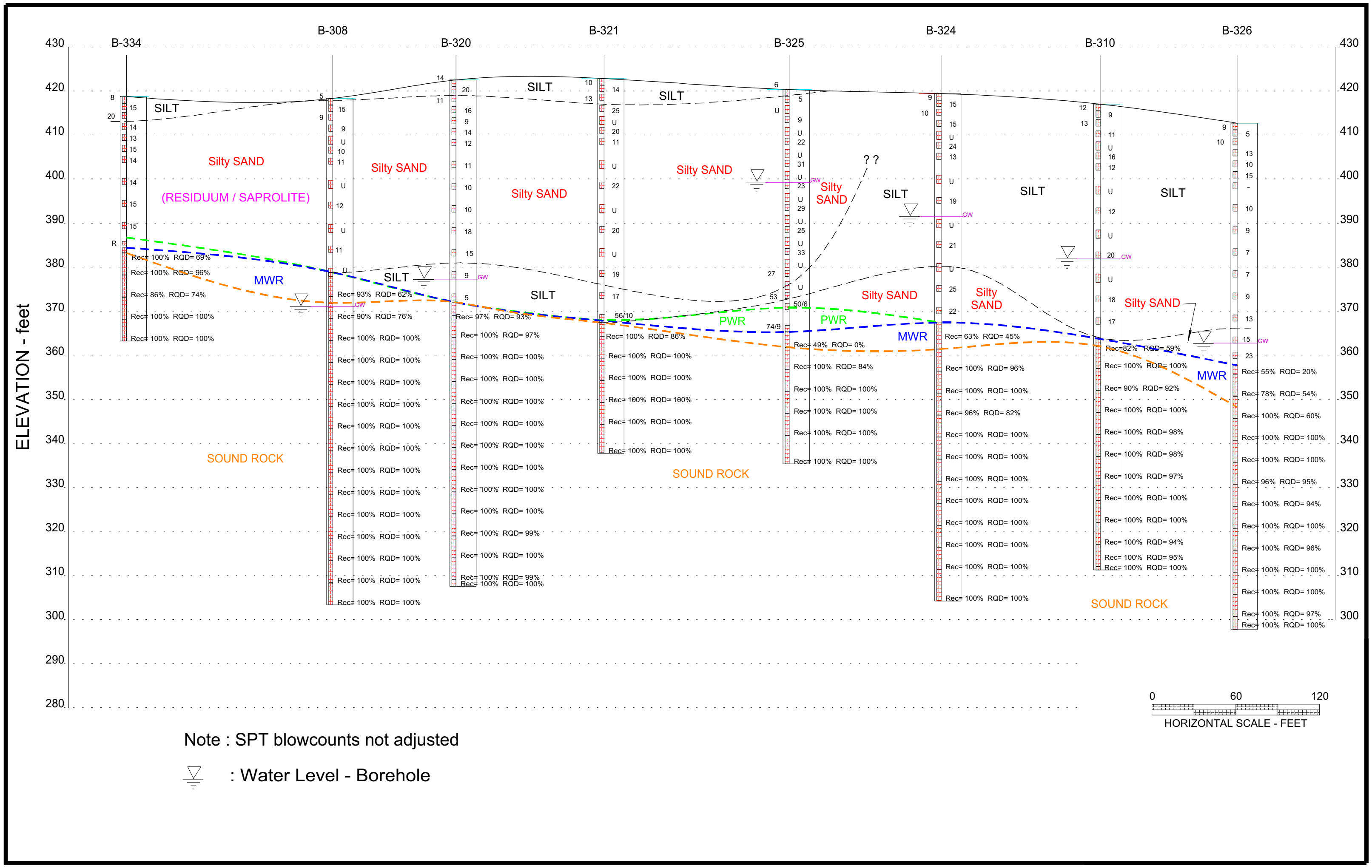


Figure 2.5.4-206 Inferred Subsurface Profiles Unit 3 East-West: C-C (Sheet 1 of 2)

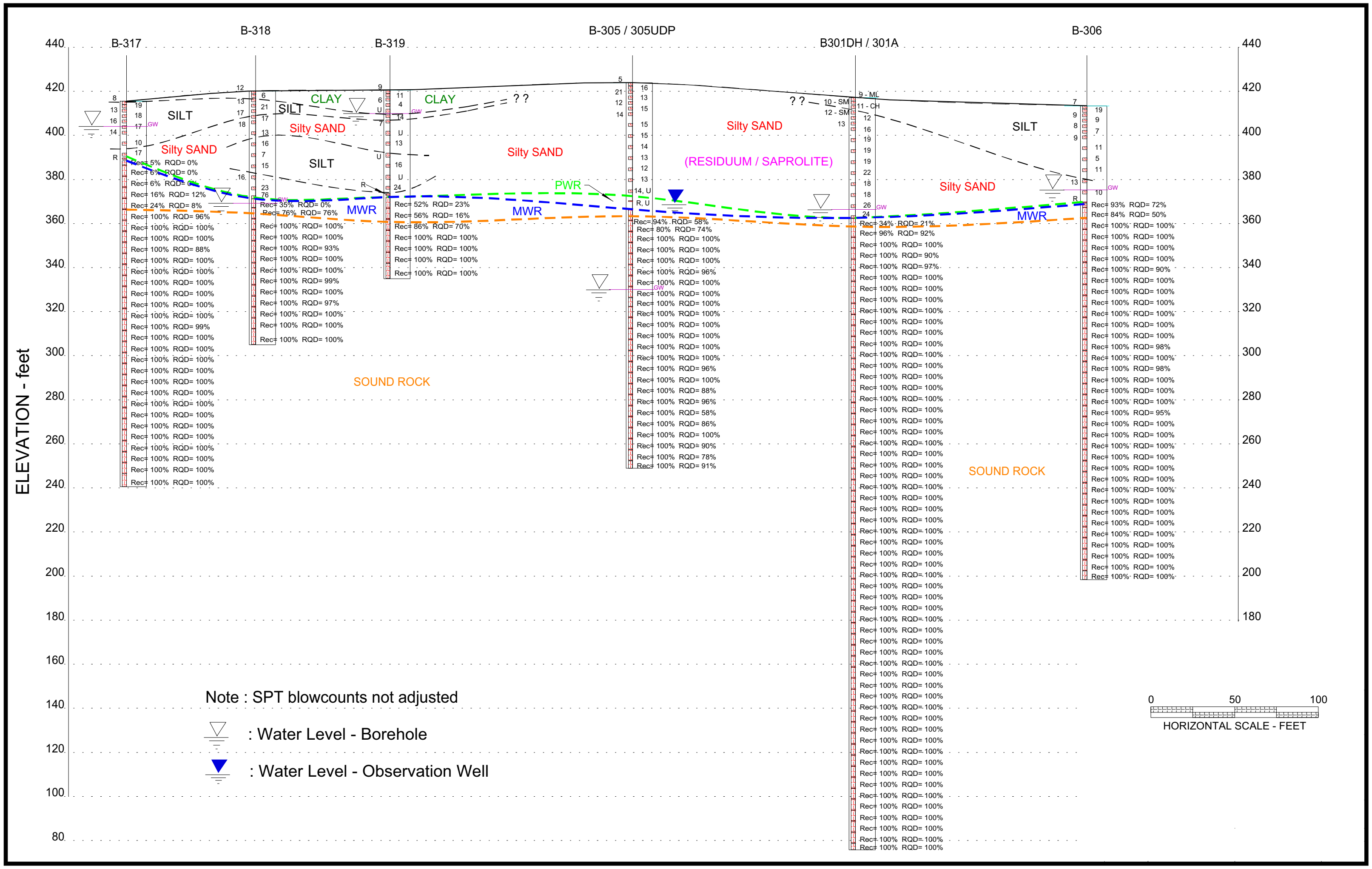


Figure 2.5.4-206 Figure 2.5.4-206 Inferred Subsurface Profiles Unit 3 East-West: D-D (Sheet 2 of 2)

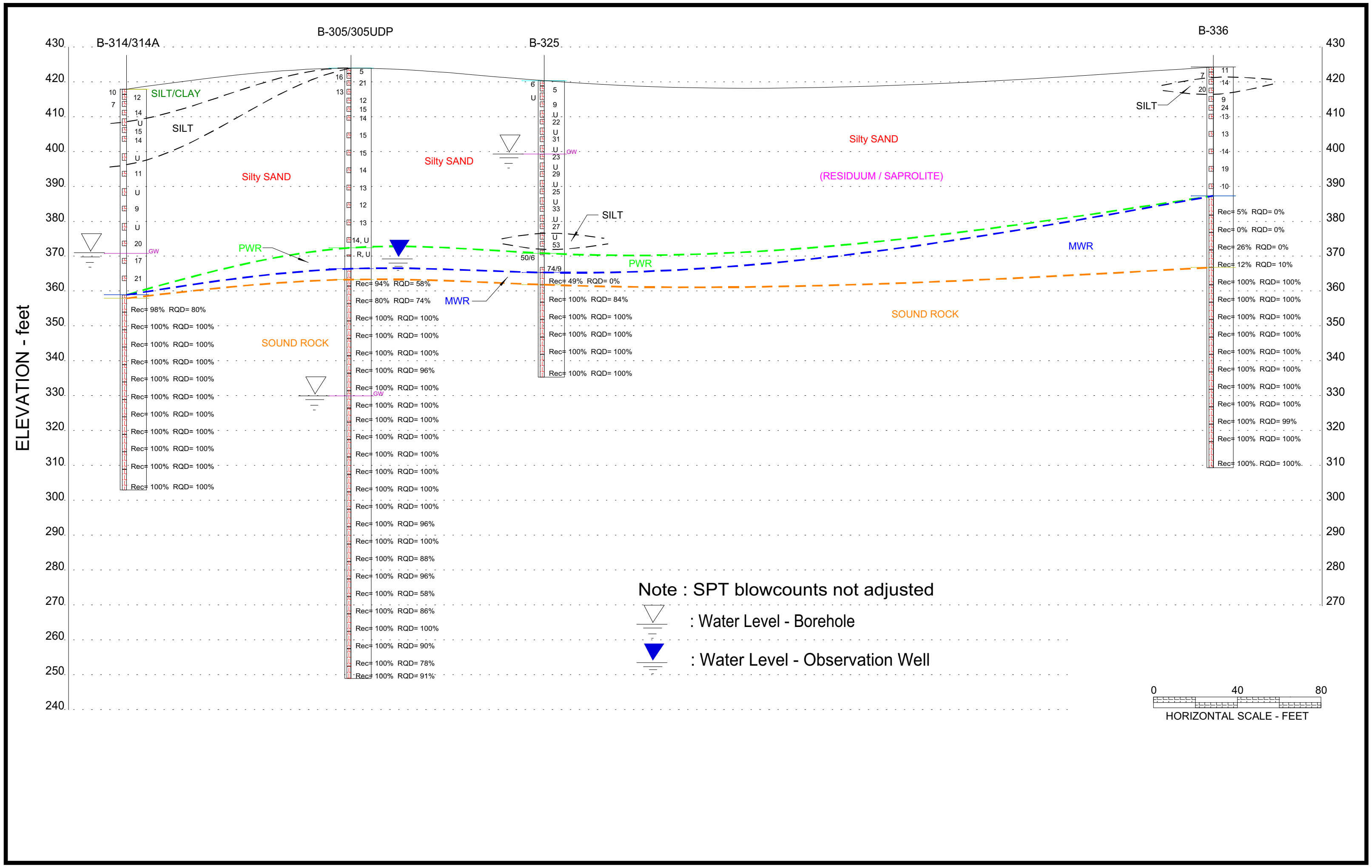


Figure 2.5.4-207 Inferred Subsurface Profiles Unit 3 North-South: I-I (Sheet 1 of 3)

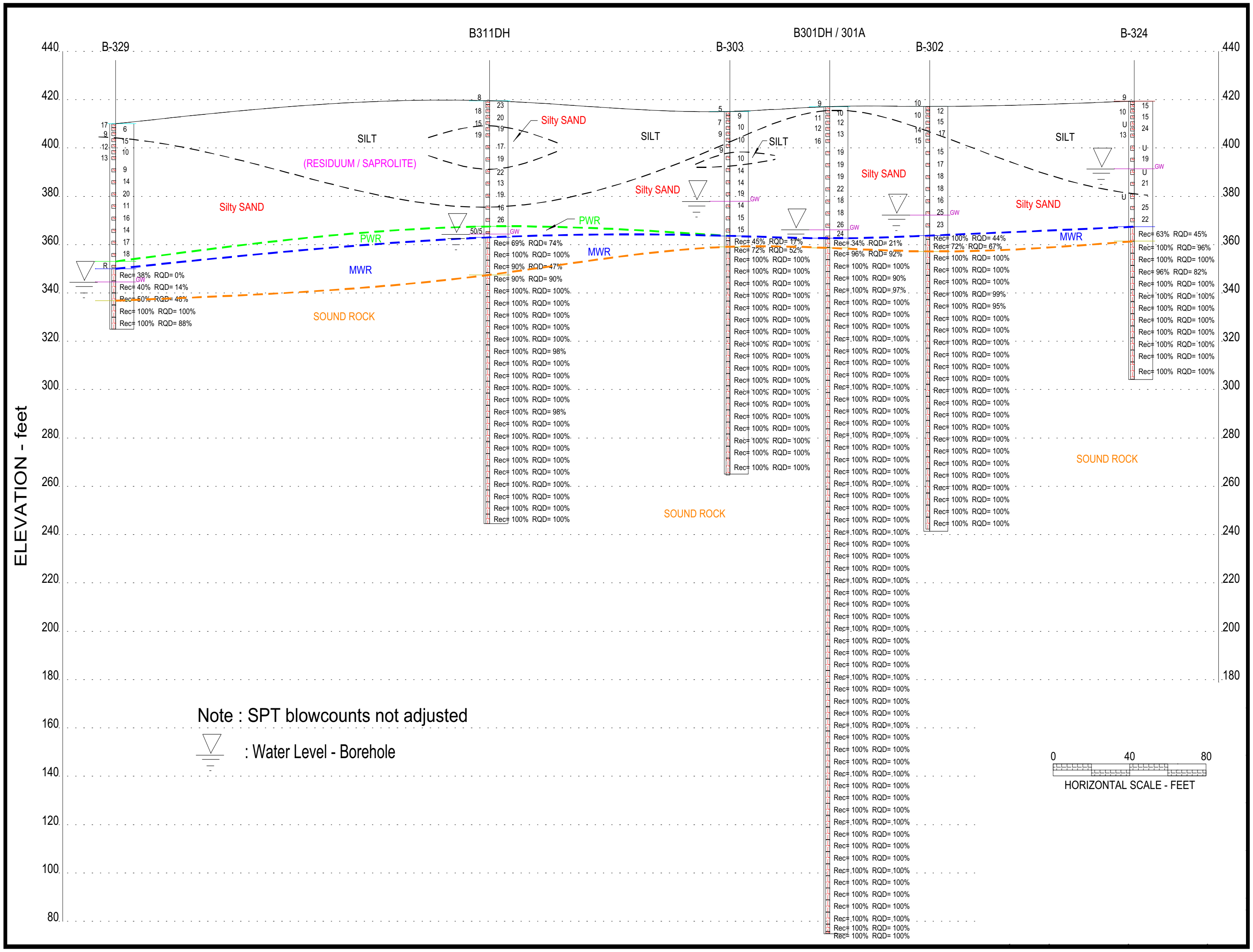


Figure 2.5.4-207 Inferred Subsurface Profiles Unit 3 North-South: J-J (Sheet 2 of 3)

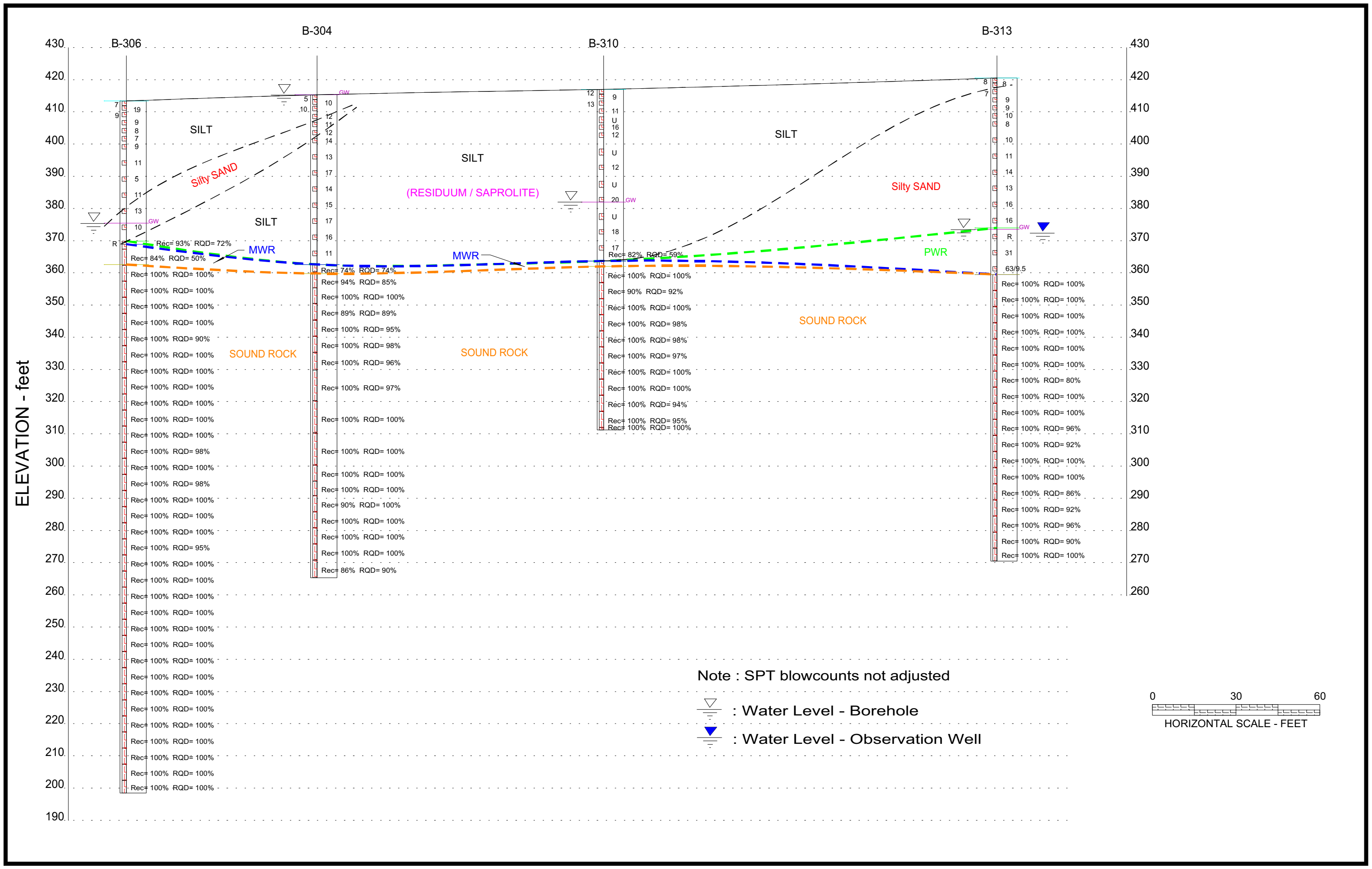
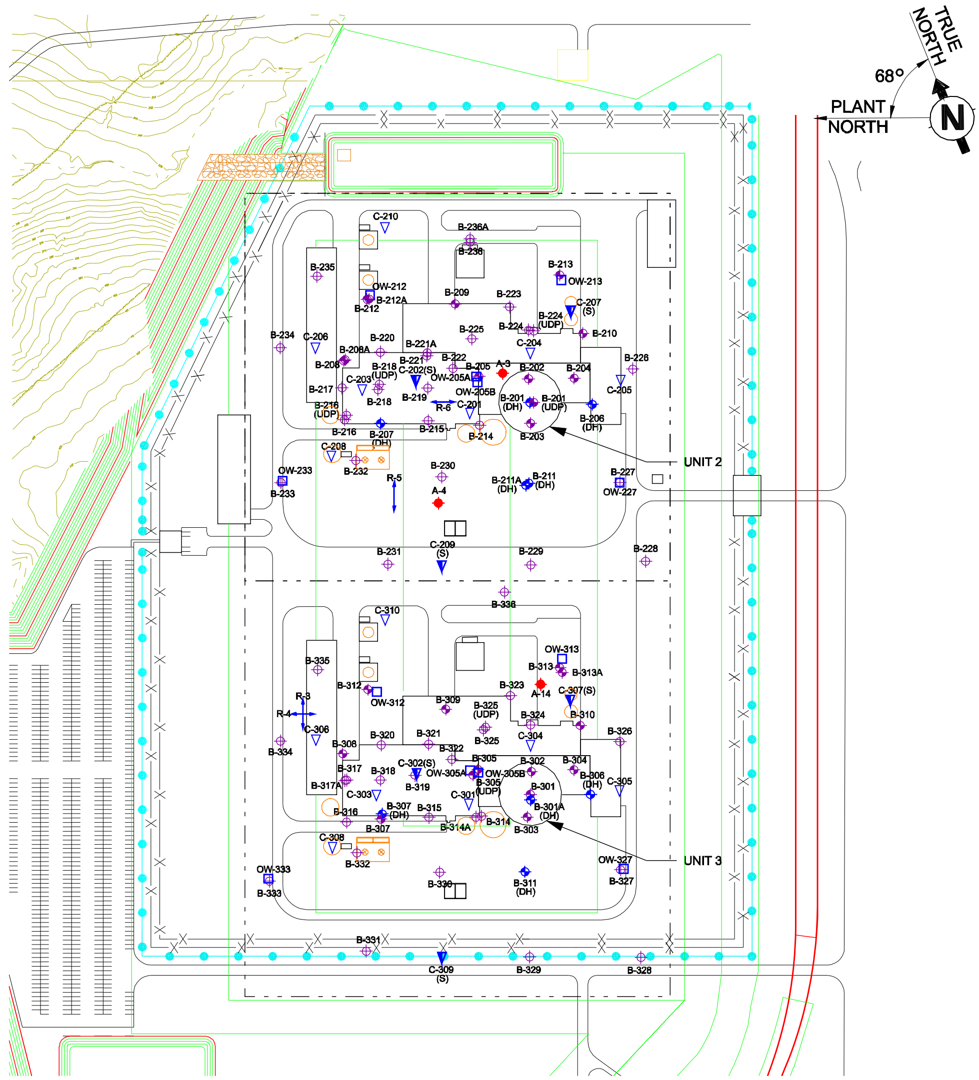


Figure 2.5.4-207 Inferred Subsurface Profiles Unit 3 North-South: K-K (Sheet 3 of 3)



LEGEND

◆	A-4	SITE SCOPING BOREHOLE
◆	B-202	BOREHOLE (HQ CORE SIZE)
◆	B-201(DH)	BOREHOLE (HQ CORE SIZE) WITH DOWN-HOLE GEOPHYSICAL TESTING
⊕	B-236	BOREHOLE (NQ CORE SIZE)
▽	C-210	CONE PENETROMETER TEST
▼	C-202(S)	CONE PENETROMETER TEST WITH SEISMIC TESTING
□	OW-212	OBSERVATION WELL
→	R-1	ELECTRICAL RESISTIVITY TEST

0 300 600 900FT

Figure 2.5.4-208 Boring Location Plan (Power Block)

