2.5S.4 Stability of Subsurface Materials and Foundations

The following site-specific supplement addresses COL License Information Item 2.26.

Presented in this subsection are the details of the subsurface materials and foundation conditions for STP 3 & 4. It was prepared based on guidance presented in the relevant sections of RG 1.206 (Reference 2.5S.4-1).

The geotechnical information presented in this subsection is based on the results of a subsurface investigation conducted at STP 3 & 4 and on an evaluation of the collected data from this subsurface investigation, unless indicated otherwise. The referenced collected data are contained in Reference 2.5S.4-2, which is also presented as Appendix 2.5-A.

The STP 1 & 2 Updated Final Safety Analysis Report (UFSAR) (Reference 2.5S.4-3) contains the geotechnical information from previous subsurface investigations and subsequent analyses, and from the construction of those existing units. The proposed location of STP 3 & 4 is approximately 2000 feet northwest of the existing STP 1 & 2. This subsection includes comparisons between the STP 1 & 2 UFSAR (Reference 2.5S.4-3) and the STP 3 & 4 geotechnical information presented here, where the STP 1 & 2 specific information is of similar content.

2.5S.4.1 Geologic Features

Subsection 2.5S.1.1 addresses regional geologic setting, including regional physiography and geomorphology, regional geologic history, regional stratigraphy, regional tectonic and neo-tectonic conditions, and potential regional geologic hazards, and provides related maps, cross-sections, and references.

Subsection 2.5S.1.2 addresses geologic conditions specific to the site, including site structural geology, site physiography and geomorphology, site geologic history, site stratigraphy and lithology, site seismic conditions, and potential site geologic hazards, accompanied by related maps, figures, and references.

As noted above, both Subsections 2.5S.1.1 and 2.5S.1.2 address potential geologic hazards, both regional and site-specific, including among other things subsidence, solutioning/karst, zones of irregular weathering, zones of structural weakness, and unrelieved residual stresses. Refer to those subsections for additional detail.

Pre-loading (over-consolidation) influences on soil deposits, including estimates of consolidation properties, overconsolidation ratios, pre-consolidation pressures, and methods used for their estimation are addressed in Subsection 2.5S.4.2. Related maps and subsurface profiles specific to the site are also presented in Subsection 2.5S.4.2.

The stability of site soils and their response to dynamic loading is addressed in Subsection 2.5S.4.7. The stability of site soils and their response to static (foundation) loading, including the stability of major foundations is addressed in Subsection 2.5S.4.10.

In summary, geologically the site is in the Coastal Prairies sub-province of the Gulf Coastal Plains physiographic province. The soils present at the site surface consist of Beaumont

Formation sediments. These soils are Pleistocene in age and were deposited by ancestral rivers during a period of glacial recession, or high sea level. The Beaumont Formation extends to a minimum depth of approximately 750 feet below ground surface at the STP site, and is underlain by additional soil deposits of Pleistocene, Pliocene, and Miocene ages. These additional soil deposits extend to a depth of approximately 4400 feet below ground surface, at which point they transition to the underlying Oakville Sandstone Formation sediments, with a base depth at approximately 6200 feet below ground surface (Reference 2.5S.4-3). These sediments are, in turn, underlain by Cretaceous bedrock, followed by Pre-Cretaceous bedrock ("basement rock") which occurs at a top depth of approximately 34,500 feet below ground surface (Reference 2.5S.4-4). The uppermost approximately 600 feet of Beaumont Formation (Pleistocene) sediments were the subject of the subsurface investigation described below.

2.5S.4.2 Properties of Subsurface Materials

The following site-specific supplement addresses COL License Information Items 2.28, 2.29, and 2.30.

This subsection addresses the properties of subsurface materials, as follows:

- Subsection 2.5S.4.2.1 provides an introduction to the STP 3 & 4 subsurface investigation and the soil strata encountered.
- Subsections 2.5S.4.2.1.1 through 2.5S.4.2.1.12 describe the subsurface conditions and the derived geotechnical engineering properties (both static and dynamic) of the 12 soil strata encountered with depth. Several tables and figures referenced in these subsections present the derived geotechnical engineering properties either spatially (e.g., versus plan location and/or elevation) or comparatively (e.g., comparing one parameter to another parameter).
- Subsection 2.5S.4.2.1.13 describes the chemical properties of the encountered soil strata. Conclusions are drawn in respect of the potential for attack by soil/groundwater constituents on buried steel (i.e., corrosiveness/chloride contents), and in respect of the potential for attack by soil/groundwater constituents on concrete in contact with the ground (i.e., aggressiveness/sulphate contents).
- Subsection 2.5S.4.2.1.14 described the subsurface materials below a depth of approximately 600 feet below ground surface (i.e., below the maximum depth of this subsurface investigation).
- Subsection 2.5S.4.2.1.15 provides a brief overview related to planning of the field testing program for this subsurface investigation.
- Subsection 2.5S.4.2.1.16 provides a brief overview related to planning of the laboratory testing program for this subsurface investigation.
- Subsection 2.5S.4.2.2 provides a detailed description of the field testing program for this subsurface investigation. Field testing types, numbers, and techniques are discussed. Notes regarding conformance of the work to RG 1.132 (Reference 2.5S.4-19) are additionally provided here.

- Subsection 2.5S.4.2.3 provides a detailed description of the laboratory testing program for this subsurface investigation. Laboratory testing types, numbers, and techniques are discussed. Notes regarding conformance of the work to RG 1.138 (Reference 2.5S.4-20) are additionally provided here.

2.5S.4.2.1 Description of Subsurface Materials

The STP site subsurface consists of deep Gulf Coastal Plains sediments underlain by Pre-Cretaceous bedrock ("basement rock"), which has been estimated to occur at a top depth of approximately 34,500 feet below ground surface (Reference 2.5S.4-4). The upper approximately 600 feet of site soils, consisting entirely of the Beaumont Formation, were the subject of this subsurface investigation. These soils are divided into the following strata, consistent with the STP 1 & 2 UFSAR (Reference 2.5S.4-3):

- Stratum A (Clay)
- Stratum B (Silt)
- Stratum C (Sand)
- Stratum D (Clay)
- Stratum E (Sand)
- Stratum F (Clay)
- Stratum H (Sand)
- Stratum J, divided into the following sub-strata
  - Sub-stratum J Clay 1
  - Sub-stratum J Sand/Silt Interbed 1
  - Sub-stratum J Sand 1
  - Sub-stratum J Clay 2
  - Sub-stratum J Sand/Silt Interbed 2
  - Sub-stratum J Sand 2
- Stratum K, divided into the following sub-strata
  - Sub-stratum K Clay
  - Sub-stratum K Sand/Silt
- Stratum L (Clay)
- Stratum M (Sand)

- Stratum N, divided into the following sub-strata
  - Sub-stratum N Clay 1
  - Sub-stratum N Sand 1
  - Sub-stratum N Clay 2
  - Sub-stratum N Sand 2
  - Sub-stratum N Clay 3
  - Sub-stratum N Sand 3
  - Sub-stratum N Clay 4
  - Sub-stratum N Sand 4
  - Sub-stratum N Clay 5
  - Sub-stratum N Sand 5
  - Sub-stratum N Clay 6

Note that Stratum G (Sand), identified in the STP 1 & 2 UFSAR (Reference 2.5S.4-3), was not encountered at STP 3 & 4. Note also that, consistent with the STP 1 & 2 UFSAR (Reference 2.5S.4-3), to avoid confusion with the Roman numeral, the letter "I" has not been used in the stratification system.

Information on deeper soils (i.e., those deeper that approximately 600 feet below ground surface) was obtained from the STP 1 & 2 UFSAR (Reference 2.5S.4-3), and other available literature, and is discussed later in this subsection. Identification of the 12 soil strata, (i.e., A through N, excluding G and I), as noted above, was based on their physical and engineering characteristics. The characterization of soils was based on field testing, including standard penetration testing (SPT) in soil borings with hammer energy measurements, cone penetration test (CPT) soundings, test pits (TP), geophysical downhole (DH) suspension compressional ("P"-wave, V_p) and shear ("S"-wave, V_s) (P-S) velocity logging, field electrical resistivity testing (ER), and observation well (OW) installations, as well as extensive laboratory testing. The extent of field testing is summarized in Table 2.5S.4-1. The as-built locations of subsurface investigation/field testing points are shown on Figures 2.5S.4-1 and 2.5S.4-2. A subsurface profile legend is provided on Figure 2.5S.4-3, the locations of selected subsurface profiles are shown on Figure 2.5S.4-4, and the selected subsurface profiles are shown on Figures 2.5S.4-5 through 2.5S.4-9.

The natural topography at the site at the time of this subsurface investigation was generally level. In the STP 3 & 4 area and the Ultimate Heat Sink (UHS) Basin/Reactor Service Water (RSW) area (i.e., the "Power Block" area as identified on Figures 2.5S.4-1 and 2.5S.4-2), ground surface elevations (El.) at the time of the investigation ranged from El. 24 feet to El. 32 feet, with an average of El. 30 feet. The elevation (rough grade) planned at STP 3 & 4 is El. 34 feet, while the elevation (rough grade) planned at the UHS Basin/RSW area is El. 30 feet. A non-structural earth berm surrounding the UHS Basin to El. 49 feet is additionally planned. It should be noted that all references to elevations given in this subsection are to the National Geodetic Vertical Datum of 1929 (NGVD 29).

As described above, the STP 3 & 4 subsurface conditions were established based primarily on the subsurface investigation information contained in Reference 2.5S.4-2 (Appendix 2.5-A) and reported on here. The subsurface profiles illustrate these conditions. The maximum depth explored by borings drilled as a part of this subsurface investigation was approximately 600 feet below ground surface (Borings B-305DH/DHA and B-405DH [note that Boring B-305DH did not reach planned depth because of a drill bit lost down-hole; a replacement boring, Boring B-305DHA, was offset 20 feet from the original boring, and was completed to planned depth]). The maximum depth explored by CPTs performed as a part of this subsurface investigation was approximately 100 feet below ground surface (CPTs C-304, C-309, C-310, and C-408). Note that CPTs could not consistently be advanced deeper, mainly because of high soil density and/or stiffness. Field test quantities are summarized in Table 2.5S.4-1. Field testing (i.e., borings, CPTs, TPs, P-S velocity logging, ERs, and OWs) identified as 300-series (e.g., B-301, C-301, etc.) were made in the STP 3 area. Field testing identified as 400-series (e.g., B-401, C-401, etc.) were made in the STP 4 area. Field testing identified as 900-series (e.g., B-901, C-901, etc.) were generally made in the UHS Basin/RSW area or in other areas at the site perimeter (i.e., the area "outside the Power Block" as identified on Figures 2.5S.4-1). As bedrock occurs at very significant depth (approximately 34,500 feet below ground surface, as noted above), and as such, is not of interest for earthwork and foundation design or

construction, rock properties are generally not addressed. The 12 identified soil strata from this subsurface investigation (i.e., Strata A though N, excluding G and I), are illustrated, in part, on the subsurface profiles, and are described in detail here.

2.5S.4.2.1.1 Stratum A

Stratum A soils were encountered at ground surface and were fully penetrated by all borings and CPTs made within the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and the area outside the Power Block. Stratum A typically consisted of yellowish red, brown, gray, or black clay with varying amounts of silt, sand, and/or gravel.

The thickness of Stratum A was estimated from the borings and CPTs. Inside the Power Block area, the thickness of Stratum A varied from 8 feet to 29 feet, with an average thickness of 18 feet, and the base elevation varied from El. 0 feet to El. 23 feet, with an average of El. 12 feet. Additional information on the thicknesses and base elevations of this stratum, including areas outside the Power Block is presented in Table 2.5S.4-2. Note that only data from borings and CPTs that encountered and fully penetrated the stratum were considered in evaluating the stratum thickness and in selecting the stratum base elevation.

It should be noted that at isolated locations, clayey and/or gravelly soils, in some cases similar in appearance to Stratum A, were encountered at ground surface, within the upper few feet of the stratum. These soils were suspected of being man-made fill. These Stratum A (Fill) soils were present in 31 borings, namely Borings B-305DH/DHA, B-310, B-311, B-313, B-314, B-316, B-317, B-318, B-323, B-326, B-340, B-343, B-346, B-347, B-401, B-403, B-404, B-405DH, B-406, B-407, B-408DH, B-409, B-412, B-414, B-912, B-913, B-916, B-920, B-929, B-932, and B-933. Their thickness, where present, ranged from 0.5 feet to 14 feet, with an average thickness of two feet.

In the case of all soil strata, soil samples were collected from the borings by SPT sampling and where appropriate by undisturbed (UD) three-inch-diameter tube sampling. SPT samples were collected more frequently in the upper portion of each boring than in the lower portion (e.g., typically 10 SPT samples were obtained in the upper 15 feet; thereafter, SPT samples were obtained at 5 foot intervals to a depth of 100 feet, 10 foot intervals to a depth of 200 feet, and 20 foot intervals to a depth of approximately 600 feet). SPT N-values (uncorrected) were measured during the sampling and were recorded on the boring logs. In the STP 3 area, uncorrected SPT N-values in Stratum A ranged from 0 blows/foot (weight of hammer [WOH]) to 27 blows/foot, with an average uncorrected SPT N-value of 9 blows/foot. In the STP 4 area, uncorrected SPT N-values in Stratum A ranged from 3 blows/foot to 42 blows/foot, with an average uncorrected SPT N-value of 11 blows/foot. In the UHS Basin/RSW area, uncorrected SPT N-values in Stratum A ranged from 3 blows/foot to 41 blows/foot, with an average uncorrected SPT N-value of 11 blows/foot. Additional SPT N-value information on this stratum at areas other than the STP 3 area, the STP 4 area, and the UHS Basin/RSW area is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11 through 2.5S.4-15 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively. The site-wide average uncorrected SPT N-value was 10 blows/foot for Stratum A.

The uncorrected SPT N-value, WOH, noted above, occurred at one sample interval within Stratum A, namely at Boring B-341 from depths 10.5 feet to 12 feet below ground surface. The soft soils sampled at this location, within the proximity of the planned STP 3 Radwaste Building, are excavated during construction for the building foundation.

For all soil strata, SPT N-values from each boring were corrected to an effective overburden pressure of one ton per square foot (tsf) (i.e., N_1). The correction factor for effective overburden pressure was determined for each SPT sample interval using the average unit weights for the individual soil strata as determined by laboratory testing and the soil strata thicknesses at individual borings, according to the formula below (Reference 2.5S.4-5):

$$C_n = 2.2/(1.2 + \sigma_v')$$ Equation 2.5S.4-1

where, C_n = the correction factor, which is multiplied by the uncorrected SPT N-value to yield the normalized SPT N_1-value, and which varies with depth to a maximum value of 1.70

σ_v' = the effective overburden pressure at the depth of the SPT sample interval in tons per square foot (tsf)

Note that a groundwater level at El. 25.5 feet, which was representative of levels measured in observation wells installed as a part of this subsurface investigation, was used in the calculation of effective overburden pressure. Refer to Subsection 2.5S.4.6.1 for additional detail.

Eleven drilling rigs were employed during this subsurface investigation, with SPT hammer energy measurements made at each of the drilling rigs employed. Energy measurements were made in accordance with ASTM D 6066 (Reference 2.5S.4-6). As the SPT N-value used in correlations with engineering properties is the value corrected to 60% hammer efficiency, the normalized N_1-values were further corrected based on the drilling rig-specific hammer energy measurements (energy transfer ratios [ETRs]), in accordance with ASTM D 6066 (Reference 2.5S.4-6). The average hammer energy corrections for hammers employed in this subsurface investigation for ETRs ranging from 72% to 99% were 1.21 to 1.65 (e.g., 72% measured energy/60% base line = 1.21 hammer energy correction; 99% measured energy/60% base line = 1.65 hammer energy correction). Additional correction factors for boring diameter, for rod length, and the presence/absence of an SPT sampler liner were also applied (Reference 2.5S.4-5). A summary of the measured ETR values and the resulting hammer energy corrections for each drilling rig employed is presented in Table 2.5S.4-4. Partially-corrected SPT N_1-values from each boring were then fully corrected using the appropriate hammer energy corrections, and using the additional correction factors in accordance with Reference 2.5S.4-5. The resulting fully-corrected SPT N-values area commonly termed $(N_1)_{60}$. A summary of corrected SPT $(N_1)_{60}$-values for all site areas and all soil strata is presented in Table 2.5S.4-5.

The average corrected SPT $(N_1)_{60}$-value for Stratum A was 17 blows/foot. An SPT $(N_1)_{60}$-value of 15 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values Stratum A is considered stiff to very stiff.

CPTs were additionally performed in Stratum A soils. Site-wide the CPT tip resistance, q_t, in this stratum ranged from 2 tsf to 212 tsf, with an average of 19 tsf. Also, site-wide the average normalized CPT tip resistance, q_{c1n} (normalized to an effective overburden pressure of 1 tsf) for Stratum A, was 29 (dimensionless). Note that CPT tip resistance profiles versus elevation

are shown on Figure 2.5S.4-16, Figure 2.5S.4-17, Figure 2.5S.4-18, and 2.5S.4-19 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Stratum A. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Stratum A, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture content (%)	57	16	29	23
Liquid Limit (%)	47	30	80	57
Plasticity Index (%)	47	11	58	37
Fines Content (%)	17	87	100	94
Unit Weight (pcf)	12	119	133	124

Test results are summarized in Table 2.5S.4-8. Natural moisture contents and Atterberg limits are presented versus elevation on Figure 2.5S.4-20. Atterberg limits are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes Stratum A soils were characterized, on average, as highly plasticity clay with an average fines content (materials passing the No. 200 sieve) of 94%. The Unified Soil Classification System (USCS) (References 2.5S.4-23 and 2.5S.4-31) designations for Stratum A were mainly fat clay, lean clay, and occasionally lean clay with gravel (visual classification), with the predominant USCS group symbols of CH and CL. Based on laboratory testing, an average unit weight of 124 pounds per cubic foot (pcf) was selected for Stratum A.

The undrained shear strength of Stratum A was evaluated based on laboratory testing and using correlations with corrected SPT $(N_1)_{60}$-values and CPT results. The results of this evaluation are summarized in Table 2.5S.4-9A and B.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values (Reference 2.5S.4-7), using:

$s_u = N/8$ (in kips per square foot [ksf]) Equation 2.5S.4-2

where, N = corrected SPT $(N_1)_{60}$-value in blows/foot.

Substituting the selected corrected SPT $(N_1)_{60}$-value for Stratum A (15 blows/foot), an s_u=1.9 ksf was estimated. Undrained shear strength was also estimated using the CPT data, following a CPT-s_u correlation from Reference 2.5S.4-8, as follows:

$s_u = (q_t - \sigma_v)/N_{kt}$ Equation 2.5S.4-3

where, q_t = the CPT tip resistance
σ_v = the total overburden pressure at the depth of the CPT test interval
N_{kt} = a cone factor which varies between 10 and 20

A site-specific cone factor of N_{kt}=19 was determined by comparing the results of laboratory undrained shear strength test results on soil samples collected from borings made at locations adjacent to CPTs (e.g., especially Borings B-904 and B-909 compared to CPTs C-901, C-902, C-903, and C-904).

Shear strength values calculated in this way from the CPT data indicated an average s_u=1.5 ksf. The CPT-derived values are shown versus elevation on Figure 2.5S.4-23, 2.5S.4-24, 2.5S.4-25, 2.5S.4-26, and 2.5S.4-27 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, the area outside the Power Block, and site-wide, respectively. Note that SPT correlations were based on 1099 field measurements, while CPT correlations were based on 862 field measurements made within Stratum A. The results of 11 laboratory unconsolidated undrained (UU) triaxial strength tests and unconfined compression (UNC) strength tests on selected samples indicated an average s_u=1.3 ksf. Laboratory shear strength test results are summarized in Table 2.5S.4-9A and plotted versus elevation on Figure 2.5S.4-22. UU strength results from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated s_u=0.9 ksf for the upper portion of Stratum A (i.e., Stratum A_1) and s_u=2.3 ksf for the lower portion of Stratum A (i.e., Stratum A_2), and were comparable to the results of this subsurface investigation. Based on the results of this subsurface investigation, an undrained shear strength of s_u=1.6 ksf was selected for Stratum A, averaged from the SPT $(N_1)_{60}$-value correlations, the CPT correlations, and the laboratory testing results.

Laboratory testing to determine the drained angle of shearing resistance of Stratum A was not performed. For engineering purposes, the drained friction angle (ϕ') for Stratum A, was selected at the same value as the ϕ' for Stratum D, or ϕ'=20 degrees. Note that Strata A, D, F, and J Clay (discussed in following subsections), all had similar plasticity. Laboratory soil strength test results, including drained friction angle, are summarized in Table 2.5S.4-10.

Consolidation properties and the stress history of Stratum A soils were assessed via laboratory testing and via an evaluation of the CPT results. A summary and the results of laboratory consolidation tests made on selected samples are presented in Tables 2.5S.4-11 and 2.5S.4-12, respectively. These results are also plotted versus elevation and shown on Figure 2.5S.4-28. Results of five consolidation tests made on selected samples indicated that, on average, Stratum A was preconsolidated to approximately 7 ksf, with an overconsolidation ratio (OCR)=8. Consolidation test results for Stratum A from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated that, on average, Stratum A was preconsolidated to approximately 10 ksf, with an OCR=14. CPT-derived OCR data for Stratum A indicated an average OCR=10, and were based on 855 field measurements. CPT-derived OCR data are shown on Figures 2.5S.4-29 through 2.5S.4-33 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, the area outside the Power Block, and site-wide, respectively. A summary of OCR values derived from the CPT results is shown in Table 2.5S.4-13. Overall, an OCR=7 and a preconsolidation pressure of 6.3 ksf were selected for Stratum A.

The elastic modulus (E) for fine-grained soils was evaluated using the following relationship (Reference 2.5S.4-9):

$E = 600\ s_u$ Equation 2.5S.4-4

where, s_u = undrained shear strength.

Substituting the previously established s_u for Stratum A soils (s_u=1.6 ksf), an E=960 ksf was estimated.

Other relationships for E (linked to large strain shear modulus (G) and to PI) for fine-grained soils (Reference 2.5S.4-10) were as follows:

$E = 2\ G\ (1 + \mu)$ Equation 2.5S.4-5

$G_{0.0001\%} = \gamma / g\ (V_s)^2$ Equation 2.5S.4-6

$G_{0.0001\%} / G_{.375\%} = 21/PI$ Equation 2.5S.4-7

where,
E = static (or large strain) elastic modulus
μ = Poisson's ratio
γ = total unit weight of soil
g = acceleration of gravity = 32.2 feet/second/second
V_s = shear wave velocity
$G_{.0001\%}$ = small strain shear modulus (i.e., strain in the range of 10^{-4} %);
$G_{.375\%}$ = large strain (static) shear modulus (i.e., strain in the range of 0.25% to 0.50%)
PI = plasticity index

Using the V_s=575 feet/second for Stratum A obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.45 for clay, γ=124 pcf for Stratum A, and PI=40 for Stratum A, an E=1,112 ksf was estimated. Using an average of the E-values estimated from undrained shear strength and from shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=1,050 ksf was selected for Stratum A. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) for soils was related to E by the following relationship, (i.e., reordering Equation 2.5S.4-5):

$G = E/(2\ [1 + \mu])$ Equation 2.5S.4-8

where, E = static (or large strain) elastic modulus
μ = Poisson's ratio

Using μ=0.45 for clay, a G=331 ksf was estimated based on the s_u-derived E, while a G=384 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=360 ksf was selected for Stratum A. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

Note, as above, that for all soil strata, E and G values selected for use were derived from a 2:1 weighted average of the shear wave velocity-derived values and either the s_u-derived values or

the SPT $(N_1)_{60}$-derived values. The shear wave velocity-derived values were based on more continuous downhole measurements and were thus considered more reliable.

The coefficient of subgrade reaction for 1 foot wide or 1 foot square footings, k_1, was obtained from Reference 2.5S.4-11. Based on the material characterization of Stratum A, k_1=150 kips per cubic feet (kcf) was selected for use.

Active, passive, and at-rest static earth pressure coefficients, K_a, K_p, and K_0, were estimated assuming frictionless vertical walls and horizontal backfill using Rankine's theory and based on the following relationships (Reference 2.5S.4-12):

$K_a = \tan^2 (45 - \phi'/2)$ Equation 2.5S.4-9

$K_p = \tan^2 (45 + \phi'/2)$ Equation 2.5S.4-10

$K_0 = 1 - \sin (\phi')$ Equation 2.5S.4-11

where, ϕ' = drained friction angle of the soil.

Using a drained friction angle, ϕ'=20 degrees, for Stratum A, the following earth pressure coefficients were calculated: K_a=0.49; K_p=2.04; and, K_0=0.66. Values selected for engineering purposes were then: K_a=0.5; K_p=2.0; and, K_0=0.7.

Determination of the sliding coefficient, tangent δ, where δ (generally 2/3 ϕ') is the friction angle between the soil and the foundation material bearing against it, in this case concrete, is an important factor for soils that support foundations. Based on Reference 2.5S.4-13, tangent δ=0.3 was selected for Stratum A. Note, however, that Stratum A is removed from under all STP 3 area, STP 4 area, and UHS Basin/RSW area major structure footprints (including Seismic Category I structures).

All of the material parameters selected for engineering purposes for Stratum A are summarized in Table 2.5S.4-16.

2.5S.4.2.1.2 Stratum B

Stratum B soils were encountered below Stratum A in a majority of the borings and CPTs made site-wide. Stratum B was not encountered in Borings B-307, B-312, B-313, B-412, B-427, B-433, B-434, B-908, B-928, and B-929, or in CPT C-901. Boring B-920 was additionally terminated in this stratum. Stratum B typically consisted of yellowish red, reddish brown, and brown silt, silty sand, or clay. As described below, the majority of the samples exhibited non-plastic behavior, and thus Stratum B was considered to behave as a granular soil (or more accurately, a fine-grained non-cohesive soil).

The thickness of Stratum B was estimated from the borings and CPTs. Inside the Power Block area, the thickness of Stratum B varied from 0.5 feet to 16 feet, with an average thickness of 7 feet, and the base elevation varied from El. -9 feet to El. 14 feet, with an average of El. 5 feet. Additional information on the thicknesses and base elevations of this stratum, including areas outside the Power Block, is presented in Table 2.5S.4-2. Note that only data from borings and CPTs that encountered and fully penetrated the stratum were considered in evaluating the stratum thickness and in selecting the stratum base elevation.

Soil samples were collected from the borings via SPT sampling and undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling and were recorded on the boring logs. In the STP 3 area, uncorrected SPT N-values in Stratum B ranged from 2 blows/foot to 23 blows/foot, with an average uncorrected SPT N-value of 7 blows/foot. In the STP 4 area, uncorrected SPT N-values in Stratum B ranged from 3 blows/foot to 40 blows/foot, with an average uncorrected SPT N-value of 12 blows/foot. In the UHS Basin/RSW area, uncorrected SPT N-values in Stratum B ranged from 2 blows/foot to 17 blows/foot, with an average uncorrected SPT N-value of 9 blows/foot. Additional SPT N-value information on this stratum at locations other than the STP 3 area, the STP 4 area, and the UHS Basin/RSW area is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11 through 2.5S.4-15 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively. The site-wide average uncorrected SPT N-value was 9 blows/foot for Stratum B.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum B was 14 blows/foot. An SPT $(N_1)_{60}$-value of 10 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum B is considered loose to medium dense.

CPTs were additionally performed in Stratum B soils. Site-wide, the CPT tip resistance, q_t, in this stratum ranged from 11 tsf to 204 tsf, with an average of 53 tsf. Also, site-wide the average normalized CPT tip resistance, q_{c1n} (normalized to an effective overburden pressure of 1 tsf) for Stratum B was 61 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figures 2.5S.4-16 through 2.5S.4-19 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Stratum B. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Stratum B, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture content (%)	30	18	28	24
Liquid Limit (%)	17	Non-Plastic	70	38
Plasticity Index (%)	17	Non-Plastic	45	19
Fines Content (%)	17	36	94	71
Unit Weight (pcf)	6	117	128	121

Test results are summarized in Table 2.5S.4-8. Note that 9 of the 17 Atterberg limits tests performed on Stratum B soils yielded non-plastic results. As such, the average values for Liquid Limit and Plasticity Index (PI), above, include only those tests made on plastic (PI>0) soils. Natural moisture contents and Atterberg limits are presented versus elevation on Figure 2.5S.4-20. Atterberg limits are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Stratum B soils were characterized, on average, as non-plastic silt or silty sand, to medium plasticity clay with an average fines content (materials passing the No. 200 sieve) of 71%. The USCS designations for Stratum B were mainly silt, silt with sand, sandy silt, silty sand, lean clay, lean clay with sand, clayey sand, and fat clay, with the predominant USCS group symbols of ML and SM. Based on laboratory testing, an average unit weight of 121 pcf was selected for Stratum B.

The strength of Stratum B was evaluated based on laboratory testing and using correlations with corrected SPT $(N_1)_{60}$-values and CPT results. The results of the laboratory testing are summarized in Table 2.5S.4-10.

The drained friction angle, ϕ', was estimated from empirical correlations with corrected SPT N-values, according to Reference 2.5S.4-14. Using the selected corrected SPT $(N_1)_{60}$-value for Stratum B (10 blows/foot), a value of ϕ'=30 degrees (for fine sand) was estimated. A value of ϕ'=28 degrees was considered appropriate.

The drained friction angle, ϕ', was also estimated using the CPT data, following a CPT-ϕ' correlation from Reference 2.5S.4-15, as follows:

$$\phi' = \text{arctangent}\,(\log\,[q_t/\sigma_v'] + 0.29)/2.68 \qquad \text{Equation 2.5S.4-12}$$

where, q_t = the CPT tip resistance;
σ_v' = the effective overburden pressure at the depth of the CPT test interval.

Drained friction angle values calculated from the CPT data indicated an average ϕ'=39 degrees. Note that SPT correlations were based on 198 field measurements, while CPT correlations were

based on 298 field measurements made within Stratum B. The results of two laboratory isotropically-consolidated undrained triaxial strength tests with pore water pressures measured (CIU-bar) made on selected samples indicated an average ϕ'=30 degrees. Laboratory CIU-bar test results are summarized in Table 2.5S.4-10. The CPT-derived values are shown versus elevation on Figures 2.5S.4-34 through 2.5S.4-38 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, the area outside the Power Block, and site-wide, respectively.

From the above, a summary of average ϕ' values for Stratum B is provided as follows:

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	28	39	30

Based on the above a ϕ'=30 degrees was selected for Stratum B.

Consolidation properties of the cohesionless fine-grained Stratum B were not evaluated/relevant.

The elastic modulus, E, for coarse-grained soils was evaluated using the following relationship (Reference 2.5S.4-9).

E = 36 N (in ksf) Equation 2.5S.4-13

where, N = average corrected SPT $(N_1)_{60}$-value in blows/foot.

Substituting the previously established average corrected SPT $(N_1)_{60}$-value for Stratum B soils (10 blows per foot), an E=360 ksf was estimated.

Other relationships for E for coarse-grained soils (Reference 2.5S.4-10), especially employing shear wave velocity, were according to Equations 2.5S.4-5 and 2.5S.4-6 and the following:

$G_{.0001\%}/G_{.375\%} = 10$ (for sands) Equation 2.5S.4-14

where, $G_{.0001\%}$ = small strain shear modulus (i.e., strain in the range of 10^{-4} %)
$G_{.375\%}$ = large strain (static) shear modulus (i.e., strain in the range of 0.25% to 0.50%)

Using the V_s=725 feet/second for Stratum B obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.30 for sand and γ=121 pcf for Stratum B, an E=515 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT $(N_1)_{60}$-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=460 ksf was selected for Stratum B. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=139 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=212 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and

2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=185 ksf was selected for Stratum B. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction for 1 foot wide or 1 foot square footings, k_1, was obtained from Reference 2.5S.4-11. Based on material characterization for Stratum B soils, k_1=160 kcf was selected for use.

Active, passive, and at-rest static earth pressure coefficients, K_a, K_p, and K_0, were estimated using Equations 2.5S.4-9, 2.5S.4-10, and 2.5S.4-11, respectively. Using the selected ϕ'=30 degrees, the following earth pressures coefficients are estimated for Stratum B; K_a=0.3, K_p=3.0, and K_0=0.5.

Based on Reference 2.5S.4-13, and the selected ϕ'=30 degrees for Stratum B, a sliding coefficient, tangent δ=0.35, was selected for Stratum B. Note, however, that Stratum B is removed from under all STP 3 area, STP 4 area, and UHS Basin/RSW area major structure footprints (including Seismic Category I structures).

All of the material parameters selected for engineering purposes for Stratum B are summarized in Table 2.5S.4-16.

2.5S.4.2.1.3 Stratum C

Stratum C soils were encountered below Stratum B in a majority of the borings and CPTs made site-wide. Boring B-911, and CPTs C-302, C-404, and C-916 were terminated in this stratum. Stratum C typically consisted of yellowish brown to dark brown sand with varying amounts of silt and/or clay.

The thickness of Stratum C was estimated from the borings and CPTs. Inside the Power Block area, the thickness of Stratum C varied from 3 feet to 30 feet, with an average thickness of approximately 20 feet, and the base elevation varied from El. -24 feet to El. -6 feet, with an average of El. -15 feet. Additional information on the thicknesses and base elevations of this stratum, including areas outside the Power Block, is presented in Table 2.5S.4-2. Note that only data from borings and CPTs that encountered and fully penetrated the stratum were considered in evaluating the stratum thickness and in selecting the stratum base elevation.

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling and were recorded on the boring logs. In the STP 3 area, uncorrected SPT N-values in Stratum C ranged from 0 blows/foot to 109 blows/foot, with an average uncorrected SPT N-value of 27 blows/foot. In the STP 4 area, uncorrected SPT N-values in Stratum C ranged from 3 blows/foot to 120 blows/foot, with an average uncorrected SPT N-value of 23 blows/foot. In the UHS Basin/RSW area, uncorrected SPT N-values in Stratum C ranged from 7 blows/foot to 67 blows/foot, with an average uncorrected SPT N-value of 24 blows/foot. Additional SPT N-value information on this stratum at locations other than the STP 3 area, the STP 4 area, and the UHS Basin/RSW area is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11 through 2.5S.4-15 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively. The site-wide average uncorrected SPT N-value was 25 blows/foot for Stratum C.

The uncorrected SPT N-value, 0 blows/foot, occurred at one sample interval within Stratum C, namely at Boring B-305DH/DHA from depth 28.5 feet to 30 feet below ground surface. The loose soils sampled at this location, at the center of the planned STP 3 Reactor Building, are removed during construction.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1) by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum C was 35 blows/foot, which was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum C is considered dense.

CPTs were additionally performed in Stratum C soils. Site-wide, the CPT tip resistance, q_t, in this stratum ranged from 12 tsf to 602 tsf, with an average of 166 tsf. Also, site-wide the average normalized CPT tip resistance, q_{c1n} (normalized to an effective overburden pressure of 1 tsf) for Stratum C was 156 (dimensionless). Note that CPT tip resistance profiles versus

elevation are shown on Figures 2.5S.4-16 through 2.5S.4-19 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Stratum C. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Stratum C, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	38	19	27	24
Liquid Limit (%)	2	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	2	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	36	6	96	25
Unit Weight (pcf)	4	120	124	122

Test results are summarized in Table 2.5S.4-8. Note that natural moisture contents and Atterberg limits for other soil strata are presented versus elevation on Figure 2.5S.4-20. Note also that Atterberg limits for other soil strata are shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Stratum C soils were characterized, on average, as silty sand with an average fines content (materials passing the No. 200 sieve) of 25%. Note that the maximum 96% fines content reported occurred at Boring B-405DH from depths of 43.5 feet to 45 feet. This result represents an isolated thin clay lens within the Stratum C sand. Two other fines content tests reported indicate fine-grained soils, including a fines content of 82% at Boring B-912 from depths of 43.5 feet to 45 feet, and a fines content of 53% at Boring B-914 from depths of 33.5 feet to 35 feet. These results represent isolated silt lenses within the Stratum C sand. The next highest fines content reported was 46%. The USCS designations for Stratum C were mainly silty sand, poorly graded sand with silt, silt with sand, sandy silt, and occasionally lean clay, with the predominant USCS group symbols of SM and SP-SM. Based on laboratory testing, an average unit weight of 122 pcf was selected for Stratum C.

The strength of Stratum C was evaluated based on laboratory testing, and using correlations with corrected SPT $(N_1)_{60}$-values and CPT results. The results of the laboratory testing are summarized in Table 2.5S.4-10.

The drained friction angle, ϕ', was estimated from empirical correlations with corrected SPT N-values, according to Reference 2.5S.4-14. Using the selected corrected SPT $(N_1)_{60}$-value for Stratum C (35 blows/foot), a value of ϕ'=of 38 degrees (for fine sand) was estimated. A value of ϕ'=36 degrees was considered appropriate. The drained friction angle, ϕ', was also estimated using the CPT data, following a CPT-ϕ' correlation (Reference 2.5S.4-15) given as Equation 2.5S.4-12. Drained friction angle values calculated from the CPT data indicated an average ϕ'=42 degrees. Note that SPT correlations were based on 444 field measurements, while CPT correlations were based on 1355 field measurements made within Stratum C. Results of three

laboratory direct shear tests made on selected samples indicated an average ϕ'=33 degrees. Laboratory direct shear test results are summarized in Table 2.5S.4-10. The CPT-derived values are shown versus elevation on Figures 2.5S.4-34 through 2.5S.4-38 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, the area outside the Power Block, and site-wide, respectively.

From the above, a summary of average ϕ' values for Stratum C is provided as follows:

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	36	42	33

Based on the above a ϕ'=35 degrees was selected for Stratum C.

Consolidation properties of the granular Stratum C were not evaluated/relevant.

The elastic modulus, E, for coarse-grained soils was evaluated using Equation 2.5S.4-13. Substituting the previously established average corrected SPT $(N_1)_{60}$-value for Stratum C soils (35 blows per foot) an E=1,260 ksf was estimated. Other relationships for E were available for coarse-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. Using the V_s=785 feet/second for Stratum C obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion) and using μ=0.30 for sand and γ=122 pcf for Stratum C an E=606 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT $(N_1)_{60}$-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=850 ksf was selected for Stratum C. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=485 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=233 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=320 ksf was selected for Stratum C. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction for 1 foot wide or 1 foot square footings, k_1, was obtained from Reference 2.5S.4-11. Based on material characterization for Stratum C soils, k_1=600 kcf was selected for use.

Active, passive, and at-rest static earth pressure coefficients, K_a, K_p, and K_0, were estimated using Equations 2.5S.4-9, 2.5S.4-10, and 2.5S.4-11, respectively. Using the selected ϕ'=35 degrees, the following earth pressures coefficients are estimated for Stratum C; K_a=0.3, K_p=3.7, and K_0=0.4.

Based on Reference 2.5S.4-13 and the selected ϕ'=35 degrees for Stratum C a sliding coefficient, tangent δ=0.4, was selected.

All of the material parameters selected for engineering purposes for Stratum C are summarized in Table 2.5S.4-16.

2.5S.4.2.1.4 Stratum D

Stratum D soils were encountered below Stratum C in a majority of the borings and CPTs made site-wide. Borings B-320, B-913, B-915, B-916, B-917, and B-927, and CPTs C-301, C-303, C-401, C-402, C-403, and C-411 were terminated in this stratum. Stratum D typically consisted of greenish gray, yellowish red, or reddish brown to dark brown clay with varying amounts of silt and/or sand, occasionally containing isolated thin lenses of silty sand.

The thickness of Stratum D was estimated from the borings and CPTs. Inside the Power Block area, the thickness of Stratum D varied from 1.5 feet to 34 feet, with an average thickness of 22 feet, and the base elevation varied from El. -45 feet to El. -18 feet, with an average of El. -37 feet. Additional information on the thicknesses and base elevations of this stratum, including areas outside the Power Block, is presented in Table 2.5S.4-2. Note that only data from borings and CPTs that encountered and fully penetrated the stratum were considered in evaluating the stratum thickness and in selecting the stratum base elevation.

Soil samples were collected from the borings via SPT sampling and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling and were recorded on the boring logs. In the STP 3 area, uncorrected SPT N-values in Stratum D ranged from 7 blows/foot to 34 blows/foot, with an average uncorrected SPT N-value of 16 blows/foot. In the STP 4 area, uncorrected SPT N-values in Stratum D ranged from 3 blows/foot to 34 blows/foot, with an average uncorrected SPT N-value of 15 blows/foot. In the UHS Basin/RSW area, uncorrected SPT N-values in Stratum D ranged from 5 blows/foot to 54 blows/foot, with an average uncorrected SPT N-value of 15 blows/foot. Additional SPT N-value information on this stratum at locations other than the STP 3 area, the STP 4 area, and the UHS Basin/RSW area is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11 through 2.5S.4-15 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively. The site-wide average uncorrected SPT N-value was 15 blows/foot for Stratum D.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1) by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum D was 17 blows/foot. An SPT $(N_1)_{60}$-value of 15 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum D is considered stiff to very stiff.

CPTs were additionally performed in Stratum D soils. Site-wide, the CPT tip resistance, q_t, in this stratum ranged from 11 tsf to 185 tsf, with an average of 41 tsf. Also, site-wide the average normalized CPT tip resistance, q_{c1n} (normalized to an effective overburden pressure of 1 tsf), for Stratum D was 26 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figures 2.5S.4-16 through 2.5S.4-19 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Stratum D. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Stratum D, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	55	16	43	25
Liquid Limit (%)	38	20	84	58
Plasticity Index (%)	38	2	59	38
Fines Content (%)	13	24	100	72
Unit Weight (pcf)	14	111	129	121

Test results are summarized in Table 2.5S.4-8. Note that four of the 38 Atterberg limits tests performed on Stratum D soils yielded non-plastic results. As such, the average values for Liquid Limit and Plasticity Index (PI), above, include only those tests made on plastic (PI>0) soils. Natural moisture contents and Atterberg limits are presented versus elevation on Figure 2.5S.4-20. Atterberg limits are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Stratum D soils were characterized, on average, as high plasticity clay with an average fines content (materials passing the No. 200 sieve) of 72%. The USCS designations for Stratum D were mainly fat clay, lean clay, sandy lean clay, silt, silt with sand, sandy silt, silty sand, and clayey sand, with the predominant USCS group symbols of CH and CL. Based on laboratory testing, an average unit weight of 121 pcf was selected for Stratum D.

The undrained shear strength of Stratum D was evaluated based on laboratory testing, and using correlations with corrected SPT $(N_1)_{60}$-values and the CPT results. The results of this evaluation are summarized in Table 2.5S.4-9A and B.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values (Reference 2.5S.4-7) using Equation 2.5S.4-2. Substituting the selected corrected SPT $(N_1)_{60}$-value for Stratum D (15 blows/foot), an s_u=1.9 ksf was estimated. Undrained shear strength was also estimated using the CPT data, following a CPT-s_u correlation (Reference 2.5S.4-13) given as Equation 2.5S.4-3. A site-specific cone factor of N_{kt}=19 was determined for the site soils, as noted above. Shear strength values calculated from the CPT data indicated an average s_u=3.1 ksf. The CPT-derived values are shown versus elevation on Figures 2.5S.4-23 through 2.5S.4-27 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, the area outside the Power Block, and site-wide, respectively. Note that SPT correlations were based on 449 field measurements, while CPT correlations were based on 721 field measurements made within Stratum D. Results of eight laboratory UU and UNC strength tests made on selected samples indicated an average s_u=1.7 ksf. By excluding the two lowest laboratory strength test results of s_u=0.3 ksf and 0.4 ksf (likely made on samples of poor or non-representative quality), an average s_u=2.1 ksf resulted. Laboratory shear strength test results are summarized in Table 2.5S.4-9A and plotted versus elevation on Figure 2.5S.4-22. UU

strength results from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated an average s_u=4.3 ksf for Stratum D (19 test results). Based on this, it was deemed that the CPT-derived s_u results from this subsurface investigation were more representative and an undrained shear strength of s_u=3 ksf was selected for Stratum D.

The drained friction angle of Stratum D soils was evaluated from laboratory test results. The results are shown in Table 2.5S.4-10 and summarized below. Strength parameters from two CIU-bar tests, indicated average (drained/effective) ϕ'=16 degrees, and c'=1.3 ksf, and average (undrained/total) ϕ=4 degrees and c=1.8 ksf, as noted:

Parameter	From CIU-Bar
ϕ' (degrees)	16
c' (tsf)	1.3
ϕ (degrees)	4
c (tsf)	1.8

Based on the above, a ϕ'=20 degrees was selected for Stratum D soils, and for similar fine-grained soil strata (i.e., Strata A, F, and J Clay).

Consolidation properties and the stress history of Stratum D soils were assessed via laboratory testing and via an evaluation of the CPT results. A summary, and the results of, laboratory consolidation tests made on selected samples are presented in Tables 2.5S.4-11 and 2.5S.4-12, respectively. These results are also plotted versus elevation and shown on Figure 2.5S.4-28. The results of five consolidation tests made on selected samples indicated that, on average, Stratum D was preconsolidated to approximately 12.5 ksf, with an OCR=3.5. Consolidation test results for Stratum D from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated that, on average, Stratum D was preconsolidated to approximately 18 ksf, with an OCR=6. CPT-derived OCR data for Stratum D indicated an average OCR=3, and were based on 720 field measurements. CPT-derived OCR data are shown on Figure 2.5S.4-29, Figure 2.5S.4-30, Figure 2.5S.4-31, Figure 2.5S.4-32, and Figure 2.5S.4-33 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, the area outside the Power Block, and site-wide, respectively. A summary of OCR values derived from the CPT results is shown in Table 2.5S.4-13. Overall, an OCR=3.3 and a preconsolidation pressure of 12.3 ksf were selected for Stratum D.

The elastic modulus (E) for Stratum D was evaluated using Equation 2.5S.4-4. Substituting the previously established s_u for Stratum D soils (s_u=3 ksf), an E=1800 ksf was estimated. Other relationships for E (linked to G and to PI) were also available for fine-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. Using the V_s=925 feet/second for Stratum D obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.45 for clay, γ=121 pcf for Stratum D, and PI=40 for Stratum D, an E=2,807 ksf was estimated. Using an average of the E-values estimated from the undrained shear strength and from the shear wave velocity, with the shear wave velocity-derived value

weighted 2:1, an E=2500 ksf was selected for Stratum D. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.45 for clay, a G=621 ksf was estimated based on the s_u-derived E, while a G=968 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=850 ksf was selected for Stratum D. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction for 1 foot wide or 1 foot square footings, k_1, was obtained from Reference 2.5S.4-11. Based on material characterization for Stratum D soils, k_1=300 kcf was selected for use.

Active, passive, and at-rest static earth pressure coefficients, K_a, K_p, and K_0, were estimated using Equations 2.5S.4-9, 2.5S.4-10, and 2.5S.4-11, respectively. Using the selected ϕ'=20 degrees, the following earth pressures coefficients are estimated for Stratum D; K_a=0.5, K_p=2, and K_0=0.7.

Based on Reference 2.5S.4-13, and the selected ϕ'=20 degrees for Stratum D, a sliding coefficient, tangent δ=0.3 was selected for Stratum D.

All of the material parameters selected for engineering purposes for Stratum D are summarized in Table 2.5S.4-16.

2.5S.4.2.1.5 Stratum E

Stratum E soils were encountered below Stratum D in a majority of the borings and CPTs made site-wide. Stratum E was largely absent in the vicinity of the UHS Basin, west of STP 4. Stratum E was not encountered in Borings B-420, B-901 through B-913, B-928, B-930, B-931, and B-933, and CPTs C-901 through C-904. Multiple borings and CPTs made site-wide were additionally terminated in this stratum. Stratum E typically consisted of gray or yellowish brown to dark brown sand with varying amounts of silt and/or clay.

The thickness of Stratum E was estimated from the borings and CPTs. Inside the Power Block area, the thickness of Stratum E varied from 1.5 feet to 36.5 feet, with an average thickness of 18 feet, and the base elevation varied from El. -72 feet to El. -37 feet, with an average of El. -55 feet. Additional information on the thicknesses and base elevations of this stratum, including areas outside the Power Block, is presented in Table 2.5S.4-2. Note that only data from borings and CPTs that encountered and fully penetrated the stratum were considered in evaluating the stratum thickness and in selecting the stratum base elevation.

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling, and were recorded on the boring logs. In the STP 3 area, uncorrected SPT N-values in Stratum E ranged from 7 blows/foot to 88 blows/foot, with an average uncorrected SPT N-value of 34 blows/foot. In the STP 4 area, uncorrected SPT N-values in Stratum E ranged from 11 blows/foot to 84 blows/foot, with an average uncorrected SPT N-value of 39 blows/foot. As noted above, Stratum E was largely absent in the UHS Basin/RSW area, with only one

uncorrected SPT N-value, 51 blows/foot, measured in that area. Additional SPT N-value information on this stratum at locations other than the STP 3 area, the STP 4 area, and the UHS Basin/RSW areas is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 through 2.5S.4-12 and 2.5S.4-13 through 2.5S.4-15 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively. The site-wide average uncorrected SPT N-value was 35 blows/foot for Stratum D.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum E was 34 blows/foot. An SPT $(N_1)_{60}$-value of 30 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum E is considered dense.

CPTs were additionally performed in Stratum E soils. Site-wide, the CPT tip resistance, q_t, in this stratum ranged from 20 tsf to 558 tsf, with an average of 228 tsf. Also, site-wide the average normalized CPT tip resistance, q_{c1n} (normalized to an effective overburden pressure of 1 tsf) for Stratum E was 144 (dimensionless). As noted above, Stratum E was largely absent in the UHS Basin/RSW area, with no CPT's encountering the stratum in that particular area. Note that CPT tip resistance profiles versus elevation are shown on Figures 2.5S.4-16 and 2.5S.4-17, for the STP 3 area and the STP 4 area, respectively.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Stratum E. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Stratum E, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	38	17	26	21
Liquid Limit (%)	6	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	6	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	35	3	96	18
Unit Weight (pcf)	8	113	127	122

Test results are summarized in Table 2.5S.4-8. Natural moisture contents and Atterberg limits are presented versus elevation on Figure 2.5S.4-20. Atterberg limits are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Stratum E soils were characterized, on average, as silty sand with an average fines content (materials passing the No. 200 sieve) of 18%. Note that the maximum 96% fines content reported occurred at Boring B-

343 from depths of 70 feet to 72 feet. This result represents an isolated thin clay lens within the Stratum E sand. The next highest fines content reported was 50%. The USCS designations for Stratum E were mainly poorly graded sand with silt, silty sand, poorly graded sand, clayey sand, and occasionally fat clay, with the predominant USCS group symbols of SP-SM and SM. Based on laboratory testing, an average unit weight of 122 pcf was selected for Stratum E.

The strength of Stratum E was evaluated based on laboratory testing, and using correlations with corrected SPT $(N_1)_{60}$-values and CPT results. The results of the laboratory testing are summarized in Table 2.5S.4-10.

The drained friction angle, ϕ', was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values, according to Reference 2.5S.4-14. Using the selected corrected SPT $(N_1)_{60}$-value for Stratum E (30 blows/foot), a value of ϕ'=of 39 degrees (for fine to medium sand) was estimated. A value of ϕ'=37 degrees was considered appropriate. The drained friction angle, ϕ', was also estimated using the CPT data, following a CPT-ϕ' correlation (Reference 2.5S.4-15) given as Equation 2.5S.4-12. Drained friction angle values calculated from the CPT data indicated an average ϕ'=40 degrees. Note that SPT correlations were based on 372 field measurements, while CPT correlations were based on 414 field measurements made within Stratum E. Results of two laboratory direct shear tests made on selected samples indicated an average ϕ'=33 degrees. Laboratory direct shear test results are summarized in Table 2.5S.4-10. The CPT-derived values are shown versus elevation on Figure 2.5S.4-34, 2.5S.4-35, and 2.5S.4-38 for the STP 3 area, the STP 4 area, and site-wide, respectively.

From the above, a summary of average ϕ' values for Stratum E is provided as follows:

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	37	40	33

Based on the above a ϕ'=35 degrees was selected for Stratum E.

Consolidation properties of the granular Stratum E were not evaluated/relevant.

The elastic modulus, E, for coarse-grained soils was evaluated using Equation 2.5S.4-13. Substituting the previously established average corrected SPT $(N_1)_{60}$-value for Stratum E soils (30 blows per foot), an E=1,080 ksf was estimated. Other relationships for E were available for coarse-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. Using the V_s=1,080 feet/second for Stratum E obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.30 for sand and γ=122 pcf for Stratum E, an E=1,149 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT $(N_1)_{60}$-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=1,100 ksf was selected for Stratum E. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=415 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=442 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and

2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=425 ksf was selected for Stratum E. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction for 1 foot wide or 1 foot square footings, k_1, was obtained from Reference 2.5S.4-11. Based on material characterization for Stratum E soils, k_1=600 kcf was selected for use.

Active, passive, and at-rest static earth pressure coefficients, K_a, K_p, and K_0, were estimated using Equations 2.5S.4-9, 2.5S.4-10, and 2.5S.4-11, respectively. Using the selected ϕ'=35 degrees, the following earth pressures coefficients are estimated for Stratum E; K_a=0.3, K_p=3.7, and K_0=0.4.

Based on Reference 2.5S.4-13, and the selected ϕ'=35 degrees for Stratum E, a sliding coefficient, tangent δ=0.4 was selected for Stratum E.

All of the material parameters selected for engineering purposes for Stratum E are summarized in Table 2.5S.4-16.

2.5S.4.2.1.6 Stratum F

Stratum F soils were encountered below Stratum E in a majority of the borings and CPTs made site-wide and below Stratum D in the majority of UHS Basin vicinity CPTs and borings. Stratum F was not encountered in Borings B-308DH, B-309, B-310, B-316, B-321, B-326, B-332, B-350, and B-430. Multiple borings and CPTs made site-wide were additionally terminated in this stratum. Stratum F typically consisted of reddish brown to dark grayish brown or greenish gray clay with varying amounts of silt and/or sand.

The thickness of Stratum F was estimated from the borings and CPTs. Inside the Power Block area, the thickness of Stratum F varied from one foot to 55 feet, with an average thickness of 16 feet, and the base elevation varied from El. -93 feet to El. -48 feet, with an average of El. -68 feet. Additional information on the thicknesses and base elevations of this stratum, including areas outside the Power Block, is presented in Table 2.5S.4-2. Note that only data from borings and CPTs that encountered and fully penetrated the stratum were considered in evaluating the stratum thickness and in selecting the stratum base elevation.

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling, and were recorded on the boring logs. In the STP 3 area, uncorrected SPT N-values in Stratum F ranged from 11 blows/foot to 98 blows/foot, with an average uncorrected SPT N-value of 23 blows/foot. In the STP 4 area, uncorrected SPT N-values in Stratum F ranged from 11 blows/foot to 63 blows/foot, with an average uncorrected SPT N-value of 22 blows/foot. In the UHS Basin/RSW area, uncorrected SPT N-values in Stratum F ranged from 12 blows/foot to 32 blows/foot, with an average uncorrected SPT N-value of 19 blows/foot. Additional SPT N-value information on this stratum at locations other than the STP 3 area, the STP 4 area, and the UHS Basin/RSW area is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 through 2.5S.4-12 and 2.5S.4-13 through 2.5S.4-15 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside

the Power Block, respectively. The site-wide average uncorrected SPT N-value was 22 blows/foot for Stratum F.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum F was 19 blows/foot. An SPT $(N_1)_{60}$-value of 15 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum F is considered stiff to very stiff.

CPTs were additionally performed in Stratum F soils. Site-wide, the CPT tip resistance, q_t, in this stratum ranged from 24 tsf to 118 tsf, with an average of 43 tsf. Also, site-wide the average normalized CPT tip resistance, q_{c1n} (normalized to an effective overburden pressure of 1 tsf) for Stratum F was 17 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figures 2.5S.4-16 through 2.5S.4-18, for the STP 3 area, the STP 4 area, and the UHS Basin/RSW area, respectively.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Stratum F. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Stratum F, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	46	18	29	24
Liquid Limit (%)	34	27	74	58
Plasticity Index (%)	34	6	53	38
Fines Content (%)	10	13	99	89
Unit Weight (pcf)	13	120	129	125

Test results are summarized in Table 2.5S.4-8. Natural moisture contents and Atterberg limits are presented versus elevation on Figure 2.5S.4-20. Atterberg limits are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Stratum F soils were characterized, on average, as highly plasticity clay with an average fines content (materials passing the No. 200 sieve) of 89%. Note that the minimum 13% fines content reported occurred at Boring B-328 from depths of 98.5 feet to 100 feet. This result represents an isolated thin sand lens within the Stratum F clay. All other fines contents reported were greater than 90%. The USCS designations for Stratum F were mainly fat clay, lean clay, silty clay, and occasionally silty sand, with the predominant USCS group symbols of CH and CL. Based on laboratory testing, an average unit weight of 125 pcf was selected for Stratum F.

The undrained shear strength of Stratum F was evaluated based on laboratory testing, and using correlations with corrected SPT $(N_1)_{60}$-values and the CPT results. The results of this evaluation are summarized in Table 2.5S.4-9A and B.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT $(N_1)_{60}$-value for Stratum F (15 blows/foot), an s_u=1.9 ksf was estimated. Undrained shear strength was also estimated using the CPT data, following a CPT-s_u correlation (Reference 2.5S.4-13) given as Equation 2.5S.4-3. A site-specific cone factor N_{kt}=19 was determined for the site soils, as noted above. Shear strength values calculated from the CPT data indicated an average s_u=3.6 ksf. The CPT-derived values are shown versus elevation on Figures 2.5S.4-23 through 2.5S.4-27 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and site-wide, respectively. Note that SPT correlations were based on 291 field measurements, while CPT correlations were based on 376 field measurements made within Stratum F. The results of 10 laboratory UU and UNC strength tests made on selected samples indicated an average s_u=2.7 ksf. By excluding the lowest laboratory strength test result of s_u=0.7 ksf (likely made on a sample of poor or non-representative quality), an average s_u=2.9 ksf resulted. Laboratory shear strength test results are summarized in Table 2.5S.4-9A and plotted versus elevation on Figure 2.5S.4-22. UU strength results from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated an average s_u=4.8 ksf for Stratum F (23 test results). Based on this, it was deemed that the CPT-derived s_u results from this subsurface investigation were more representative, and an undrained shear strength of s_u=3.2 ksf was selected for Stratum F.

The drained friction angle of Stratum F soils was evaluated from laboratory test results. The results are shown in Table 2.5S.4-10 and summarized below. Strength parameters from three CIU-bar tests, indicated average (drained/effective) ϕ'=8 degrees and c'=2 ksf, and average (undrained/total) ϕ=3 degrees and c=2.1 ksf.

Parameter	From CIU-Bar
ϕ' (degrees)	8
c' (tsf)	2.0
ϕ (degrees)	3
c (tsf)	2.1

Based on the results of CIU-bar tests made on Stratum D (having similar plasticity to Strata A, F, and J Clay, as noted above), ϕ'=20 degrees was selected for Stratum F soils.

Consolidation properties and the stress history of Stratum F soils were assessed via laboratory testing and via an evaluation of the CPT results. A summary, and the results of, laboratory consolidation tests made on selected samples are presented in Tables 2.5S.4-11 and 2.5S.4-12, respectively. These results are also plotted versus elevation and shown on Figure 2.5S.4-28. The results of three consolidation tests made on selected samples indicated that, on average, Stratum F was preconsolidated to approximately 16.5 ksf, with an OCR=2.9. Consolidation test

results for Stratum F from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated that, on average, Stratum F was preconsolidated to approximately 19 ksf, with an OCR=2.8. CPT-derived OCR data for Stratum F indicated an average OCR=2 and were based on 376 field measurements. CPT-derived OCR data are shown on Figure 2.5S.4-29, Figure 2.5S.4-30, Figure 2.5S.4-31, and Figure 2.5S.4-33 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and site-wide, respectively. A summary of OCR values derived from the CPT results is shown in Table 2.5S.4-13. Overall, an OCR=2.6 and a preconsolidation pressure of 15.5 ksf were selected for Stratum F.

The elastic modulus (E) for Stratum F was evaluated using Equation 2.5S.4-4. Substituting the previously established s_u for Stratum F soils (s_u=3.2 ksf), an E=1,920 ksf was estimated. Other relationships for E (linked to G and to PI) were also available for fine-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. Using the V_s=945 feet/second for Stratum F obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.45 for clay, =125 pcf for Stratum F, and PI=40 for Stratum F, an E=3028ksf was estimated. Using an average of the E-values estimated from the undrained shear strength and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=2600 ksf was selected for Stratum F. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.45 for clay, a G=662 ksf was estimated based on the s_u-derived E, while a G=1044 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=900 ksf was selected for Stratum F. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction for 1 foot wide or 1 foot square footings, k_1, was obtained from Reference 2.5S.4-11. Based on material characterization for Stratum F soils, k_1=300 kcf was selected for use.

Active, passive, and at-rest static earth pressure coefficients, K_a, K_p, and K_0, were estimated using Equations 2.5S.4-9, 2.5S.4-10, and 2.5S.4-11, respectively. Using the selected ϕ'=20 degrees (from Stratum D), the following earth pressures coefficients are estimated for Stratum F; K_a=0.5, K_p=2, and K_0=0.7.

Based on Reference 2.5S.4-13, and the selected ϕ'=20 degrees (from Stratum D), a sliding coefficient, tangent δ=0.3, was selected for Stratum F.

All of the material parameters selected for engineering purposes for Stratum F are summarized in Table 2.5S.4-16.

2.5S.4.2.1.7 Stratum H

Stratum H soils were encountered below Stratum F in a majority of the borings and CPTs made across the STP 3 and STP 4 areas. Stratum H was not encountered in Boring B-348 in the STP 3 area. Stratum H was only penetrated by Borings B-901 and B-910 and by CPT C-901 in the UHS Basin/RSW area. Multiple borings and CPTs made were additionally terminated in this stratum. Stratum H typically consisted of light yellowish brown to dark yellowish brown or grayish brown fine to medium sand with varying amounts of silt, clay, and/or gravel.

The thickness of Stratum H was estimated from the borings and CPTs. Inside the Power Block area, the thickness varied from 1 foot to 35.5 feet, with an average thickness of 17.5 feet, and the base elevation of Stratum H varied from El. -95 feet to El. -63 feet, with an average of El. -87 feet. Additional information on the thicknesses and base elevations of this stratum, including areas outside the Power Block, is presented in Table 2.5S.4-2. Note that only data from borings and CPTs that encountered and fully penetrated the stratum were considered in evaluating the stratum thickness and in selecting the stratum base elevation.

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling, and were recorded on the boring logs. In the STP 3 area, uncorrected SPT N-values in Stratum H ranged from 15 blows/foot to 100 blows/foot, with an average uncorrected SPT N-value of 42 blows/foot. In the STP 4 area, uncorrected SPT N-values in Stratum H ranged from 14 blows/foot to 150 blows/foot, with an average uncorrected SPT N-value of 47 blows/foot. In the UHS Basin/RSW area, uncorrected SPT N-values (only two tests conducted) in Stratum H ranged from 57 blows/foot to 74 blows/foot, with an average uncorrected SPT N-value of 66 blows/foot. Additional SPT N-value information on this stratum at locations other than the STP 3 area, the STP 4 area, and the UHS Basin/RSW areas is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11 through 2.5S.4-15 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively. The site-wide average uncorrected SPT N-value was 44 blows/foot for Stratum H.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum H was 34 blows/foot. An SPT $(N_1)_{60}$-value of 30 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum H is considered dense.

CPTs were additionally performed in Stratum H soils. Site-wide, the CPT tip resistance, q_t, in this stratum ranged from 88 tsf to 446 tsf, with an average of 180 tsf. Also site-wide, the average normalized CPT tip resistance, q_{c1n} (normalized to an effective overburden pressure of 1 tsf) for Stratum H was 104 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figure 2.5S.4-16, 2.5S.4-17, and 2.5S.4-18, for the STP 3 area, the STP 4 area, and the UHS Basin/RSW area, respectively.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Stratum H. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Stratum H, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	13	12	24	19
Liquid Limit (%)	1	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	1	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	12	6	95	16
Unit Weight (pcf)	2	122	135	128

Test results are summarized in Table 2.5S.4-8. Note that natural moisture contents and Atterberg limits for other soil strata are presented versus elevation on Figure 2.5S.4-20. Note also that Atterberg limits for other soil strata are shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Stratum H soils were characterized, on average, as silty sand with an average fines content (materials passing the No. 200 sieve) of 16%. Note that the maximum 95% fines content reported occurred at Boring B-305DH/DHA from depths of 103 feet to 105 feet. This result represents an isolated thin clay lens within the Stratum H sand. The next highest fines content reported was 13%. The USCS designations for Stratum H were mainly poorly graded sand with silt, poorly graded sand, and occasionally fat clay, with the predominant USCS group symbols of SP-SM and SM. Based on laboratory testing, an average unit weight of 128 pcf was selected for Stratum H.

The strength of Stratum H was evaluated based on laboratory testing, and using correlations with corrected SPT N-values and CPT results. The results of the laboratory testing are summarized in Table 2.5S.4-10.

The drained friction angle, ϕ', was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values, according to Reference 2.5S.4-14. Using the selected corrected SPT $(N_1)_{60}$-value for Stratum H (30 blows/foot), a value of ϕ'=of 39 degrees (for fine to medium sand) was estimated. A value of ϕ'=37 degrees was considered appropriate. The drained friction angle, ϕ', was also estimated using the CPT data, following a CPT-ϕ' correlation (Reference 2.5S.4-15) given as Equation 2.5S.4-12. Drained friction angle values calculated from the CPT data indicated an average ϕ'=38 degrees. Note that SPT correlations were based on 130 field measurements, while CPT correlations were based on 95 field measurements made within Stratum H. Results of one laboratory direct shear test made on selected samples indicated a ϕ'=29 degrees. Laboratory direct shear test results are summarized in Table 2.5S.4-10. The CPT-derived values are shown versus elevation on Figures 2.5S.4-34 through 2.5S.4-36, and 2.5S.4-38 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and site-wide, respectively.

From the above, a summary of average ϕ' values for Stratum E is provided as follows:

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	37	38	29

Based on the above a ϕ'=35 degrees was selected for Stratum H.

Consolidation properties of the granular Stratum H were not evaluated/relevant.

The elastic modulus, E, for coarse-grained soils was evaluated using Equation 2.5S.4-13. Substituting the previously established average corrected SPT $(N_1)_{60}$-value for Stratum H soils (30 blows per foot), an E=1080 ksf was estimated. Other relationships for E were available for coarse-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. Using the V_s=1075 feet/second for Stratum H obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.30 for sand, and γ=128 pcf for Stratum H, an E=1193 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT $(N_1)_{60}$-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=1150 ksf was selected for Stratum H. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=415 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=459 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=450 ksf was selected for Stratum H. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction for 1 foot wide or 1 foot square footings, k_1, was obtained from Reference 2.5S.4-11. Based on material characterization for Stratum H soils, k_1=600 kcf was selected for use.

Active, passive, and at-rest static earth pressure coefficients, K_a, K_p, and K_0, were estimated using Equations 2.5S.4-9, 2.5S.4-10, and 2.5S.4-11, respectively. Using the selected ϕ'=35 degrees, the following earth pressures coefficients are estimated for Stratum H; K_a=0.3, K_p=3.7, and K_0=0.4.

Based on Reference 2.5S.4-13, and the selected ϕ'=35 degrees, a sliding coefficient, tangent δ=0.4 was selected for Stratum H.

All of the material parameters selected for engineering purposes for Stratum H are summarized in Table 2.5S.4-16.

2.5S.4.2.1.8 Stratum J

Stratum J soils were encountered below Stratum H in all borings and CPTs made to sufficient depth. The stratum was fully penetrated in only two borings, B-305DH/DHA in the STP 3 area,

and B-405DH in the STP 4 area. Stratum J typically consisted of reddish brown to brown or greenish gray clay with interbedded sub-strata of sand and/or sandy silt. The following sub-strata were identified:

- Sub-stratum J Clay 1 (“Top” and “Bottom”)
- Sub-stratum J Sand/Silt Interbed 1 (J Interbed 1)
- Sub-stratum J Sand 1
- Sub-stratum J Clay 2 (“Top” and “Bottom”)
- Sub-stratum J Sand/Silt Interbed 2 (J Interbed 2)
- Sub-stratum J Sand 2

The thickness of Stratum J was estimated from the borings. No CPTs fully penetrated Stratum J or the other underlying strata. Overall, the stratum had an average thickness of 99 feet. Note that only data from borings and CPTs that encountered and fully penetrated the stratum were considered in evaluating the stratum thickness and in selecting the stratum base elevation.

Sub-stratum J Clay 1 was encountered in all borings made to sufficient depth. Twelve of 41 borings encountered a sand/silt interbed (Sub-stratum J Interbed 1) within Sub-stratum J Clay 1. Borings encountering Sub-stratum J Interbed 1 included B-306, B-308DH, B-314, B-321, B-327, B-328DH, B-330, B-332, B-343, B-405, B-414, and B-416. Sub-stratum J Clay 1 ranged in thickness from 4.5 feet to 15.5 feet, with an average thickness of 9.5 feet above Sub-stratum J Interbed 1. The average base elevation of Sub-stratum J Clay 1 above Sub-stratum J Interbed 1 (or Sub-stratum J Clay 1 “Top”) was El. -97 feet.

Where encountered, Sub-stratum J Interbed 1 ranged in thickness from 5.5 feet to 10 feet, with an average thickness of 9 feet. The average base elevation of Sub-stratum J Interbed 1 was El. -106 feet.

Sub-stratum J Clay 1 below Sub-stratum J Interbed 1 (or Sub-stratum J Clay 1 “Bottom”) ranged in thickness from 10 feet to 23 feet, with an average thickness of 13 feet. The thickness of the combined Sub-stratum J Clay 1 “Top” and “Bottom” ranged in thickness from 10 feet to 49 feet, with an average thickness of 29 feet. The average thickness of Sub-stratum J Clay 1 with Sub-stratum J Interbed 1 included was 31 feet. The average base elevation of Sub-stratum J Clay 1 was El. -120 feet.

Sub-stratum J Sand 1 was encountered below Sub-stratum J Clay 1, and was fully penetrated in 22 borings. Sub-stratum J Sand 1 ranged in thickness from 1.5 feet to 25.5 feet, with an average thickness of 14 feet. The average base elevation of Sub-stratum J Sand 1 was El. -131 feet. Note that Sub-stratum J Sand 1 generally divided Sub-stratum J Clay 1 and Sub-stratum J Clay 2.

Sub-stratum J Clay 2 was encountered below Sub-stratum J Sand 1 at 28 borings. Fifteen of 28 borings encountered a sand/silt interbed (Sub-stratum J Interbed 2) within Sub-stratum J Clay 2. Borings encountering Sub-stratum J Interbed 2 included B-301, B-302DH, B-303, B-304,

B-305DH/DHA, B-306, B-307, B-319DH, B-402DH, B-403, B-404, B-405DH, B-408DH, B-409, and B-428DH. Sub-stratum J Clay 2 ranged in thickness from 1 foot to 30 feet, with an average thickness of 13 feet above Sub-stratum J Interbed 2. The average base elevation of Sub-stratum J Clay 2 above Sub-stratum J Interbed 2 (or Sub-stratum J Clay 2 "Top") was El. -142 feet.

Where encountered, Sub-stratum J Interbed 2 ranged in thickness from 9 feet to 30 feet, with an average thickness of 15 feet. The average base elevation of Sub-stratum J Interbed 2 was El. -156 feet.

Sub-stratum J Clay 2 below Sub-stratum J Interbed 2 (or Sub-stratum J Clay 2 "Bottom") ranged in thickness from 11 feet to 38 feet, with an average thickness of 27 feet. The thickness of the combined Sub-stratum J Clay 2 "Top" and "Bottom" ranged in thickness from 21 feet to 48 feet, with an average thickness of 32 feet. The average thickness of Sub-stratum J Clay 2 with Sub-stratum J Interbed 2 included was 47 feet. The average base elevation of Sub-stratum J Clay 2 was El. -174 feet.

Five borings in the STP 3 area, namely B-301, B-304, B-307, B-316, and B-348, were terminated in Sub-stratum J Sand 2, encountered below Sub-stratum J Clay 2. This stratum was found neither in the STP 4 area borings, nor in the two borings in the STP 3 & 4 areas that fully penetrated Stratum J, namely B-305DH/DHA and B-405DH. This stratum was judged to be an isolated sand lens.

For discussion of engineering properties, the Stratum J sub-strata were grouped as follows:

- Sub-stratum J Clay, which contained Sub-stratum J Clay 1 and Sub-stratum J Clay 2
- Sub-stratum J Sand, which contained Sub-stratum J Interbed 1, Sub-stratum J Sand 1, Sub-stratum J Interbed 2, and Sub-stratum J Sand 2

2.5S.4.2.1.8.1 Sub-stratum J Clay

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling, and were recorded on the boring logs. In the STP 3 area, uncorrected SPT N-values in Sub-stratum J Clay ranged from 12 blows/foot to 89 blows/foot, with an average uncorrected SPT N-value of 30 blows/foot. In the STP 4 area, uncorrected SPT N-values in Sub-stratum J Clay ranged from 14 blows/foot to 120 blows/foot, with an average uncorrected SPT N-value of 32 blows/foot. In the UHS Basin/RSW area, borings did not reach Sub-stratum J Clay. Additional SPT N-value information on this stratum at locations other than the STP 3 area, the STP 4 area, and the UHS Basin/RSW areas is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11 through 2.5S.4-15 for the STP 3 area, the STP 4 area, and for the area outside the Power Block, respectively. The site-wide average uncorrected SPT N-value was 31 blows/foot for Sub-stratum J Clay.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values, for all site areas

and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Sub-stratum J Clay was 18 blows/foot. An SPT $(N_1)_{60}$-value of 15 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum J Clay is considered stiff to very stiff.

Only one CPT, C-408, made in the STP 4 area, reached Sub-stratum J Clay soils. The CPT tip resistance, q_t, in this stratum ranged from 28 tsf to 134 tsf, with an average of 61 tsf. Also, the average normalized CPT tip resistance, q_{c1n} (normalized to an effective overburden pressure of 1 tsf), for Stratum J Clay was 27 (dimensionless). Note that a CPT tip resistance profile versus elevation is shown on Figure 2.5S.4-17 for the STP 4 area.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Sub-stratum J Clay. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Sub-stratum J Clay, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	70	16	38	23
Liquid Limit (%)	58	30	85	54
Plasticity Index (%)	58	12	62	35
Fines Content (%)	29	18	100	89
Unit Weight (pcf)	37	104	132	125

Test results are summarized in Table 2.5S.4-8. Natural moisture contents and Atterberg limits are presented versus elevation on Figure 2.5S.4-20. Atterberg limits are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Sub-stratum J Clay soils were characterized, on average, as high plasticity clay with an average fines content (materials passing the No. 200 sieve) of 89%. Note that the minimum 18% fines content reported occurred at Boring B-401 from depths of 153 feet to 155 feet. This result represents an isolated thin sand lens within Sub-stratum J Clay. All other fines contents reported were greater than 65%. The USCS designations for Sub-stratum J Clay were mainly fat clay, lean clay, sandy lean clay, lean clay with sand, fat clay with sand, and occasionally silty sand, with the predominant USCS group symbols of CH and CL. Based on laboratory testing, an average unit weight of 125 pcf was selected for Sub-stratum J Clay.

The undrained shear strength of Sub-stratum J Clay was evaluated based on laboratory testing, and using correlations with corrected SPT N-values and the CPT results. The results of this evaluation are summarized in Table 2.5S.4-9A and B.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT $(N_1)_{60}$-value for Sub-stratum J Clay (15 blows/foot), an s_u=1.9 ksf was

estimated. Undrained shear strength was also estimated using the CPT data, following a CPT-s_u correlation (Reference 2.5S.4-13) given as Equation 2.5S.4-3. A site-specific cone factor of N_{kt}=19 was determined for the site soils, as noted above. Shear strength values calculated from the CPT data indicated an average s_u=3.8 ksf. The CPT-derived values are shown versus elevation on Figures 2.5S.4-24 and 2.5S.4-27, for the STP 4 area and site-wide, respectively. Note that SPT correlations were based on 215 field measurements, while CPT correlations were based on only five field measurements made within Sub-stratum J Clay. The results of 27 laboratory UU and UNC strength tests made on selected samples indicated an average s_u=3.2 ksf. By excluding the three lowest laboratory strength test result of s_u=0.1 ksf, 0.1 ksf, and 0.7 ksf (likely made on a samples of poor or non-representative quality), an average s_u=3.5 ksf results. Laboratory shear strength test results are summarized in Table 2.5S.4-9A and plotted versus elevation on Figure 2.5S.4-22. UU strength results from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated an average s_u=3.3 ksf for Sub-stratum J Clay (29 test results). Based on this, it was deemed that the laboratory-derived s_u results from this subsurface investigation were more representative, and an undrained shear strength of s_u=3.5 ksf was selected for Sub-stratum J Clay.

The drained friction angle of Sub-Strata J Clay soils was evaluated from laboratory test results. The results are shown in Table 2.5S.4-10 and summarized below. Strength parameters from seven CIU-bar tests, indicated average (drained/effective) ϕ'=8 degrees and c'=2.6 ksf and average (undrained/total) ϕ=4 degrees and c=2.9 ksf.

Parameter	From CIU-Bar
ϕ' (degrees)	8
C' (tsf)	2.6
ϕ (degrees)	4
C (tsf)	2.9

Based on the results of CIU-bar tests made on Stratum D (having similar plasticity to Strata A, F, and J Clay, as noted above), ϕ'=20 degrees was selected for Sub-stratum J Clay soils.

Consolidation properties and the stress history of Sub-stratum J Clay soils were assessed via laboratory testing and via an evaluation of the CPT results. A summary, and the results of, laboratory consolidation tests made on selected samples are presented in Tables 2.5S.4-11 and 2.5S.4-12, respectively. These results are also plotted versus elevation and shown on Figure 2.5S.4-28. The results of 10 consolidation tests made on selected samples indicated that, on average, Sub-stratum J Clay was preconsolidated to approximately 18.6 ksf, with an OCR=1.9. Consolidation test results for Sub-stratum J Clay from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated that, on average, Sub-stratum J Clay was preconsolidated to approximately 24 ksf, with an OCR=2. CPT-derived OCR data for Sub-stratum J Clay indicated an average OCR=1.8 and were based on five field measurements made at CPT C-408. CPT-derived OCR data are shown on Figures 2.5S.4-30 and 2.5S.4-33, for the STP 4 area and site-wide, respectively. A summary of OCR values derived from the CPT results is shown in Table

2.5S.4-13. Overall, an OCR=1.7 and a preconsolidation pressure of 18.5 ksf were selected for Sub-stratum J Clay.

The elastic modulus (E) for Sub-stratum J Clay was evaluated using Equation 2.5S.4-4. Substituting the previously established s_u for Sub-stratum J Clay soils (s_u=3.5 ksf), an E=2100 ksf was estimated. Other relationships for E (linked to G and to PI) were also available for fine-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. Using the V_s=1145 feet/second for Sub-stratum J Clay obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.45 for clay, γ=125 pcf for Sub-stratum J Clay, and PI=35 for Sub-stratum J Clay, an E=4157 ksf was estimated. Using an average of the E-values estimated from the undrained shear strength and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=3500 ksf was selected for Sub-stratum J Clay. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.45 for clay, a G=724 ksf was estimated based on the s_u-derived E, while a G=1433 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=1200 ksf was selected for Sub-stratum J Clay. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction, earth pressure coefficients, and the sliding coefficient were not considered for Sub-stratum J Clay. Foundations are not anticipated to bear at the depth of this stratum.

All of the material parameters selected for engineering purposes for Sub-stratum J Clay are summarized in Table 2.5S.4-16.

2.5S.4.2.1.8.2 Sub-stratum J Sand

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling, and were recorded on the boring logs. In the STP 3 area, uncorrected SPT N-values in Sub-stratum J Sand ranged from 22 blows/foot to 120 blows/foot, with an average uncorrected SPT N-value of 73 blows/foot. In the STP 4 area, uncorrected SPT N-values in Sub-stratum J Sand ranged from 18 blows/foot to 120 blows/foot, with an average uncorrected SPT N-value of 56 blows/foot. In the UHS Basin/RSW area and outside the Power Block area, borings did not reach Sub-stratum J Sand. Additional SPT N-value information on this stratum at locations other than the STP 3 area, and the STP 4 area is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11, and 2.5S.4-12 and 2.5S.4-13, for the STP 3 area, and the STP 4, respectively. The site-wide average uncorrected SPT N-value was 65 blows/foot for Sub-stratum J Sand.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1) by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Sub-

stratum J Sand was 36 blows/foot. An SPT $(N_1)_{60}$-value of 35 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum J Sand is considered dense.

CPTs did not reach Sub-stratum J Sand.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Sub-stratum J Sand. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Sub-stratum J Sand with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	13	19	32	23
Liquid Limit (%)	6	Non-Plastic	62	Non-Plastic
Plasticity Index (%)	6	Non-Plastic	35	Non-Plastic
Fines Content (%)	12	10	77	43
Unit Weight (pcf)	5	122	128	125

Test results are summarized in Table 2.5S.4-8. Natural moisture contents and Atterberg limits are presented versus elevation on Figure 2.5S.4-20. Atterberg limits are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Sub-stratum J Sand soils were characterized, on average, as silty sand to sandy silt with an average fines content (materials passing the No. 200 sieve) of 43%. Note that the maximum values for Liquid Limit and for Plasticity Index (PI) reported occurred at Boring B-409 from depths of 160 feet to 162 feet. These results represent an isolated thin clay lens within Sub-stratum J Sand. All other Atterberg Limits tests were reported as non-plastic. The USCS designations for Sub-stratum J Sand were mainly, silty sand, sandy silt, silt with sand, poorly graded sand with silt, and sandy lean clay and occasionally fat clay, with the predominant USCS group symbols of SM and ML. Based on laboratory testing, an average unit weight of 125 pcf was selected for Sub-stratum J Sand.

The strength of Sub-stratum J Sand was evaluated based on laboratory testing, and using a correlation with corrected SPT $(N_1)_{60}$-values. The results of the laboratory testing are summarized in Table 2.5S.4-10.

The drained friction angle, ϕ', was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values, according to Reference 2.5S.4-14. Using the selected corrected SPT $(N_1)_{60}$-value for Sub-stratum J Sand (35 blows/foot), a value of ϕ'=of 39 degrees (for fine to medium sand) was estimated. A value of ϕ'=37 degrees was considered appropriate. Results of one laboratory direct shear test made on selected samples indicated a ϕ'=32 degrees. Laboratory direct shear test results are summarized in Table 2.5S.4-10.

From the above, a summary of average ϕ' values for Sub-stratum J Sand is provided as follows:

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	37	---	32

Based on the above a ϕ'=33 degrees was selected for Sub-stratum J Sand.

Consolidation properties of the granular Sub-stratum J Sand were not evaluated/relevant.

The elastic modulus, E, for coarse-grained soils was evaluated using Equation 2.5S.4-13. Substituting the previously established average corrected SPT $(N_1)_{60}$-value for Sub-stratum J Sand soils (35 blows per foot), an E=1260 ksf was estimated. Other relationships for E were available for coarse-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. Using the V_s=1275 feet/second for Sub-stratum J Sand obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.30 for sand and γ=125 pcf for Sub-stratum J Sand, an E=1641 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT $(N_1)_{60}$-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=1500 ksf was selected for Sub-stratum J Sand. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=485 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=631 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=600 ksf was selected for Sub-stratum J Sand. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction, earth pressure coefficients, and the sliding coefficient were not considered for Sub-stratum J Sand. Foundations are not anticipated to bear at the depth of this stratum.

All of the material parameters selected for engineering purposes for Sub-stratum J Sand are summarized in Table 2.5S.4-16.

2.5S.4.2.1.9 Stratum K

Stratum K soils were encountered below Stratum J in Boring B-305DH/DHA in the STP 3 area and in Boring B-405DH in the STP 4 area. The stratum was fully penetrated in both borings. Stratum K typically consisted of greenish gray to gray clay with varying amounts of sand, grading to a silty sand or silt in the lower portions. The following sub-strata were identified:

- Sub-stratum K Clay
- and, Sub-stratum K Sand/Silt

The thickness of Stratum K was estimated from the borings. No CPTs reached Stratum K or the other underlying strata. Overall, the stratum had an average thickness of 44 feet.

Sub-stratum K Clay was encountered in both borings (B-305DH/DHA and B-405DH). Sub-stratum K Clay ranged in thickness from 15 feet to 22 feet, with an average thickness of 19 feet. The average base elevation of Sub-stratum K Clay was El. -203 feet.

Sub-stratum K Sand/Silt below Sub-stratum K Clay was also encountered in both borings (B-305DH/DHA and B-405DH). Sub-stratum K Sand/Silt ranged in thickness from 20 feet to 30 feet, with an average thickness of 25 feet. The average base elevation of Sub-stratum K Sand/Silt was El. -228 feet.

For discussion of engineering properties, the Stratum K sub-strata were grouped as follows:

- Sub-stratum K Clay
- Sub-stratum K Sand/Silt

2.5S.4.2.1.9.1 Sub-stratum K Clay

Soil samples were collected from the borings via SPT sampling and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling and were recorded on the boring logs. In the STP 3 & 4 area, uncorrected SPT N-values (only two tests conducted) in Sub-stratum K Clay ranged from 15 blows/foot to 15 blows/foot, with an average uncorrected SPT N-value of 15 blows/foot. In the UHS Basin/RSW area, borings did not reach Sub-stratum K Clay. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11, and 2.5S.4-12 and 2.5S.4-13, for the STP 3 area, and the STP 4 area, respectively. The site-wide average uncorrected SPT N-value was 15 blows/foot for Sub-stratum K Clay.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Sub-stratum K Clay was 7 blows/foot. An SPT $(N_1)_{60}$-value of 6 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum K Clay is considered firm (although this stratum is likely much stiffer as the average

corrected SPT $(N_1)_{60}$-value results from a low correction factor, C_n, which was extrapolated beyond its normal stress range).

CPTs did not reach Sub-stratum K Clay.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Sub-stratum K Clay. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Sub-stratum K Clay, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	3	17	22	20
Liquid Limit (%)	2	33	45	39
Plasticity Index (%)	2	18	31	25
Fines Content (%)	1	75	75	75
Unit Weight (pcf)	2	127	132	129

Test results are summarized in Table 2.5S.4-8. Natural moisture contents and Atterberg limits are presented versus elevation on Figure 2.5S.4-20. Atterberg limits are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Sub-stratum K Clay soils were characterized, on average, as lean clay with an average fines content (materials passing the No. 200 sieve) of 75%. The USCS designations for Sub-stratum K Clay were mainly lean clay and lean clay with sand, with the predominant USCS group symbol of CL. Based on laboratory testing, an average unit weight of 129 pcf was selected for Sub-stratum K Clay.

The undrained shear strength of Sub-stratum K Clay was evaluated based on laboratory testing, and using correlations with corrected SPT $(N_1)_{60}$-values. The results of this evaluation are summarized in Table 2.5S.4-9A and B.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT $(N_1)_{60}$-value for Sub-stratum K Clay (6 blows/foot), an s_u=0.8 ksf was estimated. Note, however, that this average value is based on only two corrected SPT $(N_1)_{60}$-values. Also note that CPT data were not available for this sub-stratum. Results of two laboratory UU and UNC strength tests made on selected samples indicated an average s_u=3.4 ksf. Laboratory shear strength test results are summarized in Table 2.5S.4-9A and plotted versus elevation on Figure 2.5S.4-22. Shear strength test results for Sub-stratum K Clay from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) were also not available. Based on this, it was deemed that the laboratory derived s_u results from this subsurface investigation were more representative, and an undrained shear strength of s_u=3.0 ksf was selected for Sub-stratum K Clay.

The drained friction angle of Sub-stratum K Clay soils was not evaluated/relevant.

