RAI 02.04.04-9, Supplement 1, Part A:

The following changes will be made to COLA, Part 2, Tier 1, Section 5.0, Tier 2 Subsections 2.0, 2.4S.1, 2.4S.2, 2.4S.4, 2.4S.6, 2.4S.10, 2.4S.14, 2.5S.5, 3.4, and 3H, and COLA Part 7, Subsection 2.1, to incorporate the revised main cooling reservoir (MCR) embankment breach analysis as described in the responses to RAI questions 02.04.04-9 and 02.04.04-10.

Tier 1 Table 5.0 is changed as follows:

Table 5.0 ABWR Site Parameters

Maximum Flood (or Tsunami) Level:

~~*30.5 cm below grade*~~ ~~**1478.3**~~ **1219.2 cm above MSL** **Nominal plant grade of 1036.3 cm MSL** **Flood Level = ~~442.0~~ 182.9 cm above nominal plant grade**

Tier 2 Table 2.0-2 is changed as follows:

Table 2.0-2 Comparison of ABWR Standard Plant Site Design Parameters and STP 3 & 4 Site Characteristics (Continued)

Subject	ABWR Standard Plant Site Design Parameters	STP 3 & 4 Site Characteristics	Bounded (Yes/No)	Discussion
Maximum Flood (or Tsunami) Level [8]	30.5 cm (1 ft) below grade	~~442.0~~182.9 cm (~~14.5~~6.0 ft) above nominal plant grade	No	As discussed in Subsection 2.4S.4, the ~~maximum~~ design basis flood level is ~~48.5~~ 40.0 ft (~~1478.3~~ 1219.2 cm) above MSL (MSL NVGD29). This level is ~~due to~~ based on the breach of the Main Cooling Reservoir (MCR) resulting in a flood level of approximately ~~14.5~~6.0 ft (~~442.0~~182.9 cm) above nominal plant grade, which is 34 ft (1036.3 cm) MSL (see STP DEP T1 5.0-1). STP 3 & 4 safety-related structures, systems, and components (SSCs) are designed for or protected from this flooding event by watertight doors to prevent the entry of water into the Reactor Buildings and Control Buildings in case of a flood. Flooding protection requirements due to the maximum flood level are discussed in Section 3.4. In addition, an external flooding PRA analyses for STP 3 & 4 concluded that the risk from external flooding is acceptably low as discussed in Section 19R.

The fourth paragraph of Tier 2 Subsection 2.4S.1.1 is changed as follows:

> The critical safety-related flood levels resulting from a postulated instantaneous breach of the MCR embankment are discussed in Subsection 2.4S.4. Calculations show a maximum flood water level at the safety-related facilities, including the power block and the UHSs, to be El. ~~48.5~~38.8 ft MSL~~, which is determined as t~~. The Design-Basis Flood (DBF) elevation is conservatively established as 40.0 ft MSL. Specific elevations of safety-related structures and plant flood protection measures are discussed in Subsections 2.4S.2 and 2.4S.10.

The third paragraph of Tier 2 Subsection 2.4S.2.2 is changed as follows:

> The impacts of postulated dam failures on the STP 3 & 4 safety-related SSCs are discussed in Subsection 2.4S.4. Two aspects of flooding are considered. First, flood elevation at the site is investigated as a result of cascading failure of dams in the Colorado River basin and its tributaries upstream of the site. The resulting water level at the site, including coincidental wind set-up and wave run-up is ~~41.9~~34.4 ft MSL. Second, the flood elevation at the site is investigated due to the failure of the Main Cooling Reservoir (MCR) embankment. A maximum flood elevation of ~~48.5~~ 38.8 ft MSL was determined at the STP 3 & 4 site as a result of the MCR embankment breach. ~~This flood elevation of 48.5 ft MSL also constitutes the DBF at the site.~~ Based on this, the DBF is conservatively established as 40.0 ft MSL. The MCR embankment breach flood level is above the site grade and the ground floor elevation of the safety-related SSCs in the power block area. Therefore, all power block safety-related structures will require appropriate flood protection measures below elevation ~~48.5~~40.0 ft MSL, such as water tight doors and components that will prevent any flooding of the safety-related SSCs. The UHS and reactor service water (RSW) pump house is contiguous with the UHS basin. The UHS basin and RSW pump house are water tight below elevation 50 ft MSL. Flooding of these structures due to DBF is therefore precluded. Flood protection requirements are discussed in Subsection 2.4S.10.

The first paragraph of Tier 2 Subsection 2.4S.2.3.5 is changed as follows:

> The HEC-RAS computer model simulation was used to estimate the maximum water surface elevation within the STP 3 & 4 power block area. Model simulation results showed that the maximum water surface elevation within the power block area was elevation 36.6 ft MSL. This flooding elevation is higher than the power block grade elevation and the ground floor slab elevation of the safety-related SSCs. However, the local PMP water surface elevation is less than the flood elevation estimated from the postulated breach of the MCR embankment, which was estimated to be at elevation ~~48.5~~38.8 ft MSL, as discussed in Subsection 2.4S.4. Flood protection measures for the safety-related SSCs against flooding due to the MCR embankment breach are sufficient to provide protection against flood elevation due to the local PMP storm event.

Tier 2 Subsection 2.4S.4 is changed as follows:

2.4S.4 Potential Dam Failures

The following site-specific supplement addresses COL License Information Items 2.14 and 3.5.

This section addresses the SRP Section 2.4.4 Acceptance Criteria Limits from the reference Table 2.1-1, which states that the flood level from failure of existing and potential upstream or downstream water control structures will not exceed 30.5 cm (1.0 ft) below grade. The nominal plant grade for the safety facilities of STP 3 & 4 is 34.0 ft mean sea level (MSL) and the design entrance level slab elevation is 35.0 ft MSL. The design basis flood~~ing~~ level at STP 3 & 4 ~~resulting from~~ based on the worst case dam failure scenario, the postulated MCR embankment breach, was ~~estimated to be 48.5~~ conservatively established as 40.0 ft MSL, exceeding the reference ABWR DCD site parameter flood level criteria. The departure from the DCD site parameter flood level and the evaluation summary are documented in STP DEP T1 5.0-1. Subsection 2.4S.4 develops the flooding design basis for considering potential hazards to the safety-related facilities due to potential dam failures.

The STP 3 & 4 site is located on the west bank of the Colorado River in Matagorda County, Texas, about 10.5 river miles upstream of the Gulf Intracoastal Waterway (GIWW). There are a total of 68 dams with storage capacity in excess of 5000 acre-feet (AF) on the Colorado River and its tributaries upstream of the STP site. These dams and reservoirs are owned and operated by different entities including the Lower Colorado River Authority (LCRA), the U.S. Bureau of Reclamation (USBR), the Colorado River Municipal Water District (CRMWD), other local municipalities and utilities. Figures 2.4S.4-1(a) and 2.4S.4-1(b) show the locations of the 68 dams. Specific information of these dams that are relevant to the flood risk assessment of STP 3 & 4 is summarized in Table 2.4S.4-1, based on data collected primarily from the Texas Water Development Board (TWDB), Texas Commission for Environmental Quality (TCEQ), and LCRA. The six hydroelectric dams – Buchanan, Roy Inks, Alvin Wirtz, Max Starcke, Mansfield, and Tom Miller, owned and operated by LCRA are known as the Highland Lake dams.

In Texas, both private and public dams are monitored and regulated by TCEQ under the Dam Safety Program. Existing dams, as defined in Rule §299.1 Title 30 of the Texas Administrative Code (Reference 2.4S.4-1), are subject to periodic re-evaluation in consideration of continuing downstream development. Hydrologic criteria contained in Rule §299.14 of Title 30 (Table 3) on Hydrologic Criteria for Dams are the minimum acceptable spillway evaluation flood (SEF) for re-evaluating dam and spillway capacity for existing dams to determine whether upgrading is required. Similarly, on the structural considerations, evaluation of an existing dam includes, but is not limited to, visual inspections and evaluations of potential problems such as seepage, cracks, slides, conduit and control malfunctions, and other structural and maintenance deficiencies which could lead to failure of a structure.

Following the 1987 National Dam Safety Inspection Program recommendations of the Texas Water Commission, a predecessor agency of the TCEQ, to upgrade two of the Highland Lake dams due to unsafe condition, LCRA initiated a program to evaluate all six HighlandLake dams with respect to hydrologic, structural and geotechnical criteria. In 1990, LCRA began a 15-year plan of Dam Modernization Program to address the safety condition of five of the six dams. A 1992 dam safety evaluation study commissioned by LCRA (Reference 2.4S.4-2) indicates that Wirtz, Starcke, and Tom Miller Dams would be overtopped during a Probable Maximum Flood (PMF) event, and certain sections of Buchanan, Wirtz, and Tom Miller Dams could have instability problems during severe flood conditions. The concrete dam sections of Mansfield Dam, however, would be stable during the PMF. At the completion of LCRA's Dam Modernization Program in January of 2005, substantial upgrade work had been undertaken at Buchanan, Inks, Wirtz, and Tom Miller Dams to address the unsafe conditions (Reference 2.4S.4-3). Upgrade at Mansfield Dam was considered not necessary as it is able to withstand the PMF without further reinforcement. Even in the event of failures of either Buchanan, Inks, Wirtz, or Starcke dams, Mansfield Dam would hold their flood volumes without overtopping (Reference 2.4S.4-4).

The UFSAR of STP 1 & 2 (Reference 2.4S.4-5) identifies two dam failure scenarios that are most critical to the flooding at the STP site. They are: (1) the breaching of the embankment of the onsite Main Cooling Reservoir (MCR); and (2) the postulated cascade failure of the major upstream dams on the Colorado River. These two scenarios also form the basis of the maximum flood level evaluation for STP 3 & 4 resulting from potential dam failures because the watershed and topographic conditions remain relatively unchanged since the preparation of the UFSAR for STP 1 & 2, and also because there are no new dams (including the previously proposed Columbus Bend Dam) planned for the Colorado River in the next 50 years, according to the 2007 State Water Plan (Reference 2.4S.3-6, also discussed in Subsection 2.4S.3.4.2) The dam failure scenarios and the postulated flood risk are discussed further in the following subsections.

2.4S.4.1.2 Postulated Failure of the Main Cooling Reservoir

The MCR is enclosed by a rolled-earthen embankment, rising an average of 40 ft above the natural ground surface south of the plant site. The interior reservoir side slopes of the MCR embankment are lined with 2 feet thick soil cement. The centerline of the north embankment is approximately 2340 ft south of the centerline of the reactor buildings of STP 3 & 4. Site grade near the northern embankment is in the range of El. 27 ft MSL to El. 29 ft MSL, and the top of the embankment is at about El. 65.75 ft MSL. Normal maximum operating level of the reservoir is at El. 49.0 ft MSL, which is about 20 to 22 ft higher than the site grade near the northern embankment. Postulated failure mechanisms of the earth embankment includes excessive seepage from piping through the foundations of the embankment, seismic activity leading to potential liquefaction of the foundation soils, and erosion of the embankment due to overtopping from flood or wind-wave events.

As discussed in the STP 1 & 2 UFSAR (Reference 2.4S.4-5), failure of the MCR embankment due to any of these probable mechanisms is not considered a credible event. Nevertheless, it is conceivable that a failure of the internal drainage system within the MCR embankment could saturate the embankment and allow seepage through it, which could then initiate a piping failure. Therefore, a piping failure of the MCR embankment was investigated and analyzed.

The northern MCR embankment, near the proposed circulating water intake and discharge pipeline, is the most critical location for piping failure because it is closest to, and inline with, Units 3 and 4. Two breach locations were considered for the analysis, one immediately east and one immediately west of the circulating water pipeline. Further discussion of breach parameter selection is presented in Subsection 2.4S.4.2.2.2.2.

~~Nevertheless, a conservative approach was adopted in the flood risk evaluation to assume that the embankment would fail. The most conservative conjecture of such a failure suggested that an embankment section of several hundred feet long would translate downstream several tens of feet off of its original location (Reference 2.4S.4-5). This failure scenario was modeled using a 2-dimensional flood model as described in STP 1 & 2 UFSAR by assuming an instantaneous removal of a 400 ft long section of the embankment. In order to ensure sufficient freeboard in the design of the safety related facilities for flood protection, the postulated breach length was further increased from 400 ft to 4000 ft, incrementally, to determine the most critical flooding impact to the site. A 2000 ft or wider breach was found to produce the highest flood level at the safety facilities of STP 1 & 2.~~

~~A similar approach was used for STP 3 & 4 by varying the breach length in an effort to predict the maximum flood level that would be experienced by STP 3 & 4 safety related facilities as a result of the highly improbable MCR failure event.~~

2.4S.4.1.3 Potential for Landslide and Waterborne Missiles

The potential for major scale landslide, and hence blockage of streams on the Lower Colorado River in the vicinity of the STP site, is highly improbable due to the flat terrain. This is consistent with the conclusion of the UFSAR for STP 1 & 2 (Reference 2.4S.4-5). According to the investigation, there is no threat posed to the STP site due to surge from bank material sliding into the Lower Colorado River. The potential for waterborne missiles reaching the STP site due to upstream dam failure is not considered to be critical because the site is located in the flood plain of the Lower Colorado River where the flood flow velocities are in general substantially lower than that in the main channel. Although there is a potential for waterborne missiles due to the MCR's embankment breach, these missiles are not considered to be critical to the design of the safety related structures compared to tornado missiles. The static and dynamic effects of the MCR embankment breach on the plant structures are discussed in Section 3.4.

2.4S.4.2.2 MCR Embankment Breach Analysis

FLDWAV, a computer program developed by the National Weather Service (Reference 2.4S.4-12), was used to generate the outflow flood hydrograph from the MCR embankment breach, based on breach parameters discussed in Subsection 2.4S.4.2.2.2.2. This flood hydrograph was used as input to the two-dimensional flow model downstream of the breach.

RMA2 is a two-dimensional (2-D), depth-averaged finite-element hydrodynamic numerical model developed by the United States Army Corps of Engineers (USACE) (Reference 2.4S.4-12a). RMA2 was used to determine the flood elevations and velocities at the safety-related facilities of STP Units 3 and 4. The computer program can simulate dynamic water surface elevations and horizontal velocity components for subcritical, free-surface flow in a 2-dimensional flow field. The governing equations of RMA2 are the depth-integrated equations of fluid mass and momentum conservation in two horizontal directions. The governing equations are solved by finite-element method using the Galerkin Method of weighted residuals, and the integration in space is performed by Gaussian integration. Derivatives in time are replaced by a nonlinear finite difference approximation. The solution is fully implicit and the set of simultaneous equations is solved by the Newton-Raphson nonlinear iteration scheme. The computer code executes the solution by means of a front-type solver, which assembles a portion of the matrix and solves it before assembling the next portion of the matrix.

A 2-D model grid was developed based on topographic information and assigned parameters, such as Manning's roughness coefficient. Breach characteristics and a breach outflow hydrograph were incorporated into the 2-D grid, based on the breach analysis and FLDWAV results. A sensitivity analysis was conducted to evaluate the RMA2 results.

RMA2 does not have sediment transport modeling capability, and therefore, SED2D computer model (Reference 2.4S.4-12b) was used to conduct sediment transport simulation using RMA2 results as the driving hydrodynamics. The SED2D model, developed by the USACE, included a dynamic inflow load of sediments that was developed based on the breach erosion and sediment load analysis. The SED2D results were then evaluated for sediment concentrations and deposition depths at any given location. The Surface Water Modeling System (SMS) (Reference 2.4S.4-12c) was used as the pre- and post-processor for RMA2 and SED2D models.

~~The depth averaged two-dimensional (2-D) feature of the Delft3D-FLOW (Reference 2.4S.4-12) was used to evaluate the flooding potential due to the breaching of the MCR embankment. Delft3D-FLOW is a multi-dimensional hydrodynamic and transport numerical model whichsimulates non-steady flow andtransport phenomenathat result from tidal and meteorological forcing on a rectilinear or a curvilinear boundary-fitted grid length. The model solves the Navier-Stokes equation for incompressible fluid using the shallow water and the Boussinesq assumptions. In addition, for 3D simulations, the vertical turbulence eddy viscosity and turbulent diffusivity are computed by employing a turbulence closure model. The set of partial~~

~~differential equations from the Navier-Stokes equation and the turbulence closure model are solved by using finite difference based numerical schemes. Delft3D-FLOW is capable of simulating water levels and flow rates of the flood waves resulting from a breached section in an embankment (in a 2D domain). Obstructions, such as buildings and embankments can be incorporated into the model.~~

~~For simulating flood levels from the breach of the MCR, the model domain was delineated in such a way that the entire MCR is included, together with the areas surrounding the power blocks of STP 1 & 2 and STP 3 & 4, the Essential Cooling Pond (ECP) of STP 1 & 2, and the Ultimate Heat Sinks (UHS) of STP 3 & 4. The southern and eastern limits of the model domain align closely with the southern and eastern embankments of the MCR. The western and northern boundaries of the model were selected with the consideration that the maximum flood level would occur at the STP 3 & 4 power block before the flood waves reach these two downstream boundaries. No flow boundary condition was applied to the four external boundaries of the model domain.~~

~~The model domain covers an area of approximately 6910 hectares (or 17,080 acres): 6990 m (or 4.3 miles) in the west-east direction and 9890 m (or about 6.1 miles) in the north-south direction. Table 2.4S.4-5 lists the coordinates of the four corners of model domain. The numerical grid for the model was generated with Delft3D-RGFGRID module: the horizontal grid size at the power block for STP 3 & 4 is 10 m by 10 m (or 32.8 ft by 32.8 ft), the grid size for the areas away from the power block is 20 m by 20 m (or 65.6 ft by 65.6 ft), and the grid size for transitional region is 10 m by 20 m (or 32.8 ft by 65.6 ft). Because the principal direction of the propagation of the flood waves is from the south to the north, the model was also oriented in the north-south direction. Figures 2.4S.4-13 and 2.4S.4-14 show the numerical grid of the MCR embankment breach model.~~

~~The buildings of STP Units 1 & 2 and STP 3 & 4 shown in Figure 2.4S4-15 were represented in the Delft3D-FLOW model. In addition the Units 1 & 2 ECP and MCR embankment were represented in the model. All these features were modeled as "dry points" in which the flows perpendicular to the four faces of the grid cells, representing the buildings and the embankments, are blocked. Table 2.4S.4-6 depicts the buildings for which the "dry points" option was invoked. In addition, Figures 2.4S.4-15 and 2.4S.4-16 show the modeled and the physical locations of the building outlines, represented by black and blue lines, respectively.~~

2.4S.4.2.2.1 Assumptions in the MCR Embankment Breach Analysis

The following assumptions were used for the MCR embankment breach analysis:

(1) For modeling the flood elevation on the site, it was assumed that the large concrete structures such as STP Units 1 and 2 as well as Units 3 and 4, and several other tall and durable structures would remain in place during the flood. Other structures, such as metal skin buildings and warehouses, were

assumed to be removed by the high velocity flood flow but have steel framing and associated remaining debris that would result in higher friction to flow. This higher friction to flow was incorporated by using a higher Manning's *n* for those elements.

(2) The bottom elevation of the MCR ranges approximately between elevations 16.0 ft and 28.0 ft. It was assumed that the average bottom elevation of the MCR is 20 ft (6.1 m), which is a representative low bed level in MCR.

(3) Breach side slopes were assumed to be 1 vertical to 1 horizontal for FLDWAV modeling.

(4) During the breach simulation it was assumed that there was no rainfall and therefore, there was no inflow to the MCR.

(5) It was assumed that the lateral expansion of the breach would occur symmetrically about its centerline.

~~In the MCR breach analysis, the following assumptions were adopted:~~

~~(1) The failure and removal of the breached section in the MCR embankment would be instantaneous;~~

~~(2) All internal dikes within the MCR would also fail and be removed instantaneously, coincide with the breaching of the MCR embankment;~~

~~(3) The STP 1 & 2 Essential Cooling Pond (ECP) was modeled as a structure with vertical walls (no flow-through conditions);~~

~~(4) The bottom elevation of the MCR was assumed to be uniform at El. 20 ft MSL and the initial reservoir water level would be at El. 50.74 ft MSL corresponding to a one-half local PMP event (based on the local 72-hr PMP of 55.7 in. as stated in Subsection 2.4S.2) on top of the normal maximum MCR operating water level of El. 49 ft MSL. The reservoir storage volume at this MCR level (El. 50.74 ft MSL) is about 215,200 AF;~~

~~(5) The flow velocities in the MCR are zero before the instantaneous breach of the embankment;~~

~~(6) The Manning's *n* value was selected to be 0.046;~~

~~(7) The density of water is 1000 kg/m (or 1.94 slug/ft) and the background horizontal eddy viscosity is 1.0 m/s (or 10.8 ft/s), which are the default values of Delft3D-FLOW. Because inertial forces dominate the dam break flow field, the effect of eddy viscosity would not be significant and has been verified in a sensitivity test.~~

2.4S.4.2.2.2 FLDWAV Flow Model Simulation~~Bathymetry Elevations of the MCR Breach Model~~

2.4S.4.2.2.2.1 Initial (Starting) Water Level in the MCR

The starting water level in the MCR considered for the breach analysis was 50.9 feet. This level corresponds to the response of the MCR to one-half PMP on the normal maximum operating level plus the effect of wind set-up produced by the 2-year wind speed (50 mph) from the south (Reference 2.4S.4-7).

2.4S.4.2.2.2.2 Selection of the MCR Embankment Breach Parameters

Reference 2.4S.4-12d by the Dam Safety Office of the U.S. Bureau of Reclamation describes several dam failure case studies that support empirical breach parameter relationships, and is considered the most complete and knowledgeable source for estimation of dam breach parameters. The breach parameters for the MCR embankment breach analysis were established based on discussions within this reference.

The portion of the northern embankment in line with and due south of Units 3 and 4 is the closest to the units, and therefore is considered the most critical location for a breach of the MCR embankment, with respect to flooding at STP 3 & 4. The top elevation of the embankment in this area is approximately El. 65.75 ft. A service road runs along the toe of the exterior slope of the MCR northern embankment. Due to an anticipated large scour hole that would occur at the breach location, it was assumed that the road would be eroded. The terrain immediately downstream of the road is considered to be the control for the breach bottom elevation. Therefore the breach bottom elevation was taken to be at El. 29 ft. Breach side slopes were taken to be 1 horizontal to 1 vertical, a ratio consistent with observations for earth-filled structures described in Reference 2.4S.4-12d.

Empirical relationships presented in Reference 2.4S.4-12d were used to determine the breach parameters consisting of (1) breach width, (2) time to failure, and (3) estimated peak flow from the breach, which was compared later to the peak flow from the breach resulting from FLDWAV modeling. Table 2.4S.4-5 presents empirical equations from Reference 2.4S.4-12d and the resulting breach parameters.

From Table 2.4S.4-5, it can be seen that Froehlich's equation yields the largest breach width estimate of all methods presented in Reference 2.4S.4-12d. Therefore, the Froehlich equation was used to estimate the breach width because it provides a conservative result in comparison with observed dam failures. Given the trapezoidal geometry of the breach, the average breach width of 417 feet yields a bottom breach width of 380 feet (417 - 2(65.75 - 29)/2), which was used for FLDWAV embankment breach modeling.

The breach parameters estimated for the MCR embankment were also compared with the Teton Dam breach parameters, obtained from Reference 2.4S.4-12d. Teton Dam had more volume (310,000,000 m^3) and a greater breach height, h_b (86.9 m), which would allow significantly greater erosion to take place in creating the breach width. Froehlich's equation predicts an average breach width of 220 m (722 ft) for the Teton Dam. However, the actual average breach width of Teton Dam at failure was only 151 m (495 ft). Thus, Froehlich's equation over-predicts the breach width; therefore, the breach width determined for the MCR embankment using Froehlich's equation is considered conservative.

Time to failure, presented in Table 2.4S.4-5, was based on the equation given by

MacDonald and Langridge-Monopolis (Reference 2.4S.4-12d). The breach width erosion rate (380/2 = 190 ft in 1.7 hours) is 112 feet per hour, assuming erosion opens equally to right and left of centerline. In comparison, the Teton Dam displayed a fairly rapid embankment erosion rate (496/2 = 248 ft in 1.25 hours) of about 200 feet per hour (Reference 2.4S.4-12d). This rapid rate was due to the higher hydraulic depth at the time of failure, which provides more energy to drive breach propagation. The water depth in Teton Dam was more than 200 ft, whereas the water depth in the MCR is less than 30 ft. Therefore, the estimated time to failure and breach width for the MCR embankment breach are considered reasonable.

Reference 2.4S.4-12d states that the Froehlich equation as shown in Table 2.4S.4-5 is one of the better available methods for prediction of peak breach discharge, because it correlates well with observed dam failure peak flow rates. The peak discharge estimated from that equation is 62,600 cfs.

2.4S.4.2.2.2.3 MCR Embankment Breach Outflow Hydrograph

The outflow hydrograph from the MCR embankment breach, generated by FLDWAV based on the aforementioned initial conditions and breach parameters is presented in Table 2.4S.4-6. The peak breach outflow predicted by FLDWAV is 130,000 cfs, whereas Froehlich's equation estimates 62,600 cfs. The relationship of estimated peak discharges associated with the respective hydraulic head at time of failure from Reference 2.4S.4-12e is given in Figure 2.4S.4-13. From this figure, the peak flow for the MCR embankment breach is only 20,000 cfs, compared to 130,000 cfs as determined by the FLDWAV program. Therefore, the outflow hydrograph with a peak outflow of 130,000 cfs used in the breach analysis is conservative.

~~The model bathymetry, also the elevation of the bottom boundary, was established using: (1) 2007 aerial topographic survey data of the STP 3 & 4 site; (2) USGS Digital Elevation Model (DEM) data of the area (Matagorda, Palacios NE, Wadsworth, and Blessing SE tiles); and (3) grading plan of STP 3 & 4 power block as shown in Figure 2.4S.4-17. For the model area outside the coverage of the aerial survey and the grading plan, the USGS DEM data was used and the interface between the data sets is indicated in Figure 2.4S.4-18. Bathymetric data was incorporated into the model with the Delft3D-QUICKIN module (Reference 2.4S.4-12). Figures 2.4S.4-19 and 2.4S.4-20 show the model representation of the bathymetry for the entire model, and for the power block area where the safety related structures are located. Bathymetric data is referenced to MSL and therefore any ground elevation above MSL would have a negative value. The power block is rectangular in plan of about 1718 ft (523.6 m) by 1363 ft (415.4 m). The grade elevation at the center of the power block is at 36.6 ft MSL and slopes to El. 32.0 ft MSL at the four corners.~~

~~The bottom elevations of the MCR vary approximately between El. 16.0 ft MSL at the southern end to El. 28.0 ft MSL at the northern end. These elevations correspond more or less to the natural ground topography before the building of the MCR. In the model, the entire MCR adopted~~

~~conservatively a constant elevation of 20 ft MSL which is representative of the lowest lying area within the MCR.~~

2.4S.4.2.2.3 RMA2 Two-Dimensional Model Simulation~~Boundary Conditions of the MCR Breach Model~~

2.4S.4.2.2.3.1 Bathymetry Elevations and Two-Dimensional Grid Development

The topography of the STP site was used to determine model bathymetry for routing the flood flow resulting from the MCR embankment breach. The 2-D grid was developed using: (1) STP Site Topography; (2) STP Units 3 and 4 Site Grading Plan; and (3) STP Units 3 and 4 Plot Plan. The grading plan around Units 3 and 4 power block site is shown in Figure 2.4S.4-14. The grade elevation at the center of the power block is EL. 36.6 ft and slopes to El. 32 ft at the four corners. Facilities included in the model grid are the Reactor, Turbine, Control, Radwaste, Service and Hot Machine Shop buildings for Units 1 through 4. The Ultimate Heat Sinks for Units 3 and 4 and Essential Cooling Pond (ECP) for Units 1 and 2 were also included in the model grid.

The datums of the 2-D grid are in NAD 27 State Plane Texas South Central for the horizontal datum and NGVD 29 for the vertical datum. The northern embankment of the MCR was selected as the southern boundary of the 2-D grid, and road FM 521 was chosen as the northern boundary of the grid. The western and eastern boundaries of the grid were selected to be sufficiently far from Units 3 and 4 so the target area is not impacted by the model boundaries (Figure 2.4S.4-15). To assist the 2-D model stability and to further ensure that the target area is not impacted by model boundaries, a hypothetical sump was modeled along the east, north, and west boundaries of the developed 2-D grid. The use of the sump to help with model stability is a common practice in the 2-D modeling field, and the sensitivity analysis described below indicates that the hypothetical sump has no impact on model results in and around Units 3 and 4. As a result, the developed 2-D grid (excluding the artificial sump area) covers an area of 1,477 acres: 5,873 ft in the north-south direction, and 12,455 ft in the east-west direction. Figures 2.4S.4-16 and 2.4S.4-17 show the 2-D grid with elevations for the east breach and west breach, respectively. The 2-D grid includes 2,348 nodes and 1,088 elements. The size and location of these elements were selected to best represent physical features, particularly around Units 3 and 4. The areas of the 2-D elements range from about 2,500 square feet near the reactor buildings to about 144,000 square feet away from the units.

2.4S.4.2.2.3.2 Manning's Roughness Coefficients

The Manning's roughness coefficient (*n* value) for each model element was assigned based on typical values published by the United States Geological Survey (USGS) (References 2.4S.4-12f and 2.4S.4-12g) and the HEC-RAS manual (Reference 2.4S.4-12h). Each major building was evaluated on whether it would remain in place following the flood caused by a MCR embankment breach. Those buildings that were assumed to remain in place were considered "hard buildings." Any hard buildings higher than elevation 62 feet were considered to be a total blockage to the flow, and therefore were shown as blank areas in the 2-D grid. Those buildings assumed to fail were considered "soft buildings." Soft buildings were assumed to be destroyed with foundation slab remaining in the grid. These buildings were considered "high drag" areas with a higher roughness value to represent the effects of remaining frame and debris. Any buildings not included in the 2-D grid were represented by a higher Manning's n value. Due to the resolution of the grid, the Vehicle Barrier System around the power blocks was not built into the grid, but instead was represented by higher Manning's *n* value. Manning's n values assigned to each material type are listed in Table 2.4S.4-7. Figure 2.4S.4-18 shows the material types assigned to various elements in the 2-D grid. These Manning's *n* values were conservatively determined for each type of surface.

2.4S.4.2.2.3.3 Boundary Conditions

The downstream boundaries of the model were positioned far enough downstream so that the maximum flood level at the STP Units 3 and 4 safety-related buildings due to a MCR embankment breach would occur before the flood front reaches the two boundaries. A constant water surface elevation was defined for the downstream boundary condition. A sensitivity analysis was performed on the downstream boundary condition, as discussed in Subsection 2.4S.4.2.2.4.1. ~~The rectangular model domain is bounded by four no-flow boundaries. The northern and western boundaries were positioned far enough downstream so that the maximum flood level at the STP 3 & 4 safety related buildings due to a MCR breach would occur before the flood wave front reaches the two boundaries.~~

2.4S.4.2.2.4 Results of MCR Embankment Breach Analysis~~Initial Conditions of the MCR Breach Model~~

2.4S.4.2.2.4.1 Water Levels and Velocities

Critical STP 3 and 4 site locations for RMA2 model results are shown on Figure 2.4S.4-19. The variation in water surface elevation at these locations from 1.2 hours to 2.5 hours of the model simulation are presented in Figures 2.4S.4-20 and 2.4S.4-21 for the east breach and west breach, respectively. This selected period includes the peak water level and peak velocity near the plant buildings. The peak water level of 38.8 feet occurred at the Unit 4 Ultimate Heat Sink structure for the west breach scenario. Peak water surface elevations for the east breach and west breach are shown on the plan grid in Figures 2.4S.4-21(a) and 2.4S.4-21(b), respectively. Peak velocities associated with the east breach and west breach are shown in Figures 2.4S.4-21(c) and 2.4S.4-21(d), respectively. The maximum velocity of the flood flow was found to be 4.72 feet per second and occurred between Units 3 and 4 (point 8 on Figure 2.4S.4-19). The variation in velocity at locations 1 through 8 for the period containing peak velocities for the east and west breach scenarios is shown in Figures 2.4S.4-21(e) and 2.4S.4-21(f), respectively.

A sensitivity analysis was conducted to determine the effect of boundary condition on the resulting water levels. The analysis indicated that changing the water surface elevation at the downstream boundary from 32.5 feet to 34 feet does not affect the peak flood levels for the site.

2.4S.4.2.2.4.2 Effects of Sedimentation and Erosion

The MCR embankment breach analysis also considered the material eroded during the breach. The embankment material eroded is comprised mostly of clay, with a small percentage of sand from the internal drainage system and soil cement from the interior embankment slope lining. The erosion process will also produce a scour hole downstream of the breach that extends below the breach bottom elevation. The dimensions of this scour hole, based on lab results from Reference 2.4S.4-12i, are estimated to be 20 feet deep, 203 feet long and 380 feet wide. The scour hole contributes 1,543,000 cubic feet of clay to the flood flow. The material eroded from the MCR embankment contributes an additional 1,697,314 cubic feet of clay, 75,644 cubic feet of sand, and 117,562 cubic feet of soil cement. The flood flow from the MCR embankment breach would not erode the STP 3 and 4 plant site area because surfacing in this area is mostly concrete or asphalt pavement or compacted gravel and grass. The maximum velocity of 4.72 ft/s would not cause severe erosion of these surfaces, and any minor erosion around corners of the buildings would not impact the safety-related facilities of Units 3 and 4.

SED2D sediment modeling indicated some deposition on the outer edge of the model domain and little or no deposition within the STP site. These results are consistent with the sediment concentration results in that the majority of the clay and sand loads would be suspended in the flood flow and washed downstream, beyond the STP site. The soil cement lining on the interior wall of the embankment was not simulated. This material would likely enter the water as chunks or blocks as the embankment collapses, and these large concrete blocks would be carried only a short distance from the breach before settling to the bottom. The sediment loading would cease when the breach opening expansion ends; however, sediment-free high flow would continue for a long period afterwards until the water in MCR is totally emptied. This high flow period would prevent any remaining clay or sand particles from settling and would wash away any small depositions in the study area.

2.4S.4.2.2.4.3 Hydrodynamic Forces

The maximum water levels and velocities obtained near Units 3 and 4 were used to assess the hydrodynamic loadings on the plant buildings. Figures 2.4S.4-21(g) and 2.4S.4-21(h) show the time-dependent plots of the velocities at this location during the east and west breach scenarios, respectively. The peak velocities observed were 4.72 and 4.68 feet per second for the east and west breach scenarios, respectively. Figures 2.4S.4-21(g) and 2.4S.4-21(h) also show the sediment concentrations predicted by the SED2D model. The sediment-laden water density was used for hydrodynamic load calculations. The figures show that the sediment concentrations at the time and location of peak velocities would be 16.5 kg/m^3 and 15 kg/m^3 for the east and west breach scenarios, respectively. However, Figure 2.4S.4-21(g) shows a maximum concentration of 23 kg/m^3 occurring at approximately T = 1.3 hours. Conservatively, the maximum sediment concentration was used in conjunction with the maximum velocity to determine the hydrodynamic loads on the STP 3 and 4 plant facilities. Selecting a 23 kg/m^3 sediment concentration, a water density of 1023 kg/m^3 or 63.85 lb/ft^3 was used for load calculations. The maximum hydrostatic force on any plant building would be due to the depth of floodwater at the maximum water level. Hydrodynamic loads were calculated using the drag force formula with a drag coefficient conservatively set to 2.0, as presented below:

$$\text{Force (lb/ft}^2) = 2.0 \times \text{Density (lb/ft}^3) \times \text{Velocity}^2 \text{ (ft}^2\text{/sec}^2) / 2g$$

The maximum drag force due to the maximum velocity of flow near the plant buildings is estimated as 44 pounds per square foot of the projected submerged area of the buildings.

2.4S.4.2.2.4.4 Spatial Extent of Flooding Due To MCR Embankment Breach

For both the east and west MCR embankment breach scenarios flood water from the breach opening will flow through the area encompassing Units 1 and 2 and Units 3 and 4, and will spread into the area bounded by FM 521. This road has a top of road elevation of approximately 28 feet to 30 feet, as seen from the USGS topographic map of the area (Figure 2.4S.4-21(i)). North of FM 521 and west of the west MCR embankment there are levees with approximate top elevations of 29 feet to 30 feet. South of the MCR along its south embankment is an east – west canal with levees on both sides. The area around the STP plant has an approximate grade elevation varying from 25 feet to 30 feet.

The area around the STP plant slopes east towards the Colorado River. Therefore, most of the flood water from the breach would flow to the Colorado River. A portion of the breach flow will also reach the Little Robins Slough to the west, which flows south along the west MCR embankment. From there, the water will either flow east to the Colorado River or will flow under the east-west canal through existing siphons and may flow through several swampy areas to the intracoastal waterway.
It is unlikely that the breach flood water will overflow over FM 521 and west levees. If this happens, a small portion of the breach flood flow may reach the Tres Palacios River to the west of the STP site.

~~The initial water level in the MCR was specified at El. 50.74 ft MSL corresponding to the local one half PMP (as discussed in Subsection 2.4S.2, the local 72-hr PMP is 55.7 in.). Outside of the MCR, four different initial downstream flood levels: El. 32.0 ft, El. 34.0 ft, El. 36.0 ft and dry condition, were evaluated as part of a sensitivity test. The maximum flood level at the safety related facilities of STP 3 & 4 were found to be independent of the initial flood depths within the plant site.~~

~~The initial flow velocities in the model domain were all set to zero.~~

~~2.4S.4.2.2.5 Selection of the MCR Breach Model Parameters~~

~~The surface roughness in the model was represented by Manning's *n* value. Based on the UFSAR of STP 1 & 2, Reference 2.4S.4-5, Manning's *n* was specified as 0.046 uniformly in the two principal directions (east-west and north-south) throughout the model domain. This relatively high Manning's *n* was used to account for the smaller buildings and structures between the MCR and the power blocks of STP 1 & 2 and STP 3 & 4 that were not specifically included in the model.~~

~~The simulations were run at a model time step of 0.01 minutes (0.6 seconds), which was selected based on a verification effort to demonstrate the time step independence of the model results.~~

~~2.4S.4.2.2.6 Flood Levels from the MCR Breach~~

~~Similar to the approach used in the MCR breach simulation detailed in UFSAR of STP 1 & 2, multiple embankment breach widths (also referred to as breach lengths) were investigated with the Delft3D model. The breached widths simulated vary from 190 m (or 623 ft) to 1690 m (or 5545 ft), with the centerline of the breached section aligned with the centerline of the STP 3 & 4 reactor buildings. The resulting maximum flood levels at the safety buildings in the STP 3 & 4 power block for the various simulated breached widths are presented in Table 2.4S.4-7, which indicates that a maximum flood level of El. 48.5 ft MSL at STP 3 & 4 would occur at a breached width of about 1450 m (or 4757 ft). This maximum flood level would occur at the southern face of the STP 3 & 4 UHS. For design purpose, all safety related buildings for STP 3 & 4 are designed against the maximum flood level of 48.5 ft MSL.~~

~~Figure 2.4S.4-21 details the time history of the simulated flood level at the southern face of the STP 3 & 4 UHS. As indicated in the figure, the flood wave arrives at the building in about 1.25 minutes after the embankment breaches, and a quasi-steady state flow regime is sustained for about 15 minutes (between 4 and 19 minutes after the embankment breach). Thereafter, the flood level drops because of the receding storage volume and water level in the MCR.~~

~~Coincidental wind set-up and wave run-up were not added to the highly conservative MCR breach flooding level because this flooding has a short time scale and would not sustain for a period long enough for any considerable wind-wave action. Further, the buildings and facilities in the vicinity of the safety related structures of STP 3 & 4 would have limited the fetch to a small distance such that the generation of effective wind waves is considered unlikely.~~

~~The static and dynamic effects of the MCR's northern embankment breach on the plant structures are discussed in Section 3.4.~~

2.4S.4.3 Water Level at the STP 3 & 4 Site

Analyses of the dam failures on the Lower Colorado River and the failure of the MCR northern embankment showed that the critical flood level of the safety related structures is controlled by the MCR embankment failure. The design basis flood level for the safety related facilities of STP 3 & 4 is ~~therefore 48.5~~ conservatively established as 40.0 ft MSL as discussed below.

2.4S.4.3.1 Water Level at the STP 3 & 4 Site from the Failures of Upstream Dams

In accordance with the guidelines in ANSI/ANS-2.8, Reference 2.4S.4-7, the maximum dam breach flood level at the plant site needs to consider the wind setup and wave runup effect from the coincidental occurrence of a 2-year design wind event. The 2-year fastest mile wind speed at the site is 50 mph based on Reference 2.4S.4-7. The methodology given by the Coastal Engineering Manual (CEM), Reference 2.4S.4-13, was adopted to estimate the wave height and wave run-up at STP 3 & 4 power block. The procedures outlined in CEM use the wind speed, wind duration, water depth, and over-water fetch distance, and the run-up surface characteristics as input. As discussed in UFSAR for STP 1 & 2 (Reference 2.4S.4-5), accurate estimates of the fetch length for this flooding scenario could not be made. Based on the topographic variations and any man-made features that would limit wind effects, however, two critical fetches were identified as shown in Figure 2.4S.4-22; one in an easterly direction towards a low lying ridge and the other along the Colorado River in a northeasterly direction. The fetch in the easterly direction was estimated to be about 15.5 miles with a maximum water depth varying from 1 to 23 ft at the peak of the dam break flood. The fetch along the northeasterly direction was estimated to be about 17.6 miles, with a maximum water depth varying from 1 to 9 ft at the flood peak.

The maximum wind set-up for the critical fetch lines was estimated using a method suggested in Reference 2.4S.4-14, and was found to be about 3.9 ft. Adding to the maximum water level of El. 28.6 ft MSL, estimated by the HEC-RAS dam break model for the STP site, the water level from the dam failure flooding scenario would therefore be at El. 32.5 ft MSL. With the surrounding site grade around the power block and UHS at a nominal elevation of 28.0 ft MSL, the water depth approaching at the STP power block and UHS would be about 4.5 ft. At this shallow depth, a breaking wave condition would prevail and a breaking wave index of 0.78 was used in estimating the break wave height. The breaking wave setup is typically small and is assumed to have a negligible impact on the flood level.

All the safety-related facilities including the UHS are located in the power block island. The power block island will have a high point of grade elevation of 34.0 ft near the plant buildings and will slope towards the periphery to an elevation of 32.0 ft at the edges. The outward slope of the island will be at 10H:1V from elevation 32.0 ft to an existing grade elevation of 28.0 ft.

The maximum wave run-up was estimated using the breaking wave height of 3.5 ft and a maximum wave period equal to 1.2 times of the significant wave period which was estimated to be 3.7 seconds. Conservatively assuming that the run-up surface is smooth, impermeable and ~~at~~using a slope of ~~2H~~10H:1V for the power block island, the wave run-up was estimated to be ~~about 9.4~~1.9 ft.

The maximum flood level at STP 3 & 4 power block as a result of the probable worst case dam failure scenario coincidental with a 2-year design wind of 50 mph was estimated to be at El. ~~41.9~~34.4 ft MSL. Table 2.4S.4-8 presents the water levels due to dam break, wind set-up and wave run-up at STP 3 & 4 for the ~~two~~ critical fetches.

Because the STP is about 300 miles from Mansfield Dam, any dynamic effects of the dam break waves would have been attenuated along this distance. Therefore, the dynamic effects of the dam break flood waves are not the controlling design criterion of the safety related facilities.

2.4S.4.3.2 Water Level at the STP 3 & 4 Site from Breaching of MCR Embankment

The maximum water level at STP 3 & 4 is governed by the postulated breaching of the MCR's northern embankment. The ~~maximum water~~design basis flood level at the power block and UHS of STP 3 & 4 ~~due to~~based on the breaching of the MCR's northern embankment is at El. 40.0~~48.5~~ ft MSL. Because the ~~maximum water~~design basis flood level is higher than both the nominal plant grade of 34.0 ft MSL ~~as well as~~and the entrance level slab elevation of 35.0 ft MSL for the STP 3 & 4 safety related facilities, all safety related facilities are designed to be water tight at or below elevation ~~48.5~~40.0 ft MSL. All ventilation openings of safety buildings are located at ~~48.5~~40.0 ft MSL or above. Flood protection design is discussed in Subsection 2.4S.10 and Section 3.4.

2.4S.4.3.3 Sedimentation and Erosion

During an upstream dam failure event, because the plant site is located in the floodplains of the Colorado River, the flow velocities are expected to be relatively small compared to that in the main channel. In addition, the flow depths on the floodplain are shallower to effect any significant erosion that would impact the safety of the plant. Although some sedimentation may occur near the plant site, the safety related structures and functions would not be affected by siltation because they are located at higher grades than the surrounding area.

The erosion ~~concern~~ and sedimentation during a MCR embankment breach event is discussed in Subsections 2.4S.4.2.2.4.2 and 2.4S.10.

2.4S.4.4 References

2.4S.4-12 D. L. Fread and J. M. Lewis, "NWS FLDWAV Model Theoretical Description and User Documentation," Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, U.S. National Oceanic and Atmospheric Agency, Silver Spring, Maryland, 1998.

2.4S.4-12a "User's Guide to RMA2 WES," Version 4.5, Coastal and Hydraulics Laboratory, Waterways Experiment Station, Engineer Research and Development Center, U.S. Army Corps of Engineers, April 22, 2005.

2.4S.4-12b "User's Guide to SED2D WES", Version 4.5., Coastal and Hydraulics Laboratory, Waterways Experiment Station, Engineer Research and Development Center, U.S. Army Corps of Engineers, April 14, 2003.

2.4S.4-12c Surface-water Modeling System (SMS), Version 10.0.7, Aquaveo, August 29, 2008.

2.4S.4-12d T. L. Wahl, "Prediction of Embankment Dam Breach Parameters, A Literature Review and Needs Assessment", Dam Safety Research Report DSO-98-004, Dam Safety Office, Water Resources Research Laboratory, U.S. Department of the Interior, Bureau of Reclamation, July 1998.

2.4S.4-12e "Guidelines for Defining Inundated Areas Downstream from Bureau of Reclamation Dams," Reclamation Planning Instruction No. 82-11, Dam Safety Office, U.S. Department of Interior, Bureau of Reclamation, Denver, Colorado, 1982.

2.4S.4-12f G. J. Arcement and V. R. Schneider, "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains," Water-Supply Paper 2239, United States Geological Survey, 1989.

2.4S.4-12g "Flood Damage Evaluation Project," Chapter 1-6, Volume II-C, Volume II-B, Hall Associates, Inc., July 2002.

2.4S.4-12h "HEC-RAS, River Analysis System, User's Manual," Version 3.1.3, U.S. Army Corps of Engineers, Hydrologic Engineering Center, May 2005.

