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. The Ultimate Heat Sink (UHS) design has been updated from one central location west of the power block to two separate locations just south of each Reactor Building inside the Power Block, as shown on Figure 2.5S.4-2. The data referenced in this Subsection 2.5S.4 is compiled from the initial subsurface investigation for the previous location of the UHS. An additional subsurface investigation was conducted in June of 2008 to obtain data under the new UHS locations.

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.

A subsequent subsurface investigation was performed in the relocated UHS basins, RSW Pump Houses, RSW Tunnels, and Diesel Generator Fuel Oil Storage Vaults (Reference 2.5S.4-2C). These relocated structures are now considered within the Power Block. The subsequent investigation consisted of borings with SPT sampling, undisturbed sampling and laboratory testing. Pressuremeter testing was performed in the F Clay and H Sand strata below the RSW Tunnels. No CPT borings and no shear wave velocity measurements were performed in Reference 2.5S.4-2C.

The scope of the subsequent information is described in subsection 2.5S.4.2.2.2. The information obtained from this subsequent investigation was reviewed and compared with the existing information (Reference 2.5S.4-2, 2.5S.4-2A, and 2.5S.4-2B) from which the geotechnical parameters used for analysis were selected. From this comparison, the field and laboratory information from the subsequent investigation in Reference 2.5S.4-2C is within the range of the earlier subsurface investigations and, as such, the used geotechnical parameters herein are also applicable to the relocated UHS basins, RSW Pump Houses, RSW Tunnels and Diesel Generator Fuel Oil Storage Vaults.

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 than 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) Basins/Reactor Service Water (RSW) areas (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, which will include the new UHS locations. 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 References 2.5S.4-2, 2.5S.4-2A, and 2.5S.4-2B 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-305S, C-309, C-310, C-407S, 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 former UHS Basin/RSW area or in other areas at the site perimeter (i.e., the area “outside the Power Block” as identified on Figure 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, 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. 0.3 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 38 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-443, B-444, B-912, B-913, B-916, B-920, B-929, B-932, B-933, B-940, B-942, B-944, B-945, and B-947. 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 area outside the Power Block 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 area outside the Power Block 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.4-13 and 2.5S.4-15 for the STP 3 area, the STP 4 area, 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.

Thirteen 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 measured N-values were 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 1.20 to 1.65 (e.g., 72% measured energy/60% base line = 1.20 hammer energy correction; 99% measured energy/60% base line = 1.65 hammer energy correction). Additional correction factors for boring diameter (C_B=1.0), for rod length (C_R), and the absence of an SPT sampler liner (C_s=1.2) were also applied (Reference 2.5S.4-5). The result is N_{60} applicable to all soil layers. 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.

For all sandy soil layers, SPT N_{60}-values from each boring were corrected to an effective overburden pressure of one atmosphere (P_{atm}), which is approximately one ton per square foot (tsf). The resulting fully-corrected SPT N-values are commonly termed $(N_1)_{60}$.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):

$$C_n = 2.2/(1.2 + \sigma_v'/P_{atm}) \qquad \text{Equation 2.5S.4-1}$$

where,

C_n = the correction factor, which is multiplied by the SPT N_{60}-value to yield the normalized SPT $(N_1)_{60}$-value, and which varies with depth to a maximum limit of 1.70 and a minimum limit of 0.4 (Reference 2.5S.4-5A).

σ_v' = the effective overburden pressure at the depth of the SPT sample interval in pounds per square foot (psf) and P_{atm} = 2116 psf.

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 in Layers A through D. For

Layers E and deeper, a groundwater elevation of El. 17.0 feet was used based on the observation wells in this zone. Refer to Subsection 2.5S.4.6.1 for additional detail.

A summary of corrected SPT N_{60}-values for all site areas and all soil strata is presented in Table 2.5S.4-6. Also provided is a summary of $(N_1)_{60}$ values for the sandy samples in sandy soil strata. The SPT $(N_1)_{60}$ value is not applicable for cohesive soil samples.

The average SPT (N_{60})-value for Stratum A was 13 blows/foot. An SPT (N_{60})-value of 11 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on SPT (N_{60})-values Stratum A is considered medium 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 311 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 approximately 1 tsf) for Stratum A, was 30 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figure 2.5S.4-16, Figure 2.5S.4-17, and 2.5S.4-19 for the STP 3 area, the STP 4 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 (%)	81	16	30	24
Liquid Limit (%)	44	30	80	56
Plasticity Index (%)	44	11	58	37
Fines Content (%)	11	90	100	96
Unit Weight (pcf)	14	118	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 high plasticity clay with an average fines content (materials passing the No. 200 sieve) of 96%. 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 SPT (N_{60})-values and CPT results. The results of this evaluation are summarized in Table 2.5S.4-9.

Undrained shear strength, s_u, was estimated from an empirical correlation with SPT (N_{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 = SPT (N_{60})-value in blows/foot.

Substituting the selected SPT (N_{60})-value for Stratum A (11 blows/foot), an s_u=1.4 ksf was estimated. Undrained shear strength was also estimated using the CPT data, for cohesive soil behavior types, 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, adjusted for measured pore pressure and area ratio
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 range in S_u results of laboratory undrained shear strength test results on soil samples collected from borings to the range of S_u results computed from CPTs.

Shear strength values calculated in this way from the CPT data indicated an average s_u=1.7 ksf. The CPT-derived values are shown versus elevation on Figure 2.5S.4-23, 2.5S.4-24, 2.5S.4-26, and 2.5S.4-27 for the STP 3 area, the STP 4 area, the area outside the Power Block, and site-wide, respectively. Note that SPT correlations were based on 1235 field measurements, while CPT correlations were based on 1584 field measurements made on cohesive soil behavior types within Stratum A. The results of 12 laboratory unconsolidated undrained (UU) triaxial strength tests and unconfined compression (UNC) strength tests on selected samples indicated an average s_u=1.4 ksf. Laboratory shear strength test results are summarized in Table 2.5S.4-9 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=1.5 ksf was selected for Stratum A, averaged from the SPT N_{60}-value correlations, the CPT correlations, and the laboratory testing results.

Laboratory testing to determine the consolidated-undrained and drained (effective) shearing strengths of Stratum A was not performed. Based on the average plasticity index, Reference 2.5S.4-7 indicates the effective stress friction angle ϕ', would have a value range of $22° \leq \phi'$ 27° for Stratum A in the normally consolidated stress range. A value of ϕ'=20 degrees was selected for Stratum A to represent its effective strength at stress levels above the preconsolidation stress. Shear strength values below the preconsolidation stress range are not available for Stratum A. Stratum A is removed from under all STP 3 area, and STP 4 area (including seismic Category I) structures. Note that Strata A, D, F, and J Clay (discussed in following subsections), all had similar plasticity. Laboratory soil strength test results, including used drained (effective stress) friction angles, 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 6.7 ksf, with an overconsolidation ratio (OCR)=10.5. 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 values for OCR were based on the CPT-s_u results expressed as a ratio of the shear strength, s_u, to the vertical effective stress, σ_v', existing at the depth of the CPT test. Reference 2.5S.4-14A indicates this ratio is 0.31 for clayey soils of the Beaumont formation at OCR=1. The relationship used to estimate OCR from the undrained shear strengths is taken from Reference 2.5S.4-10A as follows:

$$\left(\frac{s_u}{\sigma_v'}\right)_{OCR} = \left(\frac{s_u}{\sigma_v'}\right)_{OCR=1} (OCR)^{0.8} = 0.31(OCR)^{0.8} \quad \text{Equation 2.5S.4-3A}$$

Which is reordered to give:

$$OCR = \left[\frac{\left(\frac{s_u}{\sigma_v'}\right)_{OCR}}{0.31}\right]^{1/0.8} \quad \text{Equation 2.5S.4-3B}$$

where, s_u = undrained shear strength, and
σ_v' = effective overburden pressure at the depth of the CPT test interval.

CPT-derived OCR data for Stratum A indicated an average OCR greater than ten, and were based on 1581 field measurements. CPT-derived OCR data are shown on Figures 2.5S.4-29, 2.5S.4-30, 2.5S.4-32 and 2.5S.4-33 for the STP 3 area, the STP 4 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 of the various soil layers is used herein to represent the soil compressibility for purposes of settlement estimates. This is justified because the soils behave as over consolidated. Settlement estimates later herein are based on the dewatered condition where the water table is kept artificially lowered to 5 ft below the bottom of the excavation throughout the process of loading the foundation areas. Even with this dewatered condition, the effective stresses in the soil layers do not exceed the preconsolidation pressures except the small amounts in limited locations described later. (The compression of the soil layers in these limited locations is modeled using the consolidation test data as described in Equation 2.5S.4-29). When construction dewatering ends and the water table rises, buoyancy will reduce the effective stresses in all soil layers below the final water table and thus the final effective stresses will be less and will not exceed the preconsolidation stress. This supports the use of the elastic modulus to model the soil for settlement purposes.

The stress-based equation for the modulus ratio does not associate with the criterion of one-half of the pre-consolidation pressure. The stress-based equation gives essentially the same modulus ratio as the equation being discussed. Therefore, it is reasonable to use either equation in spite of the fact that the loading exceeds one-half of the pre-consolidation pressure at times during the loading. When the site is re-watered, the stresses become less than the preconsolidation pressure due to buoyancy.

The elastic modulus (E) for fine-grained cohesive soils was evaluated using the following relationship (Reference 2.5S.4-55):

$E = (100 \text{ to } 500)\, s_u(OCR)^{0.5}$, (use 300) Equation 2.5S.4-4A

where, s_u = undrained shear strength.

$E = (500 \text{ to } 1500)\, s_u(OCR)^{0.5}$, (use 1000) Equation 2.5S.4-4B

where, s_u = undrained shear strength.

Equation 2.5S.4-4A is indicated by Reference 2.5S.4-55 for use if the plasticity index (PI) is greater than 30 and for clay and silt. Strata A, D, and F will be characterized by Equation 2.5S.4-4A.

Equation 2.5S.4-4B is indicated by Reference 2.5S.4-55 for use if the plasticity index is less than 30 or for silty or sandy clay. Strata J Clay, K Clay, L and N Clay are considered to be somewhat more sandy clays and will be characterized by Equation 2.5S.4-4B even though they have average plasticity index values greater than 30.

Using Equation 2.5S.4-4A and substituting the previously established s_u values for Stratum A soils (s_u=1.5 ksf, OCR=7), an E=1190 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

$G_{0.0001\%} / G_{.375\%} = 21/(PI)^{0.5}$ 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

Equation 2.5S.4-7 is a "strain-based" approach to determining the large strain static modulus from the modulus at small strains. Equation 2.5S.4-7 gives modulus ratios (large strain to small strain) of 0.28 to 0.34 for the clay layers, which have PI values between 35 and 50. Note that later herein a "stress-based" approach (Equation 2.5S.4-14) that incorporates the factor of safety with respect to the ultimate stress is applied to the sand layers. If Equation 2.5S.4-14 were applied to the clay layers in lieu of strain-based Equation 2.5S.4-7, the modulus ratio would be 0.30. The velocity-based modulus thus determined for the clay layers would be about the same value. Thus it is determined that velocity-based modulus values for the clay layers could be determined using either Equation 2.5S.4-7 or Equation 2.5S.4-14. Because of the agreement with Equation 2.5S.4-14, it is not considered necessary to correlate Equation 2.5S.4-7 with the actual strain computed in each clay layer. Note that for the layers N Sand and N Clay and deeper, the incremental stress levels applied from the construction are lower, the factor of safety is higher, and a modulus ratio equal to 0.5 is considered appropriate.

Note that the empirically-based modulus values to accompany the velocity-based modulus values are computed for clay layers using Equations 2.5S.4-4A and 2.5S.4-4B. For sand layers, the empirically-based modulus values come from Equation 2.5S.4-13. The empirically-based modulus values and the velocity-based modulus values are summarized in Table 2.5S.4-14. The values in Table 2.5S.4-14 indicate the empirically-based modulus values are compatible with the velocity-based values.

The small strain modulus (Equations 2.5S.4-5 and -6) determined from the measurement of wave velocities in-situ is the highest achievable stiffness. Because it is measured in-situ at non-destructive strains it is considered to be a "benchmark". Because of these factors, the velocity-derived results for modulus are assigned a weighting of (2:1) compared to the modulus estimate from undrained shear strength (S_u) or SPT values (N).

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=1110 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=1135 ksf was selected for Stratum A. This compares with a value range of 300 ksf ≤ E_s ≤ 1050 ksf for medium stiff clay in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

Reference 2.5S.4-14B recommends values of the effective stress (drained) Poisson's ratio. For clays, Reference 2.5S.4-14B recommends for soft clay, μ_d=0.40; for medium stiff clay, μ_d=0.30; for stiff overconsolidated clay, μ_d=0.15. For sands, Reference 2.5S.4-14B recommends an average value of μ_d=0.30.

The modulus value for the sand strata and sub-strata requires no adjustment to represent the drained modulus. The modulus value for the clay strata and sub-strata is adjusted for drained, effective stress (long term) loading using the following equation from Reference 2.5S.4-14B:

$$E = \frac{3(E_d)}{2(1+\mu_d)}$$

Which, after reordering becomes:

$$E_d = E\left[\frac{2(1+\mu_d)}{3}\right] \qquad \text{Equation 2.5S.4-8A}$$

where
E = undrained elastic modulus of clay as estimated from the weighted shear strength and seismic data;
E_d = drained (long term) elastic modulus of clay;
μ_d = drained Poisson's ratio for long term loading of clay.

For Stratum A, μ_d=0.30 and E_d=985 ksf for long term loading using Equation 2.5S.4-8A. The selected values of E_d for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) for clayey soils was related to E_d by :

$$G = E_d/(2\,[1 + \mu_d]) \qquad \text{Equation 2.5S.4-8}$$

where, E_d = static (or large strain) elastic modulus for effective stress conditions
μ_d = Poisson’s ratio for effective stress conditions

Using μ=0.30 for Stratum A, a G=369 ksf was estimated using Equation 2.5S.4-8 and the value of E_d noted above for Stratum A. A value of G=370 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_{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 and 2.5S.4-55):

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

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

$K_{0,NC} = 1 - \sin (\phi')$ Equation 2.5S.4-11A

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

$K_{0,OCR} = K_{0,NC} (OCR)^n$ Equation 2.5S.4-11B

For overconsolidated sand, Reference 2.5S.4-55 gives n=sin(Φ'). For overconsolidated clay, Reference 2.5S.4-55 gives n=0.39 for PI=40. Reference 2.5S.4-14A gives the following equation for $K_{0,OCR}$ for the overconsolidated clayey soil of the Beaumont formation for OCR values between 2 and 10:

$$K_{0,\,OCR} = (OCR)^{0.5}\left(0.4 + 0.15\sin\left(\frac{\pi(PI)}{120}\right)\right) \quad \text{Equation 2.5S.4-11C}$$

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; $K_{0,NC}$=0.66. For OCR=7, $K_{0,OCR}$=1.41 by Equation 2.5S.4-11B and $K_{0,OCR}$=1.40 by Equation 2.5S.4-11C. Values selected for engineering purposes were then: K_a=0.5; K_p=2.0; $K_{0,NC}$=0.7; and $K_{0,OCR}$=1.4.

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 and STP 4 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, B-929, and 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 descriptively, 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. -8 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 8 blows/foot. In the STP 4 area, uncorrected SPT N-values in Stratum B ranged from 2 blows/foot to 40 blows/foot, with an average uncorrected SPT N-value of 12 blows/foot. In the area outside the Power Block, uncorrected SPT N-values in Stratum B ranged from 3 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 area outside the Power Block, 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.4-13 and 2.5S.4-15 for the STP 3 area, the STP 4 area, the area outside the Power Block, respectively. The site-wide average uncorrected SPT N-value was 9 blows/foot for Stratum B.

The uncorrected SPT N-values from each boring were corrected to an energy transfer ratio of 60 percent by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections for rod length and sampler (C_s=1.2) leading to values of N_{60}. A summary of SPT N_{60} values for all site areas and all soil strata is presented in Table 2.5S.4-6. The average N_{60} value for Stratum B was 14 blows/foot; N_{60}=11 was selected for engineering purposes as shown in Table 2.5S.4-6.

As noted above, corrected SPT N_{60}-values for sandy soils from each boring were corrected to an effective overburden pressure of one atmosphere (approximately one tsf), leading to fully-corrected values of $(N_1)_{60}$. A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all sandy soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum B was 15 blows/foot. An SPT $(N_1)_{60}$-value of 12 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 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 54 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 62 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figures 2.5S.4-16,2.5S.4-17 and 2.5S.4-19 for the STP 3 area, the STP 4 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 (%)	36	18	28	24
Liquid Limit (%)	14	Non-Plastic	46	33
Plasticity Index (%)	14	Non-Plastic	26	14
Fines Content (%)	19	36	94	67
Unit Weight (pcf)	5	117	128	121

Test results are summarized in Table 2.5S.4-8. Note that 9 of the 14 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, with an average fines content (materials passing the No. 200 sieve) of 67%. Cohesive soil types were a minority. 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. The empirical tabular correlations from Table 3-4 of Reference 2.5S.4-14 may be approximately expressed as:

Φ' (fine sand) = 27 + N/4 (in degrees) Equation 2.5S.4-12A

Φ' (medium sand) = 28.5 + N/3 (in degrees) Equation 2.5S.4-12B

Φ' (coarse sand) = 28 + N/2 (in degrees) Equation 2.5S.4-12C

where, N = $(N_1)_{60}$.

Using Equation 2.5S.4-12A the selected corrected SPT $(N_1)_{60}$-value for Stratum B (12 blows/foot), a value of Φ'=30 degrees (for fine sand) was estimated.

The drained friction angle, Φ', for cohesionless soil behavior types, was also estimated using the CPT data, following a CPT-Φ' correlation from Reference 2.5S.4-15, as follows:

Φ' = arctangent (log $[q_t/\sigma_v']$ + 0.29)/2.68 Equation 2.5S.4-12D

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'=39.5 degrees. Note that SPT correlations were based on 175 field measurements, while CPT correlations were based on 258 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, 2.5S.4-35, 2.5S.4-37 and 2.5S.4-38 for the STP 3 area, the STP 4 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 Triaxial Shear Testing
Φ' (degrees)	30	39.5	30

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

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

The elastic modulus, E, for coarse-grained soils was evaluated using the following relationship adapted from Reference 2.5S.4-55, Table 5-6, using an equation footnoted as "Japanese Design Standards for Structures" and adjusting the equation, which was based on N_{55} instead of N_{60}, and C_s=1.0 instead of C_s=1.2 to apply N_{60} as used herein:.

E = 47 N (in ksf) Equation 2.5S.4-13

where, N = average corrected SPT N_{60}-value in blows/foot.

Substituting the previously established corrected SPT N_{60}-value for Stratum B soils (11 blows per foot), an E=515 ksf was estimated.

For coarse grained soil strata, relationships employing shear wave velocity, were according to Equations 2.5S.4-5 and 2.5S.4-6. Reference 2.5S.4-10A (Subsection 5.5.6) provides the following relationship between the small strain stiffness modulus, E_0 (or G_0), and the modulus at working stress levels, E (or G):

$$\frac{E}{E_0} = 1 - \left(\frac{q}{q_{ult}}\right)^{0.3}$$ Equation 2.5S.4-14

where,
E_0 = small strain modulus (i.e., strain in the range of 10^{-4}%)
E = large strain (static) modulus at the desired working stress
q = working stress
q_{ult} = ultimate stress

Using a factor of safety equal to 3,

$$\frac{q}{q_{ult}} = \frac{1}{FOS} = \frac{1}{3}$$

where,
FOS = Factor of Safety

At applied (working) stress levels equal to 1/3 of q_{ult}, corresponding to soil stresses in the range of factors of safety equal to 3, E (or G) is approximately 0.3 times E_0 (or G_0). This multiplier (0.3) is used to adjust the seismic modulus of sandy layers Stratum B, C, E, H, J Sand, Sub-stratum K Sand/Silt, and Stratum M sand to a representative static modulus corresponding to applied stress levels of about one third of the ultimate stress level. The sandy layer, Sub-stratum N Sand, and the clayey layer, Sub-stratum N Clay, have higher shear wave velocities and are at depths where the applied stresses are less than one third the ultimate value. A multiplier of 0.5 is used for these layers, as well as all the deep layers at depths beyond 600 feet as described in Section 2.5S.4.2.1.14.

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=1540 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=1200 ksf was selected for Stratum B. This compares with the range of $500 \leq E_s \leq 1000$ ksf for medium dense sand in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The E value for sandy layers is appropriate for the effective stress condition. The shear modulus, G, was related to E by Equation 2.5S.4-5, re-ordered to solve for G if E and μ are known. Using E=1200 ksf and μ =0.30 for sand, G=462 ksf is calculated. A G=465 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 $K_{0,NC}$=0.5, $K_{0,OCR}$ is not evaluated for Stratum B.

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 and STP 4 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, C-916,C-948, C-948A and C-949 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 5 feet to 30 feet, with an average thickness of approximately 20 feet, and the base elevation varied from El. -24 feet to El. -7 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 122 blows/foot, with an average uncorrected SPT N-value of 23 blows/foot. 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 through 2.5S.4-13 and 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 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.

The uncorrected SPT N-values from each boring were corrected to an energy transfer ratio of 60 percent by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections for rod length and sampler (C_s=1.2) leading to values of N_{60}. A summary of SPT N_{60} values for all site areas and all soil strata is presented in Table 2.5S.4-5. The average N_{60} value for Stratum C was 41 blows/foot; N_{60}=38 blows/foot was selected for engineering purposes as shown in Table 2.5S.4-6.

As noted above, corrected SPT N_{60}-values for sandy soils from each boring were corrected to an effective overburden pressure of one atmosphere (approximately one tsf), leading to fully-corrected values of $(N_1)_{60}$. A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all sandy soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum C was 39 blows/foot, $(N_1)_{60}$=35 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT N_{60} and $(N_1)_{60}$-values, Stratum C is considered very 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 165 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 152 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figures 2.5S.4-16,2.5S.4-17, and 2.5S.4-19 for the STP 3 area, the STP 4 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 (%)	45	17	27	23
Liquid Limit (%)	2	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	2	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	39	5	96	23
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 23%. 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 Equation 2.5S.4-12A and the selected corrected SPT $(N_1)_{60}$-value for Stratum C (35 blows/foot), a value of ϕ'=of 36 degrees (for fine sand) was estimated. 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.4-12D. Drained friction angle values calculated from the CPT data indicated an average ϕ'=42 degrees. Note that SPT correlations were based on 487 field measurements, while CPT correlations were based on 2,042 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, 2.5S.4-35, 2.5S.4-37, and 2.5S.4-38 for the STP 3 area, the STP 4 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 corrected SPT N_{60}-value for Stratum C soils (38 blows per foot) an E=1785 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=1820 ksf was estimated. Using an average of the E-values estimated from the corrected SPT N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=1810 ksf was selected for Stratum C. This compares with a value range $E_s \geq$ 1700 ksf for very dense sand in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The E value for sandy layers is appropriate for the effective stress condition. The shear modulus, G, was related to E by Equation 2.5S.4-5, re-ordered to solve for G if E and μ are known. Using E=1810 ksf and μ=0.30 for sand, G=696 ksf is calculated. A G=695 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 $K_{0,NC}$=0.4. $K_{0,OCR}$ was not evaluated for Stratum C.

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, B-927, B-941, B-942, B-943, B-945, B-946, and B-947, and CPTs C-301, C-303, C-401, C-402, C-403, C-411, C-905, C-906, C-907, C-908, C-909, C-917, C-918, C-940, C-941, C-942, C-943, C-945, C-946, C-947 and C-949 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 9.0 feet to 34 feet, with an average thickness of 21 feet, and the base elevation varied from El. -45 feet to El. -26 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 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 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 through 2.5S.4-13 and 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 15 blows/foot for Stratum D.

As noted above, uncorrected (measured) SPT N-values from each boring were corrected to an energy ration of 60 percent (i.e., N_{60}) by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections. A summary of corrected SPT N_{60} and $(N_1)_{60}$-values for all site areas and all soil strata is presented in Table 2.5S.4-56. The average corrected SPT N_{60}-value for Stratum D was 25 blows/foot. An SPT N_{60}-value of 23 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT N_{60}-values, Stratum D is considered 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 40 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, 2.5S.4-17 and 2.5S.4-19 for the STP 3 area, the STP 4 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 (%)	90	16	53	26
Liquid Limit (%)	53	20	84	57
Plasticity Index (%)	53	2	59	37
Fines Content (%)	26	18	100	79
Unit Weight (pcf)	26	111	130	123

Test results are summarized in Table 2.5S.4-8. Note that five of the 53 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 79%. 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, CL, CL-ML and ML. Based on laboratory testing, an average unit weight of 123 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_{60}-values and the CPT results. The results of this evaluation are summarized in Table 2.5S.4-9.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT N_{60}-values (Reference 2.5S.4-7) using Equation 2.5S.4-2. Substituting the selected corrected SPT N_{60}-value for Stratum D (23 blows/foot), an s_u=2.9 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. Undrained shear strength values calculated from the CPT data indicated an average s_u=3.3 ksf. The CPT-derived values are shown versus elevation on Figures 2.5S.4-23,2.5S.4-24, 2.5S.4-26, and 2.5S.4-27 for the STP 3 area, the STP 4 area, the area outside the Power Block, and site-wide, respectively. Note that SPT correlations were based on 520 field measurements, while CPT correlations were based on 958 field measurements made within Stratum D. Results of 11 laboratory UU and UNC strength tests made on selected samples indicated an average s_u=1.9 ksf. The ratio of the shear strength of Stratum D clay sample to the vertical effective stress at the depth the sample was taken for the UU and UNC tests ranges from 0.07 to 0.90 in Table 2.5S.4-10. Reference 2.5S.4-14A indicates that this ratio should be 0.31 for clay soils of the Beaumont formation at OCR=1, ranging upward to 1.2+ at OCR=10. Therefore, UU and UNC test results that produced low ratios of shear strength to vertical effective stress are considered likely to have been disturbed or to have failed prematurely due to the presence of desiccation features such as slickensides and thus are unrepresentative. The two lowest laboratory strength test results have ratios of 0.07 and 0.12. By excluding these two lowest laboratory strength test results, an average s_u=2.2 ksf resulted (9 test results). Laboratory shear strength test results are summarized in Table 2.5S.4-9 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 average of the SPT-derived and 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 (effective stress) 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'=1.2 ksf, and average (undrained/total) Φ=4 degrees and c=1.8 ksf, as noted:

Parameter	From CIU-Bar
Φ' (degrees)	16
c' (ksf)	1.2
Φ (degrees)	4
c (ksf)	1.8

The parameters above are for stresses below the preconsolidation stress of the Stratum D.

Based on the average plasticity index, Reference 2.5S.4-7 indicates a value range of $20 \leq \Phi' \leq 27°$ for Stratum D in the normally consolidated stress range. For stresses above the preconsolidation stress of Stratum D, Φ'=20 degrees was used to provide conservative values.

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 eight consolidation tests made on selected samples indicated that, on average, Stratum D was preconsolidated to approximately 13.4 ksf, with an OCR=3.9. 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 using Equation 2.5S.4-3B indicated an average OCR=4.2, and were based on 958 field measurements. CPT-derived OCR data are shown on Figure 2.5S.4-29, Figure 2.5S.4-30, Figure 2.5S.4-32, and Figure 2.5S.4-33 for the STP 3 area, the STP 4 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.4-4A. Substituting the previously established s_u and OCR for Stratum D soils (s_u=3 ksf, OCR=3.3), an E=1635 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=122 pcf for Stratum D, and PI=40 for Stratum D, an E=2830 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=2430 ksf was selected for Stratum D. This compares with a value range of 1050 ≤ E_s ≤ 2100 ksf for hard clay in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

Stratum D is characterized as a clay and the elastic modulus E requires adjustment for drained, effective stress, long term loading conditions using Equation 2.5S.4-8A. For Stratum D, the value of Poisson's ratio for drained condition μ_d=0.15 based on Reference 2.5S.4-14B, and the resulting E_d=1865 ksf. The selected E_d values for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) for clayey soils is related to the drained modulus, E_d, by Equation 2.5S.4-8. Using μ_d=0.15 for Stratum D, and the value of E_d=1865 selected above, G=811 ksf was calculated. A value of G=800 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 $K_{0,NC}$=0.7. For OCR=3.3, $K_{0,OCR}$ =1.05 by Equation 2.5S.4-11B and $K_{0,OCR}$ = 0.96 by Equation 2.5S.4-11C for Stratum D. For engineering purposes, $K_{0,OCR}$=1.0 was selected.

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 area west and northwest of the Power Block. Stratum E was not encountered in Borings B-420, B-901 through B-910, B-912, B-928, B-930, B-931, and B-933, B-940 and B-949, 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 5 feet to 36 feet, with an average thickness of 18 feet, and the base elevation varied from El. -71 feet to El. -43 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 96 blows/foot, with an average uncorrected SPT N-value of 41 blows/foot. 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 through 2.5S.4-13, and 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 37 blows/foot for Stratum E.

The uncorrected SPT N-values from each boring were corrected to an energy transfer ratio of 60 percent by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections for rod length and sampler (C_s=1.2) leading to values of N_{60}. A summary of SPT N_{60} values for all site areas and all soil strata is presented in Table 2.5S.4-6. The average N_{60} value for Stratum E was 60 blows/foot; a value of N_{60}=53 blows/foot was selected for engineering purposes as shown in Table 2.5S.4-6. As noted above, corrected SPT N_{60}-values in sandy soil strata from each boring were corrected to an effective overburden pressure of one atmosphere (approximately one tsf) leading to fully-corrected values of $(N_1)_{60}$. A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all sandy soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum E was 35 blows/foot. An SPT $(N_1)_{60}$-value of 31 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 very 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 223 tsf. Also, site-wide the average normalized CPT tip resistance, q_{c1n} (normalized to an effective overburden pressure of approximately 1 tsf) for Stratum E was 133 (dimensionless). As noted above, Stratum E was largely absent in the area west and norhtwest of STP Unit 4 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 (%)	48	15	26	21
Liquid Limit (%)	6	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	6	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	43	3	96	20
Unit Weight (pcf)	9	111	133	123

Test results are summarized in Table 2.5S.4-8. Natural moisture contents and Atterberg limits for other soil strata are presented versus elevation on Figure 2.5S.4-20. Atterberg for other soil strata 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 20%. Note that the maximum fines contents reported occurred at Boring B-343 from depths of 70 feet to 72 feet (96% fines) and Boring B-940 from depths 98.5 feet to 100 feet (87% fines). These results represent isolated thin clay and silt lenses within the Stratum E sand. The next highest fines content reported was 54%. 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 123 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 (31 blows/foot), from Equation 2.5S.4-12B a value of ϕ'=of 39 degrees (for fine to medium sand) was estimated. 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.4-12D. Drained friction angle values calculated from the CPT data indicated an average ϕ'=39 degrees. Note that SPT correlations were based on 389 field measurements, while CPT correlations were based on 461 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)	39	39	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 corrected SPT N_{60}-value for Stratum E soils (53 blows per foot), an E=2490 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=123 pcf for Stratum E, an E=3475 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=3145 ksf was selected for Stratum E. This compares with a value range of $E_s \geq$1700 ksf for very dense sand in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The E value for sandy layers is appropriate for the effective stress condition. The shear modulus, G, was related to E by Equation 2.5S.4-5, re-ordered to solve for G if E and μ are known. Using E=3145 ksf and μ=0.30 for sand, G=1210 ksf is calculated. A G=1215 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 $K_{0,NC}$=0.4. $K_{0,OCR}$ was not evaluated for Stratum E.

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 the CPTs and borings west of the Power Block. 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 2 to 30 feet, with an average thickness of 15 feet, and the base elevation varied from El. -81 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 102 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. 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 through 2.5S.4-13 and 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 22 blows/foot for Stratum F.

As noted above, uncorrected (measured) SPT N-values from each boring were corrected to an energy ratio of 60 percent (i.e., N_{60}), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed. A summary of corrected SPT N_{60}-values for all site clayey soils and $(N_1)_{60}$-values for all site sandy soils for all areas and all soil strata is presented in Table 2.5S.4-6. The average SPT N_{60}-value for Stratum F was 36 blows/foot. An SPT N_{60}-value of 34 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT N_{60}-values, Stratum F is considered hard.

CPTs were additionally performed in Stratum F soils. Site-wide, the CPT tip resistance, q_t, in this stratum ranged from 23 tsf to 118 tsf, with an average of 40 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 19 (dimensionless). 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 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 (%)	66	18	33	24
Liquid Limit (%)	47	27	74	57
Plasticity Index (%)	47	6	53	37
Fines Content (%)	14	56	99	94
Unit Weight (pcf)	18	120	131	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 plastic to highly plastic clay with an average fines content (materials passing the No. 200 sieve) of 94%. Note that the minimum 56% fines content reported occurred at Boring B-443 from depths of 103.5 feet to 105 feet. This result represents an isolated thin sand lens within the Stratum F clay. All other fines contents reported were greater than 85%. The USCS designations for Stratum F were mainly fat clay, lean clay, and silty clay, with the predominant USCS group symbols of CH, CL, ML, and CL-ML. 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_{60}-values and the CPT results. The results of this evaluation are summarized in Table 2.5S.4-9.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT N_{60}-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT N_{60}-value for Stratum F (34 blows/foot), an s_u=4.0 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=3.5 ksf. The CPT-derived values are shown versus elevation on Figures 2.5S.4-23, 2.5S.4-24, 2.5S.4-26 and 2.5S.4-27 for the STP 3 area, the STP 4 area, the area outside the Power Block, and site-wide, respectively. Note that SPT correlations were based on 315 field measurements, while CPT correlations were based on 305 field measurements made on cohesive soil behavior types within Stratum F. The results of 17 laboratory UU and UNC strength tests made on selected clay samples indicated an average s_u=2.7 ksf.

The ratio of the shear strength of Stratum F clay samples to the vertical effective stress at the depth the sample was taken for the UU and UNC tests range from 0.11 to 0.89 in Table 2.5S.4-10. Reference 2.5S.4-14A indicates this ratio should be 0.31 for soils of the Beaumont formation at OCR=1, ranging upward to 1.2+ at an OCR=10.

Therefore, UU and UNC test results that produced low ratios of shear strength to vertical effective stress are considered likely to have been disturbed or to have failed prematurely due to the presence of desiccation features such as slickensides and thus are unrepresentative. The two lowest laboratory strength test results have ratios of 0.11, and 0.19. By excluding these two lowest laboratory strength test results, an average s_u=2.9 ksf resulted (15 test results).

Laboratory shear strength test results are summarized in Table 2.5S.4-9 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). The average of the SPT-derived, CPT-derived, and filtered laboratory s_u results for Stratum F is s_u=3.5 ksf. Based on this, an undrained shear strength of s_u=3.4 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' (ksf)	2.0
Φ (degrees)	3
c (ksf)	2.1

The parameters above are for stresses below the preconsolidation stress of the Stratum F. Based on the average plasticity index, Reference 2.5S.4-7 indicates a value range of 20° ≤ Φ ≤ 26° for Stratum F in the normally consolidated stress range. The value Φ'=20 degrees was selected for Stratum F soils at stresses above the preconsolidation stress.

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 six consolidation tests made on selected samples indicated that, on average, Stratum F is preconsolidated to approximately 18.6 ksf, with an OCR=3.1. 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=2.4 and were based on 305 field measurements. CPT-derived OCR data are shown on Figure 2.5S.4-29, Figure 2.5S.4-30, and Figure 2.5S.4-33 for the STP 3 area, 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=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.4-4A. Substituting the previously established s_u and OCR for Stratum F soils (s_u=3.4 ksf, OCR=2.6), an E=1645 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, γ=125 pcf for Stratum F, and PI=40 for Stratum F, an E=3030 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=2570 ksf was selected for Stratum F. This compares to a value range of 1050 ksf ≤ E_s ≤ 2100 ksf for hard clay in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

Stratum F is characterized as a clay and the elastic modulus E requires adjustment for drained, effective stress, long term loading conditions using Equation 2.5S.4-8A. For Stratum F, the value of Poisson's ratio for drained condition μ_d=0.15 based on Reference 2.5S.4-14B and the resulting E_d=1970 ksf. A value of E_d=1970 ksf was selected for engineering use for Stratum F. The selected E_d values for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) for clayey soils is related to the drained modulus, E_d, by Equation 2.5S.4-8. Using μ_d=0.15 for Stratum F, and the value of E_d=1970 selected above, G=857 ksf was calculated. A value of G=850 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 $K_{0,NC}$=0.7. Equation 2.5S.4-11B gives $K_{0,OCR}$ = 1, while Equation 2.5S.4-11C gives $K_{0,OCR}$ = 0.9. A value of $K_{0,OCR}$ = 1.0 is selected for Stratum F.

Based on Reference 2.5S.4-13, and the selected ϕ'=20 degrees, 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. 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 2 feet to 35.5 feet, with an average thickness of 17.0 feet, and the base elevation of Stratum H varied from El. -93.5 feet to El. -64.6 feet, with an average of El. -87.1 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 18 blows/foot to 150 blows/foot, with an average uncorrected SPT N-value of 48 blows/foot. 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 throught 2.5S.4-13, and 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 44 blows/foot for Stratum H.

The uncorrected SPT N-values from each boring were corrected to an energy transfer ratio of 60 percent by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections for rod length and sampler (C_s=1.2) leading to values of N_{60}. A summary of SPT N_{60} values for all site areas and all soil strata is presented in Table 2.5S.4-6. The average N_{60} value for Stratum H was 70 blows/foot; a value of N_{60}=58 blows/foot was selected for engineering purposes as shown in Table 2.5S.4-6.

As noted above, corrected SPT N_{60}--values for sandy strata from each boring were corrected to an effective overburden pressure of one atmosphere (approximately one tsf) leading to fully-corrected values of $(N_1)_{60}$. A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all sandy soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum H was 35 blows/foot. An SPT $(N_1)_{60}$-value of 28 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 very 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 99 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figure 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 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 (%)	16	12	24	19
Liquid Limit (%)	1	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	1	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	14	6	95	18
Unit Weight (pcf)	4	121	135	125

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 18%. Note that two samples taken from borings B-305DH/DHA and B-443 at depths of 103 feet to 111.5 feet had fines contents ranging from 44% to 95%. These results represent isolated thin clay lenses within the Stratum H sand. The next highest fines content reported was 19%. The USCS designations for Stratum H were mainly poorly graded sand with silt, silty 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 125 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 (28 blows/foot) and Equation 2.5S.4-12B, a value of Φ'=of 38 degrees (for fine to medium sand) was estimated. 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.4-12D. Drained friction angle values calculated from the CPT data indicated an average Φ'=37 degrees. Note that SPT correlations were based on 134 field measurements, while CPT correlations were based on 95 field measurements made on cohesionless soil behavior types 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, 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 H is provided as follows:

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
Φ' (degrees)	38	37	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 corrected SPT N_{60}-value for Stratum H soils (58 blows per foot), an E=2725 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=125 pcf for Stratum H, an E=3500 ksf was estimated. Using an average of the E-values estimated from the corrected SPT N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=3240 ksf was selected for Stratum H. This compares to a value range of $E_s \leq$ 1700 ksf for very dense sand in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The E value for sandy layers is appropriate for the effective stress condition. The shear modulus, G, was related to E by Equation 2.5S.4-5, re-ordered to solve for G if E and μ are known. Using E=3240 ksf and μ=0.30 for sand, G=1246 ksf is calculated. A G=1250 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 $K_{0,NC}$, 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 $K_{0,NC}$=0.4. $K_{0,OCR}$ was not evaluated for Stratum H.

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)

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 107 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. Thirteen of 47 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-405DH, B-414, B-416 and B-443. Sub-stratum J Clay 1 ranged in thickness from 10.0 feet to 49 feet, with an average thickness of 23 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. -98 feet.

Where encountered, Sub-stratum J Interbed 1 ranged in thickness from 3.5 feet to 10 feet, with an average thickness of 9 feet. The average base elevation of Sub-stratum J Interbed 1 was El. -107 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 35 feet, with an average thickness of 22 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. -119 feet.

Sub-stratum J Sand 1 was encountered below Sub-stratum J Clay 1, and was fully penetrated in 23 borings. Sub-stratum J Sand 1 ranged in thickness from 1.5 feet to 25.5 feet, with an average thickness of 13 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 14 borings. Thirteen of 17 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-302DH, B-303, B-305DH/DHA, B-306, B-319DH, B-402DH, B-403, B-404, B-

405DH, B-408DH, B-409, B-428DH 428DH, and
B-443. Sub-stratum J Clay 2 ranged in thickness from 3 foot to 32 feet, with an average thickness of 15 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. -139 feet.

Where encountered, Sub-stratum J Interbed 2 ranged in thickness from 8 feet to 30 feet, with an average thickness of 15 feet. The average base elevation of Sub-stratum J Interbed 2 was El. -152 feet.

Sub-stratum J Clay 2 below Sub-stratum J Interbed 2 (or Sub-stratum J Clay 2 “Bottom”) ranged in thickness from 12 feet to 38 feet, with an average thickness of 24 feet. The thickness of the combined Sub-stratum J Clay 2 “Top” and “Bottom” ranged in thickness from 35 feet to 48 feet, with an average thickness of 41 feet. The average thickness of Sub-stratum J Clay 2 with Sub-stratum J Interbed 2 included was 56 feet. The average base elevation of Sub-stratum J Clay 2 was El. -184 feet.

Five borings in the STP 3 area, namely B-301, B-304, B-307, B-316, and B-348 encountered a sand layer below Sub-stratum J Clay 2. This sand layer was found neither in the STP 4 area borings, nor in the three borings in the STP 3 & 4 areas that fully penetrated Stratum J, namely B-305DH/DHA, B-405DH, and B-443. This layer 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, and Sub-stratum J Interbed 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 120 blows/foot, with an average uncorrected SPT N-value of 32 blows/foot. In the STP 4 area, uncorrected SPT N-values in Sub-stratum J Clay ranged from 13 blows/foot to 138 blows/foot, with an average uncorrected SPT N-value of 32 blows/foot. 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 through 2.5S.4-13 and 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 32 blows/foot for Sub-stratum J Clay.

As noted above, uncorrected SPT N-values for the Sub-stratum J Clay from each boring were corrected to an energy ratio of 60 percent by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections A summary of corrected SPT N_{60} and $(N_1)_{60}$-values, for all site areas and all soil strata is presented in Table 2.5S.4-6. The average corrected SPT -N_{60}-value for Sub-stratum J Clay was 51 blows/foot. An SPT N_{60}-value of 48 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT N_{60}-values, Stratum J Clay is considered hard.

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 138 tsf, with an average of 61 tsf. Also, the average normalized CPT tip resistance, q_{c1n} (normalized to an effective overburden pressure of approximately 1 tsf), for Stratum J Clay was 31 (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 (%)	90	14	38	23
Liquid Limit (%)	70	26	85	54
Plasticity Index (%)	70	9	62	35
Fines Content (%)	39	55	100	90
Unit Weight (pcf)	47	104	134	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 90%. 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 sandy silt, with the USCS group symbols of CH, CL, and ML. 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.4-9.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT N_{60}-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT N_{60}-value for Sub-stratum J Clay (48 blows/foot), an s_u=6.0

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.4-24 and 2.5S.4-27, for the STP 4 area and site-wide, respectively. Note that SPT correlations were based on 239 field measurements, while CPT correlations were based on only two field measurements made on cohesive soil behavior types within Sub-stratum J Clay. The CPT-derived s_u result is therefore not considered representative of Sub-stratum J Clay.The results of 34 laboratory UU and UNC strength tests made on selected samples indicated an average s_u=3.0 ksf. The ratio of the shear strength of Substratum J Clay samples to the vertical effective stress at the depth the sample was taken for the UU and UNC tests range from 0.01 to 0.77 in Table 2.5S.4-10. Reference 2.5S.4-14A indicates this ratio should be 0.31 for soils of the Beaumont formation at OCR=1, ranging upward to 1.2+ at an OCR=10. Therefore, UU and UNC test results that produced low ratios of shear strength to vertical effective stress are considered likely to have been disturbed or to have failed prematurely due to the presence of desiccation features such as slickensides and thus are unrepresentative. The 13 lowest laboratory strength test results have ratios of 0.01 to 0.15. By excluding these 13 lowest laboratory strength test results, an average s_u=4.3 ksf resulted (21 test results).

Laboratory shear strength test results are summarized in Table 2.5S.4-9 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 all of the above, and an undrained shear strength of s_u=3.8 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 eight CIU-bar tests, indicated average (drained/effective) ϕ'=11 degrees and c'=2.3 ksf and average (undrained/total) ϕ=7 degrees and c=2.7 ksf.

Parameter	From CIU-Bar
ϕ' (degrees)	11
c' (ksf)	2.3
ϕ (degrees)	7
c (ksf)	2.7

These values are for stresses below the preconsolidation stress of the Sub-stratum J clay soil.

Based on the average plasticity index, reference 2.5S.4-7 indicates a value range of 22° ≤ ϕ' ≤ 27° for Sub-stratum J Clay in the normally consolidated stress range. A drained/effective ϕ'=20 degrees was selected for Sub-stratum J Clay soils, above the preconsolidation stress range.

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 11 consolidation tests made on selected samples indicated that, on average, Sub-stratum J Clay was preconsolidated to approximately 18.7 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=1.7 and was based on two field measurements made in cohesive soil behavior types at CPT C-408. The CPT-derived OCR for Sub-stratum J Clay soils is therefore not considered representative. 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.4-4B. Substituting the previously established s_u and OCR for Sub-stratum J Clay soils (s_u=3.8 ksf, OCR=1.7), an E=4955 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=1085 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=3735 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=4140 ksf was selected for Sub-stratum J Clay. This compares to a value range of 500 ksf $\leq E_s \leq$ 5000 ksf for sandy clay in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

Sub-stratum J Clay is characterized as a clay and the elastic modulus E requires adjustment for drained, effective stress, long term loading conditions using Equation 2.5S.4-8A. For Sub-stratum J Clay, the value of Poisson's ratio for drained condition μ_d=0.15 based on Reference 2.5S.4-14B and the resulting E_d=3175 ksf. The selected E_d values for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) for clayey soils is related to the drained modulus, E_d, by Equation 2.5S.4-8. Using μ_d=0.15 for Sub-stratum J Clay, and the value of E_d=3175 ksf selected above, G=1380 ksf was calculated. A value of G=1380 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 32 blows/foot to 120 blows/foot, with an average uncorrected SPT N-value of 70 blows/foot. In the STP 4 area, uncorrected SPT N-values in Sub-stratum J Sand ranged from 20 blows/foot to 125 blows/foot, with an average uncorrected SPT N-value of 55 blows/foot. In the area outside the Power Block , 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 63 blows/foot for Sub-stratum J Sand.

The uncorrected SPT N-values from each boring were corrected to an energy transfer ratio of 60 percent by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections for rod length and sampler (C_s=1.2) leading to values of N_{60}. A summary of SPT N_{60} values for all site areas and all soil strata is presented in Table 2.5S.4-6. The average N_{60} value for Sub-stratum J Sand was 100+ blows/foot; a value of N_{60}=94 blows/foot was selected for engineering purposes as shown in Table 2.5S.4-6.

As noted above, SPT N_{60}-values for sandy soils from each boring were corrected to an effective overburden pressure of one atmosphere (approximately one tsf) leading to fully-corrected values of $(N_1)_{60}$. A summary of corrected SPT $(N_1)_{60}$-values for all site areas and sandy soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Sub-stratum J Sand was 41 blows/foot. An SPT $(N_1)_{60}$-value of 38 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 very 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 (%)	17	16	32	22
Liquid Limit (%)	9	Non-Plastic	24	Non-Plastic
Plasticity Index (%)	9	Non-Plastic	3	Non-Plastic

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Fines Content (%)	17	10	97	50
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 50%. Note that the maximum values for Liquid Limit and for Plasticity Index (PI) reported occurred at Boring B-443 from depths of 163.5 feet to 165 feet. These results represent an isolated thin silt lens within Sub-stratum J Sand. All other Atterberg Limits for Sub-stratum J Sand soils tests were reported as non-plastic. The USCS designations for Sub-stratum J Sand were mainly, silty sand, sandy silt, silt with sand, and poorly graded sand with silt, and occasionally sandy lean 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 Equation 2.5S.4-12B and the selected corrected SPT $(N_1)_{60}$-value for Sub-stratum J Sand (38 blows/foot), a value of ϕ'=of 41 degrees (for fine to medium sand) was estimated. 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)	41	---	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 corrected SPT N_{60}-value for Sub-stratum J Sand soils (94 blows per foot), an E=4420 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=4925 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=4755 ksf was selected for Sub-stratum J Sand. This compares with a value range of $E_s \geq 1700$ ksf for very dense sand in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The E value for sandy layers is appropriate for the effective stress condition. The shear modulus, G, was related to E by Equation 2.5S.4-5, re-ordered to solve for G if E and μ are known. Using E=4755 ksf and μ=0.30 for sand, G=1828 ksf is calculated. A G=1830 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 31 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. 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 energy ratio of 60 percent (i.e., N_{60}) by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed. A summary of corrected SPT N_{60}-values, for all site areas and all soil strata is presented in Table 2.5S.4-6. The average corrected SPT N_{60}-value for Sub-stratum K Clay was 25.5 blows/foot. An SPT N_{60}-value of 26 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT N_{60}-values, Stratum K Clay is indicated as very stiff to hard.

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 (%)	4	17	35	23
Liquid Limit (%)	3	33	73	50
Plasticity Index (%)	3	18	51	33
Fines Content (%)	2	75	99	87
Unit Weight (pcf)	3	115	132	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, Sub-stratum K Clay soils were characterized, on average, as lean clay with an average fines content (materials passing the No. 200 sieve) of 87%. The USCS designations for Sub-stratum K Clay were mainly lean clay and lean clay with sand, with the predominant USCS group symbols of CL and CH. Based on laboratory testing, an average unit weight of 124 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_{60}-values. The results of this evaluation are summarized in Table 2.5S.4-9.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT N_{60}-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT N_{60}-value for Sub-stratum K Clay (26 blows/foot), an s_u=3.3 ksf was estimated. Note, however, that this average value is based on only two SPT N_{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 undrained shear strength test results are summarized in Table 2.5S.4-9 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. The two laboratory UU and UNC strength tests of Sub-stratum K Clay represent s_u/s_v' ratios of 0.20 and 0.26. These compare to the value of 0.31 expected for OCR=1 (Reference 2.5S.4-14A). and a value of 0.43 expected for OCR=1.3 (estimated using Equation 2.5S.4-3B, rearranged). Based on this, it was deemed that the higher of the two laboratory derived s_u results from this subsurface investigation was the most representative, and an undrained shear strength of s_u=4.0 ksf (the highest of the two tests) was selected for Sub-stratum K Clay.

CIU and CIU-Bar triaxial testing was not performed on samples of Sub-stratum K Clay soils. The CIU and CIU-Bar triaxial strengths of Sub-stratum K Clay are assumed equal to those of Sub-stratum J Clay. This is deemed reasonable as Substratum K Clay has a similar average PI and is slightly more sandy (lower fines content) than Sub-stratum J Clay.

Parameter	From CIU-Bar
ϕ' (degrees)	11
c' (ksf)	2.3
ϕ' (degrees)	7
c (ksf)	2.7

The above strengths are applicable to stress levels below the preconsolidation stress.

Based on the average plasticity index, Reference 2.5S.4-7 indicates a value range of 22°≤ ϕ' ≤ 27°‹ for Sub-stratum K Clay in the normally consolidated stress range. The drained/effective friction angle ϕ'=20 degrees was selected for Sub-stratum K Clay in the normally consolidated range.

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.4-4B. Substituting the previously established s_u and OCR for Sub-stratum K Clay soils (s_u=3.9 ksf, OCR=1.3), an E=4445 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=1170 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, γ=124 pcf for Sub-stratum K Clay, and PI=35 for Sub-stratum K Clay, an E=4305 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=4350 ksf was selected for Sub-stratum K Clay. This compares with a value range of 500 ksf ≤ E_s ≤ 5000 ksf for sandy clay in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

Sub-stratum K Clay is characterized as a clay and the elastic modulus E requires adjustment for drained, effective stress, long-term loading conditions using Equation 2.5S.4-8A. For Sub-stratum K Clay, the value of Poisson's ratio for the drained condition μ_d=0.15 based on Reference 2.5S.4-14B and the resulting E_d= 3350 ksf. The selected E_d values for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) for clayey soils is related to the drained modulus, E_d, by Equation 2.5S.4-8. Using μ_d=0.15 for Sub-stratum K Clay, and the value of E_d=3335 selected above, G=1450 ksf was estimated. A value of G=1450 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 40 blows/foot to 120 blows/foot, with an average uncorrected SPT N-value of 80 blows/foot. 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 80 blows/foot for Sub-stratum K Sand/Silt.

The uncorrected SPT N-values from each boring were corrected to an energy transfer ratio of 60 percent by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections for rod length and sampler (C_s=1.2) leading to values of N_{60}. A summary of SPT N_{60} values for all site areas and all soil strata is presented in Table 2.5S.4-5. The average N_{60} value for Stratum K Sand/Silt was 100 plus blows/foot; a value of N_{60}=68 blows/foot was selected for engineering purposes as shown in Table 2.5S.4-6.

As noted above, SPT N_{60}-values for sandy strata from each boring were corrected to an effective overburden pressure of one atmosphere (approximately one tsf), leading to fully-corrected values of $(N_1)_{60}$. A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all sandy 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 54 blows/foot. An SPT $(N_1)_{60}$-value of 27 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 very dense.

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 Equation 2.5S.4-12A and the selected corrected SPT $(N_1)_{60}$-value for Sub-stratum K Sand/Silt (27 blows/foot), a value of ϕ'=of 34 degrees (for fine sand) was estimated. 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/Silt is provided as follows:

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	34	---	29

Based on the above a ϕ'=31 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_{60}-value for Sub-stratum K Sand/Silt soils (68 blows per foot) an E=3195 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=5775 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=4915 ksf was selected for Sub-stratum K Sand/Silt. This compares to a value range of $E_s \geq 1700$ ksf for very dense sand in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The E value for sandy layers is appropriate for the effective stress condition. The shear modulus, G, was related to E by Equation 2.5S.4-5, re-ordered to solve for G if E and μ are known. Using E=4915 ksf and μ=0.30 for sand, G=1890 ksf is calculated. A G=1890 ksf was selected for SubStratum 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 5.5 feet, with an average thickness of 5.0 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. 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 a hammer energy ratio of 60 percent 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 values of N_{60}. A summary of corrected SPT N_{60}-values, for all site areas and all soil strata is presented in Table 2.5S.4-6. The average corrected SPT N_{60}-value for Stratum L was 38 blows/foot. An SPT N_{60}-value of 36 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT N_{60}-values, Stratum L is hard.

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 87% (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 124 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.4-9.

Undrained shear strength, s_u, was estimated from empirical correlations with corrected SPT N_{60}-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT N_{60}-value for Stratum L (36 blows/foot), an s_u=4.5 ksf was estimated. Note, however, that this average value is based on only two corrected SPT N_{60}-values. Also note that neither CPT data nor laboratory shear strength data from UU and/or UNC strength tests were available for Stratum L. 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=3.9 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=1.3 and a preconsolidation pressure of 20.5 ksf were selected for Stratum L.

The elastic modulus (E) for Stratum L was evaluated using Equation 2.5S.4-4B. Substituting the previously established s_u and OCR for Stratum L soils (s_u=3.9 ksf, OCR=1.3), an E=4445 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=124 pcf for Stratum L, and PI=50 for Stratum L, an E=3575 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=3865 ksf was selected for Stratum L. This compares to a value range of 500 ksf ≤ E_s ≤ 5000 ksf for sandy clay in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

Stratum L is characterized as a clay and the elastic modulus E requires adjustment for drained, effective stress, long term loading conditions using Equation 2.5S.4-8A. For Stratum L, the value of Poisson's ratio for drained condition μ_d=0.15 based on Reference 2.5S.4-14B and the resulting E_d=2965 ksf. The selected E_d values for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) for clayey soils is related to the drained modulus, E_d, by Equation 2.5S.4-8. Using μ_d=0.15 for Stratum L, and the value of E_d=2965 selected above, G=1289 ksf was calculated. A value of G=1300 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, few laboratory index tests, and tests for the determination of engineering properties were made on samples from Stratum M. Laboratory test quantities are summarized in Table 2.5S.4-7. The following index tests were performed on Stratum M with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	1	19	19	19
Liquid Limit (%)	1	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	1	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	1	55	55	55
Unit Weight (pcf)	1	116	116	116

For engineering purposes, Stratum M soils were characterized, on average, as sand with silt to silty sand (based on visual classifications). 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 (employing the value from Sub-stratum K Sand/Silt due to the limited quantity of laboratory unit weight tests on Stratum M).

No SPT N-values specific to Stratum M are available; an $(N_1)_{60}$ value of 40 blows per foot was assigned to Stratum M sand, since it is dense to very dense judging from its shear wave velocity (1165 feet per second for Stratum M sand). A value of $(N_1)_{60}$ equal to 40 blows per foot at the depth of the M sand Stratum would correspond to N_{60} = 100 blows/foot based on C_N=0.4.

In the absence of laboratory strength test data and SPT N-value data specific to Stratum M, a drained friction angle of ϕ'=31 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 assigned average corrected SPT N_{60}-value for Sub-stratum K Sand/Silt soils (100 blows per foot, an E=4700 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=4175 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=4350 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 E value for sandy layers is appropriate for the effective stress condition. The shear modulus, G, was related to E by Equation 2.5S.4-5, re-ordered to solve for G if E and μ are known. Using E=4350 ksf and μ=0.30 for sand, G=1673 ksf is calculated. A G=1675 ksf was selected for Stratum M 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 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 347 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 5 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 borings B-305DH/DHA and B-405DH. Neither B-305DH/DHA nor B-405DH was determined to have fully-penetrated sub-stratum N Clay 6. 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. 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 energy ratio of 60 percent (i.e-N_{60}) by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed. A summary of corrected SPT N_{60}-values, for all site areas and all soil strata is presented in Table 2.5S.4-6. The average corrected SPT N_{60}-value for Sub-stratum N Clay was 56 blows/foot. An SPT N_{60}-value of 54 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT N_{60}-values, Stratum N Clay is hard.

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 (%)	17	17	38	25
Liquid Limit (%)	16	33	92	67
Plasticity Index (%)	16	22	65	46
Fines Content (%)	10	22	98	79
Unit Weight (pcf)	9	113	132	123

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 79%. Note that the minimum 22% fines content reported occurred at Boring B-405 from depths of 318 feet to 320 feet. This result represents an isolated thin sand lens within Sub-stratum N Clay. 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 123 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_{60}-values. The results of this evaluation are summarized in Table 2.5S.4-9.

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_{60}-value for Sub-stratum N Clay (54 blows/foot), an s_u=6.8 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. The ratio of the shear strength of Substratum N clay samples to the vertical effective stress at the depth the sample was taken for the UU and UNC tests ranges from 0.01 to 0.15 in Table 2.5S.4-9. Reference 2.5S.4-14A indicates that this ratio should be 0.31 for clay soils of the Beaumont formation at OCR=1, ranging upward to 1.2+ at OCR=10. Therefore, UU and UNC test results that produced low ratios of shear strength to vertical effective stress are considered likely to have been disturbed or to have failed prematurely due to the presence of desiccation features such as slickensides and thus are unrepresentative. The three lowest laboratory strength test results have ratios of 0.01, 0.02, and 0.07. By excluding these three lowest laboratory strength test results, an s_u=4.5 ksf resulted (one test result). Laboratory shear strength test results are summarized in Table 2.5S.4-9 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 highest of the laboratory derived s_u results from this subsurface investigation was more representative, and an undrained shear strength of s_u=4.5 ksf was selected for Sub-stratum N Clay.

CIU and CIU-Bar triaxial strength tests were not performed on Sub-Stratum N Clay.

Likewise, the drained friction angle of Sub-Stratum 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. The OCR value is not reasonable, as OCR should be one or, more realistically, higher than one on a deeply buried ancient layer such as Sub-stratum N Clay. 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=1.3 and a preconsolidation pressure of 37 ksf were selected for Sub-stratum N Clay.

The elastic modulus (E) for Sub-stratum N Clay was evaluated using Equation 2.5S.4-4B. Substituting the previously established s_u and OCR for Sub-stratum N Clay soils (s_u=4.5 ksf and OCR=1.3), an E=5130 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=23 pcf for Sub-stratum N Clay, and 50% reduction of the seismic modulus for Sub-stratum N Clay, an E=9220 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=7855 ksf was selected for Sub-stratum N Clay. This compares with a value range of 500 ≤ E_s ≤ 5000 ksf for sandy clay in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

Sub-stratum N Clay is characterized as a clay and the elastic modulus E requires adjustment for drained, effective stress, long term loading conditions using Equation 2.5S.4-8A. For Sub-stratum N Clay, the value of Poisson's ratio for drained condition μ_d=0.15 based on Reference 2.5S.4-14B and the resulting E_d=6020 ksf. The selected E_d values for all soil strata are shown in Table 2.5S.4-14.

The shear modulus (G) for clayey soils is related to the drained modulus, E_d, by Equation 2.5S.4-8. Using μ_d=0.15 for Stratum N Clay, and the value of E_d=6020 selected above, G=2617 ksf was calculated. A value of G=2620 ksf was selected for 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. 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.

The uncorrected SPT N-values from each boring were corrected to an energy transfer ratio of 60 percent by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections for rod length and sampler (C_s=1.2) leading to values of N_{60}. A summary of SPT N_{60} values for all site areas and all soil strata is presented in Table 2.5S.4-6. The average N_{60} value for Sub-stratum N Sand was 167 blows/foot; N_{60}=141 blows/foot for Substratum N Sand was selected for engineering purposes as shown in Table 2.5S.4-6.

As noted above, corrected SPT N_{60}-values in sandy strata from each boring were corrected to an effective overburden pressure of one atmosphere (approximately one tsf) leading to fully-corrected values of $(N_1)_{60}$. A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all sandy soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Sub-stratum N Sand was 67 blows/foot. An SPT $(N_1)_{60}$-value of 56 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 very dense.

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 (%)	12	17	28	22
Liquid Limit (%)	47	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	47	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	12	5	49	21
Unit Weight (pcf)	4	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 21%. 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 Equation 2.5S.4-12C and the selected corrected SPT $(N_1)_{60}$-value for Sub-stratum N Sand (56 blows/foot), a value of ϕ'=of 50+ degrees (for fine to coarse sand) was estimated. 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)	50+	---	---

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 corrected SPT -N_{60}-value for Sub-stratum N Sand soils (141 blows per foot) an E=6625 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=14,155 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=11,645 ksf was selected for Sub-stratum N Sand. This compares to a value range of $E_s \leq 1700$ ksf for dense sand in Reference 2.5S.4-55. Note that the selected values of E for all soil strata are shown in Table 2.5S.4-14.

The E value for sandy layers is appropriate for the effective stress condition. The shear modulus, G, was related to E by Equation 2.5S.4-5, re-ordered to solve for G if E and μ are known. Using E = 11,645 ksf and μ = 0.30 for sand, G = 4479 ksf is calculated. A value of G = 4470 ksf was selected for 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.

For modeling layers deeper than Stratum N, and thus below the deepest extent of the subsurface investigation, refer to Subsection 2.5S.4.2.1.14.

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-six 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, 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 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. 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 subsurface investigation plan met the intent of Reference 2.5S.4-19.

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. 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 performed Resonant Column Torsional Shear (RCTS) testing.

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 Subsurface Investigations (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 84 feet below nominal post-construction plant grade), 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 required. This was in accordance with the subsurface investigation technical specification. X-ray imaging, however, has been performed on undisturbed samples selected for RCTS testing.

2.5S.4.2.2.2.1 Initial Field Investigations

The STP 3 & 4 subsurface investigation was performed onsite between October 2006 and January 2007 and in the Summer of 2008. 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 132 borings with SPT sampling and collecting in excess of 200 undisturbed samples (using the Shelby push sampler or the rotary 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 44 CPTs, including six 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 13 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), chemical analyses (pH, sulfate content, and chloride content), and RCTS testing.
- 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

2.5S.4.2.2.2.2 Field Investigation 2008

A third field investigation was conducted in June 2008. This investigation focused on the relocated UHS Basins, UHS Pump Houses, RSW Tunnels, and Diesel Generator Fuel Oil Storage Vaults for Units 3 & 4. These structures are relocated south of each unit. This investigation included:

- 32 soil test borings to depths of 180 to 300 feet
- 11 offset borings to collect relatively undisturbed samples (39 samples collected) and for pressuremeter testing in two of the offset borings.
- Ten drill rigs were used for the field work. At least three hammer energy measurements were made on each drill rig.
- Pressuremeter testing in two offset borings to supplement previous field and laboratory data in the F Clay stratum.
- Boring logs were prepared for each boring.
- A laboratory testing program on disturbed and relatively undisturbed samples was conducted and consisted of the following tests:

- 34 moisture content tests
- 22 grain size distribution tests (sieve and hydrometer)
- 22 Atterberg limits tests
- 17 specific gravity tests
- 6 unconsolidated-undrained triaxial shear tests
- 8 consolidated-undrained triaxial shear tests
- 6 one-dimensional consolidation tests

- A data report was prepared presenting the information above (Reference 2.5S.4-2C).

Figures 2.5S.4-2, 2.5S.4-4, 2.5S.4-5, 2.5S.4-7, and 2.5S.4-9 have been revised based on the relevant boring information from this investigation.

The information obtained from this investigation was reviewed and compared with he existing information from previous investigations (References 2.5S.4-2, 2.5S.4- 2A and 2.5S.4-2B) from which the geotechnical parameters used for analyses were selected. From this comparison it was concluded that the field and laboratory data and results from this investigation are within the range of the previous investigations and as such, the geotechnical parameters selected for use are applicable to the relocated UHS Basins, UHS Pump Houses, RSW Tunnels and Diesel Generator Fuel Oil Storage Vaults.

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 References 2.5S.4-2, 2.5S.4-2A, 2.5S.4-2B, and 2.5S.4-2C. 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 solid or 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. Thirteen 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 1.5-feet of boring depth) to approximately 15 feet below ground surface. (One boring in each power block (B-322C in STP-3, B-422C in STP-4) was continuously sampled (every 2.5 feet) from 15 feet to 100 feet). 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 References 2.5S.4-2, 2.5S.4-2A, 2.5S.4-2B, and 2.5S.4-2C. Upon completion, each boring was tremie-grouted back to the ground surface using a cement-bentonite grout mixture.

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 duct 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 References 2.5S.4-2, 2.5S.4-2A, 2.5S.4-2B, and 2.5S.4-2C.

Energy measurements were made on the SPT hammer-rod systems on each of the 13 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 References 2.5S.4-2, 2.5S.4-2A, 2.5S.4-2B, and 2.5S.4-2C.

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.

Forty-four CPTs were performed, with termination depths ranged from approximately 36 feet to 100 feet below ground surface, including six seismic CPTs (C-305S, C-306S, C-307S, C-405S, 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.

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, 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, 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 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 .

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 . 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 performed.

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/or Visual-Manual Procedure 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)
- Dynamic Soil Response Testing
  - 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 area. The boring logs are contained in Reference 2.5S.4-2, 2.5S.4-2A, and 2.5S.4-2C.

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 six 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 .

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. Subsequent to the initial five CPTs, an additional CPT (C405S) was performed; the results from this CPT fit within the range of values for the initial CPTs.

Based on all 10 suspension P-S velocity logging locations and 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 six locations, namely CPT C-305S, C-306S, C-307S, C-405S, 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 .

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	283	1,078	637
B	595	910	745
C	640	1,006	848
D	618	1,331	843
E	760	2,378	1,315
F	760	1,246	1,023
H	983	1,814	1,188

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 proportioned to fit the average strata boundaries selected for use (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.

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 5.7 million cubic yards of materials are moved during earthworks to establish site grade inside the STP 3 & 4 Power Block area, comprising 3.5 million cubic yards of excavation and 2.2 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 . 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 2.2 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. Several potential sources have been identified and, once selected 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. Further details are provided in subsection 2.5S.4.5.3.

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

2.5S.4.5.2.1 Excavation

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. Earthwork operations are conducted to achieve the proposed site grades, as shown on the excavation plan on

Figure 2.5S.4-48 and Figures 2.5S.4-48A through 48C. The safety-related structures are contained inside the STP 3 & 4 Power Block 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):

Structure [1]	Bottom of Excavation (MSL)	Bottom of Mat (MSL)	Predominant Soil Stratum Foundation [2]
Reactor Buildings	-60.3	-50.3	F
Control Buildings	-44.3	-42.3	E
Services Buildings [3]	-50.25 , 32	-14 , 34	Structural Fill
Radwaste Buildings	-39	-23	Structural Fill
Turbine Buildings [3]	-39*	-26 , -8	Structural Fill
UHS Basins	2	4	C
RSW Pump Houses	-30	-28	D
RSW Tunnels	-9.8	-7.8	Structural Fill
Diesel Fuel Oil Storage Vaults	-9	-7	Structural Fill

[1] Seismic Category 1 Structures and/or piping from the table above include: Reactor Buildings, Control Buildings, UHS Basins, RSW Pump Houses, RSW Tunnels and Diesel Generator Fuel Oil Storage Vaults.

[2] Soil strata designation 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 ft. below the bottom elevation of the mat, with over-excavation replaced by concrete fill.)

[3] The Services Building and Turbine Buildings are stepped structures supported primarily on structural fill.

* The Turbine Buildings will have a section of the excavation down to approximately El. -58 to accommodate the installation of the Circulating Water Lines.

As noted above, foundation excavations result in removing approximately 3.5 million cubic yards of soil. The extent of excavation, filling, temporary slopes, and the approximate limits of temporary ground support for major structures are shown in plan on Figures 2.5S.4-48, and 48A through 48C and in section on Figures 2.5S.4-49A through 2.5S.4-49D (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 placed on of very stiff clay stratum (Stratum F), as discussed in Subsection 2.5S.4.10. The excavation at the deepest level (i.e., the underside of over-excavation for the Reactor Buildings at El. 60.3 feet) is approximately 94 feet below nominal post-construction plant 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 the foundations and other underground features, structural fill is extended to 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 no permanent excavation or fill slopes created by site grading. Refer to Subsection 2.5S.5 for additional discussion.

2.5S.4.5.2.2 Excavation Slopes and Benches

For the excavation of STP 3 & 4, temporary side slopes of 2 horizontal to 1 vertical (2H:1V) with 20 foot wide benches (for slope maintenance and drainage), approximately every 20 feet vertically (equating to a composite slope of approximately 3H:1V) are planned. Note that the deepest structure excavations made for STP 1 & 2 construction were approximately 35 feet shallower than the deepest structure excavations proposed for STP 3 & 4 and used a typical 1.5H:1V temporary slope, and a narrower (10 feet wide) bench width, for a composite slope of approximately 2H:1V.

In local areas where the vertical and horizontal spacing is limited, two options are considered:

(1) Maximum side slopes of 1.5H:1V with or without benches, depending on distances, or,

(2) Combination of cut slopes and retained vertical cuts.