Consolidation properties and the stress history of Sub-stratum K Clay soils were assessed via laboratory testing. A summary, and the results of, laboratory consolidation tests made on selected samples are presented in Tables 2.5S.4-11 and 2.5S.4-12, respectively. These results are also plotted versus elevation and shown on Figure 2.5S.4-28. The results of two consolidation tests made on selected samples indicated that, on average, Sub-stratum K Clay was preconsolidated to approximately 24 ksf, with an OCR=1.7. Consolidation test results for Sub-stratum K Clay from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated that, on average, Sub-stratum K Clay was preconsolidated to approximately 25 ksf, with an OCR=1.6. Overall, an OCR=1.3 and a preconsolidation pressure of 18.3 ksf were selected for Sub-stratum K Clay.

The elastic modulus (E) for Sub-stratum K Clay was evaluated using Equation 2.5S.4-4. Substituting the previously established s_u for Sub-stratum K Clay soils (s_u=3.0 ksf), an E=1800 ksf was estimated. Other relationships for E (linked to G and to PI) were also available for fine-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. Using the V_s=1145 feet/second for Sub-stratum K Clay obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.45 for clay, =129 pcf for Sub-stratum K Clay, and PI=25 for Sub-stratum K Clay, an E=3787 ksf was estimated. Using an average of the E-values estimated from the undrained shear strength and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=3100 ksf was selected for Sub-stratum K Clay. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.45 for clay, a G=621 ksf was estimated based on the s_u-derived E, while a G=1306 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=1050 ksf was selected for Sub-stratum K Clay. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction, earth pressure coefficients, and the sliding coefficient were not considered for Sub-stratum K Clay. Foundations are not anticipated to bear at the depth of this stratum.

All of the material parameters selected for engineering purposes for Sub-stratum K Clay are summarized in Table 2.5S.4-16.

2.5S.4.2.1.9.2 Sub-stratum K Sand/Silt

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling, and were recorded on the boring logs. In the STP 3 & 4 area, uncorrected SPT N-values (only two tests conducted) in Sub-stratum K Sand/Silt ranged from 30 blows/foot to 120 blows/foot, with an average uncorrected SPT N-value of 75 blows/foot. In the UHS Basin/RSW area, borings did not reach Sub-stratum K Sand/Silt. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11, and 2.5S.4-12 and 2.5S.4-13, for the STP 3 area, and the STP 4, respectively. The site-wide average uncorrected SPT N-value was 75 blows/foot for Sub-stratum K Sand/Silt.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Sub-stratum K Sand/Silt was 31 blows/foot. An SPT $(N_1)_{60}$-value of 30 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum K Sand/Silt is considered dense (although this stratum is likely more dense as the average corrected SPT $(N_1)_{60}$-value results from a low correction factor, C_n, which was extrapolated beyond its normal stress range).

CPTs did not reach Sub-stratum K Sand/Silt.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Sub-stratum K Sand/Silt. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Sub-stratum K Sand/Silt with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	2	20	22	21
Liquid Limit (%)	1	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	1	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	2	27	64	45
Unit Weight (pcf)	1	127	127	127

Test results are summarized in Table 2.5S.4-8. Note that natural moisture contents and Atterberg limits for other soil strata are presented versus elevation on Figure 2.5S.4-20. Note also that Atterberg limits for other soil strata are shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Sub-stratum K Sand/Silt soils were characterized, on average, as silty sand to sandy silt with an average fines content (materials passing the No. 200 sieve) of

45%. The USCS designations for Sub-stratum K Sand/Silt were mainly silty sand and sandy silt, with the predominant USCS group symbols of SM and ML. Based on laboratory testing, an average unit weight of 127 pcf was selected for Sub-stratum K Sand/Silt.

The strength of Sub-stratum K Sand/Silt was evaluated based on laboratory testing, and using a correlation with corrected SPT $(N_1)_{60}$-values. The results of the laboratory testing are summarized in Table 2.5S.4-10.

The drained friction angle, ϕ', was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values, according to Reference 2.5S.4-14. Using the selected corrected SPT $(N_1)_{60}$-value for Sub-stratum K Sand/Silt (30 blows/foot), a value of ϕ'=of 38 degrees (for fine sand) was estimated. A value of ϕ'=36 degrees was considered appropriate. Note, however, that this average value is based on only two corrected SPT $(N_1)_{60}$-values. Results of one laboratory direct shear test made on selected samples indicated a ϕ'=29 degrees. Laboratory direct shear test results are summarized in Table 2.5S.4-10.

From the above, a summary of average ϕ' values for Sub-stratum K Sand is provided as follows:

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	36	---	29

Based on the above a ϕ'=33 degrees was selected for Sub-stratum K Sand/Silt.

Consolidation properties of the granular Sub-stratum K Sand/Silt were not evaluated/relevant.

The elastic modulus, E, for coarse-grained soils was evaluated using Equation 2.5S.4-13. Substituting the previously established average corrected SPT $(N_1)_{60}$-value for Sub-stratum K Sand/Silt soils (30 blows per foot) an E=1080 ksf was estimated. Other relationships for E were available for coarse-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. Using the V_s=1370 feet/second for Sub-stratum K Sand/Silt obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion) and using μ=0.30 for sand and γ=127 pcf for Sub-stratum K Sand/Silt, an E=1924 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT $(N_1)_{60}$-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=1650 ksf was selected for Sub-stratum K Sand/Silt. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=415 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=740 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=650 ksf was selected for Sub-stratum K Sand/Silt. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction, earth pressure coefficients, and the sliding coefficient were not considered for Sub-stratum K Sand/Silt. Foundations are not anticipated to bear at the depth of this stratum.

All of the material parameters selected for engineering purposes for Sub-stratum K Sand/Silt are summarized in Table 2.5S.4-16.

2.5S.4.2.1.10 Stratum L

Stratum L soils were encountered below Stratum K in Boring B-305DH/DHA in the STP 3 area, and in Boring B-405DH in the STP 4 area. The stratum was fully penetrated in both borings. Stratum L typically consisted of red to brown clay with varying amounts of sand.

The thickness of Stratum L was estimated from the borings. No CPTs reached Stratum L or the other underlying strata. The thickness of Stratum L varied from 4.5 feet to 5 feet, with an average thickness of 5 feet. The average base elevation of Stratum L was El. -233 feet.

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling, and were recorded on the boring logs. In the STP 3 & 4 area, uncorrected SPT N-values (only two tests conducted) in Stratum L ranged from 21 blows/foot to 24 blows/foot, with an average uncorrected SPT N-value of 23 blows/foot. In the UHS Basin/RSW area, borings did not reach Stratum L. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11, and 2.5S.4-12 and 2.5S.4-13, for the STP 3 area, and the STP 4 area, respectively. The site-wide average uncorrected SPT N-value was 23 blows/foot for Sub-stratum L.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum L was 9 blows/foot. An SPT $(N_1)_{60}$-value of 8 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum L is firm to stiff (although this stratum is likely much stiffer as the average corrected SPT N-value results from a low correction factor, C_n, which was extrapolated beyond its normal stress range).

CPTs did not reach Stratum L.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Stratum L. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Stratum L, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	2	27	30	29
Liquid Limit (%)	2	72	74	73
Plasticity Index (%)	2	51	52	52
Fines Content (%)	---	---	---	---
Unit Weight (pcf)	---	---	---	---

Test results are summarized in Table 2.5S.4-8. Natural moisture contents and Atterberg limits are presented versus elevation on Figure 2.5S.4-20. Atterberg limits are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Stratum L soils were characterized, on average, as high plasticity clay with an average fines content (materials passing the No. 200 sieve) of 75% (employing the value from Sub-stratum K Clay in the absence of laboratory fines content tests on Stratum L). The USCS designations for Stratum L were mainly fat clay, with the predominant USCS group symbols of CH. Based on laboratory testing, an average unit weight of 129 pcf was selected for Stratum L (again, employing the value from Sub-stratum K Clay in the absence of laboratory unit weight tests on Stratum L).

The undrained shear strength of Stratum L was evaluated based on laboratory testing and using correlations with corrected SPT N-values. The results of this evaluation are summarized in Table 2.5S.4-9A and B.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT $(N_1)_{60}$-value for Stratum L (8 blows/foot), an s_u=1.0 ksf was estimated. Note, however, that this average value is based on only two corrected SPT $(N_1)_{60}$-values. Also note that neither CPT data nor laboratory shear strength data from UU and/or UNC strength tests were available for this stratum. In addition, shear strength test results for Stratum L from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) were also not available. Based on the above, it was considered that the laboratory derived s_u results reported for Sub-stratum K Clay, as above, could be similarly assigned to Stratum L, and as such, an undrained shear strength of s_u=3.0 ksf was selected for Stratum L.

The drained friction angle of Stratum L soils was not evaluated/relevant.

Consolidation properties and the stress history of Stratum L soils were assessed via laboratory testing. A summary, and the results of, laboratory consolidation tests made on selected samples are presented in Tables 2.5S.4-11 and 2.5S.4-12, respectively. These results are also plotted versus elevation and shown on Figure 2.5S.4-28. Note that there were no consolidation tests of

Stratum L soils made as a part of this subsurface investigation. Consolidation test results for Stratum L from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated that, on average, Stratum L was preconsolidated to approximately 25 ksf, with an OCR=1.3. Overall, an OCR=1.0 and a preconsolidation pressure of 16 ksf were selected for Stratum L.

The elastic modulus (E) for Stratum L was evaluated using Equation 2.5S.4-4. Substituting the previously established s_u for Stratum L soils (s_u=3.0 ksf), an E=1800 ksf was estimated. Other relationships for E (linked to G and to PI) were also available for fine-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. Using the V_s=975 feet/second for Stratum L obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.45 for clay, γ=129 pcf for Stratum L, and PI=50 for Stratum L, an E=3718 ksf was estimated. Using an average of the E-values estimated from the undrained shear strength and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=3100 ksf was selected for Stratum L. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.45 for clay, a G=621 ksf was estimated based on the s_u-derived E, while a G=1282 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=1050 ksf was selected for Stratum L. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction, earth pressure coefficients, and the sliding coefficient were not considered for Stratum L. Foundations are not anticipated to bear at the depth of this stratum.

All of the material parameters selected for engineering purposes for Stratum L are summarized in Table 2.5S.4-16.

2.5S.4.2.1.11 Stratum M

Stratum M soils were encountered below Stratum L in Boring B-305DH/DHA in the STP 3 area and in Boring B-405DH in the STP 4 area. The stratum was fully penetrated in both borings. Stratum M typically consisted of olive brown to greenish gray sand with silt to silty sand.

The thickness of Stratum M was estimated from the borings. No CPTs reached Stratum M or the other underlying strata. The thickness of Stratum M varied from 14.5 feet to 15.5 feet, with an average thickness of 15 feet. The average base elevation of Stratum M was El. -248 feet.

Soil samples were collected in Stratum M via undisturbed three-inch-diameter tube sampling (two such samples collected). Standard penetration tests (SPT) in Stratum M were not conducted due to the limited thickness and substantial depth of the stratum.

CPTs did not reach Stratum M.

Due to limited stratum thickness and available soil samples, laboratory index tests, and tests for the determination of engineering properties, were not made on samples from Stratum M. Based

on boring log visual classifications, Stratum M is considered to have index properties similar to Sub-stratum K Sand/Silt.

For engineering purposes, Stratum M soils were characterized, on average, as sand with silt to silty sand (based on visual classifications), with an average fines content (materials passing the No. 200 sieve) of 45% (employing the value from Sub-stratum K Sand/Silt in the absence of laboratory fines content tests on Stratum M). The USCS designations for Stratum M were mainly poorly graded sand with silt to silty sand (based on visual classifications), with the predominant USCS group symbol of SM. An average unit weight of 127 pcf was selected for Stratum M (again, employing the value from Sub-stratum K Sand/Silt in the absence of laboratory unit weight tests on Stratum M).

In the absence of laboratory strength test data and SPT N-value data specific to Stratum M, a drained friction angle of ϕ'=33 degrees was selected for Stratum M, based on the Sub-stratum K Sand/Silt results.

Consolidation properties of the granular Stratum M were not evaluated/relevant.

The elastic modulus, E, for coarse-grained soils was evaluated using Equation 2.5S.4-13. Substituting the previously established average corrected SPT $(N_1)_{60}$-value for Sub-stratum K Sand/Silt soils (30 blows per foot) (as above, SPT N-value data for Stratum M were not collected), an E=1080 ksf was estimated. Other relationships for E were available for coarse-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. Using the V_s=1165 feet/second for Stratum M obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.30 for sand, and γ=127 pcf for Stratum M, an E=1391 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT $(N_1)_{60}$-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=1300 ksf was selected for Stratum M. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=415 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=535 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=500 ksf was selected for Stratum M. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction, earth pressure coefficients, and the sliding coefficient were not considered for Stratum M. Foundations are not anticipated to bear at the depth of this stratum.

All of the material parameters selected for engineering purposes for Stratum M are summarized in Table 2.5S.4-16.

2.5S.4.2.1.12 Stratum N

Stratum N soils were encountered below Stratum M in Boring B-305DH/DHA in the STP 3 area, and in Boring B-405DH in the STP 4 area. The stratum extended to depths greater than the maximum depth investigated (i.e., greater than approximately 600 feet below ground surface). Stratum N typically consisted of brown to greenish gray clay with varying amounts of sand, with interbedded sub-strata of sand to silty sand. The following sub-strata were identified:

- Sub-stratum N Clay 1
- Sub-stratum N Sand 1
- Sub-stratum N Clay 2
- Sub-stratum N Sand 2
- Sub-stratum N Clay 3
- Sub-stratum N Sand 3
- Sub-stratum N Clay 4
- Sub-stratum N Sand 4
- Sub-stratum N Clay 5
- Sub-stratum N Sand 5
- Sub-stratum N Clay 6

The thickness of Stratum N encountered was estimated from the borings. No CPTs reached Stratum N. Overall, the stratum had an average thickness of greater than 327 feet.

Sub-stratum N Clay 1 was encountered in both borings (B-305DH/DHA and B-405DH), ranging in thickness from 57 feet to 62 feet, with an average thickness of 59 feet. The average base elevation of Sub-stratum N Clay 1 was El. -307 feet.

Sub-stratum N Sand 1 was encountered in both borings (B-305DH/DHA and B-405DH) ranging in thickness from 16 feet to 18 feet, with an average thickness of 17 feet. The average base elevation of Sub-stratum N Sand 1 was El. -324 feet.

Sub-stratum N Clay 2 was encountered in both borings (B-305DH/DHA and B-405DH), ranging in thickness from 4 feet to 11 feet, with an average thickness of 8 feet. The average base elevation of Sub-stratum N Clay 2 was El. -332 feet.

Sub-stratum N Sand 2 was encountered in both borings (B-305DH/DHA and B-405DH), ranging in thickness from 26 feet to 39 feet, with an average thickness of 33 feet. The average base elevation of Sub-stratum N Sand 2 was El. -365 feet.

Sub-stratum N Clay 3 was encountered in both borings (B-305DH/DHA and B-405DH), ranging in thickness from 7 feet to 10 feet, with an average thickness of 9 feet. The average base elevation of Sub-stratum N Clay 3 was El. -373 feet.

Sub-stratum N Sand 3 was encountered in both borings (B-305DH/DHA and B-405DH), ranging in thickness from 17 feet to 20 feet, with an average thickness of 19 feet. The average base elevation of Sub-stratum N Sand 3 was El. -392 feet.

Sub-stratum N Clay 4 was encountered in both borings (B-305DH/DHA and B-405DH), ranging in thickness from 25 feet to 35 feet, with an average thickness of 30 feet. The average base elevation of Sub-stratum N Clay 4 was El. -422 feet.

Sub-stratum N Sand 4 was encountered only in Boring B-305DH/DHA at a thickness of 16 feet. The average base elevation of Sub-stratum N Sand 4 was El. -435 feet.

Sub-stratum N Clay 5 was encountered in both borings (B-305DH/DHA and B-405DH), ranging in thickness from 50 feet to 58 feet, with an average thickness of 54 feet. The average base elevation of Sub-stratum N Clay 5 was El. -484 feet.

Sub-stratum N Sand 5 was encountered only in Boring B-405DH at a thickness of 35 feet. The average base elevation of Sub-stratum N Sand 5 was El. -509 feet.

Sub-stratum N Clay 6 was encountered in both borings (B-305DH/DHA and B-405DH), ranging in thickness from greater than 60 feet to greater than 77 feet, with an average thickness of greater than 69 feet. This stratum extended to the termination depth of both borings, at approximately El. -570 feet.

For discussion of engineering properties, the Stratum N sub-strata were grouped as follows:

- Sub-stratum N Clay, which contained Sub-stratum N Clay 1, Sub-stratum N Clay 2, Sub-stratum N Clay 3, Sub-stratum N Clay 4, Sub-stratum N Clay 5, and Sub-stratum N Clay 6
- Sub-stratum N Sand, which contained Sub-stratum N Sand 1, Sub-stratum N Sand 2, Sub-stratum N Sand 3, Sub-stratum N Sand 4, and Sub-stratum N Sand 5

2.5S.4.2.1.12.1 Sub-stratum N Clay

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling, and were recorded on the boring logs. In the STP 3 and STP 4 areas, uncorrected SPT N-values in Sub-stratum N Clay ranged from 2 blows/foot to 47 blows/foot, with an average uncorrected SPT N-value of 33 blows/foot. In the UHS Basin/RSW area, borings did not reach Sub-stratum N Clay. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11, and 2.5S.4-12 and 2.5S.4-13, for the STP 3 area, and the STP 4, respectively. The site-wide average uncorrected SPT N-value was 33 blows/foot for Sub-stratum N Clay.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Sub-stratum N Clay was 8 blows/foot. An SPT $(N_1)_{60}$-value of 7 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum N Clay is firm (although this stratum is likely much stiffer as the average corrected SPT $(N_1)_{60}$-value results from a low correction factor, C_n, which was extrapolated beyond its normal stress range).

CPTs did not reach Sub-stratum N Clay.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Sub-stratum N Clay. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Sub-stratum N Clay, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	12	17	38	25
Liquid Limit (%)	11	45	90	65
Plasticity Index (%)	11	25	63	44
Fines Content (%)	5	22	95	75
Unit Weight (pcf)	4	113	127	121

Test results are summarized in Table 2.5S.4-8. Natural moisture contents and Atterberg limits are presented versus elevation on Figure 2.5S.4-20. Atterberg limits are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Sub-stratum N Clay soils were characterized, on average, as high plasticity clay with an average fines content (materials passing the No. 200 sieve) of 75%. Note that the minimum 22% fines content reported occurred at Boring B-401 from depths of 318 feet to 320 feet. This result represents an isolated thin sand lens within Sub-stratum N Clay. All other fines contents reported were greater than 80%. The USCS designations for Sub-stratum N Clay were mainly fat clay, lean clay, and clayey sand,

with the predominant USCS group symbols of CH and CL. Based on laboratory testing, an average unit weight of 121 pcf was selected for Sub-stratum N Clay.

The undrained shear strength of Sub-stratum N Clay was evaluated based on laboratory testing, and using correlations with corrected SPT $(N_1)_{60}$-values. The results of this evaluation are summarized in Table 2.5S.4-9A and B.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT N-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT N-value for Sub-stratum N Clay (7 blows/foot), an s_u=0.9 ksf was estimated. Note that CPT data were not available for this sub-stratum. Results of four laboratory UU and UNC strength tests made on selected samples indicated an average s_u=1.7 ksf. By excluding the lowest laboratory strength test result of s_u=0.2 ksf (likely made on a sample of poor or non-representative quality), an average s_u=2.3 ksf resulted. Laboratory shear strength test results are summarized in Table 2.5S.4-9A and plotted versus elevation on Figure 2.5S.4-22. Shear strength test results for Sub-stratum N Clay from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) were also not available. Based on this, it was deemed that the laboratory derived s_u results from this subsurface investigation were more representative, and an undrained shear strength of s_u=3.0 ksf was selected for Sub-stratum N Clay (similar to Sub-stratum K Clay).

The drained friction angle of Sub-Strata N Clay soils was not evaluated/relevant.

Consolidation properties and the stress history of Sub-stratum N Clay soils were assessed via laboratory testing. A summary and the results of laboratory consolidation tests made on selected samples are presented in Tables 2.5S.4-11 and 2.5S.4-12, respectively. These results are also plotted versus elevation and shown on Figure 2.5S.4-28. Results of two consolidation tests made on selected samples indicated that, on average, Sub-stratum N Clay was preconsolidated to approximately 18.4 ksf, with an OCR=0.8. Consolidation test results for Sub-stratum N Clay from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated that, on average, Sub-stratum N Clay was preconsolidated to approximately 43 ksf, with an OCR=1.4. Overall, an OCR=1.0 and a preconsolidation pressure of 28.5 ksf were selected for Sub-stratum N Clay.

The elastic modulus (E) for Sub-stratum N Clay was evaluated using Equation 2.5S.4-4. Substituting the previously established s_u for Sub-stratum N Clay soils (s_u=3.0 ksf), an E=1800 ksf was estimated. Other relationships for E (linked to G and to PI) were also available for fine-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. Using the V_s=1290 feet/second for Sub-stratum N Clay obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.45 for clay, γ=121 pcf for Sub-stratum N Clay, and PI=45 for Sub-stratum N Clay, an E=5794 ksf was estimated. Using an average of the E-values estimated from the undrained shear strength and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=4500 ksf was selected for Sub-stratum N Clay. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.45 for clay, a G=621 ksf was estimated based on the s_u-derived E, while a G=1998 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An

average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=1500 ksf was selected for Sub-stratum N Clay. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction, earth pressure coefficients, and the sliding coefficient were not considered for Sub-stratum N Clay. Foundations are not anticipated to bear at the depth of this stratum.

All of the material parameters selected for engineering purposes for Sub-stratum N Clay are summarized in Table 2.5S.4-16.

2.5S.4.2.1.12.2 Sub-stratum N Sand

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling, and were recorded on the boring logs. In the STP 3 and STP 4 areas, uncorrected SPT N-values in Sub-stratum N Sand ranged from 20 blows/foot to 200 blows/foot, with an average uncorrected SPT N-value of 97 blows/foot. In the UHS Basin/RSW area, borings did not reach Sub-stratum N Sand. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11, and 2.5S.4-12 and 2.5S.4-13, for the STP 3 area, and the STP 4, respectively. The site-wide average uncorrected SPT N-value was 97 blows/foot for Sub-stratum N Sand.

As noted above, uncorrected SPT N-values from each boring were corrected to an effective overburden pressure of one tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (leading to fully-corrected values of $(N_1)_{60}$). A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Sub-stratum N Sand was 23 blows/foot. An SPT $(N_1)_{60}$-value of 20 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT $(N_1)_{60}$-values, Stratum N Sand is medium dense (although this stratum is likely more dense as the average corrected SPT $(N_1)_{60}$-value results from a low correction factor, C_n, which was extrapolated beyond its normal stress range).

CPTs did not reach Sub-stratum N Sand.

Laboratory index tests, and tests for the determination of engineering properties, were performed on selected samples from Sub-stratum N Sand. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Sub-stratum N Sand with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	10	17	28	23
Liquid Limit (%)	4	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	4	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	11	5	49	22
Unit Weight (pcf)	2	126	130	128

Test results are summarized in Table 2.5S.4-8. Note that natural moisture contents and Atterberg limits for other soil strata are presented versus elevation on Figure 2.5S.4-20. Note also that Atterberg limits for other soil strata are shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Sub-stratum N Sand soils were characterized, on average, as silty sand with an average fines content (materials passing the No. 200 sieve) of 22%. The USCS designations for Sub-stratum N Sand were mainly silty sand, poorly graded sand with silt, clayey sand, and poorly graded sand, with the predominant USCS group symbols of SM and SP-SM. Based on laboratory testing, an average unit weight of 128 pcf was selected for Sub-stratum N Sand.

The strength of Sub-stratum N Sand was evaluated based on laboratory testing, and using a correlation with corrected SPT $(N_1)_{60}$-values. The results of the laboratory testing are summarized in Table 2.5S.4-10.

The drained friction angle, ϕ', was estimated from empirical correlations with corrected SPT N-values, according to Reference 2.5S.4-14. Using the selected corrected SPT N-value for Sub-stratum N Sand (20 blows/foot), a value of ϕ'=of 38 degrees (for fine to coarse sand) was estimated. A value of ϕ'=36 degrees was considered appropriate. Note that laboratory direct shear tests made on selected samples were not available for this sub-stratum.

From the above, a summary of average ϕ' values for Sub-stratum N Sand is provided as follows:

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	36	---	---

Based on the above a ϕ'=36 degrees was selected for Sub-stratum N Sand.

Consolidation properties of the granular Sub-stratum N Sand were not evaluated/relevant.

The elastic modulus, E, for coarse-grained soils was evaluated using Equation 2.5S.4-13. Substituting the previously established average corrected SPT $(N_1)_{60}$-value for Sub-stratum N Sand soils (20 blows per foot) an E=720 ksf was estimated. Other relationships for E were

available for coarse-grained soils (Reference 2.5S.4-10), namely Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. Using the V_s=1655 feet/second for Sub-stratum N Sand obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.30 for sand, and γ=128 pcf for Sub-stratum N Sand, an E=2831 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT $(N_1)_{60}$-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=2100 ksf was selected for Sub-stratum N Sand. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=277 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=1089 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=800 ksf was selected for Sub-stratum N Sand. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.

The coefficient of subgrade reaction, earth pressure coefficients, and the sliding coefficient were not considered for Sub-stratum N Sand. Foundations are not anticipated to bear at the depth of this stratum.

All of the material parameters selected for engineering purposes for Sub-stratum N Sand are summarized in Table 2.5S.4-16.

2.5S.4.2.1.13 Chemical Properties of Soils

Laboratory chemical tests and field electrical resistivity tests were made on selected soil and groundwater samples collected as a part of this subsurface investigation and as a part of the groundwater characterization addressed in Subsection 2.4S.12. A brief summary of the available information is evaluated and provided below.

2.5S.4.2.1.13.1 Laboratory Chemical Testing

Laboratory chemical tests consisting of pH, chloride content, and sulfate content, were performed on selected soil samples collected as a part of this subsurface investigation. Forty sets of chemical tests were made on site soils, from samples collected at depths ranging from 1.5 feet to 80 feet below ground surface. Twenty additional pH tests on collected soils samples were also performed, with the maximum depth tested (i.e., for pH alone) of 95 feet. Test results are presented in Reference 2.5S.4-2 (Appendix 2.5-A), and are summarized in Table 2.5S.4-8.

2.5S.4.2.1.13.2 Field Electrical Resistivity Testing

Field electrical resistivity tests were performed along four arrays at the locations shown on Figures 2.5S.4-1 and 2.5S.4-2. Test results are presented with Reference 2.5S.4-2 (Appendix 2.5-A) and are summarized in Table 2.5S.4-17. Note that Table 2.5S.4-17 additionally presents test results correlated with depth/soil strata based on the field test array spacing.

2.5S.4.2.1.13.3 Evaluation of Chemical Testing Data

Guidelines for the interpretation of chemical test results are provided in Table 2.5S.4-18, based on various references, especially References 2.5S.4-16, 2.5S.4-17, and 2.5S.4-18. The following can be concluded from the test results presented in Tables 2.5S.4-8 and 2.5S.4-17, and the guidelines presented in Table 2.5S.4-18.

The following paragraph relates to the potential for attack by soil/groundwater constituents on buried steel (i.e., corrosiveness/chloride contents). Field electrical resistivity test results indicated that all soils are “corrosive.” Chloride content tests in Stratum A samples yielded a wide range of results. Two of 20 Stratum A samples tested yielded “very corrosive” results, or chloride contents greater than 1000 parts per million (ppm). One Stratum A sample yielded a chloride content in the “corrosive” range, 300-1000 ppm. Four Stratum A samples yielded chloride contents in the “moderately corrosive” range, 200-300 ppm. The remaining thirteen Stratum A samples yielded chloride contents in the “mildly corrosive” range (less than 200 ppm). All chloride content tests performed on Stratum B, C, D, E, and F samples yielded chloride contents in the “mildly corrosive” range, less than 200 ppm. Laboratory pH test results indicated that all soils are “mildly corrosive,” with pH between 5 and 10. It is noted that laboratory chemical tests were not made on soil strata deeper than Stratum F, as STP 3 & 4 major structures (including Seismic Category I structures and/or piping) do not bear on, or contact, these deeper soil strata. Based on the available laboratory and field test results, Stratum A soils were deemed “corrosive” to “moderately corrosive,” while all other underlying soil strata tested were deemed as “moderately corrosive.” Protection of buried steel against corrosion from the ground may include specialty coatings, cathodic protection, or other measures, as determined during project detailed design stage. Additional pH testing on groundwater samples obtained from the observation wells (refer to Subsection 2.4S.12) indicated pH values in the range of “mildly corrosive” conditions. Note that observation wells installed as a part of this subsurface investigation were mainly screened in Strata C, E, or H soils.

The following paragraph relates to the potential for attack by soil/groundwater constituents on concrete in contact with the ground (i.e., aggressiveness/sulphate contents). Laboratory sulfate content tests made on soil samples as noted above, all indicated “mild” potential for sulphate attack on concrete in contact with the ground (up to 0.10%). As noted above, laboratory chemical tests were not made on soil strata deeper than Stratum F, as STP 3 & 4 major structures (including Seismic Category I structures [and/or piping]) do not bear on, or contact, these deeper soil strata.

2.5S.4.2.1.14 Subsurface Conditions Deeper than Approximately 600 Feet Below Ground Surface

As indicated above, the maximum depth explored by this subsurface investigation was approximately 600 feet below ground surface (Borings B-305DH/DHA and B-405DH). From the subsurface investigation reported on in the STP 1 & 2 UFSAR (Reference 2.5S.4-3), one boring, B-233, was extended to a greater depth, or approximately 2620 feet below ground surface. That one boring generally found alternating layers of clays and sands with depth, transitioning to soft sedimentary claystones and siltstones at depths greater than approximately 1100 feet below ground surface. Approximately two-thirds of the sediments encountered in the

boring were fine-grained, consisting mainly of lean clay, fat clay, silty clay, silt, claystone, or siltstone. The remaining one-third of the sediments encountered in the boring were coarse-grained, consisting mainly of silty sand or sand.

From Reference 2.5S.4-4, these alternating fine-grained and coarse-grained sediments extend to substantial depth. Refer to Subsection 2.5S.4.1 for a brief description of geologic conditions at depths below approximately 600 feet below ground surface, a key point being that the top depth of pre-Cretaceous bedrock ("basement rock") has been estimated to occur at approximately 34,500 feet below ground surface (Reference 2.5S.4-4).

2.5S.4.2.1.15 Field Testing Program

Planning for field testing made as a part of this subsurface investigation referred to guidance given in RG 1.132 (Reference 2.5S.4-19). References to industry standards used for field testing are shown in Table 2.5S.4-1. Field testing details and results are provided in Reference 2.5S.4-2 (Appendix 2.5-A).Details of the field testing are discussed further in Subsection 2.5S.4.2.2 The work was performed under an approved quality assurance program with work procedures developed specifically for STP 3 & 4, including a subsurface investigation plan developed by Bechtel. The initial subsurface investigation plan met the intent of Reference 2.5S.4-19. A supplemental subsurface investigation commenced onsite in mid-July 2007, to accommodate the addition of a Radwaste Building in the STP 4 area, and a revised routing of the Reactor Service Water Lines, all as shown on Figure 2.5S.4-2. Subsurface conditions substantially different from those described here are not anticipated as a result of this supplemental subsurface investigation. Following completion of this confirmatory investigation, STP will update the FSAR in accordance with 10 CFR 50.71(e) (COM 2.5S-2).

2.5S.4.2.1.16 Laboratory Testing Program

Planning for laboratory testing made as a part of this subsurface investigation referred to guidance provided in RG 1.138 (Reference 2.5S.4-20). References to industry standards used for laboratory testing are shown in Table 2.5S.4-7. Laboratory testing details and results are provided in Reference 2.5S.4-2 (Appendix 2.5-A). The work was performed under an approved quality assurance program with work procedures developed specifically for STP 3 & 4, including a subsurface investigation plan developed by Bechtel. Soil samples collected were shipped under chain-of-custody from the onsite storage area to the testing laboratories. Laboratory testing was performed at several laboratories in the following cities: Atlanta, Georgia (MACTEC); Charlotte, North Carolina (MACTEC); Phoenix, Arizona (MACTEC); St. Louis, Missouri (Severn Trent Laboratories); Houston, Texas (Fugro); and Austin, Texas (University of Texas - Austin Soils Laboratory). Both the Fugro and the University of Texas - Austin laboratories are currently performing Resonant Column Torsional Shear (RCTS) testing, with complete results available at a later date (refer to the statement on COM 2.5S-1 at Subsection 2.5S.4.7).

Note that a brief review of five recently-available (late July 2007) laboratory RCTS tests results is made in Subsection 2.5S.4.7.3.3. All other laboratories have completed their testing, with results included in Reference 2.5S.4-2 (Appendix 2.5-A). The supplemental subsurface investigation described in Subsection 2.5S.4.2.1.15 also includes limited laboratory testing of selected soils samples recovered (refer to the statement on COM 2.5S-2 under Subsection

2.5S.4.2.1.15). The laboratory testing program reported on here is discussed further in Subsection 2.5S.4.2.3.

2.5S.4.2.2 Exploration

Subsection 2.5S.4.2.2.1 describes the previous subsurface investigation performed for STP 1 & 2. Subsection 2.5S.4.2.2.2 describes the subsurface investigation performed for STP 3 & 4, reported on here.

2.5S.4.2.2.1 Previous Subsurface Investigations (STP 1 & 2)

Based on information available from the STP 1 & 2 UFSAR (Reference 2.5S.4-3), the subsurface investigations for STP 1 & 2 were performed from approximately 1974 to 1985, and consisted of a total of 157 exploratory borings, ranging in depth from 6 feet to approximately 2620 feet below ground surface. Soil samples were obtained at regular intervals for soil identification and testing. Piezometers were installed for groundwater observation and monitoring. In addition, static Dutch cone penetration tests were completed adjacent to selected borings. Soil laboratory testing included moisture content, Atterberg limits, sieve analysis, specific gravity, dry unit weight, bulk unit weight, UU triaxial and UNC strength testing, consolidation, swell potential, permeability, moisture-density (Proctor compaction), cyclic triaxial testing, cyclic torsional testing, and mineralogy.

Geologic data were gathered by drilling one deep boring (B-233) with associated Paleomagnetic sampling and analysis and performing trench excavations, remote sensing, field surface inspection and mapping, and construction-phase excavation and mapping.

Geophysical data were gathered using seismic cross-hole surveys, seismic refraction surveys, seismic reflection surveys, and borehole logging.

Site stratigraphy at depth was additionally investigated by a review of deep oil well logs at locations in the vicinity of the STP site. These found undifferentiated Pleistocene deposits, including the upper Beaumont Formation, extending to approximately 2800 feet below ground surface.

2.5S.4.2.2.2 Current Subsurface Investigation (STP 3 & 4)

RG 1.132 (Reference 2.5S.4-19) 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. To accommodate the need for subsurface investigations to be site specific, Reference 2.5S.4-19 recognizes the requirement 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.

Reference 2.5S.4-19 also provides guidance on spacing and depths of borings, sampling procedures, insitu testing procedures, and geophysical investigation methods. This guidance was used in preparing a technical specification, addressing the basis for the STP 3 & 4

subsurface investigation. The quantity of borings and CPTs for major structures (including Seismic Category I structures and/or piping) was based on a minimum of one boring per structure and one boring per 10,000-square feet of structure plan area. Reference 2.5S.4-19 recommends that borings for Seismic Category I structures extend to a depth approximately equal to the width of the structure below the planned foundation level. This criterion was met for the two deep borings (B-305DH/DHA and B-405DH) made at the centers of the Reactor Buildings (each approximately 190 feet wide, on average, with planned foundation level at approximately 85 feet below ground surface), each of which was advanced to approximately 600 feet below ground surface. At each Reactor Building, eight additional borings were made to approximately 200 feet depth below ground surface. These borings were terminated in either dense sands or stiff to very stiff clays that, from a review of STP 1 & 2 data and the completed 600 foot deep borings, become stronger with increasing depth.

The sampling intervals employed in the borings varied slightly from the guidance document recommendations, but were in accordance with the subsurface investigation technical specifications. Sample spacing in the uppermost 15 feet was shortened at each boring, with typically 10 SPT samples collected over that depth. For SPT sampling five-foot sample intervals were maintained to a depth of 100 feet, 10-foot sample intervals were maintained to a depth of 200 feet and, 20-foot sample intervals were maintained to the maximum depth of approximately 600 feet below ground surface. In most cases, additional undisturbed samples were obtained, especially between the 20-foot sample intervals at the two deep borings (B-305DH/DHA and B-405DH). Continuous sampling was also performed, as described later. CPTs obtained continuous data to a maximum depth of approximately 100 feet below ground surface.

Subsection 4.3.1.2 of Reference 2.5S.4-19, "Drilling Procedures," states that borings with depths greater than approximately 100 feet should be surveyed for deviation. Deviation surveys were conducted in the 10 suspension P-S velocity logging borings, including the two deep borings (B-305DH/DHA and B-405DH) in accordance with the subsurface investigation technical specifications. Per conventional investigation practice, deviation surveys for other borings were neither called for in the technical specifications nor performed. It should be noted that all borings and field testing points were advanced as vertical as possible by starting the drilling rigs/field testing equipment in a level position and by regularly observing the verticality of the drilling rig masts, the drilling rods, etc., as the work progressed.

Subsection 4.3.2 of Reference 2.5S.4-19, "Sampling," states that color photographs of all cores should be taken soon after removal from the boring to document the condition of the soils at the time of drilling. Undisturbed soil samples are sealed in metal tubes, and cannot be photographed. SPT soil samples are disturbed and, as a result, do not resemble the condition of the material insitu. Sample photography is a practice typically limited to rock core, rather than soil samples, and therefore, was not employed. This was in accordance with the subsurface investigation technical specification. X-ray imaging, however, has been performed on undisturbed samples selected for RCTS testing.

The STP 3 & 4 subsurface investigation was performed onsite between October 2006 and January 2007. This work consisted of an extensive investigation to define the subsurface conditions at the site. The field testing locations are shown on Figures 2.5S.4-1 and 2.5S.4-2. The scope of work and investigation methods used by the subsurface investigation

subcontractor, MACTEC Engineering and Consulting, Inc. (MACTEC) and its subcontractors, were as follows:

- Surveying to establish the horizontal coordinates and vertical elevations of field testing locations
- Evaluating the potential presence of underground utilities at field testing locations
- Drilling 120 borings with SPT sampling and collecting in excess of 200 undisturbed samples (using the Shelby push sampler or the Pitcher sampler depending on the material) to a maximum depth of approximately 600 feet below ground surface, including two borings with continuous SPT sampling (B-322C and B-422C) each made to 100 feet below ground surface. Note that "continuous sampling" was defined as one SPT sample for every 2.5 feet of boring depth, with a one foot interval between each SPT sample
- Performing 32 CPTs, including five seismic CPTs to a maximum depth of approximately 100 feet below ground surface, including making pore water pressure dissipation measurements at selected depths in 10 CPTs
- Excavating six test pits to a maximum depth of approximately 9 feet below ground surface, and collecting bulk soil samples
- Installing and developing 28 groundwater observation wells to a maximum depth of approximately 121 feet below ground surface, including slug testing each well for the determination of insitu permeability
- Performing borehole geophysical logging, consisting of suspension P-S velocity logging, natural gamma, long and short resistivity, spontaneous potential, three-arm caliper, and deviation survey for the 10 logging borings
- Conducting field electrical resistivity testing along four arrays (each array consisting of two orthogonal survey lines)
- Conducting SPT hammer energy measurements for each of the 11 drilling rigs employed
- Performing laboratory testing of soils, consisting of moisture content, Atterberg limits, sieve and hydrometer analysis, specific gravity, unit weight, UU triaxial and UNC strength testing, CIU-bar triaxial strength testing, direct shear strength testing, consolidation, moisture-density (Proctor compaction), California Bearing Ratio (CBR), and chemical analyses (pH, sulfate content, and chloride content). RCTS testing was also commissioned, with testing currently underway, and with complete results reported at a later date (refer to the statement on COM 2.5S-1 at Subsection 2.5S.4.7).
- Performing laboratory testing on groundwater samples obtained from the observation wells, including pH, conductivity, dissolved oxygen, alkalinity, ammonia, nitrogen, bromide, chloride, dissolved solids, fluoride, nitrate as N, nitrite as N, sulfate, and sulfide, including cation exchange testing on soils in the well screen area. These results are discussed in Subsection 2.4S.12

As noted earlier, the STP 3 & 4 subsurface investigation was performed according to guidelines outlined in Reference 2.5S.4-19. The field work was performed under an audited and approved quality assurance program and work procedures developed specifically for STP 3 & 4. The subsurface investigation and sample collection were directed by the MACTEC site manager, who was onsite full-time during the investigation period. MACTEC's designated project quality assurance/quality control manager made periodic visits to the site to audit their work and that of their subcontractors. A Bechtel geotechnical engineer and/or geologist, along with a representative of STPNOC, were also onsite during the field work. Additionally, field boring logs, well logs, test pit logs, and hydraulic conductivity logs were prepared by MACTEC engineers or geologists who oversaw the entire subsurface investigation on a full-time basis. A visit to the STP site during the subsurface investigation work was also made by NRC in early December 2006.

Each field testing location was checked for the presence of underground utilities prior to commencing work at that location. The locations of several field testing points were revised due to their proximity to utilities or their inaccessibility as a result of wet conditions. The ground occupied by each drilling or CPT rig was temporarily covered with plastic sheeting to prevent accidental release of hydraulic fluid onto the ground.

An onsite storage facility for soil sample retention was established before the subsurface investigation commenced. Each sample was logged into an inventory system. Samples removed from the facility were noted in the inventory log book. 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 (Reference 2.5S.4-21).

Complete results of the subsurface investigation are in Reference 2.5S.4-2 (Appendix 2.5-A). Additional details related to field testing activities, including borings, CPTs, observation wells and slug testing, test pits, field electrical resistivity testing, geophysical logging, etc., are summarized below.

2.5S.4.2.2.3 Boring and Sampling

Borings were advanced using mud-rotary drilling methods, with hollow-stem augers used in the upper portions of some borings, as noted on the boring logs. Drilling mud was a mixture of water and bentonite. Clean water, obtained from the site water supply was used for drilling. Eleven drilling rigs were used to advance the borings, including, both truck-mounted and all-terrain vehicle (ATV) rigs. The make and model of each rig is given in Table 2.5S.4-4. Each rig was equipped with an automatic SPT hammer.

Soils were sampled using a standard SPT sampler, in accordance with ASTM D 1586 (Reference 2.5S.4-22). Soils were sampled at continuous intervals (one sample every 2.5-feet of boring depth) to approximately 15 feet below ground surface. Subsequent SPT sampling was performed at regular 5-foot intervals to a depth of approximately 100 feet below ground surface. From depths of approximately 100 feet to 200 feet below ground surface SPT samples were obtained at 10-foot intervals, and finally, from depths of approximately 200 feet to 600 feet below ground surface, SPT samples were obtained at 20-foot intervals. The recovered soil samples were visually described and classified by the rig engineer or geologist in accordance with ASTM D 2488 (Reference 2.5S.4-23). A representative portion of the SPT sample was placed in a glass jar with a moisture-preserving lid. The sample jars were labeled, placed in boxes, and transported to the onsite storage facility. Table 2.5S.4-19 provides a summary of as-built boring locations and other details. Boring locations are shown on Figures 2.5S.4-1 and 2.5S.4-2. Boring logs are included with Reference 2.5S.4-2 (Appendix 2.5-A). Upon completion, each boring was tremie-grouted back to the ground surface using a cement-bentonite grout.

Undisturbed three-inch-diameter tube samples were also obtained, in accordance with ASTM D 1587 (Reference 2.5S.4-24), using either a Shelby push sampler or a rotary Pitcher sampler, depending on the material being sampled. Upon sample retrieval, any disturbed materials at the ends of the sample were removed, the ends were trimmed square to establish an effective seal, and for fine-grained cohesive soils a pocket penetrometer (PP) measurement was taken on the trimmed lower end of the sample. Both ends of the sample tube were then sealed with hot wax, covered with plastic caps, and sealed once again using electrical tape and wax. The sample tubes were labeled and transported to the onsite storage area. Table 2.5S.4-20 provides a summary of undisturbed soil samples collected as part of the subsurface investigation. Undisturbed samples are also identified on the boring logs included in Reference 2.5S.4-2 (Appendix 2.5-A).

Energy measurements were made on the SPT hammer-rod systems on each of the 11 drilling rigs employed in the subsurface investigation. A PAK model Pile Driving Analyzer (PDA) was used to acquire and process the data. A summary of the measured hammer energies and related data is provided in Table 2.5S.4-4. Between three and five hammer energy measurements were made at each drilling rig. Energy transfer to the PDA gauge positions was estimated using the Case Method, in accordance with ASTM D 4633 (Reference 2.5S.4-25). The average energy transfer ratios measured at each drilling rig ranged from 72% to 99%. Detailed results of this testing are presented in Reference 2.5S.4-2 (Appendix 2.5-A).

2.5S.4.2.2.4 Cone Penetration Testing

CPTs were advanced using an electronic seismic piezocone compression model with a 15 cm^2 tip area and a 225 cm^2 friction sleeve area. CPTs were performed in accordance with ASTM D 5778 (Reference 2.5S.4-26). The CPT equipment was mounted on a 15-ton track-mounted rig which was dedicated to the CPT work. Cone tip resistance, sleeve friction, and dynamic pore pressure were recorded every 5 centimeters (approximately every 2 inches) as the cone was advanced into the ground. Shear wave velocity measurements were also made at selected CPTs using a geophone mounted above the cone and a digital oscilloscope. An anchored beam struck at the ground surface with a sledge hammer served as the vibration source. Pore pressure dissipation data were also obtained in selected CPTs, with the data recorded at 5 second intervals.

Thirty-two CPTs were performed, with termination depths ranged from approximately 36 feet to 100 feet below ground surface, including five seismic CPTs (C-305S, C-306S, C-307S, C-406S, and C-407S). Pore pressure dissipation tests were performed at 10 CPTs, and at 19 depths. Table 2.5S.4-21 provides a summary of as-built CPT locations and other details. CPT locations are shown on Figures 2.5S.4-1 and 2.5S.4-2. CPT logs, shear wave velocity measurements, and pore pressure dissipation test results are included in Reference 2.5S.4-2/ Appendix 2.5-A.

2.5S.4.2.2.5 Observation Wells and Slug Testing

Twenty-eight observation wells were installed, with well depths ranging from approximately 36 feet to 121 feet below ground surface. Observation wells were installed under the full-time supervision of a geotechnical engineer and/or geologist either in sampled borings or in offset borings, with installation in accordance with ASTM D 5092 (Reference 2.5S.4-27). For observation wells installed in sampled borings, the borings were grouted to the base level of the well, and the portion above was reamed to a diameter of at least 6 inches using rotary methods and a biodegradable drilling fluid. Observation wells installed at offset locations were installed in borings made using the rotary drilling method and biodegradeable drilling fluid (one observation well was installed using a hollow stem auger), with an effective well diameter of 8 inches. Each well was developed by pumping and/or flushing with clean water. Table 2.5S.4-22 provides a summary of as-built observation well locations and other details. Observation well locations are shown on Figures 2.5S.4-1 and 2.5S.4-2. Complete observation well details are included in Reference 2.5S.4-2 (Appendix 2.5-A), and are discussed further in Subsection 2.4S.12.

Slug testing, for the purpose of measuring the insitu hydraulic conductivity of soil strata, was performed in all 28 observation wells. Slug tests were conducted using the falling head method, in accordance with Section 8 of ASTM D 4044 (Reference 2.5S.4-28). Slug testing included establishing the static water level, lowering a solid cylinder (slug) into the well to cause an increase in water level in the well, and monitoring the time rate for the well water to return to the pre-test static level. Electronic transducers and data loggers were used to measure the water levels and times during the test. Table 2.5S.4-23 provides a summary of the hydraulic conductivity values resulting. Complete slug testing details are provided with Reference 2.5S.4-2/Appendix 2.5A, and are discussed further in Subsection 2.4S.12.

2.5S.4.2.2.6 Test Pits

Six test pits were excavated to a maximum depth of approximately 9 feet below ground surface, each using a mechanical excavator. Bulk samples were collected at selected soil horizons in the test pits for laboratory testing. A summary of test pits completed and bulk soil samples collected is included in Table 2.5S.4-24. Test pits were made adjacent to selected borings and CPTs, as noted in the test pit number. For example, Test Pit TP-B322C was made adjacent to Boring B-322C. Reference 2.5S.4-2 (Appendix 2.5-A) contains test pit records and other details.

2.5S.4.2.2.7 Field Electrical Resistivity Testing

Four field electrical resistivity tests were performed to obtain apparent resistivity values of the site soils. Table 2.5S.4-25 provides a summary of the as-built field electrical resistivity test locations and other details. Field electrical resistivity testing was conducted using a MiniRes HP earth resistivity meter, a Wenner four-electrode array, and “a” spacings of 3 feet, 5 feet, 7.5 feet, 10 feet, 15 feet, 30 feet, 50 feet, 100 feet, 200 feet, and 300 feet, in accordance with ASTM G 57 (Reference 2.5S.4-29) and IEEE 81 (Reference 2.5S.4-30). The arrays were centered on each of the staked locations, namely ER-301, ER-401, ER-901, and ER-902, as shown on Figures 2.5S.4-1 and 2.5S.4-2. The electrodes were positioned using a 300-foot measuring tape along the appropriate bearings using a Brunton compass. Field electrical resistivity test results are summarized in Table 2.5S.4-17. The raw field electrical resistivity test data are provided in Reference 2.5S.4-2 (Appendix 2.5-A).

2.5S.4.2.2.8 Geophysical Logging Including Suspension P-S Velocity Logging

Geophysical logging consisted of suspension P-S velocity logging, natural gamma, long and short resistivity, spontaneous potential, three-arm caliper, and deviation surveys for the 10 logging borings. Detailed geophysical logging results are provided in Reference 2.5S.4-2 (Appendix 2.5-A). Suspension P-S velocity logging results are discussed further in Subsection 2.5S.4.4.

2.5S.4.2.3 Laboratory Testing

As noted above, RG 1.138 (Reference 2.5S.4-20) addresses laboratory testing of soil and rock for nuclear power plants. This guidance document describes the requirements for laboratory equipment (including calibration), handling and storage of samples, selection and preparation of test specimens, and testing procedures for determining static and dynamic soil and rock properties. The laboratory tests listed in Reference 2.5S.4-20 are common tests performed in most well-equipped soil and rock testing laboratories, and are covered by ASTM and related standards. Some tests not covered in Reference 2.5S.4-20 were also performed for the STP 3 & 4 subsurface investigation (e.g., the state-of-the-art RCTS testing method was used in lieu of resonant column tests and/or cyclic triaxial tests to obtain shear modulus degradation and damping ratios over a range of strains).

Reference 2.5S.4-20 does not provide specific guidance on the quantity of laboratory tests to conduct. The number of laboratory tests made for the STP 3 & 4 subsurface investigation was based on engineering judgment, and on experience with similar projects, to obtain necessary data for characterizing engineering properties of materials that impact ground stability and the

suitability of construction for critical foundations. An initial laboratory testing assignment was based on information developed from the subsurface investigation, such as the numbers and positions of soil strata, their thicknesses, strengths, vertical and lateral uniformity, relevance to planned foundations, and knowledge of planned construction at the time, followed by supplementary testing assignments to fill data gaps and/or to confirm previous test data.

ASTM D 4220 (Reference 2.5S.4-21) provides guidance on standard practices for preserving and transporting soil samples. This guidance was referenced in preparing the technical specifications for the STP 3 & 4 subsurface investigation work.

Laboratory testing for the STP 3 & 4 subsurface investigation included testing of soil and groundwater samples recovered from the field testing points (e.g., borings, observation wells, test pits, etc.). Laboratory testing of groundwater samples is addressed in Subsection 2.4S.12. Laboratory testing of soil samples consisted of index and engineering property tests on selected SPT, undisturbed, and bulk soil samples. SPT and undisturbed soil samples were recovered from borings. Bulk soil samples were recovered from test pits. Laboratory testing on recovered soils samples included: moisture content, Atterberg limits, sieve and hydrometer analysis, specific gravity, unit weight, UU triaxial and UNC strength testing, CIU-bar triaxial strength testing, direct shear strength testing, consolidation, moisture-density (Proctor compaction), CBR, and chemical analyses (pH, chloride content, and sulfate content). RCTS testing was also commissioned, with testing currently underway, with complete results reported at a later date (refer to the statement on COM 2.5S-1 at Subsection 2.5S.4.7).

Laboratory tests were performed in accordance with the following standards:

- Identification and Index Testing
  - Unified Soil Classification System (USCS) – ASTM D 2487 (Reference 2.5S.4-31) and ASTM D 2488 (Reference 2.5S.4-23)
  - Moisture Content – ASTM D 2216 (Reference 2.5S.4-32)
  - Atterberg Limits – ASTM D 4318 (Reference 2.5S.4-33)
  - Sieve and Hydrometer Analysis – ASTM D 422 (Reference 2.5S.4-34) and ASTM D 6913 (Reference 2.5S.4-35)
  - Specific Gravity – ASTM D 854 (Reference 2.5S.4-36)
  - Unit Weight – measured (included as a part of related ASTM standards)
- Strength Testing
  - Unconsolidated-Undrained Triaxial Compression – ASTM D 2850 (Reference 2.5S.4-37)
  - Unconfined Compression – ASTM D 2166 (Reference 2.5S.4-38)
  - Consolidated-Undrained Triaxial Compression – ASTM D 4767 (Reference 2.5S.4-39)

  - Direct Shear – ASTM D 3080 (Reference 2.5S.4-40)
- Compressibility Testing
  - Consolidation – ASTM D 2435 (Reference 2.5S.4-41)
- Compaction and Related Testing
  - Moisture-Density Relationship – ASTM D 1557 (Reference 2.5S.4-42)
  - California Bearing Ratio – ASTM D 1883 (Reference 2.5S.4-43)
- Chemical Testing – Soils
  - pH – ASTM D 4972 (Reference 2.5S.4-44)
  - Chloride Content – EPA 300.0 (Reference 2.5S.4-45)
  - Sulfate Content – EPA 300.0 (Reference 2.5S.4-45)
  - RCTS Testing – Stokoe, et al. (Reference 2.5S.4-46)

2.5S.4.3 Foundation Interfaces

The following site-specific supplement addresses COL License Information Item 2.30.

Subsurface profiles depicting the inferred subsurface stratigraphy are presented on Figures 2.5S.4-5 through 2.5S.4-9. A subsurface profile legend is Figure 2.5S.4-3, and subsurface profile locations are shown on Figure 2.5S.4-4. Note that subsurface profiles shown on Figures 2.5S.4-5 and 2.5S.4-6 illustrate typical conditions in the STP 3 area, subsurface profiles shown on Figures 2.5S.4-7 and 2.5S.4-8 illustrate typical conditions in the STP 4 area, and the subsurface profile shown on Figure 2.5S.4-9 illustrates typical conditions in the UHS Basin/RSW area.

Profiles illustrating the planned foundation excavation geometries and the locations and depths of STP 3 & 4 major structures (including Seismic Category I structures), as well as the relationship of planned structure foundations with the various subsurface strata, are addressed in Subsection 2.5S.4.5.

2.5S.4.4 Geophysical Surveys

The following site-specific supplement addresses, in part, COL License Information Item 2.34. Refer to Subsection 2.5S.4.7 for additional discussion.

This Subsection provides a summary of the geophysical surveys undertaken at the SPT site. Subsection 2.5S.4.4.1 summarizes previous geophysical surveys made for the STP 1 & 2 subsurface investigations. Subsection 2.5S.4.4.2 summarizes geophysical surveys made as a part of the STP 3 & 4 subsurface investigation.

2.5S.4.4.1 Previous Geophysical Surveys for STP 1 & 2

Various geophysical methods were employed during the original subsurface investigations made for STP 1 & 2. These investigations are addressed in detail in the STP 1 & 2 UFSAR (Reference 2.5S.4-3). A brief summary of geophysical survey methods employed, as reproduced from the STP 1 & 2 UFSAR (Reference2.5S.4-3), is below.

2.5S.4.4.1.1 Seismic Cross-Hole Measurements

Shear wave velocity measurements were obtained initially in late 1973 by cross-hole method at two locations, one each in the STP 1 and STP 2 areas, with measurements completed to depths of 280 feet and 298 feet, respectively, at depth intervals ranging from 5 feet to 40 feet.

In mid-1974, additional cross-hole measurements were completed at both the STP 1 and STP 2 areas to depths of 305 feet, at depth intervals of 5 feet. A plot summary of these results is provided on Figure 2.5S.4-39. Shear wave velocity measurements for depths greater than 305 feet were not obtained.

2.5S.4.4.1.2 Geophysical Refraction Surveys

Refraction measurements were completed for the PSAR through the future location of the center of the STP 1 and STP 2 reactors, oriented in both north-south and east-west directions. A series of geophones was placed at either 50 feet or 100 feet spacing. Explosive charges were set at distances from 50 feet to 250 feet from the end geophone, and served as the vibration source. Compressional wave velocity was estimated from the inverse of the arrival time plots obtained during measurement. From the results, compressional wave velocities were judged to be consistent to a depth of 400 feet. Two distinct compressional wave velocity layers were identified (a) a 5500 feet/second layer extending to depth ranging from 60 feet to 100 feet beneath the surface and (b) an underlying 6000 feet/second layer. Also a thin upper layer of compressional velocity less than 5000 feet/second was observed, indicative of soils above the water table.

2.5S.4.4.1.3 Geophysical Reflection Surveys

In late 1985, approximately 98.5 miles of existing reflection records and several geophysical well logs were assessed. Based on the data review, eight seismic stratigraphic cross-sections were developed. These results are available in the STP 1 & 2 UFSAR (Reference 2.5S.4-3, Subsection 2.5.1.2.5.4).

2.5S.4.4.1.4 Geophysical Borehole Logging

Geophysical logging of selected STP 1 & 2 geotechnical borings was performed. Also, a review was made of oil and gas well geophysical logs obtained in the vicinity of the STP site.

Data collected during geotechnical boring logging included: electrical resistivity, self (spontaneous) potential, and gamma ray. Data collected was interpreted to develop subsurface stratigraphy for STP 1 & 2.

2.5S.4.4.2 Geophysical Survey for STP 3 & 4

Suspension P-S velocity logging and seismic CPT tests were performed at 10 borings and five CPTs, respectively, as a part of the STP 3 & 4 subsurface investigation. The results are discussed below.

2.5S.4.4.2.1 Suspension P-S Velocity Logging

Suspension P-S velocity logging was performed at 10 borings (B-302DH, B-305DH/DHA, B-308DH, B-319DH, B-328DH, B-402DH, B-405DH, B-408DH, B-419DH, and B-428DH). Borings were uncased and filled with drilling fluid. Borings B-305DH/DHA and B-405DH were logged to approximately 470 feet and 600 feet below ground surface, respectively, while the remaining borings were logged to approximately 200 feet depth each. The OYO/Robertson Model 3403 unit and the OYO Model 170 suspension logging recorder and probe were employed. Details of the equipment are described in Reference 2.5S.4-47. The velocity measurement technique used for the STP 3 & 4 work is briefly described below. The results are provided as tables and graphs in Reference 2.5S.4-2 (Appendix 2.5-A).

At the time of this subsurface investigation, an ASTM standard was not available for the suspension P-S velocity logging method, therefore, a brief description follows here. Suspension P-S velocity logging uses a 23-foot-(7-meter-) long probe containing a source near the bottom, and two geophone receivers spaced 3.3 feet (1 meter) apart, suspended by a cable. The probe is lowered into the boring to a specified depth where the source generates a pressure wave in the boring fluid (drilling mud). The pressure wave is converted to seismic waves (compressional/"P"-waves, and shear/"S"-waves) at the boring wall. At each receiver position, the P- and S-waves are converted to pressure waves in the fluid and received by the geophones mounted in the probe, which in turn send the data to a recorder on the surface. At each measurement depth, two opposite horizontal records and one vertical record are obtained. This procedure is typically repeated every 1.6 feet (0.5 meter) or 3.3 feet (1 meter) as the probe is moved from the bottom of the boring towards the ground surface. The elapsed times between wave arrivals at the geophone receivers is used to determine the average velocity of a 3.3-feet-(1-meter) high column of soil around the boring. For quality assurance analysis is also performed on source-to-receiver data.

P-S velocity measurements obtained were sorted by soil stratum through a review of the stratigraphic changes on the boring logs, and a review of the geophysical logs for depths where soil samples were collected less frequently (i.e., especially for the deepest Stratum N).

Compressional wave velocity (V_p) and shear wave velocity (V_s) results from the STP 3 & 4 subsurface investigation, including results from both the suspension P-S velocity logging method and from the seismic CPT method, are discussed further here.

Minimum, maximum, and average V_p measurements obtained in the various soil strata from the STP 3 & 4 subsurface investigation were as follows:

Stratum	STP 3 & 4 Minimum V_p (feet/second)	STP 3 & 4 Maximum V_p (feet/second)	STP 3 & 4 Average V_p (feet/second)
A	790	5,560	2,644
B	1,180	5,560	4,631
C	2,980	6,010	5,112
D	4,660	6,170	5,511
E	4,220	6,350	5,527
F	5,050	6,060	5,540
H	4,730	7,840	5,669
J Clay	4,980	6,800	5,632
J Sand	5,130	7,250	5,699
K Clay	5,050	6,170	5,596
K Sand/Silt	5,170	6,170	5,601
L	5,210	5,750	5,388
M	5,010	5,700	5,364
N Clay	5,050	6,410	5,712
N Sand	5,210	6,600	5,853

Minimum, maximum, and average V_s measurements obtained in the various soil strata from the STP 3 & 4 subsurface investigation, and the STP 1 & 2 subsurface investigation (as noted) were as follows:

Stratum	STP 3 & 4 Minimum V_s (feet/second)	STP 3 & 4 Maximum V_s (feet/second)	STP 3 & 4 Average V_s (feet/second)	STP 1 & 2 [1] Average V_s (feet/second)
A	290	1,000	559	663
B	400	1,090	719	905
C	440	1,430	776	910
D	540	1,550	937	1,030
E	720	1,430	1,072	1,155
F	720	1,280	947	1,316
H	730	2,190	1,061	1,560
J Clay	640	1,880	1,089	1,201
J Sand	720	3,210	1,275	1,201
K Clay	730	1,650	1,170	1,541
K Sand/Silt	940	2,010	1,371	1,541
L	750	1,410	979	1,271
M	800	1,600	1,165	1,520
N Clay	700	2,540	1,296	1,324
N Sand	870	2,430	1,654	1,585

[1] Values taken from the STP 1 & 2 UFSAR (Reference 2.5S.4-3), Table 2.5S.4-27. For strata A, B, D, E, F, and J, the values shown above are the average of the sub-stratum values provided in the referenced table.

Figures 2.5S.4-40 and 2.5S.4-41 illustrate V_s measurements at the STP 3 area and at the STP 4, respectively, to depths of approximately 200 feet below ground surface. Figure 2.5S.4-42 illustrates V_s measurements at both the STP 3 area and at the STP 4 area to depths of approximately 200 feet to 600 feet below ground surface.

Note that V_s results consistently are slightly higher for STP 1 & 2 (refer especially to Figure 2.5S.4-39), than for STP 3 & 4, with the exception of Sub-Strata J Sand, N Clay, and N Sand. Average V_s results for STP 1 & 2 compared to STP 3 & 4, are shown versus depth on Figures 2.5S.4-43 and 2.5S.4-44.

Based on all 10 suspension P-S velocity logging locations and all five seismic CPT locations, an average V_s profile was developed for the upper approximately 600 feet at STP 3 & 4, as shown on Figures 2.5S.4-45 through 2.5S.4-47. Note that Figure 2.5S.4-45 illustrates Strata A through J. Figure 2.5S.4-46 illustrates Strata J through Stratum N. Figure 2.5S.4-47 illustrates the lower reaches of Stratum N to a depth of approximately 600 feet below ground surface.

Poisson's ratio (μ) values were determined based on the V_p and the V_s measurements. Overall, average Poisson's ratios were approximately 0.42 at depths above the groundwater level (El.

25.5 feet) and approximately 0.47 at depths below the groundwater level. Poisson's ratio results are summarized below. In general, Poisson's ratio results from the STP 3 & 4 subsurface investigation, by geophysical methods (i.e., small strain) are higher than those reported in the STP 1 & 2 UFSAR (Reference 2.5S.4-3), albeit for large strain.

Stratum	STP 3 & 4 Minimum μ (small strain)	STP 3 & 4 Maximum μ (small strain)	STP 3 & 4 Average μ (small strain)	STP 1 & 2 [1] Average μ (large strain)
A	0.29	0.50	0.45	0.42
B	0.32	0.49	0.48	0.42
C	0.45	0.50	0.49	0.35
D	0.47	0.49	0.48	0.42
E	0.47	0.49	0.48	0.35
F	0.47	0.49	0.48	0.42
H	0.44	0.49	0.48	0.35
J Clay	0.45	0.49	0.48	0.42
J Sand	0.38	0.49	0.47	0.42
K Clay	0.46	0.49	0.48	0.35
K Sand/Silt	0.44	0.48	0.47	0.35
L	0.47	0.49	0.48	0.42
M	0.45	0.49	0.47	0.35
N Clay	0.40	0.49	0.47	0.42
N Sand	0.41	0.49	0.46	0.35

[1] Values taken from the STP 1 & 2 UFSAR (Reference 2.5S.4-3), Table 2.5S.4-27. As noted in Subsection 2.5.4.7.2.3 of Reference 2.5S.4-3, these values were taken from published typical values and were not calculated from site-specific Vp and Vs measurements.

Note that the above V_p, V_s, and μ values (at small strain) can be assumed to reflect the STP 3 & 4 subsurface profile to a depth of approximately 600 feet below ground surface (i.e., to approximately El. -570 feet). Information on deeper subsurface soils is discussed in Subsection 2.5S.4.7.

2.5S.4.4.2.2 Seismic CPT Measurements

Shear wave velocity measurements were made using a seismic CPT at five locations, namely CPT C-305S, C-306S, C-307S, C-406S, and C-407S. The maximum depth tested by the seismic CPTs was approximately 95 feet below ground surface. As noted above, seismic CPT V_s results are included together with the suspension P-S velocity logging V_s results on Figures 2.5S.4-40 and 2.5S.4-41,and are typically within the range of the suspension P-S velocity logging results. Seismic CPT V_s results are summarized below. Individual seismic CPT V_s results are included in Reference 2.5S.4-2 (Appendix 2.5-A).

Stratum	STP 3 & 4 Minimum V_S (feet/second)	STP 3 & 4 Maximum V_S (feet/second)	STP 3 & 4 Average V_S (feet/second)
A	428	1,078	701
B	595	910	788
C	763	1,006	865
D	631	1,331	886
E	760	1,627	1,172
F	844	1,043	960
H	983	1,814	1,274

2.5S.4.4.2.3 Shear Wave Velocity Profile Selection

Suspension P-S velocity logging results and seismic CPT measurements were combined for the development of a shear wave velocity profile from ground surface to a depth of approximately 600 feet below ground surface. The data collected at the individual suspension P-S velocity logging borings and collected at the seismic CPT locations were sorted by soil strata. The average thicknesses of individual soil strata were determined at each of the test locations, and the collected data were sorted to fit average strata boundaries (refer to Subsection 2.5S.4.7.2.1 for additional detail).

As noted above, the average STP 3 & 4 V_S profiles are illustrated on Figures 2.5S.4-45, through 2.5S.4-47. Further discussion on these, and on a V_S profile for STP 3 & 4 site soils below approximately 600 feet below ground surface, is provided in Subsection 2.5S.4.7.

Weighted average shear wave velocities were calculated for both the STP 3 and the STP 4 Reactor Buildings to depths of two times the largest foundation dimension. Shear wave velocity values of the insitu soils were considered between El. -61 feet (the level of Reactor Building over-excavation) and El. -443 feet (two times the maximum Reactor Building foundation dimension of 196 feet, or 392 feet below the Reactor Building underside of foundation at El. -51 feet. The shear wave velocity of the concrete fill between the underside of foundation (El. -51) and the over-excavation level (El. -61 feet) was not included in the weighted averages. The resulting weighted average shear wave velocities were 1250 feet/second for the STP 3 Reactor Building and 1243 feet/second for the STP 4 Reactor Building.

2.5S.4.5 Excavations and Backfill

The following site-specific supplement addresses COL License Information Items 2.31 and 2.39.

2.5S.4.5.1 Source and Quantity of Backfill and Borrow

A significant amount of earthwork is anticipated in order to establish the rough grades at the site and to provide for the embedment of major structures (including Seismic Category I structures). Current estimates are that approximately 3.56 million cubic yards of materials are moved during earthworks to establish site grade inside the STP 3 & 4 Power Block area, comprising 2.125 million cubic yards of excavation and 1.435 million cubic yards of structural fill.

The materials excavated as part of the site grading are primarily the upper soils belonging to Strata A through F, consisting mostly of clays (Strata A, D, and F), silts (Stratum B), and fine sands (Strata C and E). To evaluate the uppermost soil stratum (Stratum A) for construction purposes, six test pits were excavated at STP 3 & 4, as shown on Figure 2.5S.4.2 and summarized in Table 2.5S.4-24. The maximum depth of test pits was limited to approximately 9 feet below ground surface. The results of laboratory testing on bulk samples collected from the test pits for moisture-density (Proctor compaction), CBR, and other index tests are summarized in Table 2.5S.4-26, with details included in Reference 2.5S.4-2 (Appendix 2.5-A). These tests indicated that the Stratum A soils had high plasticity and with an average fines content of 94%, and occurred at natural moisture contents, on average, approximately 10% to 13% above their optimum moisture contents. This material (Stratum A), as well as other upper clay and/or silt strata excavated (i.e., Strata B, D, and F), in their natural states are unsuitable for use as structural fill, and have limited suitability for reuse as common fill. Upper sand strata excavated (i.e., Strata C and E) are unsuitable for use as structural fill, but are suitable for reuse as common fill provided they are adequately separated from the clay and/or silt strata during excavation and provided they are adequately dried-back prior to placement in fill areas. Note that the upper sand strata (Strata C and E), both of which occur below the normal groundwater table, have natural moisture contents in a similar range to those measured for the tested Stratum A bulk samples, which may similarly be higher than their respective optimum moisture contents.

Given the state of the current knowledge regarding the soils excavated, and the past experience in constructing STP 1 & 2, it is expected that the bulk of the estimated 1.435 million cubic yards of required structural fill needs to come from offsite sources. Note that structural fill used in constructing STP 1 & 2 was a well-graded sand obtained from the Eagle Lake/Gifford Hill source, approximately 55 miles north of the site. The structural fill for STP 3 & 4 are sound, durable, well-graded sand or sand and gravel; maximum 25% fines content; and free of organic matter, trash, and deleterious materials. Once the potential sources of structural fill have been identified, the candidate materials are sampled and tested in the laboratory to establish their static and dynamic properties. Chemical tests are also performed on candidate structural fill materials.

2.5S.4.5.2 Extent of Excavations, Fills, and Slopes

At the STP 3 and STP 4 areas, existing ground surface elevations at field testing locations (e.g., borings, CPTs, etc.) ranged from approximately El. 27 feet to El. 32 feet, with an average at approximately El. 30 feet. The proposed rough grade at the STP 3 and STP 4 areas is approximately El. 34 feet. At the UHS Basin/RSW area, existing ground surface elevations at field testing locations ranged from approximately El. 24 feet to El. 31 feet, with an average at

approximately El. 29 feet. The proposed rough grade at the UHS Basin/RSW area is approximately El. 30 feet. A non-structural earth berm, raised to approximately El. 49 feet, surrounds the exterior of the UHS Basin. Earthwork operations are conducted to achieve the proposed site grades, as shown on the excavation plan on Figure 2.5S.4-48. All safety-related structures are contained inside the STP 3 & 4 Power Block area (including the UHS Basin/RSW area), as shown on Figure 2.5S.4-2.

A listing of major structures (including Seismic Category I structures and/or piping), with proposed underside of foundation elevations, and identification of the predominant soil strata at the underside of foundation elevation follows (noting that foundation elevations may be subject to minor change at this time):

Structure [1]	Foundation El. [2] (feet)	Predominant Soil Stratum [2] at Foundation El.
Reactor Buildings	-51 {-61}	F {F}
Control Buildings	-41	E
Service Buildings	-14, 4, 34 [3]	Structural Fill [3]
Radwaste Buildings	-20 {-35}	D {E}
Turbine Buildings	2	C
UHS Basin	-2	C
RSW Pump Houses	-9	C
RSW Lines	9 {5}	B {C}

[1] Seismic Category I structures and/or piping from the table above include: Reactor Buildings, Control Buildings, Radwaste Buildings (foundations), UHS Basin, RSW Pump Houses, and RSW Lines (in cut-and cover tunnel).

[2] Foundation Els. and soils strata designations shown in "{ }" symbols denote the Els. and conditions at the base of significant over-excavation at the particular structure (e.g., at the Reactor Buildings, Stratum F is over-excavated 10 feet below the underside of foundations, with over-excavation replaced by concrete fill; at the Radwaste Buildings, Stratum D is over-excavated 15 feet below the underside of foundations, with over-excavation replaced by structural fill; and at the RSW Lines, Stratum B is over-excavated 4 feet below the underside of tunnel foundations, with over-excavation replaced by structural fill).

[3] The Service Buildings are stepped structures supported primarily on structural fill (i.e., the Service Buildings fall primarily within the limits of the sloped excavations made to the underside of adjoining, more deeply founded, structures).

As noted above, foundation excavations result in removing approximately 2.135 million cubic yards of soils. The extent of excavation, filling, temporary slopes, and the approximate limits

of temporary ground support for major structures are shown in plan on Figure 2.5S.4-48 and in section on Figures 2.5S.4-49 through 2.5S.4-54 (note that the sections are taken at locations identified on Figure 2.5S.4-48). These figures illustrate that the excavations for foundations at major structures result in most major structures being founded either directly on dense sand strata (i.e., especially Strata C and E) or on structural fill bearing on dense sand strata, except that the Reactor Buildings are founded on concrete fill over a reduced thickness of very stiff clay stratum (Stratum F) (e.g. the reduced thickness/remaining Stratum F at the STP 3 Reactor Building ranges from absent to 17.5 feet thick, and averages 7.6 feet thick; the reduced thickness/remaining Stratum F at the STP 4 Reactor Building ranges from 16 feet to 17.7 feet thick, and averages 16.5 feet thick).

The excavation at the deepest level (i.e., the underside of over-excavation for the Reactor Buildings at El. -61 feet) is approximately 95 feet below proposed rough grade (El. 34 feet at the STP 3 and STP 4 areas). The subsurface investigation made at STP 3 & 4 has indicated that the subsurface strata to bear foundations are relatively horizontal. However, it should be noted that the extent of excavation to final subgrade and/or to final over-excavation level is determined during construction, based on observation of actual subsurface conditions encountered, and verification of their suitability for foundation support. Once subgrade suitability at the proposed bearing stratum has been confirmed, excavations are backfilled with either concrete fill (in the case of the Reactor Buildings) or compacted structural fill up to the foundation level of structures. Following construction of structure foundations and other underground features, structural fill is extended to the proposed rough grade, or near the proposed rough grade, depending on the details of the project detailed design stage (civil engineering elements). Compaction and quality control/quality assurance programs for filling are addressed in Subsections 2.5S.4.5.3.

There are not any permanent excavation or fill slopes created by site grading. Refer to Subsection 2.5S.5 for additional discussion.

Temporary excavation slopes, such as those for foundation excavation and/or over-excavation, would be graded on an inclination of approximately 1.5 horizontal:1 vertical (1.5:1 H/V). Note that the excavation plan and sections, Figures 2.5S.4-48 and 2.5S.4-49 through 2.5S.4-54, respectively, are based on temporary slopes typically at approximately 1.5:1 H/V, with 15-foot-wide berms (for slope maintenance) every 20-foot vertical interval (equating to a composite slope of approximately 2.25:1 H/V). Note that the deepest structure excavations made for STP 1 & 2 construction, while approximately 35 feet shallower than the deepest structure excavations proposed for STP 3 & 4, used the same typical 1.5:1 H/V temporary slope, but with a narrower (10 feet wide) bench width (for a composite slope of approximately 2:1 H/V). In any event, at project detailed design stage, slope stability analyses are conducted to show that temporary slopes have an adequate factor of safety (typically at least 1.30 for temporary slopes [static conditions]), including, among other things, the effects of surcharge loading from construction equipment, and the effects of the excavation dewatering scheme.

Note also on the excavation plan and sections, Figures 2.5S.4-48 and 2.5S.4-49 through 2.5S.4-54, respectively, that the approximate limits of temporary ground support are shown along the south edge of the Turbine Buildings and along the south edge of the Reactor Buildings. At the south edge of the Turbine Buildings, there is an abrupt change in grade (from the subgrade levels of the Control Buildings at El. -41 feet, to the subgrade levels of the Turbine Buildings

at El. 2 feet) that cannot be accommodated by a stable soil slope. Also, at the south edge of the Reactor Buildings, a steeper slope is also required to accommodate the reach of a heavy lift crane needed to place the reactor vessels. It is understood that this crane is capable of performing a 1250 metric tonne lift at a reach of approximately 200 feet. In both cases, temporary ground support is planned at project detailed design stage. Note also in the case of the temporary ground support at the south edge of the Reactor Buildings that either the temporary ground support is designed to accommodate the crane loads or the crane loads are carried below the base level of structure excavation (i.e., by piling, etc.). It is envisioned that temporary ground support would likely be tied-back concrete slurry (diaphragm) walls, or perhaps tied-back soldier piles and lagging (with soldier piles pre-drilled in place). There is also the possibility at project detailed design stage of considering an option where the deepest excavation levels are completely contained by a concrete slurry (diaphragm) wall cofferdam, either tied-back, or formed into a ring or “figure eight” shape supported by internal ring beams. This option could reduce excavation and structural fill volume, as well as reduce construction dewatering pumpage, provided the concrete slurry (diaphragm) wall can be toed into a relatively impermeable (clay) layer at depth.

Excavations for the UHS Basin and RSW Lines are similar to those described above, except that excavations are made to shallower depths (i.e., approximately 39 feet below ground surface at the UHS Basin and approximately 23 feet below ground surface at the RSW Lines). Note also that the RSW Lines, as currently planned, are contained within relatively shallow-depth cut-and-cover concrete tunnels. The excavation limits for these cut-and-cover tunnels are not detailed on Figure 2.5S.4-49.

2.5S.4.5.3 Compaction Specifications

Once structural fill sources are identified, as discussed in Subsection 2.5S.4.5.1, several samples of materials are obtained and tested for index properties and for engineering properties, especially including grain size and plasticity characteristics, and moisture-density relationships. For foundation support and for backfill against walls, structural fill needs is compacted to a minimum of 95% of its maximum dry density and within 3% of its optimum moisture content, as determined based on the modified Proctor compaction test procedure (Reference 2.5S.4-42).

Fill placement and compaction control procedures are addressed in a technical specification prepared at project detailed design stage. This includes requirements for suitable fill, sufficient testing to address potential material variations, and in-place density and moisture content testing frequency (e.g., a minimum of one test per 10,000 square feet of fill placed per lift). The technical specification also includes requirements for an onsite testing firm for quality control, especially to ensure specified material gradation and plasticity characteristics, the achievement of specified moisture-density criteria, fill placement/compaction, and other requirements to ensure that fill operations conform to a high standard of practice. The onsite testing firm is required to be independent of the earthwork contractor and to have an approved quality assurance program. A sufficient number of laboratory tests are required to ensure that any variations in fill materials are accounted for. As with STP 1 & 2 construction, a materials testing laboratory is established onsite to serve the STP 3 & 4 construction work. A trial fill program is normally conducted for the purposes of determining the optimum number of compactor coverages (passes), the maximum loose lift thickness, and other relevant data for optimum achievement of the specified moisture-density (compaction) criteria.

2.5S.4.5.4 Dewatering and Excavation Methods

Groundwater control in excavations is required during construction. Groundwater conditions and construction-stage dewatering are addressed in more detail in Subsection 2.5S.4.6.

Given the subsurface conditions, excavations are made using conventional earth-moving equipment, likely using self-propelled scrapers with push dozers for loading, and excavators and dump trucks in the more confined excavation areas and for final slope trimming. Note that scrapers are ideally loaded by pushing down a slight incline in the excavation surface. This practice makes separating horizontally-bedded strata (e.g., like the interlayered clays and sands present at STP 3 & 4) more challenging, requiring close monitoring onsite where sand soils (but not clay soils) are reused for fill (refer to Subsection 2.5S.4.5.1 for additional discussion). Excavations are planned primarily as open cuts, with limited temporary ground support, as described above.

Upon reaching final excavation levels (i.e., foundation subgrade or required over-excavation level), all excavations are cleaned of loose material, by either removal or by compaction in place. Final subgrades are inspected and approved prior to being covered by concrete fill or structural fill. Inspection and approval procedures are addressed in the foundation and earthworks specifications developed at project detailed design stage. These specifications are planned to include, among other things, measures such as proof-rolling, over-excavation and replacement of unsuitable soils, protection of surfaces from deterioration. Excavations are to comply with applicable OSHA regulations (Reference 2.5S.4-48).

Foundation subgrade rebound (or heave) is monitored in excavations for selected STP 3 & 4 major structures. Subgrade rebound estimates are addressed in Subsection 2.5S.4.10. Selected STP 1 & 2 major structures and selected lengths of the Main Cooling Reservoir (MCR) earth dike are additionally monitored during STP 3 & 4 excavation and dewatering. Monitoring program specifications are developed during the detailed design stage of the project. The specification document addresses issues such as the installation of a sufficient quantity of instruments in the excavation zone, monitoring and recording frequency, and evaluation of the magnitude of subgrade rebound and structure settlement during excavation, dewatering, and subsequent foundation construction.

2.5S.4.6 Groundwater Conditions

The following site-specific supplement addresses COL License Information Item 2.32. Refer to Subsection 2.5S.4.10 for additional detail on groundwater conditions relative to the foundation stability of Seismic Category I structures.

2.5S.4.6.1 Site-Specific Data Collection and Monitoring

A groundwater data collection and monitoring program is in progress at the time of writing this FSAR, following from the installation of observation wells during the STP 3 & 4 subsurface investigation. The details of existing groundwater conditions at STP 3 & 4 are given in Subsection 2.4S.12. The details of measured groundwater levels from the period late December 2006 through late July 2007 are shown on Figures 2.5S.4-55 and 2.5S.4-56. Based on the available data, a shallow (likely “perched”) groundwater level, primarily measured in Stratum C (sand), inside the STP 3 & 4 Power Block area ranged from approximately El. 23 feet to El.

26 feet, and averaged approximately El. 25 feet, while outside the Power Block it ranged from approximately El. 19 feet to El. 27 feet, and averaged approximate El. 24 feet. Similarly, the groundwater level associated with a deeper hydrostatic surface, primarily measured in Stratum E (sand), inside the STP 3 & 4 Power Block area ranged from approximately El. 16 feet to El. 18 feet, and averaged approximate El. 17 feet, while outside the Power Block it ranged from approximately El. 13 feet to El. 18 feet, and averaged approximate El. 16 feet. For engineering purposes, a groundwater level at El. 25.5 feet was selected based on the data available.

Note that foundations for major structures are proposed both within the shallow and deeper water-bearing soils, and as such, both the shallow and deeper groundwater conditions could impact foundation subgrade stability during construction if not properly controlled, resulting in loss of density, bearing, and equipment trafficability.

2.5S.4.6.2 Construction-Stage Dewatering

Temporary dewatering is required for groundwater control during the project construction stage. The detailed analysis of groundwater conditions at the site is in progress at this time, given continued groundwater monitoring, as addressed in Subsection 2.4S.12, and the need to analyze and interpret the results once the data collection phase has been completed. Nevertheless, on the basis of the defined subsurface conditions, it is understood that groundwater control/construction dewatering is needed at STP 3 & 4 during excavation for major structures. A construction-stage dewatering specification is developed during project detailed design stage. Construction-stage dewatering likely includes both a system of deep wells and/or well-points to dewater or to depressurize the major water-bearing strata in advance of excavation, and a system of shallow drains and/or ditches to collect and direct minor seepage. Generally, groundwater levels are maintained a minimum of 3 feet below final excavation levels. Additionally, water-bearing sand strata (e.g., Stratum H especially) below final excavation levels that are overlain by more impermeable clay strata are depressurized to ensure the base stability of excavations. A construction-stage dewatering design and specification is developed during project detailed design stage.

2.5S.4.6.3 Analysis and Interpretation of Seepage

As above, the detailed analysis of the groundwater conditions at the site is in progress at this time, given continued groundwater monitoring, as addressed in Subsection 2.4S.12. Once the monitoring phase is completed, the available data are analyzed to obtain an estimate of seepage into the major structure excavations.

2.5S.4.6.4 Permeability Testing

The permeabilities of site soils were measured insitu by slug testing, as discussed in Subsection 2.5S.4.2.2.5. A detailed description of the tests and the results is included in Subsection 2.4S.12. A summary of hydraulic conductivity values calculated from those tests is provided in Table 2.5S.4-23.

2.5S.4.6.5 History of Groundwater Fluctuations

A detailed description of the groundwater conditions at the STP site is included in Subsection 2.4S.12.

2.5S.4.7 Response of Soil and Rock to Dynamic Loading

The following site-specific supplement addresses, in part, COL License Information Item 2.34. Refer to Subsection 2.5S.4.4 for additional discussion.

STP continues to augment existing data in this subsection with additional confirmatory dynamic soil parameters, including the results of confirmatory RCTS testing and the results of deep sonic logging data (converted to shear wave velocity) comparisons with the shear wave velocity profiles on Figure 2.5S.4-57. Following completion of these activities, the FSAR will be updated in accordance with 10 CFR 50.71(e) by the third quarter of 2008 to provide this confirmatory information (COM 2.5S-1).

Detailed descriptions of the development of the Ground Motion Response Spectrum (GMRS) and the associated Probabilistic Seismic Hazard Assessment (PSHA), as well as the geologic characteristics of the site, are addressed in Subsection 2.5S.2.

2.5S.4.7.1 Site Seismic History

The seismic history of the area and of the site, including any prior history of seismicity, and evidence of liquefaction or boils is addressed in Subsections 2.5S.1.1.4.4.5 and 2.5S.1.2.6.4.

2.5S.4.7.2 P- and S-Wave Velocity Profiles

Given the extreme thickness of sediments at the site (refer to Subsection 2.5S.4.1) compared to the depth of compressional and shear wave velocity measurements made during the STP 3 & 4 subsurface investigation (to approximately 600 feet below ground surface), additional information was required to complete the velocity profile for the site. Velocities in the upper 600 feet were measured at the site, while velocities deeper than 600 feet were obtained from available references. Additional discussion follows.

2.5S.4.7.2.1 Seismic Velocity in the Upper 600 Feet

Geophysical measurements in the upper 600 feet at STP 3 & 4 were obtained by suspension P-S velocity logging methods, and by seismic CPT methods, as discussed in Subsection 2.5S.4.4.2. An average shear wave velocity profile for the upper 600 feet at STP 3 & 4 is shown on Figures 2.5S.4-45, 2.5S.4-46, and 2.5S.4-47. Average shear wave velocities (V_S), Poisson's ratios (μ), and related parameters are summarized in Table 2.5S.4-27.

Suspension P-S velocity logging measurements were made at 10 borings, five each at the STP 3 area and the STP 4 area, with depths ranging from approximately 200 feet to 600 feet below ground surface, and at locations shown on Figure 2.5S.4-2. Seismic CPT measurements were made at five CPTs, three at the STP 3 area and two at the STP 4 area, with depths ranging from approximately 65 feet to 95 feet below ground surface, and at locations shown on Figure 2.5S.4-2. The suspension P-S logging data and the seismic CPT data are contained in Reference 2.5S.4-2 (Appendix 2.5-A). As shown on Figures 2.5S.4-40 and 2.5S.4-41, the trends in V_S profiles between the STP 3 area and the STP 4 area are generally consistent. Also for comparison, the V_S profiles obtained previously for STP 1 & 2 (Reference 2.5S.4-3) to a depth of approximately 300 feet below ground surface are shown along with the V_S profiles obtained from the STP 3 & 4 subsurface investigation on Figures 2.5S.4-43 and 2.5S.4-44.

