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16. Abstract

The objective of this project was to develop an improved correlation between Texas Cone Penetrometer (TCP) blow count and undrained shear strength for soft, clay soils in the upper approximately 30 feet of the ground. Subsurface explorations were carried out by Tolunay-Wong Engineers, Inc. (TWEI) at six sites in the Texas Gulf Coast region where soft soils were expected to exist to obtain the data necessary to establish the improved correlation. A series of borings was made at each site with TCP tests, conventional ("Dutch") piezocone penetration tests, thin-walled tube sampling, and vane shear tests. Laboratory testing was subsequently performed at The University of Texas at Austin on the thin-walled tube samples obtained in the field. The results of these field and laboratory tests were used to develop average, lower- and upper-bound undrained shear strength profiles for each of the sites (Varathungarajan, 2008). The data were used to evaluate existing correlations between TCP blow count and undrained shear strength as well as to establish an improved correlation. The following improved correlation was proposed:

$$s_u = 300 + 60N$$

where s_u is undrained shear strength in lbs. per square foot (psf) and *N* is the TCP blow count.

To evaluate the improved correlation, bearing capacity analyses were performed using the computer software UTEXAS4 (Wright, 1999) to compute factors of safety for undrained strength profiles developed using the improved correlation as well as for the undrained strength profiles developed by Varathungarajan (2008). Factors of safety computed using undrained shear strengths based on the improved correlation generally showed good agreement with the factors of safety determined using the strength profiles developed by Varathungarajan (2008).

The improved correlation is recommended over the existing correlations and is intended primarily for soft, shallow (less than 30 feet) clays with undrained shear strengths of 1200 psf (approximately 0.47 atmospheres) or less (TCP blow counts of 15 or less). Accordingly, this correlation should be used cautiously. Significant benefits can also be realized by performing more extensive field and laboratory testing, rather than relying on simple, approximate TCP correlations.

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Improved Correlation between Texas Cone Penetrometer Blow Count and Undrained Shear Strength of Soft Clays

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Chapter 1. Introduction

The Texas Department of Transportation (TxDOT) routinely performs Texas Cone Penetrometer tests as part of subsurface site investigations. The data obtained from these tests are often the only data available to TxDOT for estimating undrained shear strengths of the soil for preliminary design. Accordingly, correlations between Texas Cone Penetrometer blow count and undrained shear strength are frequently used to estimate undrained shear strength. A need exists for estimating undrained shear strengths of softer soils at shallow depths (30 feet or less).

The objective of this project was to develop an improved correlation between Texas Cone Penetrometer blow count and undrained shear strength for soft, clay soils in the upper approximately thirty feet of the ground. The Texas Cone Penetrometer test is discussed in detail in Chapter 2 and the existing correlation between Texas Cone Penetrometer blow count and undrained shear strength are described and evaluated in Chapter 3.

Subsurface explorations were carried out by Tolunay-Wong Engineers, Inc. (TWEI) at six sites in the Texas Gulf Coast region where soft soils were expected to exist. A series of borings was made at each site with Texas Cone Penetrometer tests, conventional ("Dutch") piezocone penetration tests, thin-walled tube sampling, and vane shear tests. In some cases, depending upon the strength of the soils encountered and availability of field testing equipment, only some of the in-situ tests were performed. In addition to the field testing, laboratory testing was performed at The University of Texas at Austin on thin-walled tube samples collected in the field. Laboratory testing included index property tests, unconsolidated-undrained (UU) triaxial compression tests, isotropically consolidated-undrained triaxial compression tests (ICU), and one-dimensional consolidation tests. Chapter 4 summarizes the field and laboratory testing and provides a stratigraphic profile for each site.

The field and laboratory data were analyzed for each site by Varathungarajan (2008) and representative undrained shear strength profiles were developed (Appendix A). In Chapter 5, these profiles are used to evaluate the existing correlations between Texas Cone Penetrometer blow count and undrained shear strength as well as to develop an improved correlation for soft soils.

Several series of bearing capacity analyses were performed to provide a basis for evaluating the improved correlation presented in Chapter 5. These analyses were carried out assuming a uniform load of varying width and magnitude applied at the ground surface. The goal was to model a typical shallow foundation. The computer software UTEXAS4 (Wright 1999) was utilized to carry out the analyses. Bearing capacity analyses were performed for the undrained shear strength profiles established from laboratory, field vane, and piezocone penetration tests by Varathungarajan (2008) as well as for the undrained shear strength profiles determined from the Texas Cone Penetrometer blow counts using the improved correlation developed in this study. These analyses are presented in Chapter 6, where factors of safety are compared to estimate the reliability of the improved correlation. Conclusions and recommendations are presented in Chapter 7.

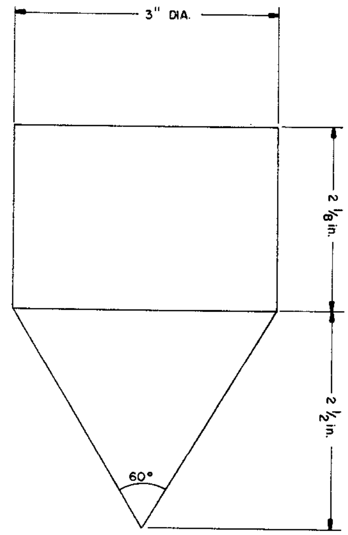
Chapter 2. Texas Cone Penetrometer (TCP) Test

A variety of in-situ tests are used in geotechnical engineering to estimate the undrained shear strength of saturated clays. One of the primary in-situ tests utilized by the Texas Department of Transportation (TxDOT) is the Texas Cone Penetrometer (TCP) test.

2.1 Apparatus and Procedure

The TCP test is a dynamic penetration test performed to determine in-situ properties of the subsurface soil. The test is performed in accordance with the TxDOT Test Procedure TEX-132-E. The TCP apparatus consists of a 3-inch diameter cone (see Figure 2.1) attached to a 1 and 3/4-inch O.D. drilling rod with a 3/16 inch wall thickness. A 170-pound hammer is positioned at the top of the drilling rod and allowed to fall freely a distance of 2 feet.

When performing the TCP test, a hole is advanced to the desired depth using an appropriate drilling method, which may vary depending upon the subsurface conditions. The cone is then seated into the undisturbed soil by driving the cone 12 blows or 12 inches, whichever is achieved first, into the soil. Once the cone is seated it is driven an additional 12 inches or 100 blows, whichever is reached first. In cases where the cone is driven the full 12 inches, the number of blows required to drive each 6 inches of penetration up to 12 inches is recorded. The total number of blows required for the two 6-inch increments are then recorded as the TCP blow count, denoted as N_{TCP}. In cases where the cone is unable to be driven the full 12 inches, the penetration is recorded after every 50 blows up to 100 blows. In the event that the cone penetrates the soil at least 12 inches under its own weight without driving or travels a distance greater than 12 inches after 1 blow, the N_{TCP} value is recorded as “weight of hammer (WOH).” In typical practice the TCP test is performed at 5-foot to 10-foot intervals and where a significant change in soil is detected.



FIG–1 DETAILS OF THD CONE PENETROMETER
AFTER VIJAYVERGIYA , HUDSON AND REESE (32)
(1.0 in = 25.4 mm)

Figure 2.1: Details of the Texas Cone Penetrometer after Vijayvergiya, Hudson and Reese (1969)

2.2 Minimum Strength Required to Support the Texas Cone Penetrometer (TCP) Cone

For the present study, a bearing capacity analysis was performed to estimate the minimum undrained shear strength of saturated clays required to support the TCP cone at a given depth below the ground surface. This is believed to represent the maximum possible undrained shear strength of the soil when the blow count is reported as "weight of hammer." Classical bearing capacity theory relates bearing capacity and undrained shear strength by an equation of the form

$$q_{ult} = N s_u + \sigma_{v0} \qquad (2.1)$$

where q_{ult} is ultimate bearing capacity, N is a bearing capacity factor, s_u is undrained shear strength, and σ_{v0} is total overburden stress prior to drilling. The total overburden stress is determined as follows,

$$\sigma_{v0} = D\gamma_s \qquad (2.2)$$

where D is the depth which the TCP test is being performed and γ_s is the total unit weight of the overlying soil.

There have been numerous studies to determine the bearing capacity factor, N, in Equation 2.1. Most of these have been for either spread footings or deep foundations. None relate directly to a cone such as the one used in the Texas Cone Penetrometer test. However, studies have been performed for the Standard ("Dutch") Cone Penetrometer (CPT) test and literature is available that discusses an empirical bearing capacity factor (also known as the cone factor) that is used for this test. The CPT test uses a smaller cone than the TCP test, and the cone is pushed rather than driven with a hammer. Lunne et al. (1997) reported that the bearing capacity factor used in the CPT test generally ranges from 10 to 20. An empirical bearing capacity factor of 15 was used for the present analysis.

The stress imposed by the cone can be determined from the weight and dimension of the equipment used in performing the Texas Cone Penetrometer test. Pertinent information is shown in Table 2.1. The depth at which the TCP test is performed will also influence the imposed stress because of the length and weight of the drilling rod.

Table 2.1: Weight and Dimension of TCP Test Equipment

Cone Weight, W_C (lbs)	7
Cone Area, A_C (ft^2)	0.049
Drilling Rod Weight, W_{DR} (lbs/ft)	4
Drilling Rod Length, L_{DR} (ft)	Varies
Drilling Rod Area, A_{DR} (ft^2)	0.006
Hammer Weight, W_H (lbs)	170

Finally, the drilling fluid used imposes a stress on the soil at the bottom of the borehole. Mud rotary drilling with a mixture of the groundwater and drilling fluid was used for all the borings performed in this study. The total unit weight of the drilling fluid was estimated to be 70 lbs. per cubic foot (pcf). Based on the above the total stress (q_{TCP}) imposed on the bottom of the borehole by the cone can be expressed by the following equation:

$$q_{TCP} = \frac{W_c + W_H + W_{DR} L_{DR} + \gamma_{DF} D\left(A_c - A_{DR}\right))}{A_c} \qquad (2.3)$$

where W_c is cone weight, W_H is hammer weight, W_{DR} is drilling rod weight, L_{DR} is drilling rod length, γ_{DF} is total unit weight of the drilling fluid, D is depth at which the TCP test is being performed, A_C is cone area, and A_{DR} is drilling rod area. The weights of the cone (W_C), hammer (W_H) and drill rod (W_{DR}) are the total weights in air rather than the weights when submerged in drilling fluid.

The minimum undrained shear strength required to support the TCP cone can be back-calculated by equating the ultimate bearing capacity (q_{ult}) in Equation 2.1 to the total imposed stress (q_{TCP}) in Equation 2.2.

$$Ns_u + \sigma_{v0} = q_{TCP} \qquad (2.4)$$

Solving for the minimum undrained shear strength required to support the cone then gives,

$$s_u = \frac{q_{TCP} - \sigma_{v0}}{N} \qquad (2.5)$$

Equation 2.5 is used in subsequent chapters to determine the minimum strength required to support the TCP cone for each site investigated.

2.3 Example of Application

An example of the minimum strength required to support the TCP cone has been calculated for a simplified subsurface profile. The subsurface conditions consist of 25 feet of homogeneous clay with the groundwater table at the surface. The clay has a total unit weight of 100 lbs. per cubic foot (pcf). Table 2.2 summarizes the pertinent computations for undrained shear strength required to support the TCP cone for any given depth below the ground surface. The undrained strength profile is also plotted in Figure 2.2. For depths up to 30 feet, which are the depths of primary interest in this study, the required undrained shear strength varies from somewhat less than 300 psf (240 psf) to somewhat greater than 300 psf (325 psf). A value of 300 psf is a reasonable average value of the minimum undrained shear strength required to support the Texas Cone Penetrometer.

Table 2.2: Computations for Bearing Capacity Analysis Example

Depth	s_{vo} (psf)	q_{TCP} (psf)	s_u (psf)
0	0	3606	240
5	500	4318	255
10	1000	5030	269
15	1500	5741	283
20	2000	6453	297
25	2500	7165	311
30	3000	7877	325

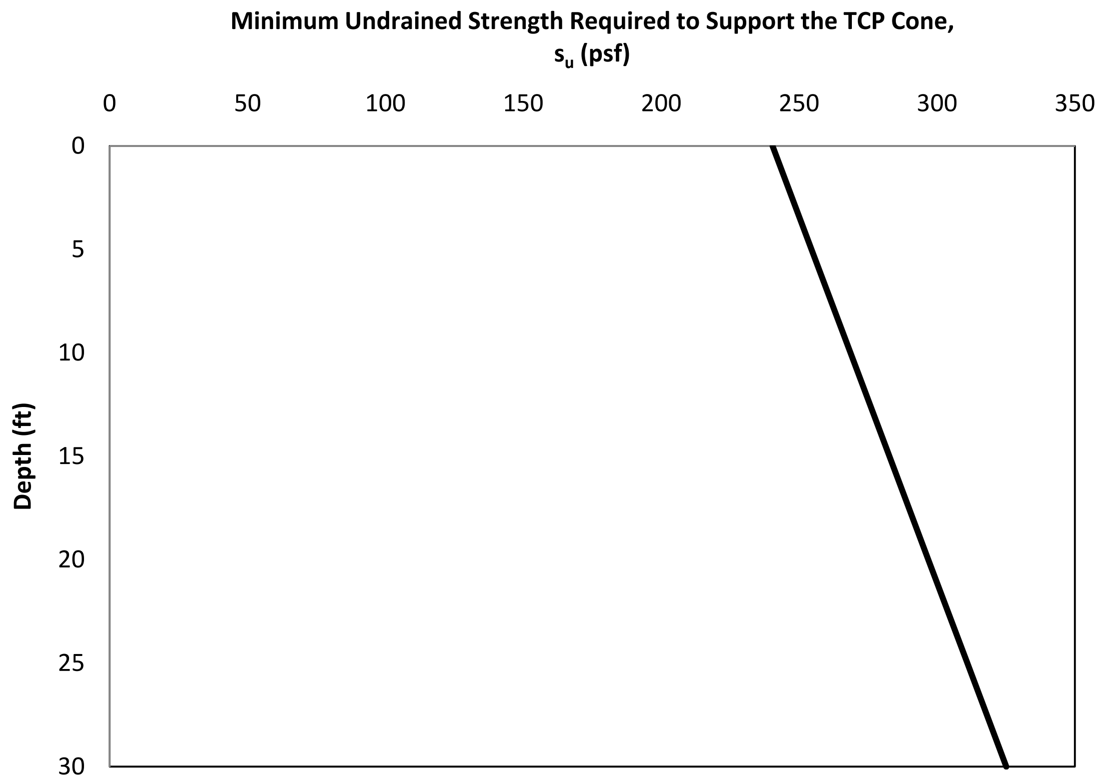


Figure 2.2: Minimum Strength Required to Support the TCP Cone for Example

Chapter 3. Correlation of the Texas Cone Penetrometer Blow Count and Undrained Shear Strength

Several correlations exist between Texas Cone Penetrometer blow count and undrained shear strength. These were developed primarily from data for stronger soils and are typically used for the design of deep foundations. Studies have been performed by Hamoudi et al. (1974), Duderstadt et al. (1977), and Kim et al. (2007) to develop correlations between Texas Cone Penetrometer blow count and undrained shear strength of fine-grained soils. These studies are reviewed in this chapter.

3.1 Texas A&M University – Hamoudi et al. (1974), Research Report 10-1

Hamoudi et al. (1974) completed a study to improve the correlation between Texas Cone Penetrometer blow count and unconsolidated-undrained shear strength of fine-grained soils. Hamoudi et al. (1974) considered the correlation previously being used by TxDOT to be overly conservative. They tested soils at four locations along the upper Texas Gulf Coast. The soil types investigated belonged to one of the following three categories based on the Unified Soil Classification System (USCS):

1) inorganic clays of high plasticity (CH classification);
2) inorganic clays of low plasticity which includes sandy clays, silty clays and lean clays (CL classification); and
3) clayey sands (SC classification).

These classifications were broken down further into six categories that are described later.

At each of the four sites investigated, soil borings were made, Texas Cone Penetrometer tests were performed, and undisturbed samples were collected. Laboratory tests were performed on the undisturbed samples to classify the soil and measure the undrained shear strength. Undrained strength tests included the Texas Triaxial Test, Transmatic Triaxial Test, and ASTM Standard Unconsolidated-Undrained (UU) Triaxial Test. It has since been shown by O'Malley and Wright (1987) that the Texas Triaxial Test is unreliable for measuring the undrained shear strength of soft soils. Only the results obtained using the UU tests are examined for the purposes of this current report and analysis.

The ranges in Texas Cone Penetrometer blow counts and undrained shear strengths from UU triaxial tests reported by Hamoudi et al. (1974) are summarized in Table 3.1. Soil types were divided into the six sub-groups shown in this table. Based on their data Hamoudi et al. (1974) proposed the following linear relationship between Texas Cone Penetrometer blow count and undrained shear strength:

$$s_u = KN_{TCP} \tag{3.1}$$

where s_u is undrained shear strength in pounds per sq. foot (psf), K is a constant of proportionality and N_{TCP} is Texas Cone Penetrometer blow count. Hamoudi et al. (1974) found the constant of proportionality varied depending on soil type and reported the values shown in Table 3.2 for four of the soil types. Values for K weren't developed for the stratified CL soils or SC soils due to lack of sufficient data. The correlation between N_{TCP} and s_u from the UU tests is also illustrated in Figure 3.1.

Table 3.3: Texas Cone Penetrometer Blow Counts and Undrained Shear Strengths from Hamoudi et al. (1974)

Soil Type	Number of Tests	N_{TCP}		s_u (psf)	
		Range	Average	Range	Average
Homogeneous CH Soils	17	12 - 32	23	760 - 6860	3040
CH Soils with Secondary Structure	15	46 - 212	140	1100 - 9000	5540
Silty CL Soils	7	12 - 32	25	900 - 4340	2480
Sandy CL Soils	7	22 - 44	30	2100 - 4860	3360
Stratified CL Soils	1	40	40	2500	2500
SC Soils	1	8	8	1960	1960

Table 3.4: Constant of Proportionality for Various USCS Soil Classifications from Hamoudi et al. (1974)

USCS Soil Classification	Constant of Proportionality, K, for UU Tests
Homogeneous CH Soils	140
CH Soils with Secondary Structure	36
Silty CL Soils	126
Sandy CL Soils	106

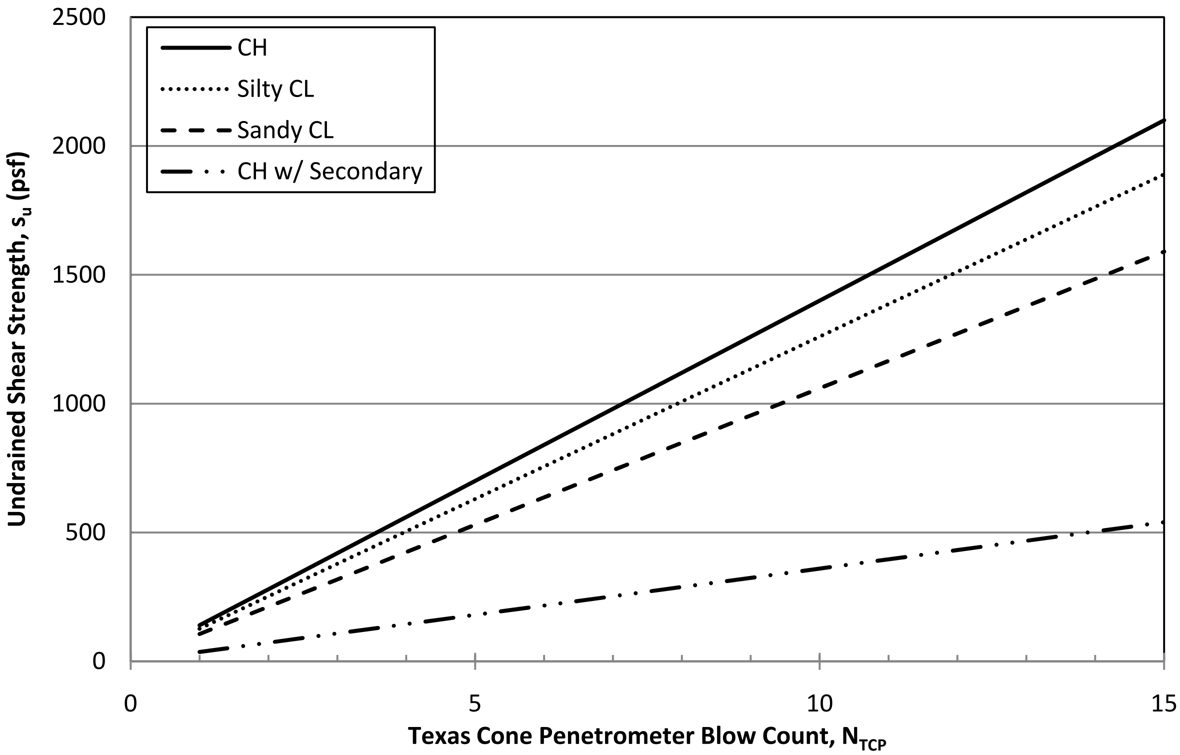


Figure 3.3: Correlation of N_{TCP} and s_u from Hamoudi et al. (1974)

3.2 Texas A&M University – Duderstadt et al. (1977), Research Report 10-3F

Duderstadt et al. (1977) later extended the work by Hamoudi et al. (1974). Duderstadt et al. (1977) tested one additional site located in the Texas Gulf Coast region. The additional site consisted exclusively of fine-grained soils. Two adjacent borings were made to obtain Texas Cone Penetrometer data and undisturbed samples. The field and laboratory investigation followed procedures similar to those of Hamoudi et al. (1974).

The ranges in Texas Cone Penetrometer blow counts and undrained shear strengths from UU triaxial tests reported by Duderstadt et al. (1977) are summarized in Table 3.3. Duderstadt et al. (1977) also assumed a linear relationship between Texas Cone Penetrometer blow count and undrained shear strength (Equation 3.1). The values for the constant of proportionality (K) determined are shown in Table 3.4. The relationship between N_{TCP} and s_u is also illustrated in Figure 3.2.

Table 3.5: TCP Blow Counts and Undrained Shear Strengths from Duderstadt et al. (1977)

Soil Type	Number of Tests	N_{TCP}		s_u (psf)	
		Range	Average	Range	Average
Homogeneous CH Soils	5	13 - 17	15	1880 - 2260	2030
Silty CL	1	7	7	2360	2360

Table 3.6: Constant of Proportionality for Various USCS Soil Classifications from Duderstadt et al. (1977)

USCS Soil Classification	Constant of Proportionality, K, for UU Tests
Homogeneous CH Soils	134
Silty CL Soils	108
Sandy CL Soils	106

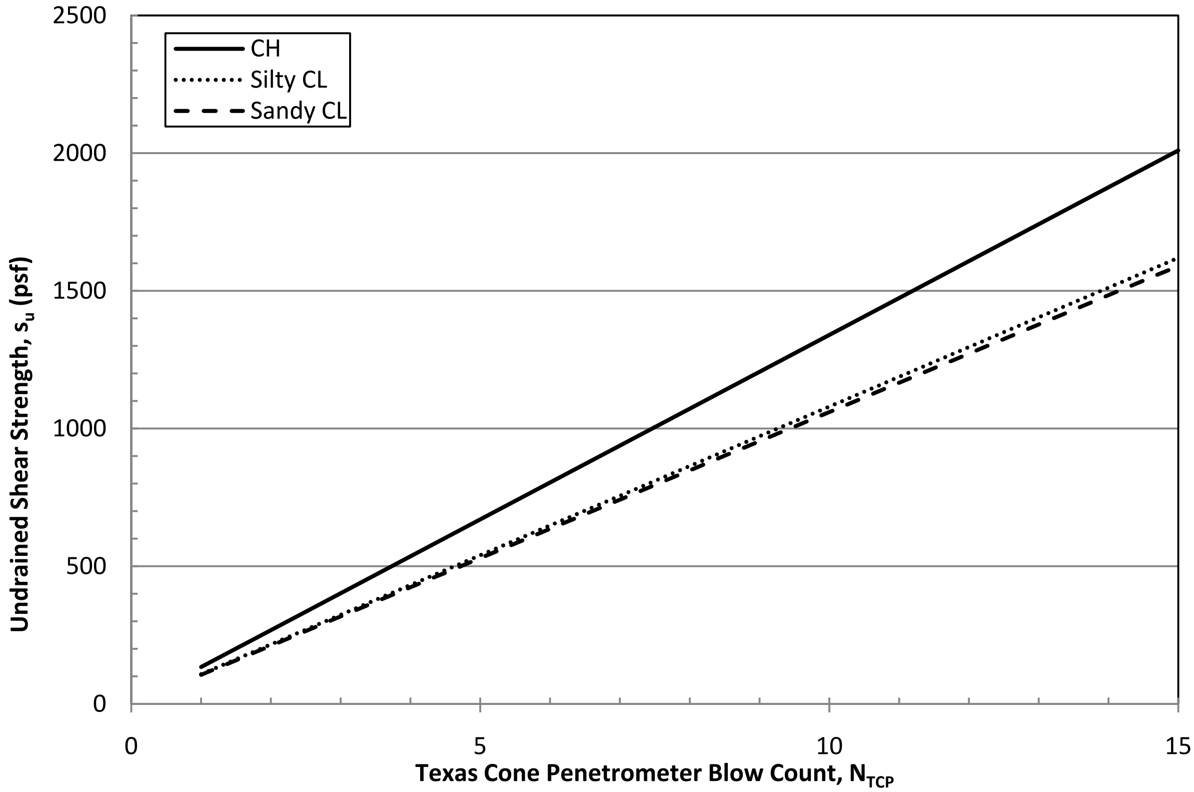


Figure 3.4: Correlation of N_{TCP} and s_u from Duderstadt et al. (1977)

3.3 University of Houston – Kim et al.

Kim et al. (2007) correlated undrained shear strength of soft clay soil ($s_u \leq 520$ psf) to Texas Cone Penetrometer blow count using statistical methods. Soil data were collected over 10 years from various TxDOT projects along the Texas Gulf Coast. Although the method for determining undrained shear strength was not reported, it seems likely that a variety of methods were used. Variables that were examined by Kim et al. (2007) in their statistical analyses included natural moisture content, liquid limit, plasticity index, bulk density, undrained shear strength, and N_{TCP} values. The ranges and averages of the natural moisture content, undrained shear strength and N_{TCP} are shown in Table 3.5 for both CH and CL soils.

Table 3.7: Ranges and Averages for Variables from Kim et al. (2007)

Soil Type		w (%)	s_u (psf)	N_{TCP}
CH Soils	Range	24.6 - 79	115 - 533	2 - 25
	Mean	43.3	418	7.6
CL Soils	Range	19 - 59	58 - 511	2 - 55
	Mean	34.6	346	18

Kim et al. (2007) concluded that Texas Cone Penetrometer blow count and undrained shear strength of cohesive soils was affected by both depth and moisture content. Based on their analysis of the data, the following equation was proposed to relate the undrained shear strength of soft clay in the Texas Gulf Coast to Texas Cone Penetrometer blow count:

$$s_u = \left(\alpha \frac{N_{TCP}}{d} + \beta\right) w \qquad (3.2)$$

where s_u is undrained shear strength in lbs. per square inch (psi), d is the depth in feet and w is the moisture content in percent. The parameters α and β were determined for both fat clays (CH) and lean clays (CL) by least squares fitting of the data and are summarized in Table 3.6.

Table 3.8: Parameter from Least Squares Fitting of the Data for the Correlation from Kim et al. (2007)

Soil Type	α	β
CH Soils	0.036	0.045
CL Soils	0.12	0.021

3.4 Summary

There are several limitations in the correlation developed by Kim et al. (2007). The correlation suggests that strength may increase with an increase in moisture content which is not the typical behavior for most soils. Although moisture content probably reduces the N_{TCP} value in Equation 3.2 to partially offset the increase in strength due to the moisture content multiplier (w), the equation still appears fundamentally illogical. Thus, basing a new correlation upon the findings of Kim et al. (2007) seems inappropriate.

Although the correlations by Hamoudi et al. (1974) and Duderstadt et al. (1977) may be reasonable, there are only a total of 6 samples between the two studies where Texas Cone Penetrometer blow counts were less than 15 as shown in Table 3.7. Only one blow count value shown in this table is less than 10. A need exists to at least verify the correlations by Hamoudi et al. (1974) and Duderstadt et al. (1977) for weaker soils and possibly develop an alternative or improved correlation.

Table 3.9: Data from Hamoudi et al. (1974) and Duderstadt et al. (1977) where Texas Cone Penetrometer Blow Counts were less than 15

Soil Type	N_{TCP}	s_u (psf)
CH Soils	14	1640
	12	760
	13	1960
	13	1880
Silty CL Soils	12	900
	7	2360

Chapter 4. Overview of Subsurface Explorations

Subsurface explorations were carried out by Tolunay-Wong Engineers, Inc. (TWEI) at six sites in the Texas Gulf Coast region where soft soils were expected to exist. Field testing included a series of borings with Texas Cone Penetrometer tests, conventional ("Dutch") piezocone penetration tests, thin-walled tube sampling, and vane shear tests. Laboratory testing was subsequently performed at The University of Texas at Austin on the samples obtained in the field. The test boring logs with the subsurface data and observations are included in Appendix B.