2.4S.4-12i Z. Xiuzhong and W. Guangqian, "Flow Analysis and Scour Hole Computation of Dyke-Breach.," Proceedings of the International Association for Hydraulic Researchers XXIX Congress, Theme E, Tsinghua University, Beijing, China, September 16-21, 2001

The following is a list of new and revised tables and figures for Tier 2, Subsection 2.4S:

Revised Tables:	
Table	Title
2.4S.4-5	MCR Dike Breach Parameters and Peak Discharge Based on Empirical Equations from Reference 2.4S.4-12d
2.4S.4-6	MCR Dike Breach Outflow Hydrograph
2.4S.4-7	Material Types and Associated Manning's n
2.4S.4-8	Estimated Water Levels due to Dam Break, Wind Setup and Wave Run-up

Revised Figures:	
Figure	Title
2.4S.4-13	Outflow Rates Experienced from Breached Dams
2.4S.4-14	Units 3 and 4 Site Grading Plan
2.4S.4-15	STP Site Layout
2.4S.4-16	Two-Dimensional View of Developed 2-D Grid with an East Breach
2.4S.4-17	Two-Dimensional View of Developed 2-D Grid with an West Breach
2.4S.4-18	Assigned Material Types within Developed 2-D Grid
2.4S.4-19	Locations for RMA2 Modeling Results
2.4S.4-20	Time-Dependent Water Surface Elevations Associated with East Breach Scenario
2.4S.4-21	Time-Dependent Water Surface Elevations Associated with West Breach Scenario

New Figures:	
2.4S.4-21(a)	Peak Water Surface Elevations Associated with East Breach Scenario (at time = 1.75 hours after initiation of breach)
2.4S.4-21(b)	Peak Water Surface Elevations Associated with West Breach Scenario (at time = 1.75 hours after initiation of breach)
2.4S.4-21(c)	Peak Velocities Associated with East Breach Scenario (at time = 1.75 hours after initiation of breach)
2.4S.4-21(d)	Peak Velocities Associated with West Breach Scenario (at time = 1.75 hours after initiation of breach)
2.4S.4-21(e)	Time-Dependent Velocities Associated with East Breach Scenario
2.4S.4-21(f)	Time-Dependent Velocities Associated with West Breach Scenario
2.4S.4-21(g)	Velocities and Sediment Concentrations In Between Units 3 and 4 with East Breach Scenario
2.4S.4-21(h)	Velocities and Sediment Concentrations In Between Units 3 and 4 with West Breach Scenario
2.4S.4-21(i)	Stream System around STP Site

COLA Tier 2 Table 2.4S.4-5 is deleted in its entirety and replaced with the following:

Table 2.4S.4-5 MCR Embankment Breach Parameters and Peak Discharge Based on Empirical Equations from Reference 2.4S.4-12d

Parameter	Equation	Results
Time to Failure (hrs)	$t_f = 0.0179(0.0261(V*h_w)^{0.769})^{0.364}$	1.7 hours
Average Breach Width (m)	$B_{ave} = 0.1803\,V^{0.32}\,h_b^{0.19}$	127 m (417 ft)
Peak Q (m^3/s)	$Q_p = 0.607\,V^{0.295}\,h_w^{1.24}$	1172.8 m^3/s (62,600 cfs)
B_{ave} = average breach width h_w = depth of water above breach in m = 50.9' – 29' = 21.9' = 6.7 m h_b = the height of breach from the top of embankment in m = 66' – 29' = 37' = 11.3 m V = volume of water in the MCR between El. 29' and El. 50.9' in m^3 = 188,400,000 m^3 (152,700 ac-ft)		

COLA Tier 2 Table 2.4S.4-6 is deleted in its entirety and replaced with the following:

Table 2.4S.4-6 MCR Embankment Breach Outflow Hydrograph

Time (hours)	Flow (cfs)	MCR Water Surface Elevation (ft)
0	0	50.90
0.1	1,100	50.90
0.2	3,970	50.89
0.3	8,570	50.88
0.4	15,500	50.87
0.5	24,700	50.85
0.6	30,600	50.82
0.7	37,200	50.78
0.8	47,300	50.73
0.9	54,700	50.68
1.0	79,600	50.59
1.1	85,900	50.49
1.2	92,300	50.40
1.3	100,700	50.28
1.4	108,500	50.15
1.5	116,100	50.03
1.6	123,500	49.88
1.7	130,000	49.74
1.8	126,700	49.58
1.9	124,500	49.46
2.0	122,600	49.29
2.1	120,800	49.13
2.2	119,000	49.00
2.3	117,400	48.86
2.4	115,600	48.70
2.5	113,900	48.56
3	112,800	47.88
6	83,150	44.44
9	63,030	41.86
12	48,890	39.88
15	38,680	38.32
18	31,110	37.08
21	25,390	36.07
24	21,000	35.24
27	17,560	34.56
30	14,840	33.98

COLA Tier 2 Table 2.4S.4-7 is deleted in its entirety and replaced with the following:

Table 2.4S.4-7 Material Types and Associated Manning's *n*

Material Type	Manning's *n*
Water	0.030
Short Hard Building	0.100
Soft Building / High Drag	0.085
Vehicle Barrier Walls (VBW)	0.085
Gravel	0.035
Open Space	0.040
Concrete Slab	0.012
Road (Concrete)	0.013
Channel	0.040
Pipeline	0.100
Artificial Sump	0.100

COLA Tier 2 Table 2.4S.4-8 is revised as follows:

Table 2.4S.4-8 Estimated Water Levels due to Dam Break, Wind Setup and Wave Run-up

	Dam Break Water Level (ft MSL)	Wind Setup (ft)	Wave Run-up (ft)	Water level at STP Site (ft MSL)
Fetch A ~~(I)~~	28.6	3.9	~~9.4~~ 1.9	~~41.9~~ 34.4
~~Fetch A (II)~~	~~27.8~~	~~4.2~~	~~8.4~~	~~40.4~~
~~Fetch B (I)~~	~~28.6~~	~~3.9~~	~~9.3~~	~~41.8~~
~~Fetch B (II)~~	~~27.8~~	~~4.0~~	~~7.9~~	~~39.7~~

~~Note: (I) - Base Case; (II) - Sensitivity Case~~

COLA Tier 2 Figure 2.4S.4-13 is deleted in its entirety and replaced with the following:

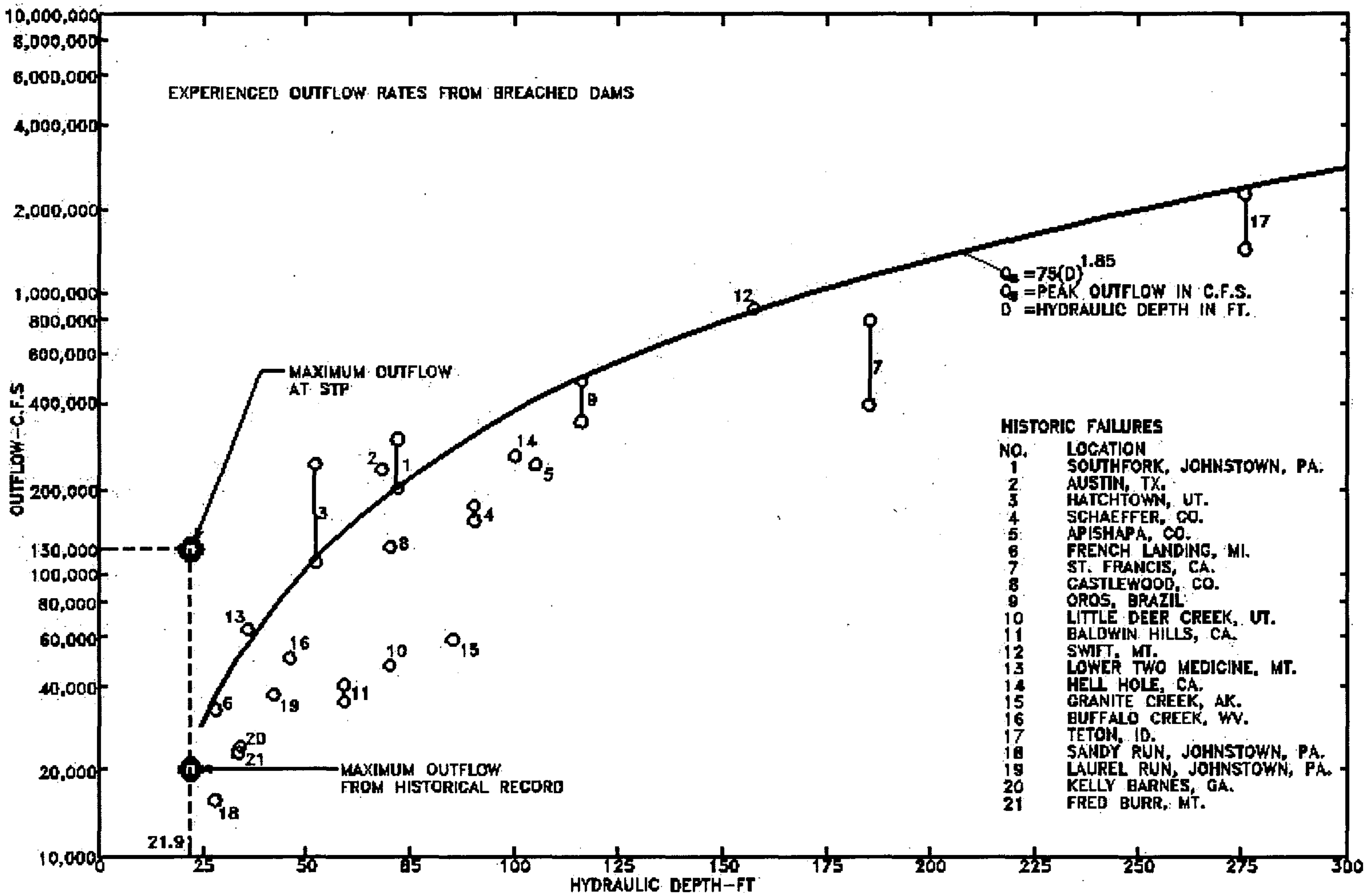

Figure 2.4S.4-13: Outflow Rates Experienced from Breached Dams (Reference 2.4S.4-12e)

COLA Tier 2 Figure 2.4S.4-14 is deleted in its entirety and replaced with the following:

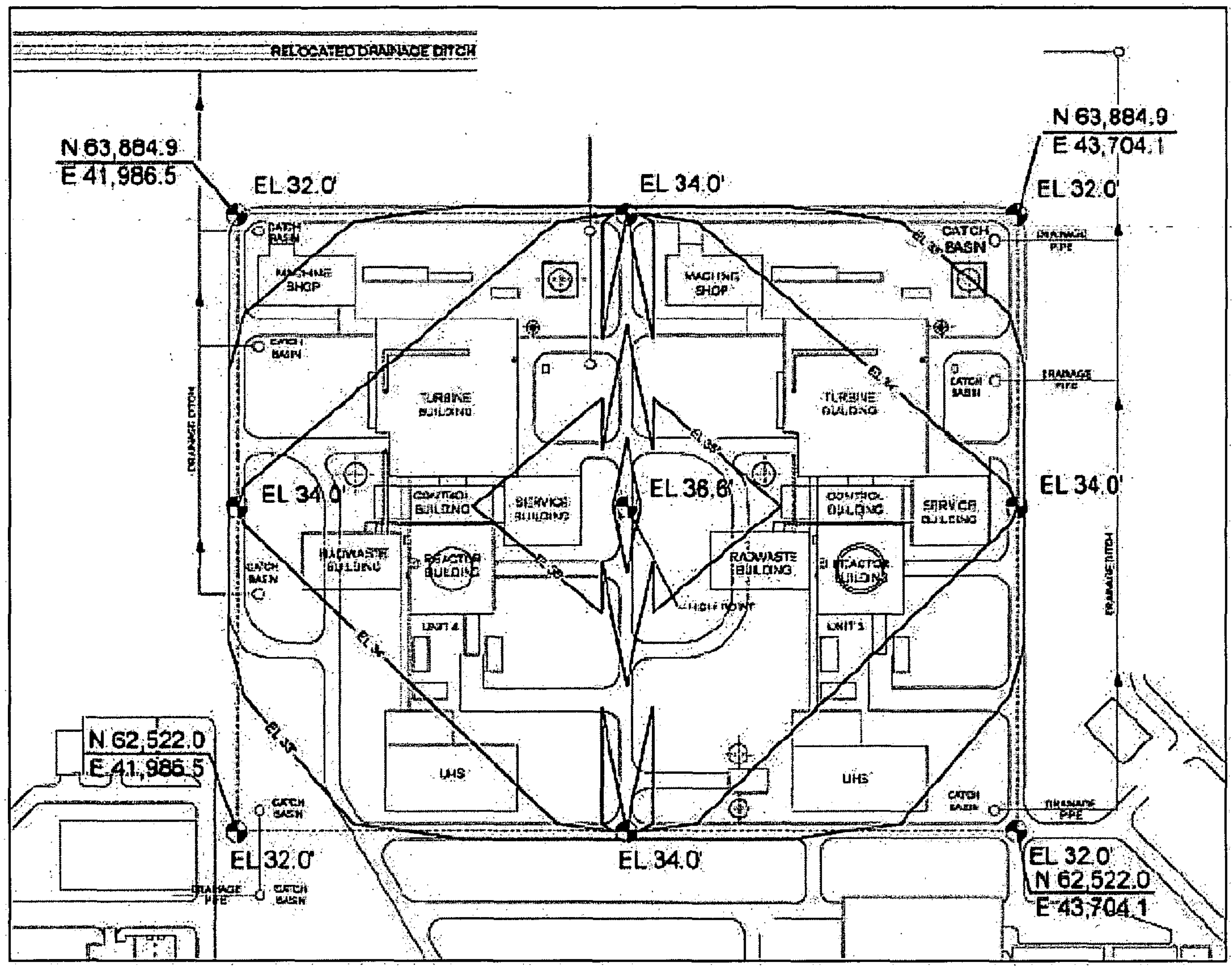

Figure 2.4S.4-14: Units 3 and 4 Site Grading Plan

COLA Tier 2 Figure 2.4S.4-15 is deleted in its entirety and replaced with the following:

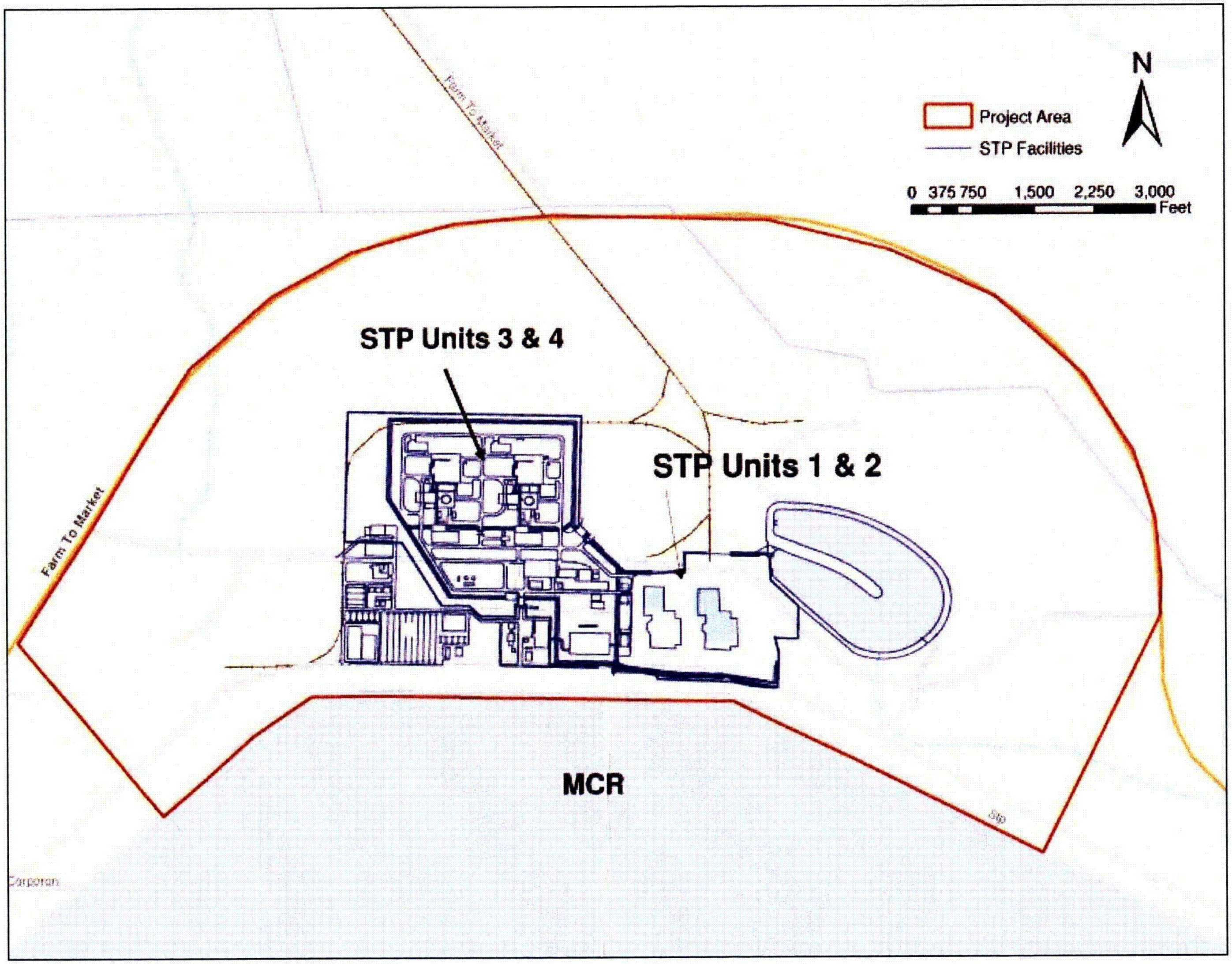

Figure 2.4S.4-15: STP Site Layout

COLA Tier 2 Figure 2.4S.4-16 is deleted in its entirety and replaced with the following:

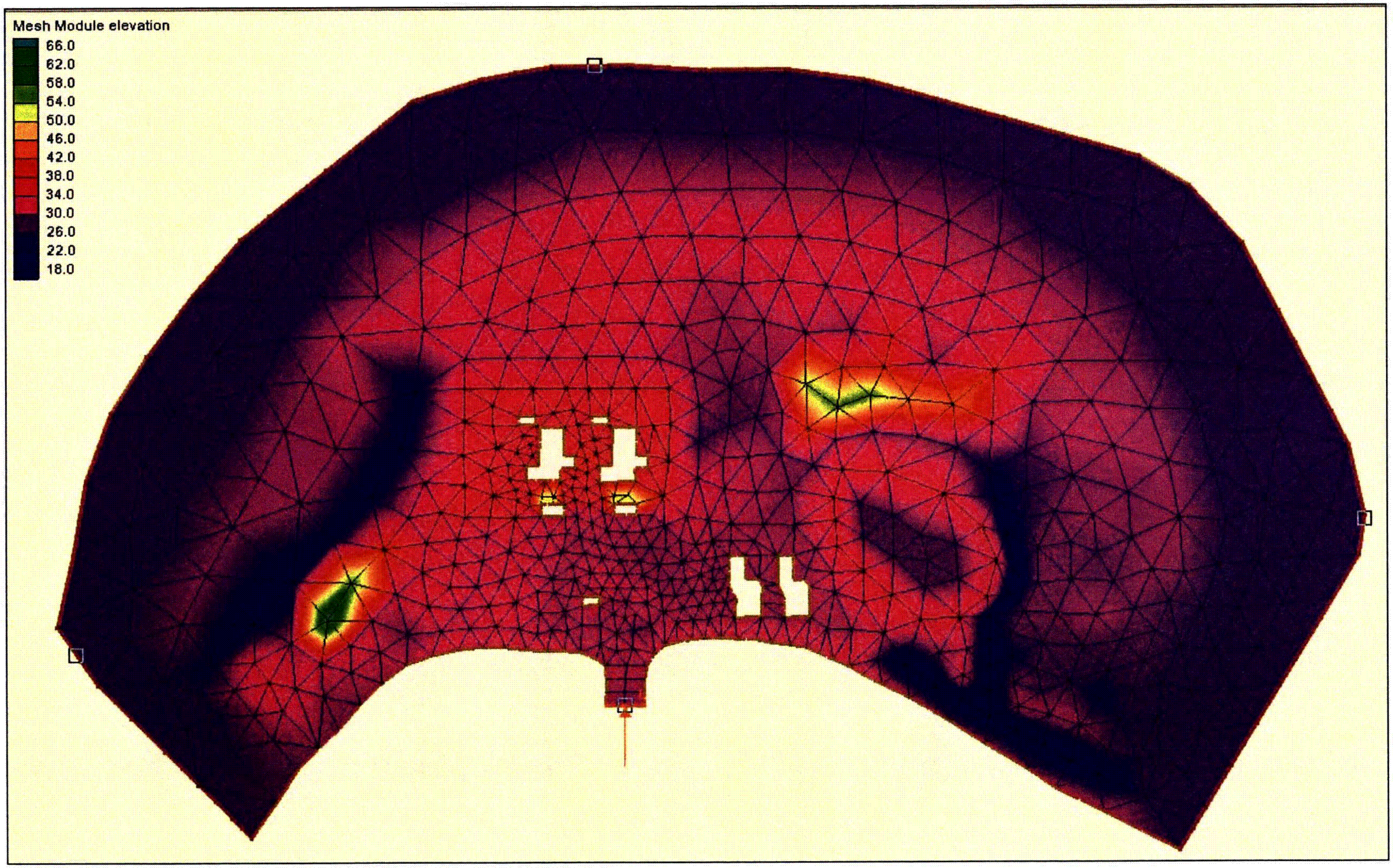

Figure 2.4S.4-16: Two-Dimensional View of Developed 2-D Grid with an East Breach

COLA Tier 2 Figure 2.4S.4-17 is deleted in its entirety and replaced with the following:

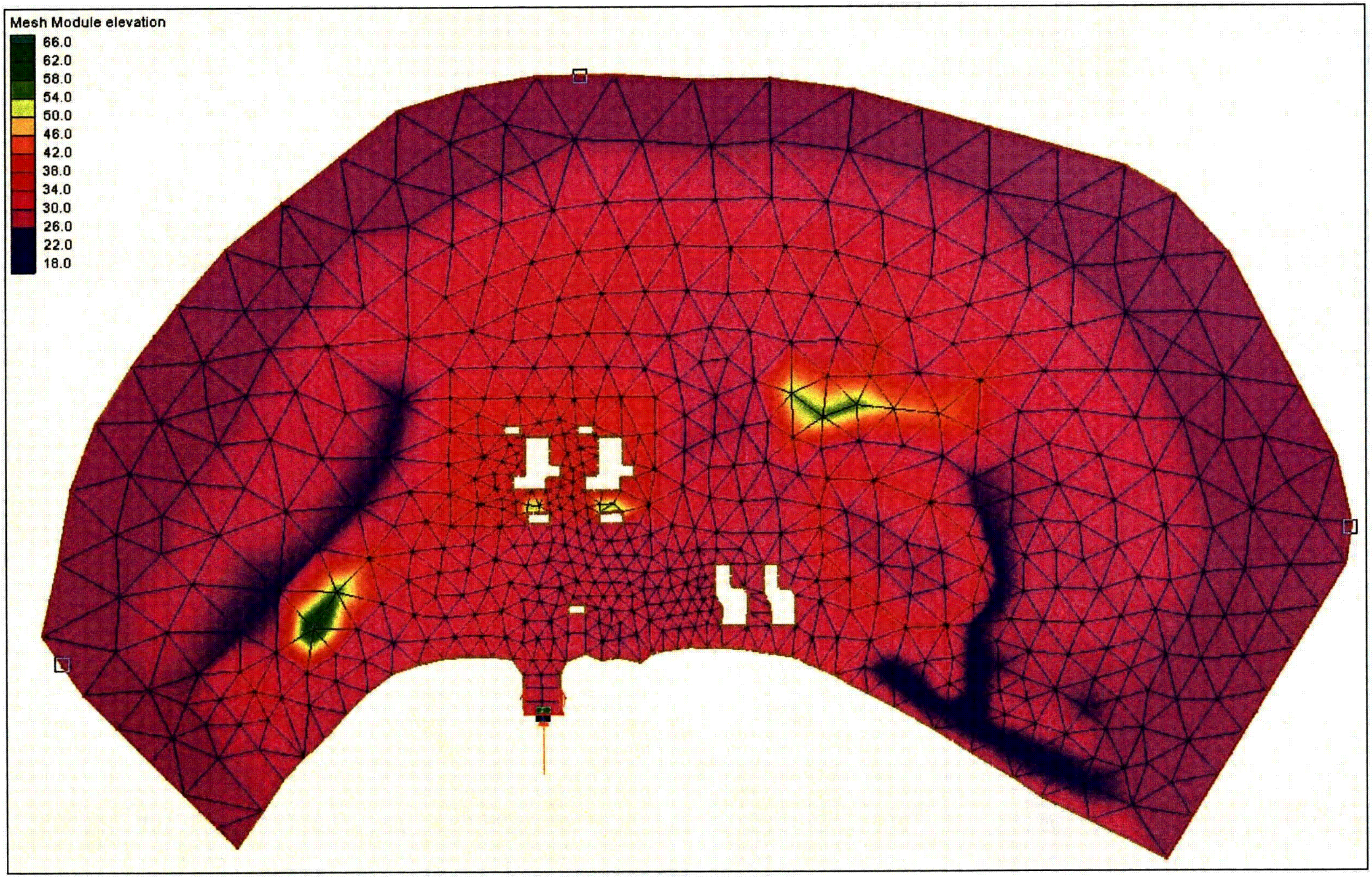

Figure 2.4S.4-17: Two-Dimensional View of Developed 2-D Grid with a West Breach

COLA Tier 2 Figure 2.4S.4-18 is deleted in its entirety and replaced with the following:

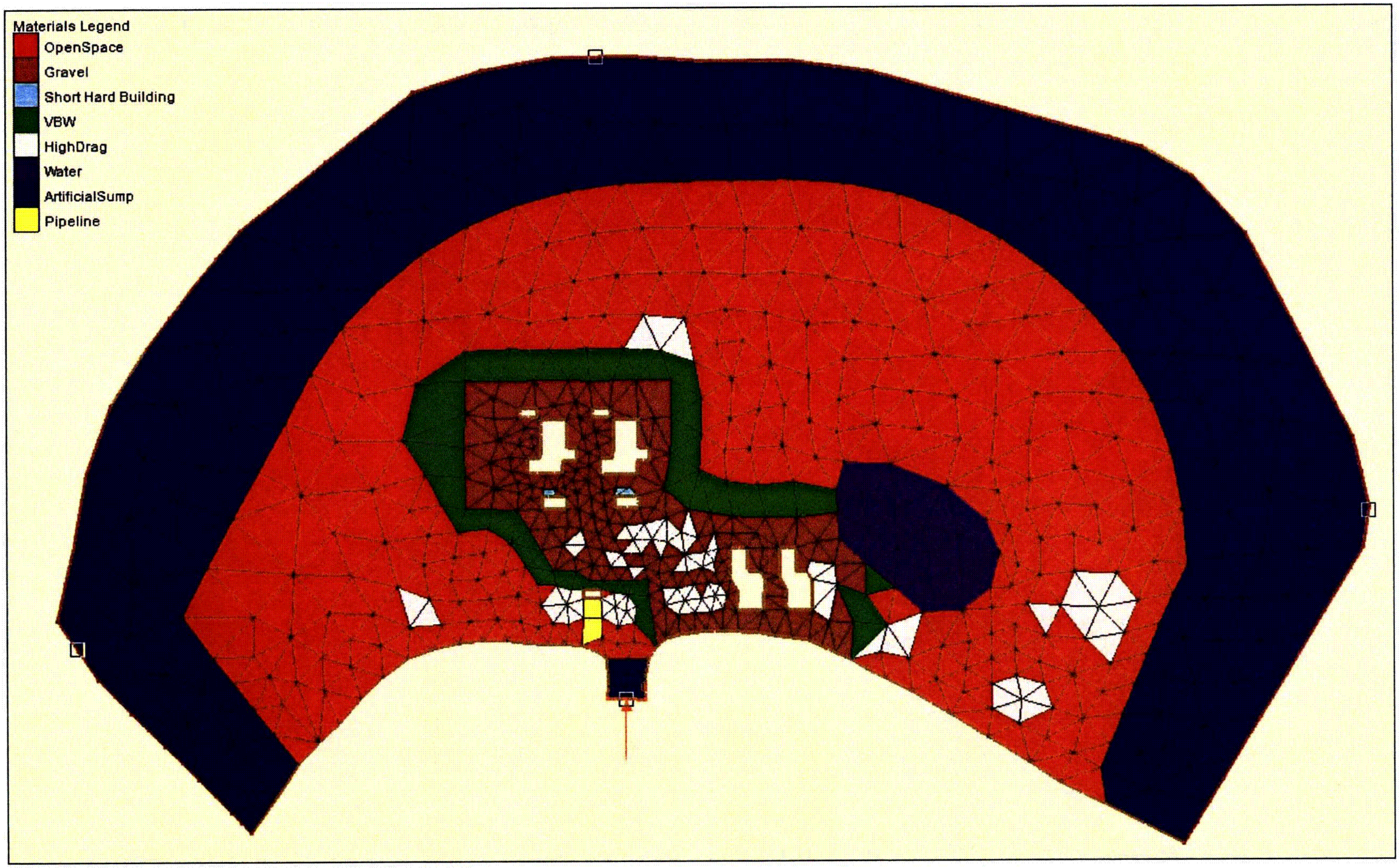

Figure 2.4S.4-18: Assigned Material Types of Developed 2-D Grid

COLA Tier 2 Figure 2.4S.4-19 is deleted in its entirety and replaced with the following:

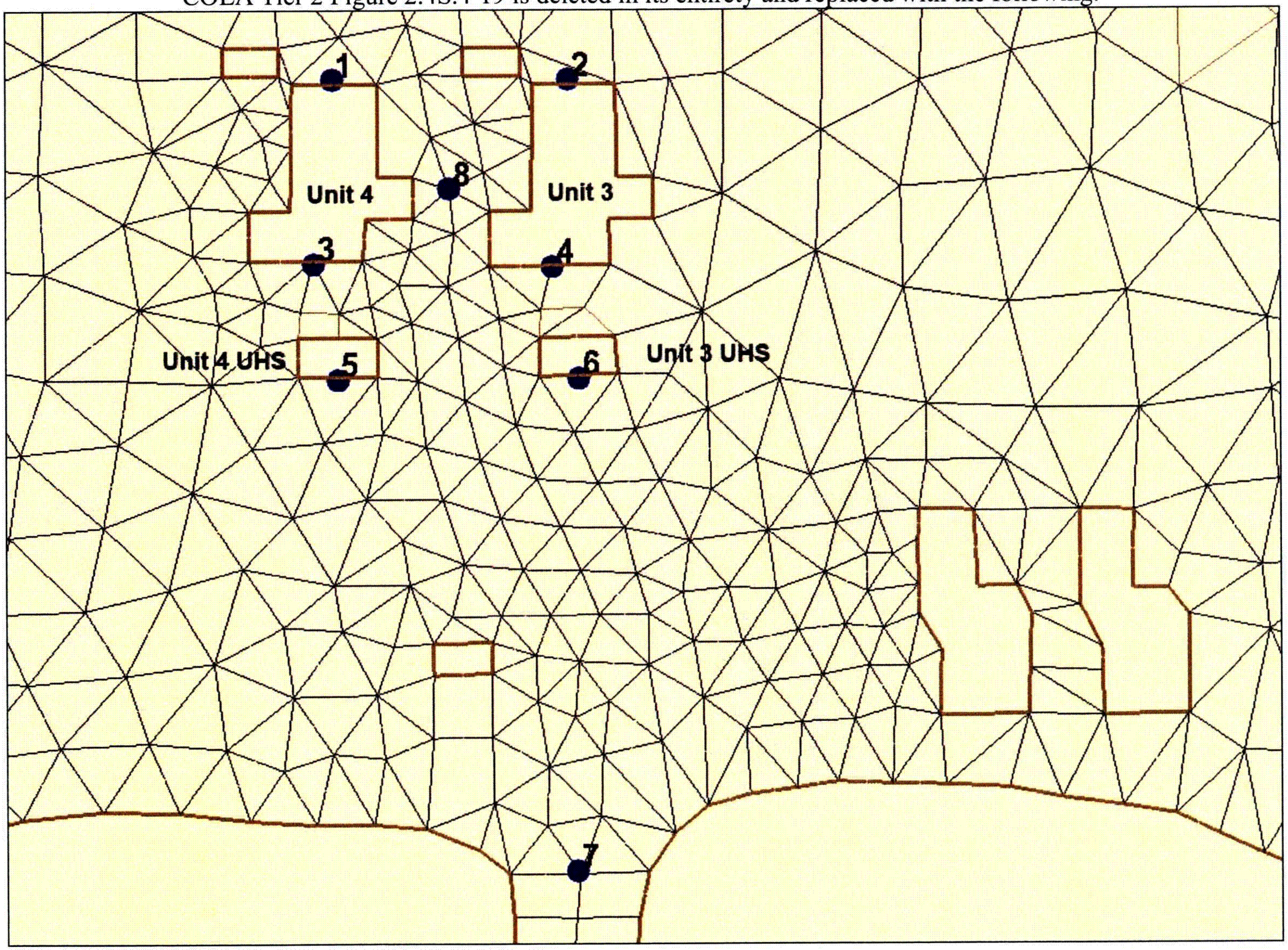

Figure 2.4S.4-19: Locations for RMA2 Modeling Results

COLA Tier 2 Figure 2.4S.4-20 is deleted in its entirety and replaced with the following:

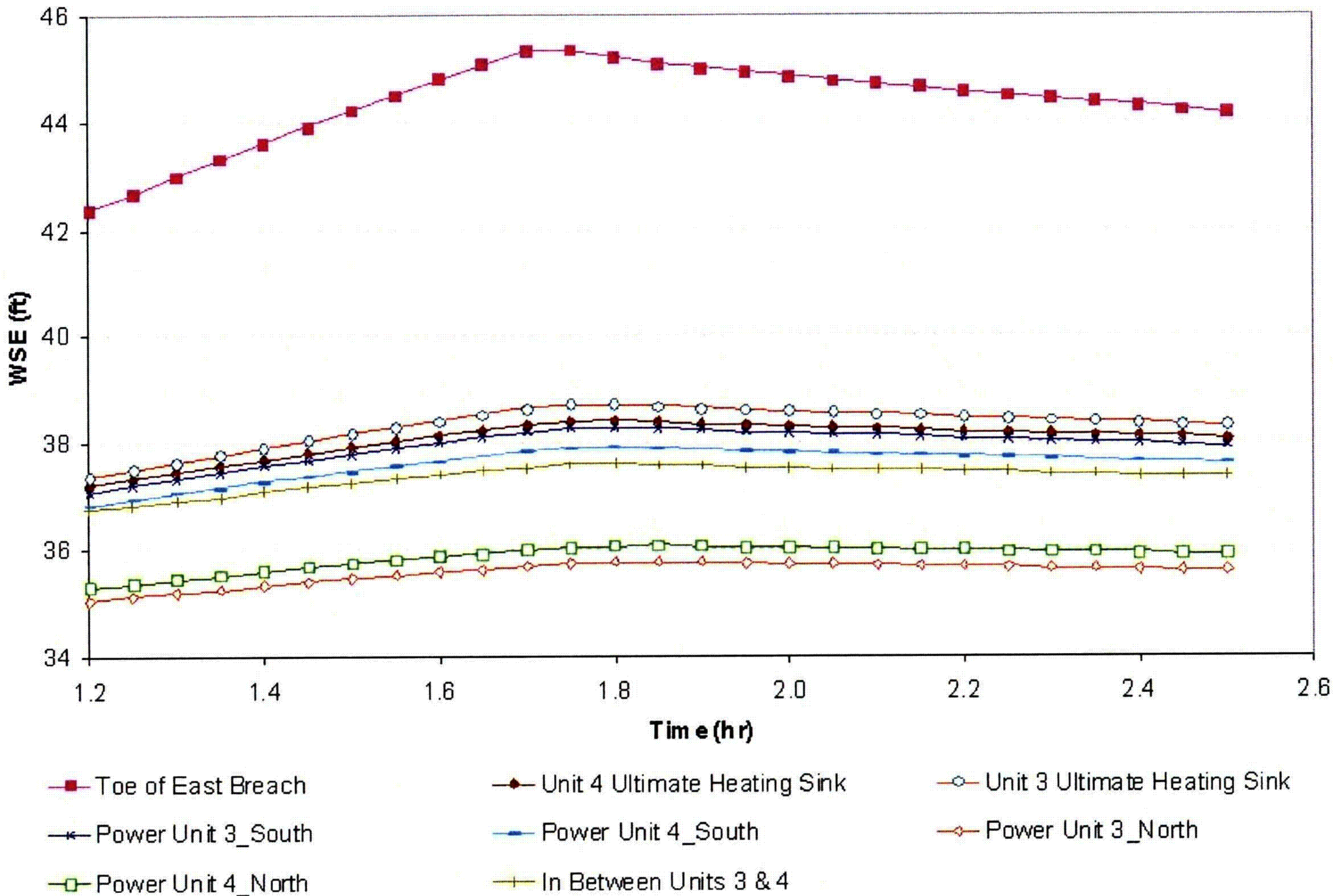

Figure 2.4S.4-20: Time-Dependent Water Surface Elevations Associated with East Breach Scenario

COLA Tier 2 Figure 2.4S.4-21 is deleted in its entirety and replaced with the following:

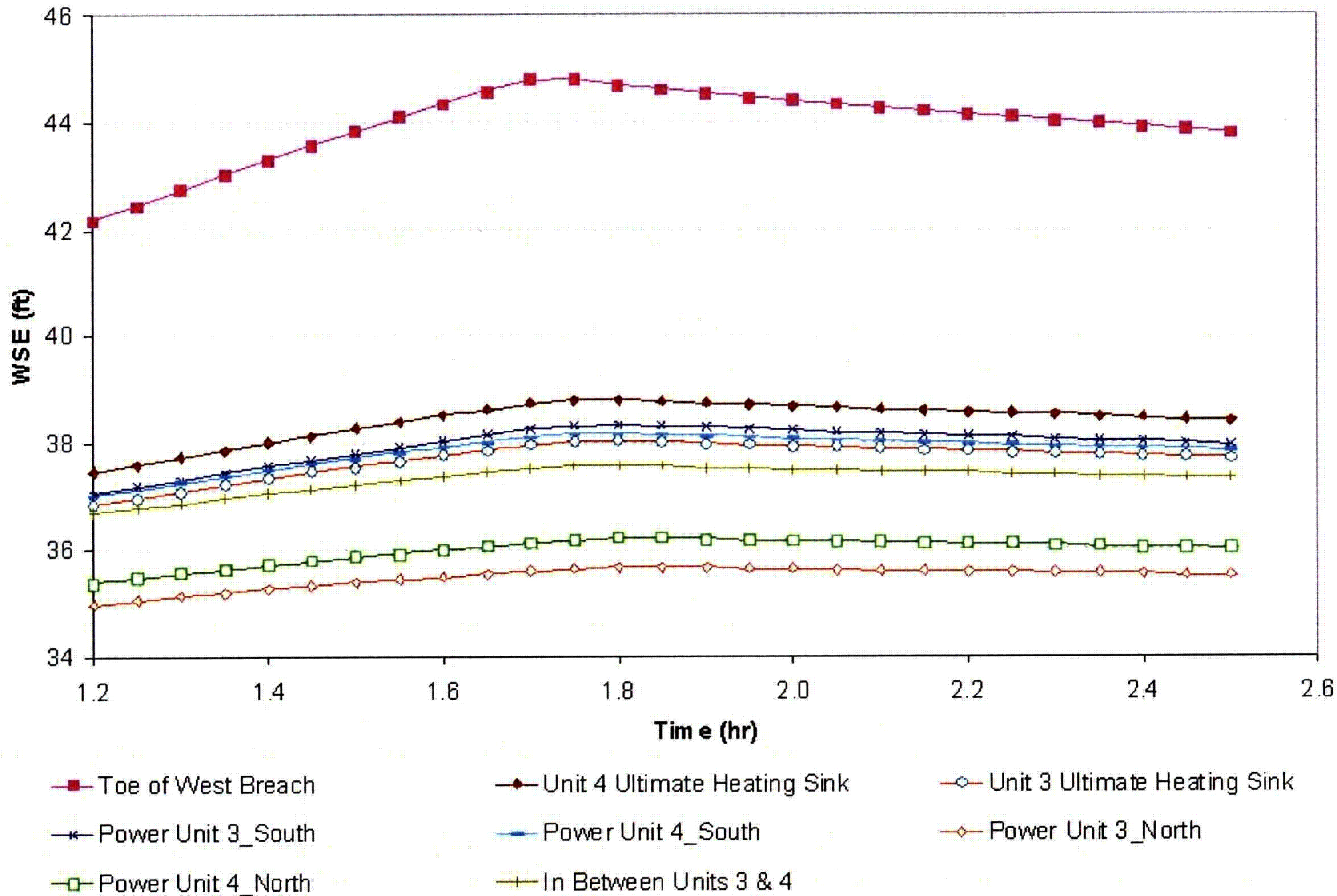

Figure 2.4S.4-21: Time-Dependent Water Surface Elevations Associated with West Breach Scenario

COLA Tier 2 Figure 2.4S.4-21(a) is a new figure:

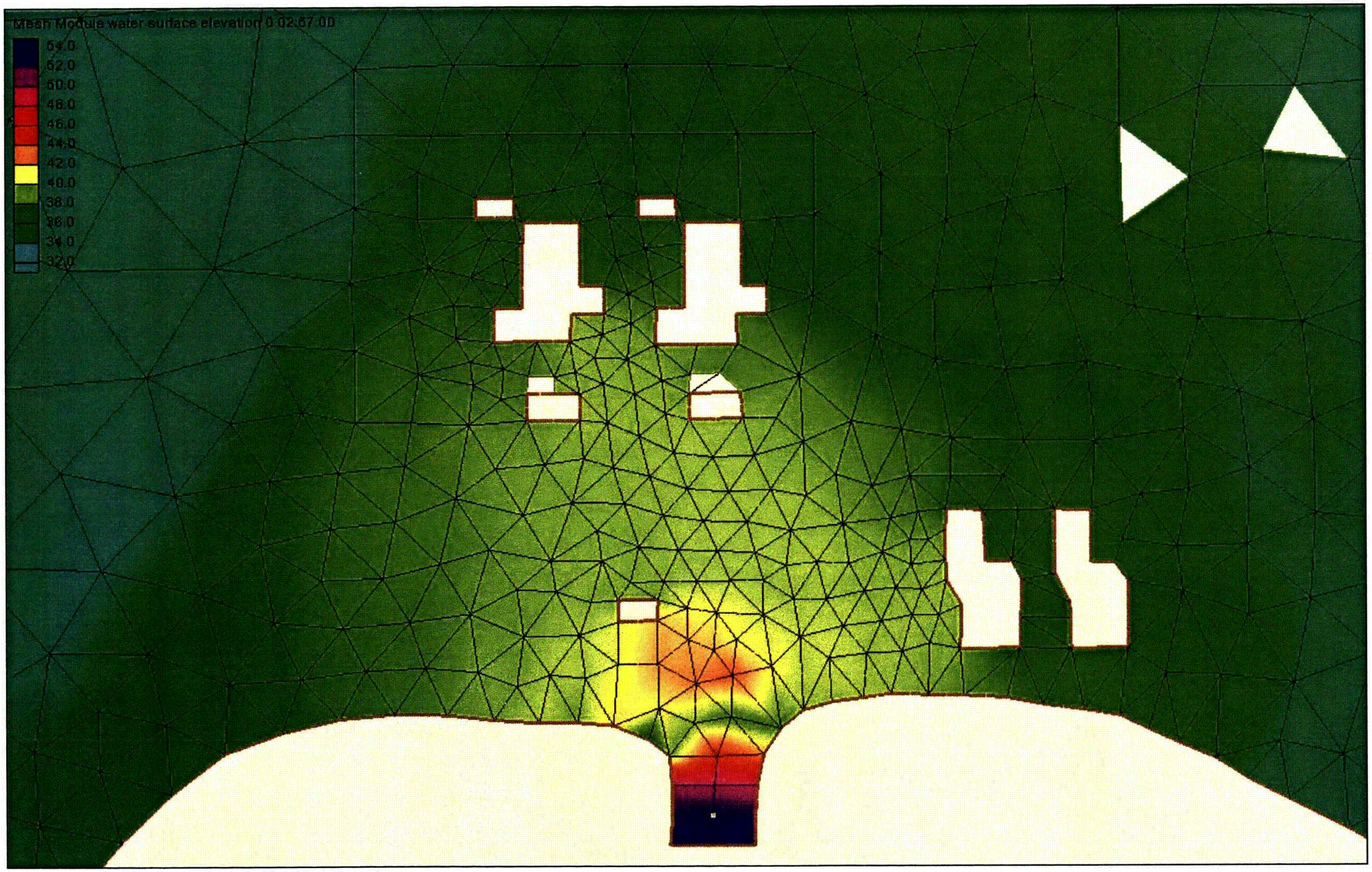

Figure 2.4S.4-21(a): Peak Water Surface Elevations Associated with East Breach Scenario (at time = 1.75 hours after initiation of breach)

COLA Tier 2 Figure 2.4S.4-21(b) is a new figure:

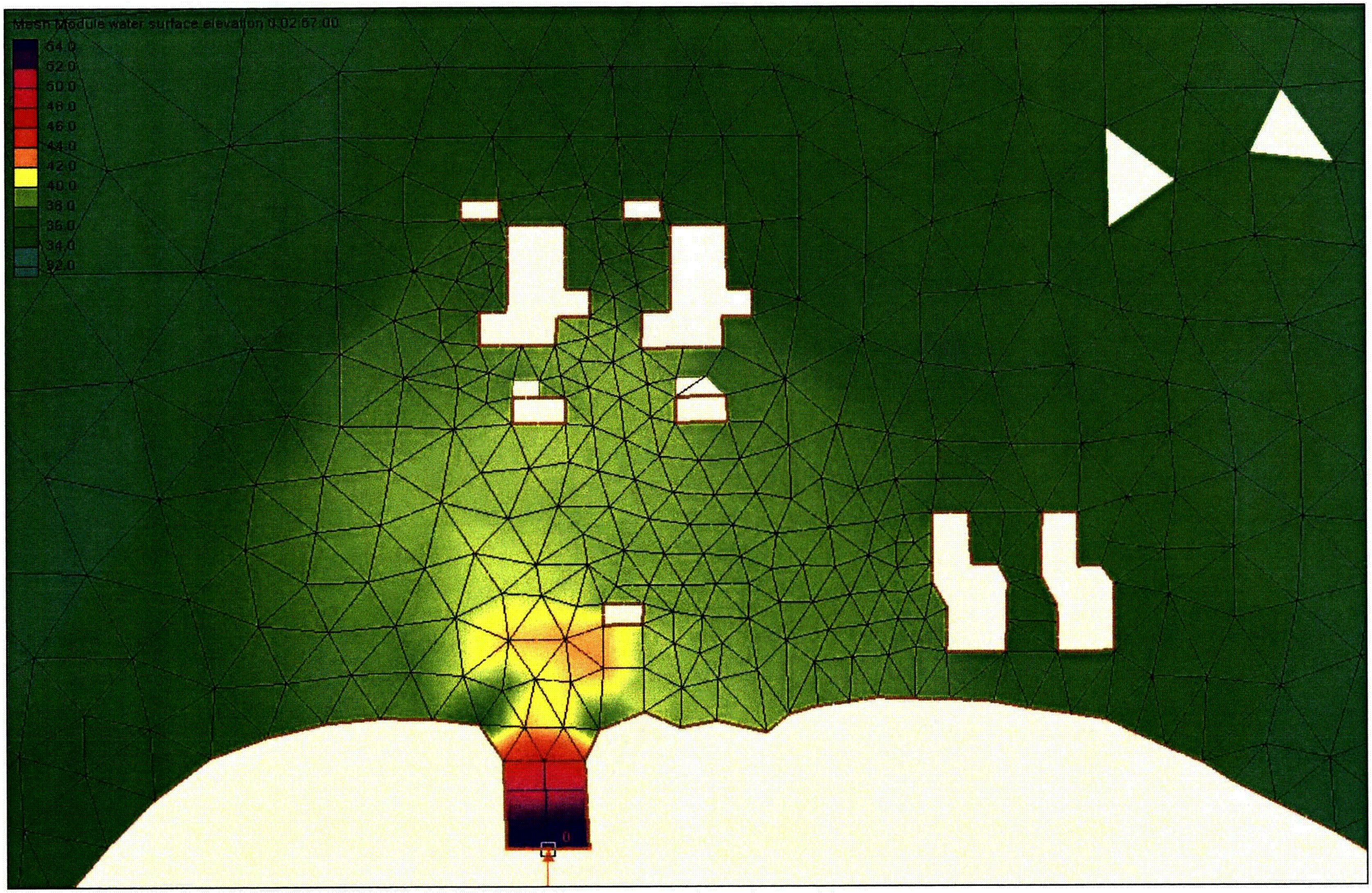

Figure 2.4S.4-21(b): Peak Water Surface Elevations Associated with West Breach Scenario (at time = 1.75 hours after initiation of breach)

COLA Tier 2 Figure 2.4S.4-21(c) is a new figure:

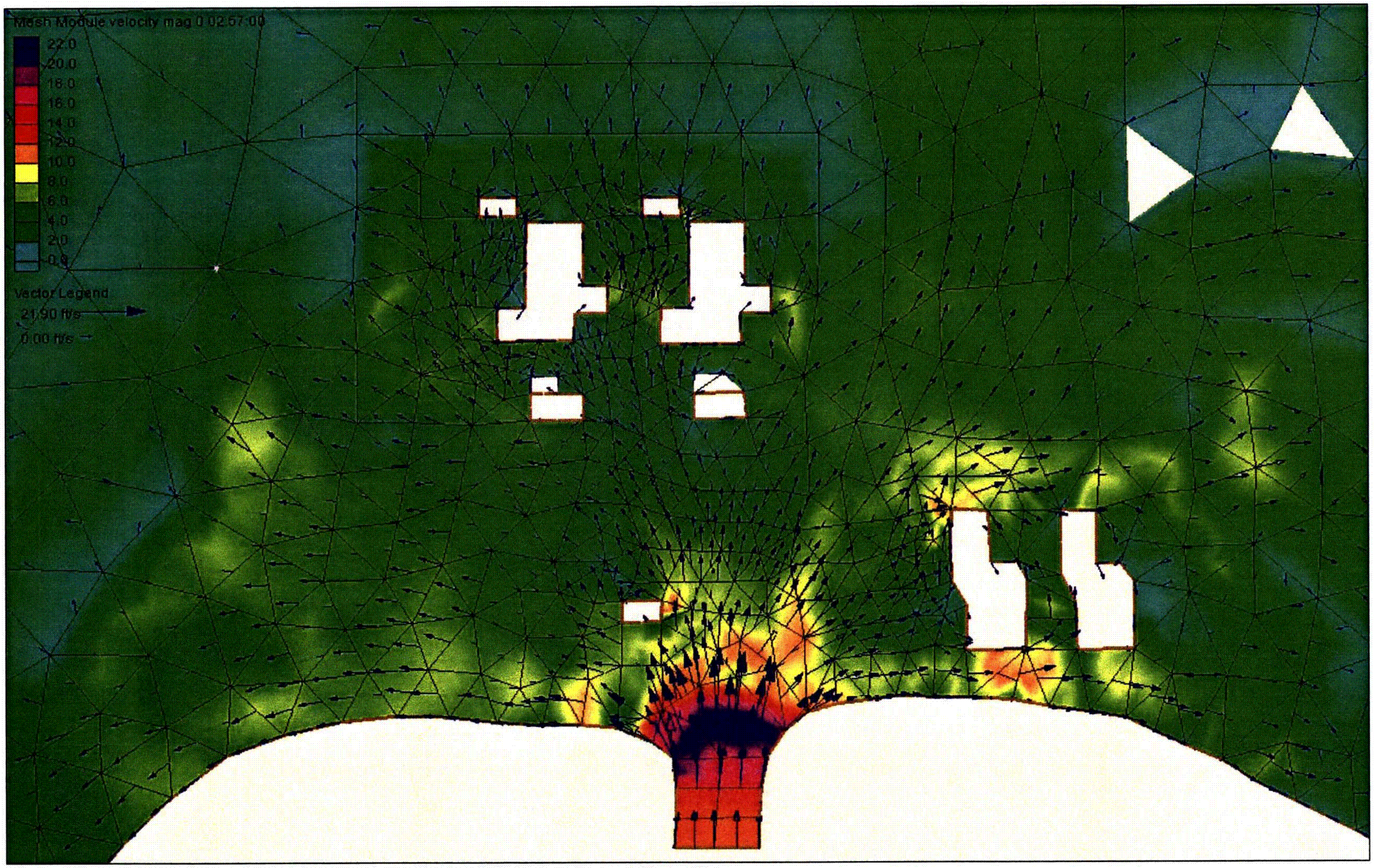

Figure 2.4S.4-21(c): Peak Velocities Associated with East Breach Scenario (at time = 1.75 hours after initiation of breach)

COLA Tier 2 Figure 2.4S.4-21(d) is a new figure:

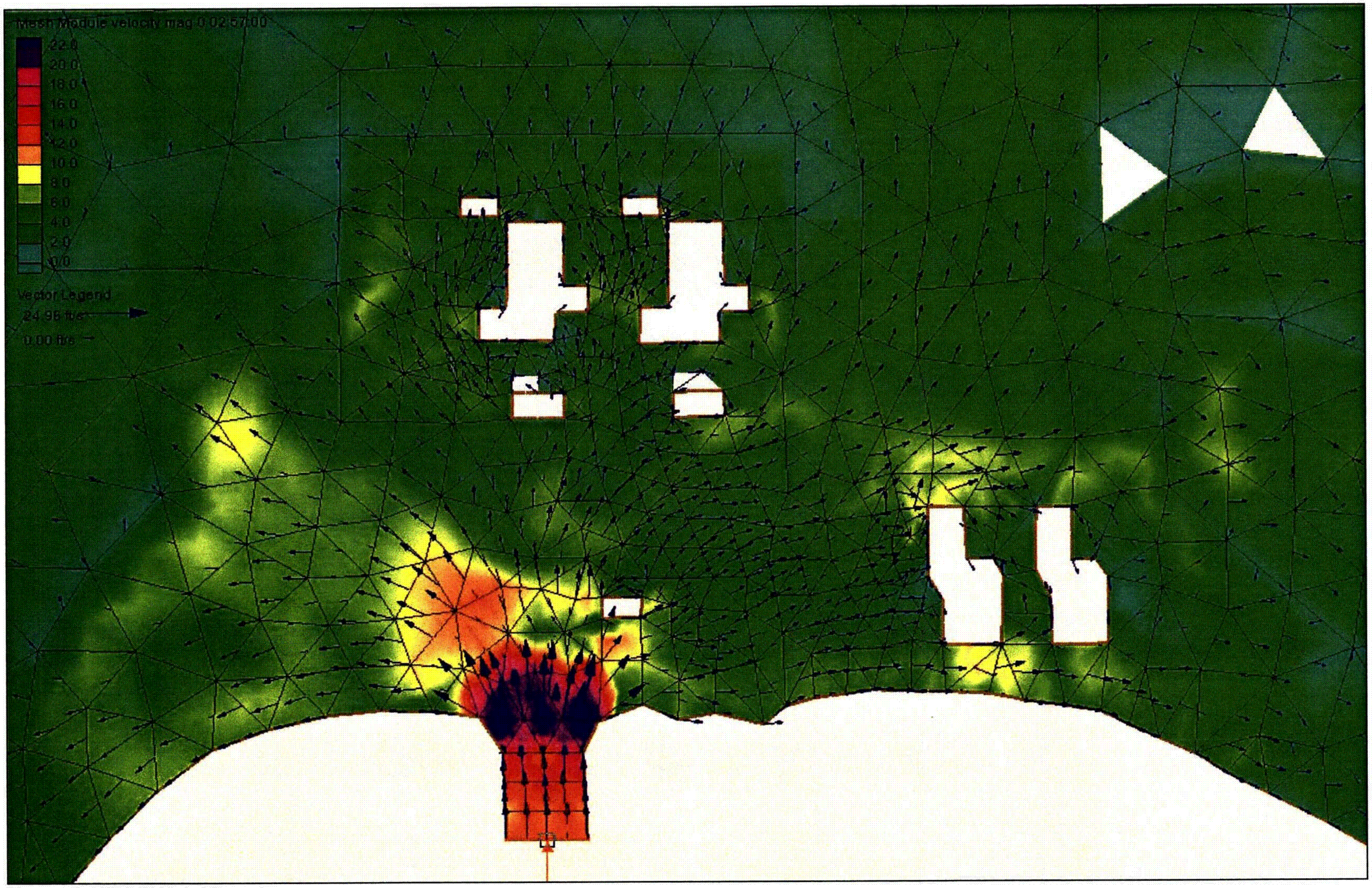

Figure 2.4S.4-21(d): Peak Velocities Associated with West Breach Scenario (at time = 1.75 hours after initiation of breach)

COLA Tier 2 Figure 2.4S.4-21(e) is a new figure:

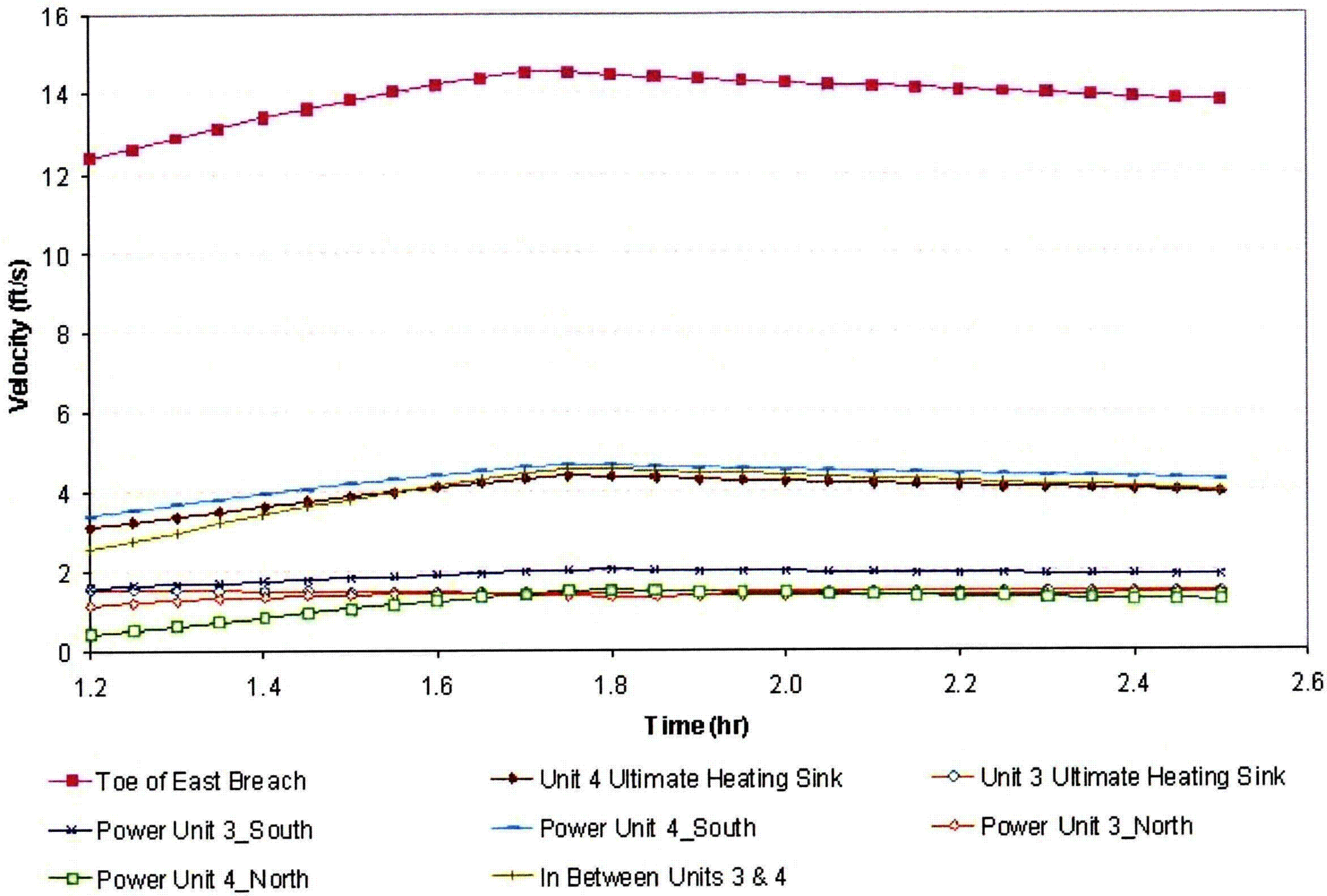

Figure 2.4S.4-21(e): Time-Dependent Velocities Associated with East Breach Scenario

COLA Tier 2 Figure 2.4S.4-21(f) is a new figure:

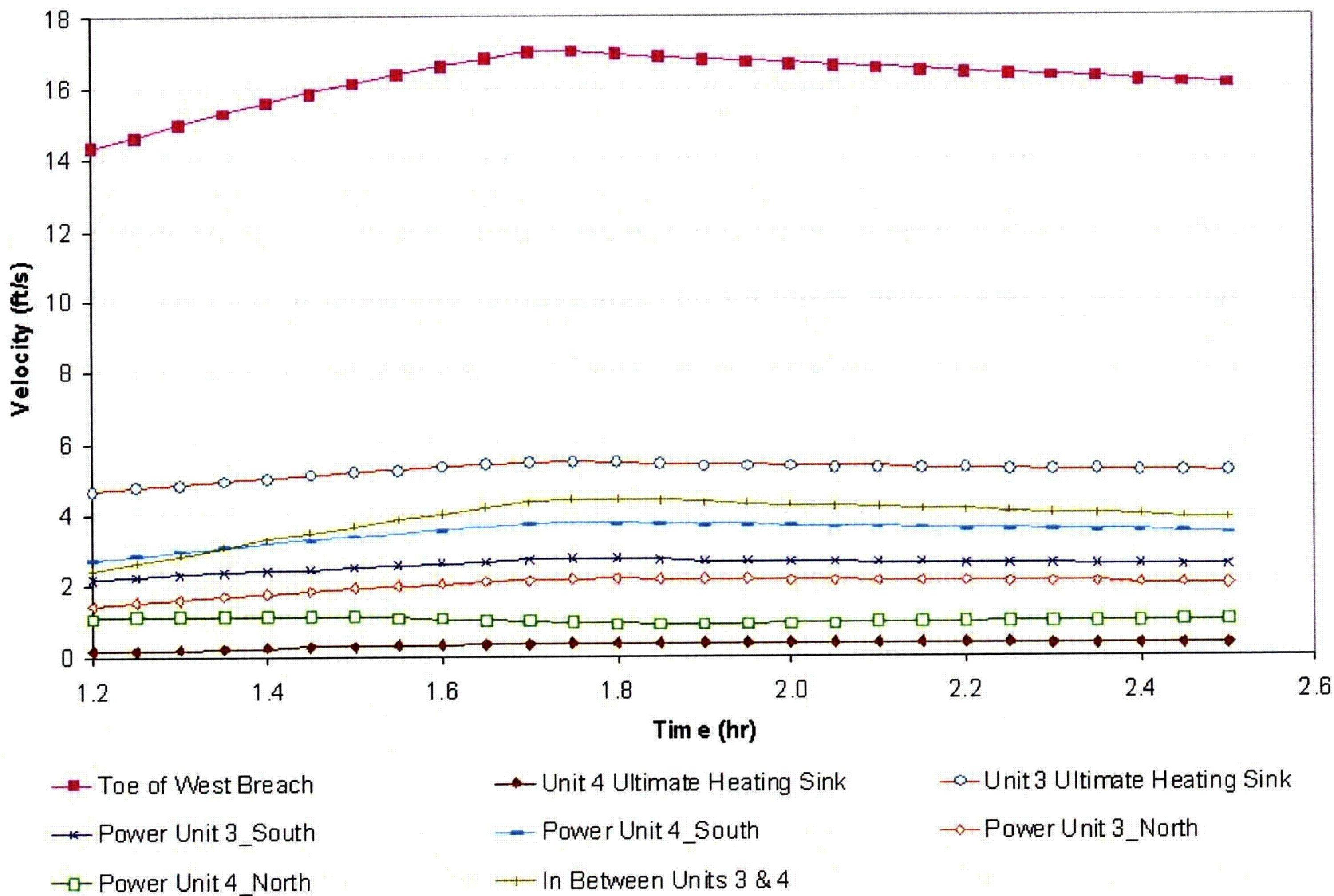

Figure 2.4S.4-21(f): Time-Dependent Velocities Associated with West Breach Scenario

COLA Tier 2 Figure 2.4S.4-21(g) is a new figure:

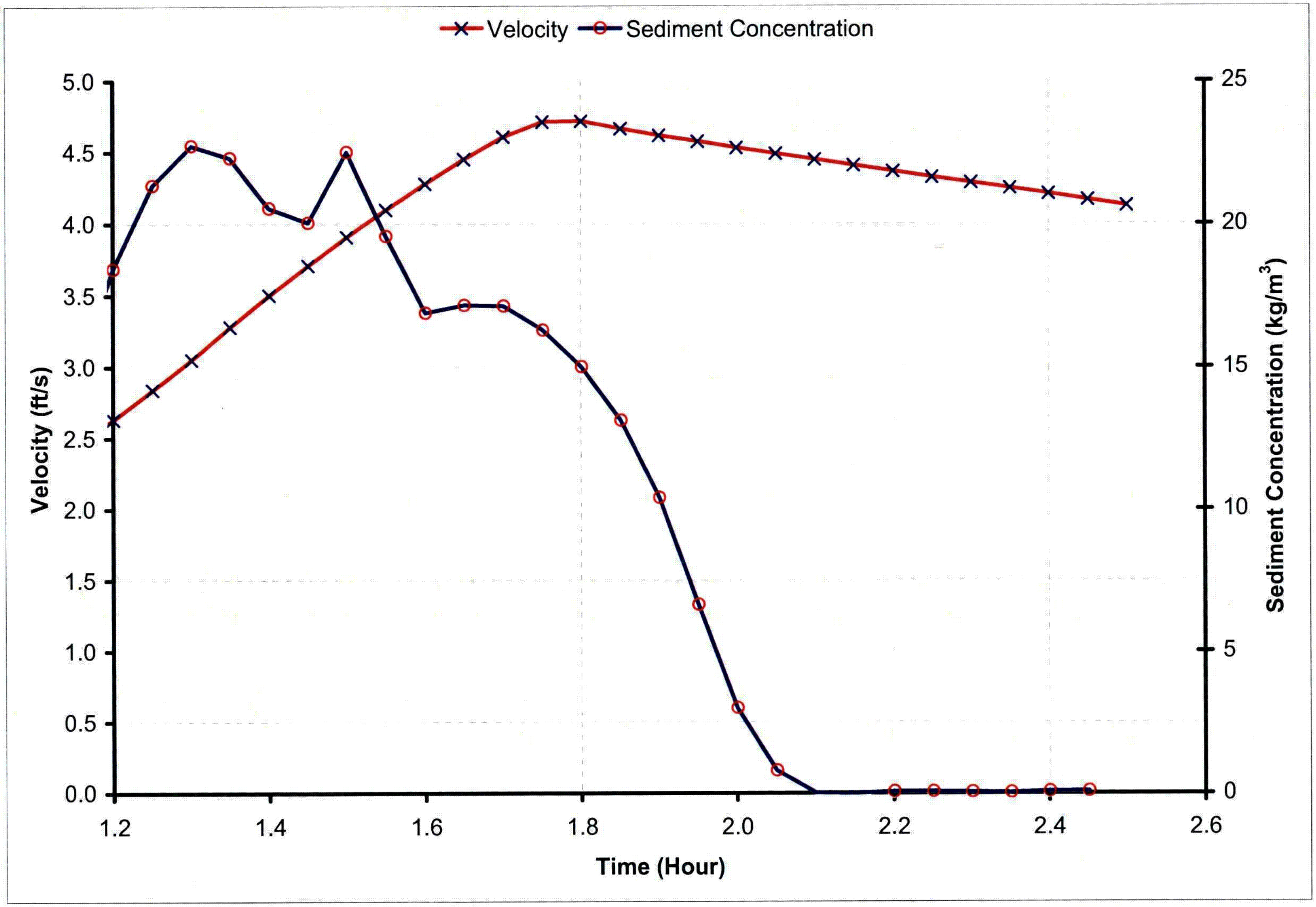

Figure 2.4S.4-21(g): Velocities and Sediment Concentrations In Between Units 3 and 4 with East Breach Scenario

COLA Tier 2 Figure 2.4S.4-21(h) is a new figure:

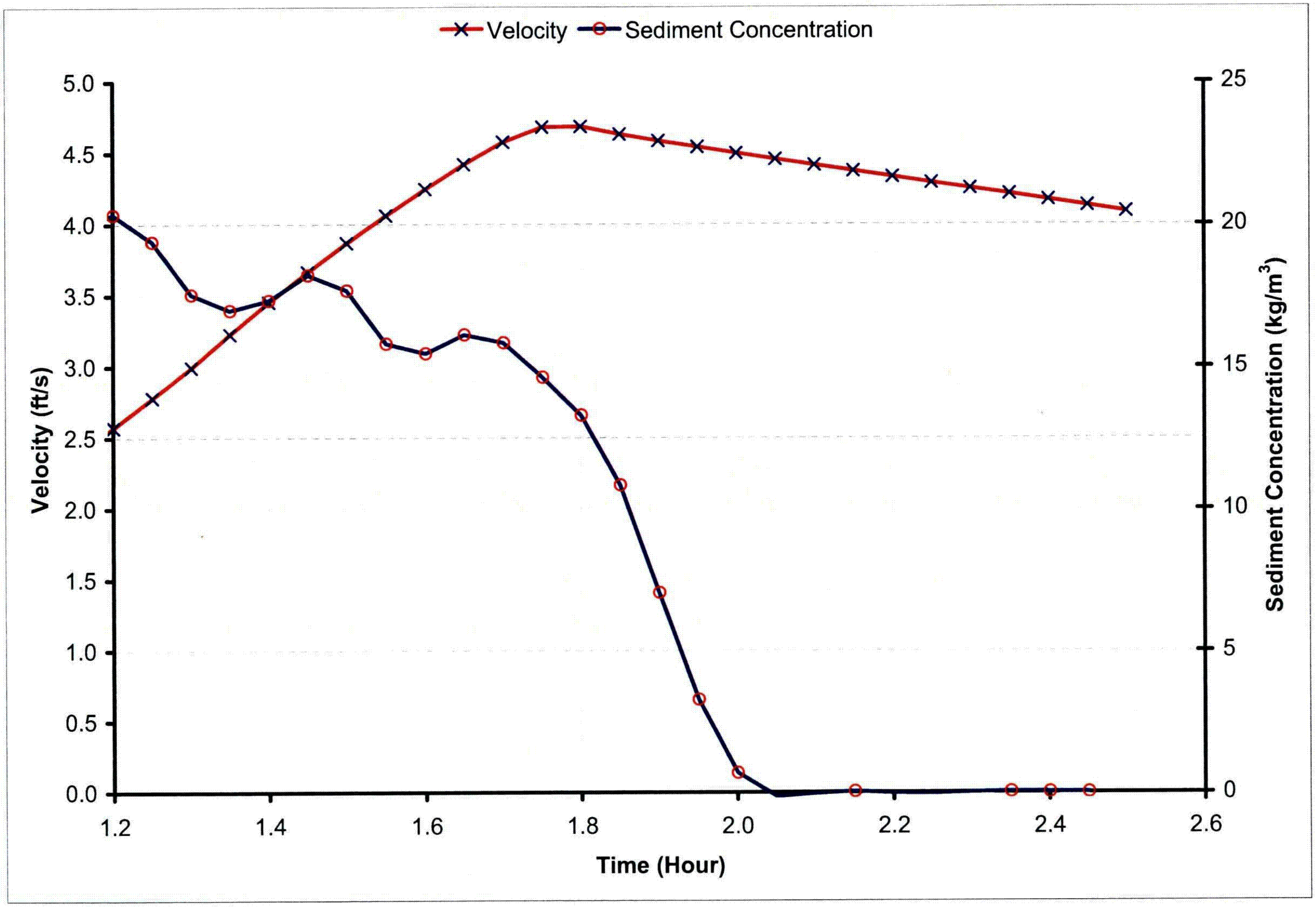

Figure 2.4S.4-21(h): Velocities and Sediment Concentrations In Between Units 3 and 4 with West Breach Scenario

COLA Tier 2 Figure 2.4S.4-21(i) is a new figure:

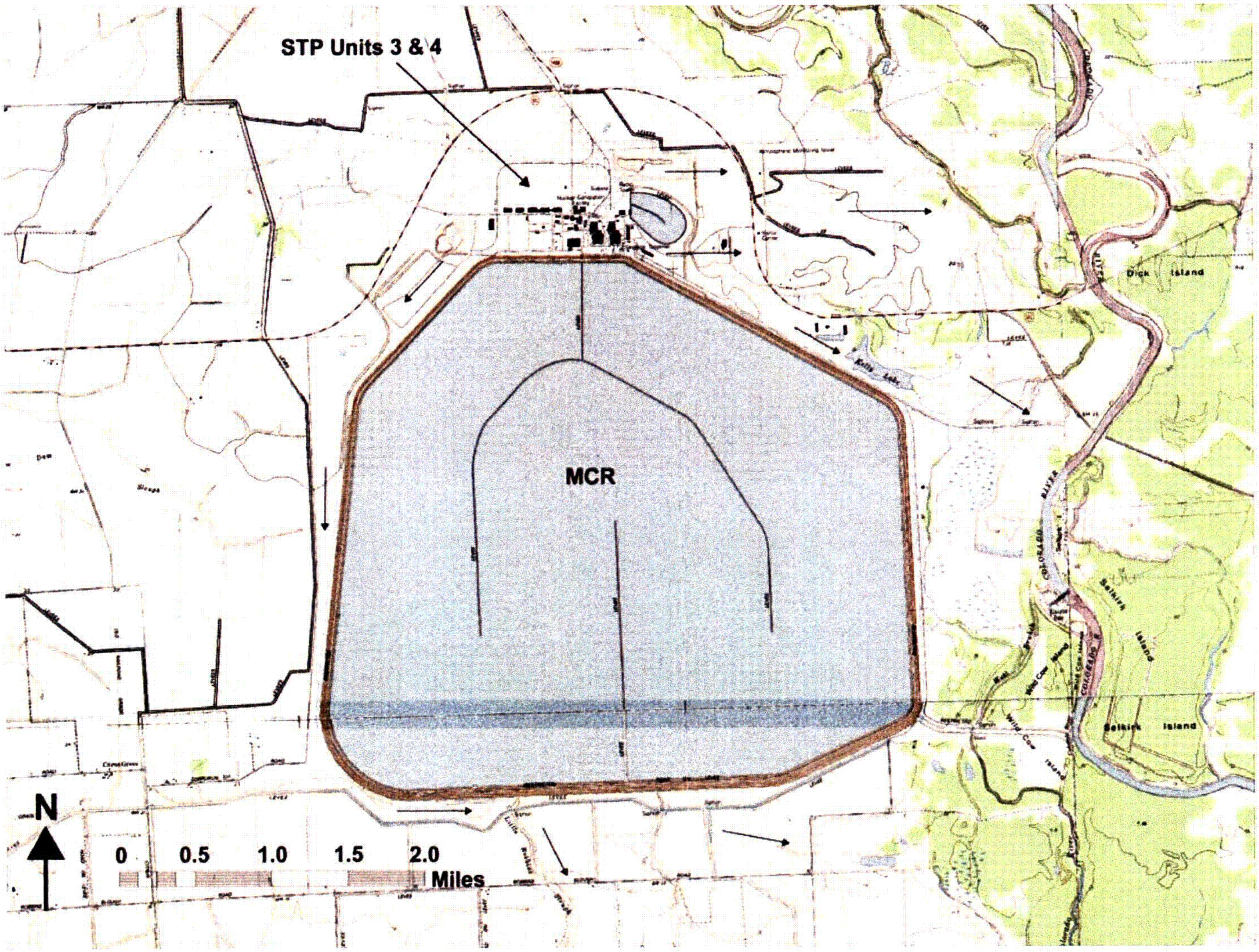

Figure 2.4S.4-21(i): Stream System around STP Site

The fourth paragraph of Tier 2 Subsection 2.4S.6.5 is changed as follows:

Based on the discussion above, it is concluded that the flood elevation at STP 3 & 4 due to the postulated probable maximum tsunami event will not be the controlling design basis flood elevation for STP 3 & 4 because it is lower than the ~~maximum~~design basis flood elevation of ~~48.5~~40.0 feet MSL~~, predicted for~~established based on a hypothetical breach event of the MCR embankment as described in Section 2.4S.4. Coincident wind waves are not considered in the analysis since the PMT event will have no flooding impacts on safety related facilities of STP 3 & 4.

The second paragraph of Tier 2 Subsection 2.4S.10 is changed as follows:

As discussed in Section 2.4S.2, the design basis flood elevation in the STP 3 & 4 power block area is at elevation ~~48.5~~40.0 ft mean sea level (MSL), or NGVD (National Geodetic Vertical Datum) 29. Therefore, all safety-related facilities require flood protection measures up to at least elevation ~~48.5~~40.0 ft MSL. The design basis flood elevation is determined by assessing a number of different flooding scenarios as a result of man made structures and various types of meteorological and hydrological events presented in Subsection 2.4S.2. The design basis flood for the STP 3 & 4 site is ~~the result of~~based on the MCR embankment breach (non-seismic Category 1 embankment), and it is discussed in detail in Subsection 2.4S.4. STP 3 & 4 safety-related facilities are the Reactor Building, Control Building, and the Ultimate Heat Sink (UHS) basin, cooling towers, and reactor service water (RSW) pump houses. The Reactor and Control Buildings and the UHC and RSW pump houses are located in the power block area of the site. Site grade elevations in the STP 3 & 4 power block area range from 32 ft MSL to 36.6 ft MSL. Thus, the Reactor Buildings, Control Buildings, UHS water storage basins, and RSW pump houses are subject to flooding and require flood protection.

The fifth paragraph of Tier 2 Subsection 2.4S.10 is changed as follows:

In addition to structural protection against static, dynamic, and erosion flood forces, the safety-related facilities must remain free from flooding and intrusion of water into areas that contain safety-related equipment. All safety-related facilities in the power block are designed to be water tight at or below elevation ~~48.5~~40.0 ft MSL. All water tight doors and hatches are normally closed under administrative controls and open outward. All ventilation openings are located above elevation ~~48.5~~40.0 ft MSL. The UHS and Pump House is designed to be watertight below 50 ft MSL.

Tier 2 Subsection 2.4S.14 is changed as follows:

2.4S.14 Technical Specifications and Emergency Operation Requirements

The following site-specific supplement addresses COL License Information Item 2.22.

For STP 3 & 4, safe plant operations are not affected by any of the extreme high water levels discussed in previous subsections of Section 2.4S because required systems and equipment are protected against such levels during extreme flood conditions and, therefore, will remain operational. The following events have been analyzed in Section 2.4S to determine high water level:

- Floods due to probable maximum precipitation (Subsection 2.4S.2).
- Probable maximum precipitation induced river flooding incident (Subsection 2.4S.3).
- Failure of the upstream dams on the Colorado River (Subsection 2.4S.4).
- Breach of the main cooling reservoir (MCR) embankment (Subsection 2.4S.4).
- Probable maximum hurricane surge (Subsection 2.4S.5).
- Probable maximum tsunami (Subsection 2.4S.6).

As discussed in Subsection 2.4S.4, the design basis flood elevation in the power block area is at El. ~~48.5~~40.0 ft MSL (NGVD 29). The design basis flood is ~~the result of~~based on a breach of the Main Cooling Reservoir (MCR) embankment (non-seismic Category 1 embankment); details are provided in Subsection 2.4S.4. Site grade elevations in the STP 3 & 4 power block area range from 32 ft MSL to 36.6 ft MSL. For structures located within the Power Block the top of concrete (TOC) floor elevation is 35 ft MSL.

Structures and components whose failure could prevent the safe shutdown of STP 3 & 4 are protected from high water levels, flood or storm surge, and maximum wave run-up, or are designed to preclude the adverse impacts of the design basis flood as described in Section 3.4.

Specific flood protection measures are described in Subsection 2.4S.10. To withstand the static and dynamic forces as a result of the MCR embankment breach, watertight flood protection measures and structural measures are applied to any STP 3 & 4 facilities that have an open passageway to any safety-related facility. Since all watertight doors and hatches for these facilities, at or below ~~48.5~~40.0 ft. MSL are to remain in a closed position under administrative control, no emergency operating procedures or plant technical specifications (plant shutdown) are required for implementation of flood protection measures.

In addition to the features included in the STP 3 & 4 design to prevent water intrusion into safety-related facilities, for the analyzed postulated events, adequate time exists to allow for remedial actions prior to any detrimental impacts on structures other than safety related. The occurrences of high water levels at STP 3 & 4 under most of the postulated events (Colorado River flooding, hurricanes, or upstream dam failure) are not sudden events. Thus, adequate time is available to take mitigating actions precluding the effects of potential flooding.

In the case of precipitation caused river level increases, the river level rise is slow enough to allow for appropriate responsive actions. For potential hurricane surge, the approach of a hurricane is a forecasted and trackable event and will allow sufficient time to take mitigating actions. In the case of an upstream dam failure on the Colorado River, the shortest warning time is estimated to be 58 hours, as discussed in Subsection 2.4S.4 allowing for the implementation of remedial, protective actions.

As described in Subsection 2.4S.11.4, no additional emergency protective measures are required to safely shut down STP 3 & 4 in the event of extreme low water levels in the MCR. Due to the large contained volume of the MCR the drop in water level (other than a major breach) would be a gradual event, which provides adequate time to place STP 3 & 4 in safe shutdown conditions. Appropriate emergency operating procedures (EOPs) will include applicable provisions for the MCR, similar to those provided for STP 1 & 2, prior to fuel load (COM 2.4S-1).

Normal and emergency operations of the ultimate heat sink (UHS) are described in Subsection 9.2.5. The UHS basin and the Reactor Service Water (RSW) pumphouse are designed to withstand the hydrodynamic forces associated with the design basis flood event. Furthermore, the top of the UHS basin wall is at elevation 97.5 ft MSL, and the roof of the RSW pump house, located at the UHS, is at elevation 50.0 ft MSL. The top of both structures are above the design basis flood level elevation of ~~48.5~~40.0 ft MSL. No emergency protective measures are required to safely shut down STP 3 & 4 provided that the minimum water level in the UHS basin is maintained at or above El. 77.3 ft MSL which ensures a minimum 30 days inventory supply without any makeup. Plant shutdown is initiated when the water level in the UHS basin falls below 77.3 ft MSL.

The fourth paragraph of Tier 2 Subsection 2.5S.5.2.1 is changed as follows:

STP 3 & 4 Seismic Category I structures are to have ~~similar~~ flood protection mechanisms to withstand a possible breach in the MCR embankment. ~~Because STP 3 & 4 structures are designed against this condition, a failure in the MCR embankment similarly does not impact the safety of STP 3 & 4 Seismic Category I structures. Because the STP 3 & 4 Power Block area is about 2000 feet from the MCR embankment, or approximately three times the distance of the MCR embankment to the STP 1 & 2 Power Block area, the maximum flood level at STP 3 & 4 (El. 48.5 feet) is somewhat lower than the maximum flood level at STP 1 & 2 (El. 50.8 feet).~~ Based on a postulated credible MCR embankment breach (see Subsection 2.4S.4), the design basis flood level at STP 3 & 4 is El. 40.0 feet.

The first paragraph of Tier 2 Subsection 3.4.2 is changed as follows:

> *Since the design* basis *flood elevation is* ~~*30.5 cm below* 442.0~~ at El. 40.0 ft (see Subsection 2.4S.2.2), 182.9 cm above *the finished plant grade,* ~~*there is no dynamic force due to flood.* The~~ the *lateral hydrostatic* and hydrodynamic *pressure on the structures due to the design flood water level, as well as ground and soil pressures, are calculated.*

The second paragraph of Tier 2 Subsection 3.4.3.1 is changed as follows:

> The site specific design basis flood elevation is defined as ~~442.0~~182.9 cm above grade. The design basis flood is described in Subsection 2.4S.2.

The design flood level and design groundwater level entries in Tier 2 Table 3.4-1 are changed as follows:

Table 3.4-1 Structures, Penetrations, and Access Openings Designed for Flood Protection

Structure	**Reactor Building**	**Service Building**	**Control Building**	**Radwaste Building**	**Turbine Building**	**Ultimate Heat Sink**
Design Flood Level ~~(mm)~~	*~~11,695~~* **~~14,783~~ 12,192** mm **~~(48.5 ft)~~ (40.0 ft)**	*~~11,695~~* **10058 mm** **(33 ft)**	*~~11,695~~* **~~14,783~~ 12,192** mm **~~(48.5 ft)~~ (40.0 ft)**	*~~11,695~~* **10058 mm** **(33 ft)**	*~~11,695~~* **10058 mm** **(33 ft)**	**~~14,783~~ 12,192 mm** **~~(48.5 ft)~~ (40.0 ft)**
Design Ground Water Level ~~(mm)~~	*~~11,390~~* **9,753 mm** **(32 ft)**	*~~11,390~~* **9,753 mm** **(32 ft)**	*~~11,390~~* **9,753 mm** **(32 ft)**	*~~11,390~~* **9,753 mm** **(32 ft)**	*~~11,390~~* **9,753 mm** **(32 ft)**	**~~8,534~~ 9,753 mm** **~~(28 ft)~~ (32 ft)**

Item Number (3) in Tier 2 Subsection 3H.1.4.2 is changed as follows:

(3) ~~*Maximum*~~ Design Basis *Flood Level:*

— ~~*0.305 m*~~ ~~442.0~~ 182.9 cm ~~*below*~~ above *grade*

The fifth paragraph of Tier 2 Subsection 3H.1.6 is changed as follows:

As documented in Subsection 3.4, the STP 3 & 4 site has a design basis flood elevation that is ~~442.0~~ 182.9 cm above grade. This results in an increase in the flood level over what was used in the ABWR Standard Plant, however the load due to the revised flood level on the RB is less than the ABWR Standard Plant RB seismic load, hence it doesn't ~~e~~affect the Standard Plant RB structural design.

Tier 2 Subsection 3H.2.4.2.3 is changed as follows:

3H.2.4.2.3 Design Basis Flood Level

Design basis *flood level is at* ~~*0.305m*~~ ~~442.0~~ 182.9 *cm* ~~*below*~~ above *grade level.*

The fifth paragraph of Tier 2 Subsection 3H.2.6 is changed as follows:

As documented in Subsection 3.4, the STP 3 & 4 site has a design basis flood elevation that is ~~442.0~~ 182.9 cm above grade. This results in an increase in the flood level over what was used in the ABWR Standard Plant, however the load due to the revised flood level on the CB is less than the ABWR Standard Plant seismic load, hence it does not ~~e~~affect the Standard Plant CB structural design.

Tier 2 Subsection 3H.6.4.2.3 is changed as follows:

3H.6.4.2.3 Design Basis Flood Level

Design basis flood level is at ~~14.8~~ 12.2 meters MSL. This elevation is defined in Subsection 2.4S.2.2.

The second and third paragraphs of the Description for STP DEP T1 5.0-1 provided in COLA Part 7, Subsection 2.1, are changed as follows:

> The site design basis flood level is increased from that specified in the DCD. The certified design site parameter for site flooding is changed from 30.5 cm below grade to ~~442.0~~ ~~442.0~~182.9 cm above grade (grade being 1036.3 cm above mean sea level (MSL)) in order to handle a main cooling reservoir failure as a design basis event at STP.
>
> The main cooling reservoir at the South Texas site is a non-seismic category 1 dam; hence, its failure must be assumed in the worst possible location. This results in the site design basis flood. ~~The maximum flood level is 1478.3 cm above MSL; however it decreases with distance from the main cooling reservoir.~~ ~~The maximum flood level is 1478.3 cm above MSL; however it decreases with distance from the main cooling reservoir.~~

Supplement 1, Part B, to RAI 02.04.04-9 Response:

FSAR Chapter 19R is changed as shown below as a result of the Main Cooling Reservoir breach re-analysis described in the responses to RAI questions 02.04.04-9 and 02.04.04-10:

Section 19R.5.3 is changed as shown below:

> *The turbine building does not contain any safety-related equipment* with the exception of instrumentation associated with Reactor Protection System and condensate pump motor trip circuit breakers. ~~*But*~~Although the instrumentation and the circuit breakers are located at or above elevation 19700 TMSL (59'-3 1/2" MSL) well above the internal flood level described below and the external flood level of ~~47.6'~~ 40.0' MSL and prevented from the floods, *the flooding of the turbine building can initiate a reactor trip and may impact the safe shutdown of the plant if the water reaches the control building through the service building access tunnel. There are several water sources listed in Table 19R-1 that may leak into the turbine building. Only the two unlimited water sources (circulating water and turbine service water) are capable of flooding the turbine building and threatening safety equipment in the control building.*

The second paragraph in supplemental Section 19R.7 is changed as shown below:

> Summarized in the sections below is the external flooding PRA analyses for the STP 3 & 4 plants. External flooding is defined as intrusion of water from sources outside of plant buildings such that the ability of the plant to achieve safe shutdown is affected. The analysis determined the potential core damage frequency (CDF) that could result from external flooding events for each of the new units and was developed assuming that the watertight door providing normal access to the main control room is open. This assumption provides a conservative and bounding assessment of risk from external flooding ~~because the watertight door to the main control room would be closed except for intermittent ingress and egress (Refer to FSAR Section 2.4S.10)~~.

In supplemental Section 19R.7.2, the first through the sixth paragraphs are changed as shown below:

> External flooding at the STP site potentially can be initiated by several basic sources: river flooding which includes ice flooding, upstream dam breaks and landslides, tsunamis, hurricane surge, intense rainstorms, and onsite sources including Main Cooling Reservoir breach and failure of an ultimate heat sink (UHS). Events from these sources could, potentially, be related. For example, a storm could cause both a breach of an upstream dam and local flooding at the site. These correlated flooding events are aanalyzed in Section 2.4S of this application. This analysis considers independent and correlated flooding events.
>
> Ice flooding of the Colorado River adjacent to the STP site is not considered a potential hazard because the warm temperatures of the area and the tidal effects that are felt on the river in the area. Therefore, ice flooding is ~~screened~~ excluded as a potential initiating event.

Based on analysis performed for STP 1&2 (Reference 19R.7-1), landslides are not considered a threat to the STP site. Therefore, landslides are ~~screened~~ excluded as potential external flooding initiating events.

Analysis for STP Units 3 and 4 ~~1 and 2~~ (Reference 19R.7-1) also concluded that tsunamis cannot affect the site. Therefore, tsunamis are ~~screened~~ excluded from consideration as initiating events.

The storm surge or seiche resulting from a hurricane could potentially cause flooding at the STP site. However, the maximum water level at the STP site that would be expected from such an event would be elevation ~~26.74~~ 31.1 feet. Since this elevation is below grade level, hurricane storm surge or seiche can be excluded as an external flooding initiating event.

Section 2.4S.2.3 of this COLA determined the maximum height of floodwater from a probable maximum precipitation (PMP) event at 36.6' MSL. Section 2.4S.4 identifies the grade at the center of the site as 36.6 dropping to 32' at the perimeter of the site. ANS 2.8-1992 (Reference 19R-4) defines PMP as the estimated depth of precipitation for a given duration, drainage area, and time of year for which there is virtually no risk of exceedance. Using example methodology of Appendix B to ANS2.8-1992, floods from PMP events that potentially challenge safety-related SSCs are screened from further consideration due to the very low frequency of exceedance. The results of intense precipitation events that approach the PMP depth are bound by the design basis flood level of 40.0' MSL based on the MCR breach analysis. ~~Intense precipitation can result in flooding local to the STP site because plant buildings will be constructed so that all external entrances are at least one foot above the flood level expected from a probable maximum precipitation event. Since the maximum flood level expected from intense precipitation is one foot below grade level for Units 3 and 4, intense local precipitation is screened from consideration as an external flooding initiating event.~~

After the ninth paragraph in Section 19R.7.2, a new paragraph is inserted along with the following changes:

Section 2.4S.3 of the COLA determined the maximum height of floodwater from a probable maximum flood (PMF) event on the Colorado River at 26.1' MSL. The still water elevation of this event is lower than the still water elevation for the multiple cascading dam flood event of 28.4' MSL, so no wave runup was determined in 2.4S.3. As the water level from the PMF event is lower than the multiple cascading dam failure flood event, the PMF on the Colorado River is screened from further analysis.

In addition the potential flooding effects from multiple, cascading failures of Colorado River dams upstream of the STP site has the potential to affect safety-related structures. That analysis shows that a peak still water elevation of 28.6 feet MSL, with wind setup on the longest fetch of 3.94 feet for a maximum water level of 32.54 feet. The calculated maximum water level including wave runup with the relocated UHS is 34.4 feet. ~~34.1 feet with wave runup to the 43.7 foot elevation~~. Therefore, multiple, ~~concurrent~~ cascading dam failures are considered as an external flooding initiating event.

The second and fourth paragraphs in supplemental Section 19R.7.3 are changed as shown below:

The frequency of multiple, ~~concurrent~~ cascading upstream dam breaks considers the failure of three dams, the S. W. Freese, Buchanan, and Mansfield Dams. The analysis assumes that the first dam failure can occur randomly and that the second and third failures are dependent on the previous dam failures. The sequence of events analyzed begins with failure of the S. W. Freese Dam which began operation in 1990.

Failure of the third dam, the Mansfield Dam, given failure of the first two dams, is calculated using the Gamma factor given in Table 19R-4. The frequency of multiple ~~concurrent~~ cascading dam failures considered as external flooding initiating events is calculated to be very low.

The first through the fourth paragraphs in supplemental section 19R.7.4.1 are changed as shown below:

Note that this analysis is developed assuming that the watertight door providing normal access to the main control room is open. This assumption provides a conservative and bounding assessment of risk from external flooding~~because the watertight door to the main control room would be closed except for intermittent ingress and egress (Refer to FSAR Section 2.4S.10)~~.

A breach of the main cooling reservoir could occur suddenly or progress over many minutes. A discussion of previous dam breaches notes that the failure time of most breaches is 15 minutes to one hour from the time of inception to completion of the breach. However, some breaches became fully developed in as little as 6 minutes while others took more than 7 hours. It was also noted that half the breaches identified occurred in less than 1.5 hours. Therefore, it is concluded that, while there is a good deal of uncertainty and variability associated with the breach time, 15 minutes to one hour would likely be conservative. Breach width was also noted to be typically 2 to 5 times dam height (Reference ~~19R.7-2~~19R-2). The timing of the breach along with the width of the breach affects the height of water that reaches plant buildings. Smaller breaches or breaches that take longer to develop would result in a lower level of water on plant buildings. For smaller and slower-developing breaches, it can be expected that water would not rise above grade elevation on plant buildings. For larger and faster-developing breaches, water level on plant buildings would be higher. The analysis, originally documented in the IPEEE of Units 1&2 (Reference ~~19R.7-3~~19R-3), considered that failures of the MCR are equally likely to occur anywhere along the perimeter and excluded from consideration that portion of MCR failures that would direct water away from plant buildings. MCR failures that would result in water flowing away from the site would not be considered as external flooding initiating events, consistent with the analysis presented in Reference ~~19R.7-3~~19R-3. This assumption is considered reasonable since the land around the ~~MRC~~ MCR generally slopes southward towards the Colorado River. This analysis assumed that any breach of the main cooling reservoir that is included in the initiating event definition is sufficiently large that water level will rise above the entrances to plant buildings. This analysis also assumed that the main cooling reservoir breach would cause a loss of offsite power either because of failure of the switchyard equipment or the plant auxiliary transformers that are impacted by the floodwaters. Furthermore, this analysis assumed that the loss of offsite power is not recoverable for several days.

A breach of the main cooling reservoir would cause water to flow across lighted roadways and open areas between the main cooling reservoir and the plant. Security personnel are stationed such that they have a clear view of these areas. On seeing the developing breach or water flow, they would notify the main control room in accordance with their training and procedures.

With the exception of the normally open access door to the control building from the service building, external ~~External~~ access points to the control and reactor buildings are provided with normally-closed, watertight barriers or doors designed to withstand the maximum loadings of any potential main cooling reservoir breach. All these doors are alarmed at the central alarm station so it is unlikely that one would be left open. Failure of any one of these doors would allow water to enter the building and flow through drains, stairways, and non-watertight doors to the essential electrical switchgear rooms below grade. Since there are no internal watertight barriers to protect the rooms on lower elevations from water entering the upper elevations, it is conservatively assumed that failure of one of the watertight doors on the reactor building would result in core damage.

Supplemental Section 19R.7.4.2 title and paragraphs are changed as shown below:

19R.7.4.2 Multiple, ~~Concurrent~~ Cascading Upstream Dam Failures

~~Note that this analysis is developed assuming that the watertight door providing normal access to the main control room is open. This assumption provides a conservative and bounding assessment of risk from external flooding because the watertight door to the main control room would be closed except for intermittent ingress and egress (Refer to FSAR Section 2.4S.10).~~

The accident progression for multiple, ~~concurrent~~ cascading upstream dam failures is similar to that of the main cooling reservoir breach except for timing. Since the last dam that would fail, the Mansfield Dam, is nearly 300 miles upstream of the STP site, flood waters from that dam failure would not reach the STP site for many hours. In that time, closure of the normally-open main control room access door would be assured. In addition, compensatory actions such as sandbagging or installation of other temporary flood barriers can be installed around access doors. These additional compensatory actions, however, are not quantified as part of this analysis. This analysis also assumes that the flooding that results from multiple, ~~concurrent~~ cascading upstream dam failures will cause a loss of offsite power either because of failure of the switchyard equipment or the plant auxiliary transformers that are impacted by the floodwaters. Furthermore, this analysis assumed that the loss of offsite power is not recoverable for several days.

With the exception of the normally open access door to the control building from the service building, external ~~External~~ access points to the control and reactor buildings are provided with normally-closed, watertight barriers or doors designed to withstand the maximum loadings of any potential main cooling reservoir breach, a more severe event than multiple, ~~concurrent~~ cascading upstream dam failures. All these doors are alarmed at the central alarm station so it is unlikely that one would be left open. Failure of any one of these doors would allow water to enter the building and flow through drains, stairways, and non-watertight doors to the essential electrical switchgear rooms below grade. Since there are no internal watertight barriers to protect the rooms on the lower elevations from water that entered the upper elevations, it is conservatively assumed that failure of one of the watertight doors on the reactor building will result in core damage.

The normal access to the main control building is via the service building through a watertight door on the 2950 mm elevation. In addition, there are other normally-closed watertight doors that provide access to the control building from the service building and that are located either at or below grade. Since the service building is not designed to withstand flooding, it is conservatively assumed that the flooding that results from multiple, ~~concurrent~~ cascading upstream dam failures would result in water entering the service building. If any one of the doors from the service building to the control building fails, then water could enter the control building and cause failure of all three divisions of reactor cooling water (RCW) or DC power since these are located below grade. Since there are no internal watertight barriers to protect the rooms below grade in the control building, it is conservatively assumed that failure of one of the watertight doors on the control building will result in core damage.

The turbine building and service building are not designed to withstand flooding. Therefore, it is conservatively assumed that any equipment in the turbine building or service building is failed by the flooding caused by multiple, ~~concurrent~~ cascading upstream dam failures. PRA-related equipment housed in the turbine building includes the condensate and feedwater systems and the combustion turbine generator (CTG).

When notified of an upstream dam failure, steps will be taken (Refer to Section 19.9.3) to ensure that the watertight main control room access door will be closed prior to flood waters reaching the STP site. Since many hours are available to effect this action and the action is simple and visually verifiable, the probability of failing to ensure closure of the door is considered sufficiently small as to be neglected. Closing this door prevents water from entering the control building.

Since the flooding is assumed to cause a loss of offsite power, all equipment powered from non-essential electrical buses would be lost. The loss of offsite power will result in the EDGs starting and loading to their respective essential electrical buses. The CTG is conservatively assumed failed by the flood so failure of all three EDGs would result in a station blackout (SBO). For this analysis, a SBO is assumed to be non-recoverable and results in core damage.

If one or more EDG starts and loads its respective buses, then the reactor can be brought to safe shutdown using equipment powered from the essential AC buses.

The accident progression for this event tree is similar to that of a loss of offsite power. However, for multiple, ~~concurrent~~ cascading upstream dam failures, it is assumed that offsite power is not recovered and that failure to insert control rods or a subsequent station blackout result in core damage.

The fourth and fifth paragraphs in supplemental Section 19R.7.5.1 are changed as shown below:

The MCR breach analysis described in Section 2.4S.4 was used to develop a minimum available warning time from water at the South Security Gate House, approximately El. 32.0' MSL, to water at the entrances to safety-related buildings, El. 35.0' MSL. At least 30 minutes is available for operator action to close the normally open access door between the Service Building and the Control Building once water reaches the South Security Gate House. ~~As discussed above, development of a main cooling reservoir breach is expected to take from 15 minutes to one hour.~~ Once the security staff notifies the control room of the breach, closing and securing the watertight door takes less than one minute. Therefore, it is assumed that a moderate and adequate amount of time is available to effect the actions to close the control room access door. Then the failure probability for this event was assigned using the values in the ABWR Standard Safety Analysis Report (SSAR) Table 19R-4.

Even if operator action to close the normally-open door is successful, failure of any one of the watertight doors that allow access to the reactor building or control building could randomly fail. Using the values in the SSAR Table 19R-4, the probability of random door failures that allow water to enter either the control building or the reactor building was calculated.

Supplemental Section 19R.7.5.2 title, second and last paragraphs are is changed as shown below:

19R.7.5.2 Multiple, ~~Concurrent~~ Cascading Upstream Dam Failures Accident

IEDAM - Multiple ~~Concurrent~~ Cascading Upstream Dam Failures
This top event represents the failure of the three dams upstream of the STP site on the Colorado River. This event is described above.

Since failure of each of the top nodes on the IEDAM event tree results in core damage and since each of the top nodes is independent of the others, the total CDF for an external flooding event caused by multiple, ~~concurrent~~ cascading upstream dam failures is the product of the initiating event frequency, the success probability of any previous nodes, and the top node failure probability. The total CDF for multiple, cascading upstream dam failure ~~a breach of the main cooling reservoir~~ is determined to be very low.

Supplemental Section 19R.7.9 is changed as shown below:

> The conclusions from the ABWR probabilistic external flooding analysis are that the risk from external flooding is acceptably low, even with the ~~conservative~~ assumption that the watertight normal access door to the control room is open. ~~The risk from external flooding would be significantly lower if analyzed assuming that the door is closed, as described in FSAR Section 2.4S.10.~~ It is also concluded that the incremental risk from external flooding events is within the goals for an increase in CDF or LERF.

An additional reference is added to Section 19R.8 as shown below:

19R-4 ANS 2.8-1992, Determining Design Basis Flooding at Power Reactor Sites, American Nuclear Society, 1992

RAI 02.05.04-13, Supplement 1:

The following proposed revisions to COLA, Part 2, Tier 2, Subsections 2.5S.4.2, 2.5S.4.10, 2.5S.4.11 and 2.5S.4.13 incorporate the sample calculation methodology and the changes described in the response to RAI 02.05.04-13 and RAI 02.05.04-15. These proposed changes to the COLA also provide clarification to RAI responses and support closure of FSAR commitments COM 2.5S-2 and COM 3H-2.

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 B405DH [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 ~~previous location of the~~ former UHS Basin/RSW area ~~(from here on referred to as "west of the Power Block")~~ 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, ~~west of the Power Block,~~ 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~~-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 ~~31~~ 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 ~~west of~~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 ~~west of~~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 ~~and~~ through 2.5S.4-~~11~~ 13 ~~through~~ and 2.5S.4-15 for the STP 3 area, the STP 4 area, ~~the area west of the Power Block, and~~ for the area outside the Power Block, respectively. The site-wide average uncorrected SPT N-value was 10 blows/foot for Stratum A.

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

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 ~~strata~~ 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). ~~(i.e., N_1).~~ 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 ~~uncorrected~~ SPT N_{60}~~N~~-value to yield the normalized SPT $(N_1)_{60}$~~N_1~~-value, and which varies with depth to a maximum ~~value~~ 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 ~~tons~~ pounds per square foot (~~tsf~~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.