An initial evaluation was performed for the deepest cut to determine the minimum factor of safety associated with the overall global slope stability, as well as the stability for each individual slope and combination of intermediate slopes. For each slope(s) analyzed, the following conditions were considered:

(1) Phreatic surface within the excavation slopes during dewatering, i.e. steady state condition.

(2) Surcharge loading applied separately to each bench, with the phreatic surface during dewatering.

Results of the initial analyses indicate a factor of safety of 1.3 or greater for these conditions. A minimum global factor of safety for stability of 1.3 is required. (Reference 2.5S.4-66).

Slope stability analyses were performed with the aid of the computer program GSTABL7 (Reference 2.5S.4-67). Analyses were performed using the modified Bishop circular-arc method, in accordance with conventional soil mechanics practice. The results were checked using the Spencer method. An evaluation of each scenario for the various slopes/cuts is performed for normal construction for a minimum factor of safety for stability of 1.3.

2.5S.4.5.2.3 Retaining Structures for Adjacent Foundations

Excavation plan and sections, Figures 2.5S.4-48, 2.5S.4-48A through 48C and 2.5S.4-49A through 2.5S.4-49B show the approximate limits of temporary ground support. These will remain in place and will not support permanent structural loads. A soil retaining structure is provided for three sides of the STP 3 & 4 Control building foundations. This structure is required due to the proximity and difference in elevation of the Reactor Building foundation to the south and the Turbine Building foundation to the north of the Control Building foundation. 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. -42 feet, to the subgrade levels of the Turbine Buildings at El. -26 feet) that cannot be accommodated by a stable soil slope. A retaining wall will be required on the east side of the Radwaste Buildings to facilitate excavation and construction activities. In order to facilitate the installation of the Circulating Water Pipes under the Turbine Building additional retaining structures will be installed. Both the Turbine retaining structures and the Radwaste retaining structures are anticipated to be left in place and backfill placed around both sides.

2.5S.4.5.2.4 Reinforced Concrete Retaining Walls

At the east edge of the Reactor Buildings and Turbine Buildings, a retaining wall is required to accommodate the reach of a heavy lift crane needed to place the reactor vessels. This crane is capable of performing a 1275 metric tonne lift at a reach of approximately 235 feet.

Non-safety related reinforced concrete retaining walls are installed on the east side of STP 3 and also on the east side of STP 4. The sole purpose for these walls is to facilitate excavation activities. These two walls will retain the soil next to the deep excavations of the Reactor, Control and Turbine Building foundations and allow the crane areas to be at grade and near the buildings. The area on the west side of the retaining walls will be backfilled as construction progresses and the walls will be abandoned in place.

The reinforced concrete retaining walls will vary in exposed height to a maximum of 90 feet. Lateral support of the retaining wall is provided by a tieback and whaler system with horizontal and vertical spacing to be determined by analysis of the wall and soil interaction. The analysis is based on lateral pressure profiles for soil and hydrostatic conditions both during and after construction.

The locations of the walls are shown in plan on Figures 2.5S.4-48, and 2.5S.4-48A through 48B and a typical wall section is shown on Figure 2.5S.4-54.

At grade crane areas are provided east of the STP 3 & 4 Reactor and Turbine Buildings for an equipment setting crane. A pile supported reinforced concrete foundation is provided for each area to support the equipment setting crane. The foundation and piles are designed to accommodate the crane loads and to minimize the surcharge load on the adjacent reinforced concrete retaining wall. The crane areas are shown in plan on Figures 2.5S.4-48, and 2.5S.4-48A through 48B.

2.5S.4.5.2.5 Slurry Cut Off Wall

A slurry cut off wall is utilized for groundwater control. The wall is located outside the foundation and excavation areas, at least 30 feet from the top edge of the excavation, and is continuous around the perimeter. The low permeability wall will hydraulically isolate the excavation inside the wall and allow the excavation to be dewatered, minimizing the effect on the groundwater outside the wall.

The slurry wall is designed to have a minimum permeability of 1 x 10-6 centimeters per second. The backfill material for this wall is select excavated material mixed with bentonite slurry and dry bentonite or borrowed material.

The wall is constructed from a level work pad a minimum of 4 feet above the existing groundwater table. The top of this work pad is the top of the finished slurry wall. The slurry wall around the main STP 3 & 4 excavations will have a total depth of approximately 125 feet from top of work pad to bottom of key and is keyed a minimum of 3 feet into the J Clay stratum. The slurry wall around the southern circulating water lines excavation will have a total depth of approximately 60 feet from top of work pad to bottom of key and is keyed a minimum of 3 feet into the D clay stratum. The slurry wall is 3 feet to 5 feet wide. The slurry cut off wall is shown in plan on Figures 2.5S.4-48, 2.5S.4-48A through 48C and Figure 2.5S.4-50.

2.5S.4.5.2.6 Dewatering Wells

A dewatering system is installed inside the slurry wall. A more detailed description of this system is in Subsection 2.5S.4.5.4 "Dewatering and Excavation Methods".

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, including grain size and plasticity characteristics, moisture-density relationships, and dynamic properties. For foundation support and for backfill against walls, structural fill is compacted to a minimum of 95% of its maximum dry density and within + or -3% of its optimum moisture content, as determined based on the modified Proctor compaction test procedure (Reference 2.5S.4-42).

A clayey soil layer will be placed above the granular structural backfill around the structures within the excavation area. The clay layer will be a minimum of 2 feet thick and comprise the layer between the granular backfill and the final surface treatment (i.e., crushed stone, paving, etc.). The clay layer will minimize any flood water infiltration into the groundwater table.

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.

Quality control for structural fill placement includes observation of borrow area excavation, moisture conditioning, and compaction. Representative samples of the structural fill material are selected and tested to verify that material classification and compaction characteristics are within range of the materials specified and used for design. Prior to the delivery of the material to the project site, each off-site source of backfill will be sampled at the source and tested for compliance with the specifications. Tests will include grain size (ASTM D6913), organic matter (ASTM D2488) and compaction tests (ASTM D-1557). Testing of materials sampled at the source will also include consolidation (ASTM D2435), triaxial shear (USACE Procedure) and Resonant Column Torsional Shear (RCTS) (University of Texas procedure PBRCTS-1).

The results of the triaxial shear tests will be evaluated to determine that the strength of the material will be at least as good as the values used in the engineering analyses of lateral earth pressure and bearing capacity.

The granular structural backfill will be relatively low in compressibility and therefore no specific acceptance criterion is applied. The results of the consolidation tests will be evaluated to determine that the compression of the fill layers results in settlement consistent with values computed during design.

The results of the RCTS tests will be evaluated to determine that the low strain shear modulus of the material, when placed and compacted, will lie within the range used in the analysis for soil-structure interaction and also that the modulus and damping variations with shear strain are within the range used for the analysis.

The materials from each source will be stockpiled separately to permit sampling and verification of the material properties before placement. These tests will include grain size (ASTM D6913) and organic matter (ASTM D2488). Additional compaction tests (ASTM D1557) at the site will be performed on samples obtained from the backfill material as it is placed for compaction.

Prior to placing backfill in the excavation for the plant structure, a test fill pad will be constructed on-site using the equipment and granular fill materials to be used in the backfill. The test pad will be used to confirm requirements for the size of compaction equipment, number of passes, lift thickness and other relevant data for achieving the specified compaction. The low strain shear wave velocity achieved in the test pad will be measured in-situ using surface wave and downhole methods.

Prior to placing the materials as backfill, an engineering report will be prepared to confirm that the materials, construction equipment and methods used to construct the test pad are capable of producing acceptable and consistent results.

Depending on the on-site handling of the material, moisture content adjustment may be necessary to achieve proper compaction. If water is added, it is uniformly applied and thoroughly mixed into the soil by discing. Testing of the backfill material during construction is required to verify that the engineering properties are compatible with the pre-construction qualification testing. Periodic density testing is performed on compacted fill as the material is placed. A quality control sampling and testing program inclusive of the items provided by Table 2.5S.4.5.3-1 is implemented during placement of the structural fill. This quality control sampling and testing program verifies that the structural fill is placed in accordance with the design parameters described in this Subsection.

Table 2.5S.4.5.3-1 Quality Control Recommendations for Structural Fill

Material	Test	Minimum Sampling and Testing Frequency[1]
Structural Fill	Field Density	For backfill placed in trenches and surrounding structures: Minimum 1 sample per 200 cubic yards placed, sample taken at suspect areas, and at least one per every lift. Elsewhere: Minimum 1 sample per 500 cubic yards placed, sample taken at suspect areas, and at least one per every lift.
	Moisture	One test for each Field Density test
	Moisture-Density Relationship (Modified Proctor)	One test for every borrow area and material type and any time material type changes. Additional test for every 10 Field Density test (ASTM D1557)
	Gradation	One test for each Moisture-Density test. (ASTM D 6913)
	Atterberg Limits	One test for each Moisture-Density test. (ASTM D 4318) for backfill types appropriate for this test.
	Material Type	Soil must come from an approved borrow source. Other soil sources must be tested and approved.

Note 1: Consistent with the requirements of NQA-1 (1994) Subpart 2.5, the need for each specific test shall be established in site-specific construction specifications. In-process tests shall be performed more frequently if the test results are erratic, or if the trend of results or an apparent change in material characteristics indicates that the frequency should be increased. These test frequencies shall be considered minimum unless documentary

test data are available to establish adequate confidence in conformance with specification requirements.

The following laboratory tests will be performed on samples of the proposed granular fill materials before they are approved for use. An engineering report will be prepared to confirm that the granular fill material will produce a backfill having acceptable engineering properties.

Test	Minimum No. of Tests	Criterion for Acceptance Unless Approved by Engineer of Record
Grain Size ASTM D6913	1 per material type per source	Complies with Specifications
Organic Matter ASTM D2488	1 per material type per source	Complies with Specifications
Specific Gravity ASTM D854	1 per material type per source	Complies with Specifications
Modified Proctor ASTM D1557	1 per material type per source	Maximum Dry Density Will Result in a Saturated Total Unit Weight ≥120 lb/ft^3
Constant Head Permeability ASTM D2434	1 per material type per source	Complies with Specifications
pH ASTM G51	1 per material type per source	Complies with Specifications
Chloride Content EPA SW-846 9056/300.0	1 per material type per source	Complies with Specifications
Sulfate Content EPA SW-846 9056/300.0	1 per material type per source	Complies with Specifications
Resistivity ASTM G 57	1 per material type per source	Complies with Specifications
Consolidated Drained Triaxial Shear USACE EM-1110-2-1906 Appendix X (30 Nov. 70)	1 per material type per source	$\phi' \geq 30°$
Consolidation ASTM D2435	1 per material type per source	Compression of fill layer results in settlement consistent with values computed during design
Resonant Column Torsional Shear University of Texas Procedure PBRCTS-1	1 per material type per source Test at 4 to 6 isotropic confining stress values	Maximum shear modulus, modulus ratio, and damping ratio consistent with upper range and lower range values used for soil-structure interaction analysis

The static and dynamic properties of structural fill are presented in Tables 2.5S.4.5.3-2, 2.5S.4.5.3-3, and 2.5S.4.5.3-4 and Figures 2.5S.4-80 and 2.5S.4-81. The following criteria are required for structural fill placement beneath and around the STP Units 3 & 4 Seismic Category I Structures:

- The on-site equipment includes earthwork equipment for both drying and wetting of soils
- Materials selected for use as structural fill are free from roots and other organic matter, trash, debris, frozen soil, and stones larger than 6 inches in any dimension. The following soil types are considered unsuitable for use as structural fill: PT, OH, OL, MH, ML, CL, and CH (Referenced from Unified Soil Classification System).
- Suitable structural fill soils of the types (SM, SC, SW and GW) are placed in accordance with specifications developed following testing. The soil is compacted by mechanical means such as steel drum, tamping, or rubber-tired rollers.
- Structural fill is compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557) to within 3 percent of the optimum moisture content.

Table 2.5S.4.5.3-2 Engineering Parameters for Backfill

Parameter	Acceptance Criteria
Grain Size Distribution	Well graded granular material Percent passing #200 sieve < 25 percent
Total Unit Weight	≥ 120 lb/ft^3
Phi Angle	≥ 30 degrees
Shear Modulus Reduction	Within UB and LB in Figure 2.5S.4-80
Damping Ratio	Within UB and LB in Figure 2.5S.4-81

Table 2.5S.4.5.3-3 Representative Range for Grain Size Distribution of Backfill

Sieve Size		Acceptance Criteria for Percent Passing
1 inch	25.4 mm	100
¾ inches	19 mm	97 – 100
No. 4	4.75 mm	30 – 100
No. 40	425 µm	10 – 100
No. 100	150 µm	0 – 65.
No. 200	75 µm	0 – 25

Table 2.5S.4.5.3-4 Dynamic Engineering Parameters for Backfill

	Shear Modulus Parameter K2 and Damping Ratio Values for SSI Analyses					
			Mean			
Shear Strain γ (%)	Lower Bound Parameter K2 (2)	Upper Bound Soil Damping Ratio (4,5)	Parameter K2 (1)	Soil Damping Ratio (4,5)	Upper Bound Parameter K2 (3)	Lower Bound Soil Damping Ratio (4,5)
0.0001	45	0.7	67	0.5	101	0.3
0.0003	44	1.2	66	0.9	99	0.4
0.0005	43	1.6	65	1.2	97	0.5
0.001	42	2.7	63	1.7	94	0.7
0.003	37	5.5	56	3.1	84	1.4
0.005	34	7.3	51	4.0	77	1.9
0.01	29	9.9	43	5.6	65	2.8
0.03	20	14.9	29	9.6	44	5.1
0.05	15	17.5	23	12.0	35	6.8
0.1	11	20.9	16	15.4	24	9.8
0.3	5	25.6	8	20.7	11	15.5
0.5	3	26.9	5	22.8	8	18.0
1	2	27.9	3	24.5	5	21.1

Notes:

1) For parameter K2 see Figure 2.5S.4-80. K2 values are for relative density, Dr = 85% (interpolated from Dr =75% and Dr = 90%) from Figure 5 of Report EERC 70-10, December 1970 by Seed and Idriss.

2) K2 values for Lower Bound are K2 values for Mean divided by 1.5.

3) K2 values for Upper Bound are K2 values for Mean multiplied by 1.5.

4) Damping values are from Figure 10 of Report EERC 70-10, December 1970 by Seed and Idriss. Note that Upper Bound damping values shall be used with Lower Bound Shear Modulus and Lower Bound damping values shall be used with Upper Bound Shear Modulus.

5) Damping values used in analysis shall not exceed 15% per SRP 3.7.2.

Lateral pressures applied against the below grade Nuclear Island walls are evaluated and discussed in Subsection 2.5S.4.10.3. Evaluation and discussion of liquefaction issues related to the structural fill materials is provided in Subsection 2.5S.4.8.

Measurements of the shear wave velocity will be made when the backfill surface has reached an elevation corresponding to placement of approximately half the total backfill thickness below the Category I structures, when the backfill surface has reached the elevation corresponding to approximately the foundation (base of concrete fill) level of the structure and at the finish grade elevation. The shear wave velocity measurements will be analyzed and compared to the acceptance criteria in COLA Part 9, Table 3.0-11. The shear wave velocity measurement methods will be the SASW technique and at least one of the following techniques: Seismic CPT (downhole) or crosshole.

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.

2.5S.4.5.4.1 Dewatering Method

Due to the existing groundwater condition at the site, groundwater control within the excavations is required during excavation and construction. Groundwater control for the excavations will include a dewatering system and a perimeter slurry wall. The slurry wall will aid in reducing the magnitude of the dewatering system.

The dewatering system is designed to lower and maintain the free-water and hydrostatic pressures inside the construction and foundation area to a minimum of at least 3 feet below the bottom of the excavation..

Dewatering is accomplished with a series of perimeter deepwell systems installed outside the excavation slopes and inside the slurry wall. The initial dewatering rate is estimated to be 6700 gpm and is expected to decline due to the slurry wall. These deepwells are installed to the top of the J Clay stratum. Depending on the hydrostatic pressure in the J Sand 1 layer, a series of deepwells for pressure relief wells may be required for the deepest excavations. If the pressure relief wells are required, a series of recharge wells may be installed outside the slurry wall to maintain the groundwater levels outside the slurry wall.

The overall system will have sufficient capacity to accomplish this desired result allowing for normal variations in soil properties and foundation conditions. The entire dewatering system consists of a combination of deepwells, recharge wells, jet eductors, sand drains, wellpoints, pumps, standby pumps, sumps, sump pumps, trenches, and necessary appurtenances capable of achieving the design requirements to dewater or to depressurize the major water-bearing strata. Figure 2.5S.4-50 shows the anticipated location of the dewatering systems. Figures 2.5S.4-51 through 53 show the typical sections and details for each type of dewatering system that may be utilized.

Due to the nature of the clay and sand materials, sumping may be required to handle any seepage, trapped water, perched water, or surface water on top of these formations. A system of shallow drains and/or ditches is utilized inside and outside the excavation to collect and direct minor seepage to sumps. This system will also be utilized to handle storm water that will enter the excavation. Sand drains may also be installed to allow the trapped and/or perched water to migrate to the lower permeable formations that are pumped by the active dewatering systems. The effluent from the dewatering well system will be controlled, and discharged into drop structures. The discharge points are located in the existing MCR.

The dewatering plan, Figure 2.5S.4-50, shows the anticipated dewatering zones. Figures 2.5S.4-51 and 52 show typical sections and details for each type of dewatering system that may be utilized. Figure 2.5S.4-53 shows a typical section of the slurry wall and dewatering systems. Due to the magnitude of the excavation, it is anticipated that a series of deepwells, sand drains, eductors and/or wellpoints may also be required within the excavation to maintain the piezometric levels a minimum of 3 feet below the bottom of the excavation and 5 feet below the faces of the slopes to achieve a minimum slope stability factor of safety of 1.3. Multiple piezometers or monitoring wells are installed inside and outside the slurry wall to monitor the effect the dewatering system will have on the groundwater elevation. The monitoring system is established to measure and evaluate the effectiveness of the dewatering system and to assess the stability of the cut slopes. Instruments are monitored as necessary to provide data.

Threshold values for instrument measurements are established to alert key project personnel of potential changed conditions or that the dewatering system is not operating as planned. If any of the limits are exceeded, the monitoring representative will immediately notify the resident engineer and the construction manager.

2.5S.4.5.4.2 Slurry Wall Installation

The slurry wall is installed utilizing the "slurry trench" method. This method consists of excavating a "one bucket wide" trench that is continuously filled with "slurry". The slurry exerts positive hydrostatic pressure against the trench wall, thereby maintaining vertical excavation sidewalls, even below the groundwater table, which enables the placement of a low permeability backfill to create a groundwater cut-off wall.

The sequence of operations for slurry wall construction includes:

- Bentonite Slurry Production
- Slurry Trench Excavation
- Backkfill Mixing
- Backfill Placement

2.5S.4.5.4.3 Excavation Method

The subsurface conditions consist mainly of clay and sand allowing for the excavations to be constructed using conventional earth-moving equipment. Excavations are planned primarily as open cuts, with limited temporary ground support, as described above in Subsection 2.5S.4.5.2.

Dewatering wells and a slurry cut off wall will allow for a dry open cut mass excavation for each unit. Mobile excavation and haul equipment is used to excavate, load and transport the excavated material to stock piles. Earth moving equipment will excavate and load haul trucks. The haul trucks will utilize the access roads and haul roads to transport the excavated material to stock piles. Dozers and graders will assist in the excavation and in cutting the slopes and benches. The clay and sand may be segregated, loaded and hauled to separate stock piles for possible reuse.

For the deepest portions of the excavations, conveyors may be used to lift the excavated material to a higher elevation and possibly to the top of the excavation at grade. The material would then be loaded and hauled to the stock piles.

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. General compaction specifications are discussed in Subsection 2.5S.4.5.3. Specifications will also include, among other things, measures such as proof-rolling, over-excavation and replacement of unsuitable soils, and protection of surfaces from deterioration. Excavations are to comply with applicable OSHA regulations (Reference 2.5S.4-48).

2.5S.4.5.4.4 Concrete Retaining Wall Installation

The concrete retaining walls are installed utilizing the "slurry trench" method. This method consists of excavating a "one bucket wide" trench that is continuously filled with "slurry". The slurry exerts positive hydrostatic pressure against the trench wall, thereby maintaining vertical excavation sidewalls, even below the groundwater table, which enables the placement of reinforcing and concrete.

The anticipated sequence for construction of the retaining wall is as follows:

- A full depth and width slurry excavation is made with the excavation being maintained by the slurry
- Reinforcing is placed in the slurry filled trench
- Concrete is placed by tremie in the excavation from the bottom up
- As the site construction excavation proceeds on the west side of the wall, tiebacks and whalers are installed

At grade, crane foundations are installed at each of the heavy lift crane areas. Auger cast or slurry displaced drilled shafts are drilled and installed. A reinforced concrete pile cap is installed on the piles. The pile cap will provide a stable foundation for the heavy lift crane.

2.5S.4.5.4.5 Quality Program and Monitoring

Quality Assurance (QA) is covered by the STP 3 & 4 Quality Assurance Plan Description provided in Section 17.5S.

A formal Field Quality Control (QC) Plan is developed and issued as a project document prior to commencement of field work. This plan is reviewed and approved for its completeness in monitoring materials and installations of materials and assessing their performance through the duration of the project. The purpose of the plan is to define and establish the requirements for planning, controlling, monitoring and documentation of the work to ensure compliance with the project drawings, specifications, and applicable standards.

The plan defines the actions and monitoring of site inspection activities to provide confidence that engineering design, construction techniques, materials and equipment required to perform the work meet the specific project requirements. The QC plan addresses inspection coordination procedures and verifies that they meet the quality requirements according to the Project Specifications. Critical to this effort are development of Inspection Test Plans and Deliverable Matrices to verify conformance to project requirements though a process of reviews, checks, measurements, tests and inspections.

2.5S.4.5.4.5.1 Excavation

Monuments and benchmarks for the overall site will be established for horizontal and vertical control. These controls will be provided for initial layout, inspection, and checkout to ensure that the work conforms to lines, grades, cross sections and details indicated on Project Documents. Primary control points will be protected throughout the duration of work.

Excavations will meet the alignments, grades, dimensions, and shape shown on the Construction Drawings. The Geotechnical Engineer will verify that the final bearing elevations are founded within the existing proper soil stratum. Retaining walls will be adequate to minimize loss of adjacent ground. Excavated areas will be protected against flooding from adjacent areas.

Excavations and embankments will be kept shaped and effectively drained. Matting, plastics, or gunite may be employed as preventative erosion control methods. Appropriate dust control measures will be implemented. Once final subgrade is accepted, concrete base materials, "concrete fill", will be utilized as necessary to control the effects of subgrade moisture change and general degradation during construction activities.

The QC program will use several methods to ensure the quality of materials and installation procedures. These inspection and testing methods may include the use of field soil density monitoring, laboratory soil testing, survey control, concrete test methods, monitoring of groundwater level fluctuations, and continuous visual inspections. The QC program will define the continuous monitoring, collection and distribution of data available which indicates how dewatering and earth retaining structures are performing throughout the construction operation.

2.5S.4.5.4.5.2 Slurry Wall

The slurry wall is utilized to control groundwater infiltration into the excavation and to control the lowering of the groundwater table outside the excavation.

Quality control procedures during installation will ensure that the wall is properly located. Field sampling from the installation areas and subsequent laboratory testing is used to determine the mixing proportions to ensure a proper installation. The slurry wall depth is determined using onsite testing along the installation perimeter to define the termination depth to ensure adequate keying into the designated clay strata. During installation, inspection of excavated samples will confirm termination into the clay strata.

2.5S.4.5.4.5.3 Retaining Walls

Inspection of the materials for the retaining walls will be in accordance with the project specifications. Once the material has been accepted, the excavation will commence and an engineered tieback system will be installed as required and tested. A tilt monitoring system will be implemented to provide periodic measurements of wall movements throughout the construction phase. The QC plan will define the criteria for collection of monitoring data throughout open excavation duration to ensure retaining structures are performing as designed.

2.5S.4.5.4.5.4 Monitoring

The construction monitoring program includes monitoring the following:

- Selected STP 1 & 2 major structures and selected lengths of the Main Cooling Reservoir (MCR) earth dike
- Heave at bottom of excavations
- Piezometric levels within and outside the slurry wall
- Horizontal and vertical control
  - Side slopes
  - Retaining walls

Heave of the excavation bottoms are monitored by the installation of borehole heave points or extensometers. These are installed prior to any excavation to measure movement of the bottom due to the relief of the overburden soils. Subgrade rebound estimates are addressed in Subsection 2.5S.4.10.

A system of piezometers is installed throughout the excavation areas to monitor the groundwater level for the various structures. Additional piezometers are installed outside the slurry wall between the potential recharge system and the existing STP 1 & 2. This second set of piezometers are installed to monitor and measure the influence the dewatering system has on the groundwater levels in the various layers outside the excavation and slurry wall zones. Measurements in the piezometers are taken as is necessary.

Survey benchmarks and monuments are established to monitor horizontal and vertical movements of the excavation slopes and retaining wall. Inclinometers and tiltmeters are installed and monitored as necessary.

The monitoring program is in effect before and during excavation and construction activities. Specifications are developed for the gathering of instrumented data and its interpretation. The quantity of instruments varies depending on the size and depth of the excavation. Frequency of instrumentation readings will be established based on the rate of material removal during excavation and the construction activities.

Threshold values for instrument measurements are established to alert key project personnel of potential destabilizing conditions. If the values are exceeded, the monitoring representative will immediately notify the resident engineer and the construction manager. Contingency plans are developed for the excavation and dewatering items being monitored to include base line ranges, frequency of monitoring, evaluation of monitoring data, actions to address any changed conditions, and required response time for corrective or recovery action. Contingency actions may consist of the following:

- Instability of bottom excavation → Increase dewatering to lower the groundwater below the excavation
- Excess pore pressures/piezometric readings → Increase dewatering to lower the groundwater
- Excessive movement of slope → Flatten the slope or increase dewatering to lower the groundwater
- Excessive movement of retaining wall → Install additional tiebacks

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 monitoring plan has been prepared for the construction of STP 3 & 4. The monitoring plan has been designed to address possible impacts to the existing plant structures by examining water level changes in sealed and open tube piezometers. The current site has an extensive groundwater monitoring program and the existing wells and piezometers will be incorporated into the STP Units 3 & 4 groundwater monitoring plan. Observation wells installed during the site investigation work will also be included.

At the site, the strata of interest are the C and E sands (shallow aquifer) and H and J sands (deep aquifer). The existing and new piezometers are sited around STP Units 3 & 4 to provide sufficient data to understand the possible changes in groundwater levels as construction progresses.

The groundwater data from the piezometers related to the excavation of Units 3 & 4 will be collected manually or remotely. Baseline data will be collected prior to the start of construction and data will also be collected at assigned intervals during construction.

This groundwater data will be evaluated for impact to foundation subgrade stability and for impact to existing structures. A course of action will be implemented, as needed.

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.

2.5S.4.6.2 Construction-Stage Dewatering

2.5S.4.6.2.1 Dewatering

Temporary dewatering is required for groundwater control during the project construction stage. The dewatering system is designed to lower and maintain the free-water and hydrostatic pressures inside the construction and foundation area to a minimum of at least 3 feet below earth slopes and excavation surfaces, inclusive of the interior slopes of any retaining structure embankments. The perimeter slurry wall will also aid in the dewatering during construction. A more detailed discussion is presented in Subsection 2.5S.4.5.4. Upon completion of the requirement for dewatering, the groundwater will be monitored during the decommissioning of the system. The dewatering system will be turned off and removed in a sequence that will allow the groundwater to return to the original elevation in a controlled and gradual manner.

2.5S.4.6.2.2 Storm Water System for Excavated Area

The storm water/sumping system will consist of a series of trenches, sumps, pumps and piping inside the excavation area that will collect and remove the storm water conveyed through the trenches to the sumps located on various benches of the slopes and at the subgrade for the different Structures, and have the ability to transfer that discharge water to a temporary detention basin.

The trenches, sumps, pumps and piping is designed to handle a 25-year 24-hour event as indicated on the Technical Paper No. 40 Charts provided by the U.S. Dept. of Commerce and the Weather Bureau. The detention basin is designed to handle a 25-year 24-hour event and allow for a 2-hour detention time. The location of the storm water detention pond is anticipated to be located between STP 3 & 4 and the MCR and west of the new circulating cooling lines. The storm water will then be pumped into the MCR.

Additional excavation storm water protection will consist of a temporary berm that is installed around the perimeter of the entire excavation for STP 3 & 4. This berm will hinder storm water from the surrounding water shed from entering into the excavation area. Since there may be a lag time between the excavations for STP 3 & 4, a temporary berm is installed between the two excavations so that the storm water system for STP 3 or 4 will not be required to handle the capacity of the entire excavation area.

2.5S.4.6.3 Analysis and Interpretation of Seepage

The use of a slurry wall around the perimeter of the excavation will aid in minimizing groundwater seepage into the excavation. In order to control potential piping from the J Sand 1 stratum, pressure relief wells or additional dewatering may be utilized. The groundwater is also lowered at least 3 feet below the subgrades as indicated in Subsection 2.5S.4.6.2.1 to minimize the potential that piping will occur. Seepage, trapped water, perched water, and surface water is handled by the storm water system described in Subsection 2.5S.4.6.2.2. As the excavation progresses, seepage quantities will be monitored and the need for additional dewatering systems will be evaluated.

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.

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 six CPTs, three at the STP 3 area and three 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 . 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).

Soil unit weight information is limited at depths greater than 600 feet, with available information from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) provided 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, STP conducted a search of geophysical logging results (especially sonic logs) from 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 19,900 feet below ground surface).

The average shear wave velocity obtained from converting the data in the three sonic logs was used for the deep layers as input to the site response analysis. These average shear wave velocities (and average +/- 1 standard deviation) are plotted versus depth in Figure 2.5S.4-57. Based on the conversion, in general, the shear wave velocity profile is as follows:

- At a depth of 2,500 feet the sonic logging data showed the shear wave velocity to be in the range of 2,900 to 3,200 feet/second. This range continues to a depth of 3,000 feet;
- Increases from 3,000 feet/second at a depth of 3,000 feet to 5,000 feet/second at 6,000 feet depth;
- Decreases to around 3,500 feet/second at an 8,000 feet depth;
- Increases linearly to 5,500 feet/second at an 18,000 feet depth; and
- Increases to about 6,500 feet/second just beyond 18,000 feet depth, then falls back to 5,000 feet/second at a 19,000 feet depth.

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 was performed, 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. A total of 16 undisturbed soil samples, from depths of 10 feet to 590 feet below ground surface, were subjected to RCTS tests. A summary of the samples tested is included in Table 2.5S.4-31. Prior to these test being completed and the results becoming available, the, shear modulus versus shear strain curves from available literature were used for dynamic soil properties characterization. Once the laboratory testing was completed an evaluation was made of the RCTS test-derived modulus reduction and damping curves and a comparison made with the literature-derived curves.

A total of 16 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 completed RCTS tests are discussed here and compared with the selected (literature) curves.

2.5S.4.7.3.1 Selected Shear Modulus Degradation Curves from Literature

Generic shear modulus curves for cohesionless soil or sub-strata strata B, C, E, H, J Sand/Silt, K Sand/Silt, M, and N Sand were developed from Reference 2.5S.4-49, based on stratum depths. The depths of soil strata or sub-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.4-32. 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.4-58, with numerical values given in Table 2.5S.4-32. Note 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.4-32.

2.5S.4.7.3.2 Selected Damping Ratio Curves from Literature

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.4-33. 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.4-33.

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.4-33.

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 Measured Shear Modulus Degradation and Damping Ratios for Soils

A summary of the results of the RCTS tests is provided in Table 2.5S.4-34, with comparisons of individual test results to the selected (literature) curves given on Figures 2.5S.4-62 through 2.5S.4-68. 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.

Details of the RCTS results are contained in Reference 2.5S.4-2A. The soil samples subjected to RCTS tests were divided into the following categories:

1) Sand
   a) Deep sand with depth greater than 105 ft
   b) Shallow sand with depth no more than 105 ft
2) Clay
   a) High PI clay with PI greater than 30
   b) High PI clay with PI no more than 30
3) Silt

2.5S.4.7.3.4 Comparison of Selected and Measured Shear Modulus Degradation for Soils

The shear modulus degradation (G) curves for all the sand samples are presented in Figure 2.5S.4-62, along with the EPRI curves derived versus depth in Reference 2.5S.4-50. Note that the results plotted on Figures 2.5S.4-62 through 2.5S.4-65 are those results obtained using a confining pressure equal to or very close to the in-situ confining pressure. Figure 2.5S.4-62 shows that at the same strain level the normalized shear modulus (G/G_{max}) generally increases with depth. The one exception is sample B306-UD3 located at 75 ft depth, which is considered an outlier. The G curves for the deep sand samples are also presented in Figure 2.5S.4-63, along with the average of the deep curves, and the EPRI curves. The G curves of the deep soil samples generally agree with each, and their average is close to the EPRI curve for depth = 500 – 1000 ft. Figure 2.5S.4-62 shows that soil sample B-306-UD-6 located at the depth of 104.7 ft is consistent with the EPRI curve for depth = 250 - 500 ft. Therefore, it is recommended that the following shear modulus degradation curves be used for sand strata at the STP site:

- For sands located at depths greater than 100 ft, use the EPRI curve for depth = 500 - 1000 ft
- For sands located at depths less than 100 ft, use the EPRI curve for depth = 250 - 500 ft

The shear modulus degradation curves for all the high PI clay samples are presented in Figure 2.5S.4-64, along with their average, which is quite close to Vucetic & Dobry (1991) curve for PI = 100 (Reference 2.5S.4-65). The shear modulus degradation curves for the low PI clay sample and the silt sample are presented in Figure 2.5S.4-65. Based on comparison between the test curves and the published curves, it is recommended that the following shear modulus degradation curves be used for clay and silt:

- For clays with PI greater than 30, use the Vucetic & Dobry curve for PI = 100
- For clays with PI less than 30, use the Vucetic & Dobry curve for PI = 50.
- For silt, use the EPRI curve for PI = 50.

2.5S.4.7.3.5 Comparison of Selected and Measured Damping Ratio Curves for Soils

The damping ratio (D) curves for all the sand samples subject to TS tests are presented in Figure 2.5S.4-66, along with the average of the curves. The results consistently show that the D curves are close to the EPRI curve for depth = 500 - 1000 ft. Therefore, it is suggested that the following D curve be used for all the sands:

- For all sands, use EPRI curve for depth = 500 - 1000 ft

The D curves for high PI clay are presented in Figure 2.5S.4-67, along with their average, which is quite consistent with the Vucetic & Dobry curve for PI = 200.

- For clays with PI greater than 30, use the Vucetic & Dobry curve for PI = 200

The D curves for the low PI clay sample and the silt sample are presented in Figure 2.5S.4-68. These curves do not consistently follow any single EPRI or Vucetic & Dobry curve. At strains below about 0.005%, the test curves are close to the Vucetic & Dobry curve for PI = 200. However, at higher strains, the slope for both tests becomes steeper and is closer to the EPRI PI = 50 or PI = 70 curves.

- For low PI clay and silt samples, use Vucetic & Dobry curve for PI = 200 up to strains of 0.005% and use EPRI interpolated PI = 60 curve for strains above 0.05%.

2.5S.4.7.3.6 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.4.7.3.7 Dynamic Properties of Structural Fill

Confirmation that the properties of proposed fill material under Seismic Category I structures meet the values of engineering parameters used in the site-specific design analyses of these structures will be documented in an engineering report as required by COLA Part 9, Section 3.0, Table 3.0-11, Backfill Under Category I Structures.

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.4.8.2.6.

As described in Subsection 2.5S.4.7.5, the peak horizontal ground surface acceleration of 0.10gand a Moment Magnitude 7.7 earthquake was selected for use in liquefaction analysis. 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.

Reference 2.5S.4-5 contains the so-called "Chinese Method" to assess the vulnerability to liquefaction or serious loss of strength in clayey soils. For the remaining soils, the state-of-the-art (as defined in Reference 2.5S.4-5) 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 0.935 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 Liquefaction Assessment of Clayey Soils

Laboratory tests and field performance data have shown that the great majority of clayey soils will not liquefy during earthquakes. Criteria to express these observations have been formulated as contained in Reference 2.5S.4-5 and are hereafter referred to as the "Chinese Method". The criteria state that clayey soils which satisfy all of the three following conditions should be judged to be vulnerable to liquefaction or serious loss of strength during a seismic event:

- Laboratory-determined water content greater than 90 percent of the laboratory-determined liquid limit;
- Liquid limit is less than 35 percent; and
- Clay content (<0.005 mm) is less than 15 percent.

The criteria are generally applicable to fine-grained soils (more than 50 percent of particles passing the No. 200 sieve). Initially, the criteria are assessed for both the fine-grained and silty and clayey sand soils below the water table (which is also a necessary condition for liquefaction to occur) for which test data are available. The liquid limit and natural water content data are assessed first, as they are the most abundant. If they indicate no liquefaction susceptibility, assessment by the clay content criterion is not necessary.

Liquid limit and natural water content for SM, SC, ML, MH, CL, and CH samples are available from References 2.5S.4-2B and 2.5S.4-2C, and are assessed to check their liquefaction potential according to the above criteria for clayey soils. The application of the criteria to the individual samples for which data are available show that the vulnerability of the clayey fine-grained soil, as well as the clayey sands (SC), to seismic liquefaction is negligible (see Figure 2.5S.4-78). Nevertheless, the clayey sand (SC) samples are also assessed by the other methods (SPT, CPT, V_s). Those samples having liquid limit = 0 (NP) on Figure 2.5S.4-78 are SM and ML samples and also are assessed by other methods (SPT, CPT, V_s) discussed later herein. All soil types except CL and CH are assessed by other methods.

A total of 299 samples for which test data are available were assessed according to the Chinese Method. Based on the liquefaction assessment by the water content and liquid limit of the 299 samples, it is judged not necessary to assess the clay content for 295 (98.7%) of the samples, as the first two conditions are sufficient to show the clay soils are not vulnerable to liquefaction or severe loss of strength (see Figure 2.5S.4-78).

For the remaining four of the 299 samples assessed:

- One sample (Boring U4-3; El. -28.8 feet; Stratum D) has water content equal to 20.9% and liquid limit equal to 23%. The sample has measured clay content equal to 27%, and thus is greater than 15%, meaning the sample is not vulnerable to liquefaction or serious loss of strength.

- One sample (Boring U4-3A; El. -54.4 feet; Stratum F) has water content equal to 26.2% and liquid limit equal to 30%. The clay content of the sample was not measured, but 14 other samples in Stratum F were tested and have an average clay content equal to 65%, with the minimum measured clay content equal to 32%. Thus, the sample is judged not vulnerable to liquefaction or serious loss of strength.
- One sample (Boring B-443; El. -133.9 feet; Sub-stratum J Clay 2) has water content equal to 26.2% and liquid limit equal to 24%. Laboratory grain-size analysis classifies the sample as ML, with the clay content equal to 15%. The sample is also assessed by the SPT method discussed later herein, from which the factor of safety against liquefaction is 3.75. Thus, the sample is not vulnerable to liquefaction or serious loss of strength.
- One sample (Boring B-306; El. -35.2 feet; Stratum E) has water content equal to 21.9% and liquid limit equal to 20%. Laboratory grain-size analysis classifies the samples as SM, with the clay content equal to 10%. The sample only marginally crosses the threshold shown on Figure 2.5S.4-88 and is judged to be an outlier. An adjacent sample at El. -31.7 feet shows a factor of safety against liquefaction equal to 3.63 when assessed by the SPT method discussed later herein.

Thus, the clay soils at the site are judged not vulnerable to liquefaction or serious loss of strength during a seismic event.

2.5S.4.8.2.3 FOS Against Liquefaction Based on SPT Data

Uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10, 2.5S.4-12, 2.5S.4-14, and 2.5S.4-15 for the STP 3 area, the STP 4 area, the former UHS Basin/RSW area (the area west of the Power Block, and for the area outside the Power Block, now outside the Power Block), and for the remaining area outside the Power Block,respectively. Uncorrected SPT N-values verses elevation are presented on Figures 2.5S.4-11 and 2.5S.4-13 for Boring B-305DH/DHA and Boring B-405DH, respectively. SPT data from all 68 borings made within the STP 3 area, all 60 borings made within the STP 4 area, all 26 borings made within the former UHS Basin/RSW area (west of the Power Block), and all 11 borings made within the remaining area outside the Power Block were evaluated for liquefaction potential. For completeness, all SPT N-values, including those measured in clay soils (CH, CL) and those measured in soils above the groundwater level are identified, but the FOS is not calculated for the clay soils; clay soils (consisting of CL and CH materials) were evaluated by the Chinese Method (subsection 2.5S.4.8.2.2) and were found to be not vulnerable to liquefaction. Of the 4944 total SPT tests, 2965 tests, or 60.0% of the total, are CL or CH samples which are not liquefiable.

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.4-5, (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.4-69 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, 1205 tests of 4944 total tests, or approximately 24.4% of tests, had $(N_1)_{60c\ s} \geq 30$ blows/foot.

Of the 4944 SPT N-values, all but 11 tests were either CL and CH soils not liquefiable by the Chinese Method, or were other soil types including ML soils that had FOS ≥ 1.10 (refer to Subsection 2.5S.4.11 for discussion on the selection of an appropriate FOS). The 11 tests having FOS<1.10 amounted to 0.2% of all the STP tests evaluated; in other words, 99.8% of the SPT samples were either not vulnerable or had calculated FOS values that exceeded 1.10. For completeness, an examination of each FOS < 1.10 is provided in Table 2.5S.4-34. From Table 2.5S.4-35, it can be noted that: seven of the 11 tests were within areas/depths excavated for structure foundations; one of the 11 tests as within areas where no structureis placed, and where soils at similar elevations in adjoining borings had minimum FOS=1.54 the three remaining tests are discussed separately next.

For the remaining three of the 11 tests:

- One test (Boring B-337; El. 5.8 feet; Stratum C) occurred at shallow depth at the STP 3 Machine Shop, which is not a safety-related structure; Note that soils at similar elevations in adjoining borings had minimum FOS=1.42)
- One test (Boring U3-5; El. -193.5 feet; Sub-stratum K Sand/Silt) occurred at the location of the STP 3 UHS Basin, which is a safety-related structure. Excavation plans indicate that the soil at this location will be excavated to El. 2.0 feet, thus the low-FOS soil encountered will remain below the foundation of the STP 3 UHS Basin. Other SPTs in the K Sand/Silt Sub-stratum of adjoining borings at the UHS Basin had minimum FOS = 1.38.
- One test (Boring T3-7; El. -190.6 feet; Sub-stratum K Sand/Silt) occurred at the location of the STP 3 RSW Tunnel, which is a safety-related structure. Excavation plans indicate that the soil at this location will be excavated to El. -50.0 feet, thus the low-FOS soil encountered will remain below the STP 3 RSW Tunnel. Other SPTs in the K Sand/Silt Sub-stratum of adjoining borings had minimum FOS = 1.45.

The two SPT samples in the K Sand/Silt Sub-stratum described above have computed FOS values < 1.10. These soils are geologically old and on this basis could potentially be declared immune to liquefaction, or at least more resistant than shown by their computed FOS values, as described in Reference 2.5S.4-53 and Reference 2.5S.4-5. Reference 2.5S.4-5 notes that detailed information to assess the effect of geologic age when evaluating liquefaction behavior in terms of a quantitative FOS calculation is generally not available. Reference 2.5S.4-5 notes that sometimes the effect of geologic age is at least partially accounted for by the factor $K_{\underline{\sigma}} = 1$ in geologically old materials that are being evaluated using SPT (or CPT) for a FOS calculation. The value of K_σ used when computing the FOS at the two SPTs being discussed was:

Boring	Elevation	K_σ
U3-5	-193.5 feet	0.632
T3-7	-190.6 feet	0.648

If K_σ were assigned as 1 to partially account for the geologic age of these two SPT samples with low FOS values, the resulting FOS would be 1.74 for boring U3-5 at El. -193.5 feet, and 1.60 for boring T3-7 at El. -190.6 feet:

Thus, if the geologic age of the Sub-stratum K Sand/Silt had been at least partially accounted for by assigning K_σ = 1, the resulting FOS values would be greater than 1.10. On the basis of its geologic age and depth below the ground surface, the low FOS calculated for two of the individual SPT samples in the K Sand/Silt Sub-stratum are judged to be of no concern.

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

2.5S.4.8.2.4 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.4.8.2.5.

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 former UHS Basin/RSW area (area west of the Power Block, now outside the Power Block), and for the remaining 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, 21 CPTs made within the former UHS Basin/RSW area (C-947 was excluded due to erroneous data) and the one CPT made within the remaining 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 included in the FOS calculation spreadsheet, despite their known high resistance to liquefaction. The CPT method identifies clay soils by their soil behavior type index, Ic, and no FOS is calculated for clay-rich soils. The spreadsheet also is set to identify soils above the water table, and no FOS is calculated for soils above the water table.

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.4-70 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, 1013 tests of 6272 total tests, or 16.2% 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 2576 tests of 6272 total tests, or 41.1% of tests, had $I_c \geq 2.60$ and thus are considered too clay rich to liquefy.

Of the 6272 CPT values, all but 176 tests had $I_c \geq 2.60$, are above the water table, or had FOS ≥ 1.10. The 76 tests having FOS < 1.10 amounted to 2.8% of all the tests evaluated; in other words, 97.2% of the CPT tests have $I_c \geq 2.60$, are above the water table, or had calculated FOS values by this method that 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: 55 of the 176 tests were within areas/depths excavated for structure foundations, 101 of the 176 tests were within areas where no structures are placed and 20 of the 176 tests were made at locations where non-safety related structures are planned, but design details such as foundation elevations are unknown at this time. None of the CPT tests that had FOS < 1.10 are located where the soil will remain in-place beneath safety-related structures.

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

2.5S.4.8.2.5 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 all three CPTs (C-405S, C-406S and C-407S) 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 included in the FOS calculation, despite their known high resistance to liquefaction.

Shear wave velocity measurements provide no information about the soil classifications, so it is possible that liquefaction FOS values may indicate soil liquefaction potential when, in fact, the soil in which the measurement was taken is a CL or CH material which is not vulnerable to liquefaction. For P-S logs, the shear wave velocity data are associated with soil type and fines content by examination of the SPT boring log, in the same way that fines content and soil type were determined for the SPT method. FOS values by the V_s method in the P-S logs were calculated for all soil types, including CL and CH clay soils which are not liquefiable. For the seismic CPTs, the fines content at approximately the middle of the V_s measurement interval was estimated from the CPT tip resistance and sleeve friction data, but no filter was imposed for clay-rich soil ($I_c > 2.60$). FOS values were calculated from the V_s in seismic CPTs without considering soil type other than estimated fines content. Low FOS values by the V_s method in the seismic CPTs were evaluated by examination of the associated I_c results to determine if they were in a soil type that is too clay-rich to be liquefiable.

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 the V_s method is not applicable at depths exceeding about 40 feet based on the recommendation in Reference 2.5S.4-5B, page 120. Therefore, V_s data at depths more than about 40 feet are not used to evaluate liquefaction. This left a total of 287 V_s tests to be utilized. Note that at STP 3 & 4, 150 tests of 287 tests utilized, or 52.3% of tests, had $V_{s1} \geq V_{s1}$*.

Of the 287 V_s values, all but 19 tests were above the water table, in clay soils, or had FOS ≥ 1.10. The 19 tests having FOS < 1.10 amounted to 6.6% of all the tests evaluated; in other words, 93.4% 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.4-37. From Table 2.5S.4-37, it can be noted that: all 19 tests with FOS < 1.10 were within areas/depths to be excavated for structure foundations.

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

2.5S.4.8.2.6 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 more than 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 the 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. Higher liquefaction resistance would be expected from these deeper soils.

On these bases, liquefaction of STP 3 & 4 site soils more than a depth of 600 feet below ground surface was not considered possible.

2.5S.4.8.2.7 Spatial Distribution of Liquefaction FOS Values

Tables 2.5S.4-35, 2.5S.4-36, and 2.5S.4-37 summarize the low liquefaction FOS values obtained by the SPT, CPT, and V_s methods, respectively, including the layer name and the range in elevation at which low FOS values were encountered. Figure 2.5S.4-79 identifies the soil borings and CPT locations in which FOS values < 1.10 were encountered in Stratum A through K Sand/Silt. No low FOS values were encountered in strata below Stratum K Sand/Silt. The information presented in Figure 2.5S.4-79 is discussed in this subsection.

Stratum A has no low FOS values by the SPT method. Stratum A has low FOS values by the CPT method at 13 locations scattered about the site. Stratum A has low FOS values by the V_s method at three locations. Stratum A is not used for support of any safety related structures.

Stratum B has low FOS values by the SPT method in four borings. Stratum B has low FOS values by the CPT method at 13 locations scattered about the site. Stratum B has low FOS values by the V_s method in four borings. Stratum B is not used for support of any safety related structures.

Stratum C has low FOS values by the SPT method in four borings (each low FOS value occurs at a single test depth in the respective boring). Stratum C has low FOS values by the CPT method at eight locations across the site with varying thicknesses. Stratum C has low FOS values by the V_s method at three borings scattered about the site with varying thicknesses. None of the low FOS locations in Stratum C will remain in-place beneath safety-related structures.

Stratum D has no low FOS values by the SPT or V_s method. Stratum D has low FOS values by the CPT method at 17 locations scattered about the site. Stratum D is characterized as predominantly a fine grained layer and, as such, would not be expected to experience liquefaction. None of the low FOS locations in Stratum D will remain in-place beneath safety-related structures.

Stratum E has no low FOS values by the SPT method. Stratum E has a low FOS value by the CPT method at one location, within the STP-3 Turbine Building. The low FOS location will be removed by the construction excavation. The V_s method is not applicable to Stratum E soils (and deeper strata) because they occur at depths greater than the recommended depth of approximately 40 feet (per Reference 2.5S.4-5B).

Stratum F has no low FOS values by the SPT method. Stratum F has a low FOS value by the CPT method at one location, within the Machine Shop northwest of the STP-4 Power Block. The V_s method is not applicable to Stratum F soils and below.

Sub-stratum K Sand/Silt has two low FOS values by the SPT method. Sub-stratum K Sand/Silt was not penetrated by the CPT, and thus no FOS values by the CPT method are available. Sub-stratum K Sand/Silt was not penetrated by the CPT The V_S method is not applicable to Sub-stratum K Sand/Silt soils.

No layer deeper than Sub-stratum K Sand/Silt indicated low FOS values by the SPT method. No layer deeper than Sub-stratum K Sand/Silt was penetrated by the CPT. The V_S method is not applicable at these depths.

The arrangement of the soil borings and CPT locations where low FOS values were computed do not indicate spatial "clustering" of the low FOS values horizontally between adjacent borings or vertically between strata. Reference 2.5S.4-5 identifies the SPT method as the most reliable of the three methods. The SPT method indicates low FOS values at four borings in Stratum B, four borings in Stratum C, and two boring in the K Sand/Silt Sub-stratum. No other strata have low FOS values by the SPT method. The low FOS values in Stratum B and Stratum C are either to be excavated during construction, or are associated with single test depths in the borings and do not occur in adjacent borings, and are thus shown to be of limited lateral and vertical extent. Two of the low FOS locations by the SPT method will remain in-place beneath safety-related structures. These occurred in the K Sand/Silt Sub-stratum and, as discussed in Subsection 2.5S.4.8.2.3, are judged to be of no concern.

2.5S.4.8.2.8 Concluding Remarks

A liquefaction analysis was performed using state-of-the-art procedures outlined in Reference 2.5S.4-5. Liquid limit, water content, and clay content data of fine grained soils were evaluated using the Chinese Method and showed that the clayey soils are not vulnerable to liquefaction or serious loss of strength. Even though the liquid limit and water content data available for clayey sands (SC) indicate that they are not vulnerable to liquefaction, all soil types other than CL and CH were evaluated for liquefaction behavior by other methods (SPT, CPT, V_S). SPT data points, 4544 total, were analyzed from 165 borings, from which 99.8% of the samples were either CL, CH, located above the water table, or hadcalculated FOS values that exceeded 1.10. CPT data points, 6272 total, were analyzed from 43 CPTs, from which 97.2% of the tests indicated too clay-rich to liquefy, were from above the water table, or had calculated FOS values that exceeded 1.10. Finally, shear wave velocity (V_S) data points to a maximum depth of about 40 feet, 287 total, were analyzed from 10 suspension P-S velocity logging borings and six seismic CPTs, from which 93.4% of the tests were either associated with clay-rich soil types, were from above the water table, or had calculated FOS values that 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. In the liquefaction evaluation, the effects of overconsolidation and geologic age were generally not considered, both of which tend to increase resistance to liquefaction. 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).

2.5S.4.8.2.9 Conformance 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 clayey the finer-grained STP 3 & 4 site soils are not vulnerable to liquefaction or to serious loss of strength according to the Chinese Method in Reference 2.5S.4-5. In the liquefaction analyses by the SPT, CPT and V_S methods, the groundwater level for calculation purposes was selected at El. 25.5 feet for evaluating Strata A-D. This groundwater level is likely a “perched” condition within Stratum D, as measured in the Stratum C sand (refer to Figures 2.5S.4-55 and 2.5S.4-56). For evaluating Stratum E and deeper layers, a lower water level was used as measured in the deeper Stratum E sand at an average El. 16.5 feet (El. 17.0 was used) (also refer to Figures 2.5S.4-55 and 2.5S.4-56). 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 test that had FOS < 1.10, detailed above, is additionally of limited thickness and/or lateral extent. The spatial distributions of low FOS<1.10 locations are dispersed around the site and do not define clusters or local areas of weak soil.

Under “Procedures for Evaluating Liquefaction Potential,” Reference 2.5S.4-52 lists CPT, SPT, cyclic triaxial, and shear wave velocity tests as acceptable methods. 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. The CPT, SPT, and shear wave velocity test results were used in these liquefaction potential analyses.

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 Power Block is approximately El. 34 feet. As noted above, the Reactor Buildings, Control Buildings, UHS Basins/RSW Pump Houses, the RSW Tunnels, and the Diesel Generator Fuel Oil Storage Vaults 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, Radwaste Buildings, and the Service Buildings, are not Seismic Category I structures, and are, therefore, not considered here.

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. Given the position of the groundwater level (once construction dewatering is terminated and the groundwater level recovers) and the foundation depth, buoyancy effects on the foundations must be considered. The foundation and fill elevations shown below may vary slightly from analysis input values shown in tables in this section. As noted in Subsection 2.5S.4.5.2.1, foundation elevations are be subject to minor changes.

Structure [1]	Approximate Foundation Dimensions (feet)	Foundation El. [2] (feet)	Foundation Depth [2] (feet)	Estimated Pressure for Bearing Calculations (ksf)	Estimated Pressure for Settlement Calculations (ksf)
Reactor Buildings	187.7 by 197.5	-50.3 {-60.3}	84.3 {94.3}	15.0	12.74
Control Buildings	183.8 by 78.8	-42.3 {-44.3}	76.3 {78.3}	15.0	7.51
UHS Basins	312.0 by 164.0	4.0 {2.0}	30.0 {32.0}	8.9	7.4
RSW Pump Houses	170.0 by 94.0	-28.0 {-30.0}	62.0 {64.0}	6	5.02
RSW Tunnels	17.0 wide	-7.8 {-9.8)	41.8 {43.8}	3	2.49
Diesel Generator Fuel Oil Storage Vaults	40.0 by 73.5	-5.0-7.0 {-9.0}	41.0 {43.0}	2.1	1.74

[1] All structures listed above are Seismic Category I structures.

[2] At the Reactor Buildings, Stratum F is over-excavated 10 feet below the underside of foundations, with over-excavation replaced by concrete fill. Foundation elevations and depths shown in "{ }" symbols denote base of significant over-excavation with the over-excavation to be replaced with concrete fill. At the Control Buildings, UHS Basins, RSW Pump Houses, RSW Tunnels, and Diesel Generator Fuel Oil Storage Vaults, underlying soil will be over-excavated 2 feet below the underside of the foundation and the over-excavation replaced by concrete fill.

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.The properties of structural fill are taken from the STP 1 & 2 UFSAR (Reference 2.5S.4-3). Structural fill properties were taken as: saturated unit weight (γ) of 134 pcf, static elastic modulus (E) of 3000 ksf; drained friction angle (ϕ') of 43 degrees (36 degrees was used in the bearing capacity analyses for conservatism), and drained cohesion (c') of 0 ksf (Reference 2.5S.4-3). The moist unit weight (unsaturated, as compacted) of structural fill above the water table was estimated based on information in Reference 2.5S.4-3. A moist unit weight of 121 pcf was assumed for the structural fill.

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.4-71 through 2.5S.4-74C. Associated elevations and soil properties for these profiles are shown in Tables 2.5S.4-37A, 38A, 39A, 40A, 40C and 40E. 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). For depths below El. -570 feet, strata boundary and soil properties estimated for the deep layers were used, extending the geotechnical model for settlement estimates down to El. -2466 feet. Refer to Subsection 2.5S.4.2.1.14 for information on these deep layers. Bearing capacity calculations considered construction phase loading conditions with groundwater artificially lowered. Based on measurements at STP 3 & 4 there was an upper groundwater level at El. 25.5 feet (refer to Subsection 2.5S.4.6.1). This uppermost groundwater level at El. 25.5 feet (Subsection 2.5S.4.6.1) was assumed to be perched on the upper clay layers; it was assumed to be eliminated in the vicinity of the STP 3 & 4 structures when these upper clay layers are removed and replaced with granular structural fill. Based on measurements at STP 3 & 4, the piezometric level in the deeper layers (Stratum E Sand) was at El. 17.0 feet. A future groundwater level at El. 17.0 in the granular backfill adjacent to the Category I structures was assumed as a long term loading condition in the bearing capacity estimates for the Category I structures. Future groundwater elevations higher than El. 17.0 feet may occur, and would increase the buoyant loading of the structures and increase the bearing capacity factor of safety.

2.5S.4.10.3 STP 3 & 4 Bearing Capacity Evaluation

The ultimate bearing capacity, q_{ult}, of a foundation was calculated by Hansen's equations (Reference 2.5S.4-55):

$q_{ult} = cN_cS_cN_c + qN_qS_qd_q + 0.5\gamma_eBN_\gamma S_\gamma d_\gamma R_b$ Equation 2.5S.4-15

If $\phi = 0$, use

$q_{ult} = 5.14c(1 + S_c' + d_c') + q$ Equation 2.5S.4-15A

where,

c = undrained shear strength of the soil (s_u)
q = effective overburden pressure at the foundation base
γ_e = effective unit weight of the soil
B = foundation width
S_c, S_q, and S_γ are shape factors
d_c, d_q, and d_γ are depth factors
N_c, N_q, and N_γ are bearing capacity factors
R_b is a reduction factor for large foundation size (See Equation 2.5S.4-21A)

$$N_c = \frac{N_q - 1}{\tan(\phi)}$$ Equation 2.5S.4-15B

$$N_q = e^{(\pi - \tan(\phi))}\left[\tan\left(45 + \frac{\phi}{2}\right)\right]^2$$ Equation 2.5S.4-15C

$$N_\gamma = 1.5(N_q - 1)\tan(\phi)$$ Equation 2.5S.4-15D

For rectangular foundations, the shape factors were given by Reference 2.5S.4-55 as:

$$S_c' = 0.2\left(\frac{B}{L}\right) \text{ where } \phi = 0\text{, else}$$ Equation 2.5S.4-16

$$S_c = 1 + \left(\frac{N_q}{N_c}\right)\left(\frac{B}{L}\right)$$ Equation 2.5S.4-16A

$$S_q = 1 + \sin(\phi)\left(\frac{B}{L}\right)$$ Equation 2.5S.4-17

$$S_\gamma = 1 - 0.4\left(\frac{B}{L}\right) \geq 0.60$$ Equation 2.5S.4-18

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

For rectangular foundations, the depth factors were given by Reference 2.5S.4-55 as:

$$d_c' = 0.4k \text{ for } \phi = 0\text{, else} \qquad \text{Equation 2.5S.4-19}$$

$$d_c = 1 + 0.4k \qquad \text{Equation 2.5S.4-19A}$$

where,

$$k = \frac{D_f}{B} \text{ if}\left(\frac{D_f}{B}\right) \le 1\text{;and}$$

$$k = \tan^{-1}\left(\frac{D_f}{B}\right) \quad \text{if}\left(\frac{D_f}{B}\right) > 1 \qquad \text{Equation 2.5S.4-19B}$$

D_f = the depth of soil beside the foundation.

$$d_q = 1 + 2\tan(\phi) \times k(1 - \sin(\phi))^2 \qquad \text{Equation 2.5S.4-20}$$

$$d_\gamma = 1.00 \qquad \text{Equation 2.5S.4-21}$$

For large foundations with width, B, greater than 6 feet, Reference 2.5S.4-55 recommends R_b as follows:

$$R_b = 1 - 0.25\log\left(\frac{B}{6}\right) \qquad \text{Equation 2.5S.4-21A}$$

The factor of safety (FOS) against exceeding the ultimate loading that can be sustained by the soil at the bottom of the foundation, or the bottom of concrete fill, where present, is calculated as follows:

$$FOS = \frac{Q_{ult}}{Q_{gross} - Q_{uplift}} \qquad \text{Equation 2.5S.4-22}$$

where,

Q_{ult} = (qult)(bearing area)

$Q_{gross} = Q_{gross\_building} + Q_{gross\_concrete\ fill} + Q_{gross\_fill}$

$Q_{gross\_building}$ = building load input including any seismic increment to vertical load

$Q_{gross\_concrete\ fill}$ = (volume of concrete fill)(150 pcf)

Q_{gross_fill} = weight of soil backfill above fill concrete or mat foundation exposed beyond building wall perimeters.

$Q_{uplift} = Q_{uplift\_concrete\ fill} + Q_{uplift\_fill} + Q_{uplift\_building}$ = Buoyancy effect

$Q_{uplift\_concrete\ fill}$ = (bearing area of concrete fill)(thickness of concrete fill)(unit weight of water)

Q_{uplift_fill} = (bearing area of concrete fill – bearing area of mat foundation)(height from groundwater level to bottom of mat foundation)(unit weight of water)

$Q_{uplift\_building}$ = (building area)(height from groundwater level to top of mat foundation)(unit weight of water)

The volume and soil bearing area contact dimensions for concrete fill were based on right rectangular prism shapes extending beyond (outside) the base mat dimensions by an amount equal to the thickness of the concrete fill for the Reactor Buildings and Diesel Generator Fuel Oil Storage Vaults. For other buildings, the concrete fill has the same lateral dimensions as the base mat.

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 but acceptable approach is to average the shear strength parameters in the foundation deformation zone, as proposed by Reference 2.5S.4-55 and to use the formulation in Reference 2.5S.4-55 (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.4-75 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 determining the weighted average of cohesion, *c* (s_u), and friction angle,, as follows:

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

$$\tan(\phi) = \frac{\sum \tan(\phi_i) H_i}{\sum H_i}$$ 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'*

$$H' = 0.5B\tan(\alpha) = 0.5B\tan\left(45 + \frac{\phi}{2}\right)$$ Equation 2.5S.4-24A

Equations 2.5S.4-23, 2.5S.4-24, and 2.5S.4-24A were used for deriving average shear strength properties for soils beneath each of the STP 3 & 4 foundations. For bearing capacity estimating, soil layering beneath the foundation edge judged most susceptible was used, rather than the average layering conditions.The material properties derived for each foundation are shown in Tables 2.5S.4-37B, 38B, 39B, 40B, 40D and 40F. Two soil strength cases are considered. The undrained shear strength (s_u) of the clays is a short term condition where the loading is applied so rapidly that the clay does not consolidate (drain) under the applied loading. Secondly, the consolidated undrained effective (CUE) shear strength of the clays is a long term condition wherein the clay has consolidated fully (drained) under the applied loads. The strength of the sand layers was the same in all cases.

The properties of the stronger concrete fill below the foundations were ignored in estimating bearing capacity except to deepen the bottom of the foundation bearing level and, for the Reactor Building and Diesel Generator Fuel Oil Storage Vaults, to distribute the bearing pressure as explained below.

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.4-75. With a 10-foot-thickness of concrete fill, then, the pressure from each Reactor Building was distributed on an area having B = 187.7 feet + 20 feet = 207.7 feet, and L = 197.5 feet + 20 feet = 217.5 feet. Thus, the effective foundation pressure at the base of the concrete fill for each Reactor Building was estimated as {[(15 ksf) (187.7 feet) (197.7 feet)]÷[(207.7 feet) (217.5 feet)]} + (0.150 kcf) (10 feet) = 13.82 ksf, using a unit weight of concrete fill of 0.150 kcf and not accounting for soil backfill above the exposed 10-foot projections of the concrete fill.

Foundation bearing capacities were estimated using the material properties in Tables 2.5S.4-37B, 38B, 39B, 40B, 40D and 40F and using Equations 2.5S.4-15 through 2.5S.4-21. A summary of the material parameters, as well as the derived bearing capacity factors, are shown in Tables 2.5S.4-41A and 2.5S.4-41B. Estimated ultimate bearing capacities and factors of safety are shown in Table 2.5S.4-41B. The results of the analyses show that the factor of safety is equal to or higher than the required minimum for all structures. The FOS values ranging from 3.03 to 123.6 for short term conditions with full backfill in place prior to fuel storage and the water table lowered below the underside of the concrete fill, to a FOS range of 6.0 to 207.6 for long term conditions with full backfill in place and the water table at El. 17.0 feet.

The allowable bearing pressure due to seismic loads would be calculated from the allowable bearing pressure under equivalent static loads. For a transient (dynamic) loading condition applied to the foundation after it has adjusted to its applied static loading, the allowable bearing pressure is computed using the consolidated-undrained (CU) total stress shear strength parameters in the clay soils layers. The effective stress shear strength parameters are used in the sand soil layers.

The bearing capacity calculation for seismic loading utilizes the CU (total) strength parameters for the clay layers, the effective strength for the sand layers and the same bearing capacity equations as for static loading, and a reduced foundation width and length due to the eccentricity caused by the seismic loading. The equation for the reduced foundation width and length is:

$B' = B - 2e_x$, Equation 2.5S.4-24B

$L' = L - 2e_y$, where

B' = Reduced foundation width,

L' = Reduced foundation length,

e_x = eccentricity of load in direction parallel to B, and

e_y = eccentricity of load in direction parallel to L.

The criterion factor of safety (FOS) is 1.5 when dynamic or transient loading conditions such as seismic apply (Reference 2.5S.4-69). The calculated FOS values during dynamic or transient loading for the Reactor Buildings, Control Buildings, UHS/RSW Pump Houses, RSW Piping Tunnels, and Diesel Generator Fuel Oil Storage Vaults are shown on Table 2.5S.4-41C. FOS values for the Diesel Generator Fuel Oil Tunnels are not shown in this table; however, in comparison to the RSW Piping Tunnels and Diesel Generator Fuel Oil Storage Vaults, these structures are lightly loaded. Therefore, considering the large FOS values for the RSW Piping Tunnels and Diesel Generator Fuel Oil Storage Vaults, FOS values for the Diesel Generator Fuel Oil Tunnels are judged to be greater than the required value of 1.5.

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:

$$\delta = \Sigma\left(\frac{p_i}{M_i}\right)h_i = \Sigma\delta_i \qquad \text{Equation 2.5S.4-25}$$

where,

δ = settlement

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

p_i = vertical applied pressure increment at center of layer i (also called σ_{zi} or $\Delta\sigma_i$)

h_i = thickness of layer i

M_i = elastic constrained modulus of layer i

$$M_i = E_{di}\left[\frac{(1-\mu_{di})}{(1+\mu_{di})(1-2\mu_{di})}\right] \qquad \text{Equation 2.5S.4-26}$$

E_{di} = elastic modulus of layer i for drained (long term) conditions

μ_{di} = Poisson's ratio of layer i for drained (long term) conditions

The elastic modulus of the various soil layers was used to represent the soil compressibility for purposes of settlement estimates. This is justified because the soils behave as overconsolidated. Estimated settlements were based on the dewatered condition where the water table was kept artificially lowered at the bottom of the excavation throughout the process of loading the foundation areas. Even with this dewatered condition, the effective stresses in the soil layers did not exceed the preconsolidation pressures except by small amounts in limited locations described later. (The compression of the sub layers in these limited locations was modeled using the consolidation test data as described by Equation 2.5S.4-29). When construction dewatering ends and the water table rises, buoyancy will reduce the effective stresses in all soil layers below the final water table and the final effective stresses will be less and will not exceed the preconsolidation stress. This supports the use of the elastic modulus to model the soil for settlement purposes.

The stress distribution below rectangular, flexible foundations was based on a Boussinesq-type distribution. Reference 2.5S.4-57 provides a convenient equation for performing the calculation that allows the addition of stresses from loaded areas other than the one for which settlements is being calculated.

$$\Delta\sigma_Z = \frac{q}{2\cdot\pi}(T_1 - T_2 + T_3 - T_4 + T_5 - T_6 + T_7 + T_8) \qquad \text{Equation 2.5S.4-27}$$

where,

$\Delta\sigma_Z$ = calculated pressure at depth z
q = applied foundation pressure
z = depth below the foundation from which the pressure is calculated

$$T_1 = \tan^{-1}\left[\frac{(x+a)(y+b)}{z\sqrt{(x+a)^2+(y+b)^2+z^2}}\right]$$

$$T_2 = \tan^{-1}\left[\frac{(x+a)(y-b)}{z\sqrt{(x+a)^2+(y-b)^2+z^2}}\right]$$

$$T_3 = \tan^{-1}\left[\frac{(x-a)(y-b)}{z\sqrt{(x-a)^2+(y-b)^2+z^2}}\right]$$

$$T_4 = \tan^{-1}\left[\frac{(x-a)(y+b)}{z\sqrt{(x-a)^2+(y+b)^2+z^2}}\right]$$

$$T_5 = \left[\frac{z(x+a)(y+b)[(x+a)^2+(y+b)^2+2z^2]}{[(x+a)^2+z^2][(y+b)^2+z^2]\sqrt{(x+a)^2+(y+b)^2+z^2}}\right]$$

$$T_6 = \left[\frac{z(x+a)(y-b)[(x+a)^2+(y-b)^2+2z^2]}{[(x+a)^2+z^2][(y-b)^2+z^2]\sqrt{(x+a)^2+(y-b)^2+z^2}}\right]$$

$$T_7 = \left[\frac{z(x-a)(y-b)[(x-a)^2+(y-b)^2+2z^2]}{[(x-a)^2+z^2][(y-b)^2+z^2]\sqrt{(x-a)^2+(y-b)^2+z^2}}\right]$$

$$T_8 = \left[\frac{z(x-a)(y+b)[(x-a)^2+(y+b)^2+2z^2]}{[(x-a)^2+z^2][(y+b)^2+z^2]\sqrt{(x-a)^2+(y+b)^2+z^2}}\right]$$

Where:

x = coordinate of point in longer direction (with respect to the center of the loaded area) at which the stresses are being calculated

y = coordinate of point in shorter direction (with respect to the center of the loaded area) at which the stresses are being calculated

a = half of the length or width of foundation in longer direction (or concrete fill when concrete fill is longer than foundation)

b = half of the length or width of foundation in shorter direction (or concrete fill when concrete fill is longer than foundation)

z = depth below the bottom of foundation to the mid depth of layer i

In applying Equation 2.5S.4-25 the E-values for the various soil strata are shown in Tables 2.5S.4-37A, 38A, 39A, 40A, 40C and 40E. The μ_{di} values for the strata are given in Table 2.5S.4-16. 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. Because of the large surface area occupied by the various structures (loaded area) including Category I and non-Category I loaded areas, whose stress bulbs overlap at depth, the calculations were extended to include the deep layers (e.g., Table 2.5S.4-37A) beginning at a depth of 527 feet below El. 34 feet and extending to a depth of 2500 feet for any minor contributions they might make to the total settlement. The applied vertical pressure increments, calculated using the Boussinesq distribution and Equation 2.5S.4-26, with contributions from all loaded areas, were added to the vertical effective stresses below the excavation bottom and the result was compared to preconsolidation pressure (P_c') of the various soil strata in Table 2.5S.4-13. Results showed that the strata preconsolidation pressures exceeded the final vertical stresses at the mid-point of each layer, except at a few select depths in the Reactor Buildings, RSW Tunnels and Diesel Generator Fuel Oil Storage Vaults No. 1 in Units 3 & 4, the Control Building in Unit 4, and the Diesel Generator Fuel Oil Storage Vault No. 3 in Unit 3.