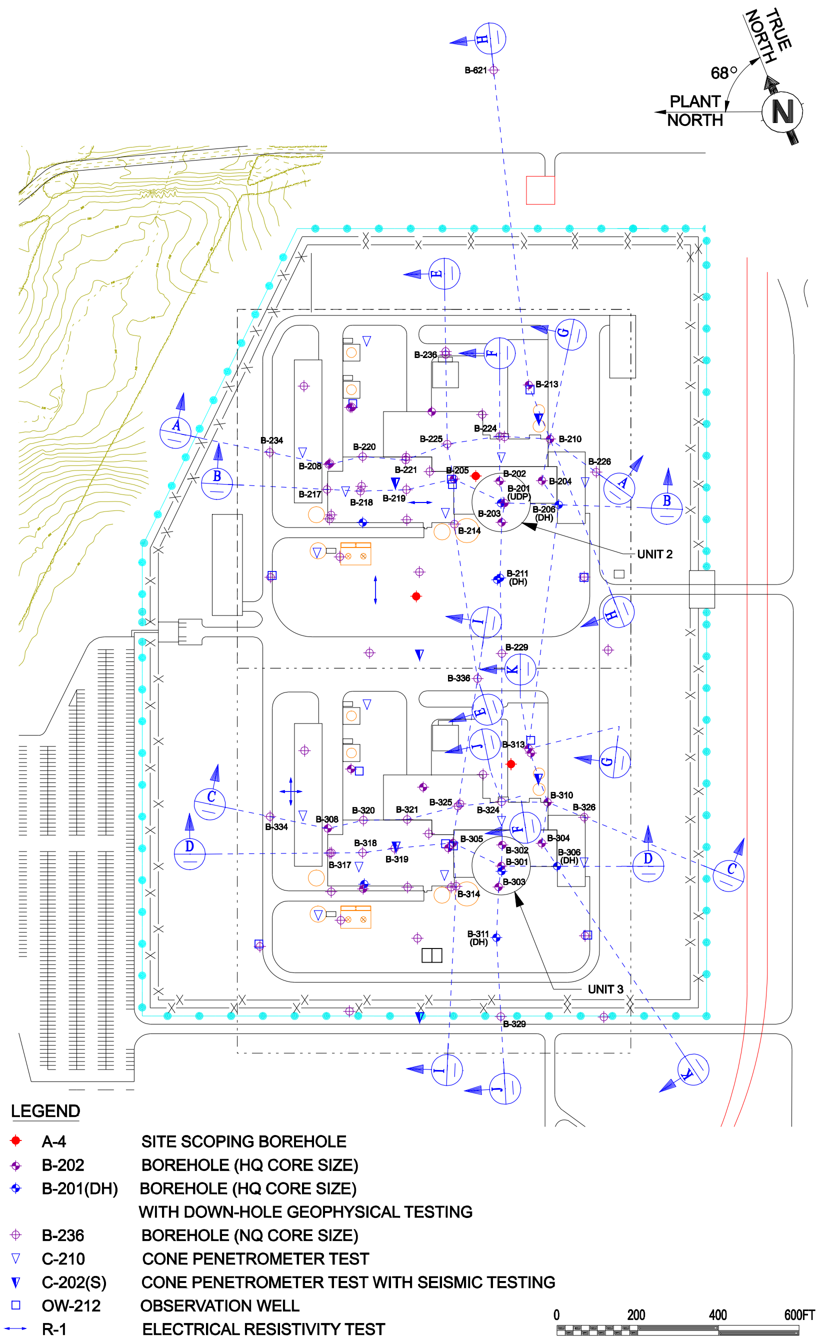


Figure 2.5.4-209 Boring Location Plan with Subsurface Profiles (Power Block)

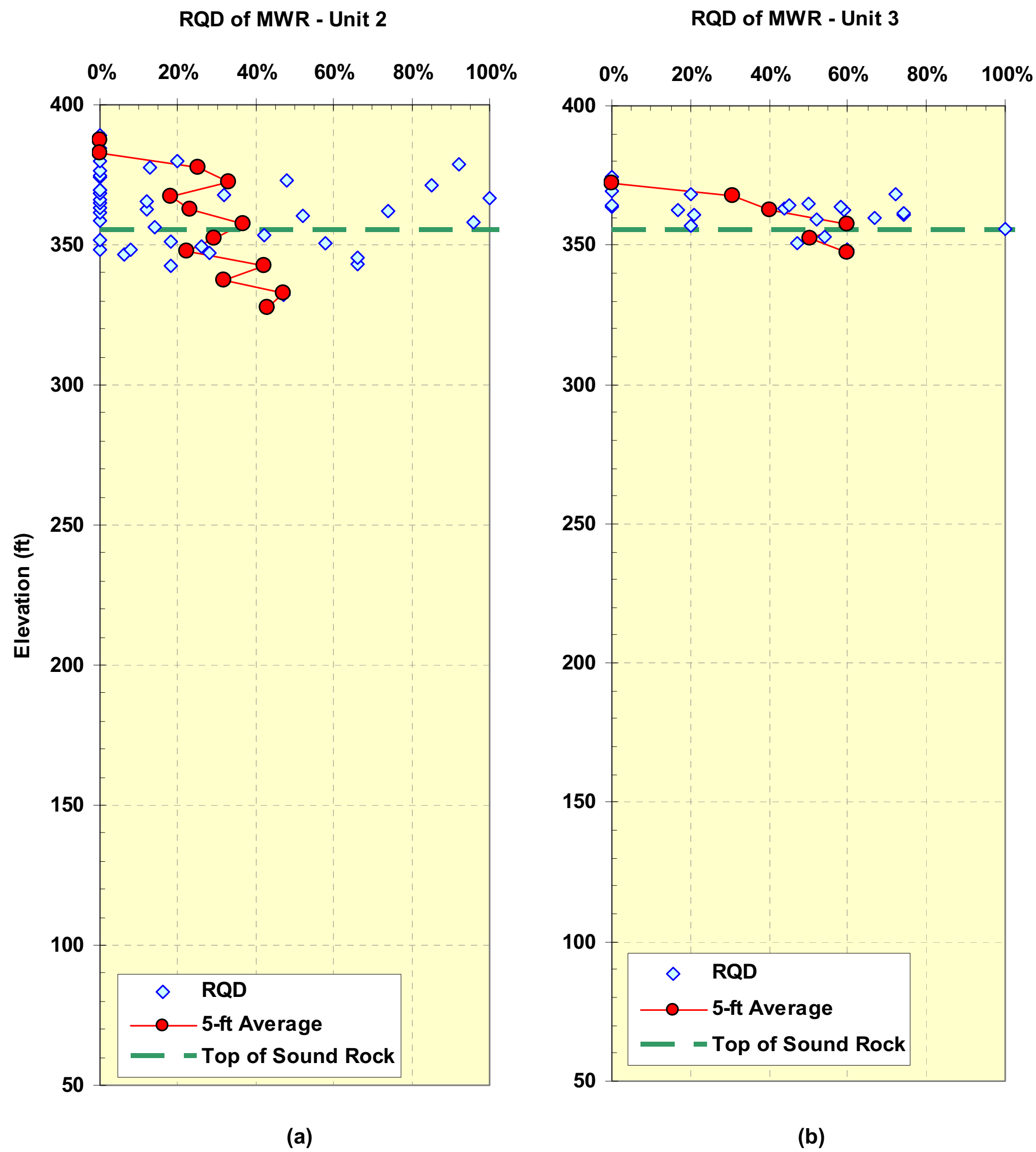


Figure 2.5.4-210 RQD of Layer IV (MWR)

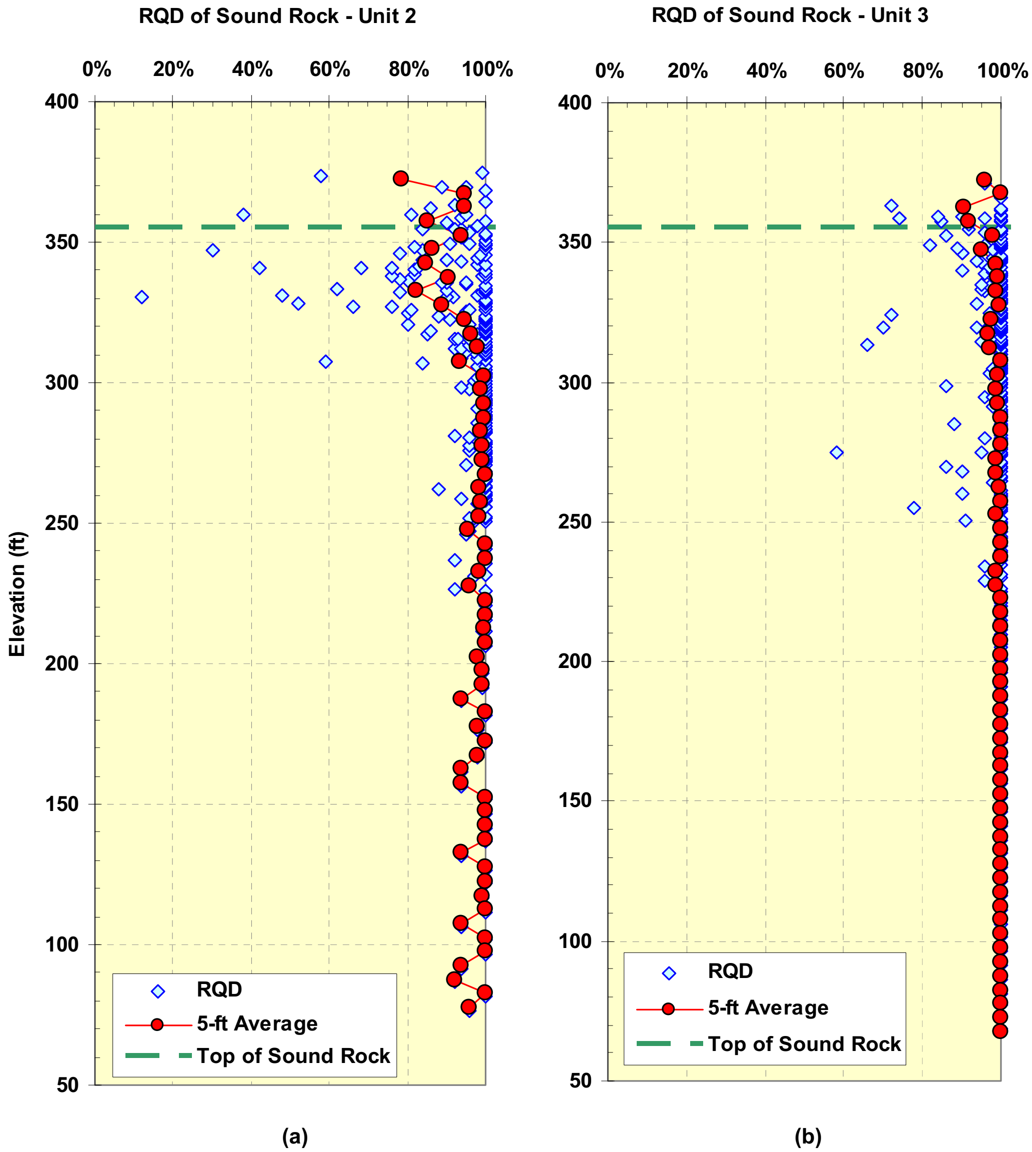


Figure 2.5.4-211 RQD of Layer V (Sound Rock)

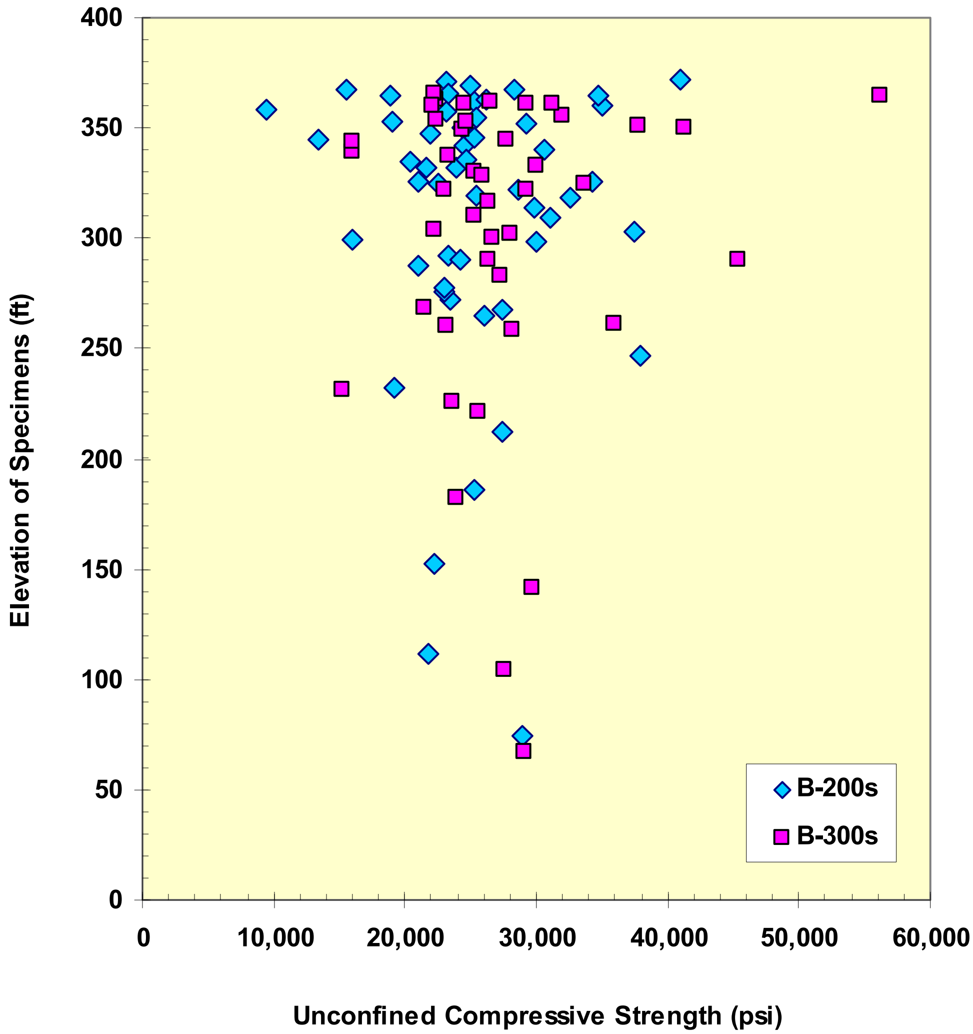


Figure 2.5.4-212 Unconfined Compressive Strength of Rock Specimens

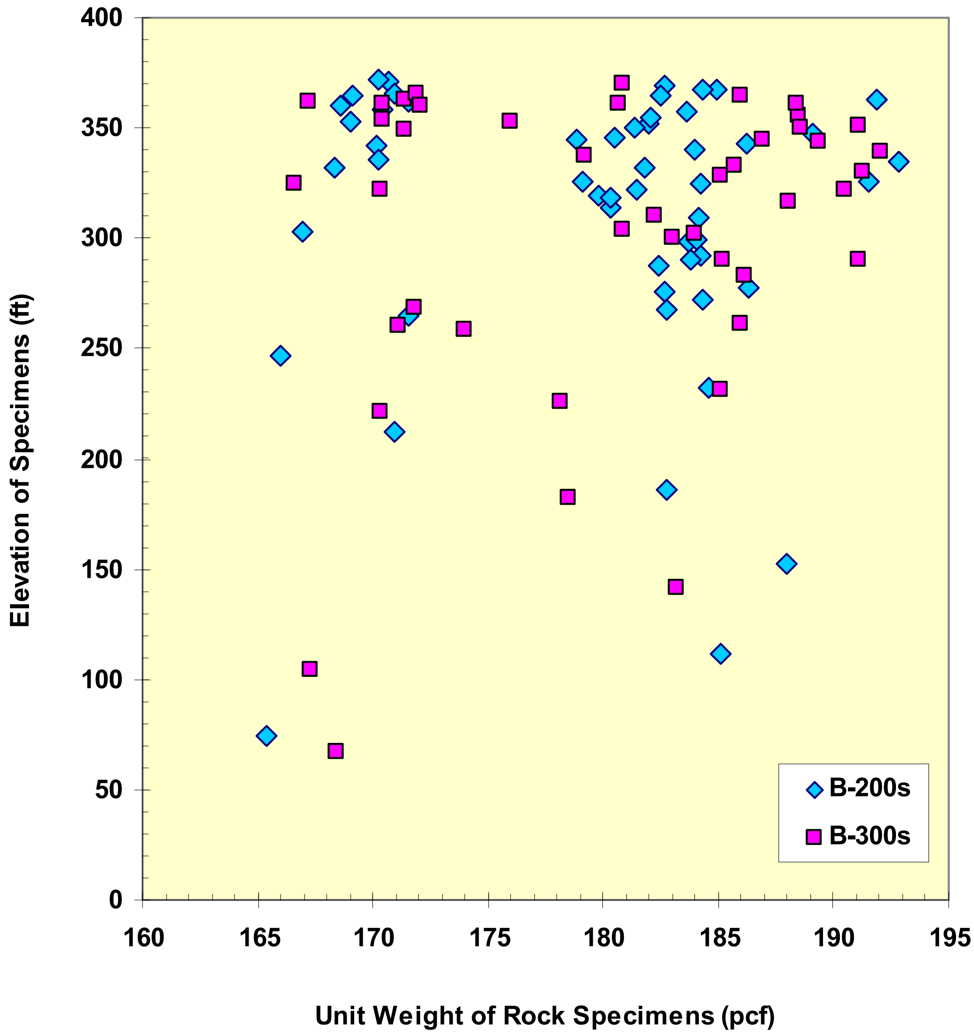


Figure 2.5.4-213 Unit Weight of Rock Specimens

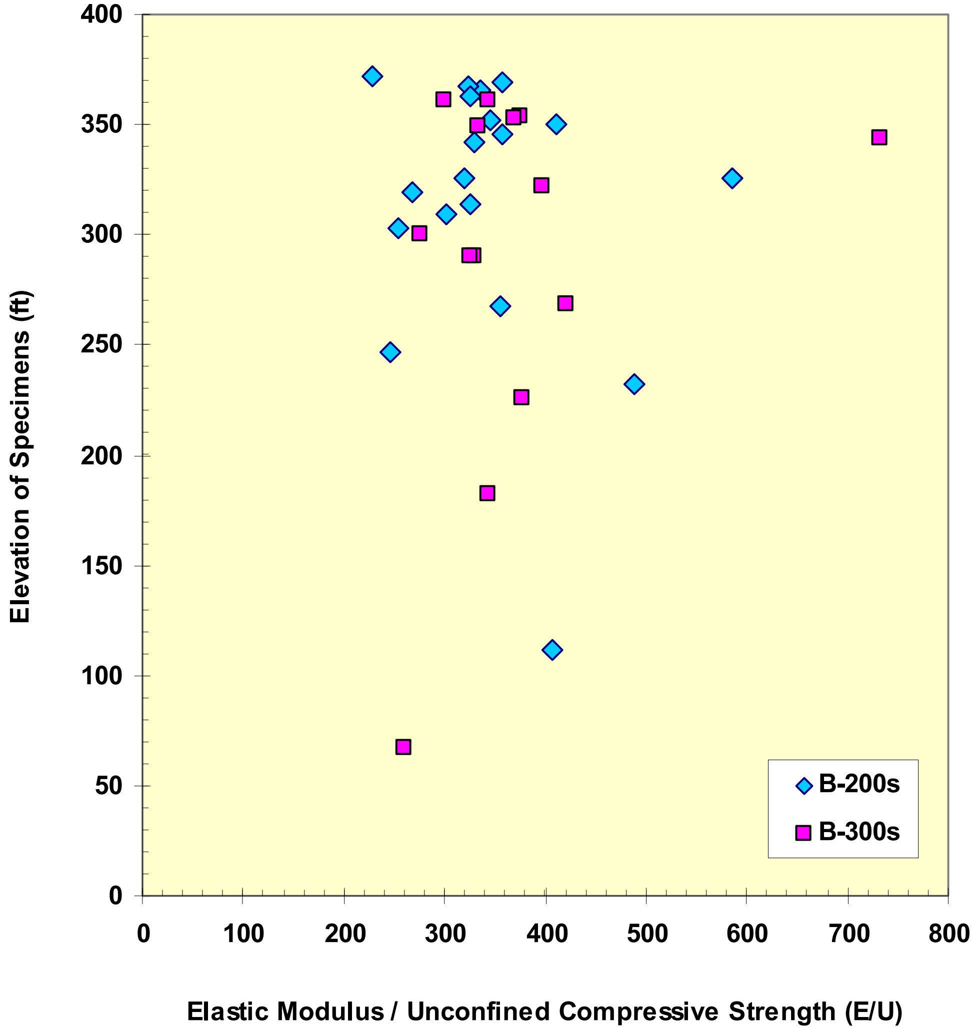


Figure 2.5.4-214 Ratio of Elastic Modulus to Compressive Strength of Rock Specimens