~~Eleven drilling rigs were employed during this subsurface investigation, with SPT hammer energy measurements made at each of the drilling rigs employed. Energy measurements were made in accordance with ASTM D 6066 (Reference 2.5S.4-6). As the SPT N-value used in correlations with engineering properties is the value~~

~~corrected to 60% hammer efficiency, the normalized N_1-values were further corrected based on the drilling rig-specific hammer energy measurements (energy transfer ratios [ETRs]), in accordance with ASTM D 6066 (Reference 2.5S.4-6). The average hammer energy corrections for hammers employed in this subsurface investigation for ETRs ranging from 72% to 99% were 1.21 to 1.65 (e.g., 72% measured energy/60% base line = 1.21 hammer energy correction; 99% measured energy/60% base line = 1.65 hammer energy correction). Additional correction factors for boring diameter, for rod length, and the presence/absence of an SPT sampler liner were also applied (Reference 2.5S.4-5). A summary of the measured ETR values and the resulting hammer energy corrections for each drilling rig employed is presented in Table 2.5S.44. Partially corrected SPT N_1-values from each boring were then fully corrected using the appropriate hammer energy corrections, and using the additional correction factors in accordance with Reference 2.5S.4-5. The resulting fully-corrected SPT N-values area commonly termed $(N_1)_{60}$. A summary of corrected SPT $(N_1)_{60}$-values for all site areas and all soil strata is presented in Table 2.5S.4-5.~~

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 ~~corrected~~ SPT N_{60}~~$(N_1)_{60}$~~-value for Stratum A was ~~17~~ 13 blows/foot. An SPT N_{60}~~$(N_1)_{60}$~~-value of ~~15~~ 11 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on ~~corrected~~ SPT N_{60}~~$(N_1)_{60}$~~-values Stratum A is considered medium stiff~~stiff to very stiff~~.

CPTs were additionally performed in Stratum A soils. Site-wide the CPT tip resistance, q_t, in this stratum ranged from 2 tsf to ~~212~~ 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 ~~29~~ 30 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figure 2.5S.4-16, Figure 2.5S.4-17, ~~Figure 2.5S.4-18,~~ and 2.5S.4-19 for the STP 3 area, the STP 4 area, ~~area west of the Power Block~~, and for the area outside the Power Block, respectively.

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

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	~~57~~ 81	16	~~29~~30	~~23~~ 24
Liquid Limit (%)	~~47~~ 44	30	80	~~57~~ 56
Plasticity Index (%)	~~47~~ 44	11	58	37
Fines Content (%)	~~17~~ 11	~~87~~ 90	100	~~94~~ 96
Unit Weight (pcf)	~~12~~ 14	~~119~~ 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~~ly~~ plasticity clay with an average fines content (materials passing the No. 200 sieve) of ~~94~~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 ~~corrected~~ SPT N_{60}~~$(N_1)_{60}$~~-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~~s~~ with ~~corrected~~SPT N_{60}~~$(N_1)_{60}$~~-values (Reference 2.5S.4-7), using:

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

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

Substituting the selected ~~corrected~~ SPT N_{60}~~$(N_1)_{60}$~~-value for Stratum A (~~15~~ 11 blows/foot), an s_u=~~1.9~~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 ~~made at locations adjacent to CPTs (e.g., especially Borings B-904 and~~

~~B-909 compared to CPTs C-901, C-902, C-903, and C-904)~~ 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=~~1.5~~1.7 ksf. The CPT-derived values are shown versus elevation on Figure 2.5S.4-23, 2.5S.4-24, ~~2.5S.4-25~~2.5S.4-26, and 2.5S.4-27 for the STP 3 area, the STP 4 area, ~~the area west of the Power Block,~~ the area outside the Power Block, and site-wide, respectively. Note that SPT correlations were based on ~~1099~~ 1235 field measurements, while CPT correlations were based on ~~862~~ 1584 field measurements made on cohesive soil behavior types within Stratum A. The results of ~~11~~12 laboratory unconsolidated undrained (UU) triaxial strength tests and unconfined compression (UNC) strength tests on selected samples indicated an average s_u=~~1.3~~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=~~1.6~~1.5 ksf was selected for Stratum A, averaged from the SPT ~~$(N_1)_{60}$~~N_{60}-value correlations, the CPT correlations, and the laboratory testing results.

Laboratory testing to determine the consolidated-undrained and drained (effective) shearing strengths ~~drained angle of shearing resistance~~ 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° ≤ ϕ' ≤ 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. ~~For engineering purposes, the drained friction angle (ϕ') for Stratum A, was selected at the same value as the ϕ' for Stratum D, or ϕ'=20 degrees.~~ Note that Strata A, D, F, and J Clay (discussed in following subsections), all had similar plasticity. Laboratory soil strength test results, including 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 ~~7~~ 6.7 ksf, with an overconsolidation ratio (OCR)=~~8~~ 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:

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

Which is reordered to give:

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

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~~=1.0~~ greater than 10 and were based on ~~855~~ 1581 field measurements. CPT-derived OCR data are shown on Figures 2.5S.4-29, ~~through~~ 2.5S.4-30, 2.5S.4-32 and 2.5S.4-33 for the STP 3 area, the STP 4 area, ~~the area west of the Power Block,~~ 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 overconsolidated. 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 elastic modulus (E) for fine-grained cohesive soils was evaluated using the following relationships (Reference 2.5S.4-~~9~~55):

$E = (100 \text{ to } 500)\ s_u (OCR)^{0.5}$, (use 300)~~600 s_u~~ 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~~ substituting the previously established s_u values for Stratum A soils (s_u=~~1.6~~1.5 ksf, OCR=7), an E=~~960~~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

$G_{0.0001\%} / G_{.375\%} = 21/(PI)^{0.5}$~~PI~~ Equation 2.5S.4-7

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

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=~~1, 112~~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=~~1,050~~ 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 the following relationship, (i.e., reordering Equation 2.5S.4-5)~~:

$G = E_d/(2\,[1 + \mu_d])$ 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=~~0.45~~0.30 for Stratum A ~~for clay~~, a G=~~331~~ 369 ksf was estimated ~~based on the s_u-derived E, while a G=384 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a~~ using Equation 2.5S.4-8 and the value of E_d noted above for Stratum A. A value of G=~~360~~ 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 ~~$(N_1)_{60}$~~ 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 (References 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 $n=\sin(\phi')$. 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:

$K_{0,OCR} = (OCR)^{0.5}\left(0.4 + 0.15\sin\left(\frac{\pi(PI)}{120}\right)\right)$ 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; $K_{0,NC}$ ~~and K_o~~=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; ~~and~~ $K_{0,NC}$ ~~K_o~~=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~~, and UHS Basin/RSW area~~ major structure footprints (including Seismic Category I structures).

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

2.5S.4.2.1.2 Stratum B

Stratum B soils were encountered below Stratum A in a majority of the borings and CPTs made site-wide. Stratum B was not encountered in Borings B-307, B-312, B-313, B-412, B-427, B-433, B-434, B-908, B-928, ~~and~~ B-929, ~~or in~~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 ~~accurately~~descriptively, a fine-grained noncohesive soil).

The thickness of Stratum B was estimated from the borings and CPTs. Inside the Power Block area, the thickness of Stratum B varied from 0.5 feet to 16 feet, with an average thickness of 7 feet, and the base elevation varied from El. ~~-9~~-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 ~~7~~ 8 blows/foot. In the STP 4 area, uncorrected SPT N-values in Stratum B ranged from ~~3~~ 2 blows/foot to 40 blows/foot, with an average uncorrected SPT N-value of 12 blows/foot. In the area ~~west of~~outside the Power Block, uncorrected SPT N-values in Stratum B ranged from ~~2~~ 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 ~~west of~~ 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 ~~and~~ through 2.5S.4-~~11~~ 13 ~~through~~ and 2.5S.4-15 for the STP 3 area, the STP 4 area, the area ~~west of the Power Block and for 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, ~~u~~corrected SPT ~~N~~N_{60}-values for sandy soils from each boring were corrected to an effective overburden pressure of one ~~tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections~~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 ~~14~~ 15 blows/foot. An SPT $(N_1)_{60}$value of ~~10~~ 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 ~~loose to~~ medium dense.

CPTs were additionally performed in Stratum B soils. Site-wide, the CPT tip resistance, q_t, in this stratum ranged from 11 tsf to 204 tsf, with an average of ~~53~~ 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 ~~61~~ 62 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figures 2.5S.4-16, 2.5S.4-17 and ~~through~~ 2.5S.4-19 for the STP 3 area, the STP 4 area, and for the area ~~west of and for the area~~ outside the Power Block, respectively.

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

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	~~30~~ 36	18	28	24
Liquid Limit (%)	~~17~~ 14	Non-Plastic	~~70~~ 46	~~38~~ 33
Plasticity Index (%)	~~17~~ 14	Non-Plastic	~~45~~ 26	~~19~~ 14
Fines Content (%)	~~17~~ 19	36	94	~~71~~ 67
Unit Weight (pcf)	~~6~~ 5	117	128	121

Test results are summarized in Table 2.5S.4-8. Note that 9 of the ~~17~~ 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, ~~to medium plasticity clay~~ with an average fines content (materials passing the No. 200 sieve) of ~~71~~ 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 and the selected corrected SPT $(N_1)_{60}$-value for Stratum B (~~10~~ 12 blows/foot), a value of ϕ'=30 degrees (for fine sand) was estimated. ~~A value of ϕ'=28 degrees was considered appropriate.~~

The drained friction angle, ϕ', 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~~ 39.5 degrees. Note that SPT correlations were based on ~~198~~ 175 field measurements, while CPT correlations were based on ~~298~~ 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 ~~through~~ 2.5S.4-38 for the STP 3 area, the STP 4 area, the area ~~west of the Power Block, the area~~ outside the Power Block, and site-wide, respectively.

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

Parameter	From SPT Correlation	From CPT Correlation	From ~~Direct~~ Triaxial Shear Testing
ϕ' (degrees)	~~28~~30	~~39~~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 (Reference 2.5S.4-9)~~ 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 = ~~36~~ 47 N (in ksf) Equation 2.5S.4-13

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

Substituting the previously established ~~average~~ corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Stratum B soils (~~10~~ 11 blows per foot), an E=~~360~~ 515 ksf was estimated.

~~Other relationships for E for coarse-grained soils (Reference 2.5S.4-1.0), especially~~ For coarse grained soil strata, relationships employing shear wave velocity were according to Equations 2.5S.4-5 and 2.5S.4-6. ~~and the following:~~ 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.

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

~~where,~~

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

Using the V_s=725 feet/second for Stratum B obtained from measurements at the site (refer to Subsection 2.5S.4.4 for further discussion), and using μ=0.30 for sand and γ=121 pcf for Stratum B, an E=~~515~~ 1540 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=~~460~~ 1200 ksf was selected for Stratum B. This compares with the range of 500 ≤ E_s ≤ 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 shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=139 ksf was estimated based on the SPT $(N_1)_{60}$ value derived E, while a G=212 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=185ksf 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 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.4-1 0, 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_o~~ $K_{0,NC}$ =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~~, and UHS Basin/RSW area~~ major structure footprints (including Seismic Category I structures).

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

2.5S.4.2.1.3 Stratum C

Stratum C soils were encountered below Stratum B in a majority of the borings and CPTs made site-wide. Boring B-911, and CPTs C-302, C-404, ~~and~~ C-916, 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 ~~3~~5 feet to 30 feet, with an average thickness of approximately 20 feet, and the base elevation varied from El.-24 feet to El. ~~-6~~-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 ~~120~~ 122 blows/foot, with an average uncorrected SPT N-value of 23 blows/foot. ~~In the area west of the Power Block, uncorrected SPT N-values in Stratum C ranged from 7 blows/foot to 67 blows/foot, with an average uncorrected SPT N-value of 24 blows/foot.~~ Additional SPT N-value information on this stratum at locations other than the STP 3 area~~,~~ and the STP 4 area~~, and the area west of 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 ~~and~~ through 2.5S.4-~~11~~13, ~~through~~and 2.5S.4-15 for the STP 3 area, the STP 4 area, and ~~for the area west of the Power Block 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, ~~un~~corrected SPT ~~N~~N_{60}-values for sandy soils from each boring were corrected to an effective overburden pressure of one ~~tsf (i.e., N_1) by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections (~~ 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 ~~35~~39 blows/foot, $(N_1)_{60}$=35 blows/foot ~~which~~ 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 ~~166~~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 ~~156~~ 152 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figures 2.5S.4-16, 2.5S.4-17, and ~~through~~ 2.5S.4-19 for the STP 3 area, the STP 4 area, ~~area west of the Power Block,~~ and for the area outside the Power Block, respectively.

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

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	~~38~~ 45	~~19~~17	27	~~24~~23
Liquid Limit (%)	2	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	2	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	~~36~~ 39	~~6~~5	96	~~25~~ 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 ~~25~~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 ~~38~~ 36 degrees (for fine sand) was estimated. ~~A value of ϕ'=36 degrees~~

~~was considered appropriate.~~ The drained friction angle, φ', was also estimated using the CPT data, following a CPT- φ' correlation (Reference 2.5S.4-15) given as Equation 2.5S.4-12D. Drained friction angle values calculated from the CPT data indicated an average φ'=42 degrees. Note that SPT correlations were based on ~~444~~ 487 field measurements, while CPT correlations were based on ~~1355~~ 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 ~~through~~ 2.5S.4-38 for the STP 3 area, the STP 4 area, ~~the area west of the Power Block,~~ the area outside the Power Block, and site-wide, respectively.

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

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

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

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

The elastic modulus, E, for coarse-grained soils was evaluated using Equation 2.5S.4-13. Substituting the previously established ~~average~~ corrected SPT ~~$(N_1)_{60}$~~ N_{60} value for Stratum C soils (~~35~~ 38 blows per foot) an E=~~1,260~~ 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=~~606~~ 1820 ksf was estimated. Using an average of the E-values estimated from the ~~average~~ corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=~~850~~ 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 shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=485 ksf was estimated based on the SPT $(N_1)_{60}$-value derived E, while a G=233 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=320 ksf was selected for Stratum C.~~ 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~~ $K_{0,NC}$=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, ~~and~~ 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, ~~and~~ 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 ~~1.5~~ 9.0 feet to 34 feet, with an average thickness of ~~22~~ 21 feet, and the base elevation varied from El. -45 feet to El. ~~-18~~ -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 ~~34~~ 54 blows/foot, with an average uncorrected SPT N-value of 15 blows/foot. ~~In the area west of the Power Block, uncorrected SPT N-values in Stratum D ranged from 5 blows/foot to 54 blows/foot, with an average uncorrected SPT N-value of 15 blows/foot.~~ Additional SPT N-value information on this stratum at locations other than the STP 3 area and~~,~~ the STP 4 area~~, and the UHS Basin/RSW area~~ is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 ~~and 2.5S.4-11~~ through 2.5S.4-~~15~~ 13 and 2.5S.4-15 for the STP 3 area, the STP 4 area, and ~~for the area west of~~ 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 ~~effective overburden pressure of one tsf~~energy ratio of 60 percent (i.e., ~~N_1~~ 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 ~~(leading to fully corrected values of $(N_1)_{60}$)~~. 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-~~5~~6. The average corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Stratum D was ~~17~~ 25 blows/foot. An SPT ~~$(N_1)_{60}$~~ N_{60}-value of ~~15~~ 23 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT N_{60}~~$(N_1)_{60}$~~-values, Stratum D is considered ~~stiff to~~ very stiff.

CPTs were additionally performed in Stratum D soils. Site-wide, the CPT tip resistance, q_t, in this stratum ranged from 11 tsf to 185 tsf, with an average of ~~41~~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, ~~through~~2.5S.4-17 and 2.5S.4-19 for the STP 3 area, the STP 4 area, and for the ~~area west of the Power Block and for the~~ area outside the Power Block, respectively.

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

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	~~55~~ 90	16	~~43~~ 53	~~25~~ 26
Liquid Limit (%)	~~38~~ 53	20	84	~~58~~ 57
Plasticity Index (%)	~~38~~ 53	2	59	~~38~~ 37
Fines Content (%)	~~13~~ 26	~~24~~ 18	100	~~72~~ 79
Unit Weight (pcf)	~~14~~ 26	111	~~129~~ 130	~~121~~ 123

Test results are summarized in Table 2.5S.4-8. Note that ~~four~~ five of the ~~38~~ 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 ~~72~~ 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 ~~and~~, CL, CL-ML and ML. Based on laboratory testing, an average unit weight of ~~121~~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 ~~$(N_1)_{60}$~~ 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 ~~$(N_1)_{60}$~~ N_{60}-values (Reference 2.5S.4-7) using Equation 2.5S.4-2. Substituting the selected corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Stratum D (~~15~~ 23 blows/foot), an s_u = ~~1.9~~ 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 ~~S~~shear strength values calculated from the CPT data indicated an average s_u = ~~3.1~~ 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 ~~through~~ 2.5S.4-27 for the STP 3 area, the STP 4 area, ~~the area west of the Power Block,~~ the area outside the Power Block, and site-wide, respectively. Note that SPT correlations were based on ~~449~~ 520 field measurements, while CPT correlations were based on ~~721~~ 958 field measurements made within Stratum D. Results of ~~eight~~ 11 laboratory UU and UNC strength tests made on selected samples indicated an average s_u = ~~1.7~~ 1.9 ksf. ~~By excluding the two lowest laboratory strength test results of s_u =0.3 ksf and 0.4 ksf (likely made on samples of poor or non-representative quality), an average s_u =2.1ksf resulted.~~ 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'= ~~1.3~~ 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' (~~tsf~~ ksf)	~~1.3~~ 1.2
ϕ (degrees)	4
c (~~tsf~~ 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° ≤ ϕ' ≤ 27° for Stratum D in the normally consolidated stress range. ~~Based on the above, a ϕ'=20 degrees was selected for Stratum D soils, and for similar fine-grained soil strata (i.e., Strata A, F, and J Clay).~~ 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 ~~five~~ eight consolidation tests made on selected samples indicated that, on average, Stratum D was preconsolidated to approximately ~~12.5~~13.4 ksf, with an OCR=~~3.5~~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=~~3~~4.2, and were based on ~~720~~ 958 field measurements. CPT-derived OCR data are shown on Figure 2.5S.4-29, Figure 2.5S.4-30, ~~Figure 2.5S.4-31,~~ Figure 2.5S.4-32, and Figure 2.5S.4-33 for the STP 3 area, the STP 4 area, ~~the area west of the Power Block,~~ 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=~~1800~~ 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=~~121~~ 122 pcf for Stratum D, and PI=40 for Stratum D, an E=~~2,807~~ 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=~~2500~~ 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.

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

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 $\cancel{K_0}$ $K_{0,NC}$=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-913~~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 ~~1.5~~5 feet to ~~36.5~~36 feet, with an average thickness of 18 feet, and the base elevation varied from El. ~~-72~~-71 feet to El. ~~-37~~-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 ~~84~~96 blows/foot, with an average uncorrected SPT N-value of ~~39~~41 blows/foot. ~~As noted above, Stratum E was largely absent in the area west of the Power Block, with only one uncorrected SPT N-value, 51 blows/foot, measured in that area.~~ Additional SPT N-value information on this stratum at locations other than the STP 3 area~~,~~ and the STP 4 area ~~and the UHS Basin/RSW areas~~ is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 through ~~2.5S.4-12 and~~ 2.5S.4-13, ~~through~~ and 2.5S.4-15 for the STP 3 area, the STP 4 area, ~~the area west of the Power Block,~~ and for the area outside the Power Block, respectively. The site-wide average uncorrected SPT N-value was ~~35~~37 blows/foot for Stratum ~~D~~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 (Cs=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, ~~un~~corrected SPT ~~N~~N_{60}-values in sandy soil strata from each boring were corrected to an effective overburden pressure of one ~~tsf (i.e., N_1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (~~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 ~~34~~35 blows/foot. An SPT $(N_1)_{60}$value of ~~30~~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 ~~228~~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 ~~144~~133 (dimensionless). As noted above, Stratum E was largely absent in the area west ~~of the Power Block~~ and northwest 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 (%)	~~38~~ 48	~~17~~ 15	26	21
Liquid Limit (%)	6	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	6	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	~~35~~ 43	3	96	~~18~~ 20
Unit Weight (pcf)	~~8~~ 9	~~113~~ 111	~~127~~ 133	~~122~~ 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 limits for other soil strata are also shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Stratum E soils were characterized, on average, as silty sand with an average fines content (materials passing the No. 200 sieve) of ~~18~~20%. Note that the maximum ~~96%~~ 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). ~~This~~ These results represent~~s an~~ isolated thin clay and silt lenses within the Stratum E sand. The next highest fines content reported was ~~50~~ 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 ~~122~~ 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 (~~30~~ 31 blows/foot), from Equation 2.5S.4-12B a value of ϕ'=of 39 degrees (for fine to medium sand) was estimated. ~~A value of ϕ'=37 degrees was considered appropriate.~~ The drained friction angle, ϕ', was also estimated using the CPT data, following a CPT ϕ' correlation (Reference 2.5S.4-15) given as Equation 2.5S.4-12D. Drained friction angle values calculated from the CPT data indicated an average ϕ'=~~40~~ 39 degrees. Note that SPT correlations were based on ~~372~~ 389 field measurements, while CPT correlations were based on ~~414~~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)	~~37~~ 39	~~40~~ 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 ~~average~~ corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Stratum E soils (~~30~~ 53 blows per foot), an E=~~1,080~~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=~~122~~ 123 pcf for Stratum E, an E=~~1,149~~3475 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=~~1,100~~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 shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=415 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=442ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=425ksf 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 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~~ $K_{0,NC}$=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 ~~one foot~~2 to ~~55~~ 30 feet, with an average thickness of ~~16~~ 15 feet, and the base elevation varied from El. ~~-93~~ -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 ~~98~~ 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. ~~In the area west of the Power Block, uncorrected SPT N-values in Stratum F ranged from 12 blows/foot to 32 blows/foot, with an average uncorrected SPT N-value of 19 blows/foot.~~ Additional SPT N-value information on this stratum at locations other than the STP 3 area and the STP 4 area, ~~and the UHS Basin/RSW area~~ is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 through ~~2.5S.4-12 and~~ 2.5S.4-13 and through

2.5S.4-15 for the STP 3 area, the STP 4 area, ~~area west of the Power Block,~~ 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 ~~effective overburden pressure of one tsf~~energy ratio of 60 percent (i.e., ~~N_1~~ 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. (leading to fully corrected values of $(N_1)_{60}$).~~ A summary of corrected SPT N_{60}-values ~~$(N_1)_{60}$-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-~~5~~6. The average ~~corrected~~ SPT ~~$(N_1)_{60}$~~ N_{60}-value for Stratum F was ~~19~~ 36 blows/foot. An SPT ~~$(N_1)_{60}$~~ N_{60}-value of ~~15~~ 34 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT ~~$(N_1)_{60}$~~ N_{60}-values, Stratum F is considered ~~stiff to very stiff~~hard.

CPTs were additionally performed in Stratum F soils. Site-wide, the CPT tip resistance, q_t, in this stratum ranged from ~~24~~ 23 tsf to 118 tsf, with an average of ~~43~~ 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 ~~17~~ 19 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figures 2.5S.4-16 ~~through~~ and 2.5S.4-~~18~~17, for the STP 3 area, and the STP 4 area, ~~and the area west of the Power Block,~~ respectively.

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

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	~~46~~ 66	18	~~29~~ 33	~~24~~ 24
Liquid Limit (%)	~~34~~ 47	27	74	~~58~~ 57
Plasticity Index (%)	~~34~~ 47	6	53	~~38~~ 37
Fines Content (%)	~~10~~ 14	~~13~~ 56	99	~~89~~ 94
Unit Weight (pcf)	~~13~~ 18	120	~~129~~ 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~~ity~~ clay with an average fines content (materials passing the No. 200 sieve) of ~~89~~94%. Note that the minimum ~~13~~56% fines content reported occurred at Boring B-~~328~~443 from depths of ~~98.5~~103.5 feet to ~~100~~105 feet. This result represents an isolated thin sand lens within the Stratum F clay. All other fines contents reported were greater than ~~90~~85%. The USCS designations for Stratum F were mainly fat clay, lean clay, and silty clay, ~~and occasionally silty sand,~~ 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 ~~$(N_1)_{60}$~~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 ~~$(N_1)_{60}$~~N_{60}-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Stratum F (~~15~~34 blows/foot), an s_u=~~1.9~~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=~~3.6~~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 ~~through~~ 2.5S.4-27 for the STP 3 area, the STP 4 area, area ~~west of~~outside the Power Block, and site-wide, respectively. Note that SPT correlations were based on ~~291~~ 315 field measurements, while CPT correlations were based on ~~376~~ 305 field measurements made on cohesive soil behavior types within Stratum F. The results of ~~10~~ 17 laboratory UU and UNC strength tests made on selected clay samples indicated an average s_u=2.7 ksf. ~~By excluding the lowest laboratory strength test result of s_u=0.7 ksf (likely made on a sample of poor or non-representative quality), an average s_u=2.9 ksf resulted.~~

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~~, it was deemed that the CPT-derived s_u results from this subsurface investigation were more representative, and~~ an undrained shear strength of s_u=~~3.2~~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' (~~tsf~~ksf)	2.0
ϕ (degrees)	3
c (~~tsf~~ksf)	2.1

The parameters above are for stresses below the preconsolidation stress of the Stratum F. Based on the ~~results of CIU-bar tests made on Stratum D (having similar plasticity to Strata A, F, and J Clay, as noted above)~~ average plasticity index, Reference 2.5S.4-7 indicates a value range of $20° \leq \phi' \leq 26°$ 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 ~~three~~ six consolidation tests made on selected samples indicated that, on average, Stratum F ~~was~~ is preconsolidated to approximately ~~16.5~~18.6 ksf, with an OCR=~~2.9~~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~~ 2.4 and were based on ~~376~~ 305 field measurements. CPT-derived OCR data are shown on Figure 2.5S.4-29, Figure 2.5S.4-30, ~~Figure 2.5S.4-31,~~ and Figure 2.5S.4-33 for the STP 3 area, the STP 4 area, ~~the area west ofoutside 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=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=~~3.2~~3.4 ksf, OCR=2.6), an E=~~1,920~~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=~~3028~~ 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=~~2600~~ 2570 ksf was selected for Stratum F. This compares to a value range of 1050 ksf $\leq E_s \leq$ 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.

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

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~~ $K_{0,NC}$=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 ~~(from Stratum D)~~, a sliding coefficient, tangent δ=0.3, was selected for Stratum F.

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

2.5S.4.2.1.7 Stratum H

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

The thickness of Stratum H was estimated from the borings and CPTs. Inside the Power Block area, the thickness varied from ~~1 foot~~ 2 feet to 35.5 feet, with an average thickness of ~~17.5~~17.0 feet, and the base elevation of Stratum H varied from El. ~~-95~~ -93.5 feet to El. ~~-63~~ -64.6 feet, with an average of El. ~~-87~~ -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 ~~14~~ 18 blows/foot to 150 blows/foot, with an average uncorrected SPT N-value of ~~47~~ 48 blows/foot. ~~In the area west of the Power Block, uncorrected SPT N-values (only two tests conducted) in Stratum H ranged from 57 blows/foot to 74 blows/foot, with an average uncorrected SPT N-value of 66 blows/foot.~~ Additional SPT N-value information on this stratum at locations other than the STP 3 area, and the STP 4 area ~~and the UHS Basin/RSW areas~~ is presented in Table 2.5S.4-3. Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 through 2.5S.4-13, and ~~2.5S.4-11 through~~ 2.5S.4-15 for the STP 3 area, the STP 4 area~~, the area west of the Power Block,~~ 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, ~~un~~corrected SPT ~~N~~N_{60}-values for sandy strata from each boring were corrected to an effective overburden pressure of one ~~tsf (i.e., N_1),~~ atmosphere (approximately one tsf) ~~by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (~~leading to fully-corrected values of $(N_1)_{60}$. A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all sandy soil strata is presented in Table 2.5S.4-5. The average corrected SPT $(N_1)_{60}$-value for Stratum H was ~~34~~ 35 blows/foot. An SPT $(N_1)_{60}$ value of ~~30~~ 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 ~~104~~ 99 (dimensionless). Note that CPT tip resistance profiles versus elevation are shown on Figure 2.5S.4-16 and~~,~~ 2.5S.4-17~~, and 2.5S.4-18,~~ for the STP 3 area~~,~~ and the STP 4 area, ~~and the area west of the Power Block,~~ respectively.

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

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	~~13~~ 16	12	24	19
Liquid Limit (%)	1	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	1	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	~~12~~ 14	6	95	~~16~~ 18
Unit Weight (pcf)	~~2~~ 4	~~122~~121	135	~~128~~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 ~~16~~ 18%. Note that ~~the maximum 95% fines content reported occurred at Boring B-305DH/DHA from depths of 103 feet to 105 feet~~ two samples taken from borings B-305DH/DHA, ~~B-440,~~ and B-443 at depths of ~~98.5~~103 feet to ~~110~~ 111.5 feet had fines contents ranging from 44% to 95%. ~~This~~ These results represent~~s an~~ isolated thin clay lenses within the Stratum H sand. The next highest fines content reported was ~~13~~ 19%. The USCS designations for Stratum H were mainly poorly graded sand with silt, ~~poorly graded sand~~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 ~~128~~ 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 (~~30~~ 28 blows/foot) and Equation 2.5S.4-12B, a value of ϕ'=of ~~39~~ 38 degrees (for fine to medium sand) was estimated. ~~A value of ϕ'=37 degrees was considered appropriate.~~ The drained friction angle, ϕ', was also estimated using the CPT data, following a CPT-ϕ' correlation (Reference 2.5S.4-15) given as Equation 2.5S.4-12D. Drained friction angle values calculated from the CPT data indicated an average ϕ'=~~38~~ 37 degrees. Note that SPT correlations were based on ~~130~~ 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~~, through~~ 2.5S.4-~~35~~36, and 2.5S.4-38 for the STP 3 area, the STP 4 area, ~~the area west of the Power Block,~~ and site-wide, respectively.

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

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	~~37~~ 38	~~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 ~~average~~ corrected SPT ~~$(N_1)_{60}$~~N_{60}-value for Stratum H soils (~~30~~ 58 blows per foot), an E=~~1080~~ 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=~~128~~ 125 pcf for Stratum H, an E=~~1193~~ 3500 ksf was estimated. Using an average of the E-values estimated from the ~~average~~ corrected SPT N_{60}~~$(N_1)_{60}$~~-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=~~1150~~ 3240 ksf was selected for Stratum H. 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 shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=415 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=459 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=450ksf was selected for Stratum H.~~ 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)
- ~~Sub-stratum J Sand 2~~

The thickness of Stratum J was estimated from the borings. No CPTs fully penetrated Stratum J or the other underlying strata. Overall, the stratum had an average thickness of ~~99~~ 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. ~~Twelve~~ Thirteen of ~~41~~ 47 borings encountered a sand/silt interbed (Sub-stratum J Interbed 1) within Substratum 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, ~~and~~ B-416 and B-443. Sub-stratum J Clay 1 ranged in thickness from ~~4.5~~ 10.0 feet to ~~15.5~~ 49 feet, with an average thickness of ~~9.5~~ 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. ~~-97~~ -98 feet.

Where encountered, Sub-stratum J Interbed 1 ranged in thickness from ~~5.5 feet~~ 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. ~~-106~~ -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 ~~49~~ 35 feet, with an average thickness of ~~29~~ 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. ~~-120~~ -119 feet.

Sub-stratum J Sand 1 was encountered below Sub-stratum J Clay 1, and was fully penetrated in ~~22~~ 23 borings. Sub-stratum J Sand 1 ranged in thickness from 1.5 feet to 25.5 feet, with an average thickness of ~~14~~ 13 feet. The average base elevation of Substratum J Sand 1 was El. -131 feet. Note that Sub-stratum J Sand 1 generally divided Sub-stratum J Clay 1 and Sub-stratum J Clay 2.

Sub-stratum J Clay 2 was encountered below Sub-stratum J Sand 1 at ~~28~~ 14 borings. ~~Fifteen~~ Thirteen of ~~28~~ 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-301,~~ B-302DH, B-303, ~~B-304,~~ B-305DH/DHA, B-306, ~~B-307,~~ B-319DH, B-402DH, B-403, B404, B-405DH, B-408DH, B-409, ~~and~~ B-428DH, and B-443. Sub-stratum J Clay 2 ranged in thickness from ~~1~~ 3 foot to ~~30~~ 32 feet, with an average thickness of ~~13~~ 15 feet above Substratum J Interbed 2. The average base elevation of Sub-stratum J Clay 2 above Substratum J Interbed 2 (or Sub-stratum J Clay 2 "Top") was El. ~~-142~~ -139 feet.

Where encountered, Sub-stratum J Interbed 2 ranged in thickness from ~~9~~ 8 feet to 30 feet, with an average thickness of 15 feet. The average base elevation of Sub-stratum J Interbed 2 was El. ~~-156~~ -152 feet.

Sub-stratum J Clay 2 below Sub-stratum J Interbed 2 (or Sub-stratum J Clay 2 "Bottom") ranged in thickness from ~~11~~ 12 feet to 38 feet, with an average thickness of ~~27~~ 24 feet. The thickness of the combined Sub-stratum J Clay 2 "Top" and "Bottom" ranged in thickness from ~~21~~ 35 feet to 48 feet, with an average thickness of ~~32~~ 41 feet. The average thickness of Sub-stratum J Clay 2 with Sub-stratum J Interbed 2 included was ~~47~~ 56 feet. The average base elevation of Sub-stratum J Clay 2 was El. ~~-174~~ -184 feet.

Five borings in the STP 3 area, namely B-301, B-304, B-307, B-316, and B-348~~,~~ ~~were terminated in Sub-stratum J Sand 2,~~ encountered a sand layer below Sub-stratum J Clay 2. This ~~stratum~~ sand layer was found neither in the STP 4 area borings, nor in the ~~two~~ three borings in the STP 3 & 4 areas that fully penetrated Stratum J, namely B-305DH/DHA, ~~and~~ B-405DH, and B-443. This ~~stratum~~ 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~~, and Sub-stratum J Sand 2~~

2.5S.4.2.1.8.1 Sub-stratum J Clay

Soil samples were collected from the borings via SPT sampling, and via undisturbed three-inch-diameter tube sampling. SPT N-values (uncorrected) were measured during the sampling, and were recorded on the boring logs. In the STP 3 area, uncorrected SPT N-values in Sub-stratum J Clay ranged from 12 blows/foot to ~~89~~120 blows/foot, with an average uncorrected SPT N-value of ~~30~~ 32 blows/foot. In the STP 4 area, uncorrected SPT N-values in Sub-stratum J Clay ranged from ~~14~~ 13 blows/foot to ~~120~~ 138 blows/foot, with an average uncorrected SPT N-value of 32 blows/foot. ~~In the area west of the Power Block, borings did not reach Sub-stratum J Clay.~~ Additional SPT N-value information on this stratum at locations other than the STP 3 area and the STP 4 area ~~, and the area west of 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 ~~and~~ through 2.5S.4-~~11~~ 13 ~~through~~ 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 ~~31~~ 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 ~~effective overburden pressure of one tsf (i.e., N_1),~~ 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 fully corrected values of $(N_1)_{60}$).~~ 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-~~5~~6. The average corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Sub-stratum J Clay was ~~18~~ 51 blows/foot. An SPT ~~$(N_1)_{60}$~~ N_{60}-value of ~~15~~48 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT ~~$(N_1)_{60}$~~ N_{60}-values, Stratum J Clay is considered ~~stiff to very stiff~~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 ~~134~~ 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 ~~27~~ 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 (%)	~~70~~ 90	~~16~~14	38	23
Liquid Limit (%)	~~58~~ 70	~~30~~ 26	85	54
Plasticity Index (%)	~~58~~ 70	~~12~~ 9	62	35
Fines Content (%)	~~29~~ 39	~~18~~ 55	100	~~89~~ 90
Unit Weight (pcf)	~~37~~ 47	104	~~132~~ 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 ~~89~~ 90%. ~~Note that the minimum 18% fines content reported occurred at Boring B-401 from depths of 153 feet to 155 feet. This result represents an isolated thin sand lens within Sub-stratum J Clay. All other fines contents reported were greater than 65%.~~ The USCS designations for Sub-stratum J Clay were mainly fat clay, lean clay, sandy lean clay, lean clay with sand, fat clay with sand, and occasionally ~~silty sand~~sandy silt, with the ~~predominant~~ USCS group symbols of CH~~, and~~ 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 ~~$(N_1)_{60}$~~N_{60}-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT ~~(N_1)~~N_{60}-value for Sub-stratum J Clay (~~15~~48 blows/foot), an s_u =~~1.9~~ 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=~~3.8~~ 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 ~~215~~ 239 field measurements, while CPT correlations were based on only ~~five~~ 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 ~~27~~ 34 laboratory UU and UNC strength tests made on selected samples indicated an average s_u=~~3.2~~ 3.0 ksf. The ratio of the shear strength of Sub-stratum 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).

~~By excluding the three lowest laboratory strength test result of s_u=0.1 ksf, 0.1 ksf, and 0.7 ksf (likely made on a samples of poor or non-representative quality), an average s_u=3.5 ksf results.~~ Laboratory shear strength test results are summarized in Table 2.5S.4-9A and plotted versus elevation on Figure 2.5S.4-22. UU strength results from the STP 1 & 2 UFSAR (Reference 2.5S.4-3) indicated an average s_u=3.3 ksf for Sub-stratum J Clay (29 test results). Based on all of the above, ~~this, it was deemed that the laboratory-derived s_u results from this subsurface investigation were more representative, and~~ an undrained shear strength of s_u=~~3.5~~ 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 ~~seven~~ eight CIU-bar tests, indicated average (drained/effective) ϕ'=~~8~~ 11 degrees and c'=~~2.6~~ 2.3 ksf and average (undrained/total) ϕ=~~4~~ 7 degrees and c=~~2.9~~ 2.7 ksf.

Parameter	From CIU-Bar
ϕ' (degrees)	~~8~~ 11
c' (~~tsf~~ksf)	~~2.6~~ 2.3
ϕ (degrees)	~~4~~ 7
c (~~tsf~~ksf)	~~2.9~~ 2.7

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

Based on the ~~results of CIU bar tests made on Stratum D (having similar plasticity to Strata A, F, and J Clay, as noted above),~~ 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. ~~Note that~~ 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 ~~10~~ 11 consolidation tests made on selected samples indicated that, on average, Sub-stratum J Clay was preconsolidated to approximately ~~18.6~~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=~~1.8~~1.7 and ~~were~~was based on ~~five~~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=~~3.5~~3.8 ksf, OCR=1.7), an E=~~2100~~ 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=~~1145~~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=~~4157~~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=~~3500~~ 4140 ksf was selected for Sub-stratum J Clay. 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.

~~The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.45 for clay, a G=724 ksf was estimated based on the s_u-derived E, while a G=1433 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=1200 ksf was selected for Sub-stratum J Clay. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.~~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 ~~22~~ 32 blows/foot to 120 blows/foot, with an average uncorrected SPT N-value of ~~73~~ 70 blows/foot. In the STP 4 area, uncorrected SPT N-values in Sub-stratum J Sand ranged from ~~18~~ 20 blows/foot to ~~120~~ 125 blows/foot, with an average uncorrected SPT N-value of ~~56~~ 55 blows/foot. In the area ~~west of the Power Block, and~~ 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 ~~65~~ 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, ~~uncorrected~~ SPT ~~N~~N_{60}-values for sandy soils from each boring were corrected to an effective overburden pressure of one ~~tsf (i.e., N_1) by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed and by other corrections~~ (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 J Sand was ~~36~~ 41 blows/foot. An SPT $(N_1)_{60}$-value of ~~35~~ 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 Substratum J Sand with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	~~13~~ 17	~~19~~ 16	32	~~23~~ 22
Liquid Limit (%)	~~6~~ 9	Non-Plastic	~~62~~ 24	Non-Plastic
Plasticity Index (%)	~~6~~ 9	Non-Plastic	~~35~~ 3	Non-Plastic
Fines Content (%)	~~12~~ 17	10	~~77~~ 97	~~43~~ 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 ~~43~~ 50%. Note that the maximum values for Liquid Limit and for Plasticity Index (PI) reported occurred at Boring B~~409~~ 443 from depths of ~~160~~ 163.5 feet to ~~162~~ 165 feet. These results represent an isolated thin ~~clay~~ silt lens within Sub-stratum J Sand. All other Atterberg Limits tests for Sub-stratum J Sand soils 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 ~~and sandy lean clay and occasionally fat clay~~, with the predominant USCS group symbols of SM and ML. Based on laboratory testing, an average unit weight of 125 pcf was selected for Substratum 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 (~~35~~ 38 blows/foot), a value of ϕ'=of ~~39~~ 41 degrees (for fine to medium sand) was estimated. ~~A value of ϕ'=37 degrees was considered appropriate.~~ Results of one laboratory direct shear test made on selected samples indicated a ϕ'=32 degrees. Laboratory direct shear test results are summarized in Table 2.5S.4-10.

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

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	~~37~~ 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 ~~average~~ corrected SPT ~~$(N_1)_{60}$~~ N_{60} value for Sub-stratum J Sand soils (~~35~~ 94 blows per foot), an E=~~1260~~ 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=~~1641~~ 4925 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=~~1500~~ 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 shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=485 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=631 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=600ksf was selected for Sub-stratum J Sand.~~ 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 substrata were identified:

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

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

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

Sub-stratum K Sand/Silt below Sub-stratum K Clay was also encountered in both borings (B-305DH/DHA and B-405DH). Sub-stratum K Sand/Silt ranged in thickness from 20 feet to ~~30~~ 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. ~~In the area west of the Power Block, borings did not reach Sub-stratum K Clay.~~ Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11, and 2.5S.4-12 and 2.5S.4-13, for the STP 3 area, and the STP 4 area, respectively. The site-wide average uncorrected SPT N-value was 15 blows/foot for Sub-stratum K Clay.

As noted above, uncorrected SPT N-values from each boring were corrected to an ~~effective overburden pressure of one tsf~~energy ratio of 60 percent (i.e., ~~N_1~~ 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 ~~(leading to fully corrected values of $(N_1)_{60}$)~~. 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-~~5~~6. The average corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Sub-stratum K Clay was ~~7~~ 25.5 blows/foot. An SPT ~~$(N_1)_{60}$~~ N_{60}-value of ~~6~~ 26 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT ~~$(N_1)_{60}$~~ N_{60}-values, Stratum K Clay is ~~considered~~ indicated as ~~firm~~ very stiff to hard~~(although this stratum is likely much stiffer as the average corrected SPT $(N_1)_{60}$-value results from a low correction factor, C_n, which was extrapolated beyond its normal stress range)~~.

CPTs did not reach Sub-stratum K Clay.

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

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	~~3~~ 4	17	~~22~~ 35	~~20~~ 23
Liquid Limit (%)	~~2~~ 3	33	~~45~~ 73	~~39~~ 50
Plasticity Index (%)	~~2~~ 3	18	~~31~~ 51	~~25~~ 33
Fines Content (%)	~~1~~ 2	75	~~75~~ 99	~~75~~ 87
Unit Weight (pcf)	~~2~~ 3	~~127~~ 115	132	~~129~~ 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 ~~75~~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 ~~129~~ 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 ~~$(N_1)_{60}$~~ 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 ~~$(N_1)_{60}$~~ N_{60}-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Sub-stratum K Clay (~~6~~26 blows/foot), an s_u=~~0.8~~3.3 ksf was estimated. Note, however, that this average value is based on only two ~~corrected~~ SPT ~~$(N_1)_{60}$~~ N_{60}-values. Also note that CPT data were not available for this substratum. 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 ~~were more~~was the most representative, and an undrained shear strength of s_u=~~3.0~~4.0 ksf (the highest of the two tests) was selected for Substratum 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 Sub-stratum 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. ~~The drained friction angle of Sub-stratum K Clay soils was not evaluated/relevant.~~

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

The elastic modulus (E) for Sub-stratum K Clay was evaluated using Equation 2.5S.4-4B. Substituting the previously established s_u and OCR for Sub-stratum K Clay soils (s_u=~~3.0~~3.9 ksf, OCR=1.3), an E=~~1800~~ 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=~~1145~~ 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=~~129~~ 124 pcf for Sub-stratum K Clay, and PI=~~25~~35 for Sub-stratum K Clay, an E=~~3787~~ 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=~~3100~~ 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.

~~The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.45 for clay, a G=621 ksf was estimated based on the s_u-derived E, while a G=1306 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An average of these two values, with the shear~~

~~wave velocity derived value weighted 2:1, was considered, and a value of G=1050 ksf was selected for Sub-stratum K Clay. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.~~

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 ~~30~~ 40 blows/foot to 120 blows/foot, with an average uncorrected SPT N-value of ~~75~~ 80 blows/foot. ~~In the area west of the Power Block borings did not reach Substratum K Sand/Silt.~~ Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11, and 2.5S.4-12 and 2.5S.4-13, for the STP 3 area, and the STP 4, respectively. The site-wide average uncorrected SPT N-value was ~~75~~ 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, ~~uncorrected~~ SPT ~~N~~N_{60}-values for sandy strata from each boring were corrected to an effective overburden pressure of one ~~tsf (i.e., N_1)~~ atmosphere (approximately one tsf), ~~by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections (~~leading to fully-corrected values of $(N_1)_{60}$~~)~~. A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all 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 ~~31~~ 54 blows/foot. An SPT $(N_1)_{60}$-value of ~~30~~ 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 ~~(although this stratum is likely more dense as the average corrected SPT $(N_1)_{60}$-value results from a low correction factor, C_N, which was extrapolated beyond its normal stress range).~~

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

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

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

Test results are summarized in Table 2.5S.4-8. Note that natural moisture contents and Atterberg limits for other soil strata are presented versus elevation on Figure 2.5S.4-20. Note also that Atterberg limits for other soil strata are shown on a plasticity chart on Figure 2.5S.4-21. For engineering purposes, Sub-stratum K Sand/Silt soils were characterized, on average, as silty sand to sandy silt with an average fines content (materials passing the No. 200 sieve) of 45%. The USCS designations for Sub-stratum K Sand/Silt were mainly silty sand and sandy silt, with the predominant USCS group symbols of SM and ML. Based on laboratory testing, an average unit weight of 127 pcf was selected for Sub-stratum K Sand/Silt.

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

The drained friction angle, ϕ', was estimated from empirical correlations with corrected SPT $(N_1)_{60}$-values, according to Reference 2.5S.4-14. Using Equation 2.5S.4-12A and the selected corrected SPT $(N_1)_{60}$-value for Sub-stratum K Sand/Silt (~~30~~ 27 blows/foot), a value of ϕ'=of ~~38~~ 34 degrees (for fine sand) was estimated. ~~A value of ϕ'=36 degrees was considered appropriate.~~ Note, however, that this average value is based on only two corrected SPT $(N_1)_{60}$-values. Results of one laboratory direct shear test made on selected samples indicated a ϕ'=29 degrees. Laboratory direct shear test results are summarized in Table 2.5S.4-10.

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

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	~~36~~ 34	---	29

Based on the above a ϕ'=~~33~~ 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 ~~$(N_1)_{60}$~~ N_{60}-value for Sub-stratum K Sand/Silt soils (~~30~~ 68 blows per foot) an E=~~1080~~ 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=~~1924~~ 5775 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=~~1650~~ 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 shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=415 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=740 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=650ksf was selected for Sub-stratum K Sand/Silt. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.~~ The 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~~ 5.5 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. ~~In the area west of the Power Block, borings did not reach Stratum L.~~ Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11, and 2.5S.4-12 and 2.5S.4-13, for the STP 3 area, and the STP 4 area, respectively. The site-wide average uncorrected SPT N-value was 23 blows/foot for Sub-stratum L.

As noted above, uncorrected SPT N-values from each boring were corrected to ~~an effective overburden pressure of one tsf (i.e., N_1),~~ 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 ~~fully-corrected~~ values of N_{60}~~$(N_1)_{60}$~~). A summary of corrected SPT ~~$(N_1)_{60}$~~ N_{60}-values, for all site areas and all soil strata is presented in Table 2.5S.4-6. The average corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Stratum L was ~~9~~ 38 blows/foot. An SPT ~~$(N_1)_{60}$~~ N_{60}-value of ~~8~~ 36 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT ~~$(N_1)_{60}$~~ N_{60}-values, Stratum L is ~~firm to stiff (although this stratum is likely much stiffer as the average corrected SPT N-value results from a low correction factor, C_N, which was extrapolated beyond its normal stress range)~~ 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 ~~75~~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 ~~129~~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 ~~$(N_1)_{60}$~~N_{60}-values (Reference 2.5S.4-7), using Equation 2.5S.4-2. Substituting the selected corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Stratum L (~~8~~36 blows/foot), an s_u =~~1.0~~4.5 ksf was estimated. Note, however, that this average value is based on only two corrected SPT ~~$(N_1)_{60}$~~ N_{60}-values. Also note that neither CPT data nor laboratory shear strength data from UU and/or UNC strength tests were available for ~~this stratum~~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 =~~3.0~~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=~~1.0~~1.3 and a preconsolidation pressure of ~~16~~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 =~~3.0~~3.9 ksf, OCR=1.3), an E=~~1800~~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=~~129~~124 pcf for Stratum L, and PI=50 for Stratum L, an E=~~3718~~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=~~3100~~ 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.