The post-construction stresses exceeded the preconsolidation pressures in the Stratum F layer at the southeast and southwest corners of the Reactor Buildings in Units 3 & 4 and at the southeast corner of the Control Building in Unit 4; in the Stratum

F, Stratum J, and Stratum K layers at the centers of the RSW Tunnels in Units 3 & 4; in the Stratum J layer at the centers and west sides of the Reactor Buildings in Units 3 & 4 and north side of the reactor Building in Unit 3; in the Stratum J and Stratum K layers at the south side of the Diesel Generator Fuel Oil Storage Vault No. 1 in Unit 3 and north and west sides of the Diesel Generator Fuel Oil Storage Vault No.3 in Unit 3; in the Stratum K layer at the centers and east and west sides of the Diesel Generator Fuel Oil Storage Vaults No. 1 in Units 3 & 4 and north side of the Diesel Generator Fuel Oil Storage Vault No. 1 in Unit 4, and the center of the Diesel Generator Fuel Oil Storage Vault No. 3 in Unit 3; and in the Stratum K and Stratum L layers at the north edge of the Diesel Generator Fuel Oil Storage Vault No. 3 in Unit 3. Once the buoyancy is considered on the building, the stresses applied will diminish and the post construction stresses will be less than preconsolidation pressure. If the applied vertical stresses were to exceed the preconsolidation pressure, the additional virgin compression of the stratum, ΔS_c, at a particular foundation and layer would be computed using (Reference 2.5S.4-55):

$$\Delta S_c = \frac{\Delta e}{1 + e_0} H \qquad \text{Equation 2.5S.4-28}$$

$$\Delta e = \frac{C_c}{1 + e_0} \log\left(\frac{P_c' + \Delta\sigma_{normal}}{P_c'}\right) \qquad \text{Equation 2.5S.4-29}$$

where, H = thickness of the soil layer

e_0 = initial void ratio

Δe= void ratio change

C_c = compression index

P_c’ = preconsolidation pressure

$\Delta\sigma_{normal}$ = the increment in vertical stress above P_c’

Foundation settlements were calculated based on Equations 2.5S.4-25 through 2.5S.4-29, the subsurface profiles shown in Figures 2.5S.4-71 through 2.5S.4-74C and the material parameters shown in Tables 2.5S.4-37B, 38B, 39B, 40B, 40D and 40F. Settlement estimates, which included the total settlement at the center, corners and the middles of the edges of foundations are shown in Table 2.5S.4-42. Total settlements calculated at the centers of foundations for the Reactor Buildings, were estimated in the range of approximately 10.1 to 10.7 inches. Total settlements calculated at the centers of foundations for the Control Buildings were estimated in the range of approximately 7.8 to 8.3 inches. Total settlements calculated at the centers of foundations for the UHS Basins were estimated to be in the range of 8.2 to 8.5 inches. Total settlements calculated at the centers of foundations for the RSW Pump Houses were estimated to be 7.0 to 7.2 inches. Total settlements calculated at the centers of foundations for the RSW Tunnels were estimated to be in the range of 11.8 to 12.0 inches. Settlements calculated for the Diesel Generator Fuel Oil Storage Vaults were estimated to be in the range of 5.8 to 7.9 inches.

The values presented above are considered ultimate settlements at a point in time after the loading of backfill and adjacent structures are totally applied and the soil has fully adjusted to the applied load. The settlements are for a case that may be interpreted as no settlement occurs until all loads, including backfills, are in place, after which settlement starts and continues until the soil is fully adjusted. The settlement calculations also assume no buoyancy on the structures; buoyancy on the structures during rewatering will reduce the calculated settlement. In order to verify buoyancy effect, a sample calculation was conducted. Water table was located at El. +17.0 feet in the Reactor Building in Unit 3. Water table at other structures remained at the bottom of the concrete fills. Settlement due to the loading of the structure itself, s_{ss}, decreased from 7.13 to 4.26 inches, and settlement due to the consolidation of clay layers for load exceeding the preconsolidation pressures, s_c, decreased from 0.26 to 0.00 inches. Other settlement components remained the same. This example calculation also indicated that the final effective stress in the cohesive soil layer did not exceed the preconsolidation stress due to buoyancy. The settlements of some of the structures are thus overstated to varying amounts depending on the sequence of construction and rewatering.

As an additional consideration, soil rebound or heave resulting from the maximum 90 to 95 feet of excavation (i.e., Reactor Buildings over-excavation to El. -60.3 feet), was estimated, with calculated values in the range of approximately 3.5 inches to 3.7 inches when using the lower bound method, and approximately 6.3 to 6.5 inches when using the upper bound method. Actual soil rebounds are anticipated to vary between the calculated lower bound values and upper bound values, depending on sequence of construction and dewatering. Soil rebounds measured for the STP 1 & 2 Reactor Building foundation excavations, which extended to El. -31 feet, were approximately 4 inches. This value of heave resulted in a calculated “spring” value of approximately 1060 psf per inch of rebound (effective pressure at El. -31 feet before excavation divided by 4 inches of rebound/heave).. Note that soil rebound at selected foundation excavations is monitored during construction.

The settlements described above were calculated assuming a perfectly flexible structure and with no reduction in applied loading due to buoyancy. Reference 2.5S.4-55 (Article 10-4) notes that the rigidity of the superstructure and its mat foundation reduce the differential settlement within the mat to a fraction of the differential settlement between the center and edge calculated for the flexible case. Reference 2.5S.4-55 gives Equation 10-2 thereof for a rigidity factor, K_r, which expresses the ratio of the flexural rigidity of the superstructure and mat to the product of the Young’s modulus of elasticity of the soil multiplied by the cube of an appropriate base width of the foundation perpendicular to the direction of interest. From the rigidity factor, the expected differential settlement on the mat is as follows:

K_r	Expected Differential Settlement
0	0.5 times total settlement(1) for long base 0.35 times total settlement(1) for square base

0.5	0.1 times total settlement(1)
Greater than 0.5	Rigid structure; no differential settlement

(1) For a mat foundation, the total settlement is indicated to be the calculated interior settlement minus the calculated edge settlement.

Total settlements such as calculated in Table 2.5S.4-42 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. Reference 2.5S.4-55A recommends an angular distortion /tilt criterion of 1/500. This value (1/500) includes additional safety factor and will be used as the acceptance criterion for assessing the Seismic Category I structures.

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

Structure	Estimated Maximum Flexible Differential Settlement (inches)(1)	Estimated Maximum Flexible Angular Distortion/Tilt(1)
Reactor Buildings	1.5 to 1.8	1/600 to 1/750
Control Building	1.8 to 2.0	1/400 to 1/450
UHS Basins	2.2 to 2.3	1/650 to 1/700
RSW Pump Houses	0.5	1/1700 to 1/1750
RSW Tunnels	5.0	1/700
Diesel Generator Fuel Oil Storage Vaults (No.1)	0.5	1/1000 to 1/1050
Diesel Generator Fuel Oil Storage Vaults (No. 2)	0.5	1/500 to 1/550
Diesel Generator Fuel Oil Storage Vaults (No. 3)	0.4	1/650 to 1/750

(1) Note that structural rigidity will reduce these values to 0.5 or less of tabulated values.

Foundations evaluated had estimated differential settlements in the range of approximately 0.4 inches to 2.3 inches (measured from center to edge of structure) for the flexible case. From the differential settlement values, angular distortions/tilts were estimated (based on average foundation plan dimension), and for all evaluated structures were within the acceptable limit of 1/500. From the differential settlement values, calculated angular distortion/tilt values for the flexible case exceeded the 1/750 criterion for the special case of foundations supporting sensitive machinery for only the RSW Pump Houses and Diesel Generator Fuel Oil Storage Vaults (No.1). The calculated angular distortion/tilt values were less than the 1/750 criterion for the

Reactor Buildings and Control Buildings, UHS Basins, RSW Tunnels, and Diesel Generator Fuel Oil Storage Vaults No. 2 and No. 3. However, it should be noted that despite the calculated total settlement for the referenced foundations, and the angular distortion/tilt values, actual angular distortion/tilt values are much less even for the flexible case, 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 1/1200 to 1/1500 for the Reactor Buildings, and 1/800 to 1/900 for the Control Buildings, 1/1300 to 1/1400 for the UHS Basins, 1/3400 to 1/3500 for the RSW Pump Houses, 1/1400 for the RSW Tunnels, 1/2000 to 1/2100 for the Diesel Generator Fuel Oil Storage Vault No. 1, 1/1000 to 1/1100 for the Diesel Generator Fuel Oil Storage Vault No. 2, and 1/1300 to 1/1500 for the Diesel Generator Fuel Oil Storage Vault No. 3.These are well within the stricter criterion for the special case of foundations supporting sensitive machinery and the 1/500 limit of Reference 2.5S.4-55A. 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 applied 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 37,070 square feet, or approximately 73% larger than the plan area of an individual STP 1 & 2 structure. In addition, the applied loading of each Reactor Containment Building at STP 1 & 2 was about 9.4 ksf, while the effective foundation pressure of each Reactor Building at STP 3 & 4 is 12.74 ksf at the bottom of the basemat. As anticipated, the STP 3 & 4 larger foundation sizes and higher effective foundation pressures found at STP 3 & 4 are expected to result in larger, but still tolerable, foundation settlements at STP 3 & 4 than those found at STP 1 & 2.

Construction sequencing will be necessary to address the time-rate of settlement for the Category 1 structures. The structural and mechanical considerations (addressed during design) will influence differential settlement tolerances between structures. Experience during settlement monitoring of STP Units 1 & 2 (Reference 2.5S.4-3) will be used to assist with the time-rate of settlement projections. The acceptance criteria for differential settlement between Category 1 structures will be developed during design of these structures and will be consistent with the DCD.

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 $(k_h)(g)$: a peak horizontal ground surface acceleration of 0.10g (refer to Subsection 2.5S.4.7.5) was applied. Vertical ground accelerations $(k_v)(g)$ 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):

$p_{0S} = K_{0S} \cdot \gamma \cdot z$ 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:

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

$p_W = \gamma_W \cdot z_w$ 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,
ΔK_{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:

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

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

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

(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:

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

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.

Hydrodynamic groundwater pressure was considered for active condition. Seismic lateral earth pressure computation for at-rest condition includes hydrodynamic pressure through use of total mass density of the structural fill. The hydrodynamic pressure for active condition was calculated by (Reference 2.5S.4-60):

$$p_{hydro} = (7/8)K_h\gamma_w[H_w z_w]^{1/2} \quad \text{Equation 2.5S.4-39A}$$

where,

p_{hydro} = hydrodynamic pressure

K_h = peak horizontal ground surface acceleration

γ_w = unit weight of water = 62.4 pcf

H_w = depth from groundwater level to base of wall

z_w = depth below the groundwater level

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:

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

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.4-76 and 2.5S.4-77 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; 36 degrees was assumed for structural fill in the bearing capacity calculations; both values are 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, and liquefaction analyses whereas a groundwater level at ground surface (rough grade at El. 34 feet) was conservatively adopted for developing sample earth pressure diagrams. For bearing capacity analyses, the groundwater table was considered below the underside of concrete fill for short term and intermediate conditions, and a long term case with groundwater at El. 17.0 feet was evaluated and resulted in higher FOS than the short and intermediate cases. Groundwater at the surface (rough grade at El. 34 feet) would result in even higher FOS for the bearing capacity due to buoyancy on the structures. 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. The settlement analyses considered the groundwater table was below the underside of concrete fill, and thus no settlement reduction by buoyancy on the structure was considered. 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 Seismic Category I structures. Mat foundations will be used for all Seismic Category I structures. Table 2.5S.4-42B provides ultimate bearing capacity for Seismic Category I structures. Generally, a minimum FOS=3.0 was used when applying ultimate bearing capacity equations when static loading conditions apply. This FOS can also be applied against breakout failure due to uplift forces on buried piping. This FOS can be reduced to 1.5 when dynamic or transient loading conditions apply (Reference 2.5S.4-69). Table 2.5S.4-47 shows estimated structure total settlements under the stated 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 1/500, or 1/750 for foundations supporting sensitive machinery), then settlements do not impact foundation performance. Higher total settlements such as calculated for the STP 3 & 4 Reactor Buildings can be accommodated when critical connections to adjacent structures, utilities, and paving are delayed.

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.4-77, 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. The lateral earth pressure diagrams are shown on Figures 2.5S.4-76 and 2.5S.4-77, 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 and hard clay subgrade soils; concrete fill; and/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), is proposed to replace these soils. Partial removal of Stratum F at the STP 3 & 4 Reactor Buildings allows concrete fill to be placed, reducing the intensity of the pressure applied to the remaining soils and reducing settlement. A general over-excavation of 2 feet, and backfilling with concrete fill, at the STP 3 & 4 Control Buildings, the UHS Basins, the RSW Tunnels, RSW Pump Houses, and Diesel Generator Fuel Oil Storage Vaults, is proposed to ensure a firm subgrade for construction activities. While the foundations for these latter structures often 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.

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 hard 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.

2.5S.4.13 References

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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	132
SPT Hammer Energy Measurements	References 2.5S4-6 and 2.5S.4-25	52
Cone Penetration Tests (C)	Reference 2.5S.4-26	44
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

Stratum	Range	STP 3		STP 4		Inside Power Block		Outside Power Block		Site-Wide	
		Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)
A (Fill)	Minimum	23.7	1.0	23.3	0.5	23.3	0.5	23.1	1.0	23.1	0.5
	Maximum	30.2	4.5	30.5	4.5	30.5	4.5	29.8	13.5	30.5	13.5
	Average	27.8	1.8	29.2	1.6	28.6	1.7	28.0	3.1	28.5	1.9
A	Minimum	-0.3	7.5	2.6	8.5	-0.3	7.5	-1.9	6.0	-1.9	6.0
	Maximum	23.1	28.5	21.6	28.5	23.1	28.5	20.8	28.5	23.1	28.5
	Average	13.5	15.8	10.5	19.7	12.0	17.8	10.1	19.2	11.5	18.2
B	Minimum	-2.4	0.5	-7.9	1.0	-7.9	0.5	-9.0	2.0	-9.0	0.5
	Maximum	14.2	16.0	13.6	16.0	14.2	16.0	10.9	27.5	14.2	27.5
	Average	6.2	7.8	4.6	6.1	5.4	7.1	2.8	7.7	4.8	7.2
C	Minimum	-23.7	14.5	-21.9	5.0	-23.7	5.0	-20.4	5.0	-23.7	5.0
	Maximum	-9.0	30.0	-7.3	28.0	-7.3	30.0	-7.5	29.5	-7.3	30.0
	Average	-16.1	22.1	-13.8	18.6	-14.9	20.2	-13.9	16.9	-14.7	19.4
D	Minimum	-45.4	9.0	-45.3	15.0	-45.4	9.0	-41.5	15.0	-45.4	9.0
	Maximum	-25.9	31.3	-28.6	34.0	-25.9	34.0	-16.9	30.5	-16.9	34.0
	Average	-36.8	20.7	-36.6	22.5	-36.7	21.0	-36.6	23.1	-36.7	21.4
E	Minimum	-71.1	9.4	-66.7	5.0	-71.1	5.0	-70.9	5.0	-71.1	5.0
	Maximum	-48.0	35.8	-43.0	30.0	-43.0	35.8	-21.9	31.0	-21.9	35.8
	Average	-59.7	23.4	-50.6	13.8	-54.6	18.1	-50.8	19.1	-54.4	18.1

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

Stratum	Range	STP 3		STP 4		Inside Power Block		Outside Power Block		Site-Wide	
		Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)
F	Minimum	-80.8	2.4	-78.7	4.0	-80.8	2.4	-93.1	27.1	-93.1	2.4
	Maximum	-53.0	25.0	-47.9	30.0	-47.9	30.0	-46.9	55.0	-46.9	55.0
	Average	-70.2	11.4	-66.6	17.0	-68.0	14.9	-68.4	39.0	-68.0	16.1
H	Minimum	-93.5	1.9	-90.3	1.7	-93.5	1.7	-93.9	5.0	-93.9	1.7
	Maximum	-80.3	34.5	-64.6	35.5	-64.6	35.5	-73.8	45.0	-64.6	45.0
	Average	-88.7	18.3	-85.4	14.9	-87.1	16.6	-86.7	23.2	-87.0	17.2
J Clay 1	Minimum	-131.9	10.0	-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	28.7	-116.5	28.6	-119.2	28.6	-	-	-119.2	28.6
J Interbed 1	Minimum	-110.7	5.5	-107.9	3.5	-110.7	3.5	-	-	-110.7	3.5
	Maximum	-98.0	10.0	-93.9	9.5	-93.9	10.0	-	-	-93.9	10.0
	Average	-107.8	9.3	-103.7	6.9	-106.5	8.6	-	-	-106.5	8.6
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	-126.7	11.7	-130.9	13.4	-	-	-130.9	13.4
J Interbed 2	Minimum	-161.9	9.5	-168.6	8.0	-168.6	8.0	-	-	-168.6	8.0
	Maximum	-140.3	20.5	-127.6	30.3	-127.6	30.3	-	-	-127.6	30.3
	Average	-155.0	17.9	-151.1	13.1	-152.1	14.9	-	-	-152.1	14.9
J Clay 2	Minimum	-183.2	34.5	-185.0	48.1	-185.0	34.5	-	-	-185.0	34.5
	Maximum	-183.2	34.5	-185.0	48.1	-183.2	48.1	-	-	-183.2	48.1
	Average	-183.2	34.5	-185.0	48.1	-184.1	41.3	-	-	-184.1	41.3

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

Stratum	Range	STP 3		STP 4		Inside Power Block		Outside Power Block		Site-Wide	
		Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (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	-	-	-362.2	16.0
	Maximum	-362.2	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)

Stratum	Range	STP 3		STP 4		Inside Power Block		Outside Power Block		Site-Wide	
		Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)
N Sand 2	Minimum	-370.2	39.0	-358.9	26.0	-370.2	26	-	-	-370.2	26
	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	-392.4	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	16.0	-	-	-435.2	16.0
	Maximum	-435.2	16.0	-	-	-435.2	16.0	-	-	-435.2	16.0
	Average	-435.2	16.0	-	-	-435.2	16.0	-	-	-435.2	16.0
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	35.0	-	-	-508.9	35.0
	Maximum	-	-	-508.9	35.0	-508.9	35.0	-	-	-508.9	35.0
	Average	-	-	-508.9	35.0	-508.9	35.0	-	-	-508.9	35.0

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

Stratum	Range	STP 3		STP 4		Inside Power Block		Outside Power Block		Site-Wide	
		Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)	Base El. (feet)	Thickness (feet)
N Clay 6	Minimum	-	-	-	-	-	-	-	-	-	-
	Maximum	-	-	-	-	-	-	-	-	-	-
	Average	-	-	-	-	-	-	-	-	-	-

Maximum, minimum, and average thickness calculations did not include the last layers which were terminated at the bottoms of the borings, and averaging only included layers encountered in the borings.

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

Stratum	Range	STP 3 Power Block	STP 4 Power Block	Inside Power Block	Outside Power Block	Site Wide
A (Fill)	No. of Tests	17	17	34	15	49
	Minimum	4.0	2.0	2.0	3.0	2.0
	Maximum	12.0	22.0	22.0	14.0	22.0
	Average	8.3	8.5	8.4	8.1	8.3
A	No. of Tests	449	524	973	262	1235
	Minimum	0.0	3.0	0.0	3.0	0.0
	Maximum	27.0	42.0	42.0	41.0	42.0
	Average	8.5	11.1	9.9	10.7	10.1
B	No. of Tests	100	67	167	47	214
	Minimum	2.0	2.0	2.0	3.0	2.0
	Maximum	23.0	40.0	40.0	17.0	40.0
	Average	7.5	11.5	9.1	8.9	9.0
C	No. of Tests	232	186	418	75	493
	Minimum	0.0	3.0	0.0	4.0	0.0
	Maximum	109.0	122.0	122.0	82.0	122.0
	Average	27.3	23.1	25.5	24.1	25.3
D	No. of Tests	206	213	419	101	520
	Minimum	7.0	3.0	3.0	6.0	3.0
	Maximum	34.0	54.0	54.0	34.0	54.0
	Average	15.6	14.6	15.1	15.4	15.2
E	No. of Tests	237	132	369	23	392
	Minimum	7.0	11.0	7.0	15.0	7.0
	Maximum	88.0	96.0	96.0	67.0	96.0
	Average	34.1	41.4	36.7	33.1	36.5
F	No. of Tests	50	130	180	135	315
	Minimum	11.0	11.0	11.0	9.0	9.0
	Maximum	102.0	63.0	102.0	56.0	102.0
	Average	22.5	22.4	22.4	20.9	21.7

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

Stratum	Range	STP 3 Power Block	STP 4 Power Block	Inside Power Block	Outside Power Block	Site Wide
H	No. of Tests	58	57	115	20	135
	Minimum	15.0	18.0	15.0	10.0	10.0
	Maximum	100.0	150.0	150.0	74.0	150.0
	Average	42.0	47.9	44.9	34.9	43.5
J Clay	No. of Tests	115	114	229	10	239
	Minimum	12.0	13.0	12.0	17.0	12.0
	Maximum	120.0	138.0	138.0	58.0	138.0
	Average	32.1	32.4	32.2	30.6	32.2
J Sand	No. of Tests	40	33	73	0	73
	Minimum	32.0	20.0	20.0		20.0
	Maximum	120.0	125.0	125.0		125.0
	Average	69.7	55.2	63.1		63.1
K Clay	No. of Tests	1	1	2	0	2
	Minimum	15.0	15.0	15.0		15.0
	Maximum	15.0	15.0	15.0		15.0
	Average	15.0	15.0	15.0		15.0
K Sand/Silt	No. of Tests	1	1	2	0	2
	Minimum	120.0	40.0	40.0		40.0
	Maximum	120.0	40.0	120.0		120.0
	Average	120.0	40.0	80.0		80.0
L	No. of Tests	1	1	2	0	2
	Minimum	24.0	21.0	21.0		21.0
	Maximum	24.0	21.0	24.0		24.0
	Average	24.0	21.0	22.5		22.5
M	No. of Tests	0	0	0	0	0
	Minimum					
	Maximum					
	Average					

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

Stratum	Range	STP 3 Power Block	STP 4 Power Block	Inside Power Block	Outside Power Block	Site Wide
N Clay	No. of Tests	13	12	25	0	25
	Minimum	21.0	2.0	2.0		2.0
	Maximum	47.0	46.0	47.0		47.0
	Average	31.8	34.2	33.0		33.0
N Sand	No. of Tests	3	4	7	0	7
	Minimum	20.0	49.0	20.0		20.0
	Maximum	200.0	200.0	200.0		200.0
	Average	83.0	109.8	98.3		98.3

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

Dates Applicable [3]	Drilling Rig	Number Of Measurements	ETR Range (%)	ETR Average [1] (%)	Hammer Energy Correction (ETR%/60%)
October 2006 - January 2007	Best Failing 1500 Truck Rig	4	70-75	73	1.22
October 2006 - January 2007	Environmental Exploration CME 750 ATV	5	79-84	82	1.37
October 2006 - January 2007	Gregg Fraste Track Rig	3	79-80	80	1.33
October 2006 - January 2007	Gregg CME 55 Truck Rig	3	86-88	87	1.45
October 2006 - January 2007	Jedi CME 75 Truck Rig	5	71-77	75	1.25
July 2007 - August 2007	Jedi CME 75 Truck Rig	3	75-79	78	1.30
October 2006 - 12/7/2006	Lewis Environmental Mobile B57 (pre-12/08/2006) [2]	5	90-107	99	1.65
12/8/2006 - July 2007	Lewis Environmental Mobile B57 (post-12/08/2006)	5	83-89	87	1.45
July 2007 - August 2007	Lewis Environmental Mobile B57	3	83-86	84	1.40
12/16/2006 to January 2007	Lewis Environmental Mobile B61 (post-12/16/2006) [2]	3	94-98	96	1.60
October 2006 - January 2007	MACTEC D50 ATV Rig	4	69-74	72	1.20
October 2006 - January 2007	MACTEC CME 45 Trailer Rig	5	74-84	83	1.38
October 2006 - January 2007	Miller CME 750 ATV	4	83-86	85	1.42

[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.

[3] Dates Applicable is the range of dates corresponding to energy measurements for the appropriate drill rig. Miller CME 750 ATV and Jedi Drilling CME 75 Truck, Lewis Environmental Mobile B61 and Mobile 57 rigs were used on site more than once.

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

Stratum	Range	STP 3 Power Block	STP 4 Power Block	Inside Power Block	Outside Power Block	Site-Wide
B	No. of Tests	75	59	134	41	175
	Minimum	3.3	3.4	3.3	4.7	3.3
	Maximum	31.2	75.1	75.1	30.2	75.1
	Average	12.5	19.1	15.5	15.1	15.4
C	No. of Tests	229	184	413	74	487
	Minimum	0.0	4.4	0.0	10.2	0.0
	Maximum	160.6	200.5	200.5	123.1	200.5
	Average	42.6	35.0	39.2	36.8	38.9
E	No. of Tests	235	131	366	23	389
	Minimum	7.3	11.5	7.3	12.6	7.3
	Maximum	101.8	89.0	101.8	68.8	101.8
	Average	32.4	40.1	35.1	30.7	34.9
H	No. of Tests	57	57	114	20	134
	Minimum	12.2	13.9	12.2	7.9	7.9
	Maximum	78.2	101.9	101.9	63.9	101.9
	Average	33.4	37.7	35.5	28.3	34.5
J Sand	No. of Tests	40	30	70	0	70
	Minimum	20.5	12.8	12.8	N/A	12.8
	Maximum	76.8	87.0	87.0	N/A	87.0
	Average	44.5	36.4	41.0	N/A	41.0
K Sand/Silt	No. of Tests	1	1	2	0	2
	Minimum	81.6	27.2	27.2	N/A	27.2
	Maximum	81.6	27.2	81.6	N/A	81.6
	Average	81.6	27.2	54.4	N/A	54.4
M	No. of Tests	0	0	0	0	0
	Minimum	N/A	N/A	N/A	N/A	N/A
	Maximum	N/A	N/A	N/A	N/A	N/A
	Average	N/A	N/A	N/A	N/A	N/A

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

Stratum	Range	STP 3 Power Block	STP 4 Power Block	Inside Power Block	Outside Power Block	Site-Wide
N Sand	No. of Tests	3	4	7	0	7
	Minimum	13.6	33.3	13.6	N/A	13.6
	Maximum	136.0	136.0	136.0	N/A	136.0
	Average	56.4	74.6	66.8	N/A	66.8

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

Stratum	Average [2] Uncorrected N-Value	Average [2] Corrected N_{60}-Value	Average [2] Corrected $(N_1)_{60}$-Value	Selected [3] Corrected N_{60}-Value	Selected [3] Corrected $(N_1)_{60}$-Value
A	10	13	N/A	11	N/A
B	9	14	15	11	12
C	25	41	39	38	35
D	15	25	N/A	23	N/A
E	37	60	35	53	31
F	22	36	N/A	34	N/A
H	44	70	35	58	28
J Clay	32	51	N/A	48	N/A
J Sand	63	101	41	94	38
K Clay	15	26	N/A	26	N/A
K Sand/Silt	80	136	54	68	27
L	23	38	N/A	36	N/A
M	Not Tested	-	Not Tested	100	40
N Clay	33	56	N/A	54	N/A
N Sand	98	167	67	141	56

[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

Table 2.5S.4-7 Laboratory Testing Summary

Laboratory Test	Industry Standard	Number Of Tests
Moisture content	Reference 2.5S.4-32	534
Atterberg Limits	Reference 2.5S.4-33	286
Grain Size Analysis	References 2.5S.4-34 and 2.5S.4-35	257
Specific Gravity	Reference 2.5S.4-36	107
Unit Weight	Included with Related ASTM Standards	141
Unconsolidated Undrained (UU) Triaxial Strength	Reference 2.5S.4-37	76
Unconfined Compressive (UNC) Strength	Reference 2.5S.4-38	25
Consolidated Undrained (CIU-bar) Triaxial Strength	Reference 2.5S.4-39	17
Direct Shear (DS) Strength	Reference 2.5S.4-40	10
Consolidation	Reference 2.5S.4-41	37
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	67
Chloride Content	Reference 2.5S.4-45	47
Sulfate Content	Reference 2.5S.4-45	47
Resonant Column Torsional Shear (RCTS)	Reference 2.5S.4-46	16

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

Description of Value	USCS Group	Natural Moisture Content (%)	Total Unit Weight (pcf)	Specific Gravity	Initial Void Ratio	Liquid Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Fines Content (%)	pH	Chloride Content (mg/kg)	Sulfide Content (mg/kg)
Stratum A													
Minimum	CL, CH	15.7	117.9	2.65	0.467	30.0	11.0	0.0	0.2	89.6	7.7	26.1	6.1
Maximum		29.6	133.0	2.77	0.748	80.0	58.0	0.0	10.4	99.8	9.2	1230.0	622.0
Average		24.1	123.5	2.71	0.667	56.3	36.6	0.0	3.9	96.1	8.4	263.0	121.9
# of Tests		81	14	9	13	44	44	11	11	11	30	20	20
Stratum B													
Minimum	CL, ML, SM, SC	17.6	116.8	2.69	0.600	26.0	8.0	0.0	5.7	36.1	8.5	6.5	9.3
Maximum		28.4	127.7	2.71	0.806	46.0	26.0	4.0	63.9	94.3	8.7	124.0	13.5
Average		24.3	121.4	2.70	0.717	33.0	14.4	0.6	32.1	67.3	8.6	73.5	11.7
# of Tests		36	5	2	5	5	5	19	19	19	3	3	3
Stratum C													
Minimum	SP-SM, ML, SM	17.1	119.6	2.65	0.653	NV	NP	0.0	4.1	5.3	8.1	36.1	7.2
Maximum		27.0	124.2	2.73	0.715	NV	NP	5.9	94.7	95.9	9.1	108.0	35.5
Average		23.3	122.0	2.68	0.695	NV	NP	0.3	76.3	23.4	8.7	77.7	14.4
# of Tests		45	4	4	4	2	2	39	39	39	14	10	10

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

Description of Value	USCS Group	Natural Moisture Content (%)	Total Unit Weight (pcf)	Specific Gravity	Initial Void Ratio	Liquid Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Fines Content (%)	pH	Chloride Content (mg/kg)	Sulfide Content (mg/kg)
Stratum D													
Minimum	CH, CL,	16.3	110.8	2.65	0.523	20.0	2.0	0.0	0.0	18.3	8.5	33.4	6.7
Maximum	CL-ML,	53.4	129.6	2.77	1.030	84.0	59.0	2.2	81.7	100.0	9.1	66.9	143.0
Average	ML	25.8	122.6	2.72	0.746	57.2	36.6	0.2	21.0	78.9	8.7	48.5	40.0
# of Tests		90	26	14	26	53	53	26	26	26	8	5	5
Stratum E													
Minimum	SM, ML,	14.9	111.4	2.62	0.576	NV	NP	0.0	3.8	3.0	8.4	27.0	11.6
Maximum	SP, SC,	25.8	132.6	2.78	0.770	NV	NP	1.5	97.0	96.2	9.3	46.6	31.8
Average	SP-SM	20.8	122.6	2.68	0.678	NV	NP	0.1	80.1	19.8	8.8	37.1	23.3
# of Tests		48	9	8	8	6	6	43	43	43	6	4	4
Stratum F													
Minimum	CH, CL,	17.9	119.5	2.65	0.542	27.0	6.0	0.0	0.6	55.8	8.3	20.6	14.5
Maximum	ML, CL-	33.2	131.0	2.78	0.786	74.0	53.0	0.0	44.2	99.4	8.9	40.0	47.8
Average	ML	24.2	125.0	2.73	0.684	57.0	37.0	0.0	6.2	93.8	8.6	30.8	31.6
# of Tests		66	18	15	17	47	47	14	14	14	5	5	5

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

Description of Value	USCS Group	Natural Moisture Content (%)	Total Unit Weight (pcf)	Specific Gravity	Initial Void Ratio	Liquid Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Fines Content (%)	pH	Chloride Content (mg/kg)	Sulfide Content (mg/kg)
Stratum H													
Minimum		12.4	120.6	2.66	0.404	NV	NP	0.0	5.2	6.0	8.8	N/A	N/A
Maximum	SP-SM,	24.4	134.9	2.66	0.697	NV	NP	8.6	94.0	94.8	8.8		
Average	SM	19.1	124.9	2.66	0.551	NV	NP	1.1	80.5	18.5	8.8		
# of Tests		16	4	1	2	1	1	14	14	14	1	0	0
Stratum J (CLAY 1)													
Minimum		13.7	103.7	2.65	0.480	26.0	9.0	0.0	0.6	54.6	N/A	N/A	N/A
Maximum	CH, CL	34.0	133.7	2.80	0.991	80.0	58.0	0.0	45.4	99.4			
Average		21.9	125.0	2.71	0.654	52.5	33.7	0.0	12.1	87.9			
# of Tests		50	28	17	27	39	39	23	23	23	0	0	0
SUB-STRATUM J (SAND/SILT Interbed 1) <associated with J (CLAY 1)>													
Minimum		16.1	N/A	N/A	N/A	N/A	N/A	0.0	27.7	51.3	N/A	N/A	N/A
Maximum	CL, ML	21.0						0.0	48.7	72.3			
Average		18.6						0.0	38.2	61.8			
# of Tests		2	0	0	0	0.0	0.0	2	2	2	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 (%)	Total Unit Weight (pcf)	Specific Gravity	Initial Void Ratio	Liquid Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Fines Content (%)	pH	Chloride Content (mg/kg)	Sulfide Content (mg/kg)
SUB-STRATUM J (SAND 1)													
Minimum		18.7	121.6	2.63	0.645	NV	NP	0.0	22.7	14.8	N/A	N/A	N/A
Maximum	ML, SM	24.6	124.4	2.72	0.692	NV	NP	1.1	85.2	77.3			
Average		21.8	123.0	2.67	0.669	NV	NP	0.2	63.6	36.2			
# of Tests		9	2	3	2	4	4	9	9	9	0	0	0
STRATUM J (CLAY 2)													
Minimum		16.4	118.9	2.64	0.501	29.0	12.0	0.0	0.2	61.4	N/A	N/A	N/A
Maximum	CH, CL	38.0	129.2	2.75	0.793	85.0	62.0	0.9	34.0	99.8			
Average		24.1	124.4	2.71	0.664	55.3	35.7	0.1	7.1	92.2			
# of Tests		40	19	13	18	31	31	16	16	16	0	0	0
SUB-STRATUM J (SAND/SILT Interbed 2) <associated with J (CLAY 2)>													
Minimum		18.5	124.4	2.65	0.642	24.0	3.0	0.0	3.3	9.8	N/A	N/A	N/A
Maximum	SM, ML, SP-SM	32.0	128.0	2.67	0.749	24.0	3.0	0.0	90.2	96.7			
Average		24.4	126.5	2.66	0.696	24.0	3.0	0.0	34.9	66.8			
# of Tests		6	3	3	2	5	5	6	6	6	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 (%)	Total Unit Weight (pcf)	Specific Gravity	Initial Void Ratio	Liquid Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Fines Content (%)	pH	Chloride Content (mg/kg)	Sulfide Content (mg/kg)
COMBINED SUB-STRATA J (CLAY 1), J (CLAY 2)													
Minimum	CL, CH, ML	13.7	103.7	2.64	0.480	26.0	9.0	0.0	0.2	54.6	N/A	N/A	N/A
Maximum		38.0	133.7	2.80	0.991	85.0	62.0	0.9	45.4	99.8			
Average		22.9	124.8	2.71	0.658	53.8	34.6	0.0	10.1	89.7			
# of Tests		90	47	30	45	70	70	39	39	39	0	0	0
COMBINED SUB-STRATA J (SAND 1), J (SAND 2), J (SAND/SILT interbeds)													
Minimum	SM, ML, SP-SM, CL	16.1	121.6	2.63	0.642	24.0	3.0	0.0	3.3	9.8	N/A	N/A	N/A
Maximum		32.0	128.0	2.72	0.749	24.0	3.0	1.1	90.2	96.7			
Average		22.4	125.1	2.67	0.682	24.0	3.0	0.1	50.5	50.0			
# of Tests		17	5	6	4	9	9	17	17	17	0	0	0
Stratum K (CLAY)													
Minimum	CH, CL	16.8	114.9	2.71	0.499	33.0	18.0	0.0	1.0	74.8	N/A	N/A	N/A
Maximum		34.5	131.5	2.76	0.627	73.0	51.0	0.0	25.2	99.0			
Average		23.2	124.3	2.73	0.563	50.3	33.3	0.0	13.1	86.9			
# of Tests		4	3	3	2	3	3	2	2	2	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 (%)	Total Unit Weight (pcf)	Specific Gravity	Initial Void Ratio	Liquid Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Fines Content (%)	pH	Chloride Content (mg/kg)	Sulfide Content (mg/kg)
Stratum K (SAND/SILT)													
Minimum	SM, ML	20.1	126.8	2.67	0.596	NV	NP	0.0	34.6	27.0	N/A	N/A	N/A
Maximum		21.5	126.8	2.67	0.596	NV	NP	1.6	73.0	63.8			
Average		20.8	126.8	2.67	0.596	NV	NP	0.8	53.8	45.4			
# 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	72.0	51.0	N/A	N/A	N/A	N/A	N/A	N/A
Maximum		29.6				74.0	52.0						
Average		28.5				73.0	51.5						
# of Tests		2	0	0	0.0	2	2	0	0	0	0	0	0
Stratum M													
Minimum	SM	19.2	116.0	2.65	N/A	NV	NP	0.0	45.0	55.0	N/A	N/A	N/A
Maximum		19.2	116.0	2.65		NV	NP	0.0	45.0	55.0			
Average		19.2	116.0	2.65		NV	NP	0.0	45.0	55.0			
# of Tests		1	1	1	0.0	1	1	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 (%)	Total Unit Weight (pcf)	Specific Gravity	Initial Void Ratio	Liquid Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Fines Content (%)	pH	Chloride Content (mg/kg)	Sulfide Content (mg/kg)
Stratum N (CLAY 1)													
Minimum	CH, SC	19.7	112.9	2.67	0.835	50.0	25.0	0.0	2.0	21.7	N/A	N/A	N/A
Maximum		37.7	120.3	2.75	1.074	90.0	63.0	0.0	78.3	98.0			
Average		29.4	117.6	2.71	0.954	71.4	48.0	0.0	23.7	76.4			
# of Tests		5	3	4	2	5	5	4	4	4	0	0	0
Stratum N (SAND 1)													
Minimum	SM	16.7	130.2	2.65	0.536	NV	NP	0.0	50.1	4.7	N/A	N/A	N/A
Maximum		20.9	130.2	2.65	0.536	NV	NP	0.7	95.3	49.2			
Average		18.8	130.2	2.65	0.536	NV	NP	0.4	72.7	27.0			
# of Tests		2	1	1	1	1	1	2	2	2	0	0	0
Stratum N (CLAY 2)													
Minimum	CH	29.5	116.3	2.74	N/A	92.0	65.0	2.0	12.0	86.0	N/A	N/A	N/A
Maximum		29.5	116.3	2.74		92.0	65.0	2.0	12.0	86.0			
Average		29.5	116.3	2.74		92.0	65.0	2.0	12.0	86.0			
# of Tests		1	1	1	0	1	1	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 (%)	Total Unit Weight (pcf)	Specific Gravity	Initial Void Ratio	Liquid Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Fines Content (%)	pH	Chloride Content (mg/kg)	Sulfide Content (mg/kg)
Stratum N (SAND 2)													
Minimum	SP, SM,	21.2	128.8	2.67	N/A	NV	NP	0.0	72.3	5.4	N/A	N/A	N/A
Maximum	SP-SM,	28.0	128.8	2.67		NV	NP	6.1	89.8	25.9			
Average	SC	24.6	128.8	2.67		NV	NP	2.0	83.9	14.1			
# of Tests		4	1	1	0	1	1	4	4	4	0	0	0
Stratum N (CLAY 3)													
Minimum		17.1	N/A	N/A	N/A	46.0	31.0	N/A	N/A	N/A	N/A	N/A	N/A
Maximum	CL	17.1				46.0	31.0						
Average		17.1				46.0	31.0						
# of Tests		1	0	0	0	1	1	0	0	0	0	0	0
Stratum N (SAND 3)													
Minimum		N/A	N/A	2.69	N/A	N/A	N/A	1.1	82.6	16.3	N/A	N/A	N/A
Maximum	SM			2.69				1.1	82.6	16.3			
Average				2.69				1.1	82.6	16.3			
# of Tests		0	0	1	0	0	0	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 (%)	Total Unit Weight (pcf)	Specific Gravity	Initial Void Ratio	Liquid Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Fines Content (%)	pH	Chloride Content (mg/kg)	Sulfide Content (mg/kg)
Stratum N (CLAY 4)													
Minimum		17.4	131.7	2.66	N/A	33.0	22.0	1.0	49.0	50.0	N/A	N/A	N/A
Maximum	CH, CL	29.7	131.7	2.66		86.0	59.0	1.0	49.0	50.0			
Average		24.1	131.7	2.66		65.3	45.0	1.0	49.0	50.0			
# of Tests		3	1	1	0	3	3	1	1	1	0	0	0
Stratum N (SAND 4)													
Minimum		18.8	125.8	2.67	0.616	NV	NP	0.0	71.4	11.8	N/A	N/A	N/A
Maximum	SP-SM,	23.3	129.2	2.67	0.616	NV	NP	0.0	88.2	28.6			
Average	SM	21.4	127.5	2.67	0.616	NV	NP	0.0	79.8	20.2			
# of Tests		3	2	1	1	1	1	2	2	2	0	0	0
Stratum N (CLAY 5)													
Minimum		21.8	123.7	N/A	0.729	59.0	40.0	N/A	N/A	N/A	N/A	N/A	N/A
Maximum	CH	24.3	123.7		0.729	81.0	58.0						
Average		23.3	123.7		0.729	70.0	49.0						
# of Tests		3.0	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 (%)	Total Unit Weight (pcf)	Specific Gravity	Initial Void Ratio	Liquid Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Fines Content (%)	pH	Chloride Content (mg/kg)	Sulfide Content (mg/kg)
Stratum N (SAND 5)													
Minimum	SM	20.8	N/A	2.68	N/A	NV	NP	0.0	62.4	20.7	N/A	N/A	N/A
Maximum		25.4		2.68		NV	NP	2.0	79.3	37.2			
Average		22.6		2.68		NV	NP	0.8	70.8	28.4			
# of Tests		3	0	1	0	1	1	3	3	3	0	0	0
Stratum N (CLAY 6)													
Minimum	CH, CL	18.4	122.1	2.69	0.567	45.0	29.0	0.0	2.0	79.0	N/A	N/A	N/A
Maximum		27.2	128.8	2.70	0.567	84.0	62.0	2.0	19.0	98.0			
Average		21.5	126.1	2.70	0.567	60.8	42.5	0.7	14.1	86.6			
# of Tests		4	3	2	1	4	4	4	4	4	0	0	0

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

From Correlations with SPT N_{60}-value Data			
Stratum	Selected N_{60}-Value (blows/foot)	Calculated S_u (ksf)	
A	11	1.4	
D	23	2.9	
F	34	4.3	
J Clay	48	6.0	
K Clay	26	3.3	
L	36	4.5	
N Clay	54	6.8	
From Laboratory UU and UNC Tests (Average Excludes Tests with low S_u/σ_v')			
Stratum	Minimum s_u (ksf)	Maximum s_u (ksf)	Average s_u (ksf)
A	0.5	2.3	1.4
D	0.3	2.5	2.2
F	0.7	4.3	2.9
J Clay	0.1	6.6	4.3
K Clay	2.8	4.0	4.0
L	Not Tested	Not Tested	Not Tested
N Clay	0.2	4.5	4.5
From Correlations with CPT Data			
Stratum	Minimum s_u (ksf)	Maximum s_u (ksf)	Average s_u (ksf)
A	0.2	>10	1.7
D	0.8	>10	3.3
F	1.9	6.3	3.5
J Clay	2.3	4.0	3.1
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		1.5	
D		3.0	
F		3.4	
J Clay		3.48	
K Clay		3.9	
L		3.9	
N Clay		4.5	

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

Strata	Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Average Total Unit Weight (pcf)	Average Natural Moisture Content (percent) at e_o	Liquid Limit, LL (percent)	Plasticity Index, PI (percent)	USCS Symbol	Effective Vertical Overburden Stress (ksf), σ'_v	UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
											Test Type	Undrained Shear Strength, Su (ksf)	Ratio, Su/ σ'_v	Undrained Cohesion (ksf)	Undrained Friction Angle (degrees)	Drained Cohesion (ksf)	Drained Friction Angle (degrees)	Drained Cohesion (ksf)	Drained Friction Angle (degrees)
A	B-305-DH	UD-1	3.0	26.8	122.0	23.8	62	43	CH	0.51	UNC	0.953	1.85	-	-	-	-	-	-
A	B-332	UD-1	3.0	27.3	122.6	23.1	70	45	CH	0.55	UNC	1.946	3.56	-	-	-	-	-	-
A	B-333	UD-1	8.0	22.5	123.8	24.3	43	27	CL	0.87	UNC	1.291	1.49	-	-	-	-	-	-
A	B-333	UD-2	18.0	12.5	119.1	23.4	30	11	CL	1.48	UU	0.480	0.32	-	-	-	-	-	-
A	B-432	UD-1	3.0	28.2	123.0	21.8	65	42	CH	0.60	UNC	2.293	3.81	-	-	-	-	-	-
A	B-432	UD-2	15.0	16.2	121.4	24.0	31	11	CL	1.34	UNC	0.916	0.68	-	-	-	-	-	-
A	B-902	UD-1	5.0	24.1	124.7	21.3	65	43	CH	0.59	UNC	1.277	2.15	-	-	-	-	-	-
A	B-902	UD-2	15.0	14.1	124.5	24.3	59	39	CH	1.21	UU	0.903	0.75	-	-	-	-	-	-
A	B-904	UD-1	5.0	24.8	122.0	22.3	68	46	CH	0.64	UNC	1.170	1.83	-	-	-	-	-	-
A	B-904	UD-2	18.0	11.8	122.7	21.1	63	41	CH	1.44	UNC	1.968	1.37	-	-	-	-	-	-
A	B-918	UD-1	3.0	27.9	133.0	15.7	-	-	CH	0.58	UNC	2.138	3.66	-	-	-	-	-	-
A	B-919	UD-1	8.0	23.9	124.7	23.5	68	47	CH	0.95	UU	1.195	1.25	-	-	-	-	-	-
			Stratum A	**Minimum**	119.1	15.7	30	11	-	-	-	0.480	0.32	-	-	-	-	-	-
				Maximum	133.0	24.3	70	47	-	-	-	2.293	3.81	-	-	-	-	-	-
				Average	123.6	22.4	57	36	-	-	-	1.377	1.89	-	-	-	-	-	-
B	B-904	UD-3	28.0	1.8	121.3	25.2	NV	NP	ML	2.04	-	-	-	5.05	23.0	2.00	28.0	-	-
B	B-904	UD-3	28.0	1.8	117.1	25.0	NV	NP	ML	2.04	UU	-	-	-	-	-	-	-	-
B	B-918	UD-2	18.0	12.9	124.0	23.8	46	26	CL	1.50	UU	1.852	1.23	-	-	-	-	-	-
B	B-919	UD-2	23.0	8.9	116.8	21.6	NV	NP	ML	1.88	-	-	-	0.40	47.0	0.00	31.0	-	-
B	B-927	UD-2	28.0	-1.2	127.7	21.3	-	-	CH	1.80	UNC	0.864	0.48	-	-	-	-	-	-
			Stratum B	**Minimum**	116.8	21.3	-	-	-	-	-	0.864	0.48	0.40	23.0	0.00	28.0	-	-
				Maximum	127.7	25.2	-	-	-	-	-	1.852	1.23	5.05	47.0	2.00	31.0	-	-
				Average	121.4	23.4	NV	NP	-	-	-	1.358	0.86	2.73	35.0	1.00	29.5	-	-
C	B-421	UD-1A	33.5	-3.2	119.6	23.0	-	-	SP-SM	2.40	-	-	-	-	-	-	-	0.0	33.0
C	B-902	UD-3	23.0	6.1	121.3	24.9	NV	NP	SM	2.84	-	-	-	-	-	-	-	0.0	32.0
C	B-909	UD-1	33.0	-3.3	123.0	23.2	-	-	SM	2.33	-	-	-	-	-	-	-	0.0	34.0
			Stratum C	**Minimum**	119.6	23.0	-	-	-	-	-	-	-	-	-	-	-	0.0	32.0
				Maximum	123.0	24.9	-	-	-	-	-	-	-	-	-	-	-	0.0	34.0
				Average	121.3	23.7	NV	NP	-	-	-	-	-	-	-	-	-	0.0	33.0

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

Strata	Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Average Total Unit Weight (pcf)	Average Natural Moisture Content (percent) at e_o	Liquid Limit, LL (percent)	Plasticity Index, PI (percent)	USCS Symbol	Effective Vertical Overburden Stress (ksf), σ'_v	UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
											Test Type	Undrained Shear Strength, Su (ksf)	Ratio, Su/ σ'_v	Undrained Cohesion (ksf)	Undrained Friction Angle (degrees)	Drained Cohesion (ksf)	Drained Friction Angle (degrees)	Drained Cohesion (ksf)	Drained Friction Angle (degrees)
D	B-305-DH	UD-4	53.0	-23.2	125.2	24.6	45	25	CL	3.53	UNC	2.474	0.70	-	-	-	-	-	-
D	B-330	UD-2	53.0	-23.5	128.8	16.3	-	-	CH	3.51	-	-	-	2.17	0.0	2.15	0.0	-	-
D	B-338	UD-2	48.0	-15.9	122.3	26.8	44	26	CL	3.35	UU	2.248	0.67	-	-	-	-	-	-
D	B-909	UD-2	43.0	-13.3	121.1	25.4	62	38	CH	2.92	UU	2.219	0.76	-	-	-	-	-	-
D	B-909	UD-3	48.0	-18.3	115.9	32.8	74	53	CH	3.23	UU	2.392	0.74	-	-	-	-	-	-
D	B-909	UD-4	53.0	-23.3	110.8	30.0	66	40	CH	3.53	UU	0.440	0.12	-	-	-	-	-	-
D	B-916	UD-3	48.0	-20.2	117.5	20.8	59	35	CH	3.10	-	-	-	1.41	8.7	0.31	31.7	-	-
D	B-918	UD-4	58.0	-27.1	126.8	17.7	22	6	CL-ML	3.90	UNC	0.263	0.07	-	-	-	-	-	-
D	B-919	UD-3	43.0	-11.1	119.5	26.5	66	42	CH	3.06	UU	1.728	0.56	-	-	-	-	-	-
D	B-927	UD-3	48.0	-21.2	121.9	28.2	54	33	CH	3.00	UNC	1.803	0.60	-	-	-	-	-	-
D	B-940	UD-3	41.0	-11.3	123.7	28.4	67	41	CH	2.78	UNC	2.512	0.90	-	-	-	-	-	-
D	B-940	UD-5	56.0	-26.3	124.1	27.0	50	32	CH	3.69	UU	2.024	0.55	-	-	-	-	-	-
D	B-949	UD-3	61.0	-32.3	129.6	23.3	54	31	CH	3.94	UU	2.530	0.64	-	-	-	-	-	-
			Stratum D	**Minimum**	110.8	16.3	22.0	6.0	-	-	-	0.263	0.07	1.41	0.0	0.31	0.0	-	-
				Maximum	129.6	32.8	74.0	53.0	-	-	-	2.530	0.90	2.17	8.7	2.15	31.7	-	-
				Average	122.1	25.2	55.3	33.5	-	-	-	1.876	0.57	1.79	4.4	1.23	15.9	-	-
										Alternate Minimum		1.728	0.55						
	Cells not included in Alternate Min, Max, and Average values calculated from this table.									**Alternate Maximum**		2.530	0.90						
										Alternate Average		2.214	0.68						
E	B-314	UD-1	83.0	-53.8	122.7	20.9	NV	NP	SP	5.84	-	-	-	-	-	-	-	0.0	33.0
E	B-409	UD-1	68.0	-36.8	111.4	14.9	NV	NP	SM	5.05	-	-	-	-	-	-	-	0.0	33.0
			Stratum E	**Minimum**	111.4	14.9	20	2	-	-	-	-	-	-	-	-	-	0.0	33.0
				Maximum	127.0	25.8	20	2	-	-	-	-	-	-	-	-	-	0.0	33.0
				Average	121.9	21.8	NV	NP	-	-	-	-	-	-	-	-	-	0.0	33.0
F	B-303	UD-2	88.0	-61.4	127.9	26.6	57	39	CH	6.02	UU	3.469	0.58	-	-	-	-	-	-
F	B-306	UD-4	88.0	-60.2	125.9	22.7	57	38	CH	6.05	-	-	-	3.02	3.1	2.95	6.5	-	-
F	B-401	UD-2	88.0	-57.2	125.7	23.4	57	36	CH	6.26	-	-	-	2.35	0.8	2.06	5.1	-	-
F	B-404	UD-1	88.0	-57.0	126.7	21.8	-	-	CH	6.27	UU	3.476	0.55	-	-	-	-	-	-
F	B-404	UD-2	98.0	-67.0	123.9	25.3	50	30	CH	6.90	UU	3.500	0.51	-	-	-	-	-	-
F	B-415	UD-1	88.0	-58.0	123.8	23.9	61	44	CH	6.20	UU	1.173	0.19	-	-	-	-	-	-

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

Strata	Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Average Total Unit Weight (pcf)	Average Natural Moisture Content (percent) at e_o	Liquid Limit, LL (percent)	Plasticity Index, PI (percent)	USCS Symbol	Effective Vertical Overburden Stress (ksf), σ'_v	UNC/ UU Tests [1] Test Type	UNC/ UU Tests [1] Undrained Shear Strength, Su (ksf)	UNC/ UU Tests [1] Ratio, Su/ σ'_v	CIU-Bar Tests [1] Undrained Cohesion (ksf)	CIU-Bar Tests [1] Undrained Friction Angle (degrees)	CIU-Bar Tests [1] Drained Cohesion (ksf)	CIU-Bar Tests [1] Drained Friction Angle (degrees)	DS Tests [1] Drained Cohesion (ksf)	DS Tests [1] Drained Friction Angle (degrees)
F	B-419DH	UD-1	78.0	-48.3	127.8	22.3	47	23	CL	5.58	UU	3.713	0.67	-	-	-	-	-	-
F	B-419DH	UD-2	98.0	-68.3	119.5	27.0	61	37	CH	6.83	UU	0.738	0.11	-	-	-	-	-	-
F	B-421	UD-3	83.0	-52.7	125.9	22.9	56	36	CH	5.93	UU	3.099	0.52	-	-	-	-	-	-
F	B-443	UD-1	86.0	-54.9	123.2	22.7	61	37	CH	6.16	UU	2.656	0.43	-	-	-	-	-	-
F	B-443	UD-3	96.0	-64.9	125.1	25.6	62	37	CH	6.78	UU	2.773	0.41	-	-	-	-	-	-
F	B-443	UD-4	101.0	-69.9	119.6	29.1	55	30	CH	7.10	UU	2.764	0.39	-	-	-	-	-	-
F	B-904	UD-5	83.0	-53.2	122.3	24.0	62	41	CH	5.91	UU	1.649	0.28	-	-	-	-	-	-
F	B-909	UD-5	85.0	-55.3	126.1	21.9	49	26	CL	6.03	-	-	-	0.93	6.0	0.95	12.0	-	-
F	B-909	UD-6	93.0	-63.3	129.3	17.9	55	34	CH	6.53	UU	3.567	0.55	-	-	-	-	-	-
F	B-909	UD-7	98.0	-68.3	121.1	21.5	-	-	CH	6.85	UU	2.506	0.37	-	-	-	-	-	-
F	B-940	UD-7	76.0	-46.3	128.1	22.1	55	34	CH	4.90	UU	4.344	0.89	-	-	-	-	-	-
F	B-940	UD-8	91.0	-61.3	127.2	24.4	61	39	CH	5.81	UNC	2.380	0.41	-	-	-	-	-	-
F	B-949	UD-4	71.0	-42.3	124.6	24.9	56	32	CH	4.55	UU	1.345	0.30	-	-	-	-	-	-
F	B-949	UD-7	91.0	-62.3	121.3	28.1	60	36	CH	5.76	UNC	1.990	0.35	-	-	-	-	-	-
			Stratum F	**Minimum**	119.5	17.9	47.0	23.0	-	-	-	0.738	0.11	0.93	0.8	0.95	5.1	-	-
				Maximum	129.3	29.1	62.0	44.0	-	-	-	4.344	0.89	3.02	6.0	2.95	12.0	-	-
				Average	124.6	23.7	56.4	34.9	-	-	-	2.655	0.44	2.10	3.3	1.99	7.9	-	-
											Alternate Minimum	1.345	0.28						
	Cells not included in Alternate Min, Max, and Average values calculated from this table.										**Alternate Maximum**	4.344	0.89						
											Alternate Average	2.882	0.48						
H	B-306	UD-5	98.0	-70.2	121.7	24.4	NV	NP	SP-SM	6.68	-	-	-	-	-	-	-	0.0	29.0
			Stratum H	**Minimum**	121.7	12.4	-	-	-	-	-	-	-	-	-	-	-	0.0	29.0
				Maximum	134.9	24.4	-	-	-	-	-	-	-	-	-	-	-	0.0	29.0
				Average	128.3	18.4	NV	NP	-	-	-	-	-	-	-	-	-	0.0	29.0
JC1	B-303	UD-4	133.0	-106.4	121.3	29.5	65	39	CH	8.90	UU	0.142	0.02	-	-	-	-	-	-
JC1	B-303	UD-4	133.0	-106.4	131.1	18.7	-	-	CH	8.90	-	-	-	3.05	3.2	2.35	11.0	-	-
JC1	B-305-DH	UD-7	123.0	-93.2	129.2	18.8	-	-	CH	8.44	UNC	1.178	0.14	-	-	-	-	-	-
JC1	B-305-DH	UD-7	123.0	-93.2	128.4	20.1	-	-	CH	8.44	UU	2.984	0.35	-	-	-	-	-	-
JC1	B-305-DH	UD-8	138.0	-108.2	129.5	18.6	32	18	CL	9.38	UU	4.543	0.48	-	-	-	-	-	-
JC1	B-314	UD-2	113.0	-83.8	127.9	20.0	38	25	CL	7.74	UU	4.831	0.62	-	-	-	-	-	-

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

Strata	Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Average Total Unit Weight (pcf)	Average Natural Moisture Content (percent) at e_o	Liquid Limit, LL (percent)	Plasticity Index, PI (percent)	USCS Symbol	Effective Vertical Overburden Stress (ksf), σ'_v	UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
											Test Type	Undrained Shear Strength, Su (ksf)	Ratio, Su/ σ'_v	Undrained Cohesion (ksf)	Undrained Friction Angle (degrees)	Drained Cohesion (ksf)	Drained Friction Angle (degrees)	Drained Cohesion (ksf)	Drained Friction Angle (degrees)
JC1	B-314	UD-3	121.0	-91.8	128.5	17.8	-	-	CH	8.24	UU	3.042	0.37	-	-	-	-	-	-
JC1	B-314	UD-4	141.0	-111.8	120.3	24.2	46	31	CL	9.49	UU	0.718	0.08	-	-	-	-	-	-
JC1	B-314	UD-4	141.0	-111.8	125.6	21.7	-	-	CL	9.49	-	-	-	2.28	9.0	1.20	20.0	-	-
JC1	B-319-DH	UD-1	128.0	-99.6	125.0	19.3	70	45	CH	8.60	UU	3.394	0.39	-	-	-	-	-	-
JC1	B-321	UD-3	138.0	-108.8	127.4	20.0	46	25	CL	9.31	UU	4.913	0.53	-	-	-	-	-	-
JC1	B-330	UD-4B	123.0	-93.5	128.7	19.5	-	-	CH	8.46	UU	1.084	0.13	-	-	-	-	-	-
JC1	B-401	UD-3	118.0	-87.2	127.3	19.8	47	25	CL	8.16	UU	2.003	0.25	-	-	-	-	-	-
JC1	B-404	UD-3	121.0	-90.0	124.2	23.6	62	39	CH	8.37	-	-	-	2.50	3.5	2.20	9.0	-	-
JC1	B-404	UD-3	121.0	-90.0	126.1	23.4	-	-	CH	8.37	UU	3.485	0.42	-	-	-	-	-	-
JC1	B-404	UD-4	131.0	-100.0	124.3	20.8	52	30	CH	8.99	UU	3.461	0.38	-	-	-	-	-	-
JC1	B-404	UD-5	141.0	-110.0	129.0	18.0	30	12	CL	9.62	UU	4.149	0.43	-	-	-	-	-	-
JC1	B-405DH	UD-5	113.0	-81.9	122.7	25.8	73	50	CH	7.84	UU	2.180	0.28	-	-	-	-	-	-
JC1	B-415	UD-3	124.0	-94.0	113.9	34.0	51	35	CH	8.51	UU	0.127	0.01	-	-	-	-	-	-
JC1	B-419DH	UD-3	118.0	-88.3	127.6	21.8	56	39	CH	8.14	UU	6.305	0.77	-	-	-	-	-	-
JC1	B-419DH	UD-4	138.0	-108.3	129.3	16.1	40	25	CL	9.39	UU	6.579	0.70	-	-	-	-	-	-
JC1	B-428-DH	UD-6	113.0	-82.1	122.6	27.3	62	41	CH	7.92	UNC	1.041	0.13	-	-	-	-	-	-
JC1	B-430	UD-3	133.0	-102.1	119.1	28.5	-	-	CH	9.13	-	-	-	0.50	0.0	0.50	0.0	-	-
JC1	B-443	UD-6	112.0	-80.9	123.9	26.7	63	36	CH	7.80	UNC	1.206	0.15	-	-	-	-	-	-
JC1	B-443	UD-7A	123.0	-91.9	128.2	23.4	48	29	CL	8.49	UU	3.721	0.44	-	-	-	-	-	-
JC2	B-307	UD-3	188.0	-159.8	125.3	21.9	49	30	CL	12.42	-	-	-	3.40	1.0	3.40	2.0	-	-
JC2	B-314	UD-5A	183.0	-153.8	122.5	20.3	72	48	CH	12.12	UU	5.265	0.43	-	-	-	-	-	-
JC2	B-314	UD-5A	183.0	-153.8	124.5	26.3	-	-	CH	12.12	-	-	-	0.86	11.2	1.86	7.4	-	-
JC2	B-314	UD-6	191.0	-161.8	118.9	26.5	64	40	CH	12.62	UU	1.933	0.15	-	-	-	-	-	-
JC2	B-319-DH	UD-5	188.0	-159.6	122.7	26.5	62	41	CH	12.35	UU	3.788	0.31	-	-	-	-	-	-
JC2	B-343	UD-7	173.0	-142.5	125.0	16.9	31	17	CL	11.60	UU	1.352	0.12	-	-	-	-	-	-
JC2	B-343	UD-7	173.0	-142.5	127.8	16.4	-	-	CL	11.60	-	-	-	1.50	25.0	0.21	28.8	-	-
JC2	B-343	UD-8	198.0	-167.5	122.6	22.1	-	-	CH	13.17	UU	1.105	0.08	-	-	-	-	-	-
JC2	B-401	UD-5A	184.0	-153.2	125.2	24.1	-	-	CH	12.29	UU	3.610	0.29	-	-	-	-	-	-
JC2	B-404	UD-6	161.0	-130.0	123.1	19.5	30	15	CL	10.87	-	-	-	7.69	2.6	6.92	6.0	-	-
JC2	B-409	UD-4A	160.0	-128.8	123.5	19.6	62	35	CH	10.82	UU	0.509	0.05	-	-	-	-	-	-

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

Strata	Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Average Total Unit Weight (pcf)	Average Natural Moisture Content (percent) at e_o	Liquid Limit, LL (percent)	Plasticity Index, PI (percent)	USCS Symbol	Effective Vertical Overburden Stress (ksf), σ'_v	UNC/ UU Tests [1]: Test Type	UNC/ UU Tests [1]: Undrained Shear Strength, Su (ksf)	UNC/ UU Tests [1]: Ratio, Su/ σ'_v	CIU-Bar Tests [1]: Undrained Cohesion (ksf)	CIU-Bar Tests [1]: Undrained Friction Angle (degrees)	CIU-Bar Tests [1]: Drained Cohesion (ksf)	CIU-Bar Tests [1]: Drained Friction Angle (degrees)	DS Tests [1]: Drained Cohesion (ksf)	DS Tests [1]: Drained Friction Angle (degrees)
JC2	B-409	UD-6	198.0	-166.8	121.5	25.3	-	-	CH	13.20	UU	5.629	0.43	-	-	-	-	-	-
JC2	B-419DH	UD-5	158.0	-128.3	129.2	20.3	47	30	CL	10.64	UU	6.285	0.59	-	-	-	-	-	-
JC2	B-419DH	UD-6	178.0	-148.3	129.2	22.0	53	33	CH	11.89	UU	6.226	0.52	-	-	-	-	-	-
JC2	B-419DH	UD-7	198.0	-168.3	123.5	23.1	56	36	CH	13.15	UU	3.847	0.29	-	-	-	-	-	-
JC2	B-443	UD-14	156.0	-124.9	127.5	24.2	67	43	CH	10.55	UNC	1.521	0.14	-	-	-	-	-	-
JC2	B-443	UD-15	172.0	-140.9	127.0	25.7	51	31	CH	11.55	UNC	0.324	0.03	-	-	-	-	-	-
			Stratum J Clay	**Minimum**	113.9	16.1	30.0	12.0	-	-	-	0.127	0.01	0.50	0.0	0.21	0.0	-	-
				Maximum	131.1	34.0	73.0	50.0	-	-	-	6.579	0.77	7.69	25.0	6.92	28.8	-	-
				Average	125.2	22.3	52.6	32.5	-	-	-	3.014	0.31	2.72	6.9	2.33	10.5	-	-
											Alternate Minimum	2.003	0.25						
	Cells not included in Alternate Min, Max, and Average values calculated from this table.										**Alternate Maximum**	6.579	0.77						
											Alternate Average	4.332	0.45						
JS1	B-405DH	UD-7	148.0	-116.9	121.6	24.0	NV	NP	SM	10.03	-	-	-	-	-	-	-	0.0	32.0
			Stratum J Sand	**Minimum**	121.6	24.0	-	-	-	-	-	-	-	-	-	-	-	0.0	32.0
				Maximum	128.0	32.0	-	-	-	-	-	-	-	-	-	-	-	0.0	32.0
				Average	125.3	26.7	NV	NP	-	-	-	-	-	-	-	-	-	0.0	32.0
KCLAY	B-305-DH	UD-11	213.0	-183.2	126.6	21.8	45	31	CL	14.07	UNC	2.775	0.20	-	-	-	-	-	-
KCLAY	B-405DH	UD-11	233.0	-201.9	131.5	16.8	-	-	CH	15.33	UU	3.958	0.26	-	-	-	-	-	-
			Stratum K Clay	**Minimum**	126.6	16.8	45.0	31.0	-	-	-	2.775	0.20	-	-	-	-	-	-
				Maximum	131.5	21.8	45.0	31.0	-	-	-	3.958	0.26	-	-	-	-	-	-
				Average	129.1	19.3	45.0	31.0	-	-	-	3.366	0.23	-	-	-	-	-	-
											Alternate Minimum	3.958	0.26						
	Cells not included in Alternate Min, Max, and Average values calculated from this table.										**Alternate Maximum**	3.958	0.26						
											Alternate Average	3.958	0.26						
K SS	B-305-DH	UD-12	228.0	-198.2	126.8	21.5	NV	NP	SM	15.00	-	-	-	-	-	-	-	0.0	29.0
			Stratum K Sand/Silt	**Minimum**	126.8	21.5	-	-	-	-	-	-	-	-	-	-	-	0.0	29.0
				Maximum	126.8	21.5	-	-	-	-	-	-	-	-	-	-	-	0.0	29.0
				Average	126.8	21.5	NV	NP	-	-	-	-	-	-	-	-	-	0.0	29.0
NC1	B-305-DH	UD-14	288.0	-258.2	112.9	37.7	84	58	CH	18.82	UU	0.221	0.01	-	-	-	-	-	-

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

Strata	Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Average Total Unit Weight (pcf)	Average Natural Moisture Content (percent) at e_o	Liquid Limit, LL (percent)	Plasticity Index, PI (percent)	USCS Symbol	Effective Vertical Overburden Stress (ksf), σ'_v	UNC/ UU Tests [1] Test Type	UNC/ UU Tests [1] Undrained Shear Strength, Su (ksf)	UNC/ UU Tests [1] Ratio, Su/ σ'_v	CIU-Bar Tests [1] Undrained Cohesion (ksf)	CIU-Bar Tests [1] Undrained Friction Angle (degrees)	CIU-Bar Tests [1] Drained Cohesion (ksf)	CIU-Bar Tests [1] Drained Friction Angle (degrees)	DS Tests [1] Drained Cohesion (ksf)	DS Tests [1] Drained Friction Angle (degrees)
NC1	B-305-DH	UD-15A	316.0	-286.2	119.7	30.4	-	-	CH	20.51	UNC	1.356	0.07	-	-	-	-	-	-
NC5	B-405DH	UD-20	458.5	-427.4	123.7	23.8	-	-	CH	29.47	UU	4.487	0.15	-	-	-	-	-	-
NC6	B-405DH	UD-25	598.0	-566.9	127.3	18.4	45	29	CL	37.92	UNC	0.899	0.02	-	-	-	-	-	-
			Stratum N Clay	**Minimum**	112.9	18.4	45.0	29.0	-	-	-	0.221	0.01	-	-	-	-	-	-
				Maximum	127.3	37.7	84.0	58.0	-	-	-	4.487	0.15	-	-	-	-	-	-
				Average	120.9	27.6	64.5	43.5	-	-	-	1.740	0.06	-	-	-	-	-	-
											Alternate Minimum	4.487	0.152						
	Cells not included in <u>Alternate</u> Min, Max, and Average values calculated from this table.										**Alternate Maximum**	4.487	0.152						
											Alternate Average	4.487	0.152						
NS1	B-405DH	UD-15	343.0	-311.9	130.2	20.9	NV	NP	SP	22.20	-	-	-	-	-	-	-	-	-
NS4	B-305-DH	UD-21A	453.3	-423.5	125.8	22.0	NV	NP	SP-SM	29.22	-	-	-	-	-	-	-	-	-
			Stratum N Sand	**Minimum**	125.8	20.9	-	-	-	-	-	-	-	-	-	-	-	-	-
				Maximum	130.2	22.0	-	-	-	-	-	-	-	-	-	-	-	-	-
				Average	128.0	21.5	NV	NP	-	-	-	-	-	-	-	-	-	-	-