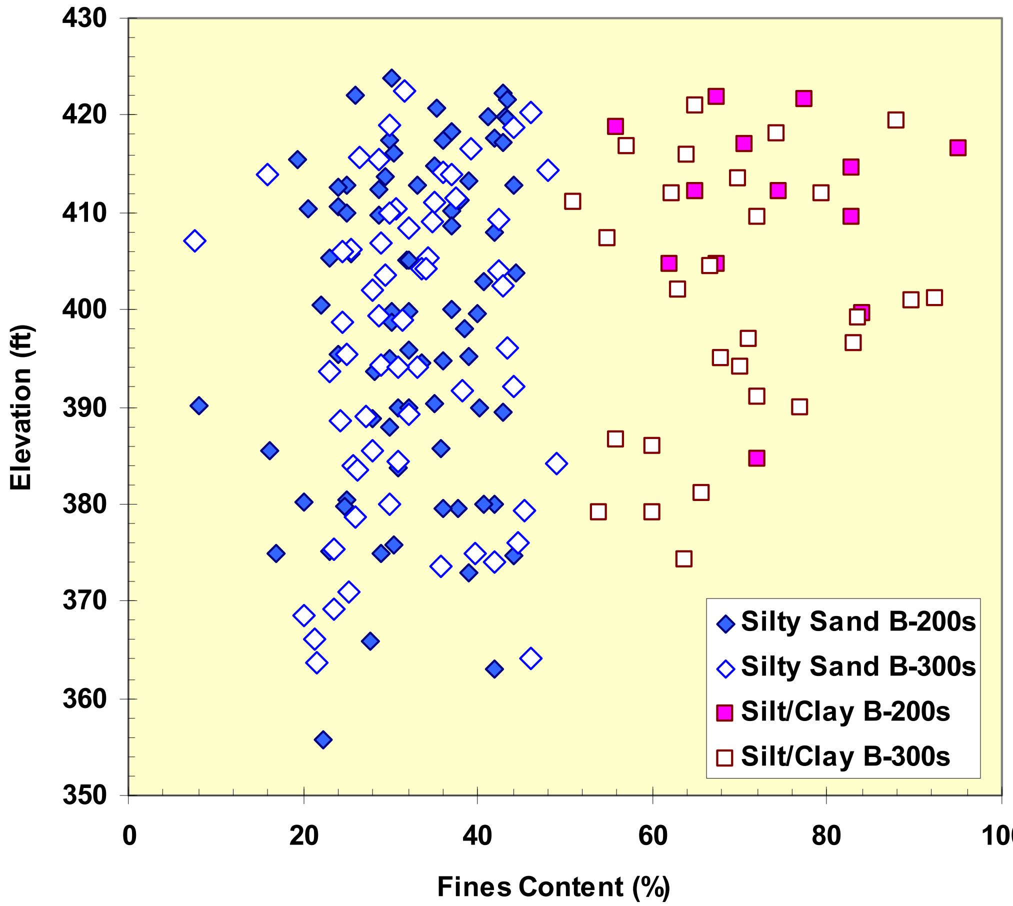


Figure 2.5.4-215 Fines Content

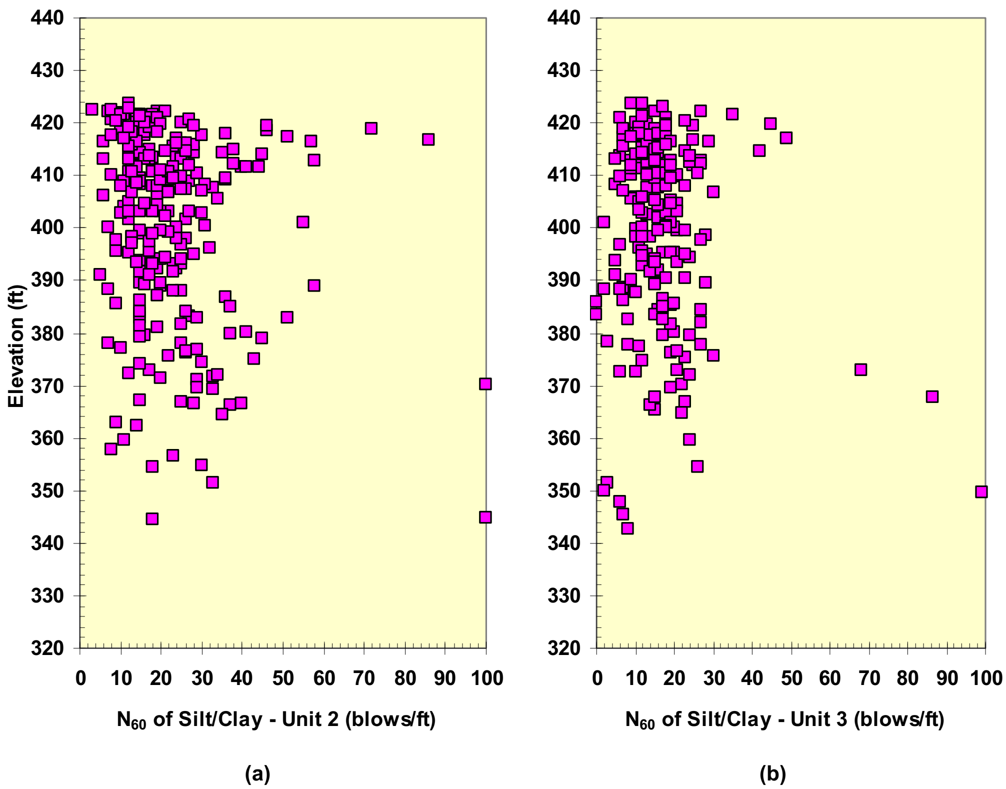


Figure 2.5.4-216 Adjusted SPT N-Values (N_{60}) — Silt/Clay

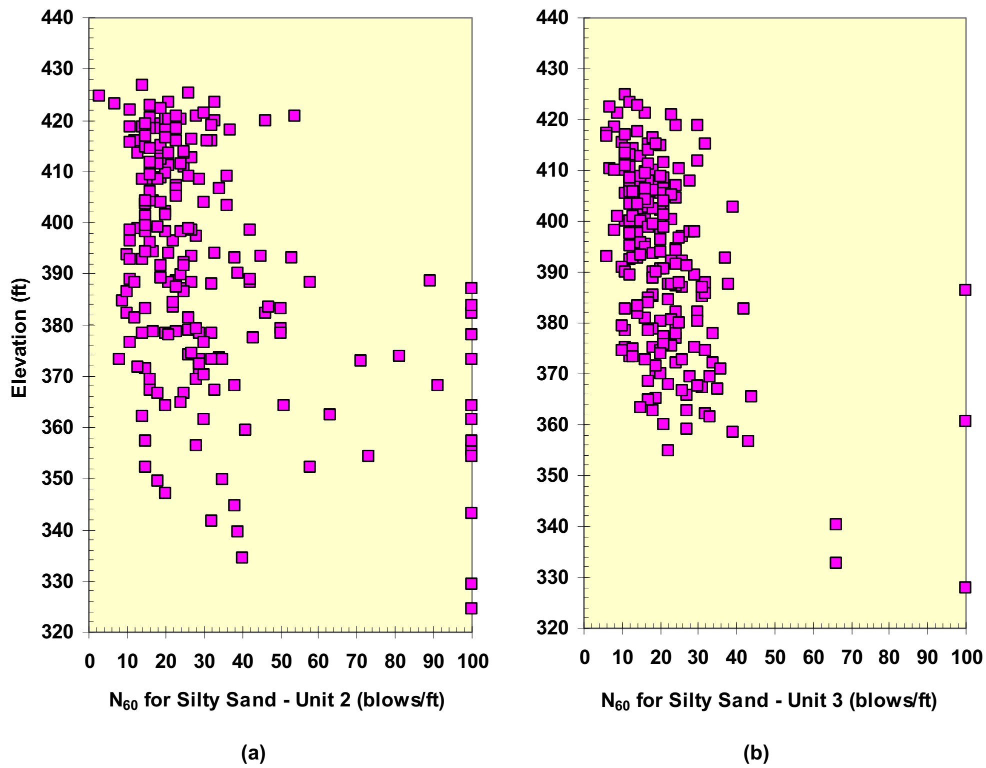
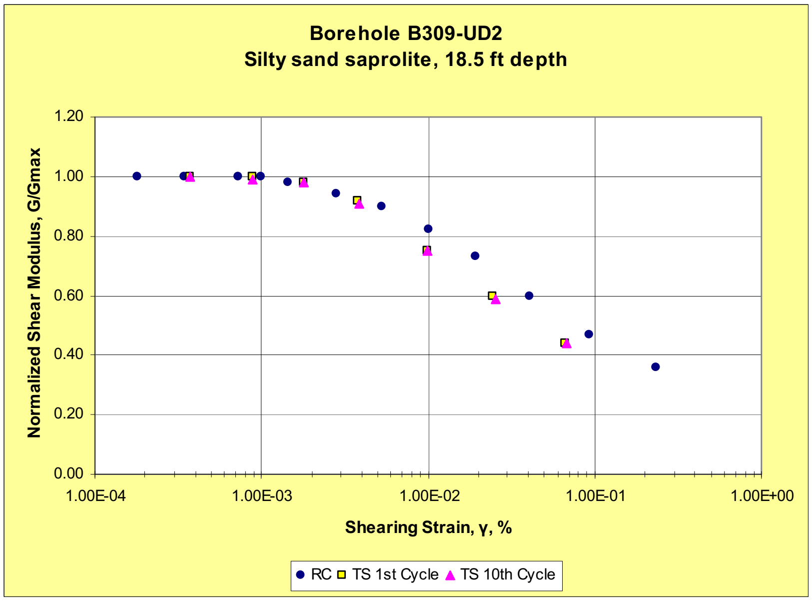


Figure 2.5.4-217 Adjusted SPT N-Values (N_{60}) – Silty Sand

a. Saprolite Sample B-309 UD-2
9.8 psi Confining Pressure



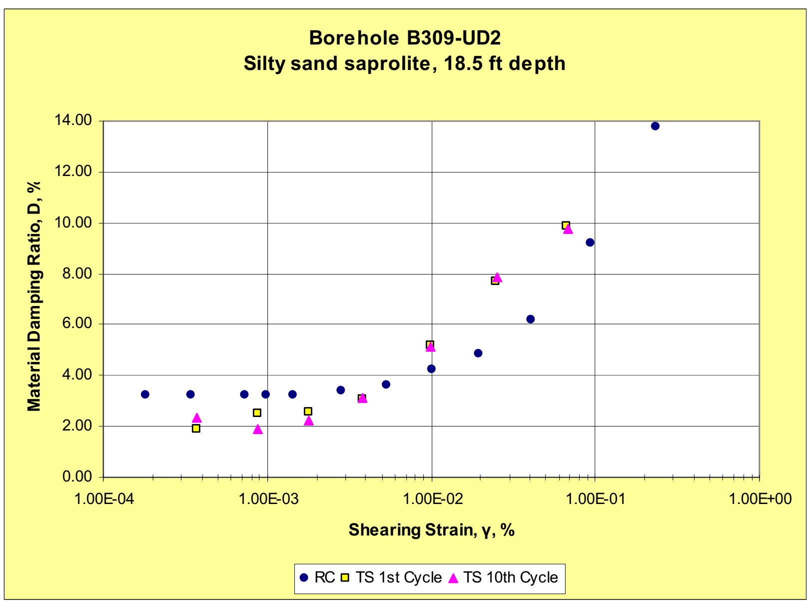
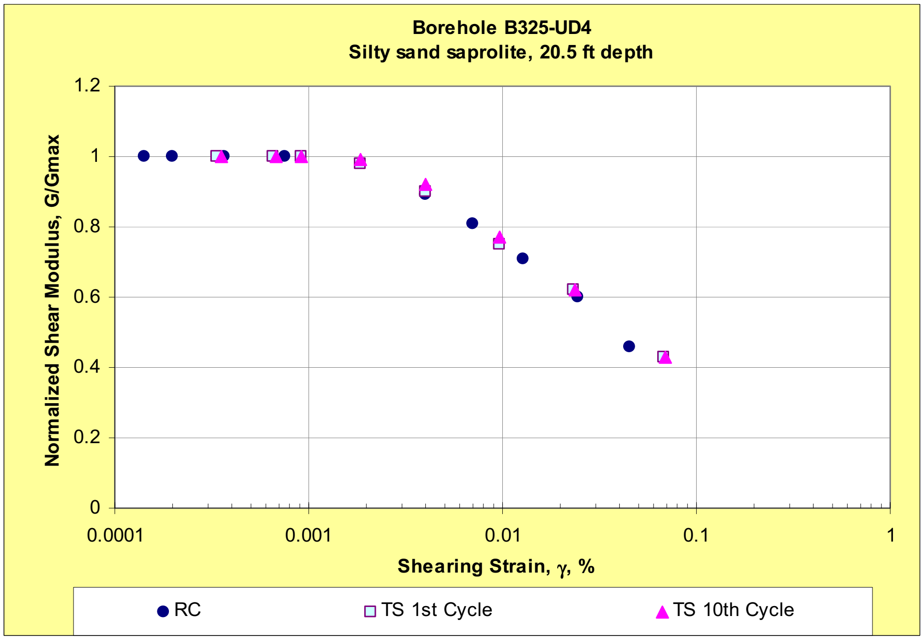


Figure 2.5.4-218 RCTS Results G/G_{MAX} and D versus Shear Strain (Sheet 1 of 5)

b. Saprolite Sample B-325 UD4
9.8 psi Confining Pressure



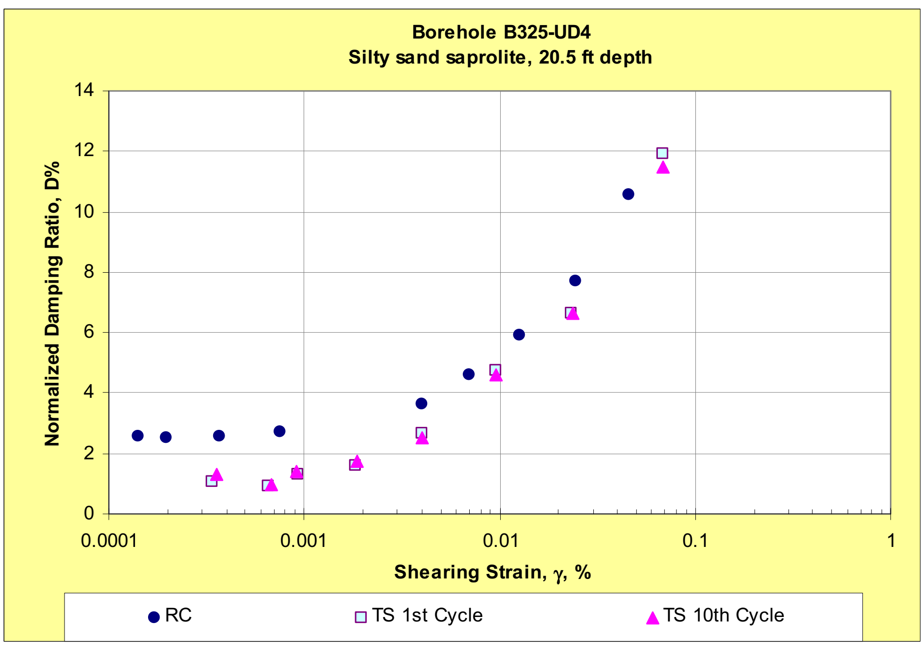
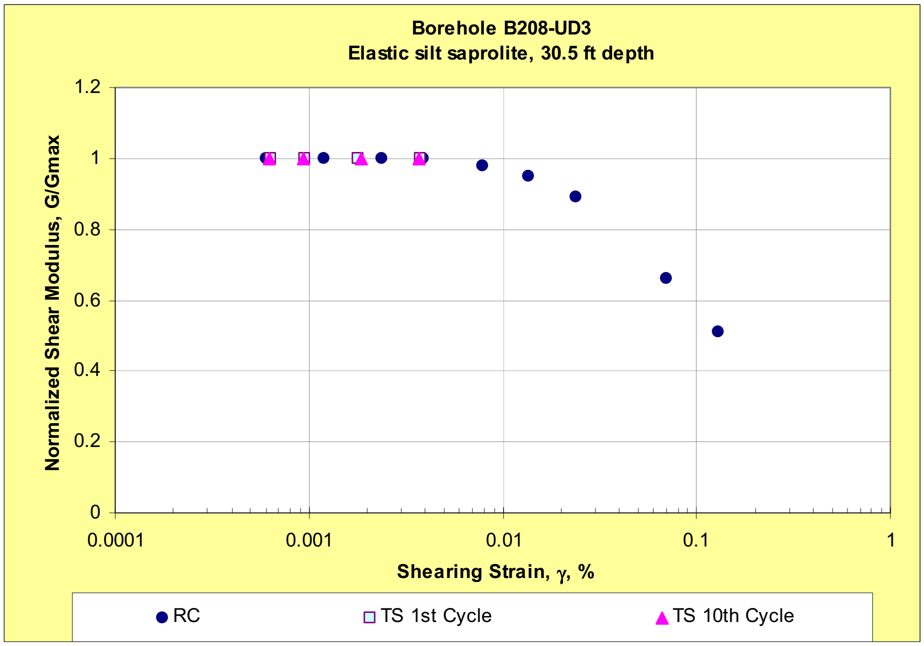


Figure 2.5.4-218 RCTS Results G/G_{MAX} and D versus Shear Strain (Sheet 2 of 5)

c. Saprolite Sample B-208 UD3
15.2 psi Confining Pressure



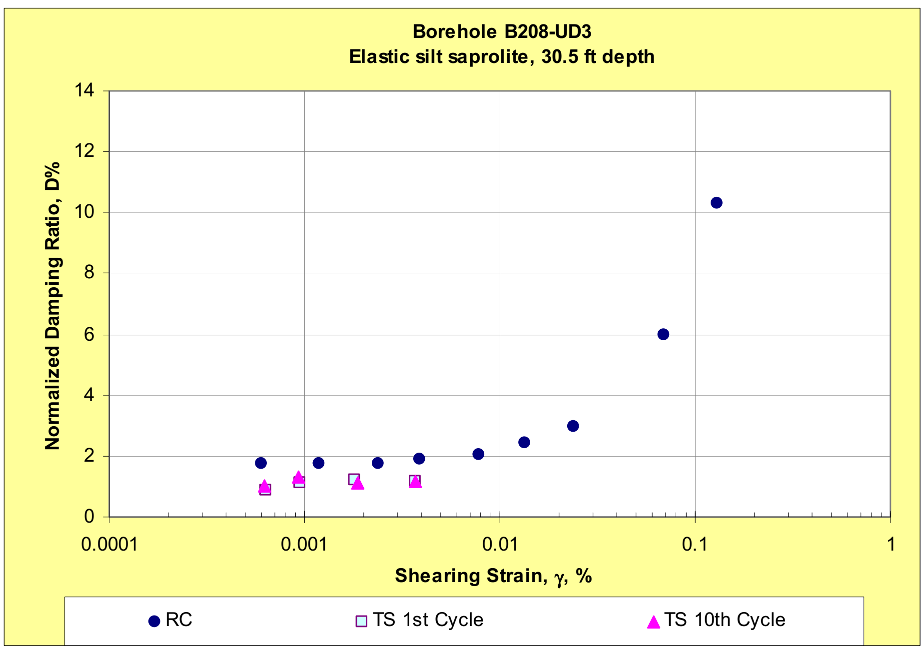
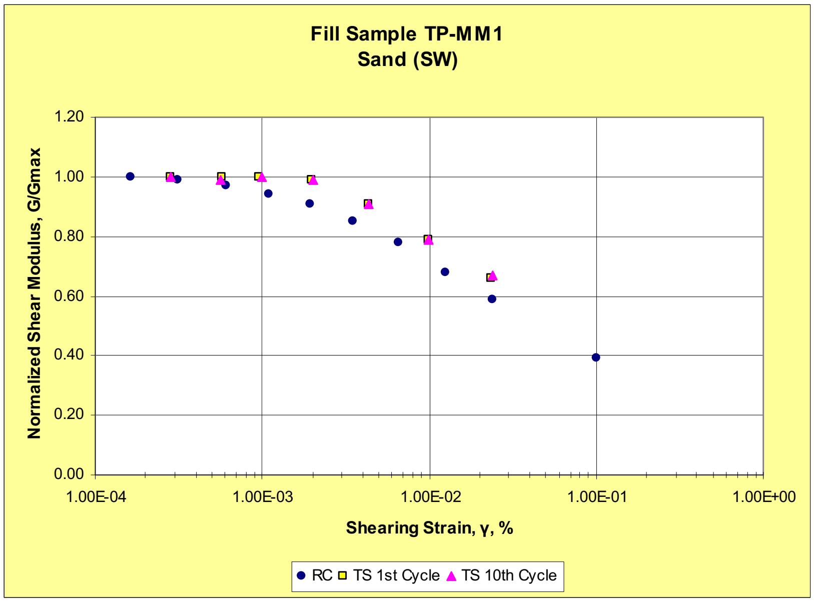


Figure 2.5.4-218 RCTS Results G/G_{MAX} and D versus Shear Strain (Sheet 3 of 5)

d. Fill Sample MM-1
12.1 psi Confining Pressure



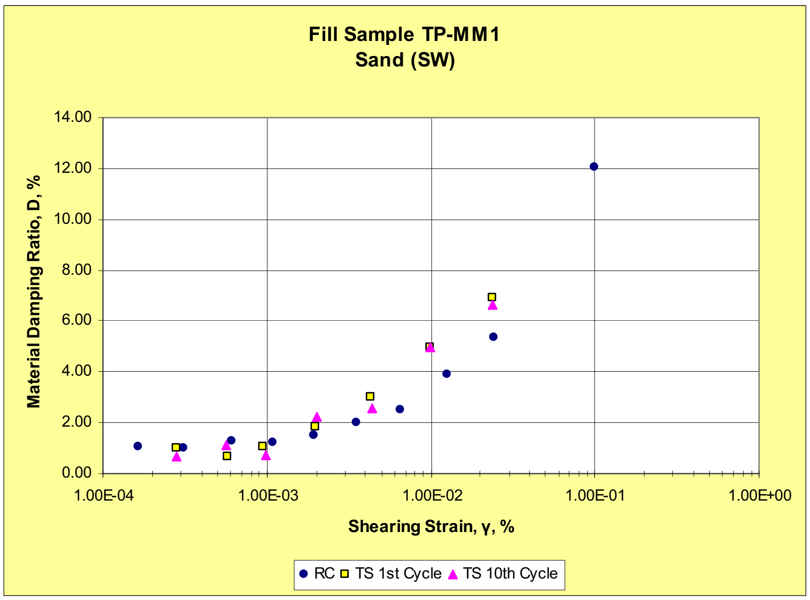
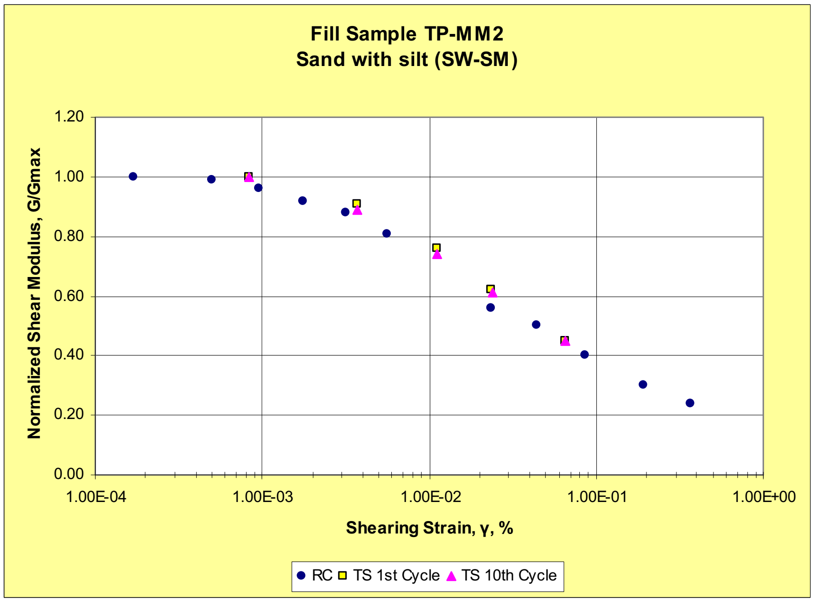


Figure 2.5.4-218 RCTS Results G/G_{MAX} and D versus Shear Strain (Sheet 4 of 5)

e. Fill Sample MM-2
12.1 psi Confining Pressure



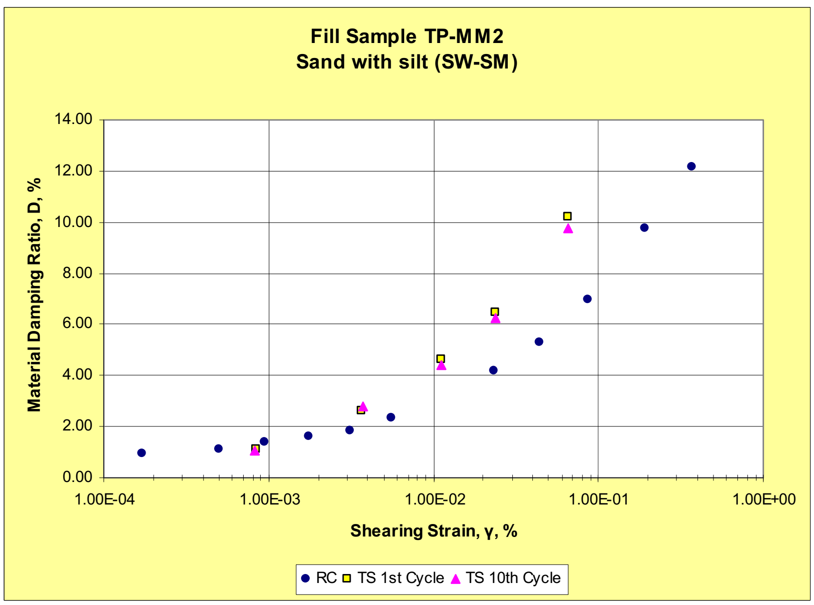


Figure 2.5.4-218 RCTS Results G/G_{MAX} and D versus Shear Strain (Sheet 5 of 5)

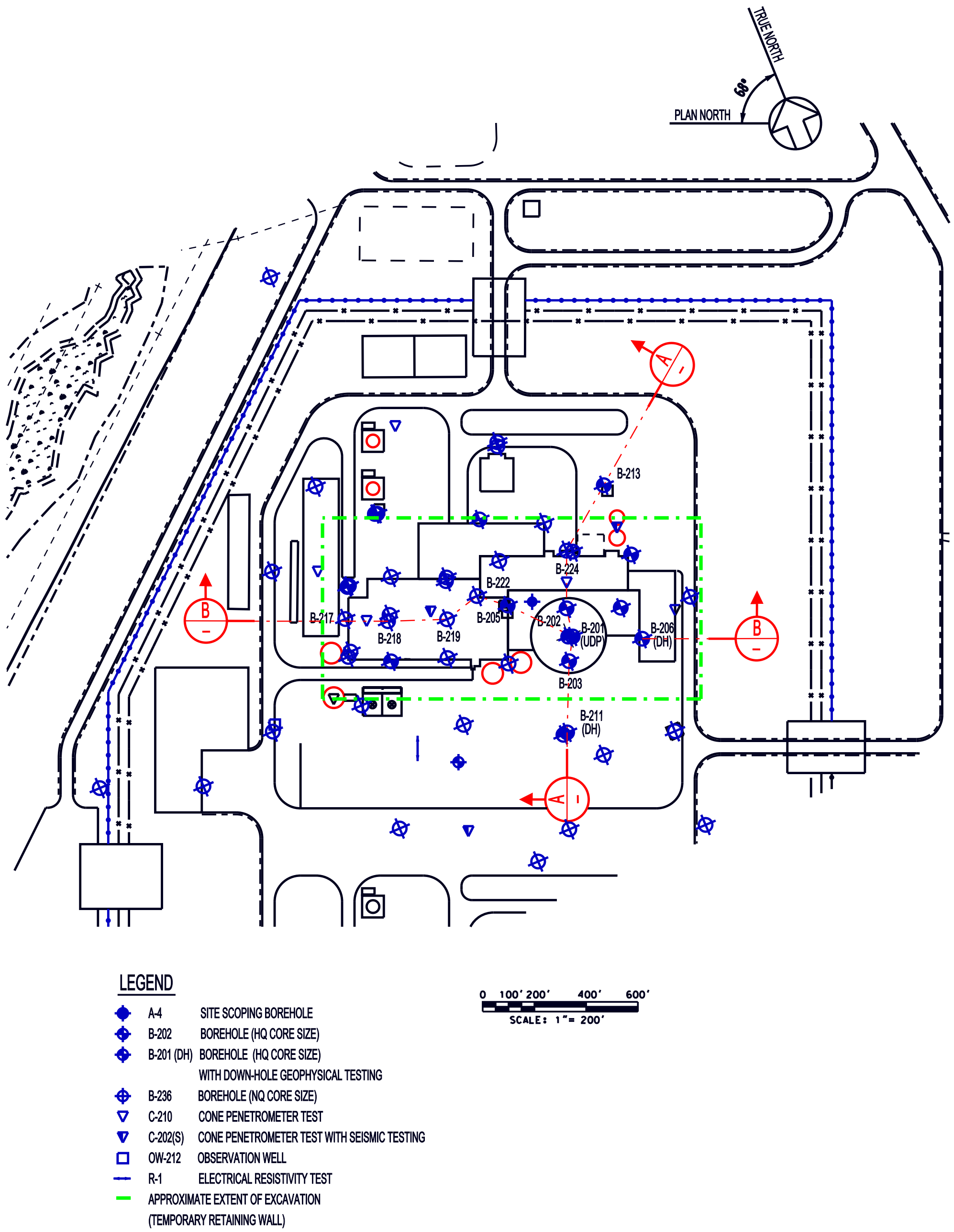


Figure 2.5.4-219 Profile Location Map Showing Excavation Geometry, Unit 2 (Sheet 1 of 2)