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

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 ~~not~~ made on samples from Stratum M. ~~Based on boring log visual classifications, Stratum M is considered to have index properties similar to Sub-stratum K Sand/Silt.~~ 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)~~, with an average fines content (materials passing the No. 200 sieve) of 45% (employing the value from Sub-stratum K Sand/Silt in the absence of laboratory fines content tests on Stratum M)~~. The USCS designations for Stratum M were mainly poorly graded sand with silt to silty sand (based on visual classifications), with the predominant USCS group symbol of SM. An average unit weight of 127 pcf was selected for Stratum M (~~again,~~ employing the value from Sub-stratum K Sand/Silt ~~in the absence~~ 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'=~~33~~ 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 ~~established~~ assigned average corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Sub-stratum K Sand/Silt soils (~~30~~ 100 blows per foot) ~~(as above, SPT N-value data for Stratum M were not collected)~~, an E=~~1080~~ 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=~~1391~~ 4175 ksf was estimated. Using an average of the E-values estimated from the average corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value and from the shear wave velocity, with the shear wave velocity-derived value weighted 2:1, an E=~~1300~~ 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 shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=415 ksf was estimated based on the SPT $(N_1)_{60}$-value-derived E, while a G=535 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=500 ksf was selected for Stratum M. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.~~

The 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 ~~327~~ 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 ~~4~~ 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 ~~both~~ borings (B-305DH/DHA and B-405DH~~), ranging in thickness from greater than 60 feet to greater than 77 feet, with an average thickness of greater than 69 feet~~. 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 Substratum N Clay 6
- Sub-stratum N Sand, which contained Sub-stratum N Sand 1, Sub-stratum N Sand 2, Sub-stratum N Sand 3, Sub-stratum N Sand 4, and Sub-stratum N Sand 5

2.5S.4.2.1.12.1 Sub-stratum N Clay

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

As noted above, uncorrected SPT N-values from each boring were corrected to an ~~effective overburden pressure of one tsf~~energy ratio of 60 percent (i.e.~~,~~ ~~N_1~~ 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 (leading to fully-corrected values of $(N_1)_{60}$)~~. A summary of corrected SPT ~~$(N_1)_{60}$~~ N_{60}-values, for all site areas and all soil strata is presented in Table 2.5S.4-~~5~~6. The average corrected SPT ~~$(N_1)_{60}$~~ N_{60}-value for Sub-stratum N Clay was ~~8~~ 56 blows/foot. An SPT ~~$(N_1)_{60}$~~ N_{60}-value of ~~7~~ 54 blows/foot was selected for engineering purposes, as shown in Table 2.5S.4-6. Based on corrected SPT ~~$(N_1)_{60}$~~N_{60}-values, Stratum N Clay is ~~firm (although this stratum is likely much stiffer as the average corrected SPT $(N_1)_{60}$-value results from a low correction factor, C_n, which was extrapolated beyond its normal stress range)~~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 Substratum N Clay, with results as noted:

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	~~12~~ 17	17	38	25
Liquid Limit (%)	~~11~~ 16	~~45~~ 33	~~90~~ 92	~~65~~ 67
Plasticity Index (%)	~~11~~ 16	~~25~~ 22	~~63~~ 65	~~44~~ 46
Fines Content (%)	~~5~~ 10	22	~~95~~ 98	~~75~~ 79
Unit Weight (pcf)	~~4~~ 9	113	~~127~~ 132	~~121~~ 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 ~~75~~79%. Note that the minimum 22% fines content reported occurred at Boring B-~~401~~ 405 from depths of 318 feet to 320 feet. This result represents an isolated thin sand lens within Sub-stratum N Clay. ~~All other fines contents reported were greater than 80%.~~ The USCS designations for Sub-stratum N Clay were mainly fat clay, lean clay, and clayey sand, with the predominant USCS group symbols of CH and CL. Based on laboratory testing, an average unit weight of ~~121~~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 ~~$(N_1)_{60}$~~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~~N_{60}-value for Sub-stratum N Clay (~~7~~54 blows/foot), an s_u=~~0.9~~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. ~~By excluding the lowest laboratory strength test result of s_u=0.2 ksf (likely made on a sample of poor or non-representative quality), an average s_u=2.3 ksf resulted.~~ The ratio of the shear strength of Sub-stratum 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 ~~were~~ was more representative, and an undrained shear strength of s_u=~~3.0~~4.5 ksf was selected for Sub-stratum N Clay ~~(similar to Sub-stratum K Clay)~~.

CIU and CIU-Bar triaxial strength tests were not performed on Sub-Stratum N Clay. Likewise, the ~~The~~ drained friction angle of Sub-~~Strata~~ 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=~~1.0~~1.3 and a preconsolidation pressure of ~~28.5~~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=~~3.0~~4.5 ksf and OCR=1.3), an E=~~1800~~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=~~121~~ 123 pcf for Sub-stratum N Clay, and ~~PI=45~~50% reduction of the seismic modulus for Sub-stratum N Clay, an E=~~5794~~ 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=~~4500~~ 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.

~~The shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.45 for clay, a G=621 ksf was estimated based on the s_u-derived E, while a G=1998 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-7. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=1500ksf was selected for Sub-stratum N Clay. Note that the selected values of G for all soil strata are shown in Table 2.5S.4-15.~~

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. ~~In the area west of the Power Block, borings did not reach Sub-stratum N Sand.~~ Note also that uncorrected SPT N-values versus elevation are presented on Figures 2.5S.4-10 and 2.5S.4-11, and 2.5S.4-12 and 2.5S.4-13, for the STP 3 area, and the STP 4, respectively. The site-wide average uncorrected SPT N-value was 97 blows/foot for Sub-stratum N Sand.

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 Sub-stratum N Sand was selected for engineering purposes as shown in Table 2.5S.4-6.

As noted above, ~~un~~corrected SPT ~~N~~N_{60}-values in sandy strata from each boring were corrected to an effective overburden pressure of one atmosphere (approximately one tsf) ~~(i.e., N1), by the appropriate hammer energy correction value shown in Table 2.5S.4-4 for the drilling rig employed, and by other corrections~~ ~~(~~leading to fully-corrected values of $(N_1)_{60}$~~)~~. A summary of corrected SPT $(N_1)_{60}$-values, for all site areas and all 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 ~~23~~ 67 blows/foot. An SPT $(N_1)_{60}$-value of ~~20~~ 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 ~~medium~~ very dense ~~(although this stratum is likely more dense as the average corrected SPT $(N_1)_{60}$ value results from a low correction factor, C_n, which was~~

~~extrapolated beyond its normal stress range).~~

CPTs did not reach Sub-stratum N Sand.

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

Test	Number of Tests	Minimum Value	Maximum Value	Average Value
Moisture Content (%)	~~10~~ 12	17	28	~~23~~ 22
Liquid Limit (%)	~~4~~ 7	Non-Plastic	Non-Plastic	Non-Plastic
Plasticity Index (%)	~~4~~ 7	Non-Plastic	Non-Plastic	Non-Plastic
Fines Content (%)	~~11~~ 12	5	49	~~22~~ 21
Unit Weight (pcf)	~~2~~ 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 ~~22~~ 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 $(N_1)_{60}$~~N-~~value for Sub-stratum N Sand (~~20~~ 56 blows/foot), a value of ϕ'=of ~~38~~50~~+~~ degrees (for fine to coarse sand) was estimated. ~~A value of ϕ'=36 degrees was considered appropriate.~~ Note that laboratory direct shear tests made on selected samples were not available for this sub-stratum.

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

Parameter	From SPT Correlation	From CPT Correlation	From Direct Shear Testing
ϕ' (degrees)	~~36~~ 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 ~~average~~ corrected SPT ~~$(N_1)_{60}$~~ N_{60} value for Sub-stratum N Sand soils (~~20~~ 141 blows per foot) an E=~~720~~ 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 Substratum N Sand, an E=~~2831~~ 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=~~2100~~ 11,645 ksf was selected for Sub-stratum N Sand. This compares to a value range of $E_s \geq 1700$ 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 shear modulus (G) was related to E by Equation 2.5S.4-8. Using μ=0.30 for sand, a G=277 ksf was estimated based on the SPT (N1)60-value derived E, while a G=1089 ksf was estimated using the shear wave velocity and other parameters, as per Equations 2.5S.4-5, 2.5S.4-6, and 2.5S.4-14. An average of these two values, with the shear wave velocity-derived value weighted 2:1, was considered, and a value of G=800 ksf was selected for Sub-stratum N Sand.~~ 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.

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 ~~initial~~ subsurface investigation plan met the intent of Reference 2.5S.4-19. ~~A supplemental subsurface investigation commenced onsite in mid-July 2007, to accommodate the addition of a Radwaste Building in the STP 4 area, and a revised routing of the Reactor Service Water Lines, all as shown on Figure 2.5S.4-2. Subsurface conditions substantially different from those described here are not anticipated as a result of this supplemental subsurface investigation. Following~~

~~completion of this confirmatory investigation, STP will update the FSAR in accordance with 10 CFR 50.71(e) (COM 2.5S-2). A third subsurface investigation was conducted during the Summer of 2008 to support the relocation of the Ultimate Heat Sink. The two new UHS unites will be located inside the Power Block just south of their respective Reactor Buildings. Upon completion of the laboratory test data, this Subsection 2.5S.4 will be updated (COM 3H-2).~~

2.5S.4.2.1.16 Laboratory Testing Program

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

~~Note that a brief review of five recently available (as of late July 2007) laboratory RCTS tests results is made in Subsection 2.5S.4.7.3.3. All other laboratories have completed their testing, with results included in Reference 2.5S.4-2. The supplemental subsurface investigation described in Subsection 2.5S.4.2.1.15 also includes limited laboratory testing of selected soils samples recovered (refer to the statement on COM 2.5S-2 under Subsection 2.5S.4.2.1.15).~~ The laboratory testing program reported on here is discussed further in Subsection 2.5S.4.2.3.

2.5S.4.2.2 Exploration

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

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

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

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

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

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

2.5S.4.2.2.2 ~~Current~~ Subsurface 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 85 feet below ground surface), each of which was advanced to approximately 600 feet below ground surface. At each Reactor Building, eight additional borings were made to approximately 200 feet depth below ground surface. These borings were terminated in either dense sands or stiff to very stiff clays that, from a review of STP 1 & 2 data and the completed 600 foot deep borings, become stronger with increasing depth.

The sampling intervals employed in the borings varied slightly from the guidance document recommendations, but were in accordance with the subsurface investigation technical specifications. Sample spacing in the uppermost 15 feet was shortened at each boring, with typically 10 SPT samples collected over that depth. For SPT sampling five-foot sample intervals were maintained to a depth of 100 feet, 10-foot sample intervals were maintained to a depth of 200 feet and, 20-foot sample intervals were maintained to the maximum depth of approximately 600 feet below ground surface. In most cases, additional undisturbed samples were obtained, especially between the 20-foot sample intervals at the two deep borings (B-305DH/DHA and B405DH). Continuous sampling was also performed, as described later. CPTs obtained continuous data to a maximum depth of approximately 100 feet below ground surface.

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

Subsection 4.3.2 of Reference 2.5S.4-19, "Sampling," states that color photographs of all cores should be taken soon after removal from the boring to document the condition of the soils at the time of drilling. Undisturbed soil samples are sealed in metal tubes, and cannot be photographed. SPT soil samples are disturbed and, as a result, do not resemble the condition of the material insitu. Sample photography is a practice typically limited to rock core, rather than soil samples, and therefore, was not ~~employed~~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.4.2.2.1 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 ~~120~~ 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 ~~32~~ 44 CPTs, including ~~five~~ 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 ~~11~~ 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), ~~and~~ chemical analyses (pH, sulfate content, and chloride content), and. RCTS testing. ~~was also commissioned, with testing currently underway, and with complete results reported at a later date (refer to the statement on COM 2.5S-1 at Subsection 2.5S.4.7).~~
- Performing laboratory testing on groundwater samples obtained from the observation wells, including pH, conductivity, dissolved oxygen, alkalinity, ammonia, nitrogen, bromide, chloride, dissolved solids, fluoride, nitrate as N, nitrite as N, sulfate, and sulfide, including cation exchange testing on soils in the well screen area. These results are discussed in Subsection 2.4S.12
- ~~The third field investigation conducted in June of 2008 consisted of 32 boring locations with 12 off-set samples. The maximum depth was approximately 300 feet.~~

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 the 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. ~~Eleven~~ 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 ~~2.5~~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 ~~electrical~~ 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 ~~11~~ 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.

~~Thirty-two~~Forty-four CPTs were performed, with termination depths ranged from approximately 36 feet to 100 feet below ground surface, including ~~five~~ six seismic CPTs (C-305S, C306S, 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. ~~commissioned, with testing currently underway with complete results reported at a later date (refer to the statement on COM 2.5S-1 at Subsection 2.5S.4.7).~~

Laboratory tests were performed in accordance with the following standards:

- Identification and Index Testing
  - Unified Soil Classification System (USCS) – ASTM D 2487 (Reference 2.5S.4-31) and/or Visual-Manual Procedure ~~and~~ ASTM D 2488 (Reference 2.5S.4-23)
  - Moisture Content – ASTM D 2216 (Reference 2.5S.4-32)
  - Atterberg Limits – ASTM D 4318 (Reference 2.5S.4-33)
  - Sieve and Hydrometer Analysis – ASTM D 422 (Reference 2.5S.4-34) and ASTM D 6913 (Reference 2.5S.4-35)
  - Specific Gravity – ASTM D 854 (Reference 2.5S.4-36)
  - Unit Weight – measured (included as a part of related ASTM standards)
- Strength Testing
  - Unconsolidated-Undrained Triaxial Compression – ASTM D 2850 (Reference 2.5S.4-37)
  - Unconfined Compression – ASTM D 2166 (Reference 2.5S.4-38)
  - Consolidated-Undrained Triaxial Compression – ASTM D 4767 (Reference 2.5S.4-39)
  - Direct Shear – ASTM D 3080 (Reference 2.5S.4-40)
- Compressibility Testing
  - Consolidation – ASTM D 2435 (Reference 2.5S.4-41)

- Compaction and Related Testing
  - Moisture-Density Relationship – ASTM D 1557 (Reference 2.5S.4-42)
  - California Bearing Ratio – ASTM D 1883 (Reference 2.5S.4-43)
- Chemical Testing – Soils
  - pH – ASTM D 4972 (Reference 2.5S.4-44)
  - Chloride Content – EPA 300.0 (Reference 2.5S.4-45)
  - Sulfate Content – EPA 300.0 (Reference 2.5S.4-45)
- Dynamic Soil Response Testing
  - RCTS Testing – Stokoe, et al. (Reference 2.5S.4-46)

Table 2.5S.4-1 Field Testing Summary

Field Test	Industry Standard	Number Of Tests
Borings (B)	References 2.5S.4-22 and 2.5S.4-24	~~120~~ 132
SPT Hammer Energy Measurements	References 2.5S.4-6 and 2.5S.4-25	~~46~~ 52
Cone Penetration Tests (C)	Reference 2.5S.4-26	~~32~~ 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 was replaced in its entirety with the following:

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
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
	Minimum	-198.2	15.0	-207.4	22.4	-207.4	15.0	--	--	-207.4	15.0

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

K Clay	Maximum	-198.2	15.0	-207.4	22.4	-198.2	22.4	n/a	n/a	-198.2	22.4
	Average	-198.2	15.0	-207.4	22.4	-202.8	18.7	n/a	n/a	-202.8	18.7
K Sand/Silt	Minimum	-228.7	30.5	-227.4	20.0	-228.7	20.0	n/a	n/a	-228.7	20.0
	Maximum	-228.7	30.5	-227.4	20.0	-227.4	30.5	n/a	n/a	-227.4	30.5
	Average	-228.7	30.5	-227.4	20.0	-228.1	25.3	n/a	n/a	-228.1	25.3
L	Minimum	-234.2	5.5	-231.9	4.5	-234.2	4.5	n/a	n/a	-234.2	4.5
	Maximum	-234.2	5.5	-231.9	4.5	-231.9	5.5	n/a	n/a	-231.9	5.5
	Average	-234.2	5.5	-231.9	4.5	-233.1	5.0	n/a	n/a	-233.1	5.0
M	Minimum	-248.7	14.5	-247.4	15.5	-248.7	14.5	n/a	n/a	-248.7	14.5
	Maximum	-248.7	14.5	-247.4	15.5	-247.4	15.5	n/a	n/a	-247.4	15.5
	Average	-248.7	14.5	-247.4	15.5	-248.1	15.0	n/a	n/a	-248.1	15.0
N Clay 1	Minimum	-310.2	61.5	-303.9	56.5	-310.2	56.5	n/a	n/a	-310.2	56.5
	Maximum	-310.2	61.5	-303.9	56.5	-303.9	61.5	n/a	n/a	-303.9	61.5
	Average	-310.2	61.5	-303.9	56.5	-307.1	59.0	n/a	n/a	-307.1	59.0
N Sand 1	Minimum	-326.2	16.0	-321.9	18.0	-326.2	16.0	n/a	n/a	-326.2	16.0
	Maximum	-326.2	16.0	-321.9	18.0	-321.9	18.0	n/a	n/a	-321.9	18.0
	Average	-326.2	16.0	-321.9	18.0	-324.1	17.0	n/a	n/a	-324.1	17.0
N Clay 2	Minimum	-331.2	5.0	-332.9	11.0	-332.9	5.0	n/a	n/a	-332.9	5.0
	Maximum	-331.2	5.0	-332.9	11.0	-331.2	11.0	n/a	n/a	-331.2	11.0
	Average	-331.2	5.0	-332.9	11.0	-332.1	8.0	n/a	n/a	-332.1	8.0
N Sand 2	Minimum	-370.2	39.0	-358.9	26.0	-370.2	26.0	n/a	n/a	-370.2	26.0
	Maximum	-370.2	39.0	-358.9	26.0	-358.9	39.0	n/a	n/a	-358.9	39.0
	Average	-370.2	39.0	-358.9	26.0	-364.6	32.5	n/a	n/a	-364.6	32.5
N Clay 3	Minimum	-377.2	7.0	-368.9	10.0	-377.2	7.0	n/a	n/a	-377.2	7.0
	Maximum	-377.2	7.0	-368.9	10.0	-368.9	10.0	n/a	n/a	-368.9	10.0
	Average	-377.2	7.0	-368.9	10.0	-373.1	8.5	n/a	n/a	-373.1	8.5
N Sand 3	Minimum	-394.2	17.0	-388.9	20.0	-394.2	17.0	n/a	n/a	-394.2	17.0
	Maximum	-394.2	17.0	-388.9	20.0	-388.9	20.0	n/a	n/a	-388.9	20.0
	Average	-394.2	17.0	-388.9	20.0	-391.6	18.5	n/a	n/a	-391.6	18.5
N Clay 4	Minimum	-419.2	25.0	-423.9	35.0	-423.9	25.0	n/a	n/a	-423.9	25.0
	Maximum	-419.2	25.0	-423.9	35.0	-419.2	35.0	n/a	n/a	-419.2	35.0
	Average	-419.2	25.0	-423.9	35.0	-421.6	30.0	n/a	n/a	-421.6	30.0
N Sand 4	Minimum	-435.2	16.0	n/a	n/a	-435.2	16.0	n/a	n/a	-435.2	16.0
	Maximum	-435.2	16.0	n/a	n/a	-435.2	16.0	n/a	n/a	-435.2	16.0
	Average	-435.2	16.0	n/a	n/a	-435.2	16.0	n/a	n/a	-435.2	16.0
N Clay 5	Minimum	-493.2	58.0	-473.9	50.0	-493.2	50.0	n/a	n/a	-493.2	50.0
	Maximum	-493.2	58.0	-473.9	50.0	-473.9	58.0	n/a	n/a	-473.9	58.0
	Average	-493.2	58.0	-473.9	50.0	-483.6	54.0	n/a	n/a	-483.6	54.0
N Sand 5	Minimum	n/a	n/a	-508.9	35.0	-508.9	35.0	n/a	n/a	-508.9	35.0
	Maximum	n/a	n/a	-508.9	35.0	-508.9	35.0	n/a	n/a	-508.9	35.0
	Average	n/a	n/a	-508.9	35.0	-508.9	35.0	n/a	n/a	-508.9	35.0
N Clay 6	Minimum	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	Maximum	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	Average	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a

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 has been reformatted and replaced in its entirety to include new data and methodologies.to reflect the new data distribution for the Ultimate Heat Sinks and the new methodology.

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
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

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

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					
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	~~1.36~~ 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 [2]	5	90-107	99	1.65
12/8/2006 – July 2007	Lewis Environmental Mobile B57 [2]	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 [2]	3	94-98	96	1.60

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

Dates Applicable [3]	Drilling Rig	Number Of Measurements	ETR Range (%)	ETR Average [1] (%)	Hammer Energy Correction (ETR%/60%)
October 2006 – January 2007	MACTEC D50 ATV Rig	4	69-74	72	~~1.21~~ 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	~~1.41~~ 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 has been replaced in its entirety to include the new distribution of data for UHS and new methodology.

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
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~~/A (Fill)~~	10	13	~~17~~ N/A	11	~~15~~ N/A
B	9	14	~~14~~ 15	11	~~10~~12
C	~~25~~25	41	~~35~~ 39	38	35
D	15	25	~~17~~ N/A	23	~~15~~N/A
E	~~35~~ 37	60	~~34~~ 35	53	~~30~~31
F	22	36	~~19~~ N/A	34	~~15~~N/A
H	44	70	~~34~~35	58	~~30~~28
J Clay	~~31~~32	51	~~18~~ N/A	48	~~15~~N/A
J Sand	~~65~~63	101	~~36~~41	94	~~35~~38
K Clay	15	26	~~7~~ N/A	26	~~6~~N/A
K Sand/Silt	~~75~~80	136	~~31~~ 54	68	~~30~~27
L	23	38	~~9~~ N/A	36	~~8~~N/A
M	Not Tested	-	Not Tested	100	~~30 [4]~~40
N Clay	33	56	~~8~~ N/A	54	~~7~~N/A
N Sand	~~97~~ 98	167	~~23~~67	141	~~20~~ 56

[1] All SPT N_{60}- and $(N_1)_{60}$-values in blows/foot. N_{60} values include correction for energy transfer ratio, rod length and no sample liner (C_s=1.2). $(N_1)_{60}$ values include corrections for energy transfer ratio, rod length, no sample liner (C_s=1.2) and vertical effective stress ($0.4 \leq C_N \leq 1.7$).

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

[3] Selected values for engineering use are based on lowest averages of the predominant grain size by area in Table 2.5S.4-3A for N_{60} and Table 2.5S.4-5 for $(N_1)_{60}$.

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

Table 2.5S.4-7 Laboratory Testing Summary

Laboratory Test	Industry Standard	Number Of Tests
Moisture content	Reference 2.5S.4-32	~~388~~ 534
Atterberg Limits	Reference 2.5S.4-33	~~226~~ 286
~~Sieve and Hydrometer~~ Grain Size Analysis	References 2.5S.4-34 and 2.5S.4-35	~~200~~ 257
Specific Gravity	Reference 2.5S.4-36	~~86~~ 107
Unit Weight	Included with Related ASTM Standards	~~109~~ 141
Unconsolidated Undrained (UU) Triaxial Strength	Reference 2.5S.4-37	~~68~~ 76
Unconfined Compressive (UNC) Strength	Reference 2.5S.4-38	~~20~~ 25
Consolidated Undrained (CIU-bar) Triaxial Strength	Reference 2.5S.4-39	~~15~~ 17
Direct Shear (DS) Strength	Reference 2.5S.4-40	~~11~~ 10
Consolidation	Reference 2.5S.4-41	~~30~~ 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	~~60~~ 67
Chloride Content	Reference 2.5S.4-~~44~~ 45	~~40~~ 47
Sulfate Content	Reference 2.5S.4-45	~~40~~ 47
Resonant Column Torsional Shear (RCTS) ~~[1]~~	Reference 2.5S.4-46	~~Pending [1]~~ 16

~~[1]RCTS testing is currently in progress~~

Table 2.5S.4-8 has been reformatted and replaced in its entirety to include new data and methodologies.

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		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	CL, CH	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		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	CL, ML	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	SM, SC	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		17.1	119.6	2.65	0.653	NV	NP	0.0	4.1	5.3	8.1	36.1	7.2
Maximum	SP-SM	27.0	124.2	2.73	0.715	NV	NP	5.9	94.7	95.9	9.1	108.0	35.5
Average	ML, SM	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
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 (mg/kg)	Sulfide (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
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,	32.0	128.0	2.67	0.749	24.0	3.0	0.0	90.2	96.7			
Average	SP-SM	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 (mg/kg)	Sulfide (mg/kg)
COMBINED SUB-STRATA J (CLAY 1), J (CLAY 2)													
Minimum		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	CL, CH,	38.0	133.7	2.80	0.991	85.0	62.0	0.9	45.4	99.8			
Average	ML	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

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 (mg/kg)	Sulfide (mg/kg)
COMBINED SUB-STRATA J (SAND 1), J (SAND 2), J (SAND/SILT interbeds)													
Minimum	SM, ML,	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	SP-SM,	32.0	128.0	2.72	0.749	24.0	3.0	1.1	90.2	96.7			
Average	CL	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

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 (mg/kg)	Sulfide (mg/kg)
Stratum K (CLAY)													
Minimum		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	CH, CL	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

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 (mg/kg)	Sulfide (mg/kg)
Stratum K (SAND/SILT)													
Minimum		20.1	126.8	2.67	0.596	NV	NP	0.0	34.6	27.0	N/A	N/A	N/A
Maximum	SM, ML	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

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 (mg/kg)	Sulfide (mg/kg)
Stratum L													
Minimum		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	CH	28.5				73.0	51.5						
# of Tests		2	0	0	0.0	2	2	0	0	0	0	0	0

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 (mg/kg)	Sulfide (mg/kg)
Stratum M													
Minimum		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	SM	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 (mg/kg)	Sulfide (mg/kg)
Stratum N (CLAY1)													
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 (SAND1)													
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 (CLAY2)													
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
Stratum N (SAND2)													
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 (CLAY3)													
Minimum	CL	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		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 (SAND3)													
Minimum	SM	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				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 (CLAY4)													
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 (SAND4)													
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 (CLAY5)													
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
Stratum N (SAND5)													
Minimum		20.8	N/A	2.68	N/A	NV	NP	0.0	62.4	20.7	N/A	N/A	N/A
Maximum	SM	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 (CLAY6)													
Minimum		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	CH, CL	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 ~~Corrected (N1)60~~-Value (blows/foot)**	**Calculated ~~su~~ S_u (ksf)**	
~~A/A (Fill)~~ A	~~15~~ 11	~~1.9~~ 1.4	
D	~~15~~ 23	~~1.9~~ 2.9	
F	~~15~~ 34	~~1.9~~ 4.3	
J Clay	~~15~~ 48	~~1.9~~ 6.0	
K Clay	~~6~~ 26	~~0.8~~ 3.3	
L	~~8~~ 36	~~1.0~~ 4.5	
N Clay	~~7~~ 54	~~0.9~~ 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/A (Fill)~~ A	0.5	2.3	~~1.3~~ 1.4
D	0.3	2.5	~~1.7~~ 2.2
F	0.7	~~3.7~~ 4.3	~~2.7~~ 2.9
J Clay	0.1	6.6	~~3.2~~ 4.3
K Clay	2.8	4.0	~~3.4~~ 4.0
L	Not Tested	Not Tested	Not Tested
N Clay	0.2	4.5	~~1.7~~ 4.5
From Correlations with CPT Data			
Stratum	**Minimum S_u ~~su~~(ksf)**	**Maximum S_u ~~su~~ (ksf)**	**Average S_u ~~su~~(ksf)**
~~A/A (Fill)~~ A	0.2	~~3.7~~ >10	~~1.5~~ 1.7
D	0.8	~~7.6~~ >10	~~3.1~~ 3.3
F	1.9	6.3	~~3.6~~ 3.5
J Clay	2.3	~~5.3~~ 4.0	~~3.8~~ 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 ~~su~~(ksf)**	
~~A/A (Fill)~~ A		~~1.6~~ 1.5	
D		3.0	
F		~~3.2~~ 3.4	
J Clay		~~3.5~~ 3.48	
K Clay		~~3.0~~ 3.9	
L		~~3.0~~ 3.9	
N Clay		~~3.0~~ 4.5	

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

											UNC/UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
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	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	21.6	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	45	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	6.0	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-915	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]: 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)
D	B-305-DH	UD-4	53.0	-23.2	125.2	24.6	45	28	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	38	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	2.812	0.90	-	-	-	-	-	-
D	B-940	UD-5	56.0	-26.3	124.1	27.0	60	32	CH	3.69	UU	2.024	0.55	-	-	-	-	-	-
D	B-949	UD-3	61.0	-32.3	129.8	23.3	54	31	CH	3.94	UU	2.830	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	-	-
			Stratum D	Maximum	129.8	32.8	74.0	53.0	-	-	-	2.830	0.90	2.17	8.7	2.15	31.7	-	-
			Stratum D	Average	122.1	25.2	58.3	33.8	-	-	-	1.876	0.67	1.79	4.4	1.23	15.9	-	-
											Alternate Minimum	1.728	0.55						
											Alternate Maximum	2.830	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	5.08	-	-	-	-	-	-	-	0.0	33.0
			Stratum E	Minimum	111.4	14.9	20	2	-	-	-	-	-	-	-	-	-	0.0	33.0
			Stratum E	Maximum	127.0	25.8	20	2	-	-	-	-	-	-	-	-	-	0.0	33.0
			Stratum E	Average	121.9	21.8	NV	NP	-	-	-	-	-	-	-	-	-	0.0	33.0

Cells not included in Alternate Min, Max, and Average values calculated from this table.

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-303	UD-2	88.0	-61.4	127.9	26.8	57	39	CH	6.02	UU	3.469	0.58	-	-	-	-	-	-
F	B-306	UD-4	88.0	-60.2	128.9	22.7	57	38	CH	6.06	-	-	-	3.02	3.1	2.98	6.8	-	-
F	B-401	UD-2	88.0	-87.2	125.7	23.4	57	38	CH	6.26	-	-	-	2.36	0.8	2.06	8.1	-	-
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	-	-	-	-	-	-
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	58	36	CH	5.93	UU	3.093	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	28	CL	6.03	-	-	-	0.93	6.0	0.98	12.0	-	-
F	B-909	UD-6	93.0	-63.3	129.3	17.9	56	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	56	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	2.350	0.41	-	-	-	-	-	-
F	B-949	UD-4	71.0	-42.3	124.6	24.9	56	32	CH	4.56	UU	1.346	0.30	-	-	-	-	-	-
F	B-949	UD-7	91.0	-62.3	121.3	28.1	60	38	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	0.98	8.1	-	-
				Maximum	129.3	29.1	62.0	44.0	-	-	-	4.344	0.89	3.02	6.0	2.98	12.0	-	-
				Average	124.6	23.7	56.4	34.9	-	-	-	2.658	0.44	2.10	3.3	1.99	7.9	-	-
											Alternate Minimum	1.346	0.28						
											Alternate Maximum	4.344	0.89						
											Alternate Average	2.882	0.46						

Cells not included in Alternate Min, Max, and Average values calculated from this table.

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)
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
			Stratum H	Maximum	134.9	24.4	-	-	-	-	-	-	-	-	-	-	-	0.0	29.0
			Stratum H	Average	128.3	18.4	NV	NP	-	-	-	-	-	-	-	-	-	0.0	29.0
JC1	B-303	UD-4	133.0	-106.4	121.3	29.5	66	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	-	-	-	-	-	-
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	136.0	-105.8	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	23.8	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	62	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	-	-	-	-	-	-

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-307	UD-3	188.0	-153.8	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	18.4	-	-	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	-	-	-	-	-	-
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	11.58	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	-	-
			Stratum J Clay	Maximum	131.1	34.0	73.0	50.0	-	-	-	6.579	0.77	7.69	25.0	6.92	28.8	-	-
			Stratum J Clay	Average	125.2	22.3	52.6	32.5	-	-	-	3.014	0.31	2.72	6.9	2.33	10.8	-	-
											Alternate Minimum	2.003	0.25						
											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
			Stratum J Sand	Maximum	128.0	32.0	-	-	-	-	-	-	-	-	-	-	-	0.0	32.0
			Stratum J Sand	Average	125.3	26.7	NV	NP	-	-	-	-	-	-	-	-	-	0.0	32.0

Cells not included in Alternate Min, Max, and Average values calculated from this table.

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

											UNC/ UU Tests [1]			CIU-Bar Tests [1]				DS Tests [1]	
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	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)
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						
											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	16.00	-	-	-	-	-	-	-	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	-	-	-	-	-	-
NC1	B-305-DH	UD-15A	316.0	-286.2	119.7	30.4	-	-	CH	20.51	UNC	1.366	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						
											Alternate Maximum	4.487	0.152						
											Alternate Average	4.487	0.152						
NS1	B-405DH	UD-16	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	-	-	-	-	-	-	-	-	-
			Straum N Sand	Minimum	125.8	20.9	-	-	-	-	-	-	-	-	-	-	-	-	-
				Maximum	130.2	22.0	-	-	-	-	-	-	-	-	-	-	-	-	-
				Average	128.0	21.5	NV	NP	-	-	-	-	-	-	-	-	-	-	-

Cells not included in Alternate Min, Max, and Average values calculated from this table.

Cells not included in Alternate Min, Max, and Average values calculated from this table.

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

Stratum	Number Of Tests	Range	C_r	C_c	e_0	P_c'(ksf)	OCR	c_v (ft^2/day)
A	5	Minimum	0.000	~~0.316~~ 0.050	~~0.316~~ 0.660	3.2	~~3.2~~ 3.7	1.73
		Maximum	0.023	~~0.050~~ 0.316	0.750	10.0	~~17.2~~ 25.0	9.85
		Average	0.017	0.235	~~0.235~~ 0.702	6.7	~~7.8~~ 10.5	5.32
D	~~5~~ 8	Minimum	0.007	0.086	~~0.710~~ 0.710	6.1	~~1.6~~ 1.7	~~0.05~~ 0.04
		Maximum	0.033	0.468	~~0.920~~ 0.980	~~16.9~~ 18.3	~~4.8~~ 5.8	0.52
		Average	~~0.023~~ 0.026	~~0.255~~ 0.285	~~0.796~~ 0.830	~~12.5~~ 13.4	~~3.5~~ 3.9	~~0.21~~ 0.20
F	~~3~~ 6	Minimum	~~0.037~~ 0.013	~~0.229~~ 0.199	0.630	13.4	~~2.2~~ 2.3	0.15
		Maximum	0.040	~~0.249~~ 0.262	0.810	~~18.0~~ 23.7	~~3.3~~ 3.7	3.41
		Average	~~0.039~~ 0.028	~~0.240~~ 0.238	~~0.713~~ 0.703	~~16.5~~ 18.6	~~2.9~~ 3.1	~~1.29~~ 0.91
J Clay	~~10~~ 11	Minimum	0.013	~~0.149~~ 0.130	~~0.520~~ 0.480	14.1	1.2	~~0.04~~ 0.01
		Maximum	0.086	0.472	0.790	27.9	2.7	14.20 14.17
		Average	~~0.040~~ 0.038	~~0.228~~ 0.224	~~0.628~~ 0.615	18.6	1.9	~~2.74~~ 2.34
K Clay	2	Minimum	0.010	0.103	0.510	20.2	~~1.3~~ 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

This table has been updated to reflect the new methodology and data.

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_o	Compression Index, C_c	Compression Ratio, CR	Recompression Index, C_r	Recompression Ratio, RR	Preconsolidation Pressure, Pc (kips per square foot)	Overconsolidation Ratio, OCR	Coefficient of Consolidation, cv (feet^2 / day)
STRATUM A																	
B-305-DH	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
MINIMUM, 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-305-DH	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

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, Pc (kips per square foot)	Overconsolidation Ratio, OCR	Coefficient of Consolidation, cv (ft^2 / day)
MINIMUM, STRATUM D			-	2.9	115.9	25.7	44	25	-	0.710	0.086	0.048	0.007	0.004	6.08	1.7	4.32E-02
MAXIMUM, STRATUM D			-	4.8	126.2	34.9	74	53	-	0.980	0.468	0.244	0.033	0.018	18.30	5.8	5.18E-01
AVERAGE, STRATUM D			-	3.5	120.9	29.3	59	36	-	0.830	0.285	0.155	0.026	0.014	13.36	3.9	1.99E-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
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
MINIMUM, STRATUM F			-	5.5	124.9	23	47	23	-	0.630	0.199	0.116	0.013	0.008	13.40	2.3	1.50E-01
MAXIMUM, STRATUM F			-	6.6	127.9	29	66	43	-	0.810	0.262	0.156	0.040	0.024	23.70	3.7	3.41
AVERAGE, STRATUM F			-	6.0	126.3	25	57	35	-	0.703	0.238	0.140	0.028	0.017	18.55	3.1	0.91
STRATUM J CLAY 1																	
B-305-DH	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-305-DH	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-319-DH	UD-1	128.0	-99.6	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

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_o	Compression Index, C_c	Compression Ratio, CR	Recompression Index, C_r	Recompression Ratio, RR	Preconsolidation Pressure, Pc (kips per square foot)	Overconsolidation Ratio, OCR	Coefficient of Consolidation, cv (ft^2 / day)
MINIMUM, STRATUM J CLAY 1				8.0	125.0	16.6	32	18	-	0.480	0.130	0.081	0.013	0.008	14.10	1.7	4.68E-02
MAXIMUM, STRATUM J CLAY 1				9.3	133.7	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	129.0	19.4	50	32	-	0.570	0.206	0.131	0.034	0.022	17.31	2.0	4.08
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	1.00E-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	1.00
MINIMUM, 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	1.00E-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	1.00
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	0.25
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
MINIMUM, 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

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, Pc (kips per square foot)	Overconsolidation Ratio, OCR	Coefficient of Consolidation, cv (ft^2 / day)
STRATUM N CLAY 1																	
B-305-DH	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	119.70	29.69	-	-	-	0.790	0.292	0.156	0.033	0.018	17.89	0.6	4.42E-02
MAXIMUM, STRATUM N CLAY				20.4	119.70	29.69	-	-	-	0.870	0.379	0.212	0.066	0.037	18.90	0.9	5.13E-02
AVERAGE, STRATUM N CLAY				20.4	119.70	29.69	-	-	-	0.830	0.336	0.184	0.050	0.027	18.40	0.8	4.78E-02

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

Stratum	Average P_c'(ksf)	Average OCR
From Laboratory Consolidation Tests		
A~~/A(Fill)~~	6.7	~~7.8~~10.0+
D	~~12.5~~13.4	~~3.5~~3.9
F	~~16.5~~18.6	~~2.9~~3.1
J Clay	18.6	1.9
K Clay	24.1	~~1.3~~1.7
L	Not Tested	Not Tested
N Clay	18.4	0.8
From Correlations with CPT Data		
A~~/A(Fill)~~	N/A	10.0~~+~~
D	N/A	~~3.0~~4.2
F	N/A	~~2.2~~2.4
J Clay	N/A	~~1.8~~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~~/A(Fill)~~	6.3	7.0
D	12.3	3.3
F	15.5	2.6
J Clay	18.5	1.7
K Clay	18.3	1.3
L	~~16.0~~20.5	~~1.0~~1.3
N Clay	~~28.5~~37	~~1.0~~1.3

This table has been updated with the new data and methodology.

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

Strata A~~/A (Fill)~~ 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~~/A (Fill)~~ through E					
Relationship Employed	**High Strain Shear Moduli By Stratum (ksf)**				
	A~~/A (Fill)~~	**B**	**C**	**D**	**E**
~~$G_{.375\%}$ = f (Vs)~~	~~N/A~~	~~212~~	~~233~~	~~N/A~~	~~442~~
~~$G_{.375\%}$ = f (PI)~~	~~384~~	~~N/A~~	~~N/A~~	~~968~~	~~N/A~~
~~$G_{.375\%}$ = f (su)~~	~~331~~	~~N/A~~	~~N/A~~	~~621~~	~~N/A~~
~~$G_{.375\%}$ = f (N)~~	~~N/A~~	~~139~~	~~485~~	~~N/A~~	~~415~~
$G = \frac{E_d}{2(1+\mu_d)}$ G Value Selected for Engineering Use	~~360~~ 370	~~185~~ 465	~~320~~ 695	~~850~~ 800	~~425~~ 1,215
Strata F through K Clay					
Relationship Employed	**High Strain Shear Moduli By Stratum (ksf)**				
	F	**H**	**J Clay**	**J Sand**	**K Clay**
~~$G_{.375\%}$ = f (Vs)~~	~~N/1~~	~~459~~	~~N/A~~	~~631~~	~~N/A~~
~~$G_{.375\%}$ = f (PI)~~	~~1,044~~	~~N/A~~	~~1,433~~	~~N/A~~	~~1,306~~
~~$G_{.375\%}$ = f (su)~~	~~662~~	~~N/A~~	~~724~~	~~N/A~~	~~621~~
~~$G_{.375\%}$ = f (N)~~	~~N/A~~	~~415~~	~~N/A~~	~~485~~	~~N/A~~
$G = \frac{E_d}{2(1+\mu_d)}$ ~~G Value Selected for Engineering Use~~	~~900~~ 850	~~450~~ 1,250	~~1,200~~ 1,380	~~600~~ 1,830	~~1,050~~ 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**
~~$G_{.375\%}$ = f (Vs)~~	~~740~~	~~N/A~~	~~535~~	~~N/A~~	~~1,089~~
~~$G_{.375\%}$ = f (PI)~~	~~N/A~~	~~1,282~~	~~N/A~~	~~1,998~~	~~N/A~~
~~$G_{.375\%}$ = f (su)~~	~~N/A~~	~~621~~	~~N/A~~	~~621~~	~~N/A~~
~~$G_{.375\%}$ = f (N)~~	~~415~~	~~N/A~~	~~415~~	~~N/A~~	~~277~~
$G = \frac{E_d}{2(1+\mu_d)}$ G Value Selected for Engineering Use	~~650~~ 1,890	~~1,050~~ 1,300	~~500~~ 1,675	~~1,500~~ 2,620	~~800~~ 4,470

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

Parameter [1]	Stratum				
	A~~/A (Fill)~~	B	C	D	E
Average Thickness, feet	~~18~~19	7	~~20~~19	~~22~~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), %	~~23~~24	24	~~24~~23	~~25~~26	21
Moist Unit Weight (γ_{moist}), pcf	124	121	122	~~121~~122	~~122~~123
Fines Content, %	~~94~~96	~~71~~67	~~25~~23	~~72~~79	~~18~~20
Liquid Limit (LL), %	~~57~~56	~~38~~NV	~~NA~~NV	~~58~~57	~~NA~~NV
Plasticity Index (PI), %	~~37~~40	~~19~~NP	~~NA~~NP	~~38~~40	~~NA~~NP
Uncorrected SPT N-value, bpf	~~10~~9	~~9~~8	~~24~~23	15	~~35~~33
Corrected SPT N_{60}-value, bpf	11	11	38	23	53
Corrected SPT $(N_1)_{60}$-value, bpf	~~15~~N/A	~~10~~12	~~35~~35	~~15~~N/A	~~30~~31
Shear Wave Velocity (V_s), feet/sec	575	725	785	925	1,080
Undrained Shear Strength (S_u), ksf	~~1.6~~1.5	N/A	N/A	3.0	N/A
Drained Friction Angle (ϕ'), degrees [8]	N/A	30	35	~~20~~16	35
Drained Cohesion (c'), ksf	N/A	N/A	N/A	~~N/A~~1.2	N/A
Elastic Modulus (High Strain) (E_s), ksf	~~1050~~1,135	~~460~~1,200	~~850~~1,810	~~2500~~2,430	~~1100~~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	~~360~~370	~~185~~465	~~320~~695	~~850~~800	~~425~~1,215
Shear Modulus (Low Strain) (G_{max}), ksf	1,270	1,970	~~2740~~2,335	~~3210~~3,240	~~4420~~,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	~~0.255~~.285	N/A
- Recompression Index (C_r)	0.017	N/A	N/A	~~0.023~~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

[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.4-16 Summary of Geotechnical Engineering Parameters (Continued)

Parameter [1]	Stratum				
	F	H	J Clay	J Sand	K Clay
Average Thickness, feet	16	~~17.5~~ 17	~~61~~70 [2]	37.5 [3]	~~18.5~~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	~~23~~22	~~20~~23
Moist Unit Weight (γ_{moist}), pcf	125	~~128~~125	125	125	~~129~~124
Fines Content, %	~~89~~94	~~16~~18	~~89~~90	~~43~~50	~~75~~87
Liquid Limit (LL), %	~~58~~57	~~N/A~~NV	54	~~N/A~~NV	~~39~~50
Plasticity Index (PI), %	~~38~~40	~~N/A~~NP	35	~~N/A~~NP	~~25~~35
Uncorrected SPT N-value, bpf	22	~~44~~ 42	~~31~~32	~~65~~55	15
Corrected SPT N_{60}-value, bpf	34	58	48	94	26
Corrected SPT $(N_1)_{60}$-value, bpf	~~15~~NA	~~30~~28	~~15~~NA	~~35~~38	~~6~~NA
Shear Wave Velocity (V_s), feet/sec	945	1,075	~~1145~~1,085	1,275	~~1145~~1,170
Undrained Shear Strength (S_u), ksf	~~3.2~~3.4	N/A	~~3.5~~3.8	N/A	~~3.0~~3.9
Drained Friction Angle (ϕ'), degrees [8]	~~20~~8	35	~~20~~11	33	~~N/A~~11
Drained Cohesion (c'), ksf	~~N/A~~2.0	N/A	~~0~~2.3	N/A	~~N/A~~2.3
Elastic Modulus (High Strain) (E_s), ksf	~~2600~~2,570	~~1150~~3,240	~~3500~~4,140	~~1500~~4,755	~~3100~~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	~~900~~850	~~450~~1,250	~~1200~~1,380	~~600~~1,830	~~1050~~1,450
Shear Modulus (Low Strain) (G_{max}), ksf	3,470	~~4590~~4,490	~~5090~~4,570	6,310	~~5480~~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
- 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)	~~0.240~~0.238	N/A	~~0.228~~0.224	N/A	0.176
- Recompression Index (C_r)	~~0.039~~0.028	N/A	~~0.040~~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

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

[2] Sub-stratum J Clay thickness = combined thickness of J Clay 1 (29 feet) + J Clay 2 (~~32~~ 41 feet)

[3] Sub-stratum J Sand thickness = combined thickness of J Interbed 1 (~~29~~ 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.4-16 Summary of Geotechnical Engineering Parameters (Continued)

Parameter [1]	Stratum				
	K Sand/Silt	L	M	N Clay	N Sand
Average Thickness, feet	~~25.5~~ 25.3	5	15	>228~~[2]~~[4]	~~93.5~~119 ~~[3]~~ [5]
USCS Group Symbol	SM, ML	CH	SM	CH, CL, SC	SM, SP-SM, SC
Natural Moisture Content (MC), %	21	29	~~21[4]~~19	25	~~23~~22
Moist Unit Weight (γ_{moist}), pcf	127	~~129[5]~~124 [7]	127 ~~[4]~~ [6]	~~121~~123	128
Fines Content, %	45	~~75[5]~~87 [7]	45 ~~[4]~~ [6]	~~75~~79	~~22~~21
Liquid Limit (LL), %	~~N/A~~NV	73	~~N/A~~NV	~~65~~67	~~N/A~~NV
Plasticity Index (PI), %	~~N/A~~NP	~~52~~50	~~N/A~~NP	~~44~~ 45	~~N/A~~NP
Uncorrected SPT N-value, bpf	~~75~~60	~~23~~21	~~75[4]~~60	~~33~~32	~~97~~83
Corrected SPT N_{60}-value, bpf	68	36	100	54	141
Corrected SPT $(N_1)_{60}$-value, bpf	~~30~~27	~~8~~N/A	~~30[4]~~40	~~7~~N/A	~~20~~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	~~3.0~~3.9	N/A	~~3.0~~4.5	N/A
Drained Friction Angle (ϕ'), degrees [8]	~~33~~31	N/A	~~33[4]~~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	~~1650~~4,915	~~3100~~3,865	~~1300~~4,350	~~4500~~7,855	~~2100~~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	~~650~~1,890	~~1050~~1,300	~~500~~1,675	~~1500~~2,620	~~800~~4,470
Shear Modulus (Low Strain) (G_{max}), ksf	7,400	~~3810~~3,660	5,350	~~6250~~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 ~~[5]~~ [7]	N/A	0.336	N/A
- Recompression Index (C_r)	N/A	0.017 ~~[5]~~ [7]	N/A	0.050	N/A
- Preconsolidation Pressure (P_c'), ksf	N/A	~~16~~ 20.5	N/A	~~28.5~~ 37	N/A
- Overconsolidation Ratio (OCR)	N/A	~~1.0~~ 1.3	N/A	~~1.0~~ 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 (~~32~~ 41 feet)

[3] Sub-stratum J Sand thickness = combined thickness of J Interbed 1 (~~29~~ 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	~~9.384~~ 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)					
Parameter	**Range For Steel Corrosiveness**				
	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		<200 [2]		300-1,000 [1]	>1,000 [1]
(ppm)					

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**	**Maximum Water/ 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 was updated to reflect the redistribution of data inside and outside the Power Block.