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	0.050	0.660	3.2	3.7	1.73
		Maximum	0.023	0.316	0.750	10.0	25.0	9.85
		Average	0.017	0.235	0.702	6.7	10.5	5.32
D	8	Minimum	0.007	0.086	0.710	6.1	1.7	0.04
		Maximum	0.033	0.468	0.980	18.3	5.8	0.52
		Average	0.026	0.285	0.830	13.4	3.9	0.20
F	6	Minimum	0.013	0.199	0.630	13.4	2.3	0.15
		Maximum	0.040	0.262	0.810	23.7	3.7	3.41
		Average	0.028	0.238	0.703	18.6	3.1	0.91
J Clay	11	Minimum	0.013	0.130	0.480	14.1	1.2	0.01
		Maximum	0.086	0.472	0.790	27.9	2.7	14.17
		Average	0.038	0.224	0.615	18.6	1.9	2.34
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)	Average 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	0.4	122.0	25.0	62	43	CH	0.750	0.269	0.154	0.023	0.013	3.22	8.0	4.42
B-333	UD-1	8.0	22.5	0.8	123.8	24.8	43	27	CL	0.680	0.282	0.168	0.020	0.012	9.20	11.5	9.85
B-333	UD-2	18.0	12.5	1.4	119.1	23.8	30	11	CL	0.660	0.050	0.030	0.000	0.000	5.22	3.7	1.73
B-432	UD-1	3.0	28.2	0.4	123.0	22.8	65	42	CH	0.680	0.316	0.188	0.020	0.012	10.00	25.0	8.81
B-432	UD-2	15.0	16.2	1.3	121.4	24.8	31	11	CL	0.740	0.259	0.149	0.020	0.011	5.70	4.4	1.79
> MIMIMUM, STRATUM A				0.4	119.1	22.8	30	11	-	0.660	0.050	0.030	0.000	0.000	3.22	3.7	1.73
> MAXIMUM, STRATUM A				1.4	123.8	25.0	65	43	-	0.750	0.316	0.188	0.023	0.013	10.00	25.0	9.85
> AVERAGE, STRATUM A				0.9	121.9	24.3	46	27	-	0.702	0.235	0.138	0.017	0.010	6.67	10.5	5.32
STRATUM D																	
B-305DH	UD-4	53.0	-23.2	3.5	125.2	26.7	45	25	CL	0.710	0.252	0.147	0.017	0.010	14.30	4.1	4.46E-02
B-338	UD-2	48.0	-15.9	3.3	122.3	27.4	44	26	CL	0.760	0.256	0.145	0.030	0.017	13.90	4.2	2.59E-01
B-421	UD-2	53.0	-22.7	3.5	120.7	28.1	63	42	CH	0.810	0.213	0.118	0.033	0.018	6.08	1.7	1.39E-01
B-909	UD-2	43.0	-13.3	2.9	121.1	25.7	62	38	CH	0.780	0.086	0.048	0.007	0.004	11.27	3.9	5.18E-01
B-909	UD-3	48.0	-18.3	3.2	115.9	34.9	74	53	CH	0.920	0.468	0.244	0.030	0.016	16.90	5.3	7.34E-02
B-940	UD-4	46.0	-16.3	3.0	117.9	32.6	61	38	CH	0.940	0.365	0.188	0.033	0.017	17.50	5.8	3.46E-01
B-940	UD-6	66.0	-36.3	4.8	126.2	26.5	54	33	CH	0.740	0.345	0.198	0.027	0.016	18.30	3.8	4.32E-02
B-949	UD-2	53.5	-24.8	3.4	118.0	32.5	65	36	CH	0.980	0.296	0.149	0.030	0.015	8.60	2.5	1.73E-01
> MIMIMUM, STRATUM D				2.9	115.9	25.7	44	25	-	0.710	0.086	0.048	0.007	0.004	6.08	1.7	4.46E-02
> MAXIMUM, STRATUM D				4.8	125.2	34.9	74	53	-	0.920	0.468	0.244	0.033	0.018	18.30	5.8	5.18E-01
> AVERAGE, STRATUM D				3.5	121.0	28.6	58	37	-	0.796	0.255	0.141	0.023	0.013	13.6	3.9	2.07E-01
STRATUM F																	
B-303	UD-2	88.0	-61.4	5.9	127.9	28.6	57	39	CH	0.810	0.249	0.138	0.040	0.022	13.40	2.3	3.08E-01
B-419DH	UD-1	78.0	-48.3	5.5	127.8	23.4	47	23	CL	0.630	0.243	0.149	0.037	0.023	18.00	3.3	1.50E-01
B-421	UD-3	83.0	-52.7	5.8	125.9	24.6	56	36	CH	0.700	0.229	0.135	0.040	0.024	17.99	3.1	3.41
B-443	UD-2A	93.0	-61.9	6.4	124.9	25.1	55	31	CH	0.710	0.199	0.116	0.013	0.008	23.70	3.7	8.64E-01
B-443	UD-3	96.0	-64.9	6.6	125.1	25.4	62	37	CH	0.690	0.246	0.146	0.020	0.012	17.30	2.6	2.50E-01

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

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Initial Effective Overburden Pressure (kips per square foot)	Average 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)
B-949	UD-6	83.5	-54.8	5.8	125.9	24.6	66	43	CH	0.680	0.262	0.156	0.020	0.012	20.90	3.6	5.00E-01
> MIMIMUM, STRATUM F				5.5	124.9	23	47	23		0.630	0.229	0.135	0.037	0.008	13.40	2.3	1.50E-01
> MAXIMUM, STRATUM F				6.6	127.9	29	57	39	-	0.810	0.249	0.149	0.040	0.024	23.70	3.7	3.41
> AVERAGE, STRATUM F				6.0	126.3	26	53	33	-	0.713	0.240	0.140	0.039	0.017	18.55	3.1	1.29
STRATUM J CLAY 1																	
B-305DH	UD-7	123.0	-93.2	8.3	129.2	21.6	-	-	CH	0.590	0.186	0.117	0.030	0.019	18.90	2.3	6.00
B-305DH	UD-8	138.0	-108.2	9.3	129.5	19.0	32	18	CL	0.550	0.173	0.112	0.023	0.015	16.20	1.7	3.37
B-319DH	UD-1	128.0	-98.0	8.5	125.0	17.6	70	45	CH	0.600	0.130	0.081	0.013	0.008	14.10	1.7	14.17
B-419DH	UD-3	118.0	-88.3	8.0	127.6	24.4	56	39	CH	0.680	0.276	0.164	0.030	0.018	21.40	2.7	4.68E-02
B-419DH	UD-4	138.0	-108.3	9.3	129.3	17.3	40	25	CL	0.520	0.289	0.190	0.086	0.057	15.48	1.7	1.90E-01
B-443	UD-9A	133.0	-101.9	9.0	133.7	16.6	52	34	CH	0.480	0.183	0.124	0.022	0.015	17.80	2.0	7.00E-01
> MIMIMUM, STRATUM J CLAY 1				8.0	125.0	17.3	32	18	-	0.520	0.130	0.081	0.013	0.008	14.10	1.7	4.68E-02
> MAXIMUM, STRATUM J CLAY 1				9.3	129.5	24.4	70	45	-	0.680	0.289	0.190	0.086	0.057	21.40	2.7	14.17
> AVERAGE, STRATUM J CLAY 1				8.7	128.1	20.0	50	32	-	0.588	0.211	0.133	0.036	0.023	17.31	2.0	4.76
STRATUM J CLAY 2																	
B-319DH	UD-4	173.0	-144.6	11.3	124.4	26.9	65	43	CH	0.730	0.173	0.100	0.040	0.023	19.25	1.7	5.02E-02
B-319DH	UD-5	188.0	-159.6	12.2	122.7	29.1	62	41	CH	0.790	0.472	0.264	0.060	0.034	27.87	2.3	1.04E-01
B-419DH	UD-5	158.0	-128.3	10.5	129.2	21.3	47	30	CL	0.600	0.199	0.124	0.047	0.029	19.30	1.8	3.64E-02
B-419DH	UD-6	178.0	-148.3	11.8	129.2	23.3	53	33	CH	0.660	0.233	0.140	0.043	0.026	18.40	1.6	8.10E-02
B-419DH	UD-7	198.0	-168.3	13.0	123.5	19.4	56	36	CH	0.560	0.149	0.096	0.027	0.017	15.35	1.2	3.33
> MIMIMUM, STRATUM J CLAY 2				10.5	122.7	19.4	47	30	-	0.560	0.149	0.096	0.027	0.017	15.35	1.2	3.64E-02
> MAXIMUM, STRATUM J CLAY 2				13.0	129.2	29.1	65	43	-	0.790	0.472	0.264	0.060	0.034	27.87	2.3	3.33
> AVERAGE, STRATUM J CLAY 2				11.8	125.8	24.0	57	37	-	0.668	0.245	0.145	0.043	0.026	20.03	1.7	0.72

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

Boring Number	Sample Number	Sample Top Depth (feet)	Sample Top Elevation (feet)	Initial Effective Overburden Pressure (kips per square foot)	Average 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 K CLAY																	
B-305DH	UD-11	213.0	-183.2	14.0	126.6	22.6	45	31	CL	0.610	0.249	0.155	0.023	0.014	27.90	2.0	1.34E-01
B-405DH	UD-11	233.0	-201.9	15.2	131.5	18.3	-	-	CH	0.510	0.103	0.068	0.010	0.007	20.20	1.3	2.09
> MIMIMUM, STRATUM K CLAY				14.0	126.6	18.3	45	31	-	0.510	0.103	0.068	0.010	0.007	20.20	1.3	1.34E-01
> MAXIMUM, STRATUM K CLAY				15.2	131.5	22.6	45	31	-	0.610	0.249	0.155	0.023	0.014	27.90	2.0	2.09
> AVERAGE, STRATUM K CLAY				14.6	129.1	20.4	45	31	-	0.560	0.176	0.111	0.017	0.010	24.05	1.7	1.11
STRATUM N CLAY 1																	
B-305DH	UD-15A	316.0	-286.2	20.4	119.7	29.7	-	-	CH	0.790	0.379	0.212	0.066	0.037	18.90	0.9	5.13E-02
STRATUM N CLAY 5																	
B-405DH	UD-20	458.5	-427.4	29.4	123.7	30.0	-	-	CH	0.870	0.292	0.156	0.033	0.018	17.89	0.6	4.42E-02
> MINIMUM, STRATUM N CLAY				20.4	119.7	29.7	-	-	-	0.790	0.292	0.156	0.033	0.018	17.89	0.6	4.42E-02
> MAXIMUM, STRATUM N CLAY				20.4	123.7	30.0	-	-	-	0.870	0.379	0.212	0.066	0.037	18.90	0.9	5.13E-02
> AVERAGE, STRATUM N CLAY				20.4	121.7	29.9	-	-	-	0.830	0.336	0.184	0.050	0.027	18.40	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	6.7	10.0+
D	13.4	3.9
F	18.6	3.1
J Clay	18.6	1.9
K Clay	24.1	1.7
L	Not Tested	Not Tested
N Clay	18.4	0.8
From Correlations with CPT Data		
A	N/A	10.0+
D	N/A	4.2
F	N/A	2.4
J Clay	N/A	1.7
K Clay	N/A	Not Reached
L	N/A	Not Reached
N Clay	N/A	Not Reached
Selected Values for Engineering Use		
A	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	20.5	1.3
N Clay	37	1.3

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

Strata A through E					
Relationship Employed	**High Strain Elastic Moduli By Stratum (ksf)**				
	A/A (Fill)	**B**	**C**	**D**	**E**
$E = f(N_{60})$	N/A	515	1,785	N/A	2,490
$E = f(S_u, OCR)$	1,190	N/A	N/A	1,635	N/A
$E = f(V_s)$	N/A	1,540	1,820	N/A	3,475
E = f(PI)	1,110	N/A	N/A	2.830	N/A
E Value Selected for Engineering Use	1,135	1,200	1,810	2,430	3,145
E_d (Drained) Effective Stress Value selected for Engineering Use	985	1,200	1,810	1,865	3,145
μ_d (Drained) Effective Stress Value Selected for Engineering Use	0.30	0.30	0.30	0.15	0.30
Strata F through K Clay					
Relationship Employed	**High Strain Elastic Moduli By Stratum (ksf)**				
	F	**H**	**J Clay**	**J Sand**	**K Clay**
$E = f(N_{60})$	N/A	2,725	N/A	4,420	N/A
$E = f(S_u, OCR)$	1,645	N/A	4,955	N/A	4,445
$E = f\ (V_s)$	N/A	3,500	N/A	4,925	N/A
E = f (PI)	3,030	N/A	3,735	N/A	4,305
E Value Selected for Engineering Use	2,570	3,240	4,140	4,755	4,350
E_d (Drained) Effective Stress Value selected for Engineering Use	1,970	3,240	3,175	4,755	3,335
μ_d (Drained) Effective Stress Value Selected for Engineering Use	0.15	0.30	0.15	0.30	0.15

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

Strata K Sand/Silt through N Sand					
Relationship Employed	**High Strain Elastic Moduli By Stratum (ksf)**				
	K Sand/Silt	**L**	**M**	**N Clay**	**N Sand**
E = f(N_{60})	3,195	N/A	4,700	N/A	6,625
E = f(S_u,OCR)	N/A	4,445	N/A	5,130	N/A
E = f (V_s)	5,775	N/A	4,175	9,220	14,155
E = f (PI)	N/A	3,575	N/A	N/A	N/A
E Value Selected for Engineering Use	4,915	3,865	4,350	7.855	11,645
E_d (Drained) Effective Stress Value selected for Engineering Use	4,915	2,965	4,350	6,020	11,645
μ_d (Drained) Effective Stress Value Selected for Engineering Use	0.30	0.15	0.30	0.15	0.30

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

Strata A through E					
Relationship Employed	High Strain Shear Moduli By Stratum (ksf)				
	A	B	C	D	E
$G = \dfrac{E_d}{2(1+\mu_d)}$ G Value Selected for Engineering Use	370	465	695	800	1,215
Strata F through K Clay					
Relationship Employed	High Strain Shear Moduli By Stratum (ksf)				
	F	H	J Clay	J Sand	K Clay
$G = \dfrac{E_d}{2(1+\mu_d)}$ G Value Selected for Engineering Use	850	1,250	1,380	1,830	1,450
Strata K Sand/Silt through N Sand					
Relationship Employed	High Strain Shear Moduli By Stratum (ksf)				
	K Sand/Silt	L	M	N Clay	N Sand
$G = \dfrac{E_d}{2(1+\mu_d)}$ G Value Selected for Engineering Use	1,890	1,300	1,675	2,620	4,470

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

Parameter [1]	Stratum				
	A	B	C	D	E
Average Thickness, feet	19	7	19	21	18
USCS Group Symbol	CH, CL	ML, CL, SM, SC	SM, SP-SM, ML	CH, CL, ML, CL-ML	SP-SM, SM, ML, SP, SC
Natural Moisture content (MC), %	24	24	23	26	21
Moist Unit Weight, (γ_{moist}), pcf	124	121	122	122	123
Fines content, %	96	67	23	79	20
Liquid Limit (LL), %	56	NV	NV	57	NV
Plasticity Index (PI), %	40	NP	NP	40	NP
Uncorrected SPT N-value, bpf	9	8	23	15	33
Corrected SPT N_{60}-value, bpf	11	11	38	23	53
Corrected SPT $(N_1)_{60}$-value, bpf	N/A	12	35	N/A	31
Shear Wave Velocity (V_s), feet/sec	575	725	785	925	1,080
Undrained shear strength (S_U), ksf	1.5	N/A	N/A	3.0	N/A
Drained Friction Angle (ϕ'), degrees [8]	N/A	30	35	16	35
Drained Cohesion, (c'), ksf	N/A	N/A	N/A	1.2	N/A
Elastic modulus (High Strain) (E_s), ksf	1,135	1,200	1,810	2,430	3,145
Elastic Modulus (High Strain) (E_d), ksf	985	1,200	1,810	1,865	3,145
Shear modulus (High Strain) (G_s), ksf	370	465	695	800	1,215
Shear modulus (Low Strain) (G_{max}), ksf	1,270	1,970	2,335	3,240	,455
Poisson's Ratio (drained) (μ_d)	0.30	0.30	0.30	0.15	0.30
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 ($K_{0, NC}$)	0.7	0.5	0.4	0.7	0.4
- At-Rest ($K_{0, OCR}$)	1.4	N/A	N/A	1.0	N/A
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	.285	N/A
- Recompression Index (C_r)	0.017	N/A	N/A	0.026	N/A
- Preconsolidation Pressure (P_c'), ksf	6.3	N/A	N/A	12.3	N/A
- Overconsolidation Ratio (OCR)	7.0	N/A	N/A	3.3	N/A

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

Parameter [1]	Stratum				
	F	H	J Clay	J Sand	K Clay
Average Thickness, feet	16	17	70 [2]	37.5 [3]	19
USCS Group Symbol	CH, CL, ML, CL-ML	SP-SM, SM	CH, CL, ML	SM, ML, SP-SM, CL	CL, CH
Natural Moisture content (MC), %	24	19	23	22	23
Moist Unit Weight, (γ_{moist}), pcf	125	125	125	125	124
Fines content, %	94	18	90	50	87
Liquid Limit (LL), %	57	NV	54	NV	50
Plasticity Index (PI), %	40	NP	35	NP	35
Uncorrected SPT N-value, bpf	22	42	32	55	15
Corrected SPT N_{60}-value, bpf	34	58	48	94	26
Corrected SPT $(N_1)_{60}$-value, bpf	NA	28	NA	38	NA
Shear Wave Velocity (V_s), feet/sec	945	1,075	1,085	1,275	1,170
Undrained shear strength (S_U), ksf	3.4	N/A	3.8	N/A	3.9
Drained Friction Angle (ϕ'), degrees [8]	8	35	11	33	11
Drained Cohesion, (c'), ksf	2.0	N/A	2.3	N/A	2.3
Elastic modulus (High Strain) (E_s), ksf	2,570	3,240	4,140	4,755	4,350
Elastic Modulus (High Strain) (E_d), ksf	1,970	3,240	3,175	4,755	3,335
Shear modulus (High Strain) (G_s), ksf	850	1,250	1,380	1,830	1,450
Shear modulus (Low Strain) (G_{max}), ksf	3,470	4,490	4,570	6,310	5,270
Poisson's Ratio (drained) (μ_d)	0.15	0.30	0.15	0.30	0.15
Coefficient of Subgrade Reaction (k_1), kcf	300	600	N/A	N/A	N/A
Earth Pressure Coefficients					
- Active (K_a)	0.5	0.3	N/A	N/A	N/A N/A
- Passive (K_p)	2.0	3.7	N/A	N/A	N/A
- At-rest ($K_{0, NC}$)	0.7	0.4	N/A	N/A	N/A
- At-rest ($K_{0, OCR}$)	1.00	N/A	N/A	N/A	N/A
Sliding Coefficient (tangent)	0.30	0.40	N/A	N/A	N/A
Consolidation Properties					
- Compression Index (C_c)	0.238	N/A	0.224	N/A	0.176
- Recompression Index (C_r)	0.028	N/A	0.038	N/A	0.017
- Preconsolidation Pressure (P_c'), ksf	15.5	N/A	18.5	N/A	18.3
- Overconsolidation Ratio (OCR)	2.6	N/A	1.7	N/A	1.3

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	25.3	5	15	>228 [4]	119 [5]
USCS Group Symbol	SM, ML	CH	SM	CH, CL, SC	SM, SP-SM, SC
Natural Moisture content (MC), %	21	29	19	25	22
Moist Unit Weight, (γ_{moist}), pcf	127	124 [7]	127 [6]	123	128
Fines content, %	45	87 [7]	45 [6]	79	21
Liquid Limit (LL), %	NV	73	NV	67	NV
Plasticity Index (PI), %	NP	50	NP	45	NP
Uncorrected SPT N-value, bpf	60	21	60	32	83
Corrected SPT N_{60}-value, bpf	68	36	100	54	141
Corrected SPT $(N_1)_{60}$-value, bpf	27	N/A	40	N/A	56
Shear Wave Velocity (V_s), feet/sec	1,370	975	1,165	1,290	1,655
Undrained shear strength (S_U), ksf	N/A	3.9	N/A	4.5	N/A
Drained Friction Angle (ϕ'), degrees [8]	31	N/A	31 [6]	N/A	36
Drained Cohesion, (c'), ksf	N/A	N/A	N/A	N/A	N/A
Elastic modulus (High Strain) (E_s), ksf	4,915	3,865	4,350	7,855	11,645
Elastic Modulus (High Strain) (E_d), ksf	4,915	2,965	4,350	6,020	11,645
Shear modulus (High Strain) (G_s), ksf	1,890	1,300	1,675	2,620	4,470
Shear modulus (Low Strain) (G_{max}), ksf	7,400	3,660	5,350	6,355	10,890
Poisson's Ratio (drained) (μ_d)	0.30	0.15	0.30	0.15	0.30
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 ($K_{0, NC}$)	N/A	N/A	N/A	N/A	N/A
- At-rest ($K_{0, OCR}$)	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 [7]	N/A	0.336	N/A
- Recompression Index (C_r)	N/A	0.017 [7]	N/A	0.050	N/A
- Preconsolidation Pressure (P_c'), ksf	N/A	20.5	N/A	37	N/A
- Overconsolidation Ratio (OCR)	N/A	1.3	N/A	1.3	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

[2] Sub-stratum J Clay thickness = combined thickness of J Clay 1 (29 feet) + J Clay 2 (41 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)

[4] 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 5 (54 feet) + N Clay 6 (>68.5 feet)

[5] 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 (16 feet) + N Sand 5 (35 feet)

[6] Value from Sub-stratum K Sand/Silt selected. $(N_1)_{60}$ based on $C_N = 0.4$.

[7] Value from Sub-stratum K Clay selected

[8] Drained friction angle, ϕ' for clays is for stresses above P_c'. See text for strength parameters for clays at stresses below P_c'.

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	9.193	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 [4]					
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
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

Table 2.5S.4-19 As-Built Boring Information (Continued)

Boring Number	Northing [1] (feet)	Easting [1] (feet)	Ground El. [2] (feet)	Depth (feet)	Base El. [2] (feet)
BORINGS - STP 3 [4] (continued)					
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
B-948	63,227.49	42,967.91	31.3	100	-68.7
BORINGS - STP 4 [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
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

Table 2.5S.4-19 As-Built Boring Information (Continued)

Boring Number	Northing [1] (feet)	Easting [1] (feet)	Ground El. [2] (feet)	Depth (feet)	Base El. [2] (feet)
BORINGS - STP 4 [4] (continued)					
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
B-443	63,182.04	42,133.51	30.6	200	-169.4
B-444	63,058.00	42,133.47	30.0	100	-70.0
B-445	63,057.99	42,240.47	31.3	100	-68.7
B-450	63,539.57	42,057.93	28.8	100.0	-71.2
B-913	63,253.07	42,031.18	30.6	50	-19.4
B-914	63,218.30	42,181.90	28.2	100	-71.8
B-915	63,357.95	42,118.79	29.0	50	-21.0
B-916	63,599.37	42,120.70	27.8	50	-22.2

Table 2.5S.4-19 As-Built Boring Information (Continued)

Boring Number	Northing [1] (feet)	Easting [1] (feet)	Ground El. [2] (feet)	Depth (feet)	Base El. [2] (feet)
B-944	62,952.54	42,205.50	30.1	100	-69.9
B-945	62,952.51	42,411.48	29.6	50	-20.4
B-946	62,952.51	42,589.48	31.0	50	-19.1
B-947	63,044.57	42,784.81	31.5	50	-18.5
BORINGS - OUTSIDE POWER BLOCK					
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-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
B-940	63,471.37	41,379.59	29.7	125	-95.3
B-941	63,077.70	41,410.59	29.8	50	-20.2
B-942	62,952.52	41,575.55	31.0	50	-19.0
B-943	62,952.50	41,801.53	31.5	50	-18.5
B-949	63,604.36	41,778.94	28.7	125	-96.3

[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] Boring B-917, made midway between STP 3 and STP 4, is included with STP 3 here
[4] Refer to Reference 2C for 2008 As-Built boring information.

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	D	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	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)	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	E	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)	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)	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
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)
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 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 Sand 5	518.0	-486.9
B-405DH	UD23	SM	N Sand 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 Interbed 2	188.0	-156.8
B-409	UD6	CH	J Clay 2	198.0	-166.8
UNDISTURBED TUBE SAMPLES - STP 4 (continued)					
B-415	UD1	CH	F	88.0	-58.0

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)
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	F	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)
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	C/ D	43.0	-12.8
B-443	UD-1	CH	F	86	-55.39
B-443	UD-2A	CH	F	93	-62.39
B-443	UD-3	CH	F	96	-65.39
B-443	UD-4	CH	F	101	-70.39
B-443	UD-6	CH	JC1	112	-81.39
B-443	UD-7A	CL	JC1	123	-92.39
B-443	UD-9A	CH	JC1	133	-102.39
B-443	UD-11	CH	JC1	141	-110.39
B-443	UD-14	CH	JC2	156	-125.39
B-443	UD-15	CH	JC2	172	-141.39
B-916	UD1	CH	A	13	14.8
B-916	UD2	NR (may be SM)	C	28	-0.2
B-916	UD2A	NR (may be SM)	C	30	-2.2
B-916	UD3	CH	D	48	-20.2
UNDISTURBED TUBE SAMPLES - OUTSIDE POWER BLOCK					
B-902	UD1	CH	A	5	24.1
B-902	UD2	CH	A	15	14.1
B-902	UD3	SM	C	23	6.1
B-904	UD1	CH	A	5	24.8
B-904	UD2	CH	A	18	11.8
B-904	UD3	ML	B	28	1.8
B-904	UD4	SC	D	53	-23.2

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)
B-904	UD5	CH	F	83	-53.2
B-907	UD1	CH	A	3	26.2
B-907	UD2	CH	A	13	16.2
B-907	UD3	SM	B	28	1.2
B-909	UD1	SM	C	33	-3.3
B-909	UD2	CH	D	43	-13.3
B-909	UD3	CH	D	48	-18.3
B-909	UD4	CH	D	53	-23.3
B-909	UD5	CL	F	85	-55.3
B-909	UD6	CH	F	93	-63.3
B-909	UD7	CH	F	98	-68.3
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)	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
B-940	UD3	CH	D	41	-12.28
B-940	UD-4	CH	D	46	-17.28
B-940	UD-5	CH	D	56	-27.28
B-940	UD-6	CH	D	66	-37.28
B-940	UD-7	CH	D	76	-47.28
B-940	UD-8	CH	D	91	-62.28
B-949	UD-2	CH	D	53.5	-25.78
B-949	UD-3	CH	D	61	-33.28

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)
B-949	UD-4	CH	D	71	-43.28
B-949	UD-6	CH	D	83.5	-55.78
B-949	UD-7	CH	D	91	-63.28
B-949	UD-8	SM	E	101	-73.28
B-949	UD-9	CL	F	111	-83.28

[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
C-947[4]	63,127.31	42,867.72	30.71	50	-19.29
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-405S	63,120.00	42,240.54	31.48	75.3	-43.82
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-907	63,219.02	41,968.73	28.5	50	-21.2
C-908	63,219.72	42,082.33	30.9	50	-19.1
C-916 [3]	63,217.32	42,280.50	31.4	39.0	-7.6
C-917	63,281.30	42,122.51	30.7	50	-19.3
C-918	63,484.09	42,118.30	25.4	50	-24.6
C-944	62,952.53	42,102.50	30.13	74.1	-43.97
C-945	62,952.55	42,308.52	31.46	50	-18.54
C-946	62,952.55	42,692.95	32.02	50	-17.98
C-949	63,375.80	41,999.97	27.72	50	-22.28
CONE PENETRATION TESTS - OUTSIDE POWER BLOCK					
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-909	63,464.25	43,948.29	30.2	40.0	-9.8

Table 2.5S.4-21 As-Built CPT Information (Continued)

CPT Number	Northing [1] (feet)	Easting [1] (feet)	Ground El. [2] (feet)	Depth (feet)	Base El. [2] (feet)
C-940	63,174.72	41,370.39	28.72	50	-21.28
C-941	62,952.49	41,462.49	31.81	50	-18.19
C-942	62,952.51	41,688.53	30.36	50	-19.64
C-943	62,952.51	41,914.51	30.71	50	-19.29
C-948	63,649.01	41,886.79	29.81	37.6	-7.79

[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] Boring CPT C-916, made between STP 3 and STP 4, is included with STP 3.
[4] Not included in site characterization for engineering properties

Table 2.5S.4-22 As-Built Observation Well Information

OW Number	Northing [1]	Easting [1]	Reference El. [2]	Well Depth	Base El. [2]
	(feet)	(feet)	(feet)	(feet)	(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 - OUTSIDE POWER BLOCK					
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
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 μ
		30	10	20	14				1,078	290	578	575	0.45
		30	25	5	6.5				670	330	451	450	0.43
A	Clay	25	20	5	11.5	124	40	1.6	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
		10	0	10	29				1,090	400	728	725	0.48
B	Silt	10	5	5	26.5	121	N/A	N/A	1,060	400	707	705	0.48
		5	0	5	31.5				1,090	470	758	755	0.49
		0	-20	20	44				1,430	440	786	785	0.49
		0	-5	5	36.5				1,430	440	756	755	0.49
C	Sand	-5	-10	5	41.5	122	N/A	N/A	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
		-20	-40	20	64				1,550	540	929	925	0.48
		-20	-25	5	56.5				1,020	540	702	700	0.49
D	Clay	-25	-30	5	61.5	122	40	3.0	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
		-40	-60	20	84				1,627	720	1,082	1,080	0.48
		-40	-45	5	76.5				1,430	940	1,196	1,195	0.48
E	Sand	-45	-50	5	81.5	123	N/A	N/A	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

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 μ
F	Clay	-60	-75	15	101.5	125	40	3.3	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
H	Sand	-75	-90	15	116.5	125	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	3.4	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

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 μ
J Clay 2	Clay	-140	-185	45	196.5	125	35	3.4	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	124	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
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	124	50	3.0	1,410	750	979	975	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 μ
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	123	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 μ
N Clay 2	Clay	-324	-332	8	362.0	123	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	123	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

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 μ
N Clay 4	Clay	-392	-422	30	441.0				1,810	910	1,207	1,205	0.47
		-392	-397	5	428.5				1,810	1,330	1,537	1,535	0.46
		-397	-402	5	433.5				1,260	1,040	1,115	1,115	0.48
		-402	-407	5	438.5	123	45	3.0	1,390	1,050	1,190	1,190	0.48
		-407	-412	5	443.5				1,400	1,040	1,260	1,260	0.47
		-412	-417	5	448.5				1,380	1,000	1,167	1,165	0.48
		-417	-422	5	453.5				1,100	910	975	975	0.48
N Sand 4	Sand	-422	-430	8	460.0				1,720	870	1,359	1,355	0.47
		-422	-427	5	458.5	128	N/A	N/A	1,720	870	1,292	1,290	0.47
		-427	-430	3	462.5				1,580	1,370	1,460	1,460	0.46
N Clay 5	Clay	-430	-484	54	491.0				1,820	970	1,223	1,220	0.48
		-430	-435	5	466.5				1,540	1,000	1,260	1,260	0.47
		-435	-440	5	471.5				1,460	970	1,184	1,180	0.48
		-440	-445	5	476.5				1,050	1,030	1,040	1,040	0.48
		-445	-450	5	481.5				1,060	1,000	1,040	1,040	0.48
		-450	-455	5	486.5	123	45	3.0	1,460	1,080	1,273	1,270	0.48
		-455	-460	5	491.5				1,280	1,110	1,167	1,165	0.48
		-460	-465	5	496.5				1,130	1,080	1,110	1,110	0.48
		-465	-470	5	501.5				1,190	1,170	1,180	1,180	0.48
		-470	-475	5	506.5				1,280	1,110	1,180	1,180	0.48
		-475	-480	5	511.5				1,420	1,190	1,330	1,330	0.47
		-478	-484	4	516.0				1,820	1,750	1,785	1,785	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 μ
N Sand 5	Sand	-484	-502	18	527.0	128	N/A	N/A	2,250	1,540	1,848	1,845	0.45
		-484	-489	5	520.5				2,250	1,790	1,972	1,970	0.44
		-489	-494	5	525.5				2,080	1,720	1,910	1,910	0.44
		-494	-499	5	530.5				2,020	1,540	1,735	1,735	0.45
		-499	-502	3	534.5				1,800	1,740	1,770	1,770	0.45
N Clay 6	Clay	-502	-575	73	572.5	123	45	3.0	1,880	1,120	1,347	1,345	0.47
		-502	-507	5	538.5				1,880	1,620	1,750	1,750	0.45
		-507	-512	5	543.5				1,250	1,180	1,217	1,217	0.48
		-512	-517	5	548.5				1,200	1,120	1,170	1,170	0.48
		-517	-522	5	553.5				1,270	1,140	1,190	1,190	0.48
		-522	-527	5	558.5				1,330	1,320	1,323	1,323	0.47
		-527	-532	5	563.5				1,190	1,130	1,160	1,160	0.48
		-532	-537	5	568.5				1,320	1,210	1,267	1,265	0.47
		-537	-542	5	573.5				1,230	1,220	1,227	1,225	0.47
		-542	-547	5	578.5				1,560	1,160	1,363	1,360	0.47
		-547	-552	5	583.5				1,400	1,270	1,317	1,315	0.47
		-552	-557	5	588.5				1,370	1,290	1,330	1,330	0.47
		-557	-562	5	593.5				1,620	1,470	1,523	1,520	0.47
		-562	-567	5	598.5				1,800	1,280	1,508	1,505	0.47
		-567	-572	5	603.5				1,620	1,420	1,520	1,520	0.47
		-572	-575	3	607.5				1,450	1,420	1,435	1,435	0.47

[1] Elevations are referenced to NGVD 29 datum
[2] Mid-point depth measured below El. 34 feet

Table 2.5S.4-28 Not Used

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	123
70 to 90	E	123
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	124;127
258 to 263	L	124 [1]
263 to 278	M	127 [1]
278 to 609	N Clay; N Sand	123; 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 Resonant Column Torsional Shear Testing Summary

Boring No.	Sample No.	Depth, Ft	Stratum	Material
B-405	UD1	11.8	A	Clay, LL = 73, PI = 52
B-306	UD3	75.0	E	Fine Sand, 8% fines
B-405	UD4	85.0	F	Clay, LL = 60, PI = 41
B-306	UD6	104.7	H	Sand (SP-SM)*
B-405	UD6	127.0	J (clay 1)	Clay, LL = 68, PI = 50
B-405	UD8	170.0	J (clay 2 and sand/silt)	Sandy Silt, 78% fines, non-plastic
B-305	UD10	195.0	J (clay 2)	Clay, LL = 70, PI = 48
B-405	UD10	224.0	K (clay)	Clay, LL = 73, PI = 51
B-305	UD13	265.5	M	Silty Sand/Sandy Silt, 54% fines
B-405	UD13	294.7	N (clay 1)	Clay, LL = 80, PI = 60
B-405	UD16	358.5	N (clay 2)	Clay, LL = 92, PI = 65
B-305	UD18	387.5	N (sand 2)	Silty Sand, 15% fines
B-405	UD19	440.5	N (clay 4)	Clay, LL = 33, PI = 22
B-305A	UD21	455.2	N (sand 4)	Fine Sand (SP-SM)*
B-405	UD24	569.2	N (clay 6)	Clay, LL = 84, PI = 62
B-305	UD25	590.5	N (clay 6)	Clay, LL = 67, PI = 48

* Gradation tests were not performed on the two samples tested at the University of Texas. The descriptions above are based on visual descriptions in the field. (The remaining 14 RCTS tests were performed by Fugro.)

Table 2.5S.4-32 Summary of Shear Modulus Degradation Curves Numerical Values Prior to RCTS

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 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

Strain (%)	Stratum (Plasticity Index in %)						
	A (35)	D (40)	F (60)	J Clay (60)	K Clay (45)	L (70)	N Clay (70)
	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.4-33 Summary of Damping Ratio Curves Numerical Values

Cohesionless Soil Strata Prior to RCTS

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.4-34A Summaryof RCTS Laboratory Test Results

Appendix A Tests Boring B-405DH Sample UD13 Sub-stratum N Clay 1	Resonant Column Stage σ_o = 87.3 psi			Torsional Shear Stage First Cycle σ_o = 87.3 psi			Torsional Shear Stage Tenth Cycle σ_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 σ'_{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.4-34A Summaryof RCTS Laboratory Test Results (Continued)

Appendix B Tests Boring B-305DH Sample UD13 Stratum M	Resonant Column Stageσ_o = 78.6 psi			Torsional Shear Stage First Cycleσ_o = 78.6 psi			Torsional Shear StageTenth Cycle σ_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 σ'_{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 σ_o = 314.3 psi			Torsional Shear Stage First Cycleσ_o = 314.3 psi			Torsional Shear Stage Tenth Cycleσ_o = 314.3 psi		
	5.60E-05	1.00	0.77	1.04E-03	1.00	0.41	1.04E-03	1.00	0.48
	1.15E-04	1.00	0.75	2.03E-03	1.00	0.59	2.05E-03	1.00	0.43
	2.27E-04	1.00	0.75	6.65E-03	0.99	0.93	6.68E-03	1.00	0.71
	4.49E-04	1.00	0.83	9.92E-03	0.97	1.00	9.93E-03	0.98	1.21
	9.22E-04	1.00	0.90	1.34E-02	0.96	1.04	1.34E-02	0.96	1.26
	3.52E-03	0.99	0.95	-	-	-	-	-	-
	6.60E-03	0.98	1.01	-	-	-	-	-	-
	1.20E-02	0.95	1.15	-	-	-	-	-	-
	2.13E-02	0.92	1.30	-	-	-	-	-	-
	3.74E-02	0.88	1.67	-	-	-7	-	-	-
	6.62E-02	0.81	2.01	-	-	-	-	-	-
	1.14E-01	0.73	2.97	-	-	-	-	-	-
	1.59E-01	0.68	3.82	-	-	-	-	-	-
	2.03E-01	0.65	4.31	-	-	-	-	-	-

Table 2.5S.4-34A Summaryof RCTS Laboratory Test Results (Continued)

Appendix C Tests Boring B-405DH Sample UD16 Sub-stratum N Clay 2	Resonant Column Stageσ_o = 106.1 psi			Torsional Shear Stage First Cycleσ_o = 106.1 psi			Torsional Shear Stage Tenth Cycleσ_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 σ'_{mean} = 106.1 psi	3.24E-04	1.00	2.45	1.11E-03	0.95	2.76	1.09E-03	0.98	1.73
	7.02E-04	1.00	2.50	2.15E-03	0.99	1.61	2.15E-03	0.99	1.36
	1.37E-03	1.00	2.61	4.25E-03	1.00	1.49	4.26E-03	1.00	1.31
	2.73E-03	1.00	2.64	9.84E-03	0.99	1.80	9.87E-03	0.99	1.61
	5.45E-03	1.00	2.74	2.00E-02	0.97	2.02	2.00E-02	0.98	1.98
	1.09E-02	0.99	2.82	4.19E-02	0.93	2.13	4.21E-02	0.93	2.10
	2.14E-02	0.99	2.88	-	-	-	-	-	-
	4.23E-02	0.97	2.98	-	-	-	-	-	-
	8.27E-02	0.93	3.09	-	-	-	-	-	-
	1.64E-01	0.84	3.34	-	-	-	-	-	-
	3.37E-01	0.71	4.34	-	-	-	-	-	-
	7.07E-01	0.55	6.68	-	-	-	-	-	-
	1.46E+00	0.40	11.88	-	-	-	-	-	-
	Resident Column Stage σ_o = 424.4 psi			**Torsional Shear Stage First Cycleσ_o = 424.4 psi**			**Torsional Shear Stage Tenth Cycleσ_o = 424.4 psi**		
	1.84E-04	1.00	2.11	1.08E-03	0.99	1.00	1.11E-03	0.96	1.08
	3.72E-04	1.00	2.1	2.16E-03	0.99	1.15	2.14E-03	1.00	0.98
	7.63E-04	1.00	2.08	4.31E-03	0.99	0.98	4.29E-03	0.99	1.30
	1.53E-03	1.00	2.15	9.67E-03	1.00	1.56	9.67E-03	1.00	1.22
	3.06E-03	1.00	2.15	1.95E-02	0.99	1.27	1.94E-02	1.00	1.26
	6.12E-03	1.00	2.17	3.11E-02	0.99	1.28	3.10E-02	0.99	1.34
	1.22E-02	1.00	2.18	-	-	-	-	-	-
	2.43E-02	0.99	2.26	-	-	-	-	-	-
	4.77E-02	0.98	2.29	-	-	-	-	-	-
	9.21E-02	0.93	2.34	-	-	-	-	-	-
	1.75E-01	0.83	2.79	-	-	-	-	-	-
	3.37E-01	0.69	4.13	-	-	-	-	-	-
	4.98E-01	0.59	5.42	-	-	-	-	-	-

Table 2.5S.4-34A Summaryof RCTS Laboratory Test Results (Continued)

Appendix D Tests Boring B-405DH Sample UD19 Sub-stratum N Clay 4	**Resonant Column Stage σ_o = 129.4 psi**			**Torsional Shear Stage First Cycle σ_o = 129.4 psi**			**Torsional Shear Stage Tenth Cycle σ_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 σ'_{mean}= 129.4 psi	1.01E-04	1.00	4.82	3.94E-04	0.98	0.91	3.97E-04	0.98	1.17
	2.02E-04	1.00	4.96	9.61E-04	1.00	1.08	9.84E-04	0.99	1.34
	3.93E-04	1.00	5.09	1.99E-03	0.97	1.20	1.95E-03	1.00	1.06
	8.26E-04	1.00	5.09	4.06E-03	0.95	1.00	3.95E-03	0.99	1.21
	1.65E-03	1.00	5.16	9.59E-03	0.88	2.07	9.56E-03	0.89	2.05
	3.32E-03	0.98	5.28	2.15E-02	0.78	3.09	2.17E-02	0.79	3.04
	6.68E-03	0.96	5.46	3.21E-02	0.72	4.02	3.26E-02	0.72	4.10
	1.37E-02	0.91	5.62	-	-	-	-	-	-
	2.75E-02	0.84	6.17	-	-	-	-	-	-
	6.54E-02	0.70	6.98	-	-	-	-	-	-
	1.73E-01	0.51	8.85	-	-	-	-	-	-
	Resonant Column Stageσ_o = 455.0 psi			**Torsional Shear Stage First Cycleσ_o = 455.0 psi**			**Torsional Shear Stage Tenth Cycleσ_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.4-34A Summaryof RCTS Laboratory Test Results (Continued)

Appendix E Tests Boring B-305DH Sample UD18 Sub-stratum N Sand 2	Resonant Column Stageσ_o = 113.9 psi			Torsional Shear Stage First Cycle σ_o = 113.9 psi			Torsional Shear Stage Tenth Cycleσ_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 σ'_{mean}= 113.9 psi	1.11E-04	1.00	0.37	7.07E-04	1.00	0.99	7.15E-04	1.00	1.04
	2.26E-04	1.00	0.37	8.75E-04	1.00	1.15	8.85E-04	1.00	1.07
	4.43E-04	1.00	0.47	1.74E-03	1.00	0.88	1.76E-03	1.00	0.63
	8.98E-04	1.00	0.47	3.59E-03	0.98	1.07	3.55E-03	1.00	1.19
	1.74E-03	0.99	0.55	7.53E-03	0.93	1.08	7.58E-03	0.93	1.08
	3.28E-03	0.98	0.63	9.71E-03	0.90	1.50	9.76E-03	0.91	1.46
	5.92E-03	0.97	0.78	-	-	-	-	-	-
	1.04E-02	0.94	0.86	-	-	-	-	-	-
	1.81E-02	0.92	0.99	-	-	-	-	-	-
	3.09E-02	0.86	1.25	-	-	-	-	-	-
	5.21E-02	0.80	1.85	-	-	-	-	-	-
	9.01E-02	0.70	2.90	-	-	-	-	-	-
	1.58E-01	0.61	4.28	-	-	-	-	-	-
	2.82E-01	0.50	5.30	-	-	-	-	-	-
	Resonant Column Stageσ_o = 455.0 psi			Torsional Shear Stage First Cycleσ_o = 455.0 psi			Torsional Shear Stage Tenth Cycleσ_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.4-34A Summary of RCTS Laboratory Test Results

Appendix F Tests Boring B-305DH Sample UD25 Sub-Stratum N Clay 6	Resonant Column Stage σ_o = 172.7 psi			Torsional Shear Stage First Cycle σ_o = 172.7 psi			Torsional Shear Stage Tenth Cycle σ_o = 172.7 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 = 590.5 feet (180.0 meters) Total Unit Weight = 128.8 pcf Moisture Content = 20.6% Estimated In-Situ K_0 = 0.5 Estimated σ'_{mean}= 172.7 psi	7.10E-05	1.00	5.04	9.58E-04	1.00	1.55	9.57E-04	1.00	1.36
	1.37E-04	1.00	4.99	1.74E-03	1.00	1.50	1.76E-03	1.00	1.46
	2.67E-04	1.00	5.04	3.43E-03	1.00	1.83	3.52E-03	1.00	1.85
	5.54E-04	1.00	5.05	9.94E-03	1.00	2.36	1.00E-02	0.98	2.17
	1.08E-03	1.00	5.12	2.08E-02	0.96	2.94	2.09E-02	0.94	3.17
	2.17E-03	1.00	5.15	4.47E-02	0.85	3.17	4.52E-02	0.84	3.30
	4.34E-03	1.00	5.23						
	8.66E-03	0.99	5.46						
	1.82E-02	0.98	5.61						
	3.89E-02	0.93	6.12						
	8.56E-02	0.83	6.69						
	2.03E-01	0.66	7.60						
	3.49E-01	0.54	10.22						
	Resonant Column Stage σ_o = 455.0 psi			Torsional Shear Stage First Cycle σ_o = 455.0 psi			Torsional Shear Stage Tenth Cycle σ_o = 455.0 psi		
	2.60E-05	1.00	4.72	1.04E-03	1.00	1.40	1.03E-03	1.00	1.59
	5.20E-05	1.00	4.69	2.01E-03	1.00	1.80	2.04E-03	1.00	1.80
	1.03E-04	1.00	4.72	4.06E-03	1.00	1.47	4.08E-03	1.00	1.55
	2.02E-04	1.00	4.72	9.41E-03	0.99	1.63	9.39E-03	1.00	1.65
	3.97E-04	1.00	4.72	1.93E-02	0.97	1.79	1.93E-02	0.98	1.78
	9.85E-04	1.00	4.79						
	4.29E-03	1.00	4.90						
	9.75E-03	0.99	5.13						
	2.09E-02	0.97	5.38						
	4.47E-02	0.91	5.63						
	1.01E-01	0.80	6.27						
	2.39E-01	0.62	7.71						

Table 2.5S.4-34A Summary of RCTS Laboratory Test Results (Continued)

Appendix G Tests Boring B-405DH Sample UD4 Stratum F	Resonant Column Stage σ_o = 26.4 psi			Torsional Shear Stage First Cycle σ_o = 26.4 psi			Torsional Shear Stage Tenth Cycle σ_o = 26.4 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 = 85.0 feet (25.9 meters) Total Unit Weight = 131.0 pcf Moisture Content = 22.6% Estimated In-Situ K_0 = 0.5 Estimated σ'_{mean}= 26.4 psi	8.90E-05	1.00	2.37	2.22E-03	1.00	1.04	2.27E-03	1.00	0.82
	1.58E-04	1.00	2.39	4.21E-03	1.00	1.19	4.39E-03	1.00	1.09
	3.21E-04	1.00	2.38	1.06E-02	1.00	1.32	1.06E-02	1.00	1.62
	6.42E-04	1.00	2.44	2.20E-02	0.97	1.81	2.22E-02	0.99	1.72
	1.25E-03	1.00	2.43	4.60E-02	0.87	2.62	4.66E-02	0.88	2.65
	2.55E-03	0.99	2.45						
	5.06E-03	0.99	2.41						
	1.01E-02	0.98	2.45						
	2.00E-02	0.97	2.58						
	3.93E-02	0.93	2.75						
	7.61E-02	0.87	3.55						
	1.44E-01	0.76	4.44						
	2.81E-01	0.58	6.95						
	Resonant Column Stage σ_o = 105.6 psi			Torsional Shear Stage First Cycle σ_o = 105.6 psi			Torsional Shear Stage Tenth Cycle σ_o = 105.6 psi		
	1.63E-04	1.00	1.87	2.04E-03	1.00	0.89	2.05E-03	1.00	0.92
	2.53E-04	1.00	1.89	4.06E-03	1.00	0.92	4.09E-03	1.00	1.02
	3.85E-04	1.00	1.90	1.04E-02	0.98	1.17	1.03E-02	0.99	1.26
	7.52E-04	1.00	1.95	2.14E-02	0.95	1.47	2.15E-02	0.95	1.38
	1.40E-03	1.00	1.93	4.83E-02	0.84	2.47	4.89E-02	0.84	2.52
	2.72E-03	1.00	2.01						
	5.27E-03	1.00	2.03						
	1.08E-02	0.99	2.04						
	2.15E-02	0.98	2.11						
	4.26E-02	0.93	2.26						
	8.56E-02	0.85	2.84						
	1.74E-01	0.73	3.43						
	3.89E-01	0.56	5.37						
	5.60E-01	0.49	6.48						

Table 2.5S.4-34A Summary of RCTS Laboratory Test Results (Continued)

Appendix H Tests Boring B-405DH Sample UD8 Sub-Stratum J Clay 2 and sand/silt	Resonant Column Stage σ_o = 51.0 psi			Torsional Shear Stage First Cycle σ_o = 51.0 psi			Torsional Shear Stage Tenth Cycle σ_o = 51.0 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 = 170.0 feet (51.8 meters) Total Unit Weight = 124.4 pcf Moisture Content = 22.9% Estimated In-Situ K_0 = 0.5 Estimated σ'$_{mean}$= 51.0 psi	4.59E-04	1.00	0.73	7.04E-04	1.00	0.55	7.10E-04	1.00	0.63
	9.74E-04	1.00	0.73	1.06E-03	1.00	0.60	1.07E-03	1.00	0.69
	1.76E-03	1.00	0.73	2.14E-03	0.99	0.85	2.15E-03	0.99	0.68
	3.34E-03	0.98	0.77	4.39E-03	0.96	1.33	4.36E-03	0.98	1.13
	5.97E-03	0.97	0.76	9.79E-03	0.86	1.76	9.80E-03	0.87	2.02
	1.04E-02	0.94	0.92	2.19E-02	0.77	3.01	2.20E-02	0.78	2.87
	1.75E-02	0.91	1.07	4.89E-02	0.66	4.65	4.87E-02	0.67	4.47
	2.84E-02	0.86	1.32						
	5.38E-02	0.78	1.95						
	9.68E-02	0.70	3.33						
	1.69E-01	0.62	4.68						
	3.03E-01	0.53	7.24						
	Resonant Column Stage σ_o = 204.0 psi			Torsional Shear Stage First Cycle σ_o = 204.0 psi			Torsional Shear Stage Tenth Cycle σ_o = 204.0 psi		
	1.76E-04	1.00	0.60	5.56E-04	1.00	0.55	5.63E-04	1.00	0.59
	3.46E-04	1.00	0.63	1.03E-03	1.00	0.41	1.02E-03	1.00	0.58
	7.12E-04	1.00	0.64	2.06E-03	1.00	0.53	2.07E-03	1.00	0.65
	1.39E-03	1.00	0.64	4.19E-03	0.98	0.59	4.22E-03	0.97	0.47
	2.65E-03	1.00	0.64	1.00E-02	0.95	1.07	1.01E-02	0.95	1.07
	4.87E-03	0.98	0.63	1.79E-02	0.92	1.51	1.79E-02	0.92	1.53
	8.71E-03	0.97	0.71						
	1.59E-02	0.93	0.82						
	2.69E-02	0.90	0.98						
	4.65E-02	0.84	1.23						
	7.99E-02	0.78	1.74						
	1.38E-01	0.69	2.48						
	2.28E-01	0.61	3.83						

Table 2.5S.4-34A Summary of RCTS Laboratory Test Results (Continued)

Appendix I Tests Boring B-305DH Sample UD10 Sub-Stratum J Clay 2	Resonant Column Stage σ_o = 58.2 psi			Torsional Shear Stage First Cycle σ_o = 58.2 psi			Torsional Shear Stage Tenth Cycle σ_o = 58.2 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 = 195.0 feet (59.4 meters) Total Unit Weight = 120.0 pcf Moisture Content = 27.9% Estimated In-Situ K_0 = 0.5 Estimated σ'_{mean} = 58.2 psi	3.50E-04	1.00	1.25	1.03E-03	1.00	0.76	9.80E-04	1.00	0.84
	6.71E-04	1.00	1.21	1.96E-03	1.00	0.39	1.95E-03	1.00	0.78
	1.28E-03	1.00	1.22	3.89E-03	1.00	0.72	3.87E-03	1.00	0.84
	2.52E-03	1.00	1.12	9.56E-03	1.00	0.84	9.50E-03	1.00	0.92
	5.07E-03	1.00	1.13	1.98E-02	1.00	0.92	1.97E-02	0.98	0.95
	1.02E-02	1.00	1.18						
	1.94E-02	1.00	1.26						
	5.06E-02	0.96	1.57						
	9.69E-02	0.90	2.04						
	1.63E-01	0.79	3.25						
	2.76E-01	0.70	5.45						
	Resonant Column Stage σ_o = 233.0 psi			Torsional Shear Stage First Cycle σ_o = 233.0 psi			Torsional Shear Stage Tenth Cycle σ_o = 233.0 psi		
	1.26E-04	1.00	1.05	1.02E-03	1.00	0.76	1.02E-03	1.00	0.79
	2.33E-04	1.00	1.03	2.03E-03	1.00	0.72	2.00E-03	1.00	0.68
	4.76E-04	1.00	1.05	4.02E-03	1.00	0.71	4.01E-03	1.00	0.77
	9.48E-04	1.00	1.09	1.00E-02	1.00	0.88	1.00E-02	1.00	0.74
	1.88E-03	1.00	1.07	2.03E-02	1.00	0.84	2.03E-02	0.99	0.88
	3.77E-03	1.00	1.03						
	7.41E-03	1.00	1.08						
	1.49E-02	1.00	1.05						
	3.05E-02	0.99	1.09						
	5.90E-02	0.95	1.28						
	1.07E-01	0.89	1.61						
	1.85E-01	0.79	2.49						
	3.03E-01	0.69	3.97						
	7.15E-01	0.47	7.65						

Table 2.5S.4-34A Summary of RCTS Laboratory Test Results (Continued)

Appendix J Tests Boring B-405DH Sample UD1 Stratum A	Resonant Column Stage σ_o = 5.2 psi			Torsional Shear Stage First Cycle σ_o = 5.2 psi			Torsional Shear Stage Tenth Cycle σ_o = 5.2 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 = 11.8 feet (3.4 meters) Total Unit Weight = 117.9 pcf Moisture Content = 28.2% Estimated In-Situ K_0 = 0.5 Estimated σ'_{mean} = 5.2 psi	4.88E-04	1.00	2.31	9.24E-04	1.00	1.42	9.13E-04	1.00	1.44
	9.37E-04	1.00	2.32	3.75E-03	1.00	1.41	3.72E-03	1.00	1.68
	1.69E-03	1.00	2.33	9.75E-03	1.00	1.50	9.73E-03	0.99	1.68
	3.36E-03	0.99	2.34						
	6.76E-03	0.99	2.33						
	1.33E-02	0.99	2.39						
	2.66E-02	0.98	2.50						
	5.24E-02	0.95	2.75						
	1.04E-01	0.78	3.09						
	2.36E-01	0.62	4.29						
	5.93E-01	0.45	5.57						
	Resonant Column Stage σ_o = 20.9 psi			Torsional Shear Stage First Cycle σ_o = 20.9 psi			Torsional Shear Stage Tenth Cycle σ_o = 20.9 psi		
	3.21E-04	1.00	2.10	4.42E-04	1.00	0.93	4.26E-04	1.00	0.91
	6.47E-04	1.00	2.10	1.01E-03	1.00	1.03	1.00E-03	1.00	0.97
	1.28E-03	1.00	2.11	2.03E-03	1.00	1.13	1.97E-03	1.00	1.18
	2.55E-03	1.00	2.16	1.00E-02	1.00	1.24	1.00E-02	1.00	1.30
	5.10E-03	1.00	2.20						
	1.03E-02	1.00	2.23						
	2.03E-02	1.00	2.26						
	4.06E-02	0.97	2.28						
	7.87E-02	0.92	2.51						
	1.56E-01	0.83	2.88						
	3.22E-01	0.68	3.49						

Table 2.5S.4-34A Summary of RCTS Laboratory Test Results (Continued)

Appendix K Tests Boring B-405DH Sample UD24 Sub-Stratum N Clay 6	Resonant Column Stage σ_o = 167.0 psi			Torsional Shear Stage First Cycle σ_o = 167.0 psi			Torsional Shear Stage Tenth Cycle σ_o = 167.0 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 = 569.2 feet (173.5 meters) Total Unit Weight = 122.1 pcf Moisture Content = 27.2% Estimated In-Situ K_0 = 0.5 Estimated σ'_{mean} = 167.0 psi	1.71E-04	1.00	2.43	9.99E-04	1.00	1.14	1.02E-03	1.00	1.08
	3.22E-04	1.00	2.44	2.00E-03	1.00	1.06	2.00E-03	1.00	1.14
	6.16E-04	1.00	2.52	4.02E-03	1.00	1.14	4.00E-03	1.00	1.19
	1.26E-03	1.00	2.58	9.90E-03	0.98	1.37	9.91E-03	0.98	1.41
	2.47E-03	1.00	2.56	2.06E-02	0.94	1.60	2.07E-02	0.94	1.65
	4.75E-03	1.00	2.64	3.83E-02	0.87	2.38	3.82E-02	0.88	2.34
	8.91E-03	1.00	2.75						
	1.75E-02	0.98	2.90						
	3.43E-02	0.95	3.02						
	6.66E-02	0.89	3.38						
	1.45E-01	0.78	4.05						
	3.05E-01	0.67	5.10						

Table 2.5S.4-34A Summary of RCTS Laboratory Test Results (Continued)

Appendix L Tests Boring B-405DH Sample UD10 Sub-Stratum K Clay	**Resonant Column Stage σ_o = 67.0 psi**			**Torsional Shear Stage First Cycle σ_o = 67.0 psi**			**Torsional Shear Stage Tenth Cycle σ_o = 67.0 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 = 224.0 feet (68.3 meters) Total Unit Weight = 114.9 pcf Moisture Content = 34.5% Estimated In-Situ K_0 = 0.5 Estimated σ'$_{mean}$ = 67.0 psi	3.14E-04	1.00	1.54	7.42E-04	1.00	0.97	7.49E-04	1.00	1.07
	6.75E-04	1.00	1.54	9.80E-04	1.00	1.05	9.75E-04	1.00	1.21
	1.32E-03	1.00	1.54	1.96E-03	1.00	1.01	1.94E-03	1.00	1.06
	2.61E-03	1.00	1.62	3.90E-03	1.00	1.12	3.91E-03	1.00	1.11
	5.22E-03	1.00	1.70	1.02E-02	1.00	1.52	1.02E-02	1.00	1.42
	1.04E-02	1.00	1.81						
	2.08E-02	0.99	1.85						
	4.00E-02	0.98	2.03						
	7.17E-02	0.94	2.27						
	9.07E-02	0.91	2.44						
	1.85E-01	0.81	3.47						
	Resonant Column Stage σ_o = 267.0 psi			**Torsional Shear Stage First Cycle σ_o = 267.0 psi**			**Torsional Shear Stage Tenth Cycle σ_o = 267.0 psi**		
	1.82E-04	1.00	1.33	1.01E-03	1.00	1.38	9.78E-04	1.00	1.39
	3.27E-04	1.00	1.33	2.02E-03	1.00	1.19	2.00E-03	1.00	1.43
	6.00E-04	1.00	1.33	4.00E-03	1.00	1.34	3.99E-03	1.00	1.55
	1.09E-03	1.00	1.33	1.02E-02	0.98	1.29	1.02E-02	0.97	1.34
	2.20E-03	1.00	1.37						
	4.34E-03	1.00	1.40						
	8.67E-03	1.00	1.45						
	1.72E-02	1.00	1.49						
	3.35E-02	0.98	1.59						
	6.13E-02	0.95	1.68						
	1.06E-01	0.89	2.07						
	1.79E-01	0.78	2.63						
	3.17E-01	0.66	3.75						
	4.86E-01	0.55	5.69						

Table 2.5S.4-34A Summary of RCTS Laboratory Test Results (Continued)

Appendix M Tests Boring B-405DH Sample UD6 Sub-Stratum J Clay 1	Resonant Column Stage σ_o = 39.0 psi			Torsional Shear Stage First Cycle σ_o = 39.0 psi			Torsional Shear Stage Tenth Cycle σ_o = 39.0 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 = 127.0 feet (38.7 meters)	2.30E-04	1.00	1.62	1.03E-03	1.00	1.08	1.03E-03	1.00	1.16
Total Unit Weight = 121.8 pcf Moisture Content = 27.2% Estimated In-Situ K_0 = 0.5 Estimated σ'_{mean} = 39.0 psi	4.50E-04	1.00	1.67	2.03E-03	1.00	1.16	2.03E-03	1.00	1.12
	9.69E-04	1.00	1.67	4.07E-03	1.00	1.26	4.08E-03	1.00	1.27
	1.95E-03	1.00	1.71	1.05E-02	0.98	1.26	1.05E-02	0.98	1.36
	3.92E-03	1.00	1.69						
	8.01E-03	1.00	1.78						
	1.62E-02	0.98	1.86						
	3.15E-02	0.97	2.03						
	5.99E-02	0.93	2.36						
	1.11E-01	0.85	2.67						
	2.11E-01	0.73	3.53						
	4.59E-01	0.55	5.85						
	Resonant Column Stage σ_o = 267.0 psi			Torsional Shear Stage First Cycle σ_o = 267.0 psi			Torsional Shear Stage Tenth Cycle σ_o = 267.0 psi		
	9.20E-05	1.00	1.47	1.01E-03	1.00	0.67	1.01E-03	1.00	0.73
	1.79E-04	1.00	1.47	2.03E-03	1.00	0.73	2.02E-03	1.00	0.67
	3.69E-04	1.00	1.47	4.02E-03	1.00	0.72	4.01E-03	1.00	0.70
	7.64E-04	1.00	1.47	1.01E-02	1.00	0.71	1.01E-02	1.00	0.68
	1.52E-03	1.00	1.46	2.05E-02	0.98	0.84	2.05E-02	0.98	0.84
	3.06E-03	1.00	1.50	3.53E-02	0.96	1.20	3.52E-02	0.95	1.21
	6.11E-03	1.00	1.52						
	1.23E-02	0.99	1.54						
	2.39E-02	0.98	1.63						
	4.53E-02	0.95	1.71						
	8.21E-02	0.89	1.97						
	1.51E-01	0.79	2.50						
	2.69E-01	0.69	3.49						
	5.26E-01	0.53	6.35						

Table 2.5S.4-34A Summary of RCTS Laboratory Test Results (Continued)

Appendix N Tests Boring B-306 Sample UD3 Stratum E	Resonant Column Stage σ_o = 24.0 psi			Torsional Shear Stage First Cycle σ_o = 24.0 psi			Torsional Shear Stage Tenth Cycle σ_o = 24.0 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 = 75.0 feet (22.9 meters)	5.91E-04	1.00	1.08	3.61E-04	1.00	0.38	3.70E-04	1.00	0.38
Total Unit Weight = 132.6 pcf Moisture Content = 24.7% Estimated In-Situ K_0 = 0.5 Estimated σ'_{mean} = 24.0 psi	1.21E-03	1.00	1.08	7.00E-04	1.00	0.39	7.05E-04	1.00	0.41
	2.30E-03	1.00	1.06	1.02E-03	1.00	0.42	1.01E-03	1.00	0.47
	4.35E-03	0.99	1.06	2.04E-03	1.00	0.57	2.05E-03	1.00	0.63
	7.86E-03	0.97	1.14	4.12E-03	0.99	0.94	4.14E-03	0.99	0.69
	1.55E-02	0.92	1.28	1.01E-02	0.92	1.40	1.01E-02	0.93	1.24
	2.56E-02	0.90	1.46						
	4.04E-02	0.85	1.82						
	5.90E-02	0.80	2.57						
	9.70E-02	0.73	3.66						
	1.69E-01	0.65	5.96						
	Resonant Column Stage σ_o = 94.0 psi			Torsional Shear Stage First Cycle σ_o = 94.0 psi			Torsional Shear Stage Tenth Cycle σ_o = 94.0 psi		
	1.85E-04	1.00	0.80	3.30E-04	1.00	0.64	3.25E-04	1.00	0.46
	3.63E-04	1.00	0.84	6.37E-04	1.00	0.49	6.37E-04	1.00	0.49
	7.45E-04	1.00	0.84	9.82E-04	1.00	0.50	9.87E-04	1.00	0.49
	1.47E-03	1.00	0.80	2.01E-03	1.00	0.73	1.99E-03	1.00	0.68
	2.86E-03	0.99	0.80	4.05E-03	0.99	0.86	4.03E-03	0.99	0.90
	5.33E-03	0.98	0.87	1.01E-02	0.95	1.08	1.01E-02	0.95	0.96
	9.79E-03	0.96	0.92						
	1.70E-02	0.94	1.09						
	2.94E-02	0.90	1.32						
	4.97E-02	0.86	1.68						
	8.53E-02	0.79	2.24						
	1.41E-01	0.71	3.15						
	1.81E-01	0.67	3.96						
	2.20E-01	0.64	4.94						

Table 2.5S.4-34A Summary of RCTS Laboratory Test Results (Continued)

Appendix O Tests Boring B-305A Sample UD21 Sub-Stratum N Sand 4	Resonant Column Stage σ_o = 133.8 psi			Torsional Shear Stage First Cycle σ_o = 133.8 psi			Torsional Shear Stage Tenth Cycle σ_o = 133.8 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 = 455.2 feet (138.7 meters) Total Unit Weight = 129.2 pcf Moisture Content = 18.8% Estimated In-Situ K_0 = 0.5 Estimated σ'_{mean} = 133.8 psi	1.38E-04	1.00	0.18	4.82E-04	1.00	0.12	4.81E-04	1.00	0.12
	2.68E-04	1.00	0.16	9.65E-04	1.00	0.09	9.64E-04	1.00	0.10
	5.26E-04	1.00	0.16	1.46E-03	0.99	0.11	1.46E-03	0.99	0.14
	1.01E-03	1.00	0.16						
	2.02E-03	0.99	0.19						
	3.81E-03	0.99	0.21						
	6.50E-03	0.98	0.26						
	1.13E-02	0.97	0.36						
	1.86E-02	0.95	0.49						
	2.91E-02	0.91	0.74						
	4.58E-02	0.86	1.15						
	6.71E-02	0.82	1.70						
	1.00E-01	0.77	2.43						
	1.34E-01	0.73	3.40						
	Resonant Column Stage σ_o = 260.0 psi			Torsional Shear Stage First Cycle σ_o = 260.0 psi			Torsional Shear Stage Tenth Cycle σ_o = 260.0 psi		
	9.34E-05	1.00	0.17	5.44E-04	1.01	0.13	5.40E-04	1.00	0.11
	1.90E-04	1.00	0.16	1.09E-03	1.00	0.09	1.09E-03	1.00	0.09
	3.78E-04	1.00	0.16	2.19E-03	1.00	0.10	2.19E-03	1.00	0.10
	7.38E-04	1.00	0.16	3.30E-03	1.00	0.16	3.30E-03	1.00	0.15
	1.45E-03	1.00	0.21	4.75E-03	1.00	0.19	4.72E-03	1.00	0.19
	2.78E-03	0.99	0.09						
	4.87E-03	0.99	0.24						
	9.46E-03	0.98	0.25						
	1.65E-02	0.97	0.47						
	2.61E-02	0.94	0.53						
	4.25E-02	0.92	0.71						
	6.46E-02	0.88	1.00						
	9.70E-02	0.81	1.59						
	1.31E-01	0.76	2.18						

Table 2.5S.4-34A Summary of RCTS Laboratory Test Results (Continued)

Appendix P Tests Boring B-306 Sample UD6 Stratum H	**Resonant Column Stage σ_o = 32.2 psi**			**Torsional Shear Stage First Cycle σ_o = 32.2 psi**			**Torsional Shear Stage Tenth Cycle σ_o = 32.2 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 = 104.7 feet (31.9 meters) Total Unit Weight = 120.6 pcf Moisture Content = 19.3% Estimated In-Situ K_0 = 0.5 Estimated σ'_{mean} = 32.2 psi	2.49E-04	1.01	0.35	2.67E-04	1.00	0.26	2.71E-04	1.00	0.18
	4.89E-04	1.00	0.29	5.41E-04	1.00	0.20	5.39E-04	0.99	0.21
	9.37E-04	0.99	0.32	1.09E-03	0.99	0.24	1.09E-03	0.99	0.21
	1.76E-03	0.99	0.28	2.04E-03	0.98	0.47	2.06E-03	0.97	0.49
	3.23E-03	0.97	0.37	4.20E-03	0.97	0.77	4.21E-03	0.96	0.62
	5.93E-03	0.96	0.51	1.04E-02	0.92	1.63	1.05E-02	0.91	1.26
	9.74E-03	0.93	0.72	2.05E-02	0.85	3.07	2.05E-02	0.85	2.29
	1.64E-02	0.89	0.96						
	2.65E-02	0.83	1.42						
	4.07E-02	0.75	2.03						
	Resonant Column Stage σ_o = 128.0 psi			**Torsional Shear Stage First Cycle σ_o = 128.0 psi**			**Torsional Shear Stage Tenth Cycle σ_o = 128.0 psi**		
	9.41E-05	1.00	0.15	2.69E-04	1.00	0.12	2.64E-04	1.00	0.12
	1.68E-04	1.00	0.24	5.07E-04	1.00	0.20	5.13E-04	1.00	0.27
	3.21E-04	1.00	0.15	1.03E-03	0.98	0.18	1.03E-03	0.99	0.15
	6.22E-04	1.00	0.22	I.96E-03	0.98	0.22	1.96E-03	0.99	0.22
	1.17E-03	0.99	0.17	3.95E-03	0.97	0.43	3.95E-03	0.98	0.37
	2.24E-03	0.99	0.22	8.68E-03	0.94	0.78	8.67E-03	0.95	0.64
	4.23E-03	0.98	0.30						
	7.23E-03	0.96	0.36						
	1.25E-02	0.94	0.50						
	2.22E-02	0.91	0.69						
	3.54E-02	0.86	0.99						
	5.53E-02	0.80	1.40						
	7.99E-02	0.74	2.15						
	1.16E-01	0.68	3.09						

Table 2.5S.4-34B G/Gmax vs. Strain Based on RCTS Results

Strain, %	Sand at ≥ 100 ft depth (EPRI 500 ft-1000 ft)	Sand at < 100 ft depth (EPRI 250 ft - 500 ft)	Clay with PI ≥ 30 (V&D PI =100)	Clay with PI < 30 (V&D PI = 50)	Silt (EPRI PI = 50)
	G/G_{max}				
1.00E+00	0.2	0.15	0.36	0.25	0.14
0.316	0.4	0.33	0.62	0.46	0.32
1.00E-01	0.65	0.57	0.82	0.67	0.58
0.0316	0.86	0.8	0.93	0.85	0.81
1.00E-02	0.95	0.94	0.98	0.96	0.95
0.00316	1	0.99	1	1	1
1.00E-03	1	1	1	1	1
0.000316	1	1	1	1	1
1.00E-04	1	1	1	1	1

Table 2.5S.4-34C Damping Ratio vs. Strain Based on RCTS Results

Strain, %	Sand (EPRI 500 ft-1000 ft)	Clay with PI ≥ 30 (V&D, PI = 200)	Low PI Clay and Silt (Hybrid)
	Damping Ratio (%)		
1.00E+00	16.66	8.08	15.72
0.316	10.70	4.86	10.96
1.00E-01	5.64	3.09	6.61
0.0316	2.67	2.22	3.54
1.00E-02	1.30	1.65	2.03
0.00316	0.83	1.33	1.33
1.00E-03	0.67	1.09	1.09
0.000316	0.60	1.09	1.09
1.00E-04	0.60	1.09	1.09

Table 2.5S.4-35 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	0.3	0.43	Reactor Building	-50 {-60}	Stratum C (to be excavated)	√
37	5.8	0.99	Machine Shopp	To Be Determined	Stratum A (excavatedC (see note [5]))	√
B-343	11	0.99	Radwaste Building	-23 {-39}	Stratum B (to be excavated)	√
B-422C	-3.3	0.95	Turbine Building	-8 {-10}	Stratum C (to be excavated)	√
B-424	5.8	0.98	Turbine Building	-8 {-10}	Stratum C (to be excavated)	√
B-912	-3.5	1.05	N/A	N/A	Stratum B (no structure at test location)	√
B-915	4.5	0.93	Circulating Water Pipes	To Be Determined {-15 to -39} [4]	Stratum B (to be excavated)	√
T3-5	12.1	1.08	RSW Tunnel	-8 {-10}	Stratum B (to be excavated)	√
T3-5	9.6	1.03	RSW Tunnel	-8 {-10}	Stratum B (to be excavated)	√
T3-7	-190.6	1.04	RSW Tunnel	-8 {-10}	Stratum K Sand/Silt (to remain)	
U3-5	-193.5	1.10 [6]	UHS Basin	4 {2}	Stratum K Sand/Silt (to remain)	

[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. No FOS is calculated for clay soils, as they are unlikely to liquefy according to the Chinese Method.