In general, comparison of measured STP 1 & 2 V_S results with those obtained from the STP 3 & 4 subsurface investigation indicate relatively consistent results, ignoring variations of about 100± feet/second, except between approximately El. -40 feet to -105 feet, where greater differences of the order of 300 to 400 feet/second are noted. Note that this comparison is only for the upper approximately 300 feet of soils at STP 3 & 4, as the STP 1 & 2 data (shown on Figures 2.5S.4-43 and 2.5S.4-44) only extended to approximately 300 feet below ground surface.

As noted above, design/average shear wave velocity (V_S) and Poisson's ratio (μ) values are summarized in Table 2.5S.4-27. Note that these design/average values were developed considering the variation in strata top/base elevations and thicknesses from boring-to-boring and from CPT-to-CPT. Note also that Sub-stratum J Sand was found to contain four separate interbedded sub-strata of sands and/or silts at various depths (i.e., Sub-stratum J Interbed 1 [sand or silt], Sub-stratum J Sand 1, Sub-stratum J Interbed 2 [sand or silt], and Sub-stratum J Sand 2) which were additionally discontinuous between boring locations. For developing Sub-stratum J Sand design/average values, shear wave velocity measurements obtained for the various interbedded sands and silts were fitted to a single sand/silt sub-stratum occurring between the two clay sub-strata (i.e., Sub-stratum J Clay 1 and Sub-stratum J Clay 2).

2.5S.4.7.2.2 Seismic Velocity Below 600 Feet

The soil sediments at STP 3 & 4 extend well below the 600 feet maximum depth of the STP 3 & 4 subsurface investigation. Additional subsurface information was sought to characterize the site conditions below this depth.

2.5S.4.7.2.2.1 Soil Shear Wave Velocity Profile

The upper 600 feet at STP 3 & 4 were investigated using borings, CPTs, and geophysical logging methods, and the design/average velocity profile to that depth is described in Subsection 2.5S.4.7.2.1. Between approximately 600 feet below ground surface and 2620 feet below ground surface, subsurface and shear wave velocity information was taken from the STP 1 & 2 UFSAR (Reference 2.5S.4-3). According to that reference, the subsurface deeper than 600 feet below ground surface consists of alternating layers of very stiff to hard clay (with some claystones and siltstones) and very dense, fine to silty fine sand. The claystones and siltstones occur at depths greater than approximately 880 feet below ground surface, with the frequency of their occurrence increasing with depth. Refer to Subsection 2.5S.4.1 for a brief description

of geologic conditions at greater depths, a key point being that the top depth of pre-Cretaceous bedrock ("basement rock") has been estimated to occur at approximately 34,500 feet below ground surface (Reference 2.5S.4-4).

Reference 2.5S.4-4 also contains deep shear wave velocity profiles developed for a later-stage review of the STP site, among others. These profiles increase in shear wave velocity to a depth of approximately 2500 feet below ground surface and then maintain a common value of V_S between 2500 feet and 5000 feet depths. According to the Reference 2.5S.4-4, these profiles were based on site-specific cross-hole measurements in the uppermost approximately 250 feet and were then attached to the deeper and more generic "Mississippi embayment lowlands profile," which is described in more detail in the reference. The resulting composite V_S profiles are reproduced and shown on Figure 2.5S.4-57. Note that the details of this figure are truncated at El. -3250 feet, corresponding to a depth of approximately 3280 feet below ground surface, or 1 kilometer. Three shear wave velocity profile cases, M1P1, M1P2, and M1P3, are provided on the figure. The three profiles in Figure 2.5S.4-57 all show an increase in shear wave velocity to 9285 feet/second at a depth of approximately 2500 feet. Numerical values from the three shear wave velocity profiles versus depth, between 600 feet and 3280 feet below ground surface, or 1 kilometer, are summarized in Table 2.5S.4-28. Soil unit weight information is limited deeper than 600 feet, with available information from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) given in Table 2.5S.4-29. Note that for completeness, Table 2.5S.4-29 also provides the selected values of unit weight for the upper 600 feet of soils from the STP 3 & 4 subsurface investigation.

2.5S.4.7.2.2.2 Bedrock Shear Wave Velocity Profile

To assess the V_S profile at substantially greater depth, a search was made of geophysical logging results (especially sonic logging) made for existing oil wells in the STP site vicinity. Three such wells were selected (LL3341, LL4537, and LL4987) from the available information, having the deepest sonic logging results (to a maximum of approximately 15,600 feet below ground surface). An initial conversion of the sonic logging data to shear wave velocities showed generally good agreement with the shear wave profiles presented on Figure 2.5S.4-57. These data will be further reduced, and the results/comparisons provided at a later date (refer to the statement on COM 2.5S-1 at Subsection 2.5S.4.7).

2.5S.4.7.3 Static and Dynamic Laboratory Testing

Extensive static laboratory testing of representative soil samples obtained from the STP 3 & 4 subsurface investigation were conducted, with results described in detail in Subsection 2.5S.4.2.

Dynamic laboratory testing, consisting of Resonant Column Torsional Shear (RCTS) tests, to obtain data on shear modulus and damping ratio characteristics of site soils over a wide range of strains, is now in progress. A total of 18 undisturbed soil samples, from depths of 10 feet to 590 feet below ground surface, were assigned for RCTS testing. Results from five of those assigned RCTS tests are available at this time and are discussed briefly below (Subsection 2.5S.4.7.3.3). In the interim, shear modulus degradation and damping ratio curves from the available literature were used for dynamic soil properties characterization. Once all assigned RCTS tests have been completed, an evaluation will be made, comparing the laboratory RCTS

test-derived curves with the selected (literature) curves (refer to the statement on COM 2.5S-1 at Subsection 2.5S.4.7). Refer to Subsection 2.5S.2 for additional discussion.

In the absence of final RCTS test results, shear modulus degradation and damping ratio curves selected from the available literature for the various STP 3 & 4 soil strata are discussed below. A brief review of the five available laboratory RCTS test results is also provided.

2.5S.4.7.3.1 Selected Shear Modulus Degradation Curves for Soils

Generic shear modulus degradation curves for cohesionless soil strata B, C, E, H, J Sand, K Sand/Silt, M, and N Sand were developed from Reference 2.5S.4-49, based on strata depths. The depths of soil strata at approximate mid-thicknesses, summarized in Table 2.5S.4-30, were used to develop strata-specific curves. The specific/recommended curves for the above-noted cohesionless soil strata are shown on Figure 2.5S.4-58, with numerical values given in Table 2.5S.4-31. An alternate set of curves for cohesionless soil strata, “Peninsular Range” curves (Reference 2.5S.4-50), were also evaluated, and are similarly shown on Figure 2.5S.4-58, with numerical values given in Table 2.5S.4-31. Note that these latter curves provide a range of values that can allow for overconsolidation and other variations.

Generic shear modulus degradation curves for cohesive soil strata A, D, F, J Clay, K Clay, L, and N Clay were similarly developed from Reference 2.5S.4-49, based on strata plasticity indices (PI). For cohesive soil strata occurring at depths greater than approximately 100 feet, an increase in the PI value was taken, equivalent to the next higher PI reference curve shown in Reference 2.5S.4-49 (as per Reference 2.5S.4-51). As an example, for a clay stratum deeper than 100 feet and having PI=10%, the next higher reference curve for PI=30% was used in selecting the shear modulus degradation relationship. The PI value (maximum) was capped at 70%. The specific/recommended curves for the above-noted cohesive soil strata are shown on Figure 2.5S.4-59, with numerical values given in Table 2.5S.4-31.

2.5S.4.7.3.2 Selected Damping Curves for Soils

Generic damping ratio curves for cohesionless soil strata B, C, E, H, J Sand, K Sand/Silt, M, and N Sand were developed from Reference 2.5S.4-49, based on strata depth. The specific/recommended curves for the above-noted cohesionless soil strata are shown on Figure 2.5S.4-60, with numerical values given in Table 2.5S.4-32. An alternate set of curves for cohesionless soil strata, “Peninsular Range” curves (Reference 2.5S.4-50), were also evaluated, and are similarly shown on Figure 2.5S.4-60, with numerical values given in Table 2.5S.4-32.

Generic damping ratio curves for cohesive soil strata A, D, F, J Clay, K Clay, L, and N Clay were also developed from Reference 2.5S.4-49, based on strata plasticity indices (PI). For cohesive strata occurring at depths greater than approximately 100 feet, an increase in the PI value was taken, as noted above (as per Reference 2.5S.4-51). The specific/recommended curves for the above noted-cohesive soil strata are shown on Figure 2.5S.4-61, with numerical values given in Table 2.5S.4-32.

Note that in the referenced figures and tables, damping ratios were provided at values exceeding 15%, although, damping is frequently cut off at this value. For the purpose of dynamic analyses, damping ratio is limited to 15%, and the portions of the referenced figures and tables above this value are not considered.

2.5S.4.7.3.3 Comparison of Selected and Measured Shear Modulus Degradation and Damping Ratios for Soils

As described previously, in the absence of site-specific dynamic test results, shear modulus degradation and damping ratio curves for site soils were selected from the available literature, as given in Tables 2.5S.4-31 and 2.5S.4-32 and as shown on Figures 2.5S.4-58 through 2.5S.4-61. A total of 18 undisturbed soil samples were assigned for RCTS testing to measure shear moduli and damping ratios for selected site soils across a wide range of strains. The results of five completed RCTS tests are discussed here and compared with the selected (literature) curves. The results of the remaining assigned tests are pending. Note that the results of the five available RCTS tests were for soils from three main strata, namely, Stratum M, (one test) Sub-stratum N Clay (one test on N Clay 1, one test on N Clay 2, and, one test on N Clay 4), and Sub-stratum N Sand (one test on N Sand 2). A summary of the results of the available tests is given in Table 2.5S.4-33, with comparisons of individual test results to selected (literature) curves given on Figures 2.5S.4-62 through 2.5S.4-64. Note that the RCTS test results shown are for a wide range of confining stresses (i.e., from less than 100 pounds per square inch [psi] to over 400 psi) and frequencies (i.e., from 0.5 Hz to over 80 Hz), therefore, some spread in the results should be expected. The following initial observations can be made from the available RCTS test results:

- For the tests made on samples from Stratum M and Sub-stratum N Sand, up to a shear strain level of approximately 10^{-2} %, there is very close agreement between the measured test results and the selected (literature) shear modulus degradation and damping curves. At shear strain levels above approximately 10^{-2} %, the differences widen, indicating either matching or higher measured shear moduli and lower measured damping ratios than those portrayed by the selected (literature) curves.

- The spread in results appears more pronounced in the tests made on the three samples from Sub-stratum N Clay (Figure 2.5S.4-64), which were taken from a range of depths spanning approximately 145 feet. The measured test results here show somewhat different responses at different depths. Note however, that the selected (literature) curve falls within the mid-range of the RCTS data.

The above observations are based on limited RCTS test results. While the available test results, in some cases, indicate a somewhat different soil response than those selected for shear strain levels exceeding approximately 10^{-2} %, any necessary modifications to the dynamic soil model should await the completion of the RCTS testing program, and a comprehensive review of all the RCTS test results (refer to the statement on COM 2.5S-1 at Subsection 2.5S.4.7). Refer to Subsection 2.5S.2 for additional discussion.

2.5S.4.7.3.4 Shear Modulus and Damping for Rock

Refer to Subsection 2.5S.4.1 for a brief description of geologic conditions at depths below approximately 600 feet below ground surface, a key point being that the top depth of pre-Cretaceous bedrock ("basement rock") has been estimated to occur at approximately 34,500 feet below ground surface (Reference 2.5S.4-4).

Refer also to Subsection 2.5S.4.7.2.2.1 for discussion of deep shear wave velocity profiles pertinent to the STP site and derived from information contained in Reference 2.5S.4-4.

It should be noted that hard rock is considered to have damping, but is not strain dependent. For the STP 3 & 4 work, a damping ratio of 0.2% was adopted for bedrock, and bedrock shear modulus was considered to remain constant (i.e., no degradation), in the shear strain range of 10^{-4} % to 1%.

2.5S.4.7.3.5 Dynamic Properties of Structural Fill

Some major structures (e.g., the Reactor Buildings, the Radwaste Buildings, and the RSW Lines) require over-excavation and the placement of either concrete fill or structural fill below their foundations. Refer to Subsection 2.5S.4.5 for structural fill requirements.

2.5S.4.7.4 Small Strain Shear Modulus Estimation

With shear wave velocity and other parameters established, small strain shear modulus values can be calculated from Equation 2.5S.4-6. Note that shear wave velocity values for use in the equation are given in Tables 2.5S.4-27 and 2.5S.4-28, and unit weight values for use in the equation are given in Table 2.5S.4-29. Refer to Subsection 2.5S.4.2.2 for a stratum-by-stratum discussion of the derivation of shear modulus (G) and other geotechnical engineering parameters for use in design.

2.5S.4.7.5 Seismic Parameters for Liquefaction Potential Analysis

Using the site-specific soil column extended to ground surface, the amplification factor, and the performance-based hazard methodology employed to develop the GMRS (refer to Subsections 2.5S.2.5 and 2.5S.2.6), a peak horizontal ground surface acceleration of 0.10g and a Moment Magnitude 7.7 earthquake was selected for use in liquefaction potential analysis. Refer in particular to Subsection 2.5S.2, Table 2.5S.2-17 entitled "Controlling Magnitudes and Distances from Deaggregation," regarding selection of the earthquake magnitude for use in liquefaction potential analysis.

2.5S.4.8 Liquefaction Potential

The following site-specific supplement addresses COL License Information Item 2.33.

The potential for soil liquefaction at STP 3 & 4 was evaluated following guidance given in RG 1.198 (Reference 2.5S.4-52). The current state-of-the-art, outlined in Reference 2.5S.4-5, was followed. The subsurface conditions and soil properties employed were those described in Subsection 2.5S.4.2. The peak horizontal ground surface acceleration and earthquake magnitude employed were those described in Subsection 2.5S.4.7.5.

2.5S.4.8.1 Liquefaction Potential of STP 1 & 2

The STP 1 & 2 UFSAR (Reference 2.5S.4-3) reports that liquefaction potential at that site was evaluated using SPT data from site-specific borings and using response analyses together with the results of cyclic triaxial laboratory tests. The site was evaluated for a peak ground surface acceleration of 0.10g and the equivalent of a Moment Magnitude 6 earthquake. The results showed that site soils either did not possess the potential to liquefy, or would not liquefy, under these seismic conditions.

2.5S.4.8.2 Liquefaction Potential of STP 3 & 4

As noted in Subsection 2.5S.4.2, subsurface stratigraphy of STP 3 & 4 is shown, in part, on the subsurface profiles, Figures 2.5S.4-5 through 2.5S.4-9. As discussed in Subsection 2.5S.1, the site soils, primarily Beaumont Formation deposits, are geologically old (Pleistocene age). Conventionally, only younger deposits, especially Holocene age and Recent age deposits are considered potentially liquefiable. To be complete and conservative, a comprehensive liquefaction analysis for all boring, CPT, and shear wave velocity data, and for all soil types, including those having high fines contents and/or predominantly fine-grained, was conducted.

For the purpose of liquefaction analysis, as well as for general subsurface stratification, each individual boring and CPT made at STP 3 & 4 was divided according to the various subsurface strata defined in Subsection 2.5S.4.2 (i.e., Strata A through N, excluding G and I). As such, the soils in the upper 600 feet of the site were evaluated for liquefaction, using the results of the STP 3 & 4 subsurface investigation. Soils deeper than 600 feet below ground surface are geologically old and are non-liquefiable, as further discussed in Subsection 2.5S.4.8.2.5.

As described in Subsection 2.5S.4.7.5, the peak horizontal ground surface acceleration for the site was selected at 0.10g, together with a Moment Magnitude 7.7 earthquake. These values were used in the STP 3 & 4 liquefaction potential analysis.

2.5S.4.8.2.1 Liquefaction Evaluation Methodology

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore water pressure and reduced effective stress (Reference 2.5S.4-52). Soil liquefaction occurrence (or lack thereof) depends on geologic age, state of soil saturation, density, gradation, plasticity, and earthquake intensity and duration. The liquefaction analysis presented here employed state-of-the-art methods (Reference 2.5S.4-5) for evaluating the liquefaction potential of STP 3 & 4 site soils.

In brief, the present state-of-the-art considers an evaluation of data from SPT, CPT, and shear wave velocity (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). And finally, a factor of safety (FOS) against liquefaction is 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, FOS, and other intermediate parameters such as the stress reduction coefficient (r_d), the magnitude scaling factor (MSF), the K_σ correction factor accounting for liquefaction resistance with increasing confining pressure, and a host of other correction factors, can be found in Reference 2.5S.4-5. Note that a MSF of 0.93 was used in the analyses, based on the selected earthquake magnitude. A review of the results of liquefaction potential analyses using the available SPT, CPT, and V_s data for the whole of STP 3 & 4 follows.

2.5S.4.8.2.2 FOS Against Liquefaction Based on SPT Data

Uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11 through 2.5S.4-15 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively. SPT data from all 52 borings made within the STP 3 area, all 41 borings made within the STP 4 area, all 16 borings made within the UHS Basin/RSW area, and all 11 borings made within the area outside the Power Block were evaluated for liquefaction potential. For completeness, all SPT N-values, including those measured in clay soils and those measured in soils above the groundwater level, were initially included in the FOS calculation, despite their known high resistance to liquefaction.

The equivalent clean-sand $CRR_{7.5}$ value, based on the SPT clean sand equivalent $(N_1)_{60cs}$, was calculated following recommendations in Reference 2.5S.4-6, (i.e., by step-wise proceeding from uncorrected SPT N value, to normalized N_1, to hammer energy corrected $(N_1)_{60}$, to clean sand equivalent $(N_1)_{60cs}$, and then calculating $CRR_{7.5}$ based on $(N_1)_{60cs}$). Refer to Figure 2.5S.4-65 for an example of this step-wise approach from uncorrected SPT N to clean sand equivalent $(N_1)_{60cs}$. Reference 2.5S.4-5 notes that clean sands and/or clean sand equivalents, having $(N_1)_{60cs} \geq 30$ blows/foot are considered too dense to liquefy, and are classified as non-liquefiable. Note that at STP 3 & 4, 1250 tests of 3389 total tests, or approximately 36.9% of tests, had $(N_1)_{60c\,s} \geq 30$ blows/foot.

Of the 3389 SPT N-values, all but 15 tests had FOS ≥ 1.10 (refer to Subsection 2.5S.4.11 for discussion on the selection of an appropriate FOS). The 15 tests having FOS<1.10 amounted to 0.4% of all the tests evaluated; in other words, 99.6% of calculated FOS values by this

method exceeded 1.10. For completeness, an examination of each FOS < 1.10 is provided in Table 2.5S.4-34. From Table 2.5S.4-34, it can be noted that: seven of the 15 tests were within areas/depths excavated for structure foundations; four of the 15 tests were within areas where no structures are placed, and where soils at similar elevations in adjoining borings had minimum FOS=1.41; of the remaining four of the 15 tests, two of those tests were made on clay soils which are unlikely to liquefy; and the two remaining tests are discussed separately next.

Of the remaining two of the 15 tests:

- One test (Boring B-350; El. 12.3 feet; Stratum B) occurred at shallow depth at the STP 3 Plant Stack, which is not a safety-related structure, and where the foundation level (likely removing Stratum B) is determined at detailed design stage (note also that soils at similar elevations in adjoining borings had minimum FOS=1.76)
- One test (Boring B-305DH/DHA; El. -348.7 feet; Sub-stratum N Sand 2) occurred at an extreme depth at the STP 3 Reactor Building and not affecting that structure (note also that Sub-stratum N Sand 2 in the STP 4 deep boring [Boring B-405DH] was "non-liquefiable" [i.e., $(N_1)_{60cs} \geq 30$])

Hence, the low FOS values from the SPT method are not significant to the safety of STP 3 & 4.

2.5S.4.8.2.3 FOS Against Liquefaction Based on CPT Data

CPT testing at STP 3 & 4 included the recording of both commonly-measured cone parameters (e.g., cone tip resistance, friction sleeve resistance, and pore pressure), and less-frequently-measured shear wave velocity. The evaluation of liquefaction potential based on commonly-measured cone parameters is addressed here. The evaluation of liquefaction potential based on shear wave velocity is addressed in Subsection 2.5S.4.8.2.4.

Corrected CPT q_t tip resistance profiles versus elevation are shown on Figure 2.5S.4-16, 2.5S.4-17, 2.5S.4-18, and 2.5S.4-19 for the STP 3 area, the STP 4 area, the UHS Basin/RSW area, and for the area outside the Power Block, respectively. CPT data from all 10 CPTs made within the STP 3 area, all 11 CPTs made within the STP 4 area, all 10 CPTs made within the UHS Basin/RSW area, and the one CPT made within the area outside the Power Block were evaluated for liquefaction potential. For completeness, all CPT values, including those measured in clay soils and those measured in soils above the groundwater level, were initially included in the FOS calculation, despite their known high resistance to liquefaction.

The equivalent clean-sand $CRR_{7.5}$ value, based on the CPT clean sand equivalent $(q_{c1n})_{cs}$, was calculated following recommendations in Reference 2.5S.4-5, (i.e., by step-wise proceeding from uncorrected CPT q_c value, to corrected q_t, to normalized q_{c1n}, to clean sand equivalent $(q_{c1n})_{cs}$, and then calculating $CRR_{7.5}$ based on $(q_{c1n})_{cs}$). Refer to Figure 2.5S.4-66 for an example of this step-wise approach from uncorrected CPT q_c to clean sand equivalent $(q_{c1n})_{cs}$. Reference 2.5S.4-5 notes that clean sands and/or clean sand equivalents, having $(q_{c1n})_{cs} \geq 160$ (dimensionless) are considered too dense to liquefy and are classified as non-liquefiable. Note that at STP 3 & 4, 751 tests of 4489 total tests, or 16.7% of tests, had $(q_{c1n})_{cs} \geq 160$ (dimensionless). Reference 2.5S.4-5 also notes that soils, having soil behavior type index $I_c \geq$ 2.60, under particular conditions, are considered too clay rich to liquefy, and are also classified

as non-liquefiable. Note that at STP 3 & 4 1670 tests of 4489 total tests, or 37.2% of tests, had $I_c \geq 2.60$.

Of the 4489 CPT values, all but 153 tests had FOS ≥ 1.10. The 153 tests having FOS < 1.10 amounted to 3.4% of all the tests evaluated; in other words, 96.6% of calculated FOS values by this method exceeded 1.10. For completeness, an examination of each FOS < 1.10 is provided in Table 2.5S.4-35. From Table 2.5S.4-35, it can be noted that: 35 of the 153 tests were within areas/depths excavated for structure foundations, 66 of the 153 tests were within areas where no structures are placed, 39 of the 153 tests were made on intermediate/fine-grained soils which are unlikely to liquefy, and the remaining 13 of the 153 tests are discussed separately next.

Of the remaining 13 of the 153 tests:

- Four tests (CPT C-310; El. -49.6 feet to -51.1 feet; Stratum E) occurred at the STP 3 Maintenance Shop, which is not a safety-related structure, and where the foundation level is determined at project detailed design stage
- One test (CPT C-408; El. -59.6 feet; Stratum H) occurred at the STP 4 Switch Yard, which is not a safety-related structure, and where the foundation level is determined at project detailed design stage
- Three tests (CPT C-410; El. 16.4 feet to 15.4 feet; Stratum B) occurred at shallow depths at the STP 4 Maintenance Shop, which is not a safety-related structure, and where the foundation level (likely removing Stratum B) is determined at project detailed design stage
- One test (CPT C-410; El. 12.4 feet; Stratum C) additionally occurred at shallow depth at the STP 4 Maintenance Shop, which is not a safety-related structure, and where the foundation level (likely removing a shallow Stratum C) is determined at project detailed design stage
- Three tests (CPT 307S; El. -10.6 to -11.6; Stratum C) occurred at the STP 3 Turbine Building, which is not a safety-related structure, and which is a very large mat-supported structure, capable of spanning limited areas with reduced subgrade support
- One test (CPT 307S; El. -52.4; Stratum E) additionally occurred at the STP 3 Turbine Building, which is not a safety-related structure, and which, as noted above, is a very large mat-supported structure, capable of spanning limited areas with reduced subgrade support

Hence, the low FOS values from the CPT method are not significant to the safety of STP 3 & 4.

2.5S.4.8.2.4 FOS Against Liquefaction Based on Shear Wave Velocity Data

Shear wave velocity (V_s) data from all five borings (B-302DH, B-305DH/DHA, B-308DH, B-319DH, and B-328DH) and all three CPTs (C-305S, C-306S, and C-307S) made within the STP 3 area, and all five borings (B-402DH, B-405DH, B-408DH, B-419DH, and B-428DH) and both CPTs (C-306S and C-307S) made within the STP 4 area were evaluated for liquefaction potential. For completeness, all V_s values, including those measured in clay soils and those measured in soils above the groundwater level, were initially included in the FOS calculation, despite their known high resistance to liquefaction.

The $CRR_{7.5}$ value, based on the normalized V_{s1}, was calculated following recommendations in Reference 2.5S.4-5, (i.e., by step-wise proceeding from uncorrected V_s value, to normalized V_{s1}, and then calculating $CRR_{7.5}$ based on V_{s1} and the threshold value of V_{s1}*). Note that the threshold value of V_{s1}* depends on fines content, and it varies linearly from 215 meters/second for soils having fines content of ≤5% to 200 meters per second for soils having fines content of 35%. Reference 2.5S.4-5 notes that soils having $V_{s1} \geq V_{s1}$* are considered too dense to liquefy, and are classified as non-liquefiable. Note that at STP 3 & 4, 1208 tests of 1687 total tests, or 71.6% of tests, had $V_{s1} \geq V_{s1}$*.

Of the 1687 V_s values, all but 76 tests had FOS ≥ 1.10. The 76 tests having FOS < 1.10 amounted to 4.5% of all the tests evaluated; in other words, 95.5% of calculated FOS values by this method exceeded 1.10. For completeness, an examination of each FOS < 1.10 is provided in Table 2.5S.4-36. From Table 2.5S.4-36, it can be noted that: 13 of the 76 tests were within areas/depths excavated for structure foundations, 53 of the 76 tests were made on clay soils which are unlikely to liquefy, and the 10 remaining of the 76 tests are discussed separately next.

Of the remaining 10 of the 76 tests:

- One test (B-328DH; El. -127.6 feet; Sub-stratum J Sand 1) occurred at the STP 3 Turbine Building, which is not a safety-related structure, and which is a very large mat-supported structure, capable of spanning limited areas with reduced subgrade support
- Five tests (B-428DH; El. -1.9 feet to -11.8 feet; Stratum C) occurred at the STP 4 Turbine Building, which is not a safety-related structure, and which, as noted above, is a very large mat-supported structure, capable of spanning limited areas with reduced subgrade support
- Two tests (Boring B-305DH/DHA; El. -170.1 feet to -175.1 feet; Stratum M) occurred at extreme depths at the STP 3 Reactor Building and not affecting that structure. Note however that FOS values calculated for the same boring/same depth interval by the SPT method were FOS=1.53 at El. -168.7 feet, and FOS=1.27 at El. -188.7 feet
- Two tests (Boring B-405DH; El. -80.7 feet to -82.3 feet; Stratum H) occurred at the STP 4 Reactor Building, with FOS=1.08. Note however that FOS values calculated for the same boring/same depth interval by the SPT method were FOS="non-liquefiable" (i.e., $(N_1)_{60cs} \geq 30$) at El. -77.4 feet, and FOS=2.08 at El. -87.4 feet

Hence, the low FOS values from the shear wave velocity method are not significant to the safety of STP 3 & 4.

2.5S.4.8.2.5 Liquefaction Resistance of Soils Deeper Than Approximately 600 Feet Below Ground Surface

Liquefaction evaluation at STP 3 & 4 focused on the soils in the upper approximately 600 feet. Site soils, however, are much deeper, with the Pleistocene Beaumont Formation extending to approximately 750 feet below ground surface. Refer to Subsection 2.5S.4.1 for a brief description of geologic conditions at depths below approximately 600 feet below ground surface, a key point being that the top depth of pre-Cretaceous bedrock ("basement rock") has been estimated to occur at approximately 34,500 feet below ground surface (Reference 2.5S.4-4).

Geologic information on soils below a depth of approximately 600 feet below ground surface was gathered from the available literature. Note that even these uppermost soils, including the Beaumont Formation, are considered geologically old (at approximately 100,000 to 24 million years for the Pleistocene, Pliocene, and Miocene deposits, as shown on Figure 2.5S.1-12). Liquefaction resistance increases markedly with geologic age, with Pleistocene soils having more resistance than Recent or Holocene soils, and pre-Pleistocene sediments being generally immune to liquefaction (Reference 2.5S.4-5). On this basis, these deeper soils are geologically too old to be prone to liquefaction. In addition, the degree of compaction and strength of these deeper soils are anticipated only to increase with depth, compared to the overlying soils which were analyzed. Finally, liquefaction analyses using shear wave velocity values of approximately 1250 feet/second at a depth of 600 feet did not indicate the potential for liquefaction, with a calculated FOS=2.6. With shear wave velocities increasing below the 600 feet depth, in the range of approximately 1585 feet/second to 2350 feet/second as indicated on Figure 2.5S.4-57, higher liquefaction resistance would be expected from these deeper soils. On these bases, liquefaction of STP 3 & 4 site soils below a depth of 600 feet below ground surface was not considered possible.

2.5S.4.8.2.6 Concluding Remarks

A liquefaction analysis was performed using state-of-the-art procedures outlined in Reference 2.5S.4-5. SPT data points, 3389 total, were analyzed from 120 borings, from which 99.6% of the calculated FOS values exceeded 1.10. CPT data points, 4489 total, were analyzed from 32 CPTs, from which 96.6% of the calculated FOS values exceeded 1.10. Finally, shear wave velocity (V_S) data points, 1687 total, were analyzed from 10 suspension P-S velocity logging borings and five seismic CPTs, from which 95.5% of the calculated FOS values exceeded 1.10. A detailed examination of the SPT, CPT, and V_S data points analyzed that had FOS < 1.10, revealed that the affected soils are not significant to the safety of STP 3 & 4.

It is also evident, from the collected subsurface investigation results, that STP 3 & 4 site soils are overconsolidated and are geologically old with respect to conventional liquefaction analysis. A very limited number of tests at isolated locations indicated potentially liquefiable soils; however, this indication could not be supported by the overwhelming percentages of the data that otherwise represent these soils as non-liquefiable. Moreover, the state-of-the-art methodology used for the liquefaction evaluation was intended to be conservative and not necessarily required to encompass every data point; therefore, the presence of a few data points beyond the CRR base curves is acceptable (Reference 2.5S.4-5). Additionally, in the liquefaction evaluation, the effects of overconsolidation and geologic age were not considered, both of which tend to increase resistance to liquefaction.

2.5S.4.8.2.7 Consultation with Regulatory Guide 1.198

Before and during the foregoing evaluation, RG 1.198 (Reference 2.5S.4-52) was consulted. The liquefaction evaluation presented here conforms closely to the RG 1.198 guidelines.

Under “Screening Techniques for Evaluation of Liquefaction Potential,” Reference 2.5S.4-52 lists the most commonly observed liquefiable soils as fluvial-alluvial deposits, eolian sands and silts, beach sands, reclaimed land, and uncompacted hydraulic fills. The geology at the STP site includes fluvial soils and man-made fill at very limited locations. The liquefaction evaluation included all STP 3 & 4 site soils. The man-made fill (Stratum A [Fill]), which is suspected at very limited locations, is removed during site grading operations. In the same section, Reference 2.5S.4-52 indicates that clay to silt, silty clay to clayey sand, or silty gravel to clayey gravel soils can be considered potentially liquefiable. This calculation treated all STP 3 & 4 site soils as potentially liquefiable, including the fine-grained soils. Note, however, that the finer-grained STP 3 & 4 site soils contain large percentages of fines, generally greatly exceeding the soils conventionally evaluated according to the state-of-the-art method, and/or are highly plastic, and are generally considered non-liquefiable. Additionally, in the liquefaction analyses, the groundwater level for calculation purposes was selected at El. 25.5 feet. This groundwater level is likely a “perched” condition within Stratum A, as measured in the Stratum C sand (refer to Figures 2.5S.4-55 and 2.5S.4-56). Note that a lower water level, measured in the deeper Stratum E sand, occurred at an average El. 16.5 feet (also refer to Figures 2.5S.4-55 and 2.5S.4-56). Despite the selected higher groundwater level, the calculated FOS against liquefaction overwhelmingly exceeded 1.10. Groundwater levels at STP 3 & 4 are not expected to rise in the future given the relief and topography, promoting positive drainage. Similarly, Reference 2.5S.4-52 indicates that potentially liquefiable soils may not pose a liquefaction risk to the facility if they are insufficiently thick and/or of limited lateral extent.

The separately discussed SPT tests (2 of 15 tests), CPT tests (13 of 153 tests), and shear wave velocity (V_S) tests (10 of 76 tests) tests that had FOS < 1.10, detailed above, are additionally all of limited thickness and/or lateral extent.

Under "Procedures for Evaluating Liquefaction Potential," Reference 2.5S.4-52 lists CPT, SPT, cyclic triaxial, and shear wave velocity tests as acceptable methods. The CPT, SPT, and shear wave velocity test results were used in these liquefaction potential analyses. Cyclic triaxial tests were not performed on STP 3 & 4 site soils, but were performed previously on STP 1 & 2 site soils (Reference 2.5S.4-3, Subsection 2.5.4.8.2.4), which are similar.

2.5S.4.9 Earthquake Site Characteristics

Refer to Subsection 2.5S.2.6 for a detailed discussion of the Ground Motion Response Spectrum (GMRS) basis.

2.5S.4.10 Static Stability

The following site-specific supplement addresses COL License Information Items 2.2, 2.35, 2.36, 2.37, 2.38, and 2.39.

As noted in Subsection 2.5S.4.5.2, a substantial amount of earthwork is required to establish site grades at STP 3 & 4. The proposed rough grade at the STP 3 and STP 4 areas is approximately El. 34 feet, and the proposed rough grade at the UHS Basin/RSW area is approximately El. 30 feet. A non-structural earth berm, raised to approximately El. 49 feet additionally surrounds the exterior of the UHS Basin. As noted above, the Reactor Buildings, Control Buildings, Radwaste Buildings (foundations), and the UHS Basin/RSW Pump Houses are all considered Seismic Category I structures. This subsection addresses the stability of foundation soils for those structures, the locations of which are shown on Figure 2.5S.4-2. The approximate structure dimensions, loads, and other details for these tunnel structures are included below for completeness. Other STP 3 & 4 major structures, including the Turbine Buildings and the Service Buildings, are not Seismic Category I structures, and are, therefore, not considered here.

Note that the RSW Lines (Seismic Category I piping), as currently planned, are contained within relatively shallow-depth cut-and-cover concrete tunnels. It is noted that the net unloading of the foundation soils due to excavation nearly equals the loading of the structure plus cover soil such that structure settlement in this instance can be considered negligible.

2.5S.4.10.1 STP 1 & 2 Foundations

The STP 1 & 2 UFSAR (Reference 2.5S.4-3) provides a description of the site soils and foundations for the STP 1 & 2 major structures. That information is summarized below.

STP 1 and STP 2 are essentially of identical design. The Reactor Containment Building (RCB) rests on a 166-foot-diameter mat foundation at approximately El. -31 ft, supported on undisturbed granular soils and compacted structural fill. The Fuel Handling Building (FHB) is approximately 88 feet by 190 feet in plan dimensions, with stepped foundation levels, ranging from approximately El. -36 feet to El. 14 feet. The deeper foundation levels of the FHB are on natural soils, while the shallower foundation levels of the FHB are on structural fill, in turn supported by Strata D and E. The Diesel Generator Building (DGB) is approximately 82 feet by 107 feet in plan dimensions, with foundations at approximately El. 20 feet, founded on structural fill, in turn supported by Stratum C in STP 1, and Stratum D in STP 2. The Auxiliary Feedwater Storage Tank (AFST) is 51 feet in diameter, supported on a mat foundation at approximately El. 19 feet, bearing on structural fill which extends into Strata C, D, and E. The foundation loading information for these structures (from the STP 1 & 2 UFSAR [Reference 2.5S.4-3]) is summarized below:

Structure	Gross Foundation Pressure (ksf)	Foundation El. (feet)	Net Foundation Pressure (ksf)
Reactor Containment Building	9.4	-31.2	2.0
Fuel Handling Building	4.4 to 9.2	-35.8 to 14.0	3.5 to -1.2
Diesel Generator Building	4.4	20.0	3.4
Aux. Feedwater Storage Tank	3.5	18.5	2.3

The gross foundation pressure was defined as dead plus equipment load. The net foundation pressure was defined as the gross foundation pressure less the overburden pressure.

The bearing capacity of STP 1 & 2 Seismic Category I foundations was analyzed using conventional and layered methods, with the groundwater level taken near the ground surface. The factors of safety against bearing capacity failure consistently exceeded a value of 3.0 for the long-term stability of foundations.

Foundation settlement analyses were also made. Foundation settlement monitoring was also undertaken during construction. Upper-bound predictions of foundation settlements, as well as measured settlements, were in the range of 2 inches to 3 inches subsequent to recovering the ground heave. Ground heave values were in the range of 3.5 inches to 5 inches.

2.5S.4.10.2 STP 3 & 4 Foundations, Subsurface Conditions, and Soil Properties

The STP 3 & 4 Seismic Category I structures, including their approximate foundation dimensions, elevations, and design pressures are indicated below (refer in part to Appendices 3H.1, 3H.2, and 3H.3). Note that the estimated foundation design pressures given are assumed

typical for these structures. Given the position of the groundwater level close to the ground surface and the foundation depth, buoyancy effects on the foundations were considered, leading to the effective foundation pressures also indicated below:

Structure [1]	Approximate Foundation Dimensions (feet)	Foundation El. [2] (feet)	Foundation Depth [2] (feet)	Estimated Foundation Design Pressure (ksf)	Effective Foundation Pressure (ksf)
Reactor Buildings	186 by 196	-51 {-61}	85 {95}	15.0	10.2 [3]
Control Buildings	79 by 184	-41	75	15.0	10.8
Radwaste Buildings	127 by 217	-20 {-35}	54 {69}	4.0	1.2
UHS Basin/ RSW Pump Houses	380 diameter	-2/-9	30/37	6.0 [4]	4.3
RSW Lines	39 wide	9 {5}	19 {33}	1.4 [4]	0.3

[1] All structures listed above are Seismic Category I structures (foundations for the Radwaste Buildings are considered Seismic Category I).

[2] Foundation Els. and soil strata designations shown in "{ }" symbols denote the Els. and conditions at the base of significant over-excavation at the particular structure (e.g., at the Reactor Buildings, Stratum F is over-excavated 10 feet below the underside of foundations, with over-excavation replaced by concrete fill; at the Radwaste Buildings, Stratum D is over-excavated 15 feet below the underside of foundations, with over-excavation replaced by structural fill; and at the RSW Lines, Stratum B is over-excavated 4 feet below the underside of tunnel foundations, with over-excavation replaced by structural fill).

[3] Value reduced to 9.2 ksf, as discussed later in this subsection.

[4] Estimated. Not available from Appendices 3H.1, 3H.2, and 3H.3.

The subsurface conditions at STP 3 & 4 are described in detail in Subsection 2.5S.4.2. The geotechnical engineering parameters of the various soil strata are similarly described in Subsection 2.5S.4.2, and are summarized in Table 2.5S.4-16. These parameters were used as the bases for the analyses of foundations, in addition to the properties of structural fill taken from the STP 1 & 2 UFSAR (Reference 2.5S.4-3). Structural fill properties were taken as: unit weight (γ) of 134 pcf, static elastic modulus (E) of 3000 ksf; drained friction angle (ϕ') of 43 degrees, and drained cohesion (c') of 0 ksf (Reference 2.5S.4-3).

For foundation evaluation purposes, specific subsurface profiles associated with each of the major structures, in both the STP 3 and STP 4 areas, were developed, as shown on Figures 2.5S.4-67 through 2.5S.4-70. Associated elevations and soil properties for these profiles are shown in Tables 2.5S.4-37A through 2.5S.4-40A. For depths below El. -180 feet, strata

boundary and soil property information was from the two deep borings (Borings B-305DH/DHA and B-405DH). Based on measurements at STP 3 & 4, the groundwater level was conservatively assumed at El. 25.5 feet (refer to Subsection 2.5S.4.6.1).

2.5S.4.10.3 STP 3 & 4 Bearing Capacity Evaluation

The ultimate bearing capacity, q_{ult}, of a foundation was calculated by (Reference 2.5S.4-54):

$$q_{ult} = c\, N_c\, \zeta_c + q\, N_q\, \zeta_q + 0.5\, \gamma'\, B\, N_g\, \zeta_g \qquad \text{Equation 2.5S.4-15}$$

where, $c\ (s_u)$ = undrained shear strength of the soil
q = effective overburden pressure at the foundation base
γ' = effective unit weight of the soil
B = foundation width
N_c, N_q, and N_g are bearing capacity factors
ζ_c, ζ_q, and ζ_g are shape factors

For rectangular foundations, the shape factors were given by Reference 2.5S.4-54 as:

$$\zeta_c = 1 + (B/L)\,(N_q/N_c) \qquad \text{Equation 2.5S.4-16}$$

$$\zeta_q = 1 + (B/L) \tan () \qquad \text{Equation 2.5S.4-17}$$

$$\zeta_g = 1 - 0.4\,(B/L) \qquad \text{Equation 2.5S.4-18}$$

where, B = foundation width
L = foundation length
 = friction angle of the soil

For square or circular foundations, the shape factors were given by Reference 2.5S.4-54 as:

$$\zeta_c = 1 + (N_q/N_c) \qquad \text{Equation 2.5S.4-19}$$

$$\zeta_q = 1 + \tan () \qquad \text{Equation 2.5S.4-20}$$

$$\zeta_g = 0.6 \qquad \text{Equation 2.5S.4-21}$$

The allowable bearing capacity, q_a, was derived as follows:

$$q_a = q_{ult}/FOS \qquad \text{Equation 2.5S.4-22}$$

where, FOS is the factor of safety.

The above bearing capacity formulation is based on the assumption that the soil within the zone of foundation deformation is uniform in terms of shear strength properties. The STP 3 & 4 site soils, however, are layered, and as such, this layering is considered in the evaluation of foundation bearing capacities. This issue of a layered subsurface has been addressed by several investigators. A simplified approach is to average the shear strength parameters in the foundation deformation zone, as proposed by References 2.5S.4-55 and 2.5S.4-56, and to use

the formulation in Reference 2.5S.4-54 (Equations 2.5S.4-15 through 2.5S.4-21). This approach was followed for estimating foundation bearing capacities, as described below.

Figure 2.5S.4-71 shows the typical failure wedge developed below a foundation, with the effective shear depth (i.e., the height of the failure wedge) as *H'*. Reference 2.5S.4-55 recommends using the weighted average of cohesion, c (s_u), and friction angle, , as follows:

$$c = \frac{\sum c_i H_i}{\sum H_i} \qquad \text{Equation 2.5S.4-23}$$

$$\tan(\phi) = \frac{\sum \tan(\phi_i) H_i}{\sum H_i} \qquad \text{Equation 2.5S.4-24}$$

where, c_i = cohesion of layer *i*
ϕ_i = friction angle of layer *i*
H_i = thickness of layer *i* within the effective shear depth *H'*

Equations 2.5S.4-23 and 2.5S.4-24 were used for deriving average shear strength properties for soils beneath each of the STP 3 & 4 foundations. The average material properties derived for each foundation are shown in Tables 2.5S.4-37B through 2.5S.40B.

Because different soils (e.g., clay, silt, and sand) were sometimes found at the same elevation across foundations, both soils were considered in the selection of average material properties. Where clay soils were present, the soil with the lowest cohesion, c (s_u), was used. Similarly, when silt or sand was present, the soil with the lowest friction angle, ϕ', was used. For conservatism, if structural fill was present along with other soil types, the properties of the stronger structural fill were ignored in estimating bearing capacity. Similarly, the properties of the stronger concrete fill below the Reactor Building foundations were ignored in estimating bearing capacity.

For each Reactor Building, where concrete fill is below the foundations, the pressure distribution at the base of the concrete fill (top of the natural soil) was calculated based on a 1:1 H/V distribution of stress through the concrete fill, as shown on Figure 2.5S.4-71. With a 10-foot-thickness of concrete fill, then, the pressure from each Reactor Building was distributed on an area having B = 186 feet + 20 feet = 206 feet, and L = 196 feet + 20 feet = 216 feet. Thus, the effective foundation pressure at the base of the concrete fill for each Reactor Building was estimated as {[(10.2 ksf) (186 feet) (196 feet)]/[(206 feet) (216 feet)]} + (0.150 ksf - 0.0624 ksf) (10 feet) = 9.2 ksf, using a unit weight of concrete fill of 0.150 ksf.

Foundation bearing capacities were estimated using the average material properties in Tables 2.5S.4-37B through 2.5S.4-40B and using Equations 2.5S.4-15 through 2.5S.4-21. A summary of the average material parameters, as well as the derived bearing capacity factors, are shown in Table 2.5S.4-41. Estimated bearing capacities are also shown in Table 2.5S.4-41. The results

of the analyses show that the allowable bearing capacity (using FOS=3) is higher than the effective foundation pressure for all structures, except for the STP 3 Reactor Building foundation, clay (Stratum F) subgrade case, which is discussed in more detail, below. The FOS values for all other subgrade cases ranged from approximately 5 (clay subgrade cases) to over 70 (sand subgrade cases), all indicative of a sufficient FOS against foundation bearing failure.

For the case of the STP 3 Reactor Building foundation, which has a mixed sand and clay subgrade, the FOS assuming a fully sand subgrade (i.e., were Stratum F is absent, as shown on Figure 2.5S.4-49, Section A) exceeds 4, whereas the FOS assuming a fully clay subgrade (i.e., when Stratum F is present, as shown on Figures 2.5S.4-49 and 2.5S.4-50, Sections A and B) is 2.9 (= 26.8/9.2). While the result for the fully clay subgrade case is slightly (i.e., less than 3%) below the typical FOS=3.0 commonly used, the FOS of the actual/ mixed subgrade case by inspection exceeds 3.0. Also, given the conservatism adopted in the selection of material parameters for the bearing capacity evaluation, it is concluded that the STP 3 Reactor Building foundation has sufficient safety against foundation bearing failure.

2.5S.4.10.4 Settlement

Foundation settlements were estimated using pseudo-elastic compression and one-dimensional consolidation. Based on a stress-strain model that computes settlement in discrete layers, the settlement, δ, of shallow foundations due to "elastic" compression of the subsurface materials was estimated as:

$$\delta = \Sigma\,(\Delta p_i\ \Delta h_i)/E_i \qquad \text{Equation 2.5S.4-25}$$

where, δ = settlement
i = 1 to n, where n is the number of layers
p_i = vertical applied pressure at center of layer i
h_i = thickness of layer i
E_i = elastic modulus of layer i

The stress distribution below rectangular, flexible foundations was based on a Boussinesq-type distribution and was calculated by Poulos and Davis (Reference 2.5S.4-57):

$$\sigma_z = (p/2\pi)\ \{\tan^{-1}\ [l\ b/(\ z\ R_3)] + (l\ b\ z/R_3)(1/R_1^{\,2} + 1/R_2^{\,2})\} \qquad \text{Equation 2.5S.4-26}$$

where, σ_z = calculated pressure at depth z
p = applied foundation pressure
l = length of the foundation
b = width of the foundation
z = depth below the foundation at which the pressure is calculated
$R_1 = (l^2 + z^2)^{0.5}$
$R_2 = (b^2 + z^2)^{0.5}$
$R_3 = (l^2 + b^2 + z^2)^{0.5}$

The vertical pressure under the center of a circular foundation, σ_z, was calculated by Poulos and Davis (Reference 2.5S.4-57):

$$\sigma_z = p\ \{1 - [1/(1 + (a/z)^2)]^{1.5}\} \qquad \text{Equation 2.5S.4-27}$$

where, a = diameter of the circular foundation and the other terms are as previously defined.

In applying Equation 2.5S.4-25 to layers with more than one soil type spanning across a particular foundation, the elastic moduli values, E, of the different soil types were compared and the lower value was selected. The E-values for the various soil strata are shown in Tables 2.5S.4-37A through 2.5S.4-40A. Note, however, that for the deepest considered stratum (Stratum N) because of its significant thickness, a composite E for the stratum was taken as a weighted average of the E of Sub-stratum N Clay, and the E of Sub-stratum N Sand, with weighting based on sub-strata thicknesses to total stratum thickness (i.e., [(4500 ksf) (228 feet) + (2100 ksf) (93.5 feet)]/(228 feet +93.5 feet) = 3802 ksf). Because Stratum N was found relatively deep below the foundations of major structures, the strain level in it is expected to be low, justifying the use of the relatively high composite E-value, calculated above. Also, in estimating elastic settlements, the compression of concrete fill below the Reactor Buildings was ignored due to its relative incompressibility in the range of loads being considered.

Spreadsheets were used for settlement calculations. The calculations were extended to a depth where the increase in vertical stress (Δp) due to the applied foundation pressure was less than or equal to 10% of the applied foundation pressure. Also, using a 1:2 H/V pressure distribution shown in Figure 2.5S.4-71, the applied vertical stresses below foundations were compared to the preconsolidation pressures (P_c') of the various soil strata. Where more than one soil type was present under a particular foundation, clay soils were selected over sand soils to represent the conditions of the layer, as discussed previously. Results showed that strata preconsolidation pressures typically exceeded the applied vertical stresses at mid-point of each layer, except for Stratum L (Clay) below the UHS Basin. At this foundation, Stratum L had P_c'=16 ksf, compared to the applied vertical stress at the mid-point of the layer of 16.1 ksf. While the difference in stress here was very small, and additionally considering that the stratum is only 5-feet-thick on average, the calculation nevertheless considered the virgin compression of the Stratum L, ΔS_c, at this particular foundation using (Reference 2.5S.4-58):

$$\Delta S_c = \frac{\Delta e}{1+e_0} H$$

Equation 2.5S.4-28

$$\Delta e = C_c \log\left(\frac{\sigma_0' + \Delta\sigma}{\sigma_0'}\right)$$

Equation 2.5S.4-29

where, H = thickness of the soil layer
e_0 = initial void ratio
Δe = void ratio change
C_c = compression index
σ_0' = initial effective overburden pressure
$\Delta\sigma$ = the increment in vertical stress

Foundation settlements were calculated based on Equations 2.5S.4-25 through 2.5S.4-29, the simplified subsurface profiles shown in Figures 2.5S.4-67 through 2.5S.4-70, and the material parameters shown in Tables 2.5S.4-37B though 2.5S.4-40B. Settlement estimates, which included the total settlement at the center and the edge of foundations, as well as their average values, are shown in Table 2.5S.4-42. With the exception of the foundations for the Radwaste

Buildings, where settlements of approximately 1 inch were estimated (due to the relatively light load of he structure), total settlements calculated at the centers of foundations for the Reactor Buildings, the Control Buildings, and the UHS Basin were in the range of approximately 7.5 to 9 inches. Note that the contribution of Stratum L virgin compression at the UHS Basin foundation, as expected, was minor (approximately 0.02 inches).

As an additional consideration, soil rebound/heave resulting from the maximum 95 feet of excavation (i.e., Reactor Buildings over-excavation to El. -61 feet), was estimated, with the calculated value in the range of approximately 3 inches to 5.5 inches. Note that soil rebound/heave at selected foundation excavations is monitored during construction.

As a guideline, tolerable total and differential settlements for mat foundations on clay subgrades are typically reported in the range of 3 inches and 1.5 inches, respectively. Note that tolerable total settlements as high as 5 inches have also been suggested for mat foundations (Reference 2.5S.4-55). Higher total settlements can be accommodated when critical connections to adjacent structures, utilities, and pavements can be delayed. Differential settlements are usually more important in the context of structure performance than total settlements, with acceptable angular distortions/tilts of the order of 1/300, generally reported for frame buildings (Reference 2.5S.4-55), to as low as 1/750 for foundations supporting sensitive machinery (Reference 2.5S.4-59), having been suggested.

Estimated differential settlement and angular distortion/tilt values (from center to edge of foundations for the referenced STP 3 & 4 structures) were as follows:

Structure	Estimated Differential Settlement (inches)	Estimated Angular Distortion/Tilt
Reactor Buildings	3.3	1/350
Control Building	2.5 to 3.2	1/250 to 1/320
Radwaste Buildings	0.4	1/1,860
UHS Basin/RSW Pump Houses	3.6	1/630
RSW Lines	Negligible	Negligible

Other than the Radwaste Buildings, with differential settlement values of approximately 0.4 inches, other foundations evaluated had estimated differential settlements in the range of approximately 2.5 inches to 3.5 inches (measured from center to edge of structure). From the differential settlement values, angular distortions/tilts were estimated (based on average foundation plan dimension), and for all evaluated structures were generally within the acceptable limit of 1/300. Except for the Radwaste Buildings, however, calculated angular distortion/tilt values exceeded the 1/750 criterion for the special case of foundations supporting sensitive machinery. It should be noted that despite the calculated 7.5 inches to 9 inches of total settlement for the referenced foundations, and the apparently high angular distortion/tilt values, actual angular distortion/tilt values are much less, given that a significant amount (i.e., more

than half) of foundation settlements are expected to have taken place by the time building superstructures are ready to receive equipment and/or piping. In this case, estimated angular distortion/tilt would similarly be one-half of those calculated above, or approximately 1/700 for the Reactor Buildings, 1/500 to 1/640 for the Control Buildings, and 1/1,260 for the UHS Basin. These are generally within the stricter criterion for the special case of foundations supporting sensitive machinery. Note, more significantly, that settlement estimates were based on the assumption of flexible mat foundations, not including the effects that thick, highly-reinforced concrete mat foundations have in mitigating differential settlements. To verify that foundations perform according to estimates, and to provide an ability to make corrections if needed, major structure foundations are monitored for movement during and after construction.

In general, the estimated foundation settlements are larger than those calculated for STP 1 & 2, as discussed in Subsection 2.5S.4.10.1. Given that subsurface conditions at STP 3 & 4 are comparable, the differences in calculated settlements are largely due to differences in net loading imposed on the subsurface soils, and differences in foundation sizes. For instance, each Reactor Containment Building at STP 1 & 2 was approximately 150-feet diameter, occupying a plan area of approximately 21,640 square feet, while each Reactor Building at STP 3 & 4 has a plan area of approximately 36,460 square feet, or approximately 70% larger than the plan area of an individual STP 1 & 2 structure. In addition, the net loading of each Reactor Containment Building at STP 1 & 2 was 2.0 ksf, while the effective foundation pressure of each Reactor Building at STP 3 & 4 is 9.2 ksf. The STP 3 & 4 larger foundation sizes and higher effective foundation pressures are expected to result in larger, albeit still tolerable, foundation settlements.

2.5S.4.10.5 Earth Pressures

Static and seismic lateral earth pressures are addressed here for below-grade walls. The development of seismic earth pressure diagrams is addressed generically. Passive earth pressures are not addressed here. As noted above, sources for structural fill materials, and their engineering properties, have not been conclusively established yet. As such, and to illustrate the earth pressure calculation method only, the following properties were assumed for structural fill: unit weight (γ) of 120 pcf and drained friction angle (ϕ') of 30 degrees. Actual structural fill properties, determined following sourcing of the materials, and following laboratory testing of those materials, are available at project detailed design stage.

Note additionally that a surcharge pressure of 500 psf was assumed in earth pressure calculations. The validity of this assumption is also reviewed at project detailed design stage. In particular note, as per Subsection 2.5S.4.5.2, the proposal to accommodate a heavy lift crane at the south edge of each Reactor Building. The imposed surcharge, and the foundation requirements for this specialty equipment are considered separately.

Lateral earth pressure increases due to compaction close to structures were not considered here. These are controlled at construction stage by limiting the size of compaction equipment within close proximity to below-grade walls. Note that the magnitude of compaction-induced earth pressure increases can only be assessed once a range of allowable equipment sizes and types has been selected/specified.

Earthquake-induced horizontal ground accelerations were included by the factor $k_h \cdot g$: a peak horizontal ground surface acceleration of 0.10g (refer to Subsection 2.5S.4.7.5) was applied. Vertical ground accelerations ($k_v \cdot g$) were considered negligible (Reference 2.5S.4-60).

2.5S.4.10.5.1 Static Lateral Earth Pressures

The static active earth pressure, p_{AS}, was estimated using (Reference 2.5S.4-60):

$$p_{AS} = K_{AS} \cdot \gamma \cdot z \qquad \text{Equation 2.5S.4-30}$$

where, K_{AS} = Rankine coefficient of static active lateral earth pressure
γ = unit weight of the structural fill (γ', effective unit weight when below the groundwater level)
z = depth below ground surface

The Rankine coefficient, K_{AS} , was calculated from:

$$K_{AS} = \tan^2 (45 - \phi'/2) \qquad \text{Equation 2.5S.4-31 (also Equation 2.5S.4-9, above)}$$

where, ϕ' = friction angle of the structural fill, in degrees.

The static at-rest earth pressure, p_{0S}, was estimated using (Reference 2.5S.4-12):

$$p_{0S} = K_{0S} \cdot \gamma \cdot z \qquad \text{Equation 2.5S.4-32}$$

where, K_{0S} = coefficient of at-rest static lateral earth pressure
γ = unit weight of the structural fill (γ', effective unit weight when below the groundwater level)
z = depth below ground surface

The coefficient, K_{0S} was calculated from:

$$K_{0S} = 1 - \sin (\phi') \qquad \text{Equation 2.5S.4-33 (also Equation 2.5S.4-11, above)}$$

where, ϕ' = friction angle of the structural fill, in degrees.

Hydrostatic groundwater pressures were considered for both active and at-rest static conditions. The hydrostatic pressure was calculated by:

$$p_W = \gamma_W \cdot z_w \qquad \text{Equation 2.5S.4-34}$$

where, p_w = hydrostatic pressure
z_w = depth below the groundwater level
γ_W = unit weight of water = 62.4 pcf

2.5S.4.10.5.2 Seismic Lateral Earth Pressures

The active seismic pressure, p_{AE}, was given by the Mononobe-Okabe equation (Reference 2.5S.4-60), represented by:

$p_{AE} = K_{AE} \cdot \gamma \cdot (H - z)$ Equation 2.5S.4-35

where, K_{AE} = coefficient of active seismic earth pressure = $K_{AE} - K_{AS}$
K_{AE} = Mononobe-Okabe coefficient of active seismic earth thrust (Equation 2.5S.4-36)
γ = unit weight of the structural fill at depth z
z = depth below the top of the structural fill
H = below-grade height of the wall

The coefficient K_{AE} was calculated from:

$K_{AE} = \cos^2 (\phi' - \theta)/\{\cos^2 \theta \cdot [1 + (\sin \phi' \sin (\phi' - \theta)/\cos (\theta))^{0.5}]^2\};$ Equation 2.5S.4-36

where, ϕ' = friction angle of the structural fill, in degrees
$\theta = \tan^{-1} (k_h)$
k_h = 0.10, as above

Note that ΔK_{AE} can be estimated using $3/4 \cdot k_h$ for k_h values less than about 0.25g, regardless of the angle of shearing resistance of the structural fill.

At-rest seismic pressures have been reported at up to three times as large as active earth pressures when calculated by the Mononobe-Okabe equation (Reference 2.5S.4-61).

Recognizing the limitations of the Mononobe-Okabe method for the design of below-grade structural walls, the evaluation of below-grade walls of specific Seismic Category I structures used either an alternate method described here (Reference 2.5S.4-62), or an elastic solution described in ASCE 4 (refer to Appendix 3H.6), to estimate seismic at-rest lateral earth pressures. The alternate method described here (Reference 2.5S.4-62) recognizes limited building wall movements due to the presence of floor diaphragms and the frequency content of the design motion, and uses the soil shear wave velocity and damping as input. It has been adopted for application to building design by the National Earthquake Hazard Reduction Program (NEHRP) (Reference 2.5S.4-63). To predict lateral seismic soil pressures for below-grade structure walls resting on firm foundations and assuming non-yielding walls, the method involves the following:

(1) Performing free-field soil column analysis and obtaining the ground response motion at the depth corresponding to the base of the wall in the free-field. The response motion in terms of acceleration response spectrum at 30% damping should be obtained. The free-field soil column analysis may be performed using the computer program SHAKE (Reference 2.5S.2-52), or similar dynamic methods, with input motion specified either at the ground surface or at the depth of the foundation mat. The choice of location of control motion is an important decision that is made

consistent with the development of the design motion. The location of input motion may significantly affect the dynamic response of the building and the seismic soil pressure amplitudes.

(2) Computing the total mass for a representative Single Degree of Freedom (SDOF) system using Poisson's ratio and the mass density of the soil, m:

$m = 0.5\ \gamma/g\ H^2\ \Psi_n$ Equation 2.5S.4-37

where, γ/g = total mass density of the structural fill
H = height of the wall
Ψ_n = factor to account for Poisson's ratio (μ), defined by

$\Psi_n = 2/[(1 - \mu)\ (2 - \mu)]^{0.5}$ Equation 2.5S.4-38

(3) Obtaining the lateral seismic force as the product of the total mass obtained from Step 2, and the acceleration spectral value of the free-field response at the soil column frequency obtained at the depth equal to the bottom of the wall from Step 1.

(4) Obtaining the maximum lateral seismic soil pressure at the ground surface by dividing the lateral force obtained from Step 3 by the area under the normalized seismic soil pressure, or 0.744 H.

(5) And finally, obtaining the soil pressure profile by multiplying the peak pressure from Step 4 by the following pressure distribution relationship:

$p(y) = -0.0015 + 5.05y - 15.84y^2 + 28.25y^3 - 24.59y^4 + 8.14y^5$ Equation 2.5S.4-39

where, y = normalized height ratio (Y/H), where "Y" is measured from bottom of the wall and Y/H ranges from a value of zero at the bottom of the wall to a value of 1.0 at the top of the wall. The area under the seismic soil pressure curve can be obtained from integration of the pressure distribution over the height of the wall. The total area is 0.744H P_{max} for a wall with a height of H and a maximum pressure of P_{max} at the top of the wall.

For well-drained backfills, seismic groundwater pressures need not be considered (Reference 2.5S.4-62). Since granular structural fill is used for STP 3 & 4, only hydrostatic pressures are considered, as given in Equation 2.5S.4-34. Note that seismic groundwater thrust greater than 35% of the hydrostatic thrust can develop for cases when k_h>0.30g (Reference 2.5S.4-64). Given the relatively low seismicity at STP 3 & 4 (i.e., k_h much < 0.30g), seismic groundwater considerations can be ignored.

2.5S.4.10.5.3 Lateral Earth Pressures Due to Surcharge

Lateral earth pressures as a result of surcharge applied at the ground surface at the top of a below-grade wall, p_{sur}, were calculated using the following:

$$p_{sur} = K\,q \qquad \text{Equation 2.5S.4-40}$$

where, K = earth pressure coefficient; K_{AS} for active; K_0 for at-rest; ΔK_{AE} or ΔK_{oE} for seismic loading, depending on the nature of the loading
q=uniform surcharge pressure

2.5S.4.10.5.4 Sample Earth Pressure Diagrams

Using the relationships outlined and the assumed structural fill properties, above, sample earth pressures were estimated. Sample earth pressure diagrams are provided on Figures 2.5S.4-72 and 2.5S.4-73 for the maximum 85-foot wall height, level ground surface, and groundwater level at the ground surface. As above, to illustrate the earth pressure calculation method only, structural fill properties (granular soils) were conservatively taken as unit weight (γ) of 120 pcf and drained friction angle (ϕ') of 30 degrees; the peak horizontal ground surface acceleration was taken as 0.10g; and, a permanent uniform surcharge load of 500 psf was included.

Actual surcharge loads, structural fill properties, and final configurations of structures are not known at this time. Final earth pressure calculations are prepared at project detailed design stage based on the actual design conditions at each structure, on a case-by-case basis. STP commits to include the final earth pressure calculations, including actual surcharge loads, structural fill properties, and final configuration of structures, following completion of the project detailed design in an update to the FSAR in accordance with 10CFR 50.71(e) (COM 2.5S-3).

2.5S.4.10.6 Selected Design Parameters and Results Overview

Field testing and laboratory testing results from the STP 3 & 4 subsurface investigation are discussed in Subsection 2.5S.4.2. The parameters employed for bearing capacity, settlement, and earth pressure evaluations are based on the material characterization addressed in Subsection 2.5S.4.2, and as summarized in Table 2.5S.4-16. The parameters reflected in that table were conservatively selected, as discussed in Subsection 2.5S.4.2. An angle of shearing resistance of 30 degrees was used for characterization of structural fill for earth pressure evaluations, which is considered conservative for granular fills compacted to 95 percent modified Proctor compaction. The groundwater level was selected at El. 25.5 feet for dynamic analyses, liquefaction analyses, and bearing capacity/settlement analyses, whereas a groundwater level at ground surface (rough grade at El. 34 feet) was conservatively adopted for developing sample earth pressure diagrams. The FOS calculated against bearing capacity failure of foundations at major structures typically exceeded 3.0, where a value of 3.0 is commonly considered adequate for foundation stability. Similarly, a peak horizontal ground surface acceleration of 0.10g was used both for liquefaction analyses and for seismic earth pressure analyses. This value was determined based on site-specific seismologic and soil dynamics analyses, as discussed in Subsection 2.5S.4.7.5.

2.5S.4.11 Design Criteria

Geotechnical criteria employed in the evaluation of each topic are addressed in the respective subsections, above, for the particular issue under consideration. The criteria summarized below are geotechnical criteria and also geotechnical-related criteria that pertain to structural design.

Subsection 2.5S.4.8 uses an FOS against liquefaction for the site soils. Under "Factor of Safety Against Liquefaction," Reference 2.5S.4-52 indicates that FOS < 1.10 is generally considered a trigger value, FOS≈1.10 to 1.40 is considered intermediate, and FOS≥1.40 is considered high. As used in Subsection 2.5S.4.8, an FOS of 1.10 was considered a threshold value to evaluate the potential effects of liquefaction of site soils. On this same issue, the Committee on Earthquake Engineering of the National Research Council (Reference 2.5S.4-53) stated that "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." This position, and the FOS < 1.10 trigger value from Reference 2.5S.4-52, is consistent with the value selected for the analyses of STP 3 & 4 site soils, also considering the conservatism employed in ignoring overconsolidation, the geologic age of the deposits, and other factors noted above.

Subsection 2.5S.4.10 specifies and discusses allowable bearing capacity and settlement values for site soils and for planned structures. Table 2.5S.4-41 provides allowable bearing capacities for Seismic Category I structures. Generally, a minimum FOS=3.0 was used when applying bearing capacity equations. This FOS can also be applied against breakout failure due to uplift forces on buried piping. This FOS can be reduced to 2.25 when dynamic or transient loading conditions apply. Table 2.5S.4-42 shows estimated structure total settlements under assumed foundation loads. As a guideline, if total and differential settlements are limited to 3 inches (up to 5 inches) and 1.5 inches, respectively, for mat foundations (and angular distortions/tilts do not exceed 1/300, or 1/750 for foundations supporting sensitive machinery), then settlements do not impact foundation performance. Higher total settlements can be accommodated when critical connections to adjacent structures, utilities, and paving can be delayed. Similarly as a guideline, if total and differential settlements are limited to 1 inch and 0.5 inches, respectively, for footing foundations, then settlement do not impact foundation performance.

Subsection 2.5S.4.10 also addresses criteria for static and seismic earth pressure estimation. The lateral earth pressure diagrams are shown on Figures 2.5S.4-72 and 2.5S.4-73, are best estimates, and thus have a FOS=1.0. A FOS=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 structures. There may be situations where such foundations are used for non-Seismic Category I, as determined at project detailed design stage. For axial pile and pier design capacity, a FOS=3.0 is used for the end bearing component, and a FOS=2.0 is used for skin friction. For lateral loading, the maximum allowable lateral load is taken as one-half of the load that produces 1 inch of lateral movement on the head of the pile, adjusted for pile spacing and for pile head fixity.

Subsection 2.5S.5.2 specifies and discusses the minimum acceptable static and seismic factors of safety for slopes, where such occur in the permanent STP 3 & 4 development.

2.5S.4.12 Techniques to Improve Subsurface Conditions

As noted in Subsections 2.5S.4.5 and 2.5S.4.10, major STP 3 & 4 structures (including Seismic Category I structures and/or piping) derive support from: dense sand subgrade soils; stiff to very stiff clay subgrade soils; concrete fill; or, compacted structural fill. Given the planned foundation depths, and the subsurface conditions occurring at those depths, as shown in part on Figures 2.5S.4-49 through 2.5S.4-54, special ground improvement measures are not deemed necessary. Ground treatment is limited to localized over-excavation of unsuitable soils, such as suspected fill and/or minor zones of loose/soft soils occurring at foundation subgrades, and their replacement with structural fill.

Over-excavation of 10 feet at the STP 3 & 4 Reactor Buildings (partially removing Stratum F), 17 feet at the STP 3 & 4 Radwaste Buildings (removing Stratum D), and 3 feet at the RSW Lines (removing Stratum B) is proposed to replace soils not adequate to bear design loads with the required FOS. Note that these structures have over-excavation backfilled with concrete fill (STP 3 & 4 Reactors Buildings), or with compacted structural fill (STP 3 & 4 Radwaste Buildings; RSW Lines). In addition, a general over-excavation of 2 feet, and backfilling with compacted structural fill, at the STP 3 & 4 Control Buildings, the STP 3 & 4 Turbine Buildings, and the UHS Basin/RSW Pump Houses, is proposed to ensure a firm subgrade for construction activities. While the foundations for these latter structures occur within dense sand strata (Stratum C or Stratum E) at depth, these are generally silty very fine sands occurring below the normal groundwater level, and may remain highly saturated (and difficult to work on initially) even following construction dewatering. For all affected structures, both concrete fill and structural fill are placed according to engineering specifications and quality control/quality assurance testing procedures established at project detailed design stage.

Ground improvement measures also include proof-rolling of foundation subgrades for the purpose of identifying any unsuitable soils for further over-excavation and replacement. In the absence of adverse subsurface conditions at STP 3 & 4 requiring significant ground improvement work, the primary focus is on maintaining the integrity of the existing dense sand and stiff to very stiff clay foundation subgrade soils during earthworks, and following on to subgrade preparation to receive foundations. These measures include such steps as groundwater control, the use of appropriate measures and equipment for excavation and compaction, subgrade protection (among other things, by concrete fill or by structural fill, as noted above), and other similar measures.

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Table 2.5S.4-1 Field Testing Summary

Field Test	Industry Standard	Number Of Tests
Borings (B)	References 2.5S4-22 and 2.5S.4-24	120
SPT Hammer Energy Measurements	References 2.5S4-6 and 2.5S.4-25	46
Cone Penetration Tests (C)	Reference 2.5S.4-26	32
Observation Wells (OW)	Reference 2.5S.4-27	28
Test Pits (TP)	No Standard	6
Field Electrical Resistivity Arrays (ER)	References 2.5S.4-29 and 2.5S.4-30	4
Suspension P-S Velocity Logging	Reference 2.5S.4-47	10

Table 2.5S.4-2 Summary of Soil Strata Thicknesses and Base Elevations

		STP 3		STP 4		UHS Basin/ RSW		Inside Power Block		Outside Power Block		Site-Wide	
Stratum	Range	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)
A (Fill)	Minimum	23.7	1.0	28.5	0.5	23.3	0.5	23.3	0.5	23.1	1.0	23.1	0.5
	Maximum	30.2	4.5	30.5	3.0	30.1	4.5	30.5	4.5	29.5	13.5	30.5	13.5
	Average	27.8	1.8	29.6	1.7	27.6	2.2	28.5	1.8	26.7	4.4	28.2	2.1
A	Minimum	-0.3	7.5	2.6	9.6	4.7	13.5	-0.3	7.5	-1.9	6.0	-1.9	6.0
	Maximum	23.1	28.5	21.5	28.5	11.9	24.0	23.1	28.5	20.8	25.0	23.1	28.5
	Average	13.5	15.7	10.6	19.9	8.7	20.3	11.5	18.1	11.4	17.0	11.5	18.0
B	Minimum	-2.4	0.5	-3.8	1.0	-9.0	2.0	-9.0	0.5	-6.7	3.7	-9.0	0.5
	Maximum	14.2	16.0	13.6	14.0	9.8	15.0	14.2	16.0	10.9	27.5	14.2	27.5
	Average	6.2	7.8	4.9	5.9	0.9	7.9	4.8	7.1	3.0	10.0	4.7	7.3
C	Minimum	-23.7	14.5	-21.9	3.1	-19.8	5.0	-23.7	3.1	-18.6	5.0	-23.7	3.1
	Maximum	-9.0	30.0	10.3	27.1	-9.5	25.1	10.3	30.0	-7.5	29.5	10.3	30.0
	Average	-16.1	22.2	-13.7	18.7	-13.8	15.4	-14.8	19.7	-12.7	16.0	-14.6	19.4
D	Minimum	-45.4	9.0	-43.7	15.0	-45.3	2.0	-45.4	1.5	-39.9	0.9	-45.4	9.0
	Maximum	-26.0	31.3	-28.6	30.7	-18.8	34.0	-18.4	34.0	-9.8	30.0	-16.9	34.0
	Average	-37.0	20.9	-36.4	22.0	-37.5	22.8	-37.0	21.8	-33.3	21.3	-36.7	21.7
E	Minimum	-71.1	9.4	-66.7	5.0	-51.3	6.0	-71.8	1.5	-70.9	5.0	-71.1	5.0
	Maximum	-48.0	35.8	-42.9	30.0	-51.3	6.0	-36.6	36.5	-21.9	40.5	-21.9	35.8
	Average	-59.9	23.3	-50.2	13.8	-51.3	6.0	-54.7	18.1	-50.8	19.1	-54.5	18.1

Table 2.5S.4-2 Summary of Soil Strata Thicknesses and Base Elevations (Continued)

		STP 3		STP 4		UHS Basin/ RSW		Inside Power Block		Outside Power Block		Site-Wide	
Stratum	Range	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)
F	Minimum	-80.8	3.5	-78.7	4.0	-93.1	27.1	-93.1	0.7	-96.4	6.5	-93.1	3.5
	Maximum	-53.0	25.0	-47.8	30.0	-67.6	55.0	-47.8	55.0	-46.9	61.5	-46.9	55.0
	Average	-70.3	11.8	-66.5	16.8	-76.7	39.0	-68.4	16.1	-59.6	28.3	-68.0	16.7
H	Minimum	-93.5	9.5	-90.3	4.5	-	-	-94.6	0.9	-93.9	15.0	-93.9	4.5
	Maximum	-80.3	34.5	-64.5	35.5	-	-	-63.1	35.5	-91.9	45.0	-64.5	45.0
	Average	-88.7	19.0	-85.7	15.9	-	-	-87.2	17.5	-92.9	33.3	-87.7	18.8
J Clay 1	Minimum	-131.9	15.5	-127.3	20.0	-	-	-131.9	10.0	-	-	-131.9	10.0
	Maximum	-114.6	49.0	-107.2	40.0	-	-	-107.2	49.0	-	-	-107.2	49.0
	Average	-121.8	29.6	-117.0	28.3	-	-	-119.3	29.0	-	-	-119.3	29.0
J Inter-bed 1	Minimum	-110.7	5.5	-106.7	5.5	-	-	-110.7	5.5	-	-	-110.7	5.5
	Maximum	-98.0	10.0	-93.9	9.5	-	-	-93.9	10.0	-	-	-93.9	10.0
	Average	-107.8	9.3	-102.3	8.0	-	-	-106.4	9.0	-	-	-106.4	9.0
J Sand 1	Minimum	-140.8	9.5	-128.8	1.3	-	-	-140.8	1.3	-	-	-140.8	1.3
	Maximum	-128.7	25.5	-118.7	21.5	-	-	-118.7	25.5	-	-	-118.7	25.5
	Average	-135.5	15.2	-126.9	12.1	-	-	-131.2	13.7	-	-	-131.2	13.7
J Inter-bed 2	Minimum	-161.9	9.5	-168.6	8.7	-	-	-168.6	8.7	-	-	-168.6	8.7
	Maximum	-140.3	20.5	-147.4	30.3	-	-	-140.3	30.3	-	-	-140.3	30.3
	Average	-155.0	16.2	-156.1	13.8	-	-	-155.5	15.1	-	-	-155.5	15.1
J Clay 2	Minimum	-183.2	21.0	-185.0	48.1	-	-	-185.0	21.0	-	-	-185.0	21.0
	Maximum	-167.5	34.5	-185.0	48.1	-	-	-167.5	48.1	-	-	-167.5	48.1
	Average	-172.1	28.9	-185.0	48.1	-	-	-173.9	32.7	-	-	-173.9	32.7

Table 2.5S.4-2 Summary of Soil Strata Thicknesses and Base Elevations (Continued)

		STP 3		STP 4		UHS Basin/ RSW		Inside Power Block		Outside Power Block		Site-Wide	
Stratum	Range	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)
K Clay	Minimum	-198.2	15.0	-207.4	22.4	-	-	-207.4	15.0	-	-	-207.4	15.0
	Maximum	-198.2	15.0	-207.4	22.4	-	-	-198.2	22.4	-	-	-198.2	22.4
	Average	-198.2	15.0	-207.4	22.4	-	-	-202.8	18.7	-	-	-202.8	18.7
K Sand/ Silt	Minimum	-228.7	30.5	-227.4	20.0	-	-	-228.7	20.0	-	-	-228.7	20.0
	Maximum	-228.7	30.5	-227.4	20.0	-	-	-227.4	30.5	-	-	-227.4	30.5
	Average	-228.7	30.5	-227.4	20.0	-	-	-228.1	25.3	-	-	-228.1	25.3
L	Minimum	-234.2	5.5	-231.9	4.5	-	-	-234.2	4.5	-	-	-234.2	4.5
	Maximum	-234.2	5.5	-231.9	4.5	-	-	-231.9	5.5	-	-	-231.9	5.5
	Average	-234.2	5.5	-231.9	4.5	-	-	-233.1	5.0	-	-	-233.1	5.0
M	Minimum	-248.7	14.5	-247.4	15.5	-	-	-248.7	14.5	-	-	-248.7	14.5
	Maximum	-248.7	14.5	-247.4	15.5	-	-	-247.4	15.5	-	-	-247.4	15.5
	Average	-248.7	14.5	-247.4	15.5	-	-	-248.1	15.0	-	-	-248.1	15.0
N Clay 1	Minimum	-310.2	61.5	-303.9	56.5	-	-	-310.2	56.5	-	-	-310.2	56.5
	Maximum	-310.2	61.5	-303.9	56.5	-	-	-303.9	61.5	-	-	-303.9	61.5
	Average	-310.2	61.5	-303.9	56.5	-	-	-307.1	59.0	-	-	-307.1	59.0
N Sand 1	Minimum	-326.2	16.0	-321.9	18.0	-	-	-326.2	16.0	-	-	-326.2	16.0
	Maximum	-326.2	16.0	-321.9	18.0	-	-	-321.9	18.0	-	-	-321.9	18.0
	Average	-326.2	16.0	-321.9	18.0	-	-	-324.1	17.0	-	-	-324.1	17.0
N Clay 2	Minimum	-331.2	5.0	-332.9	11.0	-	-	-332.9	5.0	-	-	-332.9	5.0
	Maximum	-331.2	5.0	-332.9	11.0	-	-	-331.2	11.0	-	-	-331.2	11.0
	Average	-331.2	5.0	-332.9	11.0	-	-	-332.1	8.0	-	-	-332.1	8.0

Table 2.5S.4-2 Summary of Soil Strata Thicknesses and Base Elevations (Continued)

		STP 3		STP 4		UHS Basin/ RSW		Inside Power Block		Outside Power Block		Site-Wide	
Stratum	Range	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)
N Sand 2	Minimum	-370.2	39.0	-358.9	26.0	-	-	-370.2	26.0	-	-	-370.2	26.0
	Maximum	-370.2	39.0	-358.9	26.0	-	-	-358.9	39.0	-	-	-358.9	39.0
	Average	-370.2	39.0	-358.9	26.0	-	-	-364.6	32.5	-	-	-364.6	32.5
N Clay 3	Minimum	-377.2	7.0	-368.9	10.0	-	-	-377.2	7.0	-	-	-377.2	7.0
	Maximum	-377.2	7.0	-368.9	10.0	-	-	-368.9	10.0	-	-	-368.9	10.0
	Average	-377.2	7.0	-368.9	10.0	-	-	-373.1	8.5	-	-	-373.1	8.5
N Sand 3	Minimum	-394.2	17.0	-388.9	20.0	-	-	-394.2	17.0	-	-	-394.2	17.0
	Maximum	-394.2	17.0	-388.9	20.0	-	-	-388.9	20.0	-	-	-388.9	20.0
	Average	-394.2	17.0	-388.9	20.0	-	-	-391.6	18.5	-	-	-391.6	18.5
N Clay 4	Minimum	-419.2	25.0	-423.9	35.0	-	-	-423.9	25.0	-	-	-423.9	25.0
	Maximum	-419.2	25.0	-423.9	35.0	-	-	-419.2	35.0	-	-	-419.2	35.0
	Average	-419.2	25.0	-423.9	35.0	-	-	-421.6	30.0	-	-	-421.6	30.0
N Sand 4	Minimum	-435.2	16.0	-	-	-	-	-435.2	0.0	-	-	-435.2	0.0
	Maximum	-435.2	16.0	-	-	-	-	-435.2	16.0	-	-	-435.2	16.0
	Average	-435.2	16.0	-	-	-	-	-435.2	8.0	-	-	-435.2	8.0
N Clay 5	Minimum	-493.2	58.0	-473.9	50.0	-	-	-493.2	50.0	-	-	-493.2	50.0
	Maximum	-493.2	58.0	-473.9	50.0	-	-	-473.9	58.0	-	-	-473.9	58.0
	Average	-493.2	58.0	-473.9	50.0	-	-	-483.6	54.0	-	-	-483.6	54.0
N Sand 5	Minimum	-	-	-508.9	35.0	-	-	-508.9	0.0	-	-	-508.9	0.0
	Maximum	-	-	-508.9	35.0	-	-	-508.9	35.0	-	-	-508.9	35.0
	Average	-	-	-508.9	35.0	-	-	-508.9	17.5	-	-	-508.9	17.5

Table 2.5S.4-2 Summary of Soil Strata Thicknesses and Base Elevations (Continued)

		STP 3		STP 4		UHS Basin/ RSW		Inside Power Block		Outside Power Block		Site-Wide	
Stratum	Range	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)	Base El. [1] (feet)	Thickness (feet)
N Clay 6	Minimum	-570.2	77.0	-568.9	60.0	-	-	-570.2	60.0	-	-	-570.2	60.0
	Maximum	-570.2	77.0	-568.9	60.0	-	-	-568.9	77.0	-	-	-568.9	77.0
	Average	-570.2	77.0	-568.9	60.0	-	-	-569.6	68.5	-	-	-569.6	68.5
Values for Use													
A (Fill)		28.0	2.0	29.5	1.5	27.5	2.0	28.5	2.0	26.5	4.5	28.0	2.0
A		13.5	15.5	10.5	20.0	8.5	20.5	11.5	18.0	11.5	17.0	11.5	18.0
B		6.0	8.0	5.0	6.0	1.0	8.0	5.0	7.0	3.0	10.0	4.5	7.5
C		-16.0	22.0	-13.5	18.5	-14.0	15.5	-15.0	19.5	-12.5	16.0	-14.5	19.5
D		-37.0	21.0	-36.5	22.0	-37.5	23.0	-37.0	22.0	-33.5	21.5	-36.5	21.5
E		-60.0	23.5	-50.0	14.0	-51.5	6.0	-54.5	18.0	-51.0	19.0	-54.5	18.0
F		-70.5	12.0	-66.5	17.0	-76.5	39.0	-68.5	16.0	-59.5	28.5	-68.0	16.5
H		-88.5	19.0	-85.5	16.0	-	-	-87.0	17.5	-93.0	33.5	-87.5	19.0
J Clay 1		-122.0	29.5	-117.0	28.5	-	-	-119.5	29.0	-	-	-119.5	29.0
J Interbed 1		-108.0	9.5	-102.5	8.0	-	-	-106.5	9.0	-	-	-106.5	9.0
J Sand 1		-135.5	15.0	-127.0	12.0	-	-	-131.0	13.5	-	-	-131.0	13.5
J Interbed 2		-155.0	16.0	-156.0	14.0	-	-	-155.5	15.0	-	-	-155.5	15.0
J Clay 2	-172.0	29.0	-185.0	48.0	-	-	-174.0	32.0	-	-	-174.0	32.0	J Clay 2
K Clay	-198.2	15.0	-207.4	22.4	-	-	-202.8	18.7	-	-	-203.0	19.0	K Clay
K Sand/ Silt	-228.5	30.5	-227.5	20.0	-	-	-228.0	25.5	-	-	-228.0	25.5	K Sand/ Silt
L	-234.0	5.5	-232.0	4.5	-	-	-233.0	5.0	-	-	-233.0	5.0	L
M	-248.5	14.5	-247.5	15.5	-	-	-248.0	15.0	-	-	-248.0	15.0	M

Table 2.5S.4-2 Summary of Soil Strata Thicknesses and Base Elevations (Continued)

		STP 3		STP 4		UHS Basin/ RSW		Inside Power Block		Outside Power Block		Site-Wide	
Stratum	**Range**	**Base El. [1] (feet)**	**Thickness (feet)**	**Base El. [1] (feet)**	**Thickness (feet)**	**Base El. [1] (feet)**	**Thickness (feet)**	**Base El. [1] (feet)**	**Thickness (feet)**	**Base El. [1] (feet)**	**Thickness (feet)**	**Base El. [1] (feet)**	**Thickness (feet)**
N Clay 1	-310.0	61.5	-304.0	56.5	-	-	-307.0	59.0	-	-	-307.0	59.0	N Clay 1
N Sand 1	-326.0	16.0	-322.0	18.0	-	-	-324.0	17.0	-	-	-324.0	17.0	N Sand 1
N Clay 2	-331.0	5.0	-333.0	11.0	-	-	-332.0	8.0	-	-	-332.0	8.0	N Clay 2
N Sand 2	-370.0	39.0	-359.0	26.0	-	-	-364.5	32.5	-	-	-364.5	32.5	N Sand 2
N Clay 3	-377.0	7.0	-369.0	10.0	-	-	-373.0	8.5	-	-	-373.0	8.5	N Clay 3
N Sand 3	-394.0	17.0	-389.0	20.0	-	-	-391.5	18.5	-	-	-391.5	18.5	N Sand 3
N Clay 4	-419.0	25.0	-424.0	35.0	-	-	-421.5	30.0	-	-	-421.5	30.0	N Clay 4
N Sand 4	-435.0	16.0	-	-	-	-	-429.5	8.0	-	-	-429.5	8.0	N Sand 4
N Clay 5	-493.0	58.0	-474.0	50.0	-	-	-475.5	54.0	-	-	-475.5	54.0	N Clay 5
N Sand 5	-	-	-509.0	35.0	-	-	-493.0	17.5	-	-	-493.0	17.5	N Sand 5
N Clay 6	-570.0	77.0	-569.0	60.0	-	-	-569.5	68.5	-	-	-569.5	68.5	N Clay 6

[1] Elevations are referenced to NGVD 29 datum.