4.1 Site No. 1 – Port Arthur, TX (Site A)

This site is located near the intersection of Procter Street and Main Avenue in Port Arthur, Jefferson County, Texas (Figure 4.1). The site lies within the West Crane Bayou just west of Sabine Lake. The existing ground surface at the site is at an elevation of approximately +10 feet.

Review of Published Literature

According to the regional Geologic Atlas of Texas (Houston Sheet), the site is underlain by Alluvium (Qal) and the Beaumont Formation (Qb) (Flawn, 1968). The Alluvium consists of clay, silt, sand and organic matter. The depositional environments include point bar, natural levee, stream channel, backswamp, coastal marsh, mud flat, and narrow beach deposits. The Beaumont Formation consists of sand, silt, clay, and gravel. The depositional environments include point bar, natural levee, stream channel, and backswamp deposits.

Field Exploration

The field exploration program consisted of the following:

- drilling and sampling one 26-foot-deep boring with Texas Highway Department cone penetration (TCP) tests performed at 2-foot intervals;
- drilling, logging and obtaining thin-walled tube soil samples continuously for a second 26-foot-deep boring;
- conducting three field vane shear tests at varying depths; and
- conducting seven 6- to 30-foot-deep piezocone penetration tests.

The location of the borings and additional field tests was selected by Tolunay-Wong Engineers, Inc. All of the borings were located in close proximity to one another within the subject site. The locations are shown on Figure 4.1.

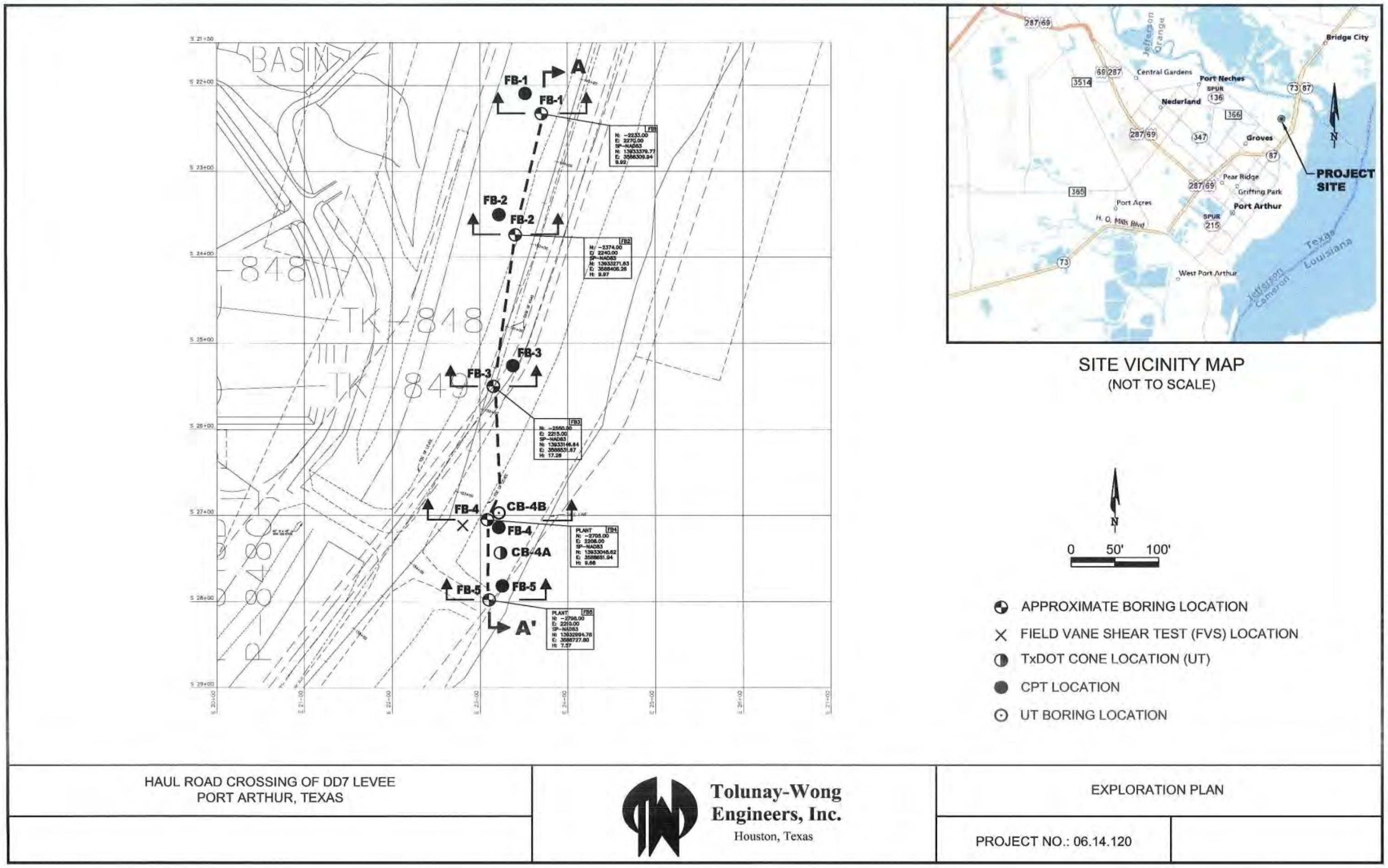


Figure 4.5: Site and Boring Location Map for Site No. 1

Laboratory Testing

Laboratory testing was performed on soil samples from the thin-walled tubes. The following tests were performed:

- eleven unconsolidated-undrained (UU) triaxial compression tests;
- one consolidated-undrained (CU) triaxial compression test;
- Liquid Limit, Plastic Limit, and Plasticity Index Tests on two specimens; and
- moisture content tests on various specimens.

The laboratory results are summarized in the following section for each soil strata. Detailed results are presented and discussed more thoroughly by Varathungarajan (2008).

Stratigraphic Profile

The subsurface profile is generalized as follows, beginning at the ground surface (Figure 4.2):

Depth: 0 to 6 feet (Elevation: +10 to +4 feet). The surficial layer consists of a firm tan and brown *sandy clay fill* with ferrous stains. Very stiff gray and black soil with sand pockets, gravel, and hydrocarbons was found near the bottom of the layer. The Texas Cone Penetrometer (TCP) blow counts (N_{TCP}) in this stratum ranged from 8 to 31 blows per foot (bpf). Undrained shear strengths were measured with a pocket penetrometer on three samples in the field; the average undrained shear strength was 2100 lbs per sq. ft. (psf), which is indicative of soil with a stiff consistency. One field vane shear test was performed in this stratum; however, the capacity of the vane shear equipment (strength of 1441 psf) was reached before failure occurred.

Laboratory testing was performed on four tube samples for this stratum. The natural moisture content varied from 22.1 to 25.4 percent. Unconsolidated-Undrained triaxial tests were performed on each sample and the undrained shear strengths ranged from 910 to 1440 psf with an average strength of 1200 psf.

Depth: 6 to 10 feet (Elevation: +4 to 0 feet). This layer consists of a dense gray and black *granular fill*. The N_{TCP} values in this stratum ranged from 7 to 38 bpf.

Depth: 10 to 13 feet (Elevation 0 to -3 feet). This layer consists of very soft gray *fat clay*. The N_{TCP} values in this stratum ranged from 3 to 7 bpf. Undrained shear strength was measured with a pocket penetrometer on one sample in the field; the undrained shear strength was 250 psf, which is indicative of soil with a very soft consistency. A single Torvane test was also performed in the field and yielded an undrained shear strength of

360 psf. A single field vane shear test was performed in this stratum and the corrected[1] undrained shear strength was measured to be 439 psf.

Laboratory testing was performed on three tube samples for this stratum. The natural moisture content varied from 30.2 to 81.1 percent. Unconsolidated-Undrained triaxial tests were performed on two samples and the undrained shear strengths were 296 and 336 psf. One consolidated-undrained triaxial test was performed and the undrained shear strength was 373 psf. Two Atterberg Limit tests were performed. The liquid limits were 49 and 97, and the plasticity indices were 26 and 69. Based on measured index properties and visual observations the soil was classified as *fat clay* (CH) by the Unified Soil Classification System (USCS).

<u>Depth: 13 to 16 feet (Elevation: -3 to -6 feet).</u> This layer consists of firm to stiff gray *lean clay* with sand and silt seams. The N_{TCP} value in this stratum was 20 bpf. Undrained shear strengths were measured with a pocket penetrometer on two samples in the field; the average undrained shear strength was 1600 psf, which is indicative of soil with a stiff consistency.

Laboratory testing was performed on two tube samples for this stratum. The natural moisture content was determined to be 19.9 and 30.2 percent. Unconsolidated-Undrained triaxial tests were performed on each sample and the representative undrained shear strength was 635 psf. Based on measured index properties and visual observations the soil was classified as *lean clay* (CL) by the Unified Soil Classification system.

<u>Depth: 16 to 20 feet (Elevation: -6 to -10 feet).</u> This layer consists of medium dense gray and tan *sand* with calcareous nodules and clay pockets. The N_{TCP} values in this stratum ranged from 15 to 20 bpf. One field vane shear test was performed in this stratum; however, the capacity of the vane shear equipment (strength of 1441 psf) was reached before failure occurred.

Laboratory testing was performed on two tube samples for this stratum. The natural moisture content was determined to be 22.7 and 37.5 percent. Based on visual observations the soil was classified as *clayey sand* (SC) by the Unified Soil Classification system.

<u>Depth: 20 to 26 feet (Elevation: -10 to -16).</u> This lowest layer consists of stiff brown and gray *fat clay* with silt pockets. The N_{TCP} values in this stratum ranged from 26 to 31 bpf. Undrained shear strengths were measured with a pocket penetrometer on three samples in the field; the average undrained shear strength was 2100 psf, which is indicative of soil with a stiff consistency.

Laboratory testing was performed on three tube samples for this stratum. The natural moisture content varied from 13.9 to 27.2 percent. Unconsolidated-Undrained triaxial

[1] Experience has shown that Vane Shear tests tend to overestimate undrained shear strength. Bjerrum (1972) developed a reduction factor based on the plasticity index of the soil, which gives a corrected undrained shear strength.

tests were performed on each sample; however, a representative undrained shear strength was not obtained due to the quality of the tests. Based on visual observations the soil was classified as *fat clay* (CH) and *silty clay* (CL-ML) by the Unified Soil Classification system.

At the time of the subsurface investigation, groundwater was determined to be at the ground surface from visual observations made by Tolunay-Wong Engineers, Inc.

4.2 Site No. 2 – Port Arthur, TX (Site B)

This site is located on the north side of Highway 87 near the border of Jefferson and Orange County in Port Arthur, Jefferson County, Texas (Figure 4.3). This site is situated approximately 4 miles northeast of Site No. 1. The existing ground surface at the site is at an elevation of approximately +3 feet. The site's geologic history is similar to that of Site No. 1.

Field Exploration

The field exploration program consisted of the following:

- drilling and sampling one 35-foot-deep boring with Texas Highway Department cone penetration (TCP) tests performed at 2-foot intervals;
- drilling, logging and obtaining soil thin-walled tube samples continuously for a second 36-foot-deep boring;
- conducting eight field vane shear tests at varying depths; and
- conducting four 46- to 133-foot-deep piezocone penetration tests.

The location of the borings and additional field tests was selected by Tolunay-Wong Engineers, Inc. All of the borings were located in close proximity to one another within the subject site. The locations are shown on Figure 4.3.

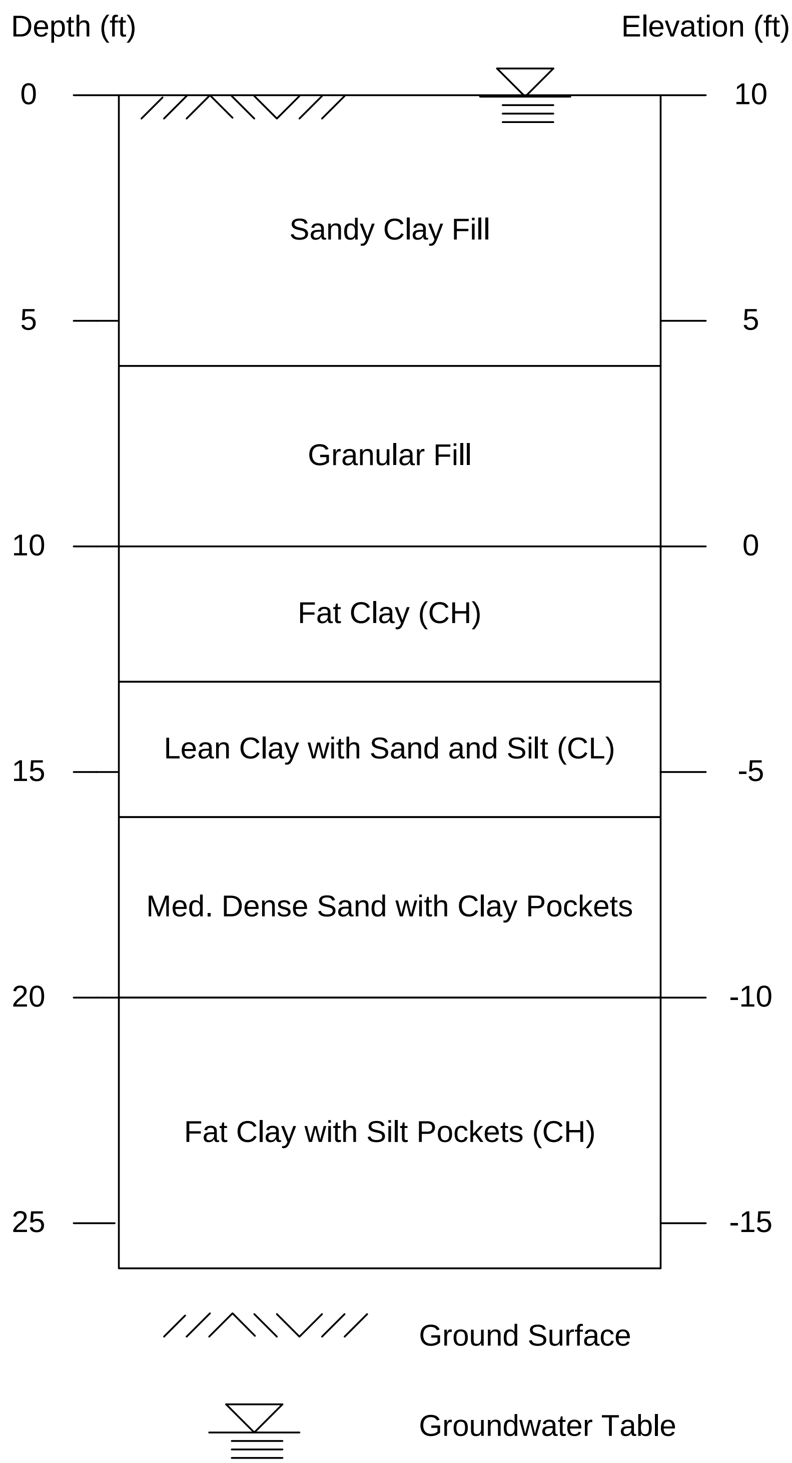


Figure 4.6: Stratigraphic Profile of Site No. 1

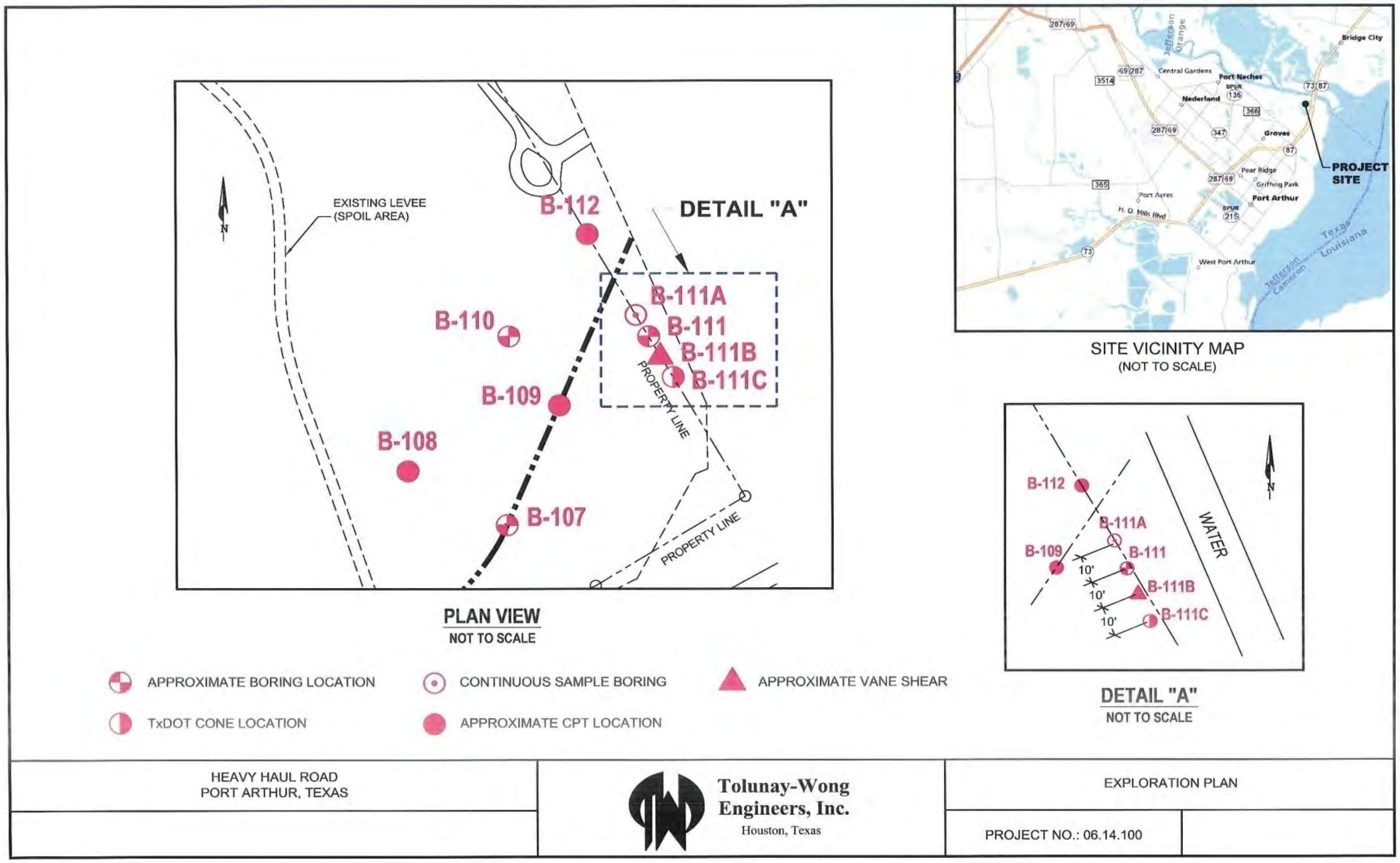


Figure 4.7: Site and Boring Location Map for Site No. 2

Laboratory Testing

Laboratory testing was performed on soil samples from the thin-walled tubes. The following tests were performed:

- twenty unconsolidated-undrained (UU) triaxial compression tests;
- two consolidated-undrained (CU) triaxial compression tests;
- two one-dimensional consolidation tests using incremental loading;
- one one-dimensional consolidation test using controlled-strain loading;
- Liquid Limit, Plastic Limit, and Plasticity Index Tests on six specimens; and
- moisture content tests on various specimens.

The laboratory results are summarized in the following section for each soil strata. Detailed results are presented and discussed more thoroughly by Varathungarajan (2008).

Stratigraphic Profile

The subsurface profile is generalized as follows, beginning at the ground surface (Figure 4.4):

Depth: 0 to 5 feet (Elevation: +3 to -2 feet). The surficial layer consists of a gray *clayey* and *silty sand* stratum. The N_{TCP} values in this stratum ranged from 2 to 11 bpf.

Depth: 5 to 29 feet (Elevation: -2 to -26 feet). This layer consists of soft gray *fat clay* with some sand. The N_{TCP} values ranged from "weight of hammer (WOH)" to 2 bpf. Ten Torvane tests were performed in the field and yielded an average undrained shear strength of 210 psf. Eight field vane shear tests were performed in this stratum; the corrected undrained shear strengths ranged from 169 to 892 psf with an average of 535 psf.

Laboratory testing was performed on twenty-two tube samples from this stratum. The natural moisture content varied from 33.7 to 100.5 percent. Unconsolidated-Undrained triaxial tests were performed on twenty samples and the representative undrained shear strengths ranged from 112 to 449 psf with an average strength of 250 psf. Consolidated-Undrained triaxial tests were performed on two samples and the undrained shear strengths were 378 and 410 psf, respectively. Six Atterberg Limit tests were performed. The liquid limit ranged from 52 to 110, and the plasticity index ranged from 31 to 93. Based on measured index properties and visual observations the soil was classified as *fat clay* (CH) by the Unified Soil Classification system.

Depth: 29 to 36 feet (Elevation: -26 to -33 feet). This layer consists of soft gray *sandy lean clay*. The N_{TCP} value in this stratum was 8 bpf.

At the time of the subsurface investigation, groundwater was determined to be at a level 4 feet below the ground surface from visual observations made by Tolunay-Wong Engineers, Inc.

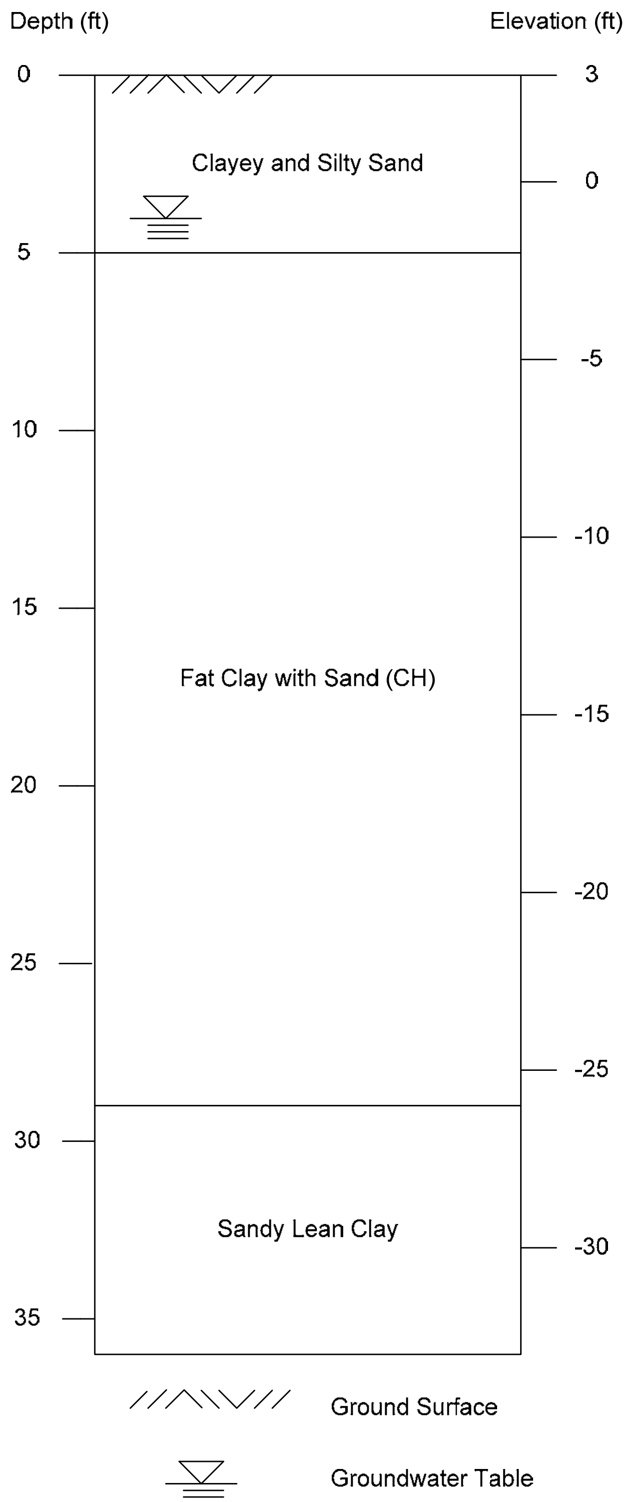


Figure 4.8: Stratigraphic Profile of Site No. 2

4.3 Site No. 3 – Mont Belvieu, TX

This site is located north of FM 1942 and west of Cedar Bayou in Mont Belvieu, Harris, and Chambers Counties, Texas (Figure 4.5). The existing ground surface at the site is at an elevation of approximately +31 feet.

Review of Published Literature

According to the regional Geologic Atlas of Texas (Houston Sheet), the site is underlain by the Beaumont Formation (Qb) (Flawn, 1968). The Beaumont Formation consists of sand, silt, clay, and gravel. The depositional environments include point bar, natural levee, stream channel, and backswamp deposits.

Field Exploration

The field exploration program consisted of the following:

- drilling and sampling one 26-foot-deep boring with Texas Highway Department cone penetration (TCP) tests performed at 2-foot intervals;
- drilling, logging and obtaining thin-walled tube soil samples continuously for a second 26-foot-deep boring; and
- conducting four 25- to 75-foot-deep piezocone penetration tests.

Vane shear tests could not be performed due to the high strength of the soil at this site. The location of the borings and additional field tests was selected by Tolunay-Wong Engineers, Inc. All of the borings were located in close proximity to one another within the subject site. The locations are shown on Figure 4.5.

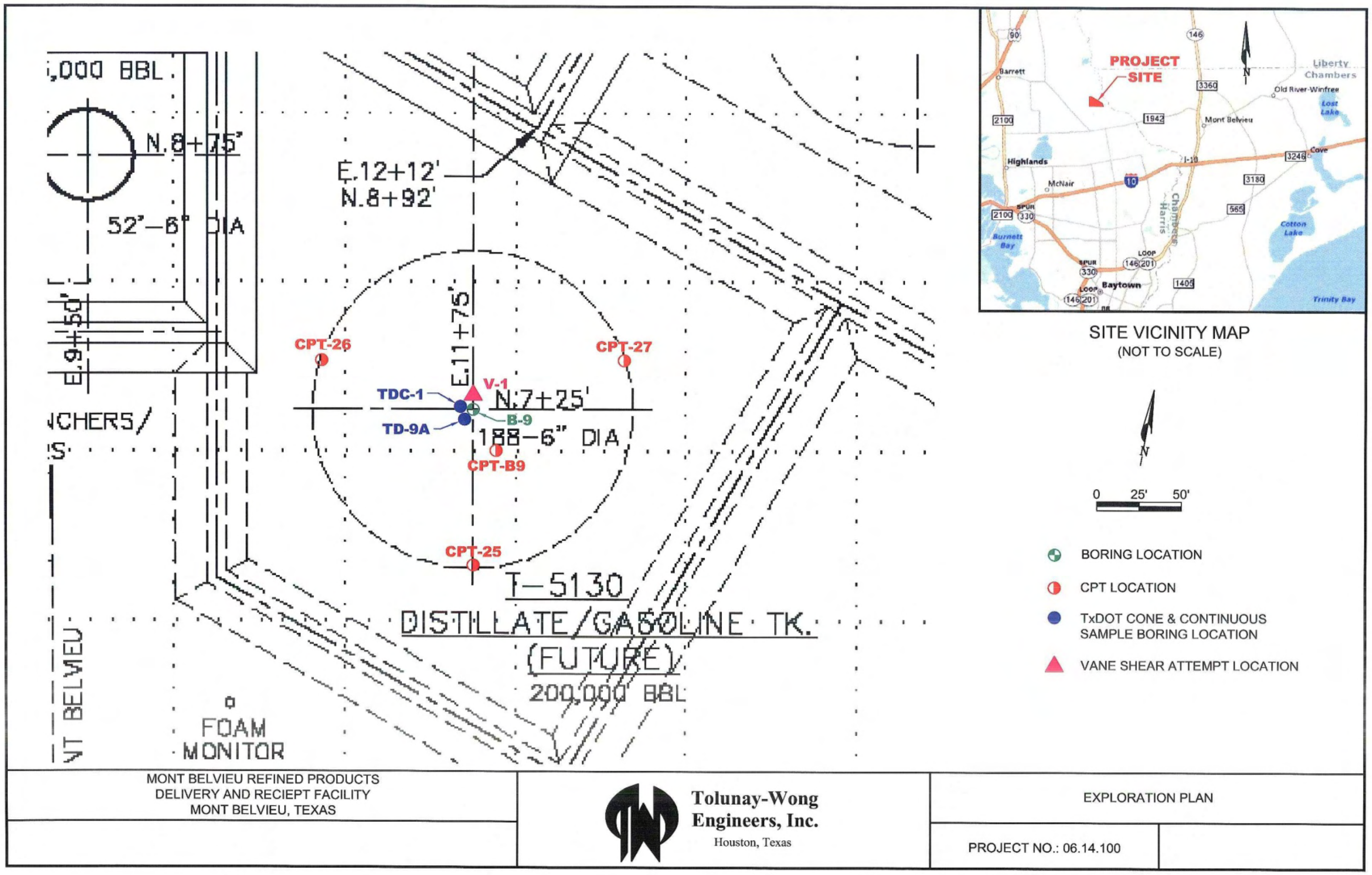


Figure 4.9: Site and Boring Location Map for Site No. 3

Laboratory Testing

Laboratory testing was performed on soil samples from the thin-walled tubes. The following tests were performed:

- nine unconsolidated-undrained (UU) triaxial compression tests;
- three consolidated-undrained (CU) triaxial compression tests;
- Liquid Limit, Plastic Limit, and Plasticity Index Tests on six specimens; and
- moisture content tests on various specimens.