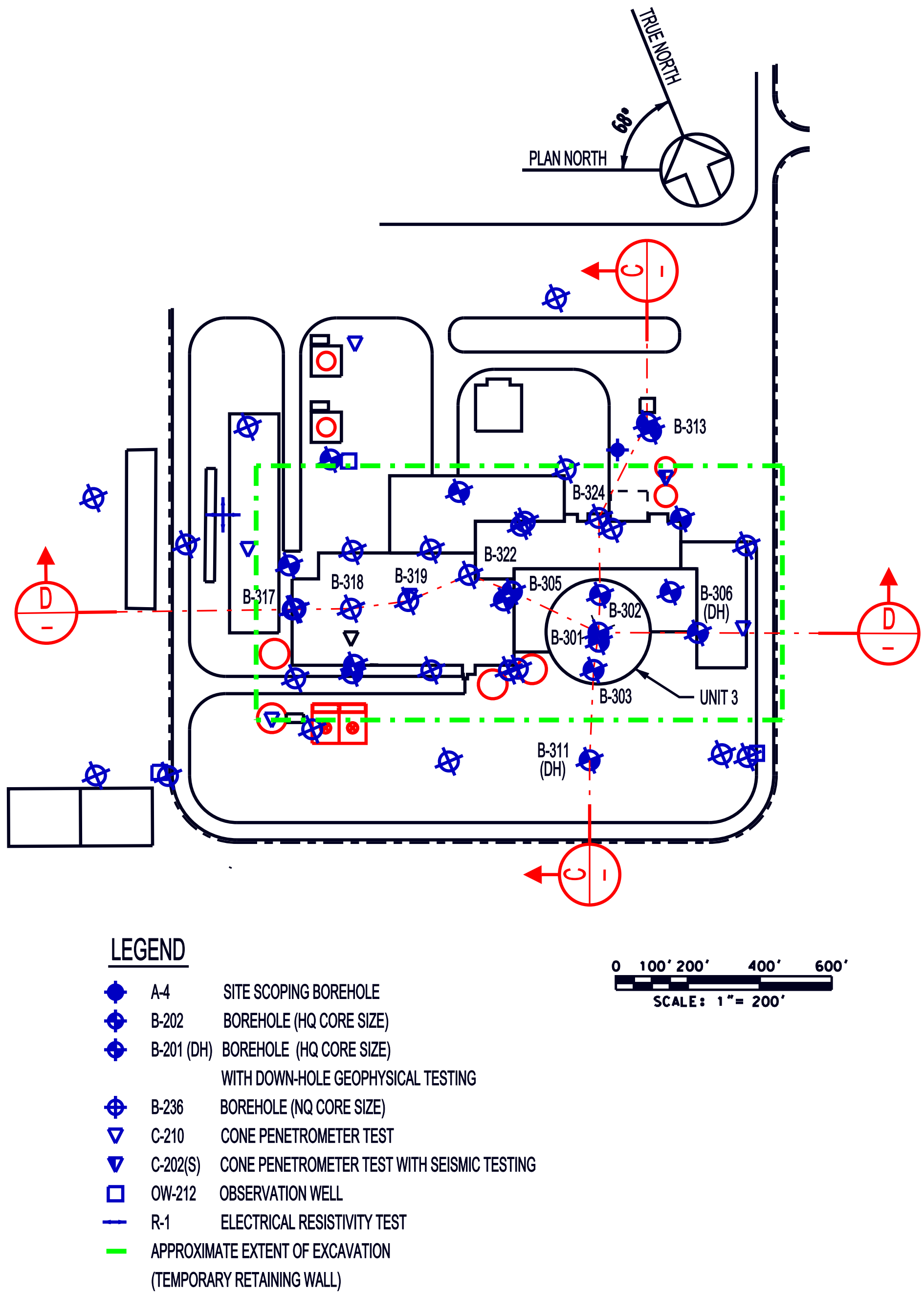


Figure 2.5.4-219 Profile Location Map Showing Excavation Geometry, Unit 3 (Sheet 2 of 2)

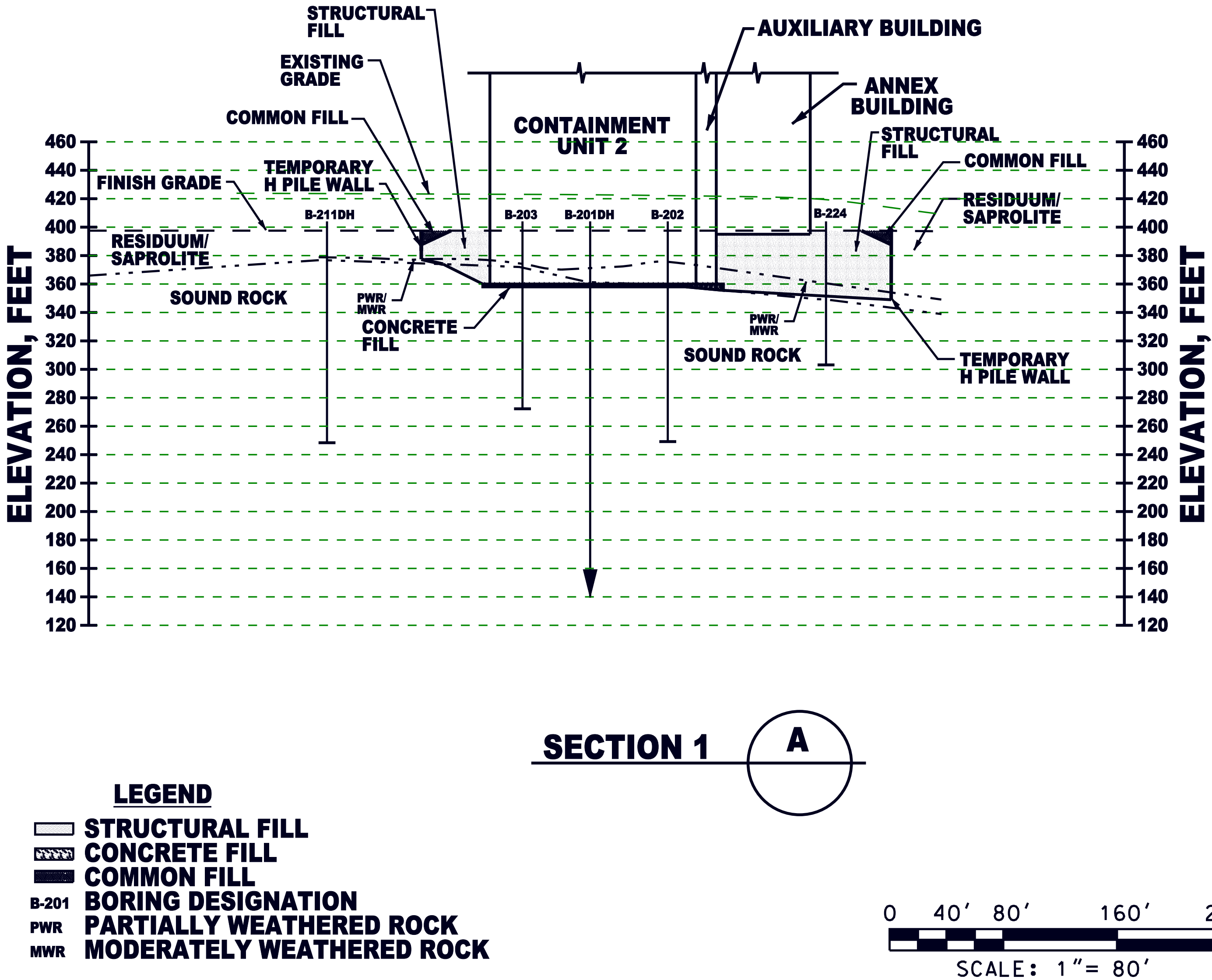


Figure 2.5.4-220 Cross-Section of Structure Foundation A-A

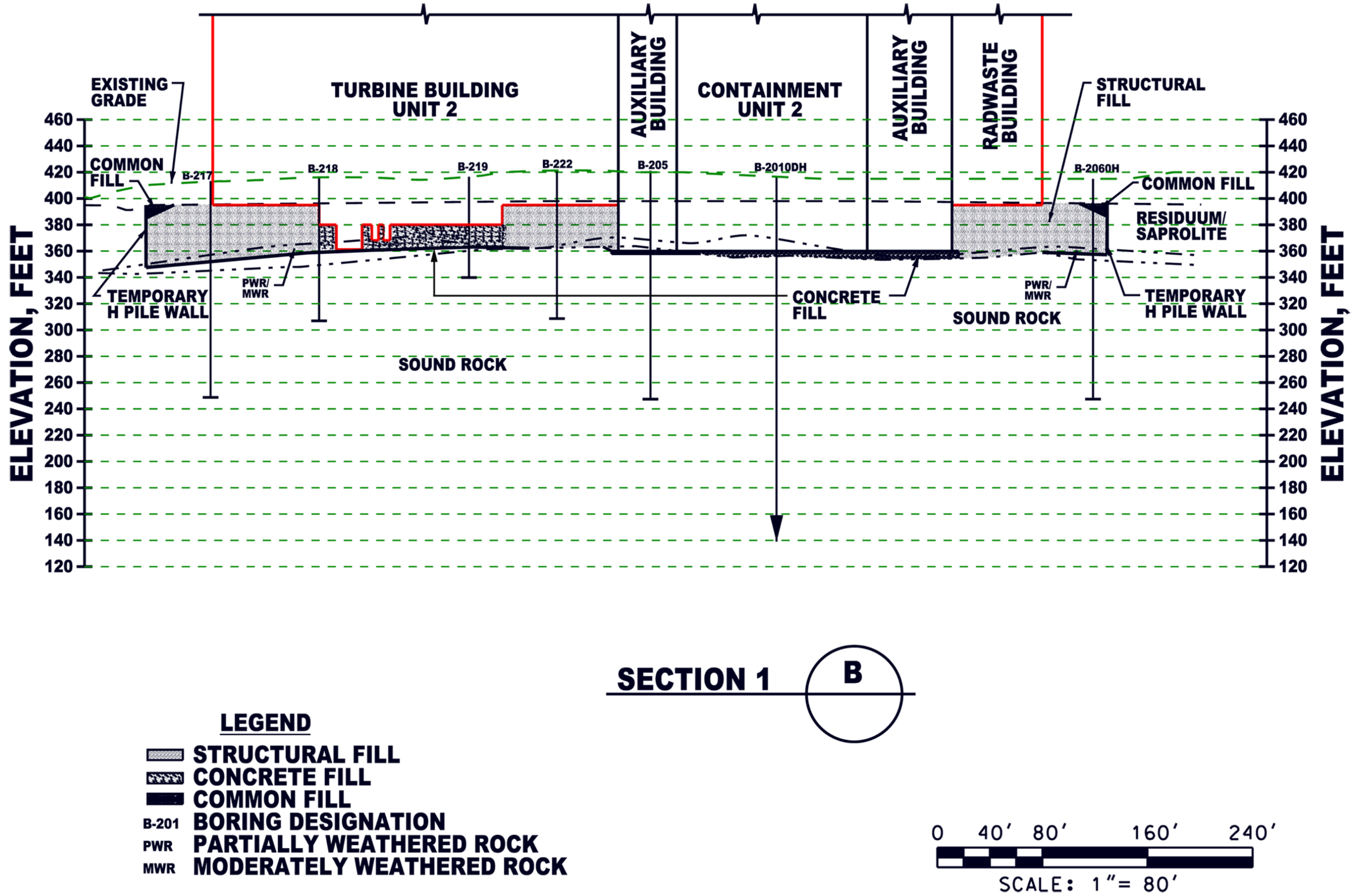


Figure 2.5.4-221 Cross-Section of Structure Foundation B-B RN-12-012

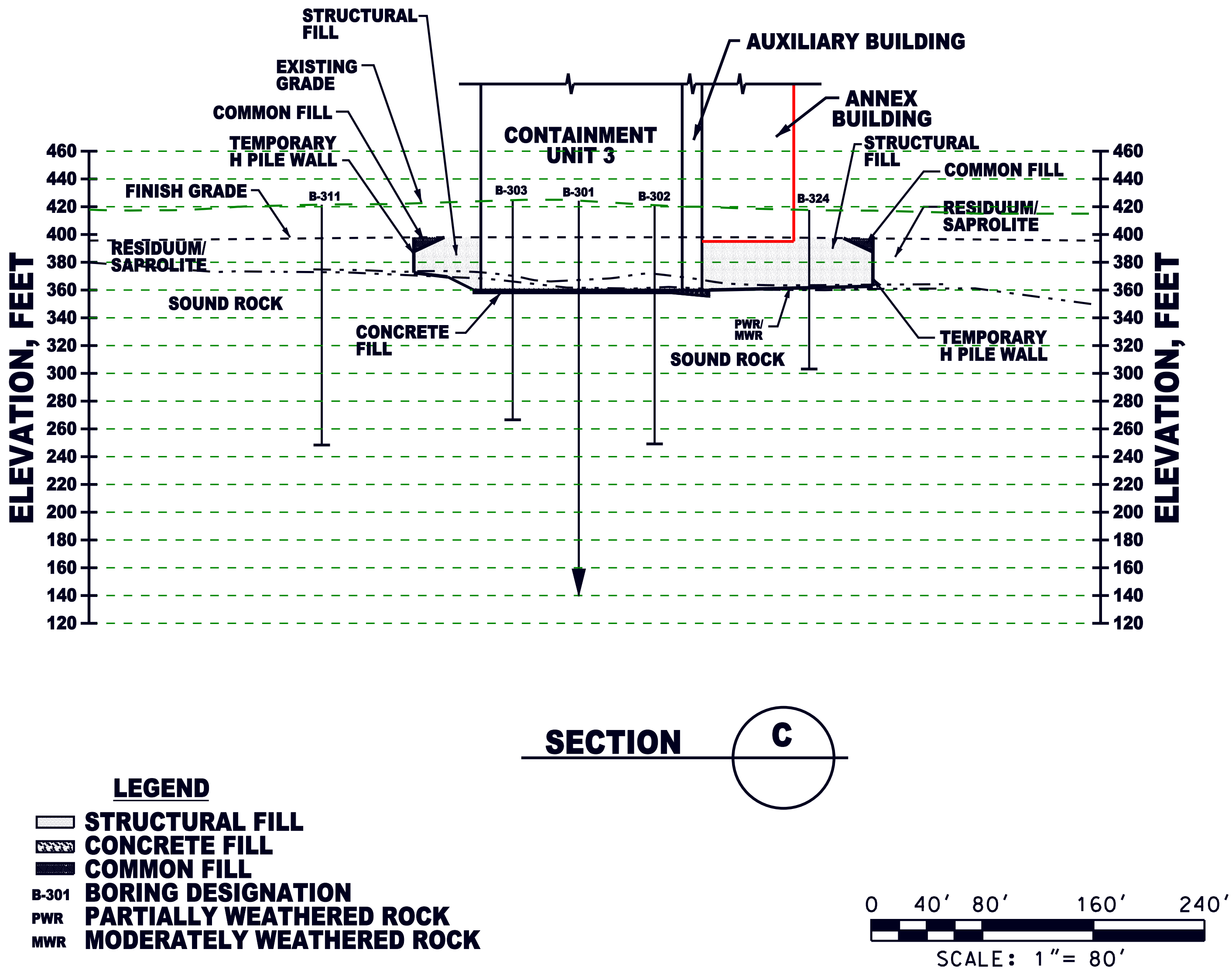


Figure 2.5.4-222 Cross-Section of Structure Foundation C-C

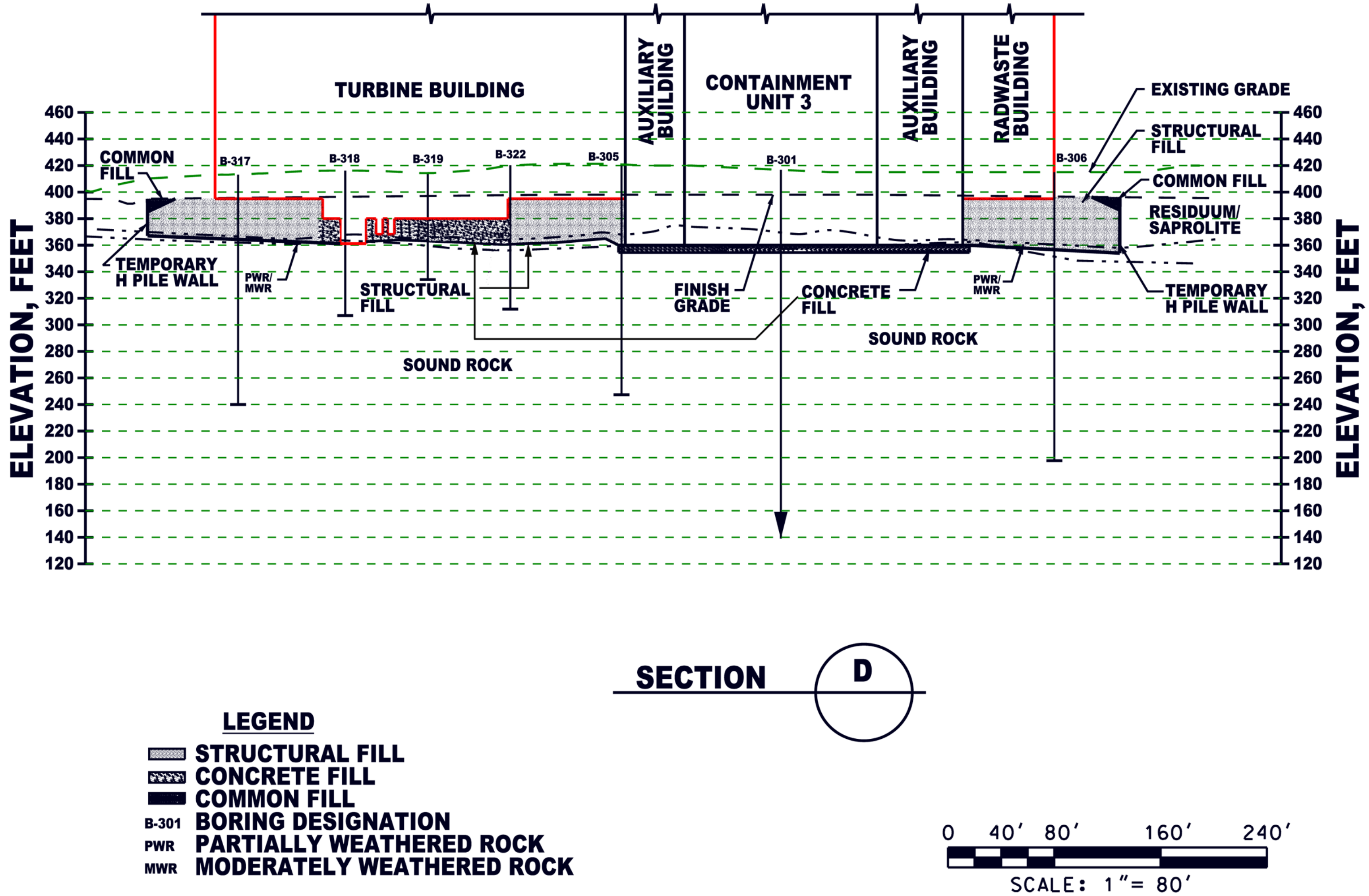


Figure 2.5.4-223 Cross-Section of Structure Foundation D-D

RN-12-012

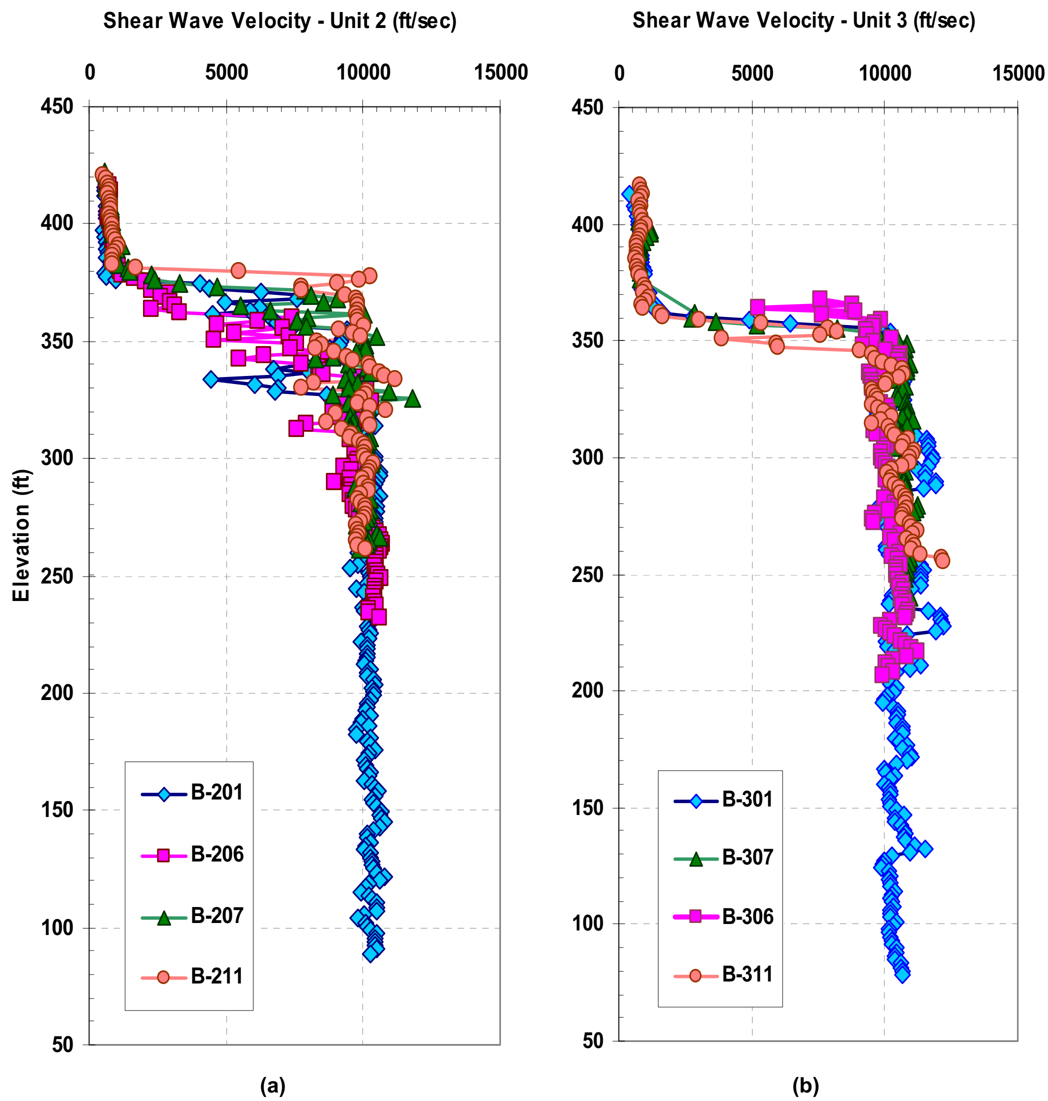


Figure 2.5.4-224 Shear Wave Velocity of Layers I through V by Suspension P-S Logging