Table 2.5S.4-3 Summary of Uncorrected SPT N-Values [1]

Stratum	Range	STP 3	STP 4	UHS Basin/RSW	Inside Power Block	Outside Power Block	Site-Wide
A (Fill)	No. of Tests	14	11	4	29	11	40
	Minimum	4	2	8	2	5	2
	Maximum	10	13	22	22	14	22
	Average	8	8	12	9	8	8
A	No. of Tests	435	417	157	1,009	90	1,099
	Minimum	0	3	3	0	4	0
	Maximum	27	42	41	42	38	42
	Average	9	11	11	10	12	10
B	No. of Tests	102	47	27	176	22	198
	Minimum	2	3	2	2	3	2
	Maximum	23	40	17	40	18	40
	Average	7	12	9	9	8	9
C	No. of Tests	227	150	40	417	27	444
	Minimum	0	3	7	0	4	0
	Maximum	109	120	67	120	48	120
	Average	27	23	24	25	23	25
D	No. of Tests	207	179	58	444	40	484
	Minimum	7	3	5	3	7	3
	Maximum	34	34	54	54	34	54
	Average	16	15	15	15	15	15
E	No. of Tests	231	117	1	349	23	372
	Minimum	7	11	51	7	15	7
	Maximum	88	84	51	88	67	88
	Average	34	39	51	36	34	35
F	No. of Tests	48	113	81	242	49	291
	Minimum	11	11	12	11	9	9
	Maximum	98	63	32	98	56	98
	Average	23	22	19	22	24	22
H	No. of Tests	56	58	2	116	14	130
	Minimum	15	14	57	14	18	14
	Maximum	100	150	74	150	63	150
	Average	42	47	66	45	32	44

Table 2.5S.4-3 Summary of Uncorrected SPT N-Values [1] (Continued)

Stratum	Range	STP 3	STP 4	UHS Basin/RSW	Inside Power Block	Outside Power Block	Site-Wide
J Clay	No. of Tests	113	99	Not Reached	212	3	215
	Minimum	12	14	Not Reached	12	36	12
	Maximum	89	120	Not Reached	120	58	120
	Average	30	32	Not Reached	31	44	31
J Sand	No. of Tests	43	32	Not Reached	75	Not Reached	75
	Minimum	22	18	Not Reached	18	Not Reached	18
	Maximum	120	120	Not Reached	120	Not Reached	120
	Average	73	56	Not Reached	65	Not Reached	65
K Clay	No. of Tests	1	1	Not Reached	Not Reached	Not Reached	2
	Minimum	15	15	Not Reached	Not Reached	Not Reached	15
	Maximum	15	15	Not Reached	Not Reached	Not Reached	15
	Average	15	15	Not Reached	Not Reached	Not Reached	15
K Sand/Silt	No. of Tests	1	1	Not Reached	Not Reached	Not Reached	2
	Minimum	120	30	Not Reached	Not Reached	Not Reached	30
	Maximum	120	30	Not Reached	Not Reached	Not Reached	120
	Average	120	30	Not Reached	Not Reached	Not Reached	75
L	No. of Tests	1	1	Not Reached	Not Reached	Not Reached	2
	Minimum	24	21	Not Reached	Not Reached	Not Reached	21
	Maximum	24	21	Not Reached	Not Reached	Not Reached	24
	Average	24	21	Not Reached	Not Reached	Not Reached	23
M	No. of Tests	0	0	Not Reached	Not Reached	Not Reached	0
	Minimum	Not Tested	Not Tested	Not Reached	Not Reached	Not Reached	Not Tested
	Maximum	Not Tested	Not Tested	Not Reached	Not Reached	Not Reached	Not Tested
	Average	Not Tested	Not Tested	Not Reached	Not Reached	Not Reached	Not Tested
N Clay	No. of Tests	14	12	Not Reached	Not Reached	Not Reached	26
	Minimum	21	2	Not Reached	Not Reached	Not Reached	2
	Maximum	47	46	Not Reached	Not Reached	Not Reached	47
	Average	31	34	Not Reached	Not Reached	Not Reached	33
N Sand	No. of Tests	4	4	Not Reached	Not Reached	Not Reached	8
	Minimum	20	49	Not Reached	Not Reached	Not Reached	20
	Maximum	200	200	Not Reached	Not Reached	Not Reached	200
	Average	85	110	Not Reached	Not Reached	Not Reached	97

[1] All SPT N-values in blows/foot

Table 2.5S.4-4 Summary of Energy Transfer Ratios/Hammer Energy Corrections

Drilling Rig	Number Of Measurements	ETR Range (%)	ETR Average [1] (%)	Hammer Energy Correction (ETR%/60%)
Best Failing 1500 Truck Rig	4	70-75	73	1.22
Environmental Exploration CME 750 ATV	5	79-84	82	1.36
Gregg Fraste Track Rig	3	79-80	80	1.33
Gregg CME 55 Truck Rig	3	86-88	87	1.45
Jedi CME 75 Truck Rig	5	71-77	75	1.25
Lewis Environmental Mobile B57 (pre-12/08/2006) [2]	5	90-107	99	1.65
Lewis Environmental Mobile B57 (post-12/08/2006) [2]	5	83-89	87	1.45
Lewis Environmental Mobile B61 (post-12/16/2006) (2)	3	94-98	96	1.60
MACTEC D50 ATV Rig	4	69-74	72	1.21
MACTEC CME 45 Trailer Rig	5	74-84	83	1.38
Miller CME 750 ATV	4	83-86	85	1.41

[1] Energy Transfer Ratio (ETR) = the percent of measured SPT hammer energy versus the theoretical SPT hammer energy (350 foot-pounds)

[2] The Lewis Environmental SPT hammer was initially mounted on the Mobile B57 drilling rig. The hammer was serviced on 12/08/2006, and was moved to the Mobile B61 drilling rig on 12/16/2006

Table 2.5S.4-5 Summary of Corrected SPT $(N_1)_{60}$-Values [1]

Stratum	Range	STP 3	STP 4	UHS Basin/ RSW	Inside Power Block	Outside Power Block	Site-Wide
A (Fill)	No. of Tests	14	11	4	29	11	40
	Minimum	8	4	13	4	8	4
	Maximum	21	25	36	36	26	36
	Average	15	16	21	16	14	15
A	No. of Tests	435	417	157	1,009	90	1,099
	Minimum	0	5	6	0	6	0
	Maximum	51	69	79	79	61	79
	Average	14	18	18	17	20	17
B	No. of Tests	102	47	27	176	22	198
	Minimum	3	4	3	3	5	3
	Maximum	34	63	28	63	26	63
	Average	12	18	14	14	13	14
C	No. of Tests	227	150	40	417	27	444
	Minimum	0	4	10	0	6	0
	Maximum	147	175	113	175	64	175
	Average	39	32	33	36	31	35
D	No. of Tests	207	179	58	444	40	484
	Minimum	8	4	6	4	7	4
	Maximum	41	39	56	56	38	56
	Average	18	17	18	17	18	17
E	No. of Tests	231	117	1	349	23	372
	Minimum	7	11	50	7	12	7
	Maximum	85	80	50	85	66	85
	Average	31	38	50	34	32	34
F	No. of Tests	48	113	81	242	49	291
	Minimum	9	10	10	9	8	8
	Maximum	72	46	30	72	41	72
	Average	19	19	18	19	20	19
H	No. of Tests	56	58	2	116	14	130
	Minimum	12	12	38	12	15	12
	Maximum	76	99	62	99	55	99
	Average	33	36	50	35	25	34

Table 2.5S.4-5 Summary of Corrected SPT $(N_1)_{60}$-Values [1] (Continued)

Stratum	Range	STP 3	STP 4	UHS Basin/ RSW	Inside Power Block	Outside Power Block	Site-Wide
J Clay	No. of Tests	113	99	Not Reached	212	3	215
	Minimum	6	7	Not Reached	6	24	6
	Maximum	51	72	Not Reached	72	36	72
	Average	17	18	Not Reached	18	29	18
J Sand	No. of Tests	43	32	Not Reached	75	Not Reached	75
	Minimum	16	11	Not Reached	11	Not Reached	11
	Maximum	65	75	Not Reached	75	Not Reached	75
	Average	40	32	Not Reached	36	Not Reached	36
K Clay	No. of Tests	1	1	Not Reached	Not Reached	Not Reached	2
	Minimum	7	6	Not Reached	Not Reached	Not Reached	6
	Maximum	7	6	Not Reached	Not Reached	Not Reached	7
	Average	7	6	Not Reached	Not Reached	Not Reached	7
K Sand/Silt	No. of Tests	1	1	Not Reached	Not Reached	Not Reached	2
	Minimum	49	12	Not Reached	Not Reached	Not Reached	12
	Maximum	49	12	Not Reached	Not Reached	Not Reached	49
	Average	49	12	Not Reached	Not Reached	Not Reached	31
L	No. of Tests	1	1	Not Reached	Not Reached	Not Reached	2
	Minimum	9	8	Not Reached	Not Reached	Not Reached	8
	Maximum	9	8	Not Reached	Not Reached	Not Reached	9
	Average	9	8	Not Reached	Not Reached	Not Reached	9
M	No. of Tests	0	0	Not Reached	Not Reached	Not Reached	0
	Minimum	Not Tested	Not Tested	Not Reached	Not Reached	Not Reached	Not Tested
	Maximum	Not Tested	Not Tested	Not Reached	Not Reached	Not Reached	Not Tested
	Average	Not Tested	Not Tested	Not Reached	Not Reached	Not Reached	Not Tested
N Clay	No. of Tests	14	12	Not Reached	Not Reached	Not Reached	26
	Minimum	5	0	Not Reached	Not Reached	Not Reached	0
	Maximum	13	10	Not Reached	Not Reached	Not Reached	13
	Average	7	8	Not Reached	Not Reached	Not Reached	8
N Sand	No. of Tests	4	4	Not Reached	Not Reached	Not Reached	8
	Minimum	5	11	Not Reached	Not Reached	Not Reached	5
	Maximum	45	40	Not Reached	Not Reached	Not Reached	45
	Average	20	27	Not Reached	Not Reached	Not Reached	23

[1] All SPT $(N_1)_{60}$-values in blows/foot

Table 2.5S.4-6 Summary of Corrected SPT $(N_1)_{60}$-Values Selected for Engineering Use [1]

Stratum	Average [2] Uncorrected N-Value	Average [2] Corrected $(N_1)_{60}$-Value	Selected [3] Corrected $(N_1)_{60}$-Value
A/A (Fill)	10	17	15
B	9	14	10
C	25	35	35
D	15	17	15
E	35	34	30
F	22	19	15
H	44	34	30
J Clay	31	18	15
J Sand	65	36	35
K Clay	15	7	6
K Sand/Silt	75	31	30
L	23	9	8
M	Not Tested	Not Tested	30 [4]
N Clay	33	8	7
N Sand	97	23	20

[1] All SPT N- and $(N_1)_{60}$-values in blows/foot

[2] Average N- and $(N_1)_{60}$-values shown above are site-wide averages

[3] Selected values for engineering use

[4] The selected $(N_1)_{60}$-value for Stratum M was taken the same as the selected (N1)60-value for Sub-stratum K Sand/Silt

Table 2.5S.4-7 Laboratory Testing Summary

Laboratory Test	Industry Standard	Number Of Tests
Moisture content	Reference 2.5S.4-32	388
Atterberg Limits	Reference 2.5S.4-33	226
Sieve and Hydrometer Analysis	References 2.5S.4-34 and 2.5S.4-35	200
Specific Gravity	Reference 2.5S.4-36	86
Unit Weight	Included with Related ASTM Standards	109
Unconsolidated Undrained (UU) Triaxial Strength	Reference 2.5S.4-37	68
Unconfined Compressive (UNC) Strength	Reference 2.5S.4-38	20
Consolidated Undrained (CIU-bar) Triaxial Strength	Reference 2.5S.4-39	15
Direct Shear (DS) Strength	Reference 2.5S.4-40	11
Consolidation	Reference 2.5S.4-41	30
Moisture-Density (Proctor Compaction)	Reference 2.5S.4-42	8
California Bearing Ratio (CBR)	Reference 2.5S.4-43	4
pH	Reference 2.5S.4-44	60
Chloride Content	Reference 2.5S.4-44	40
Sulfate Content	Reference 2.5S.4-45	40
Resonant Column Torsional Shear (RCTS) [1]	Reference 2.5S.4-46	Pending [1]

[1] RCTS testing is currently in progress

Table 2.5S.4-8 Summary of General Physical and Chemical Properties Test Results

Description of Value	USCS Group	Natural Moisture Content (percent)	Total Unit Weight (pounds/ cubic foot)	Specific Gravity	Initial Void Ratio	Liquid Limit (percent)	Plasticity Index (percent)	Gravel (percent)	Sand (percent)	Fines Content/ Silt plus Clay (percent)	pH	Chloride Content (mg/ kg)	Sulphide Content (mg/ kg)
STRATUM A													
Minimum	CH, CL	15.7	119.1	2.65	0.467	30	11	0	0	87	7.7	26.1	6.1
Maximum		29.0	133.0	2.74	0.748	80	58	1	12	100	9.2	1,230.0	622.0
Average		23.1	124.1	2.70	0.664	57	37	0	6	94	8.4	263.0	122.0
Number of Tests		57	12	7	12	47	47	17	17	17	29	20	20
STRATUM B													
Minimum	ML, CL	18.0	116.8	2.69	0.600	26	8	0	6	36	8.4	6.5	9.3
Maximum		28.0	127.7	2.74	0.806	70	45	4	64	94	8.7	124.0	13.5
Average		24.0	120.9	2.71	0.713	38	19	1	29	71	8.6	73.5	11.7
Number of Tests		30	6	3	6	8 [1]	8 [1]	17	17	17	4	3	3
STRATUM C													
Minimum	SM, SP-SM	18.8	119.6	2.65	0.653	NV	NP	0	4	6	8.1	63.8	9.8
Maximum		27.0	124.2	2.73	0.715			6	95	96	9.0	108.0	35.5
Average		23.6	122.0	2.68	0.695			0	75	25	8.7	92.9	16.2
Number of Tests		38	4	4	4	2	2	36	36	36	11	7	7
STRATUM D													
Minimum	CH, CL	16.3	110.8	2.65	0.523	20	2	0	0	24	8.5	33.4	6.7
Maximum		43.0	128.8	2.77	1.030	84	59	2	76	100	9.0	66.9	143.0
Average		25.4	121.0	2.70	0.759	58	38	0	28	72	8.7	51.9	53.3
Number of Tests		55	14	8	14	34 [2]	34 [2]	13	13	13	6	3	3

Table 2.5S.4-8 Summary of General Physical and Chemical Properties Test Results (Continued)

Description of Value	USCS Group	Natural Moisture Content (percent)	Total Unit Weight (pounds/ cubic foot)	Specific Gravity	Initial Void Ratio		Liquid Limit (percent)	Plasticity Index (percent)		Gravel (percent)	Sand (percent)	Fines Content/ Silt plus Clay (percent)		pH	Chloride Content (mg/ kg)	Sulphide Content (mg/ kg)
STRATUM E																
Minimum	SP-SM, SM	16.9	113.3	2.62	0.576		NV	NP		0	4	3		8.4	32.8	11.6
Maximum		25.8	127.1	2.78	0.783					2	97	96		8.9	46.6	31.8
Average		21.1	122.3	2.68	0.685					0	82	18		8.7	40.5	23.7
Number of Tests		38	8	8	8		6	6		35	35	35		5	3	3
STRATUM F																
Minimum	CH, CL	17.9	119.5	2.69	0.542		27	6		0	1	13		8.3	20.6	14.5
Maximum		29.3	129.3	2.78	0.844		74	53		1	86	99		8.9	40.0	47.8
Average		23.7	125.1	2.73	0.679		58	38		0	11	89		8.6	30.0	31.3
Number of Tests		46	13	11	13		34	34		10	10	10		4	4	4
STRATUM H																
Minimum	SP-SM, SM	12.4	121.7	2.66	0.404		NV	NP		0	5	6		8.8	N/A	N/A
Maximum		24.4	134.9	2.66	0.697					9	94	95		8.8	N/A	N/A
Average		18.9	128.3	2.66	0.551					1	83	16		8.8	N/A	N/A
Number of Tests		13	2	1	2		1	1		12	12	12		1	0	0
SUB-STRATUM J CLAY 1																
Minimum	CH, CL	15.7	103.7	2.65	0.502		30	12		0	1	67		N/A	N/A	N/A
Maximum		34.0	131.1	2.80	0.984		80	58		0	33	99		N/A	N/A	N/A
Average		22.0	124.6	2.71	0.661		54	35		0	11	89		N/A	N/A	N/A
Number of Tests		40	24	16	24		30	30		17	17	17		0	0	0

Table 2.5S.4-8 Summary of General Physical and Chemical Properties Test Results (Continued)

Description of Value	USCS Group	Natural Moisture Content (percent)	Total Unit Weight (pounds/ cubic foot)	Specific Gravity	Initial Void Ratio	Liquid Limit (percent)	Plasticity Index (percent)	Gravel (percent)	Sand (percent)	Fines Content/ Silt plus Clay (percent)	pH	Chloride Content (mg/ kg)	Sulphide Content (mg/ kg)
SUB-STRATUM J INTERBED 1 (associated with J CLAY 1)													
Minimum	ML	21.0	N/A	N/A	N/A	N/A	N/A	0	49	51	N/A	N/A	N/A
Maximumlm		21.0	N/A	N/A	N/A	N/A	N/A	0	49	51	N/A	N/A	N/A
Average		21.0	N/A	N/A	N/A	N/A	N/A	0	49	51	N/A	N/A	N/A
Number of Tests		1	0	0	0	0	0	1	1	1	0	0	0
SUB-STRATUM J SAND 1													
Minimum	SM, ML	18.7	121.6	2.66	0.632	Typically NV [3]	Typically NP [3]	0	40	15	N/A	N/A	N/A
Maximum		24.0	123.5	2.72	0.692			1	85	60	N/A	N/A	N/A
Average		21.1	122.6	2.68	0.662			0	68	32	N/A	N/A	N/A
Number of Tests		7	2	3	2	3	3	6	6	6	0	0	0
SUB-STRATUM J CLAY 2													
Minimum	CH, CL	16.7	118.9	2.63	0.536	30	12	0	0	18	N/A	N/A	N/A
Maximum		38.0	132.0	2.75	0.793	85	62	1	82	100	N/A	N/A	N/A
Average		24.6	124.9	2.70	0.667	55	36	0	12	88	N/A	N/A	N/A
Number of Tests		30	13	11	13	24	24	12	12	12	0	0	0
SUB-STRATUM J INTERBED 2 (associated with J CLAY 2)													
Minimum	SM, ML	20.8	127.2	2.67	0.642	NV	NP	0	23	10	N/A	N/A	N/A
Maximum		32.0	128.0	2.67	0.749			0	90	77	N/A	N/A	N/A
Average		25.7	127.6	2.67	0.696			0	50	50	N/A	N/A	N/A
Number of Tests		4	2	2	2	4	4	4	4	4	0	0	0

Table 2.5S.4-8 Summary of General Physical and Chemical Properties Test Results (Continued)

Description of Value	USCS Group	Natural Moisture Content (percent)	Total Unit Weight (pounds/ cubic foot)	Specific Gravity	Initial Void Ratio	Liquid Limit (percent)	Plasticity Index (percent)	Gravel (percent)	Sand (percent)	Fines Content/ Silt plus Clay (percent)	pH	Chloride Content (mg/ kg)	Sulphide Content (mg/ kg)
SUB-STRATUM J SAND 2													
Minimum	ML, SM	22.1	122.6	2.7	0.698	N/A	N/A	0	30	70	N/A	N/A	N/A
Maximum		22.1	122.6	2.7	0.698	N/A	N/A	0	30	70	N/A	N/A	N/A
Average		22.1	122.6	2.7	0.698	N/A	N/A	0	30	70	N/A	N/A	N/A
Number of Tests		1	1	1	1	0	0	1	1	1	0	0	0
COMBINED SUB-STRATUM J CLAY (combined J CLAY 1 and J CLAY 2)													
Minimum	CH, CL	15.7	103.7	2.63	0.502	30	12	0	0	18	N/A	N/A	N/A
Maximum		38.0	132.0	2.80	0.984	85	62	1	81	100	N/A	N/A	N/A
Average		23.1	124.7	2.71	0.663	54	35	0	11	89	N/A	N/A	N/A
Number of Tests		70	37	27	37	54	54	29	29	29	0	0	0
COMBINED SUB-STRATUM J SAND (combined J INTERBED 1, J SAND1, J INTERBED 2, and J SAND 2)													
Minimum	SM, ML	18.7	121.6	2.66	0.632	Typically NV [4]	Typically NP [4]	0	23	10	N/A	N/A	N/A
Maximum		32.0	128.0	2.73	0.749			1	90	77	N/A	N/A	N/A
Average		22.6	124.6	2.69	0.683			0	57	43	N/A	N/A	N/A
Number of Tests		13	5	6	5	5	5	12	12	12	0	0	0
SUB-STRATUM K CLAY													
Minimum	CL, CH	16.8	126.6	2.71	0.516	33	18	0	25	75	N/A	N/A	N/A
Maximum		21.8	131.5	2.76	0.627	45	31	0	25	75	N/A	N/A	N/A
Average		19.5	129.1	2.74	0.571	39	25	0	25	75	N/A	N/A	N/A
Number of Tests		3	2	2	2	2	2	1	1	1	0	0	0

Table 2.5S.4-8 Summary of General Physical and Chemical Properties Test Results (Continued)

Description of Value	USCS Group	Natural Moisture Content (percent)	Total Unit Weight (pounds/ cubic foot)	Specific Gravity	Initial Void Ratio		Liquid Limit (percent)	Plasticity Index (percent)		Gravel (percent)	Sand (percent)	Fines Content/ Silt plus Clay (percent)		pH	Chloride Content (mg/ kg)	Sulphide Content (mg/ kg)
SUB-STRATUM K SAND/ SILT																
Minimum	SM, ML	20.1	126.8	2.67	0.596		NV	NP		0	35	27		N/A	N/A	N/A
Maximum		21.5	126.8	2.67	0.596					2	73	64		N/A	N/A	N/A
Average		20.8	126.8	2.67	0.596					1	54	45		N/A	N/A	N/A
Number of Tests		2	1	1	1		1	1		2	2	2		0	0	0
STRATUM L																
Minimum	CH	27.3	N/A	N/A	N/A		72	51		N/A	N/A	N/A		N/A	N/A	N/A
Maximum		29.6	N/A	N/A	N/A		74	52		N/A	N/A	N/A		N/A	N/A	N/A
Average		28.5	N/A	N/A	N/A		73	52		N/A	N/A	N/A		N/A	N/A	N/A
Number of Tests		2	0	0	0		2	2		0	0	0		0	0	0
STRATUM M																
Minimum	SP-SM [5]	N/A	N/A	N/A	N/A		N/A	N/A		N/A	N/A	N/A		N/A	N/A	N/A
Maximum		N/A	N/A	N/A	N/A		N/A	N/A		N/A	N/A	N/A		N/A	N/A	N/A
Average		N/A	N/A	N/A	N/A		N/A	N/A		N/A	N/A	N/A		N/A	N/A	N/A
Number of Tests		0	0	0	0		0	0		0	0	0		0	0	0
SUB-STRATUM N CLAY 1																
Minimum	CH	19.7	112.9	2.67	0.835		50	25		0	5	22		N/A	N/A	N/A
Maximum		37.7	119.7	2.72	1.074		90	63		0	78	95		N/A	N/A	N/A
Average		29.5	116.3	2.69	0.954		69	45		0	31	69		N/A	N/A	N/A
Number of Tests		4	2	3	2		4	4		3	3	3		0	0	0

Table 2.5S.4-8 Summary of General Physical and Chemical Properties Test Results (Continued)

Description of Value	USCS Group	Natural Moisture Content (percent)	Total Unit Weight (pounds/ cubic foot)	Specific Gravity	Initial Void Ratio	Liquid Limit (percent)	Plasticity Index (percent)	Gravel (percent)	Sand (percent)	Fines Content/ Silt plus Clay (percent)	pH	Chloride Content (mg/ kg)	Sulphide Content (mg/ kg)
SUB-STRATUM N SAND 1													
Minimum	SM	16.7	130.2	2.65	0.565	NV	NP	0	50	5	N/A	N/A	N/A
Maximum		20.9	130.2	2.65	0.565			1	95	49	N/A	N/A	N/A
Average		18.8	130.2	2.65	0.565			0	73	27	N/A	N/A	N/A
Number of Tests		2	1	1	1	1	1	2	2	2	0	0	0
SUB-STRATUM N CLAY 2													
Minimum	CH [6]	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Maximum		N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Average		N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Number of Tests		0	0	0	0	0	0	0	0	0	0	0	0
SUB-STRATUM N SAND 2													
Minimum	SP-SM, SM	23.3	N/A	N/A	N/A	N/A	N/A	0	72	5	N/A	N/A	N/A
Maximum		28.0	N/A	N/A	N/A	N/A	N/A	6	90	26	N/A	N/A	N/A
Average		25.7	N/A	N/A	N/A	N/A	N/A	3	83	14	N/A	N/A	N/A
Number of Tests		1	0	0	0	0	0	3	3	3	0	0	0
SUB-STRATUM N CLAY 3													
Minimum	CL	17.1	N/A	N/A	N/A	46	15	N/A	N/A	N/A	N/A	N/A	N/A
Maximum		17.1	N/A	N/A	N/A	46	15	N/A	N/A	N/A	N/A	N/A	N/A
Average		17.1	N/A	N/A	N/A	46	15	N/A	N/A	N/A	N/A	N/A	N/A
Number of Tests		1	0	0	0	1	1	0	0	0	0	0	0

Table 2.5S.4-8 Summary of General Physical and Chemical Properties Test Results (Continued)

Description of Value	USCS Group	Natural Moisture Content (percent)	Total Unit Weight (pounds/ cubic foot)	Specific Gravity	Initial Void Ratio	Liquid Limit (percent)	Plasticity Index (percent)	Gravel (percent)	Sand (percent)	Fines Content/ Silt plus Clay (percent)	pH	Chloride Content (mg/ kg)	Sulphide Content (mg/ kg)
SUB-STRATUM N SAND 3													
Minimum	SM, SP-SM	N/A	N/A	2.69	N/A	N/A	N/A	1	83	16	N/A	N/A	N/A
Maximum		N/A	N/A	2.69	N/A	N/A	N/A	1	83	16	N/A	N/A	N/A
Average		N/A	N/A	2.69	N/A	N/A	N/A	1	83	16	N/A	N/A	N/A
Number of Tests		0	0	1	0	0	0	1	1	1	0	0	0
SUB-STRATUM N CLAY 4													
Minimum	CH	25.3	N/A	N/A	N/A	77	54	N/A	N/A	N/A	N/A	N/A	N/A
Maximum		29.7	N/A	N/A	N/A	86	59	N/A	N/A	N/A	N/A	N/A	N/A
Average		27.5	N/A	N/A	N/A	82	57	N/A	N/A	N/A	N/A	N/A	N/A
Number of Tests		2	0	0	0	2	2	0	0	0	0	0	0
SUB-STRATUM N SAND 4													
Minimum	SP-SM, SM	22.0	125.8	2.67	0.634	NV	NP	0	71	12	N/A	N/A	N/A
Maximum		23.3	125.8	2.67	0.634			0	88	29	N/A	N/A	N/A
Average		22.7	125.8	2.67	0.634			0	80	20	N/A	N/A	N/A
Number of Tests		2	1	1	1	1	1	2	2	2	0	0	0
SUB-STRATUM N CLAY 5													
Minimum	CH	21.8	123.7	N/A	0.729	59	40	N/A	N/A	N/A	N/A	N/A	N/A
Maximum		24.3	123.7	N/A	0.729	81	58	N/A	N/A	N/A	N/A	N/A	N/A
Average		23.3	123.7	N/A	0.729	70	49	N/A	N/A	N/A	N/A	N/A	N/A
Number of Tests		3	1	0	1	2	2	0	0	0	0	0	0

Table 2.5S.4-8 Summary of General Physical and Chemical Properties Test Results (Continued)

Description of Value	USCS Group	Natural Moisture Content (percent)	Total Unit Weight (pounds/ cubic foot)	Specific Gravity	Initial Void Ratio		Liquid Limit (percent)	Plasticity Index (percent)		Gravel (percent)	Sand (percent)	Fines Content/ Silt plus Clay (percent)		pH	Chloride Content (mg/ kg)	Sulphide Content (mg/ kg)
SUB-STRATUM N SAND 5																
Minimum	SM	20.8	N/A	2.68	N/A		N/A	N/A		0	62	21		N/A	N/A	N/A
Maximum		25.4	N/A	2.68	N/A		N/A	N/A		2	79	37		N/A	N/A	N/A
Average		22.6	N/A	2.68	N/A		N/A	N/A		1	71	28		N/A	N/A	N/A
Number of Tests		3	0	1	0		0	0		3	3	3		0	0	0
SUB-STRATUM N CLAY 6																
Minimum	CL, CH	18.4	127.3	2.69	0.567		45	29		0	11	80		N/A	N/A	N/A
Maximum		19.9	127.3	2.69	0.567		47	31		1	19	89		N/A	N/A	N/A
Average		19.1	127.3	2.69	0.567		46	30		0	15	85		N/A	N/A	N/A
Number of Tests		2	1	0	0		2	2		2	2	2		0	0	0
COMBINED SUB-STRATUM N CLAY (combined N CLAY 1, N CLAY 2, N CLAY 3, N CLAY 4, N CLAY 5, and N CLAY 6)																
Minimum	CH, CL, SC	17.1	112.9	2.67	0.567		45	25		0	5	22		N/A	N/A	N/A
Maximum		37.7	127.3	2.72	1.074		90	63		1	78	95		N/A	N/A	N/A
Average		24.9	120.9	2.69	0.801		65	44		0	25	75		N/A	N/A	N/A
Number of Tests		12	4	4	4		11	11		5	5	5		0	0	0

Table 2.5S.4-8 Summary of General Physical and Chemical Properties Test Results (Continued)

Description of Value	USCS Group	Natural Moisture Content (percent)	Total Unit Weight (pounds/ cubic foot)	Specific Gravity	Initial Void Ratio		Liquid Limit (percent)	Plasticity Index (percent)		Gravel (percent)	Sand (percent)	Fines Content/ Silt plus Clay (percent)		pH	Chloride Content (mg/ kg)	Sulphide Content (mg/ kg)
COMBINED SUB-STRATUM N SAND (combined N SAND 1, N SAND 2, N SAND 3, N SAND 4, and N SAND 5)																
Minimum	SM, SP-SM, SP, SC	16.7	125.8	2.65	0.565		NV	NP		0	50	5		N/A	N/A	N/A
Maximum		28.0	130.2	2.69	0.634					6	95	49		N/A	N/A	N/A
Average		22.8	128.0	2.67	0.600					1	77	22		N/A	N/A	N/A
Number of Tests		10	2	4	2		4	4		11	11	11		0	0	0

[1] Stratum B has nine additional Atterberg Limits tests showing Liquid Limit = NV (No Value), and Plasticity Index = NP (Non-Plastic)

[2] Stratum D has four additional Atterberg Limits tests showing Liquid Limit = NV, and Plasticity Index = NP

[3] Sub-stratum J Sand 1 has one additional Atterberg Limits test showing Liquid Limit = 62 percent, and Plasticity Index = 35 percent

[4] Combined Sub-stratum J Sand (combined J Interbed 1, J Sand 1, J Interbed 2, and J Sand 2) have one additional Atterberg Limits test showing Liquid Limit = 62 percent, and Plasticity Index = 35 percent

[5] The USCS group symbol for Stratum M is taken from visual classifications indicated on the boring logs

[6] The USCS group symbol for Sub-stratum N Clay 2 is taken from visual classifications indicated on the boring logs

Table 2.5S.4-9A Summary of Undrained Shear Strengths for Cohesive Soil Strata

From Correlations with SPT N-value Data			
Stratum	**Selected Corrected $(N_1)_{60}$-Value (blows/foot)**	**Calculated s_u (ksf)**	
A/A (Fill)	15	1.9	
D	15	1.9	
F	15	1.9	
J Clay	15	1.9	
K Clay	6	0.8	
L	8	1.0	
N Clay	7	0.9	
From Laboratory UU and UNC Tests			
Stratum	**Minimum s_u (ksf)**	**Maximum s_u (ksf)**	**Average s_u (ksf)**
A/A (Fill)	0.5	2.3	1.3
D	0.3	2.5	1.7
F	0.7	3.7	2.7
J Clay	0.1	6.6	3.2
K Clay	2.8	4.0	3.4
L	Not Tested	Tested	Tested
N Clay	0.2	4.5	1.7
From Correlations with CPT Data			
Stratum	**Minimum s_u (ksf)**	**Maximum s_u (ksf)**	**Average s_u (ksf)**
A/A (Fill)	0.2	3.7	1.5
D	0.8	7.6	3.1
F	1.9	6.3	3.6
J Clay	2.3	5.3	3.8
K Clay	Not Reached	Not Reached	Not Reached
L	Not Reached	Not Reached	Not Reached
N Clay	Not Reached	Not Reached	Not Reached
Selected Values for Engineering Use			
Stratum		**Selected s_u (ksf)**	
A/A (Fill)		1.6	
D		3.0	
F		3.2	
J Clay		3.5	
K Clay		3.0	
L		3.0	
N Clay		3.0	

Table 2.5S.4-10 Summary of Laboratory Strength Test Results

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Total Unit Weight (pounds/ cubic foot) [2]	Natural Moisture Content (percent) [2]	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
									Test Type	Undrained Shear Strength (kips/ square foot)	Ratio, Undrained Shear Strength/ Preconsolidation Pressure	Undrained Cohesion (kips/ square foot)	Undrained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)
STRATUM A [3]																	
B-305DH	UD1	3.0	26.8	122.0	23.8	62	43	CH	UNC	1.0	0.30	-	-	-	-	-	-
B-333	UD1	8.0	22.5	123.8	24.3	43	27	CL	UNC	1.3	0.14	-	-	-	-	-	-
B-333	UD2	18.0	12.5	119.1	23.4	30	11	CL	UU	0.5	0.09	-	-	-	-	-	-
B-432	UD1	3.0	28.2	123.0	21.8	65	42	CH	UNC	2.3	0.23	-	-	-	-	-	-
B-432	UD2	15.0	16.2	121.4	24.0	31	11	CL	UNC	0.9	0.16	-	-	-	-	-	-
B-902	UD1	5.0	24.1	124.7	21.3	65	43	CH	UNC	1.3	-	-	-	-	-	-	-
B-902	UD2	15.0	14.1	124.5	24.3	59	39	CH	UU	0.9	-	-	-	-	-	-	-
B-904	UD1	5.0	24.8	122.0	22.3	68	46	CH	UNC	1.2	-	-	-	-	-	-	-
B-904	UD2	18.0	11.8	122.7	21.1	63	41	CH	UNC	2.0	-	-	-	-	-	-	-
B-918	UD1	3.0	27.9	133.0	15.7	-	-	CH	UNC	2.1	-	-	-	-	-	-	-
B-919	UD1	8.0	23.9	124.7	23.5	68	47	CH	UU	1.2	-	-	-	-	-	-	-
> MINIMUM, STRATUM A				119.1	15.7	30	11	-	-	0.5	0.09	-	-	-	-	-	-
> MAXIMUM, STRATUM A				133.0	24.3	68	47	-	-	2.3	0.30	-	-	-	-	-	-
> AVERAGE, STRATUM A				123.7	22.3	55	35	-	-	1.3	0.18	-	-	-	-	-	-
STRATUM B [3]																	
B-904	UD3	28.0	1.8	121.3	25.2	NV	NP	ML	-	-	-	5.1	23.0	2.0	28.0	-	-
B-919	UD2	23.0	8.9	116.8	21.6	NV	NP	ML	-	-	-	0.4	47.0	0.0	31.0	-	-
> MINIMUM, STRATUM B				116.8	21.6	NV	NP	-	-	-	-	0.4	23.0	0.0	28.0	-	-
> MAXIMUM, STRATUM B				121.3	25.2	NV	NP	-	-	-	-	5.1	47.0	2.0	31.0	-	-
> AVERAGE, STRATUM B				119.1	23.4	NV	NP	-	-	-	-	2.7	35.0	1.0	29.5	-	-

Table 2.5S.4-10 Summary of Laboratory Strength Test Results (Continued)

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Total Unit Weight (pounds/ cubic foot) [2]	Natural Moisture Content (percent) [2]	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
									Test Type	Undrained Shear Strength (kips/ square foot)	Ratio, Undrained Shear Strength/ Preconsolidation Pressure	Undrained Cohesion (kips/ square foot)	Undrained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)
STRATUM C [3]																	
B-421	UD1A	33.5	-3.2	119.6	23.0	-	-	SP-SM	-	-	-	-	-	-	-	0.0	33.0
B-902	UD2	23.0	6.1	121.3	24.9	NV	NP	SM	-	-	-	-	-	-	-	0.0	32.0
B-909	UD1A	33.0	-3.3	123.0	23.2	-	-	SM	-	-	-	-	-	-	-	0.0	34.0
> MINIMUM, STRATUM C				116.8	21.6	NV	NP	-	-	-	-	-	-	-	-	0.0	32.0
> MAXIMUM, STRATUM C				123.0	25.2	NV	NP	-	-	-	-	-	-	-	-	0.0	34.0
> AVERAGE, STRATUM C				119.9	23.5	NV	NP	-	-	-	-	-	-	-	-	0.0	33.0
STRATUM D [3]																	
B-305DH	UD4	53.0	-23.2	125.2	24.6	45	25	CL	UNC	2.5	0.17	-	-	-	-	-	-
B-338	UD2	48.0	-15.9	122.3	26.8	44	26	CL	UU	2.2	0.16	-	-	-	-	-	-
B-909	UD2	43.0	-13.3	121.1	25.4	62	38	CH	UU	2.2	0.20	-	-	-	-	-	-
B-909	UD3	48.0	-18.3	115.9	32.8	74	53	CH	UU	2.4	0.14	-	-	-	-	-	-
B-909	UD4	53.0	-23.3	110.8	30.0	66	40	CH	UU	0.4	-	-	-	-	-	-	-
B-918	UD4	58.0	-27.1	107.7	17.7	22	6	CL-ML	UNC	0.3	-	-	-	-	-	-	-
B-919	UD3	43.0	-11.1	94.5	26.5	66	42	CH	UU	1.7	-	-	-	-	-	-	-
B-927	UD3	48.0	-21.2	121.9	28.2	54	33	CH	UNC	1.8	-	-	-	-	-	-	-
B-330	UD2	53.0	-23.5	128.8	16.3	-	-	CH	-	-	-	2.2	0.0	2.2	0.0	-	-
B-916	UD3	48	-20.2	97.3	20.8	59	35	CH	-	-	-	1.4	8.7	0.3	31.7	-	-
> MINIMUM, STRATUM D				94.5	16.3	22.0	6.0	-	-	0.3	0.14	1.4	0.0	0.3	0.0	-	-
> MAXIMUM, STRATUM D				128.8	32.8	74.0	53.0	-	-	2.5	0.20	2.2	8.7	2.2	31.7	-	-
> AVERAGE, STRATUM D				114.6	24.9	54.7	33.1	-	-	1.7	0.17	-	-	-	-	-	-

Table 2.5S.4-10 Summary of Laboratory Strength Test Results (Continued)

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Total Unit Weight (pounds/ cubic foot) [2]	Natural Moisture Content (percent) [2]	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
									Test Type	Undrained Shear Strength (kips/ square foot)	Ratio, Undrained Shear Strength/ Preconsolidation Pressure	Undrained Cohesion (kips/ square foot)	Undrained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)
STRATUM E [3]																	
B-314	UD1	83.0	-53.8	122.7	20.9	NV	NP	SP	-	-	-	-	-	-	-	0.0	33.0
B-409	UD1	68.0	-36.8	113.3	16.9	NV	NP	SM	-	-	-	-	-	-	-	0.0	33.0
> MINIMUM, STRATUM E				113.3	16.9	NV	NP	-	-	-	-	-	-	-	-	0.0	33.0
> MAXIMUM, STRATUM E				122.7	20.9	NV	NP	-	-	-	-	-	-	-	-	0.0	33.0
> AVERAGE, STRATUM E				118.0	18.9	NV	NP	-	-	-	-	-	-	-	-	0.0	33.0
STRATUM F [3]																	
B-303	UD2	88.0	-61.4	127.9	26.6	57	39	CH	UU	3.5	0.26	-	-	-	-	-	-
B-404	UD1	88.0	-57.0	126.7	21.8	-	-	CH	UU	3.5	-	-	-	-	-	-	-
B-404	UD2	98.0	-67.0	123.9	25.3	50	30	CH	UU	3.5	-	-	-	-	-	-	-
B-415	UD1	88.0	-58.0	123.8	23.9	61	44	CH	UU	1.2	-	-	-	-	-	-	-
B-419DH	UD1	78.0	-48.3	127.8	22.3	47	23	CL	UU	3.7	0.21	-	-	-	-	-	-
B-419DH	UD1	98.0	-68.3	119.5	27.0	61	37	CH	UU	0.7	-	-	-	-	-	-	-
B-421	UD3	83.0	-52.7	125.9	22.9	56	36	CH	UU	3.1	0.17	-	-	-	-	-	-
B-904	UD5	83.0	-53.2	122.3	24.0	62	41	CH	UU	1.6	-	-	-	-	-	-	-
B-909	UD6	93.0	-63.3	129.3	17.9	55	34	CH	UU	3.6	-	-	-	-	-	-	-
B909	UD7	98.0	-68.3	121.1	21.5	-	-	CH	UU	2.5	-	-	-	-	-	-	-
B-306	UD4	88	-60.2	125.9	22.7	57	38	CH	-	-	-	3.0	3.1	3.0	6.5	-	-
B-401	UD2	88	-56.9	125.7	23.4	57	36	CH	-	-	-	2.4	0.8	2.1	5.1	-	-
B-909	UD5	85	-55.3	126.1	21.9	49	26	CL	-	-	-	0.9	6.0	1.0	12.0	-	-
> MINIMUM, STRATUM F				119.5	17.9	47.0	23.0	-	-	0.7	0.17	0.9	0.8	1.0	5.1	-	-
> MAXIMUM, STRATUM F				129.3	27.0	62.0	44.0	-	-	3.7	0.26	3.0	6.0	3.0	12.0	-	-
> AVERAGE, STRATUM F				125.1	23.2	55.6	34.9	-	-	2.7	0.21	-	-	-	-	-	-

Table 2.5S.4-10 Summary of Laboratory Strength Test Results (Continued)

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Total Unit Weight (pounds/ cubic foot) [2]	Natural Moisture Content (percent) [2]	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
									Test Type	Undrained Shear Strength (kips/ square foot)	Ratio, Undrained Shear Strength/ Preconsolidation Pressure	Undrained Cohesion (kips/ square foot)	Undrained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)
STRATUM H [3]																	
B-306	UD5	98.0	-70.2	121.7	24.4	NV	NP	SP-SM	-	-	-	-	-	-	-	0.0	29.0
> MINIMUM, STRATUM H				121.7	24.4	NV	NP	-	-	-	-	-	-	-	-	0.0	29.0
> MAXIMUM, STRATUM H				121.7	24.4	NV	NP	-	-	-	-	-	-	-	-	0.0	29.0
> AVERAGE, STRATUM H				121.7	24.4	NV	NP	-	-	-	-	-	-	-	-	0.0	29.0
SUB-STRATUM J CLAY 1 [3]																	
B-303	UD4	133.0	-106.4	121.3	29.5	65	39	CH	UU	0.1	-	-	-	-	-	-	-
B-305DH	UD7	123.0	-93.2	129.2	18.8	-	-	CH	UNC	1.2	-	-	-	-	-	-	-
B-305DH	UD7	123.0	-93.2	128.4	20.1	-	-	CH	UU	3.0	0.16	-	-	-	-	-	-
B-305DH	UD8	138.0	-108.2	129.5	18.6	32	18	CL	UU	4.5	0.28	-	-	-	-	-	-
B-314	UD2	113.0	-83.8	127.9	20.0	38	25	CL	UU	4.8	-	-	-	-	-	-	-
B-314	UD3	121.0	-91.8	128.5	17.8	-	-	CH	UU	3.0	-	-	-	-	-	-	-
B-314	UD4	141.0	-111.8	120.3	24.2	46	31	CL	UU	0.7		-	-	-	-	-	-
B-319	UD1	128.0	-98.0	125.0	19.3	70	45	CH	UU	3.4	0.24	-	-	-	-	-	-
B-321	UD3	138.0	-108.8	127.4	20.0	46	25	CL	UU	4.9	-	-	-	-	-	-	-
B-330	UD4B	123.0	-93.5	128.7	19.5	-	-	CH	UU	1.1	-	-	-	-	-	-	-
B-401	UD3	118.0	-86.9	127.3	19.8	47	25	CL	UU	2.0	-	-	-	-	-	-	-
B-404	UD4	131.0	-100.0	124.3	20.8	52	30	CH	UU	3.5	-	-	-	-	-	-	-
B-404	UD5	141.0	-110.0	129.0	18.0	30	12	CL	UU	4.1	-	-	-	-	-	-	-
B-405DH	UD5	113.0	-81.9	122.7	25.8	73	50	CH	UU	2.2	-	-	-	-	-	-	-
B-415	UD3	124.0	-94.0	113.9	34.0	51	35	CH	UNC	0.1	-	-	-	-	-	-	-
B-419DH	UD3	118.0	-88.3	127.6	21.8	56	39	CH	UU	3.2	0.15	-	-	-	-	-	-
B-419DH	UD4	138.0	-108.3	129.3	16.1	40	25	CL	UU	6.6	0.42	-	-	-	-	-	-
B-428DH	UD6	113.0	-82.1	122.6	27.3	62	41	CH	UU	2.1	-	-	-	-	-	-	-
B-303	UD4	133.0	-106.4	131.1	18.7	-	-	CH	-	-	-	3.1	3.2	2.4	11.0	-	-

Table 2.5S.4-10 Summary of Laboratory Strength Test Results (Continued)

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Total Unit Weight (pounds/ cubic foot) [2]	Natural Moisture Content (percent) [2]	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
									Test Type	Undrained Shear Strength (kips/ square foot)	Ratio, Undrained Shear Strength/ Preconsolidation Pressure	Undrained Cohesion (kips/ square foot)	Undrained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)
STRATUM J CLAY 1 [3] (continued)																	
B-314	UD4	141.0	-111.8	-	-	-	-	CL	-	-	-	2.3	9.0	1.2	20.0	-	-
B-404	UD3	121.0	-90.0	124.2	23.6	62	39	CH	-	-	-	2.3	3.5	2.2	9.0	-	-
B-430	UD3	133.0	-102.1	119.1	28.5	-	-	CH	-	-	-	0.5	0.0	0.5	0.0	-	-
> MINIMUM, SUB-STRATUM J CLAY 1				113.9	16.1	30	12	-	-	0.1	0.15	0.5	0.0	0.5	0.0	-	-
> MAXIMUM, SUB-STRATUM J CLAY 1				131.1	34.0	73	50	-	-	6.6	0.42	3.1	9.0	2.4	20.0	-	-
> AVERAGE, SUB-STRATUM J CLAY 1				125.6	22.0	51	32	-	-	2.8	0.25	2.0	3.9	1.6	10.0	-	-
SUB-STRATUM J SAND 1 [3]																	
B-405DH	UD7	148.0	-116.9	121.6	24.0	NV	NP	SM	-	-	-	-	-	-	-	0.0	32.0
> MINIMUM, SUB-STRATUM J SAND 1				121.6	24.0	NV	NP	-	-	-	-	-	-	-	-	0.0	32.0
> MAXIMUM, SUB-STRATUM J SAND 1				121.6	24.0	NV	NP	-	-	-	-	-	-	-	-	0.0	32.0
> AVERAGE, SUB-STRATUM J SAND 1				121.6	24.0	NV	NP	-	-	-	-	-	-	-	-	0.0	32.0
SUB-STRATUM J CLAY 2 [3]																	
B-314	UD5A	183.0	-153.8	122.5	20.3	72	48	CH	UU	5.3	-	-	-	-	-	-	-
B-314	UD6	191.0	-161.8	118.9	26.5	64	40	CH	UU	1.9	-	-	-	-	-	-	-
B-319DH	UD5	188.0	-159.6	122.7	26.5	62	41	CH	UU	3.8	0.14	-	-	-	-	-	-
B-343	UD7	173.0	-142.5	125.0	16.9	31	17	CL	UU	1.4	-	-	-	-	-	-	-
B-401	UD5A	184.0	-152.9	125.2	24.1	-	-	CH	UU	3.6	-	-	-	-	-	-	-
B-409	UD6	198.0	-166.8	121.5	25.3	-	-	CH	UU	2.8	-	-	-	-	-	-	-
B-419DH	UD5	158.0	-128.3	129.2	20.3	47	30	CL	UU	6.3	0.33	-	-	-	-	-	-
B-419DH	UD6	178.0	-148.3	129.2	22.0	53	33	CH	UU	6.2	0.34	-	-	-	-	-	-
B-419DH	UD7	198.0	-168.3	123.5	23.1	56	36	CH	UU	3.8	0.25	-	-	-	-	-	-
B-307	UD3	188.0	-159.8	125.3	21.9	49	30	CL	-	-	-	3.4	1.0	3.4	2.0	-	-
B-314	UD5A	183.0	-153.8	-	-	-	-	CH	-	-	-	0.9	11.2	1.9	7.4	-	-
B-404	UD6	161.0	-130.0	132.0	19.8	30	15	CL	-	-	-	7.7	2.6	6.9	6.0	-	-

Table 2.5S.4-10 Summary of Laboratory Strength Test Results (Continued)

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Total Unit Weight (pounds/ cubic foot) [2]	Natural Moisture Content (percent) [2]	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
									Test Type	Undrained Shear Strength (kips/ square foot)	Ratio, Undrained Shear Strength/ Preconsolidation Pressure	Undrained Cohesion (kips/ square foot)	Undrained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)
SUB-STRATUM J CLAY 2 [3] (Continued)																	
> MINIMUM, SUB-STRATUM J CLAY 2				118.9	16.9	30	15	-	-	1.4	0.14	0.9	1.0	1.9	2.0	-	-
> MAXIMUM, SUB-STRATUM J CLAY 2				132.0	26.5	72	48	-	-	6.3	0.34	7.7	11.2	6.9	7.4	-	-
> AVERAGE, SUB-STRATUM J CLAY 2				125.0	22.4	52	32	-	-	3.9	0.26	4.0	4.9	4.1	5.1	-	-
SUB-STRATUM K CLAY [3]																	
B-305DH	UD11	213.0	-183.2	126.6	21.8	45	31	CL	UU	2.8	0.10	-	-	-	-	-	-
B-405DH	UD11	233.0	-210.9	131.5	16.8	-	-	CH	UU	4.0	0.20	-	-	-	-	-	-
> MINIMUM, SUB-STRATUM K CLAY				126.6	16.8	45	31	-	-	2.8	0.10	-	-	-	-	-	-
> MAXIMUM, SUB-STRATUM K CLAY				131.5	21.8	45	31	-	-	4.0	0.20	-	-	-	-	-	-
> AVERAGE, SUB-STRATUM K CLAY				129.1	19.3	45	31	-	-	3.4	0.15	-	-	-	-	-	-
SUB-STRATUM K SAND/ SILT [3]																	
B-305DH	UD12	228.0	-198.2	126.8	21.5	NV	NP	SM	-	-	-	-	-	-	-	0.0	29.0
B-902	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
B-909	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
> MINIMUM, SUB-STRATUM K SAND/ SILT				126.8	21.5	NV	NP	-	-	-	-	-	-	-	-	0.0	29.0
> MAXIMUM, SUB-STRATUM K SAND/ SILT				126.8	21.5	NV	NP	-	-	-	-	-	-	-	-	0.0	29.0
> AVERAGE, SUB-STRATUM K SAND/ SILT				126.8	21.5	NV	NP	-	-	-	-	-	-	-	-	0.0	29.0
SUB-STRATUM N CLAY 1 [3]																	
B-305DH	UD14	288.0	-258.2	112.9	37.7	84	58	CH	UU	0.2	-	-	-	-	-	-	-
B-305DH	UD15A	316.0	-286.2	119.7	30.4	-	-	CH	UU	1.4	0.07	-	-	-	-	-	-
SUB-STRATUM N CLAY 5 [3]																	
B-405DH	UD20	458.5	-427.4	123.7	23.8	-	-	CH	UU	4.5	0.25	-	-	-	-	-	-

Table 2.5S.4-10 Summary of Laboratory Strength Test Results (Continued)

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Total Unit Weight (pounds/ cubic foot) [2]	Natural Moisture Content (percent) [2]	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
									Test Type	Undrained Shear Strength (kips/ square foot)	Ratio, Undrained Shear Strength/ Preconsolidation Pressure	Undrained Cohesion (kips/ square foot)	Undrained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)	Drained Cohesion (kips/ square foot)	Drained Friction Angle (degrees)
SUB-STRATUM N CLAY 6 [3]																	
B-405DH	UD25	598.0	-566.9	127.3	18.4	45.0	29.0	CL	UNC	0.9	-	-	-	-	-	-	-
> MINIMUM, SUB-STRATUM N CLAY				112.9	18.4	45	29	-	-	0.2	0.07	-	-	-	-	-	-
> MAXIMUM, SUB-STRATUM N CLAY				127.3	37.7	84	58	-	-	4.5	0.25	-	-	-	-	-	-
> AVERAGE, SUB-STRATUM N CLAY				120.9	27.6	65	44	-	-	1.7	0.16	-	-	-	-	-	-

[1] UNC = unconfined compression. UU = unconsolidated undrained triaxial compression. CIU-Bar = consolidated undrained triaxial compression with pore pressures measured. DS = direct shear

[2] Initial (pre-saturation) unit weights and moisture contents. Note that all strength tests were conducted on fully-saturated test specimens

[3] UNC and UU tests made on predominantly coarse-grained and fine grained/ non-cohesive strata are not shown. DS tests made on predominantly fine-grained/ cohesive strata, similarly, are not shown

Table 2.5S.4-11 Summary of Laboratory Consolidation Test Properties

Stratum	Number Of Tests	Range	C_r	C_c	e_0	P_c'(ksf)	OCR	c_v (ft^2/day)
A	5	Minimum	0.000	0.316	0.316	3.2	3.2	1.73
		Maximum	0.023	0.050	0.750	10.0	17.2	9.85
		Average	0.017	0.235	0.235	6.7	7.8	5.32
D	5	Minimum	0.007	0.086	0.710	6.1	1.6	0.05
		Maximum	0.033	0.468	0.920	16.9	4.8	0.52
		Average	0.023	0.255	0.796	12.5	3.5	0.21
F	3	Minimum	0.037	0.229	0.630	13.4	2.2	0.15
		Maximum	0.040	0.249	0.810	18.0	3.3	3.41
		Average	0.039	0.240	0.713	16.5	2.9	1.29
J Clay	10	Minimum	0.013	0.149	0.520	14.1	1.2	0.04
		Maximum	0.086	0.472	0.790	27.9	2.7	14.20
		Average	0.040	0.228	0.628	18.6	1.9	2.74
K Clay	2	Minimum	0.010	0.103	0.510	20.2	1.3	0.13
		Maximum	0.023	0.249	0.610	27.9	2.0	2.09
		Average	0.017	0.176	0.560	24.1	1.7	1.11
L	0	Minimum	N/A	N/A	N/A	N/A	N/A	N/A
		Maximum	N/A	N/A	N/A	N/A	N/A	N/A
		Average	N/A	N/A	N/A	N/A	N/A	N/A
N Clay	2	Minimum	0.033	0.292	0.790	17.9	0.6	0.04
		Maximum	0.066	0.379	0.870	18.9	0.9	0.05
		Average	0.050	0.336	0.830	18.4	0.8	0.05

C_r = recompression index
e_0 = void ratio
OCR = overconsolidation ratio
C_c = compression index
P_c' = preconsolidation pressure
c_v = coefficient of consolidation

Table 2.5S.4-12 Summary of Laboratory Consolidation Test Results

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Initial Effective Overburden Pressure (kips per square foot)	Total Unit Weight (pounds/ cubic foot) [1]	Natural Moisture Content (percent) [1]	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	Initial Void Ratio, e_0	Compression Index, C_c	Compression Ratio, CR	Recompression Index, C_r	Recompression Ratio, RR	Preconsolidation Pressure, P_c' (kips per square foot)	Overconsolidation Ratio, OCR	Coefficient of Consolidation, c_v ($feet^2$ / day)
STRATUM A																	
B-305DH	UD-1	3.0	26.8	0.496	122.0	25.0	62	43	CH	0.750	0.269	0.154	0.023	0.013	3.2	5.5	4.42
B-333	UD-1	8.0	22.5	0.866	123.8	24.8	43	27	CL	0.680	0.282	0.168	0.020	0.012	9.2	9.2	9.85
B-333	UD-2	18.0	12.5	1.482	119.1	23.8	30	11	CL	0.660	0.050	0.030	0.000	0.000	5.2	3.2	1.73
B-432	UD-1	3.0	28.2	0.496	123.0	22.8	65	42	CH	0.680	0.316	0.188	0.020	0.012	10.0	17.2	8.81
B-432	UD-2	15.0	16.2	1.341	121.4	24.8	31	11	CL	0.740	0.259	0.149	0.020	0.011	5.7	4.0	1.79
> MIMIMUM, STRATUM A				-	119.1	22.8	30	11	-	0.660	0.050	0.030	0.000	0.000	3.2	3.2	1.73
> MAXIMUM, STRATUM A				-	123.8	25.0	65	43	-	0.750	0.316	0.188	0.023	0.013	10.0	17.2	9.85
> AVERAGE, STRATUM A				-	121.9	24.3	46	27	-	0.702	0.235	0.138	0.017	0.010	6.7	7.8	5.32
STRATUM D																	
B-305DH	UD-4	53.0	-23.2	3.510	125.2	26.7	45	25	CL	0.710	0.252	0.147	0.017	0.010	14.3	3.7	4.46E-02
B-338	UD-2	48.0	-15.9	3.335	122.3	27.4	44	26	CL	0.760	0.256	0.145	0.030	0.017	13.9	4.0	2.59E-01
B-421	UD-2	53.0	-22.7	3.554	120.7	28.1	63	42	CH	0.810	0.213	0.118	0.033	0.018	6.1	1.6	1.39E-01
B-909	UD-2	43.0	-13.3	2.921	121.1	25.7	62	38	CH	0.780	0.086	0.048	0.007	0.004	11.3	3.5	5.18E-01
B-909	UD-3	48.0	-18.3	3.214	115.9	34.9	74	53	CH	0.920	0.468	0.244	0.030	0.016	16.9	4.8	7.34E-02
> MIMIMUM, STRATUM D				-	115.9	25.7	44	25	-	0.710	0.086	0.048	0.007	0.004	6.1	1.6	4.46E-02
> MAXIMUM, STRATUM D				-	125.2	34.9	74	53	-	0.920	0.468	0.244	0.033	0.018	16.9	4.8	5.18E-01
> AVERAGE, STRATUM D				-	121.0	28.6	58	37	-	0.796	0.255	0.141	0.023	0.013	12.5	3.5	2.07E-01
STRATUM F																	
B-303	UD-2	88.0	-61.4	5.182	127.9	28.6	57	39	CH	0.810	0.249	0.138	0.040	0.022	13.4	2.2	3.08E-01
B-419DH	UD-1	78.0	-48.3	4.990	127.8	23.4	47	23	CL	0.630	0.243	0.149	0.037	0.023	18.0	3.3	1.50E-01
B-421	UD-3	83.0	-52.7	5.339	125.9	24.6	56	36	CH	0.700	0.229	0.135	0.040	0.024	18.0	3.2	3.41
> MIMIMUM, STRATUM F				-	125.9	23.4	47	23		0.630	0.229	0.135	0.037	0.022	13.4	2.2	1.50E-01
> MAXIMUM, STRATUM F				-	128	29	57	39	-	0.810	0.249	0.149	0.040	0.024	18.0	3.3	3.41
> AVERAGE, STRATUM F				-	127	26	53	33	-	0.713	0.240	0.140	0.039	0.023	16.5	2.9	1.29

Table 2.5S.4-12 Summary of Laboratory Consolidation Test Results

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Initial Effective Overburden Pressure (kips per square foot)	Total Unit Weight (pounds/ cubic foot) [1]	Natural Moisture Content (percent) [1]	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	Initial Void Ratio, e_0	Compression Index, C_c	Compression Ratio, CR	Recompression Index, C_r	Recompression Ratio, RR	Preconsolidation Pressure, P_c' (kips per square foot)	Overconsolidation Ratio, OCR	Coefficient of Consolidation, c_v ($feet^2$ / day)
SUB-STRATUM J CLAY 1																	
B-305DH	UD-7	123.0	-93.2	7.845	129.2	21.6	-	-	CH	0.590	0.186	0.117	0.030	0.019	18.9	2.3	6.00
B-305DH	UD-8	138.0	-108.2	8.784	129.5	19.0	32	18	CL	0.550	0.173	0.112	0.023	0.015	16.2	1.8	3.37
B-319DH	UD-1	128.0	-98.0	7.985	125.0	17.6	70	45	CH	0.600	0.130	0.081	0.013	0.008	14.1	1.7	14.17
B-419DH	UD-3	118.0	-88.3	7.551	127.6	24.4	56	39	CH	0.680	0.276	0.164	0.030	0.018	21.4	2.7	4.68E-02
B-419DH	UD-4	138.0	-108.3	8.803	129.3	17.3	40	25	CL	0.520	0.289	0.190	0.086	0.057	15.5	1.7	1.90E-01
> MIMIMUM, STRATUM J CLAY 1				-	125.0	17.3	32	18	-	0.520	0.130	0.081	0.013	0.008	14.1	1.7	4.68E-02
> MAXIMUM, STRATUM J CLAY 1				-	129.5	24.4	70	45	-	0.680	0.289	0.190	0.086	0.057	21.4	2.7	14.17
> AVERAGE, STRATUM J CLAY 1				-	128.1	20.0	50	32	-	0.588	0.211	0.133	0.036	0.023	17.2	2.0	4.76
SUB-STRATUM J CLAY 2																	
B-319DH	UD-4	173.0	-144.6	10.802	124.4	26.9	65	43	CH	0.730	0.173	0.100	0.040	0.023	19.3	1.7	5.02E-02
B-319DH	UD-5	188.0	-159.6	11.741	122.7	29.1	62	41	CH	0.790	0.472	0.264	0.060	0.034	27.9	2.3	1.04E-01
B-419DH	UD-5	158.0	-128.3	10.055	129.2	21.3	47	30	CL	0.600	0.199	0.124	0.047	0.029	19.3	1.9	3.64E-02
B-419DH	UD-6	178.0	-148.3	11.307	129.2	23.3	53	33	CH	0.660	0.233	0.140	0.043	0.026	18.4	1.6	8.10E-02
B-419DH	UD-7	198.0	-168.3	12.559	123.5	19.4	56	36	CH	0.560	0.149	0.096	0.027	0.017	15.4	1.2	3.33
> MIMIMUM, SUB-STRATUM J CLAY 2				-	122.7	19.4	47	30	-	0.560	0.149	0.096	0.027	0.017	15.4	1.2	3.64E-02
> MAXIMUM, SUB-STRATUM J CLAY 2				-	129.2	29.1	65	43	-	0.790	0.472	0.264	0.060	0.034	27.9	2.3	3.33
> AVERAGE, SUB-STRATUM J CLAY 2				-	125.8	24.0	57	37	-	0.668	0.245	0.145	0.043	0.026	20.0	1.7	0.72

Table 2.5S.4-12 Summary of Laboratory Consolidation Test Results

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Initial Effective Overburden Pressure (kips per square foot)	Total Unit Weight (pounds/ cubic foot) [1]	Natural Moisture Content (percent) [1]	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	Initial Void Ratio, e_0	Compression Index, C_c	Compression Ratio, CR	Recompression Index, C_r	Recompression Ratio, RR	Preconsolidation Pressure, P_c' (kips per square foot)	Overconsolidation Ratio, OCR	Coefficient of Consolidation, c_v ($feet^2$ / day)
SUB-STRATUM K CLAY																	
B-305DH	UD-11	213.0	-183.2	13.483	126.6	22.6	45	31	CL	0.610	0.249	0.155	0.023	0.014	27.9	2.0	1.34E-01
B-405DH	UD-11	233.0	-201.9	14.839	131.5	18.3	-	-	CH	0.510	0.103	0.068	0.010	0.007	20.2	1.3	2.09
> MIMIMUM, SUB-STRATUM K CLAY				-	126.6	18.3	45	31	-	0.510	0.103	0.068	0.010	0.007	20.2	1.3	1.34E-01
> MAXIMUM, SUB-STRATUM K CLAY				-	131.5	22.6	45	31	-	0.610	0.249	0.155	0.023	0.014	27.9	2.0	2.09
> AVERAGE, SUB-STRATUM K CLAY				-	129.1	20.4	45	31	-	0.560	0.176	0.111	0.017	0.010	24.1	1.7	1.11
SUB-STRATUM N CLAY 1																	
B-305DH	UD-15A	316.0	-286.2	20.068	119.7	29.7	-	-	CH	0.790	0.379	0.212	0.066	0.037	18.9	0.9	5.13E-02
SUB-STRATUM N CLAY 5																	
B-405DH	UD-20	458.5	-427.4	28.860	123.7	30.0	-	-	CH	0.870	0.292	0.156	0.033	0.018	17.9	0.6	4.42E-02
> MINIMUM, SUB-STRATUM N CLAY				20.068	119.7	29.7	-	-	-	0.790	0.292	0.156	0.033	0.018	17.9	0.6	4.42E-02
> MAXIMUM, SUB-STRATUM N CLAY				28.860	123.7	30.0	-	-	-	0.870	0.379	0.212	0.066	0.037	18.9	0.9	5.13E-02
> AVERAGE, SUB-STRATUM N CLAY				24.464	121.7	29.9	-	-	-	0.830	0.336	0.184	0.050	0.027	18.4	0.8	4.78E-02

[1] Initial (pre-saturation) unit weights and moisture contents. Note that all consolidation tests were conducted on fully-saturated test specimens

Table 2.5S.4-13 Summary of Overconsolidation Ratios and Past Preconsolidation Pressures

Stratum	Average P_c'(ksf)	Average OCR
From Laboratory Consolidation Tests		
A/A (Fill)	6.7	7.8
D	12.5	3.5
F	16.5	2.9
J Clay	18.6	1.9
K Clay	24.1	1.3
L	Not Tested	Not Tested
N Clay	18.4	0.8
From Correlations with CPT Data		
A/A (Fill)	N/A	10.0
D	N/A	3.0
F	N/A	2.2
J Clay	N/A	1.8
K Clay	N/A	Not Reached
L	N/A	Not Reached
N Clay	N/A	Not Reached
Selected Values for Engineering Use		
A/A (Fill)	6.3	7.0
D	12.3	3.3
F	15.5	2.6
J Clay	18.5	1.7
K Clay	18.3	1.3
L	16.0	1.0
N Clay	28.5	1.0

Table 2.5S.4-14 Summary of High Strain Elastic Moduli Estimates

Strata A/A (Fill) through E					
Relationship Employed	**Stratum**				
	A/A (Fill)	**B**	**C**	**D**	**E**
E = 36 N	N/A	360	1,260	N/A	1,080
E = 600 S_u	960	N/A	N/A	1,800	N/A
$E_{.375\%}$ = f (V_s)	N/A	515	606	N/A	1,149
$E_{.375\%}$ = f (PI)	1,112	N/A	N/A	2,807	N/A
E Value Selected for Engineering Use	1,050	460	850	2,500	1,100
Strata F through K Clay					
Relationship Employed	**Stratum**				
	F	**H**	**J Clay**	**J Sand**	**K Clay**
E = 36 N	N/A	1,080	N/A	1,260	N/A
E = 600 S_u	1,920	N/A	2,100	N/A	1,800
$E_{.375\%}$ = f (V_s)	N/A	1,193	N/A	1,641	N/A
$E_{.375\%}$ = f (PI)	3,028	N/A	4,157	N/A	3,787
E Value Selected for Engineering Use	2,600	1,150	3,500	1,500	3,100
Strata K Sand/Silt through N Sand					
Relationship Employed	**Stratum**				
	K Sand/Silt	**L**	**M**	**N Clay**	**N Sand**
E = 36 N	1,080	N/A	1,080	N/A	720
E = 600 S_u	N/A	1,800	N/A	1,800	N/A
$E_{.375\%}$ = f (V_s)	1,924	N/A	1,391	N/A	2,831
$E_{.375\%}$ = f (PI)	N/A	3,718	N/A	5,794	N/A
E Value Selected for Engineering Use	1,650	3,100	1,300	4,500	2,100

Table 2.5S.4-15 Summary of High Strain Shear Moduli Estimates

Strata A/A (Fill) through E					
Relationship Employed	Stratum				
	A/A (Fill)	B	C	D	E
$G_{.375\%} = f(V_s)$	N/A	212	233	N/A	442
$G_{.375\%} = f(PI)$	384	N/A	N/A	968	N/A
$G_{.375\%} = f(s_u)$	331	N/A	N/A	621	N/A
$G_{.375\%} = f(N)$	N/A	139	485	N/A	415
G Value Selected for Engineering Use	360	185	320	850	425
Strata F through K Clay					
Relationship Employed	Stratum				
	F	H	J Clay	J Sand	K Clay
$G_{.375\%} = f(V_s)$	N/1	459	N/A	631	N/A
$G_{.375\%} = f(PI)$	1,044	N/A	1,433	N/A	1,306
$G_{.375\%} = f(s_u)$	662	N/A	724	N/A	621
$G_{.375\%} = f(N)$	N/A	415	N/A	485	N/A
G Value Selected for Engineering Use	900	450	1,200	600	1,050
Strata K Sand/Silt through N Sand					
Relationship Employed	Stratum				
	K Sand/Silt	L	M	N Clay	N Sand
$G_{.375\%} = f(V_s)$	740	N/A	535	N/A	1,089
$G_{.375\%} = f(PI)$	N/A	1,282	N/A	1,998	N/A
$G_{.375\%} = f(s_u)$	N/A	621	N/A	621	N/A
$G_{.375\%} = f(N)$	415	N/A	415	N/A	277
G Value Selected for Engineering Use	650	1,050	500	1,500	800

Table 2.5S.4-16 Summary of Average Geotechnical Engineering Parameters

Parameter [1]	Stratum				
	A/A (Fill)	B	C	D	E
Average Thickness, feet	18	7	20	22	18
USCS Group Symbol	CH, CL	ML, CL, SM	SM, SP-SM	CH, CL	SP-SM, SM
Natural Moisture content (MC), %	23	24	24	25	21
Moist Unit Weight, ($_{moist}$), pcf	124	121	122	121	122
Fines content, %	94	71	25	72	18
Liquid Limit (LL), %	57	38	N/A	58	N/A
Plasticity Index (PI), %	37	19	N/A	38	N/A
Uncorrected SPT N-value, bpf	10	9	24	15	35
Corrected SPT $(N_1)_{60}$-value, bpf	15	10	35	15	30
Shear Wave Velocity (V_s), feet/sec	575	725	785	925	1,080
Undrained shear strength (s_U), ksf	1.6	N/A	N/A	3.0	N/A
Drained Friction Angle (Ø'), degrees	N/A	30	35	20	35
Drained Cohesion, (c’), ksf	N/A	N/A	N/A	N/A	N/A
Elastic modulus (High Strain) (E_s), ksf	1,050	460	850	2,500	1,100
Shear modulus (High Strain) (G_s), ksf	360	185	320	850	425
Shear modulus (Low Strain) (G_{max}), ksf	1,270	1,970	2,740	3,210	4,420
Coefficient of Subgrade Reaction (k_1), kcf	150	160	600	300	600
Earth Pressure Coefficients					
- Active (K_a)	0.5	0.3	0.3	0.5	0.3
- Passive (K_p)	2.0	3.0	3.7	2.0	3.7
- At-rest (K_0)	0.7	0.5	0.4	0.7	0.4
Sliding Coefficient (tangent)	0.30	0.35	0.40	0.30	0.40
Consolidation Properties					
- Compression Index (C_c)	0.235	N/A	N/A	0.255	N/A
- Recompression Index (Cr)	0.017	N/A	N/A	0.023	N/A
- Preconsolidation Pressure (Pc’), ksf	6.3	N/A	N/A	12.3	N/A
- Overconsolidation Ratio (OCR)	7.0	N/A	N/A	3.3	N/A

[1] The values tabulated above are guidelines. Reference should be made to the specific boring log, CPT log, and laboratory test results for appropriate modifications at specific locations and/or for specific calculations

Table 2.5S.4-16 Summary of Average Geotechnical Engineering Parameters (Continued)

Parameter [1]	Stratum				
	A/A (Fill)	B	C	D	E
Average Thickness, feet	16		17.5		61 [2]
USCS Group Symbol	CH, CL		SP-SM, SM		CH, CL
Natural Moisture content (MC), %	24		19		23
Moist Unit Weight, ($_{moist}$), pcf	125		128		125
Fines content, %	89		16		89
Liquid Limit (LL), %	58		N/A		54
Plasticity Index (PI), %	38		N/A		35
Uncorrected SPT N-value, bpf	22		44		31
Corrected SPT $(N_1)_{60}$-value, bpf	15		30		15
Shear Wave Velocity (V_s), feet/sec	945		1,075		1,145
Undrained shear strength (s_U), ksf	3.2		N/A		3.5
Drained Friction Angle (Ø'), degrees	20		35		20
Drained Cohesion, (c'), ksf	N/A		N/A		0
Elastic modulus (High Strain) (E_s), ksf	2,600		1,150		3,500
Shear modulus (High Strain) (G_s), ksf	900		450		1,200
Shear modulus (Low Strain) (G_{max}), ksf	3,470		4,590		5,090
Coefficient of Subgrade Reaction (k_1), kcf	300		600		N/A
Earth Pressure Coefficients					
- Active (K_a)	0.5		0.3		N/A
- Passive (K_p)	2.0		3.7		N/A
- At-rest (K_0)	0.7		0.4		N/A
Sliding Coefficient (tangent)	0.30		0.40		N/A
Consolidation Properties					
- Compression Index (C_c)	0.240		N/A		0.228
- Recompression Index (Cr)	0.039		N/A		0.040
- Preconsolidation Pressure (Pc'), ksf	15.5		N/A		18.5
- Overconsolidation Ratio (OCR)	2.6		N/A		1.7

[1] The values tabulated above are guidelines. Reference should be made to the specific boring log, CPT log, and laboratory test results for appropriate modifications at specific locations and/or for specific calculations

[2] Sub-stratum J Clay thickness = combined thickness of J Clay 1 (29 feet) + J Clay 2 (32 feet)

[3] Sub-stratum J Sand thickness = combined thickness of J Interbed 1 (9 feet) + J Sand 1 (13.5 feet)+ J Interbed 2 (15 feet)

Table 2.5S.4-16 Summary of Average Geotechnical Engineering Parameters (continued)

Parameter [1]	Stratum				
	K Sand/Silt	L	M	N Clay	N Sand
Average Thickness, feet	25.5	5	15	>228 [2]	93.5 [3]
USCS Group Symbol	SM, ML	CH	SM	CH, CL	SM, SP-SM
Natural Moisture content (MC), %	21	29	21 [5]	25	23
Moist Unit Weight, ($_{moist}$), pcf	127	129 [4]	127 [5]	121	128
Fines content, %	45	75 [4]	45 [5]	75	22
Liquid Limit (LL), %	N/A	73	N/A	65	N/A
Plasticity Index (PI), %	N/A	52	N/A	44	N/A
Uncorrected SPT N-value, bpf	75	23	75 [5]	33	97
Corrected SPT $(N_1)_{60}$-value, bpf	30	8	30 [5]	7	20
Shear Wave Velocity (V_s), feet/sec	1,370	975	1,165	1,290	1,655
Undrained shear strength (s_U), ksf	N/A	3.0	N/A	3.0	N/A
Drained Friction Angle (Ø'), degrees	33	N/A	33 [5]	N/A	36
Drained Cohesion, (c'), ksf	N/A	N/A	N/A	N/A	N/A
Elastic modulus (High Strain) (E_s), ksf	1,650	3,100	1,300	4,500	2,100
Shear modulus (High Strain) (G_s), ksf	650	1,050	500	1,500	800
Shear modulus (Low Strain) (G_{max}), ksf	7,400	3,810	5,350	6,250	10,890
Coefficient of Subgrade Reaction (k_1), kcf	N/A	N/A	N/A	N/A	N/A
Earth Pressure Coefficients					
- Active (K_a)	N/A	N/A	N/A	N/A	N/A
- Passive (K_p)	N/A	N/A	N/A	N/A	N/A
- At-rest (K_0)	N/A	N/A	N/A	N/A	N/A
Sliding Coefficient (tangent)	N/A	N/A	N/A	N/A	N/A
Consolidation Properties					
- Compression Index (C_c)	N/A	0.176 [4]	N/A	0.336	N/A

Table 2.5S.4-16 Summary of Average Geotechnical Engineering Parameters (continued)

- Recompression Index (Cr)	N/A	0.017 [4]	N/A	0.050	N/A
- Preconsolidation Pressure (Pc'), ksf	N/A	16.0	N/A	28.5	N/A
- Overconsolidation Ratio (OCR)	N/A	1.0	N/A	1.0	N/A

[1] The values tabulated above are guidelines. Reference should be made to the specific boring log, CPT log, and laboratory test results for appropriate modifications at specific locations and/or for specific calculations

[2] Sub-stratum N Clay thickness = combined thickness of N Clay 1 (59 feet) + N Clay 2 (8 feet) + N Clay 3 (8.5 feet) + N Clay 4 (30 feet) + N Clay (54 feet) + N Clay 6 (>68.5 feet)

[3] Sub-stratum N Sand thickness = combined thickness of N Sand 1 (17 feet) + N Sand 2 (32.5 feet) + N Sand 3 (18.5 feet) + N Sand 4 (8 feet) + N Sand 5 (17.5 feet)

[4] Value from Sub-stratum K Clay selected

[5] Value from Sub-stratum K Sand/Silt selected

Table 2.5S.4-17 Summary of Field Electrical Resistivity Test Results

Test Number	Ground Surface El. (feet)	Electrical Resistivity (ohm-meters)									
		Electrode Spacing (feet)									
		3	5	7.5	10	15	30	50	100	200	300
		Sensed Strata; Inferred									
		A	A	A	A	A/B	C	D/E	F/H	J	N
ER-301	30.5	11.554	10.868	10.169	5.152	5.113	8.101	10.533	11.874	13.406	13.789
ER-401	31.5	7.021	6.588	6.076	6.033	6.176	7.871	9.671	11.682	12.257	13.214
ER-901	31.1	7.699	6.425	5.228	4.960	5.085	7.469	9.384	11.491	13.023	13.214
ER-902	31.1	6.492	5.899	4.869	4.941	5.113	7.354	9.193	10.533	12.640	12.640
Minimum		6.492	5.899	5.228	4.941	5.085	7.354	9.384	10.533	12.257	12.640
Maximum		11.554	10.868	10.169	6.033	6.176	8.101	10.533	11.874	13.406	13.789
Average		8.192	7.445	6.586	5.272	5.372	7.699	9.695	11.395	12.832	13.214

Table 2.5S.4-18 Guidelines for the Evaluation of Soil Chemistry

Potential for Attack on Buried Steel (Corrosiveness/Chlorides)					
	Range For Steel Corrosiveness				
Parameter	**Non-Corrosive**	**Mildly Corrosive**	**Moderately Corrosive**	**Corrosive**	**Very Corrosive**
Resistivity (ohm-meters)	>100 [1, 2]	20-100 [1] 50-100 [2] >30 [2, 3]	10-20 [1] 20-50 [2]	5-10 [1] 7-20 [2]	<5 [1] <7 [2]
pH		>5 and <10 [2]		5-6. 5 [1]	<5 [1]
Chlorides (ppm)		<200 [2]		300-1,000 [1]	>1,000 [1]

Potential for Attack on Concrete in Contact with the Ground (Aggressiveness/Sulphates)			
Recommendations For Normal Weight Concrete Subject To Sulphate Attack [4]			
Concrete Exposure	**Water Soluble Sulfate (SO_4) in Soil, %**	**Cement Type**	**MaximumWater/ Cement Ratio**
Mild	0.00-0.10	---	---
Moderate	0.10-0.20	II, IP(MS), IS(MS)	0.5
Severe	0.20-2.00	V [5]	0.45
Very Severe	Over 2.00	V with pozzolan	0.45

[1] After Reference 2.5S.4-16

[2] After Reference 2.5S.4-17

[3] After Reference 2.5S.4-17, provided that 5<pH<10, chlorides <200 ppm, and sulfates <1,000 ppm

[4] After Reference 2.5S.4-18

[5] Alternatively, a blend of Type II cement and a ground granulated blast furnace slag or a pozzolan that gives equivalent sulfate resistance, can be considered

Table 2.5S.4-19 As-Built Boring Information

Boring Number	Northing [1] (feet)	Easting [1] (feet)	Ground El. [2] (feet)	Depth (feet)	Base El. [2] (feet)
BORINGS - STP 3					
B-301	63,000.83	43,271.38	28.1	200.0	-171.9
B-302DH	63,000.73	43,364.78	30.0	220.0	-190.0
B-303	63,001.22	43,456.09	26.6	200.0	-173.4
B-304	63,095.40	43,268.83	28.2	200.0	-171.8
B-305DH	63,099.59	43,364.19	29.8	495.0	-465.2
B-305DHA	63,100.87	43,343.98	29.8	618.0	-588.2
B-306	63,098.22	43,472.95	27.8	200.0	-172.2
B-307	63,196.58	43,269.07	28.2	200.0	-171.8
B-308DH	63,196.49	43,363.84	29.8	215.0	-185.2
B-309	63,197.07	43,455.89	26.6	200.0	-173.4
B-310	63,283.70	43,265.50	28.2	200.0	-171.8
B-311	63,286.55	43,363.47	29.9	100.0	-70.1
B-312	63,286.42	43,473.97	28.3	100.0	-71.7
B-313	63,149.10	43,486.09	28.2	100.0	-71.8
B-314	63,148.73	43,617.01	29.2	200.0	-170.8
B-315	63,366.12	43,511.58	27.7	150.0	-122.3
B-316	63,304.98	43,617.51	28.9	200.0	-171.1
B-317	63,364.01	43,235.44	28.5	150.0	-121.5
B-318	63,363.37	43,297.42	28.5	100.0	-71.5
B-319DH	63,364.17	43,407.90	28.4	215.0	-186.6
B-320	62,903.74	43,116.74	30.5	50.0	-19.5
B-321	63,483.05	43,231.24	29.2	150.0	-120.8
B-322C	63,483.40	43,406.69	30.1	100.0	-69.9
B-323	63,484.30	43,515.99	29.8	100.0	-70.2
B-324	63,570.87	43,233.90	29.5	100.0	-70.5
B-325	63,569.94	43,299.20	30.2	100.0	-69.8
B-326	63,572.01	43,519.56	30.4	150.0	-119.6
B-327	63,658.77	43,233.17	29.8	150.0	-120.2
B-328DH	63,660.26	43,298.12	29.9	218.0	-188.1

Table 2.5S.4-19 As-Built Boring Information

Boring Number	Northing [1] (feet)	Easting [1] (feet)	Ground El. [2] (feet)	Depth (feet)	Base El. [2] (feet)
BORINGS - STP 3 (continued)					
B-329	63,658.33	43,410.29	29.6	100.0	-70.4
B-330	63,660.32	43,518.07	29.5	150.0	-120.5
B-331	63,635.24	43,541.59	29.8	100.0	-70.2
B-332	63,738.50	43,601.33	30.3	150.0	-119.7
B-333	63,744.16	43,360.57	30.5	100.0	-69.5
B-334	63,751.04	43,254.47	30.5	100.0	-69.5
B-335	63,735.38	43,042.50	31.2	75.0	-43.8
B-336	63,680.97	42,936.21	31.1	75.0	-43.9
B-337	63,680.83	43,151.07	30.3	75.0	-44.7
B-338	63,791.50	42,935.72	32.1	75.0	-42.9
B-339	63,790.00	43,148.53	30.8	75.0	-44.2
B-340	63,281.77	43,151.48	30.5	100.0	-69.5
B-341	63,215.13	43,096.25	30.6	100.0	-69.4
B-342	63,215.34	43,175.33	30.7	100.0	-69.3
B-343	63,125.99	43,095.29	30.5	200.0	-169.5
B-344	63,056.54	43,096.13	30.6	100.0	-69.4
B-345	63,040.70	43,173.35	30.7	100.0	-69.3
B-346	62,809.88	43,006.37	30.4	75.0	-44.6
B-347	62,746.63	42,985.26	31.2	75.0	-43.8
B-348	62,683.87	43,004.72	30.0	125.0	-95.0
B-349	62,901.92	43,593.47	29.2	125.0	-95.8
B-350	63,539.30	42,960.25	30.8	100.0	-69.2
B-917 [3]	63,694.58	42,832.71	31.1	50.0	-18.9
BORINGS - STP 4					
B-401	62,999.23	42,370.55	31.1	200.0	-168.9
B-402DH	62,998.09	42,462.29	30.9	215.0	-184.1
B-403	62,998.59	42,555.20	31.5	200.0	-168.5
B-404	63,097.53	42,369.54	31.0	200.0	-169.0
B-405DH	63,098.12	42,462.95	31.1	618.0	-586.9
B-406	63,098.20	42,556.69	31.2	200.0	-168.8
B-407	63,195.82	42,369.78	31.3	200.0	-168.7
B-408DH	63,194.11	42,463.86	31.2	200.0	-168.8

Table 2.5S.4-19 As-Built Boring Information

Boring Number	Northing [1] (feet)	Easting [1] (feet)	Ground El. [2] (feet)	Depth (feet)	Base El. [2] (feet)
BORINGS - STP 4 (continued)					
B-409	63,195.47	42,557.98	31.2	200.0	-168.8
B-410	63,286.47	42,369.53	31.7	100.0	-68.3
B-411	63,285.65	42,461.25	31.3	100.0	-68.7
B-412	63,287.51	42,553.81	31.4	100.0	-68.6
B-413	63,148.27	42,585.19	31.2	100.0	-68.8
B-414	63,147.67	42,746.89	32.2	150.0	-117.8
B-415	63,355.53	42,599.76	30.0	150.0	-120.0
B-416	63,301.73	42,746.36	31.8	150.0	-118.2
B-417	63,361.95	42,331.19	29.6	150.0	-120.4
B-418	63,361.76	42,433.17	29.8	100.0	-70.2
B-419DH	63,362.12	42,506.69	29.7	215.0	-185.3
B-420	62,900.80	42,008.75	31.9	125.0	-93.1
B-421	63,483.06	42,328.30	30.3	100.0	-69.7
B-422C	63,483.67	42,510.68	31.2	100.0	-68.8
B-423	63,485.34	42,615.65	31.6	100.0	-68.4
B-424	63,571.98	42,329.57	30.3	100.0	-69.7
B-425	63,571.49	42,397.45	30.5	100.0	-69.5
B-426	63,571.71	42,615.14	31.4	100.0	-68.6
B-427	63,660.84	42,331.92	30.6	150.0	-119.4
B-428DH	63,660.05	42,398.55	30.9	218.0	-187.1
B-429	63,660.04	42,505.46	31.2	100.0	-68.8
B-430	63,624.24	42,617.30	30.9	150.0	-119.1
B-431	63,634.57	42,641.92	31.1	75.0	-43.9
B-432	63,739.93	42,701.18	31.2	150.0	-118.8
B-433	63,747.31	42,458.80	31.6	100.0	-68.4
B-434	63,752.98	42,354.31	31.1	100.0	-68.9
B-435	63,736.38	42,141.62	28.9	75.0	-46.1
B-436	63,681.44	42,034.98	30.3	75.0	-44.7
B-437	63,679.95	42,247.72	28.2	75.0	-46.8
B-438	63,791.36	42,003.39	30.2	125.0	-94.8
B-439	63,790.82	42,250.03	28.7	125.0	-96.3
B-440	63,281.42	42,249.68	31.1	200.0	-168.9