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	-171.9
B-302DH	63,000.73	43,364.78	30.0	220	-190.0
B-303	63,001.22	43,456.09	26.6	200	-173.4
B-304	63,095.40	43,268.83	28.2	200	-171.8
B-305DH	63,099.59	43,364.19	29.8	495	-465.2
B-305DHA	63,100.87	43,343.98	29.8	618	-588.2
B-306	63,098.22	43,472.95	27.8	200	-172.2
B-307	63,196.58	43,269.07	28.2	200	-171.8
B-308DH	63,196.49	43,363.84	29.8	215	-185.2
B-309	63,197.07	43,455.89	26.6	200	-173.4
B-310	63,283.70	43,265.50	28.2	200	-171.8
B-311	63,286.55	43,363.47	29.9	100	-70.1
B-312	63,286.42	43,473.97	28.3	100	-71.7
B-313	63,149.10	43,486.09	28.2	100	-71.8
B-314	63,148.73	43,617.01	29.2	200	-170.8
B-315	63,366.12	43,511.58	27.7	150	-122.3
B-316	63,304.98	43,617.51	28.9	200	-171.1
B-317	63,364.01	43,235.44	28.5	150	-121.5
B-318	63,363.37	43,297.42	28.5	100	-71.5
B-319DH	63,364.17	43,407.90	28.4	215	-186.6
B-320	62,903.74	43,116.74	30.5	50	-19.5
B-321	63,483.05	43,231.24	29.2	150	-120.8
B-322C	63,483.40	43,406.69	30.1	100	-69.9
B-323	63,484.30	43,515.99	29.8	100	-70.2
B-324	63,570.87	43,233.90	29.5	100	-70.5
B-325	63,569.94	43,299.20	30.2	100	-69.8
B-326	63,572.01	43,519.56	30.4	150	-119.6
B-327	63,658.77	43,233.17	29.8	150	-120.2
B-328DH	63,660.26	43,298.12	29.9	218	-188.1
B-329	63,658.33	43,410.29	29.6	100	-70.4
B-330	63,660.32	43,518.07	29.5	150	-120.5
B-331	63,635.24	43,541.59	29.8	100	-70.2
B-332	63,738.50	43,601.33	30.3	150	-119.7
B-333	63,744.16	43,360.57	30.5	100	-69.5
B-334	63,751.04	43,254.47	30.5	100	-69.5
B-335	63,735.38	43,042.50	31.2	75	-43.8
B-336	63,680.97	42,936.21	31.1	75	-43.9

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-337	63,680.83	43,151.07	30.3	75	-44.7
B-338	63,791.50	42,935.72	32.1	75	-42.9
B-339	63,790.00	43,148.53	30.8	75	-44.2
B-340	63,281.77	43,151.48	30.5	100	-69.5
B-341	63,215.13	43,096.25	30.6	100	-69.4
B-342	63,215.34	43,175.33	30.7	100	-69.3
B-343	63,125.99	43,095.29	30.5	200	-169.5
B-344	63,056.54	43,096.13	30.6	100	-69.4
B-345	63,040.70	43,173.35	30.7	100	-69.3
B-346	62,809.88	43,006.37	30.4	75	-44.6
B-347	62,746.63	42,985.26	31.2	75	-43.8
B-348	62,683.87	43,004.72	30.0	125	-95.0
B-349	62,901.92	43,593.47	29.2	125	-95.8
B-350	63,539.30	42,960.25	30.8	100	-69.2
B-917 [3]	63,694.58	42,832.71	31.1	50	-18.9
B-948	63,227.49	42,967.91	31.3	100	-68.7

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]					
B-401	62,999.23	42,370.55	31.1	200	-168.9
B-402DH	62,998.09	42,462.29	30.9	215	-184.1
B-403	62,998.59	42,555.20	31.5	200	-168.5
B-404	63,097.53	42,369.54	31.0	200	-169.0
B-405DH	63,098.12	42,462.95	31.1	618	-586.9
B-406	63,098.20	42,556.69	31.2	200	-168.8
B-407	63,195.82	42,369.78	31.3	200	-168.7
B-408DH	63,194.11	42,463.86	31.2	200	-168.8
B-409	63,195.47	42,557.98	31.2	200	-168.8
B-410	63,286.47	42,369.53	31.7	100	-68.3
B-411	63,285.65	42,461.25	31.3	100	-68.7
B-412	63,287.51	42,553.81	31.4	100	-68.6
B-413	63,148.27	42,585.19	31.2	100	-68.8
B-414	63,147.67	42,746.89	32.2	150	-117.8
B-415	63,355.53	42,599.76	30.0	150	-120.0
B-416	63,301.73	42,746.36	31.8	150	-118.2
B-417	63,361.95	42,331.19	29.6	150	-120.4
B-418	63,361.76	42,433.17	29.8	100	-70.2
B-419DH	63,362.12	42,506.69	29.7	215	-185.3
B-420	62,900.80	42,008.75	31.9	125	-93.1
B-421	63,483.06	42,328.30	30.3	100	-69.7
B-422C	63,483.67	42,510.68	31.2	100	-68.8
B-423	63,485.34	42,615.65	31.6	100	-68.4
B-424	63,571.98	42,329.57	30.3	100	-69.7
B-425	63,571.49	42,397.45	30.5	100	-69.5
B-426	63,571.71	42,615.14	31.4	100	-68.6
B-427	63,660.84	42,331.92	30.6	150	-119.4
B-428DH	63,660.05	42,398.55	30.9	218	-187.1
B-429	63,660.04	42,505.46	31.2	100	-68.8
B-430	63,624.24	42,617.30	30.9	150	-119.1
B-431	63,634.57	42,641.92	31.1	75	-43.9
B-432	63,739.93	42,701.18	31.2	150	-118.8
B-433	63,747.31	42,458.80	31.6	100	-68.4
B-434	63,752.98	42,354.31	31.1	100	-68.9
B-435	63,736.38	42,141.62	28.9	75	-46.1
B-436	63,681.44	42,034.98	30.3	75	-44.7
B-437	63,679.95	42,247.72	28.2	75	-46.8

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-438	63,791.36	42,003.39	30.2	125	-94.8
B-439	63,790.82	42,250.03	28.7	125	-96.3
B-440	63,281.42	42,249.68	31.1	200	-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	-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
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

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 – OUTSIDE POWER BLOCK					
B-901	63,771.76	41,809.14	29.3	100	-70.7
B-902	63,496.08	41,927.00	29.1	100	-70.9
B-903	63,672.23	41,664.45	30.0	100	-70.0
B-904	63,485.07	41,727.16	29.8	100	-70.2
B-905	63,348.01	41,571.36	29.2	100	-70.8
B-906	63,574.46	41,430.55	29.5	100	-70.5
B-907	63,549.17	41,252.15	29.2	100	-70.8
B-908	63,273.09	41,356.36	29.6	100	-70.4
B-909	63,521.67	41,590.66	29.7	100	-70.3
B-910	63,362.31	41,257.10	30.4	125	-94.6
B-911	63,254.68	41,663.52	30.8	50	-19.2
B-912	63,253.49	41,860.53	31.1	100	-68.9
B-918	64,814.60	42,764.10	30.9	100	-69.1
B-919	64,814.59	43,088.48	31.9	100	-68.1
B-920	62,943.94	43,897.79	28.2	30	-1.8
B-927	62,183.19	49,228.65	26.8	60	-33.2
B-928	64,932.77	40,366.26	29.6	125	-95.4
B-929	64,672.42	45,487.07	36.6	130	-93.4
B-930	60,212.08	49,516.47	25.6	120	-94.4
B-931	61,984.41	39,511.72	29.9	125	-95.1
B-932	61,899.52	42,106.11	31.0	125	-94.0
B-933	61,895.26	43,504.02	28.7	125	-96.3
B-934	62,081.37	48,244.01	28.6	110	-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 Plan (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, located between STP 3 and STP 4, is included with STP 3.
[4] Refer to Reference 2C for 2008 As-Built boring information.

This table was updated to reflect the redistribution of data inside and outside the Power Block.

Table 2.5S.4-20 Undisturbed Tube Sample Details

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
UNDISTURBED TUBE SAMPLES – STP 3					
B-303	UD1	CH (t); SM ~~(b)~~	D/~~E~~	63	-36.4
B-303	UD2	CH	F	88	-61.4
B-303	UD3	SM	H	108	-81.4
B-303	UD4	CH	J Clay 1	133	-106.4
B-303	UD5	SM	J Interbed 2	168	-141.4
B-305DH	UD1	CH	A	3	26.8
B-305DH	UD2	NR (may be SP-SM)	C	25	4.8
B-305DH	UD3	NR (may be SP-SM)	C	38	-8.2
B-305DH	UD3A	NR (may be SP-SM)	C	40	-10.2
B-305DH	UD4	CL	D	53	-23.2
B-305DH	UD5	SP-SM	E	78	-48.2
B-305DH	UD6	CH	H	103	-73.2
B-305DH	UD7	CH	J Clay 1	123	-93.2
B-305DH	UD8	CL	J Clay 1	138	-108.2
B-305DH	UD9	CH (t); ML (b)	J Clay 1	158	-128.2
B-305DH	UD10	CH	J Clay 2	193	-163.2
B-305DH	UD11	CL	K Clay	213	-183.2
B-305DH	UD12	SM	K Sand	228	-198.2
B-305DH	UD13	CH (t); SP-SM (b)	M	263	-233.2
B-305DH	UD14	CH	N Clay 1	288	-258.2
B-305DH	UD15	CH	N Clay 1	313	-283.2
B-305DH	UD15A	CH	N Clay 1	316.5	-286.7
B-305DH	UD16	CH	N Clay 1	338	-308.2
B-305DH	UD17	SP-SM	N Sand 1	353	-323.2
B-305DH	UD17A	SP-SM	N Sand 1	353.5	-323.7
B-305DH	UD18	SP-SM	N Sand 2	385	-355.2
B-305DH	UD20	SP-SM	N Sand 3	418	-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	-478.2
B-305DHA	UD24	CH	N Clay 6	553	-523.2
B-305DHA	UD25	CH	N Clay 6	588	-558.2
B-306	UD1	SM	C	38	-10.2
B-306	UD1A	SM	C	40	-12.2
B-306	UD2	SM	E	63	-35.2

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-306	UD3	SC	E	73	-45.2
B-306	UD4	CH	F	88	-60.2
B-306	UD5	SP-SM	H	98	-70.2
B-306	UD6	SP-SM	H	103	-75.2
B-306	UD7	GW (t); CH (b)	J Clay 1	118	-90.2
B-306	UD8	CH	J Clay 1	141	-113.2
B-306	UD9	CH	J Clay 1	151	-123.2
B-306	UD9A	CH (t); ML (b)	J Clay 1	153	-125.2
B-306	UD10	CH	J Clay 2	191	-163.2
B-307	UD1	CH	J Clay 1	118	-89.8
B-307	UD2	SM	J Sand 1	153	-124.8
B-307	UD3	CH	J Clay 2	188	-159.8
B-314	UD1	SP	E	83	-53.8
B-314	UD2	CL	J Clay 1	113	-83.8
B-314	UD3	CH	J Clay 1	121	-91.8
B-314	UD4	SC (t); CL (b)	J Clay 1	141	-111.8
B-314	UD5	NR (may be CH)	J Clay 2	181	-151.8
B-314	UD5A	CH	J Clay 2	183	-153.8
B-314	UD6	CH	J Clay 2	191	-161.8
B-319DH	UD1	CH	J Clay 1	128	-99.6
B-319DH	UD2	SM	J Sand 1	143	-114.6
B-319DH	UD3	SM	J Sand 1	158	-129.6
B-319DH	UD4	CH	J Clay 2	173	-144.6
B-319DH	UD5	CH	J Clay 2	188	-159.6
B-321	UD1	CH	D	43	-13.8
B-321	UD2	CH	J Clay 1	118	-88.8
B-321	UD3	CL	J Clay 1	138	-108.8
B-328DH	UD1	CL	A	13	16.9
B-328DH	UD2	NR (may be SM)	C	33	-3.1
B-328DH	UD3	CH	D	53	-23.1
B-328DH	UD4	SM	E	73	-43.1
B-328DH	UD5	NR (may be SP-SM)	E	83	-53.1
B-328DH	UD6	NR (may be SM)	H	103	-73.1
B-330	UD1A	NR (may be SM)	C	38	-8.5
B-330	UD1B	NR (may be SM)	C	40	-10.5
B-330	UD2	CH	D	53	-23.5
B-330	UD3	SP (t); SM (b)	E	63	-33.5
B-330	UD4	NR (may be SM)	H	118	-88.5

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-330	UD4B	CH	J Clay 1	123	-93.5
B-332	UD1	CH	A	3	27.3
B-332	UD2	ML	B	23	7.3
B-333	UD1	CL	A	8	22.5
B-333	UD2	CL	A	18	12.5
B-338	UD1	SM	C	28	4.1
B-338	UD2	CL	D	48	-15.9
B-343	UD1	CH (t); SM	B/ C	23	7.5
B-343	UD2	SM (t); CH (b)	D	48	-17.5
B-343	UD3	CH (t); SM	E	58	-27.5
B-343	UD4	NR (may be SM)	E	68	-37.5
B-343	UD4A	CH	E	70	-39.5
B-343	UD5	SM	J Interbed 1	123	-92.5
B-343	UD6	SM	J Sand 1	148	-117.5
B-343	UD7	CL-ML	J Clay 2	173	-142.5
B-343	UD8	CH	J Clay 2	198	-167.5
B-348	UD1	CL	A	5	25
B-348	UD2	ML (t); CL (b)	B	13	17
B-348	UD3	ML (t); SM (b)	B/ C	18	12
~~RELATIVELY~~ UNDISTURBED TUBE SAMPLES – STP 4					
B-401	UD1	CH	D	58	-26.9
B-401	UD2	CH	F	88	-56.9
B-401	UD3	CL	J Clay 1	118	-86.9
B-401	UD4	SM	J Sand 1	153	-121.9
B-401	UD5	NR (may be CH)	J Clay 2	178	-146.9
B-401	UD5A	CH	J Clay 2	184	-152.9
B-404	UD1	CH	F	88	-57
B-404	UD2	CH	F	98	-67
B-404	UD3	CH	J Clay 1	121	-90
B-404	UD4	CH	J Clay 1	131	-100
B-404	UD5	CL	J Clay 1	141	-110
B-404	UD6	CL	J Clay 2	161	-130
B-404	UD7	CH	J Clay 2	181	-150
B-404	UD8	CH	J Clay 2	191	-160
B-405DH	UD1	CH	A	10	21.1
B-405DH	UD2	CL	B	28	3.1

Table 2.5S.4-20 Undisturbed Tube Sample Details (Continued)

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
~~RELATIVELY~~ UNDISTURBED TUBE SAMPLES – STP 4 (Continued)					
B-405DH	UD3	CL	D	63	-31.9
B-405DH	UD4	CL	F	83	-51.9
B-405DH	UD5	CH	J Clay 1	113	-81.9
B-405DH	UD6	CL	J Clay 1	125	-93.9
B-405DH	UD7	SM	J Sand 1	148	-116.9
B-405DH	UD8	CH (t); ML (b)	J Interbed 2	168	-136.9
B-405DH	UD9	CL	J Clay 2	193	-161.9
B-405DH	UD10A	CH	K Clay	222	-190.9
B-405DH	UD11	CH	K Clay	233	-201.9
B-405DH	UD12	SP-SM	M	263	-231.9
B-405DH	UD13	CH	N Clay 1	293	-261.9
B-405DH	UD14	CH	N Clay 1	318	-286.9
B-405DH	UD15	SP	N Sand 1	343	-311.9
B-405DH	UD16	CH	N Clay 2	358	-326.9
B-405DH	UD17	SC	N Sand 2	388	-356.9
B-405DH	UD18	SP	N Sand 3	418	-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	-456.9
B-405DH	UD22	SM	N Sand 5	518	-486.9
B-405DH	UD23	SM	N Sand 5	538	-506.9
B-405DH	UD24	CH	N Clay 6	568	-536.9
B-405DH	UD25	CL	N Clay 6	598	-566.9
B-409	UD1	SM	E	68	-36.8
B-409	UD2	NR (may be CH)	F	93	-61.8
B-409	UD2A	NR (may be CH)	F	95	-63.8
B-409	UD3	CH	J Clay 1	128	-96.8
B-409	UD4	NR (may be SM)	J Sand 1	158	-126.8
B-409	UD4A	CH	J Clay 2	160	-128.8
B-409	UD5	CH (t); SP-SM (b)	J Interbed 2	188	-156.8
B-409	UD6	CH	J Clay 2	198	-166.8
B-415	UD1	CH	F	88	-58
B-415	UD2	CH (t); SP-SM (b)	F/ H	98	-68
B-415	UD3A	NR (may be CH)	J Clay 1	121	-91
B-415	UD3	CH	J Clay 1	124	-94
B-415	UD4A	NR (may be CH)	J Clay 1	131	-101
B-415	UD4	NR (may be CH)	J Clay 1	134	-104

Table 2.5S.4-20 Undisturbed Tube Sample Details (Continued)

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
~~RELATIVELY~~ UNDISTURBED TUBE SAMPLES – STP 4 (continued)					
B-419DH	UD1	CL	F	78	-48.3
B-419DH	UD2	CH (t); SM	F	98	-68.3
B-419DH	UD3	CH	J Clay 1	118	-88.3
B-419DH	UD4	CL	J Clay 1	138	-108.3
B-419DH	UD6	CH	J Clay 2	178	-148.3
B-419DH	UD7	CH	J Clay 2	198	-168.3
B-421	UD1	SM (t); SP-SM (b)	C	33	-2.7
B-421	UD1A	SP-SM	C	33.6	-3.3
B-421	UD2	CH	D	53	-22.7
B-421	UD3	CH	F	83	-52.7
B-428DH	UD1	CH	A	3	27.9
B-428DH	UD2	NR (may be SM)	B	23	7.9
B-428DH	UD2A	NR (may be SM)	B	25	5.9
B-428DH	UD3	CH	D	43	-12.1
B-428DH	UD4	CH (t); ML (b)	D	63	-32.1
B-428DH	UD5	NR (may be SM)	H	93	-62.1
B-428DH	UD5A	NR (may be SM)	H	95	-64.1
B-428DH	UD6	CH	J Clay 1	113	-82.1
B-430	UD1	CH	D	55	-24.1
B-430	UD2	SM	E	83	-52.1
B-430	UD3	CH	J Clay 1	133	-102.1
B-432	UD1	CH	A	3	28.2
B-432	UD2	CL	A	15	16.2
B-432	UD3	SM	B	25	6.2
B-434	UD1	CH	A	8	23.1
B-434	UD2	SM	C	28	3.1
B-434	SS11	SM	C	33.5	-2.4
B-434	UD3	CH	D	53	-21.9
B-438	UD1	CH	A	18	12.2
B-438	UD2	NR (may be SM)	C	33	-2.8
B-438	UD3	SM (t); ML ~~(b)~~	C/ D	43	-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

Table 2.5S.4-20 Undisturbed Tube Sample Details (Continued)

Boring Number	Sample Number	USCS Group	Stratum	Sample Top Depth (feet)	Sample Top El. [1] (feet)
~~RELATIVELY~~ UNDISTURBED TUBE SAMPLES – STP 4 (continued)					
B-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
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	27.9
B-918	UD2	CL	B	18	12.9
B-918	UD3	SM	C	25	5.9
B-918	UD4	CL-ML	D	58	-27.1
B-919	UD1	CH	A	8	23.9
B-919	UD2	CH (t); ML (b)	B	23	8.9
B-919	UD3	CH	D	43	-11.1
B-919	UD4	SP-SM	E	83	-51.1
B-927	UD1	SM	B	13	13.8
B-927	UD1A	NR (may be SM)	B	15	11.8
B-927	UD2	CH	B	28	-1.2
B-927	UD3	CH	D	48	-21.2
B-940	UD-3	CH	D	41	-12.28
B-940	UD-4	CH	D	46	-17.28
B-940	UD-5	CH	D	56	-27.28

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)
UNDISTURBED TUBE SAMPLES – OUTSIDE POWER BLOCK (continued)					
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
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

This table was updated to reflect the redistribution of data inside and outside the Power Block.

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	-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	-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	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	-18.9
C-402	62,824.68	42,600.77	30.8	50	-19.2
C-403	62,825.36	42,289.73	31.6	50	-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	-64.1
C-410	63,788.88	42,140.63	28.9	92	-63.1
C-411	62,902.74	42,803.77	31.1	50	-18.9
C-907	63,219.02	41,968.73	28.5	50	-21.5
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	-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
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	-18.8
C-906	63,212.72	41,758.97	30.2	50	-19.8
C-909	63,464.25	43,948.29	30.2	40	-9.8
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 Plan (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 C-916, made ~~close in to Unit 4, is included with STP 4 here~~ between STP 3 and STP 4, is included with STP 3.
[4] Not included in site characterization for engineering properties.

This table was updated to reflect the redistribution of data inside and outside the Power Block.

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

OW Number	Northing [1] (feet)	Easting [1] (feet)	Reference El.[2] (feet)	Well Depth (feet)	Base El. [2] (feet)
OBSERVATION WELLS – STP 3					
OW-308L	63,196.43	43,374.36	29.9	97.1	-67.2
OW-308U	63,195.64	43,354.04	29.9	47.1	-17.2
OW-332La-R	63,729.36	43,608.74	30	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
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
OW-438U	63,792.04	42,025.17	30.5	41	-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	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	-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	100	-71
OW-934U	62,079.87	48,234.20	28.5	41.1	-12.6

[1] Coordinates are referenced to the Texas South Central State Plan (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 Vs (Ft/ sec)	Minimum Vs (Ft/ sec)	Average Vs (Ft/ sec)	Use Vs (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	~~35~~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	~~121~~ 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	~~122~~ 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	~~3.2~~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	~~128~~ 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	~~3.5~~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	~~3.5~~ 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	~~129~~ 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	~~129~~ 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	~~121~~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				2,220	870	1,537	1,535	0.46
		-324	-329	5	360.5	~~121~~123	45	3.0	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				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	128	N/A	N/A	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				2,540	1,220	1,851	1,850	0.45
		-365	-370	5	401.5	~~121~~123	45	3.0	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				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	128	N/A	N/A	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	~~121~~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	~~121~~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	~~121~~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 Summary of Shear Wave Velocities Deeper than 600 Feet Below Ground Surface [1]

Profile	Top Depth (Feet)	Bottom Depth (Feet)	Top El. (Feet)	Bottom El. (Feet)	Mid-Point Depth [2] (Feet)	Vs (Ft/ sec)
M1P1	609	680	-575	-646	644.5	2,050
	680	780	-646	-746	730.0	2,150
	780	880	-746	-846	830.0	2,250
	880	1,300	-846	-1,266	1,090.0	2,350
	1,300	1,930	-1,266	-1,896	1,615.0	2,550
	1,930	2,500	-1,896	-2,466	2,215.0	2,850
	2,500	3,280	-2,466	-3,246	2,890.0	9,285
M1P2	609	1,000	-575	-966	804.5	1,585
	1,000	1,300	-966	-1,266	1,150.0	2,350
	1,300	1,930	-1,266	-1,896	1,615.0	2,550
	1,930	2,500	-1,896	-2,466	2,215.0	2,850
	2,500	3,280	-2,466	-3,246	2,890.0	9,285
M1P3	609	700	-575	-666	654.5	2,650
	700	780	-666	-746	740.0	2,825
	780	850	-746	-816	815.0	2,900
	850	1,000	-816	-966	925.0	3,000
	1,000	1,060	-966	-1,026	1,030.0	3,100
	1,060	1,160	-1,026	-1,126	1,110.0	3,200
	1,160	1,250	-1,126	-1,216	1,205.0	3,325
	1,250	1,700	-1,216	-1,666	1,475.0	3,575
	1,700	2,500	-1,666	-2,466	2,100.0	4,125
	2,500	3,280	-2,466	-3,246	2,890.0	9,285

[1]Shear wave velocities and depth ranges scaled from Figure B-12, "Shear Wave Velocity Profile for the South Texas Site," Reference 2.5S.4-4
[2]Mid-point depth measured below El. 34 fee

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Table 2.5S.4-29 Summary of Strata Unit Weights

Depth Below Ground Surface (feet)	Stratum and/or Soil Type	Selected Unit Weight (pcf)
Ground Surface to 20	A	124
20 to 30	B	121
30 to 50	C	122
50 to 70	D	~~121~~ 123
70 to 90	E	~~122~~ 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	~~129; 127~~ 124; 127
258 to 263	L	~~129~~ 124 [1]
263 to 278	M	127 [1]
278 to 609	N Clay; N Sand	~~121~~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

This figure has been updated.

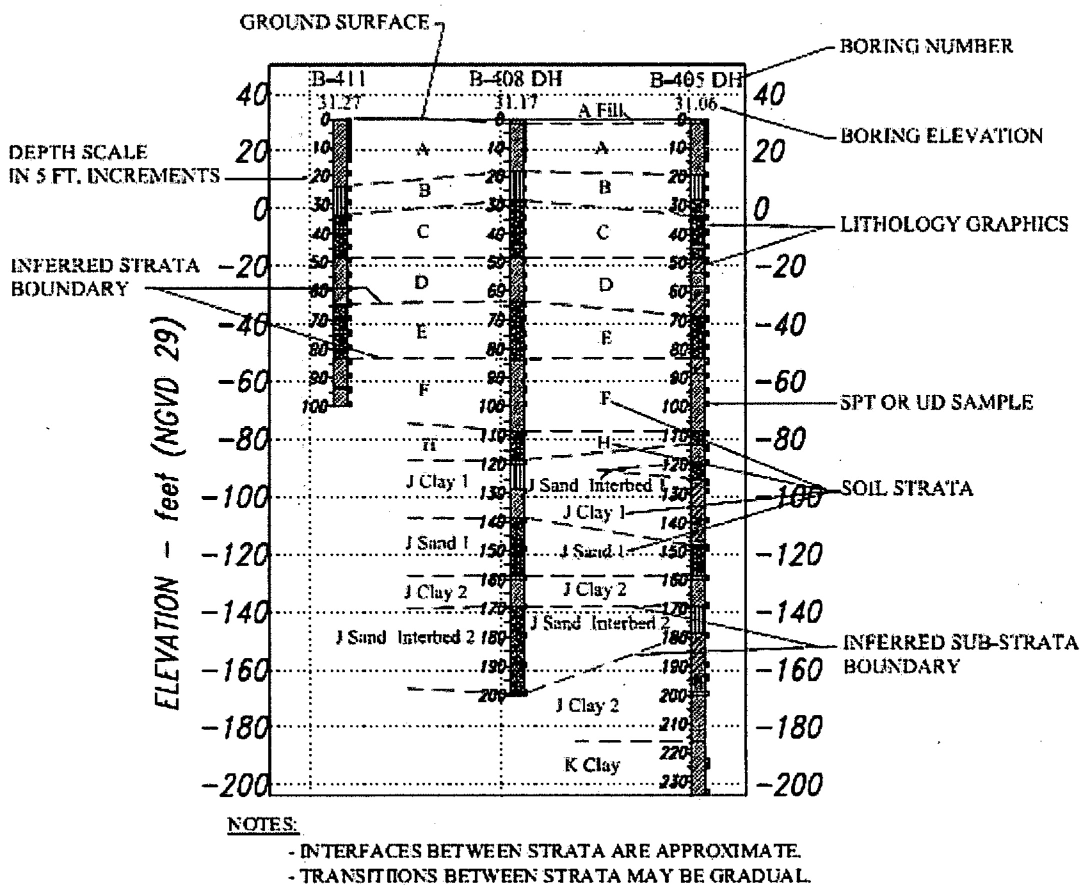

Figure 2.5S.4-3 Subsurface Profile Legend

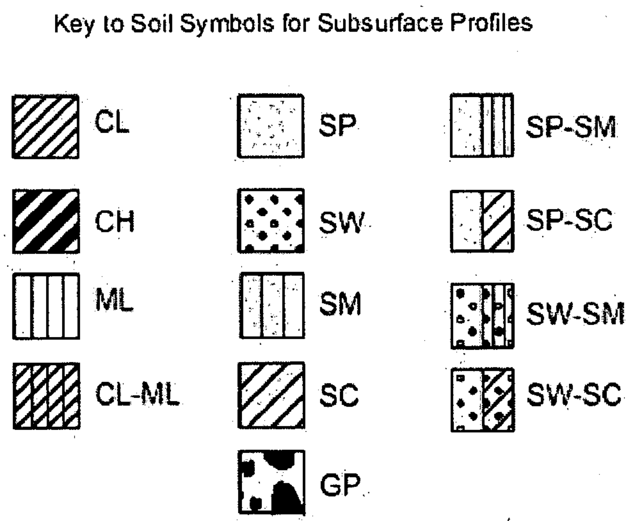

Figure 2.5S.4-3 Subsurface Profile Legend (Continued)

This figure has been updated.

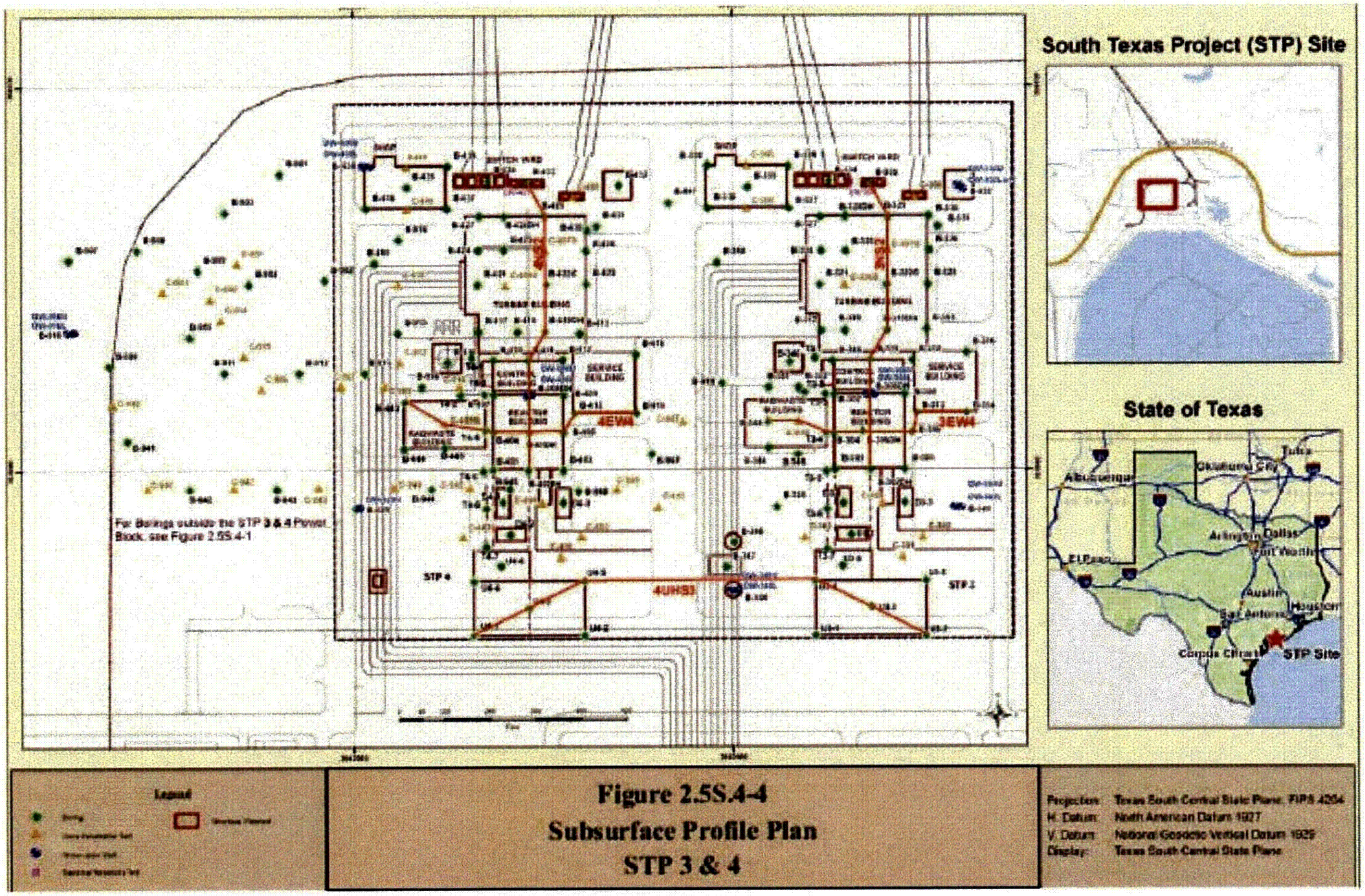

Figure 2.5S.4-4 Subsurface Profile Plan

This figure has been updated.

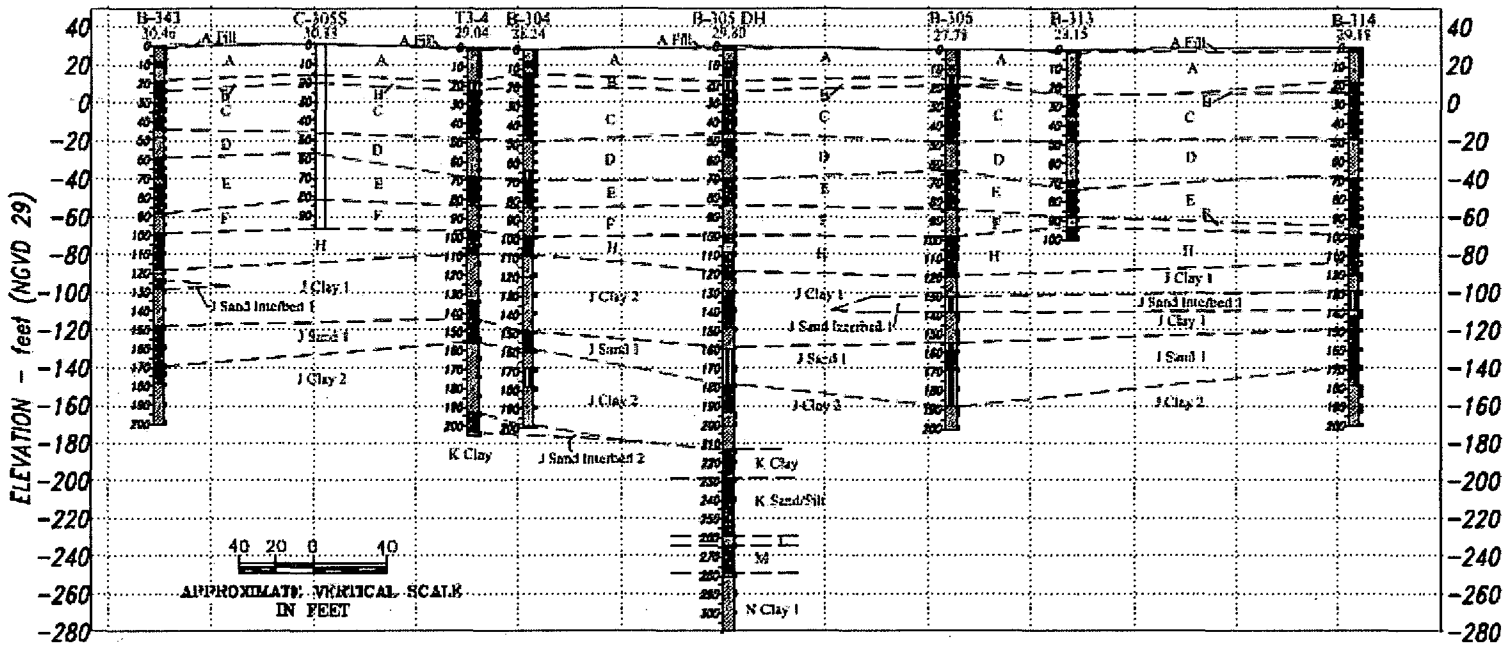

Figure 2.5S.4-5 Subsurface Profile 3EW4

This figure has been updated.

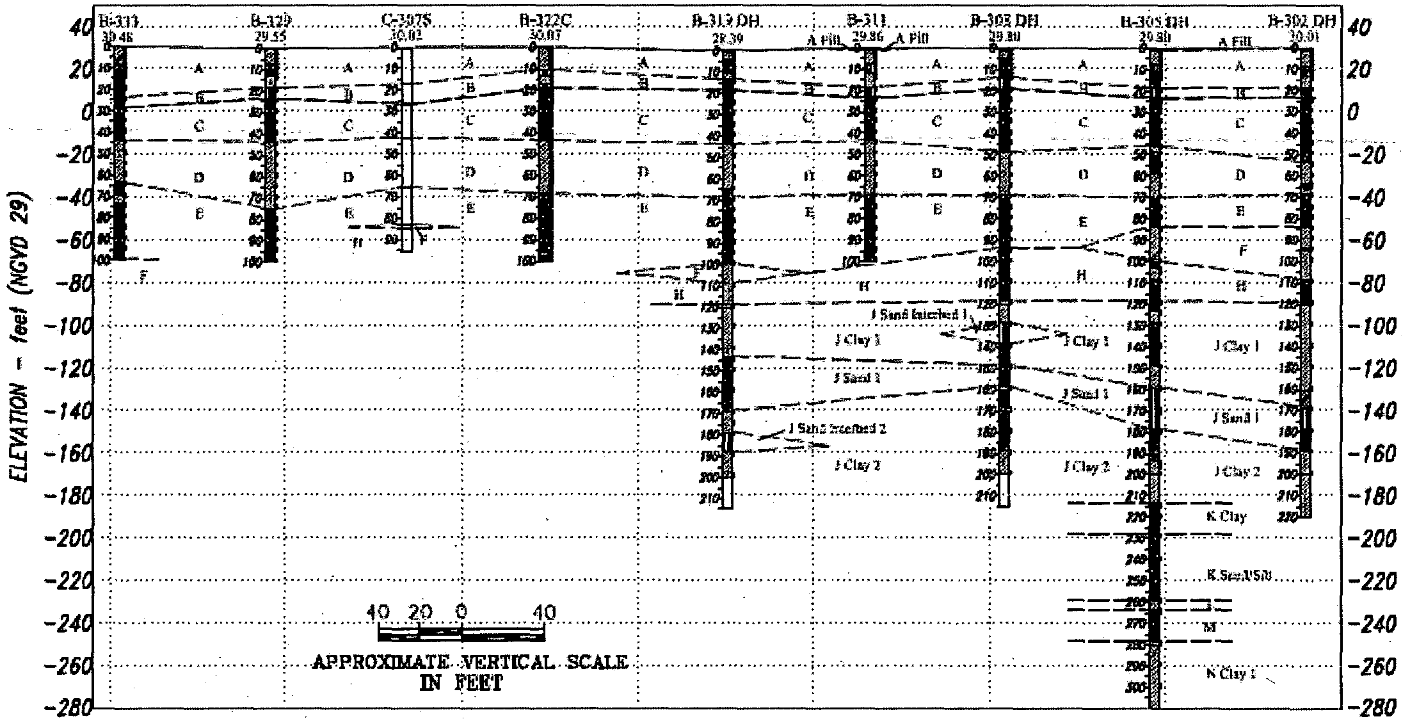

Figure 2.5S.4-6 Subsurface Profile 3N2S

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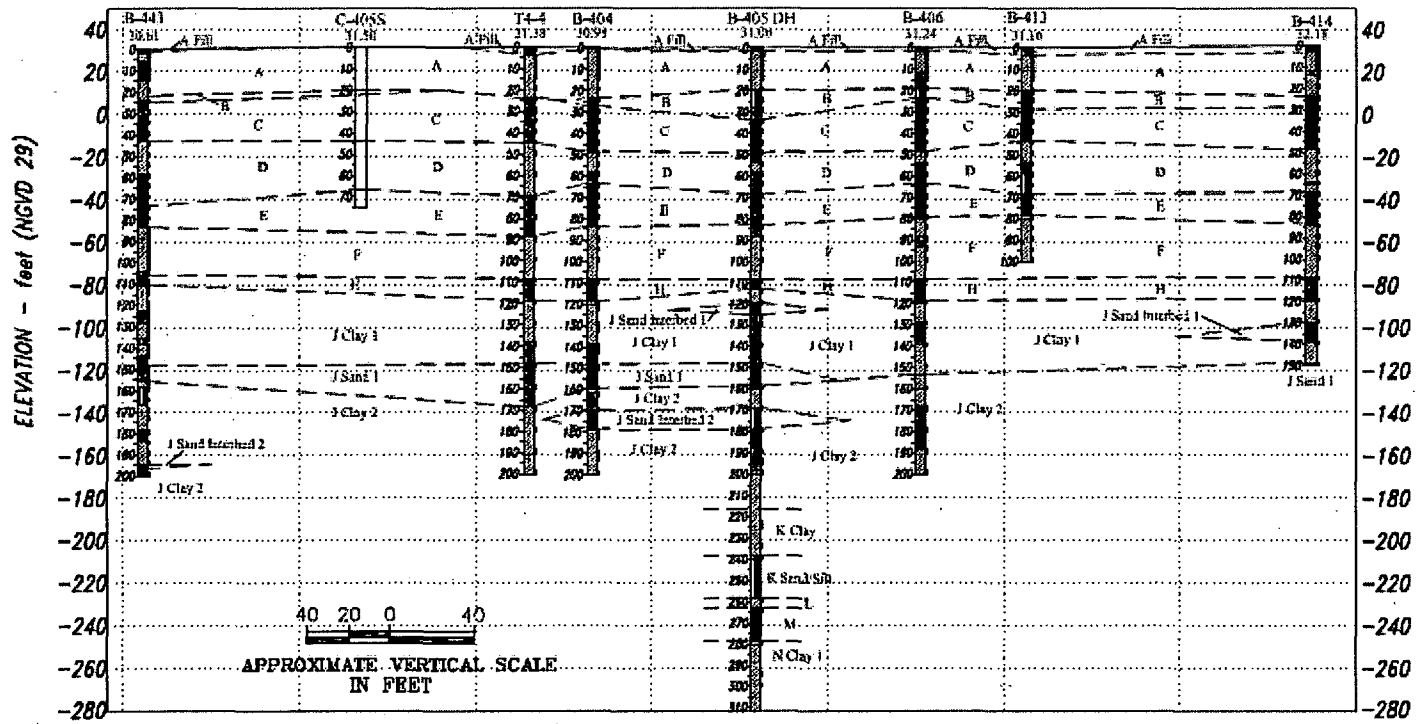

Figure 2.5S.4-7 Subsurface Profile 4EW4

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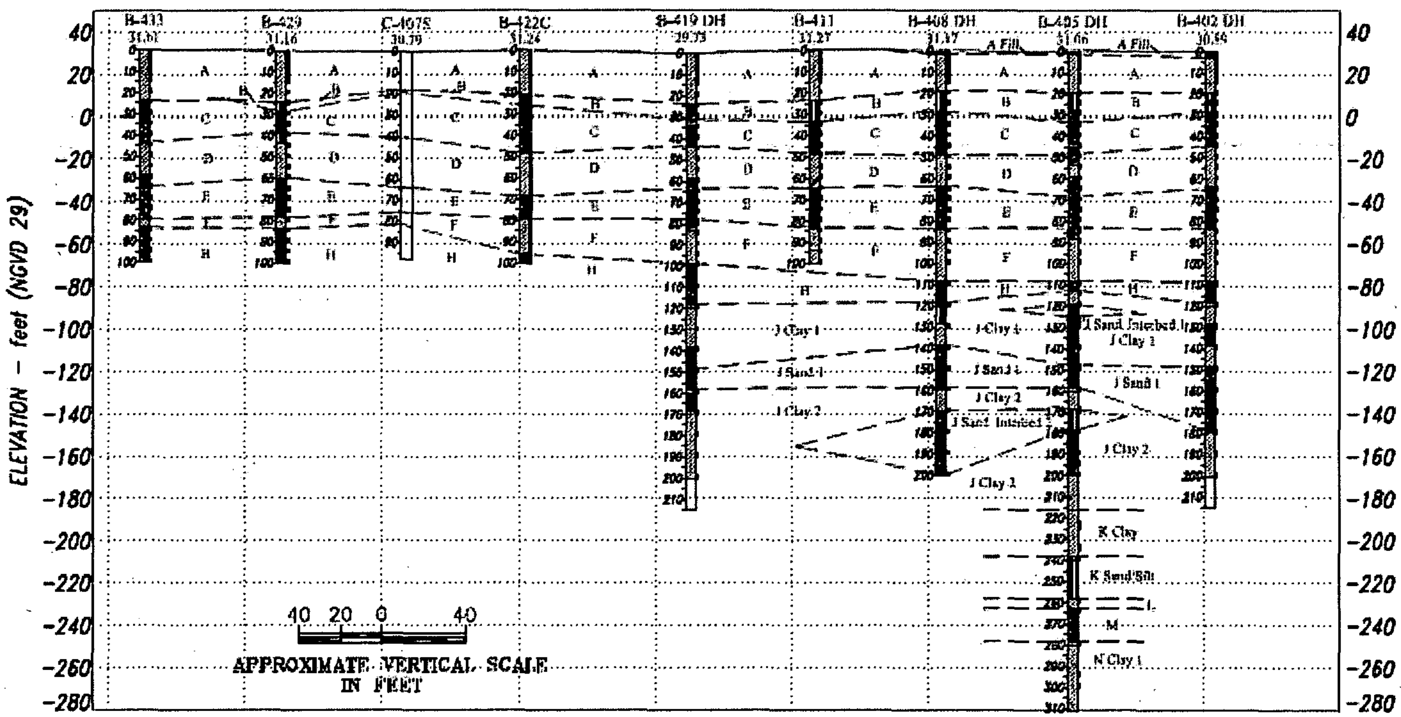


Figure 2.5S.4-8 Subsurface Profile 4NS2

This figure has been updated.

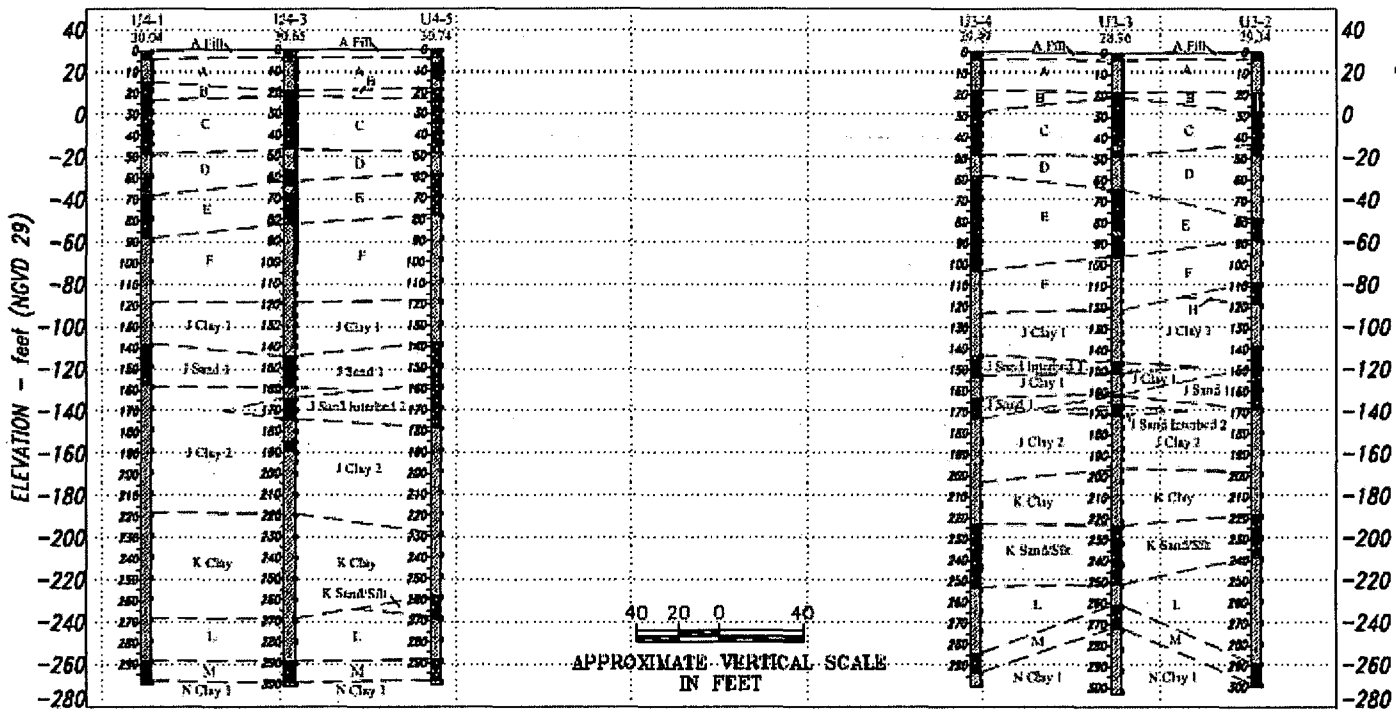


Figure 2.5S.4-9 Subsurface Profile 4UHS3

This figure has been updated.

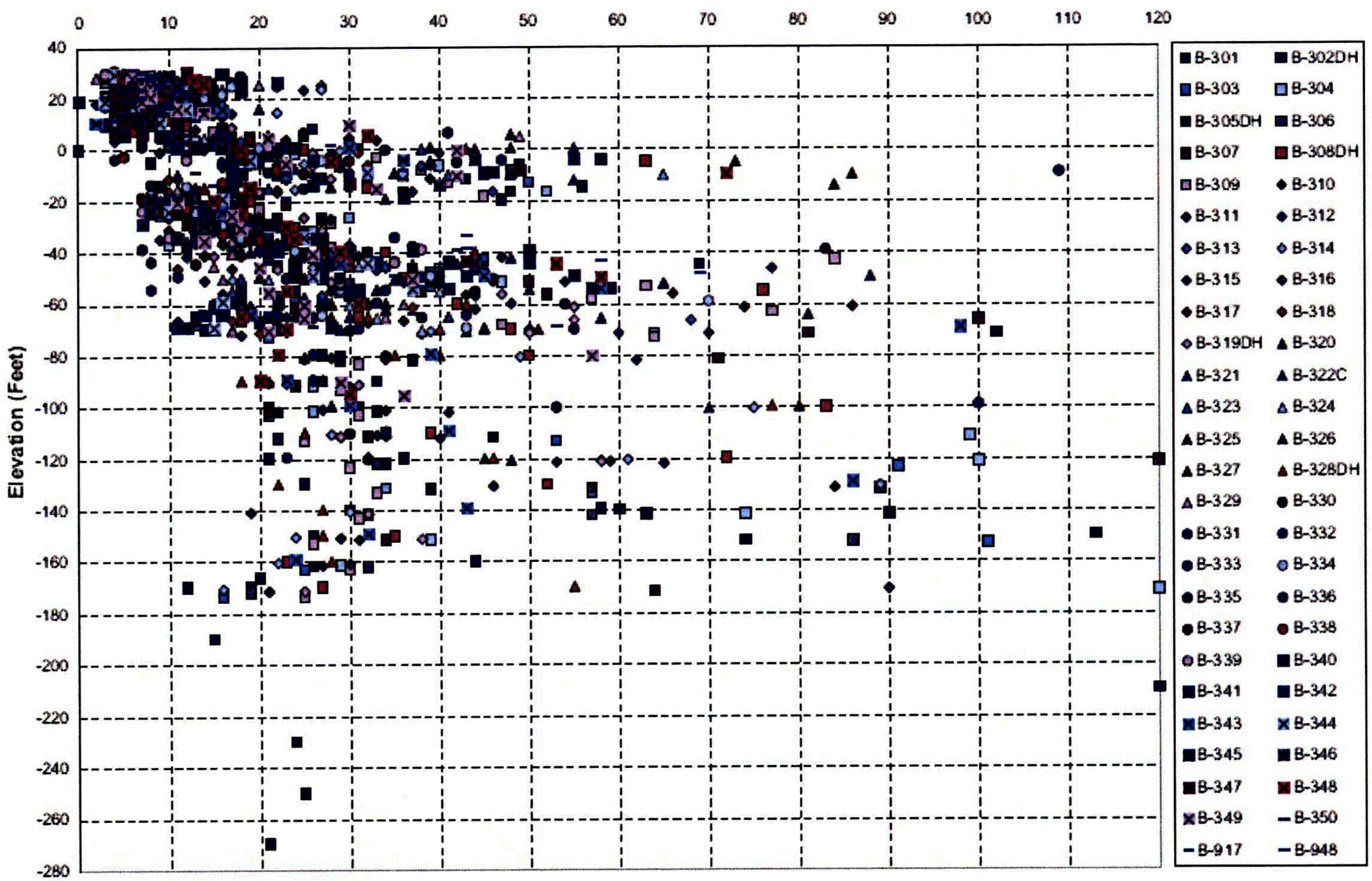


Figure 2.5S.4-10 Uncorrected SPT N-Values (STP 3) <Includes B-917>

This figure has been updated.