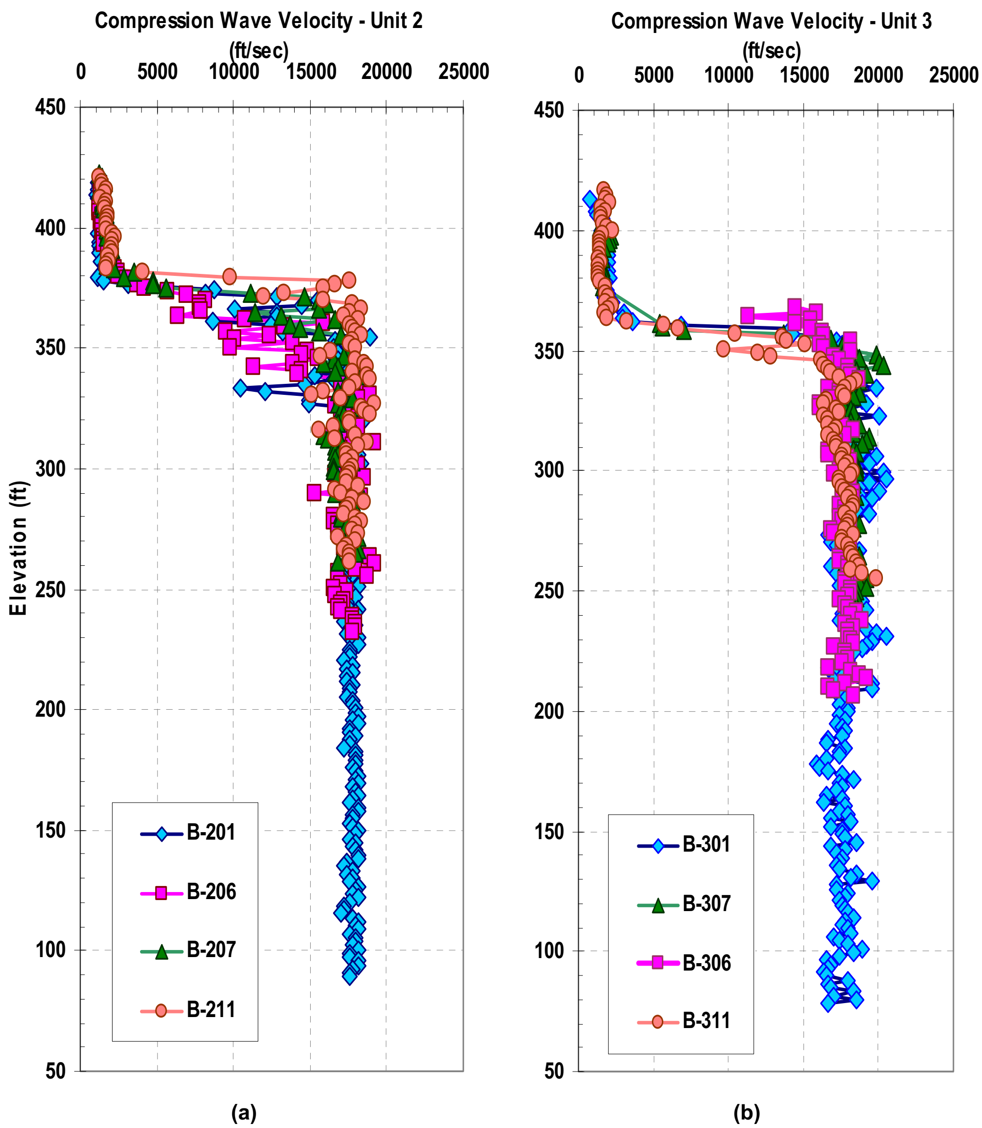


Figure 2.5.4-225 Compression Wave Velocity of Layers I Through V by Suspension P-S Logging

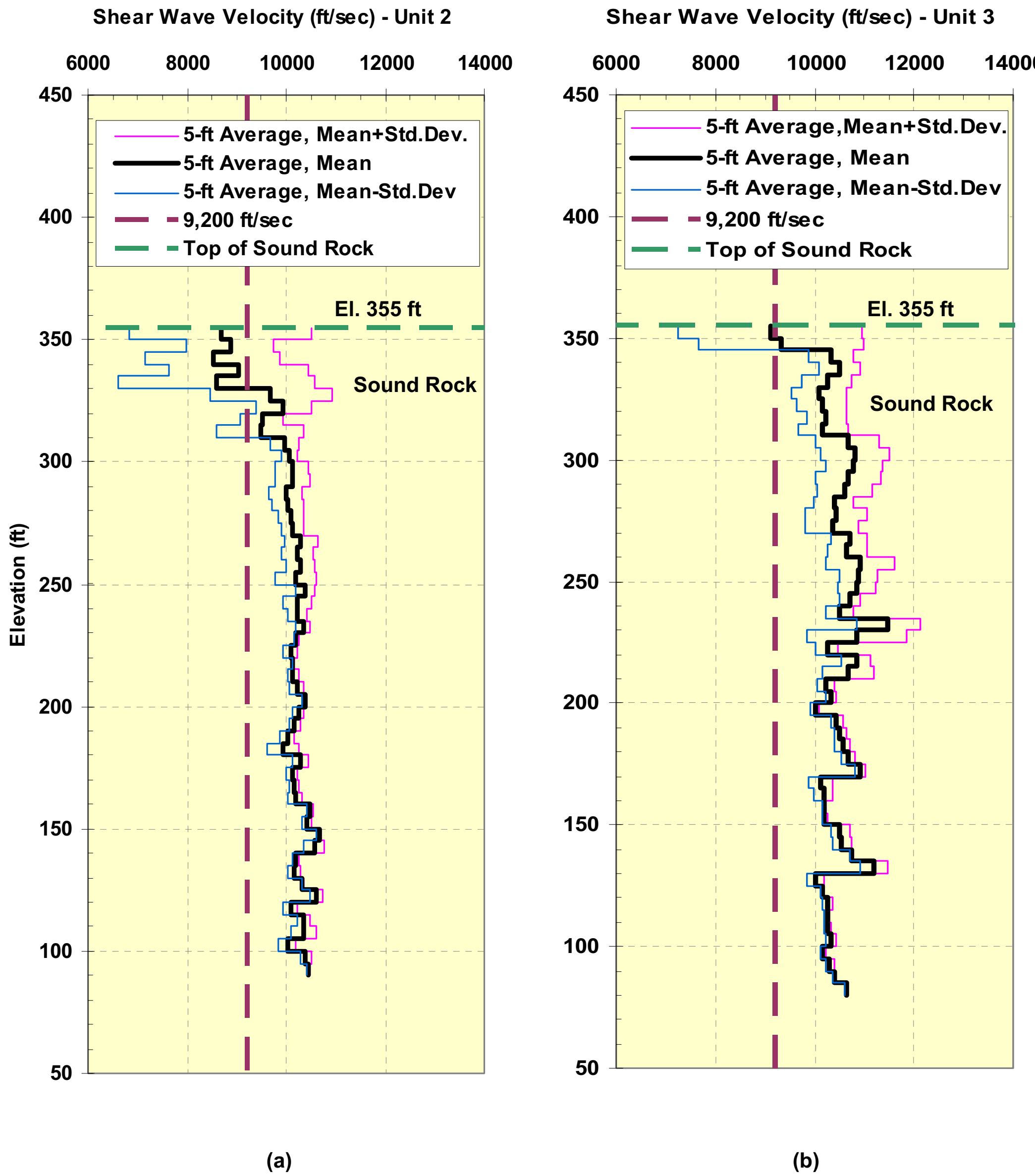


Figure 2.5.4-226 Shear Wave Velocity of Layer V with 5-Foot Vertical Distance Averaging

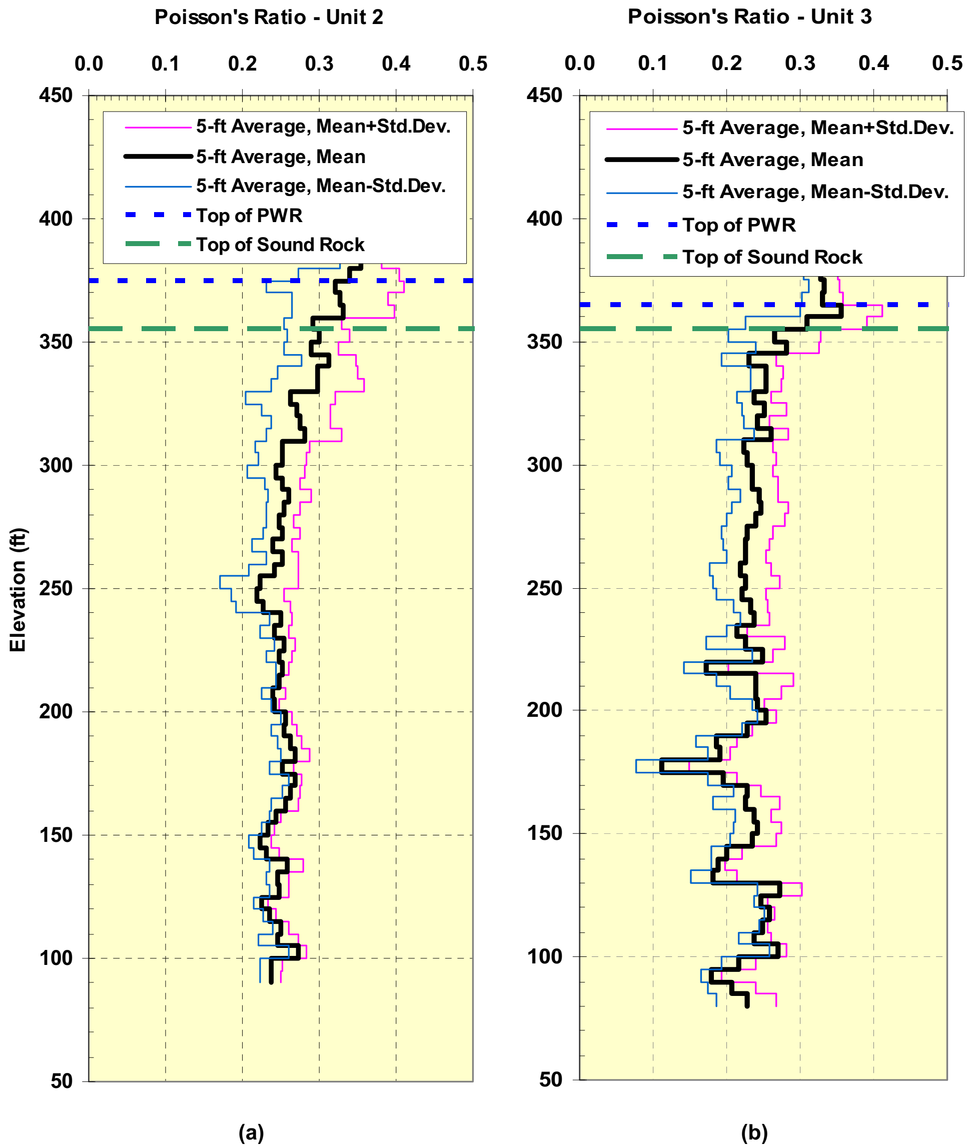


Figure 2.5.4-227 Poisson's Ratio of Layer V with 5-Foot Vertical Distance Averaging

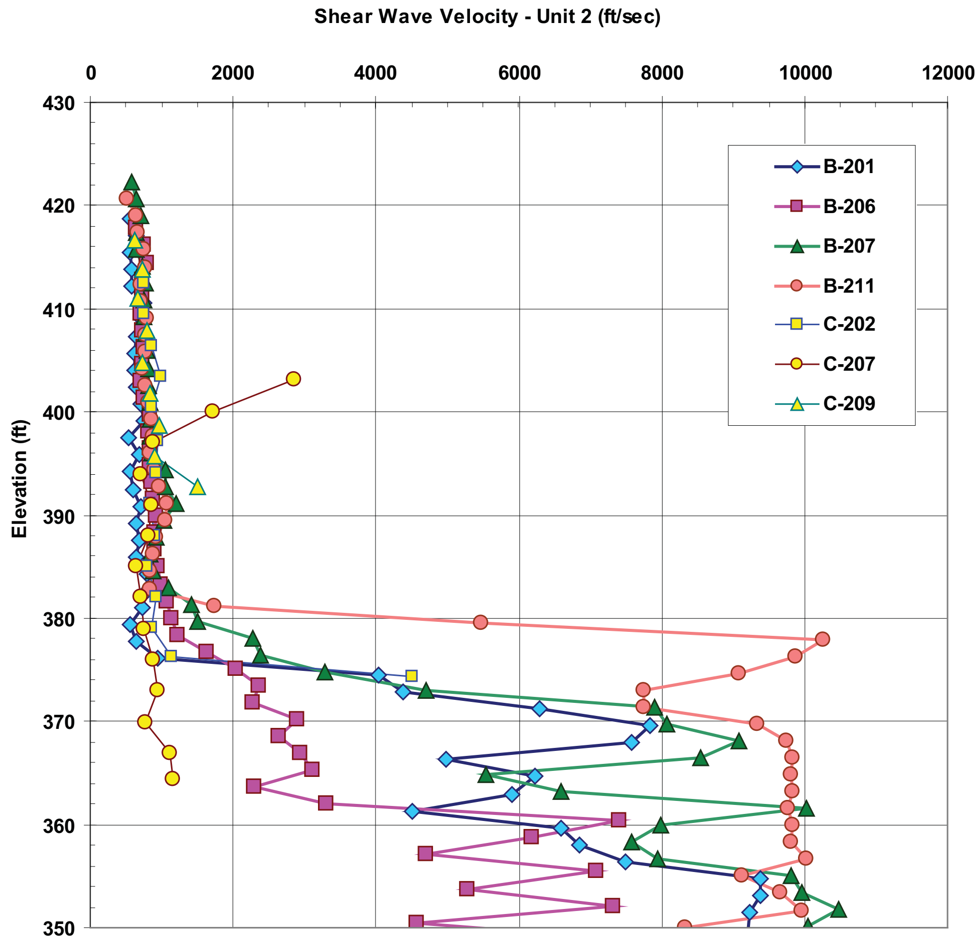


Figure 2.5.4-228 Shear Wave Velocity of Layers I Through IV by Suspension P-S Logging and Seismic CPT (Sheet 1 of 2)

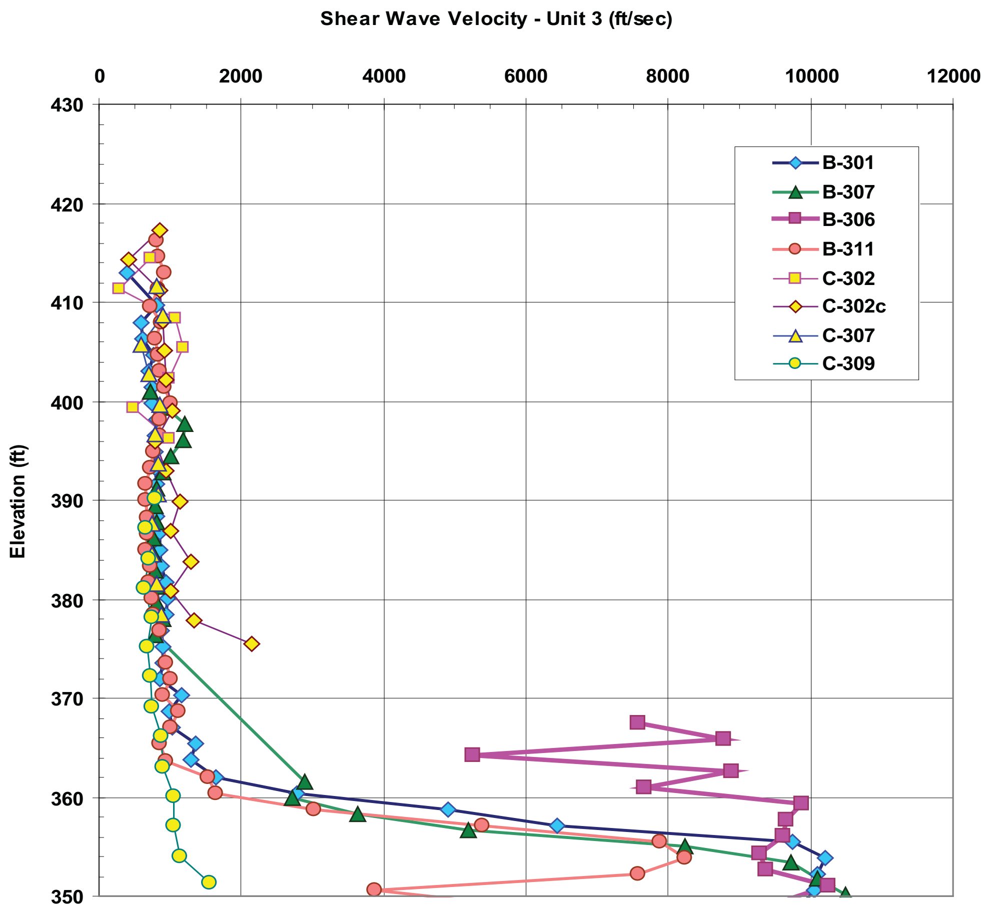


Figure 2.5.4-228 Shear Wave Velocity of Layers I Through IV by Suspension P-S Logging and Seismic CPT (Sheet 2 of 2)

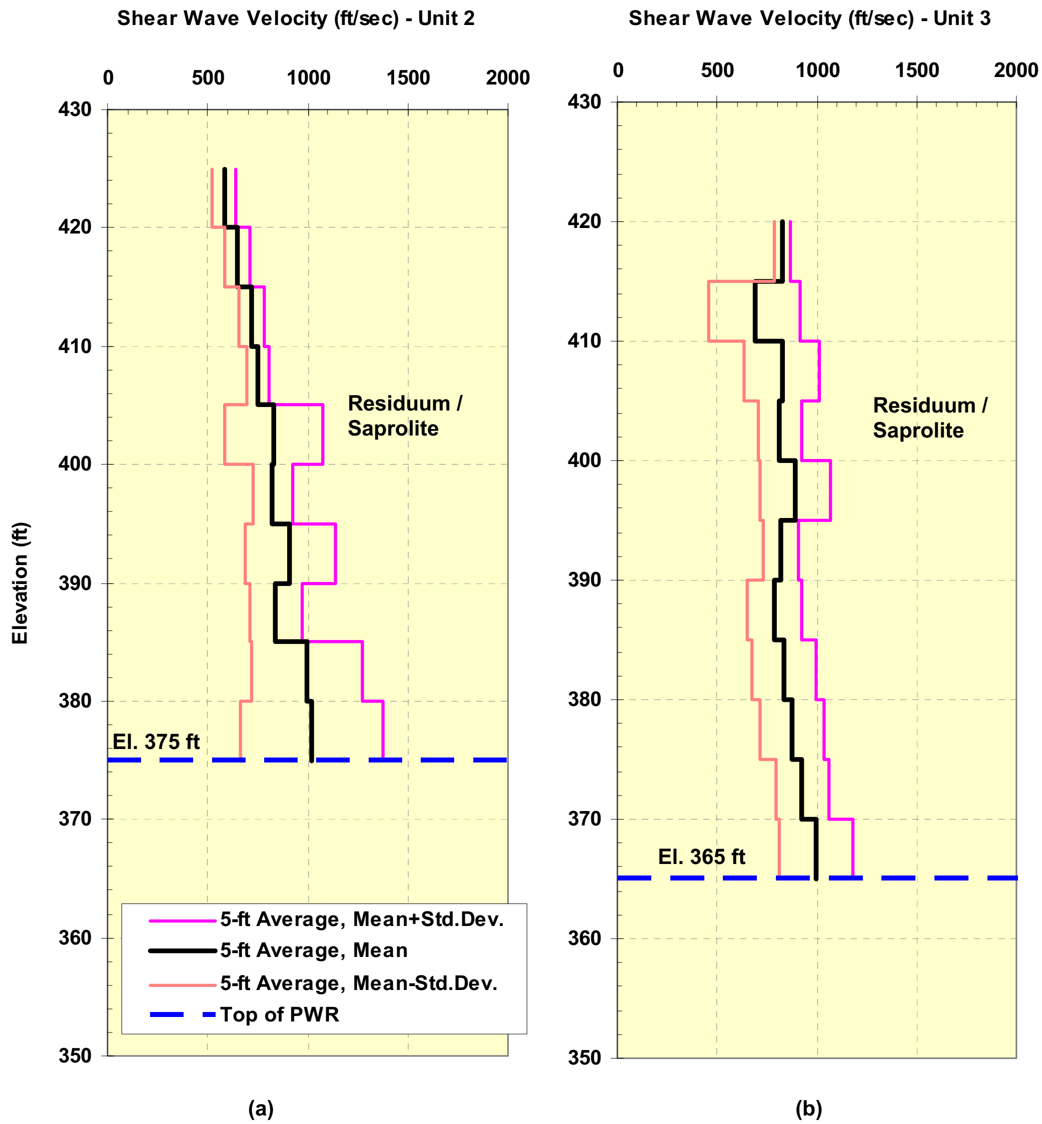


Figure 2.5.4-229 Shear Wave Velocity of Layers I and II with 5-Foot Vertical Distance Averaging

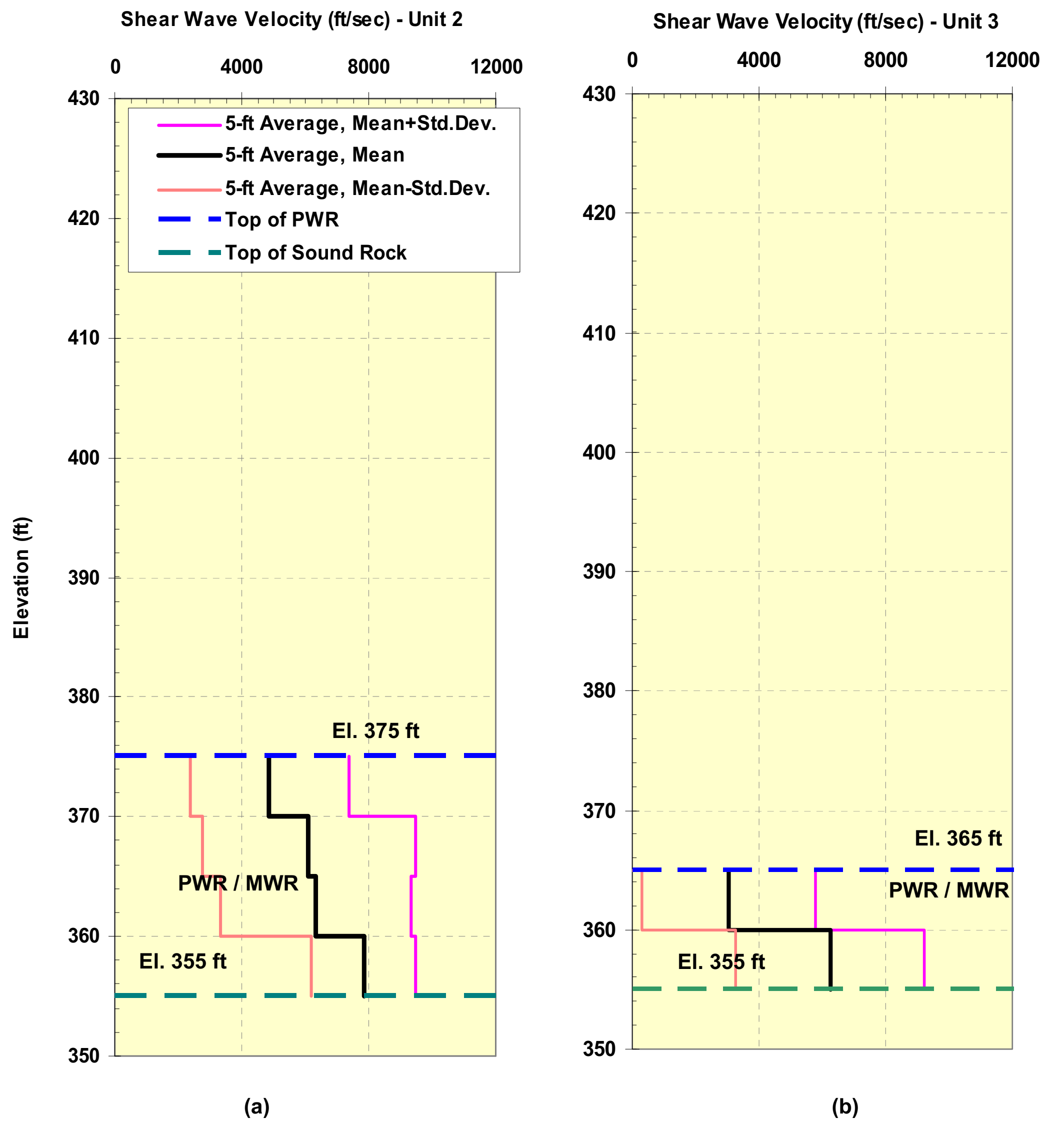


Figure 2.5.4-230 Shear Wave Velocity of Layers III and IV with 5-Foot Vertical Distance Averaging