Table 2.5S.4-19 As-Built Boring Information

Boring Number	Northing [1] (feet)	Easting [1] (feet)	Ground El. [2] (feet)	Depth (feet)	Base El. [2] (feet)
BORINGS - STP 4 (continued)					
B-450	63,539.57	42,057.93	28.8	100.0	-71.2
BORINGS - UHS BASIN/ RSW					
B-901	63,771.76	41,809.14	29.3	100.0	-70.7
B-902	63,496.08	41,927.00	29.1	100.0	-70.9
B-903	63,672.23	41,664.45	30.0	100.0	-70.0
B-904	63,485.07	41,727.16	29.8	100.0	-70.2
B-905	63,348.01	41,571.36	29.2	100.0	-70.8
B-906	63,574.46	41,430.55	29.5	100.0	-70.5
B-907	63,549.17	41,252.15	29.2	100.0	-70.8
B-908	63,273.09	41,356.36	29.6	100.0	-70.4
B-909	63,521.67	41,590.66	29.7	100.0	-70.3
B-910	63,362.31	41,257.10	30.4	125.0	-94.6
B-911	63,254.68	41,663.52	30.8	50.0	-19.2
B-912	63,253.49	41,860.53	31.1	100.0	-68.9
B-913	63,253.07	42,031.18	30.6	50.0	-19.4
B-914	63,218.30	42,181.90	28.2	100.0	-71.8
B-915	63,357.95	42,118.79	29.0	50.0	-21.0
B-916	63,599.37	42,120.70	27.8	50.0	-22.2
BORINGS - OUTSIDE POWER BLOCK					
B-918	64,814.60	42,764.10	30.9	100.0	-69.1
B-919	64,814.59	43,088.48	31.9	100.0	-68.1
B-920	62,943.94	43,897.79	28.2	30.0	-1.8
B-927	62,183.19	49,228.65	26.8	60.0	-33.2
B-928	64,932.77	40,366.26	29.6	125.0	-95.4
B-929	64,672.42	45,487.07	36.6	130.0	-93.4
B-930	60,212.08	49,516.47	25.6	120.0	-94.4
B-931	61,984.41	39,511.72	29.9	125.0	-95.1
B-932	61,899.52	42,106.11	31.0	125.0	-94.0
B-933	61,895.26	43,504.02	28.7	125.0	-96.3
B-934	62,081.37	48,244.01	28.6	110.0	-81.4

[1] Coordinates are referenced to the Texas South Central State Plane (NAD 27) grid system. Note that for brevity the "3" was eliminated from the Northing and the "29" was eliminatedfrom the Easting

[2] Elevations are referenced to NGVD 29 datum

[3] Boring B-917, made midway between STP 3 and STP 4, is included with STP 3 here

Table 2.5S.4-20 Undisturbed Tube Sample Details

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
UNDISTURBED TUBE SAMPLES - STP 3					
B-303	UD1	CH (t); SM (b)	D/ E	63.0	-36.4
B-303	UD2	CH	F	88.0	-61.4
B-303	UD3	SM	H	108.0	-81.4
B-303	UD4	CH	J Clay 1	133.0	-106.4
B-303	UD5	SM	J Interbed 2	168.0	-141.4
B-305DH	UD1	CH	A	3.0	26.8
B-305DH	UD2	NR (may be SP-SM)	C	25.0	4.8
B-305DH	UD3	NR (may be SP-SM)	C	38.0	-8.2
B-305DH	UD3A	NR (may be SP-SM)	C	40.0	-10.2
B-305DH	UD4	CL	D	53.0	-23.2
B-305DH	UD5	SP-SM	E	78.0	-48.2
B-305DH	UD6	CH	H	103.0	-73.2
B-305DH	UD7	CH	J Clay 1	123.0	-93.2
B-305DH	UD8	CL	J Clay 1	138.0	-108.2
B-305DH	UD9	CH (t); ML (b)	J Clay 1/ J Interbed 2	158.0	-128.2
B-305DH	UD10	CH	J Clay 2	193.0	-163.2
B-305DH	UD11	CL	K Clay	213.0	-183.2
B-305DH	UD12	SM	K Sand	228.0	-198.2
B-305DH	UD13	CH (t); SP-SM (b)	L/ M	263.0	-233.2
B-305DH	UD14	CH	N Clay 1	288.0	-258.2
B-305DH	UD15	CH	N Clay 1	313.0	-283.2
B-305DH	UD15A	CH	N Clay 1	316.5	-286.7
B-305DH	UD16	CH	N Clay 1	338.0	-308.2
B-305DH	UD17	SP-SM	N Sand 1	353.0	-323.2
B-305DH	UD17A	SP-SM	N Sand 1	353.5	-323.7
B-305DH	UD18	SP-SM	N Sand 2	385.0	-355.2

Table 2.5S.4-20 Undisturbed Tube Sample Details (Continued)

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
UNDISTURBED TUBE SAMPLES - STP 3 (continued)					
B-305DH	UD20	SP-SM	N Sand 3	418.0	-388.2
B-305DH	UD21	SP-SM	N Sand 4	453.3	-423.5
B-305DH	UD21A	SP-SM	N Sand 4	453.5	-423.7
B-305DHA	UD21	SP-SM	N Sand 4	453.5	-423.7
B-305DHA	UD22	CH	N Clay 5	508.0	-478.2
B-305DHA	UD24	CH	N Clay 6	553.0	-523.2
B-305DHA	UD25	CH	N Clay 6	588.0	-558.2
B-306	UD1	SM	C	38.0	-10.2
B-306	UD1A	SM	C	40.0	-12.2
B-306	UD2	SM	D	63.0	-35.2
B-306	UD3	SC	E	73.0	-45.2
B-306	UD4	CH	F	88.0	-60.2
B-306	UD5	SP-SM	H	98.0	-70.2
B-306	UD6	SP-SM	H	103.0	-75.2
B-306	UD7	GW (t); CH (b)	H/ J Clay 1	118.0	-90.2
B-306	UD8	CH	J Clay 1	141.0	-113.2
B-306	UD9	CH	J Clay 1	151.0	-123.2
B-306	UD9A	CH (t); ML (b)	J Clay 1	153.0	-125.2
B-306	UD10	CH	J Clay 2	191.0	-163.2
B-307	UD1	CH	J Clay 1	118.0	-89.8
B-307	UD2	SM	J Sand 1	153.0	-124.8
B-307	UD3	CH	J Clay 2	188.0	-159.8
B-314	UD1	SP	E	83.0	-53.8
B-314	UD2	CL	J Clay 1	113.0	-83.8
B-314	UD3	CH	J Clay 1	121.0	-91.8
B-314	UD4	SC (t); CL (b)	J Clay 1	141.0	-111.8
B-314	UD5	NR (may be CH)	J Clay 2	181.0	-151.8
B-314	UD5A	CH	J Clay 2	183.0	-153.8
B-314	UD6	CH	J Clay 2	191.0	-161.8

Table 2.5S.4-20 Undisturbed Tube Sample Details (Continued)

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
UNDISTURBED TUBE SAMPLES - STP 3 (continued)					
B-319DH	UD1	CH	J Clay 1	128.0	-99.6
B-319DH	UD2	SM	J Sand 1	143.0	-114.6
B-319DH	UD3	SM	J Sand 1	158.0	-129.6
B-319DH	UD4	CH	J Clay 2	173.0	-144.6
B-319DH	UD5	CH	J Clay 2	188.0	-159.6
B-321	UD1	CH	D	43.0	-13.8
B-321	UD2	CH	J Clay 1	118.0	-88.8
B-321	UD3	CL	J Clay 1	138.0	-108.8
B-328DH	UD1	CL	A	13.0	16.9
B-328DH	UD2	NR (may be SM)	C	33.0	-3.1
B-328DH	UD3	CH	D	53.0	-23.1
B-328DH	UD4	SM	E	73.0	-43.1
B-328DH	UD5	NR (may be SP-SM)	E	83.0	-53.1
B-328DH	UD6	NR (may be SM)	H	103.0	-73.1
B-330	UD1A	NR (may be SM)	C	38.0	-8.5
B-330	UD1B	NR (may be SM)	C	40.0	-10.5
B-330	UD2	CH	D	53.0	-23.5
B-330	UD3	SP (t); SM (b)	E	63.0	-33.5
B-330	UD4	NR (may be SM)	H	118.0	-88.5
B-330	UD4B	CH	J Clay 1	123.0	-93.5
B-332	UD1	CH	A	3.0	27.3
B-332	UD2	ML	B	23.0	7.3
B-333	UD1	CL	A	8.0	22.5
B-333	UD2	CL	A	18.0	12.5
B-338	UD1	SM	C	28.0	4.1
B-338	UD2	CL	D	48.0	-15.9
B-343	UD1	CH (t); SM (b)	B/ C	23.0	7.5
B-343	UD2	SM (t); CH (b)	D	48.0	-17.5
B-343	UD3	CH (t); SM (b)	D/ E	58.0	-27.5

Table 2.5S.4-20 Undisturbed Tube Sample Details (Continued)

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
UNDISTURBED TUBE SAMPLES - STP 3 (continued)					
B-343	UD4	NR (may be SM)	E	68.0	-37.5
B-343	UD4A	CH	E	70.0	-39.5
B-343	UD5	SM	J Interbed 1	123.0	-92.5
B-343	UD6	SM	J Sand 1	148.0	-117.5
B-343	UD7	CL-ML	J Clay 2	173.0	-142.5
B-343	UD8	CH	J Clay 2	198.0	-167.5
B-348	UD1	CL	A	5.0	25.0
B-348	UD2	ML (t); CL (b)	B	13.0	17.0
B-348	UD3	ML (t); SM (b)	B/ C	18.0	12.0
RELATIVELY UNDISTURBED TUBE SAMPLES - STP 4					
B-401	UD1	CH	D	58.0	-26.9
B-401	UD2	CH	F	88.0	-56.9
B-401	UD3	CL	J Clay 1	118.0	-86.9
B-401	UD4	SM	J Sand 1	153.0	-121.9
B-401	UD5	NR (may be CH)	J Clay 2	178.0	-146.9
B-401	UD5A	CH	J Clay 2	184.0	-152.9
B-404	UD1	CH	F	88.0	-57.0
B-404	UD2	CH	F	98.0	-67.0
B-404	UD3	CH	J Clay 1	121.0	-90.0
B-404	UD4	CH	J Clay 1	131.0	-100.0
B-404	UD5	CL	J Clay 1	141.0	-110.0
B-404	UD6	CL	J Clay 2	161.0	-130.0
B-404	UD7	CH	J Clay 2	181.0	-150.0
B-404	UD8	CH	J Clay 2	191.0	-160.0
B-405DH	UD1	CH	A	10.0	21.1
B-405DH	UD2	CL	B	28.0	3.1
B-405DH	UD3	CL	D	63.0	-31.9
B-405DH	UD4	CL	F	83.0	-51.9
B-405DH	UD5	CH	J Clay 1	113.0	-81.9

Table 2.5S.4-20 Undisturbed Tube Sample Details (Continued)

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
RELATIVELY UNDISTURBED TUBE SAMPLES - STP 4 (continued)					
B-405DH	UD6	CL	J Clay 1	125.0	-93.9
B-405DH	UD7	SM	J Sand 1	148.0	-116.9
B-405DH	UD8	CH (t); ML (b)	J Clay 2/ J Interbed 2	168.0	-136.9
B-405DH	UD9	CL	J Clay 2	193.0	-161.9
B-405DH	UD10A	CH	K Clay	222.0	-190.9
B-405DH	UD11	CH	K Clay	233.0	-201.9
B-405DH	UD12	SP-SM	M	263.0	-231.9
B-405DH	UD13	CH	N Clay 1	293.0	-261.9
B-405DH	UD14	CH	N Clay 1	318.0	-286.9
B-405DH	UD15	SP	N Sand 1	343.0	-311.9
B-405DH	UD16	CH	N Clay 2	358.0	-326.9
B-405DH	UD17	SC	N Sand 2	388.0	-356.9
B-405DH	UD18	SP	N Sand 3	418.0	-386.9
B-405DH	UD19	CH	N Clay 4	438.5	-407.4
B-405DH	UD20	CH	N Clay 5	458.5	-427.4
B-405DH	UD21	CH	N Clay 5	488.0	-456.9
B-405DH	UD22	SM	N Clay 5	518.0	-486.9
B-405DH	UD23	SM	N Clay 5	538.0	-506.9
B-405DH	UD24	CH	N Clay 6	568.0	-536.9
B-405DH	UD25	CL	N Clay 6	598.0	-566.9
B-409	UD1	SM	E	68.0	-36.8
B-409	UD2	NR (may be CH)	F	93.0	-61.8
B-409	UD2A	NR (may be CH)	F	95.0	-63.8
B-409	UD3	CH	J Clay 1	128.0	-96.8
B-409	UD4	NR (may be SM)	J Sand 1	158.0	-126.8
B-409	UD4A	CH	J Clay 2	160.0	-128.8
B-409	UD5	CH (t); SP-SM (b)	J Clay 2/ J Interbed 2	188.0	-156.8
B-409	UD6	CH	J Clay 2	198.0	-166.8

Table 2.5S.4-20 Undisturbed Tube Sample Details (Continued)

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
RELATIVELY UNDISTURBED TUBE SAMPLES - STP 4 (continued)					
B-415	UD1	CH	F	88.0	-58.0
B-415	UD2	CH (t); SP-SM (b)	F/ H	98.0	-68.0
B-415	UD3A	NR (may be CH)	J Clay 1	121.0	-91.0
B-415	UD3	CH	J Clay 1	124.0	-94.0
B-415	UD4A	NR (may be CH)	J Clay 1	131.0	-101.0
B-415	UD4	NR (may be CH)	J Clay 1	134.0	-104.0
B-419DH	UD1	CL	F	78.0	-48.3
B-419DH	UD2	CH (t); SM (b)	F/ H	98.0	-68.3
B-419DH	UD3	CH	J Clay 1	118.0	-88.3
B-419DH	UD4	CL	J Clay 1	138.0	-108.3
B-419DH	UD6	CH	J Clay 2	178.0	-148.3
B-419DH	UD7	CH	J Clay 2	198.0	-168.3
B-421	UD1	SM (t); SP-SM (b)	C	33.0	-2.7
B-421	UD1A	SP-SM	C	33.6	-3.3
B-421	UD2	CH	D	53.0	-22.7
B-421	UD3	CH	F	83.0	-52.7
B-428DH	UD1	CH	A	3.0	27.9
B-428DH	UD2	NR (may be SM)	B	23.0	7.9
B-428DH	UD2A	NR (may be SM)	B	25.0	5.9
B-428DH	UD3	CH	D	43.0	-12.1
B-428DH	UD4	CH (t); ML (b)	D	63.0	-32.1
B-428DH	UD5	NR (may be SM)	H	93.0	-62.1
B-428DH	UD5A	NR (may be SM)	H	95.0	-64.1
B-428DH	UD6	CH	J Clay 1	113.0	-82.1
B-430	UD1	CH	D	55.0	-24.1
B-430	UD2	SM	E	83.0	-52.1
B-430	UD3	CH	J Clay 1	133.0	-102.1
B-432	UD1	CH	A	3.0	28.2
B-432	UD2	CL	A	15.0	16.2

Table 2.5S.4-20 Undisturbed Tube Sample Details (Continued)

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
RELATIVELY UNDISTURBED TUBE SAMPLES - STP 4 (continued)					
B-432	UD3	SM	B	25.0	6.2
B-434	UD1	CH	A	8.0	23.1
B-434	UD2	SM	C	28.0	3.1
B-434	SS11	SM	C	33.5	-2.4
B-434	UD3	CH	D	53.0	-21.9
B-438	UD1	CH	A	18.0	12.2
B-438	UD2	NR (may be SM)	C	33.0	-2.8
B-438	UD3	SM (t); ML (b)	C/ D	43.0	-12.8
UNDISTURBED TUBE SAMPLES - UHS BASIN/ RWS					
B-902	UD1	CH	A	5.0	24.1
B-902	UD2	CH	A	15.0	14.1
B-902	UD3	SM	C	23.0	6.1
B-904	UD1	CH	A	5.0	24.8
B-904	UD2	CH	A	18.0	11.8
B-904	UD3	ML	B	28.0	1.8
B-904	UD4	SC	D	53.0	-23.2
B-904	UD5	CH	F	83.0	-53.2
B-907	UD1	CH	A	3.0	26.2
B-907	UD2	CH	A	13.0	16.2
B-907	UD3	SM	B	28.0	1.2
B-909	UD1	SM	C	33.0	-3.3
B-909	UD2	CH	D	43.0	-13.3
B-909	UD3	CH	D	48.0	-18.3
B-909	UD4	CH	D	53.0	-23.3
B-909	UD5	CL	F	85.0	-55.3
B-909	UD6	CH	F	93.0	-63.3
B-909	UD7	CH	F	98.0	-68.3
B-916	UD1	CH	A	13.0	14.8
B-916	UD2	NR (may be SM)	C	28.0	-0.2

Table 2.5S.4-20 Undisturbed Tube Sample Details (Continued)

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
UNDISTURBED TUBE SAMPLES - UHS BASIN/ RWS (continued)					
B-916	UD2A	NR (may be SM)	C	30.0	-2.2
B-916	UD3	CH	D	48.0	-20.2
UNDISTURBED TUBE SAMPLES - OUTSIDE POWER BLOCK					
B-918	UD1	CH	A	3.0	27.9
B-918	UD2	CL	B	18.0	12.9
B-918	UD3	SM	C	25.0	5.9
B-918	UD4	CL-ML	D	58.0	-27.1
B-919	UD1	CH	A	8.0	23.9
B-919	UD2	CH (t); ML (b)	A/ B	23.0	8.9
B-919	UD3	CH	D	43.0	-11.1
B-919	UD4	SP-SM	E	83.0	-51.1
B-927	UD1	SM	B	13.0	13.8
B-927	UD1A	NR (may be SM)	B	15.0	11.8
B-927	UD2	CH	B	28.0	-1.2
B-927	UD3	CH	D	48.0	-21.2

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-21 As-Built CPT Information

CPT Number	Northing [1] (feet)	Easting [1] (feet)	Ground El. [2] (feet)	Depth (feet)	Base El. [2] (feet)
CONE PENETRATION TESTS - STP 3					
C-301	62,772.55	43,448.74	27.4	59.0	-31.6
C-302	62,824.38	43,502.25	28.7	36.1	-7.4
C-303	62,823.77	43,190.19	30.2	50.0	-19.8
C-304	62,910.77	43,394.73	29.4	100.1	-70.7
C-305S	63,126.80	43,174.06	30.9	91.1	-60.2
C-306S	63,483.22	43,296.00	29.7	66.3	-36.6
C-307S	63,573.00	43,407.68	30.0	95.1	-65.1
C-308	63,711.62	43,481.16	29.9	79.4	-49.5
C-309	63,680.96	43,037.71	30.7	100.1	-69.4
C-310	63,792.39	43,037.94	31.4	100.1	-68.7
CONE PENETRATION TESTS - STP 4					
C-401	62,772.46	42,547.21	31.1	50.0	-18.9
C-402	62,824.68	42,600.77	30.8	50.0	-19.2
C-403	62,825.36	42,289.73	31.6	50.0	-18.4
C-404	62,912.73	42,499.09	31.4	37.6	-6.2
C-406S	63,481.68	42,400.33	31.1	93.3	-62.2
C-407S	63,570.38	42,507.31	30.8	98.3	-67.5
C-408	63,710.02	42,579.59	31.7	100.2	-68.5
C-409	63,678.81	42,142.10	27.9	92.0	-64.1
C-410	63,788.88	42,140.63	28.9	92.0	-63.1
C-411	62,902.74	42,803.77	31.1	50.0	-18.9
C-916 [3]	63,217.32	42,280.50	31.4	39.0	-7.6
CONE PENETRATION TESTS - UHS BASIN/ RSW					
C-901	63,539.44	41,694.20	29.6	98.1	-68.5
C-902	63,448.19	41,623.82	28.9	90.1	-61.2
C-903	63,466.93	41,498.80	29.2	93.2	-64.0
C-904	63,392.47	41,651.23	24.2	90.1	-65.9
C-905	63,298.98	41,713.69	31.2	50.0	-18.8
C-906	63,212.72	41,758.97	30.2	50.0	-19.8
C-907	63,219.02	41,968.73	28.5	50.0	-21.5
C-908	63,219.72	42,082.33	30.9	50.0	-19.1
C-917	63,281.30	42,122.51	30.7	50.0	-19.3
C-918	63,484.09	42,118.30	25.4	50.0	-24.6
CONE PENETRATION TESTS - OUTSIDE POWER BLOCK					
C-909	63,464.25	43,948.29	30.2	40.0	-9.8

[1] Coordinates are referenced to the Texas South Central State Plane (NAD 27) grid system. Note that for brevity the "3" was eliminated from the Northing and the "29" was eliminated from the Easting

[2] Elevations are referenced to NGVD 29 datum

[3] CPT C-916, made close-in to Unit 4, is included with STP 4 here

Table 2.5S.4-22 As-Built Observation Well Information

OW Number	Northing [1] (feet)	Easting [1] (feet)	Reference El. [2] (feet)	Well Depth (feet)	Base El. [2] (feet)
OBSERVATION WELLS - STP 3					
OW-308L	63,196.43	43,374.36	29.9	97.1	-67.2
OW-308U	63,195.64	43,354.04	29.9	47.1	-17.2
OW-332La-R	63,729.36	43,608.74	30.0	103.1	-73.1
OW-332U	63,739.21	43,591.02	30.2	46.1	-15.9
OW-348L	62,685.92	43,014.48	30.1	79.1	-49.0
OW-348U	62,685.23	42,994.44	30.5	39.1	-8.6
OW-349L	62,901.84	43,602.97	29.4	81.1	-51.7
OW-349U	62,902.40	43,582.28	29.4	46.1	-16.7
OBSERVATION WELLS - STP 4					
OW-408L	63,196.18	42,472.54	31.7	81.3	-49.6
OW-408U	63,194.01	42,456.01	31.5	43.1	-11.6
OW-420U	62,902.15	42,018.94	32.3	49.1	-16.9
OW-438L	63,790.77	42,045.09	30.1	104.1	-74.0
OW-438U	63,792.04	42,025.17	30.5	41.0	-10.5
OBSERVATION WELLS - UHS BASIN/ RSW					
OW-910L	63,363.45	41,266.45	30.8	92.1	-61.4
OW-910U	63,362.02	41,246.57	30.7	36.1	-5.4
OBSERVATION WELLS - OUTSIDE POWER BLOCK					
OW-928L	64,932.30	40,376.21	29.8	121.1	-91.3
OW-928U	64,933.86	40,356.48	30.0	39.6	-9.6
OW-929L	64,671.50	45,497.78	36.9	98.1	-61.2
OW-929U	64,672.34	45,477.58	36.9	60.1	-23.2
OW-930L	60,214.45	49,525.96	26.2	106.5	-80.3
OW-930U	60,209.72	49,506.58	25.6	36.1	-10.5
OW-931U	61,979.42	39,520.36	30.5	36.0	-5.5
OW-932L	61,899.37	42,115.90	31.1	79.6	-48.5
OW-932U	61,898.53	42,097.29	31.4	39.6	-8.2
OW-933L	61,898.05	43,515.01	28.7	87.1	-58.4
OW-933U	61,897.65	43,494.66	28.9	37.1	-8.2
OW-934L	62,082.08	48,254.12	29.0	100.0	-71.0
OW-934U	62,079.87	48,234.20	28.5	41.1	-12.6

[1] Coordinates are referenced to the Texas South Central State Plane (NAD 27) grid system. Note that for brevity the "3" was eliminated from the Northing and the "29" was eliminated from the Easting

[2] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-23 Insitu Hydraulic Conductivity (Slug Test Results)

Observation Well	Sand Intake El. [2] (feet)	Stratum	USCS Group	Test Type [1]					
				Rising Head Method			Falling Head Method		
				Butler	KGS	B-R	Butler	KGS	B-R
OW-308L	-52.2 to -67.2	E/H	SP-SM	64	67	65	72	73	56
OW-308U	-2.1 to -17.2	C	SP-SM	70	64	63	64	62	68
OW-332L	-57.0 to -73.1	E/H	SM	53	54	P [3]	49	49	55
OW-332U	-0.8 to -15.9	C	SM	37	36	27	19	18	11
OW-348L	-33.9 to -49.0	E	SP-SM	58	46	44	76	61	39
OW-348U	6.5 to -8.6	C	SM	P [3]	83	88	68	71	65
OW-349L	-35.6 to -51.7	D/E	SM	63	51	35	43	40	52
OW-349U	-1.6 to -16.7	C	SM	P [3]	P [3]	43	P [3]	P [3]	53
OW-408L	-34.3 to -49.6	E	SP-SM	P [3]	72	P [3]	70	68	50
OW-408U	3.5 to -11.6	C	SM	17	11	11	22	32	28
OW-420U	-1.8 to -16.9	C	SM	P [3]	33	45	ND [4]	ND [4]	ND [4]
OW-438L	-58.9 to -74.0	F/H	SM	17	27	10	15	28	14
OW-438U	4.5 to -10.5	B/C	SM	38	39	26	P [3]	P [3]	24
OW-910L	-46.3 to -61.4	F	CH	3	0.3	0.6	2	0.9	0.5
OW-910U	9.7 to -5.4	B/C	SM	26	29	21	P [3]	P [3]	P [3]
OW-928L	-76.2 to -91.3	F/H	SP	19	11	7	P [3]	24	21
OW-928U	5.5 to -9.6	C	SM	19	P [3]	8	19	16	16
OW-929L	-46.2 to -61.2	H	SP-SM	56	54	29	59	P [3]	59
OW-929U	-8.1 to -23.2	D/E/F	CH	P [3]	3	4	P [3]	12	2
OW-930L	-64.8 to -80.3	H	SP	40	37	27	24	15	19
OW-930U	4.6 to -10.5	B/C	SM	P [3]	23	32	P [3]	47	48
OW-931U	9.5 to -5.5	C	SM	34	23	20	P [3]	P [3]	49
OW-932L	-33.4 to -48.5	D/E	SM	24	23	18	22	22	25
OW-932U	6.9 to -8.3	B/C	SM	21	13	14	P [3]	16	22
OW-933L	-43.3 to -58.4	F	CH	P [3]	51	63	P [3]	P [3]	64
OW-933U	5.9 to -8.2	B/C	ML	P [3]	10	3	8	5	3
OW-934L	-56.0 to -71.0	E	SM	P [3]	P [3]	35	P [3]	P [3]	32
OW-934U	2.5 to -12.6	C	SM	P [3]	32	33	49	P [3]	40

[1] Refer to Subsection 2.4S.12 for details on testing and analysis methods
[2] Elevations are referenced to NGVD 29 datum.
[3] “P” denotes tests with a poor curve match or questionable data
[4] “ND” denotes no data (data not recovered from the data logger)

Table 2.5S.4-24 Summary of Test Pit Positions and Bulk Soil Sample Details

Test Pit Number	Position	Bulk Sample Description	Stratum (Bulk Sample Depth)
TP-B322C	Adjoining B-322C (STP 3 Turbine Building)	BEAUMONT; black; silt; CLAY (CH)	Stratum A (1.5 to 6.0 feet depth)
TP-B409	Adjoining B-409 (STP 4 Reactor Building)	BEAUMONT; black; silt; CLAY (CH)	Stratum A (1.5 to 6.5 feet depth)
TP-B919	Adjoining B-919 (Switch Yard)	BEAUMONT; black; silt; sand; CLAY (CH)	Stratum A (0.5 to 6.0 feet depth)
		BEAUMONT; red; silt; CLAY (CH)	Stratum A (6.0 to 8.5 feet depth)
TP-B927	Adjoining B-927 (Training Center)	BEAUMONT; black; silt; sand; CLAY (CL)	Stratum A (0.5 to 4.0 feet depth)
		BEAUMONT; yellow-red; silt; sand; CLAY (CL)	Stratum A (5.5 to 8.5 feet depth)
TP-C304	Adjoining C-304 (STP 3 Power Block)	BEAUMONT; black; silt; sand; CLAY (CH)	Stratum A (3.0 to 7.0 feet depth)
		BEAUMONT; red-brown; silt; sand; CLAY (CL)	Stratum A (7.0 to 9.0 feet depth)
TP-C404	Adjoining C-404 (STP 4 Power Block)	BEAUMONT; black; silt; CLAY (CH)	Stratum A (2.0 to 7.0 feet depth)
		BEAUMONT; red; silt; CLAY (CH)	Stratum A (7.0 to 9.0 feet depth)

Table 2.5S.4-25 As-Built Field Electrical Resistivity Information

ER Number	Northing [1] (feet)	Easting [1] (feet)	Ground El. [2] (feet)
ELECTRICAL RESISTIVITY TESTS - STP 3			
ER-301	63,748.20	43,308.16	30.5
ELECTRICAL RESISTIVITY TESTS - STP 4			
ER-401	63,753.46	42,407.42	31.5
ELECTRICAL RESISTIVITY TESTS - OUTSIDE POWER BLOCK			
ER-901	64,722.85	42,995.07	31.1
ER-902	64,722.85	42,995.07	31.1

[1] Coordinates are referenced to the Texas South Central State Plane (NAD 27) grid system. Note that for brevity the "3" was eliminated from the Northing and the "29"was eliminated from the Easting

[2] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-26 Summary of Laboratory Compaction and CBR Test Results

Test Number	Sample Depth (feet)	Natural Moisture Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	USCS Group	Maximum Dry Density [1] (pounds/ cubic foot)	Optimum Moisture Content [1] (percent)	California Bearing Ratio/ CBR [2] (percent)
STRATUM A (UPPER; SAMPLES GENERALLY TAKEN BETWEEN 0.5 AND 7.0 FEET BELOW GROUND SURFACE)								
TP-B919	0.50 - 6.0	20.2	53	33	CH	115.6	12.4	-
TP-B927	0.5 - 4.0	24.1	45	30	CL	118.4	13.6	3
TP-C304	3.0 - 7.0	21.7	51	36	CH	112.2	11.2	-
TP-C404	2.0 - 7.0	24.3	62	44	CH	116.6	13.8	3
MINIMUM, STRATUM A		20.2	45	30	Typically CH	112.2	11.2	3
MAXIMUM, STRATUM A (UPPER)		24.3	62	44		118.4	13.8	3
AVERAGE, STRATUM A		22.6	53	36		115.7	12.8	3
STRATUM A (LOWER; SAMPLES GENERALLY TAKEN BETWEEN 5.5 AND 9.0 FEET BELOW GROUND SURFACE)								
TP-B919	6.0 - 8.5	25.9	74	52	CH	109.1	18.2	-
TP-B927	5.5 - 8.5	22.0	41	26	CL	117.6	11.3	-
TP-C304	7.0 - 9.0	25.5	40	23	CL	121.8	9.5	2
TP-C404	7.0 - 9.0	28.4	77	56	CH	121.7	9.4	3
MINIMUM, STRATUM A		22.0	40	23	Typically CL, CH	109.1	9.4	2
MAXIMUM, STRATUM A (LOWER)		28.4	77	56		121.8	18.2	3
AVERAGE, STRATUM A		25.5	58	39		117.6	12.1	3

[1] Compaction (moisture-density) tests were conducted in accordance with Reference 2.5S.4-42, Method A

[2] CBR tests were conducted in accordance with Reference 2.5S.4-43, generally on soaked test specimens compacted to approximately 95% of modified Proctor maximum dry density (Reference 2.5S.4-42)

Table 2.5S.4-27 Summary of Shear Wave Velocities to 600 Feet Below Ground Surface

Stratum	Soil Type	Top El. [1] (Feet)	Bottom El. [1] (Feet)	Thickness (Feet)	Mid-Point Depth [2] (Feet)	Unit Weight (Feet)	PI (%)	Average s_u (KSF)	Maximum V_s (Ft/ sec)	Minimum V_s (Ft/ sec)	Average V_s (Ft/ sec)	Use V_s (Ft/ sec)	Average u
A	Clay	30	10	20	14	124	35	1.6	1,078	290	578	575	0.45
		30	25	5	6.5				670	330	451	450	0.43
		25	20	5	11.5				1,000	290	547	545	0.41
		20	15	5	16.5				1,078	370	601	600	0.47
		15	10	5	21.5				890	300	643	640	0.48
B	Silt	10	0	10	29	121	N/A	N/A	1,090	400	728	725	0.48
		10	5	5	26.5				1,060	400	707	705	0.48
		5	0	5	31.5				1,090	470	758	755	0.49
C	Sand	0	-20	20	44	122	N/A	N/A	1,430	440	786	785	0.49
		0	-5	5	36.5				1,430	440	756	755	0.49
		-5	-10	5	41.5				1,220	520	805	805	0.49
		-10	-15	5	46.5				1,070	520	828	825	0.49
		-15	-20	5	51.5				1,390	510	767	765	0.49
D	Clay	-20	-40	20	64	121	40	3.0	1,550	540	929	925	0.48
		-20	-25	5	56.5				1,020	540	702	700	0.49
		-25	-30	5	61.5				1,331	580	849	845	0.49
		-30	-35	5	66.5				1,370	790	1,026	1,025	0.48
		-35	-40	5	71.5				1,550	870	1,204	1,200	0.48
E	Sand	-40	-60	20	84	122	N/A	N/A	1,627	720	1,082	1,080	0.48
		-40	-45	5	76.5				1,430	940	1,196	1,195	0.48
		-45	-50	5	81.5				1,627	750	1,103	1,100	0.48
		-50	-55	5	86.5				1,250	770	1,038	1,035	0.48
		-55	-60	5	91.5				1,203	720	961	960	0.48
F	Clay	-60	-75	15	101.5	125	40	3.2	1,280	720	947	945	0.48
		-60	-65	5	96.5				1,280	720	905	905	0.49
		-65	-70	5	101.5				1,260	830	956	955	0.48
		-70	-75	5	106.5				1,270	780	990	990	0.48

Table 2.5S.4-27 Summary of Shear Wave Velocities to 600 Feet Below Ground Surface (Continued)

Stratum	Soil Type	Top El. [1] (Feet)	Bottom El. [1] (Feet)	Thickness (Feet)	Mid-Point Depth [2] (Feet)	Unit Weight (Feet)	PI (%)	Average s_u (KSF)	Maximum V_s (Ft/ sec)	Minimum V_s (Ft/ sec)	Average V_s (Ft/ sec)	Use V_s (Ft/ sec)	Average u
H	Sand	-75	-90	15	116.5	128	N/A	N/A	2,190	730	1,077	1,075	0.48
		-75	-80	5	111.5				1,890	740	1,078	1,075	0.48
		-80	-85	5	116.5				2,190	730	1,081	1,080	0.48
		-85	-90	5	121.5				1,814	750	1,071	1,070	0.48
J Clay 1	Clay	-90	-125	35	141.5	125	35	3.5	1,880	640	1,148	1,145	0.48
		-90	-95	5	126.5				1,350	760	981	980	0.48
		-95	-100	5	131.5				1,410	720	1,057	1,055	0.48
		-100	-105	5	136.5				1,470	640	1,068	1,065	0.48
		-105	-110	5	141.5				1,780	910	1,307	1,305	0.47
		-110	-115	5	146.5				1,880	1,000	1,337	1,335	0.47
		-115	-120	5	151.5				1,610	1,090	1,260	1,260	0.47
		-120	-125	5	156.5				1,720	680	1,178	1,175	0.48
J Sand	Sand/ Silt	-125	-140	15	166.5	125	N/A	N/A	3,210	720	1,275	1,275	0.47
		-125	-130	5	161.5				2,270	840	1,299	1,295	0.47
		-130	-135	5	166.5				2,560	840	1,277	1,275	0.47
		-135	-140	5	171.5				3,210	720	1,244	1,240	0.47
J Clay 2	Clay	-140	-185	45	196.5	125	35	3.5	1,690	700	1,033	1,030	0.48
		-140	-145	5	176.5				1,690	930	1,235	1,235	0.47
		-145	-150	5	181.5				1,260	960	1,036	1,035	0.48
		-150	-155	5	186.5				1,390	870	1,059	1,055	0.48
		-155	-160	5	191.5				1,360	700	1,034	1,030	0.48
		-160	-165	5	196.5				1,440	830	1,037	1,035	0.48
		-165	-170	5	201.5				1,290	800	965	965	0.48
		-170	-175	5	206.5				1,330	770	966	965	0.48
		-175	-180	5	211.5				1,180	760	943	940	0.48
		-180	-185	5	216.5				1,220	670	938	935	0.48
K Clay	Clay	-185	-203	18	228.0	129	25	3.0	1,650	730	1,170	1,170	0.48
		-185	-190	5	221.5				1,420	820	1,111	1,110	0.48
		-190	-195	5	226.5				1,560	810	1,117	1,115	0.48
		-195	-200	5	231.5				1,320	730	1,075	1,075	0.48
		-200	-203	3	235.5				1,650	1,430	1,510	1,510	0.47

Table 2.5S.4-27 Summary of Shear Wave Velocities to 600 Feet Below Ground Surface (Continued)

Stratum	Soil Type	Top El. [1] (Feet)	Bottom El. [1] (Feet)	Thickness (Feet)	Mid-Point Depth [2] (Feet)	Unit Weight (Feet)	PI (%)	Average s_u (KSF)	Maximum V_s (Ft/ sec)	Minimum V_s (Ft/ sec)	Average V_s (Ft/ sec)	Use V_s (Ft/ sec)	Average u
K Sand/ Silt	Sand/ Silt	-203	-228	25	249.5	127	N/A	N/A	2,010	940	1,371	1,370	0.47
		-203	-208	5	239.5				1,630	1,140	1,341	1,340	0.47
		-208	-213	5	244.5				2,010	1,100	1,573	1,570	0.46
		-213	-218	5	249.5				1,630	1,070	1,350	1,350	0.47
		-218	-223	5	254.5				1,490	1,230	1,346	1,345	0.47
		-223	-228	5	259.5				1,620	940	1,240	1,240	0.47
L	Clay	-228	-233	5	264.5	129	50	3.0	1,410	750	979	975	0.48
M	Sand	-233	-248	15	274.5	127	N/A	N/A	1,600	800	1,165	1,165	0.47
		-233	-238	5	269.5				1,600	1,130	1,343	1,340	0.47
		-238	-243	5	274.5				1,170	860	1,018	1,015	0.48
		-243	-248	5	279.5				1,400	800	1,110	1,110	0.48
N Clay 1	Clay	-248	-307	59	311.5	121	45	3.0	1,760	700	1,234	1,230	0.47
		-248	-253	5	284.5				1,180	700	957	955	0.48
		-253	-258	5	289.5				1,670	1,370	1,501	1,500	0.47
		-258	-263	5	294.5				1,650	1,320	1,510	1,510	0.46
		-263	-268	5	299.5				1,760	1,010	1,293	1,290	0.47
		-268	-273	5	304.5				1,100	980	1,053	1,050	0.48
		-273	-278	5	309.5				1,200	900	1,037	1,035	0.48
		-278	-283	5	314.5				1,160	830	966	965	0.48
		-283	-288	5	319.5				1,260	1,070	1,112	1,110	0.48
		-288	-293	5	324.5				1,570	1,210	1,408	1,405	0.47
		-293	-298	5	329.5				1,640	1,470	1,522	1,520	0.46
		-298	-303	5	334.5				1,640	1,110	1,362	1,360	0.47
		-303	-307	4	339.0				1,470	940	1,140	1,140	0.48
N Sand 1	Sand	-307	-324	17	349.5	128	N/A	N/A	2,430	1,390	1,646	1,645	0.46
		-307	-312	5	343.5				1,650	1,390	1,535	1,535	0.46
		-312	-317	5	348.5				2,430	1,540	1,843	1,840	0.45
		-317	-322	5	353.5				1,720	1,560	1,618	1,615	0.46
		-322	-324	2	357.0				1,650	1,470	1,550	1,550	0.46

Table 2.5S.4-27 Summary of Shear Wave Velocities to 600 Feet Below Ground Surface (Continued)

Stratum	Soil Type	Top El. [1] (Feet)	Bottom El. [1] (Feet)	Thickness (Feet)	Mid-Point Depth [2] (Feet)	Unit Weight (Feet)	PI (%)	Average s_u (KSF)	Maximum V_s (Ft/ sec)	Minimum V_s (Ft/ sec)	Average V_s (Ft/ sec)	Use V_s (Ft/ sec)	Average u
N Clay 2	Clay	-324	-332	8	362.0	121	45	3.0	2,220	870	1,537	1,535	0.46
		-324	-329	5	360.5				2,220	1,460	1,704	1,700	0.45
		-329	-332	3	364.5				1,670	870	1,328	1,325	0.47
N Sand 2	Sand	-332	-365	33	382.5	128	N/A	N/A	2,360	1,380	1,666	1,665	0.45
		-332	-337	5	368.5				1,790	1,380	1,642	1,640	0.46
		-337	-342	5	373.5				1,810	1,630	1,685	1,685	0.45
		-342	-347	5	378.5				1,690	1,610	1,649	1,645	0.46
		-347	-352	5	383.5				1,750	1,580	1,638	1,635	0.45
		-352	-357	5	388.5				1,620	1,470	1,561	1,560	0.46
		-357	-362	5	393.5				1,960	1,480	1,665	1,665	0.45
		-362	-365	3	397.5				2,360	2,020	2,190	2,190	0.43
N Clay 3	Clay	-365	-373	8	403.0	121	45	3.0	2,540	1,220	1,851	1,850	0.45
		-365	-370	5	401.5				2,540	1,220	2,053	2,050	0.43
		-370	-373	3	405.5				1,680	1,430	1,498	1,495	0.47
N Sand 3	Sand	-373	-392	19	416.5	128	N/A	N/A	2,060	1,360	1,572	1,570	0.46
		-373	-378	5	409.5				2,060	1,410	1,682	1,680	0.46
		-378	-383	5	414.5				1,710	1,460	1,577	1,575	0.46
		-383	-388	5	419.5				1,630	1,360	1,475	1,475	0.46
		-388	-392	4	424.0				1,630	1,460	1,552	1,550	0.46
N Clay 4	Clay	-392		-422	30		441.0	121	45	3.0	1,810	910	1,207
		-392		-397	5		428.5				1,810	1,330	1,537
		-397		-402	5		433.5				1,260	1,040	1,115
		-402		-407	5		438.5				1,390	1,050	1,190
		-407		-412	5		443.5				1,400	1,040	1,260
		-412		-417	5		448.5				1,380	1,000	1,167
		-417		-422	5		453.5				1,100	910	975
N Sand 4	Sand	-422		-430	8		460.0	128	N/A	N/A	1,720	870	1,359
		-422		-427	5		458.5				1,720	870	1,292
		-427		-430	3		462.5				1,580	1,370	1,460

Table 2.5S.4-27 Summary of Shear Wave Velocities to 600 Feet Below Ground Surface (Continued)

Stratum	Soil Type	Top El. [1] (Feet)	Bottom El. [1] (Feet)	Thickness (Feet)	Mid-Point Depth [2] (Feet)	Unit Weight (Feet)	PI (%)	Average s_u (KSF)	Maximum V_s (Ft/ sec)	Minimum V_s (Ft/ sec)	Average V_s (Ft/ sec)	Use V_s (Ft/ sec)	Average u
N Clay 5	Clay	-430		-484	54		491.0	121	45	3.0	1,820	970	1,223
		-430		-435	5		466.5				1,540	1,000	1,260
		-435		-440	5		471.5				1,460	970	1,184
		-440		-445	5		476.5				1,050	1,030	1,040
		-445		-450	5		481.5				1,060	1,000	1,040
		-450		-455	5		486.5				1,460	1,080	1,273
		-455		-460	5		491.5				1,280	1,110	1,167
		-460		-465	5		496.5				1,130	1,080	1,110
		-465		-470	5		501.5				1,190	1,170	1,180
		-470		-475	5		506.5				1,280	1,110	1,180
		-475		-480	5		511.5				1,420	1,190	1,330
		-478		-484	4		516.0				1,820	1,750	1,785
N Sand 5	Sand	-484		-502	18		527.0	128	N/A	N/A	2,250	1,540	1,848
		-484		-489	5		520.5				2,250	1,790	1,972
		-489		-494	5		525.5				2,080	1,720	1,910
		-494		-499	5		530.5				2,020	1,540	1,735
		-499		-502	3		534.5				1,800	1,740	1,770

Table 2.5S.4-27 Summary of Shear Wave Velocities to 600 Feet Below Ground Surface (Continued)

Stratum	Soil Type	Top El. [1] (Feet)	Bottom El. [1] (Feet)	Thickness (Feet)	Mid-Point Depth [2] (Feet)	Unit Weight (Feet)	PI (%)	Average s_u (KSF)	Maximum V_s (Ft/ sec)	Minimum V_s (Ft/ sec)	Average V_s (Ft/ sec)	Use V_s (Ft/ sec)	Average u
N Clay 6	Clay	-502		-575	73		572.5	121	45	3.0	1,880	1,120	1,347
		-502		-507	5		538.5				1,880	1,620	1,750
		-507		-512	5		543.5				1,250	1,180	1,217
		-512		-517	5		548.5				1,200	1,120	1,170
		-517		-522	5		553.5				1,270	1,140	1,190
		-522		-527	5		558.5				1,330	1,320	1,323
		-527		-532	5		563.5				1,190	1,130	1,160
		-532		-537	5		568.5				1,320	1,210	1,267
		-537		-542	5		573.5				1,230	1,220	1,227
		-542		-547	5		578.5				1,560	1,160	1,363
		-547		-552	5		583.5				1,400	1,270	1,317
		-552		-557	5		588.5				1,370	1,290	1,330
		-557		-562	5		593.5				1,620	1,470	1,523
		-562		-567	5		598.5				1,800	1,280	1,508
		-567		-572	5		603.5				1,620	1,420	1,520
		-572		-575	3		607.5				1,450	1,420	1,435

[1] Elevations are referenced to NGVD 29 datum

[2] Mid-point depth measured below El. 34 feet

Table 2.5S.4-28 Summary of Shear Wave Velocities Deeper than 600 Feet Below Ground Surface [1]

Profile	Top Depth (Feet)	Bottom Depth (Feet)	Top El. (Feet)	Bottom El. (Feet)	Mid-Point Depth [2] (Feet)	V_s (Ft/ sec)
M1P1	609	680	-575	-646	644.5	2,050
	680	780	-646	-746	730.0	2,150
	780	880	-746	-846	830.0	2,250
	880	1,300	-846	-1,266	1,090.0	2,350
	1,300	1,930	-1,266	-1,896	1,615.0	2,550
	1,930	2,500	-1,896	-2,466	2,215.0	2,850
	2,500	3,280	-2,466	-3,246	2,890.0	9,285
M1P2	609	1,000	-575	-966	804.5	1,585
	1,000	1,300	-966	-1,266	1,150.0	2,350
	1,300	1,930	-1,266	-1,896	1,615.0	2,550
	1,930	2,500	-1,896	-2,466	2,215.0	2,850
	2,500	3,280	-2,466	-3,246	2,890.0	9,285
M1P3	609	700	-575	-666	654.5	2,650
	700	780	-666	-746	740.0	2,825
	780	850	-746	-816	815.0	2,900
	850	1,000	-816	-966	925.0	3,000
	1,000	1,060	-966	-1,026	1,030.0	3,100
	1,060	1,160	-1,026	-1,126	1,110.0	3,200
	1,160	1,250	-1,126	-1,216	1,205.0	3,325
	1,250	1,700	-1,216	-1,666	1,475.0	3,575
	1,700	2,500	-1,666	-2,466	2,100.0	4,125
	2,500	3,280	-2,466	-3,246	2,890.0	9,285

[1] Shear wave velocities and depth ranges scaled from Figure B-12, "Shear Wave Velocity Profile for the South Texas Site," Reference 2.5S.4-4

[2] Mid-point depth measured below El. 34 feet

Table 2.5S.4-29 Summary of Strata Unit Weights

Depth Below Ground Surface (feet)	Stratum and/or Soil Type	Selected Unit Weight (pcf)
Ground Surface to 20	A	124
20 to 30	B	121
30 to 50	C	122
50 to 70	D	121
70 to 90	E	122
90 to 105	F	125
105 to 120	H	128
120 to 215	J Clay; J Sand	125; 125
215 to 258	K Clay; K Sand/Silt	129; 127
258 to 263	L	129 [1]
263 to 278	M	127 [1]
278 to 609	N Clay; N Sand	121; 128
609 to 680	Silt/Clay	129 [2]
680 to 780	Silty Sand	126 [2]
780 to 880	Silt/Clay	130 [2]
880 to 1,300	Silty Sand	130 [2]
1,300 to 1,930	Interbedded Sand, Clay, Silt, Claystone	130 [2]
1,930 to 2,500	Interbedded Claystone, Siltstone, Sand, Clay, Silt	135 [2]
2,500 to 3,280 +	Interbedded Claystone, Sand, Silt	140 [2]

[1] The selected unit weight for Stratum L is after Sub-stratum K Clay. The selected unit weight for Stratum M is after Sub-stratum K Sand/Silt

[2] The selected unit weights for strata deeper than approximately 600 feet below ground surface are after Reference 2.5S.4-3, Boring B-233

Table 2.5S.4-30 Summary of Strata Depths for the Selection of Shear Modulus Degradation and Damping Ratio Curves

Cohesionless Soils			
Stratum	Mid-Layer Depth (feet)	Mid-Layer Depth For Curve Selection (feet)	Selected Peninsular Curve (feet)
B (Silt)	29	30	< 50
C (Sand)	44	45	< 50
E (Sand)	84	85	> 50
H (Sand)	116.5	120	> 50
J (Sand/Silt)	166.5	170	> 50
K (Sand/Silt)	249.5	250	> 50
M (Sand)	274.5	250	> 50
N (Sand)	392, 427, 571	500	> 50

Cohesive Soils			
Stratum	Depth Range (feet)	Average PI (%)	Adjusted PI (%)
A (Clay)	< 100	35	35
D (Clay)	< 100	39	40
F (Clay)	> 100	39	60
J (Clay)	> 100	36	60
K (Clay	> 100	25	45
L (Clay)	> 100	52	70
N (Clay)	> 100	49	70

Table 2.5S.4-31 Summary of Shear Modulus Degradation Curves Numerical Values

Cohesionless Soil Strata

	Stratum (Mid-Point Depth in Feet)									
	B (30)	C (45)	E (85)	H (120)	J Sand (170)	K Sand/ Silt (250)	M (250)	N Sand (500)	Peninsular (<50)	Peninsular (>50)
Strain (%)	Value of G/ G_{max}									
1.00E+00	0.08	0.09	0.11	0.12	0.14	0.15	0.15	0.20	0.09	0.20
3.16E-01	0.17	0.20	0.23	0.26	0.27	0.32	0.32	0.40	0.22	0.40
1.00E-01	0.37	0.40	0.46	0.49	0.51	0.56	0.56	0.64	0.43	0.64
3.16E-02	0.61	0.65	0.69	0.72	0.74	0.78	0.78	0.84	0.67	0.84
1.00E-02	0.83	0.85	0.87	0.89	0.90	0.91	0.91	0.95	0.85	0.95
3.16E-03	0.96	0.97	0.98	0.98	0.98	0.99	0.99	0.99	0.97	0.99
1.00E-03	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
3.16E-04	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.00E-04	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Cohesive Soil Strata

	Stratum (Plasticity Index in %)						
	A (35)	D (40)	F (60)	J Clay (60)	K Clay (45)	L (70)	N Clay (70)
Strain (%)	VALUE OF G/ GMAX						
1.00E+00	0.09	0.11	0.22	0.22	0.13	0.30	0.30
3.16E-01	0.19	0.26	0.42	0.42	0.28	0.53	0.53
1.00E-01	0.45	0.49	0.70	0.70	0.52	0.78	0.78
3.16E-02	0.69	0.75	0.88	0.88	0.77	0.94	0.94
1.00E-02	0.88	0.92	0.98	0.98	0.93	1.00	1.00
3.16E-03	0.98	0.99	1.00	1.00	0.99	1.00	1.00
1.00E-03	1.00	1.00	1.00	1.00	1.00	1.00	1.00
3.16E-04	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.00E-04	1.00	1.00	1.00	1.00	1.00	1.00	

Table 2.5S.4-32 Summary of Damping Ratio Curves Numerical Values

Cohesionless Soil Strata

Strain (%)	Stratum (Mid-Point Depth in Feet)									
	B (30)	C (45)	E (85)	H (120)	J Sand (170)	K Sand/ Silt (250)	M (250)	N Sand (500)	Peninsular (<50)	Peninsular (>50)
	VALUE OF DAMPING (%)									
1.00E+00	24.5	23.2	22.1	21.0	20.5	19.4	19.4	16.6	22.8	16.5
3.16E-01	21.0	19.6	18.5	17.3	16.6	15.5	15.5	13.0	-	-
1.00E-01	18.5	17.2	16.0	14.8	14.0	13.0	13.0	10.5	16.5	10.3
3.16E-02	12.0	10.8	9.6	8.7	8.0	7.0	7.0	5.4	10.3	5.5
1.00E-02	6.7	6.1	5.4	4.7	4.2	3.7	3.7	2.5	5.5	2.6
3.16E-03	3.8	3.4	2.7	2.4	2.2	2.0	2.0	1.4	3.0	1.4
1.00E-03	2.3	1.8	1.6	1.4	1.6	1.0	1.0	1.0	1.6	0.9
3.16E-04	1.8	1.7	1.0	0.9	0.8	0.8	0.8	0.6	1.3	0.5
1.00E-04	1.4	1.4	1.0	0.8	0.8	0.8	0.8	0.6	1.1	0.5

Cohesive Soil Strata

Strain (%)	Stratum (Plasticity Index in %)						
	A (35)	D (40)	F (60)	J Clay (60)	K Clay (45)	L (70)	N Clay (70)
	VALUE OF DAMPING (%)						
1.00E+00	18.6	18.3	15.8	15.8	18.0	13.8	13.8
3.16E-01	17.5	16.7	13.2	13.2	16.1	11.1	11.1
1.00E-01	15.3	14.7	11.1	11.1	14.0	9.3	9.3
3.16E-02	9.8	9.4	6.5	6.5	8.7	5.4	5.4
1.00E-02	5.5	5.3	3.9	3.9	4.8	3.3	3.3
3.16E-03	3.4	3.0	2.8	2.8	2.9	2.7	2.7
1.00E-03	2.4	2.0	2.6	2.6	2.5	2.6	2.6
3.16E-04	1.7	1.8	2.4	2.4	1.9	2.6	2.6
1.00E-04	1.6	1.7	2.4	2.4	1.8	2.6	2.6

Table 2.5S.4-33 Summary of Initial RCTS Laboratory Test Results

Appendix A Tests Boring B-405DH Sample UD13 Sub-stratum N Clay 1	Resonant Column Stage $_{o}$= 87.3 psi			Torsional Shear Stage First Cycle $_{o}$ = 87.3 psi			Torsional Shear Stage Tenth Cycle $_{o}$ = 87.3 psi		
	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)
Depth = 294.7 feet (89.9 meters) Total Unit Weight = 120.3 pcf Moisture Content = 29.0% Estimated In-Situ K_0 = 0.5 Estimated ' $_{mean}$ = 87.3 psi	2.09E-04	1.00	1.55	4.31E-04	1.00	1.47	4.16E-04	1.00	1.25
	3.98E-04	1.00	1.57	8.23E-04	1.00	1.24	8.04E-04	1.00	1.33
	8.01E-04	1.00	1.74	2.00E-03	1.00	1.46	1.99E-03	1.00	1.61
	1.56E-03	1.00	1.74	3.84E-03	1.00	1.68	4.01E-03	1.00	1.54
	3.07E-03	1.00	1.77	9.85E-03	1.00	1.75	9.88E-03	1.00	1.88
	6.15E-03	1.00	1.93	2.02E-02	0.98	2.19	2.02E-02	0.98	2.23
	1.17E-02	0.99	2.12	-	-	-	-	-	-
	2.11E-02	0.98	2.46	-	-	-	-	-	-
	3.93E-02	0.94	3.06	-	-	-	-	-	-
	7.74E-02	0.87	3.85	-	-	-	-	-	-
	1.58E-01	0.75	4.89	-	-	-	-	-	-
	3.55E-01	0.59	6.12	-	-	-	-	-	-
	5.76E-01	0.50	7.18	-	-	-	-	-	-
	8.46E-01	0.43	8.36	-	-	-	-	-	-

Table 2.5S.4-33 Summary of Initial RCTS Laboratory Test Results (Continued)

Appendix B Tests Boring B-305DH Sample UD13 Stratum M	Resonant Column Stage$_o$ = 78.6 psi			Torsional Shear Stage First Cycle$_o$ = 78.6 psi			Torsional Shear StageTenth Cycle $_o$ = 78.6 psi		
	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)
Depth = 265.5 feet (80.8 meters) Total Unit Weight = 116.0 pcf Moisture Content = 19.2% Estimated In-Situ K_0 = 0.5 Estimated '$_{mean}$ = 78.6 psi	1.40E-04	1.00	0.95	2.58E-04	1.00	0.88	2.66E-04	1.00	0.78
	2.86E-04	1.00	0.95	5.01E-04	1.00	0.84	4.97E-04	1.00	0.94
	5.83E-04	1.00	0.96	9.62E-04	1.00	0.84	9.67E-04	1.00	1.04
	1.15E-03	1.00	0.97	1.91E-03	1.00	0.83	1.94E-03	1.00	0.75
	2.25E-03	0.99	1.05	3.93E-03	1.00	0.88	3.96E-03	1.00	0.98
	4.29E-03	0.98	1.12	9.46E-03	0.91	1.53	9.45E-03	0.92	1.45
	7.96E-03	0.96	1.24	2.05E-02	0.84	2.25	2.06E-02	0.85	2.07
	1.43E-02	0.93	1.47	3.79E-02	0.76	3.98	3.51E-02	0.79	2.94
	2.54E-02	0.89	1.69	-	-	-	-	-	-
	4.56E-02	0.82	2.15	-	-	-	-	-	-
	8.12E-02	0.74	3.17	-	-	-	-	-	-
	1.44E-01	0.65	4.43	-	-	-	-	-	-
	2.55E-01	0.58	6.38	-	-	-	-	-	-
	Resonant Column Stage $_o$ = 314.3 psi			**Torsional Shear Stage First Cycle$_o$ = 314.3 psi**			**Torsional Shear Stage Tenth Cycle$_o$ = 314.3 psi**		
	5.60E-05	1.00	0.77	5.60E-05	1.00	0.77	5.60E-05	1.00	0.77
	1.15E-04	1.00	0.75	1.15E-04	1.00	0.75	1.15E-04	1.00	0.75
	2.27E-04	1.00	0.75	2.27E-04	1.00	0.75	2.27E-04	1.00	0.75
	4.49E-04	1.00	0.83	4.49E-04	1.00	0.83	4.49E-04	1.00	0.83
	9.22E-04	1.00	0.90	9.22E-04	1.00	0.90	9.22E-04	1.00	0.90
	3.52E-03	0.99	0.95	3.52E-03	0.99	0.95	3.52E-03	0.99	0.95
	6.60E-03	0.98	1.01	6.60E-03	0.98	1.01	6.60E-03	0.98	1.01
	1.20E-02	0.95	1.15	1.20E-02	0.95	1.15	1.20E-02	0.95	1.15
	2.13E-02	0.92	1.30	2.13E-02	0.92	1.30	2.13E-02	0.92	1.30
	3.74E-02	0.88	1.67	3.74E-02	0.88	1.67	3.74E-02	0.88	1.67
	6.62E-02	0.81	2.01	6.62E-02	0.81	2.01	6.62E-02	0.81	2.01
	1.14E-01	0.73	2.97	1.14E-01	0.73	2.97	1.14E-01	0.73	2.97
	1.59E-01	0.68	3.82	1.59E-01	0.68	3.82	1.59E-01	0.68	3.82
	2.03E-01	0.65	4.31	2.03E-01	0.65	4.31	2.03E-01	0.65	4.31

Table 2.5S.4-33 Summary of Initial RCTS Laboratory Test Results (Continued)

Appendix C Tests Boring B-405DH Sample UD16 Sub-stratum N Clay 2	Resonant Column Stage$_o$ = 106.1 psi			Torsional Shear Stage First Cycle$_o$ = 106.1 psi			Torsional Shear Stage Tenth Cycle$_o$ = 106.1 psi		
	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)
Depth = 358.5 feet (109.3 meters) Total Unit Weight - 116.3 pcf Moisture Content = 29.5% Estimated In-Situ K_0 = 0.5 Estimated '$_{mean}$ = 106.1 psi	3.24E-04	1.00	2.45	3.24E-04	1.00	2.45	3.24E-04	1.00	2.45
	7.02E-04	1.00	2.50	7.02E-04	1.00	2.50	7.02E-04	1.00	2.50
	1.37E-03	1.00	2.61	1.37E-03	1.00	2.61	1.37E-03	1.00	2.61
	2.73E-03	1.00	2.64	2.73E-03	1.00	2.64	2.73E-03	1.00	2.64
	5.45E-03	1.00	2.74	5.45E-03	1.00	2.74	5.45E-03	1.00	2.74
	1.09E-02	0.99	2.82	1.09E-02	0.99	2.82	1.09E-02	0.99	2.82
	2.14E-02	0.99	2.88	2.14E-02	0.99	2.88	2.14E-02	0.99	2.88
	4.23E-02	0.97	2.98	4.23E-02	0.97	2.98	4.23E-02	0.97	2.98
	8.27E-02	0.93	3.09	8.27E-02	0.93	3.09	8.27E-02	0.93	3.09
	1.64E-01	0.84	3.34	1.64E-01	0.84	3.34	1.64E-01	0.84	3.34
	3.37E-01	0.71	4.34	3.37E-01	0.71	4.34	3.37E-01	0.71	4.34
	7.07E-01	0.55	6.68	7.07E-01	0.55	6.68	7.07E-01	0.55	6.68
	1.46E+00	0.40	11.88	1.46E+00	0.40	11.88	1.46E+00	0.40	11.88
	Resident Column Stage $_o$ = 424.4 psi			Torsional Shear Stage First Cycle$_o$ = 424.4 psi			Torsional Shear Stage Tenth Cycle$_o$ = 424.4 psi		
	1.84E-04	1.00	2.11	1.84E-04	1.00	2.11	1.84E-04	1.00	2.11
	3.72E-04	1.00	2.1	3.72E-04	1.00	2.1	3.72E-04	1.00	2.1
	7.63E-04	1.00	2.08	7.63E-04	1.00	2.08	7.63E-04	1.00	2.08
	1.53E-03	1.00	2.15	1.53E-03	1.00	2.15	1.53E-03	1.00	2.15
	3.06E-03	1.00	2.15	3.06E-03	1.00	2.15	3.06E-03	1.00	2.15
	6.12E-03	1.00	2.17	6.12E-03	1.00	2.17	6.12E-03	1.00	2.17
	1.22E-02	1.00	2.18	1.22E-02	1.00	2.18	1.22E-02	1.00	2.18
	2.43E-02	0.99	2.26	2.43E-02	0.99	2.26	2.43E-02	0.99	2.26
	4.77E-02	0.98	2.29	4.77E-02	0.98	2.29	4.77E-02	0.98	2.29
	9.21E-02	0.93	2.34	9.21E-02	0.93	2.34	9.21E-02	0.93	2.34
	1.75E-01	0.83	2.79	1.75E-01	0.83	2.79	1.75E-01	0.83	2.79
	3.37E-01	0.69	4.13	3.37E-01	0.69	4.13	3.37E-01	0.69	4.13

Table 2.5S.4-33 Summary of Initial RCTS Laboratory Test Results (Continued)

Appendix D Tests Boring B-405DH Sample UD19 Sub-stratum N Clay 4	Resonant Column Stage $_o$ = 129.4 psi			Torsional Shear Stage First Cycle $_o$ = 129.4 psi			Torsional Shear Stage Tenth Cycle $_o$ = 129.4 psi		
	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)
Depth = 440.5 feet (134.3 meters) Total Unit Weight = 131.7 pcf Moisture Content = 17.4% Estimated In-Situ K_0 = 0.5 Estimated '$_{mean}$= 129.4 psi	1.01E-04	1.00	4.82	1.01E-04	1.00	4.82	1.01E-04	1.00	4.82
	2.02E-04	1.00	4.96	2.02E-04	1.00	4.96	2.02E-04	1.00	4.96
	3.93E-04	1.00	5.09	3.93E-04	1.00	5.09	3.93E-04	1.00	5.09
	8.26E-04	1.00	5.09	8.26E-04	1.00	5.09	8.26E-04	1.00	5.09
	1.65E-03	1.00	5.16	1.65E-03	1.00	5.16	1.65E-03	1.00	5.16
	3.32E-03	0.98	5.28	3.32E-03	0.98	5.28	3.32E-03	0.98	5.28
	6.68E-03	0.96	5.46	6.68E-03	0.96	5.46	6.68E-03	0.96	5.46
	1.37E-02	0.91	5.62	1.37E-02	0.91	5.62	1.37E-02	0.91	5.62
	2.75E-02	0.84	6.17	2.75E-02	0.84	6.17	2.75E-02	0.84	6.17
	6.54E-02	0.70	6.98	6.54E-02	0.70	6.98	6.54E-02	0.70	6.98
	1.73E-01	0.51	8.85	1.73E-01	0.51	8.85	1.73E-01	0.51	8.85
	Resonant Column Stage$_o$ = 455.0 psi			Torsional Shear Stage First Cycle$_o$ = 455.0 psi			Torsional Shear Stage Tenth Cycle$_o$ = 455.0 psi		
	8.00E-06	1.00	4.37	3.62E-03	1.00	5.71	3.72E-03	1.00	6.20
	1.60E-05	1.00	4.41	9.92E-03	0.91	5.85	9.68E-03	0.96	6.37
	3.00E-05	1.00	4.47	-	-	-	-	-	-
	5.70E-05	1.00	4.62	-	-	-	-	-	-
	1.15E-04	1.00	4.66	-	-	-	-	-	-
	2.30E-04	1.00	4.79	-	-	-	-	-	-
	4.60E-04	0.99	4.76	-	-	-	-	-	-
	9.53E-04	0.99	4.73	-	-	-	-	-	-
	1.91E-03	0.99	4.83	-	-	-	-	-	-
	3.85E-03	0.98	4.85	-	-	-	-	-	-
	7.74E-03	0.95	5.05	-	-	-	-	-	-
	1.60E-02	0.90	5.49	-	-	-	-	-	-
	3.54E-02	0.79	5.82	-	-	-	-	-	-
	8.13E-02	0.65	7.10	-	-	-	-	-	-

Table 2.5S.4-33 Summary of Initial RCTS Laboratory Test Results (Continued)

Appendix E Tests Boring B-305DH Sample UD18 Sub-stratum N Sand 2	Resonant Column Stage$_o$ = 113.9 psi			Torsional Shear Stage First Cycle $_o$ = 113.9 psi			Torsional Shear Stage Tenth Cycle$_o$ = 113.9 psi		
	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)	Peak Shear Strain (%)	Normalized Shear Modulus (G/ G_{max})	Damping Ratio (%)
Depth = 387.5 feet (118.1 meters) Total Unit Weight = 128.8 pcf Moisture Content = 21.2% Estimated In-Situ K_0 = 0.5 Estimated '$_{mean}$= 113.9 psi	1.11E-04	1.00	0.37	1.11E-04	1.00	0.37	1.11E-04	1.00	0.37
	2.26E-04	1.00	0.37	2.26E-04	1.00	0.37	2.26E-04	1.00	0.37
	4.43E-04	1.00	0.47	4.43E-04	1.00	0.47	4.43E-04	1.00	0.47
	8.98E-04	1.00	0.47	8.98E-04	1.00	0.47	8.98E-04	1.00	0.47
	1.74E-03	0.99	0.55	1.74E-03	0.99	0.55	1.74E-03	0.99	0.55
	3.28E-03	0.98	0.63	3.28E-03	0.98	0.63	3.28E-03	0.98	0.63
	5.92E-03	0.97	0.78	5.92E-03	0.97	0.78	5.92E-03	0.97	0.78
	1.04E-02	0.94	0.86	1.04E-02	0.94	0.86	1.04E-02	0.94	0.86
	1.81E-02	0.92	0.99	1.81E-02	0.92	0.99	1.81E-02	0.92	0.99
	3.09E-02	0.86	1.25	3.09E-02	0.86	1.25	3.09E-02	0.86	1.25
	5.21E-02	0.80	1.85	5.21E-02	0.80	1.85	5.21E-02	0.80	1.85
	9.01E-02	0.70	2.90	9.01E-02	0.70	2.90	9.01E-02	0.70	2.90
	1.58E-01	0.61	4.28	1.58E-01	0.61	4.28	1.58E-01	0.61	4.28
	2.82E-01	0.50	5.30	2.82E-01	0.50	5.30	2.82E-01	0.50	5.30
	Resonant Column Stage$_o$ = 455.0 psi			**Torsional Shear Stage First Cycle$_o$ = 455.0 psi**			**Torsional Shear Stage Tenth Cycle$_o$ = 455.0 psi**		
	4.80E-05	1.00	0.22	9.10E-04	1.00	0.51	9.20E-04	0.96	0.75
	8.90E-05	1.00	0.43	1.82E-03	1.00	0.54	1.81E-03	1.00	0.31
	1.79E-04	1.00	0.39	3.58E-03	1.00	0.85	3.62E-03	1.00	0.74
	3.57E-04	1.00	0.30	-	-	-	-	-	-
	7.28E-04	1.00	0.40	-	-	-	-	-	-
	1.41E-03	0.99	0.50	-	-	-	-	-	-
	2.68E-03	0.98	0.54	-	-	-	-	-	-
	5.02E-03	0.97	0.57	-	-	-	-	-	-
	9.22E-03	0.96	0.57	-	-	-	-	-	-
	1.57E-02	0.95	0.59	-	-	-	-	-	-
	2.56E-02	0.92	0.79	-	-	-	-	-	-
	4.24E-02	0.86	0.95	-	-	-	-	-	-
	6.77E-02	0.80	1.60	-	-	-	-	-	-
	1.05E-01	0.72	2.20	-	-	-	-	-	-

Table 2.5S.4-34 Summary of Liquefaction Potential FOS Values <1.10; SPT Method

Boring	Test El. [1] (feet)	FOS	Structure	Foundation El. [2] (feet)	Stratum (Disposition)	[3]
B-305DH/DHA	1.3	0.59	Reactor Building	-51 {-61}	Stratum C (excavated)	√
B-305DH/DHA	-348.7	1.03	Reactor Building	-51 {-61}	Sub-stratum N Sand 2 (remains)	
B-311	19.4	1.06	Control Building	-41	Stratum A (excavated)	√
B-321	10.7	1.02	Turbine Building	2	Stratum B (excavated)	√
B-341	20.1	0.76	Radwaste Building	-20 {-35}	Stratum A (excavated)	√
B-341	12.1	1.08	Radwaste Building	-20 {-35}	Stratum B (excavated)	√
B-343	12.0	0.92	Radwaste Building	-20 {-35}	Stratum B (excavated)	√
B-350	12.3	1.00	Plant Stack	Determined at Detailed Design	Stratum B (determined at detailed design)	
B-405DH	-547.4	0.74	Reactor Building	-51 {-61}	Stratum N Clay (clay; unlikely to liquefy)	√
B-424	6.8	0.92	Turbine Building	2	Stratum C (excavated)	√
B-439	-19.8	0.97	Maintenance Shop	Determined at Detailed Design	Stratum D (clay; unlikely to liquefy)	√
B-912	-2.4	0.99	RSW Line (former location)	N/A	Stratum B (no structure at test location)	√
B-915	5.5	0.88	RSW Line (former location)	N/A	Stratum B (no structure at test location)	√
B-920	14.7	1.08	N/A	N/A	Stratum B (no structure at test location)	√
B-934	-4.9	1.00	N/A	N/A	Stratum B (no structure at test location)	

[1] Elevations are referenced to NGVD 29 datum

[2] Foundation Els. shown in "{ }" symbols denote the elevations of significant over-excavation at the particular structure

[3] Denotes tests having FOS<1.10, but made in strata that are excavated, in areas without structures, or in clay soils unlikely to liquefy

Table 2.5S.4-35 Summary of Liquefaction Potential FOS Values <1.10; CPT Method

CPT (Number of Test Points)	Test El. [1,2] (feet)	FOS [2]	Structure	Foundation El. [3] (feet)	Stratum (Disposition)	[4]
C-301 (9)	22.2 11.9	0.80 1.04	N/A	N/A	Stratum A (no structure at test location)	√
C-301 (1)	11.4	1.04	N/A	N/A	Stratum B (no structure at test location)	√
C-301 (2)	-18.6 -19.1	0.78 0.85	N/A	N/A	Stratum D (no structure at test location)	√
C-302 (3)	-3.0 -6.0	1.05 1.10 [5]	N/A	N/A	Stratum C (no structure at test location)	√
C-303 (1)	16.7	1.09	N/A	N/A	Stratum A (no structure at test location)	√
C-303 (2)	-16.8 -17.3	0.77 0.83	N/A	N/A	Stratum D (no structure at test location)	√
C-304 (2)	29.1 19.3	0.98 1.01	N/A	N/A	Stratum A (no structure at test location)	√
C-305S (3)	14.4 12.4	0.95 1.04	Radwaste Building	-20 {-35}	Stratum B (excavated)	√
C-306S (5)	22.5 17.1	0.78 1.09	Turbine Building	2	Stratum A (excavated)	√
C-306S (4)	14.2 12.2	0.92 1.08	Turbine Building	2	Stratum B (excavated)	√
C-306S (9)	-11.9 -34.5	0.78 0.92	Turbine Building	2	Stratum D (fine-grained; unlikely to liquefy)	√
C-307S (3)	-10.6 -11.6	0.95 1.05	Turbine Building	2	Stratum C (remains)	
C-307S (1)	-52.4	1.07	Turbine Building	2	Stratum E (remains)	
C-308 (2)	29.6 20.3	1.02 1.05	Switch Yard	Determined at Detailed Design	Stratum A (fine-grained; unlikely to liquefy)	√
C-308 (2)	-17.6 -24.5	1.01 1.03	Switch Yard	Determined at Detailed Design	Stratum D (fine-grained; unlikely to liquefy)	√
C-309 (1)	17.7	1.08	Maintenance Shop	Determined at Detailed Design	Stratum A (fine-grained; unlikely to liquefy)	√
C-310 (1)	31.1	1.08	Maintenance Shop	Determined at Detailed Design	Stratum A (fine-grained; unlikely to liquefy)	√
C-310 (6)	-10.7 -13.2	0.96 1.09	Maintenance Shop	Determined at Detailed Design	Stratum D (fine-grained; unlikely to liquefy)	√
C-310 (4)	-49.6 -51.1	1.01 1.05	Maintenance Shop	Determined at Detailed Design	Stratum E (determined at detailed design)	
C-401 (2)	19.5 14.6	0.98 1.10 [5]	N/A	N/A	Stratum B (no structure at test location)	√

Table 2.5S.4-35 Summary of Liquefaction Potential FOS Values <1.10; CPT Method (Continued)

CPT (Number of Test Points)	Test El. [1,2] (feet)	FOS [2]	Structure	Foundation El. [3] (feet)	Stratum (Disposition)	[4]
C-401 (1)	-12.5	0.94	N/A	N/A	Stratum D (no structure at test location)	√
C-402 (2)	-11.7 -14.7	1.04 1.07	N/A	N/A	Stratum D (no structure at test location)	√
C-403 (3)	-13.0 -15.9	0.95 1.06	N/A	N/A	Stratum D (no structure at test location)	√
C-404 (3)	31.1 30.2	0.83 0.94	N/A	N/A	Stratum A (no structure at test location)	√
C-404 (1)	13.4	1.06	N/A	N/A	Stratum B (no structure at test location)	√
C-406S (1)	-16.4	0.86	Turbine Building	2	Stratum D (fine-grained; unlikely to liquefy)	√
C-407S (2)	30.5 30.0	0.98 1.08	Turbine Building	2	Stratum A (excavated)	√
C-407S (1)	10.9	1.06	Turbine Building	2	Stratum B (excavated)	√
C-408 (1)	-59.6	1.10 [5]	Switch Yard	Determined at Detailed Design	Stratum H (determined at detailed design)	
C-408 (1)	-66.0	1.07	Switch Yard	Determined at Detailed Design	Stratum J Clay 1 (fine-grained; unlikely to liquefy)	√
C-409 (1)	-15.1	0.94	Maintenance Shop	Determined at Detailed Design	Stratum D (fine-grained; unlikely to liquefy)	√
C-409 (1)	-62.4	1.05	Maintenance Shop	Determined at Detailed Design	Stratum F (fine-grained; unlikely to liquefy)	√
C-410 (1)	23.8	1.07	Maintenance Shop	Determined at Detailed Design	Stratum A (fine-grained; unlikely to liquefy)	√
C-410 (3)	16.4 15.4	0.97 1.05	Maintenance Shop	Determined at Detailed Design	Stratum B (determined at detailed design)	
C-410 (1)	12.4	1.09	Maintenance Shop	Determined at Detailed Design	Stratum C (determined at detailed design)	
C-410 (6)	-10.2 -24.0	0.87 1.07	Maintenance Shop	Determined at Detailed Design	Stratum D (fine-grained; unlikely to liquefy)	√
C-410 (6)	-44.6 -51.5	0.80 1.08	Maintenance Shop	Determined at Detailed Design	Stratum F (fine-grained; unlikely to liquefy)	√
C-411 (1)	21.5	1.10 [5]	N/A	N/A	Stratum A (no structure at test location)	√

Table 2.5S.4-35 Summary of Liquefaction Potential FOS Values <1.10; CPT Method (Continued)

CPT (Number of Test Points)	Test El. [1,2] (feet)	FOS [2]	Structure	Foundation El. [3] (feet)	Stratum (Disposition)	[4]
C-411 (1)	-18.4	0.82	N/A	N/A	Stratum D (no structure at test location)	√
C-902 (1)	28.7	1.03	UHS Basin	-2	Stratum A (excavated)	√
C-904 (17)	23.9 16.1	0.43 1.04	UHS Basin	-2	Stratum A (excavated)	√
C-904 (1)	7.7	1.02	UHS Basin	-2	Stratum B (excavated)	√
C-904 (1)	-20.9	0.88	UHS Basin	-2	Stratum D (fine-grained; unlikely to liquefy)	√
C-907 (3)	28.3 27.3	0.88 1.06	RSW Line (former location)	N/A	Stratum A (no structure at test location)	√
C-907 (2)	10.1 9.6	0.86 0.96	RSW Line (former location)	N/A	Stratum B (no structure at test location)	√
C-907 (7)	-9.1 -13.6	0.69 1.08	RSW Line (former location)	N/A	Stratum C (no structure at test location)	√
C-916 (1)	11.0	1.10 [5]	Radwaste Building (future)	-20 {-35}	Stratum B (excavated)	√
C-917 (8)	10.2 6.3	0.87 1.03	N/A	N/A	Stratum B (no structure at test location)	√
C-917 (2)	-10.4 -11.9	0.92 1.01	N/A	N/A	Stratum D (no structure at test location)	√
C-918 (9)	24.6 19.7	0.73 1.06	RSW Line (former location)	N/A	Stratum A (no structure at test location)	√
C-918 (1)	-12.8	1.08	RSW Line (former location)	N/A	Stratum C (no structure at test location)	√

[1] Elevations are referenced to NGVD 29 datum
[2] Range of Test Els. And FOS values are given where multiple test points occur
[3] Foundation Els. shown in "{ }" symbols denote the elevations of significant over-excavation at the particular structure
[4] "" denotes tests having FOS<1.10, but made in strata that are excavated, in areas without structures, or in clay soils unlikely to liquefy
[5] FOS value slightly <1.10, but which rounds up to 1.10 at two decimal places

Table 2.5S.4-36 Summary of Liquefaction Potential FOS Values <1.10; Shear Wave Velocity Method

V_s Boring (Number of Test Points)	Test El. [1,2] (feet)	FOS [2]	Structure	Foundation El. [3] (feet)	Stratum (Disposition)	[4]
B-302DH (1)	7.0	0.85	Reactor Building	-51 {-61}	Stratum B (excavated)	√
B-302DH (2)	-1.2 -2.8	0.70 0.84	Reactor Building	-51 {-61}	Stratum C (excavated)	√
B-302DH (4)	-170.1 -175.1	0.93 1.06	Reactor Building	-51 {-61}	Sub-stratum J Clay 2 (clay; unlikely to liquefy)	√
B-305DH (1)	-21.1	0.74	Reactor Building	-51 {-61}	Stratum D (excavated)	√
B-305DH (4)	-175.3 -180.2	0.78 1.04	Reactor Building	-51 {-61}	Sub-stratum J Clay 2 (clay; unlikely to liquefy)	√
B-305DH (2)	-231.0 -232.7	0.95 1.07	Reactor Building	-51 {-61}	Stratum L (clay soil unlikely to liquefy)	√
B-305DH (2)	-242.5 -244.2	0.74 0.92	Reactor Building	-51 {-61}	Stratum M (remains)	√
B-305DH (5)	-250.7 -308.1	0.93 1.08	Reactor Building	-51 {-61}	Sub-stratum N Clay 1 (clay; unlikely to liquefy)	√
B-305DH (2)	-411.5 -418.0	0.81 1.06	Reactor Building	-51 {-61}	Sub-stratum N Clay 4 (clay; unlikely to liquefy)	√
B-305DH (2)	-419.7 -421.3	0.66 0.74	Reactor Building	-51 {-61}	Sub-stratum N Sand 4 (remains)	√
B-308DH (2)	23.2 16.7	0.41 0.61	Reactor Building	-51 {-61}	Stratum A (excavated)	√
B-308DH (1)	15.0	0.79	Reactor Building	-51 {-61}	Stratum B (excavated)	√
B-319DH (1)	16.9	0.65	Turbine Building	2	Stratum A (excavated)	√
B-319DH (1)	13.6	0.72	Turbine Building	2	Stratum B (excavated)	√
B-328DH (1)	-127.6	0.83	Turbine Building	2	Sub-stratum J Sand 1 (remains)	√
B-328DH (1)	-132.5	0.69	Turbine Building	2	Sub-stratum J Clay 2 (clay; unlikely to liquefy)	√
B-402DH (2)	-80.7 -82.3	1.02 1.08	Reactor Building	-51 {-61}	Stratum H (remains)	
B-405DH (1)	-16.5	0.95	Reactor Building	-51 {-61}	Stratum C (excavated)	√
B-405DH (1)	-18.1	0.95	Reactor Building	-51 {-61}	Stratum D (excavated)	√
B-405DH (1)	-101.8	0.96	Reactor Building	-51 {-61}	Sub-stratum J Clay 1 (clay; unlikely to liquefy)	√
B-405DH (1)	-155.9	0.68	Reactor Building	-51 {-61}	Sub-stratum J Clay 2 (clay; unlikely to liquefy)	√
B-405DH (6)	-190.4 -200.2	0.66 0.98	Reactor Building	-51 {-61}	Sub-stratum K Clay (clay; unlikely to liquefy)	√
B-405DH (1)	-226.5	0.65	Reactor Building	-51 {-61}	Sub-stratum K Silt (clay; unlikely to liquefy)	√

Table 2.5S.4-36 Summary of Liquefaction Potential FOS Values <1.10; Shear Wave Velocity Method (Continued)

V_s Boring (Number of Test Points)	Test El. [1,2] (feet)	FOS [2]	Structure	Foundation El. [3] (feet)	Stratum (Disposition)	[4]
B-405DH (4)	-247.8 -278.9	0.53 1.10 [5]	Reactor Building	-51 {-61}	Sub-stratum N Clay 1 (silt; unlikely to liquefy)	√
B-405DH (1)	-331.4	0.79	Reactor Building	-51 {-61}	Sub-stratum N Clay 2 (clay; unlikely to liquefy)	√
B-405DH (3)	-418.4 -421.7	0.79 0.95	Reactor Building	-51 {-61}	Sub-stratum N Clay 4 (clay; unlikely to liquefy)	√
B-405DH (3)	-424.9 -438.1	0.94 1.04	Reactor Building	-51 {-61}	Sub-stratum N Clay 5 (clay; unlikely to liquefy)	√
B-419DH (1)	5.1	0.67	Turbine Building	2	Stratum B (excavated)	√
B-419DH (4)	-159.0 -173.1	0.83 0.99	Turbine Building	2	Sub-stratum J Clay 2 (clay; unlikely to liquefy)	√
B-428DH (1)	9.6	0.96	Turbine Building	2	Stratum A (excavated)	√
B-428DH (5)	-1.9 -11.8	0.67 0.97	Turbine Building	2	Stratum C (remains)	√
B-428DH (2)	-13.4 -20.0	0.92 1.00	Turbine Building	2	Stratum D (clay; unlikely to liquefy)	√
B-428DH (2)	-100.3 -102.0	0.69 0.87	Turbine Building	2	Sub-stratum J Clay 1 (clay; unlikely to liquefy)	√
B-428DH (5)	-157.8 -169.2	0.58 0.98	Turbine Building	2	Sub-stratum J Clay 2 (clay; unlikely to liquefy)	√

[1] Elevations are referenced to NGVD 29 datum

[2] Ranges of Test Els. And FOS values are given where multiple test points are reported

[3] Foundation Els. shown in "{ }" symbols denote the elevations of significant over-excavation at the particular structure

[4] "" denotes tests having FOS<1.10, but made in strata that are excavated, in areas without structures, or in clay soils unlikely to liquefy

[5] FOS value slightly <1.10, but which rounds up to 1.10 at two decimal places

Table 2.5S.4-37A Subsurface Conditions for the STP 3 & 4 Reactor Buildings

Soil Properties									
	Simplified Subsurface Conditions					Selection for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Elastic Modulus, E_s (ksf)
3	0.0	85.0	34.0	-51.0	(Underside of Foundation)	-	-	-
	85.0	95.0	-51.0	-61.0	Concrete Fill	-	-	-
	95.0	121.0	-61.0	-87.0	F (Clay) or H (Sand)	H (Sand)	128	1,150
	121.0	216.0	-87.0	-182.0	J (Clay)	J (Clay)	125	3,500
	216.0	232.0	-182.0	-198.0	K (Clay)	K (Clay)	129	3,100
	232.0	262.0	-198.0	-228.0	K (Sand/Silt)	K (Sand)	127	1,650
	262.0	267.0	-228.0	-233.0	L (Clay)	L (Clay)	129	3,100
	267.0	282.0	-233.0	-248.0	M (Sand)	M (Sand)	127	1,300
	282.0	603.5	-248.0	-569.5	N (Clay) or N (Sand)	N (Clay/Sand)	123	3,802
4	0.0	85.0	34.0	-51.0	(Underside of Foundation)	-	-	-
	85.0	95.0	-51.0	-61.0	Concrete Fill	-	-	-
	95.0	111.0	-61.0	-77.0	F (Clay)	F (Clay)	125	2,600
	111.0	121.0	-77.0	-87.0	H (Sand)	H (Sand)	128	1,150
	121.0	151.0	-87.0	-117.0	J (Clay)	J (Clay)	125	3,500
	151.0	181.0	-117.0	-147.0	J (Sand)	J (Sand)	125	1,500
	181.0	216.0	-147.0	-182.0	J (Clay)	J (Clay)	125	3,500
	216.0	241.0	-182.0	-207.0	K (Clay)	K (Clay)	129	3,100
	241.0	262.0	-207.0	-228.0	K (Sand/Silt)	K (Sand/Silt)	127	1,650
	262.0	267.0	-228.0	-233.0	L (Clay)	L (Clay)	129	3,100
	267.0	282.0	-233.0	-248.0	M (Sand)	M (Sand)	127	1,300
	282.0	603.5	-248.0	-569.5	N (Clay) or N (Sand)	N (Clay/Sand)	123	3,802

Table 2.5S.4-37B Subsurface Conditions for the STP 3 & 4 Reactor Buildings

Average Properties within the Foundation Deformation Zone										
					Shear Strength					
					Layer		Average			
STP	Soil Layer Selection Preference	Soil Layer	Thickness (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	Clay	F (Clay)	26	-61	3.2	0	3.4	0.0	196	98
		J (Clay)	72	-87	3.5	0				
	Sand	H (Sand)	26	-61	0	35	2.7	9.0	196	115
		J (Clay)	89	-87	3.5	0				
4	-	F(Clay)	16	-61	3.2	0	2.3	12.2	196	122
		H (Sand)	10	-77	0	35				
		J (Clay)	30	-87	3.5	0				
		J (Sand)	30	-117	0	33				
		J (Clay)	36	-147	3.5	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-38A Subsurface Conditions for the STP 3 & 4 Control Buildings