The laboratory results are summarized in the following section for each soil strata. Detailed results are presented and discussed more thoroughly by Varathungarajan (2008).

Stratigraphic Profile

The subsurface profile is generalized as follows, beginning at the ground surface (Figure 4.6):

<u>Depth: 0 to 18 feet (Elevation: +31 to +13 feet).</u> The subsurface soil consists of a medium stiff to very stiff tan and gray *clay* stratum with varying amounts of silt and sand. The N_{TCP} values in this stratum ranged from 6 to 14 bpf. Undrained shear strengths were measured with a pocket penetrometer on nine samples in the field; the average undrained shear strength was 1670 psf, which is indicative of soil with a stiff consistency.

Laboratory testing was performed on fourteen tube samples from this stratum. The natural moisture content varied from 25.0 to 42.3 percent. Unconsolidated-Undrained triaxial tests were performed on ten samples and the representative undrained shear strengths ranged from 172 to 1204 psf with an average of 700 psf. Consolidated-Undrained triaxial tests were performed on four samples and the representative undrained shear strengths ranged from 591 to 1262 psf with an average strength of 860 psf. Six Atterberg Limit tests were performed. The liquid limit ranged from 38 to 74, and the plasticity index ranged from 22 to 49. Based on measured index properties and visual observations the soil was classified as *fat clay* (CH) and *lean clay* (CL) by the Unified Soil Classification system.

<u>Depth: 18 to 26 feet (Elevation: +13 to +5 feet).</u> This layer consists of tan and gray *clayey sand*. The N_{TCP} values in this stratum ranged from 12 to 14 bpf. Undrained shear strengths were measured with a pocket penetrometer on three samples in the field; the average undrained shear strength was 1000 psf.

Laboratory testing was performed on four tube samples from this stratum. The natural moisture content varied from 25.4 to 28.8 percent. Unconsolidated-Undrained triaxial tests were performed on each sample; however, only one test was considered reasonable and the undrained shear strength was 1385 psf.

At the time of the subsurface investigation, groundwater was determined to be at a level 4 feet below the ground surface from visual observations made by Tolunay-Wong Engineers, Inc.

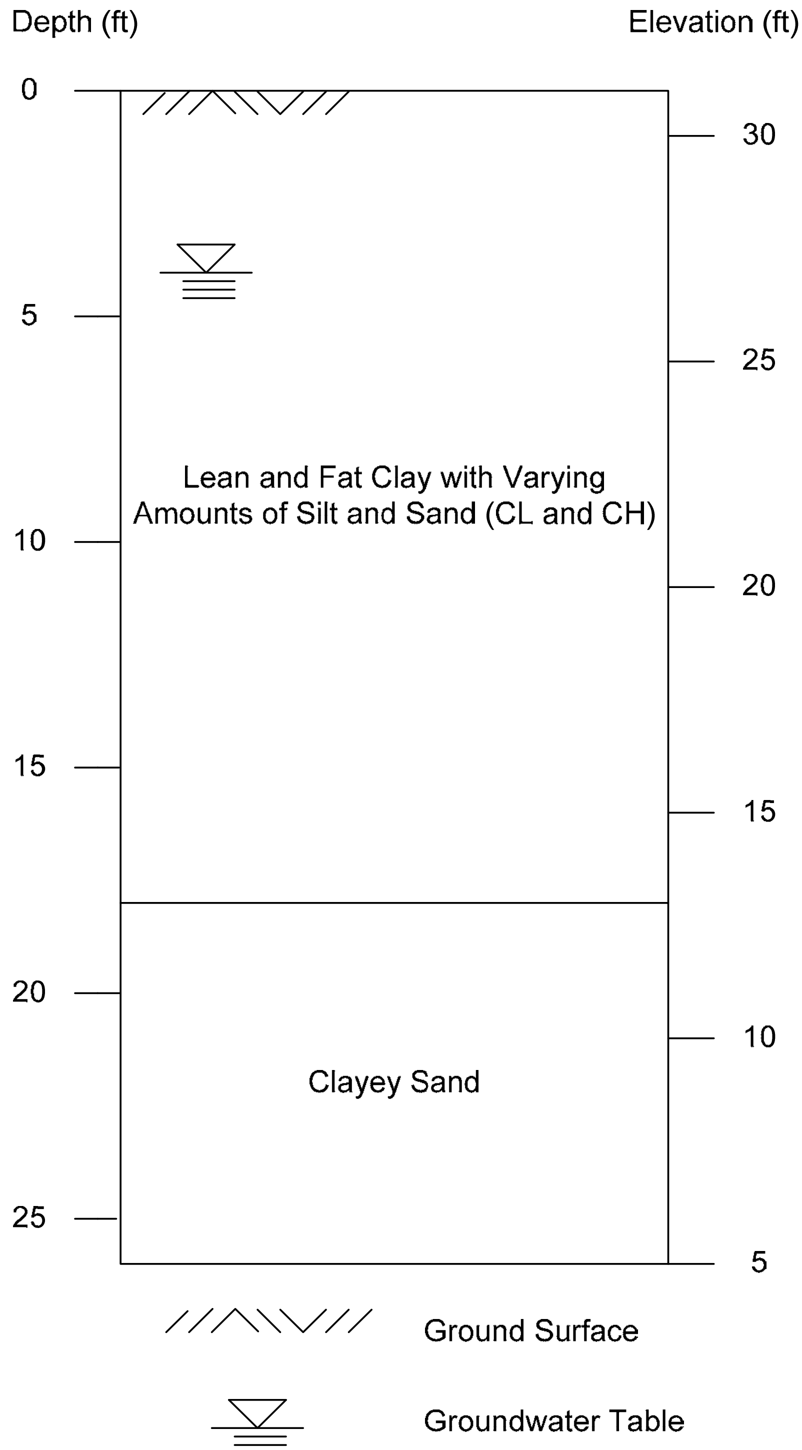


Figure 4.10: Stratigraphic Profile of Site No. 3

4.4 Site No. 4 – Beaumont, TX

This site is located just north of the intersection of US-287 and TX-347W in Beaumont, Jefferson County, Texas (Figure 4.7). The site is bordered by the Kansas City Southern Railway to the west and the Neches River to the east. The existing ground surface lies at an elevation of approximately +10 feet.

Review of Published Literature

According to the regional Geologic Atlas of Texas (Beaumont Sheet), the site is underlain by Alluvium (Qal) and the Beaumont Formation (Qb) (Flawn, 1968). The Alluvium consists of clay, silt, sand, and organic matter. The depositional environments include point bar, natural levee, stream channel, backswamp, coastal marsh, mud flat, and narrow beach deposits. The Beaumont Formation consists of sand, silt, clay, and gravel. The depositional environments include point bar, natural levee, stream channel, and backswamp deposits.

Field Exploration

The field exploration program consisted of the following:

- drilling and sampling one 26-foot-deep boring with Texas Highway Department cone penetration (TCP) tests performed at 2-foot intervals;
- drilling, logging and obtaining thin-walled tube soil samples continuously for a second 26-foot-deep boring;
- conducting three field vane shear tests at varying depths; and
- conducting three 25- to 50-foot-deep piezocone penetration tests.

The location of the borings and additional field tests was selected by Tolunay-Wong Engineers, Inc. All of the borings were located in close proximity to one another within the subject site. The locations are shown on Figure 4.7.

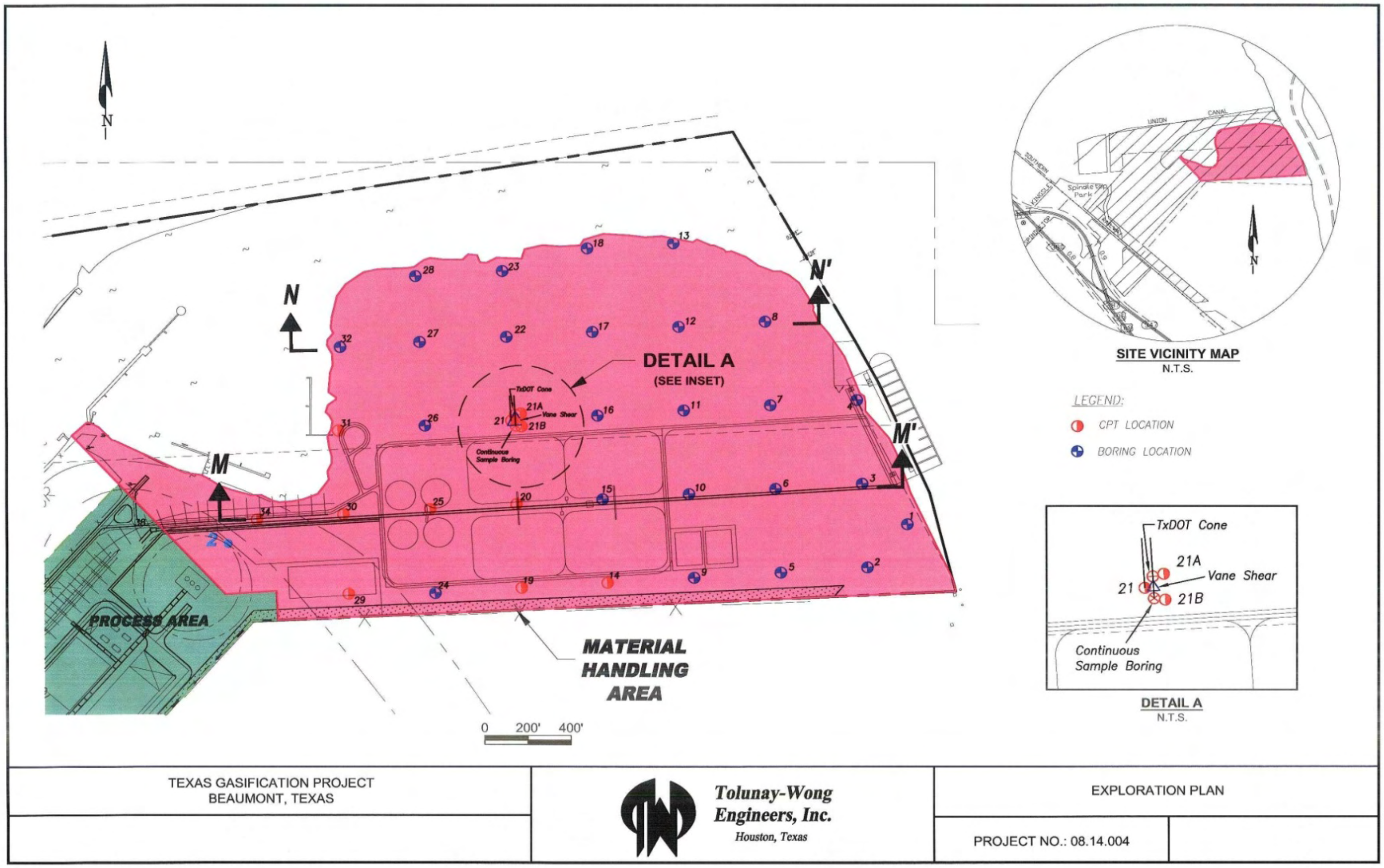


Figure 4.11: Site and Boring Location Map for Site No. 4

Laboratory Testing

Laboratory testing was performed on soil samples from the thin-walled tubes. The following tests were performed:

- sixteen unconsolidated-undrained (UU) triaxial compression tests;
- six consolidated-undrained (CU) triaxial compression tests;
- one one-dimensional consolidation test using controlled-strain loading;
- Liquid Limit, Plastic Limit, and Plasticity Index tests on five specimens; and
- moisture content tests on various specimens.

The laboratory results are summarized in the following section for each soil strata. Detailed results are presented and discussed more thoroughly by Varathungarajan (2008).

Stratigraphic Profile

The subsurface profile is generalized as follows, beginning at the ground surface (Figure 4.8):

Depth: 0 to 4 feet (Elevation: +10 to 6 feet). The surficial layer consists of a stiff tan and gray *sandy clay fill*. The N_{TCP} values in this stratum were 5 and 8 bpf. Undrained shear strengths were measured with a pocket penetrometer on three samples in the field; the average undrained shear strength was 1670 psf, which is indicative of soil with a stiff consistency.

Laboratory testing was performed on three tube samples for this stratum. The natural moisture contents ranged from 20.2 to 28.5 percent. Unconsolidated-Undrained triaxial tests were performed on each sample and the undrained shear strengths ranged from 614 to 1919 psf with an average strength of 1180 psf.

Depth: 4 to 11 feet (Elevation: +6 to -1 feet). This layer consists of medium dense tan and light gray *sand* with clay pockets. The N_{TCP} values in this stratum ranged from 3 to 18 bpf. Undrained shear strength was measured with a pocket penetrometer on one sample in the field; the undrained shear strength was 250 psf.

Depth: 11 to 22 feet (Elevation: -1 to -12 feet). This layer consists of gray *organic clay* with plant fragments. The N_{TCP} values in this stratum ranged from WOH to 3 bpf. Undrained shear strengths were measured with a pocket penetrometer on two samples in the field; the undrained shear strengths were 500 and 750 psf, which is indicative of soil with a medium stiff consistency. Three Torvane tests were also performed in the field and yielded undrained shear strengths of 240, 260, and 320 psf. Three field vane shear tests were performed in this stratum and the corrected undrained shear strengths were measured to be 415, 532, and 650 psf.

Laboratory testing was performed on sixteen tube samples for this stratum. The natural moisture content varied from 74.1 to 263.9 percent. Unconsolidated-Undrained triaxial tests were performed on eleven samples and the undrained shear strengths ranged from 343 to 611 psf with an average strength of 530 psf. Consolidated-Undrained triaxial tests were performed on five samples and the undrained shear strengths ranged from 536 to 808 psf with an average strength of 640 psf. Three Atterberg Limit tests were performed. The liquid limit ranged from 137 to 255, and the plasticity index ranged from 89 to 206. Based on measured index properties and visual observations the soil was classified as *organic clay* (OH) by the Unified Soil Classification system.

<u>Depth: 22 to 26 feet (Elevation: -12 to -16 feet).</u> The deepest layer consists of light gray *sandy fat clay*. The N_{TCP} values in this stratum were 4 and 7 bpf. Two Torvane tests were also performed in the field and yielded undrained shear strengths of 200 and 1800 psf. One field vane shear test was performed in this stratum and the corrected undrained shear strength was measured to be 681 psf.

Laboratory testing was performed on three tube samples for this stratum. The natural moisture content varied from 13.9 to 27.2 percent. Unconsolidated-Undrained triaxial tests were performed on two samples and the undrained shear strengths were 626 and 709 psf. One consolidated-undrained triaxial test was performed and the undrained shear strength was 676 psf. Two Atterberg Limit tests were performed. The liquid limit ranged from 91 to 150, and the plasticity index ranged from 62 to 105. Based on measured index properties and visual observations the soil was classified as *sandy fat clay* (CH) by the Unified Soil Classification system.

At the time of the subsurface investigation, groundwater was determined to be at a level 2.5 feet below the ground surface based on a piezometer installed near the borehole by Tolunay-Wong Engineers, Inc.

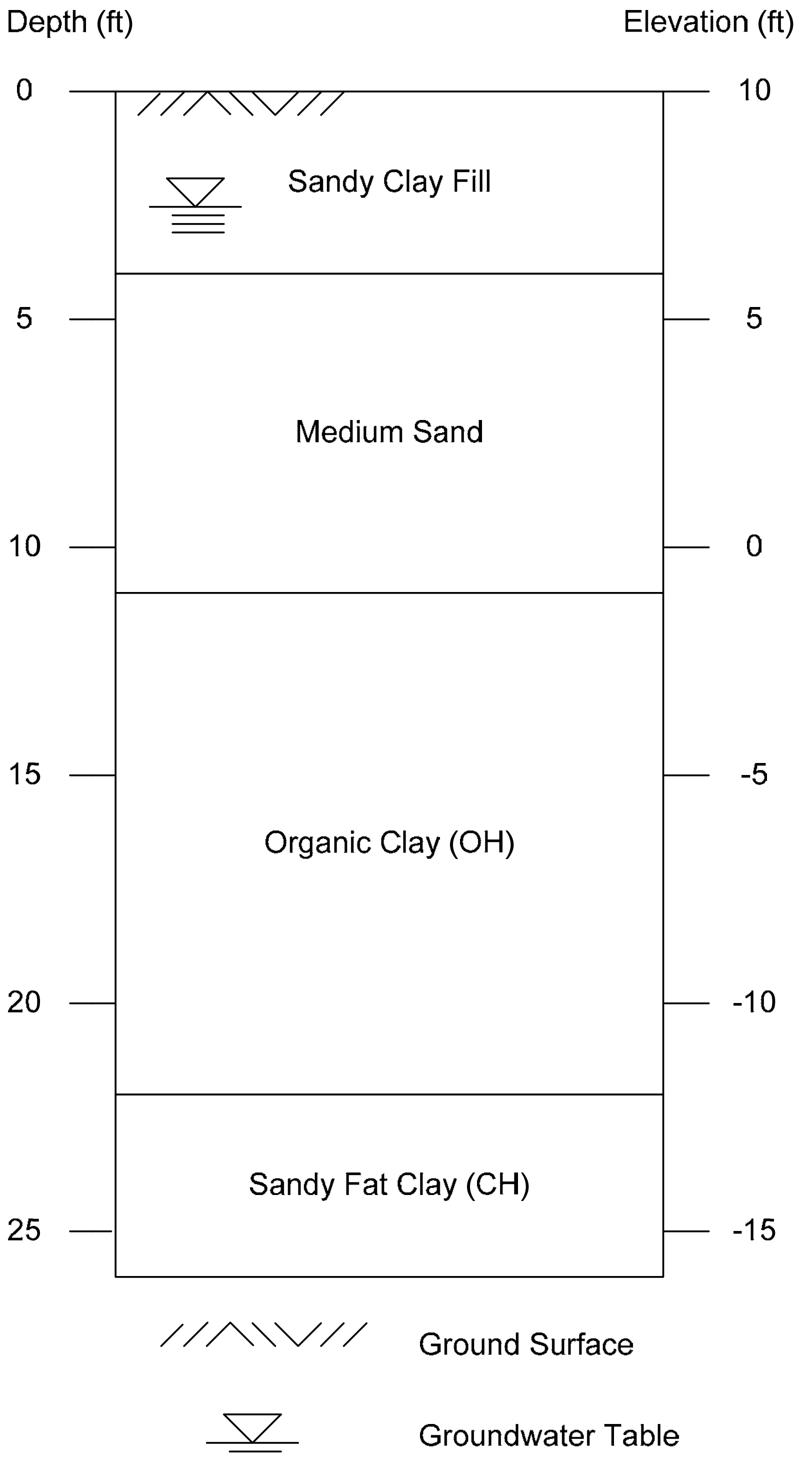


Figure 4.12: Stratigraphic Profile of Site No. 4

4.5 Site No. 5 – Cameron Parish, LA (Site A)

This site and Site No. 6 are located south of the Gulf Coast Highway and slightly east of Sabine Pass in Cameron Parish, Louisiana (Figure 4.9). The location of the Site No. 5 exploration is denoted as B-1 on this figure. The existing ground surface at the site is at an elevation of approximately 0 feet.

Review of Published Literature

According to the regional Geologic Atlas of Texas (Houston Sheet), the site is underlain by Alluvium (Qal) (Flawn, 1968). The Alluvium consists of clay, silt, sand, and organic matter. The depositional environments include point bar, natural levee, stream channel, backswamp, coastal marsh, mud flat, and narrow beach deposits.

Field Exploration

The field exploration program consisted of the following:

- drilling and sampling one 26-foot-deep boring with Texas Highway Department cone penetration (TCP) tests performed at 2-foot intervals;
- drilling, logging and obtaining thin-walled tube soil samples continuously for a second 26-foot-deep boring; and
- conducting eight field vane shear tests at varying depths.

Standard ("Dutch") piezocone penetration tests were not performed for this site. The location of the borings and additional field tests was selected by Tolunay-Wong Engineers, Inc. All of the borings were located in close proximity to one another within the subject site.

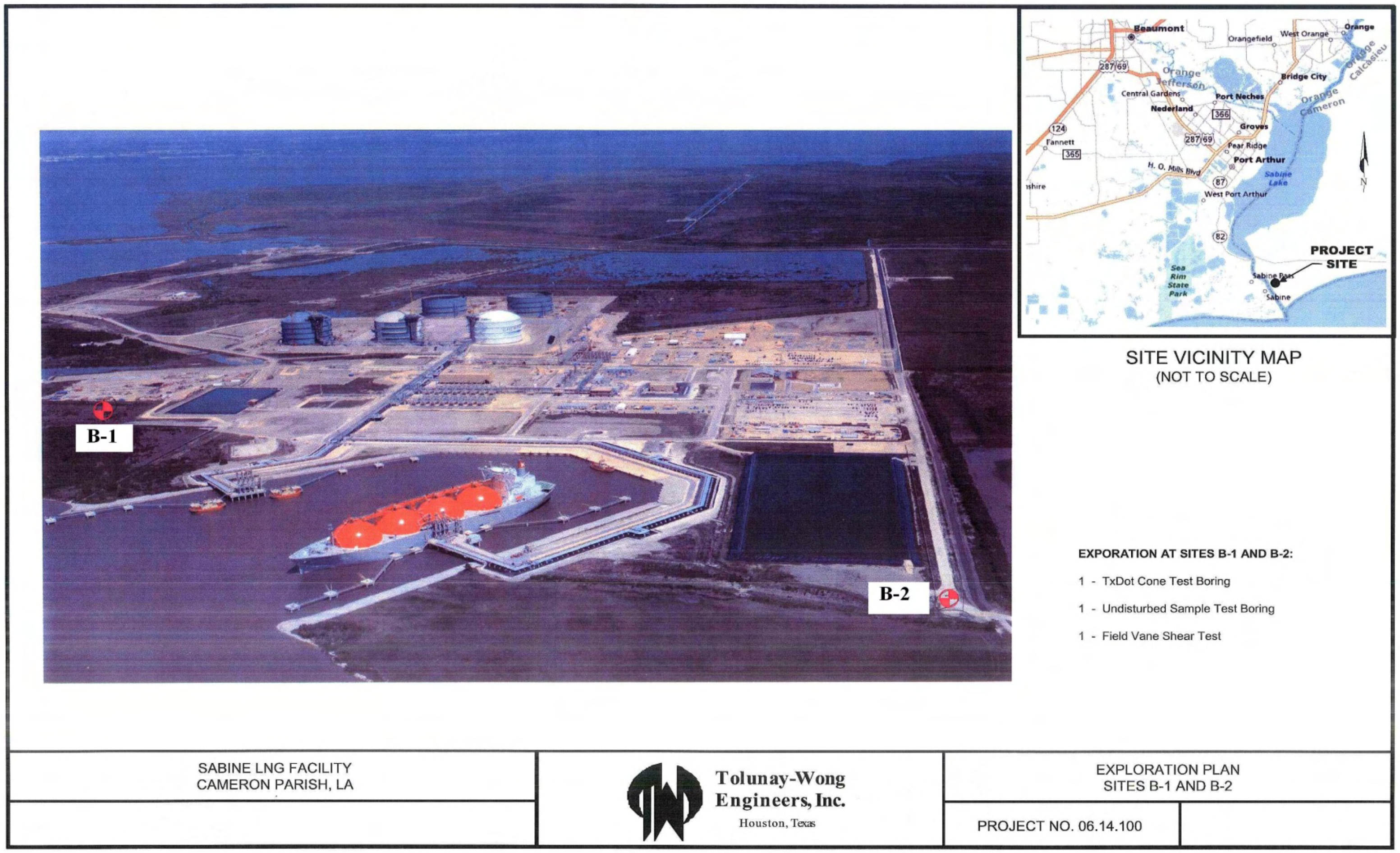


Figure 4.13: Site and Boring Location Map for Sites No. 5 and No. 6

Laboratory Testing

Laboratory testing was performed on soil samples from the thin-walled tubes. The following tests were performed:

- sixteen unconsolidated-undrained (UU) triaxial compression tests;
- five consolidated-undrained (CU) triaxial compression tests;
- one one-dimensional consolidation test using controlled-strain loading;
- Liquid Limit, Plastic Limit, and Plasticity Index Tests on four specimens; and
- moisture content tests on various specimens.

The laboratory results are summarized in the following section for each soil strata. Detailed results are presented and discussed more thoroughly by Varathungarajan (2008).

Stratigraphic Profile

The subsurface profile is generalized as follows, beginning at the ground surface (Figure 4.10):

> <u>Depth: 0 to 2 feet (Elevation: 0 to -2 feet).</u> The surficial layer consists of a firm gray *sandy clay fill*. Undrained shear strength was measured with a pocket penetrometer on one sample in the field; the undrained shear strength was 750 psf, which is indicative of soil with a medium stiff consistency.
>
> <u>Depth: 2 to 26 feet (Elevation: -2 to -26 feet).</u> The next layer consists of soft to very soft gray *fat clay* and *sandy fat clay*. The N_{TCP} values in this stratum ranged from 2 to 8 bpf. Twelve Torvane tests were performed in the field and yielded an average undrained shear strength of 180 psf. Eight field vane shear tests were performed in this stratum; the corrected undrained shear strengths measured ranged from 150 psf to 326 psf with an average of 244 psf.
>
> Laboratory testing was performed on twenty-one tube samples from this stratum. The natural moisture content varied from 40.6 to 105.1 percent. Unconsolidated-Undrained triaxial tests were performed on sixteen samples and the representative undrained shear strengths ranged from 136 to 419 psf with an average of 250 psf. Consolidated-Undrained triaxial tests were performed on five samples and the undrained shear strengths ranged from 370 to 774 psf with an average strength of 470 psf. Four Atterberg Limit tests were performed. The liquid limit ranged from 51 to 98, and the plasticity index ranged from 31 to 60. Based on measured index properties and visual observations the soil was classified as *fat clay* (CH) by the Unified Soil Classification system.

At the time of the subsurface investigation, groundwater was determined to be at a level 8 feet below the ground surface from visual observations made by Tolunay-Wong Engineers, Inc.

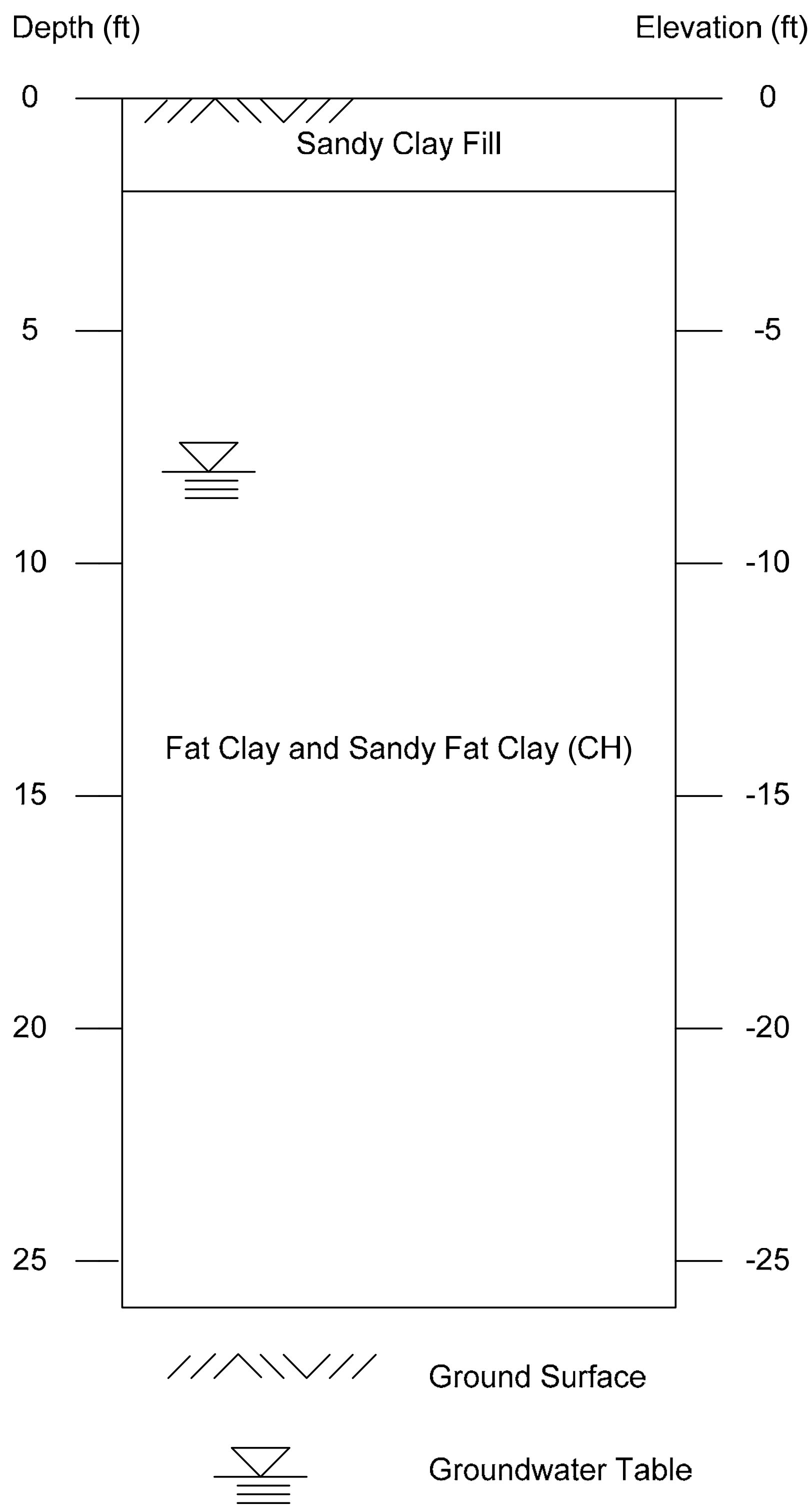


Figure 4.14: Stratigraphic Profile of Site No. 5

4.6 Site No. 6 – Cameron Parish, LA (Site B)

This site is located approximately one mile southeast of Site No. 5 in Cameron Parish, Louisiana (Figure 4.9). The location of the Site No. 6 exploration is denoted as B-2 on this figure. The existing ground surface at the site is at an elevation of approximately 0 feet. The site's geologic history is similar to that of Site No. 5.