[4] Foundation El. of the Circulating Water Pipes is to be determined. Excavation plans indicate over-excavation to the approximate elevation indicated in "{ }" symbols.

[5] Not a safety-related structure and therefore does not affect site safety.

[6] 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; CPT Method

CPT (Number of Test Points)	Test El. [1,2] (feet)	FOS [2]	Structure	Foundation El. [3] (feet)	Stratum (Disposition)	[4]
C-301 (8)	22.2 18.3	0.80 1.04	N/A	N/A	Stratum A (no structure at test location)	√
C-301 (2)	11.9 11.4	0.91 1.04	N/A	N/A	Stratum A/B (no structure at test location)	√
C-301 (2)	-18.6 -19.1	0.82 0.89	N/A	N/A	Stratum D (no structure at test location)	√
C-302 (2)	-3.0 -3.5	1.05 1.06	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	0.83 0.89	N/A	N/A	Stratum D (no structure at test location)	√
C-304 (1)	19.3	0.98	N/A	N/A	Stratum A (no structure at test location)	√
C-305S (3)	14.4 12.5	0.95 1.04	Radwaste Building	-19 {-39}	Stratum B (to be excavated)	√
C-306S (5)	22.6	0.78	Turbine Building	-8 {-39}	Stratum A (to be excavated)	√
C-306S (4)	14.2 12.2	0.93 1.08	Turbine Building	-8 {-39}	Stratum B (to be excavated)	√
C-306S (1)	-11.9	0.87	Turbine Building	-8 {-39}	Stratum D (to be excavated)	√
C-306S (5)	-17.3 -19.8	0.84 0.93	Turbine Building	-8 {-39}	Stratum D (to be excavated)	√
C-306S (1)	-23.2	0.97	Turbine Building	-8 {-39}	Stratum D (to be excavated)	√
C-306S (1)	-32.1	0.92	Turbine Building	-8 {-39}	Stratum D (to be excavated)	√
C-306S (1)	-34.5	1.09	Turbine Building	-8 {-39}	Stratum E (to be excavated)	√
C-307S (3)	-10.6 -11.6	0.98 1.08	Turbine Building	-8 {-39}	Stratum C (to be excavated)	√
C-308 (1)	20.3	1.05	Switchyard	To Be Determined	Stratum A (see note [7])	√
C-308 (1)	-17.1	1.08	Switchyard	To Be Determined	Stratum D (see note [7])	√
C-308 (1)	-24.5	1.07	Switchyard	To Be Determined	Stratum D (see note [7])	√
C-309 (1)	17.7	1.08	Machine Shop	To Be Determined	Stratum A (see note [7])	√

Table 2.5S.4-36 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-310 (4)	-10.7 -12.7	1.01 1.06	Machine Shop	To Be Determined	Stratum D (see note [7])	√
C-401 (1)	14.6	0.98	N/A	N/A	Stratum B (no structure at test location)	√
C-401 (1)	-12.5	0.98	N/A	N/A	Stratum D (no structure at test location)	√
C-402 (1)	-11.8	1.07	N/A	N/A	Stratum D (no structure at test location)	√
C-403 (2)	-12.9 -13.4	0.99 1.04	N/A	N/A	Stratum D (no structure at test location)	√
C-404 (1)	13.4	1.06	N/A	N/A	Stratum B (no structure at test location)	√
C-405S (2)	9.5 8.9	1.02 1.08	Radwaste Building	-19 {-39}	Stratum C (to be excavated)	√
C-405S (1)	6.9	1.05	Radwaste Building	-19 {-39}	Stratum C (to be excavated)	√
C-406S (1)	-16.4	0.91	Turbine Building	-8 {-39}	Stratum D (to be excavated)	√
C-407S (1)	10.9	1.06	Turbine Building	-8 {-39}	Stratum B (to be excavated)	√
C-409 (1)	-15.2	0.97	Machine Shop	To Be Determined	Stratum D (see note [7])	√
C-410 (1)	23.7	1.07	Machine Shop	To Be Determined	Stratum A (see note [7])	√
C-410 (4)	16.4 12.4	0.97 1.09	Machine Shop	To Be Determined	Stratum B/C (see note [7])	√
C-410 (5)	-10.2 -15.6	0.90 0.98	Machine Shop	To Be Determined	Stratum D (see note [7])	√
C-410 (1)	-50.6	1.02	Machine Shop	To Be Determined	Stratum F (see note [7])	√
C-411 (1)	21.5	1.10 [5]	N/A	N/A	Stratum A (no structure at test location)	√
C-411 (1)	-18.4	0.89	N/A	N/A	Stratum D (no structure at test location)	√
C-904 (17)	24.0 16.1	0.13 0.93	N/A	N/A	Stratum A (no structure at test location)	√
C-904 (2)	8.2 7.7	0.97 1.04	N/A	N/A	Stratum B (no structure at test location)	√
C-904 (2)	-3.1 -3.6	1.07 1.09	N/A	N/A	Stratum C (no structure at test location)	√

Table 2.5S.4-36 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-904 (1)	-20.8	0.89	N/A	N/A	Stratum D (no structure at test location)	√
C-907 (2)	10.1 9.6	0.86 0.95	N/A	N/A	Stratum B (no structure at test location)	√
C-907 (6)	-9.2 -13.6	0.72 1.09	N/A	N/A	Stratum C (no structure at test location)	√
C-908 (1)	-14.1	1.01	Circulating Water Pipes	To Be Determined {-15 to -39} [6]	Stratum D (to be excavated)	√
C-916 (1)	11.0	1.09	Control Building	-42 {-44}	Stratum B (to be excavated)	√
C-917 (8)	10.3 6.3	0.88 1.08	Circulating Water Pipes	To Be Determined {-15 to -39} [6]	Stratum B (to be excavated)	√
C-917 (2)	-10.4 -11.9	0.96 1.05	Circulating Water Pipes	To Be Determined {-15 to -39} [6]	Stratum D (to be excavated)	√
C-918 (13)	25.2 18.3	0.37 1.10 [5]	Circulating Water Pipes	To Be Determined {-15 to -39} [6]	Stratum A (to be excavated)	√
C-918 (1)	-12.7	1.08	Circulating Water Pipes	To Be Determined {-15 to -39} [6]	Stratum C (to be excavated)	√
C-940 (1)	-21.2	0.82	N/A	N/A	Stratum D (no structure at test location)	√
C-941 (1)	10.8	1.05	N/A	N/A	Stratum B (no structure at test location)	√
C-945 (1)	-18.4	0.90	N/A	N/A	Stratum D (no structure at test location)	√
C-946 (7)	20.5 18.6	0.97 1.09	N/A	N/A	Stratum A (no structure at test location)	√
C-948 (26)	25.2 15.7	0.84 1.10 [5]	N/A	N/A	Stratum A (no structure at test location)	√
C-948 (1)	8.2	1.09	N/A	N/A	Stratum B (no structure at test location)	√
C-948a (1)	10.4	1.07	N/A	N/A	Stratum B (no structure at test location)	√
C-949 (2)	23.4 23.1	1.06 1.09	N/A	N/A	Stratum A (no structure at test location)	√

Table 2.5S.4-36 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-949 (5)	-20.9 -22.2	0.70 0.77	N/A	N/A	Stratum C (no structure at test location)	√

NOTES:

[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 to be excavated or areas without structures. No FOS is calculated for clay soils, as they are unlikely to liquefy according to the Chinese Method.

[5] FOS value slightly < 1.10, but which rounds up to 1.10 at two decimal places.

[6] Foundation El. of the Circulating Water Pipes is to be determined. Excavation plans indicate over-excavation to the approximate elevation indicated in "{ }" symbols.

[7] Not a safety-related structure and therefore does not affect site safety.

Table 2.5S.4-37 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) [5]	[4]
B-302DH (1)	7.0	0.85	Reactor Building	20 {-60}	Stratum B (to be excavated)	√
B-302DH (2)	-1.2 -2.8	0.73 0.84	Reactor Building	-50 {-60}	Stratum C (to be excavated)	√
B-308DH (2)	23.2 21.6	0.61 0.93	Reactor Building	-50 {-60}	Stratum A (to be excavated)	√
B-308DH (2)	15.0 16.7	0.41 0.79	Reactor Building	-50 {-60}	Stratum A/B (to be excavated)	√
B-319DH (4)	18.6 13.6	0.65 0.91	Turbine Building	-8 {-39}	Stratum A/B (to be excavated)	√
B-319DH (1)	7.1	1.02	Turbine Building	-8 {-39}	Stratum C (to be excavated)	√
B-419DH (1)	5.1	0.67	Turbine Building	-8 {-39}	Stratum B (to be excavated)	√
B-428DH (1)	9.6	0.97	Turbine Building	-8 {-39}	Stratum A (to be excavated)	√
B-428DH (3)	-1.9 -5.2	0.85 1.04	Turbine Building	-8 {-39}	Stratum C (to be excavated)	√
B-428DH (2)	-10.1 -11.8	0.79 0.92	Turbine Building	-8 {-39}	Stratum C (to be excavated)	√

[1] Elevations are referenced to NGVD 29 datum. Vs Method not applicable for depths greater than approximately 40 feet (Test El. below approximately -10 to -15).

[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 excavatedor areas without structures. No FOS is calculated for clay soils, as they are unlikely to liquefy according to the Chinese Method.

Table 2.5S.4-37A Subsurface Conditions for Settlement Analysis for the STP 3&4 Reactor Buildings (Page 1 of 3)

Soil Properties									
	Average Subsurface Conditions				Properties for Settlement Calculation				
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3 Remainder of Building Area F(Clay) Present	0	18.0	-32.3	-50.3	Mat Foundation	Mat Foundation	-	-	-
	18.0	28.0	-50.3	-60.3	Concrete Fill	Concrete Fill	-	-	-
	28.0	44.7	-60.3	-77.0	F(Clay)	F(Clay)	125	0.15	1,970
	44.7	54.7	-77.0	-87.0	H(Sand)	H(Sand)	125	0.30	3,240
	54.7	149.7	-87.0	-182.0	J(Clay)	J(Clay)	125	0.15	3,175
	149.7	165.7	-182.0	-198.0	K(Clay)	K(Clay)	124	0.15	3,335
	165.7	195.7	-198.0	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	195.7	200.7	-228.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	200.7	215.7	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	215.7	277.9	-248.0	-310.2	N(Clay)	N(Clay)	123	0.15	6,020
	277.9	293.9	-310.2	-326.2	N(Sand)	N(Sand)	128	0.30	11,645
	293.9	298.9	-326.2	-331.2	N(Clay)	N(Clay)	123	0.15	6,020
	298.9	337.9	-331.2	-370.2	N(Sand)	N(Sand)	128	0.30	11,645
	337.9	344.7	-370.2	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	344.7	361.7	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	361.7	386.7	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	386.7	402.7	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,625
	402.7	537.2	-435.0	-569.5	N(Clay)	N(Clay)	123	0.15	6,020
	537.2	613.7	-569.5	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	613.7	713.7	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	713.7	813.7	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	813.7	933.7	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	933.7	1233.7	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1233.7	1863.7	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1863.7	2433.7	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-37A Subsurface Conditions for Settlement Analysis for the STP 3&4 Reactor Buildings (Page 2 of 3)

Soil Properties									
	Average Subsurface Conditions				Properties for Settlement Calculation				
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3 North Edge F(Clay) Absent	0	18.0	-32.3	-50.3	Mat Foundation	Mat Foundation	-	-	-
	18.0	28.0	-50.3	-60.3	Concrete Fill	Concrete Fill	-	-	-
	28.0	54.7	-60.3	-87.0	H(Sand)	H(Sand)	125	0.30	3,240
	54.7	149.7	-87.0	-182.0	J(Clay)	J(Clay)	125	0.15	3,175
	149.7	165.7	-182.0	-198.0	K(Clay)	K(Clay)	124	0.15	3,335
	165.7	195.7	-198.0	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	195.7	200.7	-218.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	200.7	215.7	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	215.7	277.9	-248.0	-310.2	N(Clay)	N(Clay)	123	0.15	6,020
	277.9	293.9	-310.2	-326.2	N(Sand)	N(Sand)	128	0.30	11,645
	293.9	298.9	-326.2	-331.2	N(Clay)	N(Clay)	123	0.15	6,020
	298.9	337.9	-331.2	-370.2	N(Sand)	N(Sand)	128	0.30	11,645
	337.9	344.7	-370.2	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	344.7	361.7	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	361.7	386.7	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	386.7	402.7	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,645
	402.7	537.2	-493.0	-569.5	N(Clay)	N(Clay)	123	0.15	6,020
	537.2	613.7	-569.5	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	613.7	713.7	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	713.7	813.7	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	813.7	933.7	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	933.7	1233.7	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1233.7	1863.7	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1863.7	2433.7	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-37A Subsurface Conditions for Settlement Analysis for the STP 3&4 Reactor Buildings (Page 3 of 3)

Soil Properties									
STP	Average Subsurface Conditions				Properties for Settlement Calculation				
	Depth (feet)		Elevation [1] (feet)						
	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
4	0	18.0	-32.3	-50.3	Mat Foundation	-	-	-	-
	18.0	28.0	-50.3	-60.3	Concrete Fill	-	-	-	-
	28.0	44.7	-60.3	-77.0	F(Clay)	F(Clay)	125	0.15	1,970
	44.7	54.7	-77.0	-87.0	H(Sand)	H(Sand)	125	0.30	3,240
	54.7	84.7	-87.0	-117.0	J(Clay)	J(Clay)	125	0.15	3,175
	84.7	114.7	-117.0	-147.0	J(Sand)	J(Sand)	125	0.30	4,755
	114.7	149.7	-147.0	-182.0	J(Clay)	J(Clay)	125	0.15	3,175
	149.7	174.7	-182.0	-207.0	K(Clay)	K(Clay)	124	0.15	3,335
	174.7	195.7	-207.0	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	195.7	200.7	-228.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	200.7	215.7	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	215.7	271.7	-248.0	-304.0	N(Clay)	N(Clay)	123	0.15	6,020
	271.7	289.7	-304.0	-322.0	N(Sand)	N(Sand)	128	0.30	11,645
	289.7	300.7	-322.0	-333.0	N(Clay)	N(Clay)	123	0.15	6,020
	300.7	326.7	-333.0	-359.0	N(Sand)	N(Sand)	128	0.30	11,645
	326.7	336.7	-359.0	-369.0	N(Clay)	N(Clay)	123	0.15	6,020
	336.7	356.7	-369.0	-389.0	N(Sand)	N(Sand)	128	0.30	11,645
	356.7	441.7	-389.0	-474.0	N(Clay)	N(Clay)	123	0.15	6,020
	441.7	476.7	-474.0	-509.0	N(Sand)	N(Sand)	128	0.30	11,645
	476.7	537.2	-509.0	-569.5	N(Clay)	N(Clay)	123	0.15	6,020
	537.2	613.7	-569.5	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	613.7	713.7	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	713.7	813.7	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	813.7	933.7	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	933.7	1233.7	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1233.7	1863.7	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1863.7	2433.7	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-37B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Reactor Buildings (Page 1 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	Alternate Water Table at el. -60.3 ft Short Term Conditions $c = S_u$ for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	3.4	3.6	207.7	110.6
		Concrete Fill	10.0	-50.3	-	-				
		F(Clay)	16.7	-60.3	3.4	0				
		H(Sand)	10.0	-77.0	0	35				
		J(Clay)	83.9	-87.0	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-37B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Reactor Buildings (Page 2 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3	Design Water Table at el. +17.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	3.4	3.6	207.7	110.6
		Concrete Fill	10.0	-50.3	-	-				
		F(Clay)	16.7	-60.3	3.4	0				
		H(Sand)	10.0	-77.0	0	35				
		J(Clay)	83.9	-87.0	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-37B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Reactor Buildings (Page 3 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	Alternate Water Table at el. -60.3 ft	Mat Foundation	-	-32.3	-	-	-2.08	12.8	207.7	130.0
	Long Term Conditions	Concrete Fill	10.0	-50.3	-	-				
		F(Clay)	16.7	-60.3	2.0	8				
	c' and ϕ' for Clays	H(Sand)	10.0	-77.0	0	35				
	Backfill to el. +34.0 ft	J(Clay)	95.0	-87.0	2.3	11				
		K(Clay)	8.3	-182.0	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-37B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Reactor Buildings (Page 4 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3	Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	2.08	12.8	207.7	130.0
		Concrete Fill	10.0	-50.3	-	-				
		F(Clay)	16.7	-60.3	2.0	8				
		H(Sand)	10.0	-77.0	0	35				
		J(Clay)	95.0	-87.0	2.3	11				
		K (Clay)	8.3	-182.0	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-37B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Reactor Buildings (Page 5 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4	Alternate Water Table at el. -60.3 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	2.56	11.7	207.7	127.6
		Concrete Fill	10.0	-50.3	-	-				
		F(Clay)	16.7	-60.3	3.4	0				
		H(Sand)	10.0	-77.0	0	35				
		J(Clay)	30.0	-87.0	3.8	0				
		J(Sand)	30.0	-117.0	0	33				
		J(Clay)	35.0	-147.0	3.8	0				
		K(Clay)	5.9	-182.0	3.9	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-37B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Reactor Buildings (Page 6 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4	Design Water Table at el. +17.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	2.56	11.7	207.7	127.6
		Concrete Fill	10.0	-50.3	-	-				
		F(Clay)	16.7	-60.3	3.4	0				
		H(Sand)	10.0	-77.0	0	35				
		J(Clay)	30.0	-87.0	3.8	0				
		J(Sand)	30.0	-117.0	0	33				
		J(Clay)	35.0	-147.0	3.8	0				
		K(Clay)	5.9	-182.0	3.9	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-37B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Reactor Buildings (Page 7 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4	Alternate Water Table at el. -60.3 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	1.62	17.7	207.7	142.2
		Concrete Fill	10.0	-50.3	-	-				
		F(Clay)	16.7	-60.3	2.0	8				
		H(Sand)	10.0	-77.0	0	35				
		J(Clay)	30.0	-87.0	2.3	11				
		J(Sand)	30.0	-117.0	0	33				
		J(Clay)	35.0	-147.0	2.3	11				
		K(Clay)	20.5	-182.0	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-37B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Reactor Buildings (Page 8 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4	Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	1.62	17.7	207.7	142.2
		Concrete Fill	10.0	-50.3	-	-				
		F(Clay)	16.7	-60.3	2.0	8				
		H(Sand)	10.0	-77.0	0	35				
		J(Clay)	30.0	-87.0	2.3	11				
		J(Sand)	30.0	-117.0	0	33				
		J(Clay)	35.0	-147.0	2.3	11				
		K(Clay)	20.5	-182.0	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-38A Subsurface Conditions for Settlement Analysis for the STP 3 & 4 Control Buildings (Page 1 of 2)

Soil Properties									
	Average Subsurface Conditions		Properties for Settlement Calculation			Average Subsurface Conditions		Properties for Settlement Calculation	
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3	0	10.0	-32.3	-42.3	Mat Foundation	Mat Foundation	-	-	-
	10.0	12.0	-42.3	-44.3	Concrete Fill	Concrete Fill	-	-	-
	12.0	38.7	-44.3	-71.0	E(Sand)	E(Sand)	123	0.30	3,145
	38.7	58.7	-71.0	-91.0	H(Sand)	H(Sand)	125	0.30	3,240
	58.7	94.8	-91.0	-127.1	J(Clay)	J(Clay)	125	0.15	3,175
	94.8	108.7	-127.1	-141.0	J(Sand)	J(Sand)	125	0.30	4,755
	108.7	149.7	-141.0	-182.0	J(Clay)	J(Clay)	125	0.15	3,175
	149.7	165.7	-182.0	-198.0	K(Clay)	K(Clay)	124	0.15	3,335
	165.7	195.7	-198.0	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	195.7	200.7	-228.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	200.7	215.7	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	215.7	277.7	-248.0	-310.0	N(Clay)	N(Clay)	123	0.15	6,020
	277.7	293.7	-310.0	-326.0	N(Sand)	N(Sand)	128	0.30	11,645
	293.7	298.7	-326.0	-331.0	N(Clay)	N(Clay)	123	0.15	6,020
	298.7	337.7	-331.0	-370.0	N(Sand)	N(Sand)	128	0.30	11,645
	337.7	344.7	-370.0	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	344.7	361.7	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	361.7	386.7	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	386.7	402.7	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,645
	402.7	537.2	-435.0	-569.5	N(Clay)	N(Clay)	123	0.15	6,020
	537.2	613.7	-469.5	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	613.7	713.7	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	713.7	813.7	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	813.7	933.7	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	933.7	1233.7	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1233.7	1863.7	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/ Siltstone*	130	0.30	34,130
	1863.7	2433.7	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/ Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-38A Subsurface Conditions for Settlement Analysis for the STP 3 & 4 Control Buildings (Page 2 of 2)

	Soil Properties									
	Average Subsurface Conditions		Properties for Settlement Calculation			Average Subsurface Conditions		Properties for Settlement Calculation	
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
4	0	10.0	-32.3	-42.3	Mat Foundation	-	-	-	-
	10.0	12.0	-42.3	-44.3	Concrete Fill	-	-	-	-
	12.0	19.7	-44.3	-52.0	E(Sand)	E(Sand)	123	0.30	3,145
	19.7	39.7	-52.0	-72.0	F(Clay)	F(Clay)	125	0.15	1,970
	39.7	54.7	-72.0	-87.0	H(Sand)	H(Sand)	125	0.30	3,240
	54.7	79.7	-87.0	-112.0	J(Clay)	J(Clay)	125	0.15	3,175
	79.7	94.7	-112.0	-127.0	J(Sand)	J(Sand)	125	0.30	4,755
	94.7	149.7	-127.0	-182.0	J(Clay)	J(Clay)	125	0.15	3,175
	149.7	174.7	-182.0	-207.0	K(Clay)	K(Clay)	124	0.15	3,335
	174.7	195.7	-207.0	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	195.7	200.7	-228.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	200.7	215.7	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	215.7	271.7	-248.0	-304.0	N(Clay)	N(Clay)	123	0.15	6,020
	271.7	289.7	-304.0	-322.0	N(Sand)	N(Sand)	128	0.30	11,645
	289.7	300.7	-322.0	-333.0	N(Clay)	N(Clay)	123	0.15	6,020
	300.7	326.7	-333.0	-359.0	N(Sand)	N(Sand)	128	0.30	11,645
	326.7	336.7	-359.0	-369.0	N(Clay)	N(Clay)	123	0.15	6,020
	336.7	356.7	-369.0	-389.0	N(Sand)	N(Sand)	128	0.30	11,645
	356.7	441.7	-389.0	-474.0	N(Clay)	N(Clay)	123	0.15	6,020
	441.7	476.7	-474.0	-509.0	N(Sand)	N(Sand)	128	0.30	11,645
	476.7	537.2	-509.0	-569.5	N(Clay)	N(Clay)	123	0.15	6,020
	537.2	613.7	-569.5	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	613.7	713.7	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	713.7	813.7	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	813.7	933.7	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	933.7	1233.7	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1233.7	1863.7	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/ Siltstone*	130	0.30	34,130
	1863.7	2433.7	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/ Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-38B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Control Buildings (Page 1 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3	Alternate Water Table at el. -44.3 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	1.07	26.7	80.1	65.0
		Concrete Fill	2.0	-42.3	-	-				
		E(Sand)	26.7	-44.3	0.0	35				
		H(Sand)	20.0	-71.0	0.0	35				
		J(Clay)	18.3	-91.0	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-38B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Control Buildings (Page 2 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3	Design Water Table at el. +17.0 ft Short Term Conditions $c = S_u$ for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	1.07	26.7	80.1	65.0
		Concrete Fill	2.0	-42.3	-	-				
		E(Sand)	26.7	-44.3	0.0	35				
		H(Sand)	20.0	-71.0						
		J(Clay)	18.3	-91.0						

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-38B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Control Buildings (Page 3 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3	Alternate Water Table at el. -44.3 ft Long Term Conditions c' and ϕ ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	0.71	28.6	80.1	67.4
		Concrete Fill	2.0	-42.3	-	-				
		E(Sand)	26.7	-44.3	0.0	35				
		H(Sand)	20.0	-71.0	0.0	35				
		J(Clay)	20.7	-91.0	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-38B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Control Buildings (Page 4 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3	Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	0.71	28.6	80.1	67.4
		Concrete Fill	2.0	-42.3	-	-				
		E(Sand)	26.7	-44.3	0.0	35				
		H(Sand)	20.0	-71.0	0.0	35				
		J(Clay)	20.7	-91.0	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-38B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Control Buildings (Page 5 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4	Alternate Water Table at el. -44.25 ft Short Term Conditions c = S_u Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	2.04	16.6	80.1	53.7
		Concrete Fill	2.0	-42.3	-	-				
		E(Sand)	7.7	-44.3	0.0	35				
		F(Clay)	20.0	-52.0	3.4	0				
		H(Sand)	15.1	-72.0	0.0	35				
		J(Clay)	10.9	-87.1	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-38B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Control Buildings (Page 6 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4	Design Water Table at el. +17.0 ft Short Term Conditions $c = S_u$ Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	2.04	16.6	80.1	53.7
		Concrete Fill	2.0	-42.3	-	-				
		E(Sand)	7.7	-44.3	0.0	35				
		F(Clay)	20.0	-52.0	3.4	0				
		H(Sand)	15.1	-72.0	0.0	35				
		J(Clay)	10.9	-87.1	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-38B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Control Buildings (Page 7 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4	Alternate Water Table at el. -44.25 ft Long Term Conditions c' and ϕ ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	1.29	20.5	80.1	57.8
		Concrete Fill	2.0	-42.3	-	-				
		E(Sand)	7.7	-44.3	0.0	35				
		F(Clay)	20.0	-52.0	2.0	8				
		H(Sand)	15.0	-72.0	0.0	35				
		J(Clay)	15.1	-87.0	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-38B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Control Buildings (Page 8 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4	Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-32.3	-	-	1.29	20.5	80.1	57.8
		Concrete Fill	2.0	-42.3	-	-				
		E(Sand)	7.7	-44.3	0.0	35				
		F(Clay)	20.0	-52.0	2.0	8				
		H(Sand)	15.0	-72.0	0.0	35				
		J(Clay)	15.1	-87.0	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39A Subsurface Conditions for Settlement Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 1 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3 (No. 1)	0	4.0	-1.0	-5.0	Mat Foundation	Mat Foundation	-	-	-
	4.0	6.0	-5.0	-7.0	Concrete Fill	Concrete Fill	-	-	-
	6.0	49.3	-7.0	-50.3	Structural Fill	Structural Fill	134	0.30	3,000
	49.3	60.4	-50.3	-61.4	E(Sand)	E(Sand)	123	0.30	3,145
	60.4	80.4	-61.4	-81.4	F(Clay)	F(Clay)	125	0.15	1,970
	80.4	101.3	-81.4	-102.3	J(Clay)	J(Clay)	125	0.15	3,175
	101.3	110.4	-102.3	-111.4	J(Sand)	J(Sand)	125	0.30	4,755
	110.4	140.4	-111.4	-141.4	J(Clay)	J(Clay)	125	0.15	3,175
	140.4	141.4	-141.4	-142.4	J(Sand)	J(Sand)	125	0.30	4,755
	141.4	182.0	-142.4	-183.0	J(Clay)	J(Clay)	125	0.15	3,175
	182.0	197.0	-183.0	-198.0	K(Clay)	K(Clay)	124	0.15	3,335
	197.0	228.0	-198.0	-229.0	K(Sand)	K(Sand)	127	0.30	4,915
	228.0	233.0	-229.0	-234.0	L(Clay)	L(Clay)	124	0.15	2,965
	233.0	247.5	-234.0	-248.5	M(Sand)	M(Sand)	127	0.30	4,350
	247.5	309.0	-248.5	-310.0	N(Clay)	N(Clay)	123	0.15	6,020
	309.0	325.0	-310.0	-326.0	N(Sand)	N(Sand)	128	0.30	11,645
	325.0	330.0	-326.0	-331.0	N(Clay)	N(Clay)	123	0.15	6,020
	330.0	369.0	-331.0	-370.0	N(Sand)	N(Sand)	128	0.30	11,645
	369.0	376.0	-370.0	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	376.0	393.0	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	393.0	418.0	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	418.0	434.0	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,645
	434.0	569.0	-435.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	569.0	645.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	645.0	745.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	745.0	845.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	845.0	965.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	965.0	1265.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1265.0	1895.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1895.0	2465.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-39A Subsurface Conditions for Settlement Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 2 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3 (No. 2)	0	4.0	-1.0	-5.0	Mat Foundation	Mat Foundation	-	-	-
	4.0	6.0	-5.0	-7.0	Concrete Fill	Concrete Fill	-	-	-
	6.0	49.3	-7.0	-50.3	Structural Fill	Structural Fill	134	0.30	3,000
	49.3	50.9	-50.3	-51.9	E(Sand)	E(Sand)	123	0.30	3,145
	50.9	80.6	-51.9	-81.6	F(Clay)	F(Clay)	125	0.15	1,970
	80.6	90.6	-81.6	-91.6	H(Sand)	H(Sand)	125	0.30	3,240
	90.6	120.6	-91.6	-121.6	J(Clay)	J(Clay)	125	0.15	3,175
	120.6	130.6	-121.6	-131.6	J(Sand)	J(Sand)	125	0.30	4,755
	130.6	141.2	-131.6	-142.2	J(Clay)	J(Clay)	125	0.15	3,175
	141.2	150.6	-142.2	-151.6	J(Sand)	J(Sand)	125	0.30	4,755
	150.6	182.0	-151.6	-183.0	J(Clay)	J(Clay)	125	0.15	3,175
	182.0	197.0	-183.0	-198.0	K(Clay)	K(Clay)	124	0.15	3,335
	197.0	228.0	-198.0	-229.0	K(Sand)	K(Sand)	127	0.30	4,915
	228.0	233.0	-229.0	-234.0	L(Clay)	L(Clay)	124	0.15	2,965
	233.0	247.5	-234.0	-248.5	M(Sand)	M(Sand)	127	0.30	4,350
	247.5	309.0	-248.5	-310.0	N(Clay)	N(Clay)	123	0.15	6,020
	309.0	325.0	-310.0	-326.0	N(Sand)	N(Sand)	128	0.30	11,645
	325.0	330.0	-326.0	-331.0	N(Clay)	N(Clay)	123	0.15	6,020
	330.0	369.0	-331.0	-370.0	N(Sand)	N(Sand)	128	0.30	11,645
	369.0	376.0	-370.0	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	376.0	393.0	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	393.0	418.0	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	418.0	434.0	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,645
	434.0	569.0	-435.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	569.0	645.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	645.0	745.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	745.0	845.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	845.0	965.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	965.0	1265.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1265.0	1895.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1895.0	2465.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-39A Subsurface Conditions for Settlement Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 3 of 6)

Soil Properties										
	Average Subsurface Conditions					Properties for Settlement Calculation				
	Depth (feet)		Elevation [1] (feet)							
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3 (No. 3)	0	4.0	-1.0	-5.0	Mat Foundation	Mat Foundation	-	-	-
	4.0	6.0	-5.0	-7.0	Concrete Fill	Concrete Fill	-	-	-
	6.0	33.0	-7.0	-34.0	Structural Fill	Structural Fill	134	0.30	3,000
	33.0	37.3	-34.0	-38.3	D(Clay)	D(Clay)	122	0.15	1,865
	37.3	57.3	-38.3	-58.3	E(Sand)	E(Sand)	123	0.30	3,145
	57.3	67.1	-58.3	-68.1	F(Clay)	F(Clay)	125	0.15	1,970
	67.1	77.3	-68.1	-78.3	H(Sand)	H(Sand)	125	0.30	3,240
	77.3	107.3	-78.3	-108.3	J(Clay)	J(Clay)	125	0.15	3,175
	107.3	117.6	-108.3	-118.6	J(Sand)	J(Sand)	125	0.30	4,755
	117.6	147.7	-118.6	-148.7	J(Clay)	J(Clay)	125	0.15	3,175
	147.7	152.3	-148.7	-153.3	J(Sand)	J(Sand)	125	0.30	4,755
	152.3	182.0	-153.3	-183.0	J(Clay)	J(Clay)	125	0.15	3,175
	182.0	197.0	-183.0	-198.0	K(Clay)	K(Clay)	124	0.15	3,335
	197.0	228.0	-198.0	-229.0	K(Sand)	K(Sand)	127	0.30	4,915
	228.0	233.0	-229.0	-234.0	L(Clay)	L(Clay)	124	0.15	2,965
	233.0	247.5	-234.0	-248.5	M(Sand)	M(Sand)	127	0.30	4,350
	247.5	309.0	-248.5	-310.0	N(Clay)	N(Clay)	123	0.15	6,020
	309.0	325.0	-310.0	-326.0	N(Sand)	N(Sand)	128	0.30	11,645
	325.0	330.0	-326.0	-331.0	N(Clay)	N(Clay)	123	0.15	6,020
	330.0	369.0	-331.0	-370.0	N(Sand)	N(Sand)	128	0.30	11,645
	369.0	376.0	-370.0	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	376.0	393.0	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	393.0	418.0	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	418.0	434.0	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,645
	434.0	569.0	-435.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	569.0	645.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	645.0	745.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	745.0	845.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	845.0	965.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	965.0	1265.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1265.0	1895.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1895.0	2465.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-39A Subsurface Conditions for Settlement Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 4 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
4 (No. 1)	0	4.0	-1.0	-5.0	Mat Foundation	Mat Foundation	-	-	-
	4.0	6.0	-5.0	-7.0	Concrete Fill	Concrete Fill	-	-	-
	6.0	49.3	-7.0	-50.3	Structural Fill	Structural Fill	134	0.30	3,000
	49.3	55.2	-50.3	-56.2	E(Sand)	E(Sand)	123	0.30	3,145
	55.2	75.2	-56.2	-76.2	F(Clay)	F(Clay)	125	0.15	1,970
	75.2	85.2	-76.2	-86.2	J(Clay)	J(Clay)	125	0.15	3,175
	85.2	146.0	-86.2	-147.0	J(Sand)	J(Sand)	125	0.30	4,755
	146.0	184.0	-147.0	-185.0	J(Clay)	J(Clay)	125	0.15	3,175
	184.0	206.0	-185.0	-207.0	K(Clay)	K(Clay)	124	0.15	3,335
	206.0	227.0	-207.0	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	227.0	232.0	-228.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	232.0	247.0	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	247.0	303.0	-248.0	-304.0	N(Clay)	N(Clay)	123	0.15	6,020
	303.0	321.0	-304.0	-322.0	N(Sand)	N(Sand)	128	0.30	11,645
	321.0	332.0	-322.0	-333.0	N(Clay)	N(Clay)	123	0.15	6,020
	332.0	358.0	-333.0	-359.0	N(Sand)	N(Sand)	128	0.30	11,645
	358.0	368.0	-359.0	-369.0	N(Clay)	N(Clay)	123	0.15	6,020
	368.0	388.0	-369.0	-389.0	N(Sand)	N(Sand)	128	0.30	11,645
	388.0	473.0	-389.0	-474.0	N(Clay)	N(Clay)	123	0.15	6,020
	473.0	508.0	-474.0	-509.0	N(Sand)	N(Sand)	128	0.30	11,645
	508.0	569.0	-509.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	569.0	645.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	645.0	745.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	745.0	845.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	845.0	965.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	965.0	1265.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1265.0	1895.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1895.0	2465.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-39A Subsurface Conditions for Settlement Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 5 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
4 (No. 2)	0	4.0	-1.0	-5.0	Mat Foundation	Mat Foundation	-	-	-
	4.0	6.0	-5.0	-7.0	Concrete Fill	Concrete Fill	-	-	-
	6.0	49.3	-7.0	-50.3	Structural Fill	Structural Fill	134	0.30	3,000
	49.3	56.2	-30.0	-57.2	E(Sand)	E(Sand)	123	0.30	3,145
	56.2	76.5	-57.2	-77.5	F(Clay)	F(Clay)	125	0.15	1,970
	76.5	106.2	-77.5	-107.2	J(Clay)	J(Clay)	125	0.15	3,175
	106.2	131.7	-107.2	-132.7	J(Sand)	J(Sand)	125	0.30	4,755
	131.7	137.4	-132.7	-138.4	J(Clay)	J(Clay)	125	0.15	3,175
	137.4	184.0	-138.4	-185.0	J(Sand)	J(Sand)	125	0.30	4,755
	184.0	206.0	-185.0	-207.0	K(Clay)	K(Clay)	124	0.15	3,335
	206.0	227.0	-207.0	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	227.0	232.0	-228.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	232.0	247.0	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	247.0	303.0	-248.0	-304.0	N(Clay)	N(Clay)	123	0.15	6,020
	303.0	321.0	-304.0	-322.0	N(Sand)	N(Sand)	128	0.30	11,645
	321.0	332.0	-322.0	-333.0	N(Clay)	N(Clay)	123	0.15	6,020
	332.0	358.0	-333.0	-359.0	N(Sand)	N(Sand)	128	0.30	11,645
	358.0	368.0	-359.0	-369.0	N(Clay)	N(Clay)	123	0.15	6,020
	368.0	388.0	-369.0	-389.0	N(Sand)	N(Sand)	128	0.30	11,6450
	388.0	473.0	-389.0	-474.0	N(Clay)	N(Clay)	123	0.15	6,020
	473.0	508.0	-474.0	-509.0	N(Sand)	N(Sand)	128	0.30	11,645
	508.0	569.0	-509.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	569.0	645.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	645.0	745.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	745.0	845.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	845.0	965.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	965.0	1265.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1265.0	1895.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1895.0	2465.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-39A Subsurface Conditions for Settlement Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 6 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
4 (No. 3)	0	4.0	-1.0	-5.0	Mat Foundation	Mat Foundation	-	-	-
	4.0	6.0	-5.0	-7.0	Concrete Fill	Concrete Fill	-	-	-
	6.0	33.0	-7.0	-34.0	Structural Fill	Structural Fill	134	0.30	3,000
	33.0	56.9	-34.0	-57.9	E(Sand)	E(Sand)	123	0.30	3,145
	56.9	77.2	-57.9	-78.2	F(Clay)	F(Clay)	125	0.15	1,970
	77.2	116.9	-78.2	-117.9	J(Clay)	J(Clay)	125	0.15	3,175
	116.9	152.0	-117.9	-153.0	J(Sand)	J(Sand)	125	0.30	4,755
	152.0	184.0	-153.0	-185.0	J(Clay)	J(Clay)	125	0.15	3,175
	184.0	206.0	-185.0	-207.0	K(Clay)	K(Clay)	124	0.15	3,335
	206.0	227.0	-207.0	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	227.0	232.0	-228.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	232.0	247.0	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	247.0	303.0	-248.0	-304.0	N(Clay)	N(Clay)	123	0.15	6,020
	303.0	321.0	-304.0	-322.0	N(Sand)	N(Sand)	128	0.30	11,645
	321.0	332.0	-322.0	-333.0	N(Clay)	N(Clay)	123	0.15	6,020
	332.0	358.0	-333.0	-359.0	N(Sand)	N(Sand)	128	0.30	11,645
	358.0	368.0	-359.0	-369.0	N(Clay)	N(Clay)	123	0.15	6,020
	368.0	388.0	-369.0	-389.0	N(Sand)	N(Sand)	128	0.30	11,645
	388.0	473.0	-389.0	-474.0	N(Clay)	N(Clay)	123	0.15	6,020
	473.0	508.0	-474.0	-509.0	N(Sand)	N(Sand)	128	0.30	11,645
	508.0	569.0	-509.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	569.0	645.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	645.0	745.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	745.0	845.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	845.0	965.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	965.0	1265.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1265.0	1895.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1895.0	2465.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 1 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3 (No. 1)	Assume Failure Along East or West Edge of Mat Alternate Water Table at el. -7.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.0	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	3.6				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 2 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3 (No. 1)	Assume Failure Along East or West Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.0	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 3 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3 (No. 1)	Assume Failure Along East or West Edge of Mat Alternate Water Table at el. -7.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.0	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 4 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3 (No. 1)	Assume Failure Along East or West Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.0	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 5 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3 (No. 2)	Assume Failure Along East or West Edge of Mat Alternate Water Table at el. -7.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.0	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 6 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3 (No. 2)	Assume Failure Along East or West Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.0	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 7 of 24)

<table>
<tr><th colspan="11">Average Properties within the Foundation Deformation Zone for Bearing Capacity</th></tr>
<tr><th rowspan="3">STP</th><th rowspan="3">Soil Selection Preference</th><th rowspan="3">Soil Layer</th><th rowspan="3">Thickness of Layer Below Mat (feet)</th><th rowspan="3">Top Elevation [1] (feet)</th><th colspan="4">Shear Strength</th><th rowspan="3">Foundation Width, B (feet)</th><th rowspan="3">Effective Shear Depth, H' (feet)</th></tr>
<tr><th colspan="2">Layer</th><th colspan="2">Average</th></tr>
<tr><th>c (ksf)</th><th>Φ (°)</th><th>c (ksf)</th><th>Φ (°)</th></tr>
<tr><td rowspan="3">3
(No. 2)</td><td rowspan="3">Assume Failure Along East or West Edge of Mat

Alternate Water Table at el. -7.0 ft

Long Term Conditions

c' and ϕ' for Clays

Backfill to el. +34.0 ft</td><td>Mat Foundation</td><td>-</td><td>-1.0</td><td>-</td><td>-</td><td rowspan="3">0.0</td><td rowspan="3">36.0</td><td rowspan="3">44.0</td><td rowspan="3">43.2</td></tr>
<tr><td>Concrete Fill</td><td>2.0</td><td>-5.0</td><td>-</td><td>-</td></tr>
<tr><td>Structural Fill</td><td>43.2</td><td>-7.0</td><td>0.0</td><td>36</td></tr>
</table>

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 8 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3 (No. 2)	Assume Failure Along East or West Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.0	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 9 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3 (No. 3)	Assume Failure Along North or South Edge of Mat Alternate Water Table at el. -7.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.32	32.7	44.0	40.3
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	27.0	-7.0	0.0	36				
		D(Clay)	4.3	-34.0	3.0	0				
		E(Sand)	9.0	-38.3	0.0	35				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 10 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3 (No. 3)	Assume Failure Along North or South Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions $c = S_u$ for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.32	32.7	44.0	40.3
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	27.0	-7.0	0.0	36				
		D(Clay)	4.3	-34.0	3.0	0				
		E(Sand)	9.0	-38.3	0.0	35				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 11 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
3 (No. 3)	Assume Failure Along North or South Edge of Mat Alternate Water Table at el. -7.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.12	34.0	44.0	41.4
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	27.0	-7.0	0.0	36				
		D(Clay)	4.3	-34.0	1.2	16				
		E(Sand)	10.1	-38.3	0	35				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 12 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3 (No. 3)	Assume Failure Along North or South Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.12	34.0	44.0	41.4
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	27.0	-7.0	0.0	36				
		D(Clay)	4.3	-34.0	1.2	16				
		E(Sand)	10.1	-38.3	0	35				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 13 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4 (No. 1)	Assume Failure Along North or South Edge of Mat Alternate Water Table at el.-7.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 14 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4 (No. 1)	Assume Failure Along North or South Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 15 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4 (No. 1)	Assume Failure Along North or South Edge of Mat Alternate Water Table at el. -7.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 16 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4 (No. 1)	Assume Failure Along North or South Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 17 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4 (No. 2)	Assume Failure Along North or South Edge of Mat Alternate Water Table at el. -7.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 18 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4 (No. 2)	Assume Failure Along North or South Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 19 of 24)

<table>
<tr><th colspan="11">Average Properties within the Foundation Deformation Zone for Bearing Capacity</th></tr>
<tr><th rowspan="3">STP</th><th rowspan="3">Soil Selection Preference</th><th rowspan="3">Soil Layer</th><th rowspan="3">Thickness of Layer Below Mat (feet)</th><th rowspan="3">Top Elevation [1] (feet)</th><th colspan="4">Shear Strength</th><th rowspan="3">Foundation Width, B (feet)</th><th rowspan="3">Effective Shear Depth, H' (feet)</th></tr>
<tr><th colspan="2">Layer</th><th colspan="2">Average</th></tr>
<tr><th>c (ksf)</th><th>Φ (°)</th><th>c (ksf)</th><th>Φ (°)</th></tr>
<tr><td rowspan="3">4
(No. 2)</td><td rowspan="3">Assume Failure Along North or South Edge of Mat

Alternate Water Table at el. -7.0 ft

Long Term Conditions

c' and ϕ' for Clays

Backfill to el. +34.0 ft</td><td>Mat Foundation</td><td>-</td><td>-1.0</td><td>-</td><td>-</td><td rowspan="3">0.00</td><td rowspan="3">36.0</td><td rowspan="3">44.0</td><td rowspan="3">43.2</td></tr>
<tr><td>Concrete Fill</td><td>2.0</td><td>-5.0</td><td>-</td><td>-</td></tr>
<tr><td>Structural Fill</td><td>43.2</td><td>-7.0</td><td>0.0</td><td>36</td></tr>
</table>

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 20 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4 (No. 2)	Assume Failure Along North or South Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	36.0	44.0	43.2
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	43.2	-7.0	0.0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 21 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4 (No. 3)	Assume Failure Along North or South Edge of Mat Alternate Water Table at el. -7.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	35.6	44.0	42.8
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	27.0	-7.0	0.0	36				
		E(Sand)	15.8	-34.0	0.0	35				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 22 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4 (No. 3)	Assume Failure Along North or South Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions $c = S_u$ for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	35.6	44.0	42.8
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	27.0	-7.0	0.0	36				
		E(Sand)	15.8	-34.0	0.0	35				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 23 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	Φ (°)	c (ksf)	Φ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4 (No. 3)	Assume Failure Along North or South Edge of Mat Alternate Water Table at el. -7.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	35.6	44.0	42.8
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	27.0	-7.0	0.0	36				
		E(Sand)	15.8	-34.0	0.0	35				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-39B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 Diesel Generator Fuel Oil Storage Vaults (Page 24 of 24)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	Φ (°)	c (ksf)	Φ (°)		
4 (No. 3)	Assume Failure Along North or South Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-1.0	-	-	0.00	35.6	44.0	42.8
		Concrete Fill	2.0	-5.0	-	-				
		Structural Fill	27.0	-7.0	0.0	36				
		E(Sand)	15.8	-34.0	0.0	35				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40A Subsurface Conditions for Settlement Analysis for the STP 3 & 4 UHS Basins (Page 1 of 2)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3	0	10.0	14.0	4.0	Mat Foundation	-	-	-	-
	10.0	14.0	4.0	0.0	Concrete Fill	-	-	-	-
	14.0	32.8	0.0	-18.8	C(Sand)	C(Sand)	122	0.30	1,810
	32.8	53.6	-18.8	-39.6	D(Clay)	D(Clay)	122	0.15	1,865
	53.6	77.9	-39.6	-63.9	E(Sand)	E(Sand)	123	0.30	3,145
	77.9	100.9	-63.9	-86.9	F(Clay)	F(Clay)	125	0.15	1,970
	100.9	138.6	-86.9	-124.6	J(Clay)	J(Clay)	125	0.15	3,175
	138.6	151.1	-124.6	-137.1	J(Sand)	J(Sand)	125	0.30	4,755
	151.1	186.9	-137.1	-172.9	J(Clay)	J(Clay)	125	0.15	3,175
	186.9	207.9	-172.9	-193.9	K(Clay)	K(Clay)	124	0.15	3,335
	207.9	228.9	-193.9	-214.9	K(Sand)	K(Sand)	127	0.30	4,915
	228.9	259.7	-214.9	-245.7	L(Clay)	L(Clay)	124	0.15	2,965
	259.7	269.9	-245.7	-255.9	M(Sand)	M(Sand)	127	0.30	4,350
	269.9	324.0	-255.9	-310.0	N(Clay)	N(Clay)	123	0.15	6,020
	324.0	340.0	-310.0	-326.0	N(Sand)	N(Sand)	128	0.30	11,645
	340.0	345.0	-326.0	-331.0	N(Clay)	N(Clay)	123	0.15	6,020
	345.0	384.0	-331.0	-370.0	N(Sand)	N(Sand)	128	0.30	11,645
	384.0	391.0	-370.0	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	391.0	408.0	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	408.0	433.0	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	433.0	449.0	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,645
	449.0	584.0	-435.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	584.0	660.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	660.0	760.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	760.0	860.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	860.0	980.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	980.0	1280.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1280.0	1910.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1910.0	2480.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-40A Subsurface Conditions for Settlement Analysis for the STP 3 & 4 UHS Basins (Page 2 of 2)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
4	0	10.0	14.0	4.0	Mat Foundation	-	-	-	-
	10.0	14.0	4.0	0.0	Concrete Fill	-	-	-	-
	14.0	29.0	0.0	-15.0	C(Sand)	C(Sand)	122	0.30	1,810
	29.0	48.3	-15.0	-34.3	D(Clay)	D(Clay)	122	0.15	1,865
	48.3	65.7	-34.3	-51.7	E(Sand)	E(Sand)	123	0.30	3,145
	65.7	103.8	-51.7	-89.8	F(Clay)	F(Clay)	125	0.15	1,970
	103.8	124.4	-89.8	-110.4	J(Clay)	J(Clay)	125	0.15	3,175
	124.4	146.0	-110.4	-132.0	J(Sand)	J(Sand)	125	0.30	4,755
	146.0	203.8	-132.0	-189.8	J(Clay)	J(Clay)	125	0.15	3,175
	203.8	247.8	-189.8	-233.8	K(Clay)	K(Clay)	124	0.15	3,335
	247.8	270.9	-233.8	-256.9	L(Clay)	L(Clay)	124	0.15	2,965
	270.9	280.8	-256.9	-266.8	M(Sand)	M(Sand)	127	0.30	4,350
	280.8	318.0	-266.8	-304.0	N(Clay)	N(Clay)	123	0.15	6,020
	318.0	336.0	-304.0	-322.0	N(Sand)	N(Sand)	128	0.30	11,645
	336.0	347.0	-322.0	-333.0	N(Clay)	N(Clay)	123	0.15	6,020
	347.0	373.0	-333.0	-359.0	N(Sand)	N(Sand)	128	0.30	11,645
	373.0	383.0	-359.0	-369.0	N(Clay)	N(Clay)	123	0.15	6,020
	383.0	403.0	-369.0	-389.0	N(Sand)	N(Sand)	128	0.30	11,645
	403.0	488.0	-389.0	-474.0	N(Clay)	N(Clay)	123	0.15	6,020
	488.0	523.0	-474.0	-509.0	N(Sand)	N(Sand)	128	0.30	11,645
	523.0	584.0	-509.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	584.0	660.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	660.0	760.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	760.0	860.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	860.0	980.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	980.0	1280.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1280.0	1910.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1910.0	2480.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-40B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 UHS Basins (Page 1 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	Assume Failure Along South Edge of Mat Alternate Water Table at el. +0.0 ft Short Term Conditions c= S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	14.0	-	-	1.17	24.1	164.0	126.5
		Concrete Fill	4.0	4.0	-	-				
		C(Sand)	18.8	0.0	0.0	35				
		D(Clay)	20.8	-18.8	3.0	0				
		E(Sand)	24.3	-39.6	0.0	35				
		F(Clay)	23.0	-63.9	3.4	0				
		H(Sand)	37.7	-86.9	0.0	35				
		J(Clay)	1.9	-124.6	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 UHS Basins (Page 2 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	Assume Failure Along South Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions c= S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	14.0	-	-	1.17	24.1	164.0	126.5
		Concrete Fill	4.0	4.0	-	-				
		C(Sand)	18.8	0.0	0.0	35				
		D(Clay)	20.8	-18.8	3.0	0				
		E(Sand)	24.3	-39.6	0.0	35				
		F(Clay)	23.0	-63.9	3.4	0				
		H(Sand)	37.7	-86.9	0.0	35				
		J(Clay)	1.9	-124.6	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 UHS Basins (Page 3 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	Assume Failure Along South Edge of Mat Alternate Water Table at el. +0.0 ftt Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	14.0	-	-	0.68	26.8	164.0	133.4
		Concrete Fill	4.0	4.0	-	-				
		C(Sand)	18.8	0.0	0.0	35				
		D(Clay)	20.8	-18.8	1.2	16				
		E(Sand)	24.3	-39.6	0.0	35				
		F(Clay)	23.0	-63.9	2.0	8				
		H(Sand)	37.7	-86.9	0.0	35				
		J(Clay)	8.8	-124.6	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 UHS Basins (Page 4 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	Assume Failure Along South Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	14.0	-	-	0.68	26.8	164.0	133.4
		Concrete Fill	4.0	4.0	-	-				
		C(Sand)	18.8	0.0	0.0	35				
		D(Clay)	20.8	-18.8	1.2	16				
		E(Sand)	24.3	-39.6	0.0	35				
		F(Clay)	23.0	-63.9	2.0	8				
		H(Sand)	37.7	-86.9	0.0	35				
		J(Clay)	8.8	-124.6	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 UHS Basins (Page 5 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	ϕ (°)	c (ksf)	ϕ (°)		
4	Assume Failure Along South Edge of Mat Alternate Water Table at el. +0.0 ft Short Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	14.0	-	-	2.30	12.5	164.0	102.2
		Concrete Fill	4.0	4.0	-	-				
		C(Sand)	15.0	0	0.0	35				
		D(Clay)	19.3	-15.0	3.0	0				
		E(Sand)	17.4	-34.3	0.0	35				
		F(Clay)	38.1	-51.7	3.4	0				
		J(Clay)	12.4	-89.8	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 UHS Basins (Page 6 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4	Assume Failure Along South Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions c = S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	14.0	-	-	2.30	12.5	164.0	102.2
		Concrete Fill	4.0	4.0	-	-				
		C(Sand)	15.0	0	0.0	35				
		D(Clay)	19.3	-15.0	3.0	0				
		E(Sand)	17.4	-34.3	0.0	35				
		F(Clay)	38.1	-51.7	3.4	0				
		J(Clay)	12.4	-89.8	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 UHS Basins (Page 7 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4	Assume Failure Along South Edge of Mat Alternate Water Table at el. +0.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	14.0	-	-	1.26	19.6	164.0	116.2
		Concrete Fill	4.0	4.0	-	-				
		C(Sand)	15.0	0	0.0	35				
		D(Clay)	19.3	-15.0	1.2	16				
		E(Sand)	17.4	-34.3	0.0	35				
		F(Clay)	38.1	-51.7	2.0	8				
		J(Clay)	20.6	-89.8	2.3	11				
		J(Sand)	5.8	-110.4	0.0	33				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40B Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 UHS Basins (Page 8 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4	Assume Failure Along South Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	14.0	-	-	1.26	19.6	164.0	116.2
		Concrete Fill	4.0	4.0	-	-				
		C(Sand)	15.0	0	0.0	35				
		D(Clay)	19.3	-15.0	1.2	16				
		E(Sand)	17.4	-34.3	0.0	35				
		F(Clay)	38.1	-51.7	2.0	8				
		J(Clay)	20.6	-89.8	2.3	11				
		J(Sand)	5.8	-110.4	0.0	33				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40C Conditions for Settlement Analysis for the STP 3 & 4 RSW Tunnels (Page 1 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3 (South Area)	0	3.0	-18.0	-21.0	Mat Foundation	Mat Foundation	-	-	-
	3.0	5.0	-21.0	-23.0	Concrete Fill	Concrete Fill	-	-	-
	5.0	16.0	-23.0	-34.0	Structural Fill	Structural Fill	134	0.30	3,000
	16.0	16.6	-34.0	-34.6	D(Clay)	D(Clay)	122	0.15	1,865
	16.6	36.6	-34.6	-54.6	E(Sand)	E(Sand)	123	0.30	3,145
	36.6	61.6	-54.6	-79.6	F(Clay)	F(Clay)	125	0.15	1,970
	61.6	131.6	-79.6	-149.6	J(Clay)	J(Clay)	125	0.15	3,175
	131.6	141.6	-149.6	-159.6	J(Sand)	J(Sand)	125	0.30	4,755
	141.6	151.6	-159.6	-169.6	J(Clay)	J(Clay)	125	0.15	3,175
	151.6	180.0	-169.6	-198.0	K(Clay)	K(Clay)	124	0.15	3,335
	180.0	211.0	-198.0	-229.0	K(Sand)	K(Sand)	127	0.30	4,915
	211.0	216.0	-229.0	-234.0	L(Clay)	L(Clay)	124	0.15	2,965
	216.0	230.5	-234.0	-248.5	M(Sand)	M(Sand)	127	0.30	4,350
	230.5	292.0	-248.5	-310.0	N(Clay)	N(Clay)	123	0.15	6,020
	292.0	308.0	-310.0	-326.0	N(Sand)	N(Sand)	128	0.30	11,645
	308.0	313.0	-326.0	-331.0	N(Clay)	N(Clay)	123	0.15	6,020
	313.0	352.0	-331.0	-370.0	N(Sand)	N(Sand)	128	0.30	11,645
	352.0	359.0	-370.0	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	359.0	376.0	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	376.0	401.0	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	401.0	417.0	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,645
	417.0	552.0	-435.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	552.0	628.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	628.0	728.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	728.0	828.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	828.0	948.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	948.0	1248.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1248.0	1878.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1878.0	2448.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-40C Conditions for Settlement Analysis for the STP 3 & 4 RSW Tunnels (Page 2 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3 (Center Area)	0	3.0	-18.0	-21.0	Mat Foundation	Mat Foundation	-	-	-
	3.0	5.0	-21.0	-23.0	Concrete Fill	Concrete Fill	-	-	-
	5.0	42.3	-23.0	-60.3	Structural Fill	Structural Fill	134	0.30	3,000
	42.3	56.4	-60.3	-74.4	F(Clay)	F(Clay)	125	0.15	1,970
	56.4	66.4	-74.4	-84.4	H(Sand)	H(Sand)	125	0.30	3,240
	66.4	121.4	-84.4	-139.4	J(Clay)	J(Clay)	125	0.15	3,175
	121.4	141.4	-139.4	-159.4	J(Sand)	J(Sand)	125	0.30	4,755
	141.4	151.4	-159.4	-169.4	J(Clay)	J(Clay)	125	0.15	3,175
	151.4	161.4	-169.4	-179.4	J(Sand)	J(Sand)	125	0.30	4,755
	161.4	180.0	-179.4	-198.0	K(Clay)	K(Clay)	124	0.15	3,335
	180.0	211.0	-198.0	-229.0	K(Sand)	K(Sand)	127	0.30	4,915
	211.0	216.0	-229.0	-234.0	L(Clay)	L(Clay)	124	0.15	2,965
	216.0	230.5	-234.0	-248.5	M(Sand)	M(Sand)	127	0.30	4,350
	230.5	292.0	-248.5	-310.0	N(Clay)	N(Clay)	123	0.15	6,020
	292.0	308.0	-310.0	-326.0	N(Sand)	N(Sand)	128	0.30	11,645
	308.0	313.0	-326.0	-331.0	N(Clay)	N(Clay)	123	0.15	6,020
	313.0	352.0	-331.0	-370.0	N(Sand)	N(Sand)	128	0.30	11,645
	352.0	359.0	-370.0	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	359.0	376.0	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	376.0	401.0	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	401.0	417.0	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,645
	417.0	552.0	-435.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	552.0	628.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	628.0	728.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	728.0	828.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	828.0	948.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	948.0	1248.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1248.0	1878.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1878.0	2448.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-40C Conditions for Settlement Analysis for the STP 3 & 4 RSW Tunnels (Page 3 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3 (North Area)	0	3.0	-18.0	-21.0	Mat Foundation	Mat Foundation	-	-	-
	3.0	5.0	-21.0	-23.0	Concrete Fill	Concrete Fill	-	-	-
	5.0	21.0	-23.0	-39.0	Structural Fill	Structural Fill	134	0.30	3,000
	21.0	23.9	-39.0	-41.9	D(Clay)	D(Clay)	122	0.15	1,865
	23.9	38.9	-41.9	-56.9	E(Sand)	E(Sand)	123	0.30	3,145
	38.9	56.4	-56.9	-74.4	F(Clay)	F(Clay)	125	0.15	1,970
	56.4	66.4	-74.4	-84.4	H(Sand)	H(Sand)	125	0.30	3,240
	66.4	121.4	-84.4	-139.4	J(Clay)	J(Clay)	125	0.15	3,175
	121.4	141.4	-139.4	-159.4	J(Sand)	J(Sand)	125	0.30	4,755
	141.4	151.4	-159.4	-169.4	J(Clay)	J(Clay)	125	0.15	3,175
	151.4	161.4	-169.4	-179.4	J(Sand)	J(Sand)	125	0.30	4,755
	161.4	180.0	-179.4	-198.0	K(Clay)	K(Clay)	124	0.15	3,335
	180.0	211.0	-198.0	-229.0	K(Sand)	K(Sand)	127	0.30	4,915
	211.0	216.0	-229.0	-234.0	L(Clay)	L(Clay)	124	0.15	2,965
	216.0	230.5	-234.0	-248.5	M(Sand)	M(Sand)	127	0.30	4,350
	230.5	292.0	-248.5	-310.0	N(Clay)	N(Clay)	123	0.15	6,020
	292.0	308.0	-310.0	-326.0	N(Sand)	N(Sand)	128	0.30	11,645
	308.0	313.0	-326.0	-331.0	N(Clay)	N(Clay)	123	0.15	6,020
	313.0	352.0	-331.0	-370.0	N(Sand)	N(Sand)	128	0.30	11,645
	352.0	359.0	-370.0	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	359.0	376.0	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	376.0	401.0	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	401.0	417.0	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,645
	417.0	552.0	-435.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	552.0	628.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	628.0	728.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	728.0	828.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	828.0	948.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	948.0	1248.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1248.0	1878.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1878.0	2448.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-40C Conditions for Settlement Analysis for the STP 3 & 4 RSW Tunnels (Page 4 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
4 (South Area)	0	3.0	-18.0	-21.0	Mat Foundation	Mat Foundation	-	-	-
	3.0	5.0	-21.0	-23.0	Concrete Fill	Concrete Fill	-	-	-
	5.0	16.0	-23.0	-34.0	Structural Fill	Structural Fill	134	0.30	3,000
	16.0	36.6	-34.0	-54.6	E(Sand)	E(Sand)	123	0.30	3,145
	36.6	61.5	-54.6	-79.5	F(Clay)	F(Clay)	125	0.15	1,970
	61.5	96.6	-79.5	-114.6	J(Clay)	J(Clay)	125	0.15	3,175
	96.6	112.2	-114.6	-130.2	J(Sand)	J(Sand)	125	0.30	4,755
	112.2	121.8	-130.2	-139.8	J(Clay)	J(Clay)	125	0.15	3,175
	121.8	134.7	-139.8	-152.7	J(Sand)	J(Sand)	125	0.30	4,755
	134.7	153.9	-152.7	-171.9	J(Clay)	J(Clay)	125	0.15	3,175
	153.9	193.9	-171.9	-211.9	K(Clay)	K(Clay)	124	0.15	3,335
	193.9	210.0	-211.9	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	210.0	215.0	-228.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	215.0	230.0	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	230.0	286.0	-248.0	-304.0	N(Clay)	N(Clay)	123	0.15	6,020
	286.0	304.0	-304.0	-322.0	N(Sand)	N(Sand)	128	0.30	11,645
	304.0	315.0	-322.0	-333.0	N(Clay)	N(Clay)	123	0.15	6,020
	315.0	341.0	-333.0	-359.0	N(Sand)	N(Sand)	128	0.30	11,645
	341.0	351.0	-359.0	-369.0	N(Clay)	N(Clay)	123	0.15	6,020
	351.0	371.0	-369.0	-389.0	N(Sand)	N(Sand)	128	0.30	11,645
	371.0	456.0	-389.0	-474.0	N(Clay)	N(Clay)	123	0.15	6,020
	456.0	491.0	-474.0	-509.0	N(Sand)	N(Sand)	128	0.30	11,645
	491.0	552.0	-509.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	552.0	628.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	628.0	728.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	728.0	828.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	828.0	948.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	948.0	1248.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1248.0	1878.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1878.0	2448.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-40C Conditions for Settlement Analysis for the STP 3 & 4 RSW Tunnels (Page 5 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
4 (Center Area)	0	3.0	-18.0	-21.0	Mat Foundation	Mat Foundation	-	-	-
	3.0	5.0	-21.0	-23.0	Concrete Fill	Concrete Fill	-	-	-
	5.0	42.3	-23.0	-60.3	Structural Fill	Structural Fill	134	0.30	3,000
	42.3	59.2	-60.3	-77.2	F(Clay)	F(Clay)	125	0.15	1,970
	59.2	62.2	-77.2	-80.2	H(Sand)	H(Sand)	125	0.30	3,240
	62.2	91.7	-80.2	-109.7	J(Clay)	J(Clay)	125	0.15	3,175
	91.7	109.2	-109.7	-127.2	J(Sand)	J(Sand)	125	0.30	4,755
	109.2	149.2	-127.2	-167.2	J(Clay)	J(Clay)	125	0.15	3,175
	149.2	189.0	-167.2	-207.0	K(Clay)	K(Clay)	124	0.15	3,335
	189.0	210.0	-207.0	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	210.0	215.0	-228.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	215.0	230.0	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	230.0	286.0	-248.0	-304.0	N(Clay)	N(Clay)	123	0.15	6,020
	286.0	304.0	-304.0	-322.0	N(Sand)	N(Sand)	128	0.30	11,645
	304.0	315.0	-322.0	-333.0	N(Clay)	N(Clay)	123	0.15	6,020
	315.0	341.0	-333.0	-359.0	N(Sand)	N(Sand)	128	0.30	11,645
	341.0	351.0	-359.0	-369.0	N(Clay)	N(Clay)	123	0.15	6,020
	351.0	371.0	-369.0	-389.0	N(Sand)	N(Sand)	128	0.30	11,645
	371.0	456.0	-389.0	-474.0	N(Clay)	N(Clay)	123	0.15	6,020
	456.0	491.0	-474.0	-509.0	N(Sand)	N(Sand)	128	0.30	11,645
	491.0	552.0	-509.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	552.0	628.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	628.0	728.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	728.0	828.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	828.0	948.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	948.0	1248.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1248.0	1878.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1878.0	2448.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-40C Conditions for Settlement Analysis for the STP 3 & 4 RSW Tunnels (Page 6 of 6)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
4 (North Area)	0	3.0	-18.0	-21.0	Mat Foundation	Mat Foundation	-	-	-
	3.0	5.0	-21.0	-23.0	Concrete Fill	Concrete Fill	-	-	-
	5.0	21.0	-23.0	-39.0	Structural Fill	Structural Fill	134	0.30	3,000
	21.0	31.7	-39.0	-49.7	E(Sand)	E(Sand)	123	0.30	3,145
	31.7	59.2	-49.7	-77.2	F(Clay)	F(Clay)	125	0.15	1,970
	59.2	62.2	-77.2	-80.2	H(Sand)	H(Sand)	125	0.30	3,240
	62.2	91.7	-80.2	-109.7	J(Clay)	J(Clay)	125	0.15	3,175
	91.7	109.2	-109.7	-127.2	J(Sand)	J(Sand)	125	0.30	4,755
	109.2	149.2	-127.2	-167.2	J(Clay)	J(Clay)	125	0.15	3,175
	149.2	189.0	-167.2	-207.0	K(Clay)	K(Clay)	124	0.15	3,335
	189.0	210.0	-207.0	-228.0	K(Sand)	K(Sand)	127	0.30	4,915
	210.0	215.0	-228.0	-233.0	L(Clay)	L(Clay)	124	0.15	2,965
	215.0	230.0	-233.0	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	230.0	286.0	-248.0	-304.0	N(Clay)	N(Clay)	123	0.15	6,020
	286.0	304.0	-304.0	-322.0	N(Sand)	N(Sand)	128	0.30	11,645
	304.0	315.0	-322.0	-333.0	N(Clay)	N(Clay)	123	0.15	6,020
	315.0	341.0	-333.0	-359.0	N(Sand)	N(Sand)	128	0.30	11,645
	341.0	351.0	-359.0	-369.0	N(Clay)	N(Clay)	123	0.15	6,020
	351.0	371.0	-369.0	-389.0	N(Sand)	N(Sand)	128	0.30	11,645
	371.0	456.0	-389.0	-474.0	N(Clay)	N(Clay)	123	0.15	6,020
	456.0	491.0	-474.0	-509.0	N(Sand)	N(Sand)	128	0.30	11,645
	491.0	552.0	-509.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	552.0	628.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	628.0	728.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	728.0	828.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	828.0	948.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	948.0	1248.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1248.0	1878.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1878.0	2448.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-40D Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Tunnels (Page 1 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	ϕ (°)	c (ksf)	ϕ (°)		
3	Assume Failure Along West or East Edge of Mat Alternate Water Table at el. -23.0 ft Short Term Conditions c= S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-18.0	-	-	0.00	36.0	17.0	16.7
		Concrete Fill	2.0	-21.0	-	-				
		Structural Fill	16.7	-23.0	0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40D Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Tunnels (Page 2 of 8)