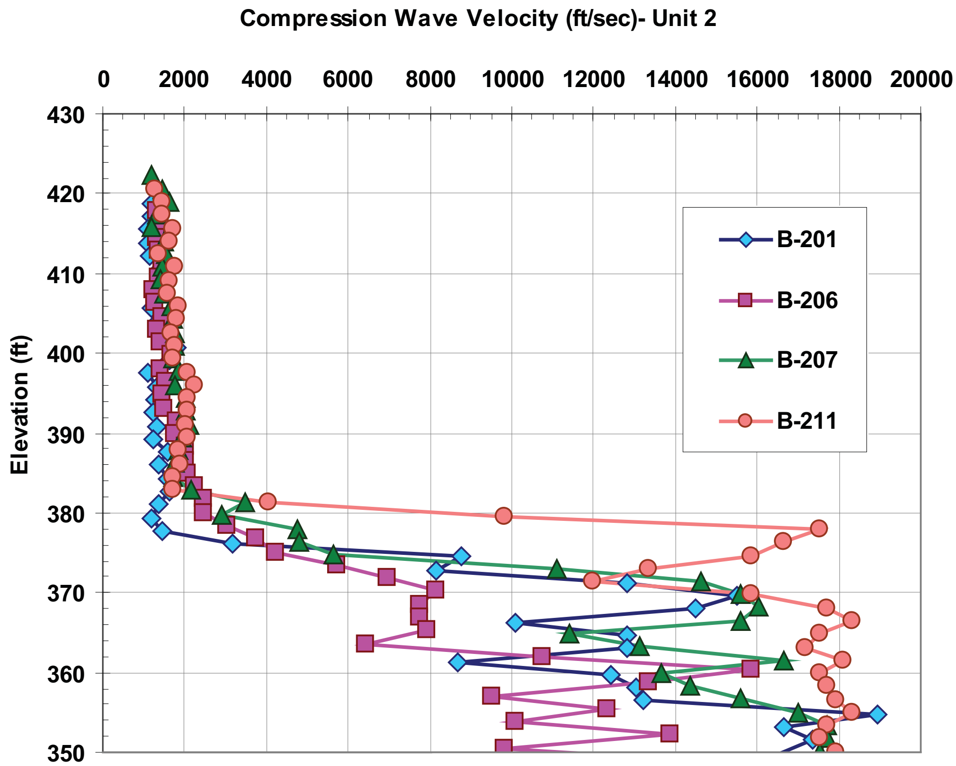


Figure 2.5.4-231 Compression Wave Velocity of Layers I Through IV by Suspension P-S Logging (Sheet 1 of 2)

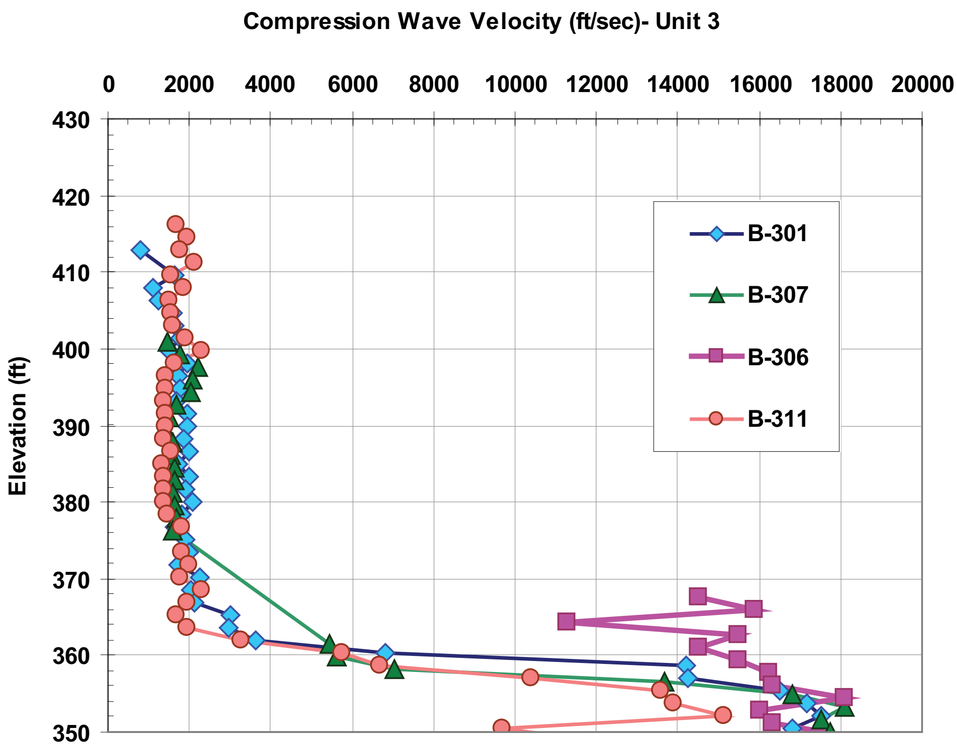


Figure 2.5.4-231 Compression Wave Velocity of Layers I Through IV by Suspension P-S Logging (Sheet 2 of 2)

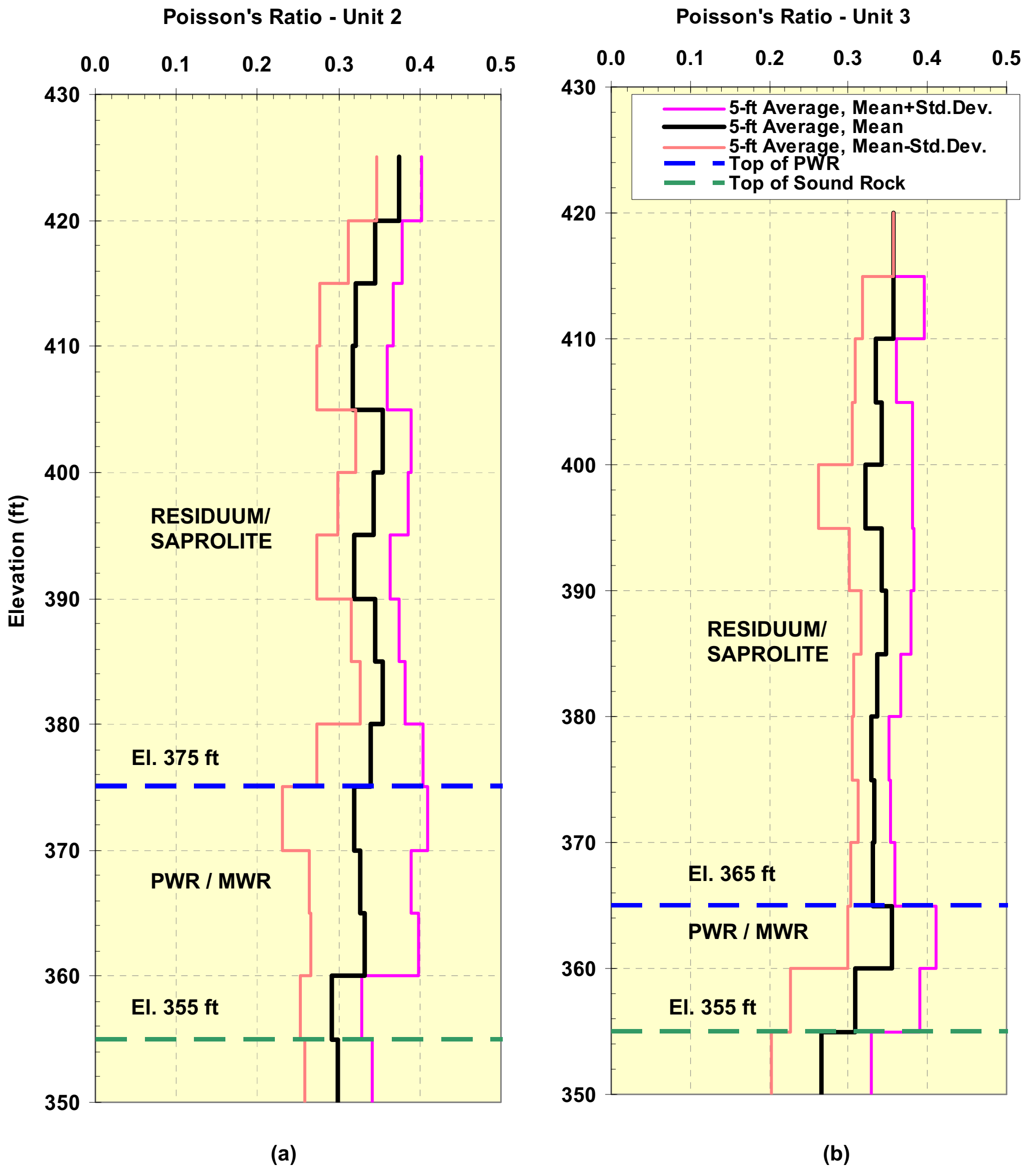


Figure 2.5.4-232 Poisson's Ratio of Layers I, II, III and IV with 5-feet Vertical Distance Averaging

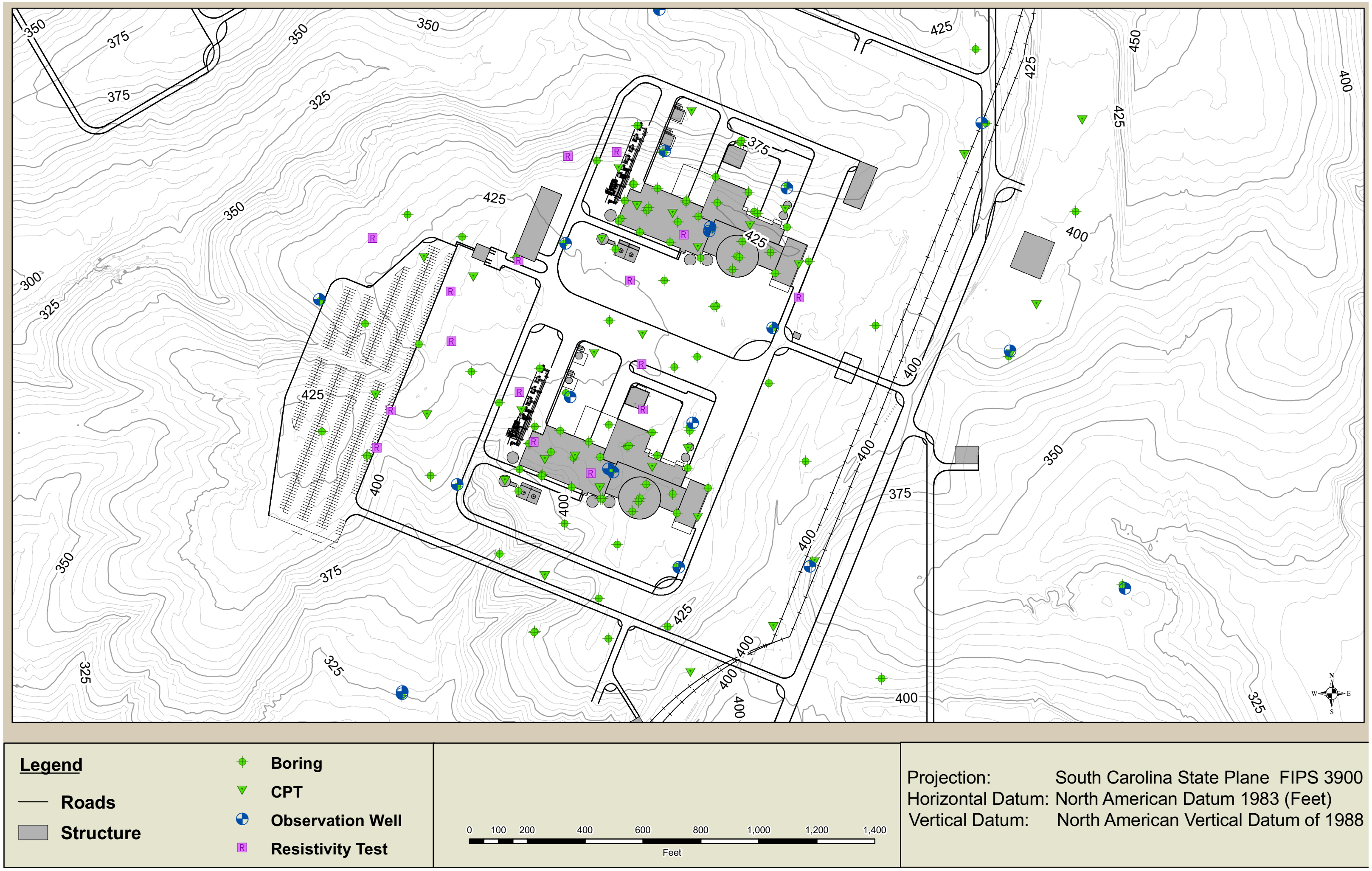
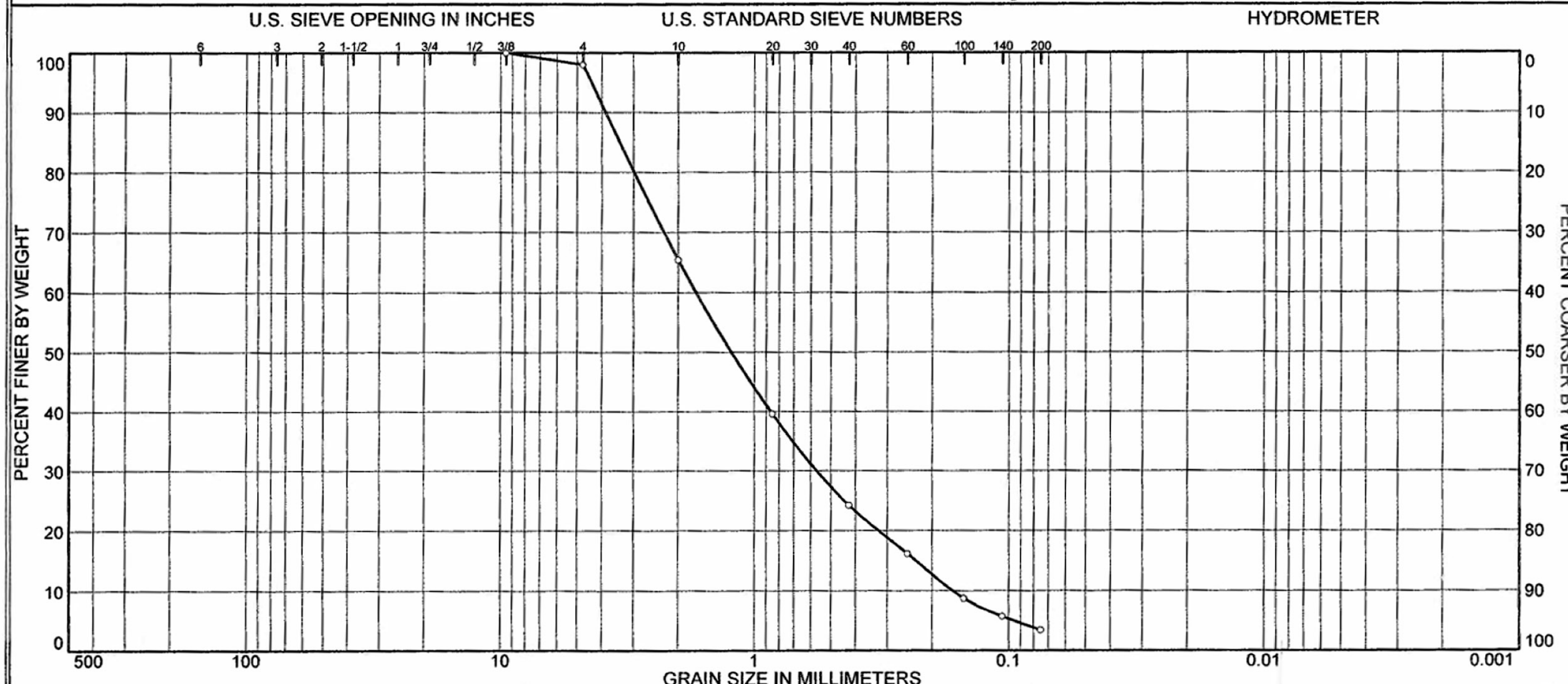


Figure 2.5.4-233 Pre-Construction Site Topography — Units 2 and 3

Particle Size Distribution Report

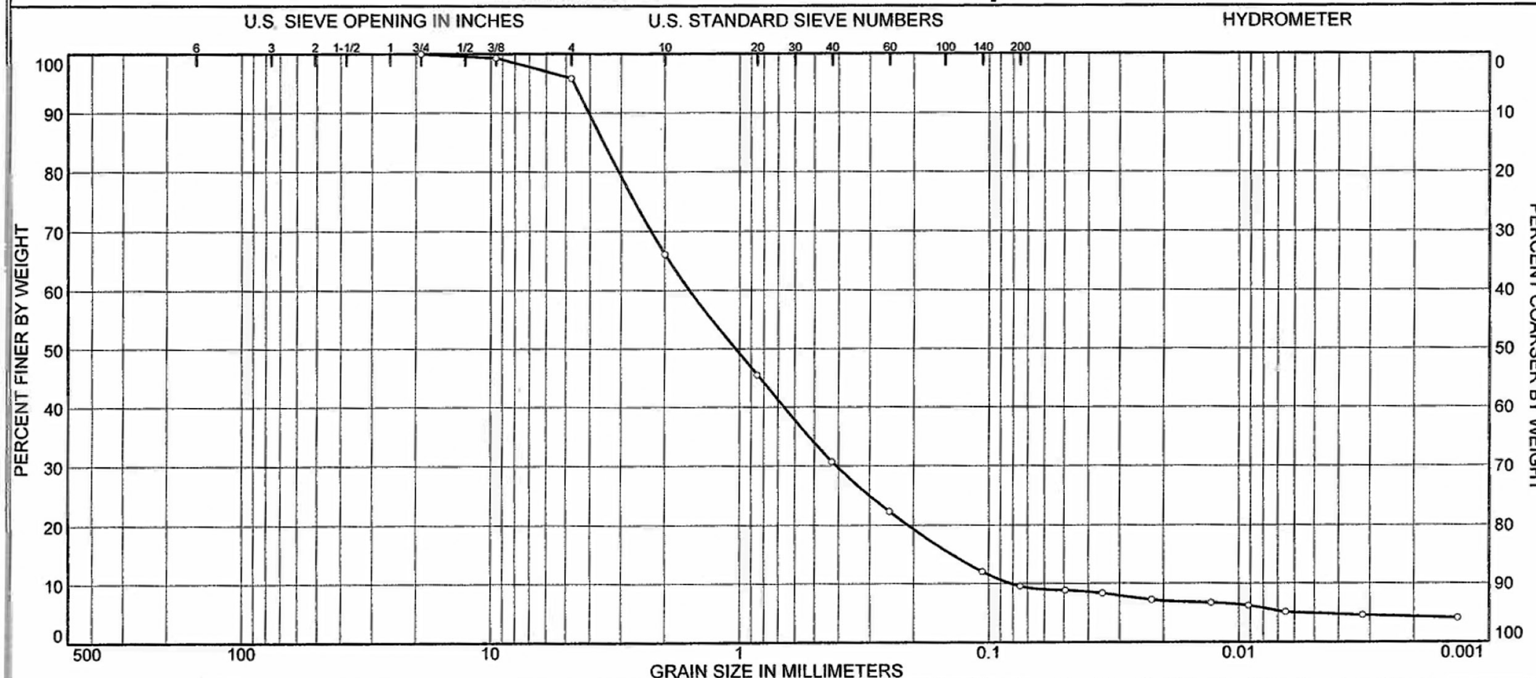


	% COBBLES	% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
○	0	0	2	33	41	21	3	

	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
○	TP-MM1	Bucket 1	n/a	9-26-06		SAND (SW), Dark Gray, Washed Granitic Screenings from Stockpile	5.0		

Figure 2.5.4-234 Particle Size Distribution of Fill Samples (Sheet 1 of 2)

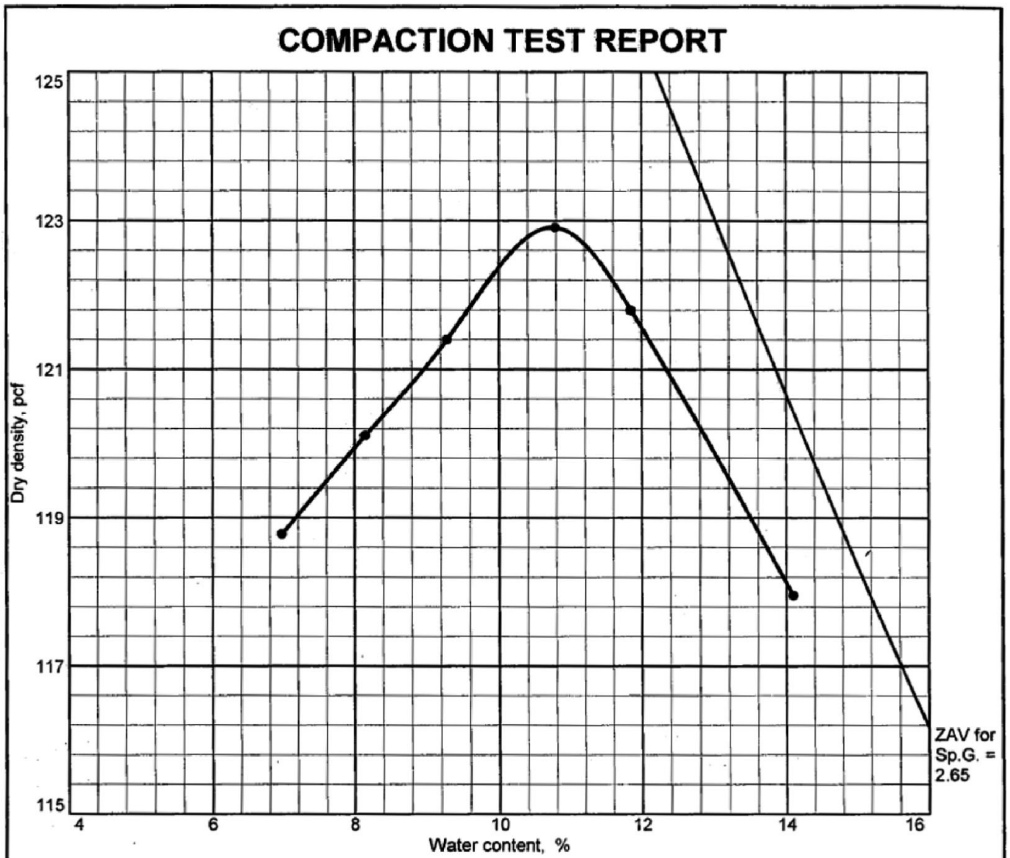
Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
○	0	0	4	30	35	21	5	5

	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PL
○	TP-MM2	Bucket 1	n/a	9-26-06	SW-SM	SAND (SW-SM) with Silt, Dark Gray, Unwashed Granitic Screenings from Stockpile	1.7		

Figure 2.5.4-234 Particle Size Distribution of Fill Samples (Sheet 2 of 2)



Test specification: ASTM D 1557-02 Method A Modified

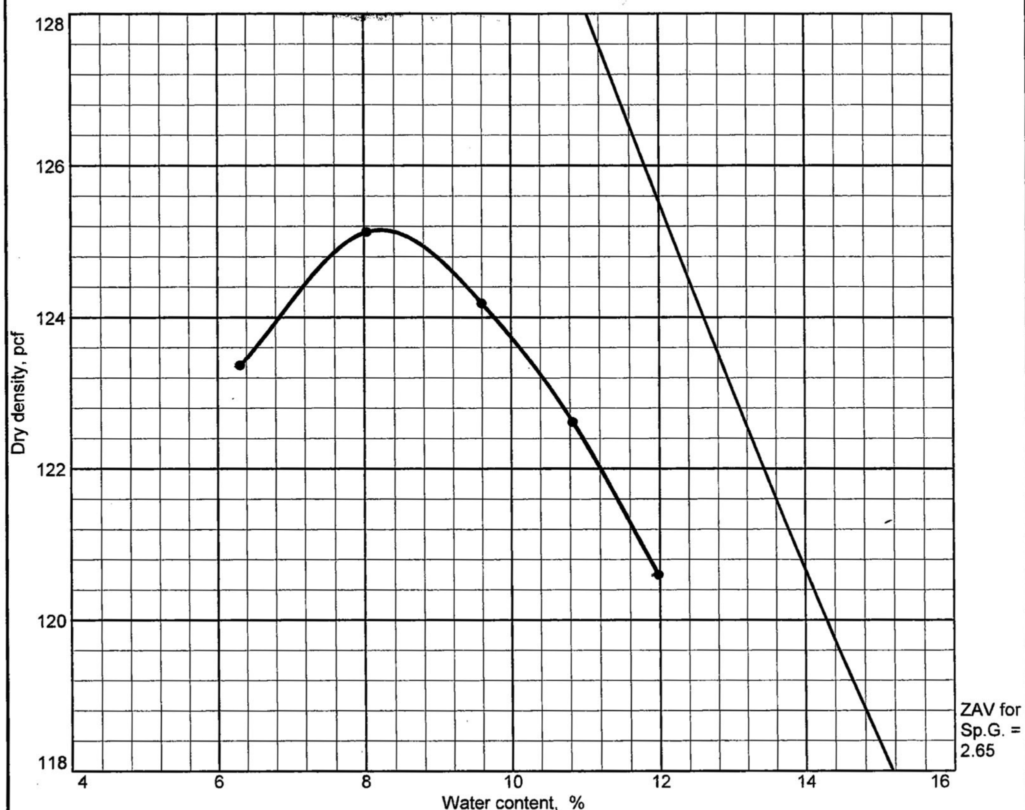
Location: TP-MMI

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
n/a			5.0%	2.65			2.0	3.4

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 122.9 pcf Optimum moisture = 10.7 %	SAND (SW), Dark Gray, Washed Granitic Screenings from Stockpile