Soil Properties								
	Simplified Subsurface Conditions					Selection for Settlement Calculation		
	Depth (feet)		Elevation [1] (feet)					
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Elastic Modulus, E_s (ksf)
3	0.0	75.0	34.0	-41.0	(Underside of Foundation)	-	-	-
	75.0	105.0	-41.0	-71.0	E (Sand) or Structural Fill	E (Sand)	122	1,100
	105.0	125.0	-71.0	-91.0	H (Sand)	H (Sand)	128	1,150
	125.0	216.0	-91.0	-182.0	J (Clay)	J (Clay)	125	3,500
	216.0	232.0	-182.0	-198.0	K (Clay)	K (Clay)	129	3,100
	232.0	262.0	-198.0	-228.0	K (Sand/Silt)	K (Sand/Silt)	127	1,650
	262.0	267.0	-228.0	-233.0	L (Clay)	L (Clay)	129	3,100
	267.0	282.0	-233.0	-248.0	M (Sand)	M (Sand)	127	1,300
	282.0	603.5	-248.0	-569.5	N (Clay) or N (Sand)	N (Clay/Sand)	123	3,802
4	0.0	75.0	34.0	-41.0	(Underside of Foundation)	-	-	-
	75.0	86.0	-41.0	-52.0	E (Sand) or Structural Fill	E (Sand)	122	1,100
	86.0	106.0	-52.0	-72.0	F (Clay)	F (Clay)	125	2,600
	106.0	121.0	-72.0	-87.0	H (Sand)	H (Sand)	128	1,150
	121.0	146.0	-87.0	-112.0	J (Clay)	J (Clay)	125	3,500
	146.0	161.0	-112.0	-127.0	J (Sand)	J (Sand)	125	1,500
	161.0	216.0	-127.0	-182.0	J (Clay)	J (Clay)	125	3,500
	216.0	241.0	-182.0	-207.0	K (Clay)	K (Clay)	129	3,100
	241.0	262.0	-207.0	-228.0	K (Sand/Silt)	K (Sand/Silt)	127	1,650
	262.0	267.0	-228.0	-233.0	L (Clay)	L (Clay)	129	3,100
	267.0	282.0	-233.0	-248.0	M (Sand)	M (Sand)	127	1,300
	282.0	603.5	-248.0	-569.5	N (Clay) or N (Sand)	N (Clay/Sand)	123	3,802

Table 2.5S.4-38B Subsurface Conditions for the STP 3 & 4 Control Buildings

Average Properties within the Foundation Deformation Zone										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	-	E (Sand)	30	-41	0	35	0.8	28.3	79	66
		H(Sand)	20	-71	0	35				
		J (Clay)	15	-91	3.5	0				
4	-	E (Sand)	11	-41	0	35	1.7	18.3	79	55
		F(Clay)	20	-52	3.2	0				
		H(Sand)	15	-72	0	35				
		J (Clay)	9	-87	3.5	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39A Subsurface Conditions for the STP 3 & 4 Radwaste Buildings

Soil Properties								
	Simplified Subsurface Conditions					Selection for Settlement Calculation		
	Depth (feet)		Elevation [1] (feet)					
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Elastic Modulus, E_s (ksf)
3	0.0	54.0	34.0	-20.0	(Underside of Foundation)	-	-	-
	54.0	69.0	-20.0	-35.0	Structural Fill	Structural Fill	134	3,000
	69.0	89.0	-35.0	-55.0	E (Sand)	E (Sand)	122	1,100
	89.0	104.0	-55.0	-70.0	F (Clay)	F (Clay)	125	2,600
	104.0	119.0	-70.0	-85.0	H (Sand)	H (Sand)	128	1,150
	119.0	154.0	-85.0	-120.0	J (Clay)	J (Clay)	125	3,500
	154.0	216.0	-120.0	-182.0	J (Sand) or J (Clay)	J (Sand)	125	1,500
	216.0	232.0	-182.0	-198.0	K (Clay)	K (Clay)	129	3,100
	232.0	262.0	-198.0	-228.0	K (Sand/Silt)	K (Sand/Silt)	127	1,650
	262.0	267.0	-228.0	-233.0	L (Clay)	L (Clay)	129	3,100
	267.0	282.0	-233.0	-248.0	M (Sand)	M (Sand)	127	1,300
	282.0	603.5	-248.0	-569.5	N (Clay) or N (Sand)	N (Clay/Sand)	123	3,802
4	0.0	54.0	34.0	-20.0	(Underside of Foundation)	-	-	-
	54.0	69.0	-20.0	-35.0	Structural Fill	Structural Fill	134	3,000
	69.0	84.0	-35.0	-50.0	E (Sand), or Structural Fill	E (Sand)	122	1,100
	84.0	109.0	-50.0	-75.0	F (Clay)	F (Clay)	125	2,600
	109.0	119.0	-75.0	-85.0	H (Sand)	H (Sand)	128	1,150
	119.0	149.0	-85.0	-115.0	J (Clay)	J (Clay)	125	3,500
	149.0	184.0	-115.0	-150.0	J (Sand) or J (Clay)	J (Sand)	125	1,500
	184.0	216.0	-150.0	-182.0	J (Clay)	J (Clay)	125	3,500
	216.0	241.0	-182.0	-207.0	K (Clay)	K (Clay)	129	3,100
	241.0	262.0	-207.0	-228.0	K (Sand/Silt)	K (Sand/Silt)	127	1,650
	262.0	267.0	-228.0	-233.0	L (Clay)	L (Clay)	129	3,100
	267.0	282.0	-233.0	-248.0	M (Sand)	M (Sand)	127	1,300
	282.0	603.5	-248.0	-569.5	N (Clay) or N (Sand)	N (Clay/Sand)	123	3,802

Table 2.5S.4-39B Subsurface Conditions for the STP 3 & 4 Radwaste Buildings

Average Properties within the Foundation Deformation Zone										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	-	Structural Fill	15	-20	0	43	1.6	22.5	124	93
		E (Sand)	20	-35	0	35				
		F (Clay)	15	-55	3.2	0				
		H (Sand)	15	-70	0	35				
		J (Clay)	28	-85	3.5	0				
4	-	Structural Fill	15	-20	0	43	1.8	19.7	124	88
		E (Sand)	15	-35	0	35				
		F (Clay)	25	-50	3.2	0				
		H (Sand)	10	-75	0	35				
		J (Clay)	23	-85	3.5	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40A Subsurface Conditions for the UHS Basin/RSW Pump Houses

Soil Properties							
Simplified Subsurface Conditions					Selection for Settlement Calculation		
Depth (feet)		Elevation [1] (feet)					
Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Elastic Modulus, E_s (ksf)
0	32	32.0	-2.0	(Underside of Foundation)	-	-	-
32	50	-2.0	-20.0	B (Silt), C (Sand), or Structural Fill	B (Silt)	121	725
50	66	-20.0	-36.0	D (Clay)	D (Clay)	121	2,500
66	101	-36.0	-71.0	F (Clay)	F (Clay)	125	2,600
101	212	-71.0	-182.0	H (Sand), J (Clay), or J (Sand)	H (Sand)	128	1,150
212	217	-182.0	-207.0	K (Clay)	K (Clay)	129	3,100
217	258	-207.0	-228.0	K (Sand/Silt)	K (Sand/Silt)	127	1,650
258	264	-228.0	-233.0	L (Clay)	L (Clay)	129	3,100
263	278	-233.0	-248.0	M (Sand)	M (Sand)	127	1,300

Table 2.5S.4-40B Subsurface Conditions for the UHS Basin/RSW Pump Houses

Average Properties within the Foundation Deformation Zone									
				Shear Strength					
				Layer		Average			
Soil Selection Preference	Soil Layer	Thickness (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
Clay	B (Silt)	20	0	0	30	3.1	3.3	380	201
	D (Clay)	16	-20	3	0				
	F (Clay)	35	-36	3.2	0				
	J (Clay)	130	-71	3.5	0				
Sand	B (Silt)	20	0	0	30	0.5	30.3	380	331
	D (Clay)	16	-20	3	0				
	F (Clay)	35	-36	3.2	0				
	H (Sand)	260	-71	0	35				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-41 Bearing Capacity Derivation

Table 2.5S.4-41A **Summary of Average Parameters**

Structure	Embedment, D (feet)	Length, L (feet)	Width, B (feet)	B/L	STP	Soil Selection	c (ksf)	ϕ (°)	N_c	N_q	N_γ	ζ_c	ζ_q	z_g
Reactor Buildings	95 (10 feet concrete fill)	196	186	0.95	3	Clay	3.4	0.0	5.14	1	0	1.18	1.00	0.62
						Sand	2.7	9.0	7.53	2.06	0.86	1.26	1.15	0.62
					4	-	2.3	12.2	9.28	2.97	1.69	1.30	1.21	0.62
Control Buildings	75	184	79	0.43	3	-	0.8	28.3	25.8	14.72	16.72	1.24	1.23	0.83
					4	-	1.7	18.3	13.1	5.26	4.07	1.17	1.14	0.83
Radwaste Buildings	54	217	127	0.59	3	-	1.6	22.5	16.88	7.82	7.13	1.27	1.24	0.77
					4	-	1.8	19.7	13.93	5.8	4.68	1.24	1.21	0.77
UHS Basin	32	380	380	1.00	3 & 4	Clay	3.1	3.3	5.9	1.31	0.24	1.22	1.06	0.60
						Sand	0.5	30.3	30.14	18.4	22.4	1.61	1.58	0.60

Table 2.5S.4-41B Bearing Capacity of Foundations

Structure	STP	Soil Selection	Overburden Pressure, q (ksf)	Ultimate Bearing Capacity, q_{ult} (ksf)	Factor of Safety, FOS	Allowable Bearing Capacity, q_a (ksf)	Effective Foundation Pressure (ksf)
Reactor Buildings	3	Clay	6.0	26.8	Slightly < 3	8.9	9.2
		Sand	6.0	42.8	3	14.3	
	4	-	6.0	55.2	3	18.4	
Control Buildings	3	-	4.9	144.9	3	48.3	10.8
	4	-	4.9	63.6	3	21.2	
Radwaste Buildings	3	-	3.6	88.6	3	29.8	1.2
	4	-	3.6	69.9	3	23.5	
UHS Basin	3 & 4	Clay	2.5	24.9	3	8.3	4.3
		Sand	2.5	172.9	3	57.6	

Table 2.5S.4-42 Estimated Foundation Settlements

Structure	STP	Elastic Settlement (inches)			Virgin Compression (inches)	Total Settlement (inches)		
		Center	Edge	Average		Center	Edge	Average
Reactor Buildings	3	8.2	4.9	6.6	N/A	8.2	4.9	6.6
	4	8.3	5.0	6.6	N/A	8.3	5.0	6.6
Control Buildings	3	9.0	5.8	7.4	N/A	9.0	5.8	7.4
	4	7.5	5.0	6.3	N/A	7.5	5.0	6.3
Radwaste Buildings	3	1.1	0.7	0.9	N/A	1.1	0.7	0.9
	4	1.0	0.6	0.8	N/A	1.0	0.6	0.8
UHS Basin/RSW Pump Houses	3 & 4	7.9	4.2	6.0	0.02	7.9	4.2	6.0
RSW Lines	3 & 4	N/A	N/A	N/A	N/A	N/A	N/A	N/A

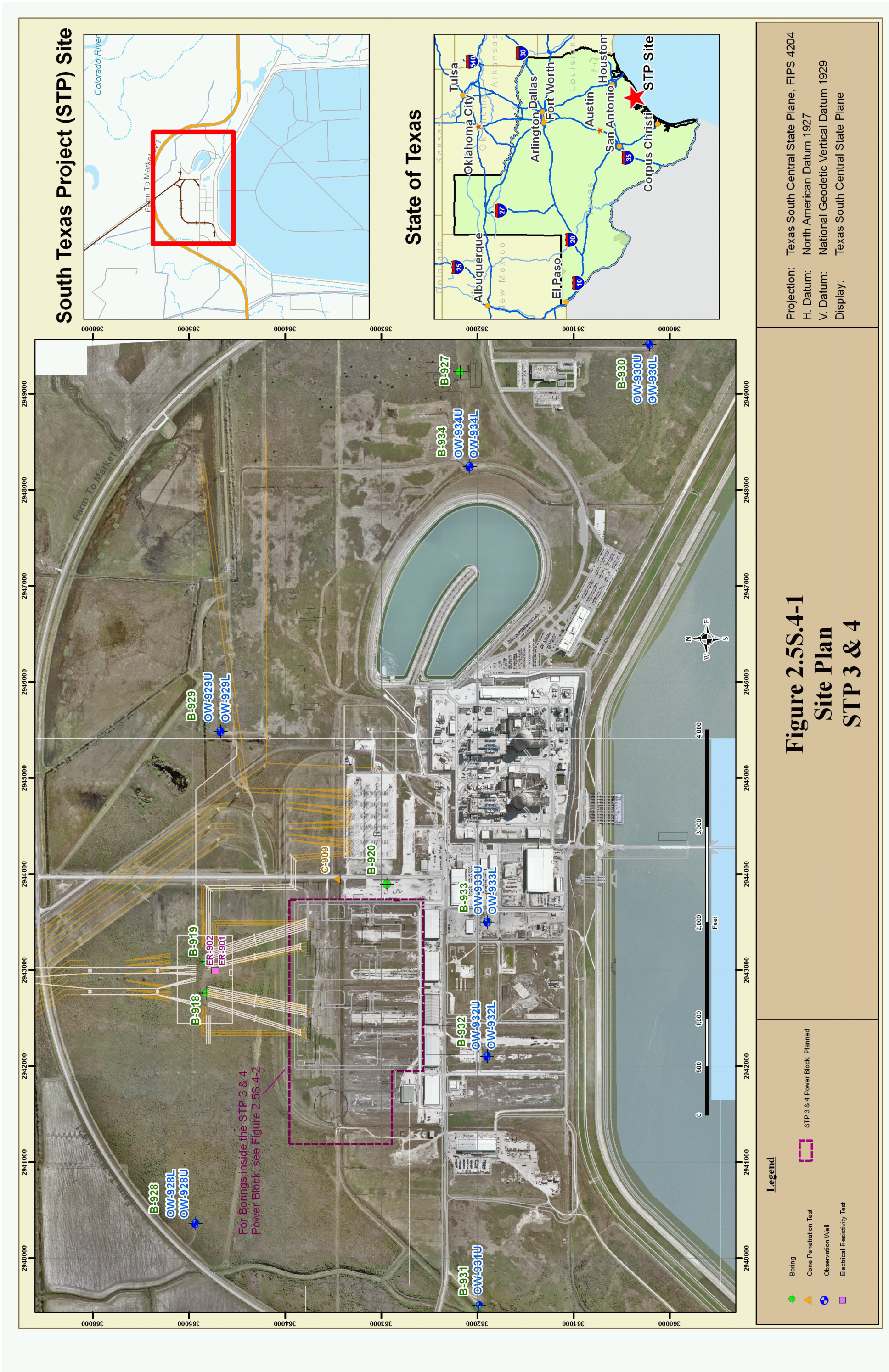


Figure 2.5S.4-1 Site Plan STP 3 & 4

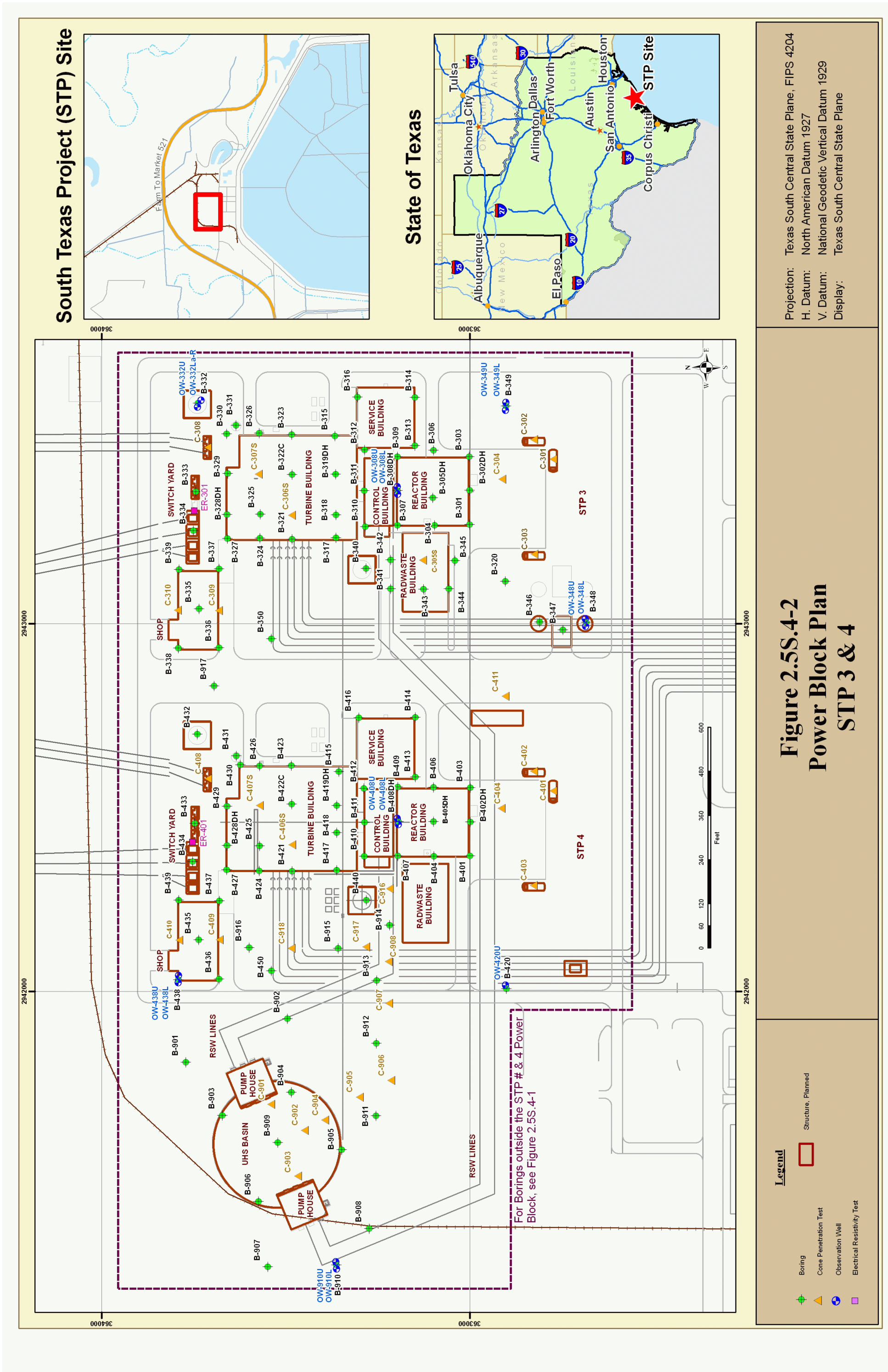


Figure 2.5S.4-2 Power Block Plan STP 3 & 4

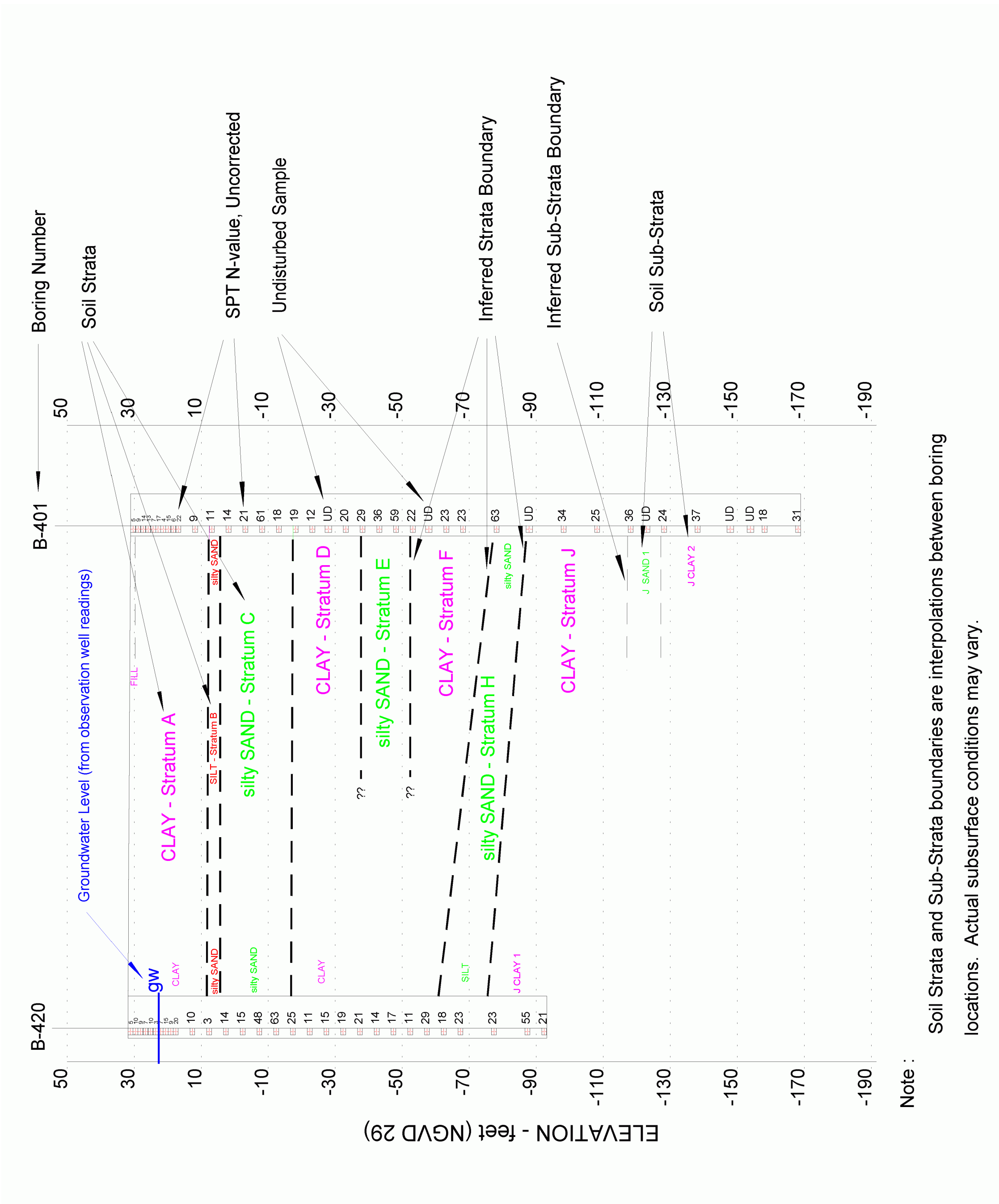


Figure 2.5S.4-3 Subsurface Profile Legend

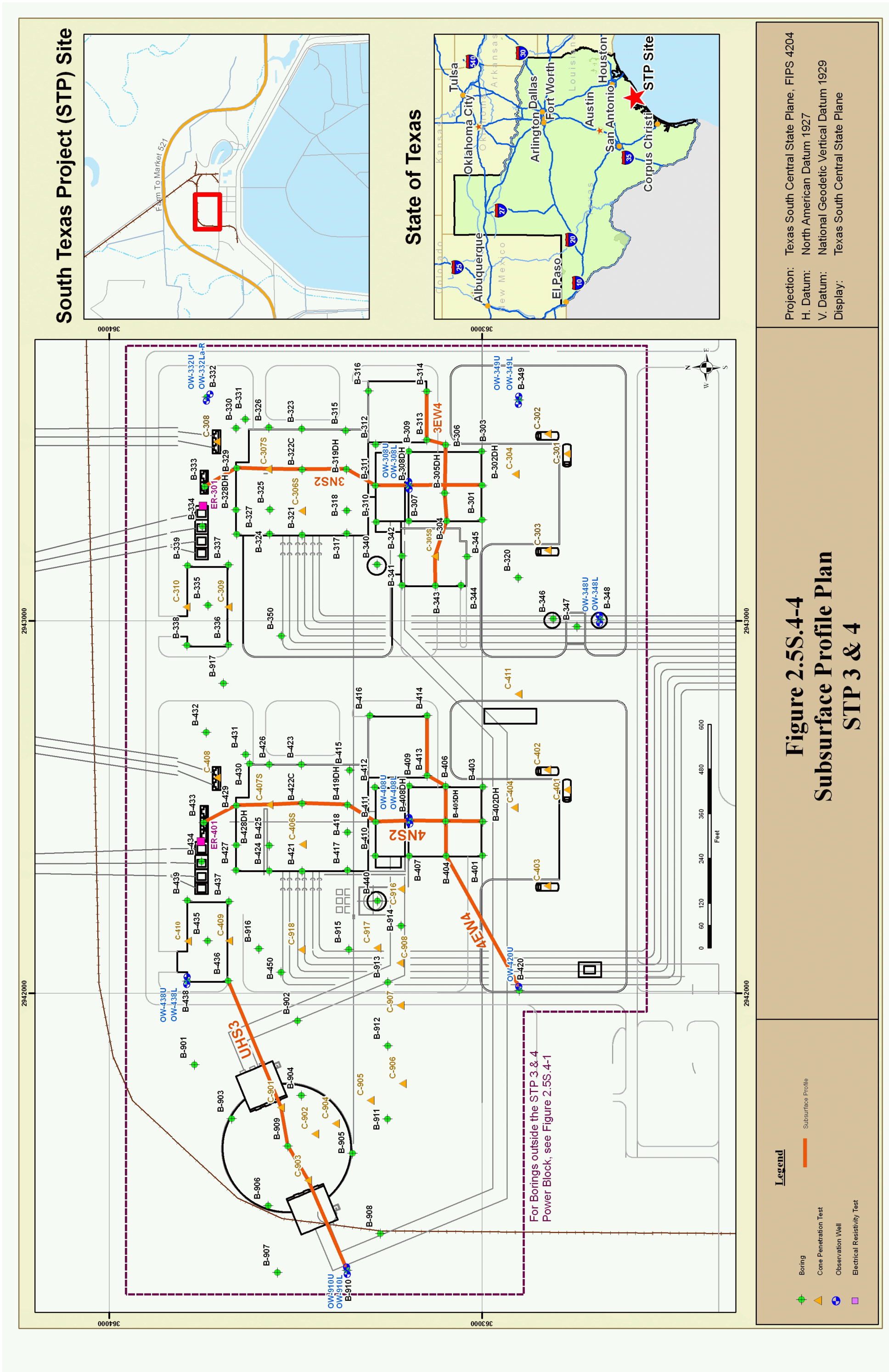


Figure 2.5S.4-4 Subsurface Profile Plan

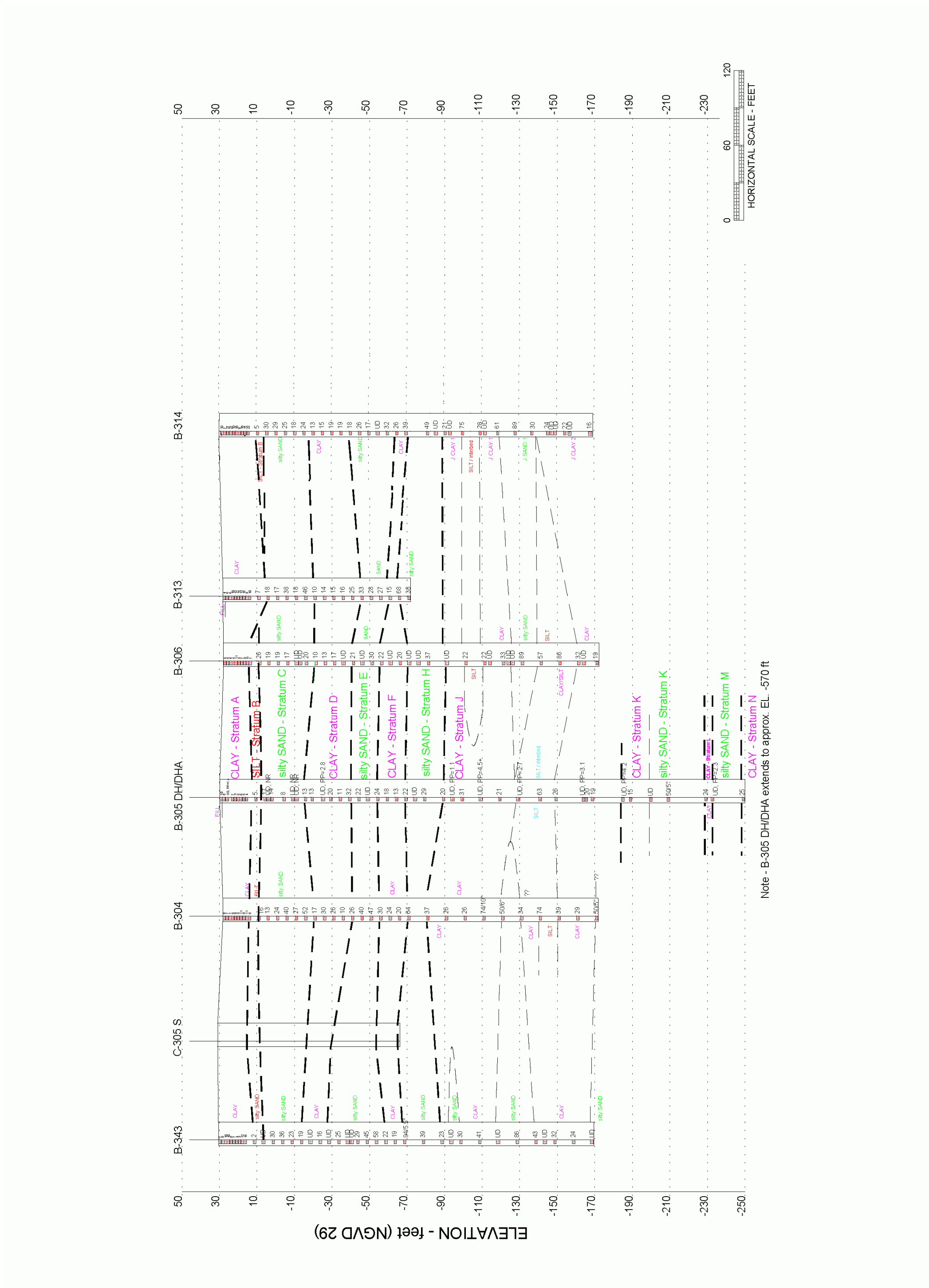


Figure 2.5S.4-5 Subsurface Profile 3EW4

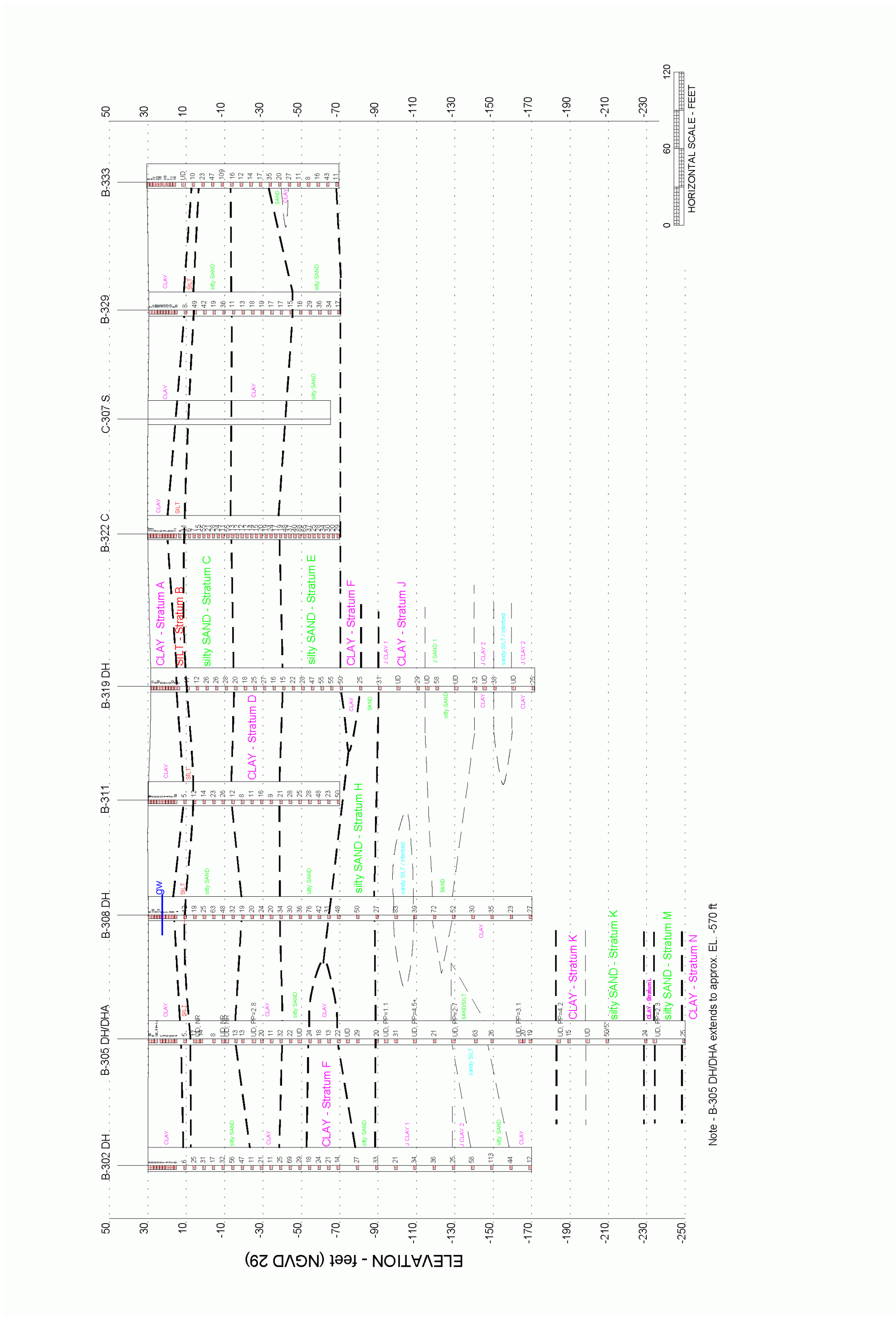


Figure 2.5S.4-6 Subsurface Profile 3NS2

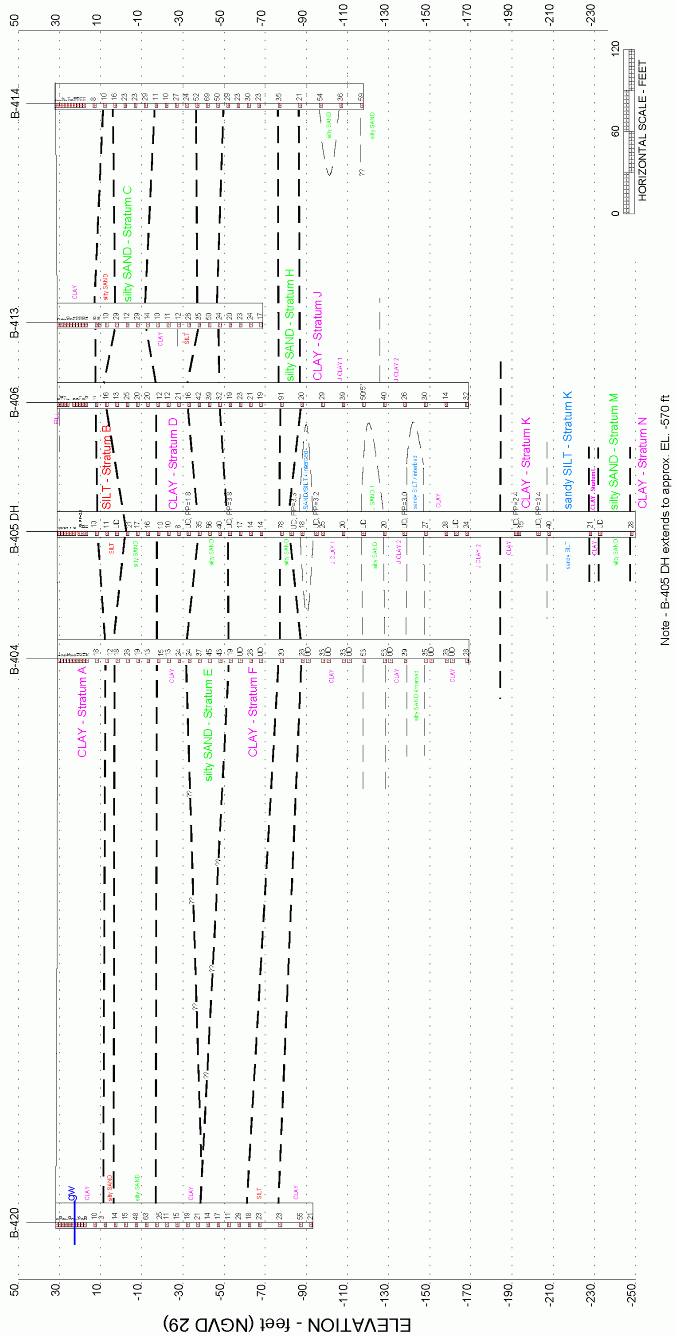


Figure 2.5S.4-7 Subsurface Profile 4EW4

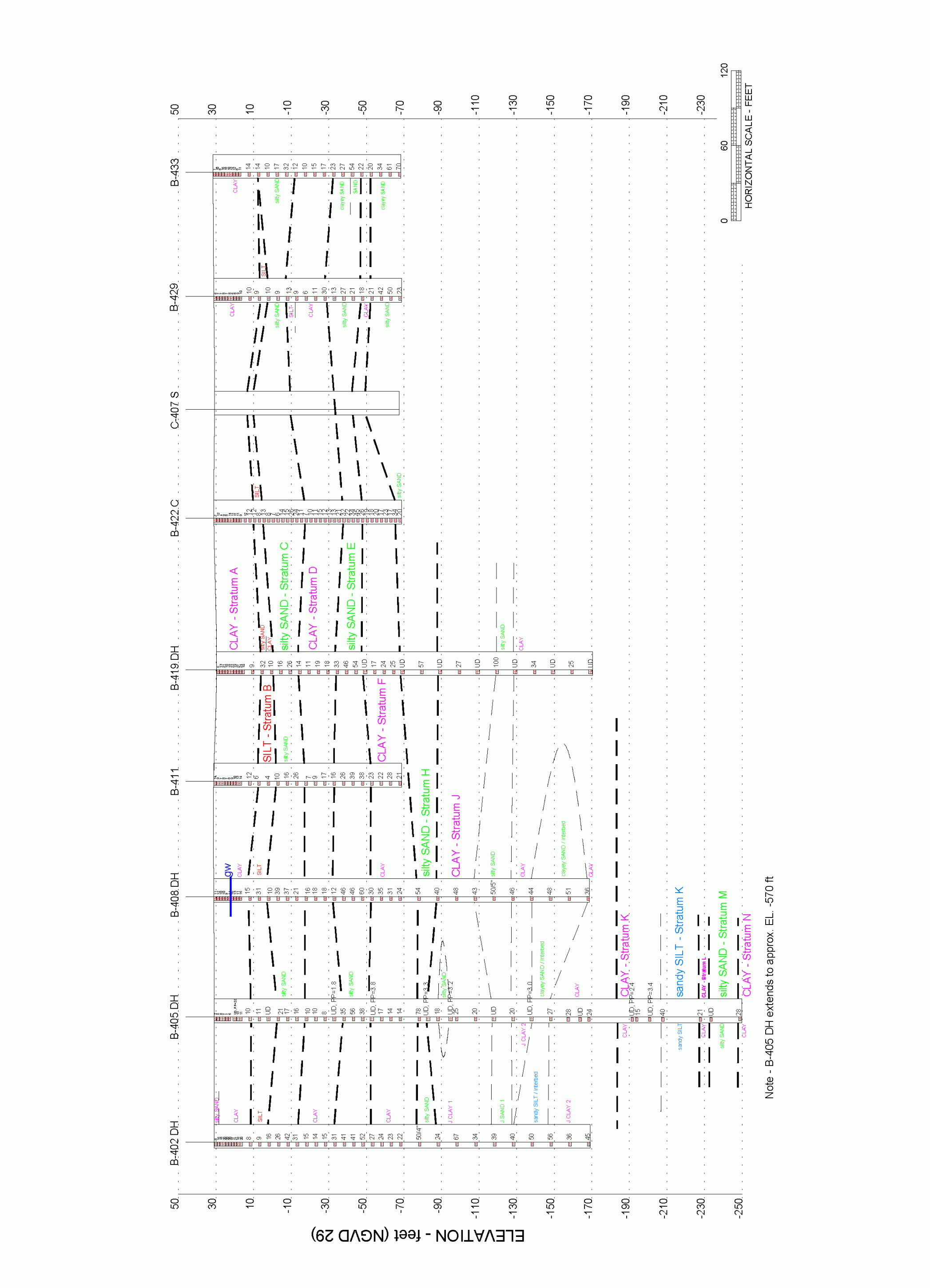


Figure 2.5S.4-8 Subsurface Profile 4NS2

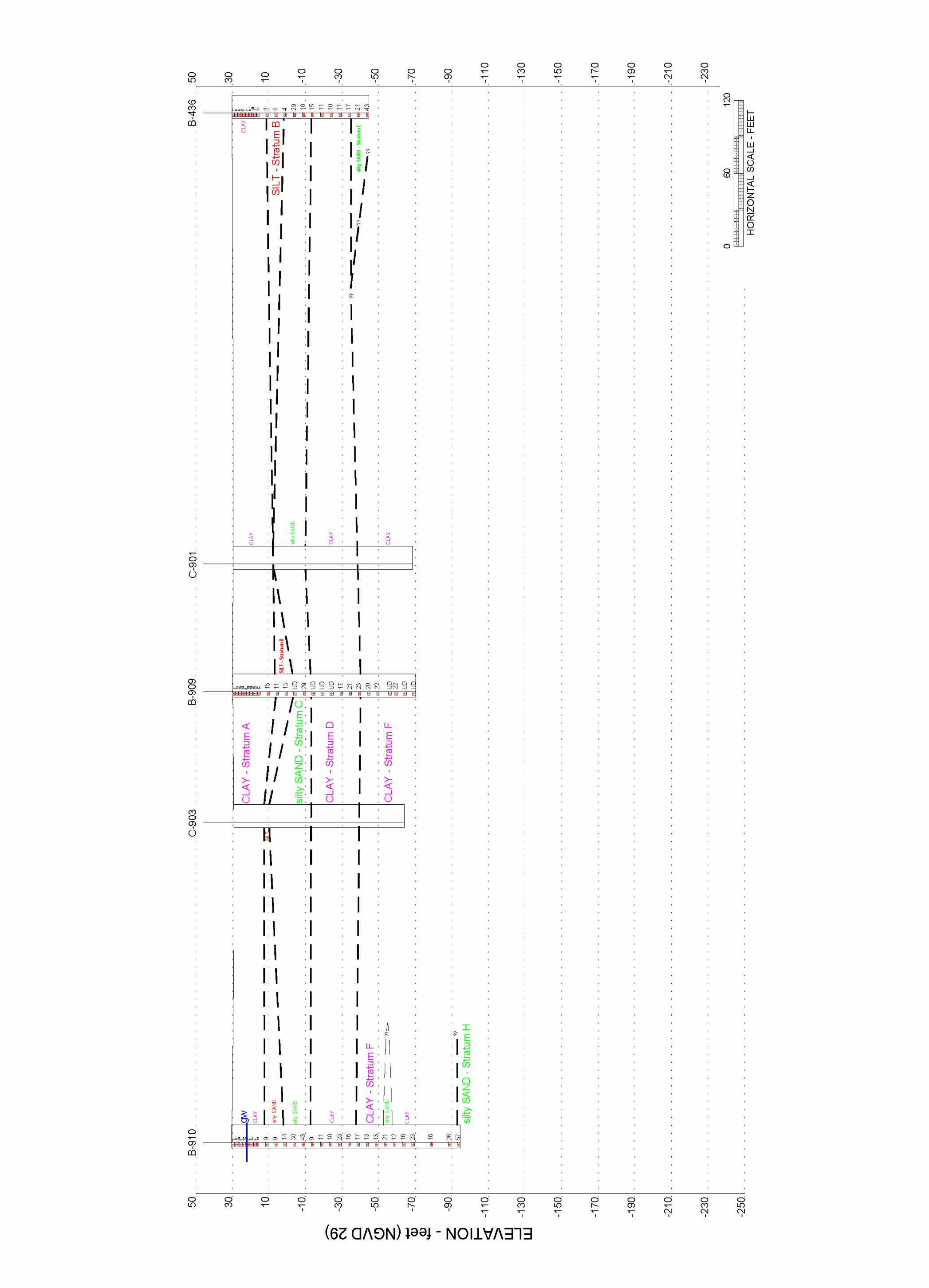


Figure 2.5S.4-9 Subsurface Profile UHS3

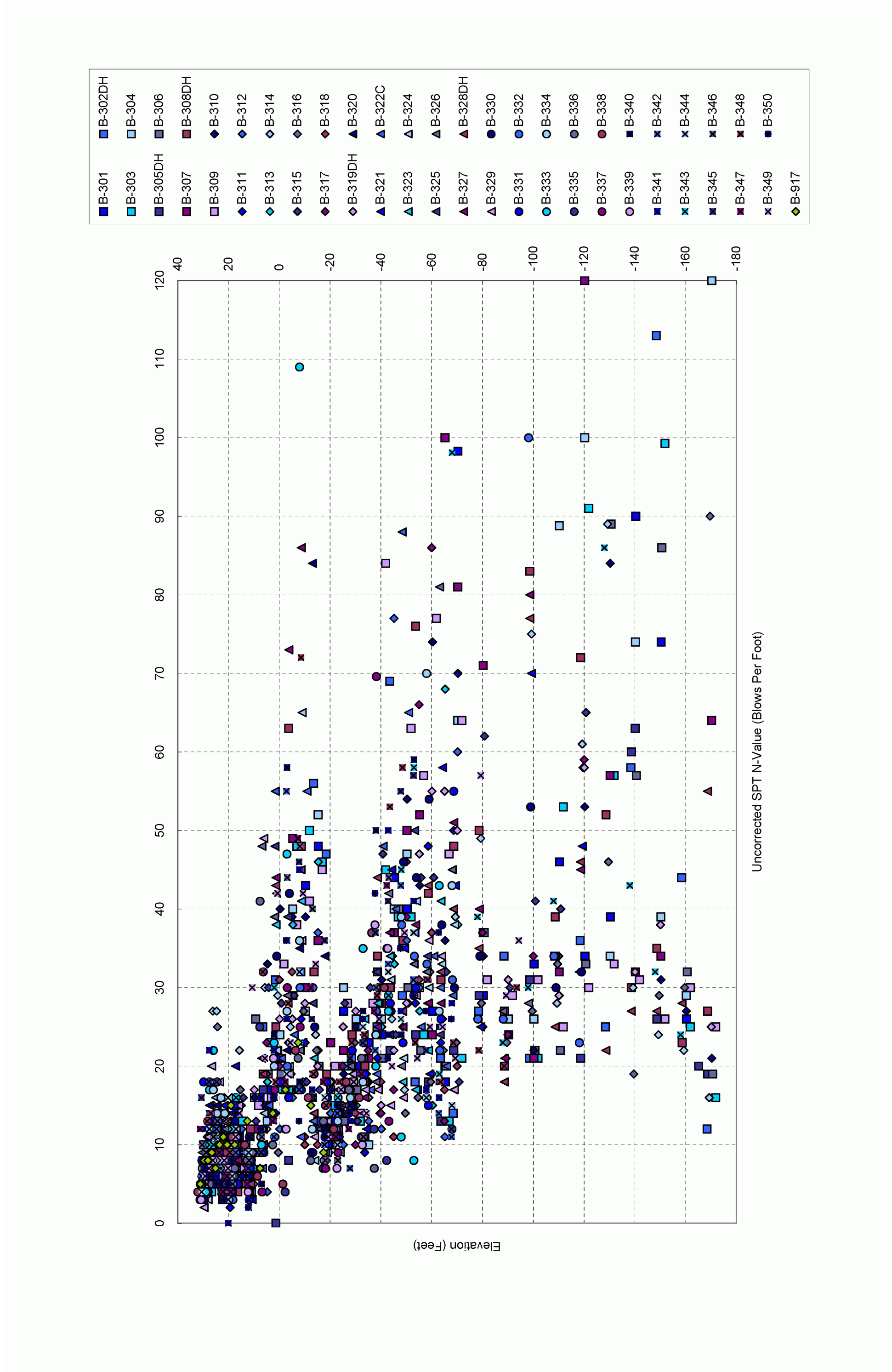


Figure 2.5S.4-10 Uncorrected SPT N-Values (STP 3) <Includes B-917>

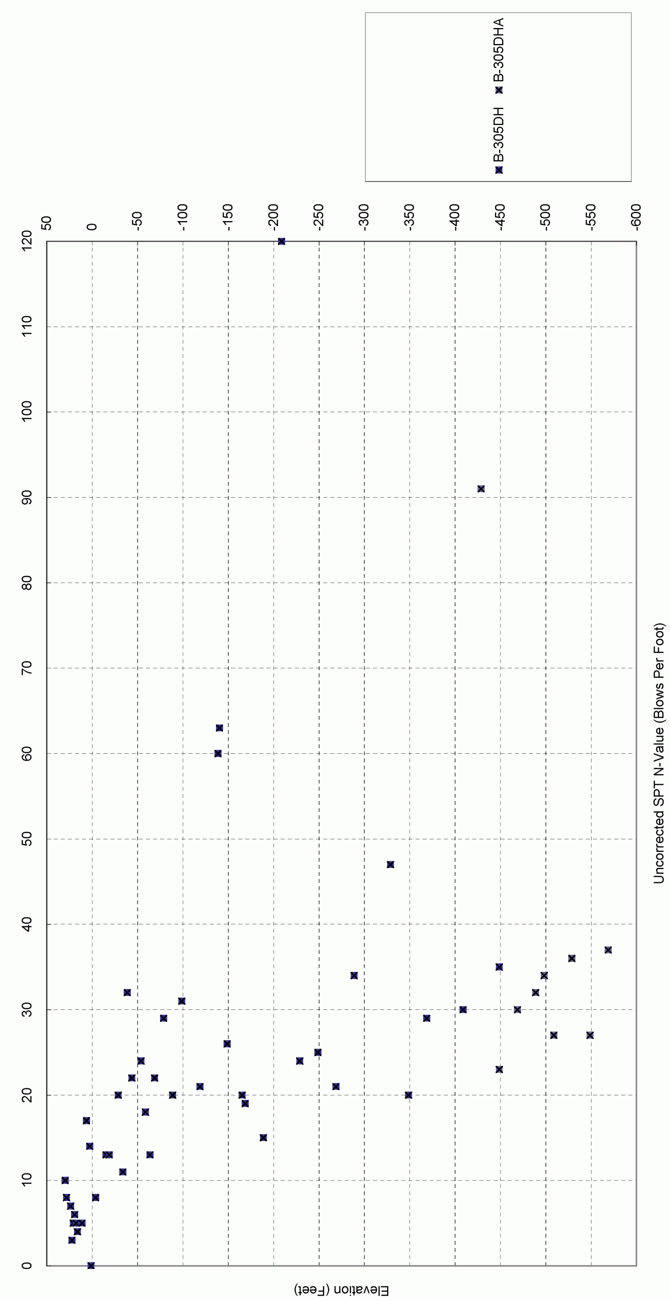


Figure 2.5S.4-11 Uncorrected SPT N-Values (STP 3; Boring B-305DH/DHA)

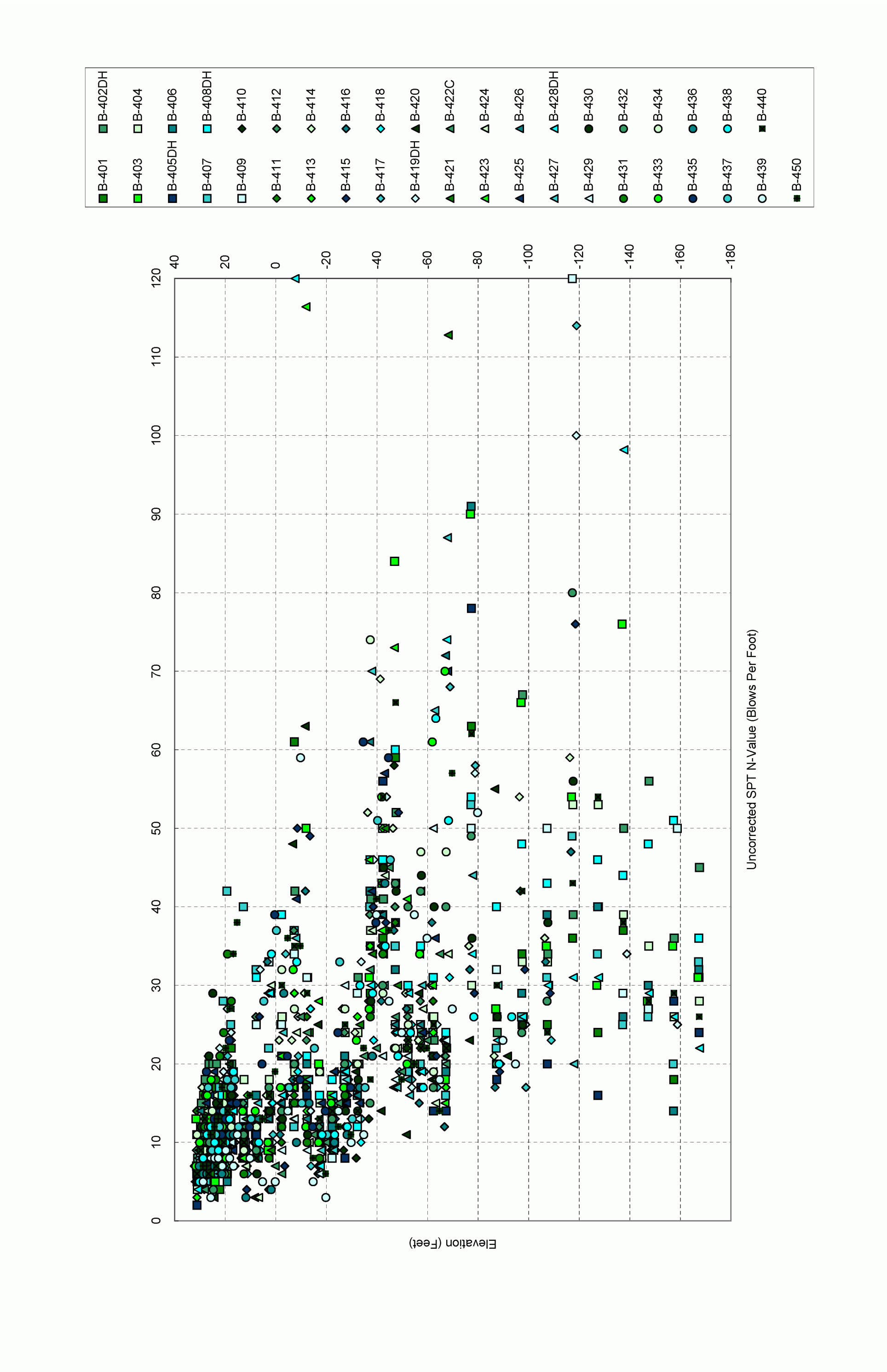


Figure 2.5S.4-12 Uncorrected SPT N-Values (STP 4)

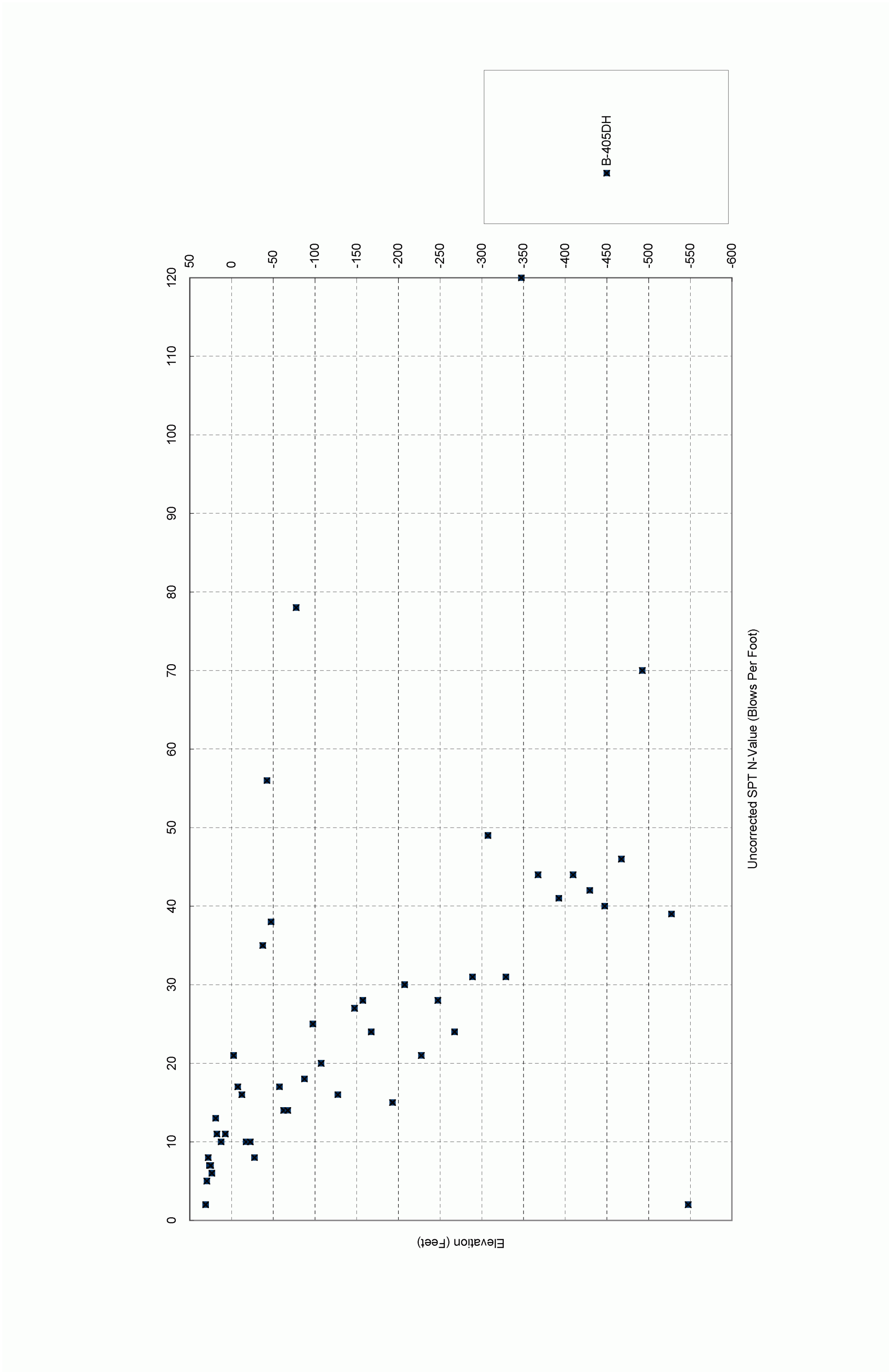


Figure 2.5S.4-13 Uncorrected SPT N-Values (STP 4; Boring B-405DH)

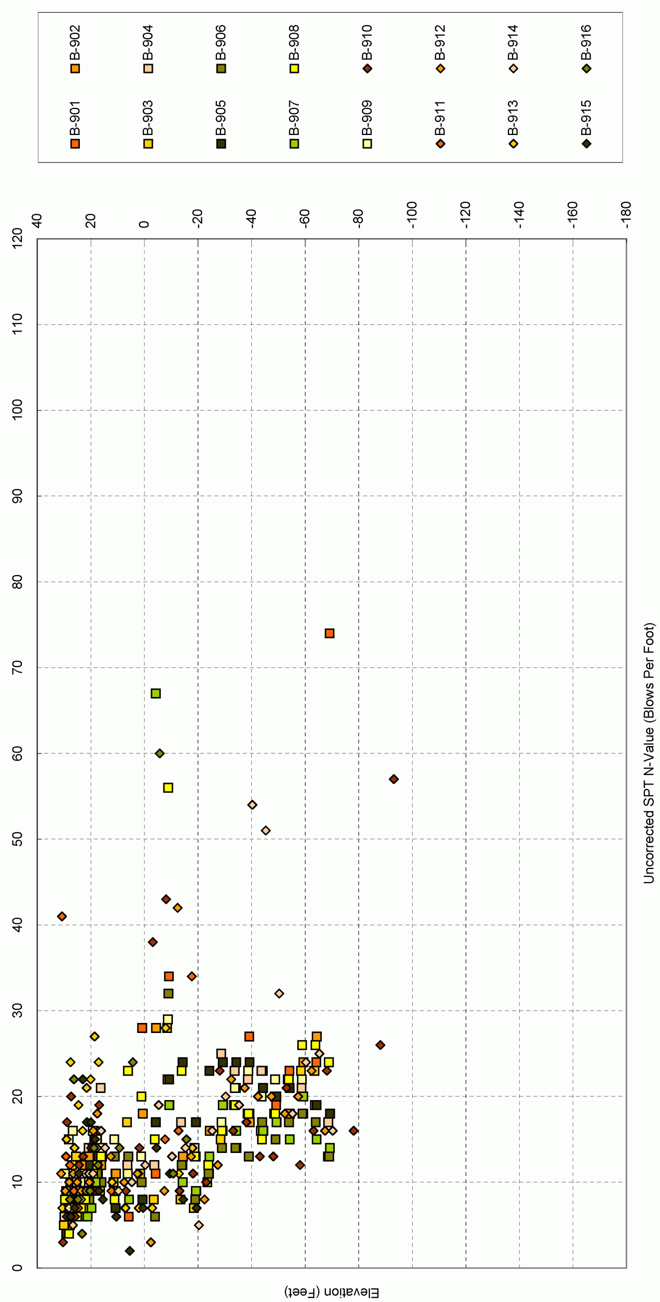


Figure 2.5S.4-14 Uncorrected SPT N-Values (UHS BASIN/RSW) <Excludes B-917>

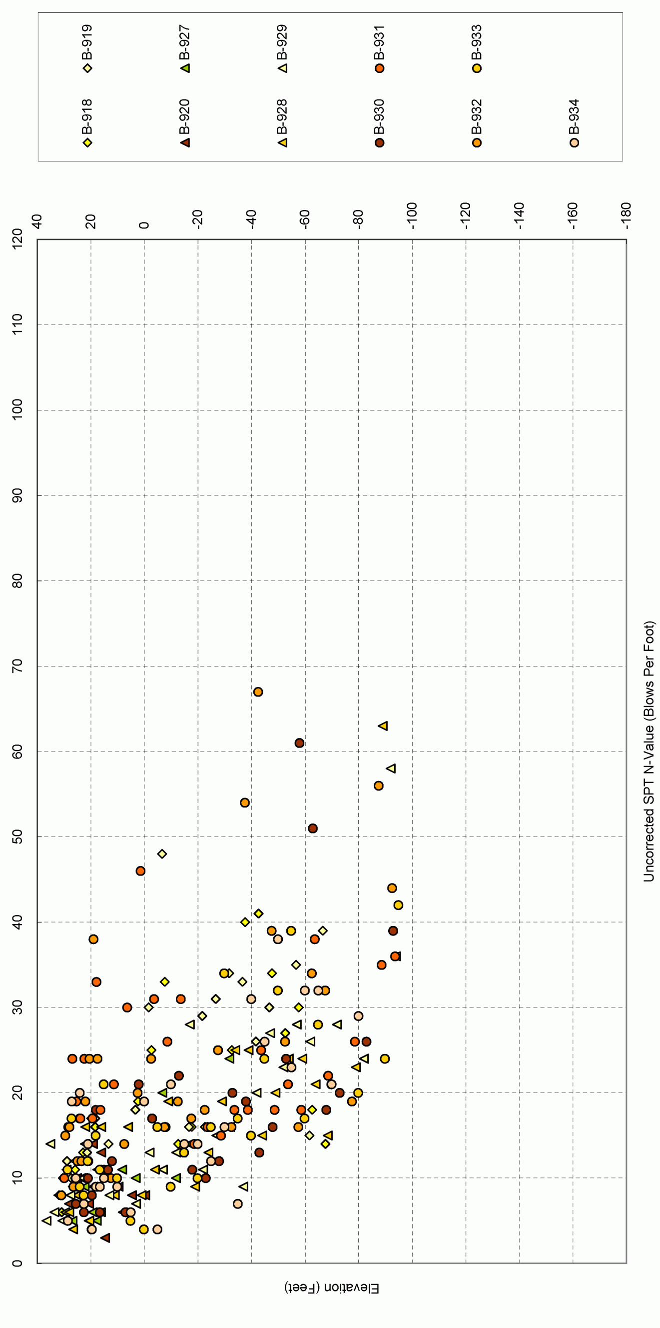


Figure 2.5S.4-15 Uncorrected SPT N-Values (Outside Power Block)

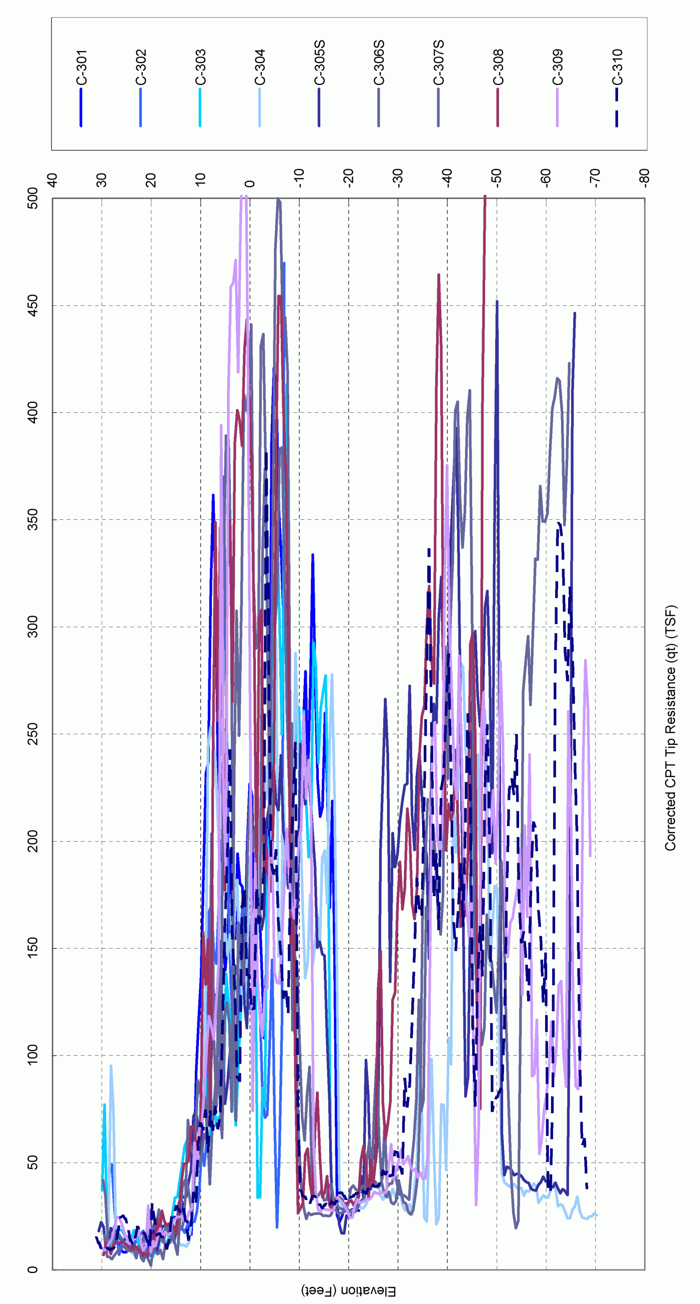


Figure 2.5S.4-16 Corrected CPT Tip Resistance (qt) (STP 3)

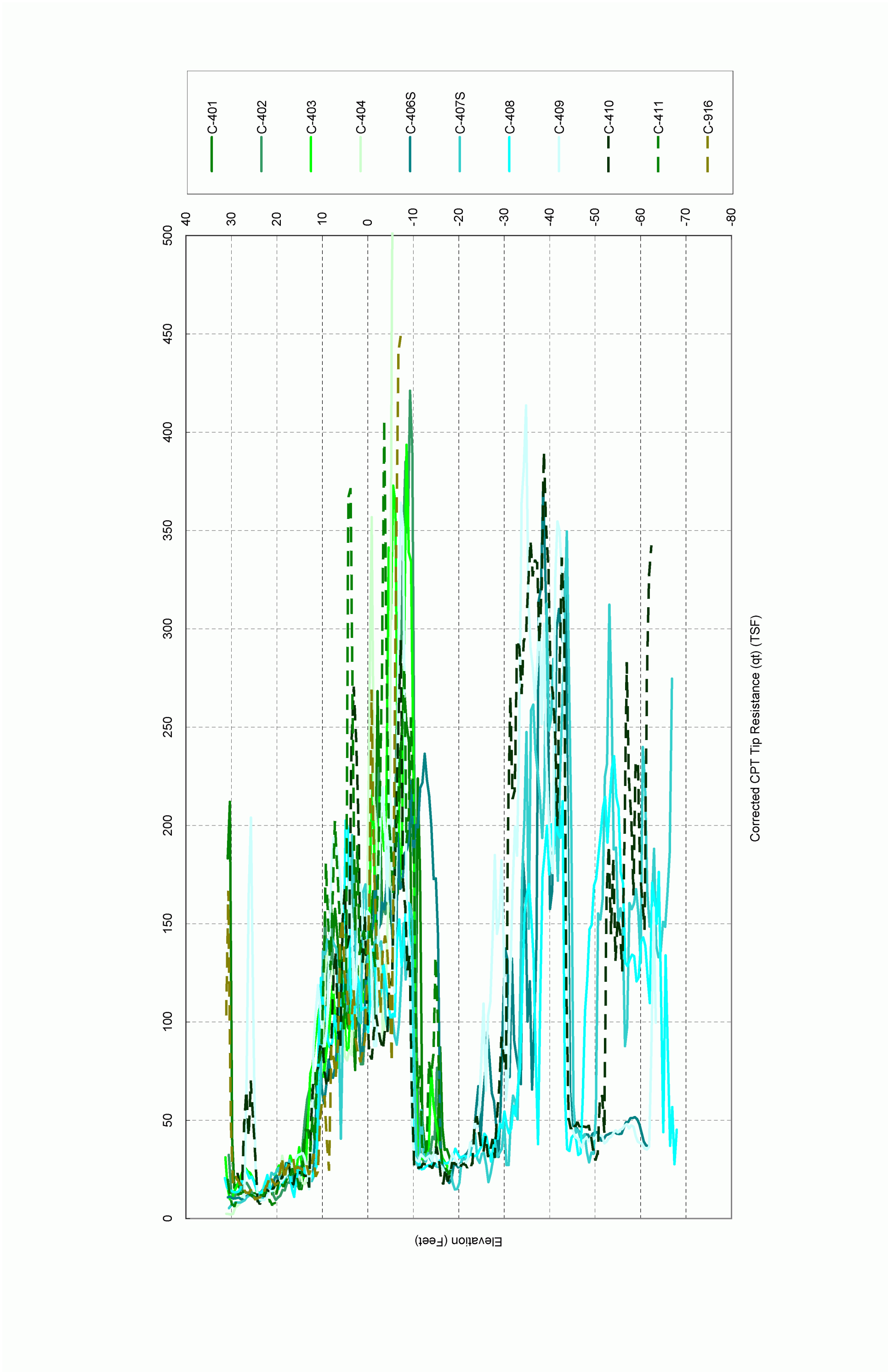


Figure 2.5S.4-17 Corrected CPT Tip Resistance (qt) (STP 3) <Includes C-916>

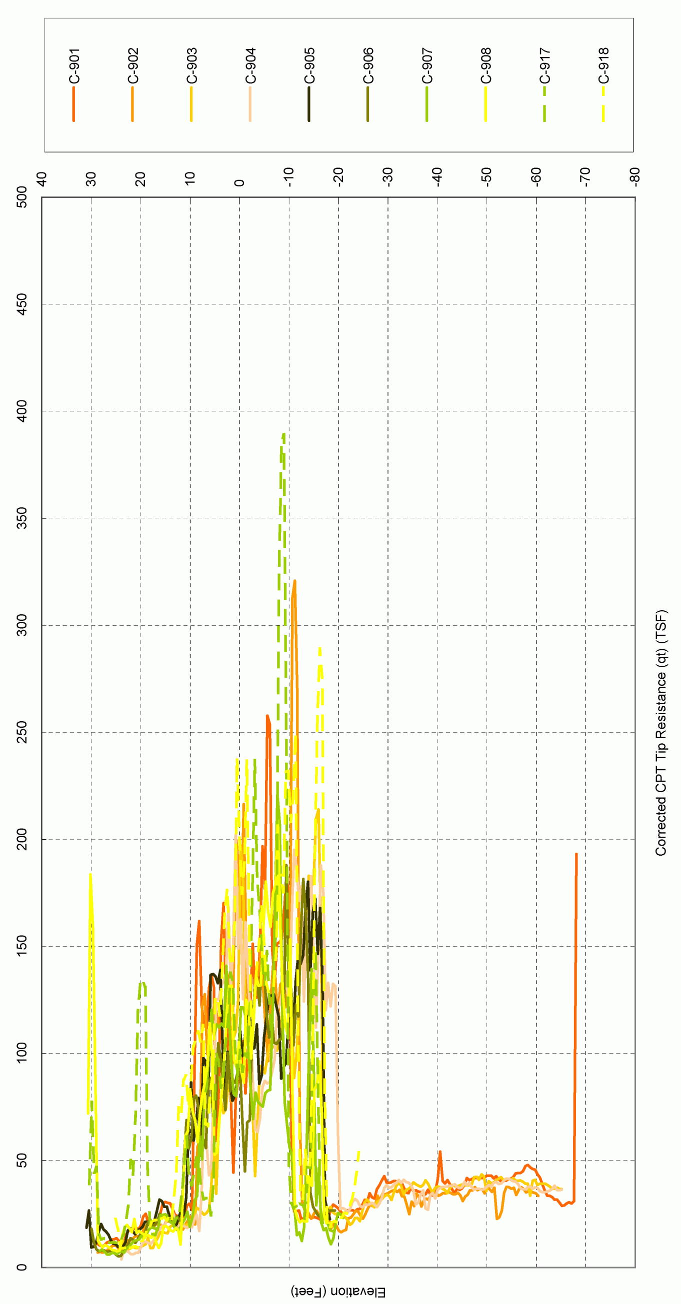


Figure 2.5S.4-18 Corrected CPT Tip Resistance (qt) (UHS Basin/RSW) <Excludes C-916>

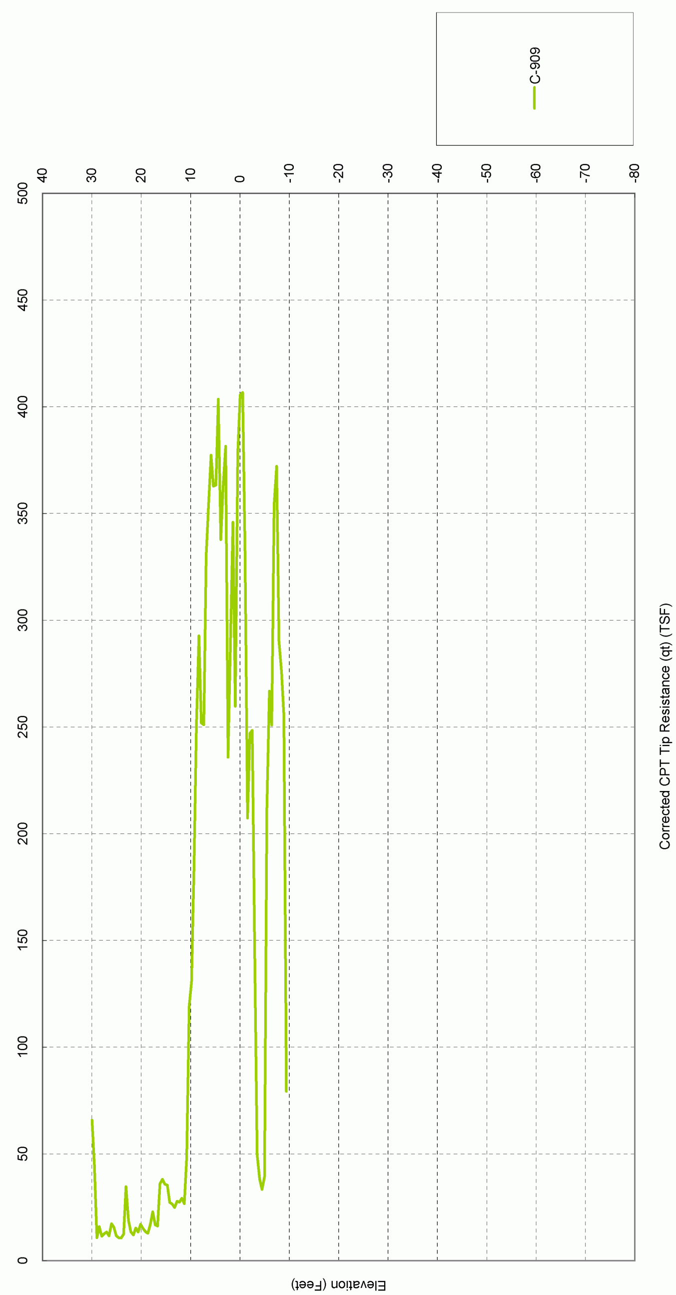


Figure 2.5S.4-19 Corrected CPT Tip Resistance (qt) (Outside Power Block)

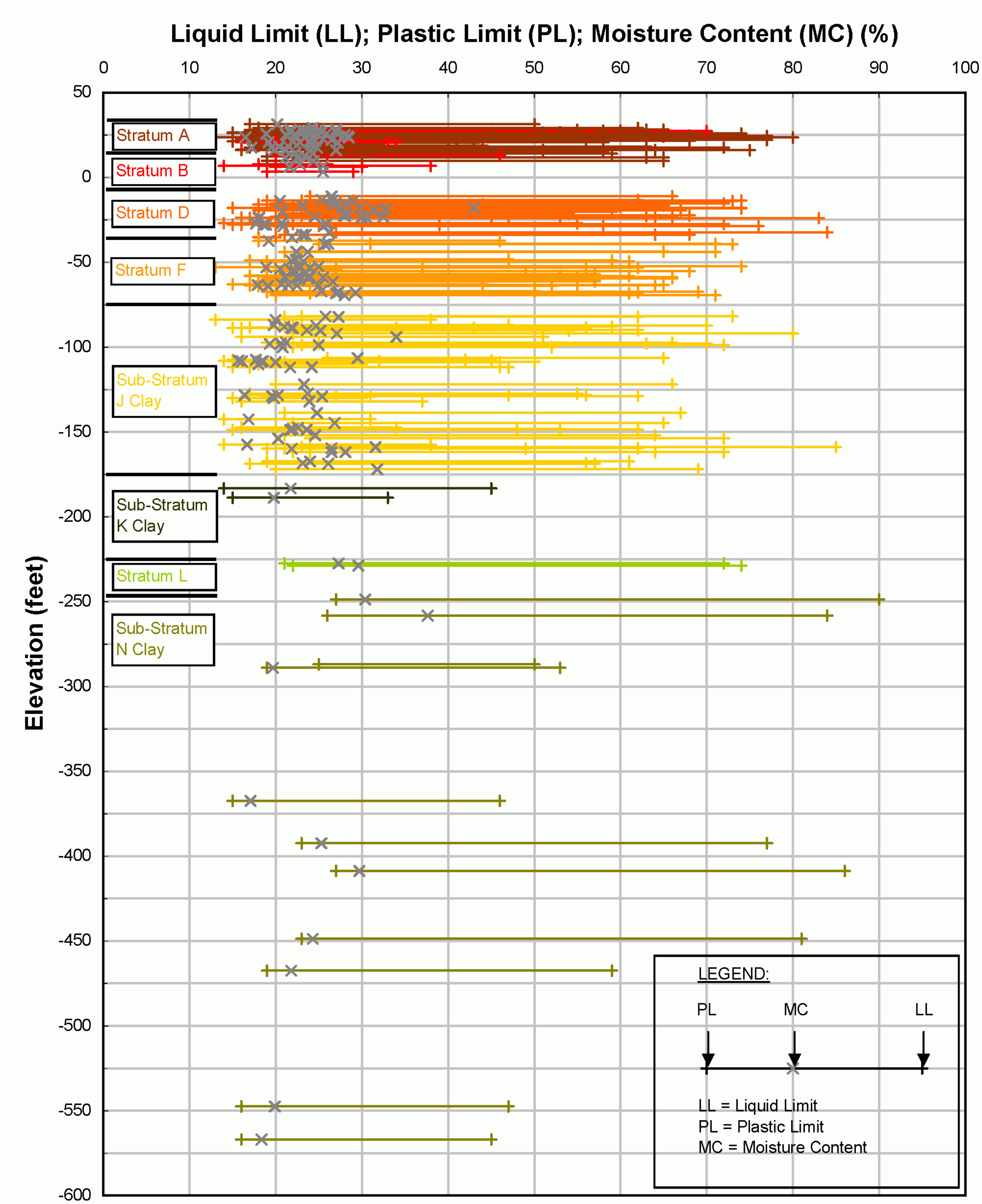


Figure 2.5S.4-20 Atterberg Limits versus Elevation

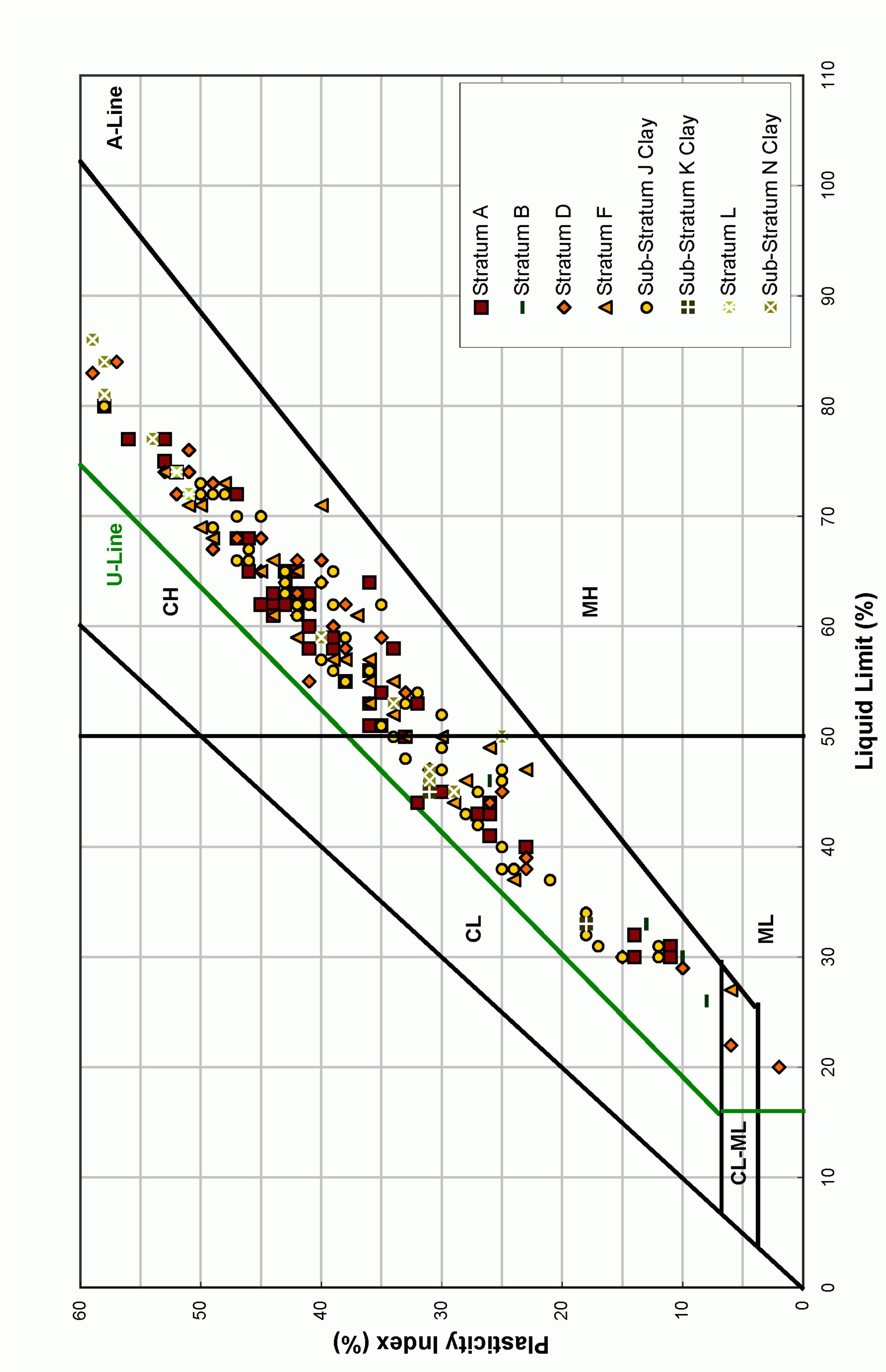


Figure 2.5S.4-21 Plasticity Chart

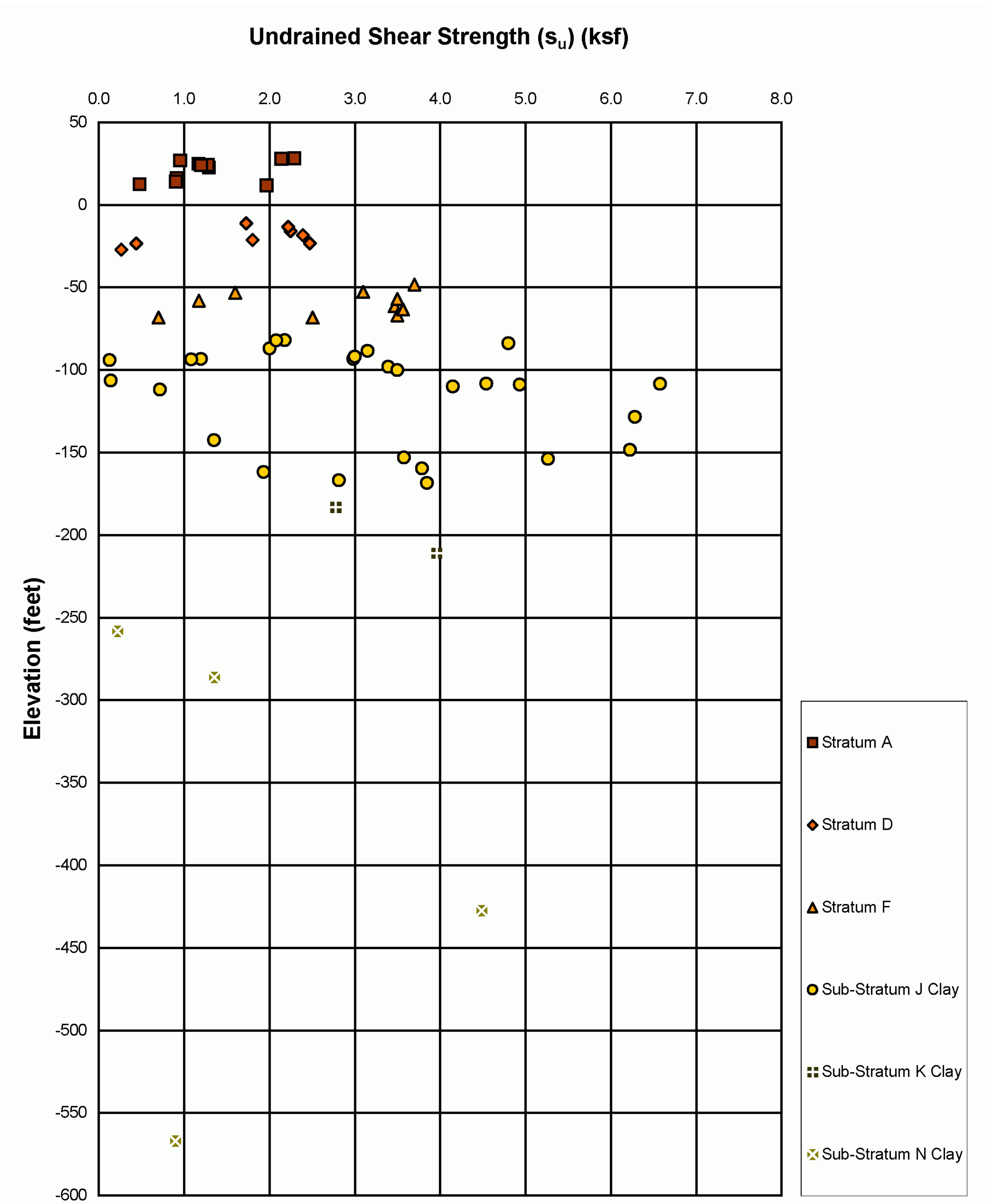


Figure 2.5S.4-22 Laboratory Test Results – Undrained Shear Strength (su) versus Elevation