Field Exploration

The field exploration program consisted of the following:

- drilling and sampling one 26-foot-deep boring with Texas Highway Department cone penetration (TCP) tests performed at 2-foot intervals;
- drilling, logging and obtaining thin-walled tube soil samples continuously for a second 26-foot-deep boring; and
- conducting eight field vane shear tests at varying depths.

Standard ("Dutch") piezocone penetration tests were not performed for this site. The location of the borings and additional field tests was selected by Tolunay-Wong Engineers, Inc. All of the borings were located in close proximity to one another within the subject site.

Laboratory Testing

Laboratory testing was performed on soil samples from the thin-walled tubes. The following tests were performed:

- seventeen unconsolidated-undrained (UU) triaxial compression tests;
- four consolidated-undrained (CU) triaxial compression tests;
- one one-dimensional consolidation test using controlled-strain loading;
- Liquid Limit, Plastic Limit, and Plasticity Index Tests on four specimens; and
- moisture content tests on various specimens.

The laboratory results are summarized in the following section for each soil strata. Detailed results are presented and discussed more thoroughly by Varathungarajan (2008).

Stratigraphic Profile

The subsurface profile is generalized as follows, beginning at the ground surface (Figure 4.11):

Depth: 0 to 2 feet (Elevation: 0 to -2 feet). The surficial layer consists of a firm gray *sandy clay fill*. Undrained shear strength was measured with a pocket penetrometer on one sample in the field; the undrained shear strength was 750 psf, which is indicative of soil with a medium stiff consistency.

Depth: 2 to 26 feet (Elevation: -2 to -26 feet). The next layer consists of soft to very soft gray *fat clay* and *sandy fat clay*. The N_{TCP} values in this stratum ranged from WOH to 7 bpf. Ten Torvane tests were performed in the field and yielded an average undrained shear strength of 250 psf. Two pocket penetrometer tests were also performed and undrained shear strengths of 750 psf were measured in each test. Eight field vane shear tests were performed in this stratum; the corrected undrained shear strengths measured ranged from 98 psf to 798 psf with an average of 406 psf.

Laboratory testing was performed on twenty-one tube samples for this stratum. The natural moisture content varied from 25.5 to 89.2 percent. Unconsolidated-Undrained triaxial tests were performed on seventeen samples and the representative undrained shear strengths ranged from 176 to 822 psf with an average of 400 psf. Consolidated-Undrained triaxial tests were performed on four samples and the representative undrained shear strengths ranged from 498 to 732 psf with an average strength of 620 psf. Four Atterberg Limit tests were performed. The liquid limit ranged from 52 to 84, and the plasticity index ranged from 30 to 49. Based on measured index properties and visual observations the soil was classified as *fat clay* (CH) by the Unified Soil Classification system.

At the time of the subsurface investigation, groundwater was determined to be at a level 8 feet below the ground surface from visual observations made by Tolunay-Wong Engineers, Inc.

4.7 Summary

Data from the conventional ("Dutch") piezocone penetration tests, vane shear tests and triaxial tests are presented and analyzed in more detail by Varathungarajan (2008). Varathungarajan (2008) also developed representative strength profiles for each site (Appendix A). These strength profiles are used in subsequent chapters to evaluate the reliability of existing Texas Cone Penetrometer correlations between N_{TCP} and undrained shear strength. The development of a new improved correlation will also be investigated.

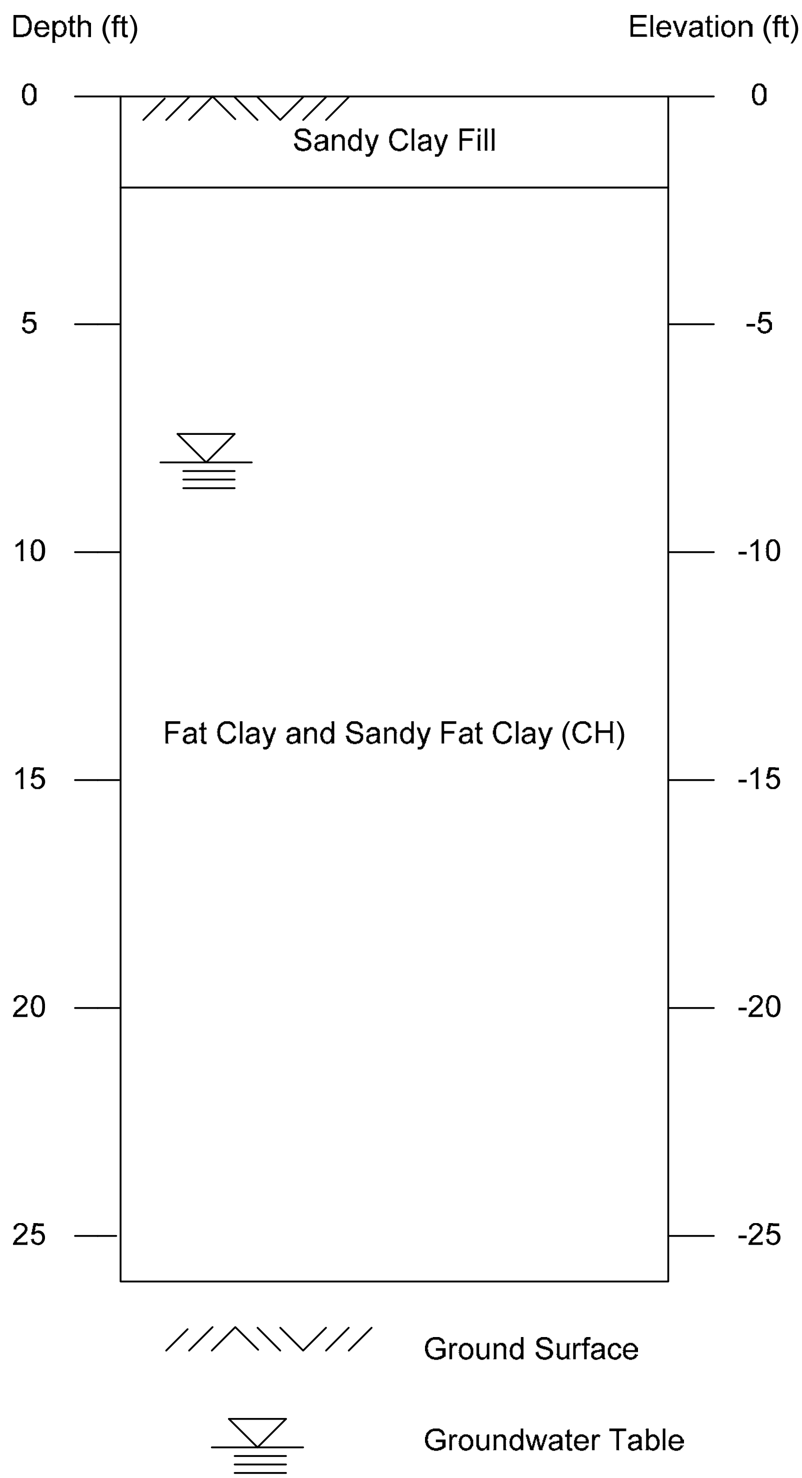


Figure 4.15: Stratigraphic Profile of Site No. 6

Chapter 5. Analysis of the Texas Cone Penetrometer Correlations

For each of the six sites described in Chapter 4, representative shear strength profiles were developed using the laboratory data and data from the field vane and piezocone penetration tests. The development of the undrained strength profiles is described by Varathungarajan (2008). He developed three strength profiles—an *average*, lower- and upper-bound—for each site (Appendix A). Once the strength profiles were established, they were used to establish and examine correlations between undrained shear strength and Texas Cone Penetrometer blow counts. The examination of these correlations is presented in this chapter and an improved correlation is proposed.

5.1 Evaluation of Existing Correlations

For each of the six sites, undrained shear strengths were computed using the correlations proposed by Hamoudi et al. (1974), Duderstadt et al. (1977) and Kim et al. (2007). The correlations from Hamoudi et al. (1974) and Duderstadt et al. (1977) produced very similar strengths, which is expected since the correlation proposed by Duderstadt et al. (1977) was developed as an improvement to the correlation proposed by Hamoudi et al. (1974).

A strength profile corresponding to the minimum strength required to support the Texas Cone Penetrometer cone was also developed for each site. Details of these strength profiles are presented in Appendix C. In most cases, when a blow count of "weight of hammer" (W.O.H.) was recorded, the undrained shear strength of the soil was less than the calculated minimum strength required to support the cone.

Site No. 1 – Port Arthur, TX (Site A)

The undrained shear strengths computed using the correlations proposed by Hamoudi et al. (1974), Duderstadt et al. (1977), and Kim et al. (2007) for Site No. 1 are shown in Table 5.1. Undrained shear strengths at several locations in the sandy clay fill, granular fill and sand strata were not computed using the correlations because the soil was considered to be non-clay. For several of the sites there are strata that contain both clay and non-clay soils and undrained shear strengths are only computed where the soil was believed to be clay. In addition, moisture content was not taken at a depth of 25 feet and thus an undrained shear strength could not be computed using the correlation by Kim et al. (2007). The undrained shear strength profiles developed by Varathungarajan (2008) along with the undrained shear strengths computed from the Texas Cone Penetrometer tests using the correlations from Duderstadt et al. (1977) and Kim et al. (2007) are plotted versus depth in Figure 5.1. Undrained shear strengths computed using the correlations ranged from approximately 10 percent to 325 percent of the average undrained shear strength profile developed by Varathungarajan (2008). The strata considered to be non-clay are represented by a shaded area.

Table 5.10: Summary of Undrained Shear Strengths for Site No. 1 using Existing Texas Cone Penetrometer Correlations

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*Water Content (%)*	TCP Correlation		
					Hamoudi, s_u (psf)	*Duderstadt, s_u (psf)*	*Kim, s_u (psf)*
0	10	Sandy Clay Fill					
1	9		8		848	848	
2	8			23.5			3929
3	7		19	25.4	2014	2014	2857
4	6			24.7			3383
5	5		31	22.1	---	---	---
6	4						
7	3	Granular Fill	38	---	---	---	---
8	2						
9	1		7	---	980	938	---
10	0						
11	-1	Fat Clay	7	81.1	980	938	793
12	-2			71.8			558
13	-3		3	30.2	420	402	232
14	-4	Lean Clay		37.9			1050
15	-5		20	19.9	2120	2120	519
16	-6						
17	-7	Sand	20	37.5	---	---	---
18	-8			22.7			---
19	-9		15	---	---	---	---
20	-10						
21	-11	Fat Clay	29	27.2	3654	3132	371
22	-12			27.6			380
23	-13		31	26.7	3906	3348	360
24	-14			13.9			168
25	-15		26	---	3276	2808	---
26	-16						

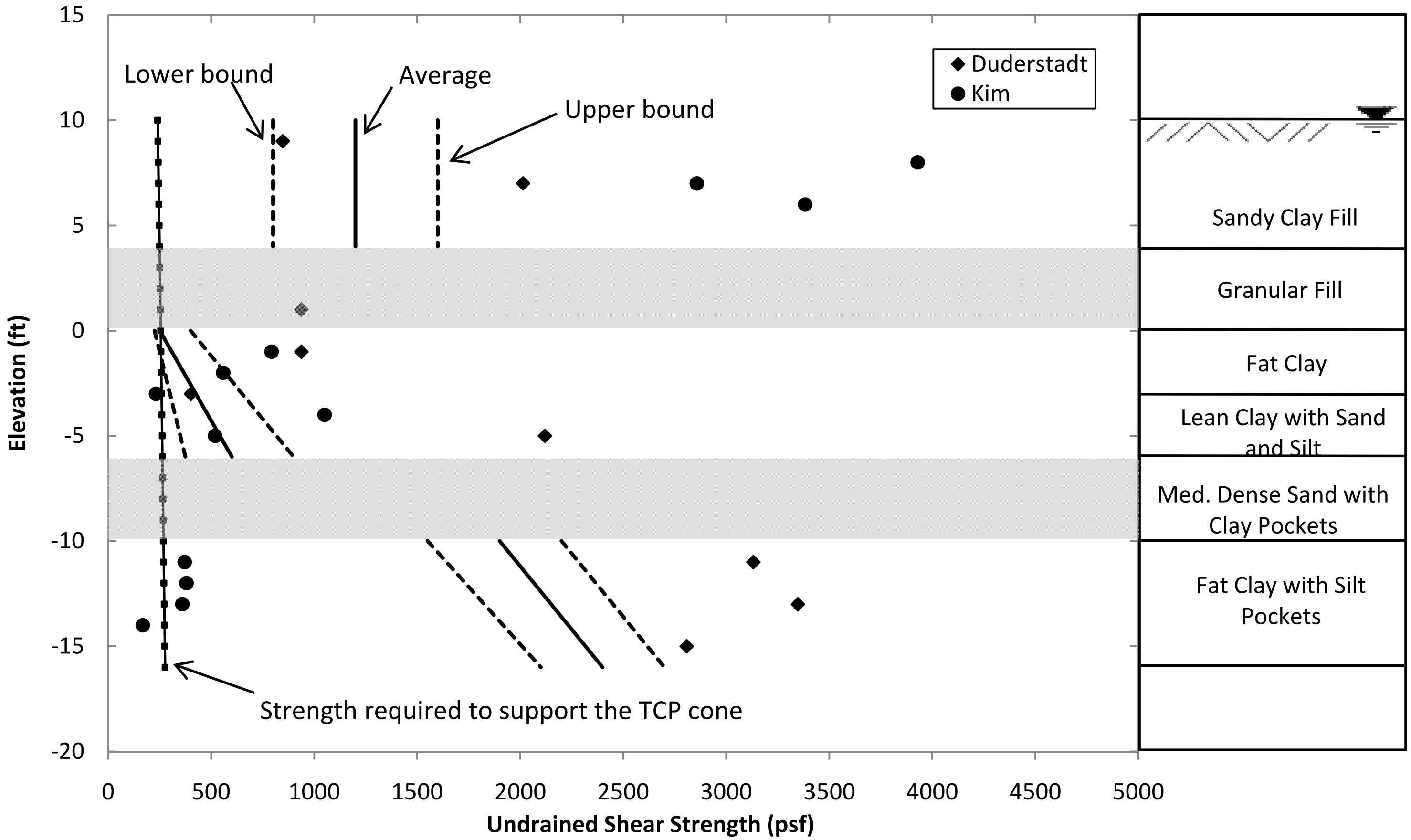


Figure 5.16: Undrained Shear Strengths for Site No. 1 using the Texas Cone Penetrometer Correlations from Duderstadt et al. (1977) and Kim et al. (2007)

Site No. 2 – Port Arthur, TX (Site B)

The undrained shear strengths computed using the correlations proposed by Hamoudi et al. (1974), Duderstadt et al. (1977), and Kim et al. (2007) for Site No. 2 are shown in Table 5.2. Undrained shear strengths at several locations in the clayey and silty sand and sandy lean clay strata were not computed using the correlations because the soil was considered to be non-clay. Also, many of the Texas Cone Penetrometer blow counts were recorded as "weight of hammer" (W.O.H.) within the fat clay and thus undrained shear strengths could also not be computed. The undrained shear strength profiles developed by Varathungarajan (2008) along with the undrained shear strengths computed from the Texas Cone Penetrometer tests using the correlations from Duderstadt et al. (1977) and Kim et al. (2007) are plotted versus depth in Figure 5.2. All undrained shear strengths computed using the correlations were within approximately 20 percent of the average undrained shear strength profile developed by Varathungarajan (2008). The arrows indicating a range and question marks in Figure 5.2 correspond to depths where the undrained shear strength was less than the strength required to support the Texas Cone Penetrometer as discussed earlier and in Appendix C. While the Texas Cone Penetrometer test could not be performed at many depths, it is worth noting that the theoretical calculations presented in Chapter 2 regarding the minimum strength required to support the cone are in agreement with the field observations at this site.

Table 5.11: Summary of Undrained Shear Strengths for Site No. 2 using Existing Texas Cone Penetrometer Correlations

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*Water Content (%)*	TCP Correlation		
					Hamoudi, s_u (psf)	*Duderstadt, s_u (psf)*	*Kim, s_u (psf)*
0	3						
1	2		11	---	---	---	---
2	1	Clayey and Silty Sand					
3	0		3	---	---	---	---
4	-1						
5	-2		2	---	280	268	---
6	-3						
7	-4		2	40.7	280	268	324
8	-5			33.7			262
9	-6			100.4			---
10	-7		W.O.H.	68.2	---	---	---
11	-8			100.5			---
12	-9			76.0			---
13	-10			46.0			---
14	-11		W.O.H.	51.6	---	---	---
15	-12			76.1			---
16	-13			86.1			---
17	-14	Fat Clay		93.6			---
18	-15			93.4			---
19	-16						
20	-17		W.O.H.	61.8	---	---	---
21	-18			59.3			---
22	-19			59.9			---
23	-20			72.5			---
24	-21			67.6			---
25	-22			35.2			---
26	-23		W.O.H.	35.8	---	---	---
27	-24			34.6			---
28	-25			65.0			---
29	-26						
30	-27						
31	-28		8	---	---	---	---
32	-29	Sandy Lean Clay					
33	-30		2	---	---	---	---
34	-31						
35	-32						

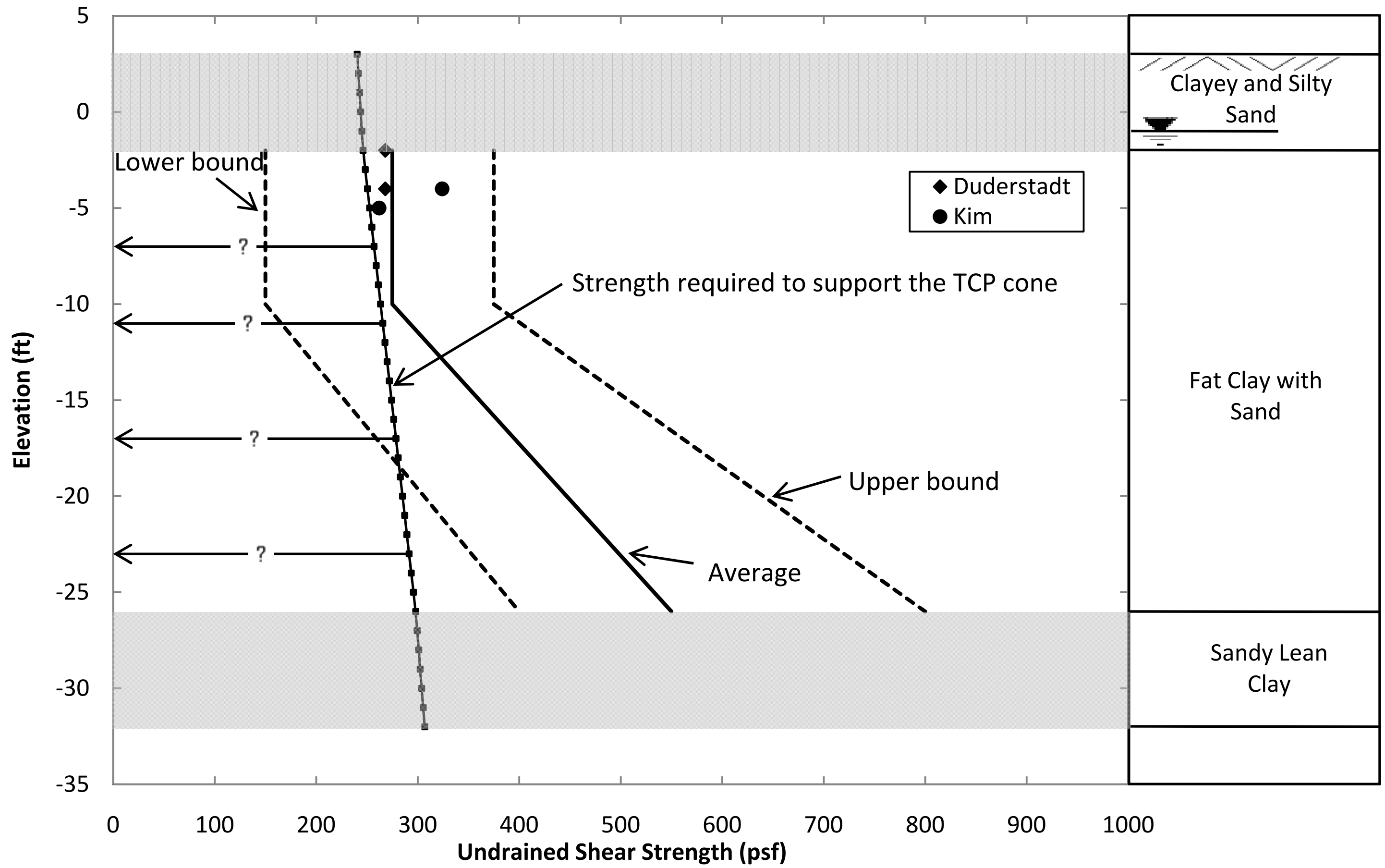


Figure 5.17: Undrained Shear Strengths for Site No. 2 using the Texas Cone Penetrometer Correlations from Duderstadt et al. (1977) and Kim et al. (2007)

Site No. 3 – Mont Belvieu, TX

The undrained shear strengths computed using the correlations proposed by Hamoudi et al. (1974), Duderstadt et al. (1977), and Kim et al. (2007) for Site No. 3 are shown in Table 5.3. Undrained shear strengths were not computed in the clayey sand stratum because the soil was considered to be non-clay. The undrained shear strength profiles developed by Varathungarajan (2008) along with the undrained shear strengths computed from the Texas Cone Penetrometer tests using the correlations from Duderstadt et al. (1977) and Kim et al. (2007) are plotted versus depth in Figure 5.3. Undrained shear strengths computed using the correlations ranged from approximately 40 percent to 155 percent of the average undrained shear strength profile developed by Varathungarajan (2008).

Table 5.12: Summary of Undrained Shear Strengths for Site No. 3 using Existing Texas Cone Penetrometer Correlations

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*Water Content (%)*	TCP Correlation		
					Hamoudi, s_u (psf)	*Duderstadt, s_u (psf)*	*Kim, s_u (psf)*
0	31	Lean and Fat CLAY					
1	30		6		840	804	
2	29			31.0			683
3	28		6	32.2	840	804	543
4	27			32.0			456
5	26		6	30.0	840	804	381
6	25			29.7			398
7	24		8	42.3	1120	1072	525
8	23						
9	22		8		1120	1072	
10	21			25.6			520
11	20		10	25.4	1060	1060	476
12	19						
13	18		12	41.2	1680	1608	464
14	17						
15	16		7	40.3	742	742	447
16	15						
17	14		14	25.0	1484	1484	1112
18	13						
19	12	Clayey SAND	14	---	---	---	---
20	11						
21	10		13	---	---	---	---
22	9			24.8			---
23	8		12	28.8	---	---	---
24	7			28.3			---
25	6		12	27.2	---	---	---
26	5						

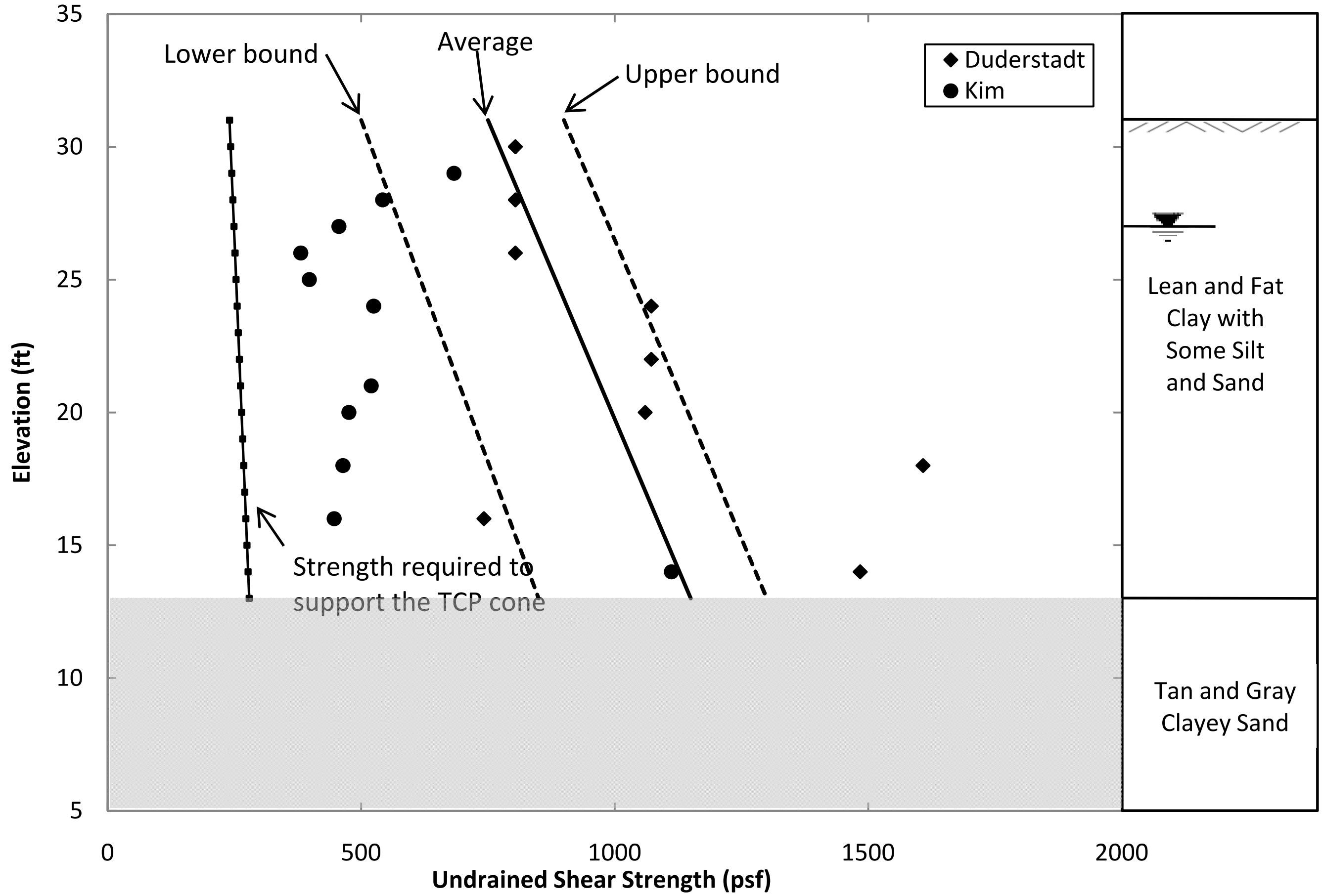


Figure 5.18: Undrained Shear Strengths for Site No. 3 using the Texas Cone Penetrometer Correlations from Duderstadt et al. (1977) and Kim et al. (2007)

Site No. 4 – Beaumont, TX

The undrained shear strengths computed using the correlations proposed by Hamoudi et al. (1974), Duderstadt et al. (1977) and Kim et al. (2007) for Site No. 4 are shown in Table 5.4. Undrained shear strengths at most locations in the sand stratum were not computed using the correlations because the soil was considered to be non-clay. Also, two of the Texas Cone Penetrometer blow counts were recorded as “weight of hammer” (W.O.H.) in the organic clay and thus undrained shear strengths could also not be computed. The undrained shear strength profiles developed by Varathungarajan (2008) along with the undrained shear strengths computed from the Texas Cone Penetrometer tests using the correlations from Duderstadt et al. (1977) and Kim et al. (2007) are plotted versus depth in Figure 5.4. Undrained shear strengths computed using the correlations ranged from approximately 45 percent to 335 percent of the average undrained shear strength profile developed by Varathungarajan (2008). The arrows indicating a range and question marks correspond to depths where the undrained shear strength was less than the strength required to support the Texas Cone Penetrometer as discussed earlier and in Appendix C.