<table>
<tr><th colspan="11">Average Properties within the Foundation Deformation Zone for Bearing Capacity</th></tr>
<tr><th rowspan="3">STP</th><th rowspan="3">Soil Selection Preference</th><th rowspan="3">Soil Layer</th><th rowspan="3">Thickness of Layer Below Mat (feet)</th><th rowspan="3">Top Elevation [1] (feet)</th><th colspan="4">Shear Strength</th><th rowspan="3">Foundation Width, B (feet)</th><th rowspan="3">Effective Shear Depth, H' (feet)</th></tr>
<tr><th colspan="2">Layer</th><th colspan="2">Average</th></tr>
<tr><th>c (ksf)</th><th>ϕ (°)</th><th>c (ksf)</th><th>ϕ (°)</th></tr>
<tr><td rowspan="3">3</td><td rowspan="3">Assume Failure Along West or East Edge of Mat

Design Water Table at el. +17.0 ft

Short Term Conditions

c= S_u for Clays

Backfill to el. +34.0 ft</td><td>Mat Foundation</td><td>-</td><td>-18.0</td><td>-</td><td>-</td><td rowspan="3">0.00</td><td rowspan="3">36.0</td><td rowspan="3">17.0</td><td rowspan="3">16.7</td></tr>
<tr><td>Concrete Fill</td><td>2.0</td><td>-21.0</td><td>-</td><td>-</td></tr>
<tr><td>Structural Fill</td><td>16.7</td><td>-23.0</td><td>0</td><td>36</td></tr>
</table>

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40D Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Tunnels (Page 3 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	ϕ (°)	c (ksf)	ϕ (°)		
3	Assume Failure Along West or East Edge of Mat Alternate Water Table at el. -23.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-18.0	-	-	0.00	36.0	17.0	16.7
		Concrete Fill	2.0	-21.0	-	-				
		Structural Fill	16.7	-23.0	0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40D Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Tunnels (Page 4 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	ϕ (°)	c (ksf)	ϕ (°)		
3	Assume Failure Along West or East Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-18.0	-	-	0.0	36.0	17.0	16.7
		Concrete Fill	2.0	-21.0	-	-				
		Structural Fill	16.7	-23.0	0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40D Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Tunnels (Page 5 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4	Assume Failure Along West or East Edge of Mat Alternate Water Table at el. -23.0 ft Short Term Conditions c= S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-18.0	-	-	0.0	36.0	17.0	16.7
		Concrete Fill	2.0	-21.0	-	-				
		Structural Fill	16.7	-23.0	0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40D Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Tunnels (Page 6 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4	Assume Failure Along West or East Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions c= S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-18.0	-	-	0.0	36.0	17.0	16.7
		Concrete Fill	2.0	-21.0	-	-				
		Structural Fill	16.7	-23.0	0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40D Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Tunnels (Page 7 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	ϕ (°)	c (ksf)	ϕ (°)		
4	Assume Failure Along West or East Edge of Mat Alternate Water Table at el. -23.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-18.0	-	-	0.0	36.0	17.0	16.7
		Concrete Fill	2.0	-21.0	-	-				
		Structural Fill	16.7	-23.0	0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40D Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Tunnels (Page 8 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4	Assume Failure Along West or East Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-18.0	-	-	0.0	36.0	17.0	16.7
		Concrete Fill	2.0	-21.0	-	-				
		Structural Fill	16.7	-23.0	0	36				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40E Subsurface Conditions for Settlement Analysis for the STP 3 & 4 RSW Pump Houses (Page 1 of 2)

Soil Properties									
	Average Subsurface Conditions					Properties for Settlement Calculation			
	Depth (feet)		Elevation [1] (feet)						
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
3	0	10.0	-22.0	-32.0	Mat Foundation	Mat Foundation	-	-	-
	10.0	12.0	-32.0	-34.0	Concrete Fill	Concrete Fill	-	-	-
	12.0	18.0	-34.0	-40.0	D(Clay)	D(Clay)	122	0.15	1,865
	18.0	39.9	-40.0	-61.9	E(Sand)	E(Sand)	123	0.30	3,145
	39.9	62.7	-61.9	-84.7	F(Clay)	F(Clay)	125	0.15	1,970
	62.7	116.0	-84.7	-138.0	J(Clay)	J(Clay)	125	0.15	3,175
	116.0	126.0	-138.0	-148.0	J(Sand)	J(Sand)	125	0.30	4,755
	126.0	152.7	-148.0	-174.7	J(Clay)	J(Clay)	125	0.15	3,175
	152.7	172.7	-174.7	-194.7	K(Clay)	K(Clay)	124	0.15	3,335
	172.7	197.9	-194.7	-219.9	K(Sand)	K(Sand)	127	0.30	4,915
	197.9	216.7	-219.9	-238.7	L(Clay)	L(Clay)	124	0.15	2,965
	216.7	226.0	-238.7	-248.0	M(Sand)	M(Sand)	127	0.30	4,350
	226.0	288.0	-248.0	-310.0	N(Clay)	N(Clay)	123	0.15	6,020
	288.0	304.0	-310.0	-326.0	N(Sand)	N(Sand)	128	0.30	11,645
	304.0	309.0	-326.0	-331.0	N(Clay)	N(Clay)	123	0.15	6,020
	309.0	348.0	-331.0	-370.0	N(Sand)	N(Sand)	128	0.30	11,645
	348.0	355.0	-370.0	-377.0	N(Clay)	N(Clay)	123	0.15	6,020
	355.0	372.0	-377.0	-394.0	N(Sand)	N(Sand)	128	0.30	11,645
	372.0	397.0	-394.0	-419.0	N(Clay)	N(Clay)	123	0.15	6,020
	397.0	413.0	-419.0	-435.0	N(Sand)	N(Sand)	128	0.30	11,645
	413.0	548.0	-435.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	548.0	624.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	624.0	724.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	724.0	824.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	824.0	944.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	944.0	1244.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1244.0	1874.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1874.0	2444.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-40E Subsurface Conditions for Settlement Analysis for the STP 3 & 4 RSW Pump Houses (Page 2 of 2)

Soil Properties										
	Average Subsurface Conditions					Properties for Settlement Calculation				
	Depth (feet)		Elevation [1] (feet)							
STP	Top	Bottom	Top	Bottom	Stratum	Stratum	Unit Weight, γ (pcf)	Poisson's Ratio μ_d	Elastic Modulus E_d (ksf)
4	0	10.0	-22.0	-32.0	Mat Foundation	Mat Foundation	-	-	-
	10.0	12.0	-32.0	-34.0	Concrete Fill	Concrete Fill	-	-	-
	12.0	35.6	-34.0	-57.6	E(Sand)	E(Sand)	123	0.30	3,145
	35.6	63.1	-57.6	-85.1	F(Clay)	F(Clay)	125	0.15	1,970
	63.1	93.1	-85.1	-115.1	J(Clay)	J(Clay)	125	0.15	3,175
	93.1	119.1	-115.1	-141.1	J(Sand)	J(Sand)	125	0.30	4,755
	119.1	148.1	-141.1	-170.1	J(Clay)	J(Clay)	125	0.15	3,175
	148.1	173.1	-170.1	-195.1	K(Clay)	K(Clay)	124	0.15	3,335
	173.1	188.1	-195.1	-210.1	L(Clay)	L(Clay)	124	0.15	2,965
	188.1	233.6	-210.1	-255.6	M(Sand)	M(Sand)	127	0.30	4,350
	233.6	282.0	-255.6	-304.0	N(Clay)	N(Clay)	123	0.15	6,020
	282.0	300.0	-304.0	-322.0	N(Sand)	N(Sand)	128	0.30	11,645
	300.0	311.0	-322.0	-333.0	N(Clay)	N(Clay)	123	0.15	6,020
	311.0	337.0	-333.0	-359.0	N(Sand)	N(Sand)	128	0.30	11,645
	337.0	347.0	-359.0	-369.0	N(Clay)	N(Clay)	123	0.15	6,020
	347.0	367.0	-369.0	-389.0	N(Sand)	N(Sand)	128	0.30	11,645
	367.0	452.0	-389.0	-474.0	N(Clay)	N(Clay)	123	0.15	6,020
	452.0	487.0	-474.0	-509.0	N(Sand)	N(Sand)	128	0.30	11,645
	487.0	548.0	-509.0	-570.0	N(Clay)	N(Clay)	123	0.15	6,020
	548.0	624.0	-570.0	-646.0	Deep Silt/Clay Vs=1585	Silt/Clay	129	0.15	11,190
	624.0	724.0	-646.0	-746.0	Deep Silty Sand Vs=1585	Silty Sand	126	0.30	12,780
	724.0	824.0	-746.0	-846.0	Deep Silt/Clay Vs=1585	Silt/Clay	130	0.15	11,275
	824.0	944.0	-846.0	-966.0	Deep Silty Sand Vs=1585	Silty Sand	130	0.30	13,185
	944.0	1244.0	-966.0	-1266.0	Deep Silty Sand Vs=2350	Silty Sand	130	0.30	28,985
	1244.0	1874.0	-1266.0	-1896.0	Deep Interbedded Vs=2550	Claystone/Siltstone*	130	0.30	34,130
	1874.0	2444.0	-1896.0	-2466.0	Deep Interbedded Vs=2850	Claystone/Sand/Silt	135	0.30	44,270

[1] Elevations are referenced to NGVD 29 datum
*/Clay/Sand/Silt

Table 2.5S.4-40F Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Pump Houses (Page 1 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	ϕ (°)	c (ksf)	ϕ (°)		
3	Assume Failure Along North Edge of Mat Alternate Water Table at el. -34.0 ft Short Term Conditions c= S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-22.0	-	-	2.19	14.2	94.0	60.4
		Concrete Fill	2.0	-32.0	-	-				
		D(Clay)	6.0	-34.0	3.0	0				
		E(Sand)	21.9	-40.0	0.0	35				
		F(Clay)	22.8	-61.9	3.4	0				
		J(Clay)	9.7	-84.7	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40F Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Pump Houses (Page 2 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	Assume Failure Along North Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions c= S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-22.0	-	-	2.19	14.2	94.0	60.4
		Concrete Fill	2.0	-32.0	-	-				
		D(Clay)	6.0	-34.0	3.0	0				
		E(Sand)	21.9	-40.0	0.0	35				
		F(Clay)	22.8	-61.9	3.4	0				
		J(Clay)	9.7	-84.7	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40F Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Pump Houses (Page 3 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	ϕ (°)	c (ksf)	ϕ (°)		
3	Assume Failure Along North Edge of Mat Alternate Water Table at el. -34.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-22.0	-	-	1.34	19.4	94.0	66.3
		Concrete Fill	2.0	-32.0	-	-				
		D(Clay)	6.0	-34.0	1.2	16				
		E(Sand)	21.9	-40.0	0.0	35				
		F(Clay)	22.8	-61.9	2.0	8				
		J(Clay)	15.6	-84.7	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40F Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Pump Houses (Page 4 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
3	Assume Failure Along North Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-22.0	-	-	1.34	19.4	94.0	66.3
		Concrete Fill	2.0	-32.0	-	-				
		D(Clay)	6.0	-34.0	1.2	16				
		E(Sand)	21.9	-40.0	0.0	35				
		F(Clay)	22.8	-61.9	2.0	8				
		J(Clay)	15.6	-84.7	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40F Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Pump Houses (Page 5 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	ϕ (°)	c (ksf)	ϕ (°)		
4	Assume Failure Along North Edge of Mat Alternate Water Table at el. -34.0 ft Short Term Conditions c= S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-22.0	-	-	2.16	15.1	94.0	61.3
		Concrete Fill	2.0	-32.0	-	-				
		E(Sand)	23.6	-34.0	0.0	35				
		F(Clay)	27.5	-57.6	3.4	0				
		J(Clay)	10.2	-85.1	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40F Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Pump Houses (Page 6 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	Shear Strength				Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
					Layer		Average			
					c (ksf)	ϕ (°)	c (ksf)	ϕ (°)		
4	Assume Failure Along North Edge of Mat Design Water Table at el. +17.0 ft Short Term Conditions c= S_u for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-22.0	-	-	2.16	15.1	94.0	61.3
		Concrete Fill	2.0	-32.0	-	-				
		E(Sand)	23.6	-34.0	0.0	35				
		F(Clay)	27.5	-57.6	3.4	0				
		J(Clay)	10.2	-85.1	3.8	0				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40F Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Pump Houses (Page 7 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4	Assume Failure Along North Edge of Mat Alternate Water Table at el. -34.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-22.0	-	-	1.36	19.4	94.0	66.4
		Concrete Fill	2.0	-32.0	-	-				
		E(Sand)	23.6	-34.0	0.0	35				
		F(Clay)	27.5	-57.6	2.0	8				
		J(Clay)	15.3	-85.1	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-40F Subsurface Conditions for Bearing Capacity Analysis for the STP 3 & 4 RSW Pump Houses (Page 8 of 8)

Average Properties within the Foundation Deformation Zone for Bearing Capacity										
					Shear Strength					
					Layer		Average			
STP	Soil Selection Preference	Soil Layer	Thickness of Layer Below Mat (feet)	Top Elevation [1] (feet)	c (ksf)	ϕ (°)	c (ksf)	ϕ (°)	Foundation Width, B (feet)	Effective Shear Depth, H' (feet)
4	Assume Failure Along North Edge of Mat Design Water Table at el. +17.0 ft Long Term Conditions c' and ϕ' for Clays Backfill to el. +34.0 ft	Mat Foundation	-	-22.0	-	-	1.36	19.4	94.0	66.4
		Concrete Fill	2.0	-32.0	-	-				
		E(Sand)	23.6	-34.0	0.0	35				
		F(Clay)	27.5	-57.6	2.0	8				
		J(Clay)	15.3	-85.1	2.3	11				

[1] Elevations are referenced to NGVD 29 datum

Table 2.5S.4-41A Summary of Average Parameters for Bearing Capacity (Page 1 of 3)

Structure	Final Embedment, D (feet) To Bottom of Concrete Fill	Length, L (feet)	Width, B (feet)	B/L	STP	Soil Strength Selection	Embedment Df (feet)	c(ksf)	φ (°)	N_c	N_q	N_γ	S_c	S_q	S_γ	d_c	d_q	d_γ	R_b
Reactor Buildings	94.3	217.5	207.7	0.95	3	Short Term	94.3	3.40	3.6	6.08	1.38	0.04	1.22	1.06	0.62	1.18	1.05	1.00	0.62
						Short Term	94.3	3.40	3.6	6.08	1.38	0.04	1.22	1.06	0.62	1.18	1.05	1.00	0.62
						Long Term	94.3	2.08	12.8	9.68	3.19	0.74	1.31	1.21	0.62	1.18	1.12	1.00	0.62
						Long Term	94.3	2.08	12.8	9.68	3.19	0.74	1.31	1.21	0.62	1.18	1.12	1.00	0.62
					4	Short Term	94.3	2.56	11.7	9.15	2.90	0.59	1.30	1.19	0.62	1.18	1.12	1.00	0.62
						Short Term	94.3	2.56	11.7	9.15	2.90	0.59	1.30	1.19	0.62	1.18	1.12	1.00	0.62
						Long Term	94.3	1.62	17.7	12.89	5.12	1.97	1.38	1.259	0.62	1.18	1.14	1.00	0.62
						Long Term	94.3	1.62	17.7	12.89	5.12	1.97	1.38	1.259	0.62	1.18	1.14	1.00	0.62
Control Buildings	78.3	185.0	80.1	0.43	3	Short Term	78.3	1.07	26.7	23.43	12.79	8.89	1.24	1.19	0.83	1.39	1.30	1.00	0.72
						Short Term	78.3	1.07	26.7	23.43	12.79	8.89	1.24	1.19	0.83	1.39	1.30	1.00	0.72
						Long Term	78.3	0.71	28.6	26.98	15.70	12.01	1.25	1.21	0.83	1.39	1.29	1.00	0.72
						Long Term	78.3	0.71	28.6	26.98	15.70	12.01	1.25	1.21	0.83	1.39	1.29	1.00	0.72
					4	Short Term	78.3	2.04	16.6	12.02	4.57	1.59	1.16	1.12	0.83	1.39	1.30	1.00	0.72
						Short Term	78.3	2.04	16.6	12.02	4.57	1.59	1.16	1.12	0.83	1.39	1.30	1.00	0.72
						Long Term	78.3	1.29	20.5	15.34	6.74	3.23	1.19	1.15	0.83	1.39	1.31	1.00	0.72
						Long Term	78.3	1.29	20.5	15.34	6.74	3.23	1.19	1.15	0.83	1.39	1.31	1.00	0.72
UHS Basins	34.0	312.0	164.0	0.53	3	Short Term	34.0	1.17	24.1	19.45	9.70	5.84	1.26	1.21	0.79	1.08	1.06	1.00	0.64
						Short Term	34.0	1.17	24.1	19.45	9.70	5.84	1.26	1.21	0.79	1.08	1.06	1.00	0.64
						Long Term	34.0	0.68	26.8	23.65	12.96	9.08	1.29	1.24	0.79	1.08	1.06	1.00	0.64
						Long Term	34.0	0.68	26.8	23.65	12.96	9.08	1.29	1.24	0.79	1.08	1.06	1.00	0.64
					4	Short Term	34.0	2.30	12.5	9.55	3.12	0.71	1.17	1.11	0.79	1.08	1.06	1.00	0.64
						Short Term	34.0	2.30	12.5	9.55	3.12	0.71	1.17	1.11	0.79	1.08	1.06	1.00	0.64
						Long Term	34.0	1.26	19.6	14.45	6.14	2.74	1.22	1.18	0.79	1.08	1.07	1.00	0.64
						Long Term	34.0	1.26	19.6	14.45	6.14	2.74	1.22	1.18	0.79	1.08	1.07	1.00	0.64

Table 2.5S.4-41A Summary of Average Parameters for Bearing Capacity (Page 2 of 3)

Structure	Final Embedment, D (feet) To Bottom of Concrete Fill	Length, L (feet)	Width, B (feet)	B/L	STP	Soil Strength Selection	Embedment Df (feet)	c(ksf)	ϕ (°)	N_c	N_q	N_γ	S_c	S_q	S_γ	d_c	d_q	d_γ	R_b
RSW Pump Houses	68.0	170.0	94.0	0.55	3	Short Term	68.0	2.19	14.2	10.51	3.67	1.02	1.19	1.14	0.78	1.29	1.21	1.00	0.70
						Short Term	68.0	2.19	14.2	10.51	3.67	1.02	1.19	1.14	0.78	1.29	1.21	1.00	0.70
						Long Term	68.0	1.34	19.4	14.25	6.00	2.64	1.23	1.18	0.78	1.29	1.23	1.00	0.70
						Long Term	68.0	1.34	19.4	14.25	6.00	2.64	1.23	1.18	0.78	1.29	1.23	1.00	0.70
					4	Short Term	68.0	2.16	15.1	11.03	3.97	1.20	1.20	1.14	0.78	1.29	1.21	1.00	0.70
						Short Term	68.0	2.16	15.1	11.03	3.97	1.20	1.20	1.14	0.78	1.29	1.21	1.00	0.70
						Long Term	68.0	1.36	19.4	14.27	6.02	2.65	1.23	1.18	0.78	1.29	1.23	1.00	0.70
						Long Term	68.0	1.36	19.4	14.27	6.02	2.65	1.23	1.18	0.78	1.29	1.23	1.00	0.70
RSW Tunnels	57.0	560.0	17.0	0.03	3	Short Term	57.0	0.00	36.0	50.59	37.75	40.05	1.02	1.02	0.99	1.51	1.32	1.00	0.89
						Short Term	57.0	0.00	36.0	50.59	37.75	40.05	1.02	1.02	0.99	1.51	1.32	1.00	0.89
						Long Term	57.0	0.00	36.0	50.59	37.75	40.05	1.02	1.02	0.99	1.51	1.32	1.00	0.89
						Long Term	57.0	0.00	36.0	50.59	37.75	40.05	1.02	1.02	0.99	1.51	1.32	1.00	0.89
					4	Short Term	57.0	0.00	36.0	50.59	37.75	40.05	1.02	1.02	0.99	1.51	1.32	1.00	0.89
						Short Term	57.0	0.00	36.0	50.59	37.75	40.05	1.02	1.02	0.99	1.51	1.32	1.00	0.89
						Long Term	57.0	0.00	36.0	50.59	37.75	40.05	1.02	1.02	0.99	1.51	1.32	1.00	0.89
						Long Term	57.0	0.00	36.0	50.59	37.75	40.05	1.02	1.02	0.99	1.51	1.32	1.00	0.89
Diesel Generator Fuel Oil Storage Vaults (No. 1)	41.0	84.5	44.0	0.52	3	Short Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Short Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Long Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Long Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
					4	Short Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Short Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Long Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Long Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78

Table 2.5S.4-41A Summary of Average Parameters for Bearing Capacity (Page 3 of 3)

Structure	Final Embedment, D (feet) To Bottom of Concrete Fill	Length, L (feet)	Width, B (feet)	B/L	STP	Soil Strength Selection	Embedment Df (feet)	c(ksf)	ϕ (°)	N_c	N_q	N_γ	S_c	S_q	S_γ	d_c	d_q	d_γ	R_b
Diesel Generator Fuel Oil Storage Vaults (No. 2)	41.0	84.5	44.0	0.52	3	Short Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Short Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Long Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Long Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
					4	Short Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Short Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Long Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
						Long Term	41.0	0.00	36.0	50.59	37.75	40.05	1.39	1.31	0.79	1.37	1.23	1.00	0.78
Diesel Generator Fuel Oil Storage Vaults (No. 3)	41.0	84.5	44.0	0.52	3	Short Term	41.0	0.32	32.7	37.81	25.32	23.46	1.35	1.28	0.79	1.37	1.25	1.00	0.78
						Short Term	41.0	0.32	32.7	37.81	25.32	23.46	1.35	1.28	0.79	1.37	1.25	1.00	0.78
						Long Term	41.0	0.12	34.0	42.15	29.43	28.76	1.36	1.29	0.79	1.37	1.24	1.00	0.78
						Long Term	41.0	0.12	34.0	42.15	29.43	28.76	1.36	1.29	0.79	1.37	1.24	1.00	0.78
					4	Short Term	41.0	0.00	35.6	48.89	36.04	37.68	1.38	1.30	0.79	1.37	1.23	1.00	0.78
						Short Term	41.0	0.00	35.6	48.89	36.04	37.68	1.38	1.30	0.79	1.37	1.23	1.00	0.78
						Long Term	41.0	0.00	35.6	48.89	36.04	37.68	1.38	1.30	0.79	1.37	1.23	1.00	0.78
						Long Term	41.0	0.00	35.6	48.89	36.04	37.68	1.38	1.30	0.79	1.37	1.23	1.00	0.78

Table 2.5S.4-41B Bearing Capacity of Foundation (Page 1 of 2)

Structure	STP	Soil Strength Selection [1]	Ultimate Bearing Capacity at Base of Concrete Fill, q_{ULT}, (ksf)	Factor of Safety (FOS) [2]
Reactor Buildings	3	Short Term [3]	47.4	3.03
		Short Term [4]	41.5	3.78
		Long Term [3]	82.8	5.29
		Long Term [4]	66.2	6.03
	4	Short Term [3]	81.7	5.23
		Short Term [4]	66.9	6.10
		Long Term [3]	124.8	7.98
		Long Term [4]	96.1	8.75
Control Buildings	3	Short Term [3]	244.0	15.95
		Short Term [4]	184.0	16.03
		Long Term [3]	282.4	18.46
		Long Term [4]	208.4	18.16
	4	Short Term [3]	105.2	6.87
		Short Term [4]	85.0	7.41
		Long Term [3]	133.9	8.75
		Long Term [4]	103.1	8.99
UHS Basins	3	Short Term [3]	97.5	10.27
		Short Term [4]	87.0	10.31
		Long Term [3]	115.8	12.19
		Long Term [4]	101.5	12.03
	4	Short Term [3]	44.7	4.71
		Short Term [4]	41.6	4.93
		Long Term [3]	62.8	6.61
		Long Term [4]	56.4	6.68
RSW Pump Houses	3	Short Term [3]	78.5	12.46
		Short Term [4]	65.8	21.11
		Long Term [3]	106.2	16.86
		Long Term [4]	84.2	27.02
	4	Short Term [3]	84.1	13.35
		Short Term [4]	70.2	25.52
		Long Term [3]	107.0	16.98
		Long Term [4]	85.0	27.25
RSW Tunnels	3	Short Term [3]	370.2	71.47
		Short Term [4]	270.3	100.69
		Long Term [3]	370.2	71.47
		Long Term [4]	270.3	100.69
	4	Short Term [3]	370.2	71.47
		Short Term [4]	270.3	100.69
		Long Term [3]	370.2	71.47
		Long Term [4]	270.3	100.69
Diesel Generator Fuel Oil Storage Vaults (No. 1)	3	Short Term [3]	340.0	123.61
		Short Term [4]	268.1	207.61
		Long Term [3]	340.0	123.61
		Long Term [4]	268.1	207.61
	4	Short Term [3]	340.0	123.61
		Short Term [4]	268.1	207.61
		Long Term [3]	340.0	123.61
		Long Term [4]	268.1	207.61

Table 2.5S.4-41B Bearing Capacity of Foundation (Page 2 of 2)

Structure	STP	Soil Strength Selection [1]	Ultimate Bearing Capacity at Base of Concrete Fill, q_{ULT}, (ksf)	Factor of Safety (FOS) [2]
Diesel Generator Fuel Oil Storage Vaults (No. 2)	3	Short Term [3]	340.0	123.61
		Short Term [4]	268.1	207.61
		Long Term [3]	340.0	123.61
		Long Term [4]	268.1	207.61
	4	Short Term [3]	340.0	123.61
		Short Term [4]	268.1	207.61
		Long Term [3]	340.0	123.61
		Long Term [4]	268.1	207.61
Diesel Generator Fuel Oil Storage Vaults (No. 3)	3	Short Term [3]	245.8	89.34
		Short Term [4]	197.6	152.98
		Long Term [3]	271.0	98.50
		Long Term [4]	214.9	166.40
	4	Short Term [3]	322.0	117.07
		Short Term [4]	253.4	196.19
		Long Term [3]	322.0	117.07
		Long Term [4]	253.4	196.19

[1] Short term - undrained condition, Long term - drained condition

[2] See Section 2.5S.4.10.3 STP 3 & 4 Bearing Capacity Evaluation

[3] Water table at bottom of concrete fill

[4] Water table at +17.0 feet

Table 2.5S.4-41C Bearing Capacity of Foundations under Dynamic or Transient Loading

Structure	STP	Soil Strength Selection [1]	Ultimate Bearing Capacity at Base of Concrete Fill, q_{ULT}, (ksf)	Factor of Safety (FOS) [2]
Reactor Building	3	Short Term	49.4	2.35
	4	Short Term	94.7	4.55
Control Building	3	Short Term	432.3	6.01
	4	Short Term	86.4	1.73
UHS/RSW Pump House	3	Short Term	65.0	5.22
	4	Short Term	87.0	6.98
RSW Piping Tunnels	3	Short Term	235.3	44.74
	4	Short Term	180.1	34.24
Diesel Generator Fuel Oil Storage Vaults	3	Short Term	250.9	68.06
	4	Short Term	332.6	90.22

[1] Short term - undrained condition

[2] See Section 2.5S.4.10.3 STP 3 & 4 Bearing Capacity Evaluation

Table 2.5S.4-42 Estimated Foundation Settlements (Page 1 of 3)

Structure	Unit	Location	S_{ss} (in)	S_{os} (in)	S_{sf} (in)	S_{bf} (in)	S_c (in)	S (in)	Max†Differential Settlement (in)	Max†Angular Distortion (in/in)
Reactor Buildings	3	Center	7.13	1.46	1.82	0.19	0.26	10.86	1.84	1/600
		North Edge*	4.80	2.66	1.51	0.29	0.03	9.30		
		South Edge*	5.11	1.22	3.39	0.13	0.00	9.85		
		East Edge*	5.30	1.07	2.50	0.15	0.00	9.02		
		West Edge*	5.28	2.56	2.40	0.35	0.24	10.84		
		NE Corner	3.58	1.84	2.02	0.23	0.00	7.67		
		NW Corner	3.58	2.80	2.27	0.37	0.00	9.02		
		SE Corner	3.89	0.95	3.45	0.10	0.33	8.72		
		SW Corner	3.90	1.77	3.91	0.17	0.23	9.97		
	4	Center	6.72	1.43	1.78	0.19	0.02	10.14	1.51	1/750
		North Edge*	5.10	2.64	1.49	0.29	0.00	9.52		
		South Edge*	4.85	1.20	3.25	0.12	0.00	9.43		
		East Edge*	5.03	1.06	2.40	0.15	0.00	8.63		
		West Edge*	5.02	2.46	2.34	0.34	0.07	10.24		
		NE Corner	3.94	1.82	2.08	0.23	0.00	8.06		
		NW Corner	3.94	2.81	2.27	0.37	0.00	9.38		
		SE Corner	3.74	0.93	3.31	0.10	0.15	8.23		
		SW Corner	3.75	1.72	3.75	0.17	0.05	9.44		
Control Buildings	3	Center	2.54	3.44	1.40	0.39	0.00	7.77	1.76	1/450
		North Edge*	1.78	3.13	1.33	0.55	0.00	6.80		
		South Edge*	1.78	5.28	1.48	0.30	0.00	8.85		
		East Edge*	1.43	2.72	1.53	0.32	0.00	6.01		
		West Edge*	1.43	3.38	2.06	0.42	0.00	7.29		
		NE Corner	1.06	2.62	1.19	0.46	0.00	5.33		
		NW Corner	1.06	3.05	2.01	0.56	0.00	6.68		
		SE Corner	1.06	4.03	1.94	0.25	0.00	7.27		
		SW Corner	1.06	4.95	2.18	0.37	0.00	8.55		
	4	Center	3.01	3.51	1.40	0.40	0.00	8.32	1.97	1/400
		North Edge*	2.03	3.25	1.33	0.58	0.00	7.18		
		South Edge*	2.03	5.67	1.49	0.31	0.00	9.49		
		East Edge*	1.66	2.78	1.59	0.32	0.00	6.35		
		West Edge*	1.66	3.51	2.08	0.42	0.00	7.67		
		NE Corner	1.18	2.72	1.19	0.48	0.00	5.57		
		NW Corner	1.18	3.21	2.03	0.58	0.00	7.00		
		SE Corner	1.18	4.38	2.08	0.25	0.03	7.91		
		SW Corner	1.18	5.40	2.23	0.37	0.00	9.18		

Notes: S_{ss} = Settlement due to the loading of the structure itself S_{os} = Settlement due to the loading of other structures S_{sf} = Settlement due to the structural backfill other than placed below the footings of all structures S_{bf} = Settlement due to the structural backfill placed underneath the footings of all structures S_c = Settlement due to the consolidation of the clay layers for load exceeding the preconsolidation pressure S = Total settlement

† Differential settlement and the angular distortion (tilt) are with respect to the center and the midpoint of an edge of the structures

* At the mid point of the edge

Table 2.5S.4-42 Estimated Foundation Settlements (Page 2 of 3)

Structure	Unit	Location	S_{ss} (in)	S_{os} (in)	S_{sf} (in)	S_{bf} (in)	S_c (in)	S (in)	Max†Differential Settlement (in)	Max†Angular Distortion (in/in)
UHS Basins	3	Center	5.86	0.68	1.58	0.03	0.00	8.15	2.15	1/700
		North Edge*	3.72	1.77	2.08	0.04	0.00	7.61		
		South Edge*	3.72	0.41	2.54	0.02	0.00	6.69		
		East Edge*	3.27	0.43	2.26	0.03	0.00	5.99		
		West Edge*	3.27	0.65	2.15	0.03	0.00	6.10		
		NE Corner	2.16	0.59	2.39	0.03	0.00	5.18		
		NW Corner	2.16	1.49	2.80	0.04	0.00	6.49		
		SE Corner	2.16	0.31	2.57	0.02	0.00	5.07		
		SW Corner	2.16	0.40	2.41	0.02	0.00	5.00		
	4	Center	6.11	0.69	1.63	0.03	0.00	8.46	2.26	1/650
		North Edge*	3.88	1.84	2.17	0.04	0.00	7.92		
		South Edge*	3.88	0.41	2.63	0.02	0.00	6.94		
		East Edge*	3.41	0.43	2.33	0.03	0.00	6.20		
		West Edge*	3.41	0.66	2.21	0.03	0.00	6.31		
		NE Corner	2.26	0.60	2.47	0.03	0.00	5.35		
		NW Corner	2.26	1.55	2.89	0.04	0.00	6.73		
		SE Corner	2.26	0.31	2.66	0.02	0.00	5.25		
		SW Corner	2.26	0.40	2.49	0.02	0.00	5.17		
RSW Pump Houses	3	Center	2.31	2.43	2.37	0.05	0.00	7.17	0.48	1/1750
		North Edge*	1.52	2.23	3.53	0.06	0.00	7.33		
		South Edge*	1.52	3.41	1.87	0.04	0.00	6.84		
		East Edge*	1.34	2.38	2.94	0.05	0.00	6.71		
		West Edge*	1.34	1.83	3.48	0.05	0.00	6.69		
		NE Corner	0.92	2.00	3.37	0.05	0.00	6.35		
		NW Corner	0.92	1.68	4.43	0.06	0.00	7.10		
		SE Corner	0.92	3.53	2.18	0.04	0.00	6.67		
		SW Corner	0.92	2.35	2.61	0.04	0.00	5.92		
	4	Center	2.21	2.38	2.32	0.05	0.00	6.95	0.49	1/1700
		North Edge*	1.45	2.17	3.39	0.06	0.00	7.08		
		South Edge*	1.45	3.30	1.83	0.04	0.00	6.62		
		East Edge*	1.28	2.33	2.81	0.04	0.00	6.46		
		West Edge*	1.28	1.79	3.34	0.05	0.00	6.46		
		NE Corner	0.89	1.95	3.23	0.05	0.00	6.12		
		NW Corner	0.89	1.64	4.28	0.06	0.00	6.87		
		SE Corner	0.89	3.40	2.10	0.04	0.00	6.43		
		SW Corner	0.89	2.27	2.52	0.04	0.00	5.72		
RSW Tunnels	3	Center	0.66	5.29	3.23	0.22	2.37	11.78	4.98	1/700
		North Edge*	0.37	3.63	2.07	0.73	0.00	6.80		
		South Edge*	0.38	4.35	2.33	0.05	0.00	7.11		
	4	Center	0.68	5.53	3.36	0.22	2.15	11.95	4.97	1/700
		North Edge*	0.38	3.79	2.13	0.77	0.00	7.07		
		South Edge*	0.37	4.26	2.31	0.05	0.00	6.98		

Notes: S_{ss} = Settlement due to the loading of the structure itself S_{os} = Settlement due to the loading of other structures S_{sf} = Settlement due to the structural backfill other than placed below the footings of all structures S_{bf} = Settlement due to the structural backfill placed underneath the footings of all structures S_c = Settlement due to the consolidation of the clay layers for load exceeding the preconsolidation pressure S = Total settlement

† Differential settlement and the angular distortion (tilt) are with respect to the center and the midpoint of an edge of the structures

* At the mid point of the edge

Table 2.5S.4-42 Estimated Foundation Settlements (Page 3 of 3)

Structure	Unit	Location	S_{ss} (in)	S_{os} (in)	S_{sf} (in)	S_{bf} (in)	S_c (in)	S (in)	Max†Differential Settlement (in)	Max†Angular Distortion (in/in)
Diesel Generator Fuel Oil Storage Vault No. 1	3	Center	0.37	2.58	4.51	0.10	0.32	7.87	-0.47	1/1000
		North Edge*	0.26	3.09	4.19	0.12	0.69	8.35		
		South Edge*	0.26	2.53	4.52	0.08	0.25	7.63		
		East Edge*	0.29	2.54	4.47	0.09	0.31	7.70		
		West Edge*	0.29	2.68	4.54	0.10	0.31	7.91		
	4	Center	0.35	2.51	4.20	0.09	0.05	7.20	-0.46	1/1050
		North Edge*	0.24	2.96	3.87	0.12	0.47	7.66		
		South Edge*	0.25	2.43	4.20	0.08	0.00	6.95		
		East Edge*	0.28	2.47	4.15	0.09	0.04	7.04		
		West Edge*	0.28	2.60	4.22	0.10	0.04	7.23		
Diesel Generator Fuel Oil Storage Vault No. 2	3	Center	0.38	1.94	3.86	0.07	0.00	6.25	0.49	1/500
		North Edge*	0.27	2.36	3.59	0.08	0.00	6.30		
		South Edge*	0.27	1.77	3.90	0.06	0.00	6.00		
		East Edge*	0.30	1.80	3.59	0.07	0.00	5.76		
		West Edge*	0.30	2.07	4.05	0.07	0.00	6.49		
	4	Center	0.36	1.85	3.51	0.07	0.00	5.79	0.45	1/550
		North Edge*	0.25	2.22	3.25	0.08	0.00	5.80		
		South Edge*	0.25	1.70	3.55	0.06	0.00	5.56		
		East Edge*	0.28	1.73	3.27	0.07	0.00	5.34		
		West Edge*	0.28	1.97	3.67	0.07	0.00	6.00		
Diesel Generator Fuel Oil Storage Vault No. 3	3	Center	0.37	2.61	3.64	0.06	0.01	6.69	0.38	1/650
		North Edge*	0.29	2.47	4.20	0.07	0.03	7.05		
		South Edge*	0.29	3.00	3.22	0.06	0.00	6.57		
		East Edge*	0.26	2.44	3.55	0.06	0.00	6.30		
		West Edge*	0.26	2.79	3.78	0.07	0.13	7.02		
	4	Center	0.36	2.60	3.62	0.06	0.00	6.65	0.38	1/750
		North Edge*	0.29	2.45	4.17	0.07	0.00	6.97		
		South Edge*	0.29	3.02	3.21	0.06	0.00	6.58		
		East Edge*	0.25	2.42	3.54	0.06	0.00	6.28		
		West Edge*	0.25	2.78	3.77	0.07	0.00	6.87		

Notes: S_{ss} = Settlement due to the loading of the structure itself S_{os} = Settlement due to the loading of other structures S_{sf} = Settlement due to the structural backfill other than placed below the footings of all structures S_{bf} = Settlement due to the structural backfill placed underneath the footings of all structures S_c = Settlement due to the consolidation of the clay layers for load exceeding the preconsolidation pressure S = Total settlement

† Differential settlement and the angular distortion (tilt) are with respect to the center and the midpoint of an edge of the structures

* At the mid point of the edge

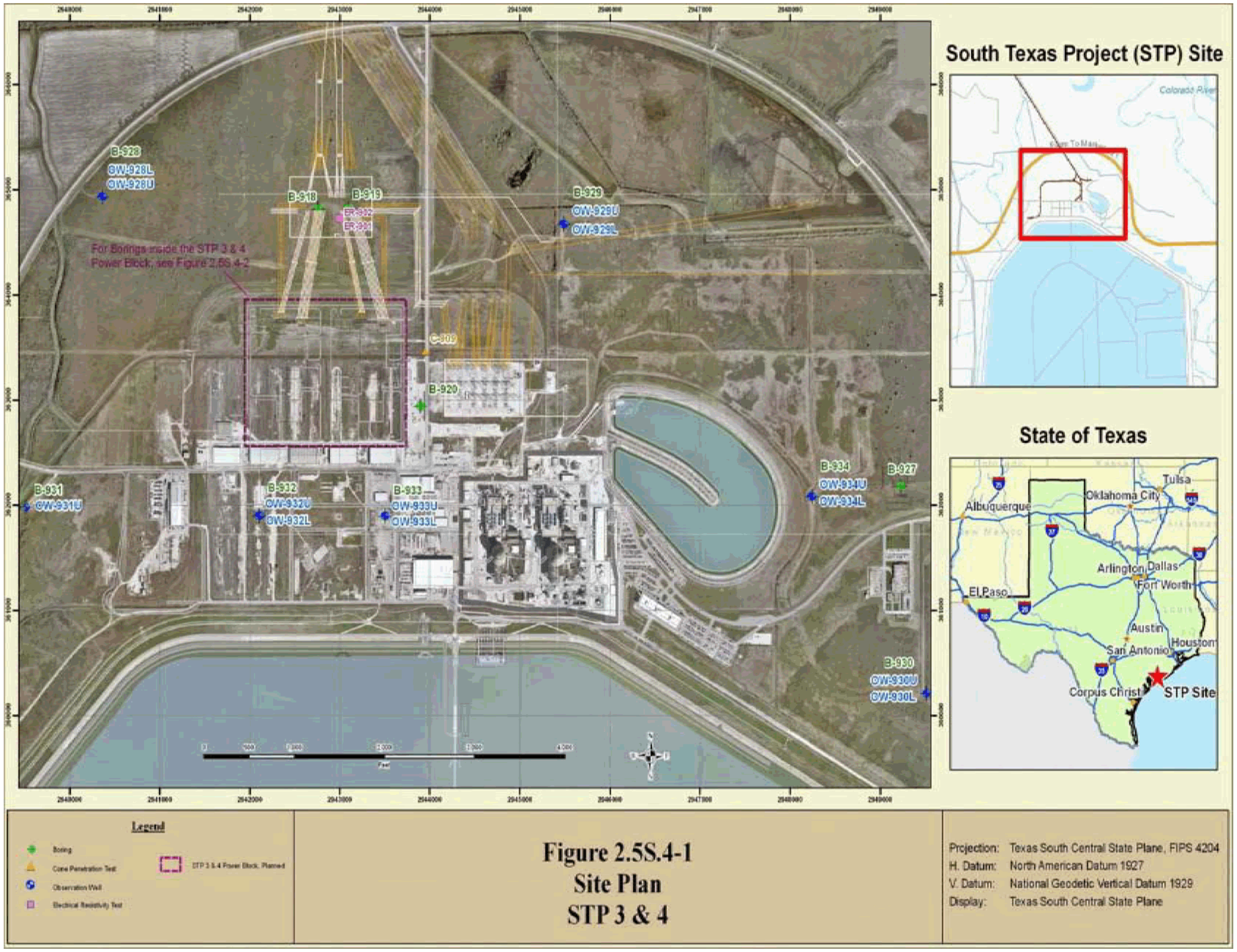


Figure 2.5S.4-1 Site Plan STP 3 & 4

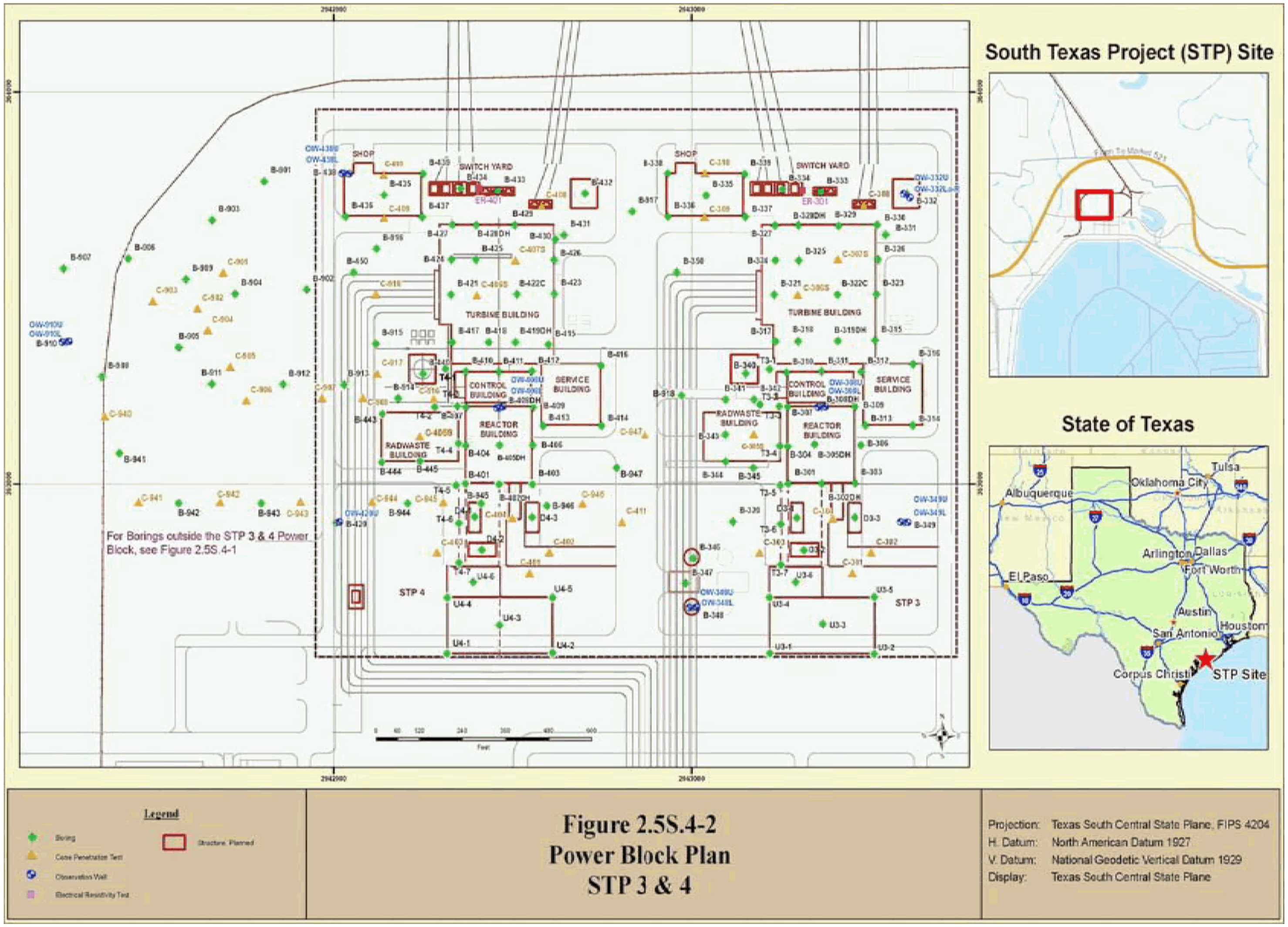
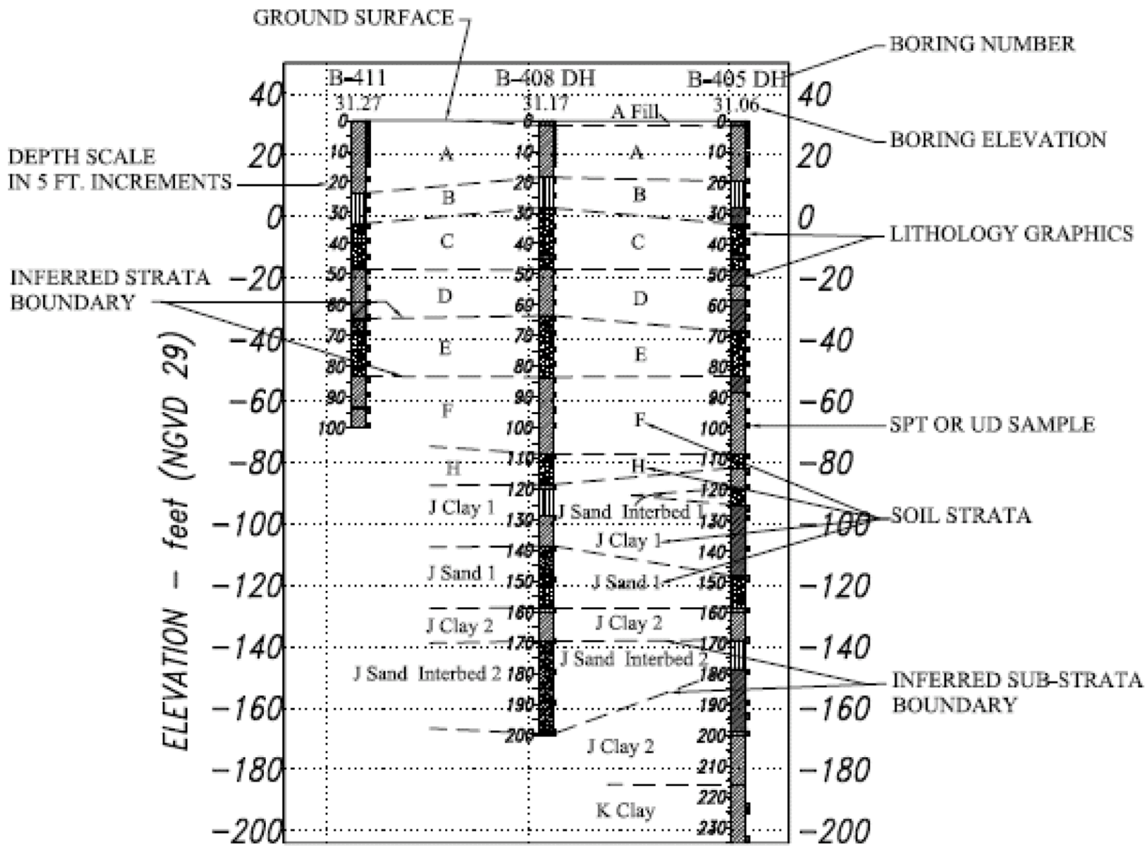


Figure 2.5S.4-2 Power Block Plan STP 3 & 4



NOTES:

- INTERFACES BETWEEN STRATA ARE APPROXIMATE.
- TRANSITIIONS BETWEEN STRATA MAY BE GRADUAL.

Figure 2.5S.4-3 Subsurface Profile Legend

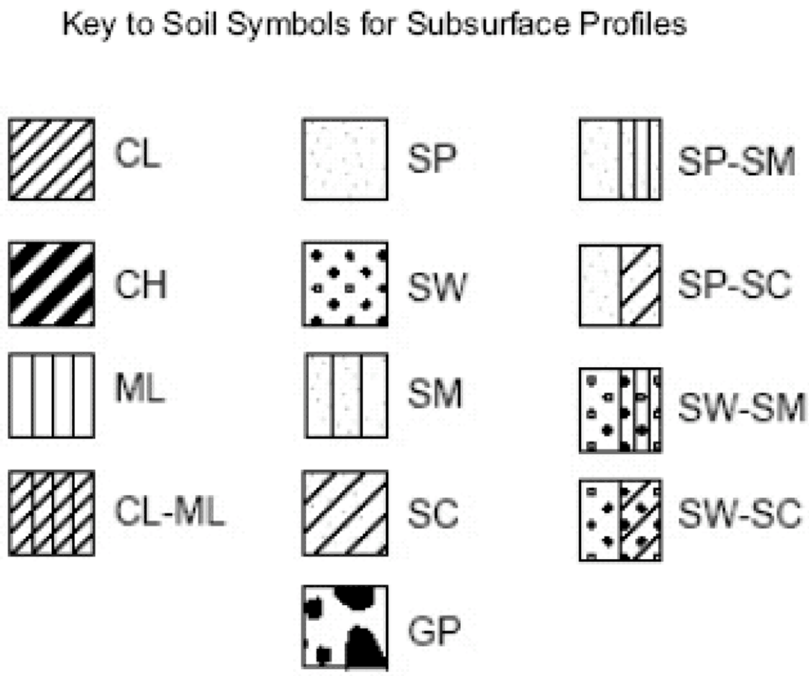


Figure 2.5S.4-3 Subsurface Profile Legend (Continued)