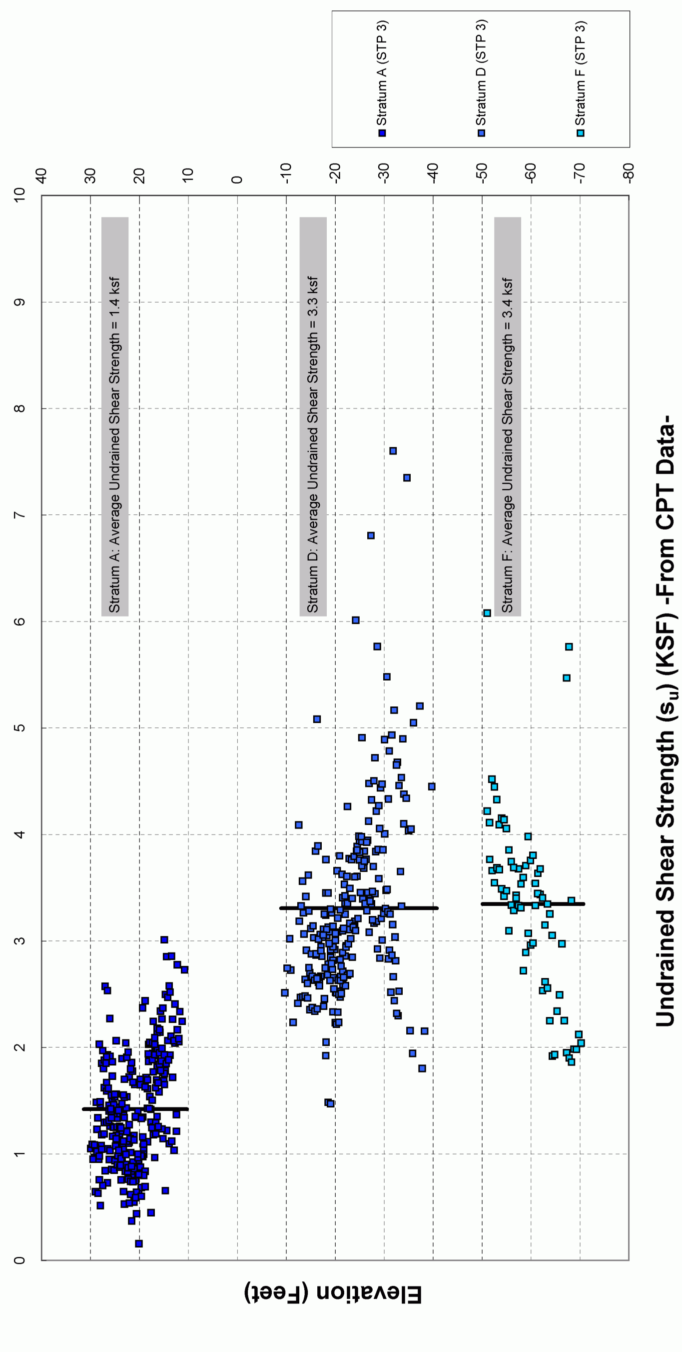


Figure 2.5S.4-23 Undrained Shear Strength (su) From CPT Data (STP 3)

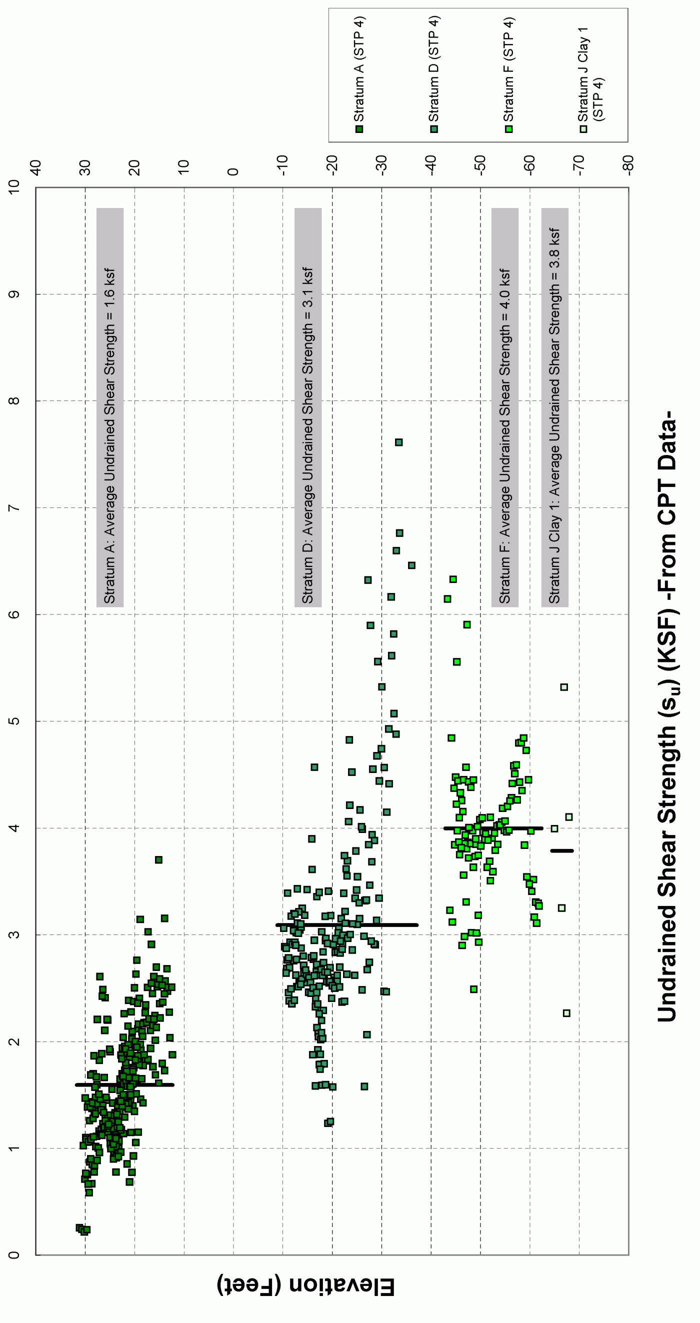


Figure 2.5S.4-24 Undrained Shear Strength (su) From CPT Data (STP 4)

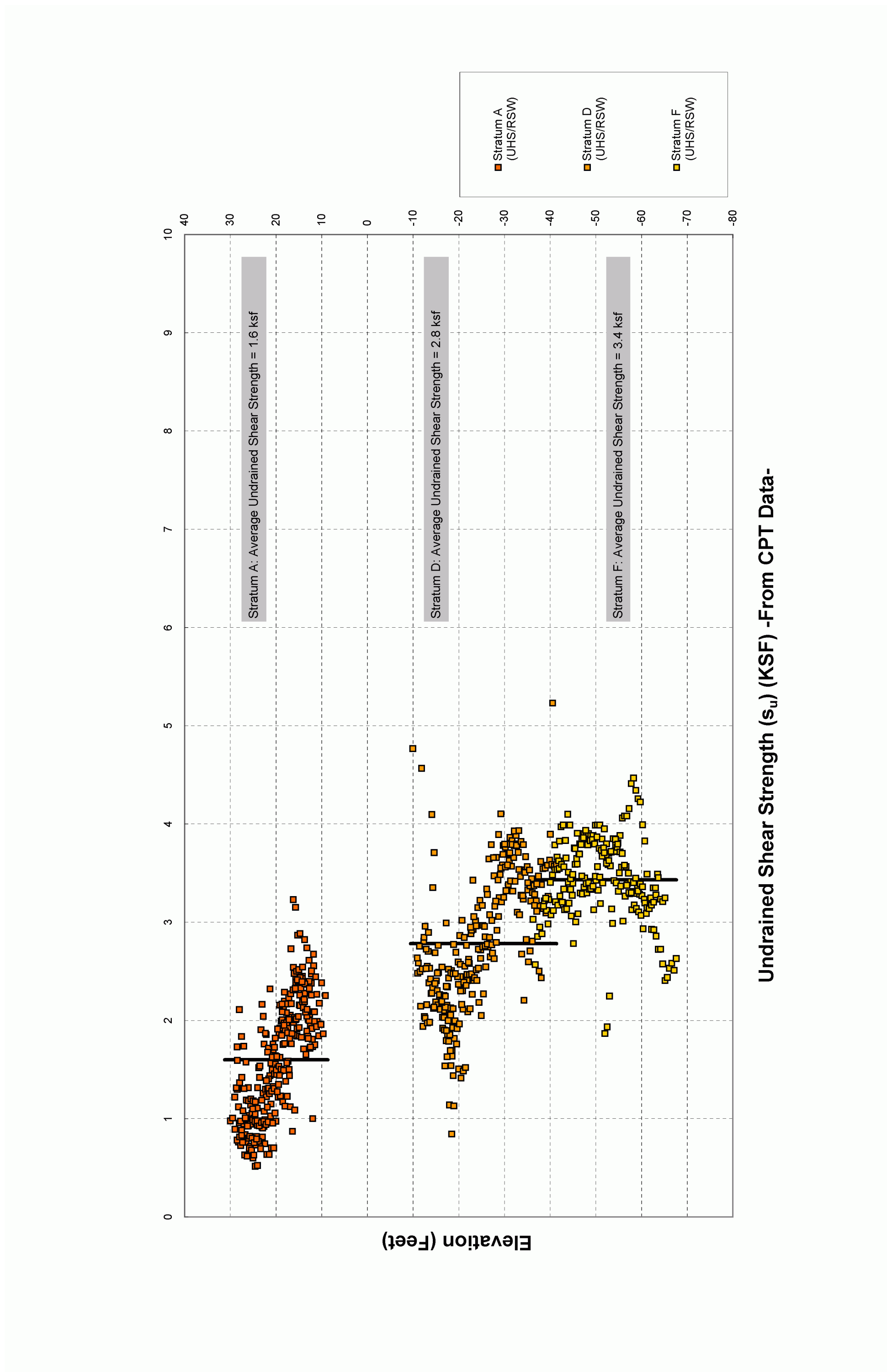


Figure 2.5S.4-25 Undrained Shear Strength (su) From CPT Data (UHS Basin/RSW)

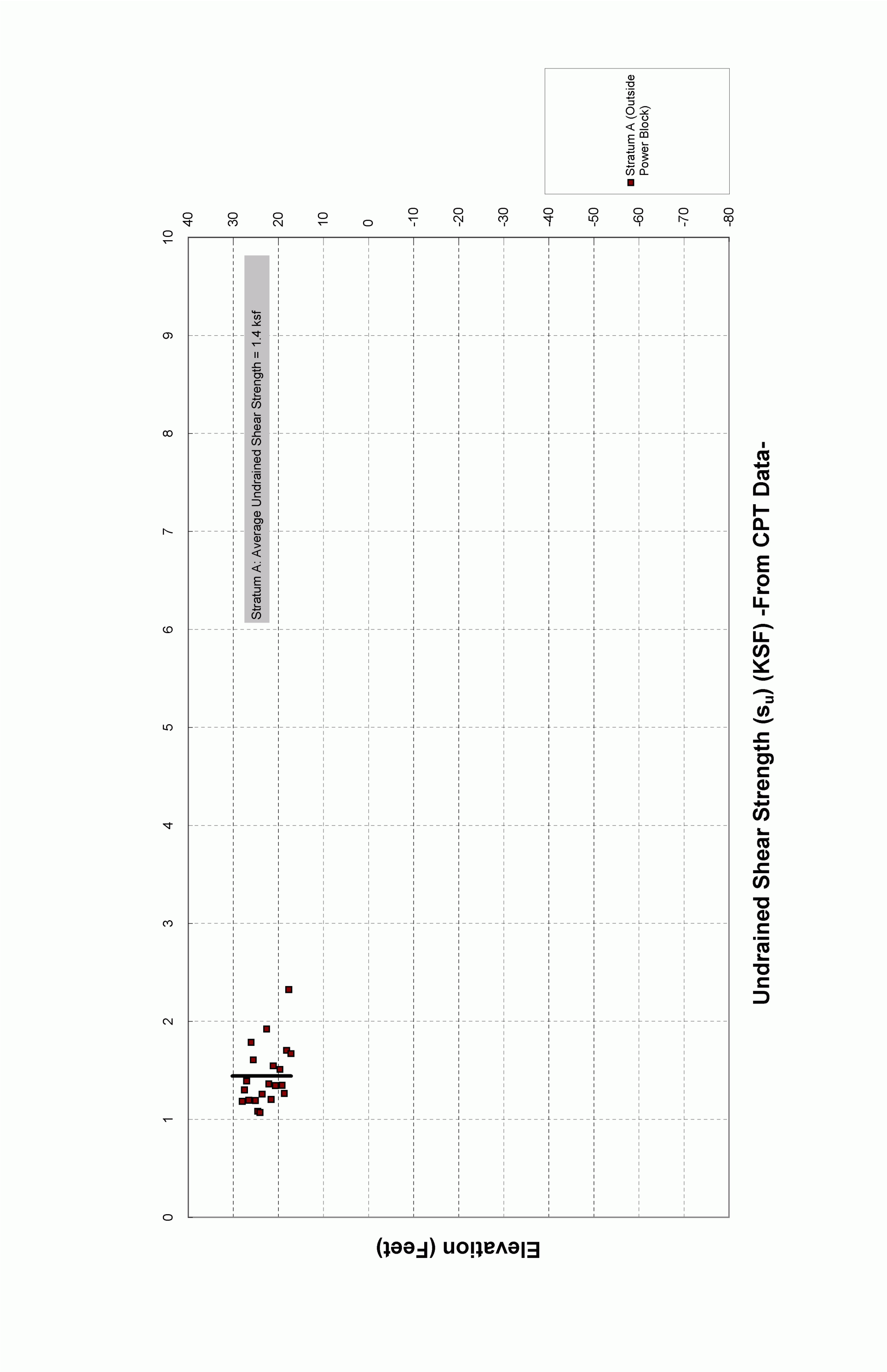


Figure 2.5S.4-26 Undrained Shear Strength (su) From CPT Data (Outside Power Block)

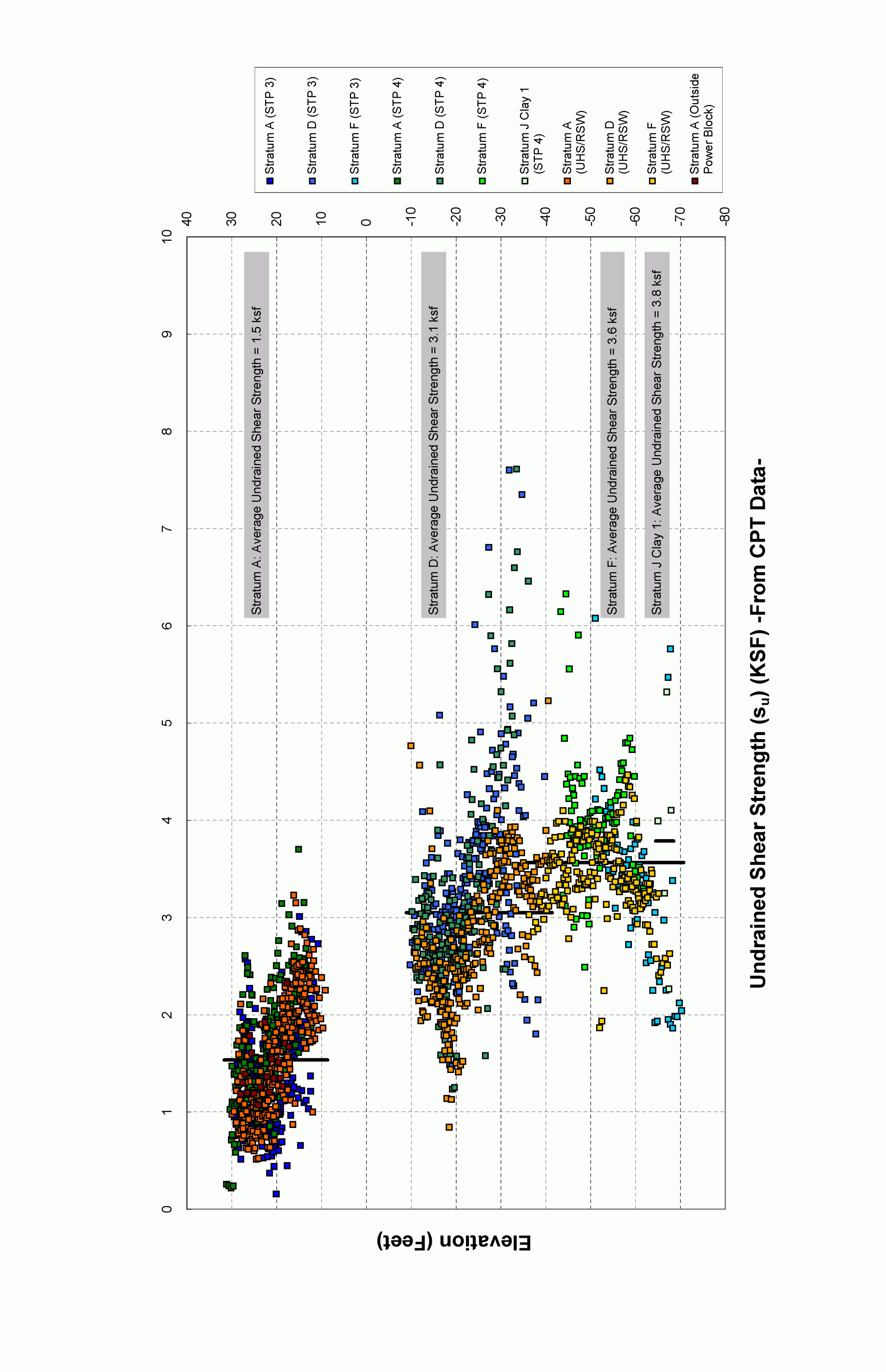


Figure 2.5S.4-27 Undrained Shear Strength (su) From CPT Data (Site-Wide)

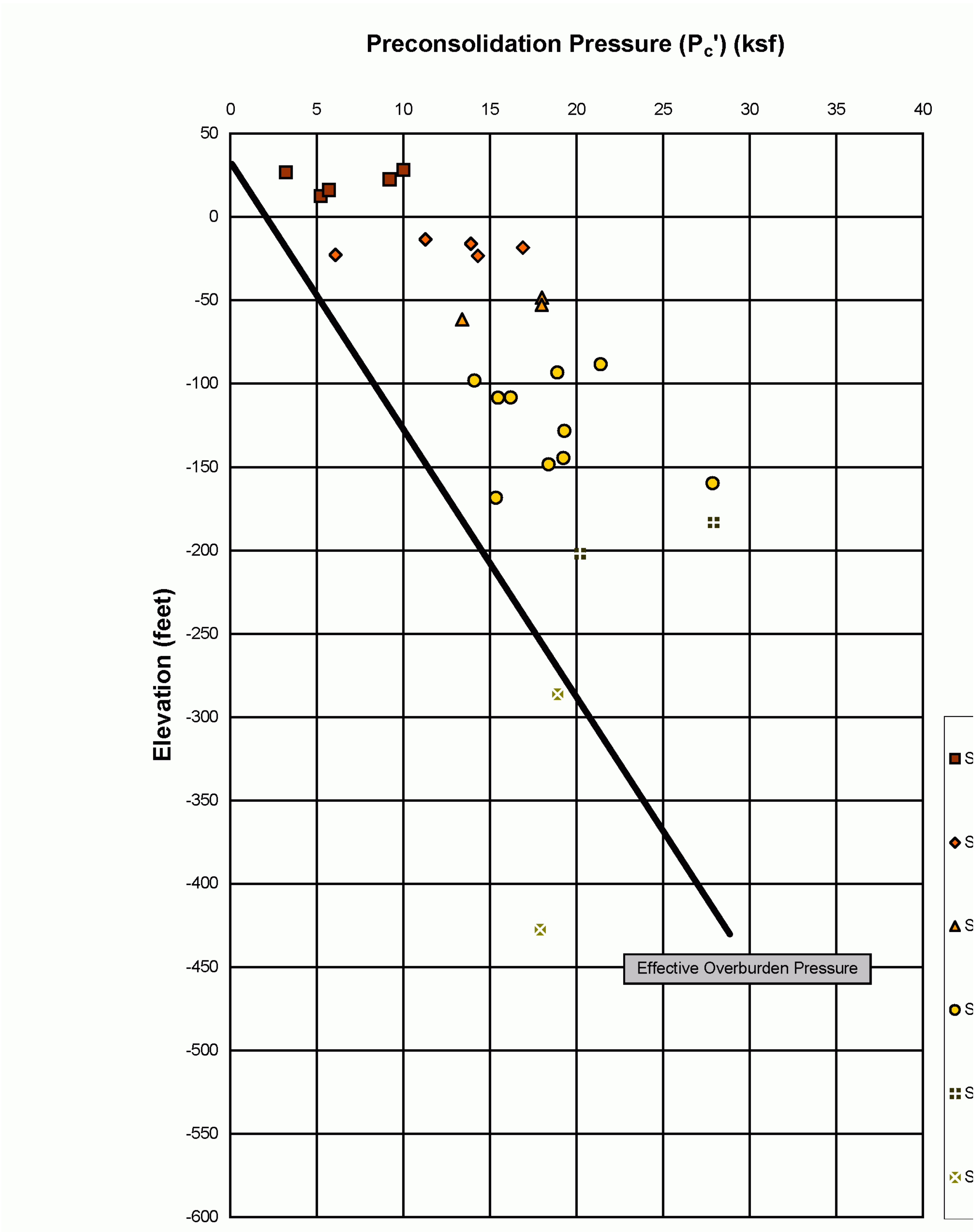


Figure 2.5S.4-28 Laboratory Test Results -Preconsolidation Pressure (P_c') versus Elevation

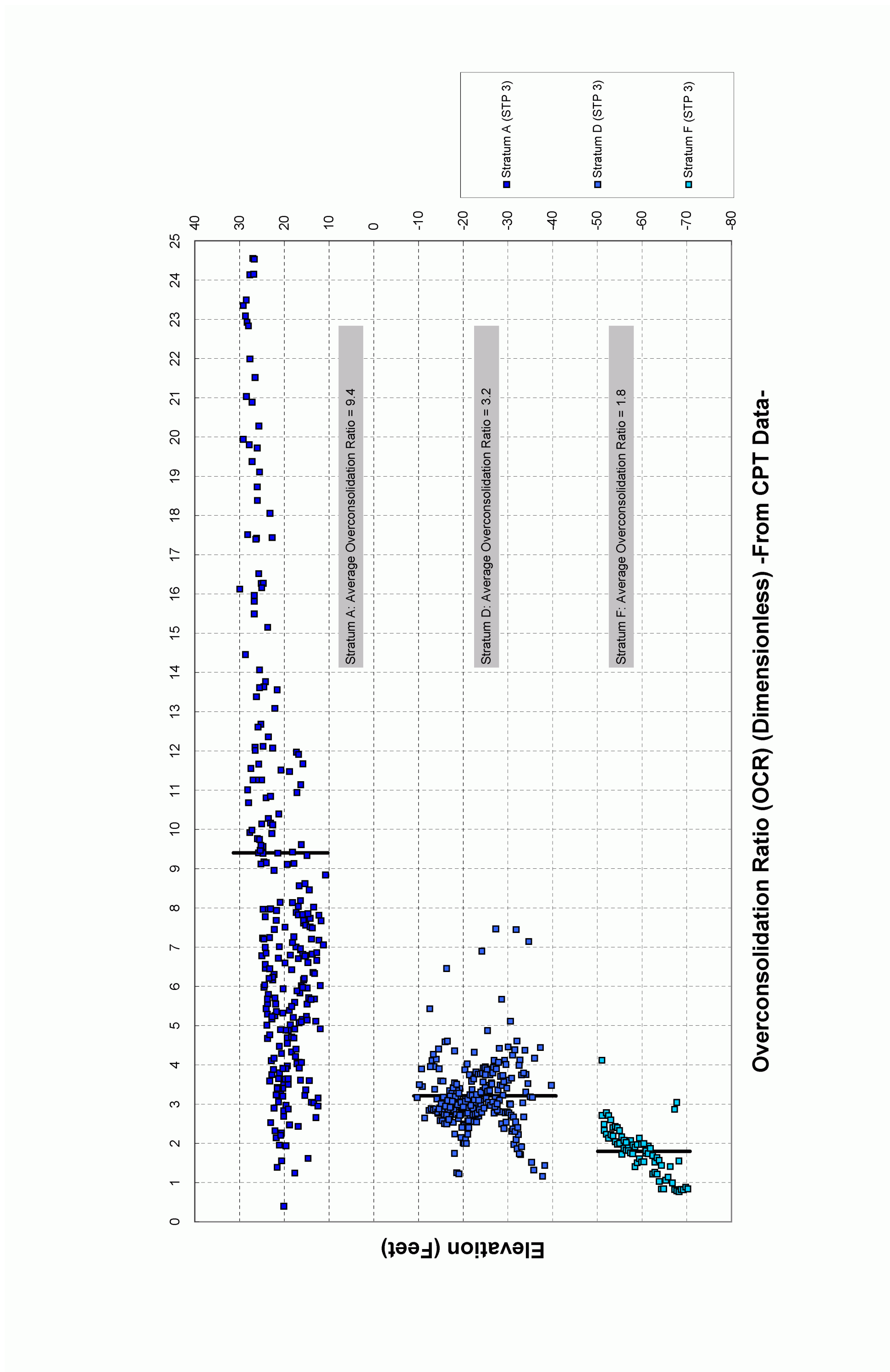


Figure 2.5S.4-29 Overconsolidation Ratio (OCR) From CPT Data (STP 3)

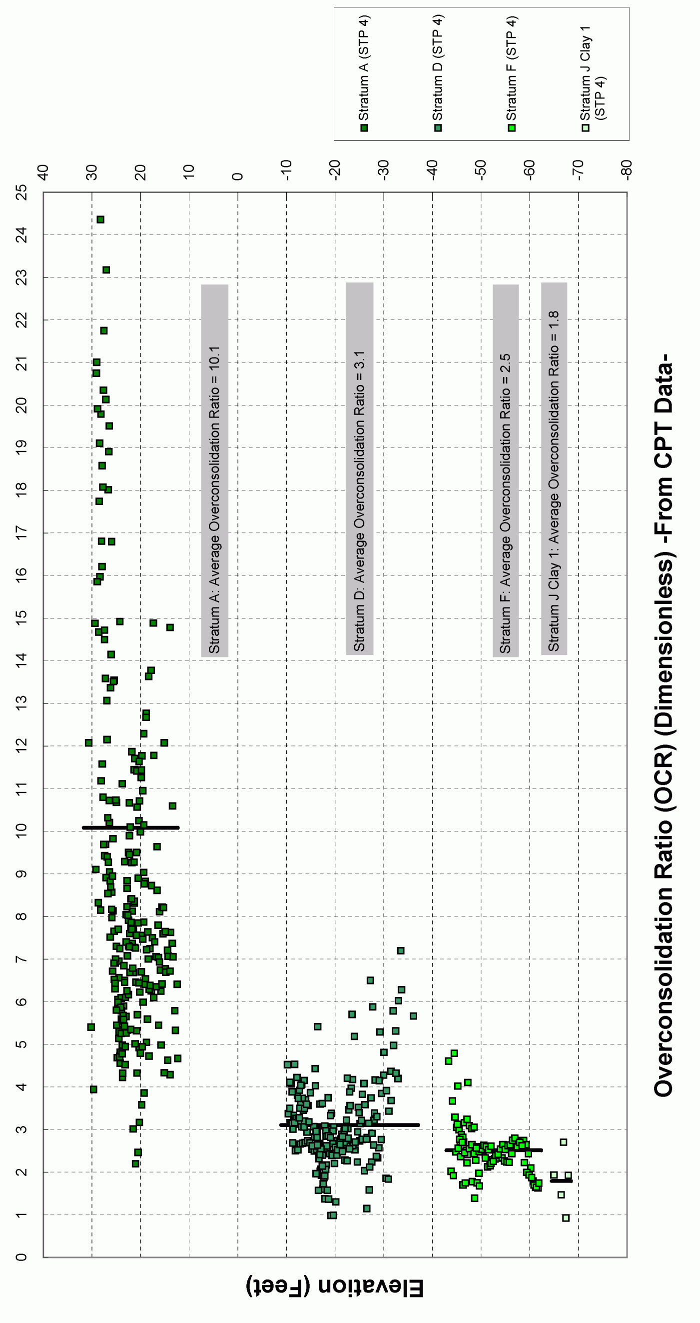


Figure 2.5S.4-30 Overconsolidation Ratio (OCR) From CPT Data (STP 4)

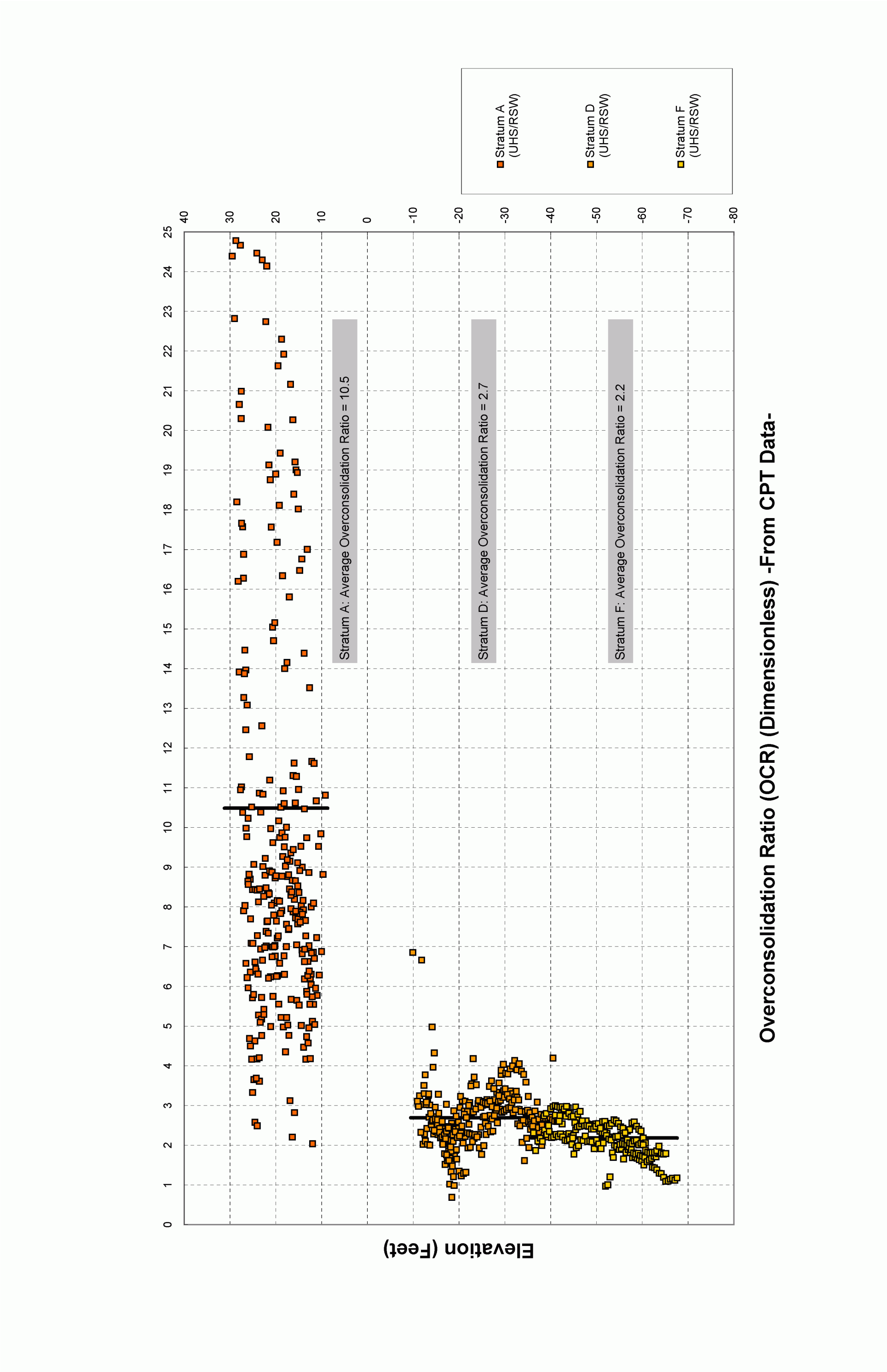


Figure 2.5S.4-31 Overconsolidation Ratio (OCR) From CPT Data (UHS Basin/RSW)

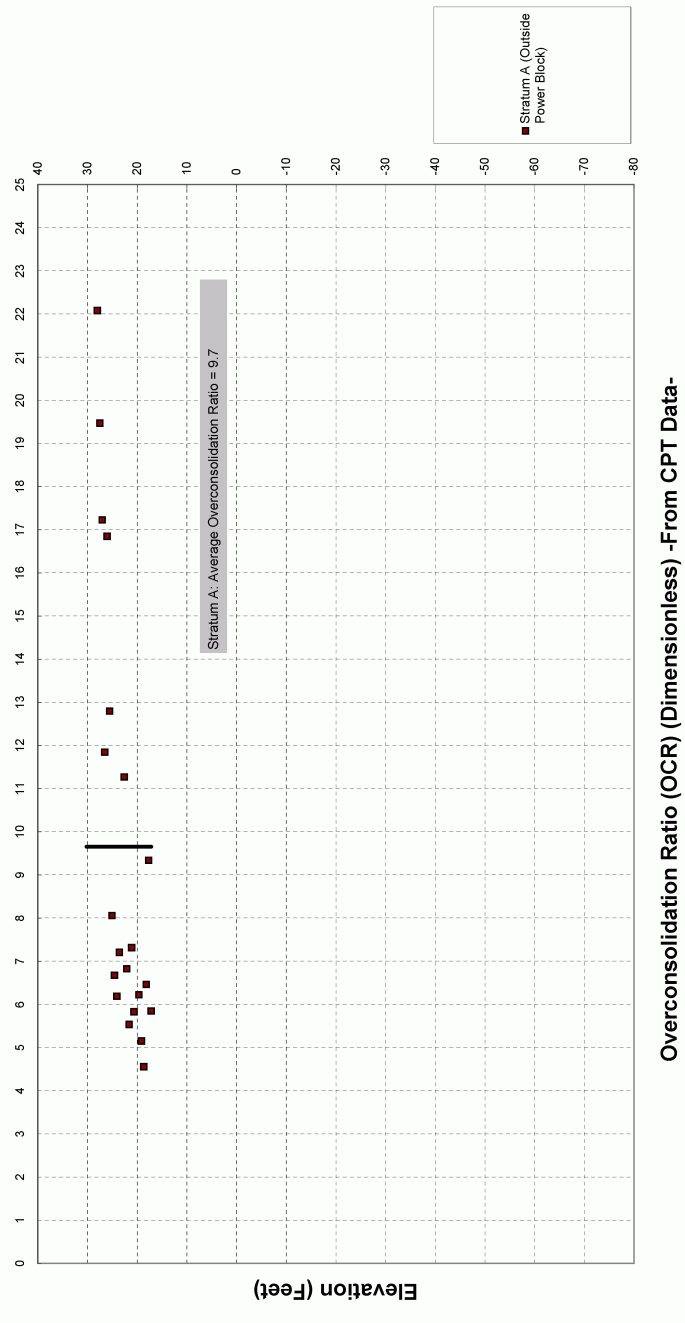


Figure 2.5S.4-32 Overconsolidation Ratio (OCR) From CPT Data (Outside Power Block)

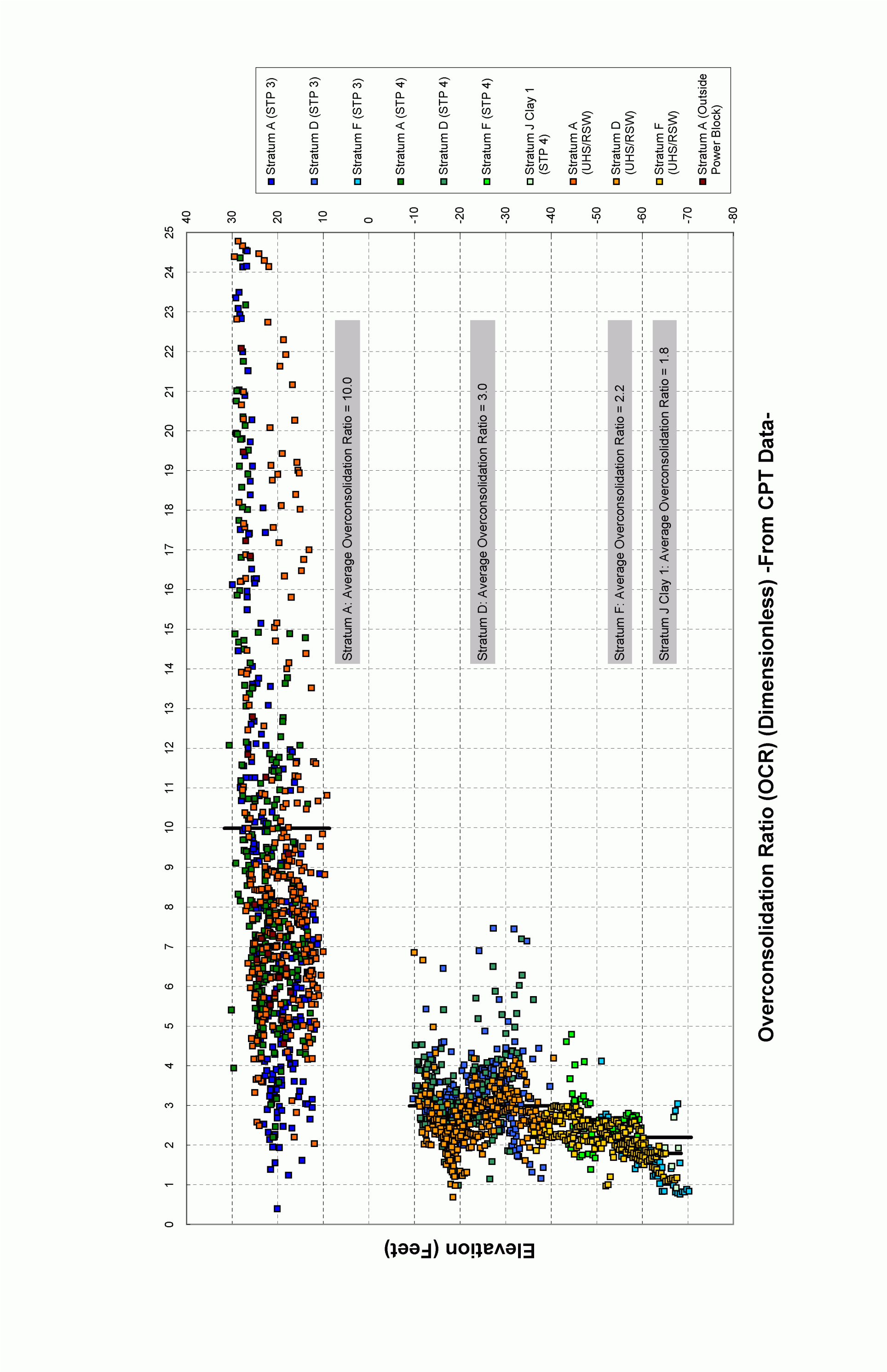


Figure 2.5S.4-33 Overconsolidation Ratio (OCR) From CPT Data (Site-Wide)

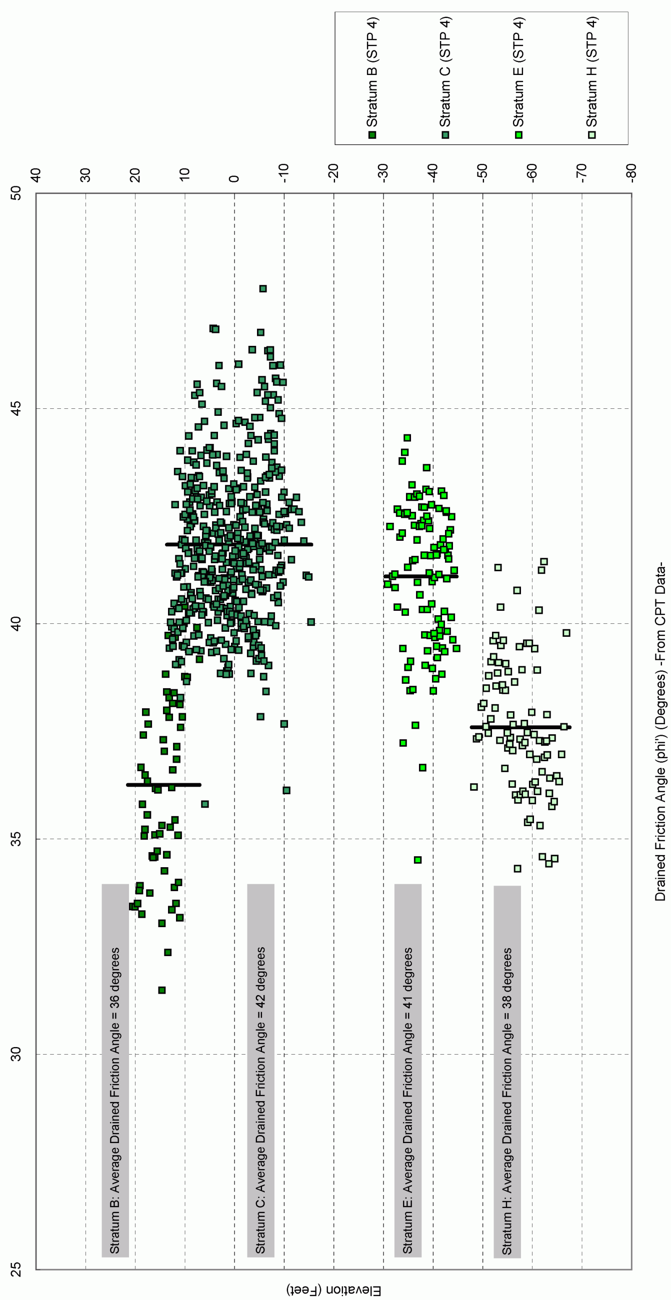


Figure 2.5S.4-34 Drained Friction Angle (phi') From CPT Data (STP 3)

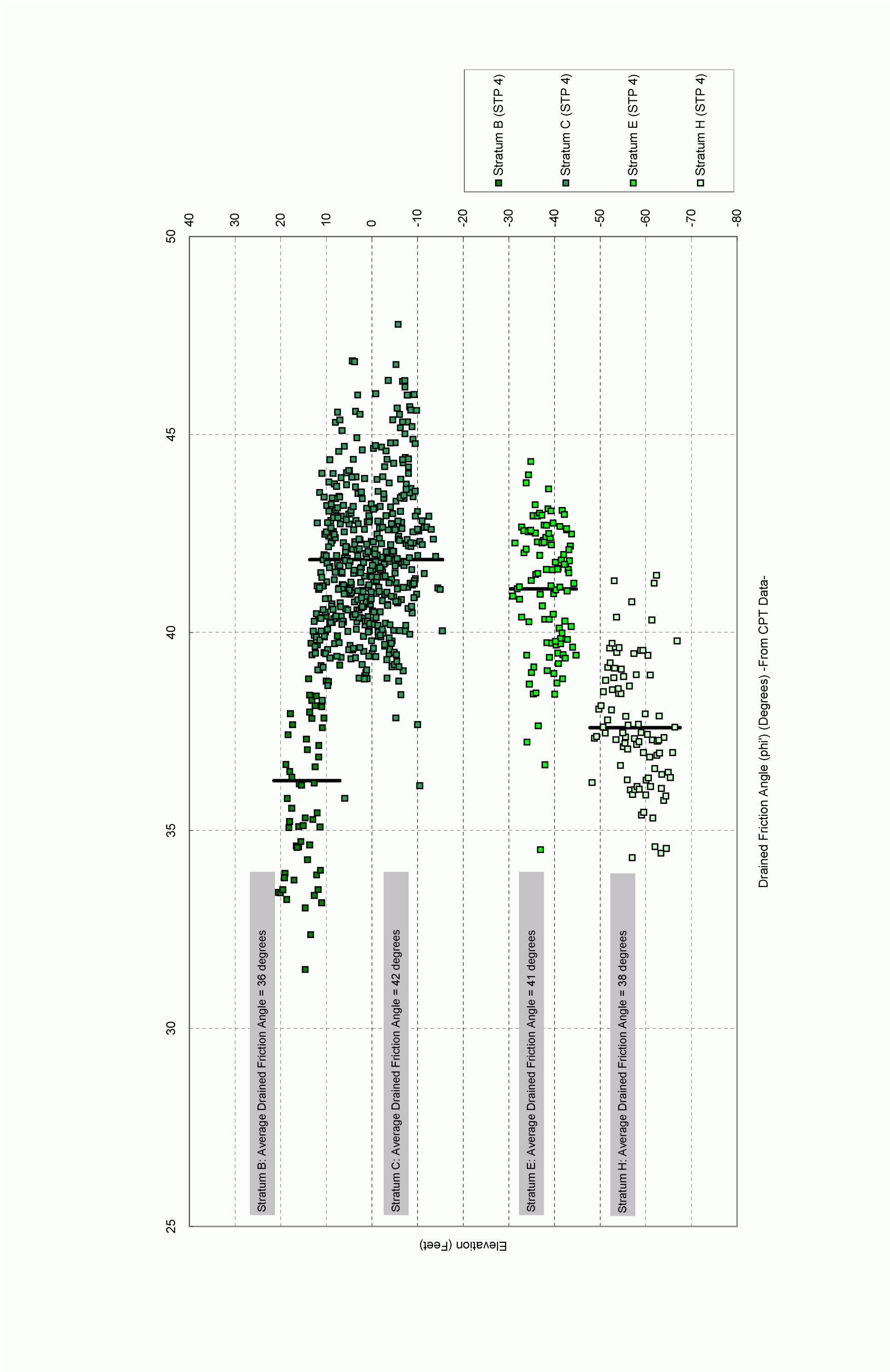


Figure 2.5S.4-35 Drained Friction Angle (phi') From CPT Data (STP 4)

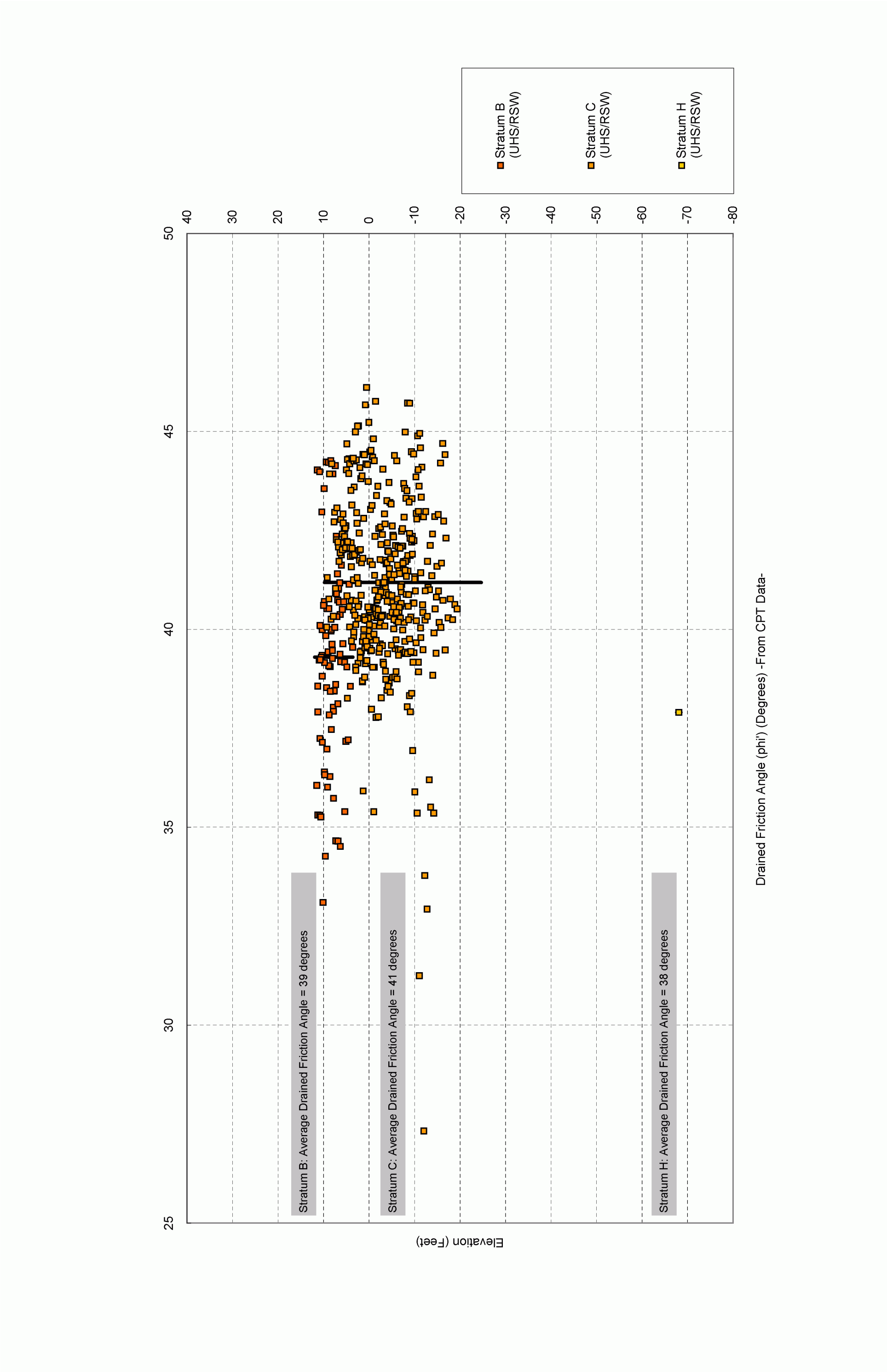


Figure 2.5S.4-36 Drained Friction Angle (phi') From CPT Data (UHS Basin/ RSW)

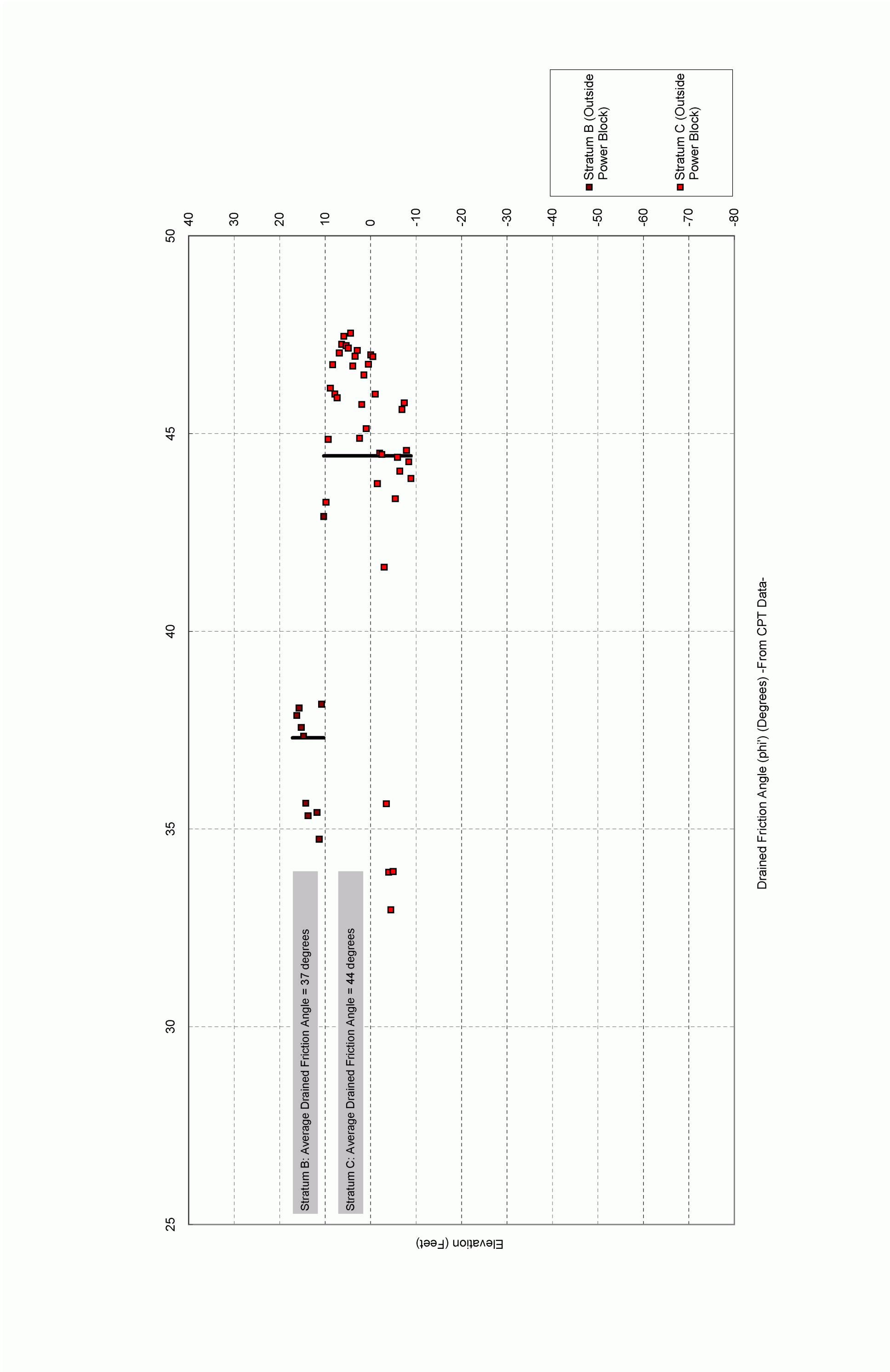


Figure 2.5S.4-37 Drained Friction Angle (phi') From CPT Data (Outside Power Block)

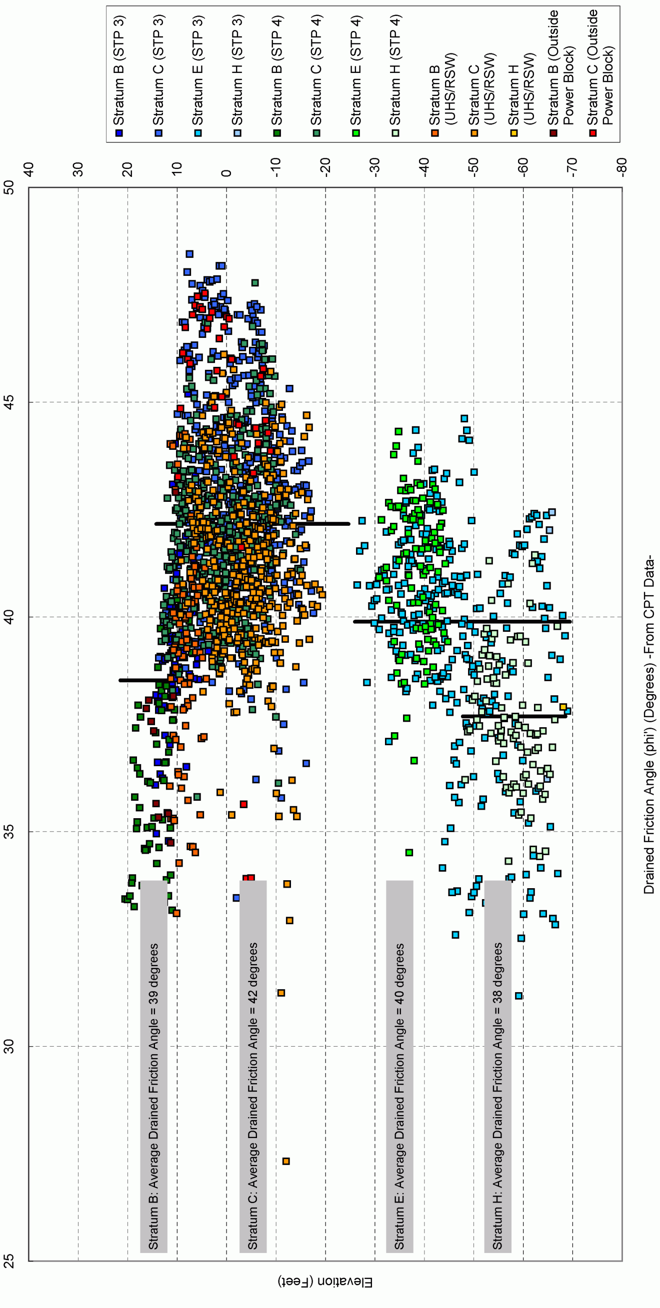


Figure 2.5S.4-38 Drained Friction Angle (phi') From CPT Data (Site-Wide)

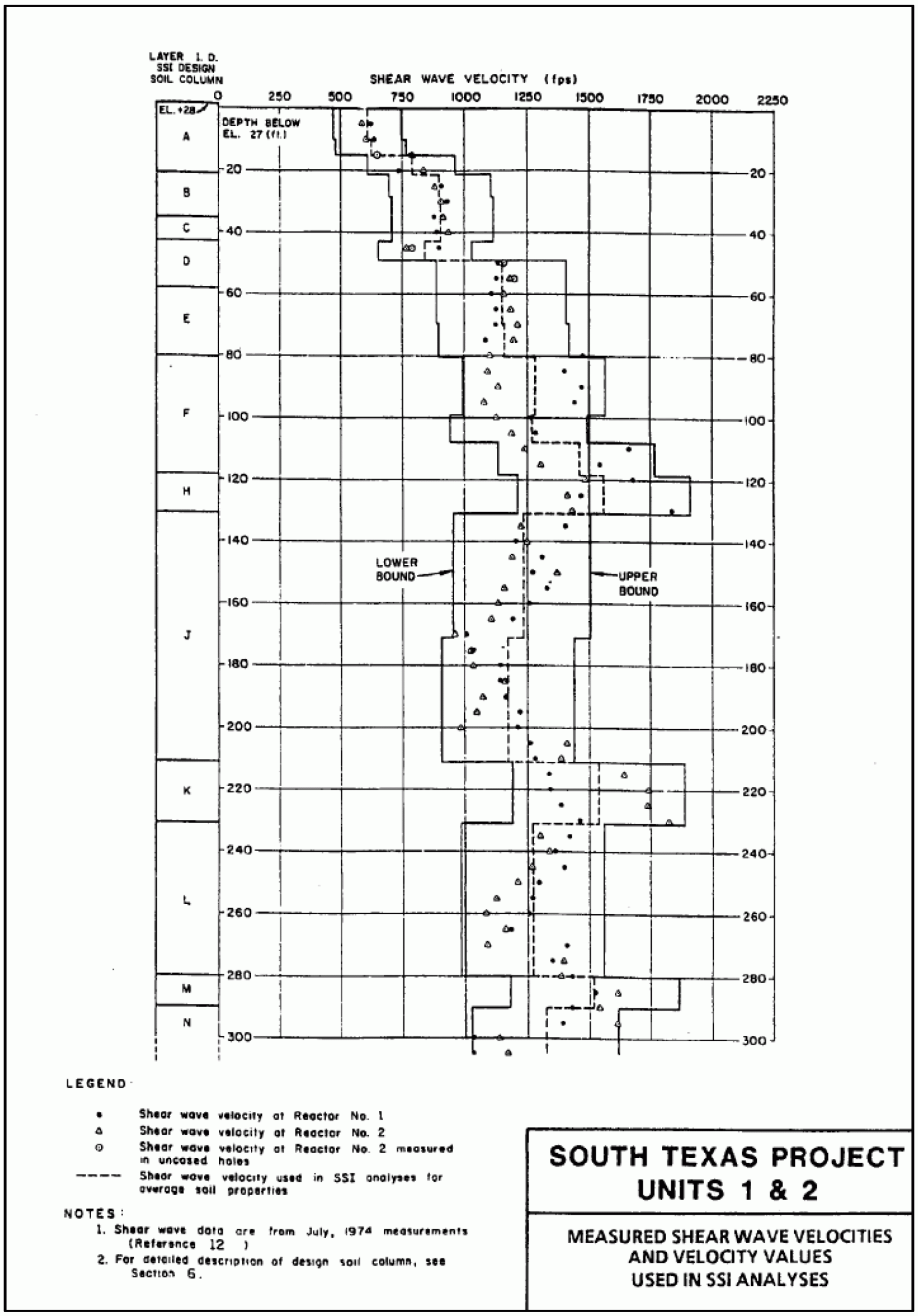


Figure 2.5S.4-39 STP 1 & 2; Shear Wave Velocity versus Depth

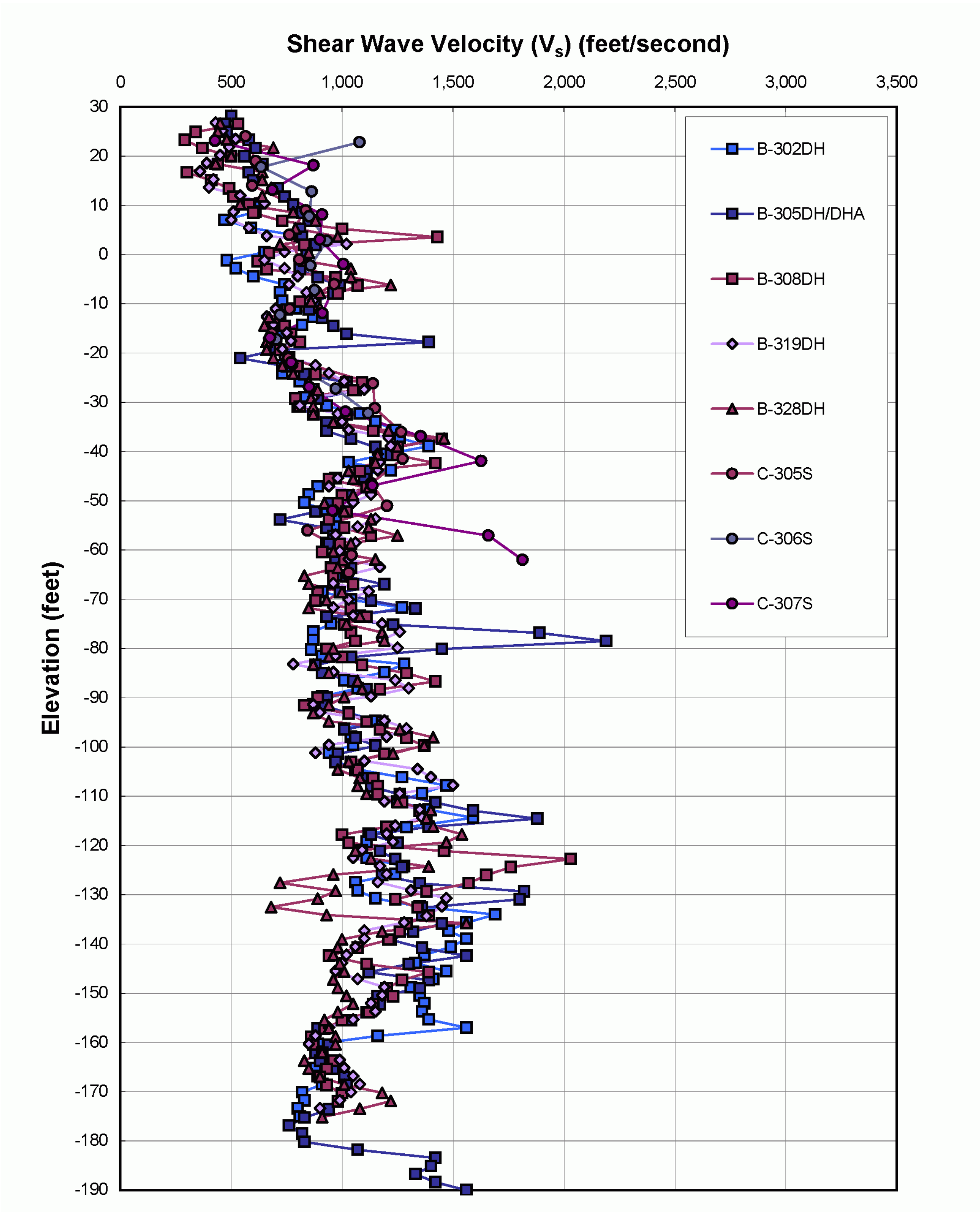


Figure 2.5S.4-40 STP 3; Shear Wave Velocity versus Depth

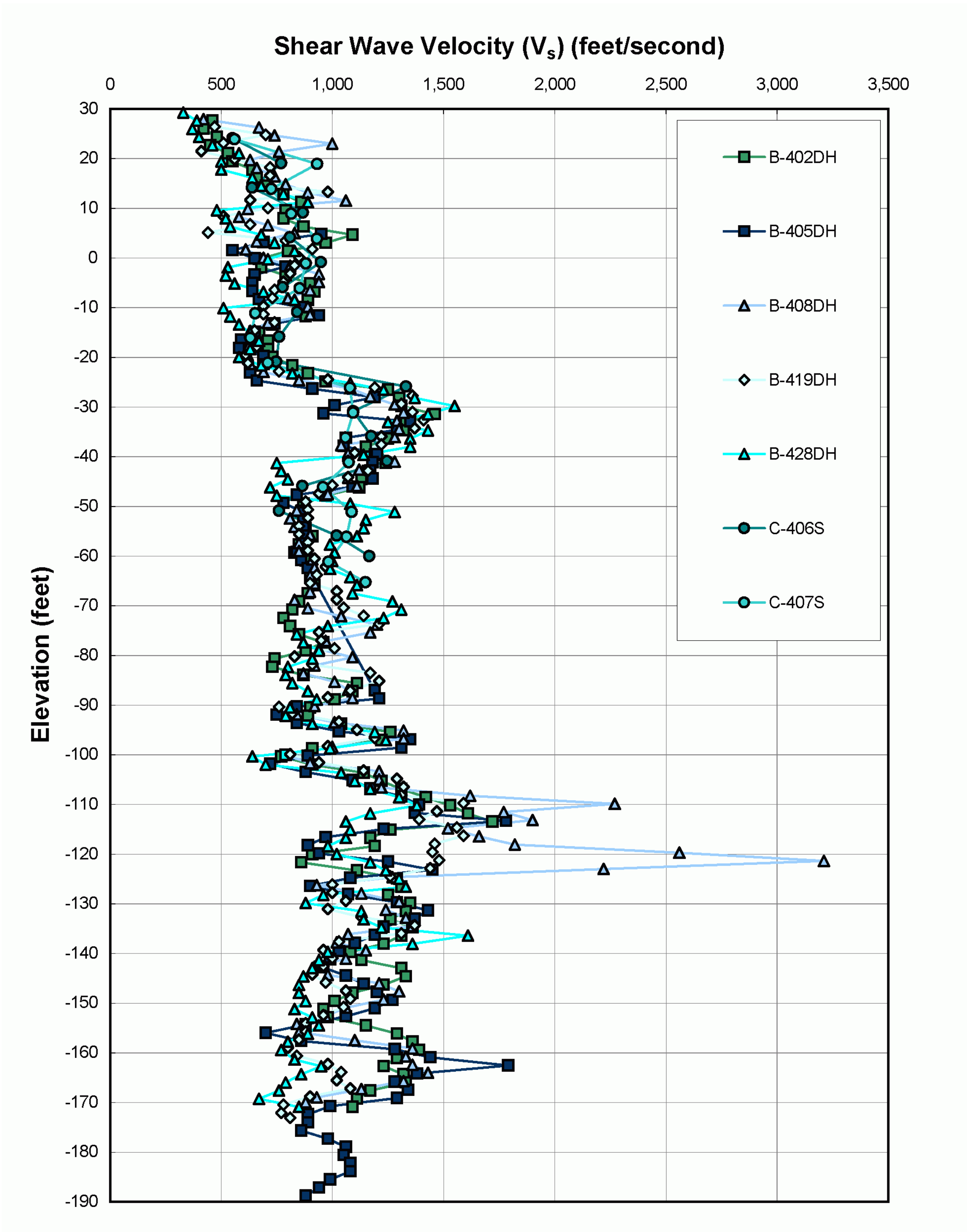


Figure 2.5S.4-41 STP 4; Shear Wave Velocity versus Depth

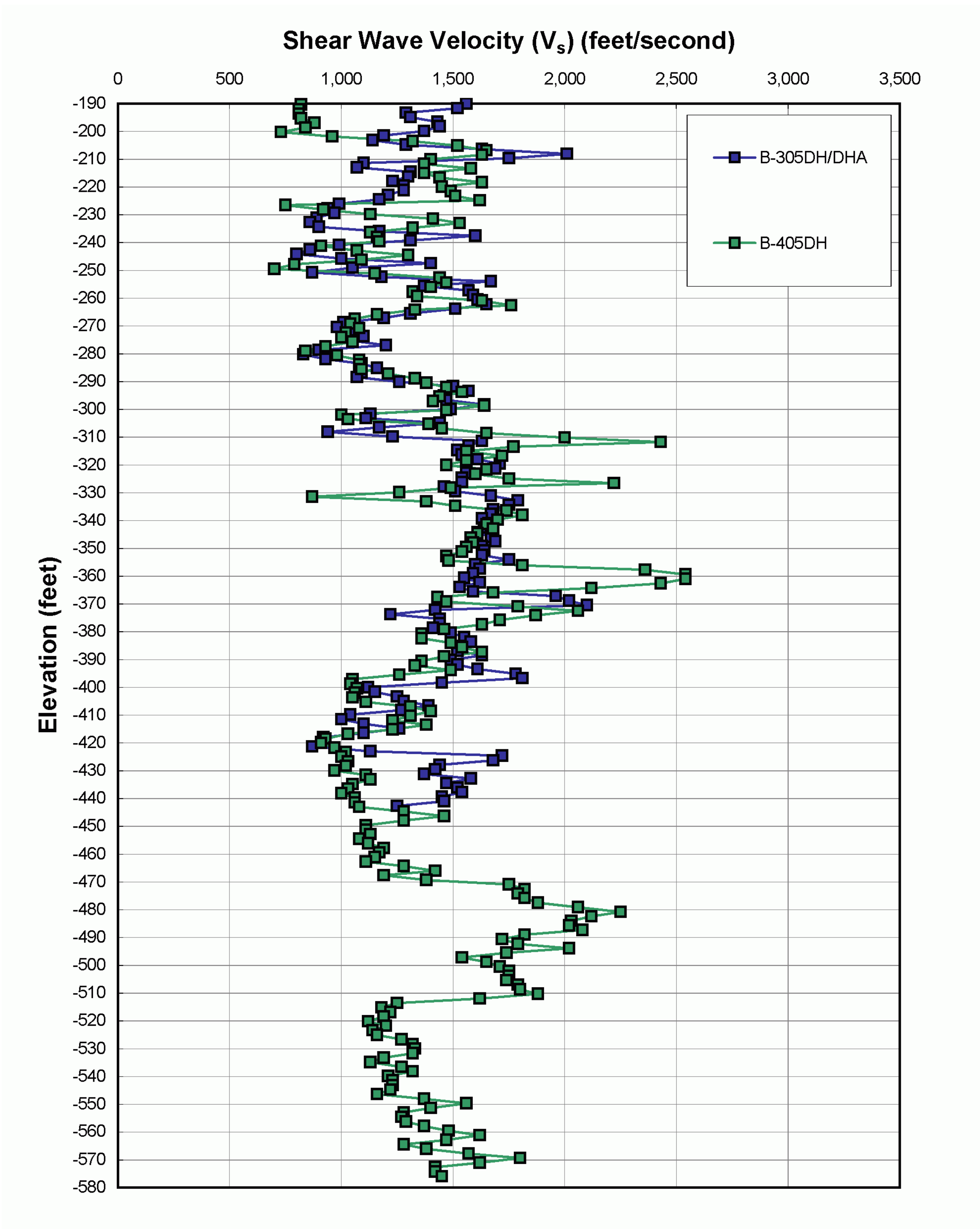


Figure 2.5S.4-42 STP 3 & 4; Shear Wave Velocity to 600 Feet Below Ground Surface

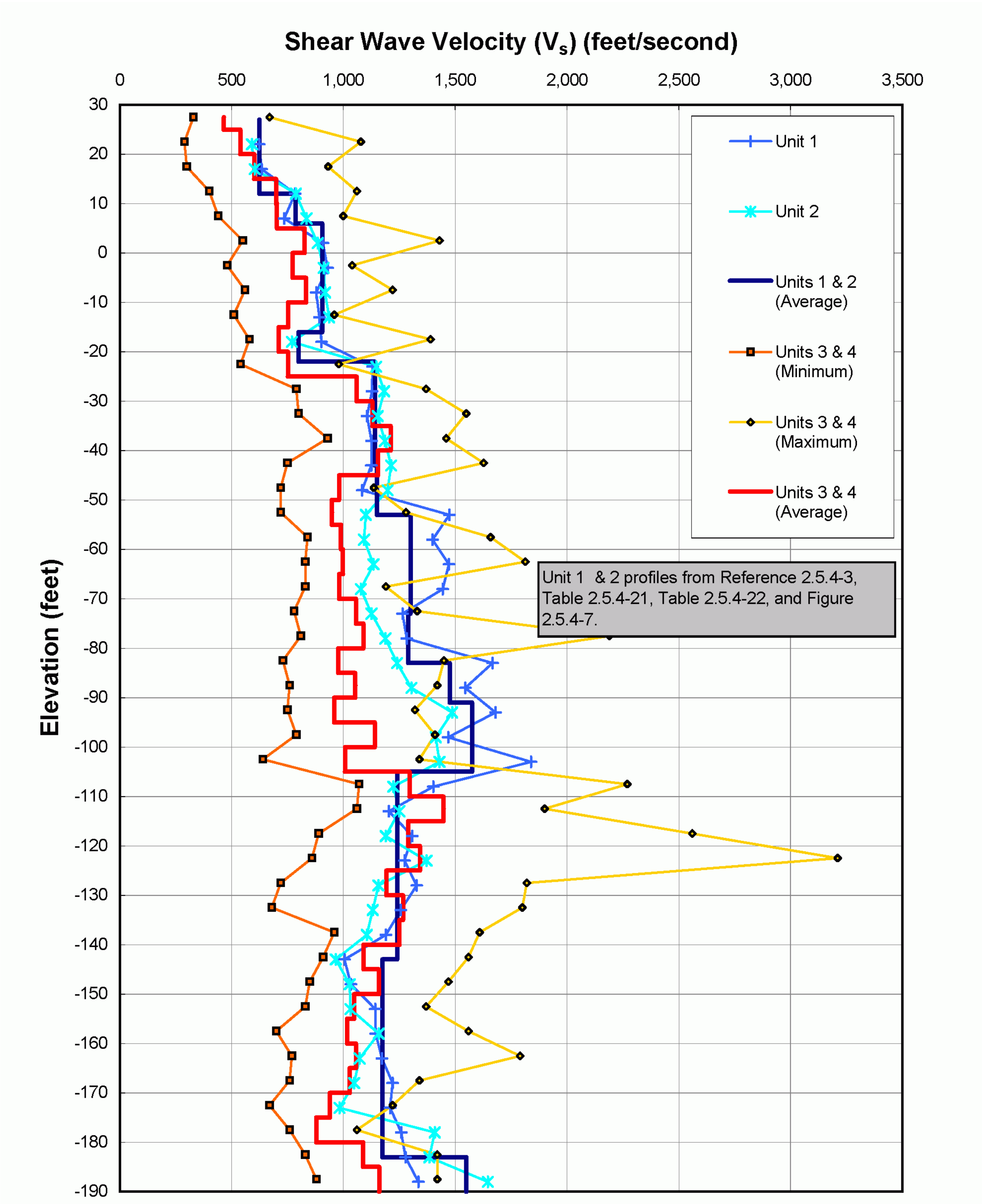


Figure 2.5S.4-43 STP 1 & 2/ STP 3 & 4; Average Shear Wave Velocity to 200 Feet Below Ground Surface

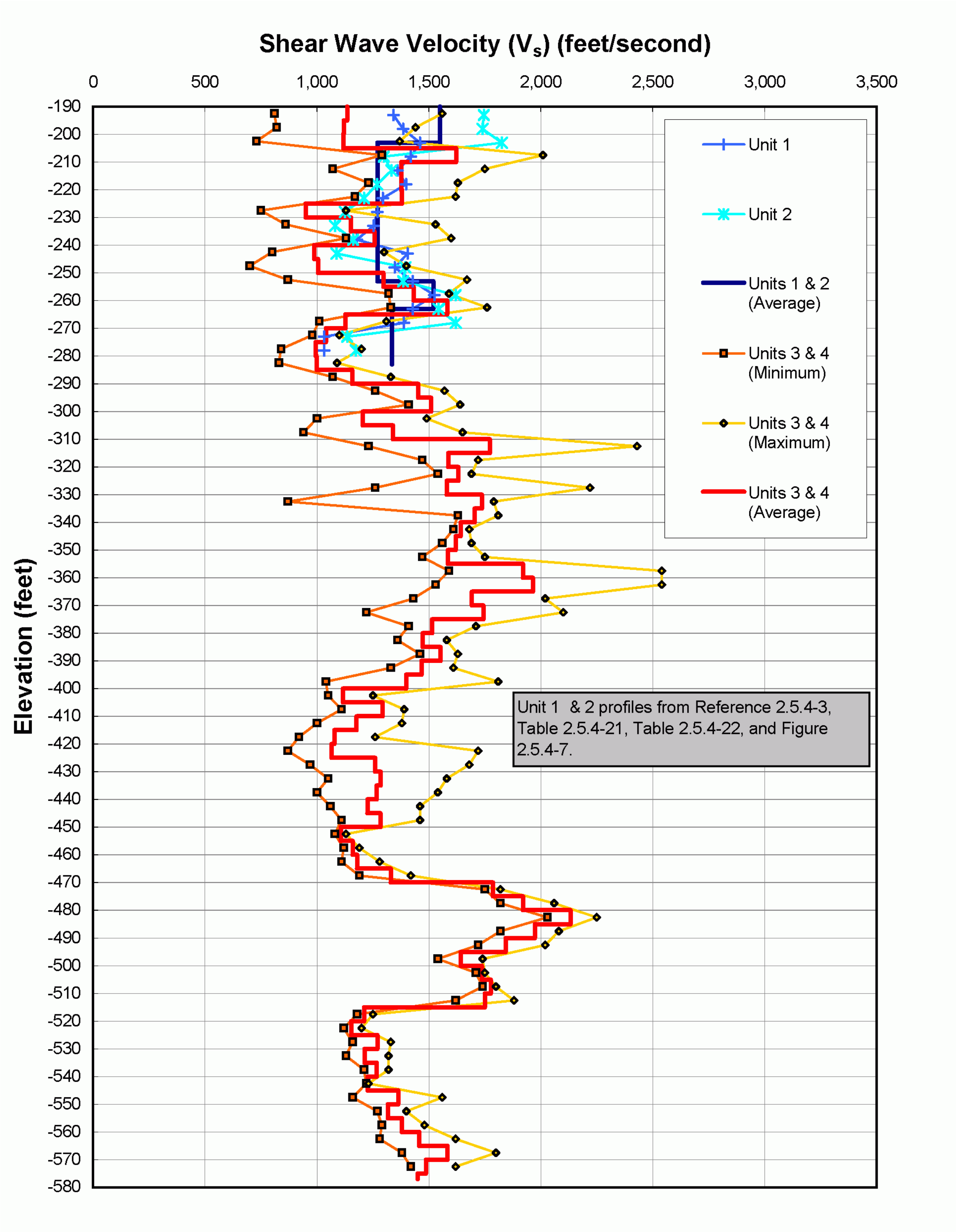


Figure 2.5S.4-44 STP 1 & 2/ STP 3 & 4; Average Shear Wave Velocity to 600 Feet Below Ground Surface

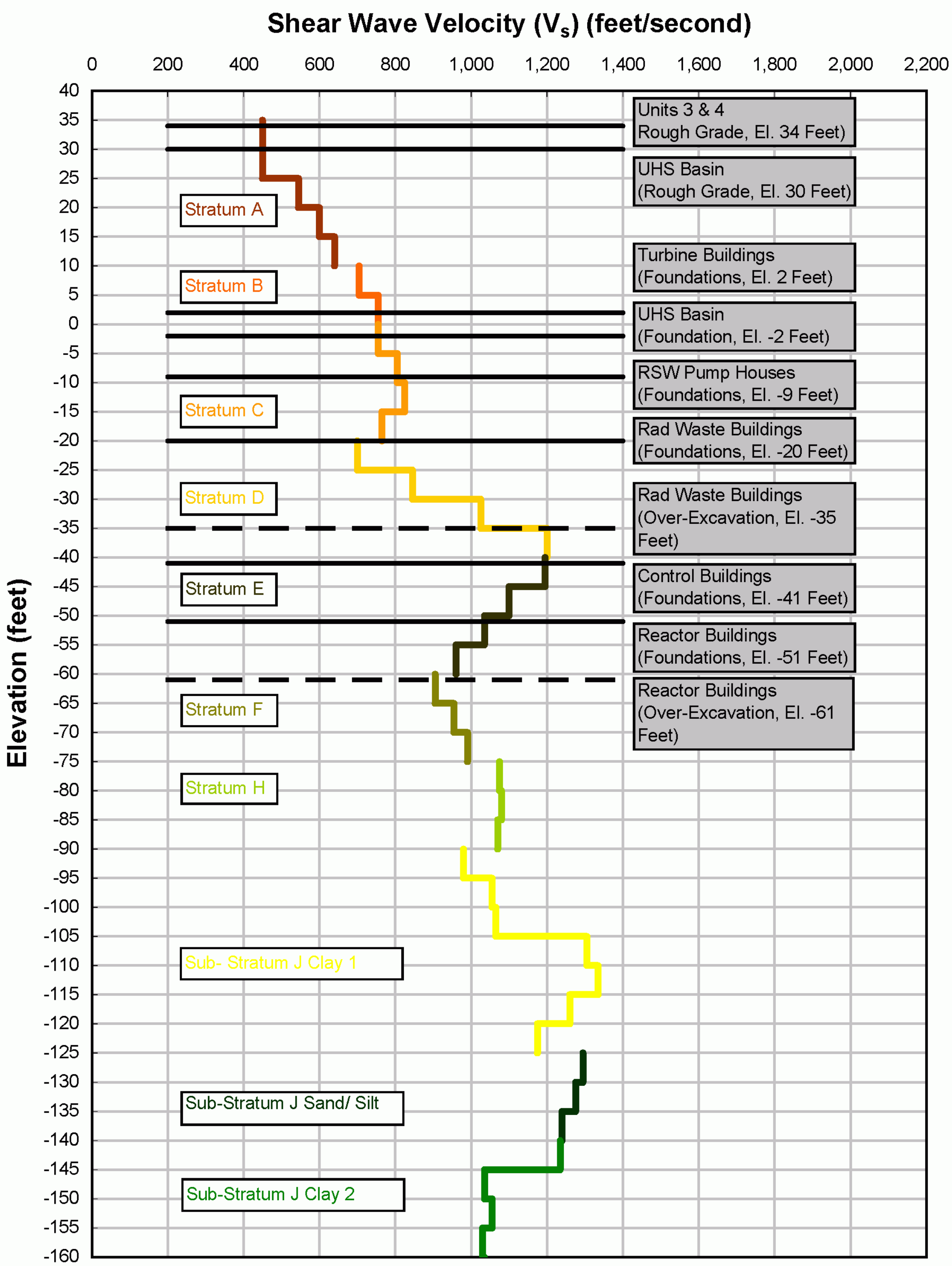


Figure 2.5S.4-45 Shear Wave Velocity Profile - Strata A to J

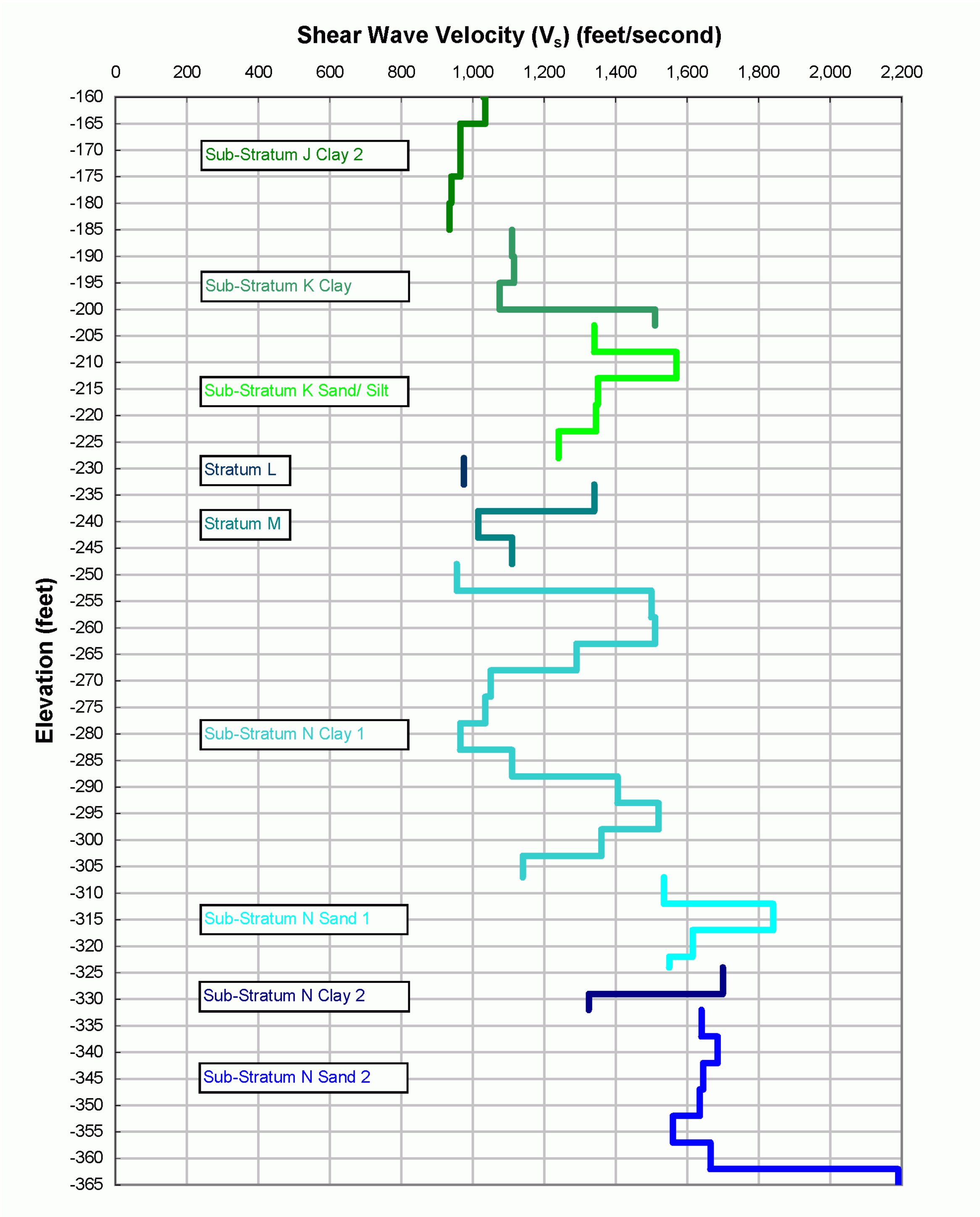


Figure 2.5S.4-46 Shear Wave Velocity Profile - Strata J to N

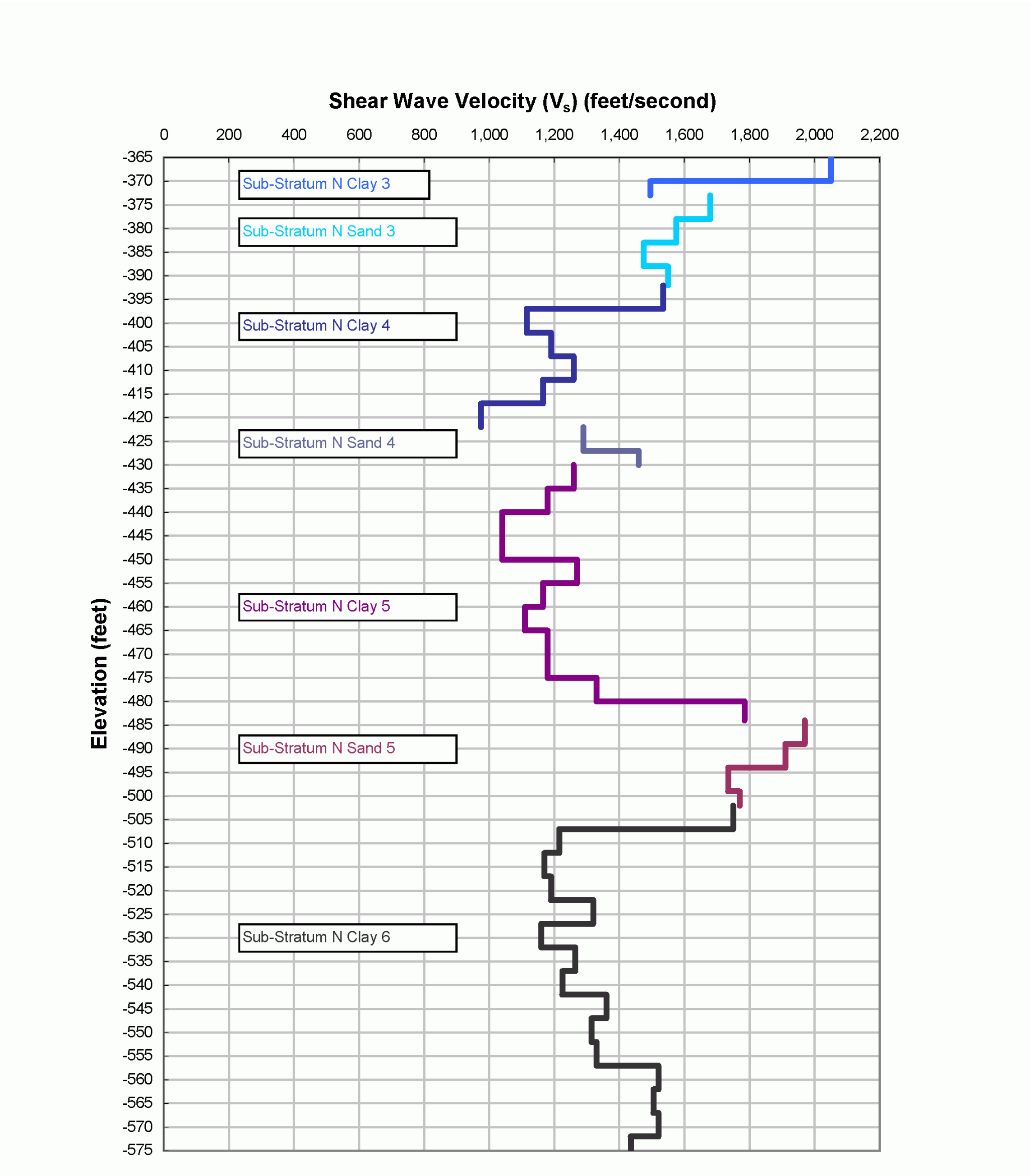


Figure 2.5S.4-47 Shear Wave Velocity Profile - Stratum N to 600 Feet Below Ground Surface