Table 5.13: Summary of Undrained Shear Strengths for Site No. 4 using Existing Texas Cone Penetrometer Correlations

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*Water Content (%)*	TCP Correlation *Hamoudi, s_u (psf)*	TCP Correlation *Duderstadt, s_u (psf)*	TCP Correlation *Kim, s_u (psf)*
0	10	Sandy Clay Fill					
0.5	9.5						
1	9		8		848	848	
1.5	8.5						
2	8			20.2			934
2.5	7.5						
3	7		5	28.5	530	530	907
3.5	6.5			25.1			696
4	6						
4.5	5.5	Sand					
5	5		17	---	---	---	---
5.5	4.5						
6	4						
6.5	3.5						
7	3		18	---	---	---	---
7.5	2.5						
8	2						
8.5	1.5						
9	1		8	---	---	---	---
9.5	0.5						
10	0						
10.5	-0.5						
11	-1		3	---	318	318	---
11.5	-1.5	Organic Clay					
12	-2						
12.5	-2.5			83.8			647
13	-3		3	114.9	420	402	882
13.5	-3.5			113.1			863
14	-4			74.1			---
14.5	-4.5						
15	-5		W.O.H.	172.7	---	---	---
15.5	-5.5			123.1			---
16	-6			94.7			---
16.5	-6.5						
17	-7		W.O.H.	127.5	---	---	---
17.5	-7.5			181.7			---
18	-8			169.7			1246
18.5	-8.5						
19	-9		3	263.9	420	402	1926
19.5	-9.5			162.9			1186
20	-10			192.7			1349
20.5	-10.5						
21	-11		2	162.7	280	268	1135
21.5	-11.5			163.0			1135
22	-12			124.6			866
22.5	-12.5	Sandy Fat Clay		134.3			994
23	-13		4	88.1	424	424	650
23.5	-13.5			99.4			732
24	-14			96.7			773
24.5	-14.5						
25	-15		7	65.8	742	742	522
25.5	-15.5						
26	-16						

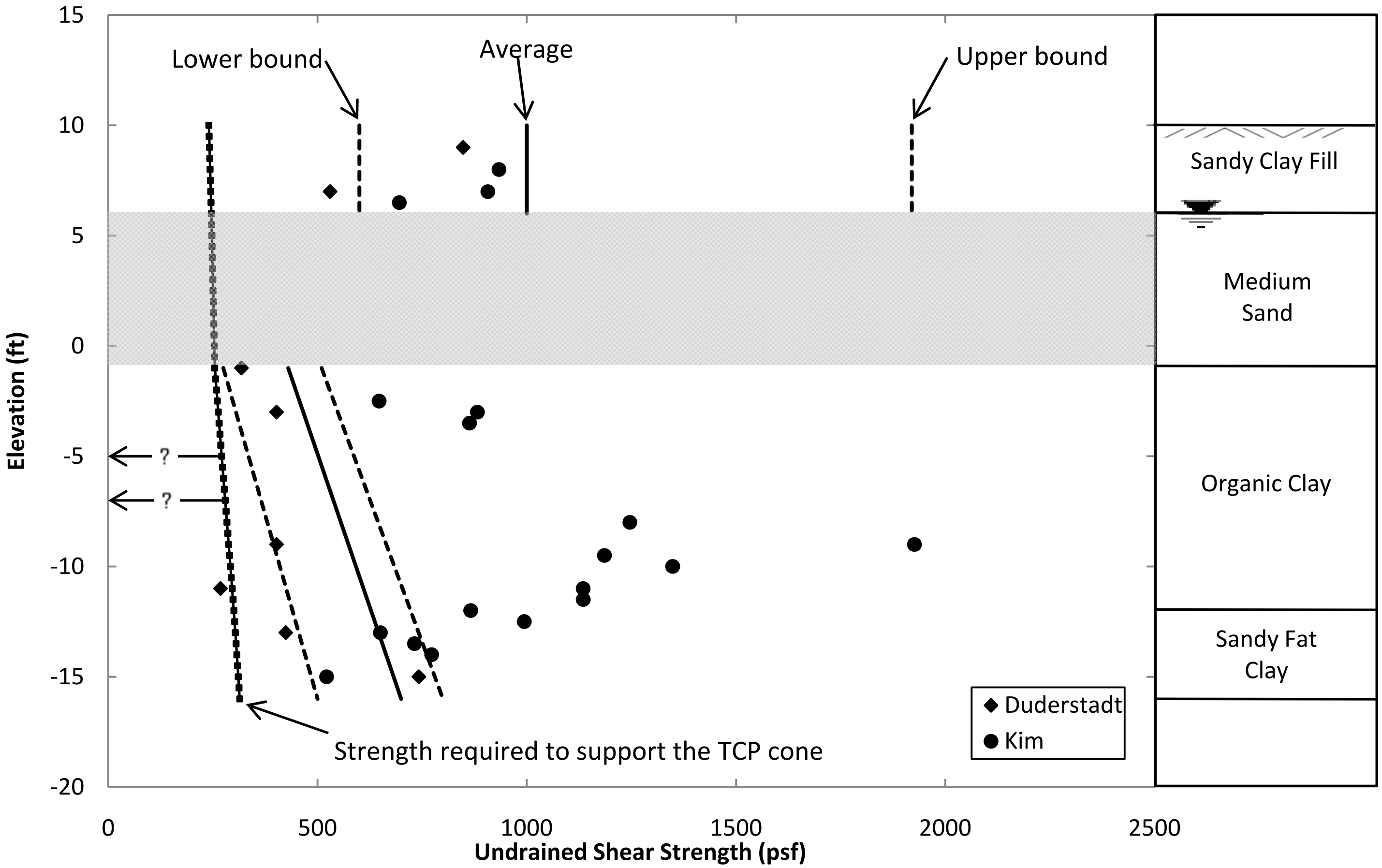


Figure 5.19: Undrained Shear Strengths for Site No. 4 using the Texas Cone Penetrometer Correlations from Duderstadt et al. (1977) and Kim et al. (2007)

Site No. 5 – Cameron Parish, LA (Site A)

The undrained shear strengths computed using the correlations proposed by Hamoudi et al. (1974), Duderstadt et al. (1977), and Kim et al. (2007) for Site No. 5 are shown in Table 5.5. Undrained shear strengths were not computed in the sandy clay fill stratum because the soil was considered to be non-clay. The undrained shear strength profiles developed by Varathungarajan (2008) along with the undrained shear strengths computed from the Texas Cone Penetrometer tests using the correlations from Duderstadt et al. (1977) and Kim et al. (2007) are plotted versus depth in Figure 5.5. Undrained shear strengths computed using the correlations ranged from approximately 105 percent to 340 percent of the average undrained shear strength profile developed by Varathungarajan (2008).

Table 5.14: Summary of Undrained Shear Strengths for Site No. 5 using Existing Texas Cone Penetrometer Correlations

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*Water Content (%)*	TCP Correlations *Hamoudi, s_u (psf)*	*Duderstadt, s_u (psf)*	*Kim, s_u (psf)*
0	0	Sandy Clay Fill					
0.5	-0.5						
1	-1						
1.5	-1.5						
2	-2						
2.5	-2.5	Fat Clay and Sandy Fat Clay					
3	-3		3	48.4	420	402	565
3.5	-3.5			57.2			625
4	-4						
4.5	-4.5						
5	-5		2	41.6	280	268	356
5.5	-5.5			40.6			340
6	-6			53.8			333
6.5	-6.5						
7	-7		2	48.1	280	268	383
7.5	-7.5			43.0			338
8	-8			92.6			720
8.5	-8.5						
9	-9		2	95.8	280	268	731
9.5	-9.5			92.7			702
10	-10			97.6			784
10.5	-10.5						
11	-11		3	96.1	318	318	759
11.5	-11.5			105.1			823
12	-12			83.6			614
12.5	-12.5						
13	-13		2	78.3	280	268	570
13.5	-13.5			88.3			640
14	-14			65.1			470
14.5	-14.5						
15	-15		2	52.5	280	268	376
15.5	-15.5			58.0			415
16	-16			83.0			592
16.5	-16.5						
17	-17		2	78.0	280	268	553
17.5	-17.5			51.3			363
18	-18						
18.5	-18.5						
19	-19		3		318	318	
19.5	-19.5						
20	-20						
20.5	-20.5						
21	-21		3	45.1	318	318	326
21.5	-21.5			50.2			362
22	-22						
22.5	-22.5						
23	-23		7	68.5	742	742	552
23.5	-23.5						
24	-24						
24.5	-24.5						
25	-25		8	76.1	848	848	619
25.5	-25.5			77.2			626
26	-26						

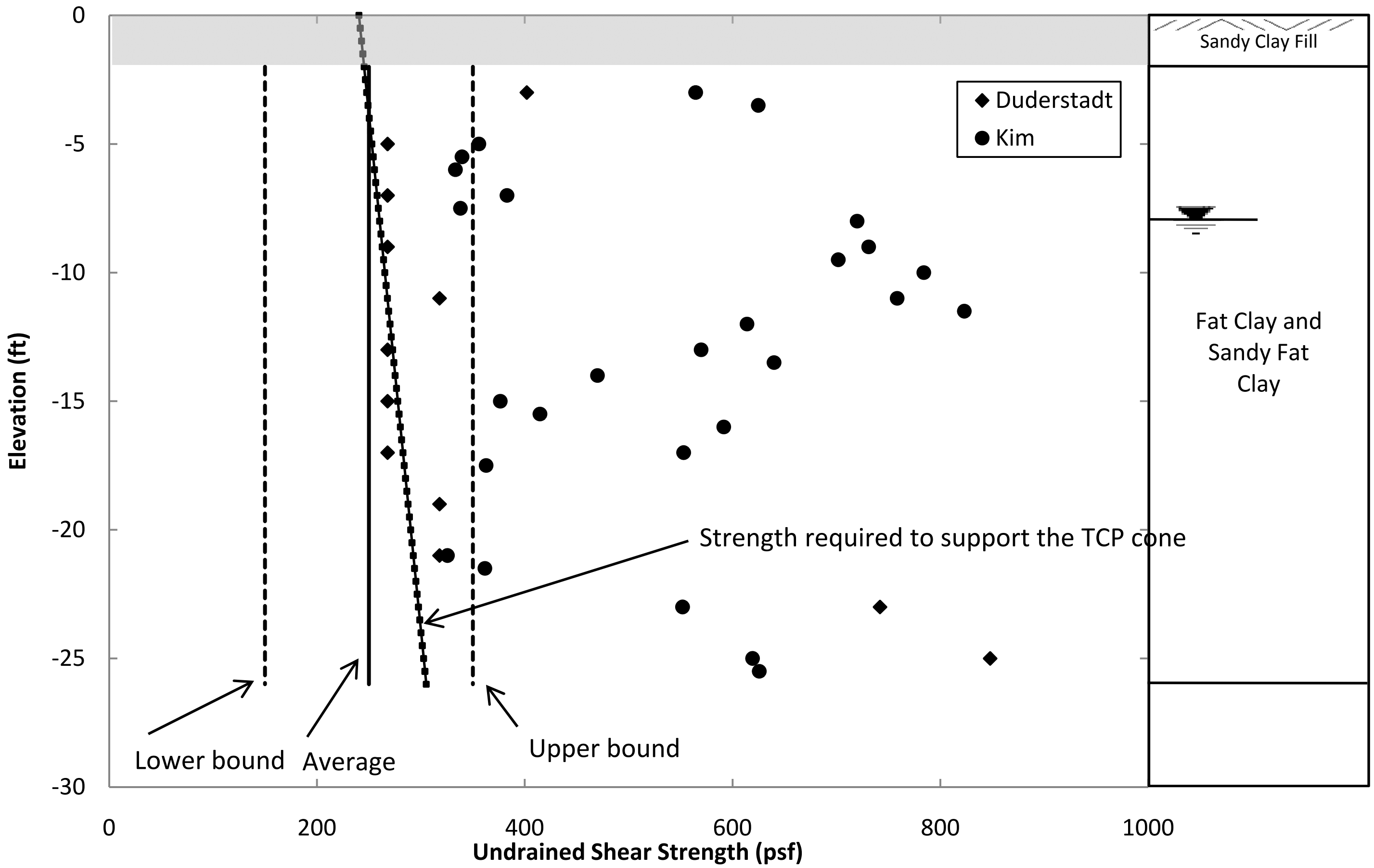


Figure 5.20: Undrained Shear Strengths for Site No. 5 using the Texas Cone Penetrometer Correlations from Duderstadt et al. (1977) and Kim et al. (2007)

Site No. 6 – Cameron Parish, LA (Site B)

The undrained shear strengths computed using the correlations proposed by Hamoudi et al. (1974), Duderstadt et al. (1977), and Kim et al. (2007) for Site No. 6 are shown in Table 5.6. Undrained shear strengths were not computed in the sandy clay fill stratum because the soil was considered to be non-clay. Also, one of the Texas Cone Penetrometer blow counts was recorded as "weight of hammer" (W.O.H.) within the Clay and thus an undrained shear strength could also not be computed. The undrained shear strength profiles developed by Varathungarajan (2008) along with the undrained shear strengths computed from the Texas Cone Penetrometer tests using the correlations from Duderstadt et al. (1977) and Kim et al. (2007) are plotted versus depth in Figure 5.6. Undrained shear strengths computed using the correlations ranged from approximately 35 percent to 175 percent of the average undrained shear strength profile developed by Varathungarajan (2008). The arrow indicating a range and question mark correspond to the depth where the undrained shear strength was less than the strength required to support the Texas Cone Penetrometer as discussed earlier and in Appendix C.

Table 5.15: Summary of Undrained Shear Strengths for Site No. 6 using Existing Texas Cone Penetrometer Correlations

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*Water Content (%)*	TCP Correlation *Hamoudi, s_u (psf)*	*Duderstadt, s_u (psf)*	*Kim, s_u (psf)*
0	0	Sandy Clay Fill					
0.5	-0.5						
1	-1						
1.5	-1.5						
2	-2						
2.5	-2.5	Fat Clay and Sandy Fat Clay					
3	-3		2		280	268	
3.5	-3.5			52.7			498
4	-4						
4.5	-4.5			49.0			430
5	-5		2	57.8	280	268	494
5.5	-5.5			37.5			314
6	-6						
6.5	-6.5						
7	-7		2	50.5	280	268	402
7.5	-7.5			37.5			295
8	-8			40.7			369
8.5	-8.5						
9	-9		4		560	536	
9.5	-9.5			45.0			390
10	-10						
10.5	-10.5						
11	-11		6		840	804	
11.5	-11.5			50.7			466
12	-12						
12.5	-12.5			69.7			---
13	-13		W.O.H	78.7	---	---	---
13.5	-13.5			76.6			---
14	-14			67.5			487
14.5	-14.5						
15	-15		2	89.2	280	268	640
15.5	-15.5						
16	-16						
16.5	-16.5			79.0			562
17	-17		2	55.5	212	212	393
17.5	-17.5			48.7			344
18	-18						
18.5	-18.5						
19	-19		3		318	318	
19.5	-19.5						
20	-20			83.2			582
20.5	-20.5						
21	-21		2	64.6	280	268	451
21.5	-21.5			62.7			437
22	-22						
22.5	-22.5						
23	-23		3	25.4	318	318	182
23.5	-23.5						
24	-24						
24.5	-24.5						
25	-25		7		980	938	
25.5	-25.5						
26	-26						

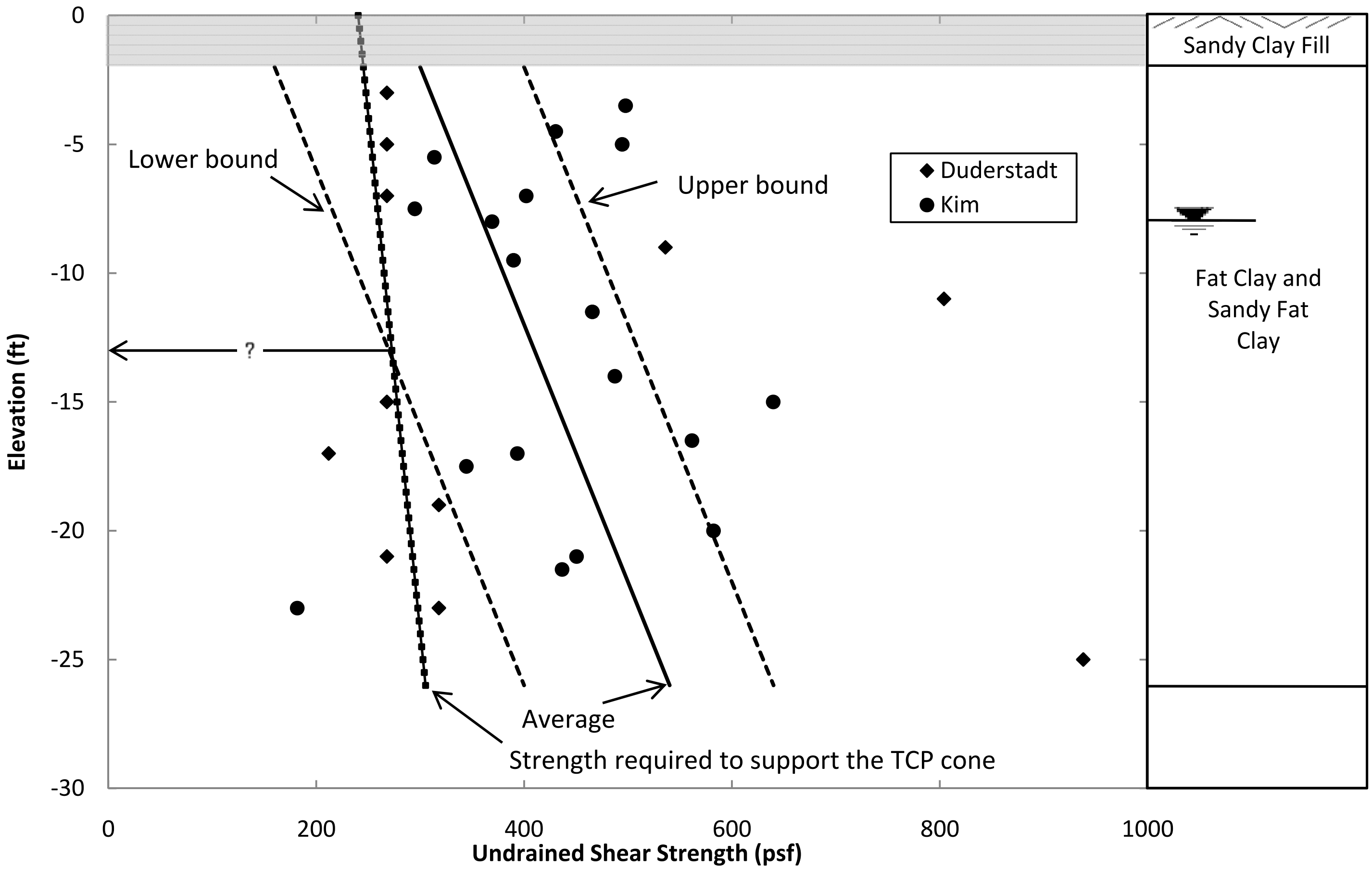


Figure 5.21: Undrained Shear Strengths for Site No. 6 using the Texas Cone Penetrometer Correlations from Duderstadt et al. (1977) and Kim et al. (2007)

5.2 Discussion of Existing Correlations

The correlations by Kim et al. (2007) tend to produce highly variable undrained strengths, in many cases ranging from either significantly greater or less than what are believed to be the actual strengths. The maximum underestimates for each site ranged from 8 to 95 percent of the average undrained shear strength profile developed by Varathungarajan (2008) while the maximum overestimates for each site ranged from 118 to 336 percent of the average undrained shear strength profile developed by Varathungarajan (2008). Furthermore, the correlation suggests that as the moisture content increases, the strength increases, which seems fundamentally unsound. As a result, this correlation was not considered further in this study.

The correlation by Duderstadt et al. (1977) showed reasonable agreement with the upper portions of the undrained shear strength profiles developed for the sites, but showed a tendency to overestimate the strengths in the deeper portion of the soil profiles. Furthermore, it seems reasonable that even with a zero ("weight of hammer") blow count, if the soil can support the weight of the cone, it has some strength. This suggests that the equation for computing the undrained shear strength of the soil should be of the form,

$$s_u = s_0 + KN \qquad (5.1)$$

where s_0 is the strength required to support the weight of the cone and K is the increase in strength associated with each blow count. Referring to Chapter 2 where the minimum strengths required to support the weight of the cone were examined, a nominal strength of approximately 300 (± 50) psf would be required to support the weight of the cone for depths up to approximately 30 feet, which is the depth range of interest. Thus, Equation 5.1 could be written as,

$$s_u = 300 + KN \qquad (5.2)$$

in units of psf for undrained shear strength. The ratio of the average undrained shear strength profile from Varathungarajan (2008) to the minimum undrained shear strength required to support the cone is plotted versus depth in Figure 5.7. All ratios were approximately equal to or greater than 1.0 suggesting that the Texas Cone Penetrometer test could be performed.

The correlation provided by Duderstadt et al. (1977) suggested that for clays the undrained shear strength increased at a rate ranging from 106 to 134 psf per blow with a nominal average value of 120 psf per blow. A line corresponding to 120 psf per blow is plotted in Figure 5.8 along with measured undrained shear strength and Texas Cone Penetrometer blow count values. Data from Hamoudi et al. (1974) and Duderstadt et al. (1977) as well as the current study are all shown in this figure. Examination of the data in this figure suggests that the strengths may be overestimated by nearly a factor of two in some cases by using a factor of 120 psf per blow. Strength increasing at a rate of 60 psf per blow count is probably a more reasonable, safe lower bound.

Based on the above reasoning, the following improved equation is proposed:

$$s_u = 300 + 60N \qquad (5.3)$$

Equation 5.3 can also be presented in the following form where the constants are independent of units:

$$s_u = p_{atmosphere}(0.142 + 0.028N) \qquad (5.4)$$

where $p_{atmosphere}$ is atmospheric pressure. In this form the units for s_u will be the same as the units for atmospheric pressure and any set of units can be chosen. For example if the units are in pounds per square inch (psi), atmospheric pressure is 14.7 psi and Equation 5.4 appears as

$$s_u = 14.7(0.142 + 0.028N) \qquad (5.5)$$

In this case Equation 5.5 yields the undrained shear strength in units of psi.

A line corresponding to Equation 5.3 for blow counts of 15 or less is plotted in Figure 5.9. Also plotted in this figure are the measured undrained shear strength and blow count values for the six sites from this study as well as the data from Hamoudi et al. (1974) and Duderstadt et al. (1977). Finally, a line corresponding to the correlation by Duderstadt eta al. (1977) using a nominal value of 120 for the strength-to-blow count value is shown. The line corresponding to Equation 5.3 is believed to represent a better, but conservative estimate of the undrained shear strengths. The correlation provided in Equation 5.3 will be referred to as the improved correlation from this point on. This correlation is intended primarily for soft clays with undrained shear strengths of 1200 psf or less (TCP blow counts of 15 or less), which is the range of interest in this present study.

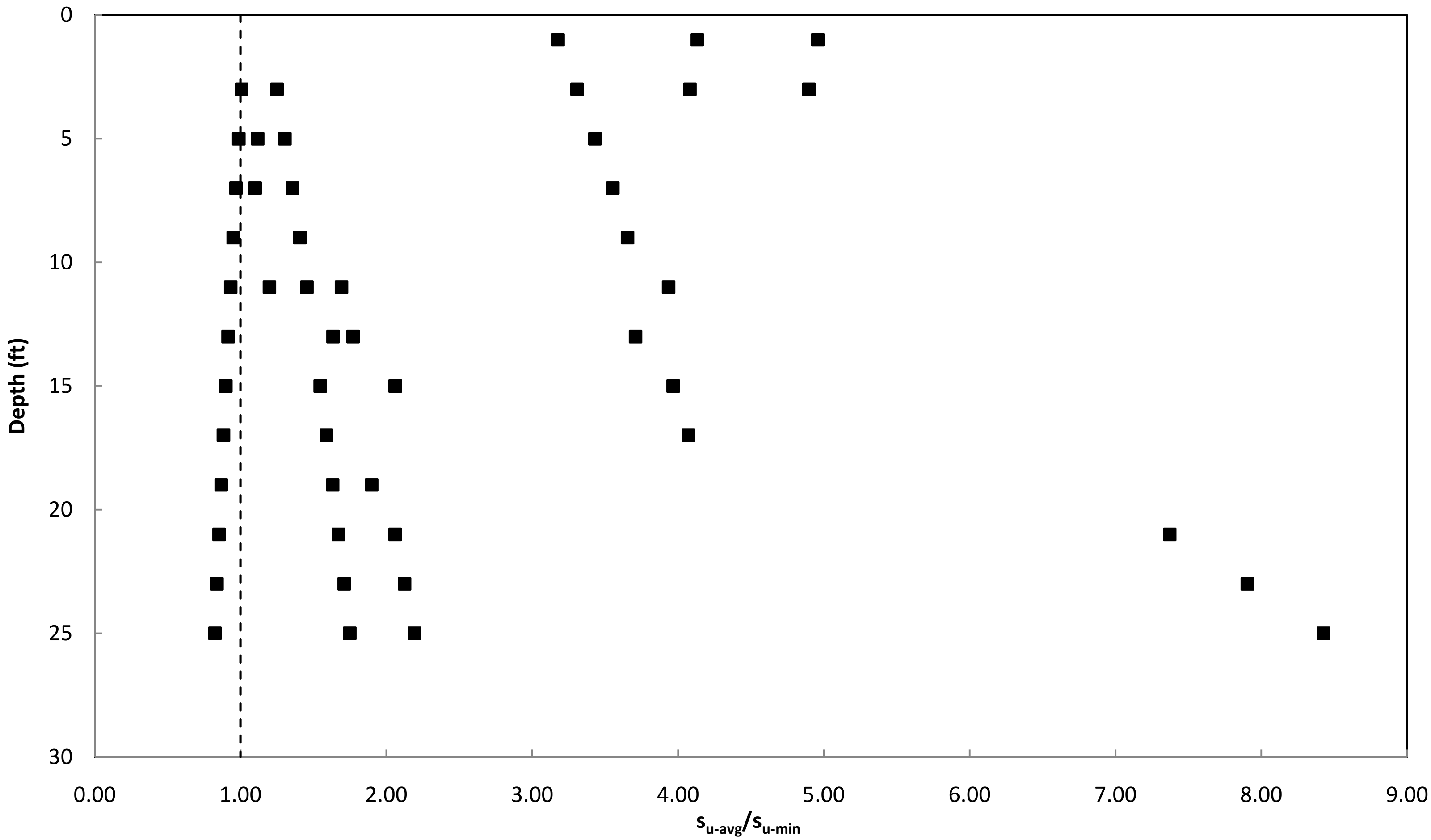


Figure 5.22: Ratio of the Undrained Shear Strength Determined Using the Average Undrained Strength Profile from Varathungarajan (2008) to the Minimum Undrained Shear Strength Required to Support the TCP Cone

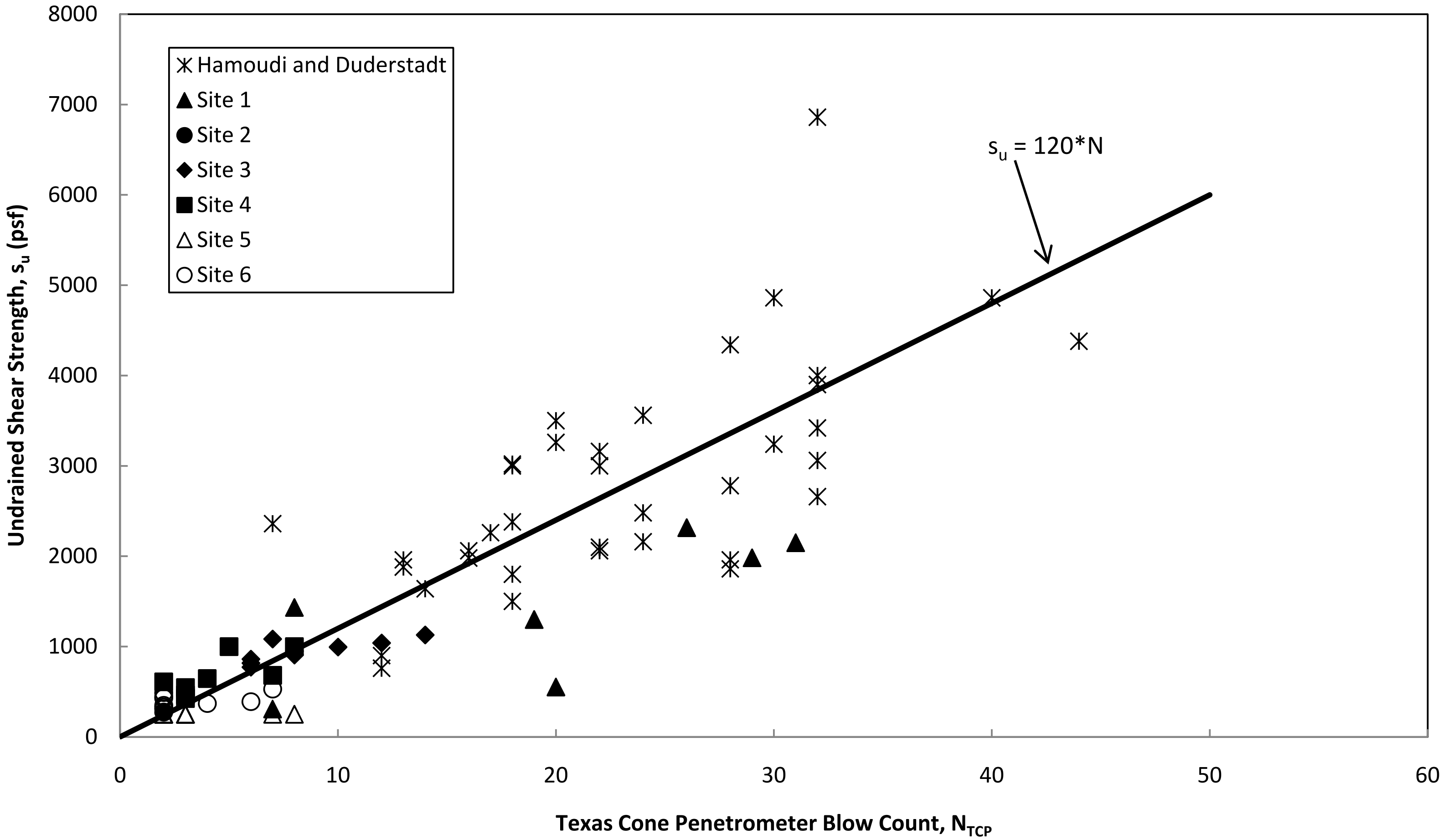


Figure 5.23: All Texas Cone Penetrometer Data from Hamoudi et al. (1974), Duderstadt et al. (1977) and the Additional Six Sites of this Study