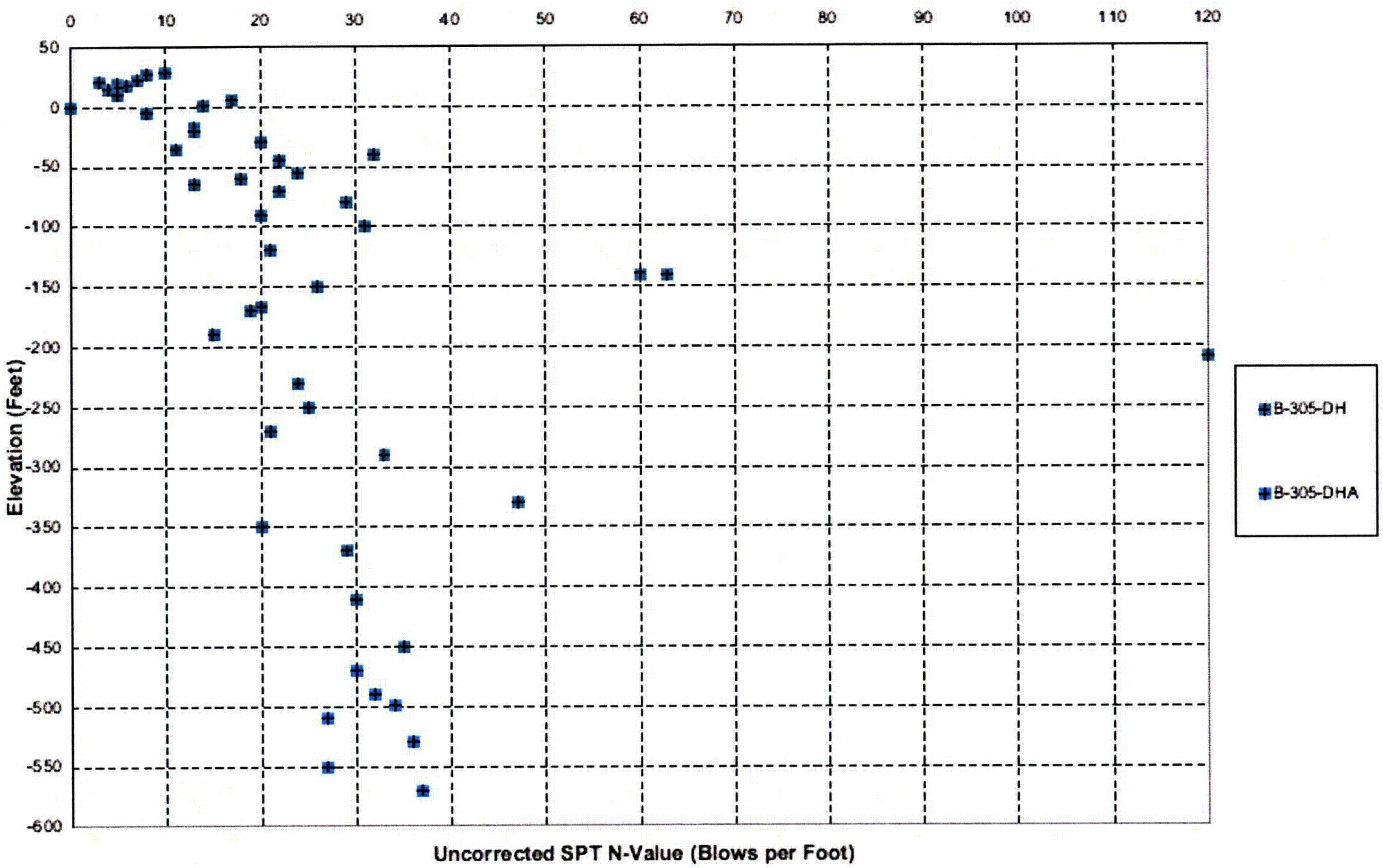


Figure 2.5S.4-11 Uncorrected SPT N-Values (STP 3; Boring B-305DH/DHA)

This figure has been updated.

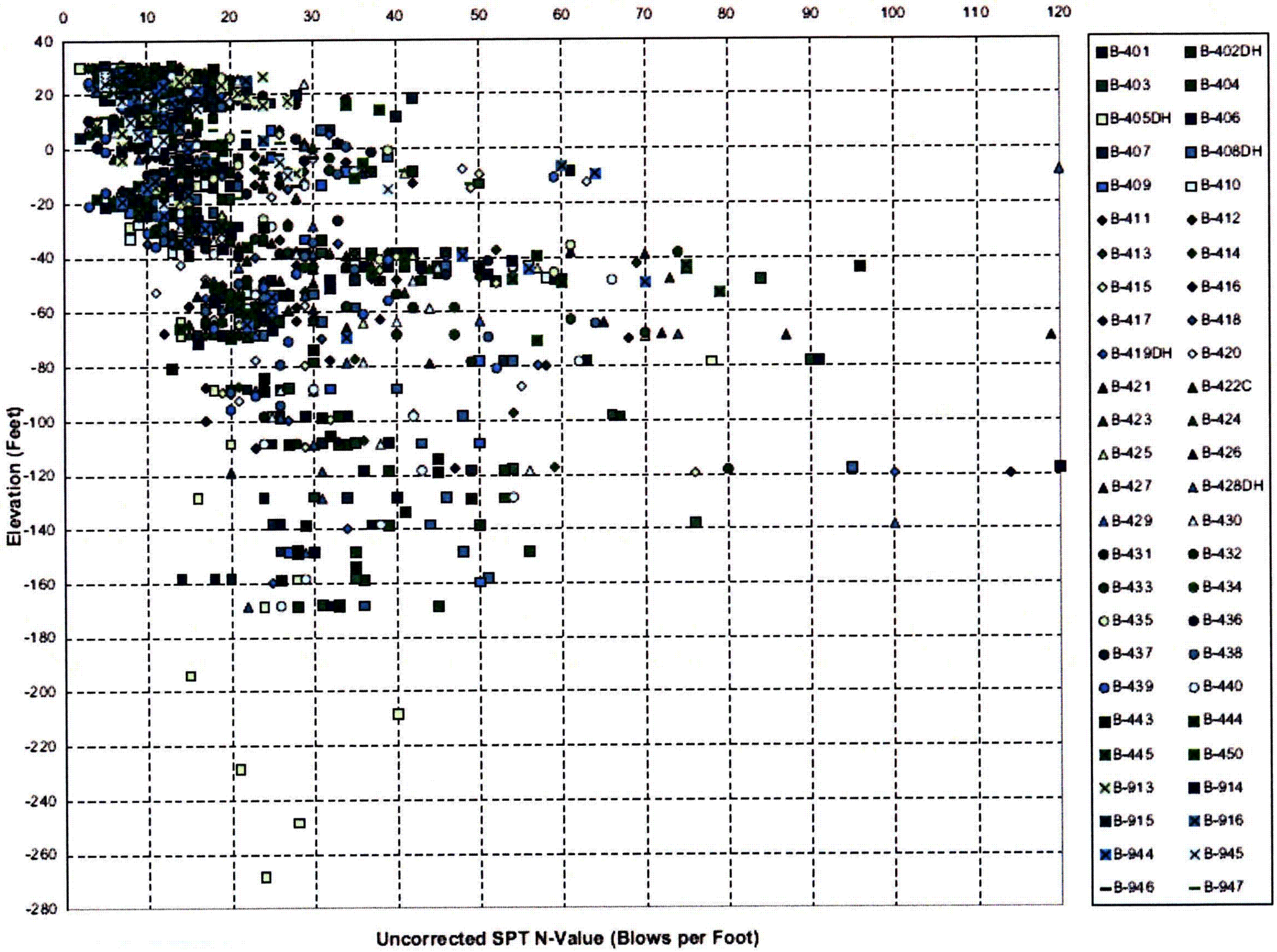


Figure 2.5S.4-12 Uncorrected SPT N-Values (STP 4)

This figure has been updated.

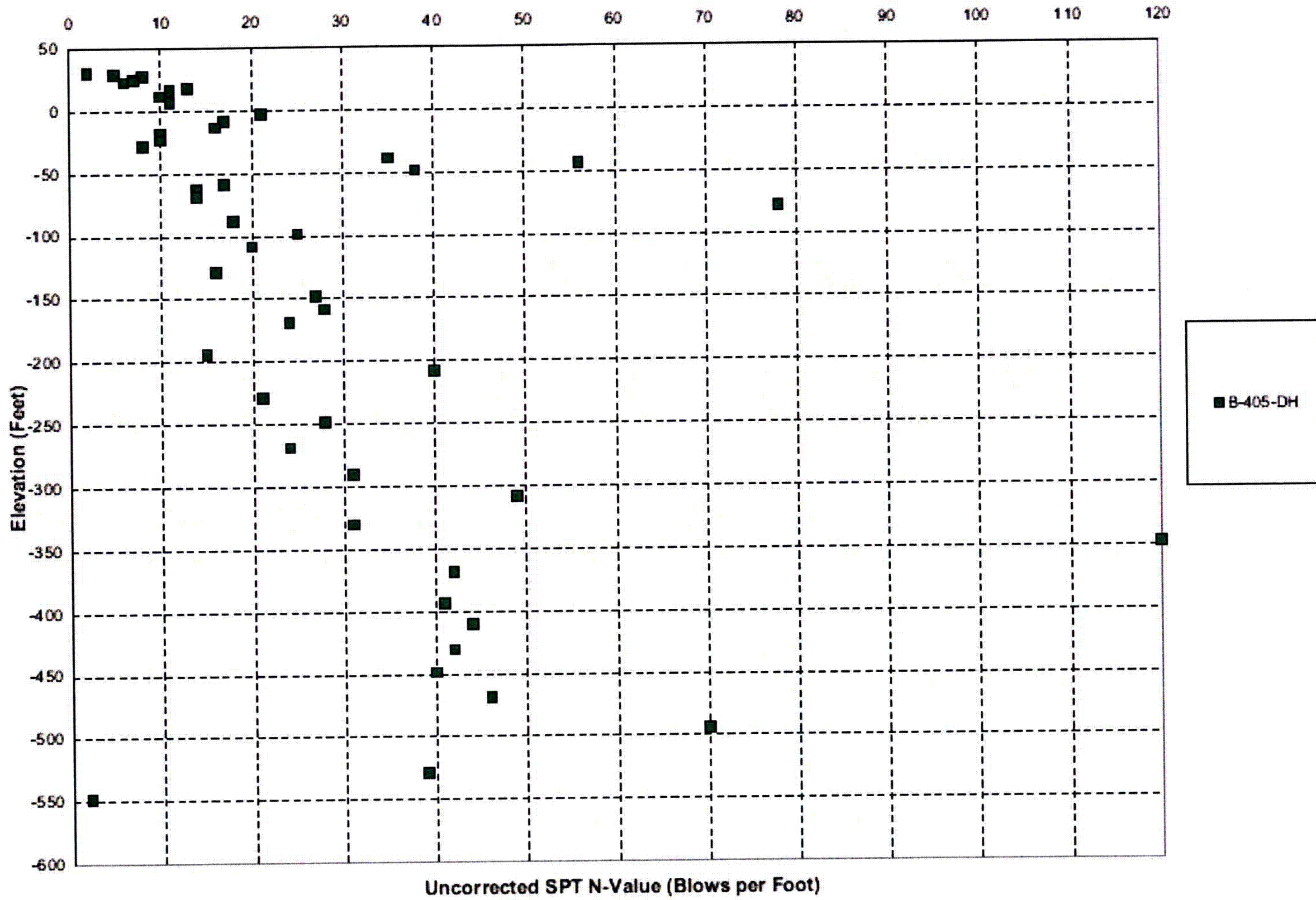


Figure 2.5S.4-13 Uncorrected SPT N-Values (STP 4; Boring B-405DH)

Figure 2.5S.4-14 Not Used
(The data has been included in Figure 2.5S.4-15)

Question 02.05.04-13, Supplement 1

This figure has been updated.

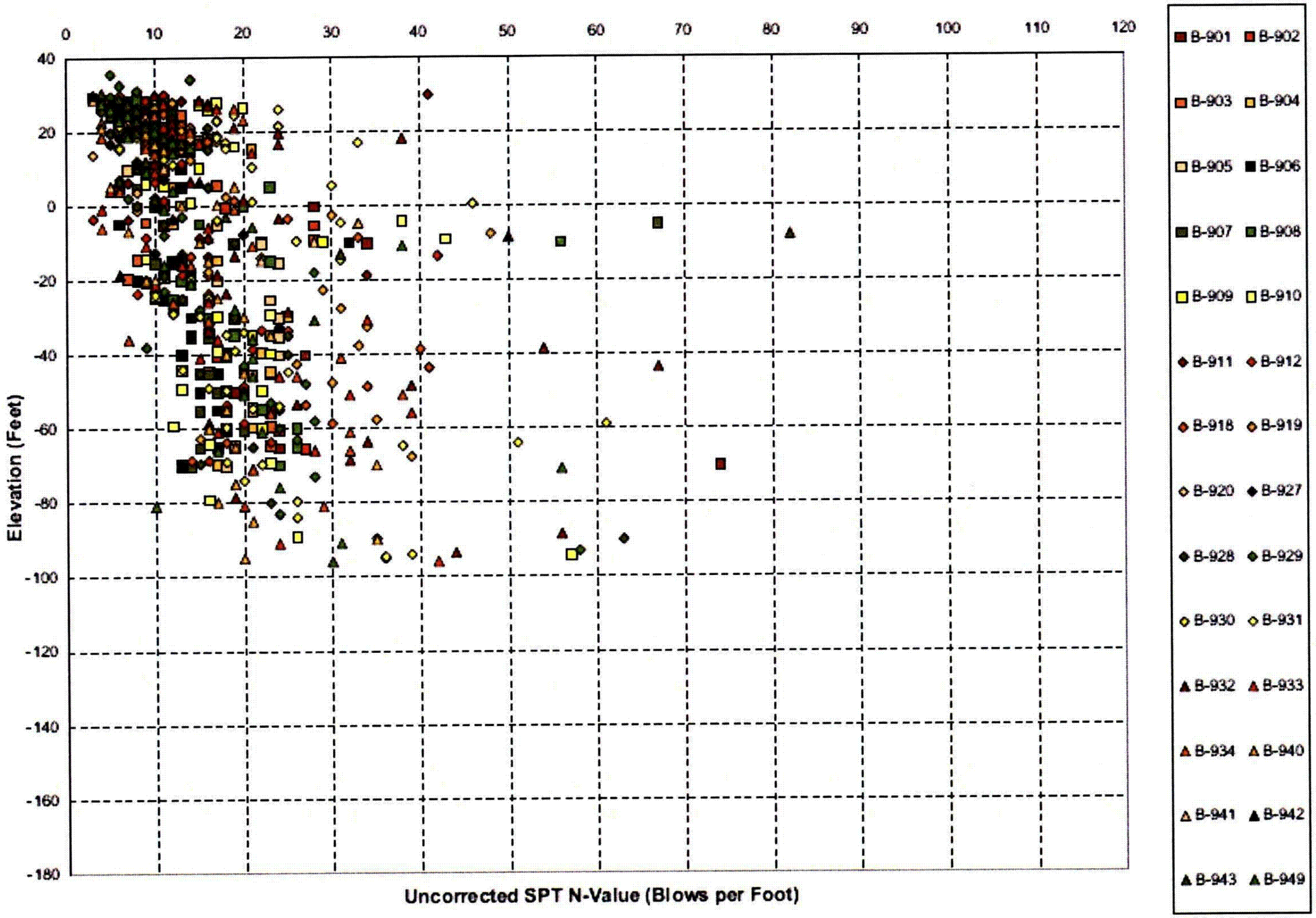
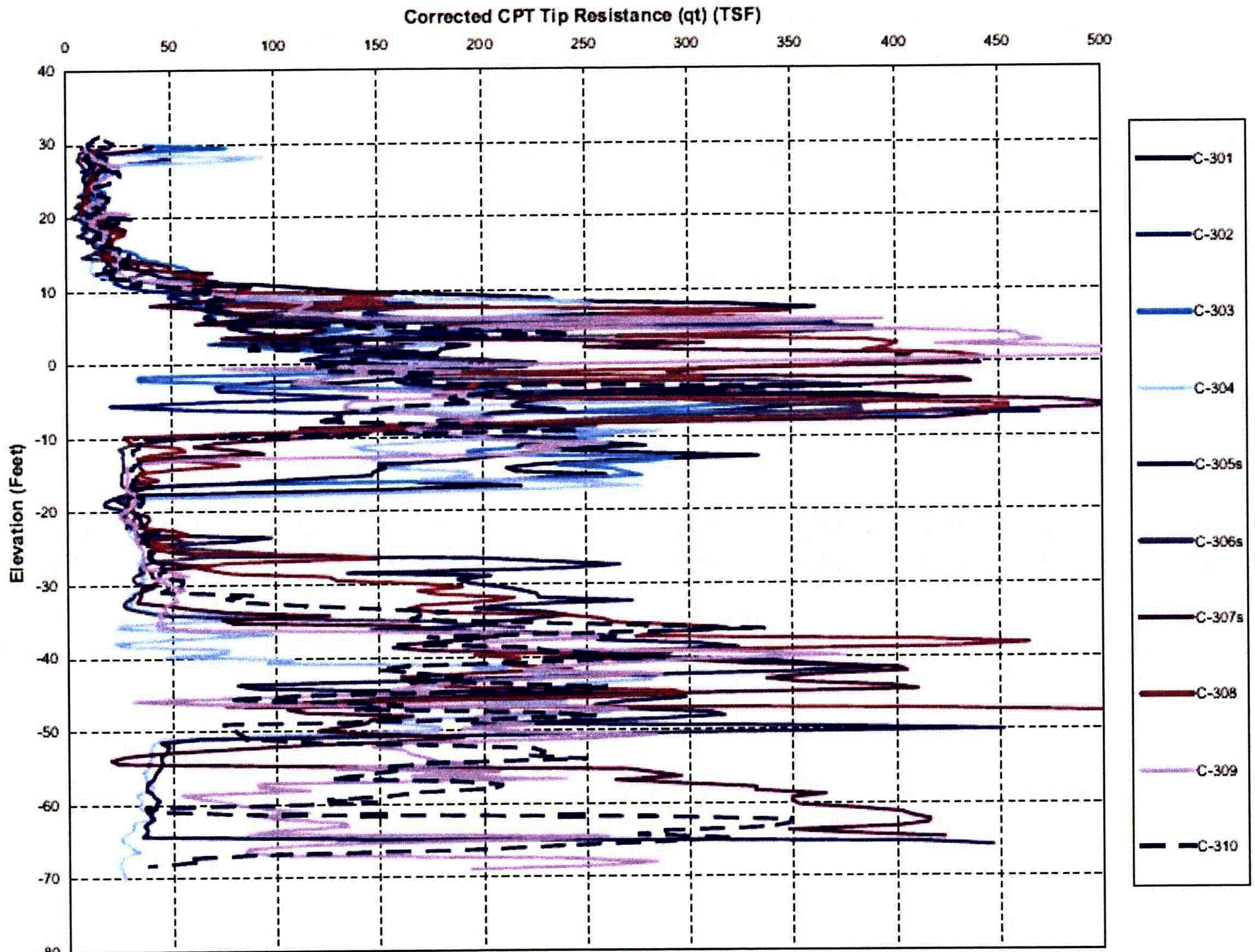


Figure 2.5S.4-15 Uncorrected SPT N-Values (Outside the Power Block)

Question 02.05.04-13, Supplement 1

This figure has been updated.



NOTE: C-947 was not representative and therefore was not used as part of this data.

Figure 2.5S.4-16 Corrected CPT Tip Resistance (qt) (STP 3)

This figure has been updated.

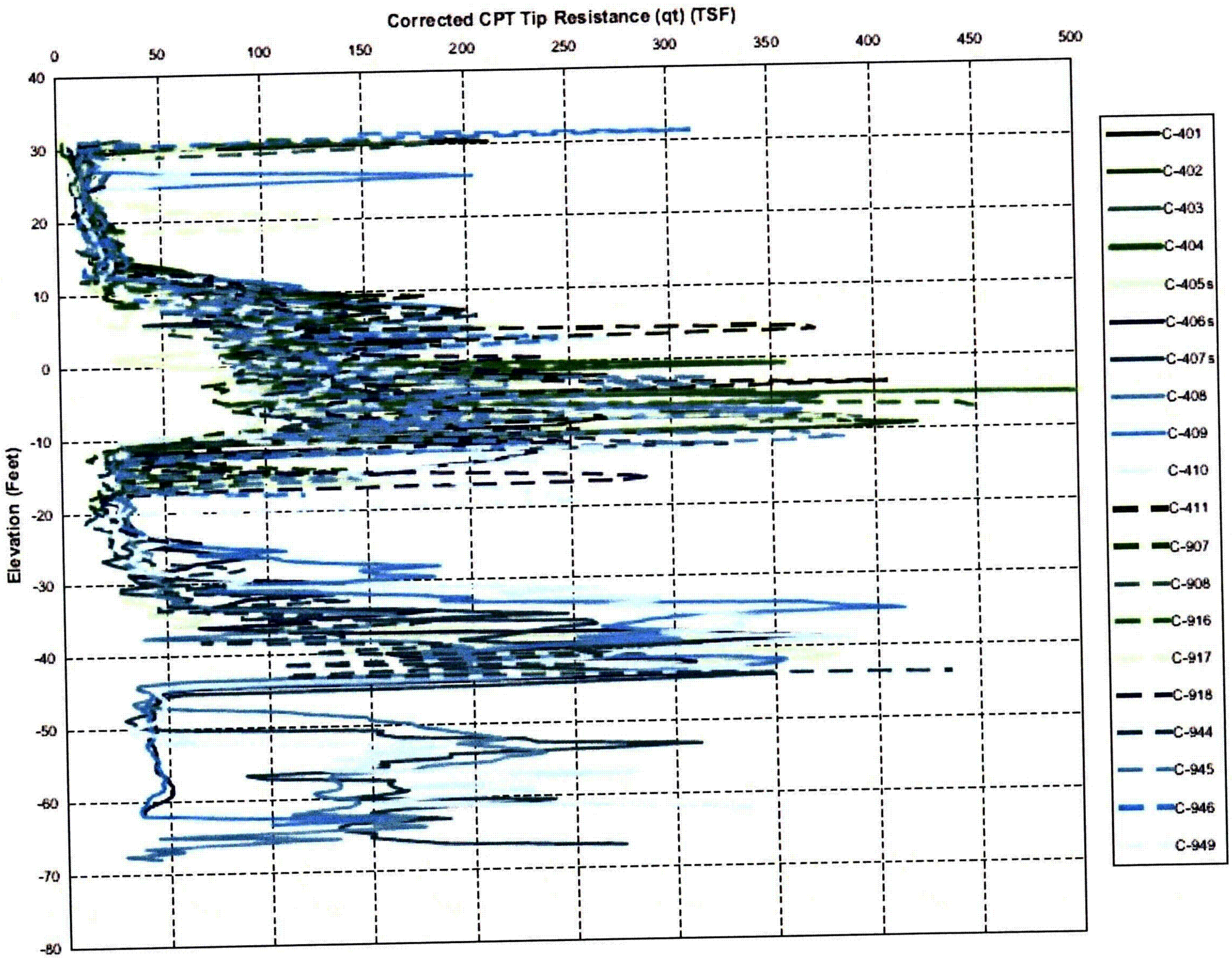


Figure 2.5S.4-17 Corrected CPT Tip Resistance (qt) (STP 4) <Includes C-916>

Figure 2.5S.4-18 Not Used
(The data has been included in Figure 2.5S.4-19)

This figure has been updated.

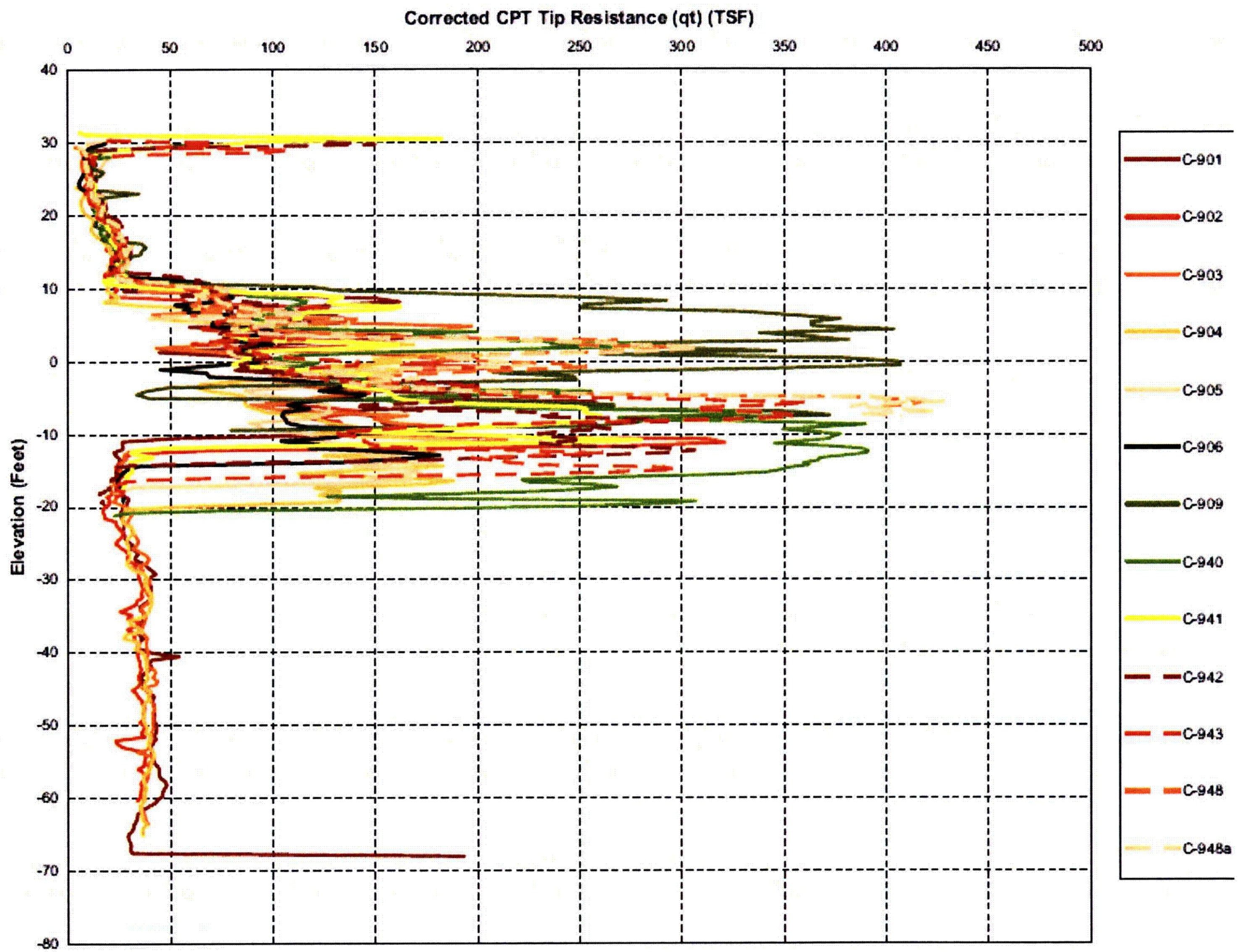


Figure 2.5S.4-19 Corrected CPT Tip Resistance (qt) (Outside Power Block)

This figure has been updated.

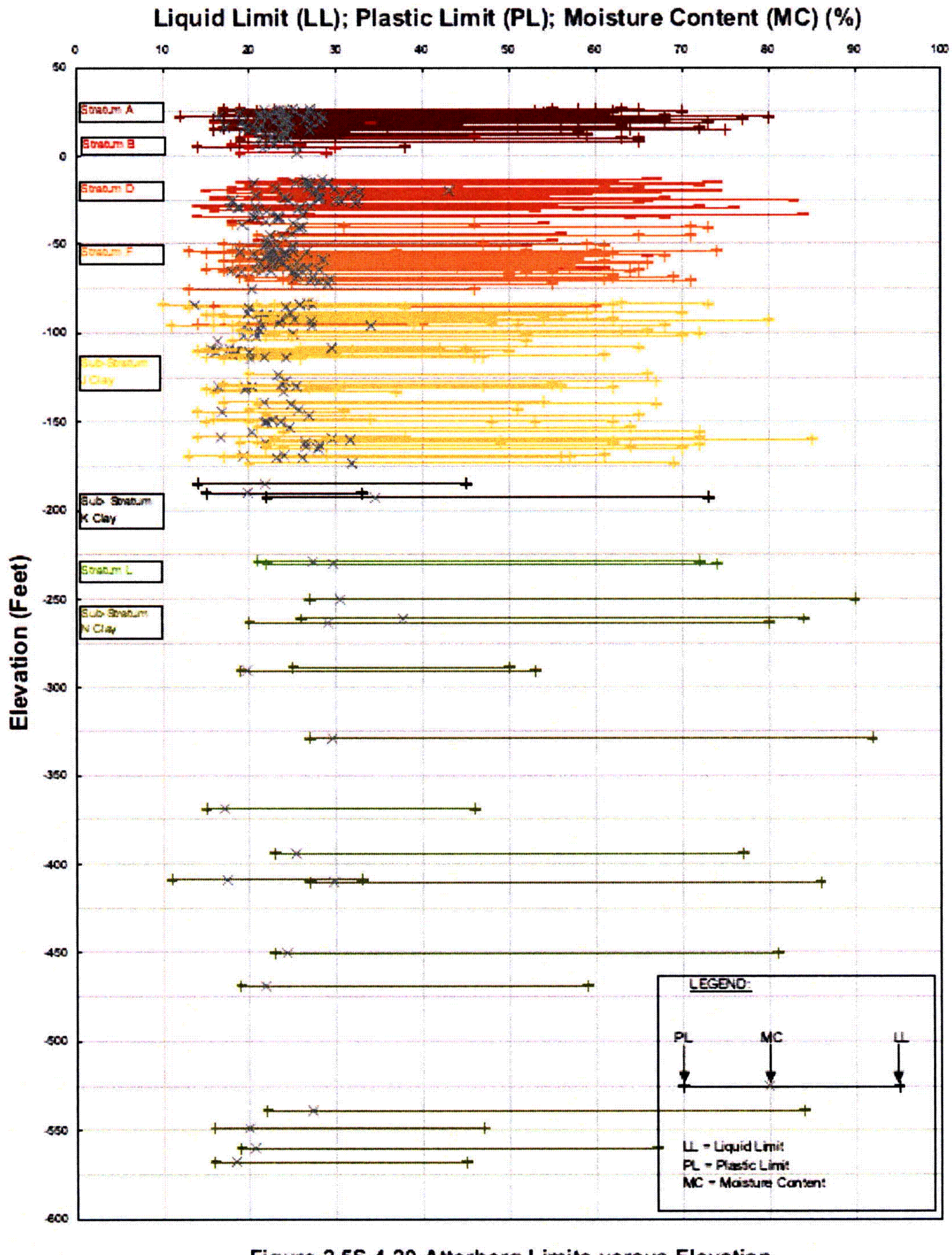


Figure 2.5S.4-20 Atterberg Limits versus Elevation

This figure has been updated.

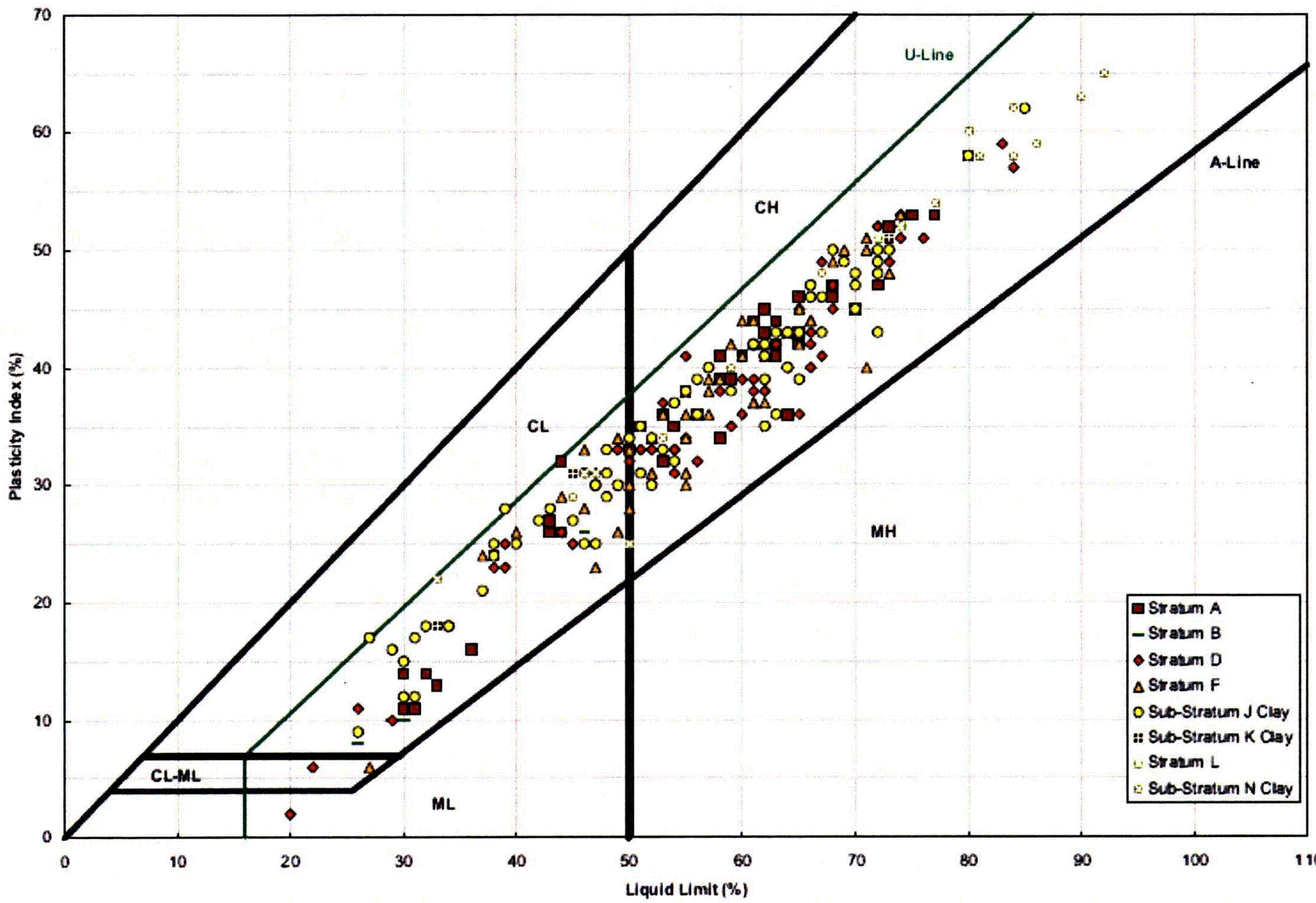


Figure 2.5S.4-21 Plasticity Chart

This figure has been updated.

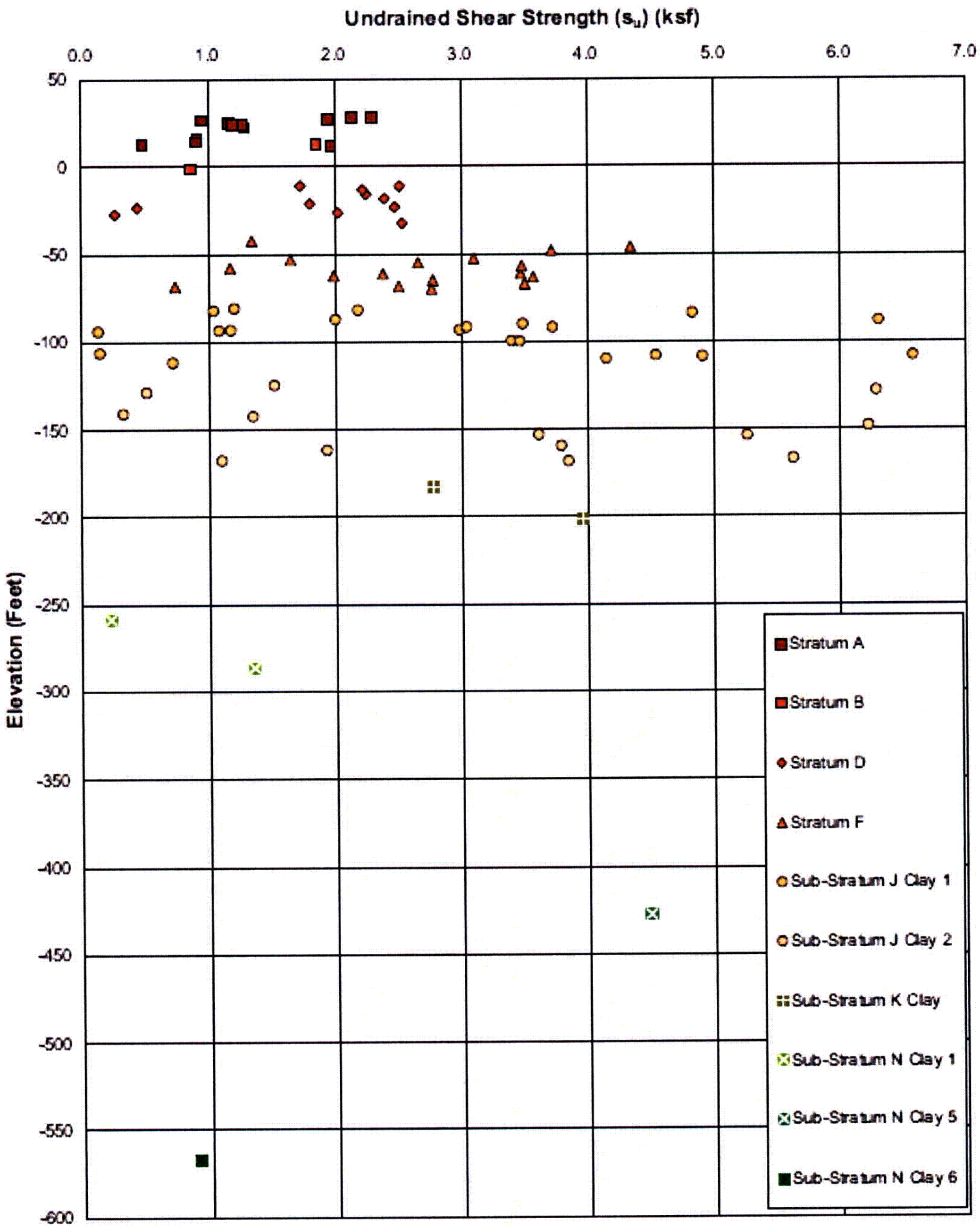
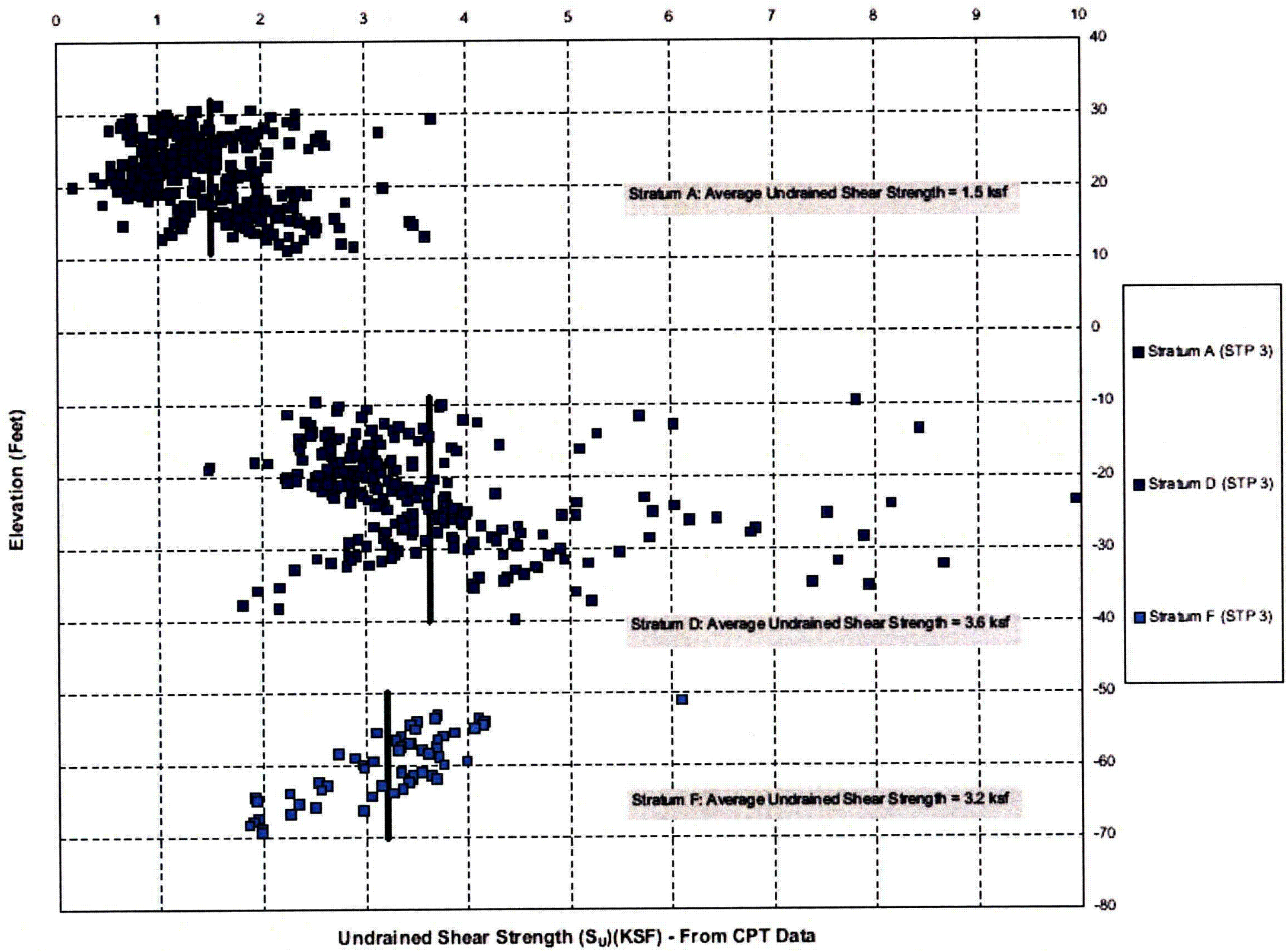


Figure 2.5S.4-22 Laboratory Test Results - Undrained Shear Strength (s_u) versus Elevation

Question 02.05.04-13, Supplement 1

This figure has been updated.



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)

This figure has been updated.

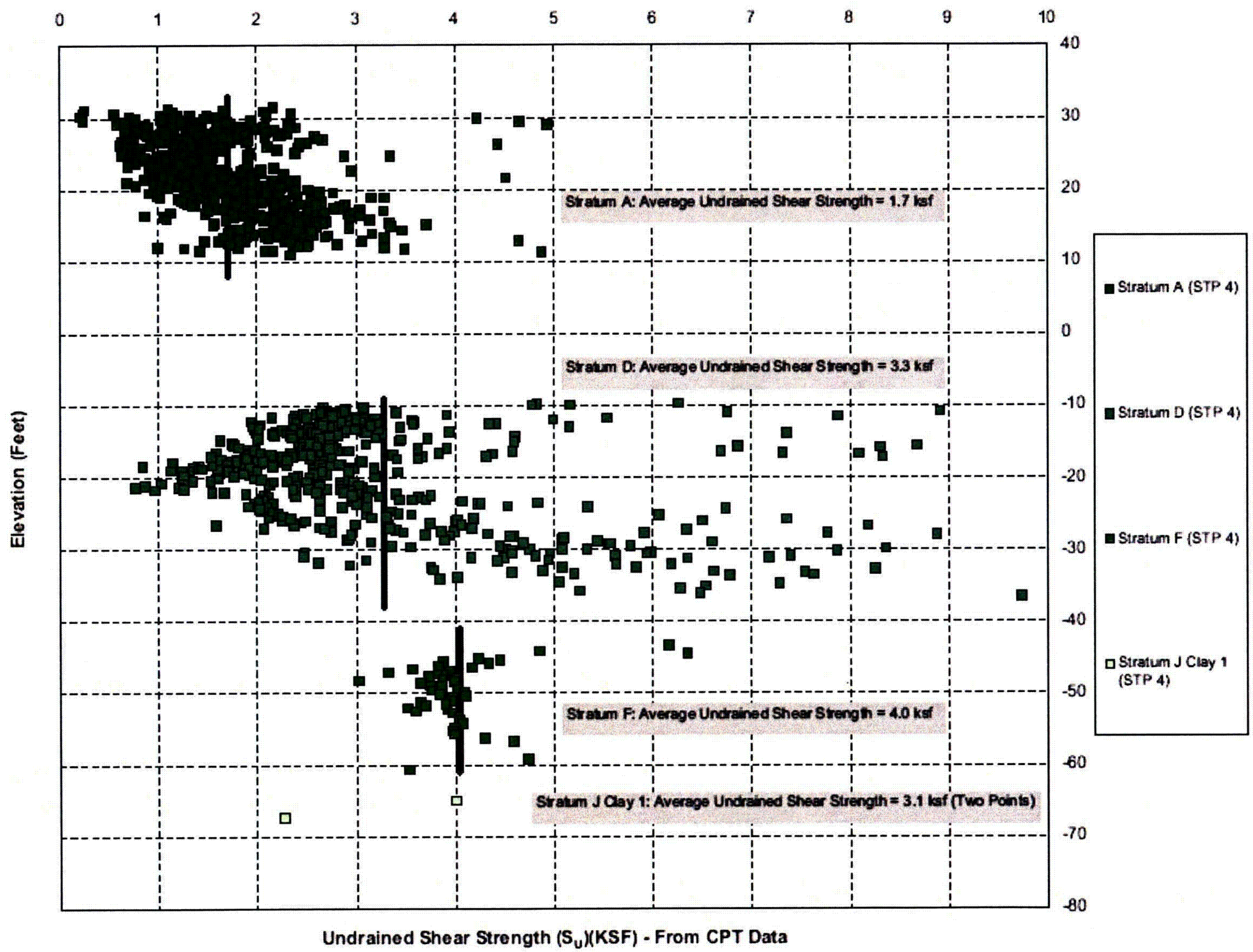


Figure 2.5S.4-24 Undrained Shear Strength (S_u) From CPT Data (STP 4)

Figure 2.5S.4-25 Not Used
(The data has been included in Figure 2.5S.4-26)

This figure has been updated.

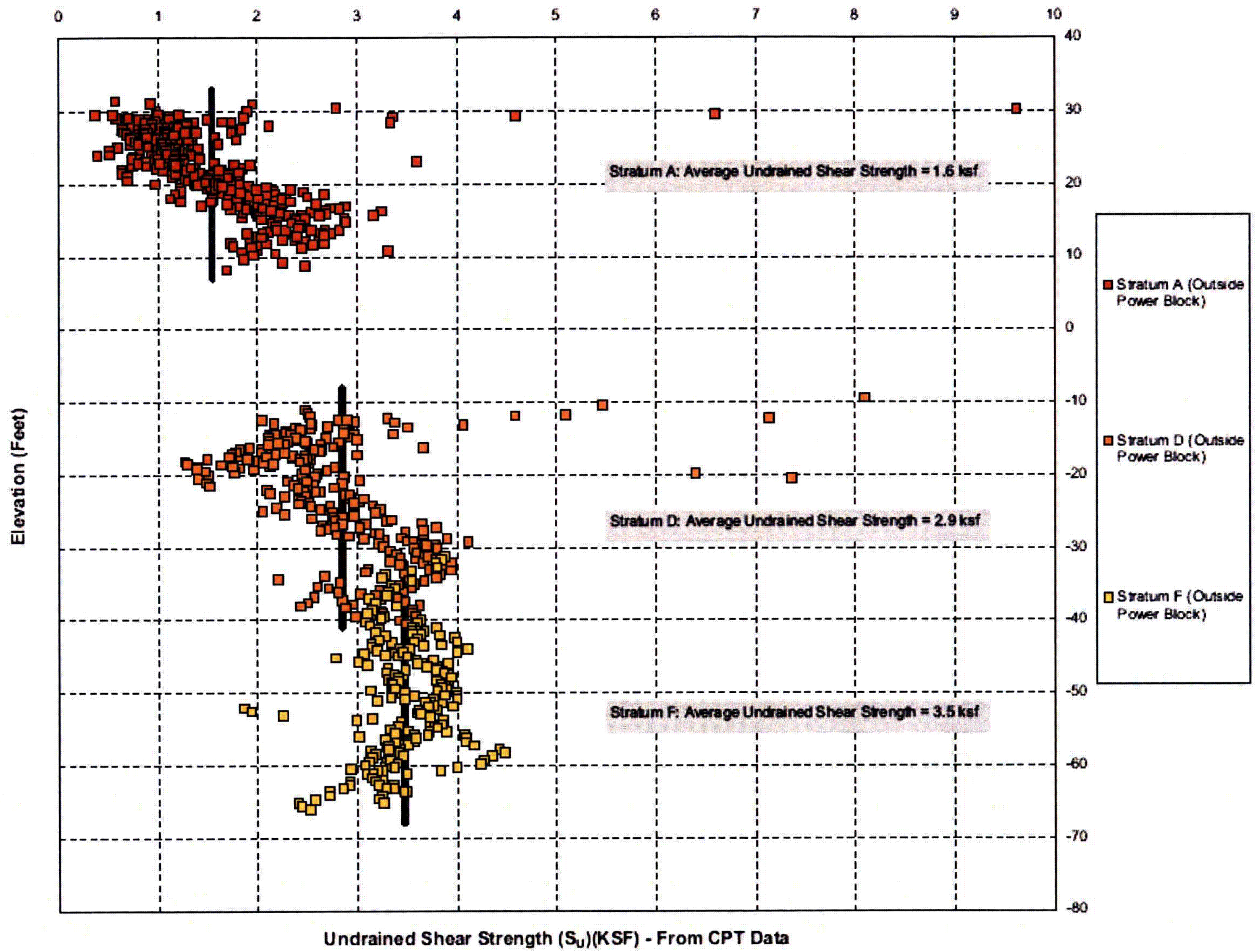


Figure 2.5S.4-26 Undrained Shear Strength (s_u) From CPT Data (Outside Power Block)

Question 02.05.04-13, Supplement 1

This figure has been updated.