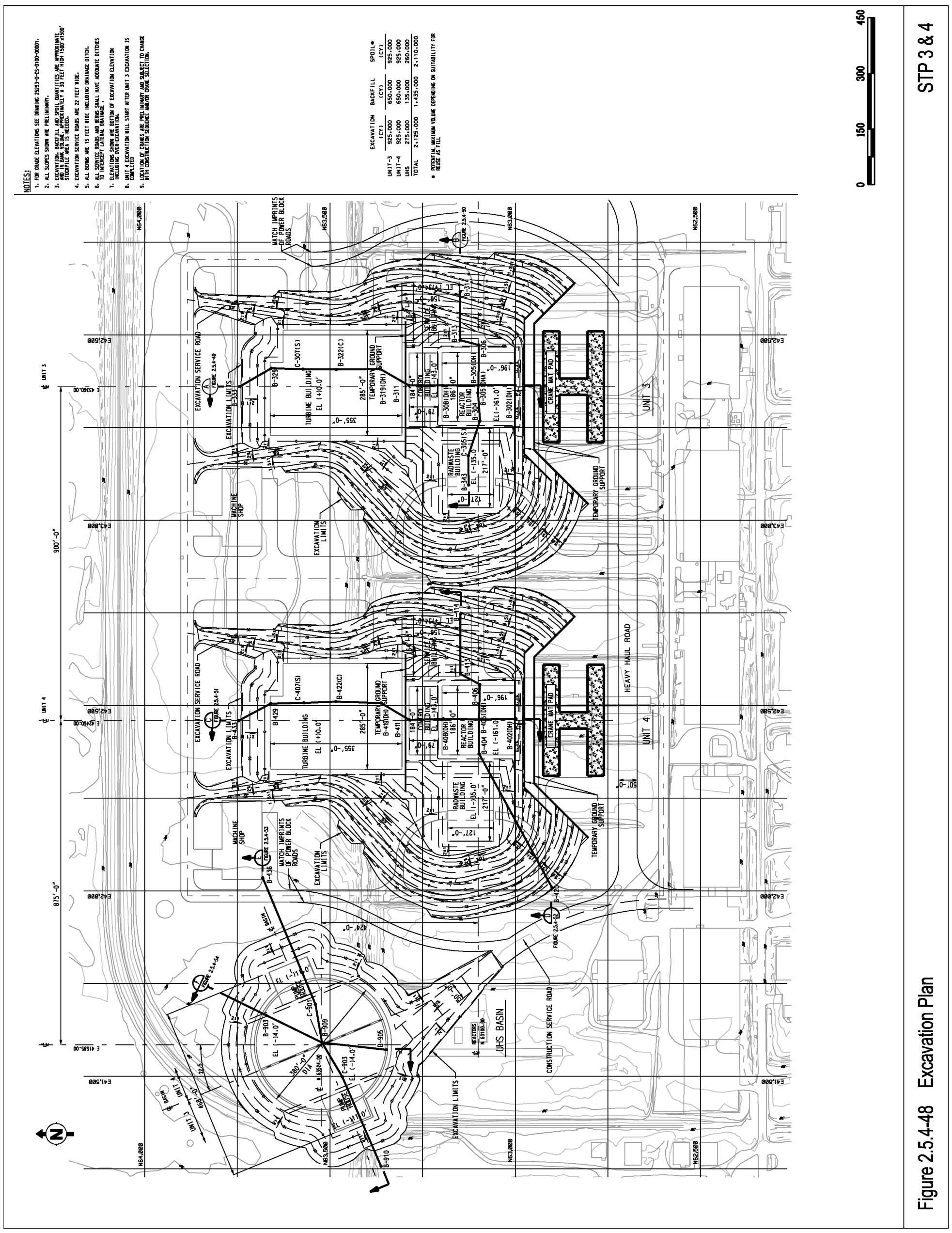


Figure 2.5S.4-48 Excavation Plan

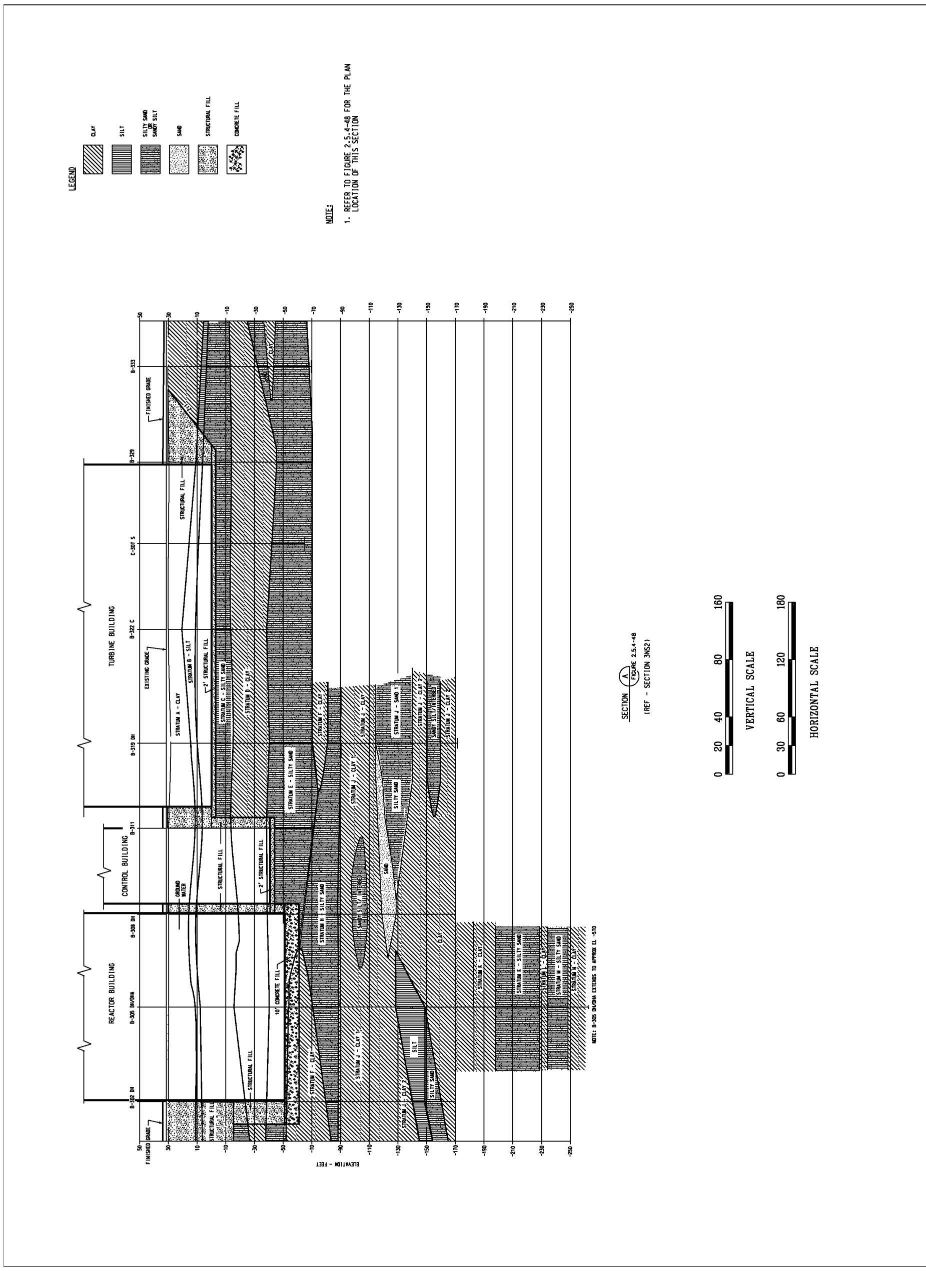


Figure 2.5.4-49 Section A

STP 3 & 4

Figure 2.5S.4-49 Section A

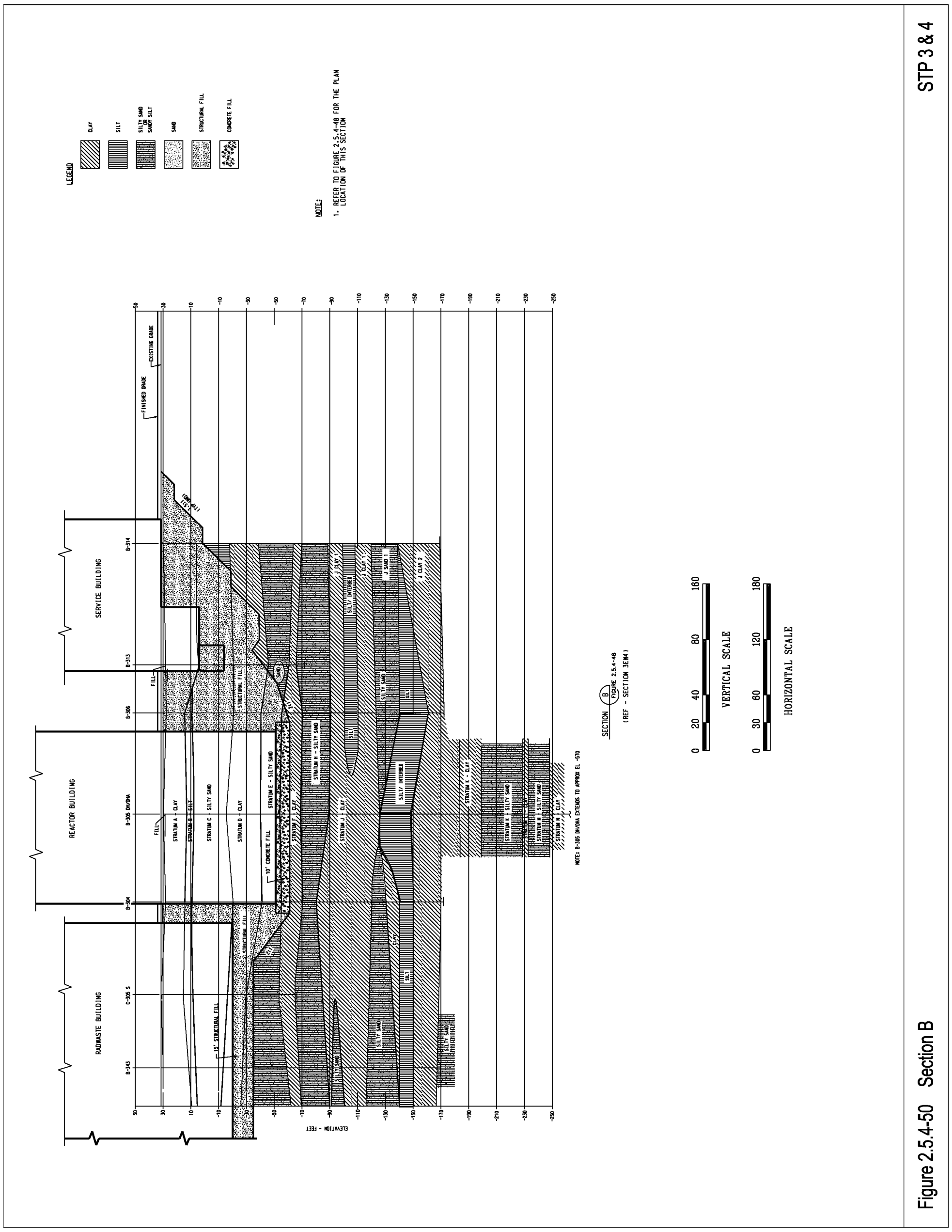


Figure 2.5S.4-50 Section B

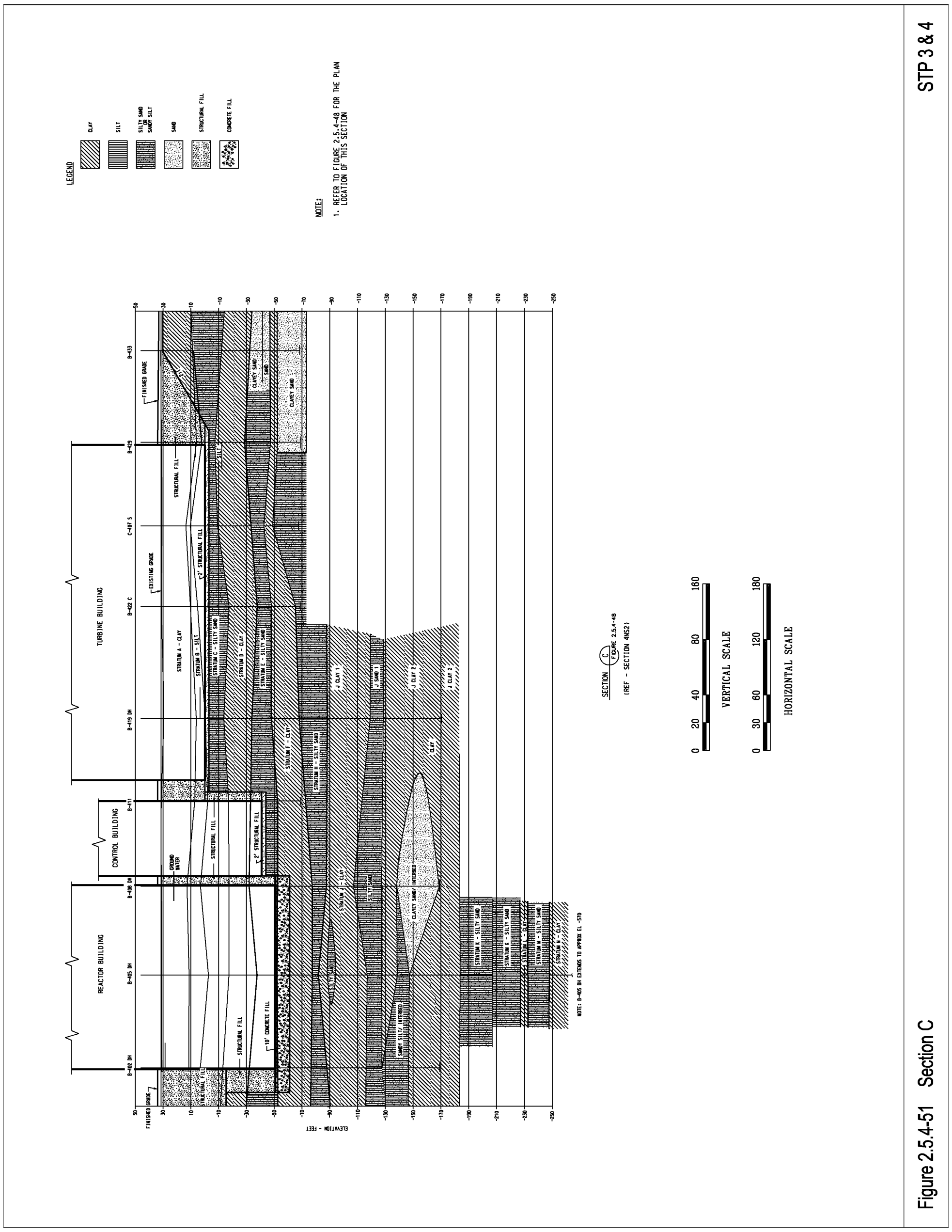


Figure 2.5S.4-51 Section C

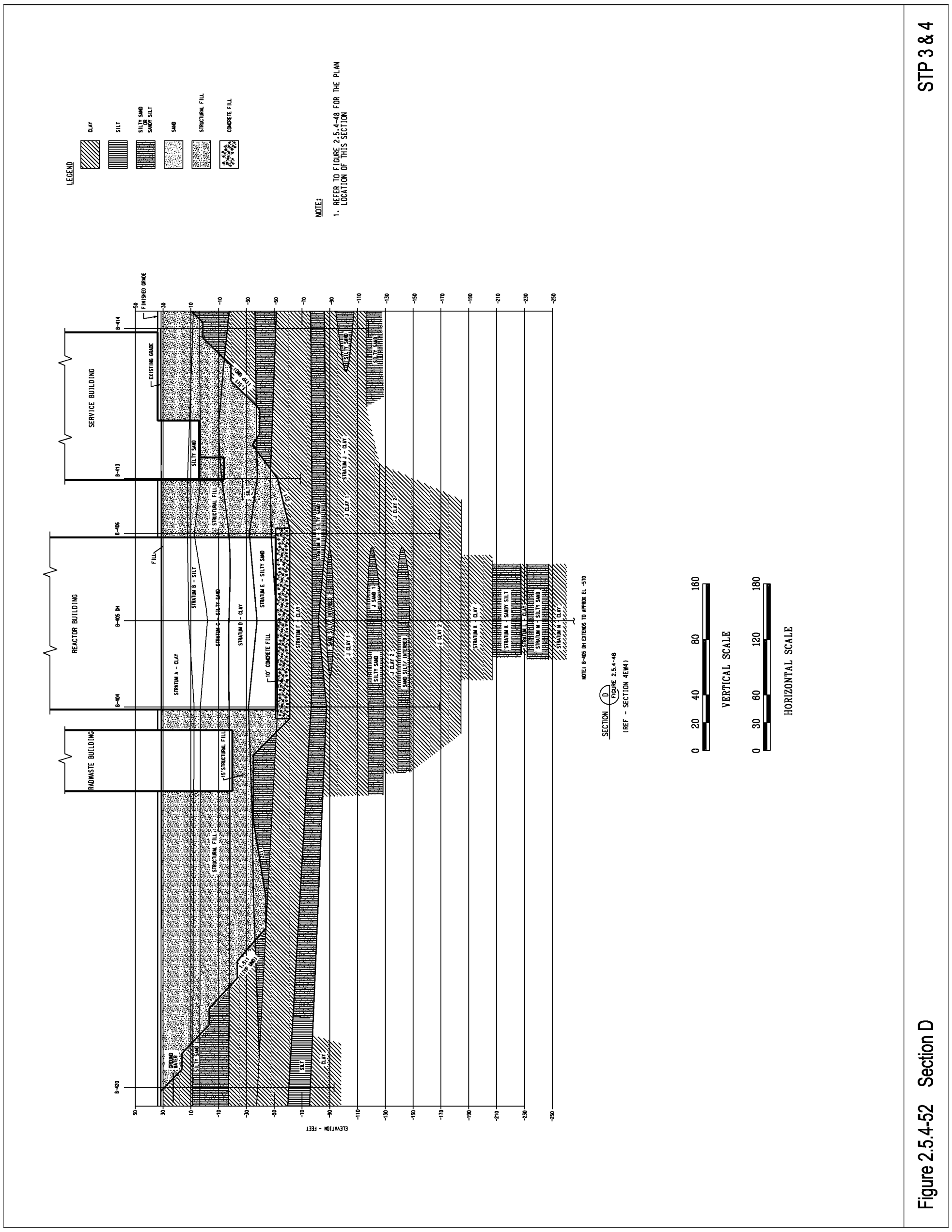


Figure 2.5.4-52 Section D

Figure 2.5S.4-52 Section D

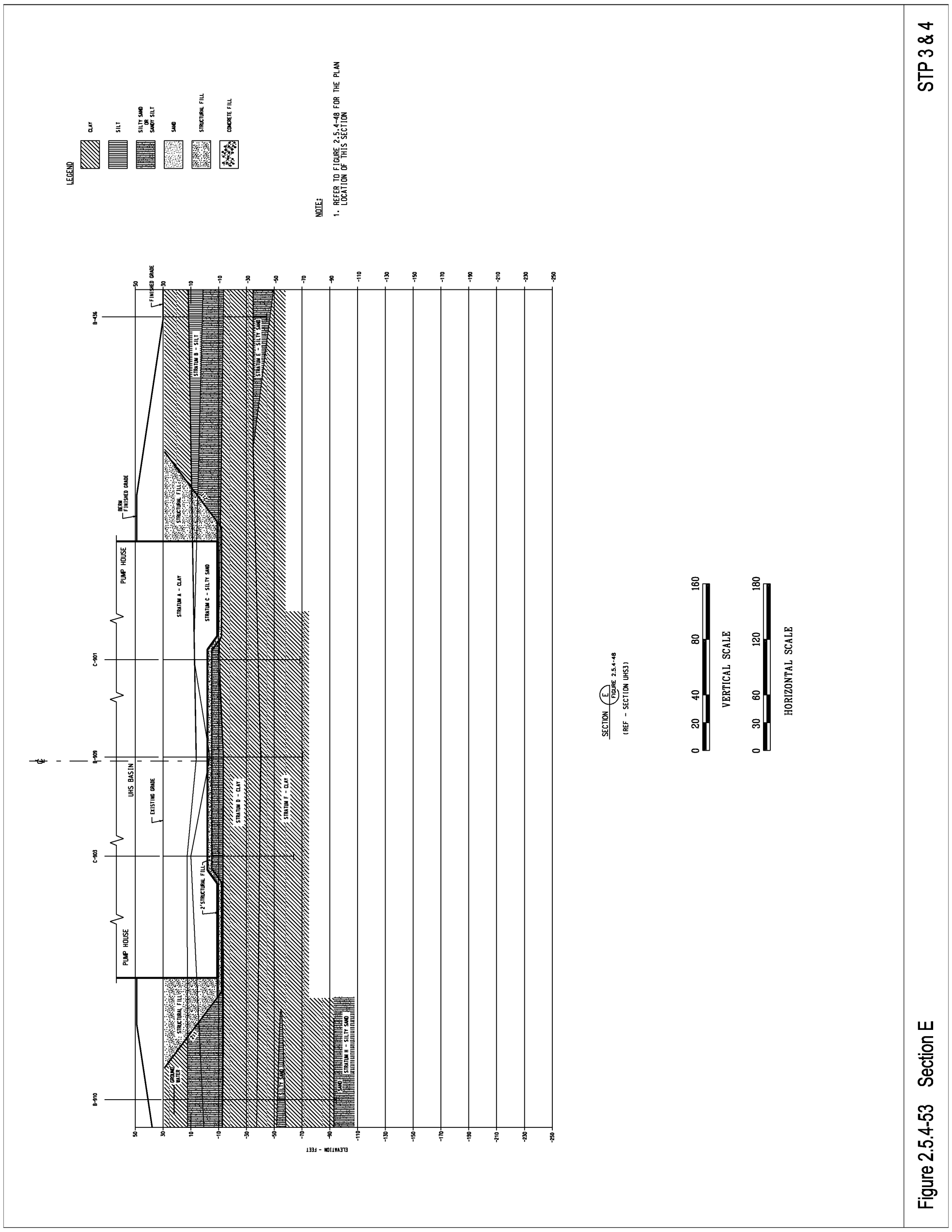


Figure 2.5S.4-53 Section E

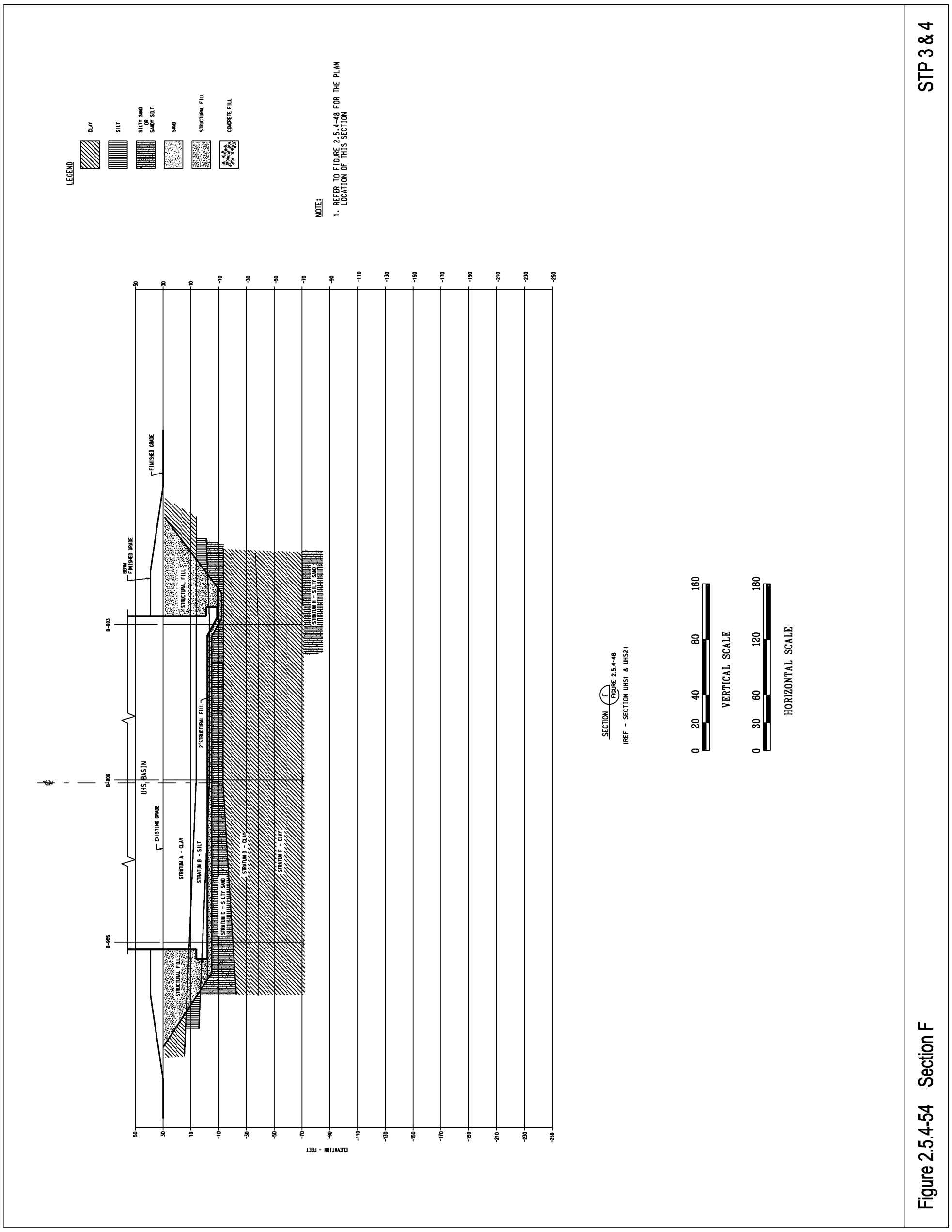


Figure 2.5S.4-54 Section F

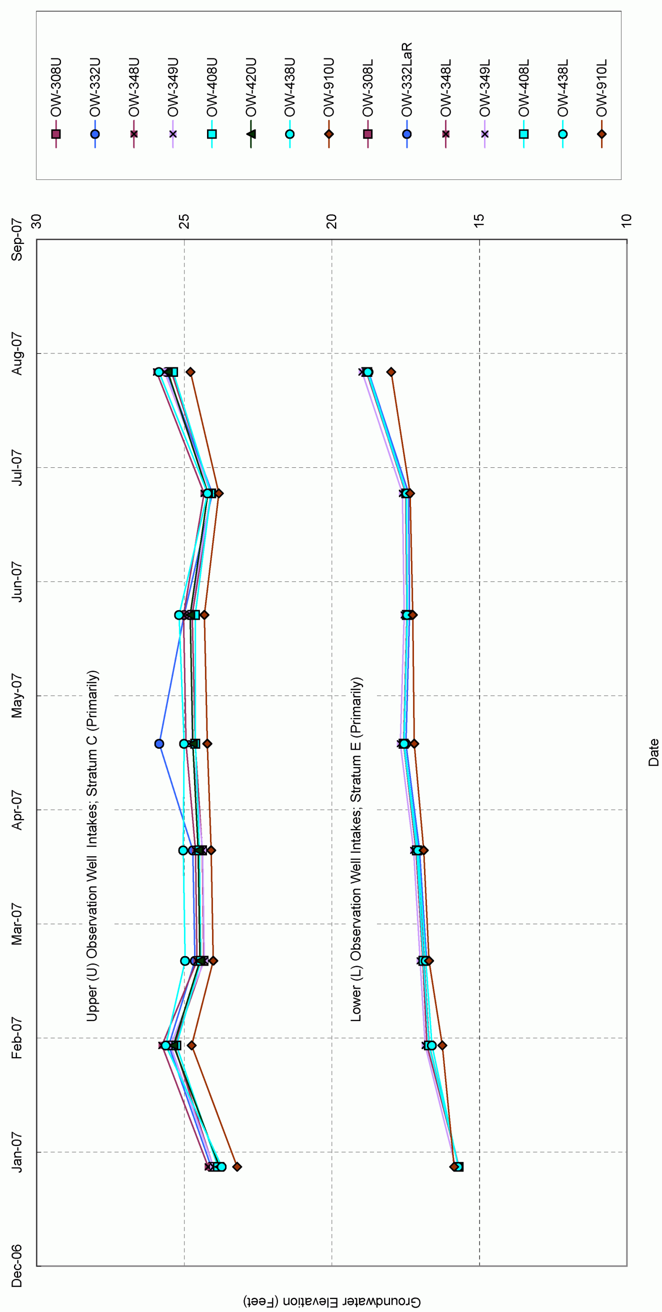


Figure 2.5S.4-55 Measured Groundwater Levels (Inside Power Block)

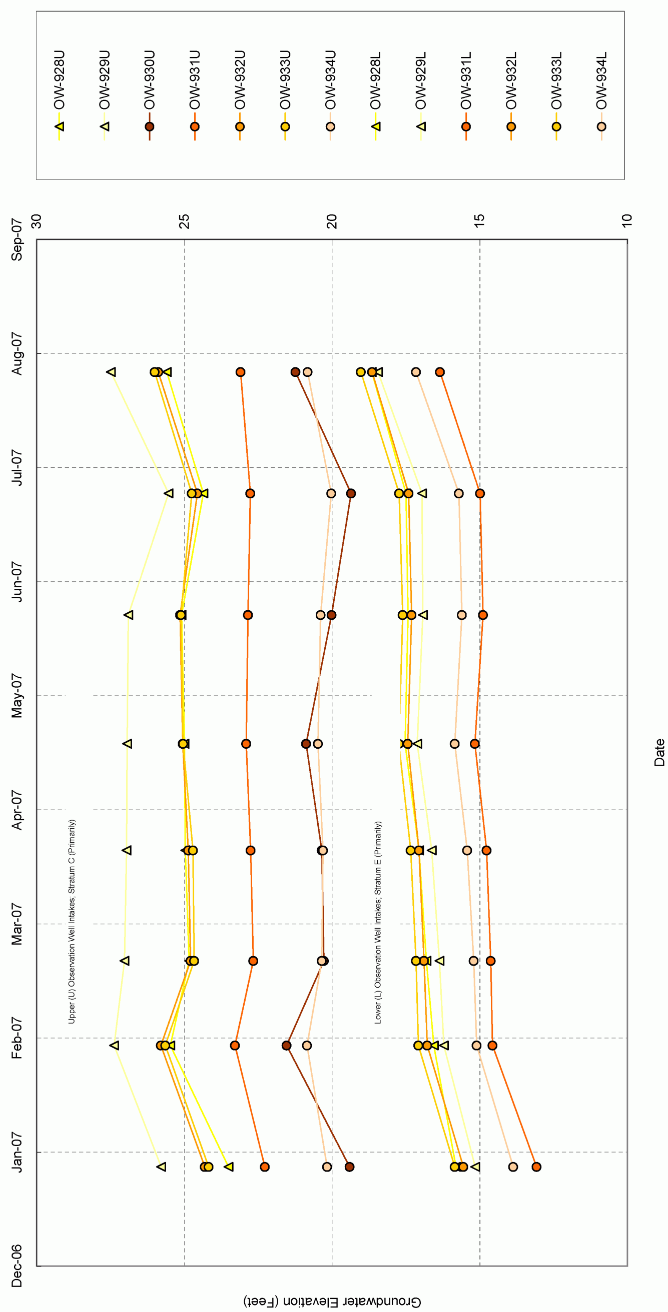


Figure 2.5S.4-56 Measured Groundwater Levels (Outside Power Block)

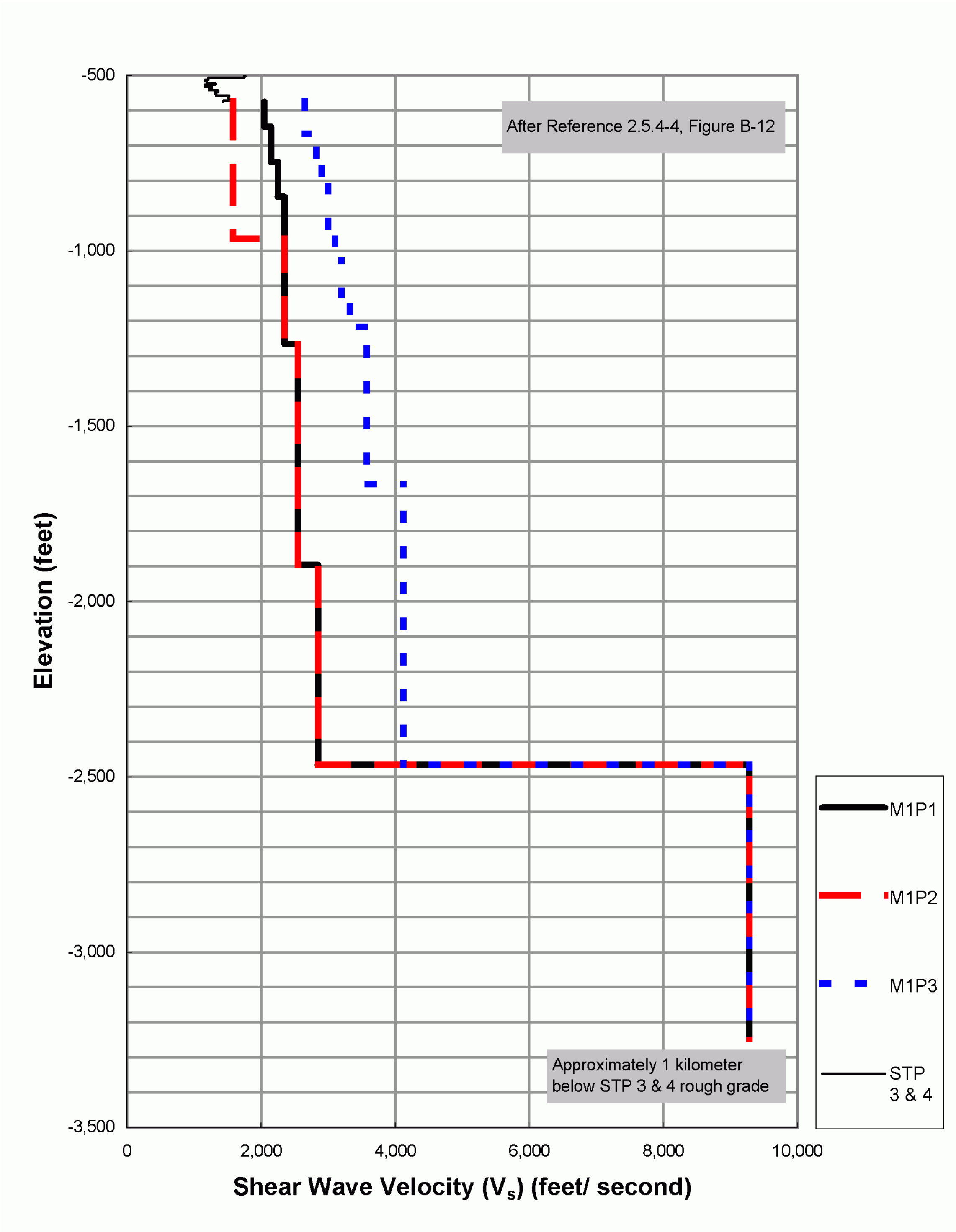


Figure 2.5S.4-57 Deep Shear Wave Velocity Profile for the STP Site

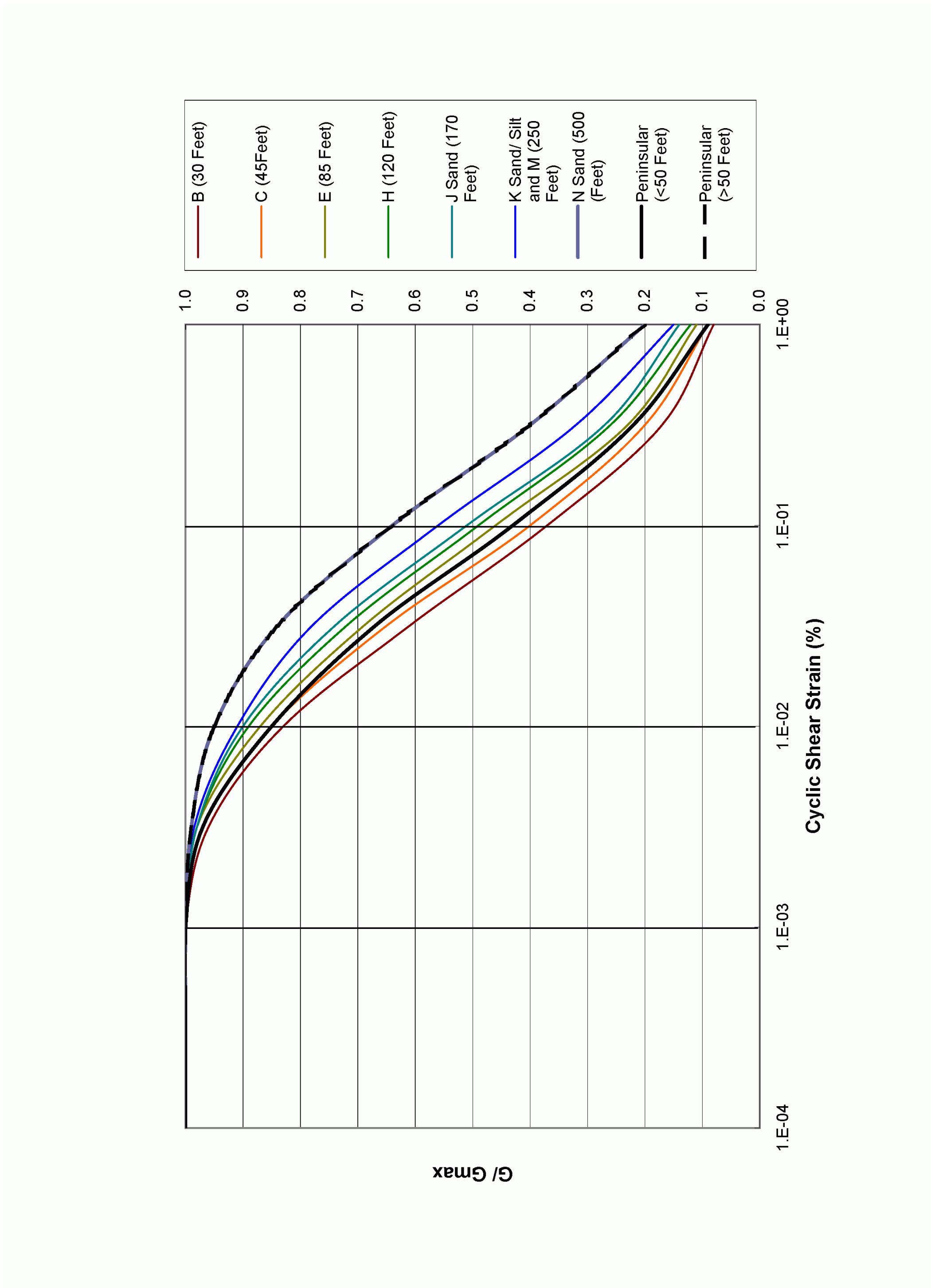


Figure 2.5S.4-58 Selected Shear Modulus Degradation Curves for Cohesionless Soil Strata

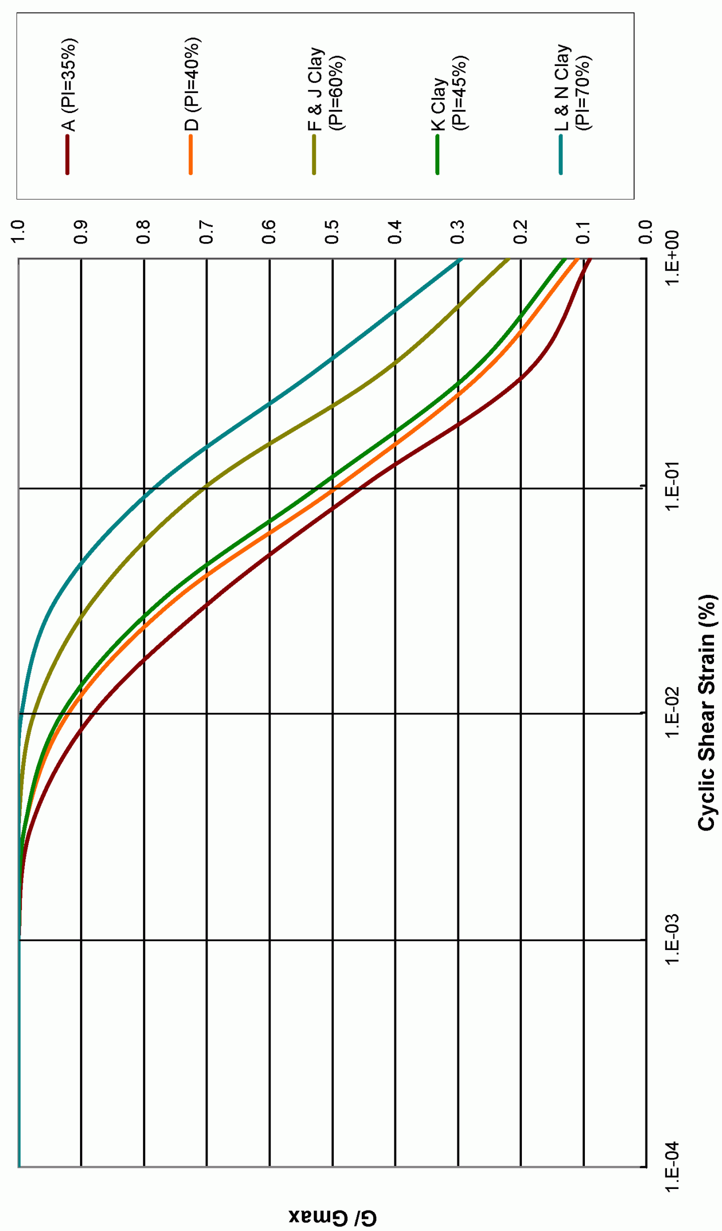


Figure 2.5S.4-59 Selected Shear Modulus Degradation Curves for Cohesive Soil Strata

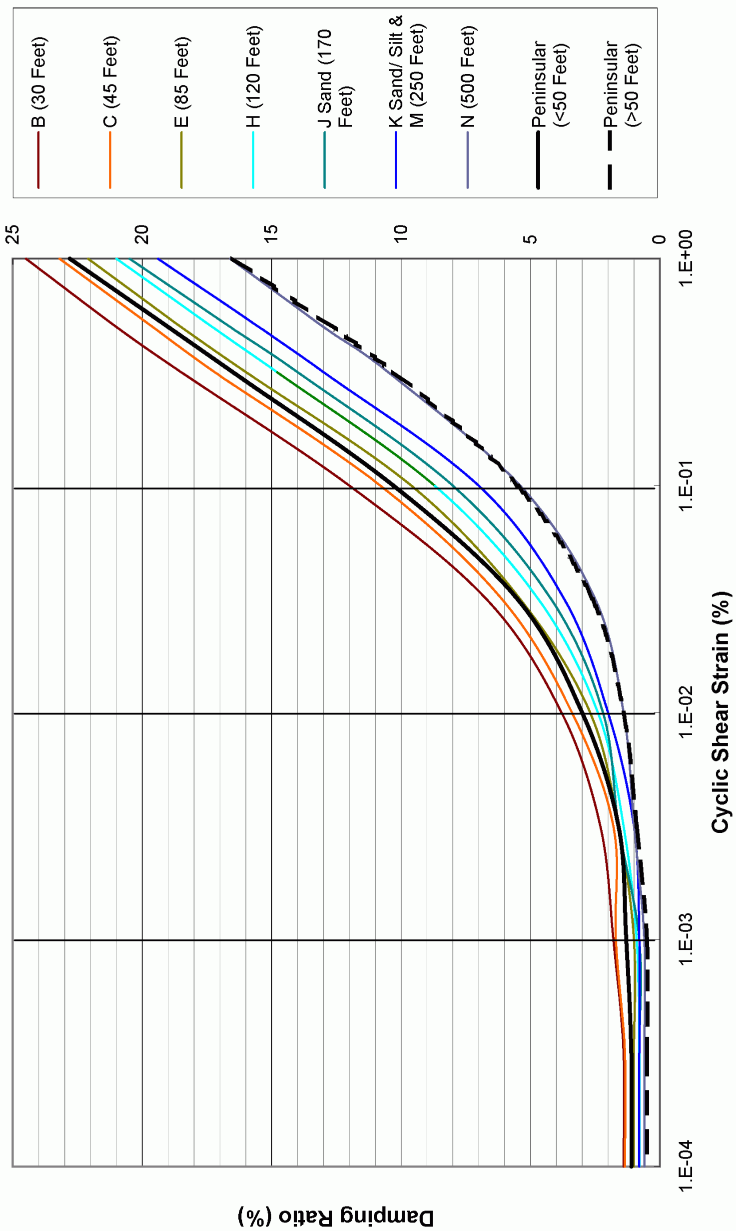


Figure 2.5S.4-60 Selected Damping Ratio Curves for Cohesionless Soil Strata

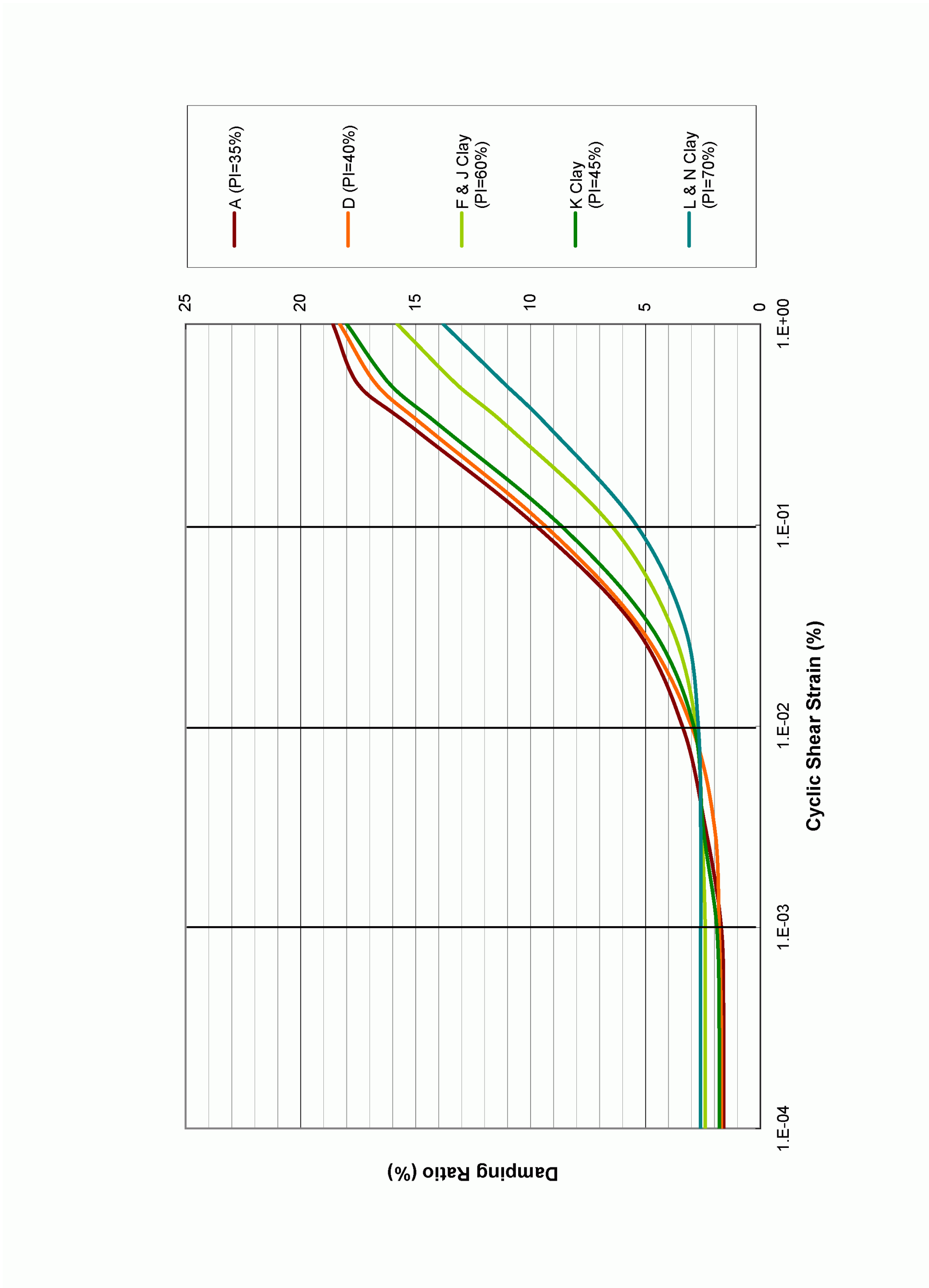
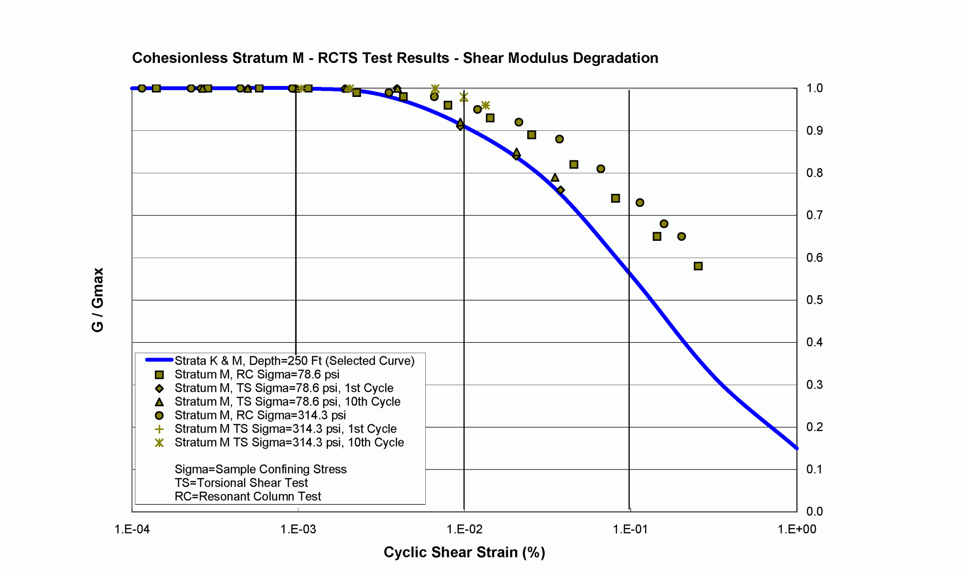


Figure 2.5S.4-61 Selected Damping Ratio Curves for Cohesive Soil Strata



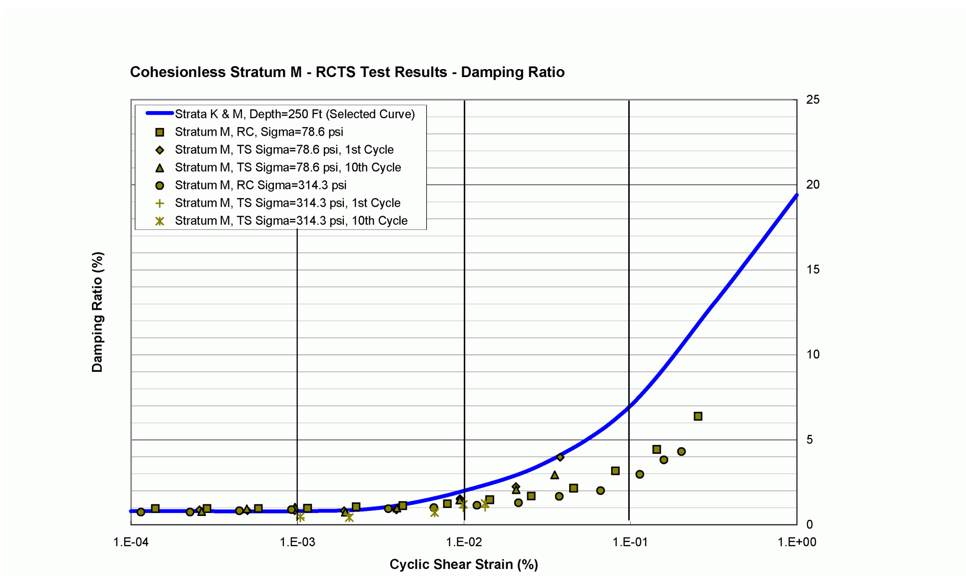
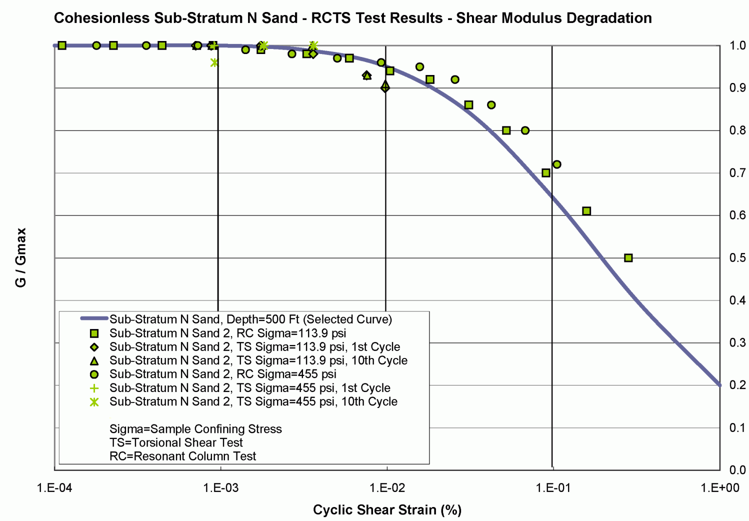


Figure 2.5S.4-62 Comparison of Selected and Measured Shear Modulus Degradation and Damping Ratio Curves; RCTS Test Made on Stratum M (B-305DH/DHA, Sample UD13)



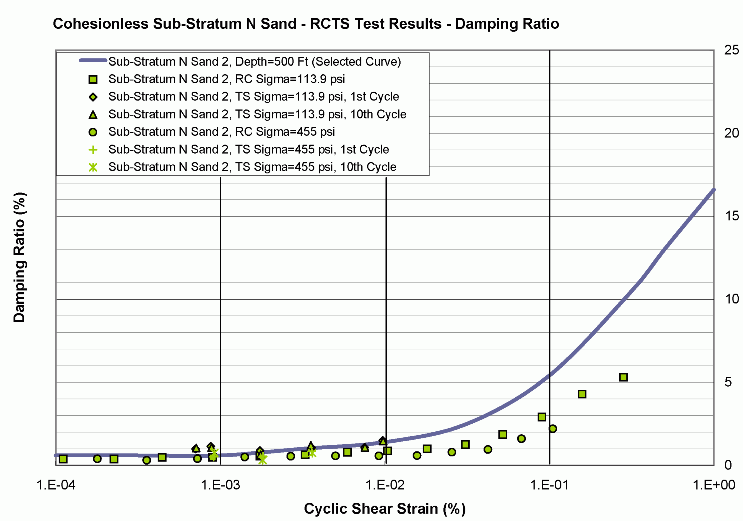
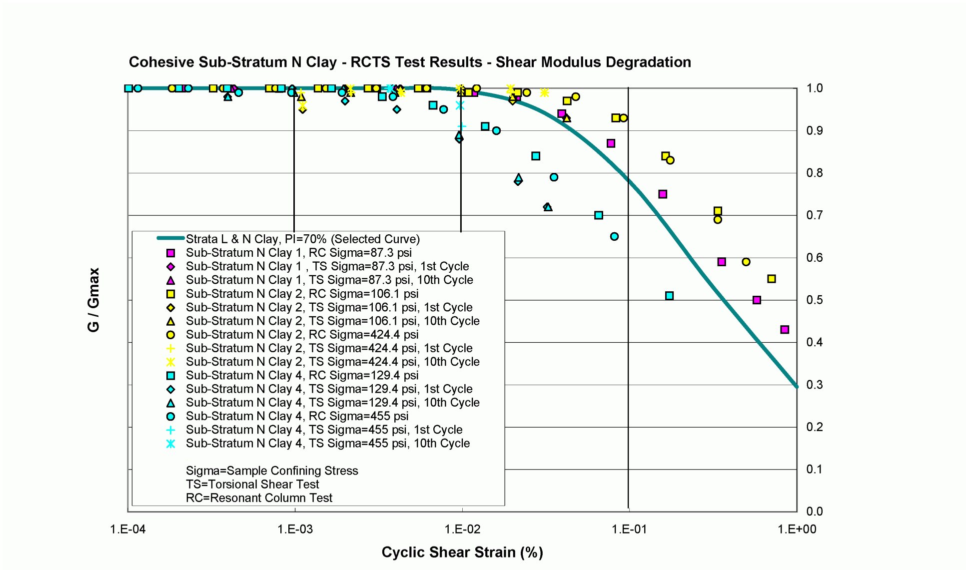


Figure 2.5S.4-63 Comparison of Selected and Measured Shear Modulus Degradation and Damping Ratio Curves; RCTS Test Made on Sub-Stratum N Sand (B-305DH/DHA, Sample UD18)



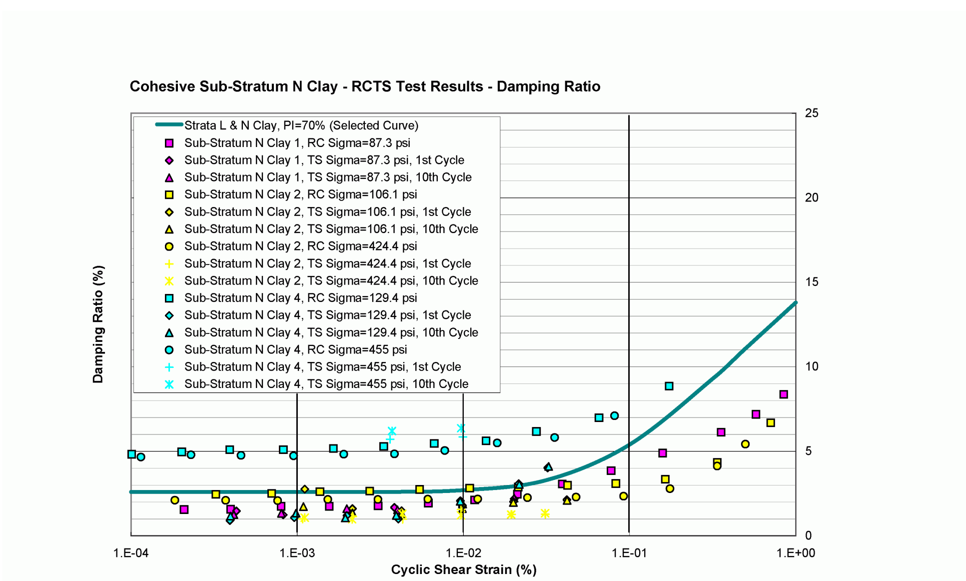


Figure 2.5S.4-64 Comparison of Selected and Measured Shear Modulus Degradation and Damping Ratio Curves; RCTS Tests Made on Sub-Stratum N Clay (B-405DH, Samples UD13, UD 16, and UD19)

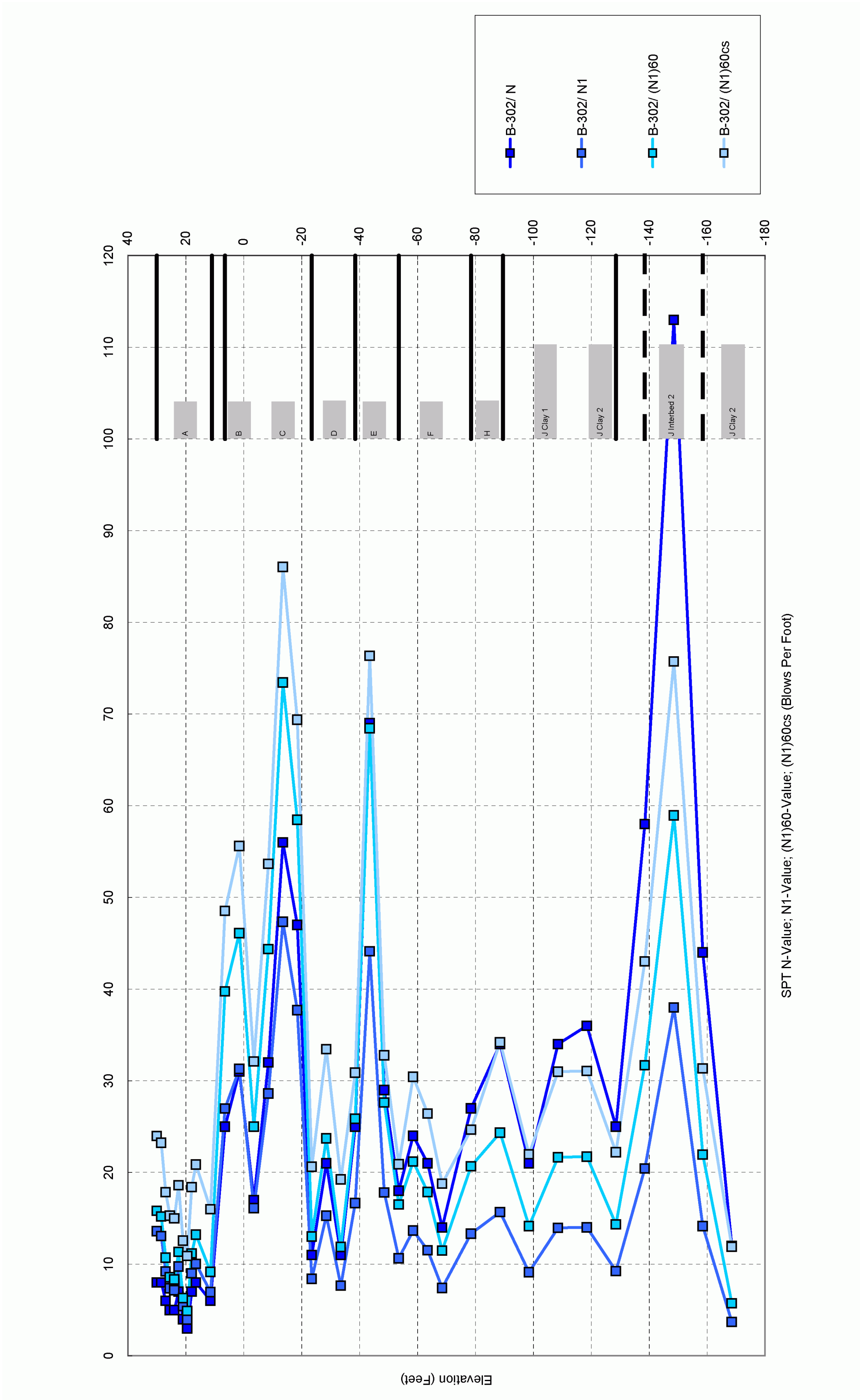


Figure 2.5S.4-65 CPT C-305S (SPT 3; Example; CPT Tip Resistance qc -to- qt -to- qc1n -to- (qc1n)cs)

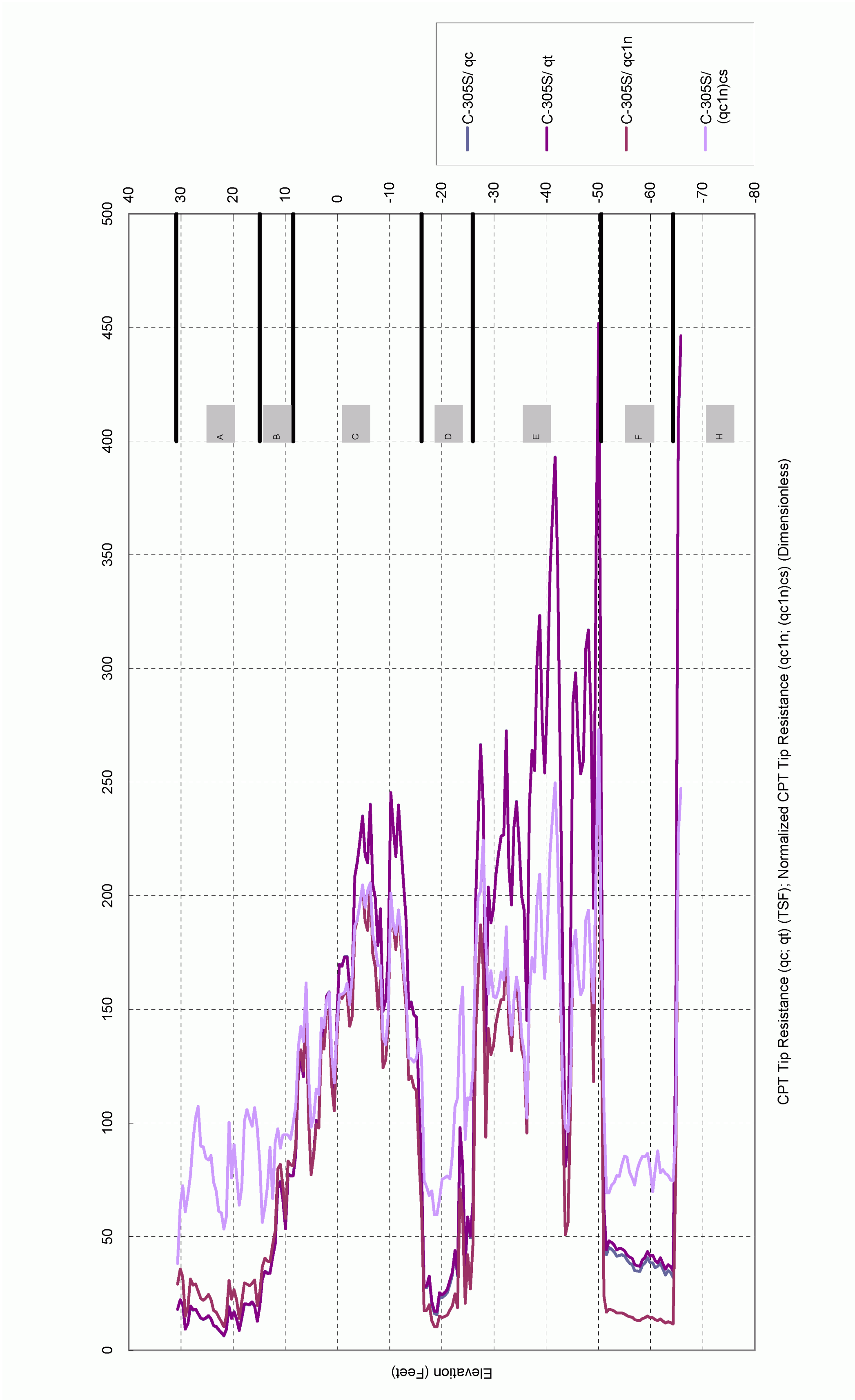


Figure 2.5S.4-66 CPT C-305S (SPT 3; Example; CPT Tip Resistance qc -to- qt -to- qc1n -to- (qc1n)cs)

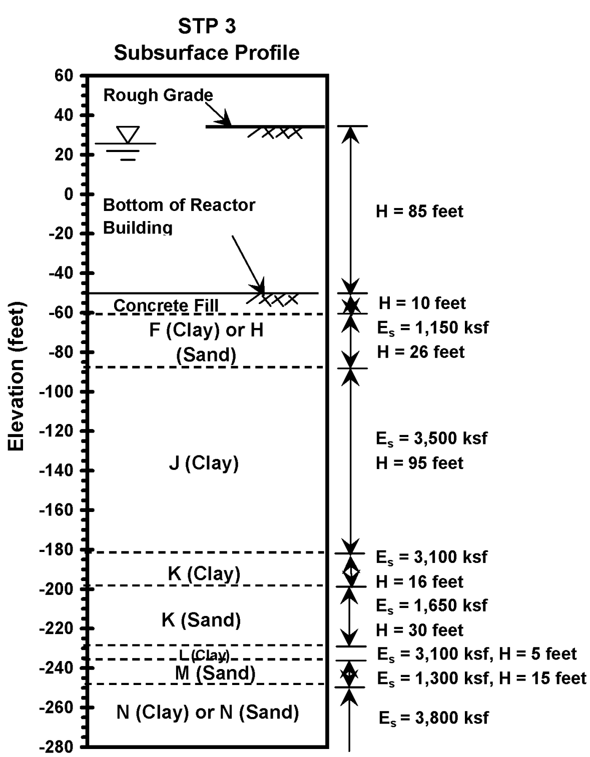


Figure 2.5S.4-67 Adopted Subsurface Profiles for the STP 3 & 4 Reactor Buildings

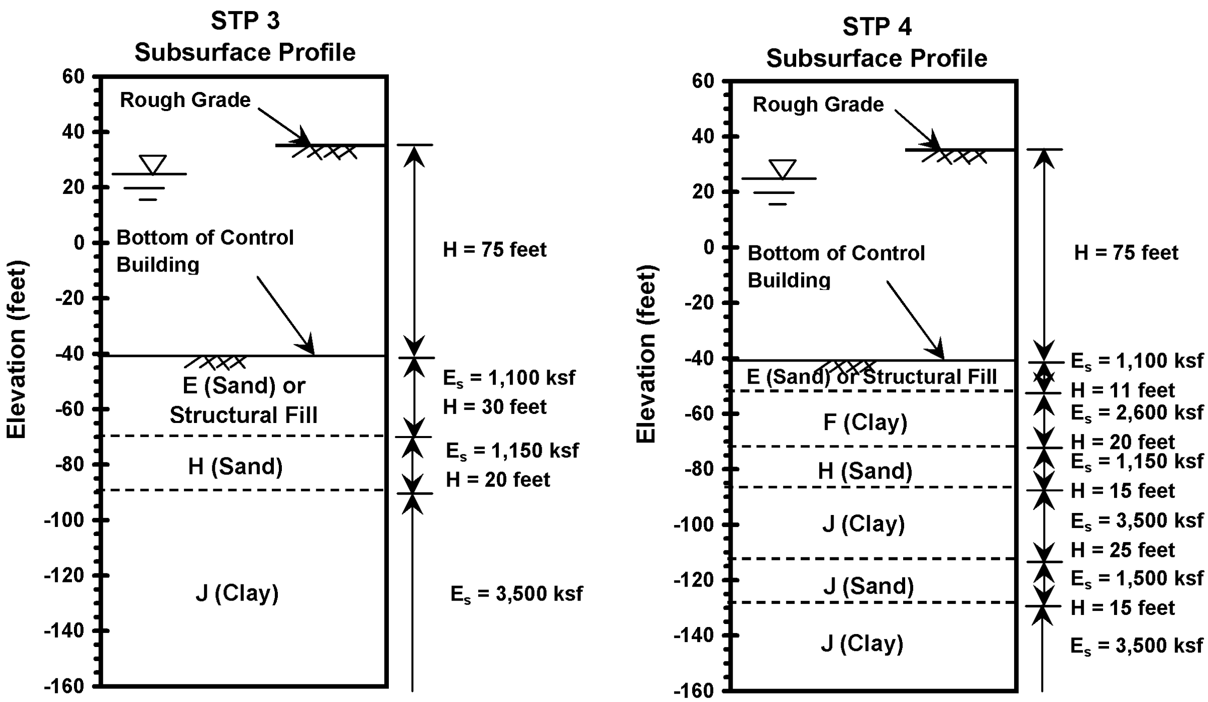


Figure 2.5S.4-68 Adopted Subsurface Profiles for the STP 3 & 4 Control Buildings

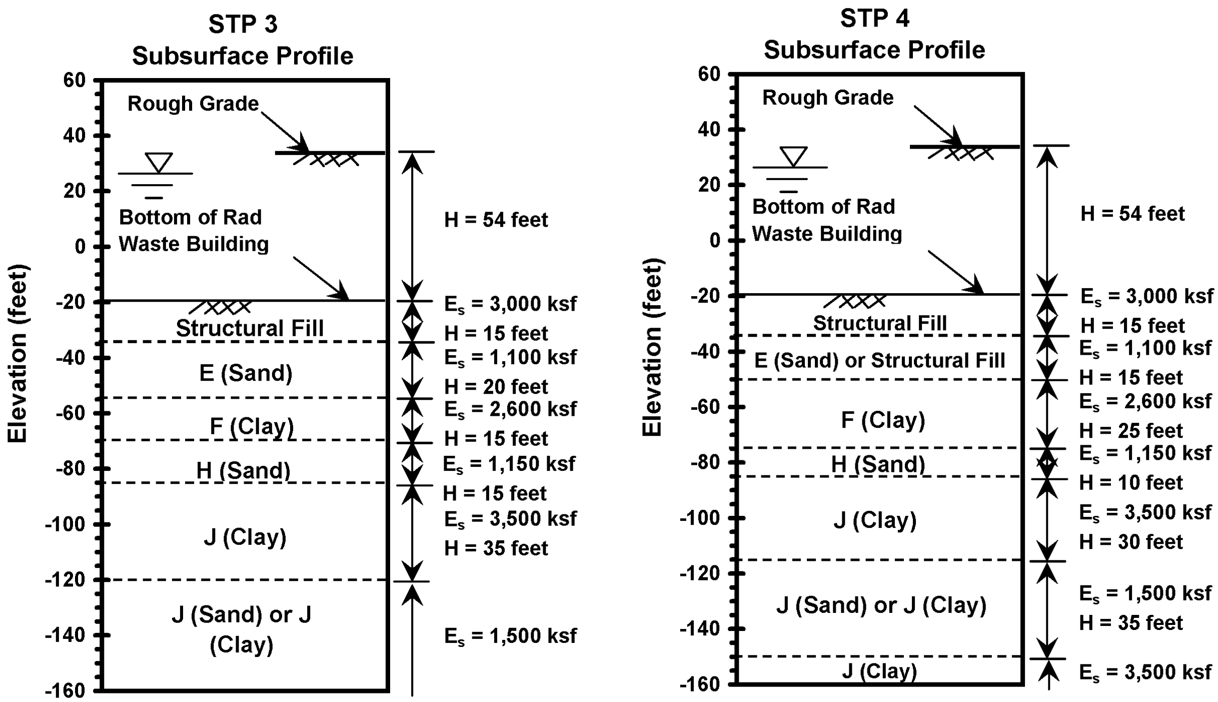


Figure 2.5S.4-69 Adopted Subsurface Profiles for the STP 3 & 4 Radwaste Buildings

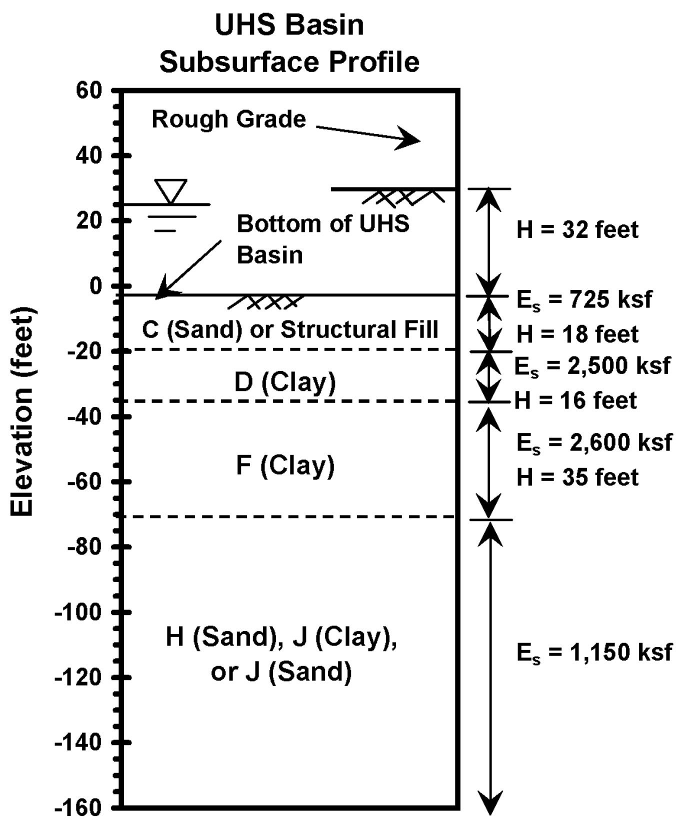
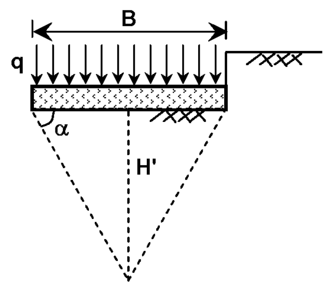


Figure 2.5S.4-70 Adopted Subsurface Profile for the UHS Basin/ RSW Pump Houses STP 3 & 4 Radwaste Buildings



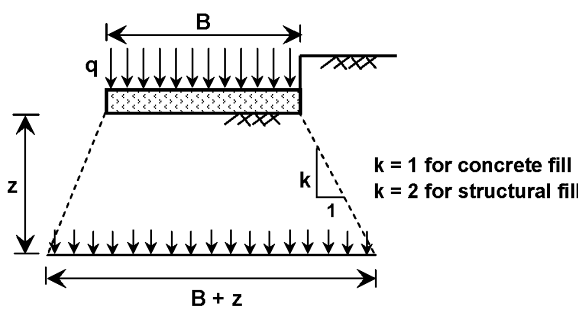


Figure 2.5S.4-71 Nomenclature for Foundation Wedge and Pressure Distribution Diagrams

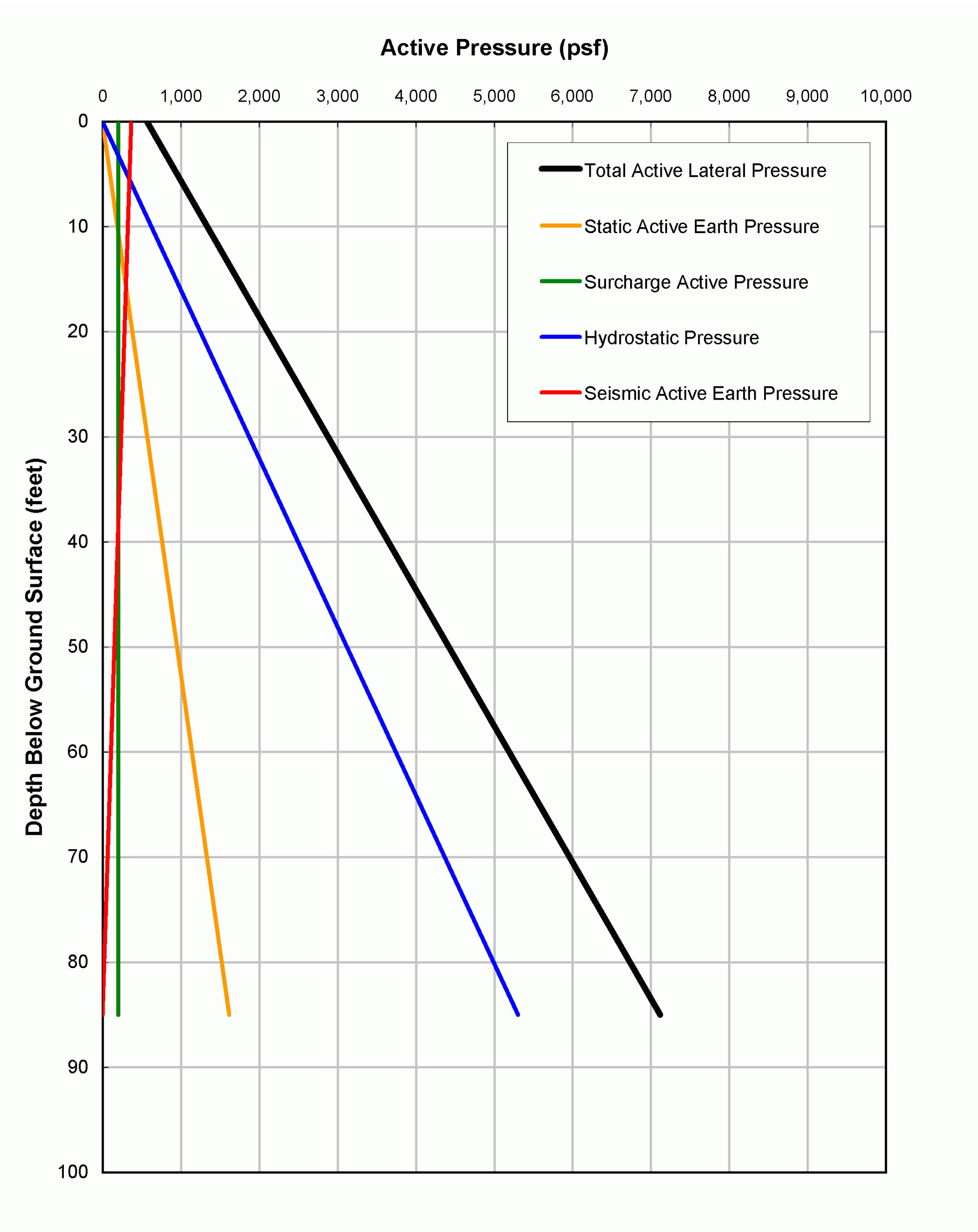


Figure 2.5S.4-72 Sample Active Lateral Earth Pressure Diagrams

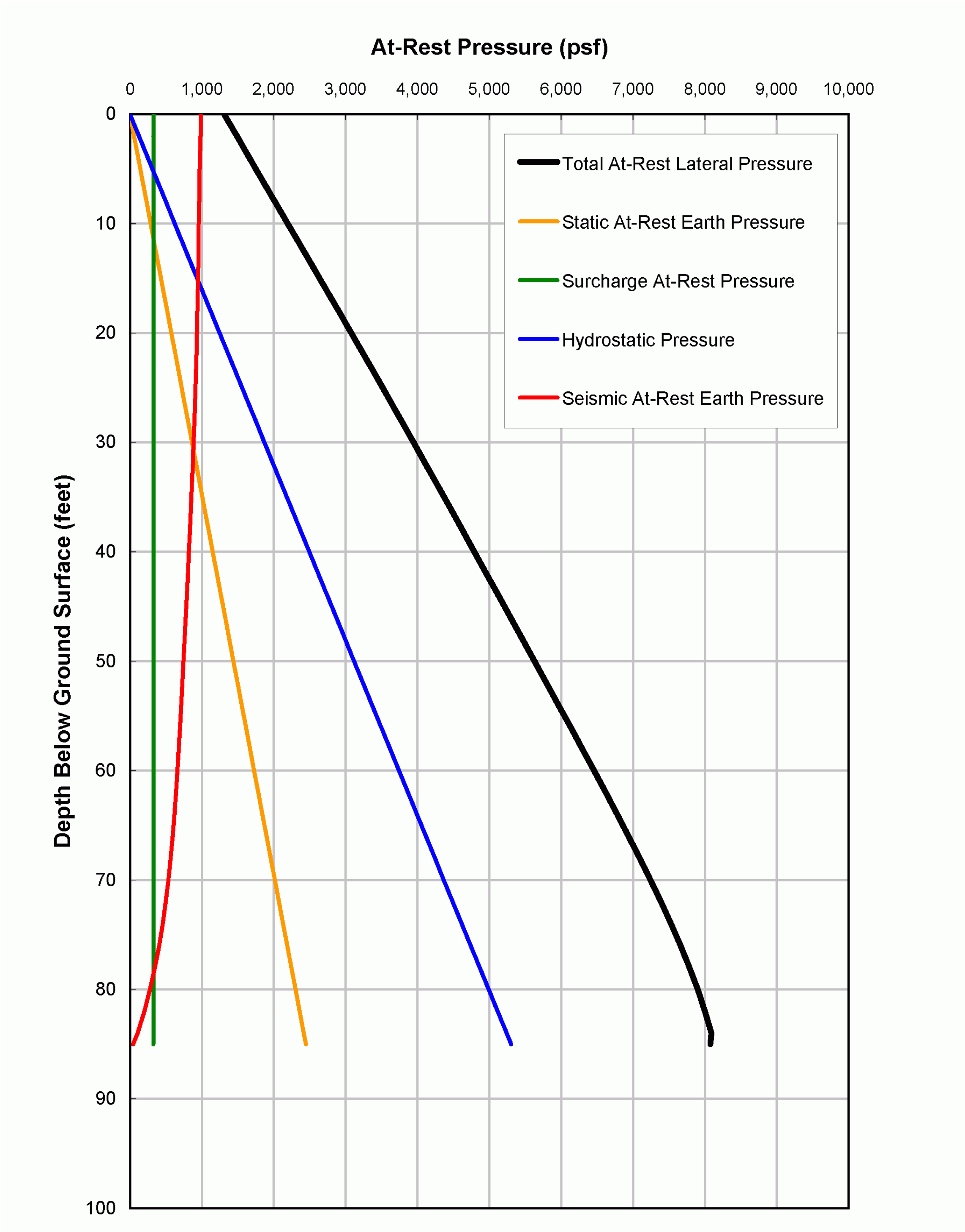


Figure 2.5S.4-73 Sample At-Rest Lateral Earth Pressure Diagrams