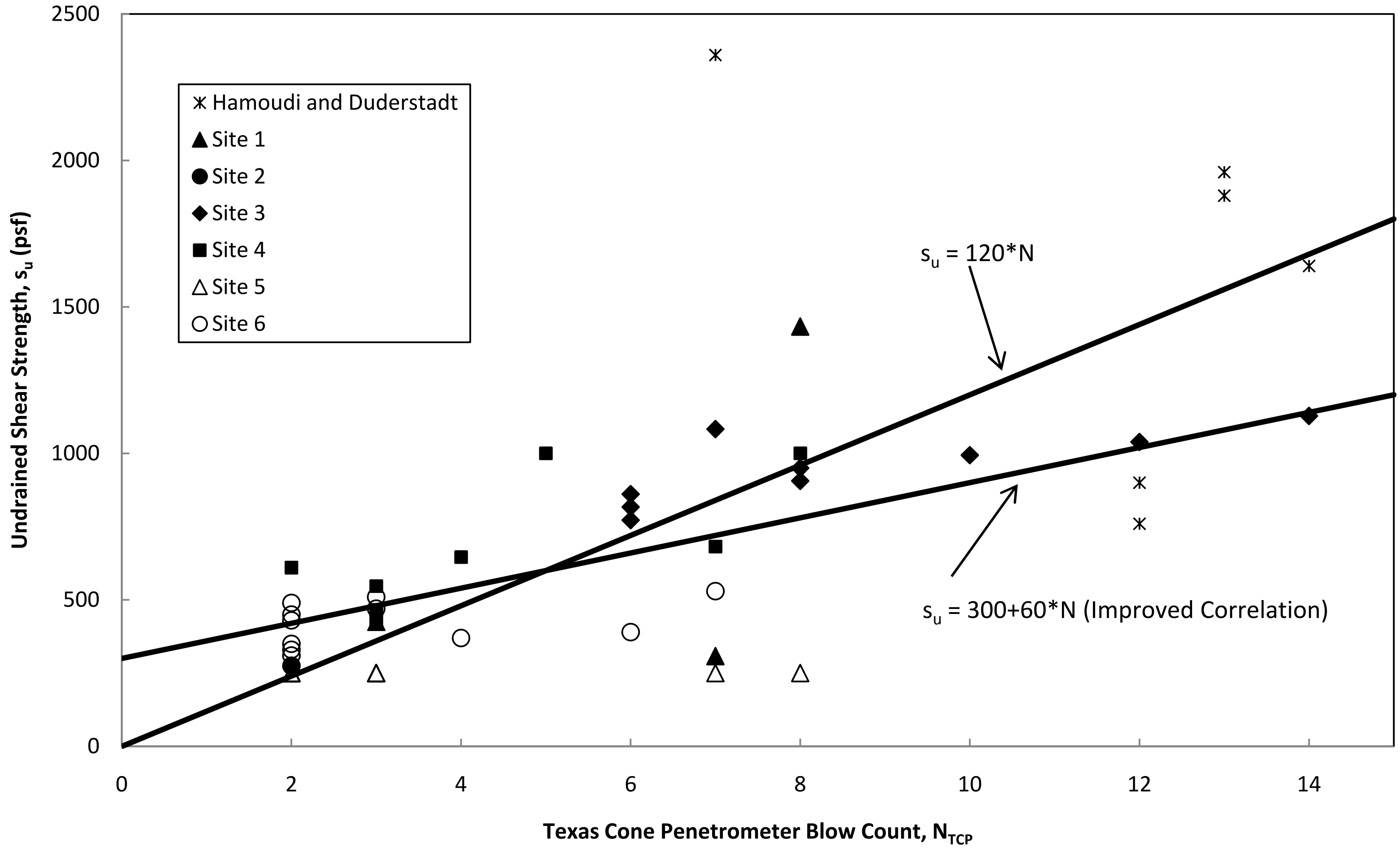


Figure 5.24: Texas Cone Penetrometer Data from Hamoudi et al. (1974), Duderstadt et al. (1977) and the Additional Six Sites of this Study for Blow Counts of 15 or less

5.3 Evaluation of Improved Correlation

A summary of the undrained shear strengths computed using the improved correlation (Equation 5.3) are shown for each of the six sites in Tables 5.7 through 5.12. The undrained shear strengths determined using this correlation for each of the six sites are plotted in Figures 5.10 through 5.15 along with the undrained shear strength profiles developed by Varathungarajan (2008). A range corresponding to ± ½ blow count is shown to indicate the range of possible error and uncertainty associated with recording the Texas Cone Penetrometer blow count to the nearest integer value.

Table 5.16: Summary of Undrained Shear Strengths for Site No. 1 using the Improved Correlation

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*s_u (psf)*
0	10	Sandy Clay Fill		
1	9		8	780
2	8			
3	7		19	1440
4	6			
5	5		31	---
6	4			
7	3	Granular Fill	38	---
8	2			
9	1		7	720
10	0			
11	-1	Fat Clay	7	720
12	-2			
13	-3		3	480
14	-4	Lean Clay		
15	-5		20	1500
16	-6			
17	-7	Sand	20	---
18	-8			
19	-9		15	---
20	-10			
21	-11	Fat Clay	29	2040
22	-12			
23	-13		31	2160
24	-14			
25	-15		26	1860
26	-16			

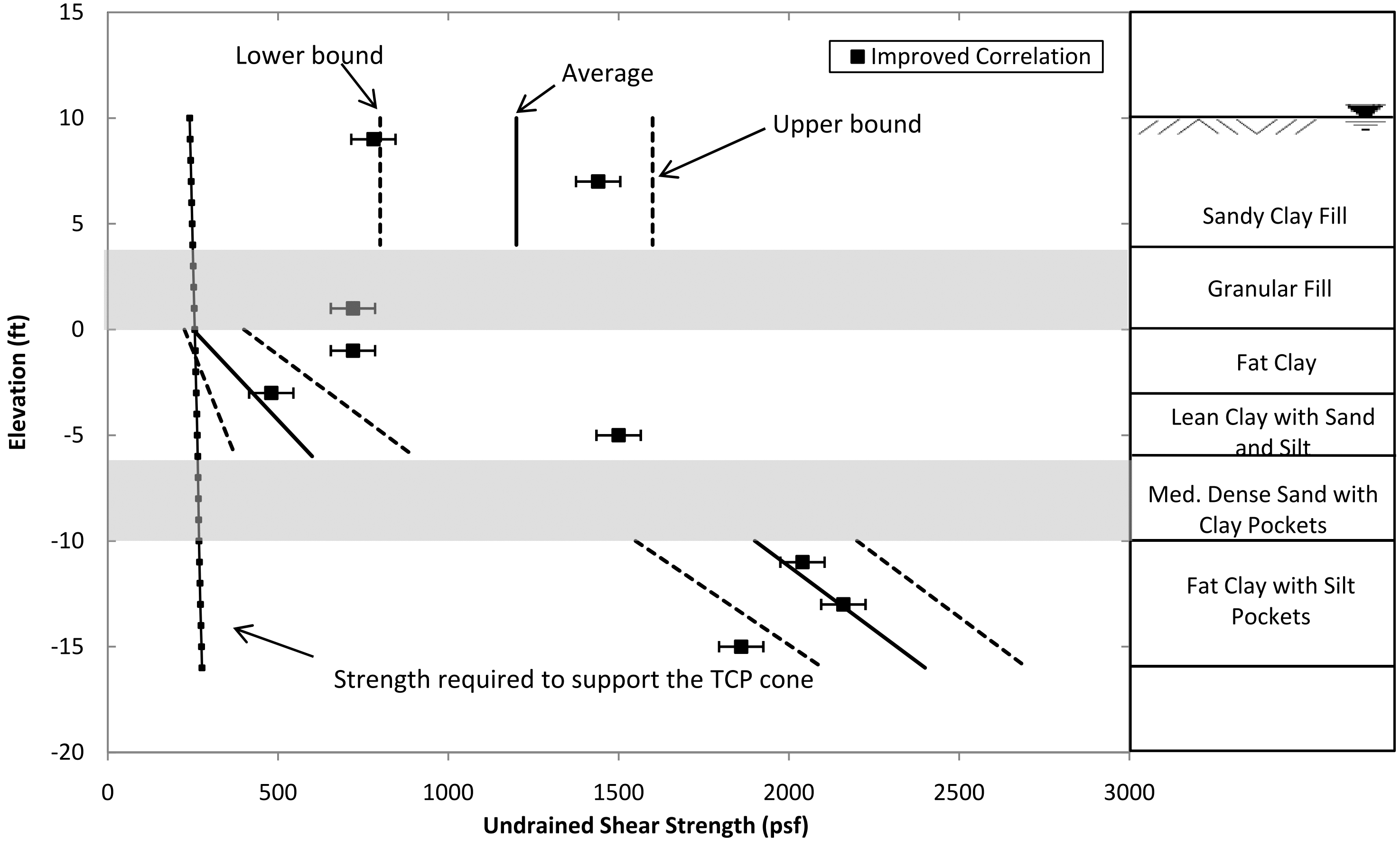


Figure 5.25: Undrained Shear Strengths for Site No. 1 using the Improved Correlation

Table 5.17: Summary of Undrained Shear Strengths for Site No. 2 using the Improved Correlation

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*s_u (psf)*
0	3	Clayey and Silty Sand		
1	2		11	---
2	1			
3	0		3	---
4	-1			
5	-2		2	420
6	-3	Fat Clay		
7	-4		2	420
8	-5			
9	-6			
10	-7		W.O.H.	---
11	-8			
12	-9			
13	-10			
14	-11		W.O.H.	---
15	-12			
16	-13			
17	-14			
18	-15			
19	-16			
20	-17		W.O.H.	---
21	-18			
22	-19			
23	-20			
24	-21			
25	-22			
26	-23		W.O.H.	---
27	-24			
28	-25			
29	-26			
30	-27	Sandy Lean Clay		
31	-28		8	---
32	-29			
33	-30		2	---
34	-31			
35	-32			

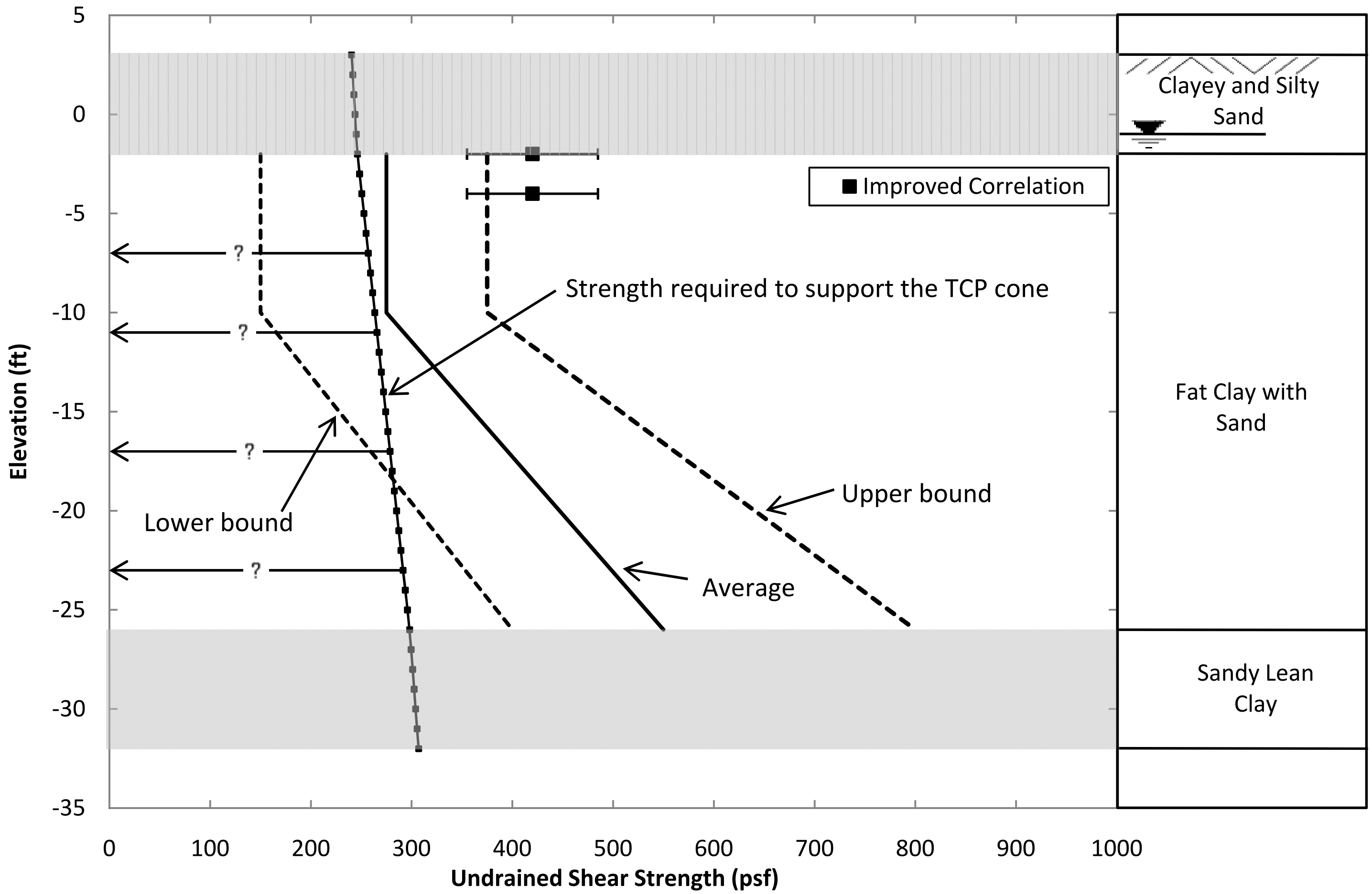


Figure 5.26: Undrained Shear Strengths for Site No. 2 using the Improved Correlation

Table 5.18: Summary of Undrained Shear Strengths for Site No. 3 using the Improved Correlation

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*s_u (psf)*
0	31	Lean and Fat CLAY		
1	30		6	660
2	29			
3	28		6	660
4	27			
5	26		6	660
6	25			
7	24		8	780
8	23			
9	22		8	780
10	21			
11	20		10	900
12	19			
13	18		12	1020
14	17			
15	16		7	720
16	15			
17	14		14	1140
18	13			
19	12	Clayey SAND	14	---
20	11			
21	10		13	---
22	9			
23	8		12	---
24	7			
25	6		12	---
26	5			

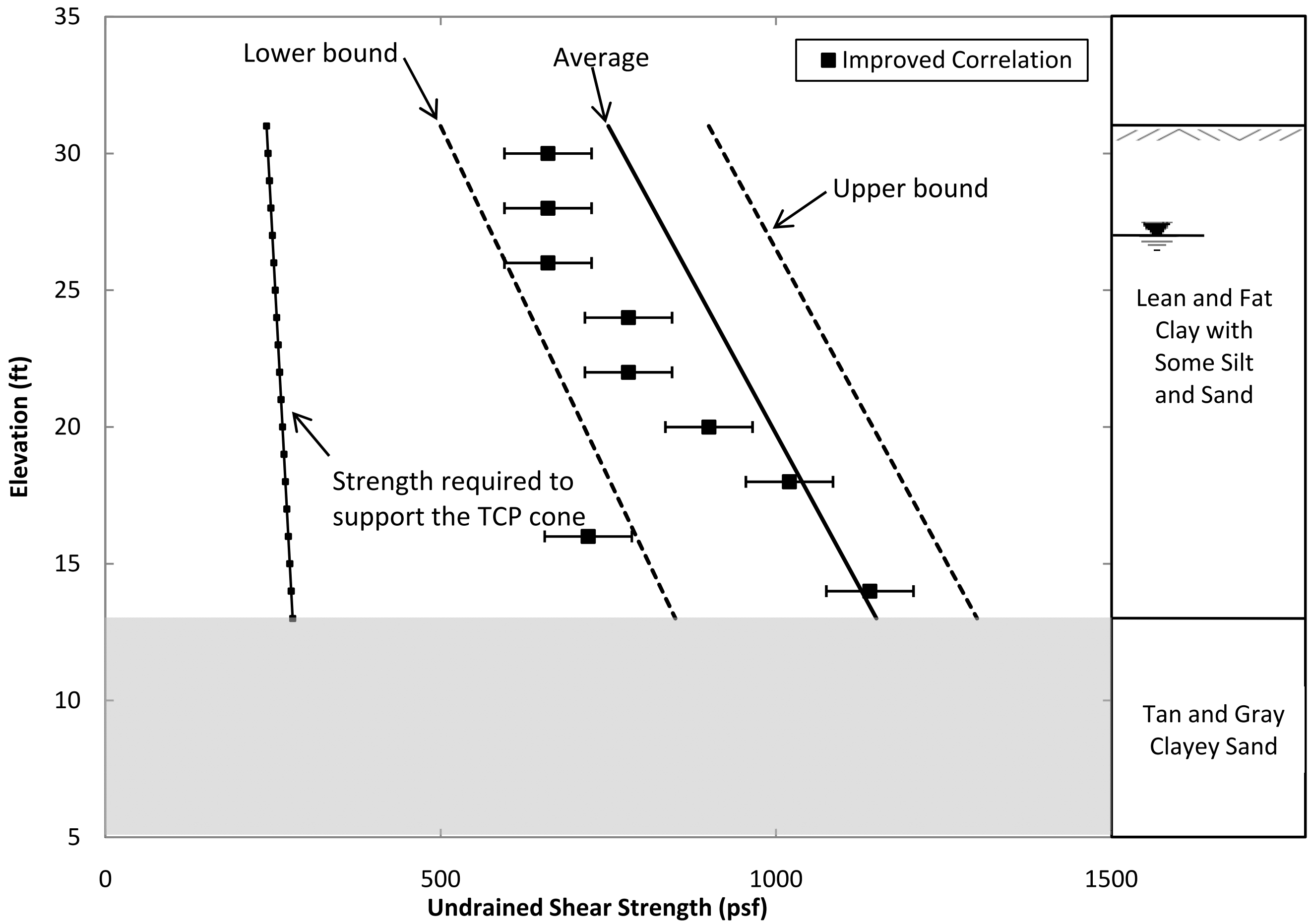


Figure 5.27: Undrained Shear Strengths for Site No. 3 using the Improved Correlation

Table 5.19: Summary of Undrained Shear Strengths for Site No. 4 using the Improved Correlation

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*s_u (psf)*
0	10	Sandy Clay Fill		
0.5	9.5			
1	9		8	780
1.5	8.5			
2	8			
2.5	7.5			
3	7		5	600
3.5	6.5			
4	6			
4.5	5.5	Sand		
5	5		17	---
5.5	4.5			
6	4			
6.5	3.5			
7	3		18	---
7.5	2.5			
8	2			
8.5	1.5			
9	1		8	---
9.5	0.5			
10	0			
10.5	-0.5			
11	-1		3	480
11.5	-1.5	Organic Clay		
12	-2			
12.5	-2.5			
13	-3		3	480
13.5	-3.5			
14	-4			
14.5	-4.5			
15	-5		W.O.H.	---
15.5	-5.5			
16	-6			
16.5	-6.5			
17	-7		W.O.H.	---
17.5	-7.5			
18	-8			
18.5	-8.5			
19	-9		3	480
19.5	-9.5			
20	-10			
20.5	-10.5			
21	-11		2	420
21.5	-11.5			
22	-12			
22.5	-12.5	Sandy Fat Clay		
23	-13		4	540
23.5	-13.5			
24	-14			
24.5	-14.5			
25	-15		7	720
25.5	-15.5			
26	-16			

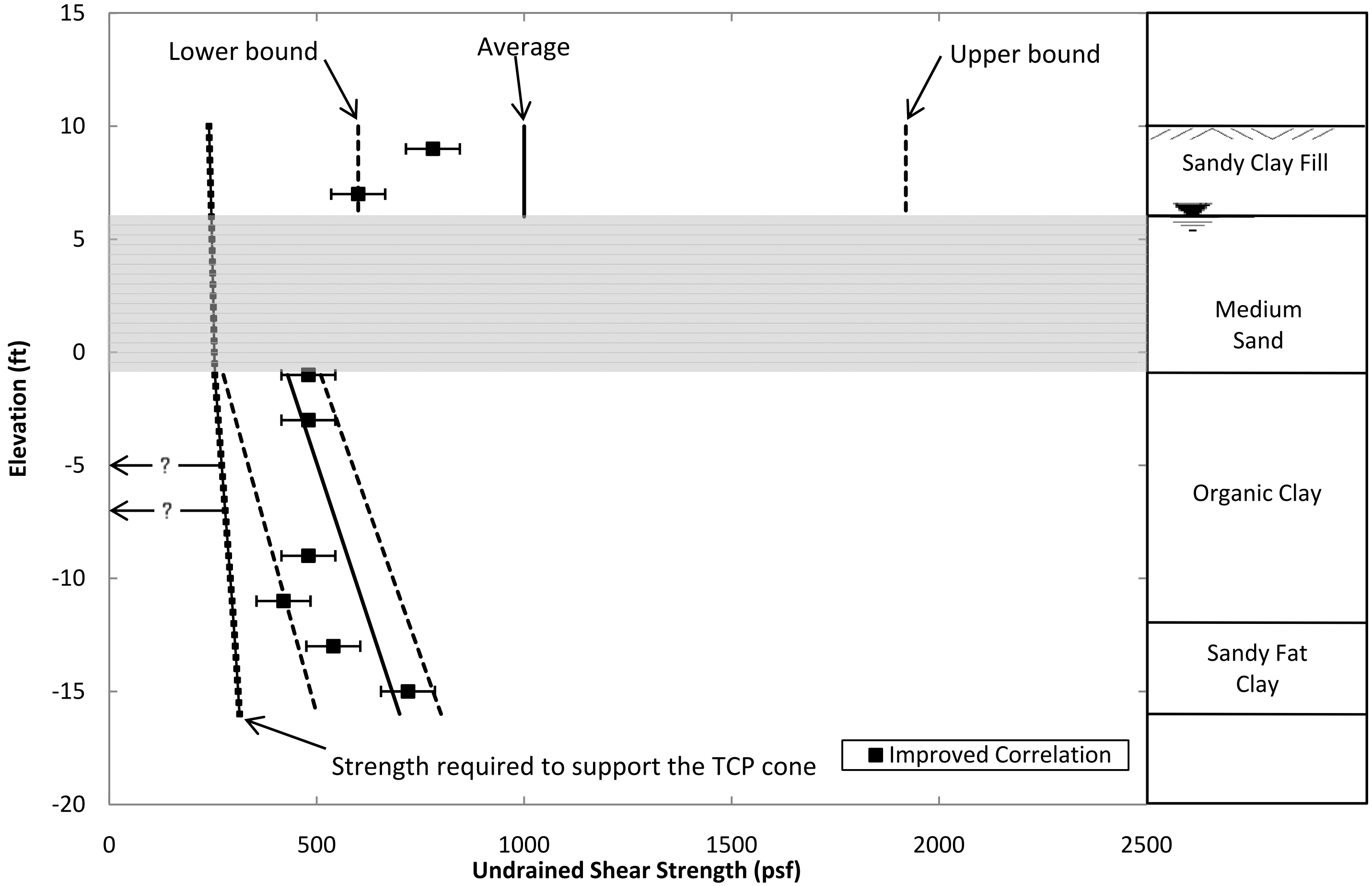


Figure 5.28: Undrained Shear Strengths for Site No. 4 using the Improved Correlation

Table 5.20: Summary of Undrained Shear Strengths for Site No. 5 using the Improved Correlation

Depth (ft)	*Elevation (ft)*	*Soil Type*	*N_{TCP} (bpf)*	*s_u (psf)*
0	0	Sandy Clay Fill		
0.5	-0.5			
1	-1			
1.5	-1.5			
2	-2			
2.5	-2.5	Fat Clay and Sandy Fat Clay		
3	-3		3	480
3.5	-3.5			
4	-4			
4.5	-4.5			
5	-5		2	420
5.5	-5.5			
6	-6			
6.5	-6.5			
7	-7		2	420
7.5	-7.5			
8	-8			
8.5	-8.5			
9	-9		2	420
9.5	-9.5			
10	-10			
10.5	-10.5			
11	-11		3	480
11.5	-11.5			
12	-12			
12.5	-12.5			
13	-13		2	420
13.5	-13.5			
14	-14			
14.5	-14.5			
15	-15		2	420
15.5	-15.5			
16	-16			
16.5	-16.5			
17	-17		2	420
17.5	-17.5			
18	-18			
18.5	-18.5			
19	-19		3	480
19.5	-19.5			
20	-20			
20.5	-20.5			
21	-21		3	480
21.5	-21.5			
22	-22			
22.5	-22.5			
23	-23		7	720
23.5	-23.5			
24	-24			
24.5	-24.5			
25	-25		8	780
25.5	-25.5			
26	-26			

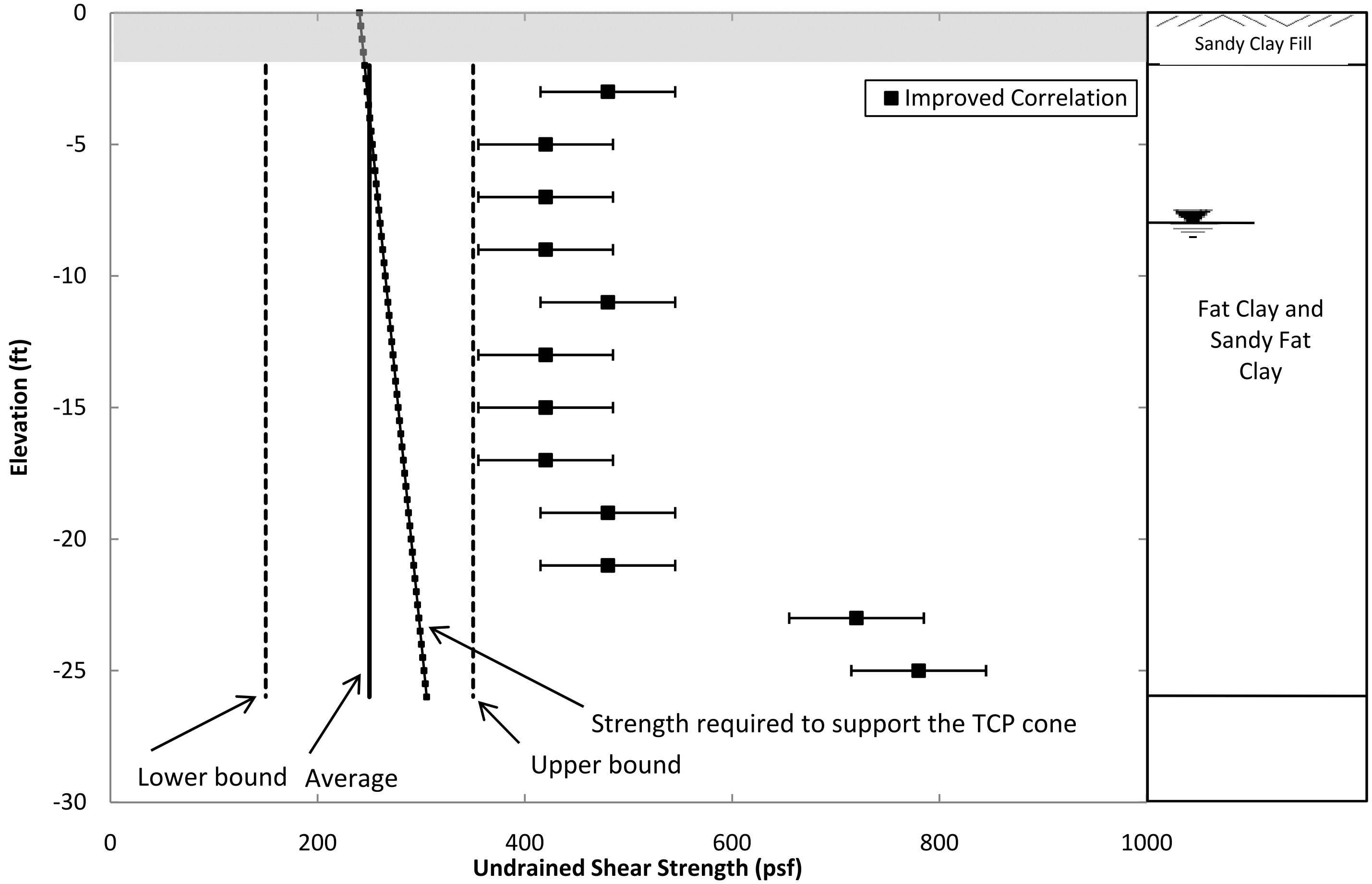


Figure 5.29: Undrained Shear Strengths for Site No. 5 using the Improved Correlation