Figure 2.5.4-235 Modified Proctor Compaction on Fill Samples (Sheet 1 of 2)

COMPACTION TEST REPORT



Test specification: ASTM D 1557-02 Method A Modified

Location: TP-MM2

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
n/a			1.7%	2.65			4.1	9.5

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 125.2 pcf Optimum moisture = 8.2 %	SAND (SW-SM) with Silt, Dark Gray, Unwashed Granitic Screenings from Stockpile

Figure 2.5.4-235 Modified Proctor Compaction on Fill Samples (Sheet 2 of 2)

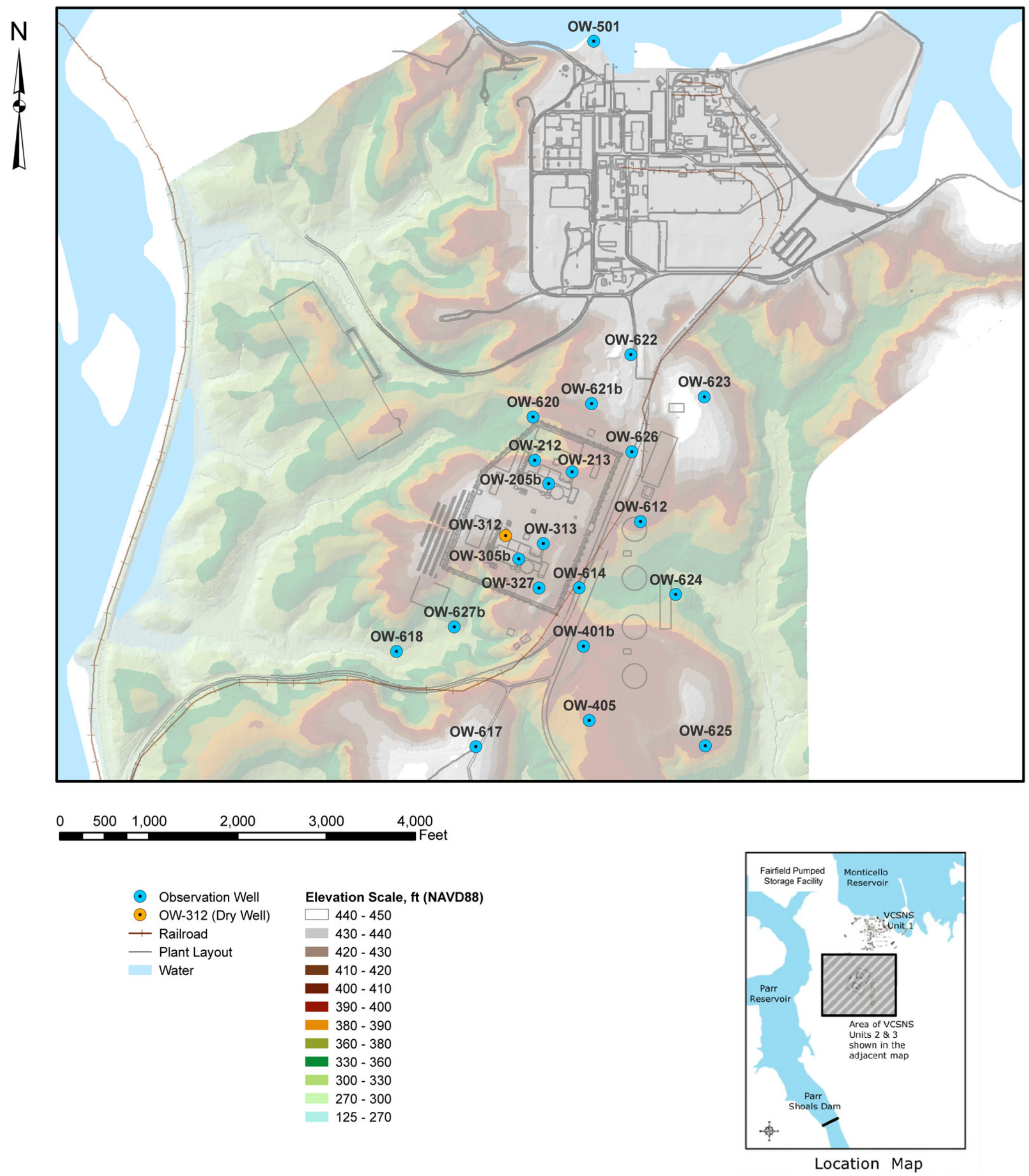


Figure 2.5.4-236 Shallow Groundwater Observation Well Locations

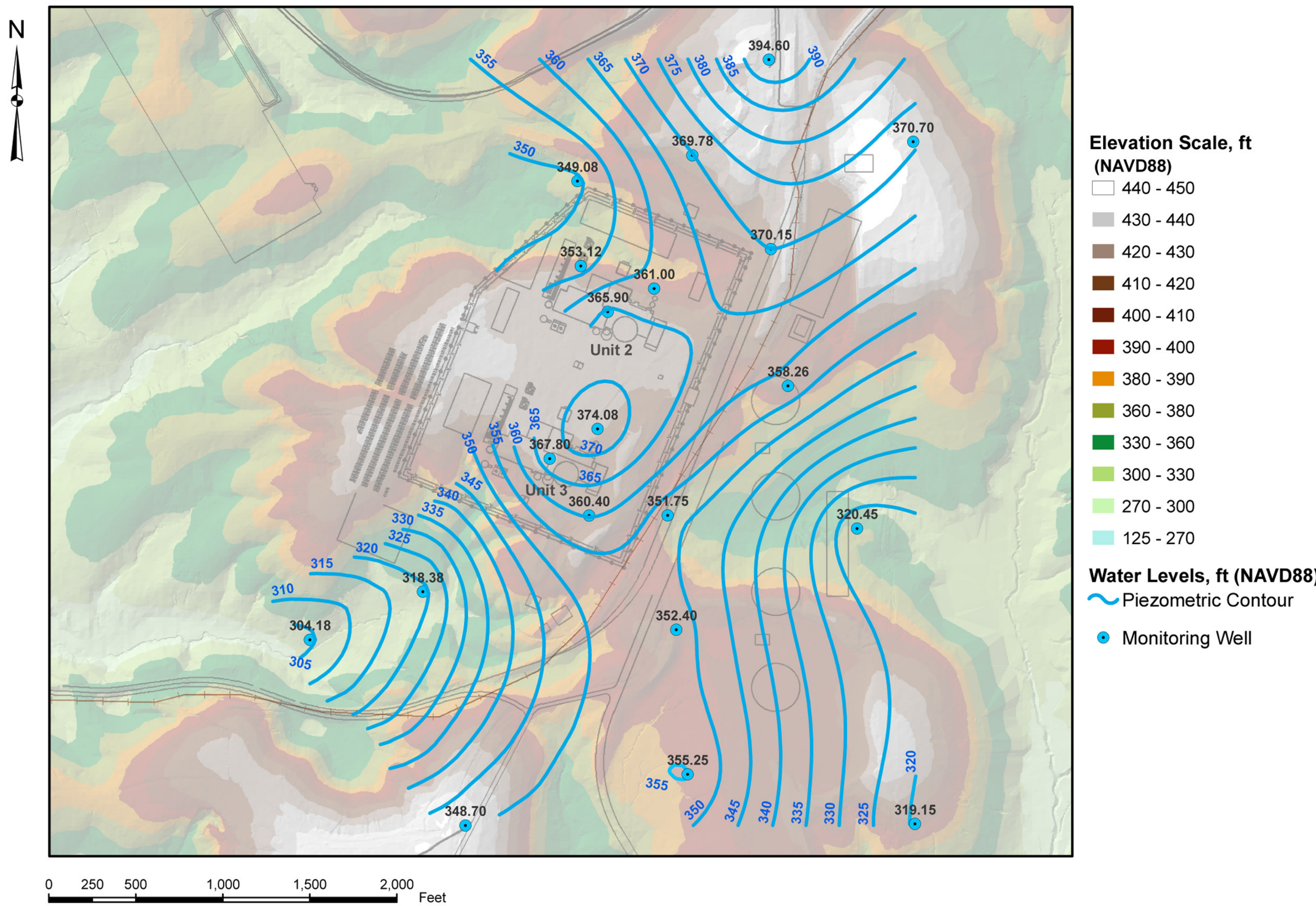


Figure 2.5.4-237 Piezometric Level Contours, 4th Quarter, March 2007 — Units 2 and 3

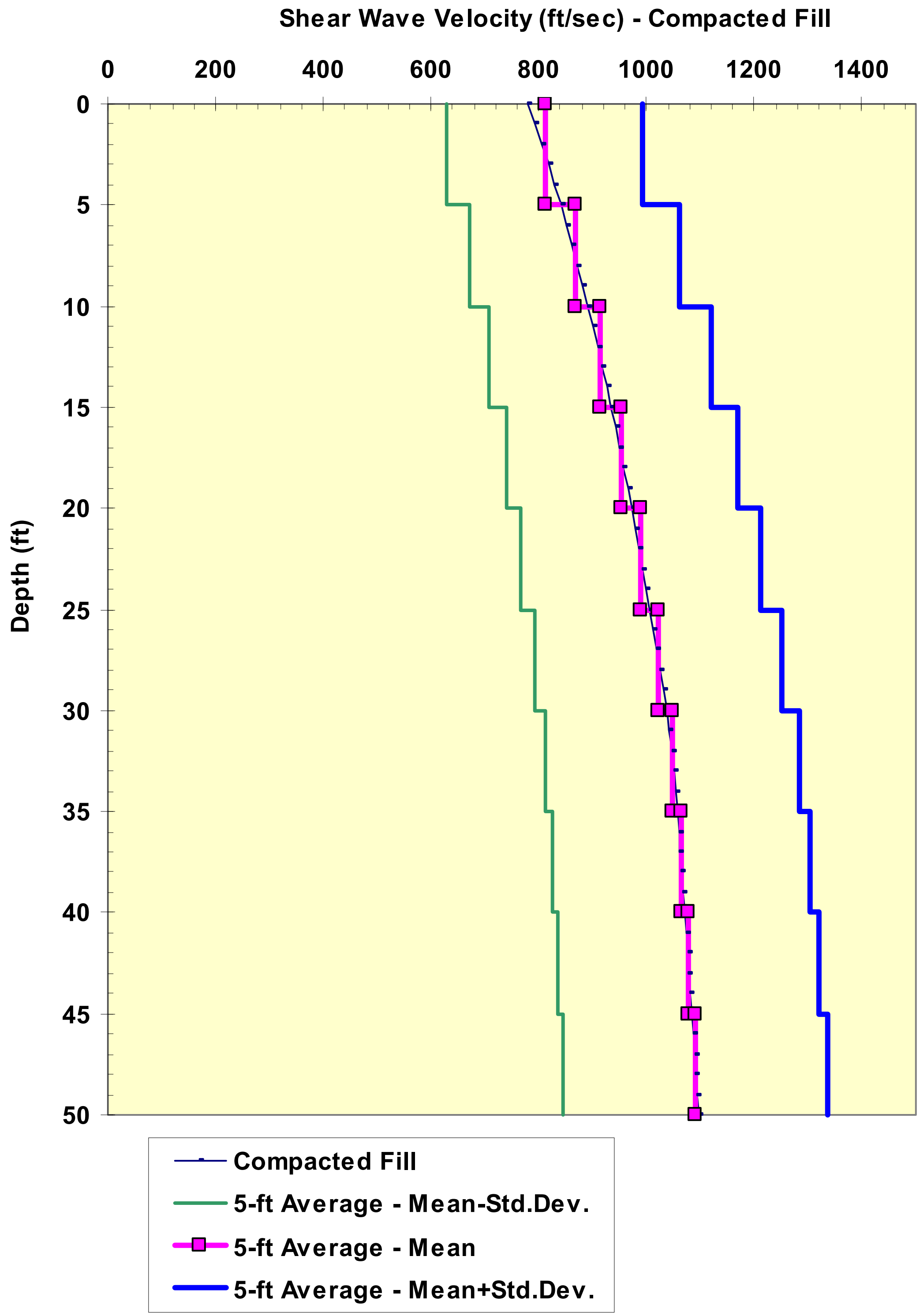


Figure 2.5.4-238 Shear Wave Velocity versus Depth for Structural Fill

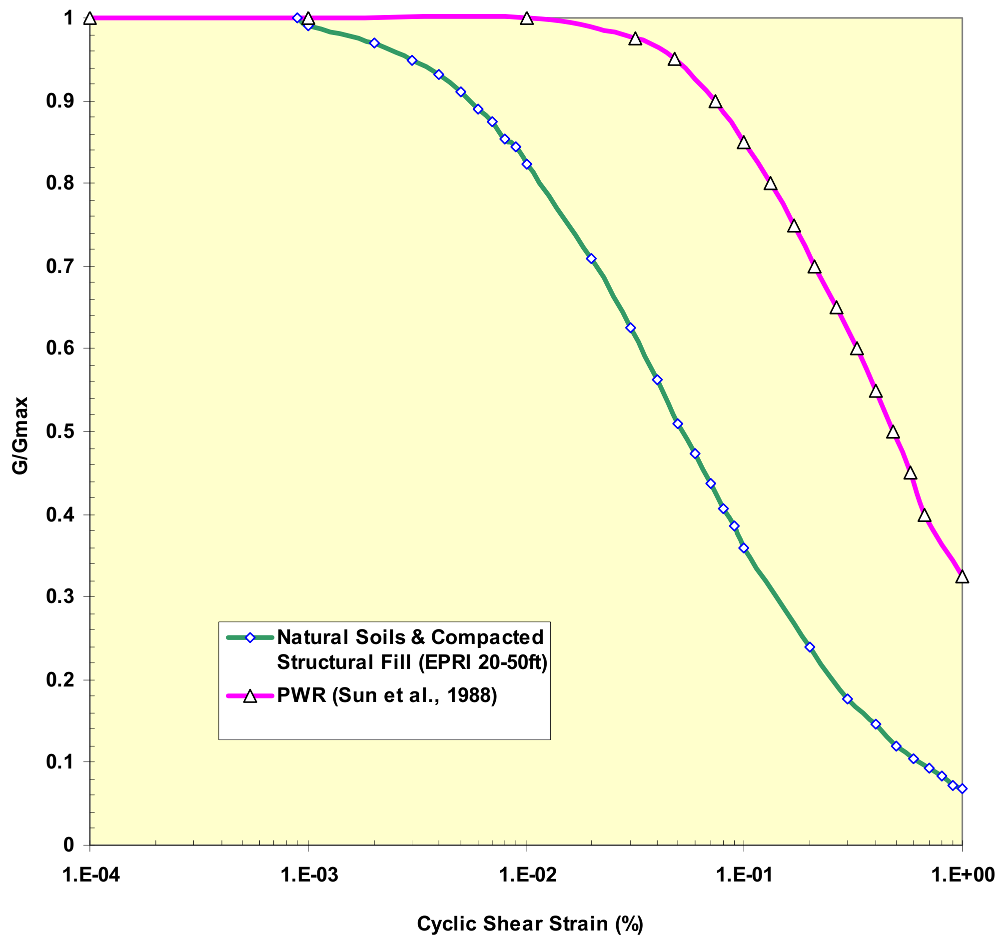


Figure 2.5.4-239 Shear Modulus Reduction Curves

a. Saprolite Sample B-309 UD-2

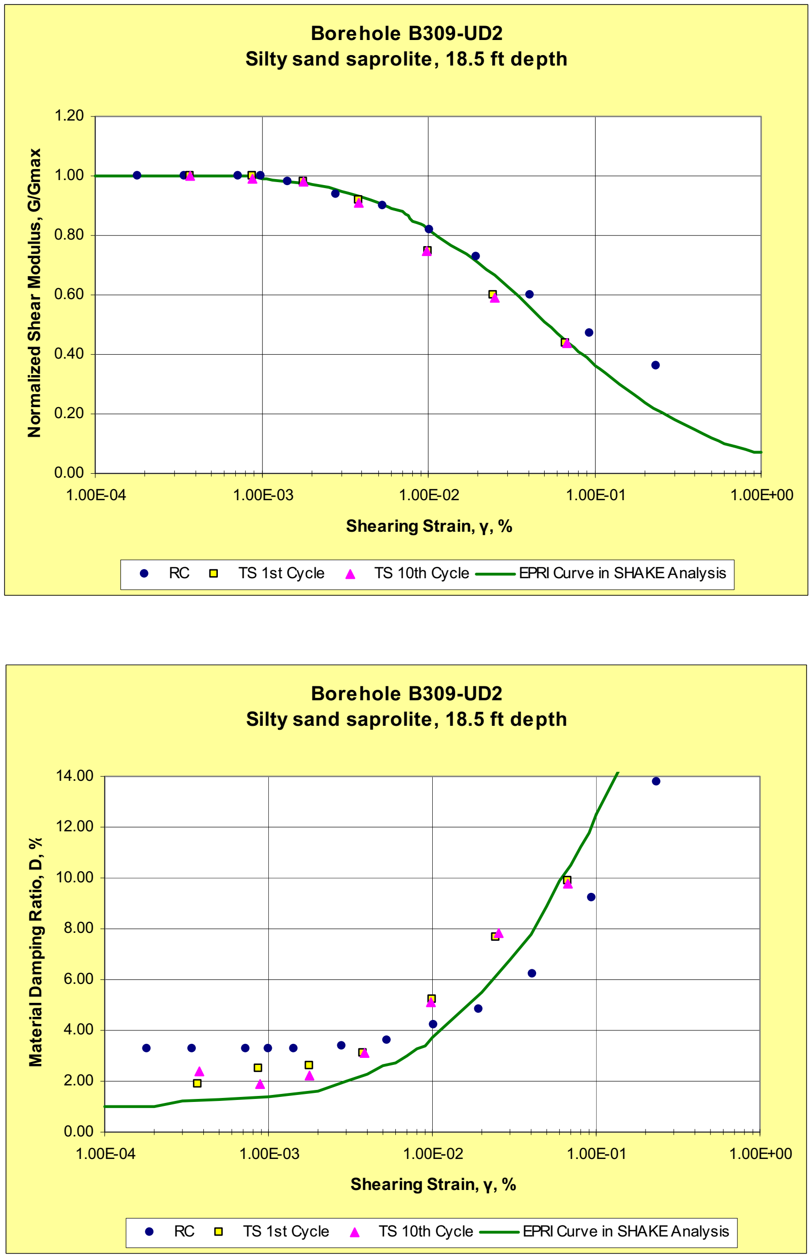
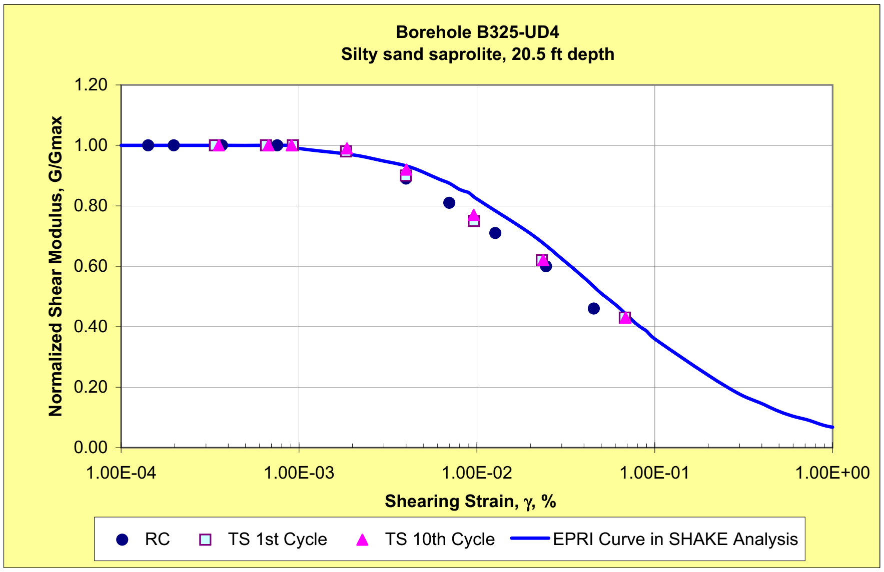