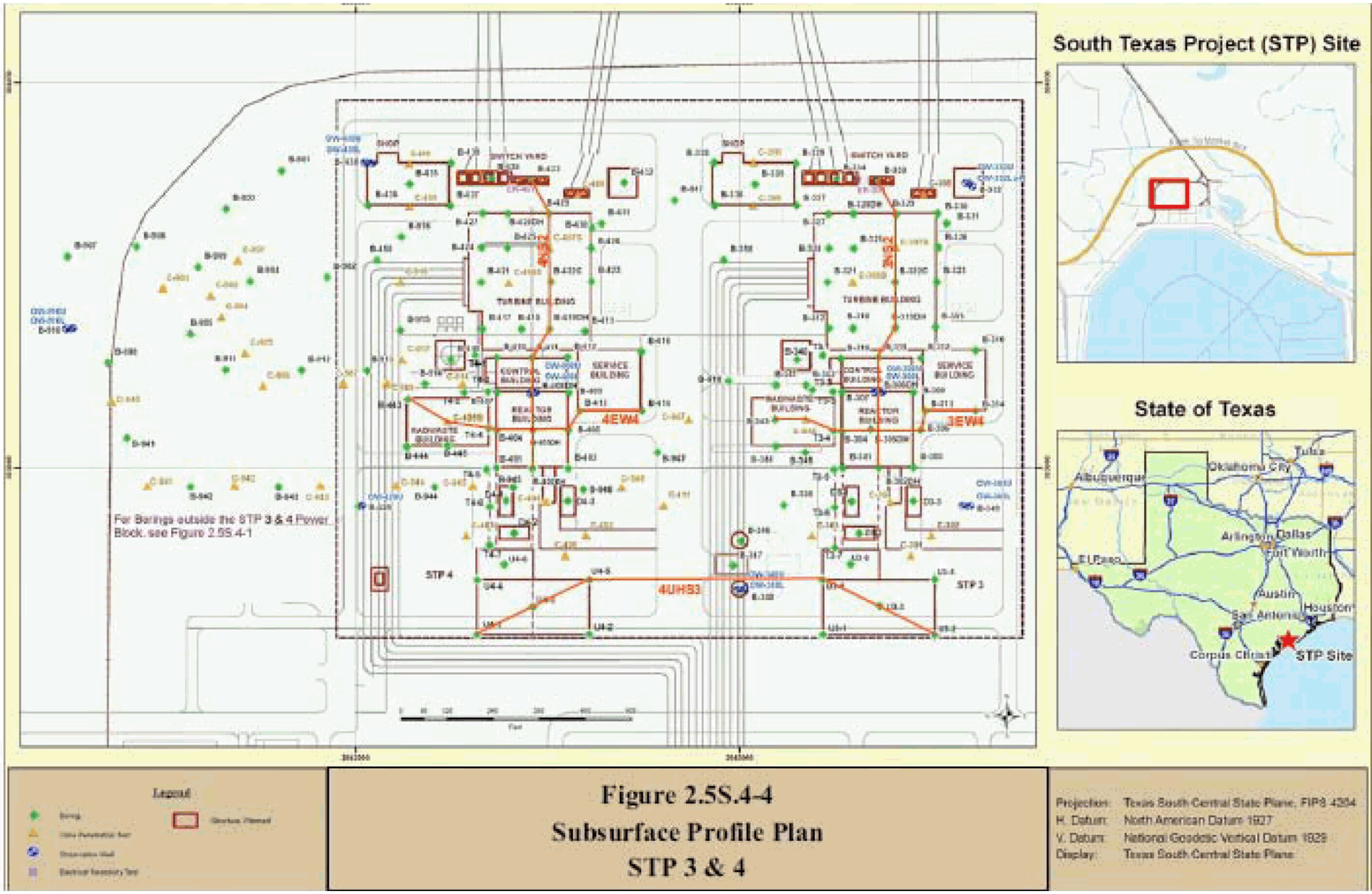


Figure 2.5S.4-4 Subsurface Profile Plan

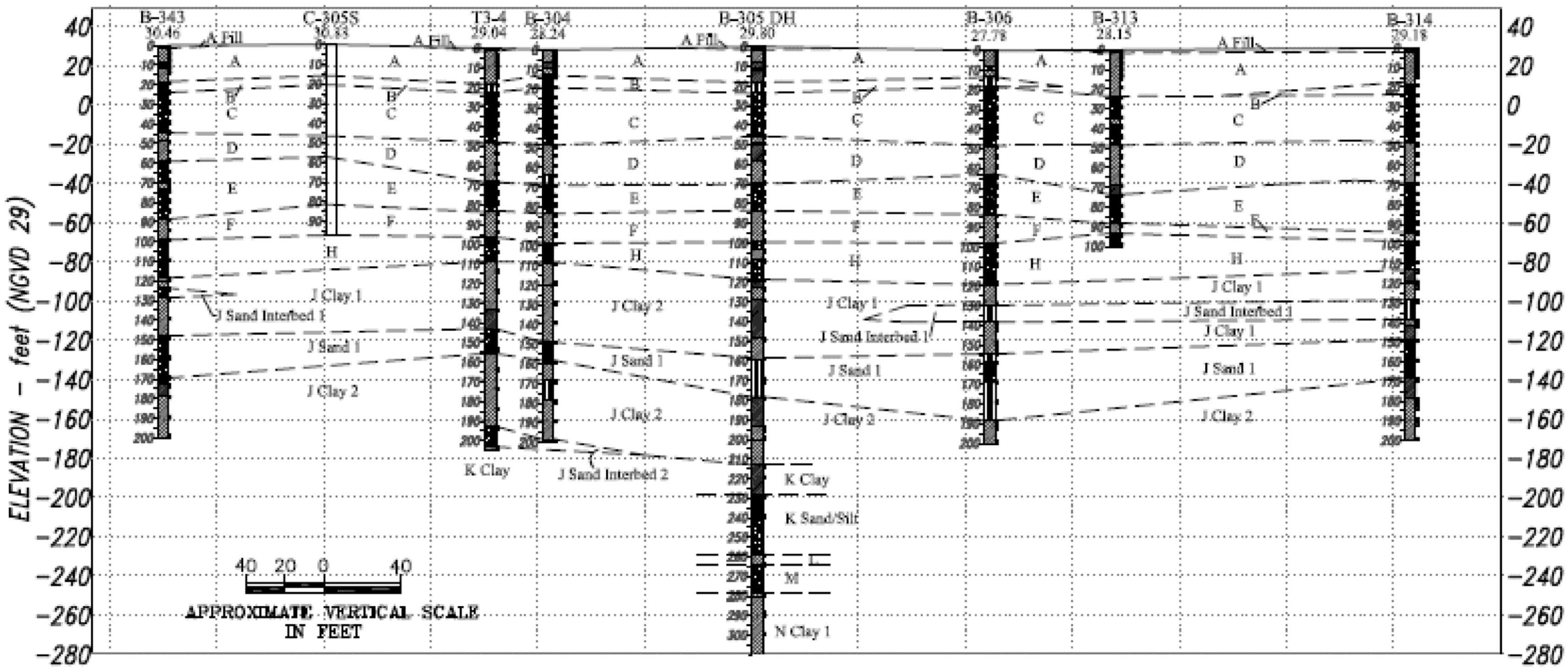


Figure 2.5S.4-5 Subsurface Profile 3EW4

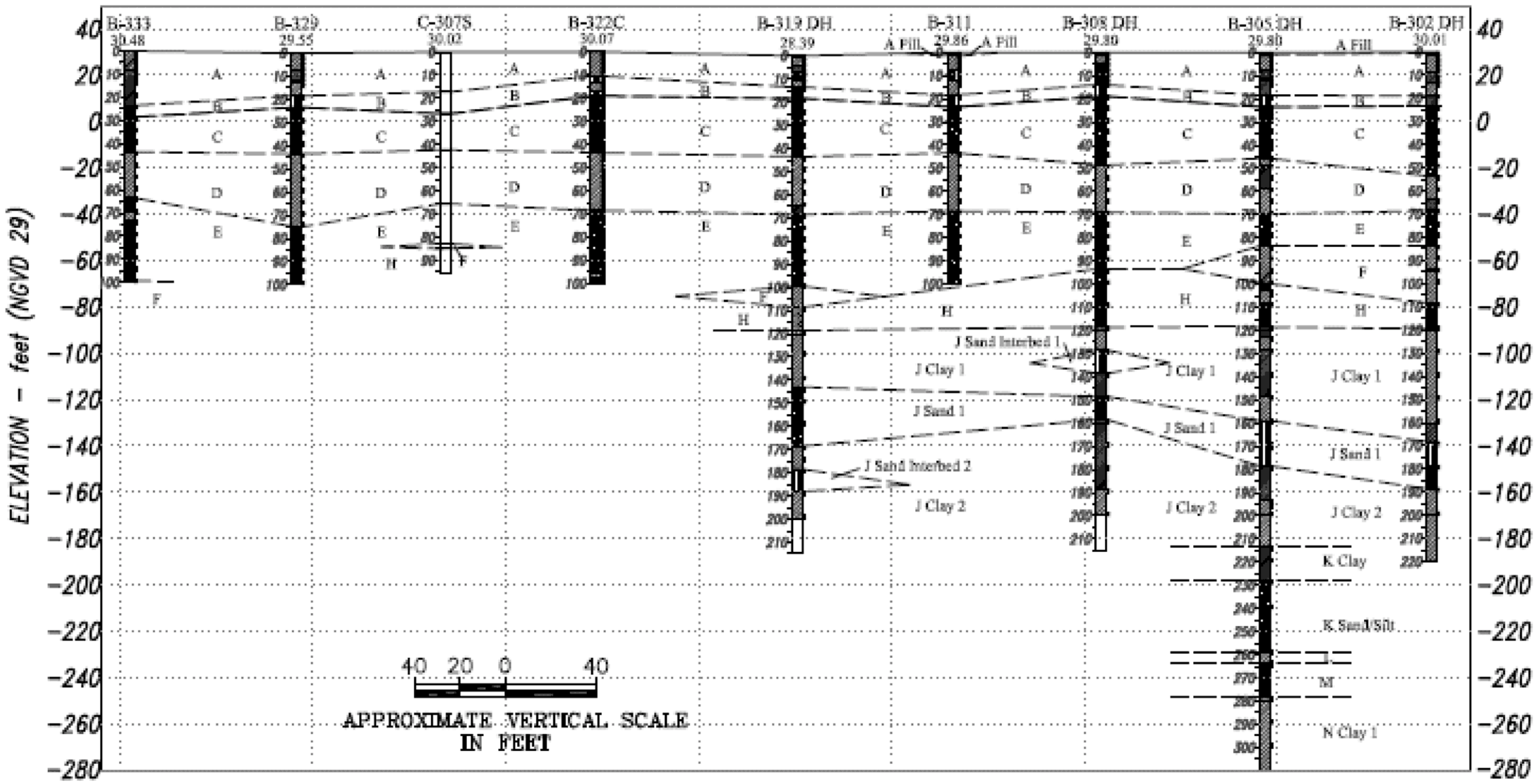


Figure 2.5S.4-6 Subsurface Profile 3NS2

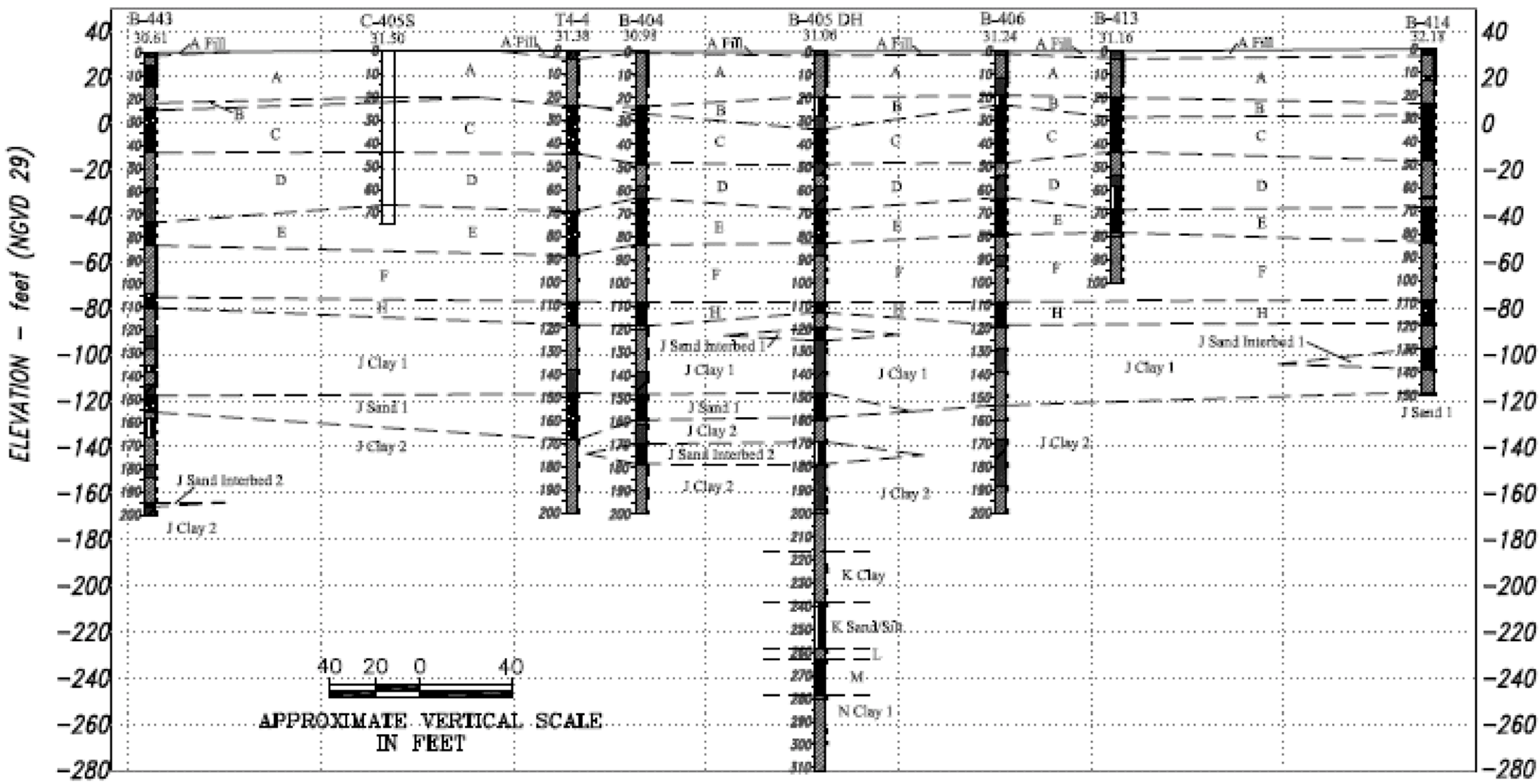


Figure 2.5S.4-7 Subsurface Profile 4EW4

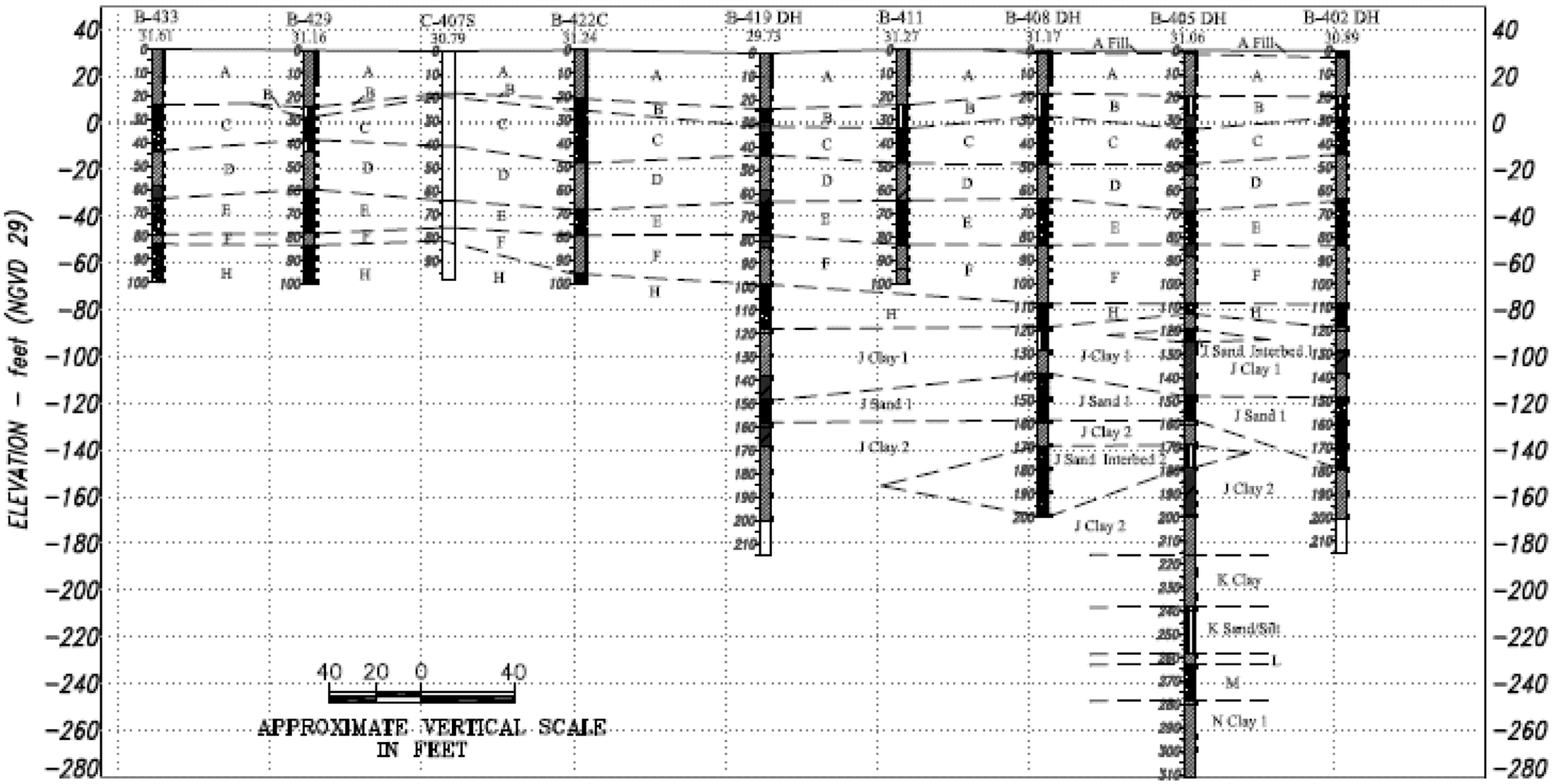


Figure 2.5S.4-8 Subsurface Profile 4NS2

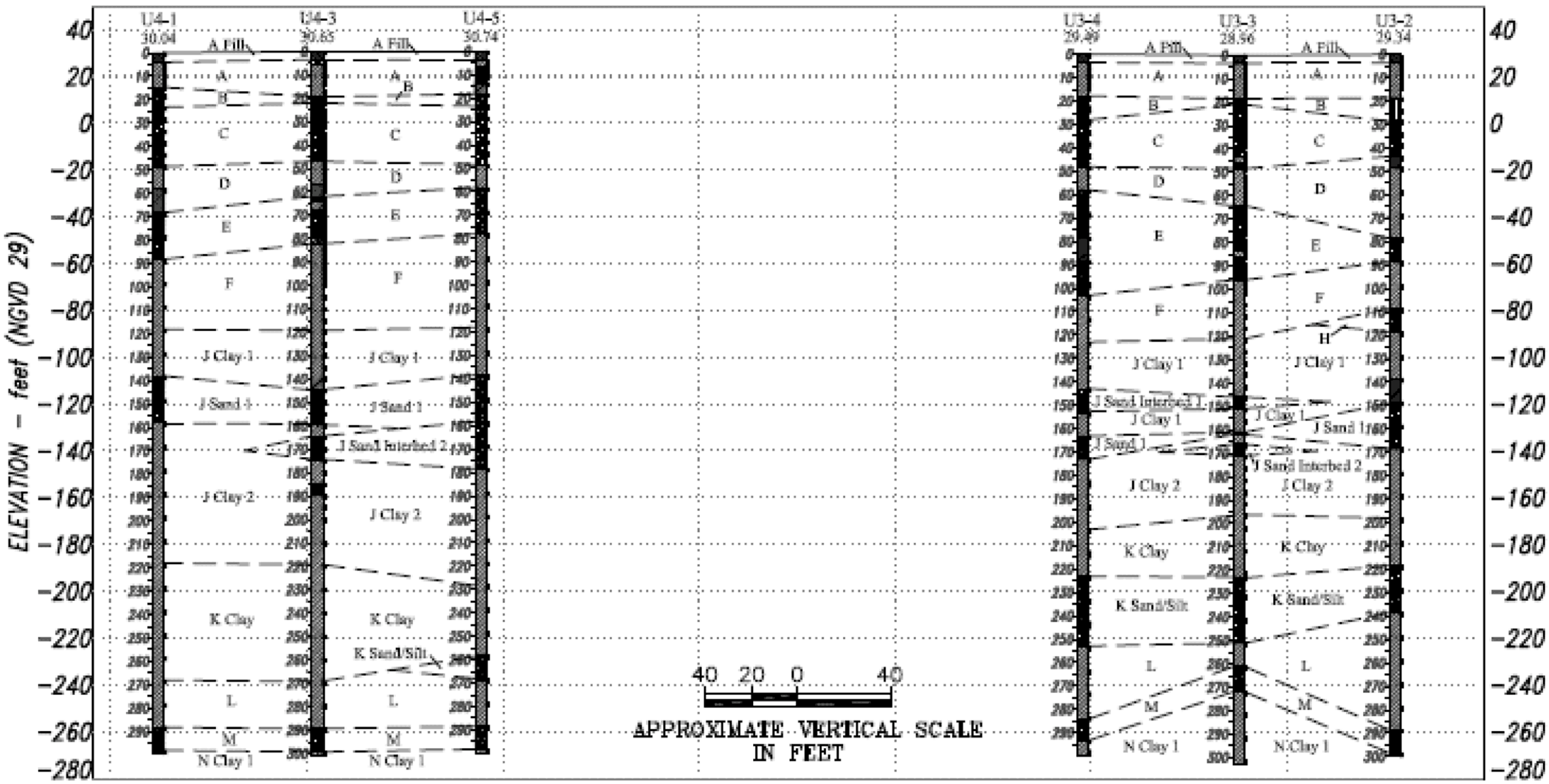


Figure 2.5S.4-9 Subsurface Profile 4UHS3

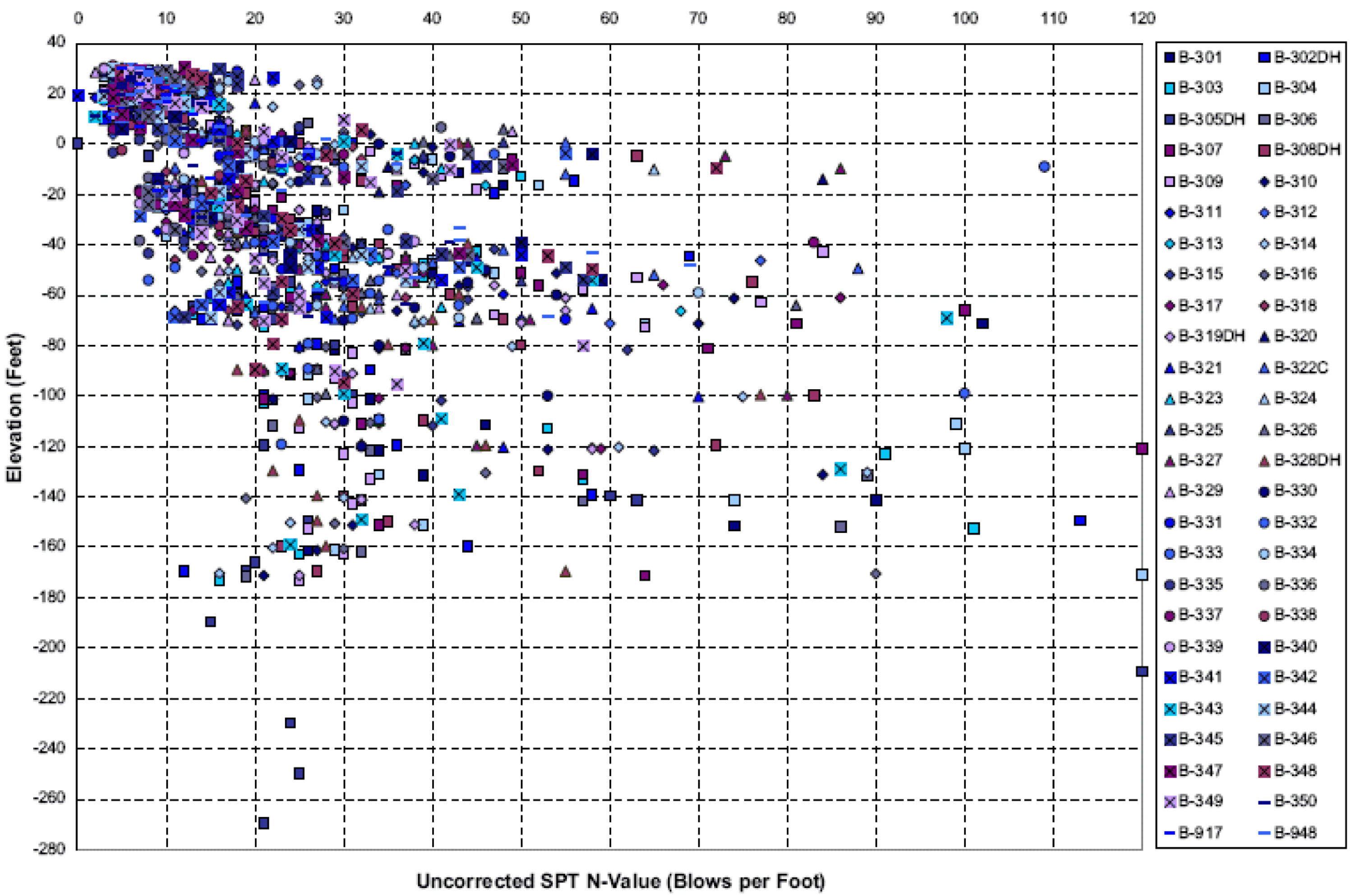


Figure 2.5S.4-10 Uncorrected SPT N-Values (STP 3) <Includes B-917>

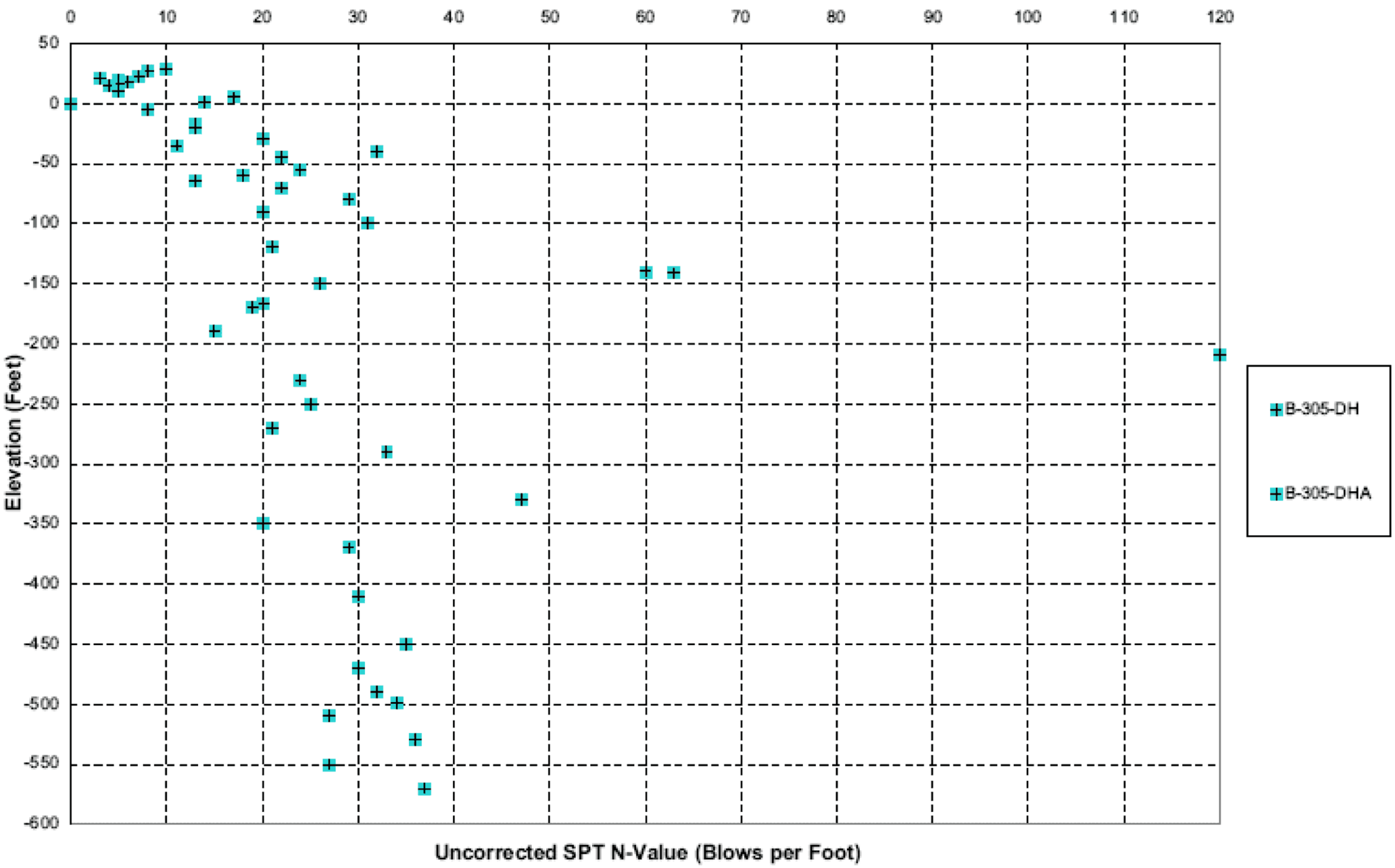


Figure 2.5S.4-11 Uncorrected SPT N-Values (STP 3; Boring B-305DH/DHA)

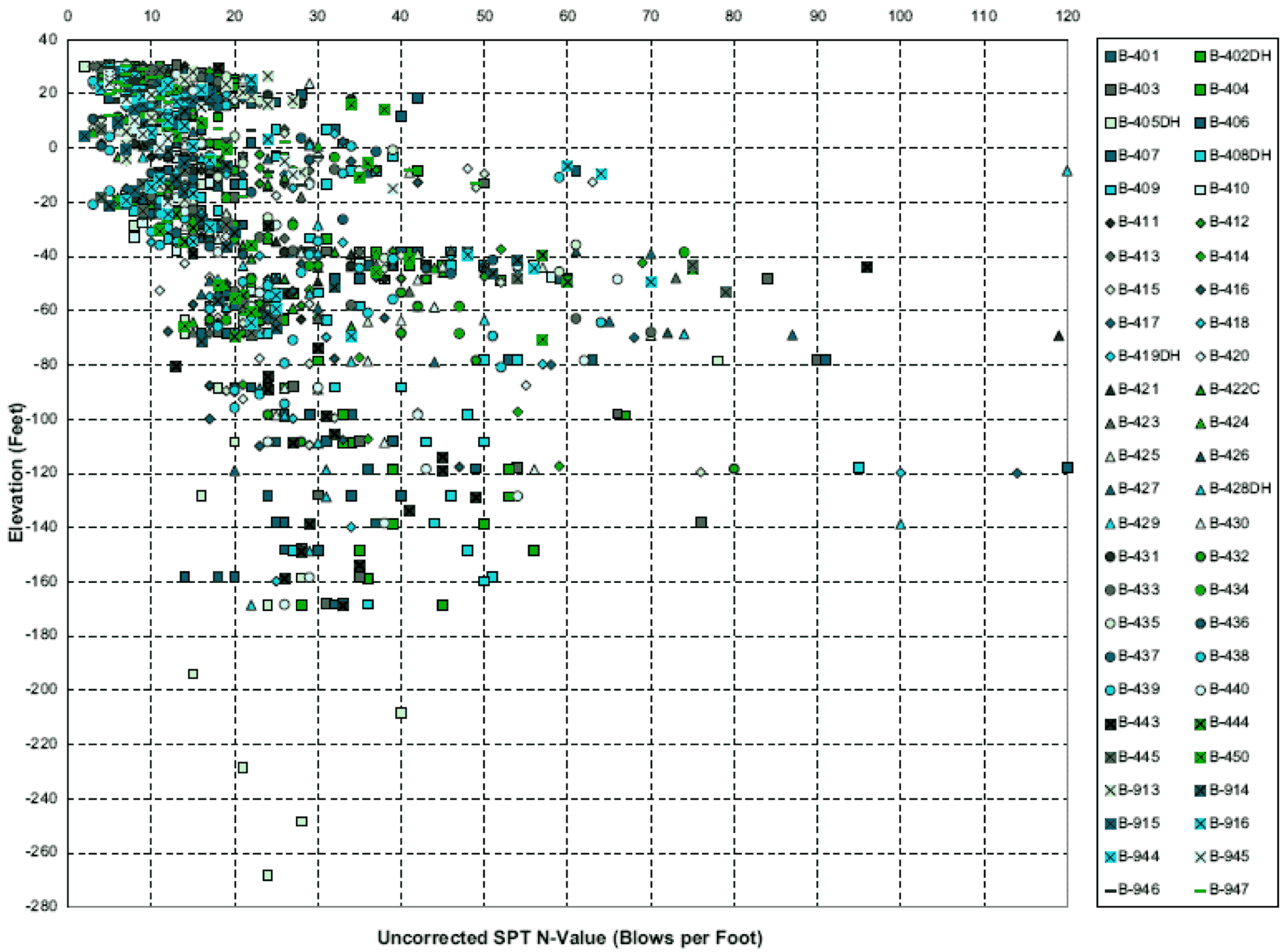


Figure 2.5S.4-12 Uncorrected SPT N-Values (STP 4)

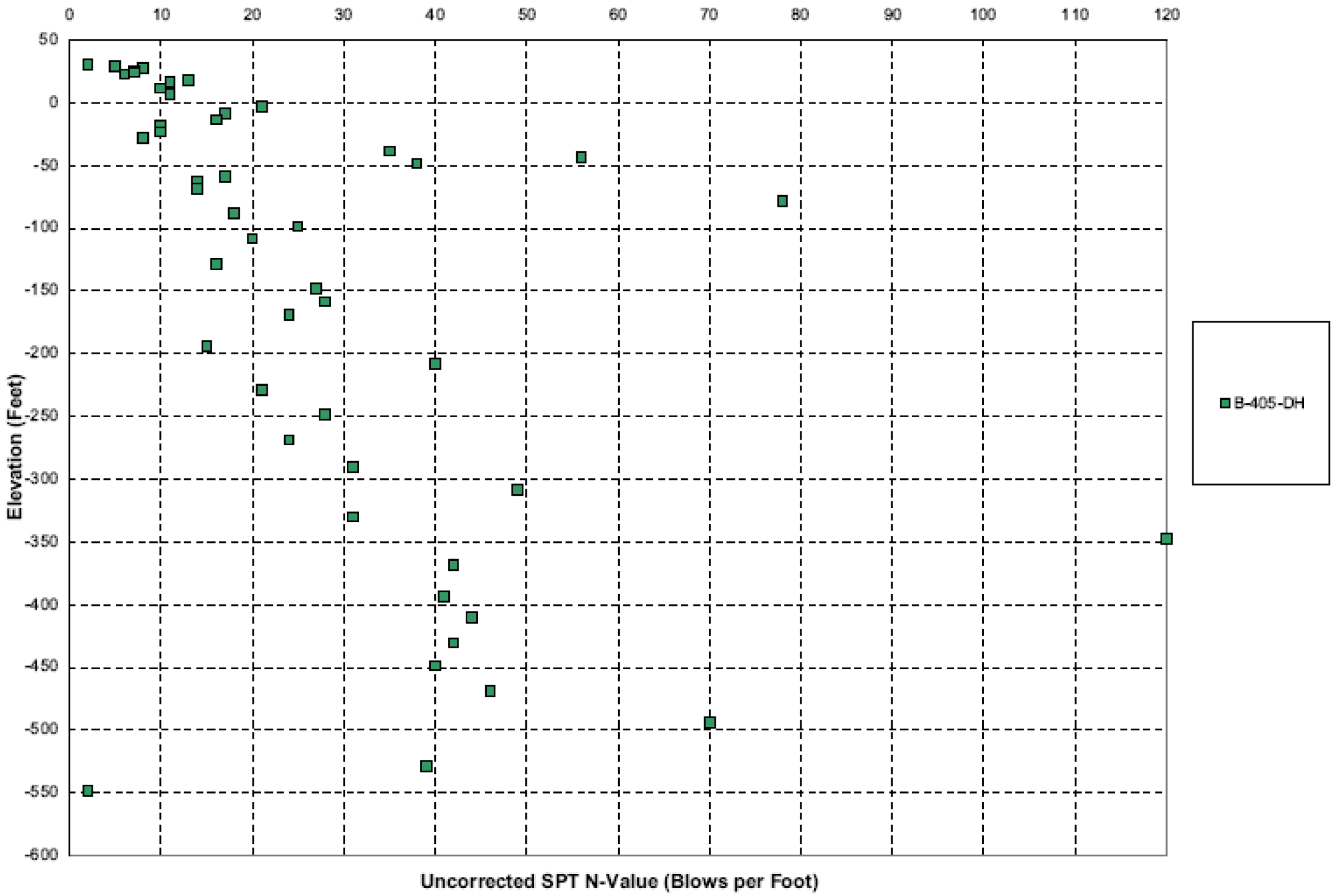


Figure 2.5S.4-13 Uncorrected SPT N-Values (STP 4; Boring B-405DH)

Figure 2.5S.4-14 Not Used
(The data has been included in Figure 2.5S.4-15)

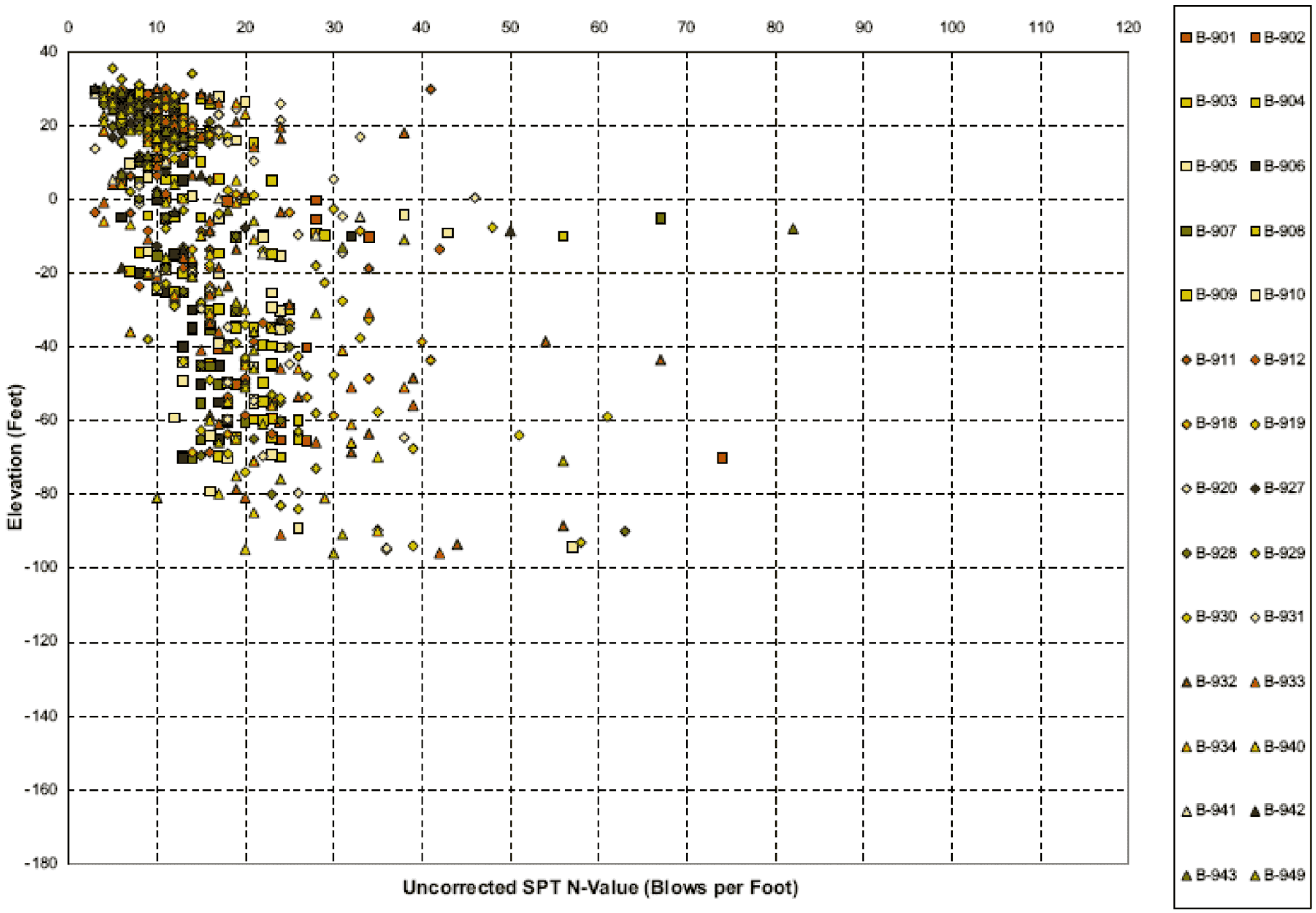


Figure 2.5S.4-15 Uncorrected SPT N-Values (Outside Power Block)

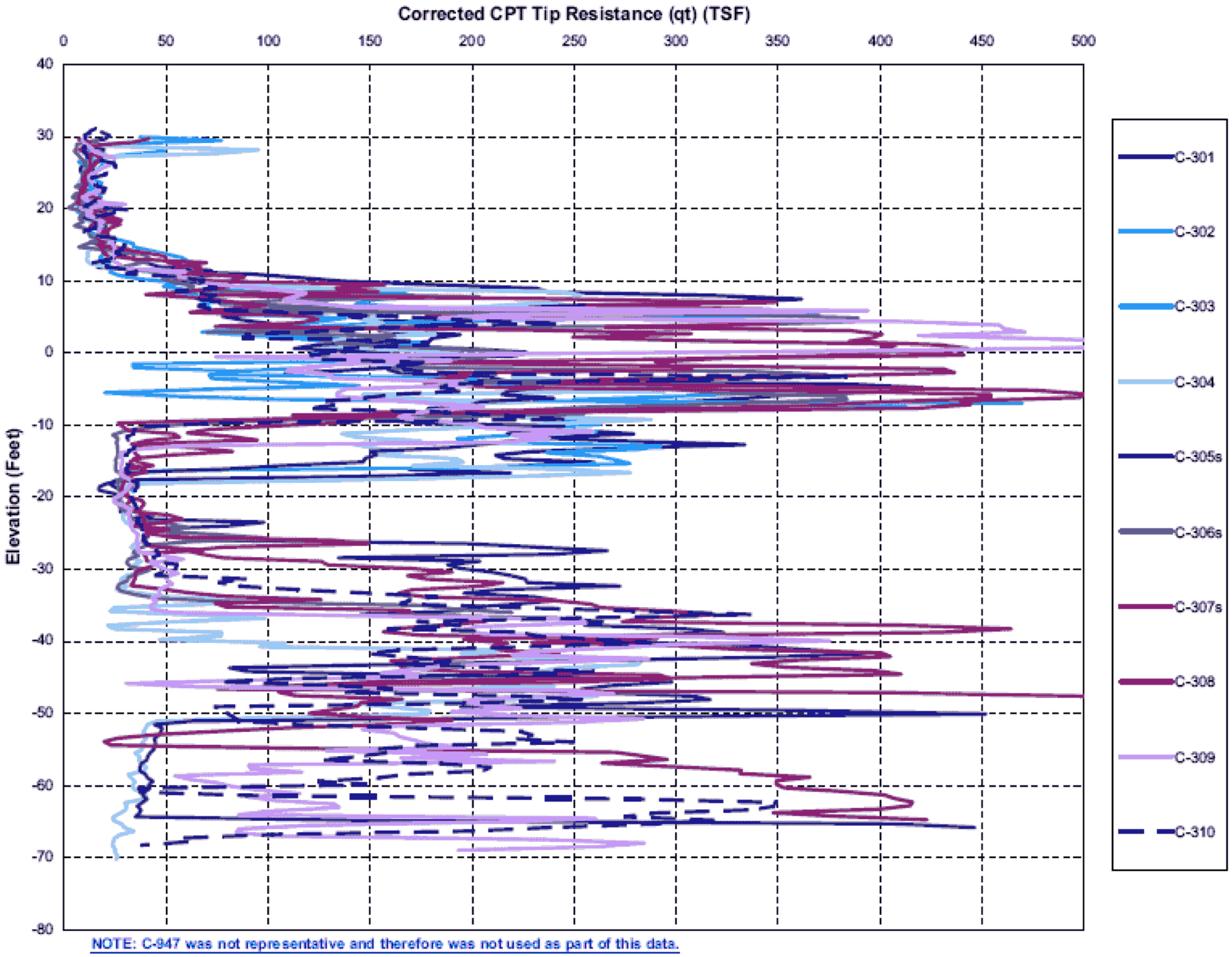


Figure 2.5S.4-16 Corrected CPT Tip Resistance (qt) (STP 3)

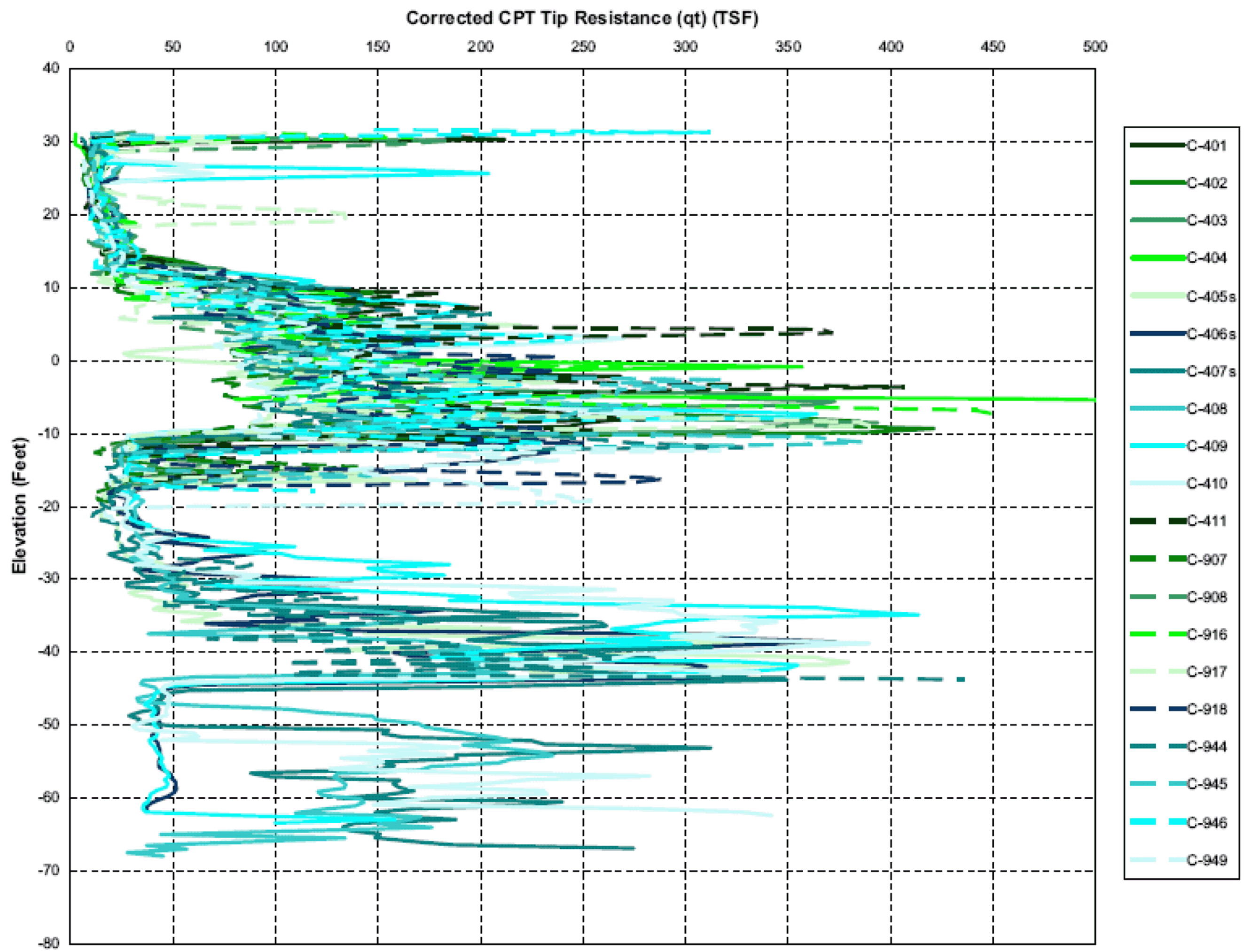


Figure 2.5S.4-17 Corrected CPT Tip Resistance (qt) (STP 4) <Includes C-916>

Figure 2.5S.4-18 Not Used
(The data has been included in Figure 2.5S.4-19)

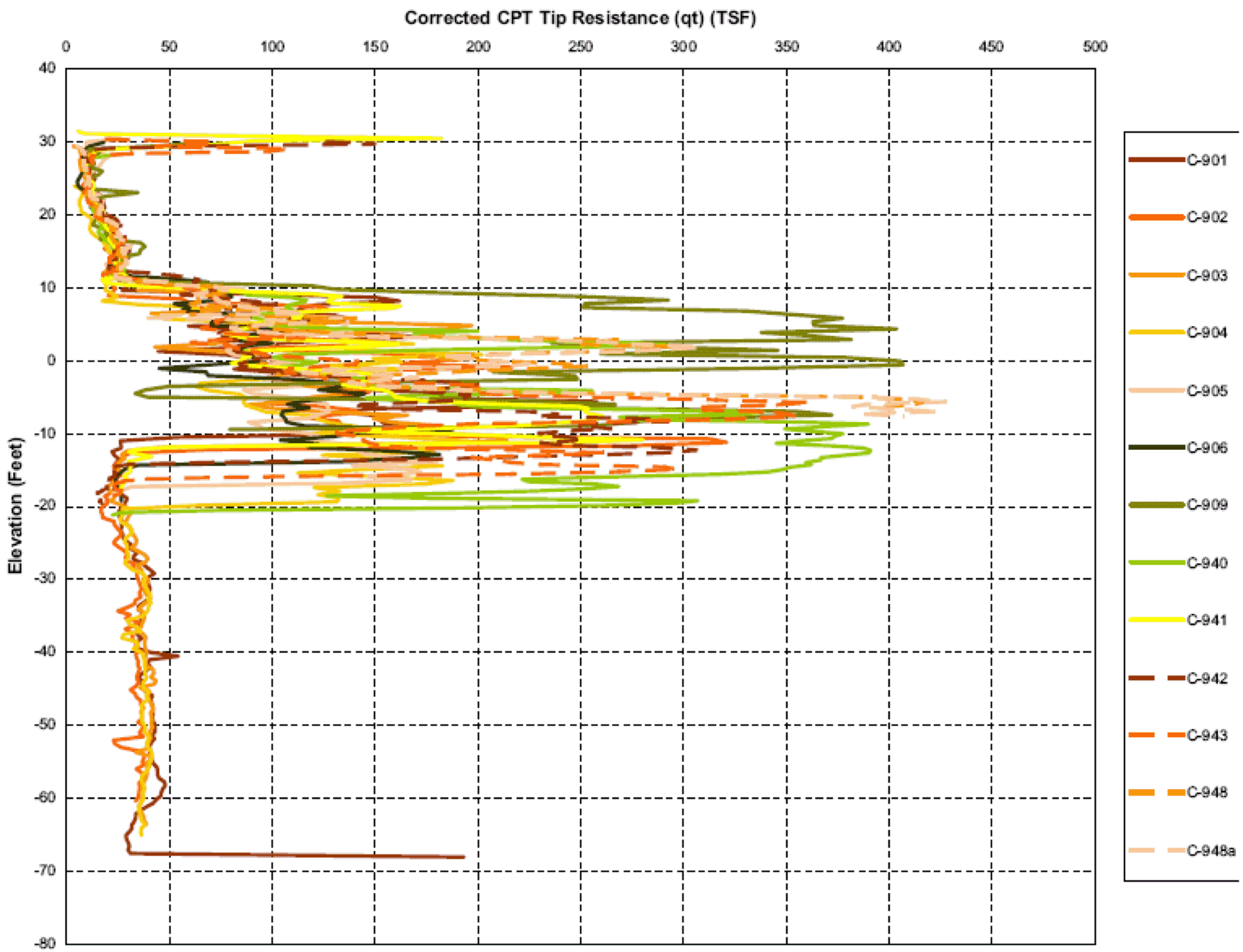


Figure 2.5S.4-19 Corrected CPT Tip Resistance (qt) (Outside Power Block)

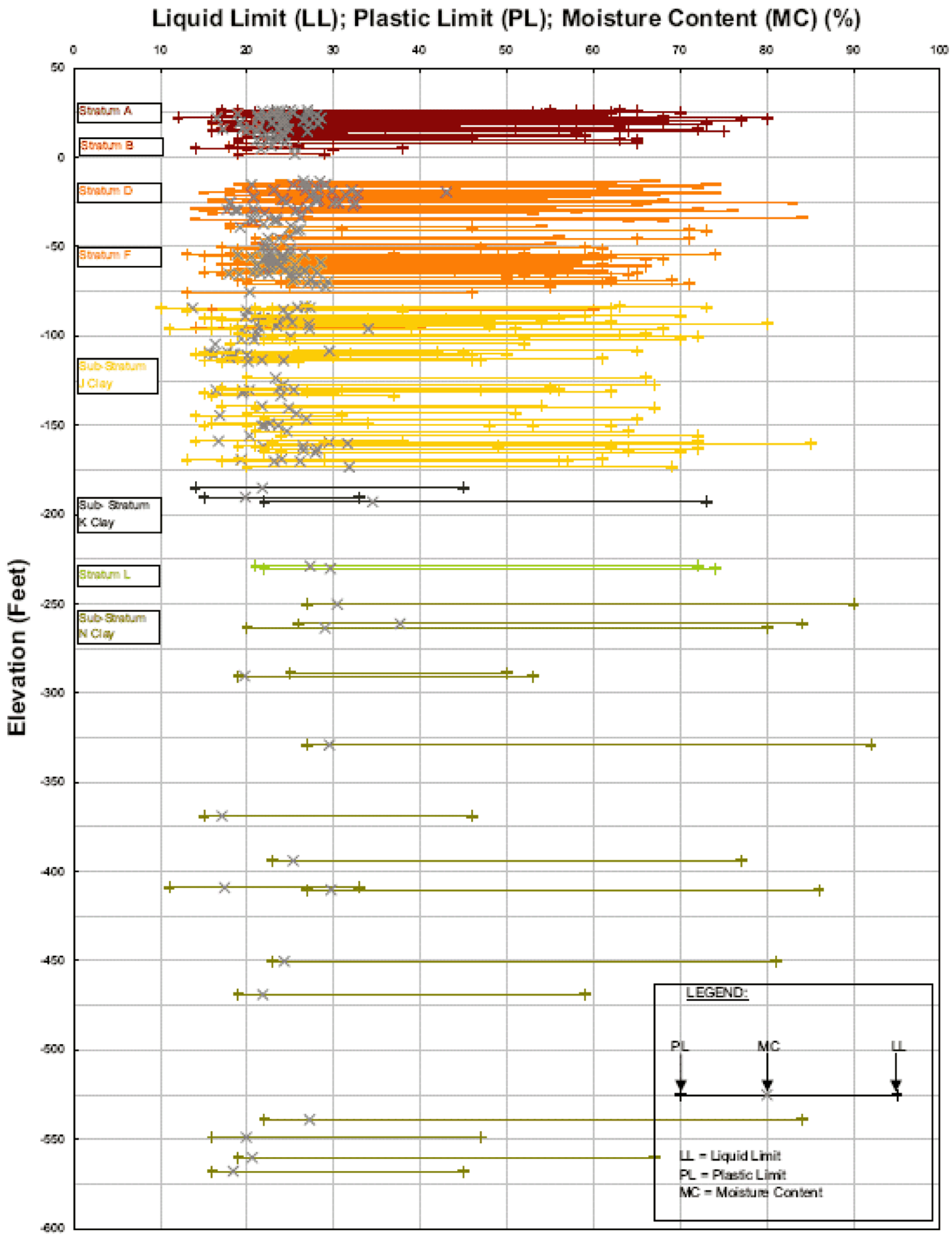


Figure 2.5S.4-20 Atterberg Limits versus Elevation

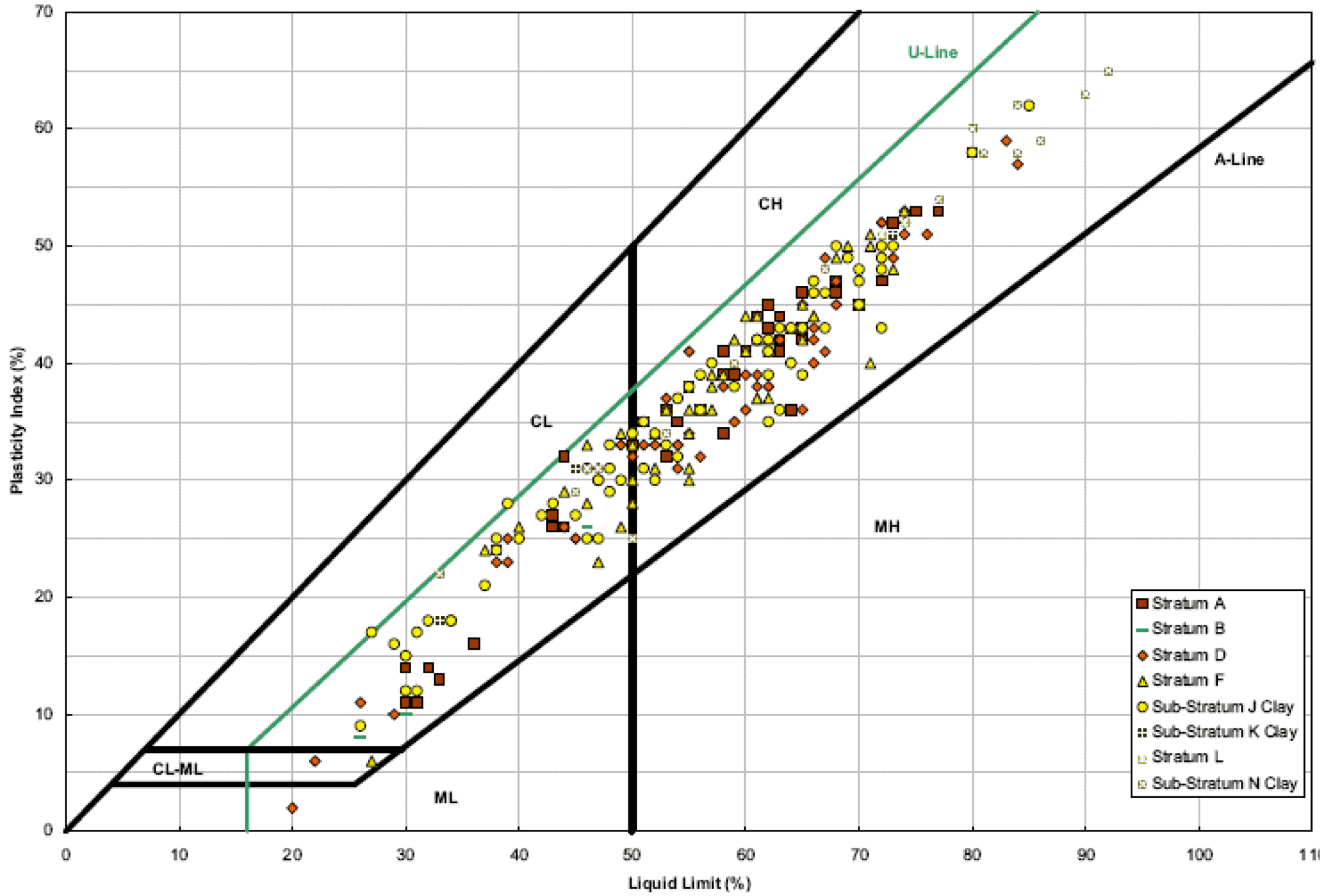


Figure 2.5S.4-21 Plasticity Chart

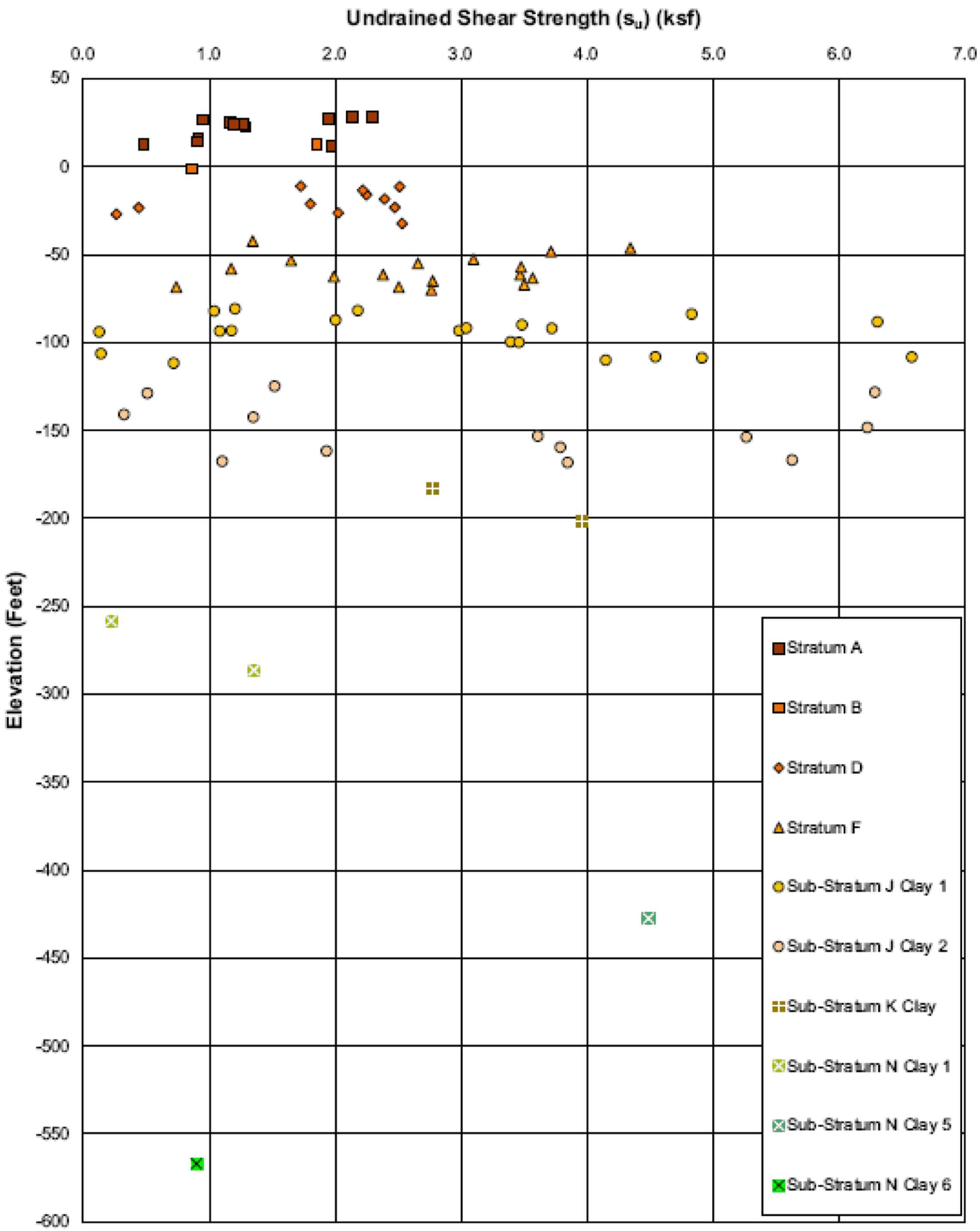
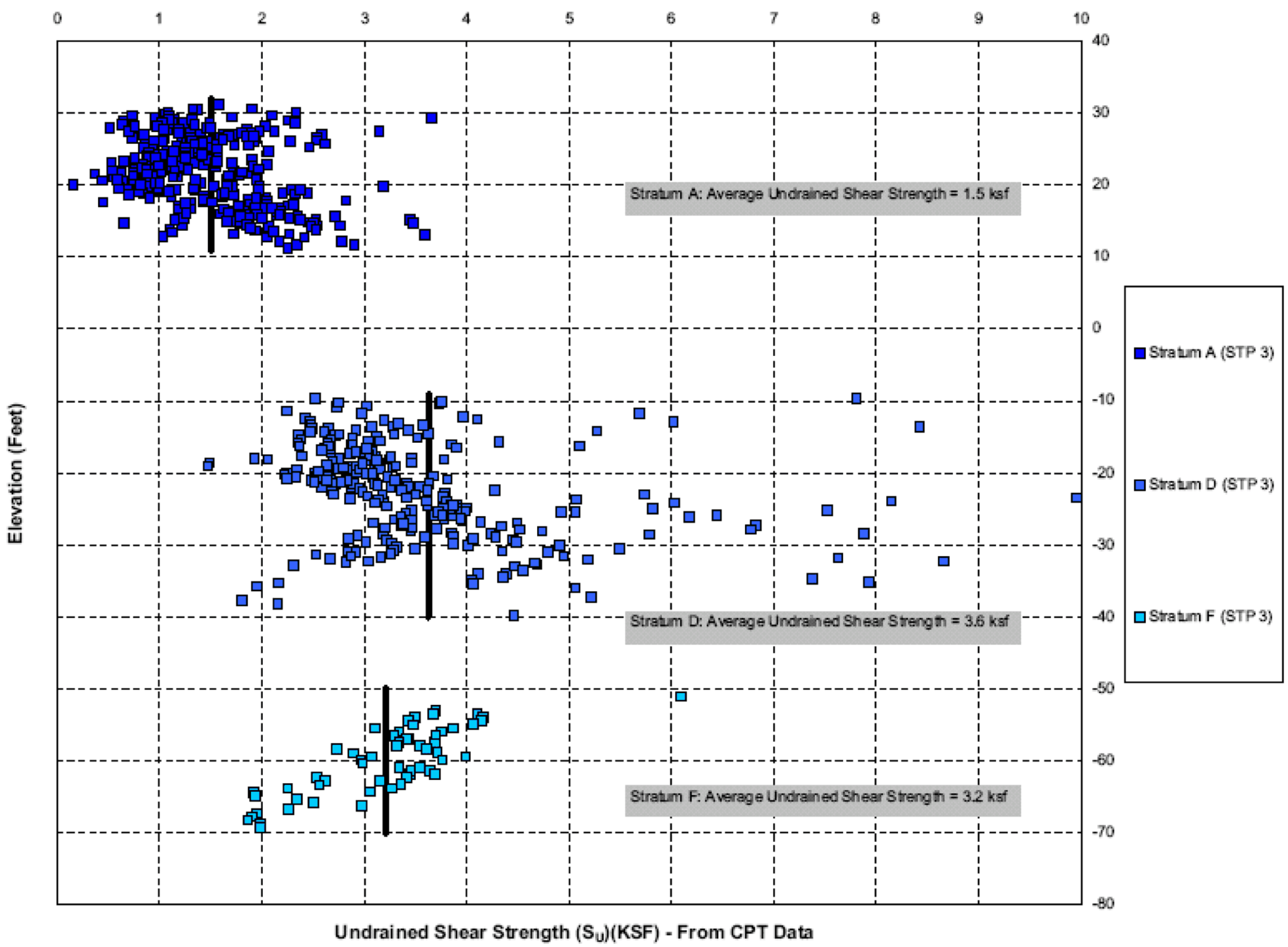


Figure 2.5S.4-22 Laboratory Test Results – Undrained Shear Strength (S_u) versus Elevation



NOTE: C-947 was not representative and therefore was not used as part of this data set.

Figure 2.5S.4-23 Undrained Shear Strength (S_u) From CPT Data (STP 3)

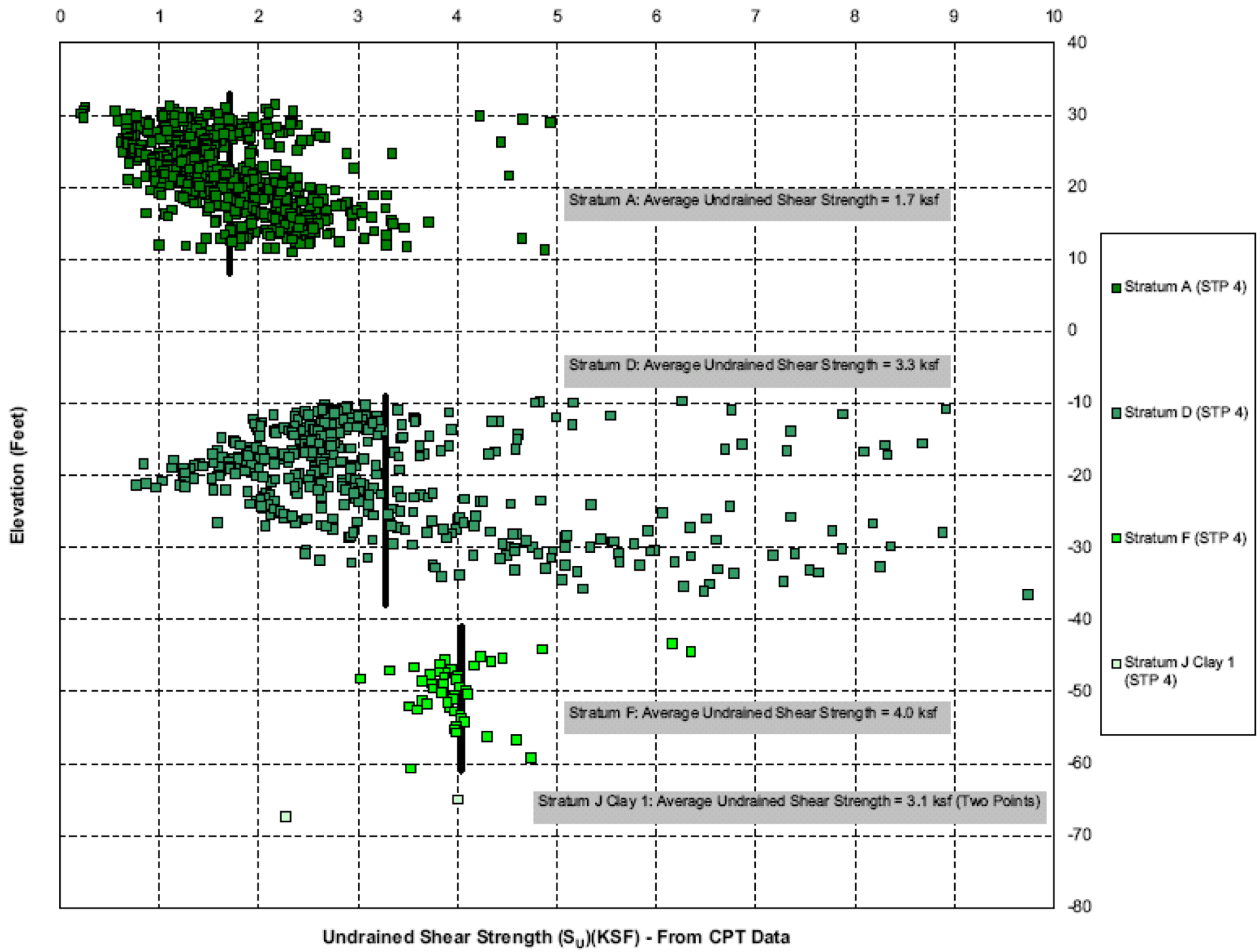


Figure 2.5S.4-24 Undrained Shear Strength (S_u) From CPT Data (STP 4)

Figure 2.5S.4-25 Not Used
(The data has been included in Figure 2.5S.4-26)

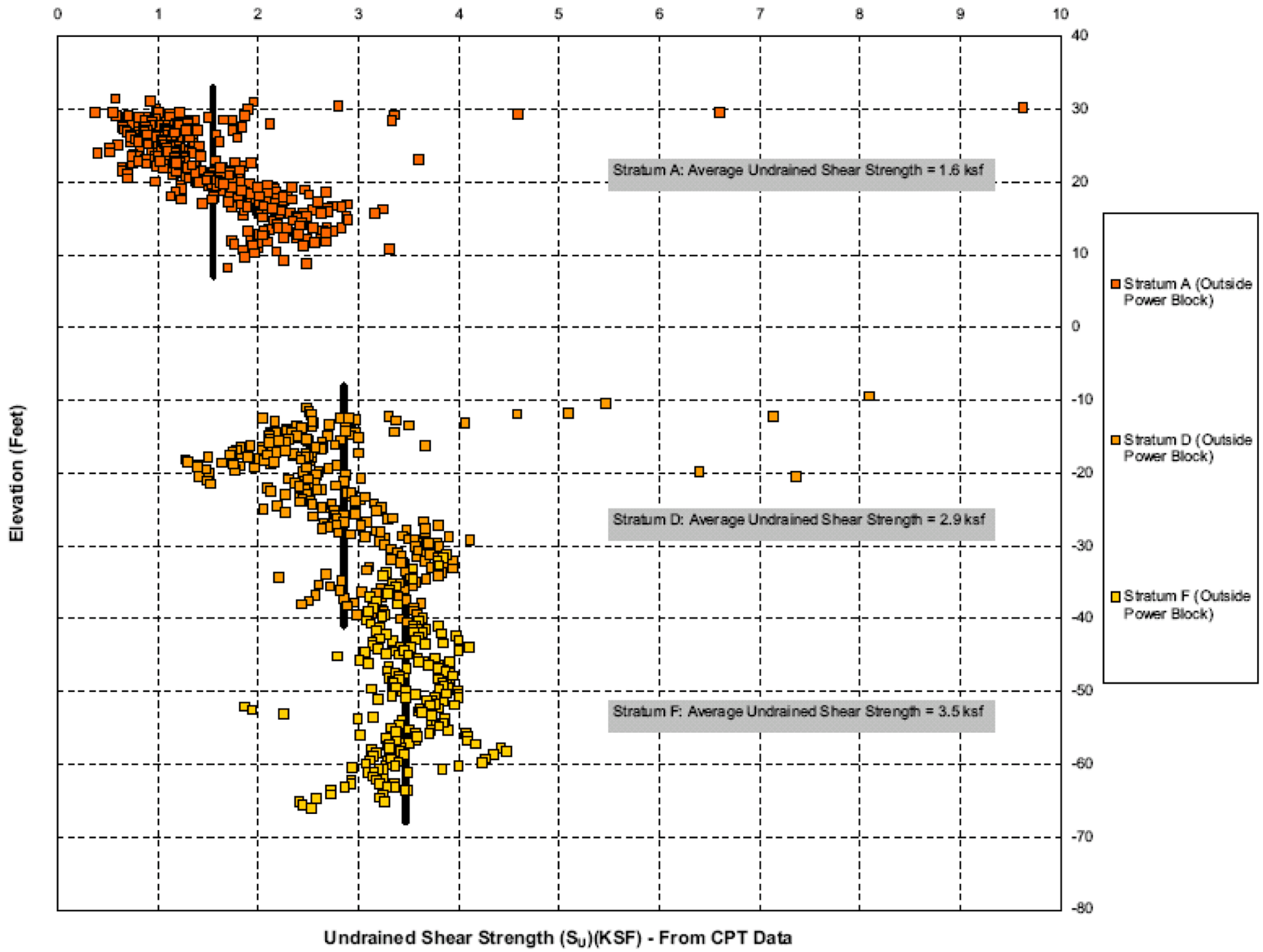


Figure 2.5S.4-26 Undrained Shear Strength (S_u) From CPT Data (Outside Power Block)

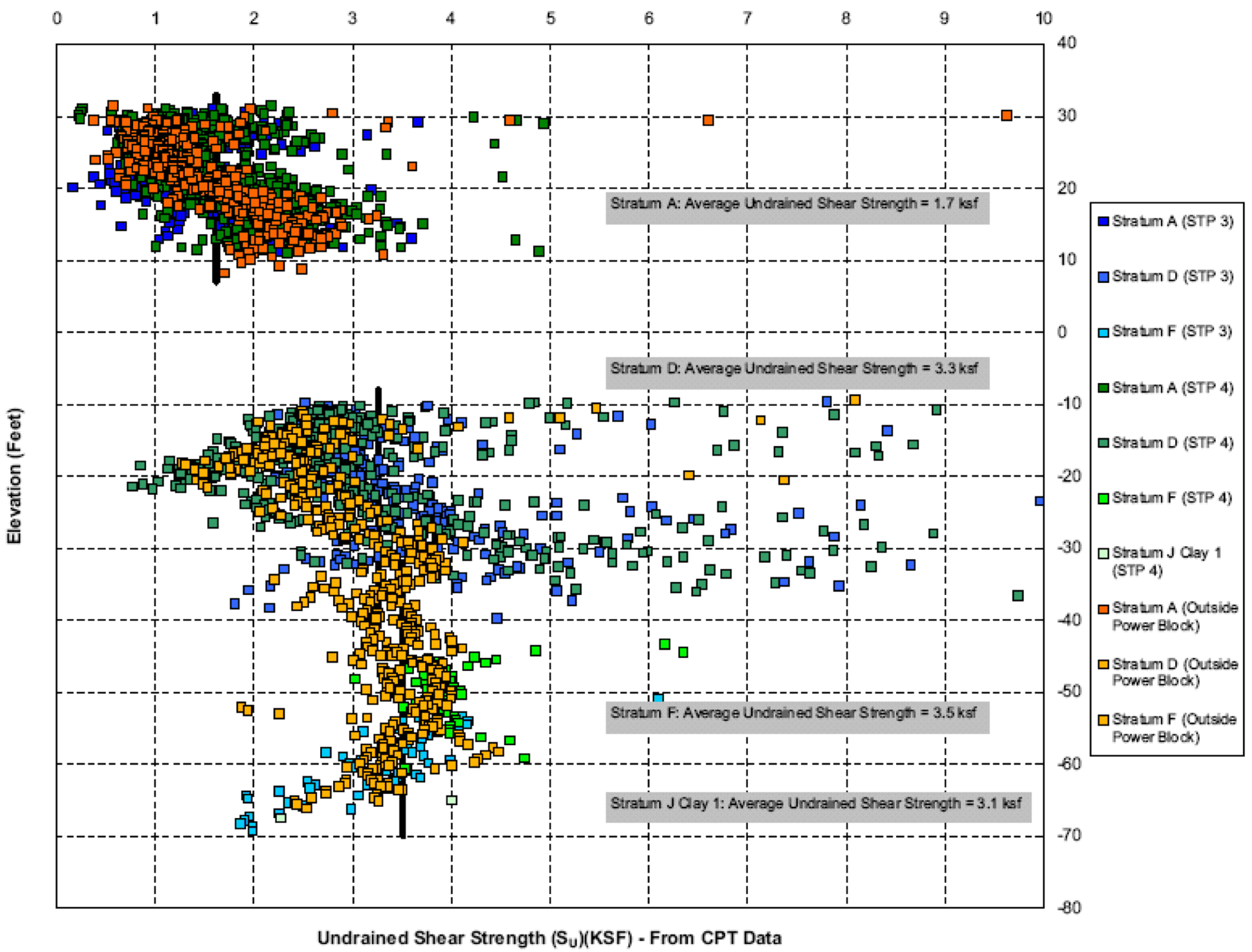


Figure 2.5S.4-27 Undrained Shear Strength (S_u) From CPT Data (Site-Wide)

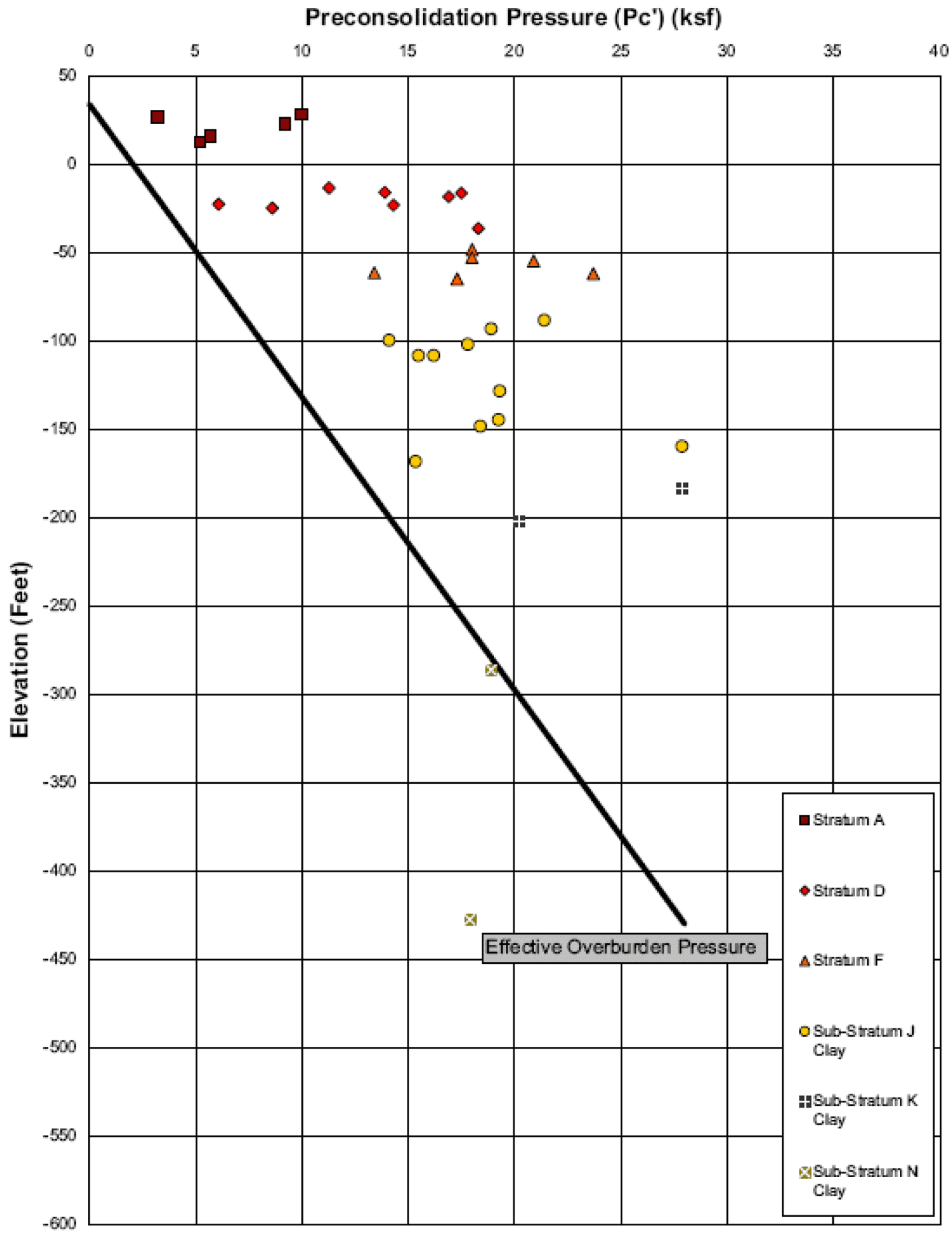


Figure 2.5S.4-28 Laboratory Test Results -Preconsolidation Pressure (P_c') versus Elevation

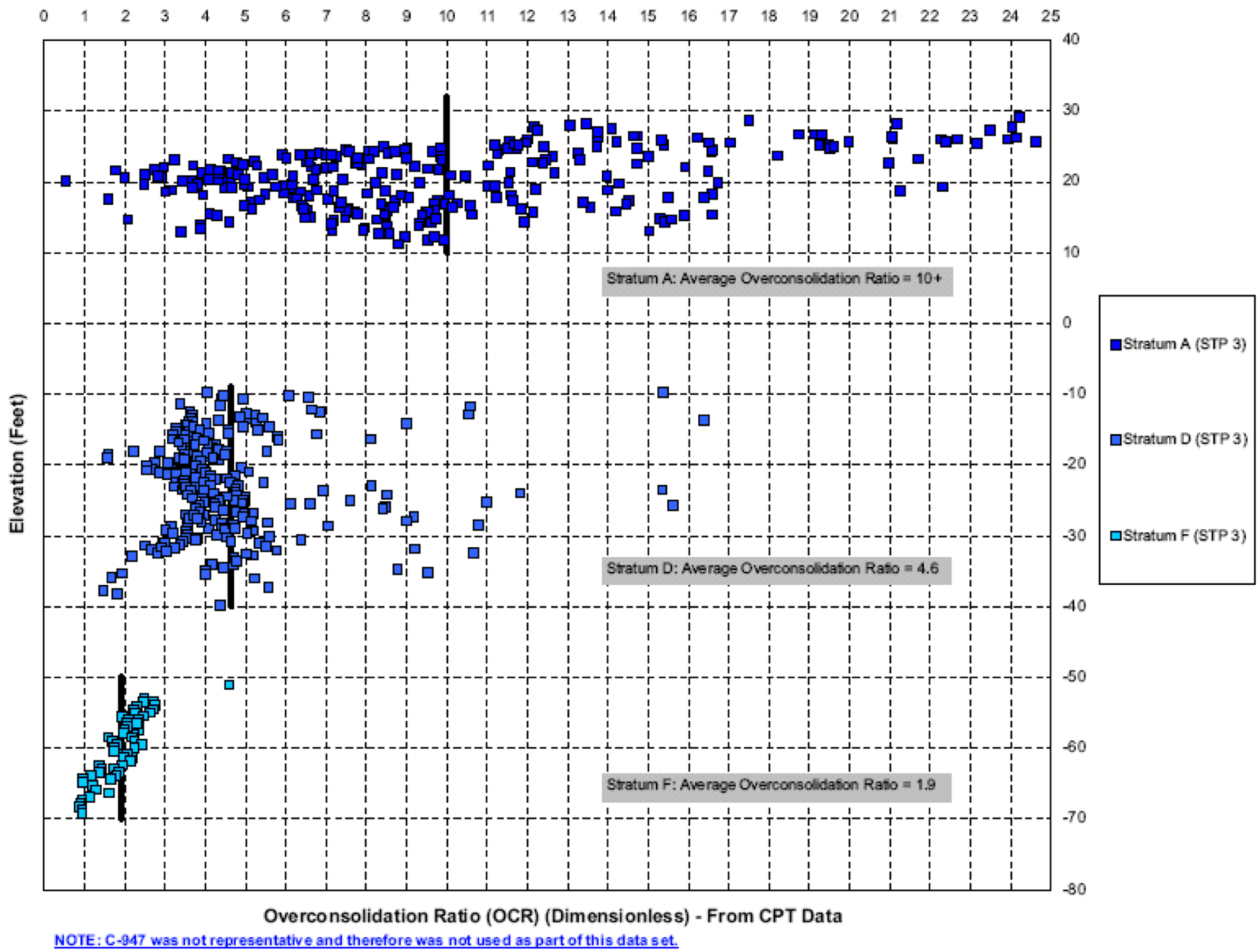


Figure 2.5S.4-29 Overconsolidation Ratio (OCR) From CPT Data (STP 3)