Table 5.21: Summary of Undrained Shear Strengths for Site No. 6 using the Improved Correlation

Depth (ft)	Elevation (ft)	Soil Type	N_{TCP} (bpf)	s_u (psf)
0	0			
0.5	-0.5			
1	-1	Sandy Clay Fill		
1.5	-1.5			
2	-2			
2.5	-2.5			
3	-3		2	420
3.5	-3.5			
4	-4			
4.5	-4.5			
5	-5		2	420
5.5	-5.5			
6	-6			
6.5	-6.5			
7	-7		2	420
7.5	-7.5			
8	-8			
8.5	-8.5			
9	-9		4	540
9.5	-9.5			
10	-10			
10.5	-10.5			
11	-11		6	660
11.5	-11.5			
12	-12			
12.5	-12.5			
13	-13		W.O.H	---
13.5	-13.5			
14	-14	Fat Clay and Sandy Fat Clay		
14.5	-14.5			
15	-15		2	420
15.5	-15.5			
16	-16			
16.5	-16.5			
17	-17		2	420
17.5	-17.5			
18	-18			
18.5	-18.5			
19	-19		3	480
19.5	-19.5			
20	-20			
20.5	-20.5			
21	-21		2	420
21.5	-21.5			
22	-22			
22.5	-22.5			
23	-23		3	480
23.5	-23.5			
24	-24			
24.5	-24.5			
25	-25		7	720
25.5	-25.5			
26	-26			

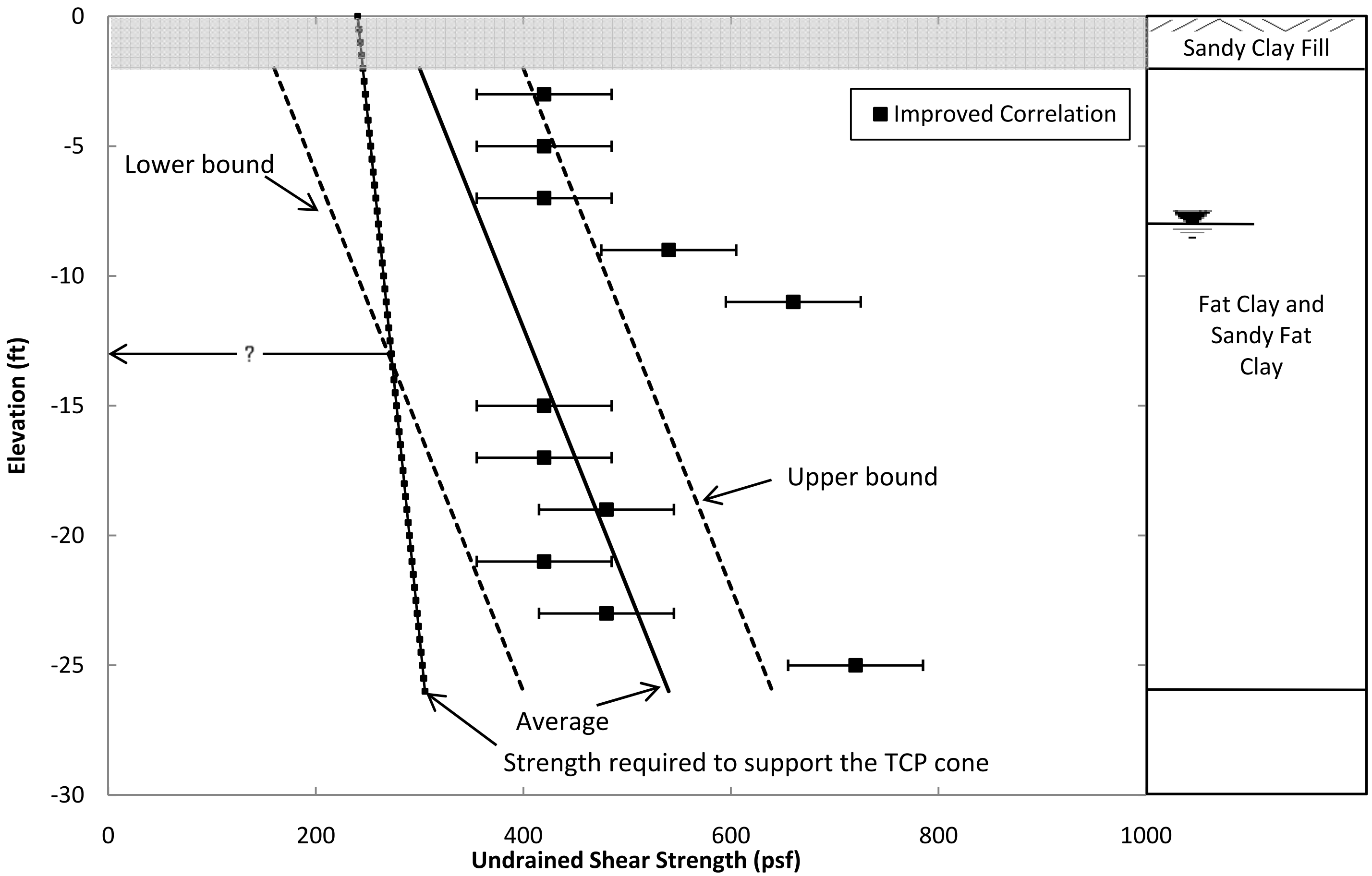


Figure 5.30: Undrained Shear Strengths for Site No. 6 using the Improved Correlation

5.4 Discussion of Improved Correlation

The improved correlation presented in this chapter is intended primarily for soft clays with undrained shear strengths of 1200 psf or less (TCP blow counts of 15 or less), which is the range of interest in this present study. The results presented in Figures 5.10 through 5.15 show that the improved correlation provides a reasonable estimate of undrained shear strength that is in good agreement with the undrained shear strength profiles provided by Varathungarajan (2008). However, with any correlation there is error and uncertainty with the correlation. The ratio of the undrained shear strength computed from the improved correlation to the undrained shear strength determined using the average undrained strength profile from Varathungarajan (2008) ranges from 0.60 to 3.12 as shown in Figure 5.16. The potential significance of this variation and uncertainty is examined in the next chapter.

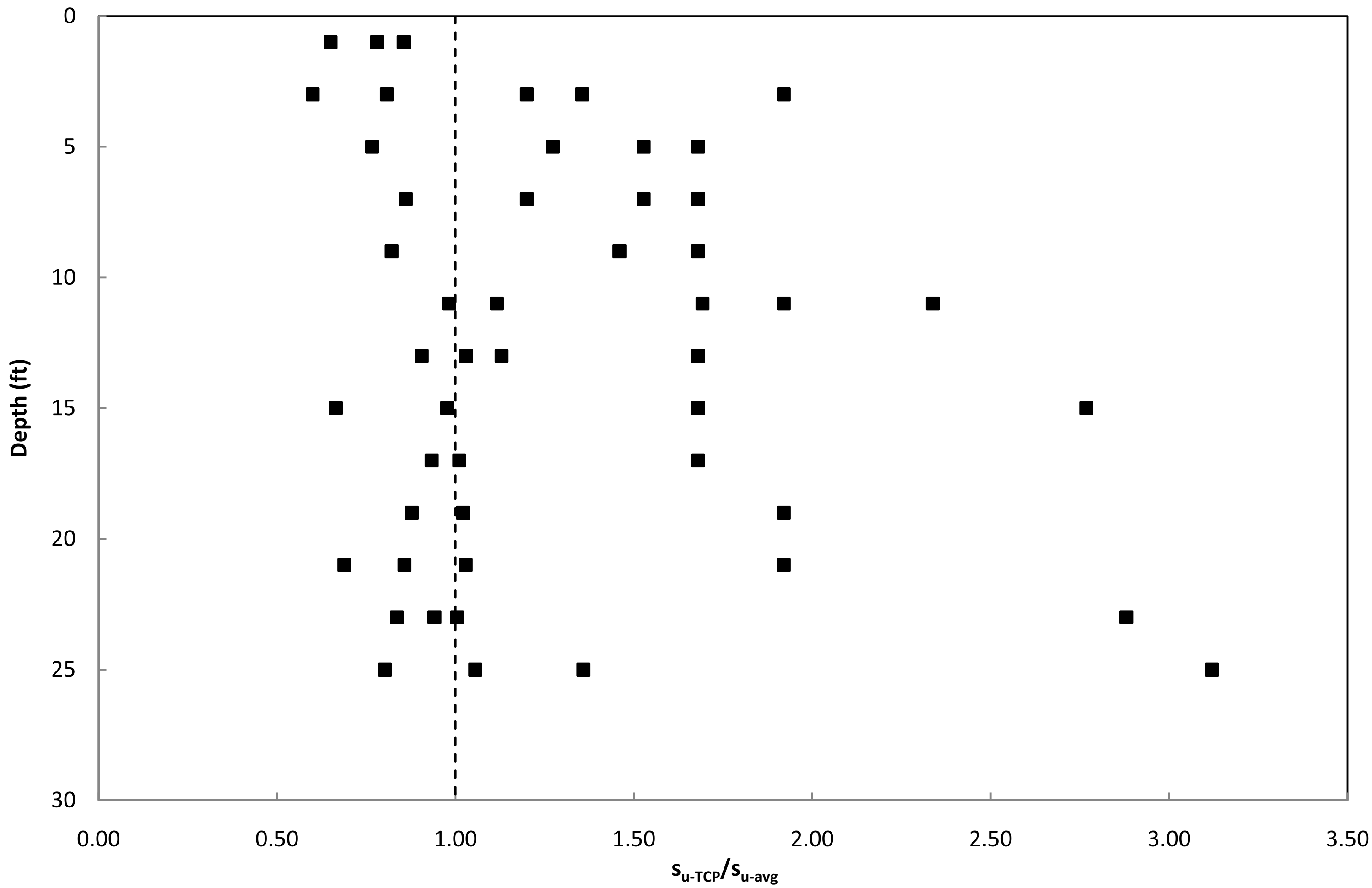


Figure 5.31: Ratio of the Undrained Shear Strength Computed from the Improved Correlation to the Undrained Shear Strength Determined Using the Average Undrained Strength Profile from Varathungarajan (2008)

Chapter 6. Application of Improved Correlation

One of the primary uses of undrained shear strengths by TxDOT, particularly for soil at shallow (less than 30 feet) depths, is to evaluate bearing capacity for embankments, retaining walls, and shallow footings. In order to evaluate the correlations between undrained shear strength and Texas Cone Penetrometer blow count developed in this study and described in Chapter 5, a series of bearing capacity analyses was performed. Bearing capacity analyses were performed first for the average and lower- and upper-bound undrained shear strength profiles established from laboratory, field vane and piezocone penetration tests by Varathungarajan (2008). These analyses are assumed to represent the correct bearing capacity and expected uncertainty. Next, similar analyses were performed using undrained shear strength profiles determined from the Texas Cone Penetrometer blow counts and the improved correlation (Equation 5.3) developed in this study. Analyses were performed for all the sites described in Chapter 5 with the exception of Site No. 2, where Texas Cone Penetrometer tests could not be performed due to the very soft soils encountered. The procedures used and results of these analyses are presented in this chapter.

6.1 Approach

For each soil strength profile the factor of safety was calculated for a uniform load at the ground surface (Figure 6.1). Three different widths—5, 10, and 20 feet—were assumed for the distributed load. As the width of the load varied the depth of the most critical (lowest factor of safety) slip surface varied also, thus influencing how much of the soil strength profile influenced the bearing capacity. As the width of the load increased the depth of the critical slip surface generally increased. Widths of 5, 10, and 20 feet were considered to represent the range of widths of retaining walls as well as include the probable width of most shallow foundations.

Because the undrained shear strength varied with depth throughout most of the undrained shear strength profiles, a conventional bearing capacity equation, which assumes a constant strength profile could not be used. Instead, bearing capacity analyses were performed using the computer software UTEXAS4 (Wright, 1999). Although this software is intended primarily for slope stability analyses, it can also be used to evaluate bearing capacity. The software allows analyses to be performed for a horizontal “slope” subjected to a distributed load (Figure 6.2).

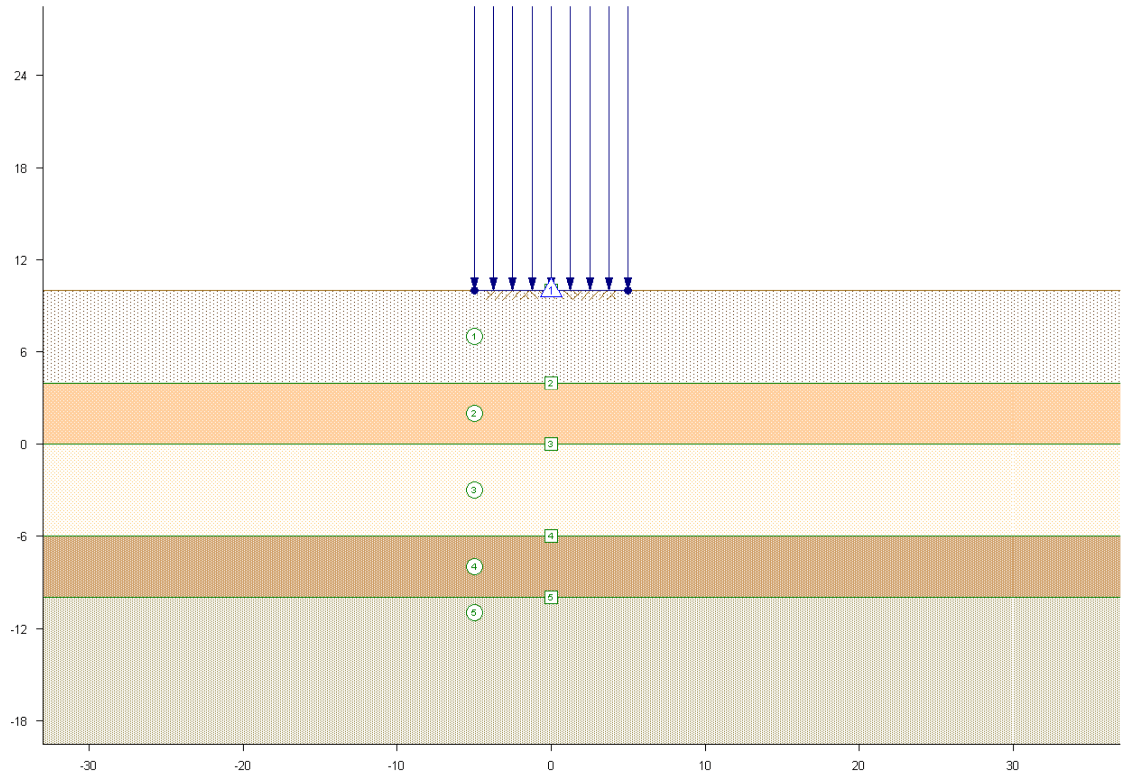


Figure 6.32: Graphic File for Site 1 using the Average Undrained Shear Strength Profile and a Load Width of 10 Feet

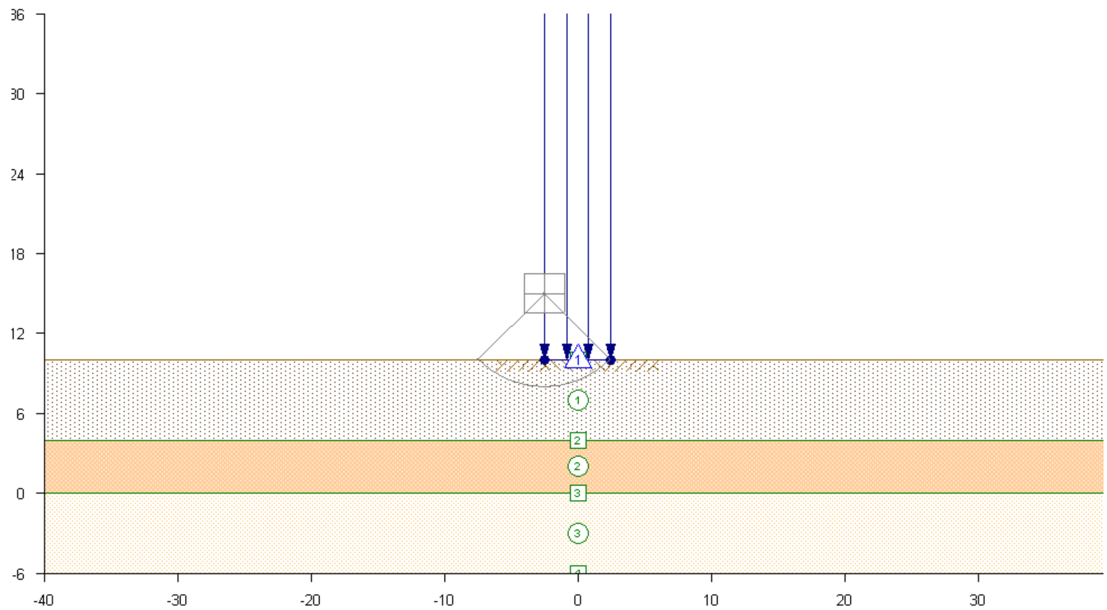


Figure 6.33: Simplified Graphic File for Site 1 using the Average Undrained Shear Strength Profile and a Load Width of 5 Feet

UTEXAS4 calculates a factor of safety applied to the soil shear strength. This differs from the factor of safety normally applied in bearing capacity analyses, where the factor of safety is defined with respect to load, rather than shear strength. However, for the case where the friction angle (ϕ) is equal to zero, which applies to the undrained shear strengths for all of the clays in the soil profiles considered, there is no difference in the numerical values of a factor of safety defined with respect to shear strength and a factor of safety defined with respect to load. The only times there are differences in factor of safety depending on definition is when the friction angle (ϕ) is greater than zero. This ($\phi > 0$) only occurs for the sands in the soil profiles, and was not considered to be significant enough to warrant further consideration of the definitions of factor of safety. All of the factors of safety reported in this chapter are defined with respect to soil shear strength as computed by the UTEXAS4 software.

UTEXAS4 permits computations to be performed using either circular or general, noncircular slip surfaces. However, for all of the analyses presented in this chapter only circular slip surfaces were used. Because the primary interest in the analyses was in comparing values of the factor of safety for different representations of the undrained shear strength profile, use of circular slip surfaces was considered adequate.

A general procedure was followed regarding the analysis and computations made by UTEXAS4. The Simplified Bishop procedure was used and a *floating grid* search scheme with a grid spacing equal to 1% of the width of the loaded area was used. The circular slip surfaces were forced through the right edge of the loaded area and the starting center point was typically positioned at the left edge of the loaded area. Judgment was used with regard to the starting center point of the search to ensure that the minimum factor of safety was determined for each analysis.

For each site the magnitude of the load was selected to produce a factor of safety of 1.0 based on the average undrained strength profile determined by Varathungarajan (2008). The magnitude of the load varied depending on the width of the load. The load (bearing pressure) required to produce a factor of safety of 1.0 for each site and load widths of 5, 10, and 20 feet are summarized in Table 6.1. Once these loads were determined they were used to compute the factors of safety for bearing capacity for the lower- and upper-bound strength profiles determined by Varathungarajan (2008) as well as the undrained shear strength profile computed from the Texas Cone Penetrometer blow counts using the improved correlation presented in Chapter 5 (Equation 5.3).

Table 6.22: Load (Bearing Pressure) Required to Produce a Factor of Safety of 1.0 for each Site

Site	Load Width (ft)	Load Magnitude (lbs)
Site No. 1	5	6620
	10	6215
	20	4355
Site No. 3	5	4392
	10	4635
	20	5088
Site No. 4	5	5520
	10	6375
	20	5325
Site No. 5	5	1815
	10	1572
	20	1502
Site No. 6	5	2145
	10	1980
	20	2097

Site No. 1 – Port Arthur, TX (Site A)

The average, lower- and upper-bound strength profiles from Varathungarajan (2008) along with the strength profile using the improved correlation are shown in Figure 6.3 for Site No. 1. The undrained shear strength values for the improved correlation were taken from Figure 5.10 in the previous chapter. For the granular fill and medium dense sand strata, the shear strength was assumed to be represented in terms of effective stresses with no cohesion and an angle of internal friction of 30 degrees. The groundwater table was at the ground surface.

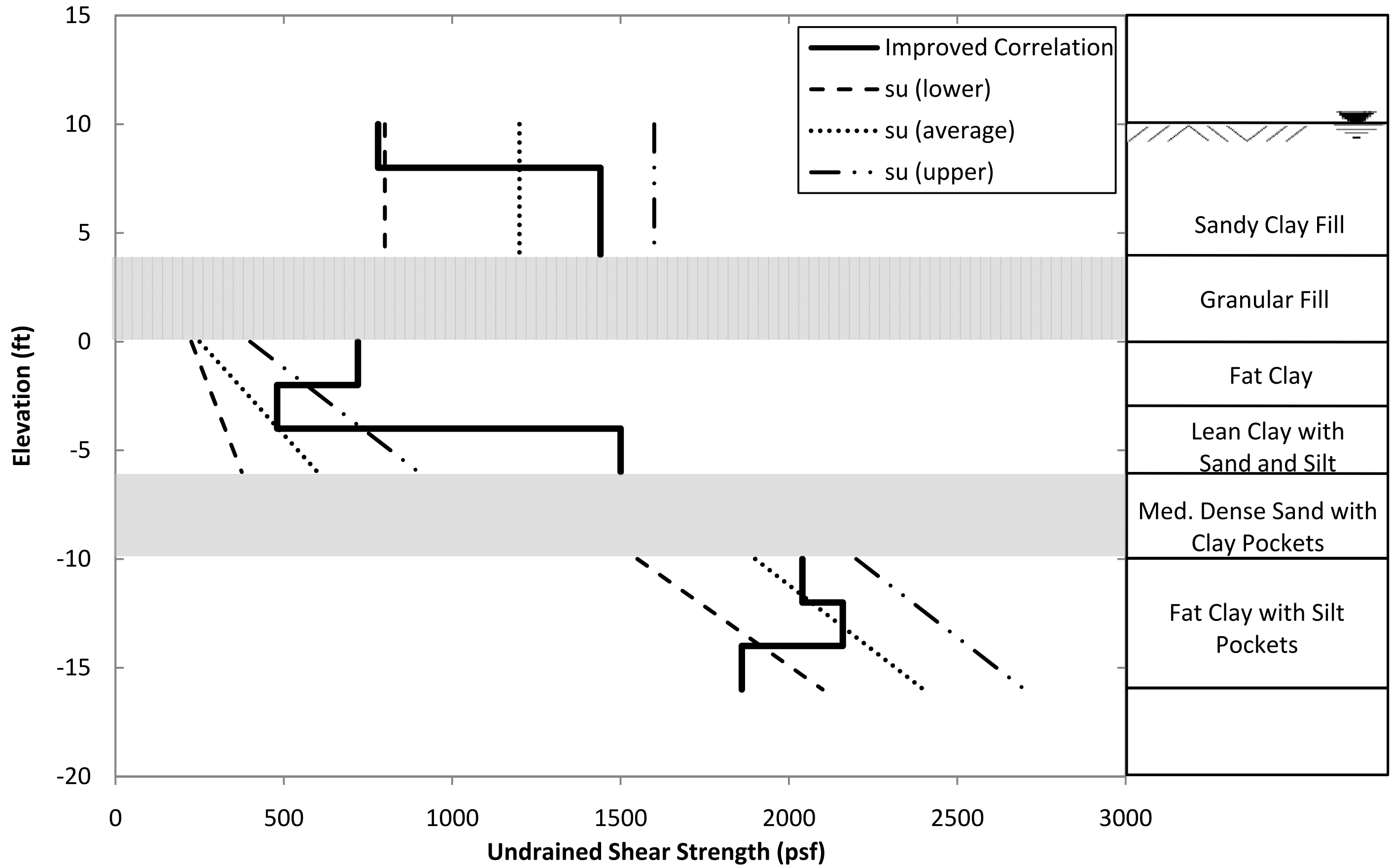


Figure 6.34: Undrained Shear Strength Profiles for Site No. 1

Site No. 2 – Port Arthur, TX (Site B)

An undrained shear strength profile could not be developed for Site No. 2 from the Texas Cone Penetrometer tests due to the numerous “weight of hammer” (W.O.H.) blow counts. As a result no analysis was conducted for this site. Referring to Figure 5.11 in Chapter 5, the Texas Cone Penetrometer test would have been expected to produce “weight of hammer” (W.O.H.) blow counts due to the insufficient strength of the soil to support the weight of the Texas Cone Penetrometer cone.

Site No. 3 – Mont Belvieu, TX

The average, lower- and upper-bound strength profiles from Varathungarajan (2008) along with the strength profile using the improved correlation are shown in Figure 6.4 for Site No. 3. The undrained shear strength values for the improved correlation were taken from Figure 5.12 in the previous chapter. For the clayey sand stratum, the shear strength was assumed to be represented in terms of effective stresses with no cohesion and an angle of internal friction of 30 degrees. The groundwater table was at a depth of 4 feet.

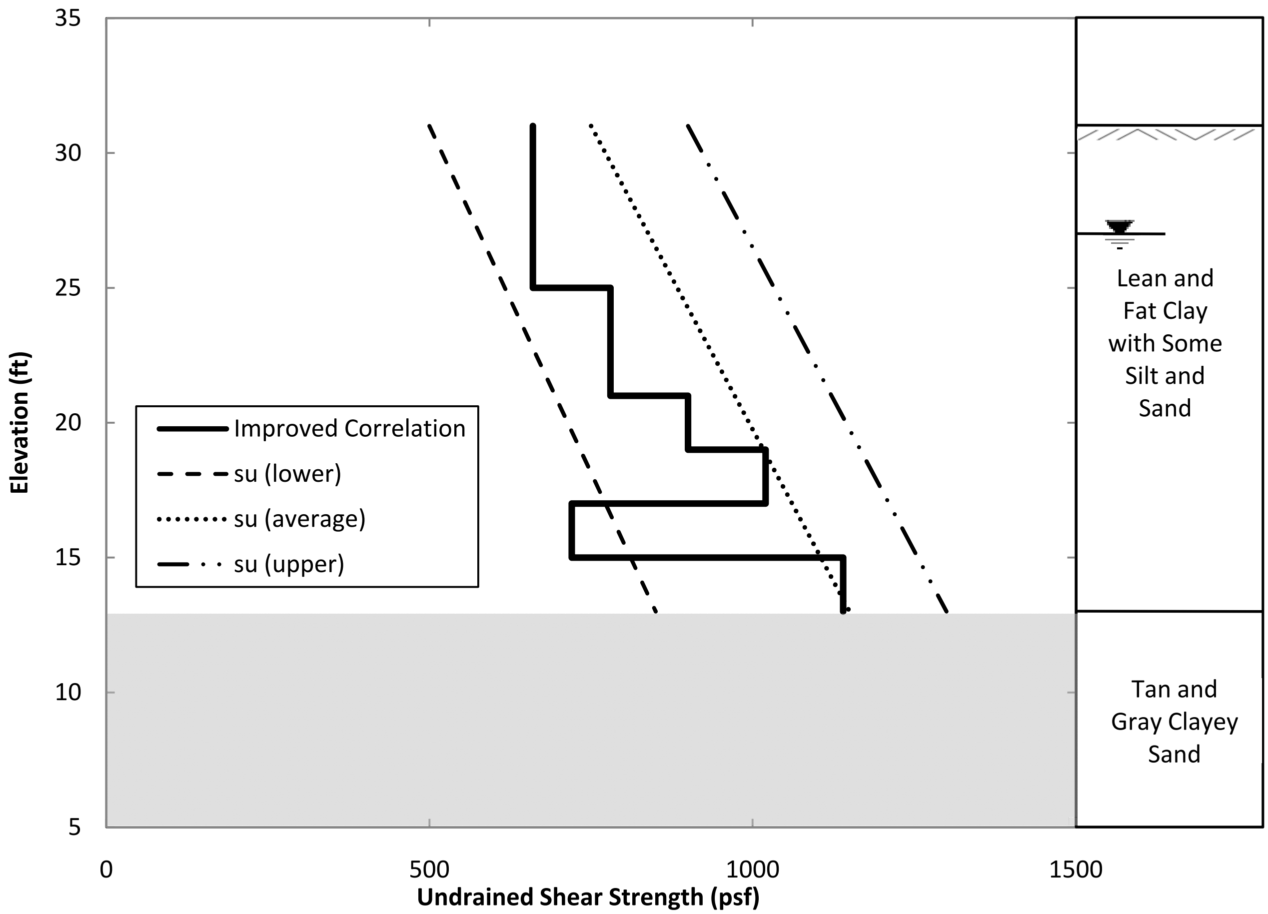


Figure 6.35: Undrained Shear Strength Profiles for Site No. 3

Site No. 4 – Beaumont, TX

The average, lower- and upper-bound strength profiles from Varathungarajan (2008) along with the strength profile using the improved correlation are shown in Figure 6.5 for Site No. 4. The undrained shear strength values for the improved correlation were taken from Figure 5.13 in the previous chapter. For the medium sand stratum, the shear strength was assumed to be represented in terms of effective stresses with no cohesion and an angle of internal friction of 30 degrees. The groundwater table was at a depth of 2.5 feet.