Figure 2.5.4-240 EPRI Curves for G/G_{MAX} and D Versus Shear Strain Superimposed on RCTS Results (Sheet 1 of 5)

b. Saprolite Sample B-325 UD-4



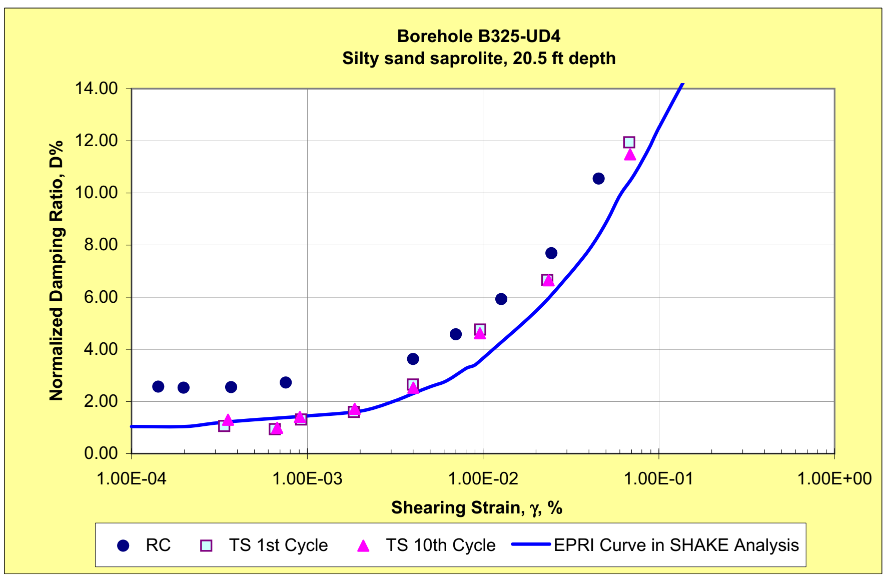


Figure 2.5.4-240 EPRI Curves for G/G_{MAX} and D versus Shear Strain Superimposed on RCTS Results (Sheet 2 of 5)

c: Saprolite Sample B-208 UD-3

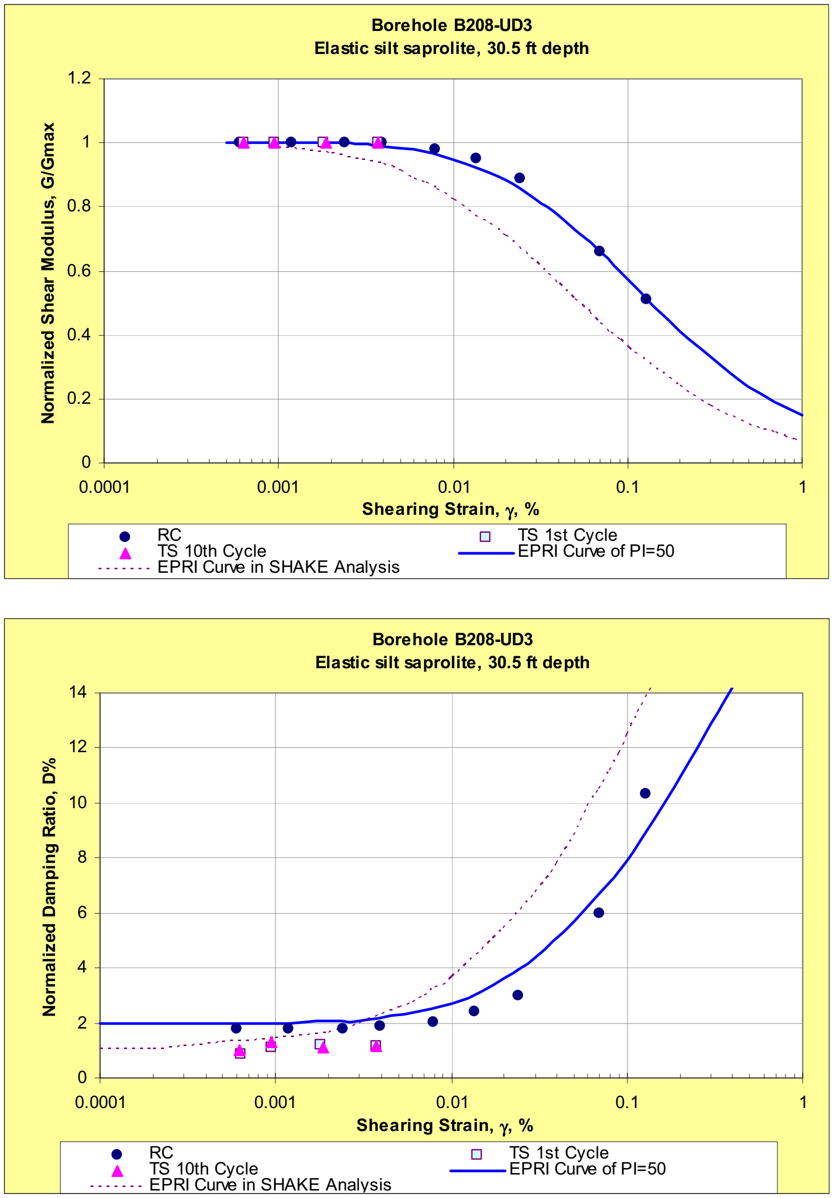
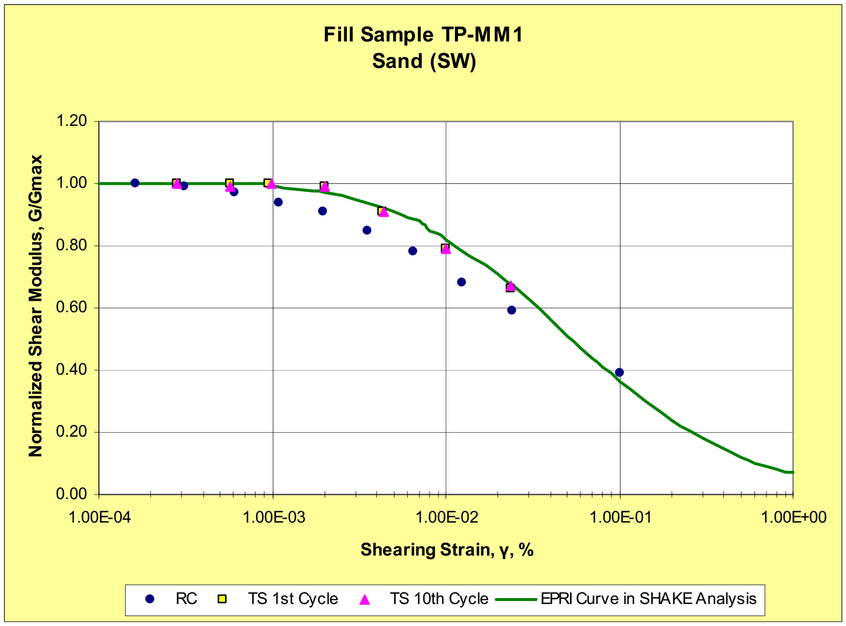


Figure 2.5.4-240 EPRI Curves for G/G_{MAX} and D versus Shear Strain Superimposed on RCTS Results (Sheet 3 of 5)

d. Fill Sample MM-1



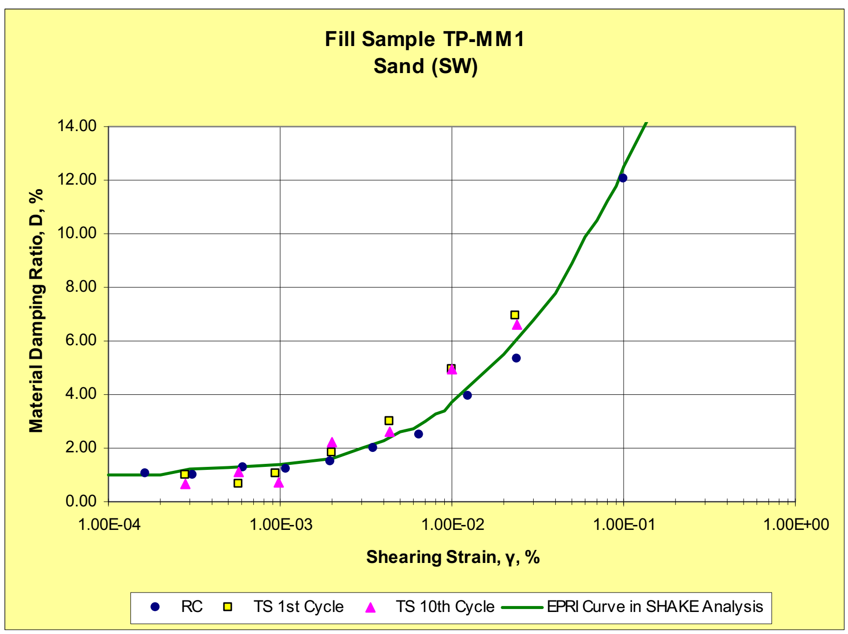
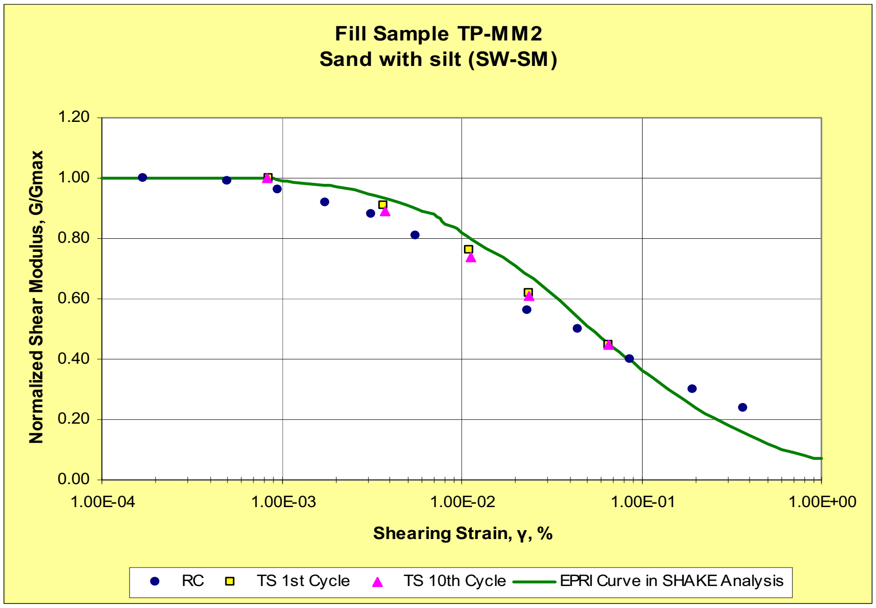


Figure 2.5.4-240 EPRI Curves for G/G_{MAX} and D Versus Shear Strain Superimposed on RCTS Results (Sheet 4 of 5)

e. Fill Sample MM-2



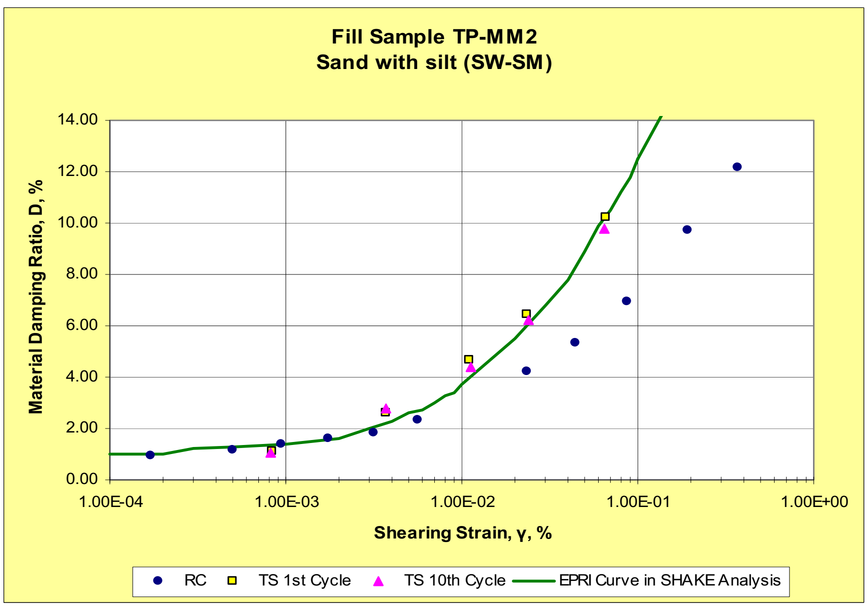


Figure 2.5.4-240 EPRI Curves for G/G_{MAX} and D Versus Shear Strain Superimposed on RCTS Results (Sheet 5 of 5)

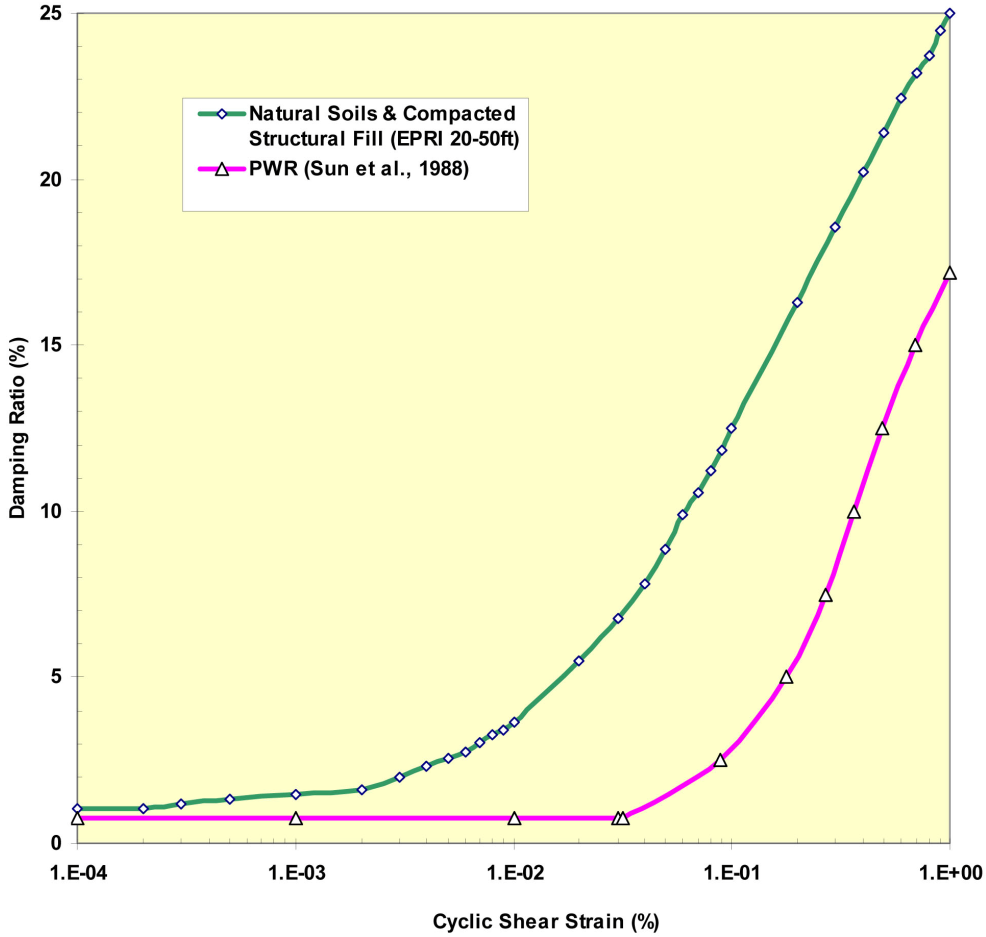


Figure 2.5.4-241 Damping Ratio Curves

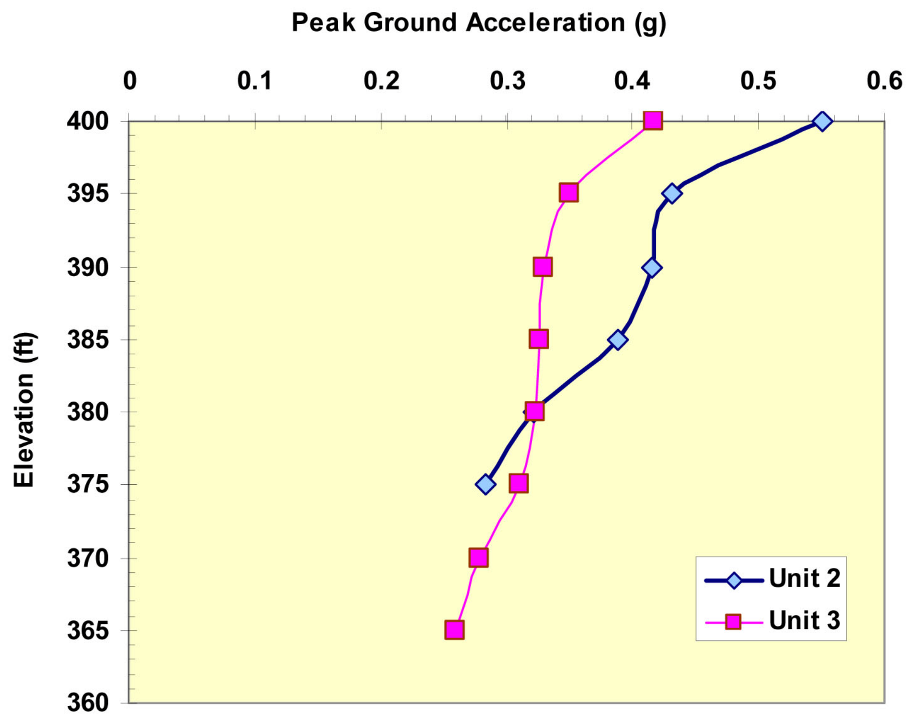


Figure 2.5.4-242 Peak Ground Acceleration Profile in Natural Soils

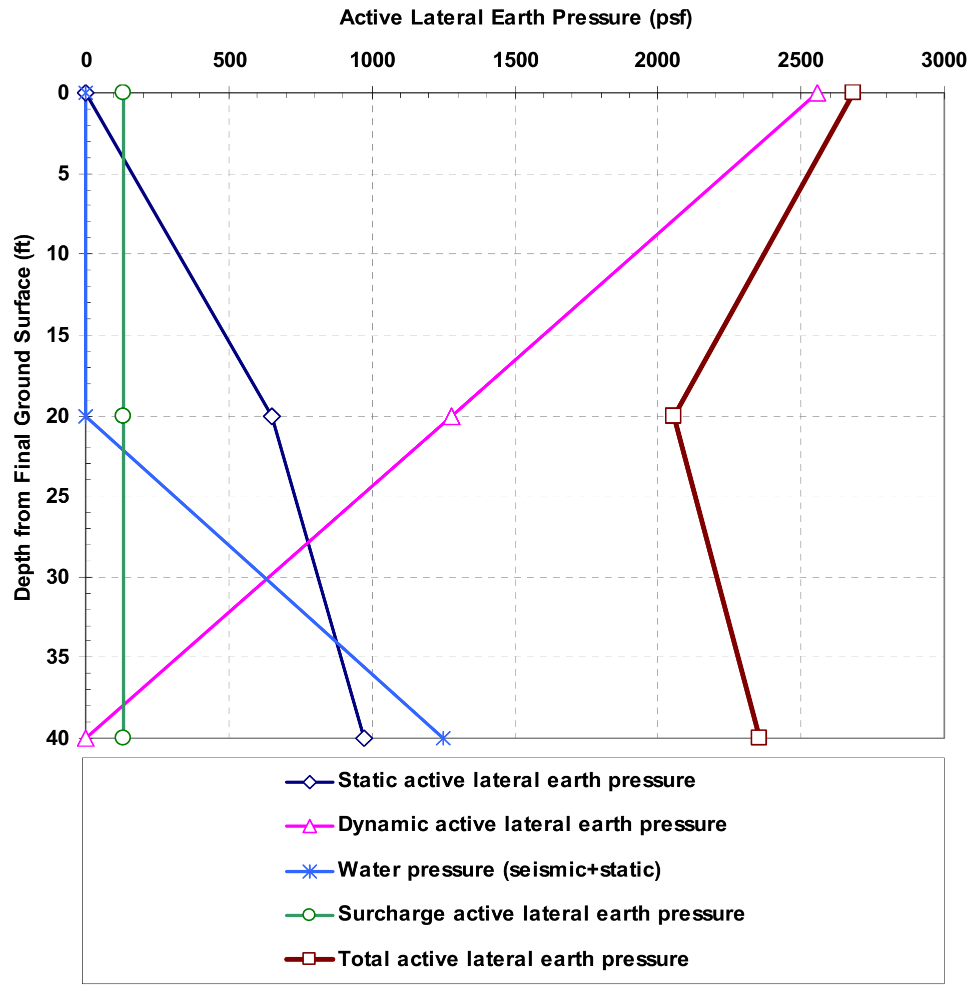


Figure 2.5.4-243 Active Lateral Earth Pressure Diagrams

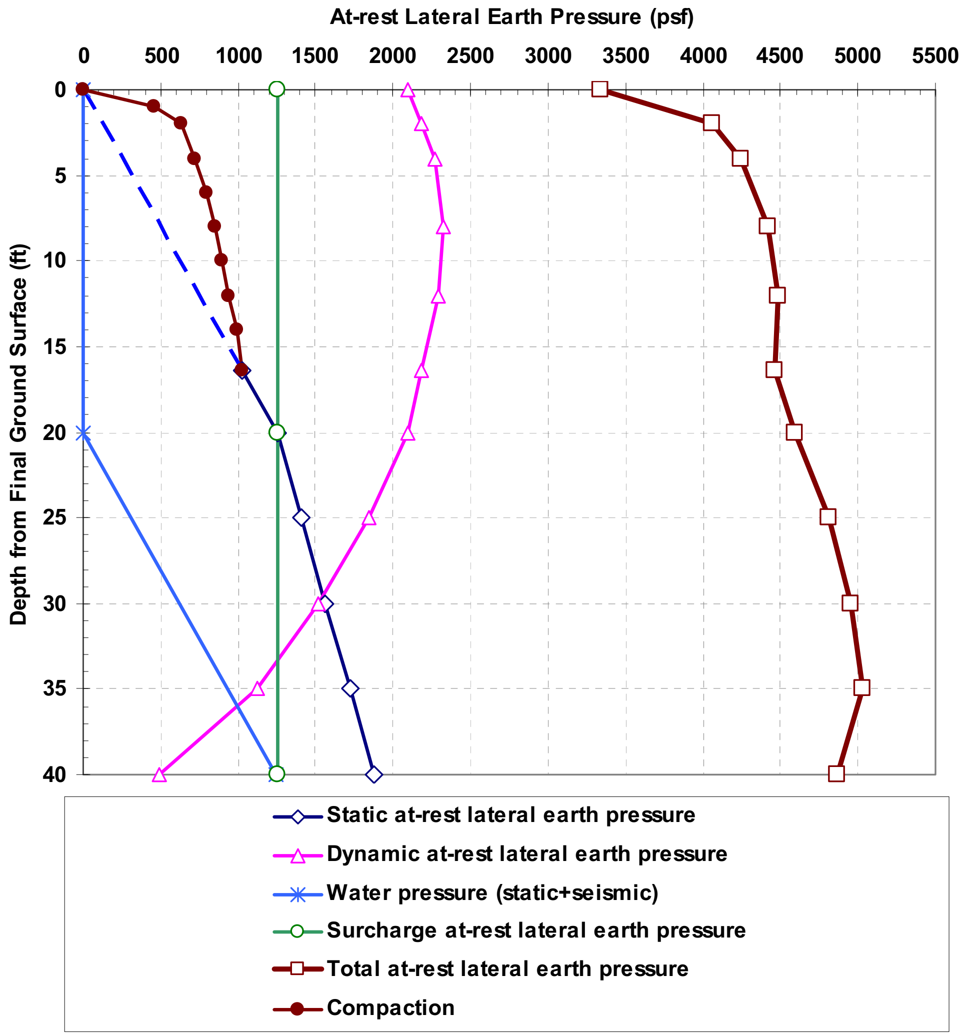


Figure 2.5.4-244 At-Rest Lateral Earth Pressure Diagrams

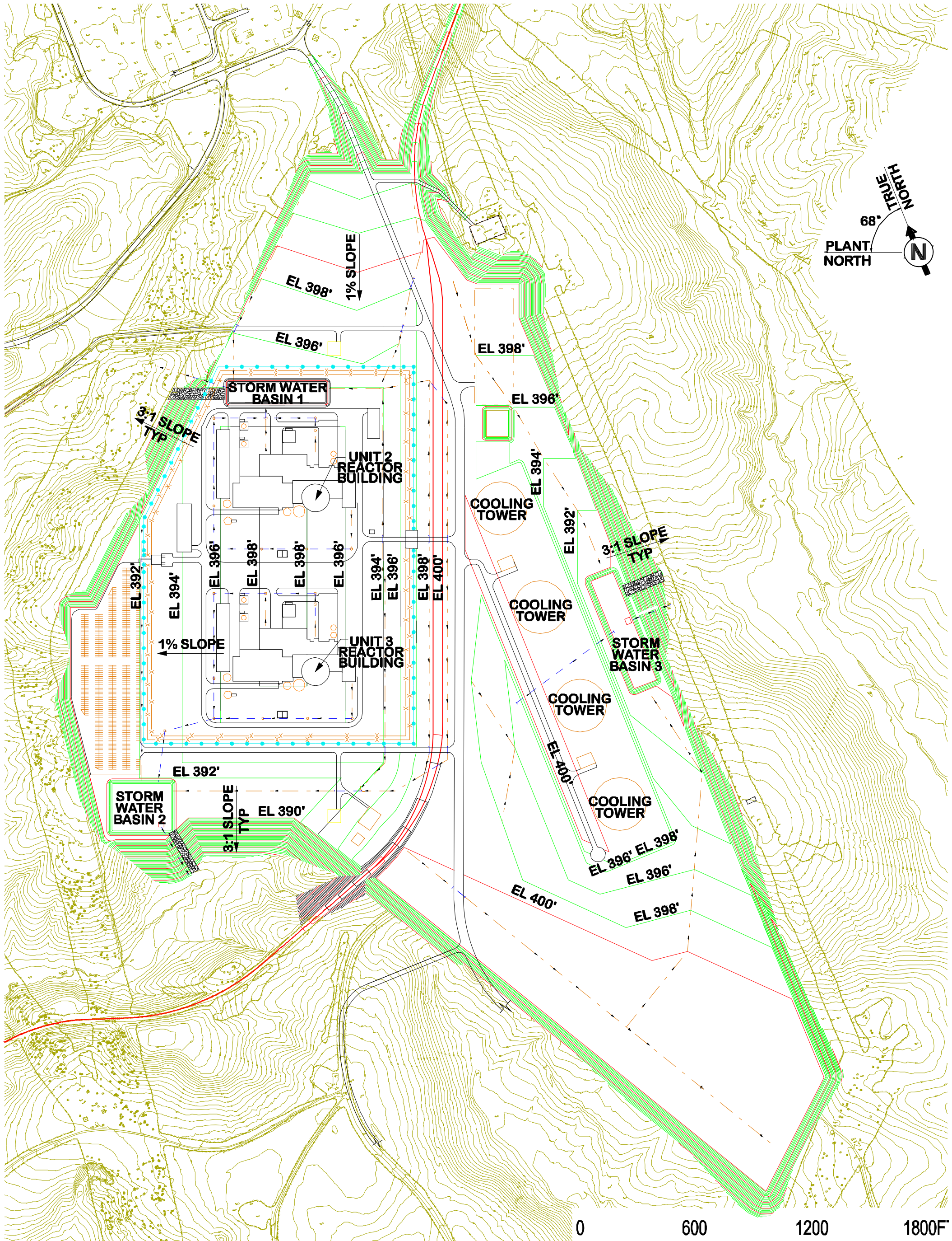


Figure 2.5.4-245 Site Grade Plan