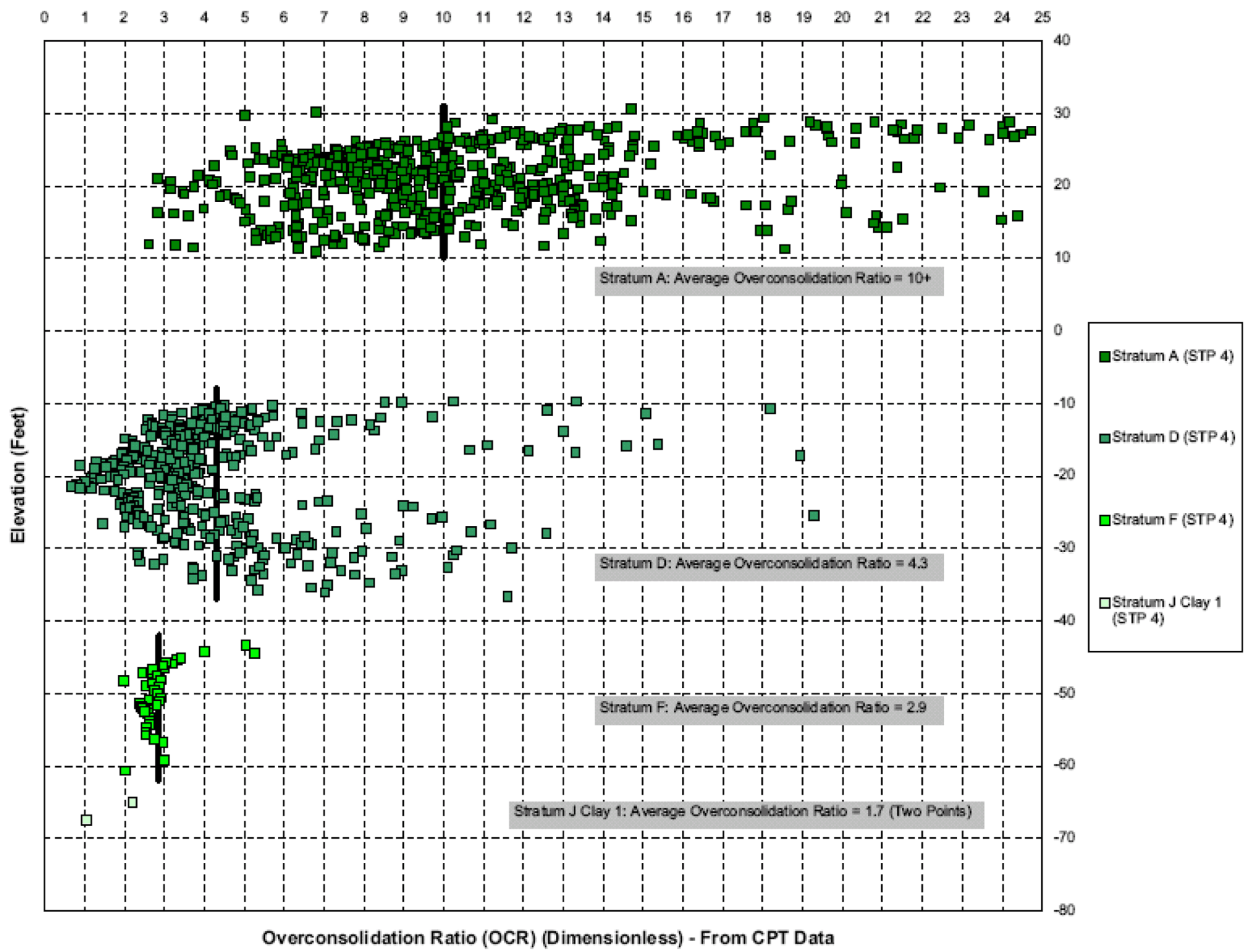


Figure 2.5S.4-30 Overconsolidation Ratio (OCR) From CPT Data (STP 4)

Figure 2.5S.4-31 Not Used
(The data has been included in Figure 2.5S.4-32)

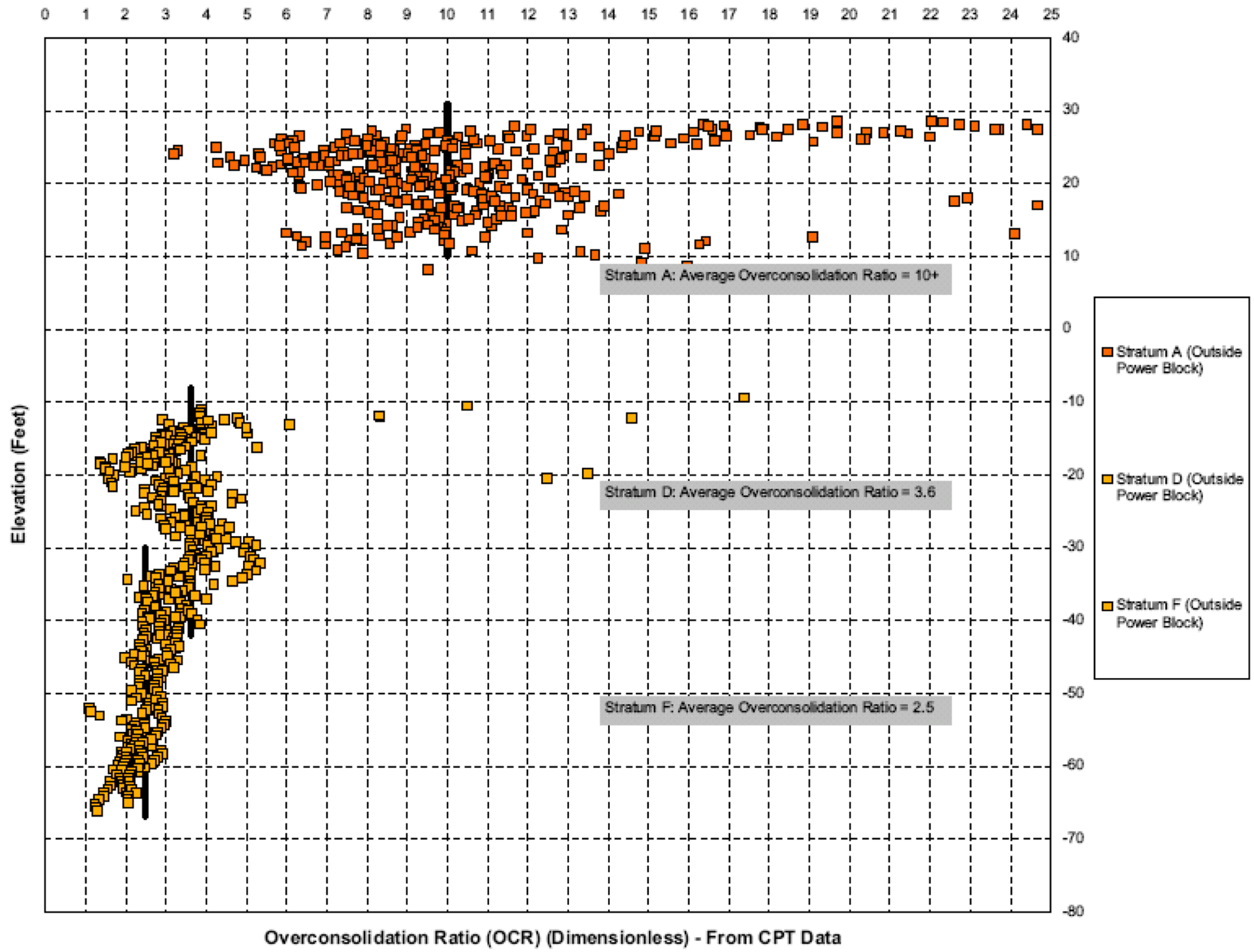


Figure 2.5S.4-32 Overconsolidation Ratio (OCR) From CPT Data (Outside Power Block)

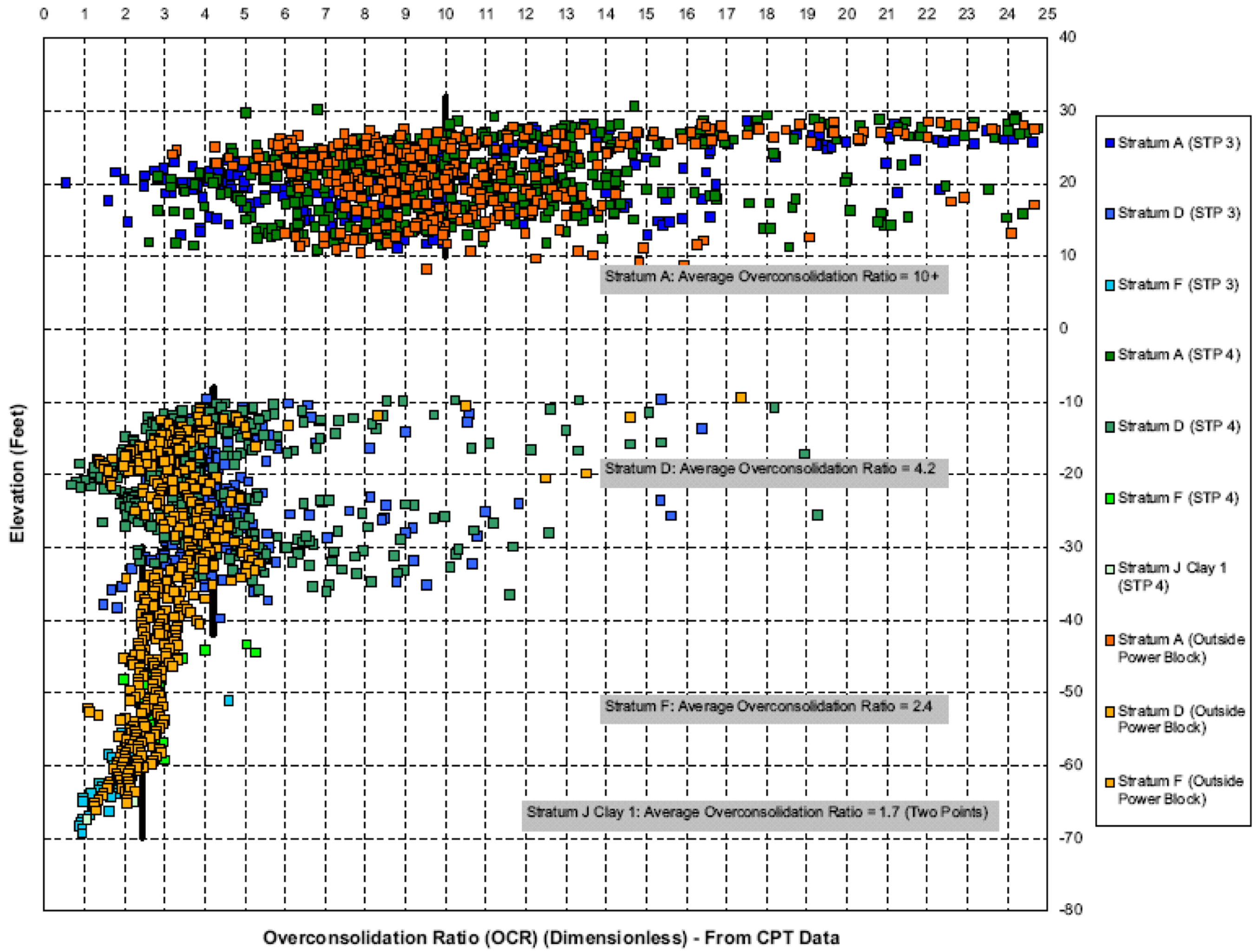
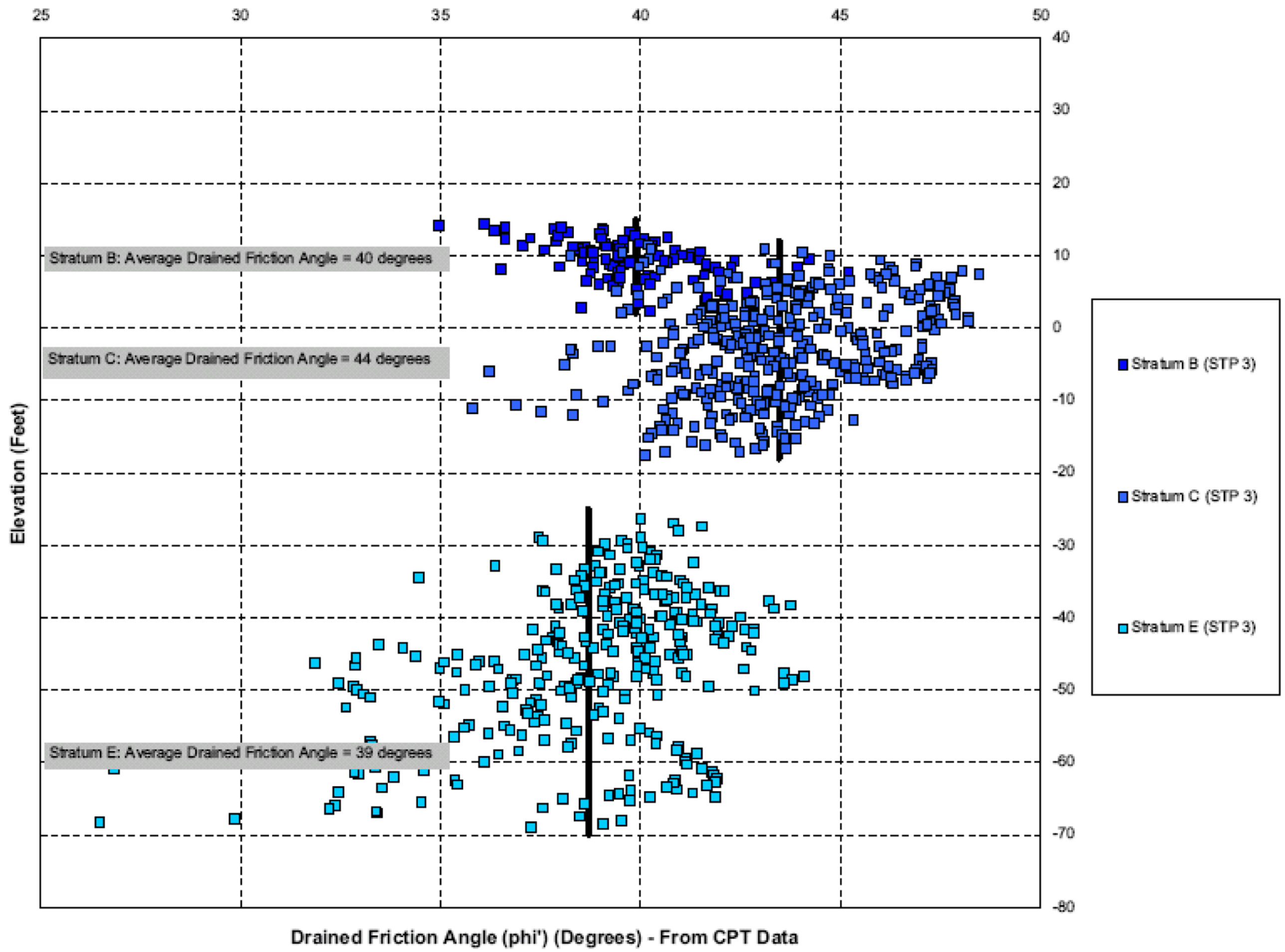


Figure 2.5S.4-33 Overconsolidation Ratio (OCR) From CPT Data (Site-Wide)



NOTE: C-947 was not representative and therefore was not used as part of this data set.

Figure 2.5S.4-34 Drained Friction Angle (phi') From CPT Data (STP 3)

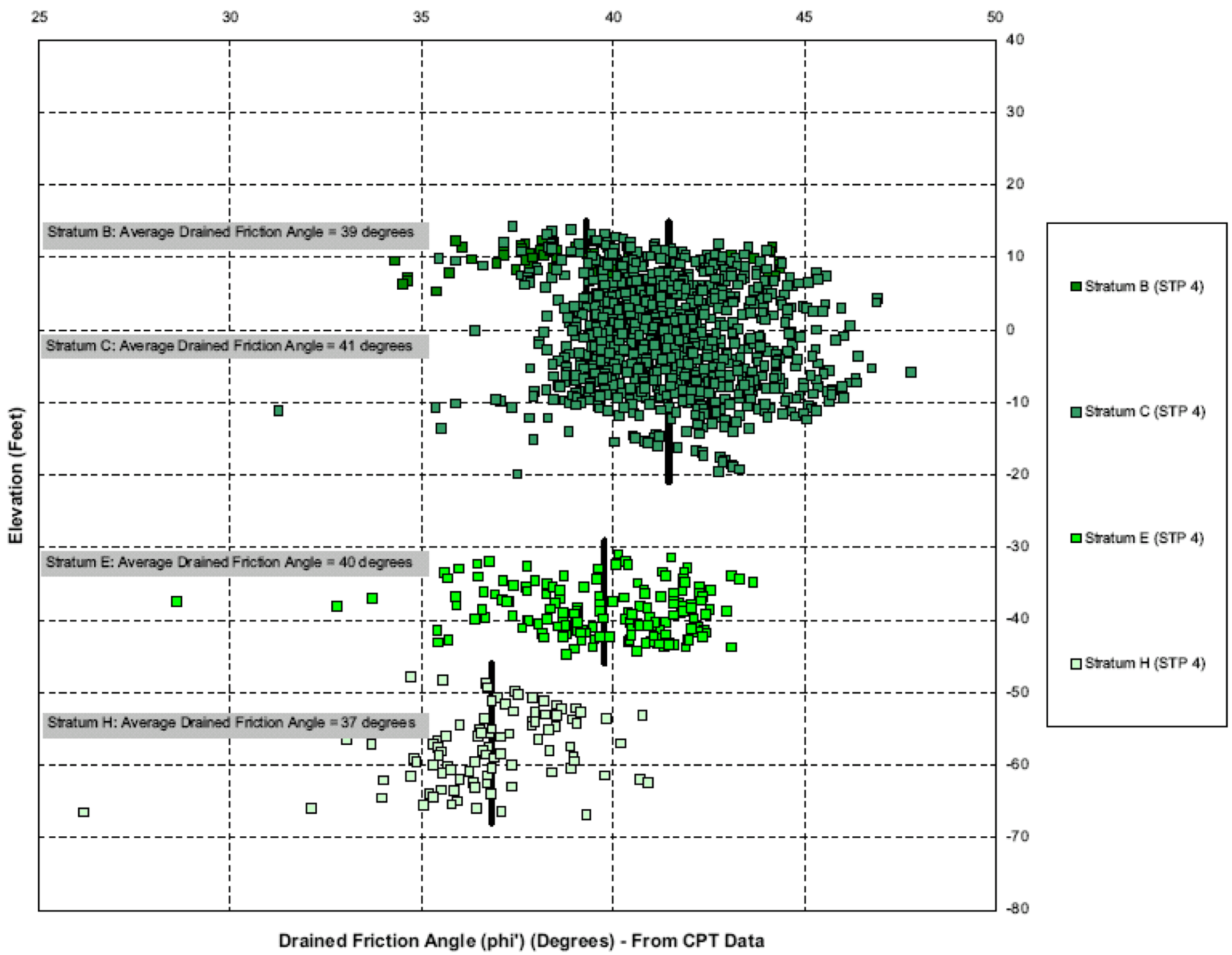


Figure 2.5S.4-35 Drained Friction Angle (phi') From CPT Data (STP 4)

Figure 2.5S.4-36 Not Used
(The data has been included in Figure 2.5S.4-37)

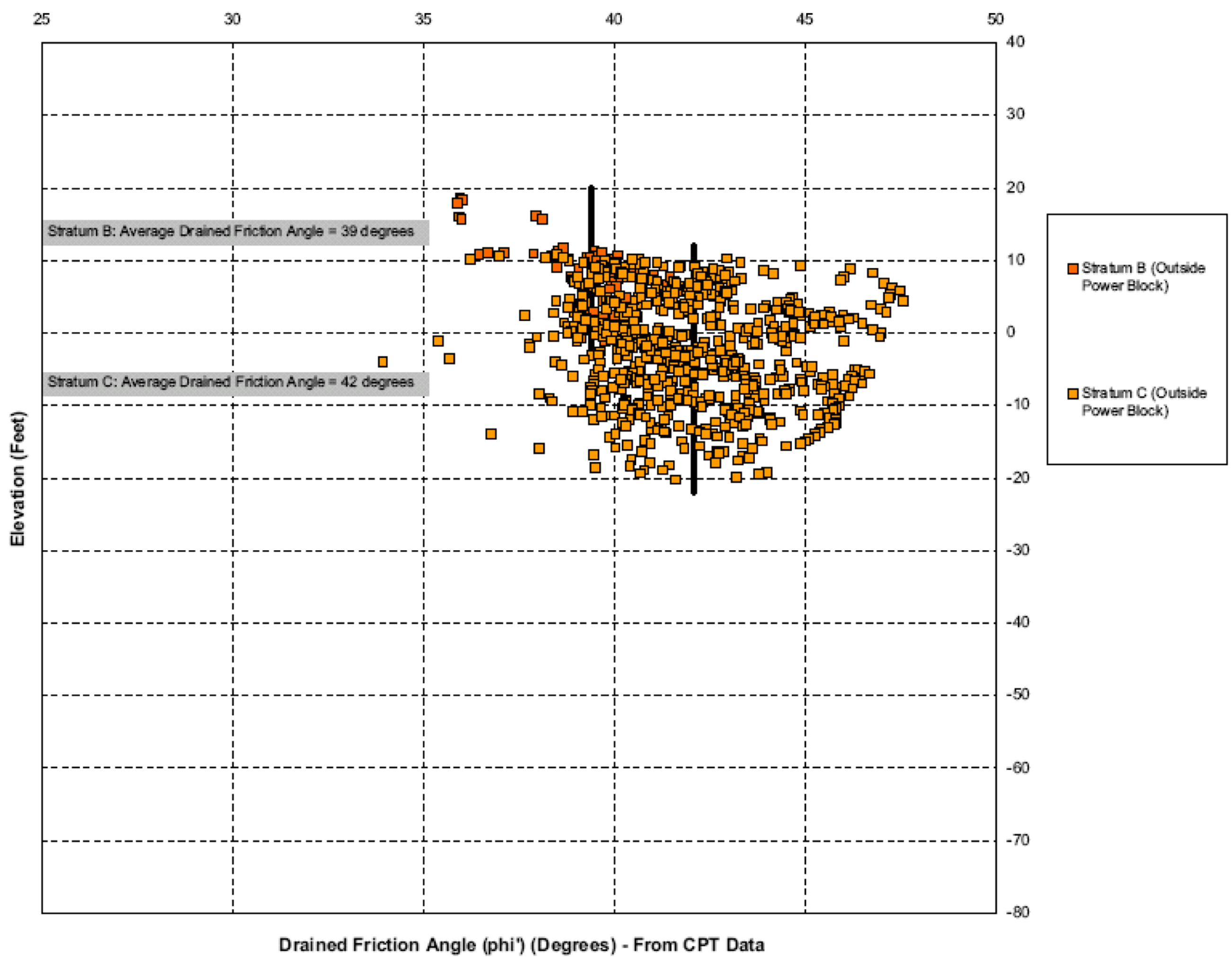


Figure 2.5S.4-37 Drained Friction Angle (phi') From CPT Data (Outside Power Block)

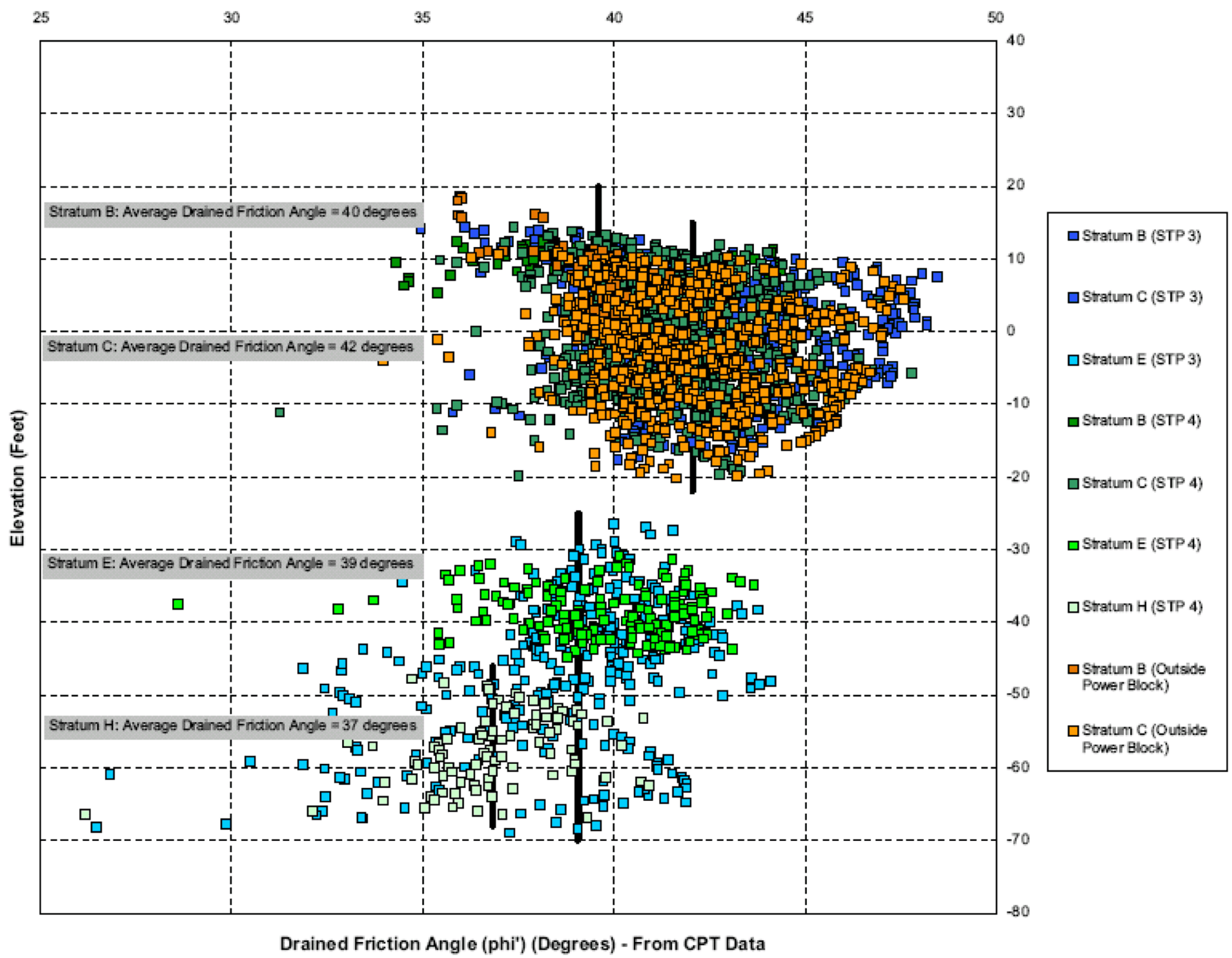


Figure 2.5S.4-38 Drained Friction Angle (phi') From CPT Data (Site-Wide)

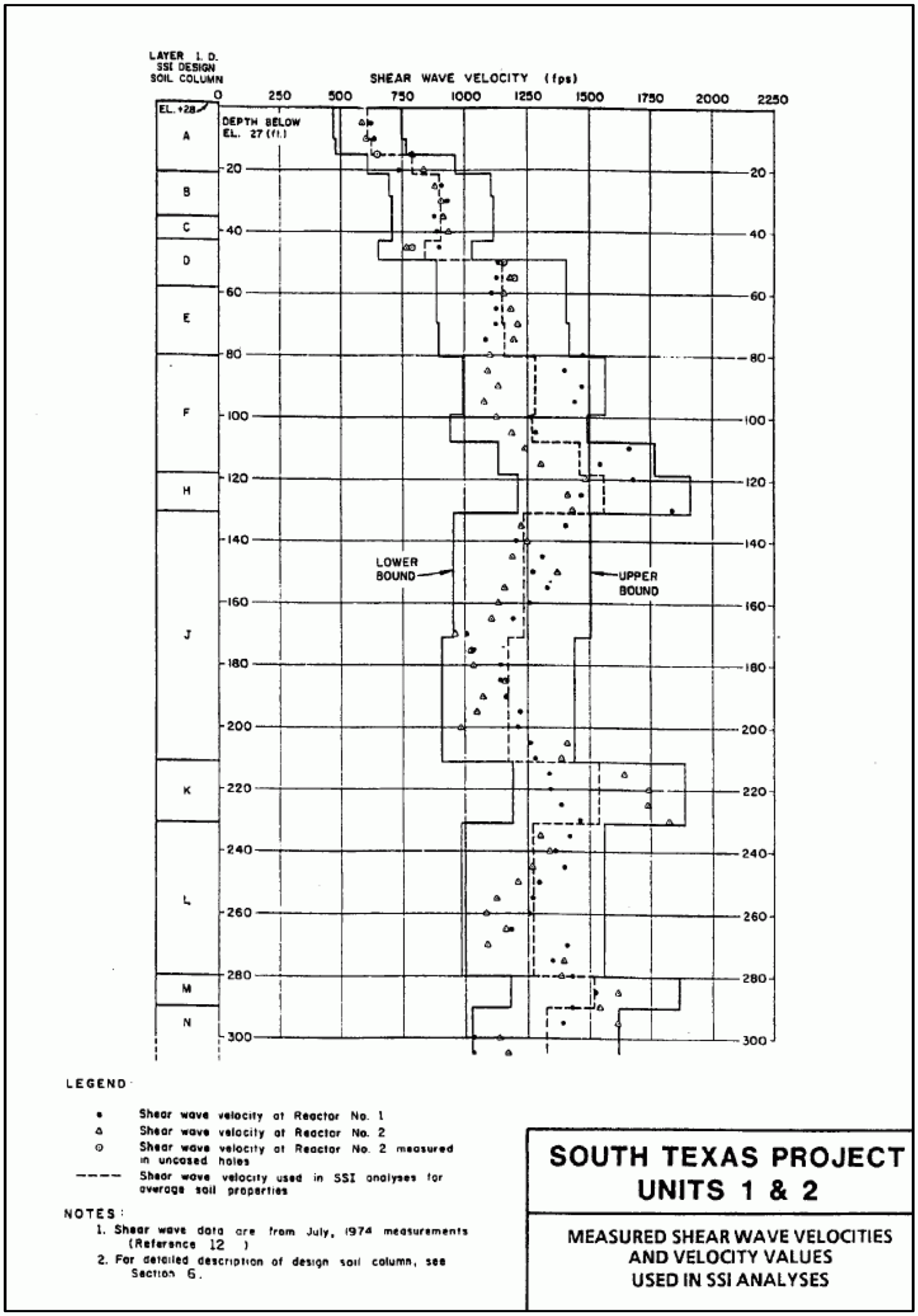


Figure 2.5S.4-39 STP 1 & 2; Shear Wave Velocity versus Depth

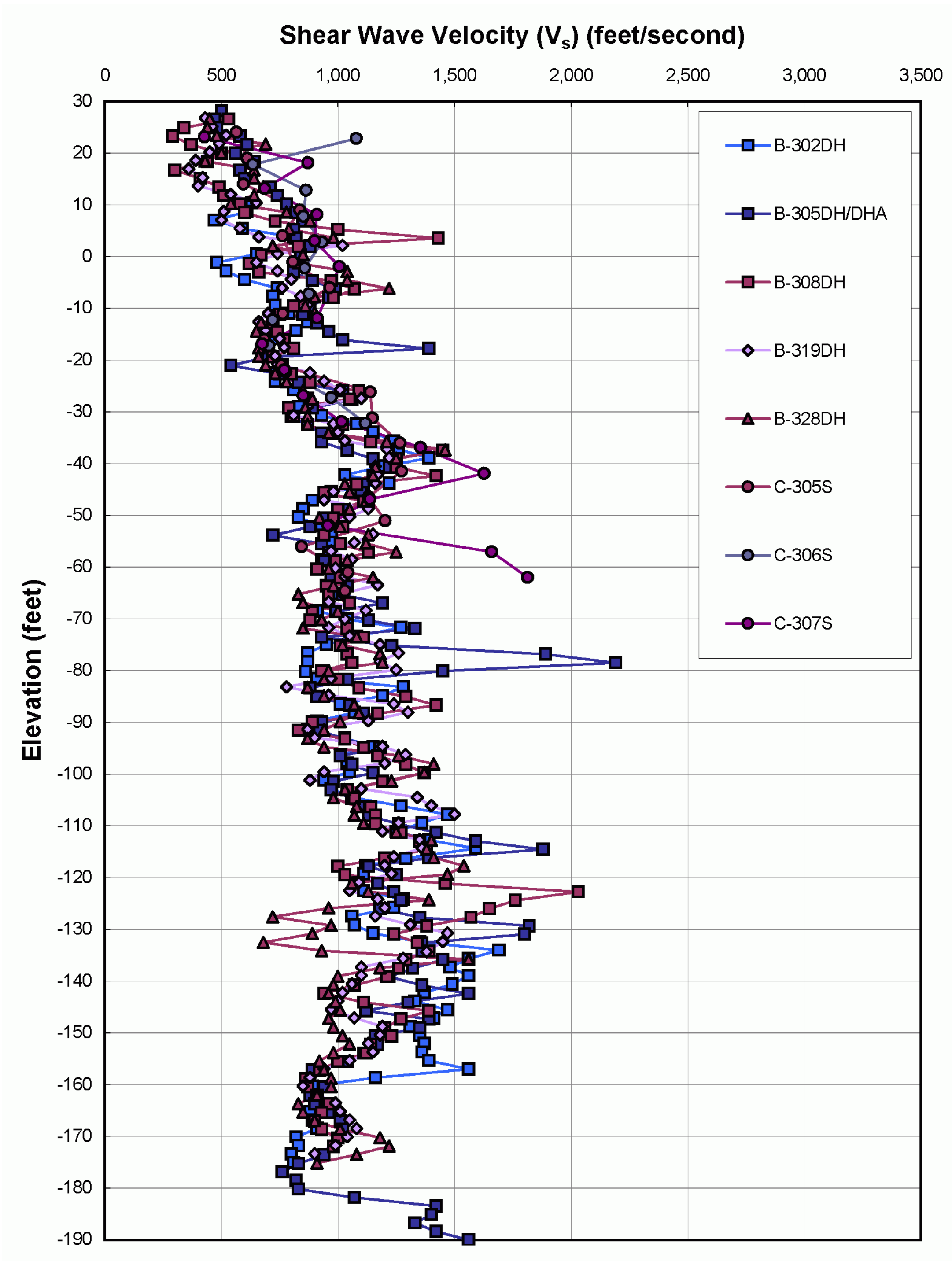


Figure 2.5S.4-40 STP 3; Shear Wave Velocity versus Depth

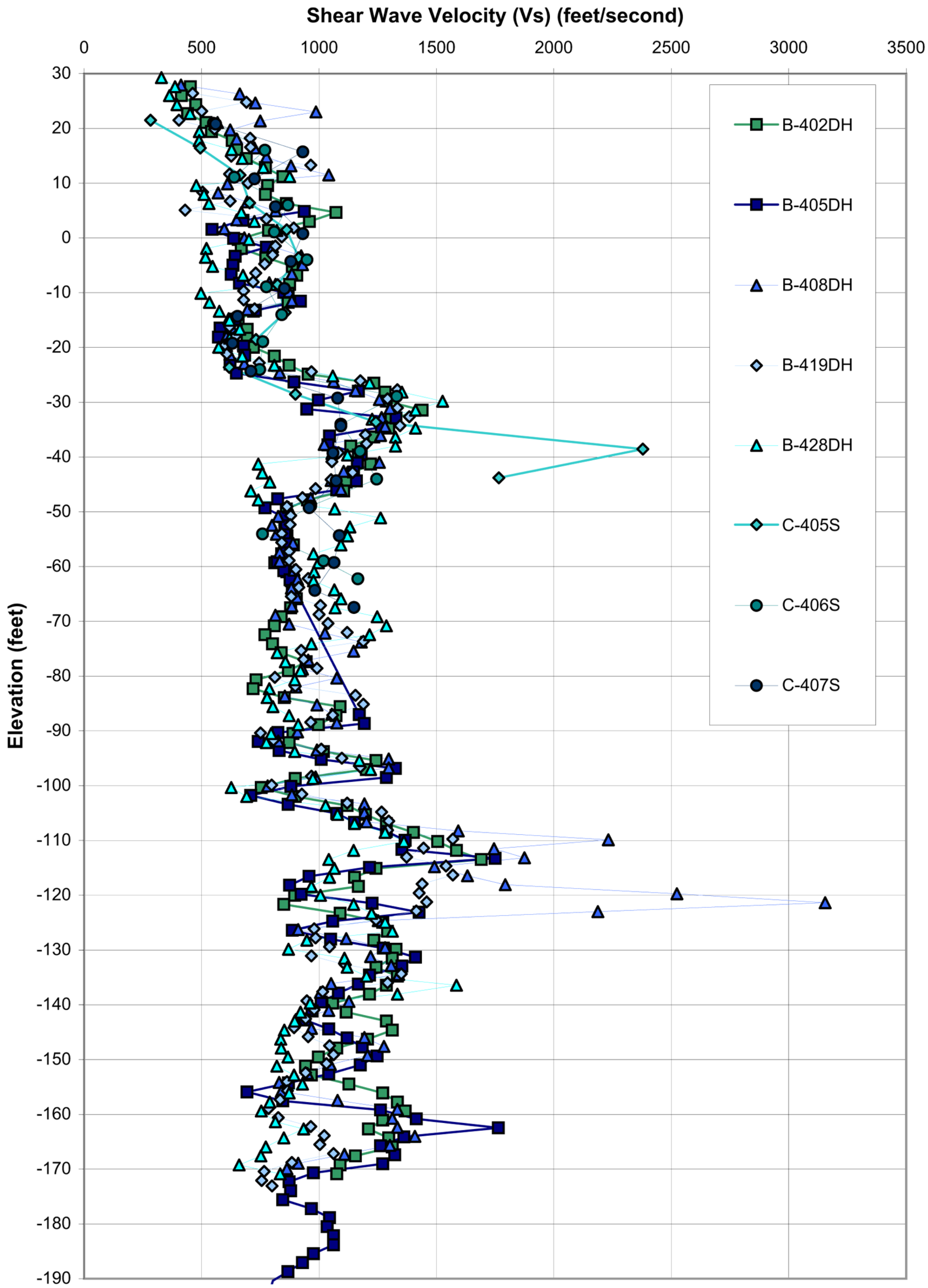


Figure 2.5S.4-41 STP 4; Shear Wave Velocity versus Depth

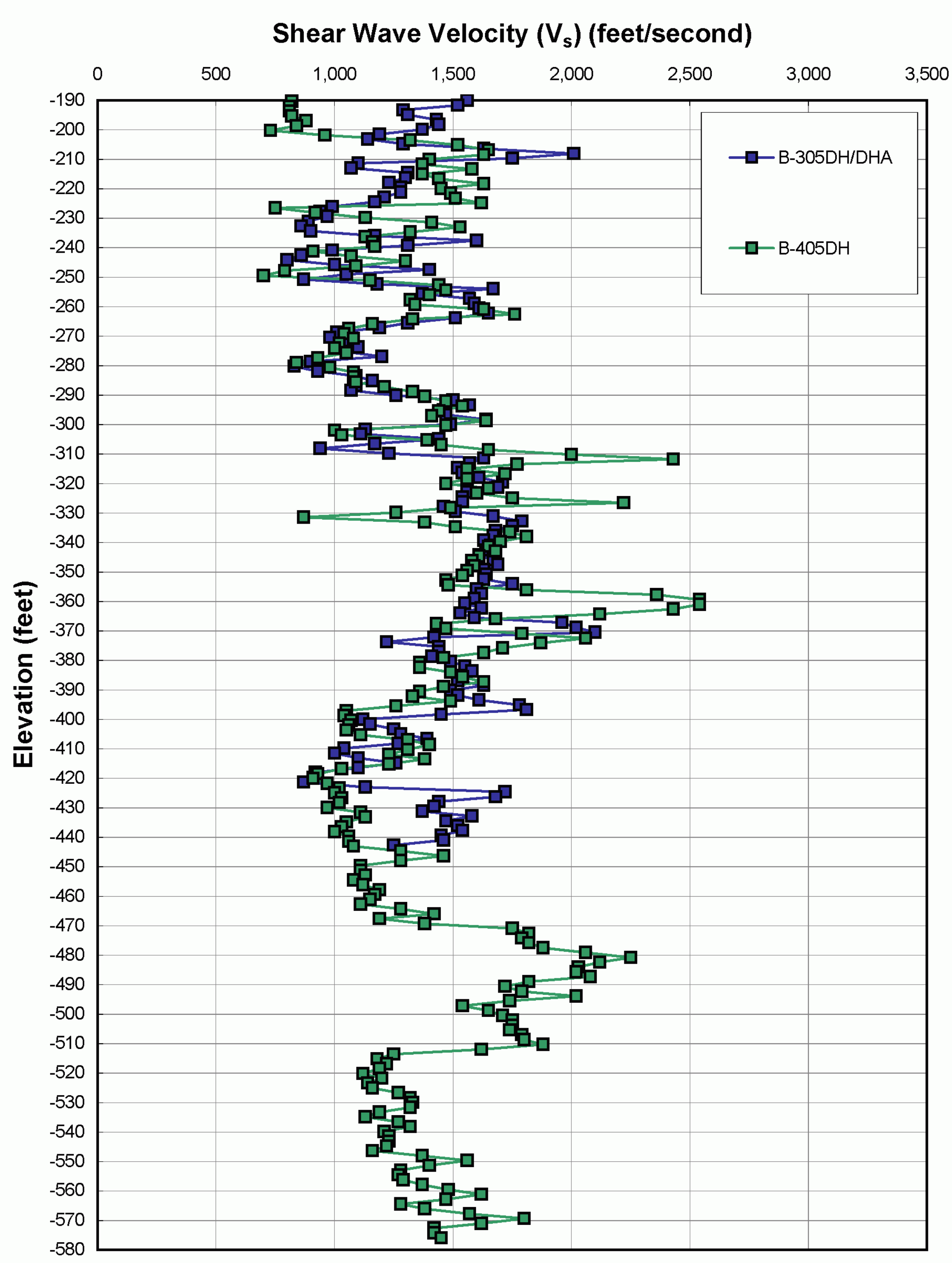


Figure 2.5S.4-42 STP 3 & 4; Shear Wave Velocity to 600 Feet Below Ground Surface

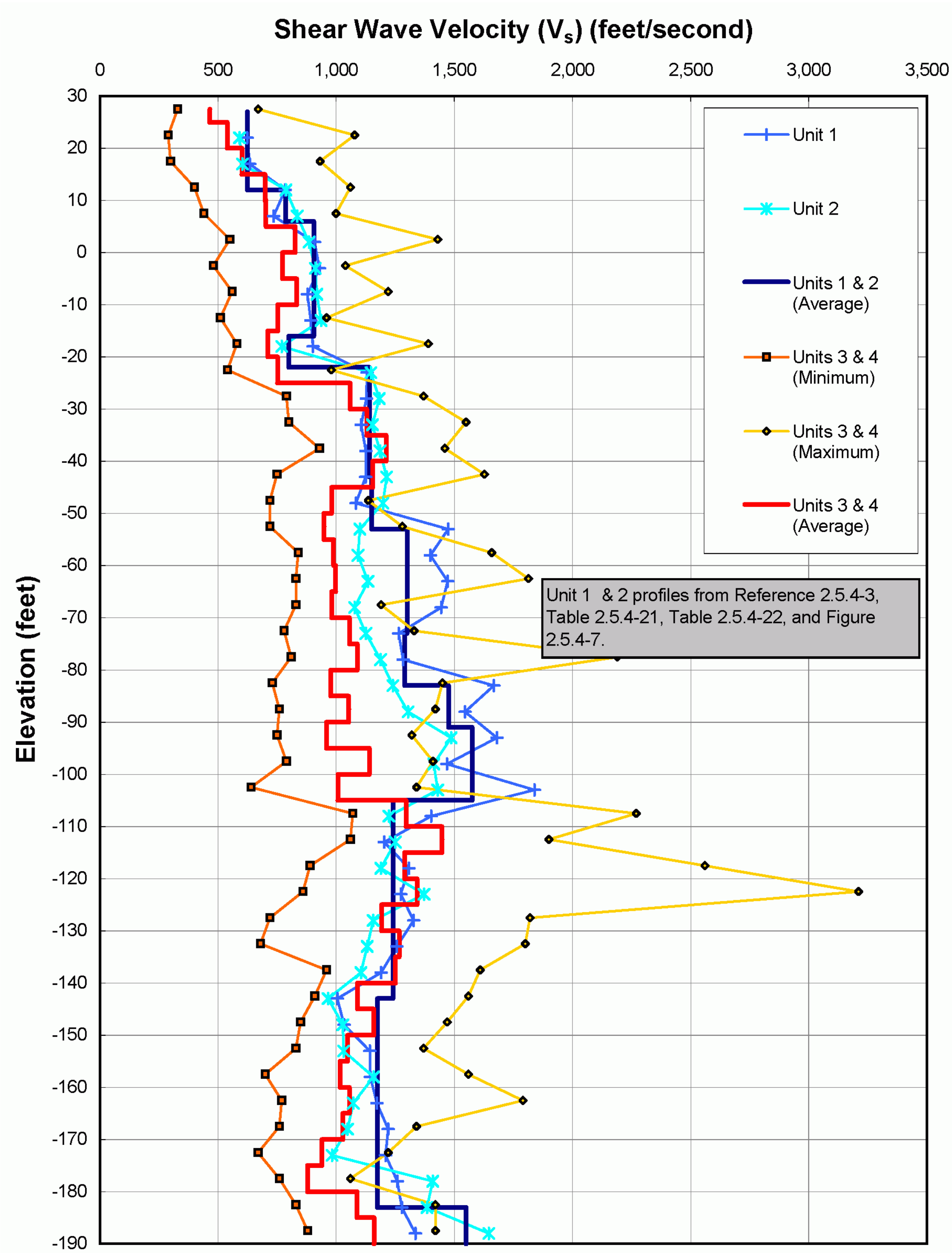


Figure 2.5S.4-43 STP 1 & 2/ STP 3 & 4; Average Shear Wave Velocity to 200 Feet Below Ground Surface

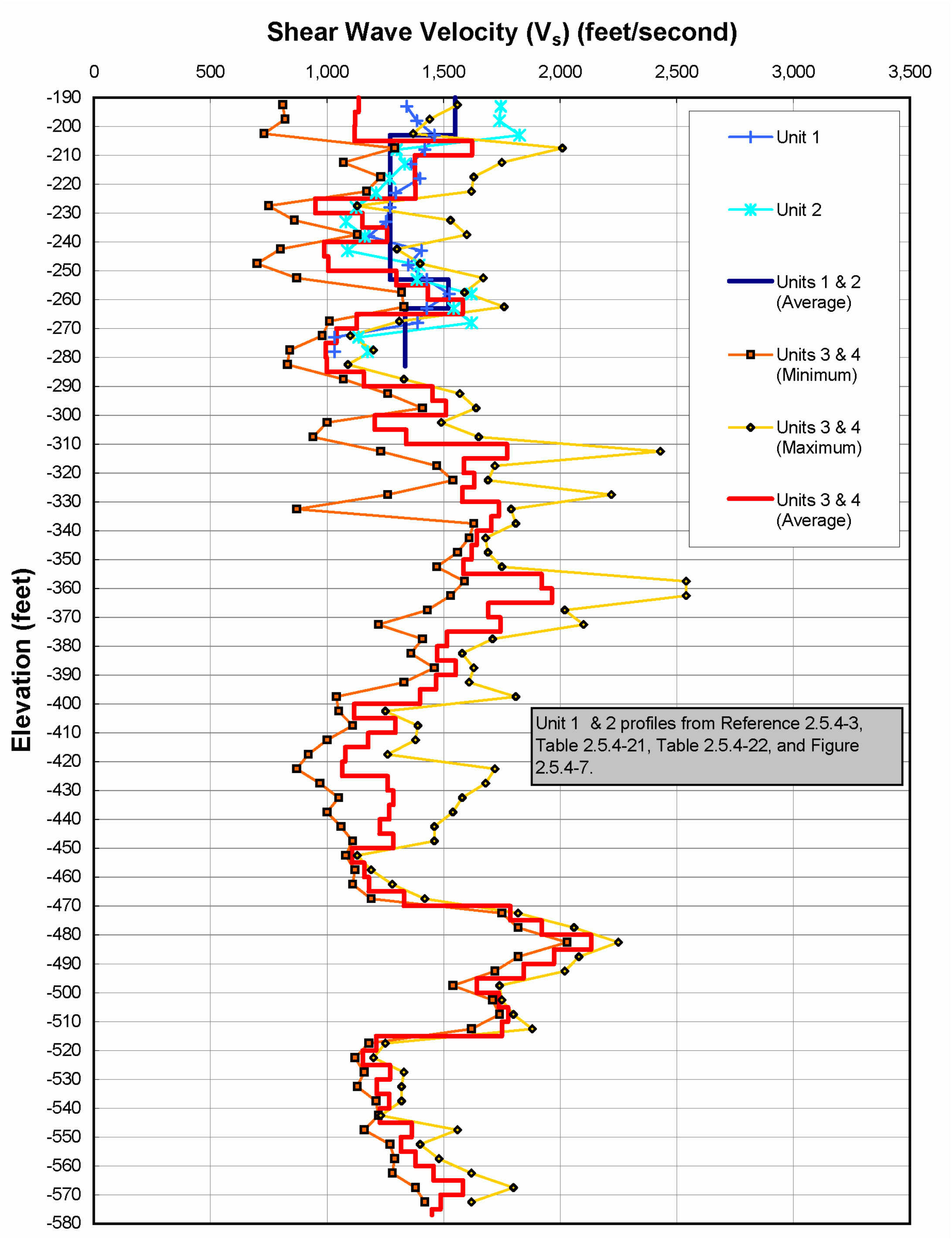


Figure 2.5S.4-44 STP 1 & 2/ STP 3 & 4; Average Shear Wave Velocity to 600 Feet Below Ground Surface

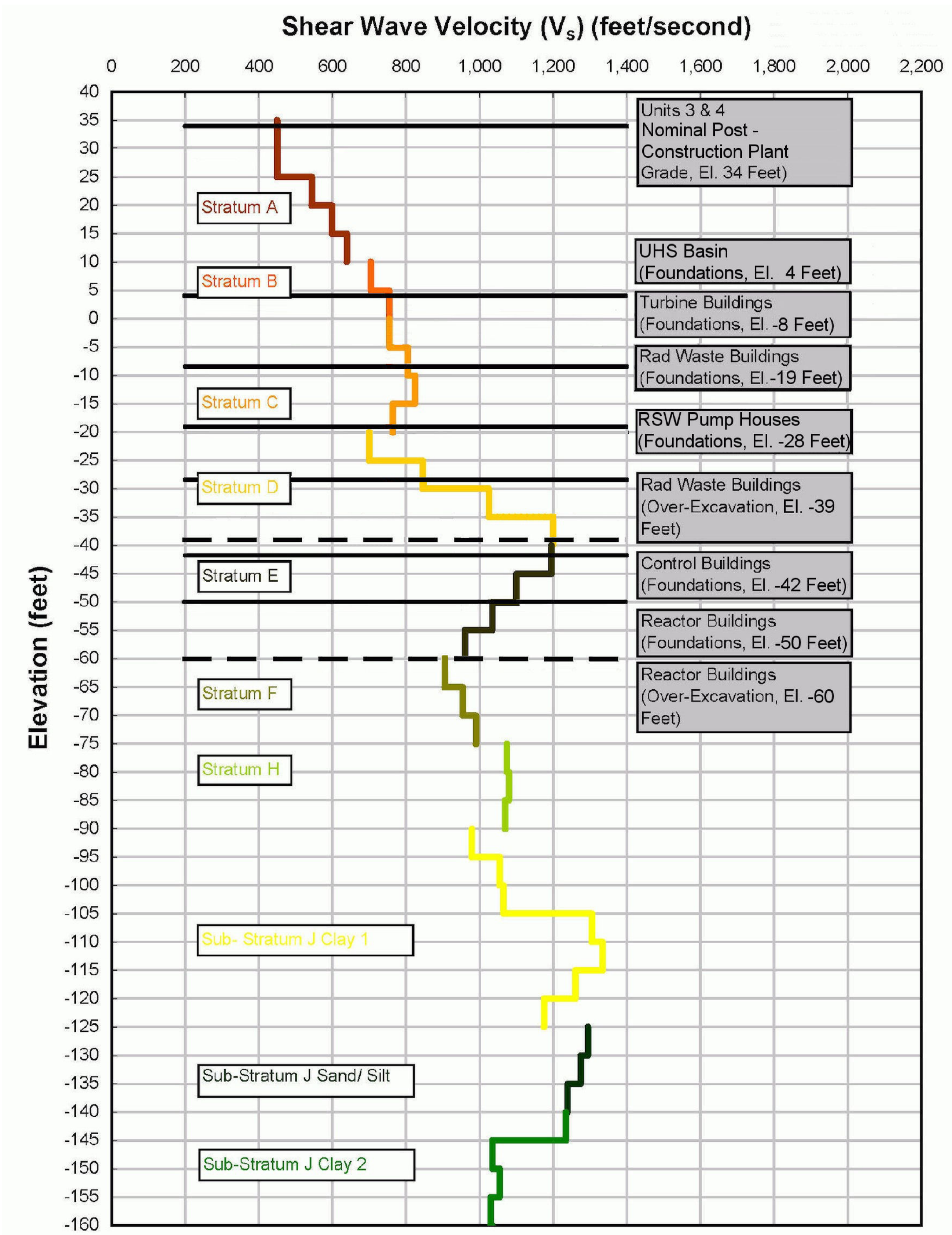


Figure 2.5S.4-45 Shear Wave Velocity Profile - Strata A to J

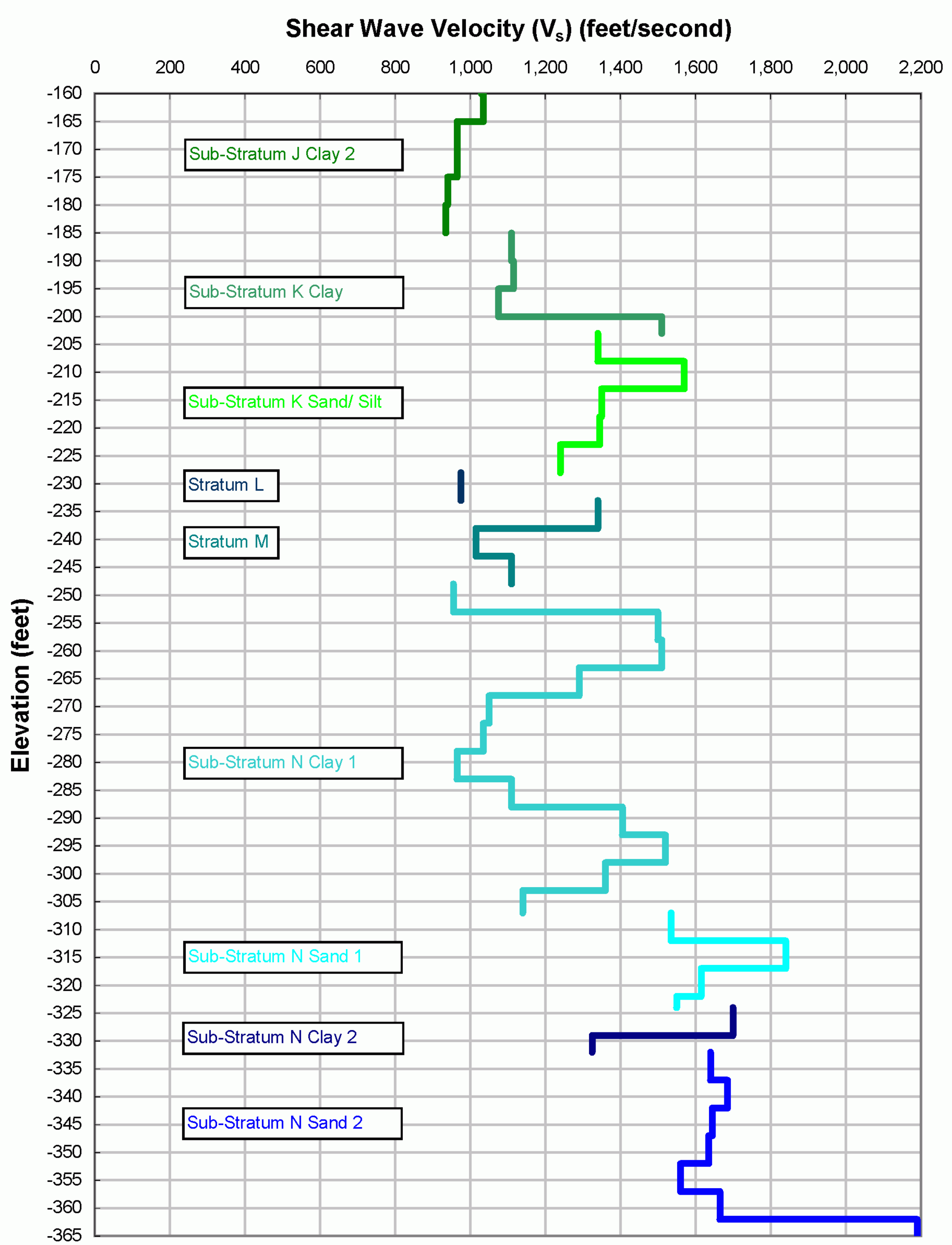


Figure 2.5S.4-46 Shear Wave Velocity Profile - Strata J to N

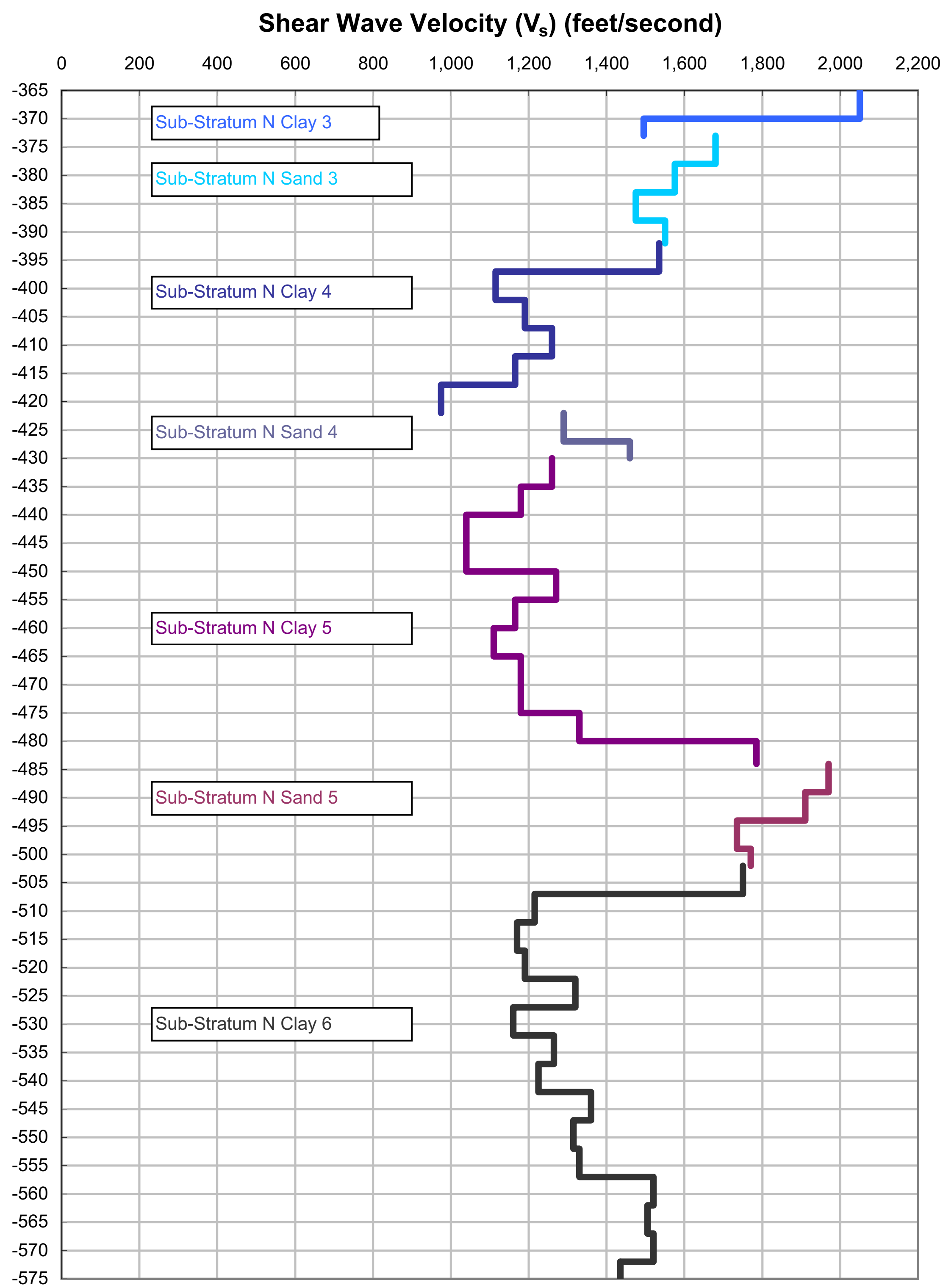


Figure 2.5S.4-47 Shear Wave Velocity Profile - Stratum N to 600 Feet Below Ground Surface

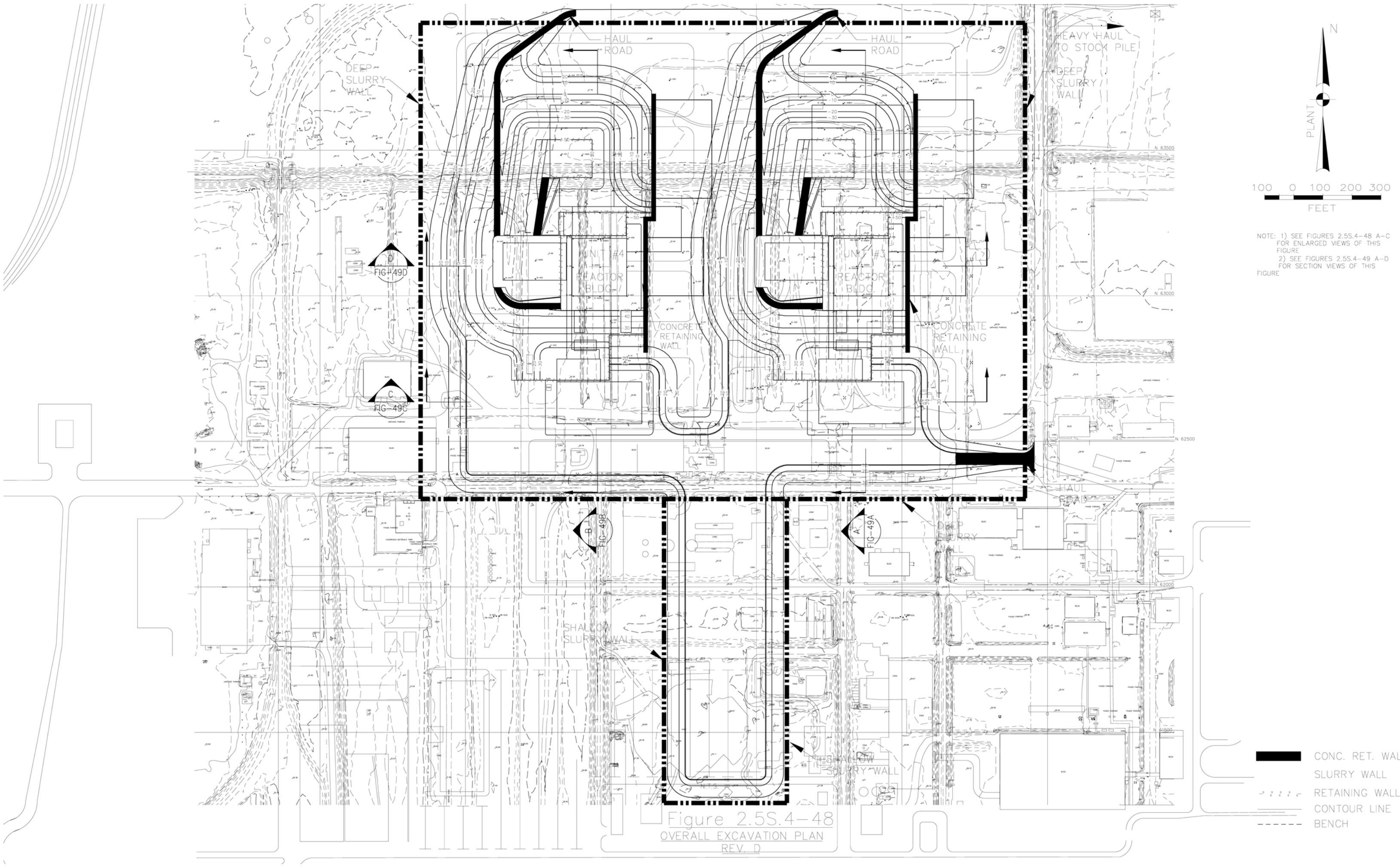


Figure 2.5S.4-48 Overall Excavation Plan Rev. D

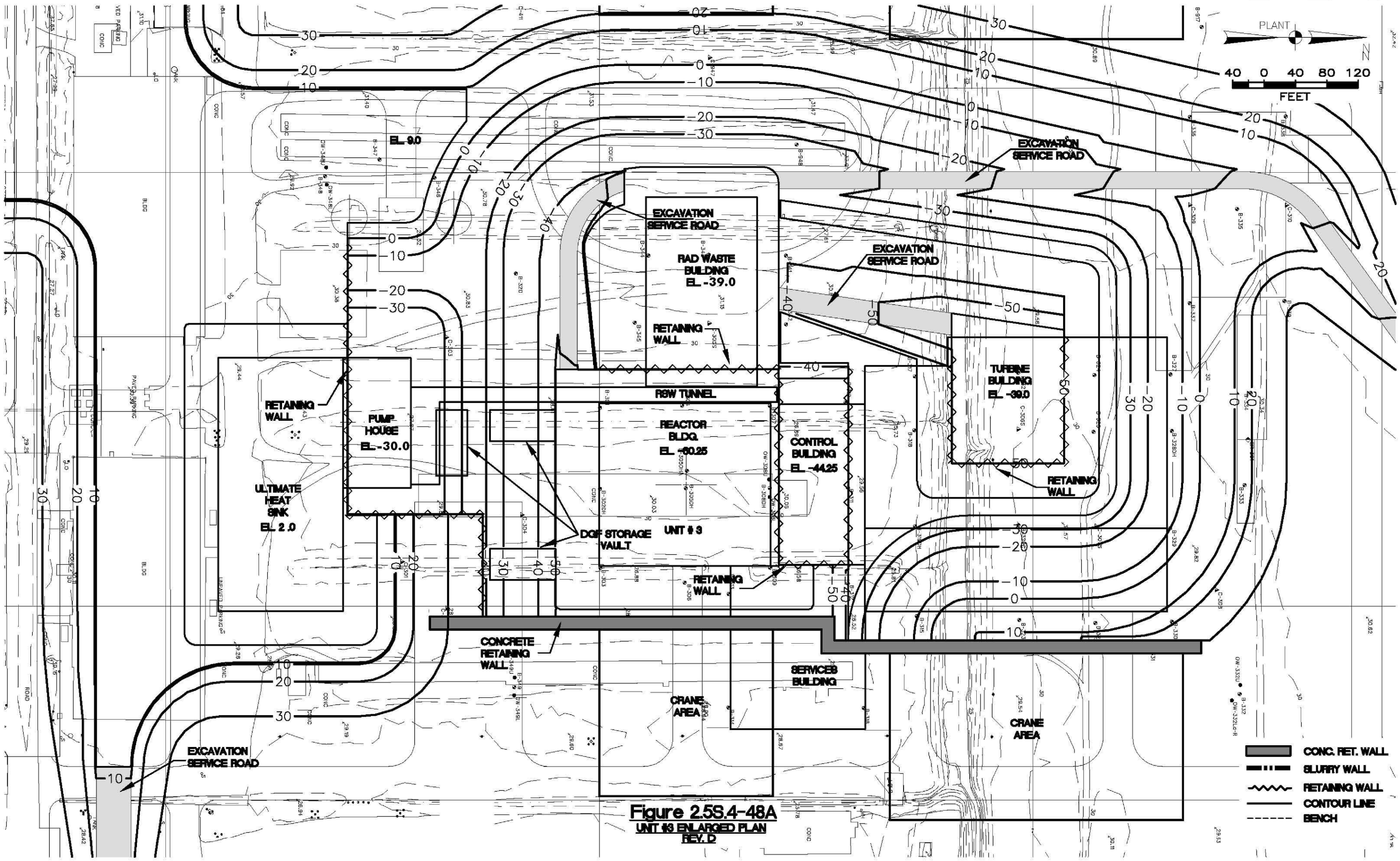


Figure 2.5S.4-48A Unit #3 Enlarged Plan Rev. D

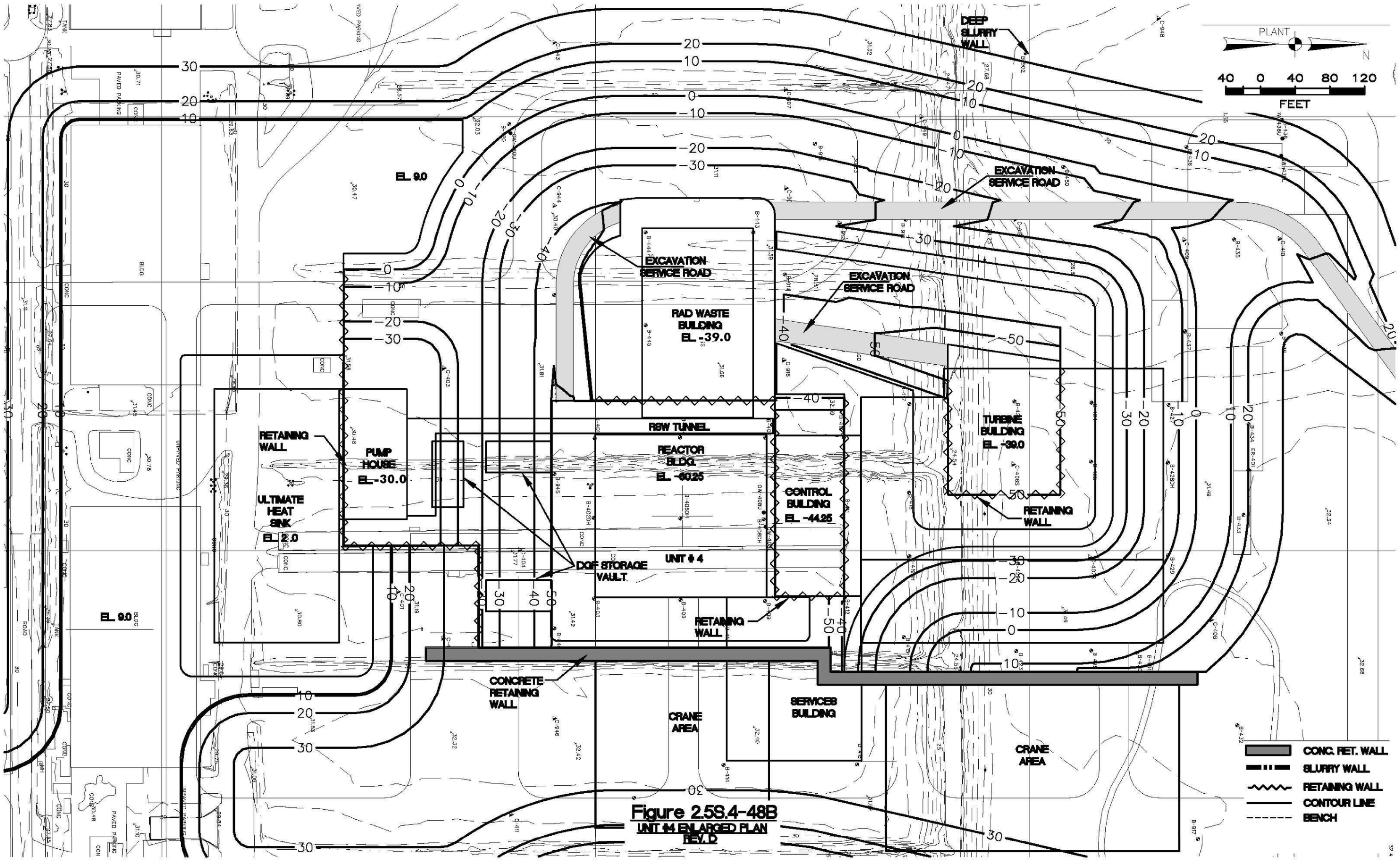


Figure 2.5S.4-48B Unit #4 Enlarged Plan Rev. D