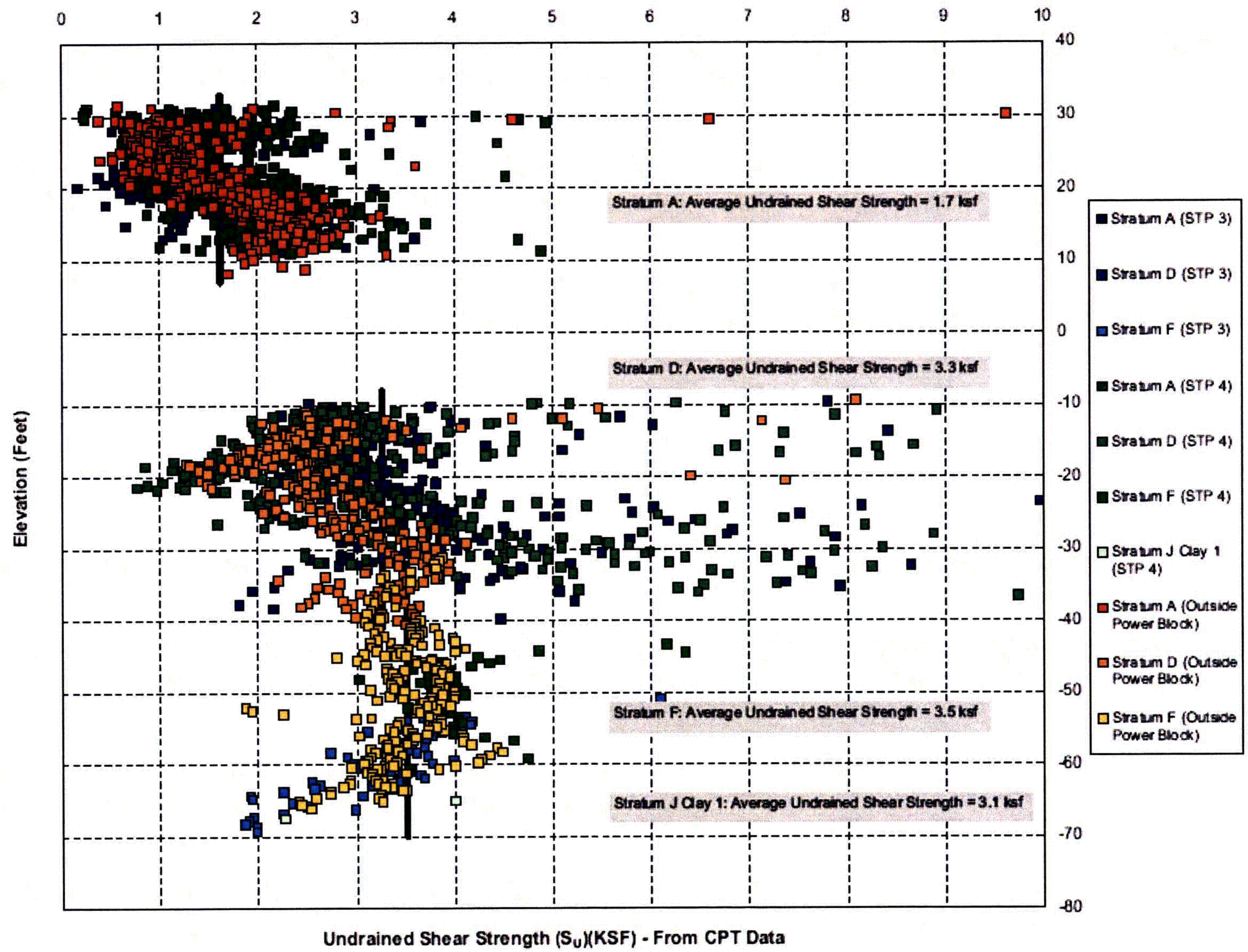


Figure 2.5S.4-27 Undrained Shear Strength (S_u) From CPT Data (Site-Wide)

This figure has been updated.

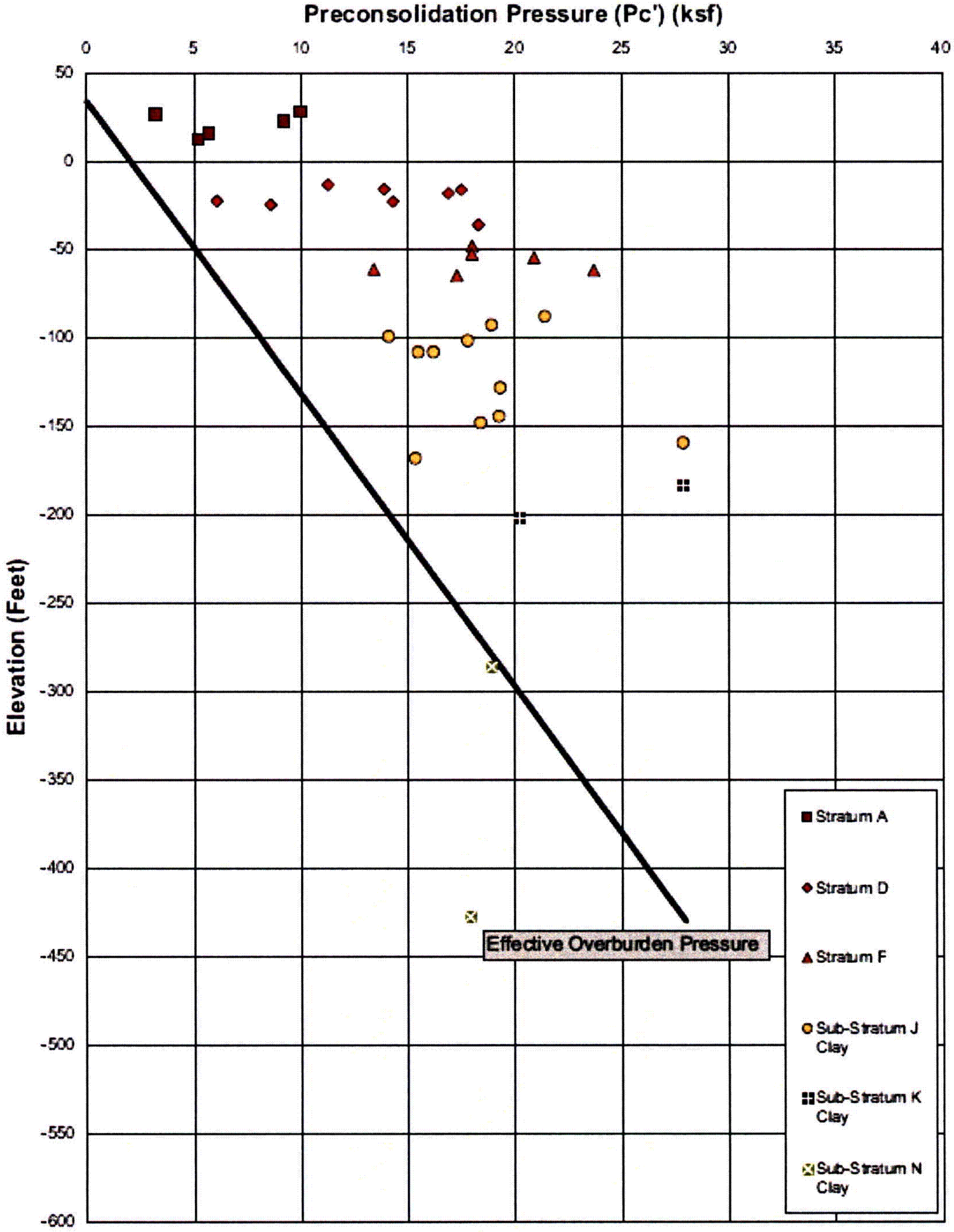
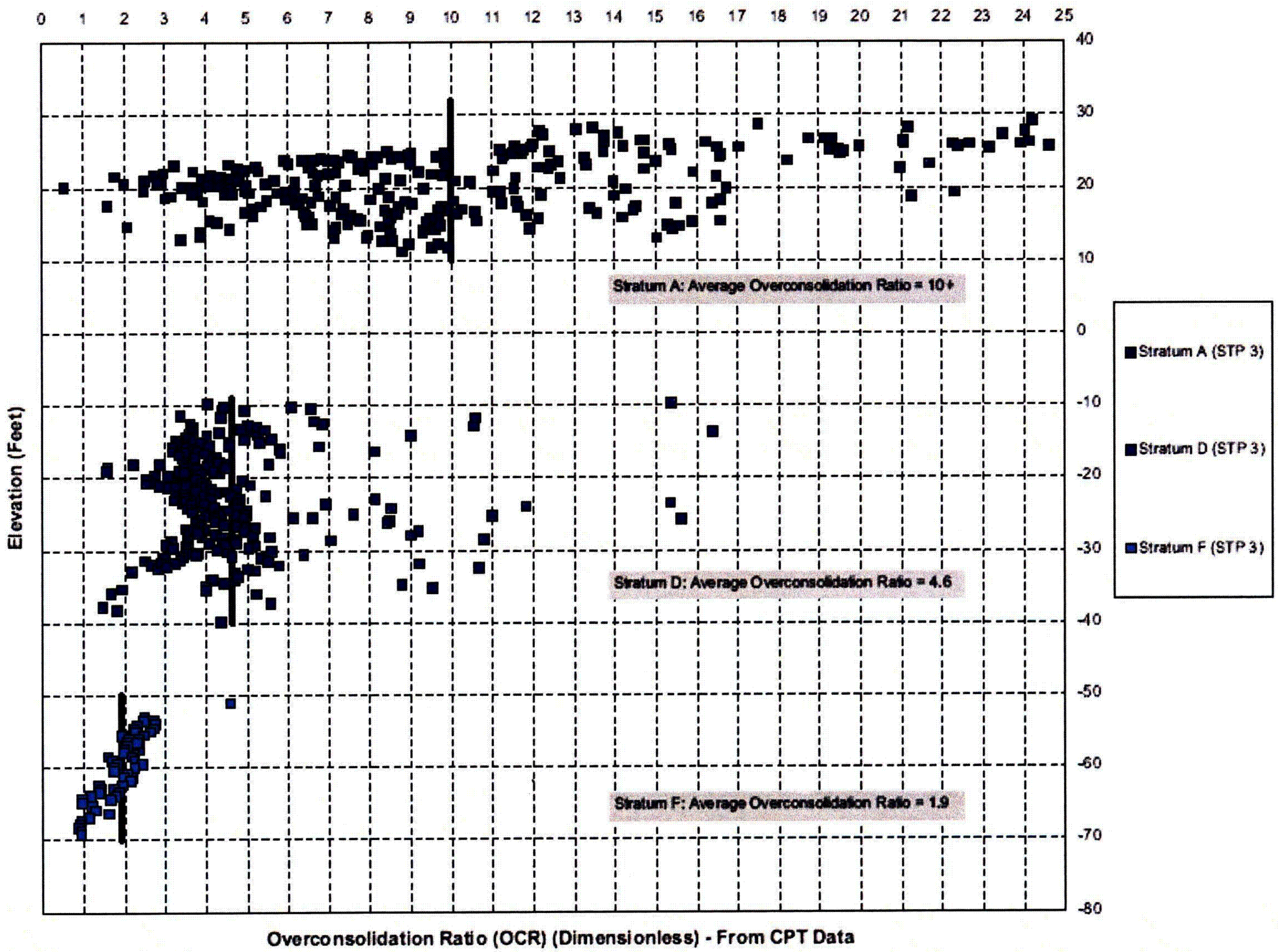


Figure 2.5S.4-28 Laboratory Test Results - Preconsolidation Pressure (P_c') versus Elevation

Question 02.05.04-13, Supplement 1

This figure has been updated.



NOTE: C-947 was not representative and therefore was not used as part of this data set.

Figure 2.5S.4-29 Overconsolidation Ratio (OCR) From CPT Data (STP 3)

This figure has been updated.

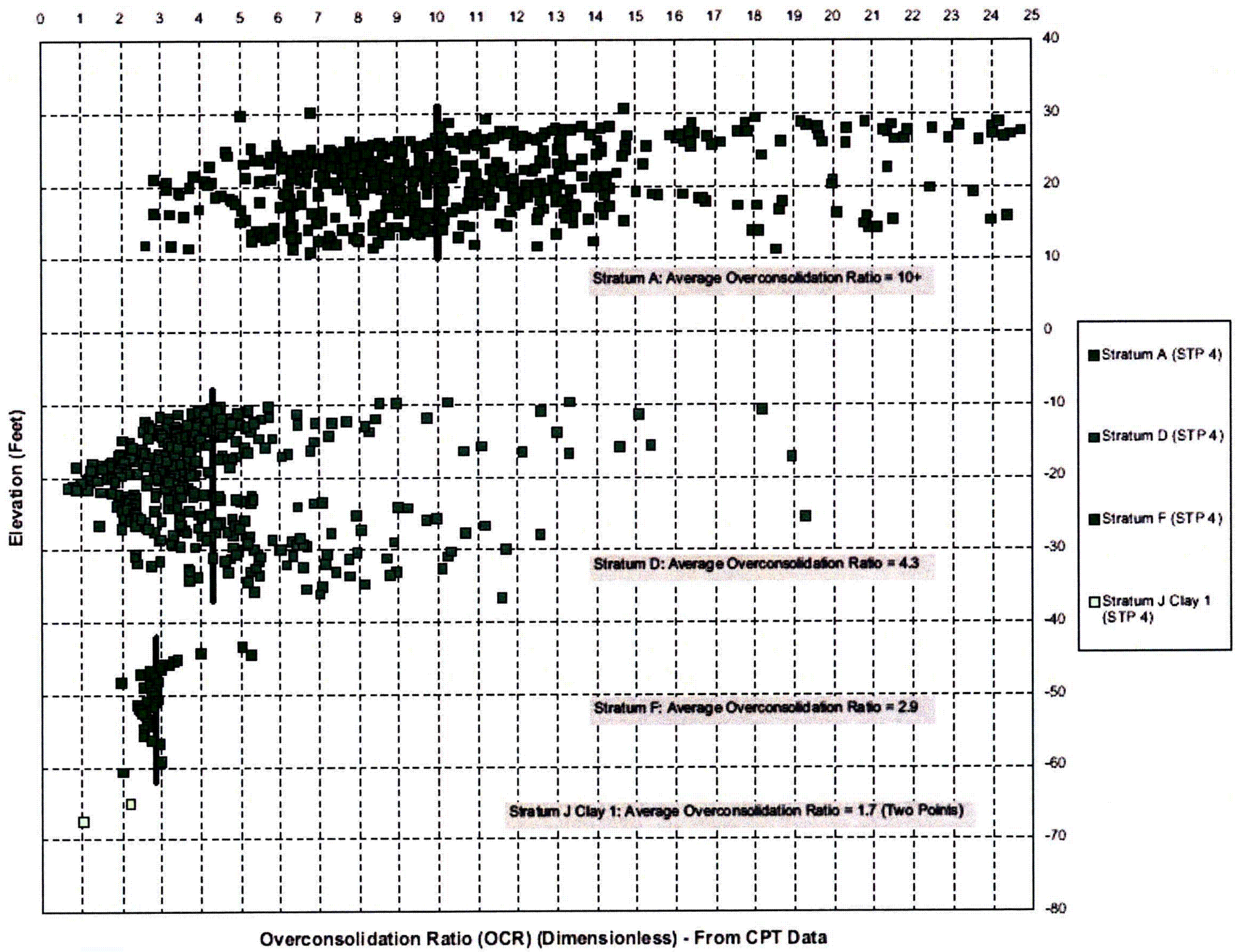


Figure 2.5S.4-30 Overconsolidation Ratio (OCR) From CPT Data (STP 4)

Figure 2.5S.4-31 Not Used
(The data has been included in Figure 2.5S.4-32)

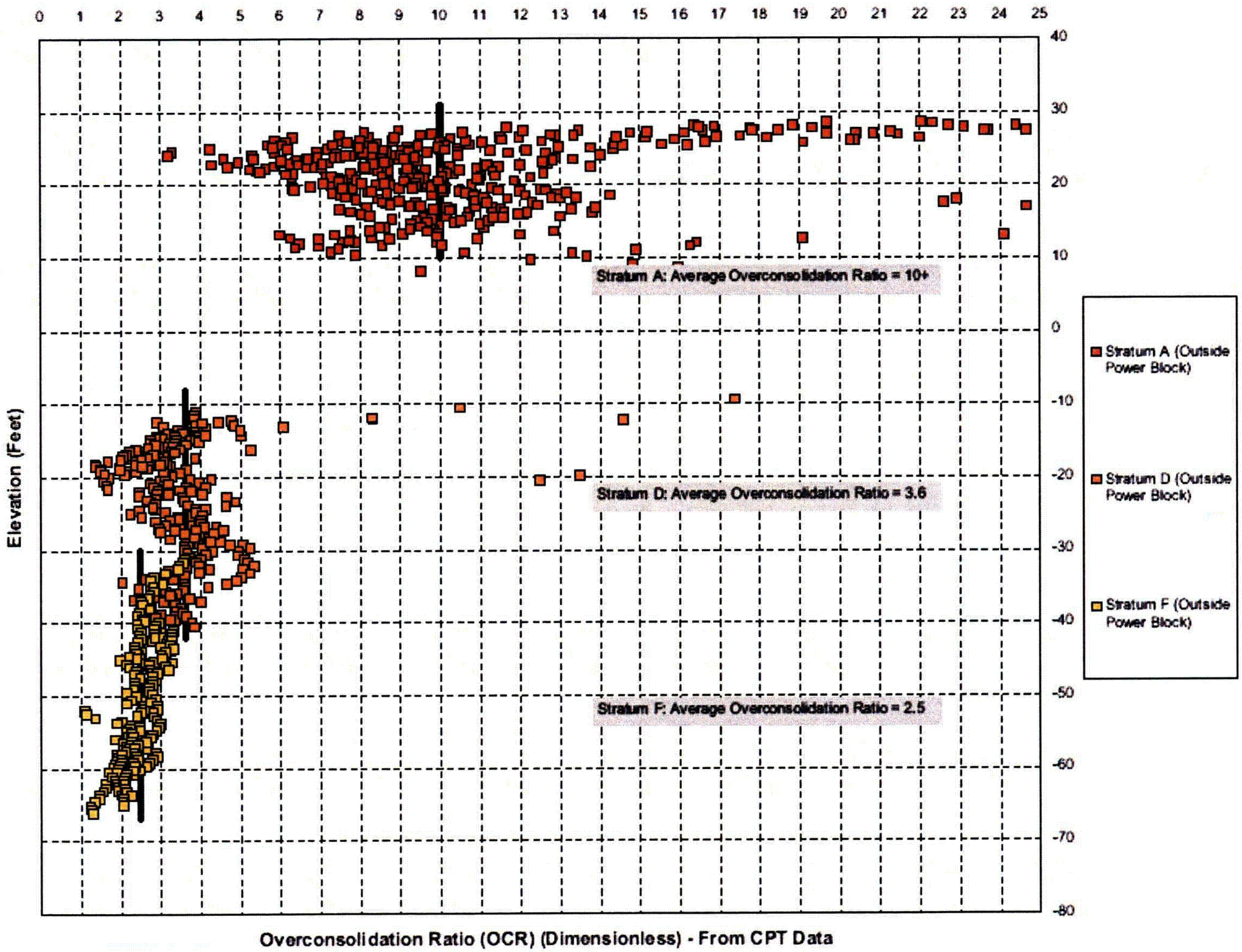


Figure 2.5S.4-32 Overconsolidation Ratio (OCR) From CPT Data (Outside Power Block)

This figure has been updated.

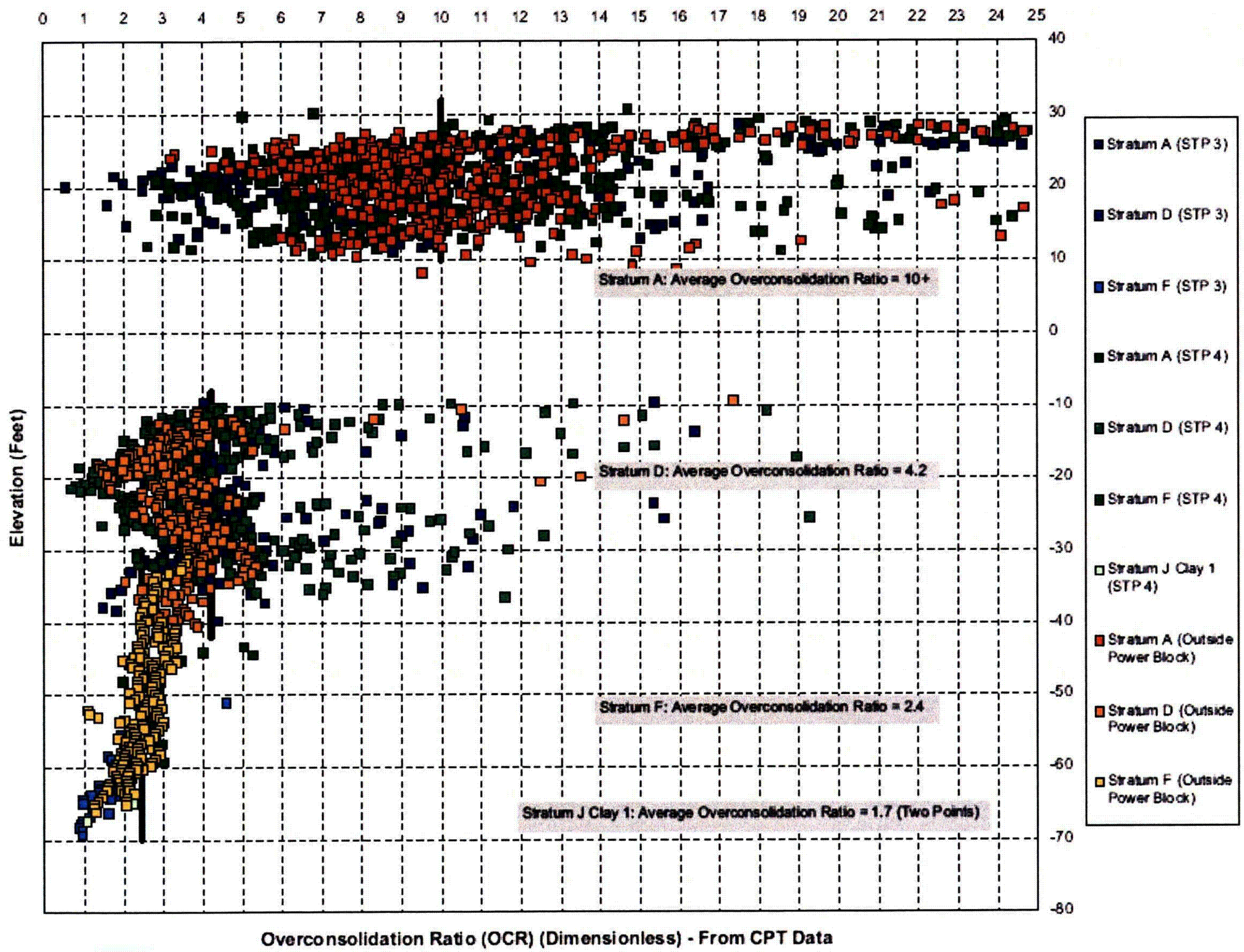
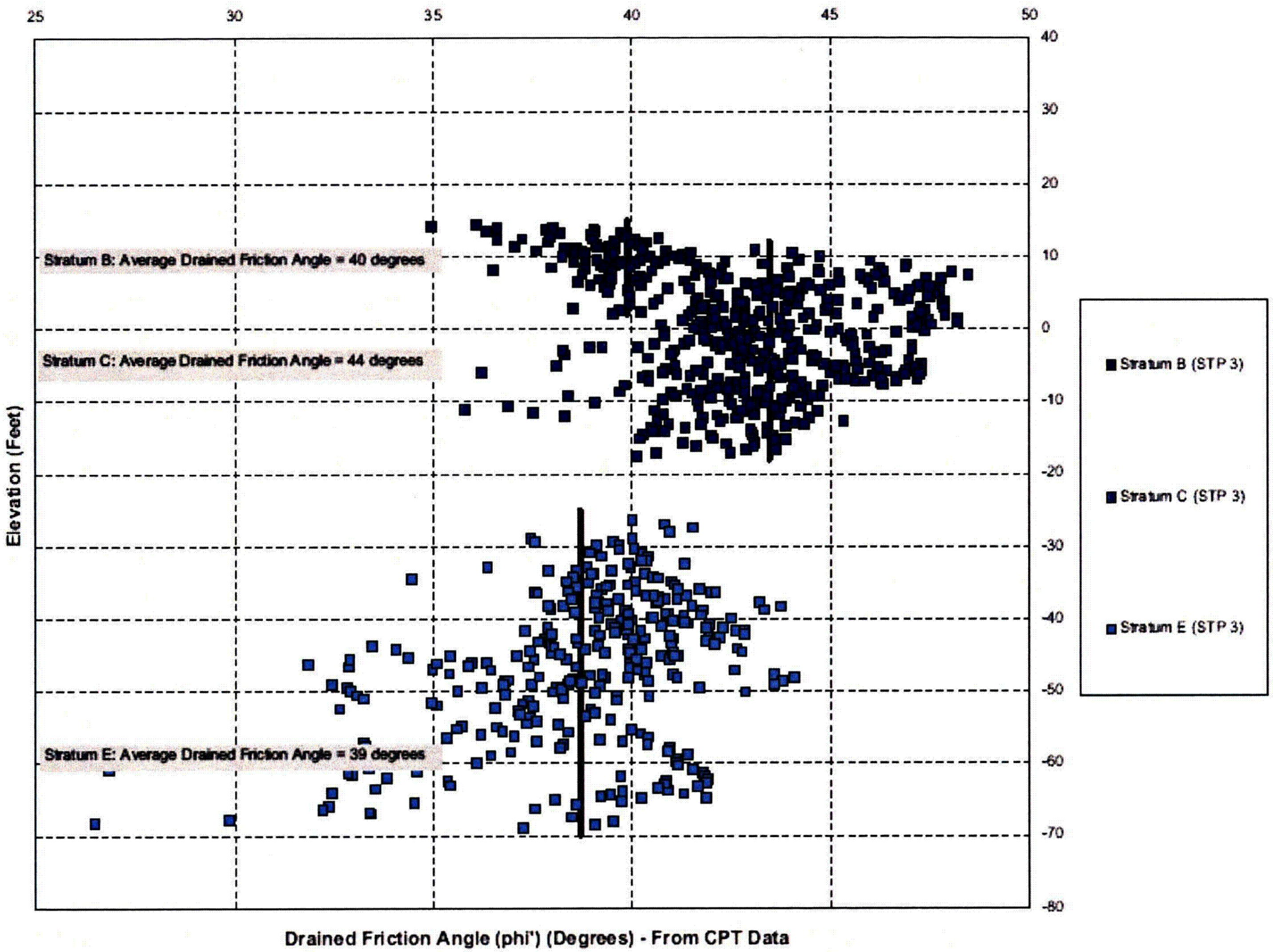


Figure 2.5S.4-33 Overconsolidation Ratio (OCR) From CPT Data (Site-Wide)

This figure has been updated.



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 (φ') From CPT Data (STP 3)

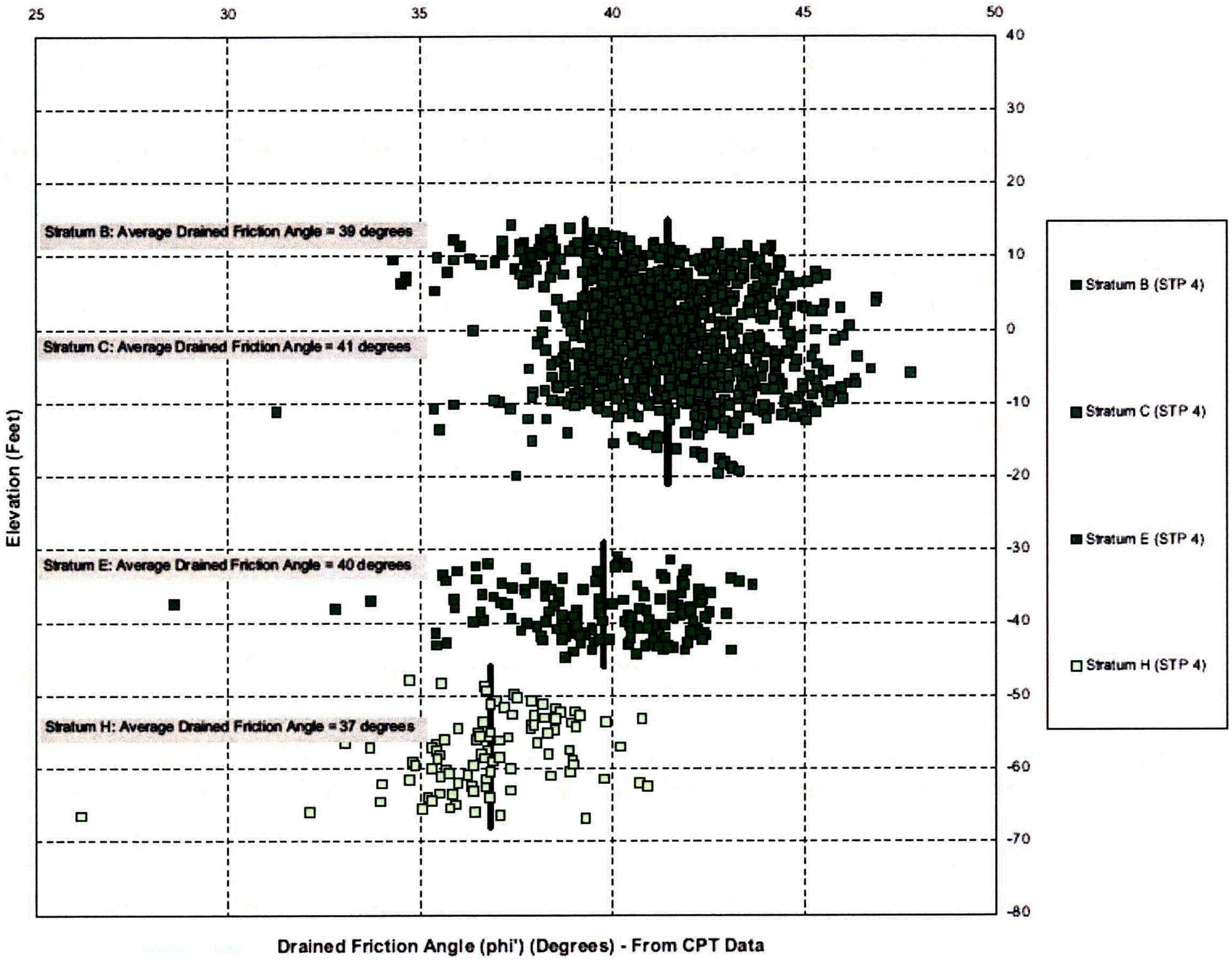


Figure 2.5S.4-35 Drained Friction Angle (ϕ') From CPT Data (STP 4)

Figure 2.5S.4-36 Not Used
(The data has been included in Figure 2.5S.4-37)

This figure has been updated.

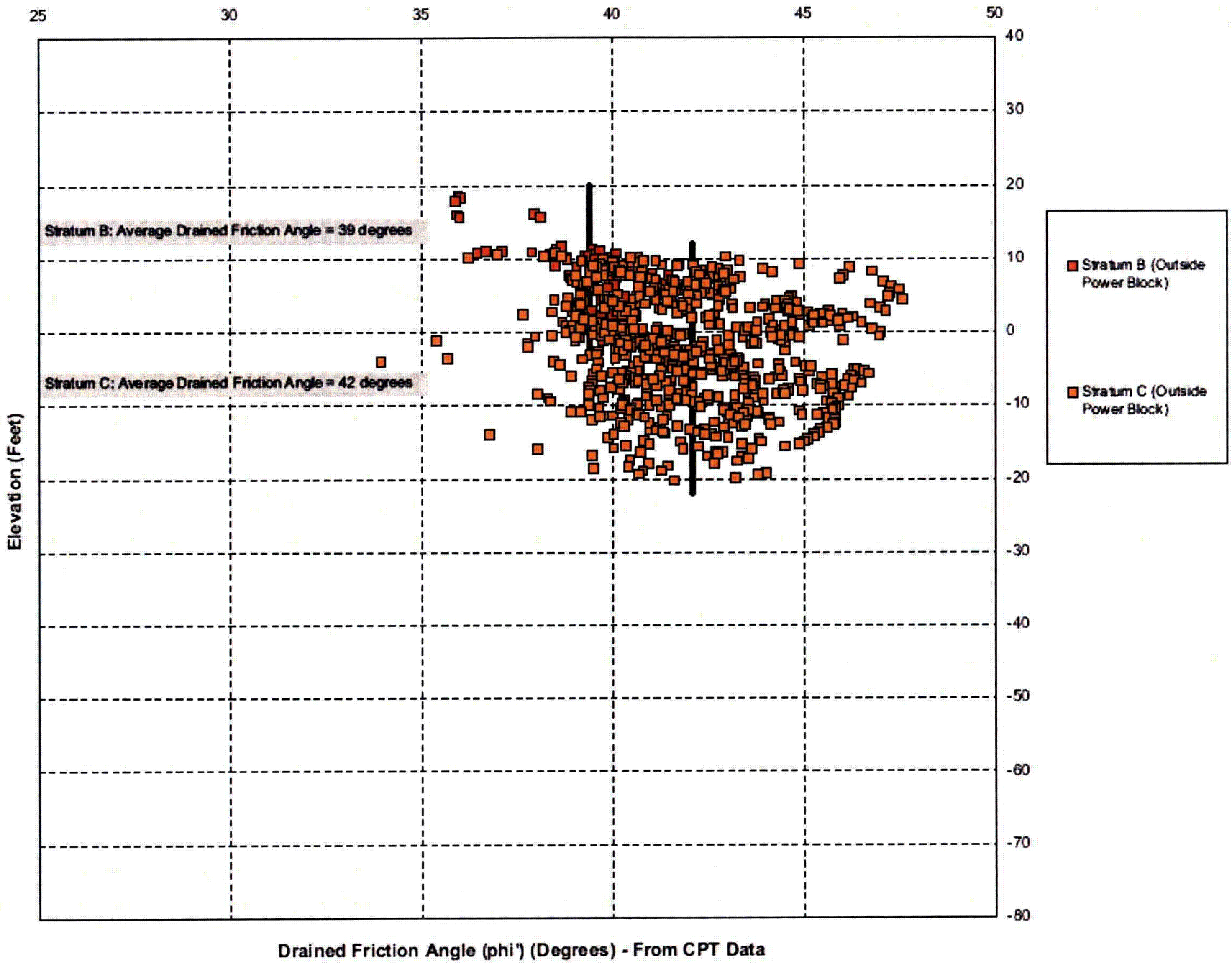


Figure 2.5S.4-37 Drained Friction Angle (ϕ') From CPT Data (Outside Power Block)

This figure has been updated.

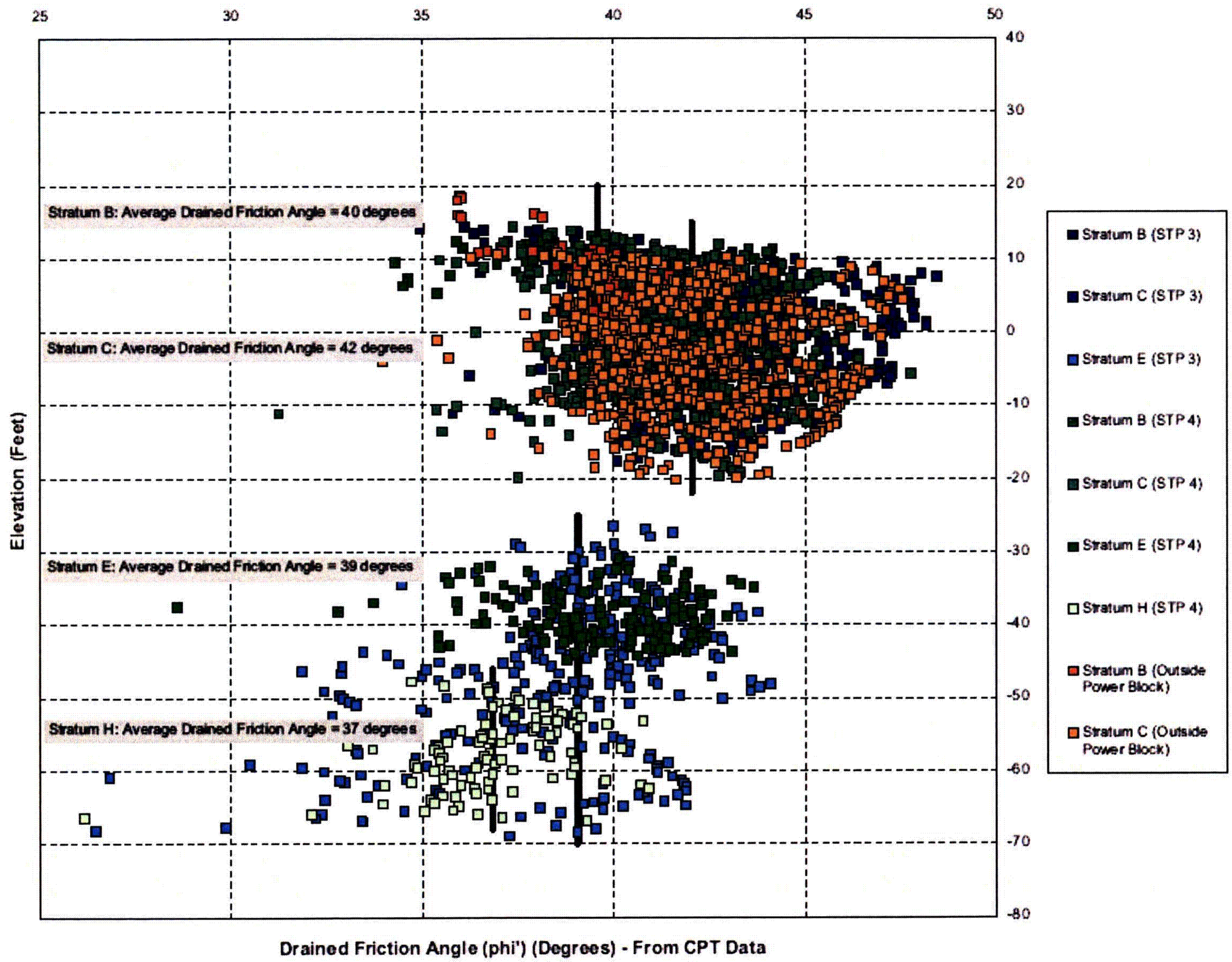


Figure 2.5S.4-38 Drained Friction Angle (ϕ') From CPT Data (Site-Wide)

COLA Tier 2 Section 2.5S.4.10 will be revised as shown:

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, ~~and the~~ 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. ~~(refer in part to Appendices 3H.1, and 3H.2.). Note that the estimated foundation design pressures given are assumed typical for these structures.~~ Given the position of the groundwater level ~~close to the ground surface~~ (once construction dewatering is terminated and the groundwater level recovers) and the foundation depth, buoyancy effects on the foundations ~~were~~ must be considered~~, leading to the effective foundation pressures also indicated below:~~

Structure [1]	Approximate Foundation Dimensions (feet)	Foundation El. [2] (feet)	Foundation Depth [2] (feet)	Estimated ~~Foundation Design~~ Pressure for Bearing Calculations (ksf)	~~Effective Foundation~~ Estimated Pressure for Settlement Calculations (ksf)
Reactor Buildings	187.7 by 197.5 ~~186 by 196~~	-50.3 {-60.3} ~~-51 {-61}~~	84.3 {94.3} ~~85 {95}~~	15.0	12.74 ~~10.2 [3]~~
Control Buildings	185.0 by 80.1 ~~79 by 184~~	-42.3 {-44.3} ~~-41~~	76.3 {78.3} ~~75~~	15.0	7.51 ~~10.8~~
UHS Basins	312.0 by 164.0 ~~312 by 164~~	4.0 {2.0} ~~4~~	30.0 {32.0} ~~30~~	8.9 ~~TBD~~	7.4 ~~TBD~~
RSW Pump Houses	170.0 by 94.0 ~~94 by 170~~	-32.0 {-34.0} ~~-32~~	66.0 {68.0} ~~66~~	6 ~~TBD~~	5.02 ~~TBD~~
RSW Tunnels	17.0 wide ~~17~~	-21.0 {-23.0} ~~-21~~	54.0 {56.0} ~~54~~	3 ~~TBD~~	2.49 ~~TBD~~
Diesel Generator Fuel Oil Storage Vaults	36.0 by 76.0	-5.0	39.0 {41.0}	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 ~~Els.~~ elevations and depths ~~and soil strata designations~~ shown in "{ }" symbols denote ~~the Els. and conditions at the~~ base of significant over-excavation ~~at the particular structure~~ with the over-excavation to be replaced with concrete fill. ~~(e.g., at the Reactor Buildings, Stratum F is over-excavated 10 feet below the underside of foundations, with over-excavation replaced by concrete fill.~~ At the 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~~.; and at the RSW Tunnels, Stratum B is over-excavated 4 feet below the underside of tunnel foundations, with over-excavation replaced by structural fill).~~

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

The subsurface conditions at STP 3 & 4 are described in detail in Subsection 2.5S.4.2. The geotechnical engineering parameters of the various soil strata are similarly described in Subsection 2.5S.4.2, and are summarized in Table 2.5S.4-16. These parameters were used as the bases for the analyses of foundations. ~~, in addition to the~~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~~-67~~-71 through 2.5S.4~~-70~~ 74C. Associated elevations and soil properties for these profiles are shown in Tables 2.5S.4-37A, 38A, 39A, 40A, 40C and 40E ~~through 2.5S.4-40A~~. For depths below El. -180 feet, strata boundary and soil property information was from the two deep borings (Borings B-305DH/DHA and B-405DH). 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~~, the~~ there was an upper groundwater level ~~was conservatively assumed~~ 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~~-54~~55):

$$\cancel{q_{ult} = c N_c \zeta_c + q N_q \zeta_q + 0.5 \gamma' B N_q \zeta_q}$$

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

If $\phi = 0$, use

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

where,
c ~~(s_u)~~ = undrained shear strength of the soil (s_u)
q = effective overburden pressure at the foundation base
~~γ'~~ 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)
~~ζ_c, ζ_q, and ζ_γ are shape factors~~

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

$$N_q = e^{(\pi \cdot \tan(\phi))}\left[\tan\left(45 + \frac{\phi}{2}\right)\right]^2$$ 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~~-54~~-55 as:

~~$\zeta_c = 1 + (B/L)(N_q/N_c)$~~ $S_c' = 0.2\left(\frac{B}{L}\right)$ where $\phi = 0$, 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

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

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

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

For ~~square or circular~~rectangular foundations, the ~~shape~~ depth factors were given by Reference 2.5S~~.454~~.4-55 as:

~~$\zeta_c = 1 + (N_q/N_c)$~~ $d_c' = 0.4k$ for $\phi = 0$, else Equation 2.5S.4-19

$d_c = 1 + 0.4k$ Equation 2.5S.4-19A

where,
$k = \frac{D_f}{B}$ if $\frac{D_f}{B} \leq 1$; and

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

D_f = the depth of soil beside the foundation.

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

$\cancel{\zeta_g = 0.6}\ d_\gamma = 1.00$ 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:

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

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

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

~~where, FOS is the factor of safety.~~
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:

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

where,
Q_{ult} = (q_{ult})(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~~s~~ 2.5S.4-55~~and 2.5S.4-56,~~ and to use the formulation in Reference 2.5S.4-~~54~~ 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-~~71~~ 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 ~~using~~ determining the weighted average of cohesion, *c* (s_u), and friction angle, as follows:

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

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

where,

c_i = cohesion of layer i

ϕ_i = friction angle of layer i

H_i = thickness of layer i within the effective shear depth H'

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

Equations 2.5S.4-23, ~~and~~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 ~~average~~ material properties derived for each foundation are shown in Tables 2.5S.4-37B, 38B, 39B, 40B, 40D and 40F ~~through 2.5S.40B~~. 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.

~~Because different soils (e.g., clay, silt, and sand) were sometimes found at the same elevation across foundations, both soils were considered in the selection of average material properties. Where clay soils were present, the soil with the lowest cohesion, c~~

~~(s_u), was used. Similarly, when silt or sand was present, the soil with the lowest friction angle, φ', was used. For conservatism, if structural fill was present along with other soil types, the properties of the stronger structural fill were ignored in estimating bearing capacity. Similarly, t~~The properties of the stronger concrete fill below the ~~Reactor Building~~ 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-~~71~~75. With a 10-foot-thickness of concrete fill, then, the pressure from each Reactor Building was distributed on an area having B = ~~186~~ 187.7 feet + 20 feet = ~~206~~ 207.7 feet, and L = ~~196~~ 197.5 feet + 20 feet = ~~216~~ 217.5 feet. Thus, the effective foundation pressure at the base of the concrete fill for each Reactor Building was estimated as {[(~~10.2~~15 ksf) (~~186~~ 187.7 feet) (~~196~~ 197.7 feet)] / ~~=~~ [(~~206~~ 207.7 feet) (~~216~~ 217.5 feet)]} + (0.150 ~~ksf~~kcf ~~– 0.0624 ksf~~) (10 feet) = ~~9.2~~ 13.82 ksf, using a unit weight of concrete fill of 0.150 ~~ksf~~ kcf and not accounting for soil backfill above the exposed 10-foot projections of the concrete fill.

Foundation bearing capacities were estimated using the ~~average~~ material properties in Tables 2.5S.4-37B, 38B, 39B, 40B, 40D and 40F ~~through Table 2.5S.4-40B~~ and using Equations 2.5S.4-15 through 2.5S.4-21. A summary of the ~~average~~ material parameters, as well as the derived bearing capacity factors, are shown in Tables 2.5S.4-41~~a~~A and 2.5S.4-41~~b~~B. Estimated ultimate bearing capacities and factors of safety are ~~also~~ shown in Table 2.5S.4-41~~b~~B. 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 ~~allowable bearing capacity (using FOS=3) is higher than the effective foundation pressure for all structures, except for the STP 3 Reactor Building foundation, clay (Stratum F) subgrade case, which is discussed in more detail, below. The FOS values for all other subgrade cases ranged from approximately 5 (clay subgrade cases) to over 70 (sand subgrade cases), all indicative of a sufficient FOS against foundation bearing failure.~~

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

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).

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:

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

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
~~E_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~~ and was calculated by Poulos and Davis (Reference 2.5S.4-57)~~. 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.

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

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

where,

$\Delta\sigma_z$ = calculated pressure at depth z

~~p~~ q = applied foundation pressure

~~l = length of the foundation~~

~~b = width of the foundation~~

z = depth below the foundation ~~at~~ from which the pressure is calculated

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

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

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

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

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

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

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

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

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

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

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

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

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

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

~~where,~~

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

In applying Equation 2.5S.4-25 ~~to layers with more than one soil type spanning across a particular foundation, the elastic moduli values, E, of the different soil types were compared and the lower value was selected. T~~the E-values for the various soil strata are shown in Tables 2.5S.4-37A, 38A, 39A, 40A, 40C and 40E ~~through 2.5S.4-40A~~. The μ_{di} values for the strata are given in Table 2.5S.4-16. ~~Note, however, that for the deepest considered stratum (Stratum N) because of its significant thickness, a composite E for the stratum was taken as a weighted average of the E of Sub-stratum N Clay, and the E of Sub-stratum N Sand, with weighting based on sub-strata thicknesses to total stratum thickness (i.e., [(4500 ksf) (228 feet) + (2100 ksf) (93.5 feet)]/(228 feet +93.5 feet) = 3802 ksf). Because Stratum N was found relatively deep below the foundations of major structures, the strain level in it is expected to be low, justifying the use of the relatively high composite E value, calculated above.~~ Also, in estimating elastic settlements, the compression of concrete fill below the Reactor Buildings was ignored due to its relative incompressibility in the range of loads being considered.

Spreadsheets were used for settlement calculations. 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~~ the calculations were extended ~~to a depth where the increase in vertical stress (Δp) due to the applied foundation pressure was less than or equal to 10% of the applied foundation pressure. Also, using a 1:2 H/V pressure distribution shown in Figure 2.5S.4-71, the applied vertical stresses~~

~~below foundations were compared to the preconsolidation pressures (P_c') of the various soil strata. Where more than one soil type was present under a particular foundation, clay soils were selected over sand soils to represent the conditions of the layer, as discussed previously. Results showed that strata preconsolidation pressures typically exceeded the applied vertical stresses at mid-point of each layer~~ 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-~~27~~29, the ~~simplified~~ subsurface profiles shown in Figures 2.5S.4-~~67~~ 71 through 2.5S.4-~~69~~74C, and the material parameters shown in Tables 2.5S.4-37B, 38B, 39B, 40B, 40D and 40F ~~though 2.5S.4-40B~~. Settlement estimates, which included the total settlement at the center, corners and the middles of the edges of foundations~~, as well as their average values,~~ are shown in Table 2.5S.4-~~41~~42. Total settlements calculated at the centers of foundations for the Reactor Buildings ~~and the Control Buildings~~ were estimated in the range of approximately ~~7.5~~10.1 to ~~9~~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-excavated to El. ~~-61~~ -60.3 feet) was estimated, with ~~the~~ calculated values in the range of approximately ~~3~~ 3.5 inches ~~to 5.5~~ 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~~/heave~~ at selected foundation excavations is to be monitored during construction.

~~As a guideline, tolerable total and differential settlements for mat foundations on clay subgrades are typically reported in the range of 3 inches and 1.5 inches, respectively. Note that tolerable total settlements as high as 5 inches have also been suggested for mat foundations (Reference 2.5S.4-55).~~ 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, 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.

~~Higher total~~ 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.

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	~~3.3~~ 1.5 to 1.8	~~1/350~~ 1/600 to 1/750
Control Buildings	~~2.5 to 3.2~~ 1.8 to 2.0	~~1/250 to 1/320~~ 1/400 to 1/450
UHS Basins~~/RSW Pump Houses~~	~~TBD~~ 2.2 to 2.3	~~TBD~~ 1/650 to 1/700
RSW Pump Houses	0.5	1/1700 to 1/1750
RSW Tunnels	~~TBD~~ 5.0	~~TBD~~ 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 ~~2.5~~ 0.4 inches to ~~3.5~~ 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 ~~generally~~ within the acceptable limit of 1/300. ~~However,~~ 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 ~~7.5 inches to 9 inches of~~ total settlement for the referenced foundations, and the ~~apparently high~~ angular distortion/tilt values, actual angular distortion/tilt values are much less 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 ~~1/700~~1/1200 to 1/1500 for the Reactor Buildings, and ~~1/500 to 1/640~~ 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 ~~generally~~ well within the stricter criterion for the special case of foundations supporting sensitive machinery. Note, more significantly, that settlement estimates were based on the assumption of flexible mat foundations, not including the effects that thick, highly-reinforced concrete mat foundations have in mitigating differential settlements. To verify that foundations perform according to estimates, and to provide an ability to make corrections if needed, major structure foundations are monitored for movement during and after construction.

In general, the estimated foundation settlements are larger than those calculated for STP 1 & 2, as discussed in Subsection 2.5S.4.10.1. Given that subsurface conditions at STP 3 & 4 are comparable, the differences in calculated settlements are largely due to differences in ~~net~~ 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 ~~36,460~~37,070 square feet, or approximately ~~70%~~73% larger than the plan area of an individual STP 1 & 2 structure. In addition, the ~~net~~ applied loading of each Reactor Containment Building at STP 1 & 2 was ~~2.0~~ about 9.4 ksf, while the effective foundation pressure of each Reactor Building at STP 3 & 4 is ~~9.2~~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, ~~albeit~~ 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.

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):

$p_{AS} = K_{AS} \cdot \gamma \cdot z$ 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:

$K_{AS} = \tan^2 (45 - \phi'/2)$ 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:

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

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.

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

$p_{hydro} = (7/8)\ K_h\ \gamma_w\ [H_w\ z_w]^{1/2}$ 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:

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

where,

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

2.5S.4.10.5.4 Sample Earth Pressure Diagrams

Using the relationships outlined and the assumed structural fill properties, above, sample earth pressures were estimated. Sample earth pressure diagrams are provided on Figures 2.5S.4-~~72~~ 76 and 2.5S.4-~~73~~ 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).