Site No. 5 – Cameron Parish, LA (Site A)

The average, lower- and upper-bound strength profiles from Varathungarajan (2008) along with the strength profile using the improved correlation are shown in Figure 6.6 for Site No. 5. The undrained shear strength values for the improved correlation were taken from Figure 5.14 in the previous chapter. For the sandy clay fill stratum, the shear strength was assumed to be represented in terms of effective stresses with no cohesion and an angle of internal friction of 30 degrees. The groundwater table was at a depth of 8 feet.

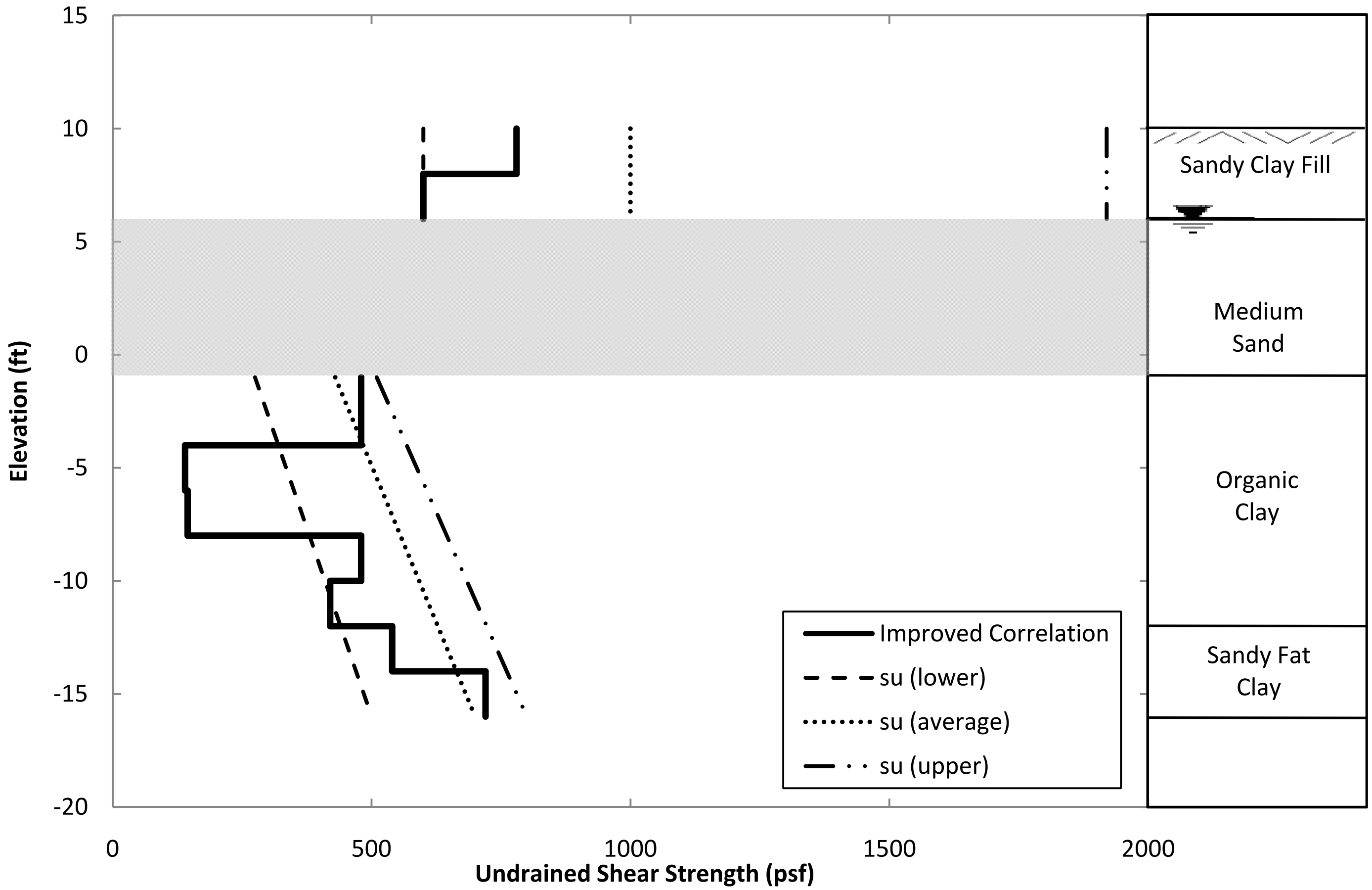


Figure 6.36: Undrained Shear Strength Profiles for Site No. 4

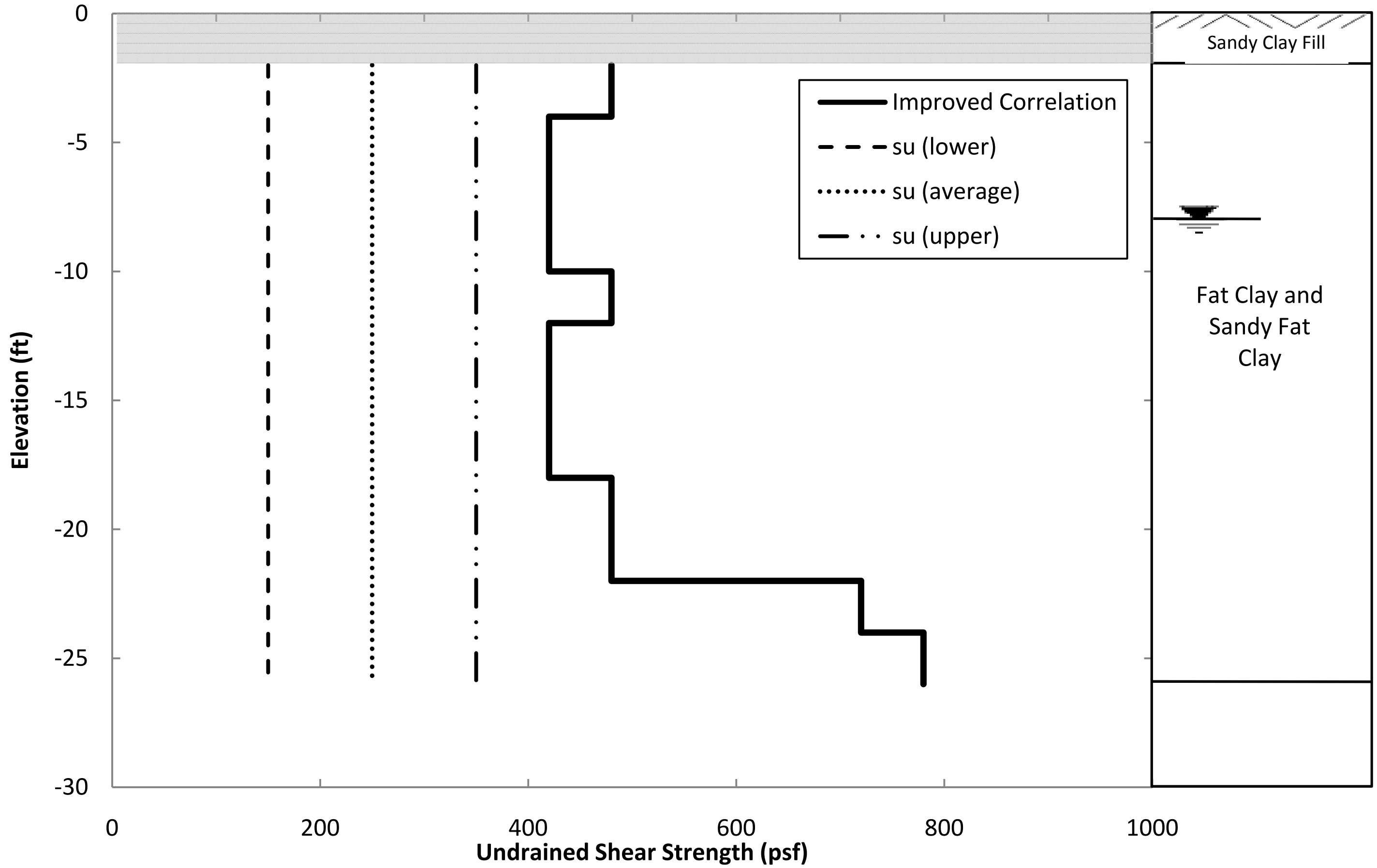


Figure 6.37: Undrained Shear Strength Profiles for Site No. 5

Site No. 6 – Cameron Parish, LA (Site B)

The average, lower- and upper-bound strength profiles from Varathungarajan (2008) along with the strength profile using the improved correlation are shown in Figure 6.7 for Site No. 6. The undrained shear strength values for the improved correlation were taken from Figure 5.15 in the previous chapter. For the sandy clay fill stratum, the shear strength was assumed to be represented in terms of effective stresses with no cohesion and an angle of internal friction of 30 degrees. The groundwater table was at a depth of 8 feet.

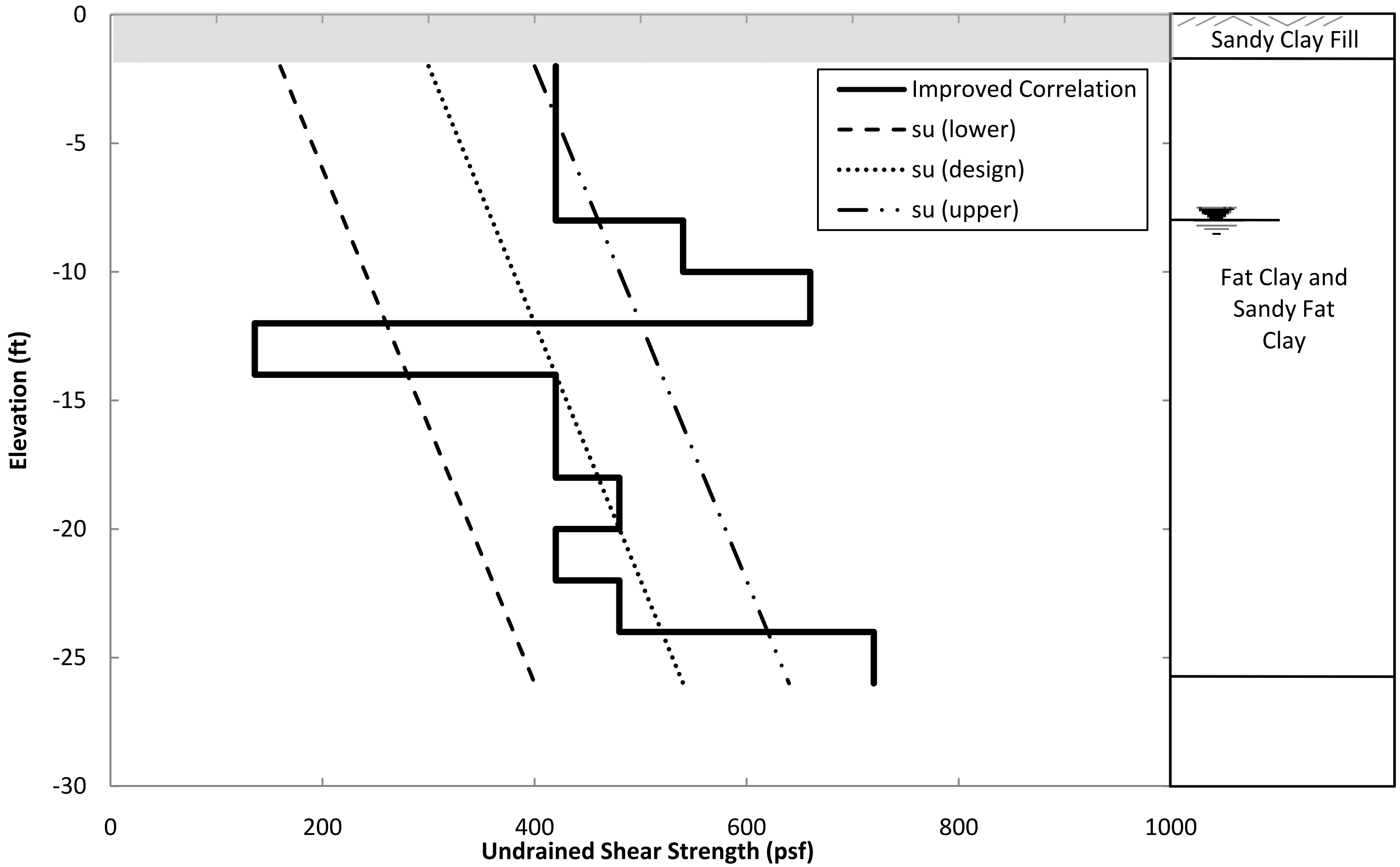


Figure 6.38: Undrained Shear Strength Profiles for Site No. 6

6.2 Results and Discussion of Bearing Capacity Analyses

Summaries of the results of the stability (bearing capacity) analyses for each site are provided in Tables 6.2 through 6.6. Because the loads were selected to produce a factor of safety of 1.0 for the average strength profile all the values shown in these tables for the average strength profile are 1.0. The factors of safety computed using the lower-bound strength profiles ranged from 0.60 to 0.88; for the upper-bound strength profiles factors of safety ranged from and 1.14 to 1.61. Factors of safety computed using the strength profile based on the improved correlation ranged from 0.64 to 1.59.

The factors of safety for the lower- and upper-bound strength profiles as well as the strength profile based on the improved correlation are plotted versus the width of the loaded area for each site in Figures 6.8 through 6.12. Again factors of safety for the average strength profiles were 1.0 as shown in the figures by a dashed line.

Table 6.23: Factors of Safety and Depth of Critical Slip Surface Computed for Site No. 1

Strength Profile		Load Width = 5 ft		Load Width = 10 ft		Load Width = 20 ft	
		FS	Depth of Critical Slip Surface (ft)	FS	Depth of Critical Slip Surface (ft)	FS	Depth of Critical Slip Surface (ft)
Site No. 1	Lower Bound	0.67	3.3	0.68	6.2	0.80	15.0
	Average	1.00	3.3	1.00	6.2	1.00	14.8
	Upper Bound	1.33	3.3	1.27	9.3	1.28	16.1
	Improved Correlation	0.77	2.0	1.07	6.2	1.12	14.0

Table 6.24: Factors of Safety and Depth of Critical Slip Surface Computed for Site No. 3

Strength Profile		Load Width = 5 ft		Load Width = 10 ft		Load Width = 20 ft	
		FS	Depth of Critical Slip Surface (ft)	FS	Depth of Critical Slip Surface (ft)	FS	Depth of Critical Slip Surface (ft)
Site No. 3	Lower Bound	0.68	3.0	0.69	5.7	0.70	10.3
	Average	1.00	3.1	1.00	5.8	1.00	10.8
	Upper Bound	1.19	3.1	1.18	5.9	1.17	11.0
	Improved Correlation	0.83	3.3	0.79	6.0	0.82	10.0

Table 6.25: Factors of Safety and Depth of Critical Slip Surface Computed for Site No. 4

Strength Profile		Load Width = 5 ft		Load Width = 10 ft		Load Width = 20 ft	
		FS	Depth of Critical Slip Surface (ft)	FS	Depth of Critical Slip Surface (ft)	FS	Depth of Critical Slip Surface (ft)
Site No. 4	Lower Bound	0.60	3.3	0.61	4.0	0.88	16.8
	Average	1.00	3.3	1.00	4.0	1.00	16.5
	Upper Bound	1.61	5.7	1.46	9.4	1.16	17.3
	Improved Correlation	0.64	4.3	0.67	4.0	0.83	18.0

Table 6.26: Factors of Safety and Depth of Critical Slip Surface Computed for Site No. 5

Strength Profile		Load Width = 5 ft		Load Width = 10 ft		Load Width = 20 ft	
		FS	Depth of Critical Slip Surface (ft)	FS	Depth of Critical Slip Surface (ft)	FS	Depth of Critical Slip Surface (ft)
Site No. 5	Lower Bound	0.85	3.6	0.77	5.4	0.73	9.4
	Average	1.00	3.4	1.00	5.8	1.00	11.1
	Upper Bound	1.16	3.3	1.26	6.2	1.30	12.1
	Improved Correlation	1.39	3.2	1.49	6.6	1.59	13.7

Table 6.27: Factors of Safety and Depth of Critical Slip Surface Computed for Site No. 6

Strength Profile		Load Width = 5 ft		Load Width = 10 ft		Load Width = 20 ft	
		FS	Depth of Critical Slip Surface (ft)	FS	Depth of Critical Slip Surface (ft)	FS	Depth of Critical Slip Surface (ft)
Site No. 6	Lower Bound	0.82	3.5	0.74	5.0	0.71	8.3
	Average	1.00	3.3	1.00	5.6	1.00	10.1
	Upper Bound	1.14	3.2	1.21	5.8	1.23	10.9
	Improved Correlation	1.16	3.3	1.20	6.1	1.05	14.0

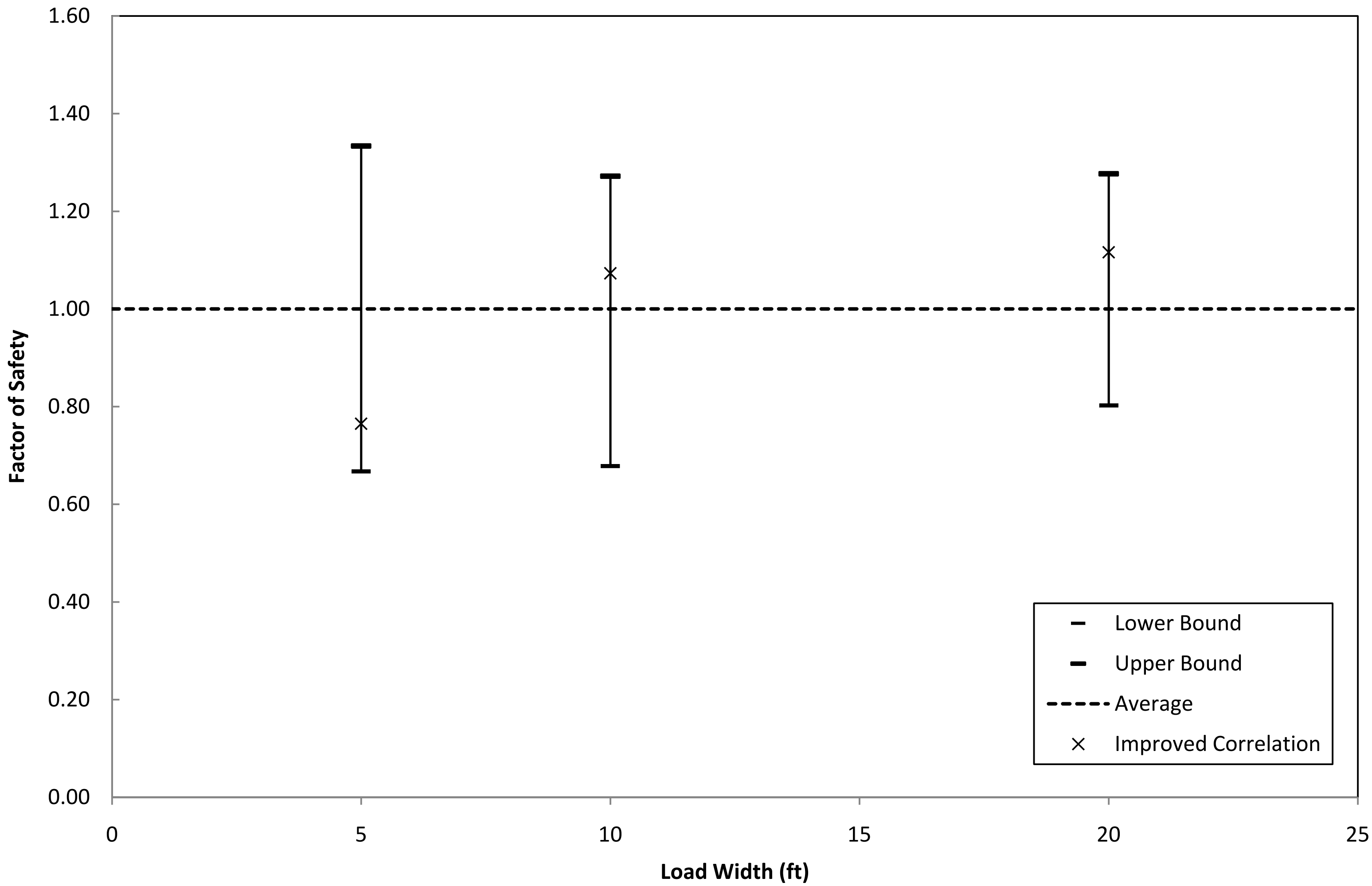


Figure 6.39: Factors of Safety Computed using UTEXAS4 for Site No. 1

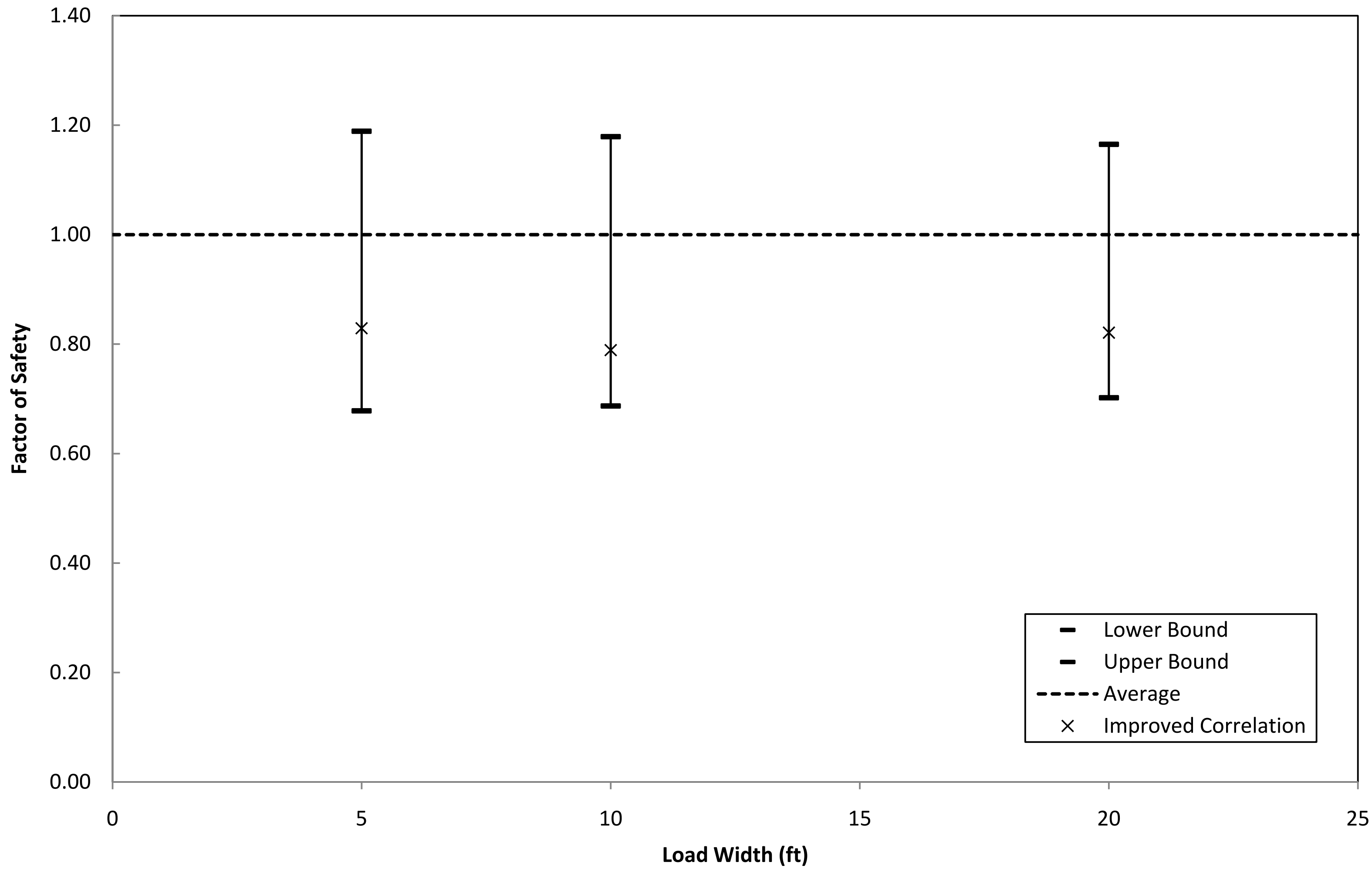


Figure 6.40: Factors of Safety Computed using UTEXAS4 for Site No. 3

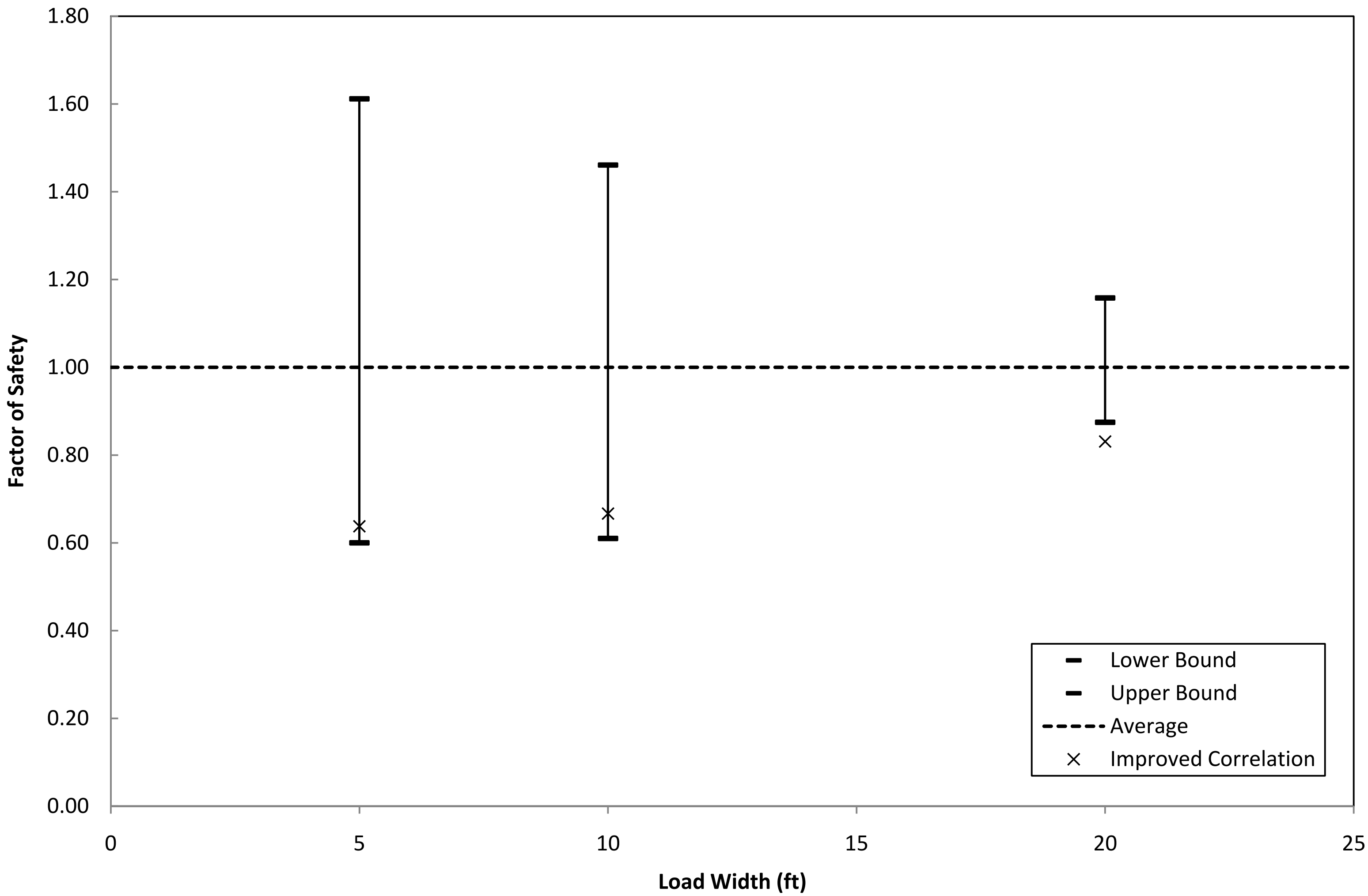


Figure 6.41: Factors of Safety Computed using UTEXAS4 for Site No. 4

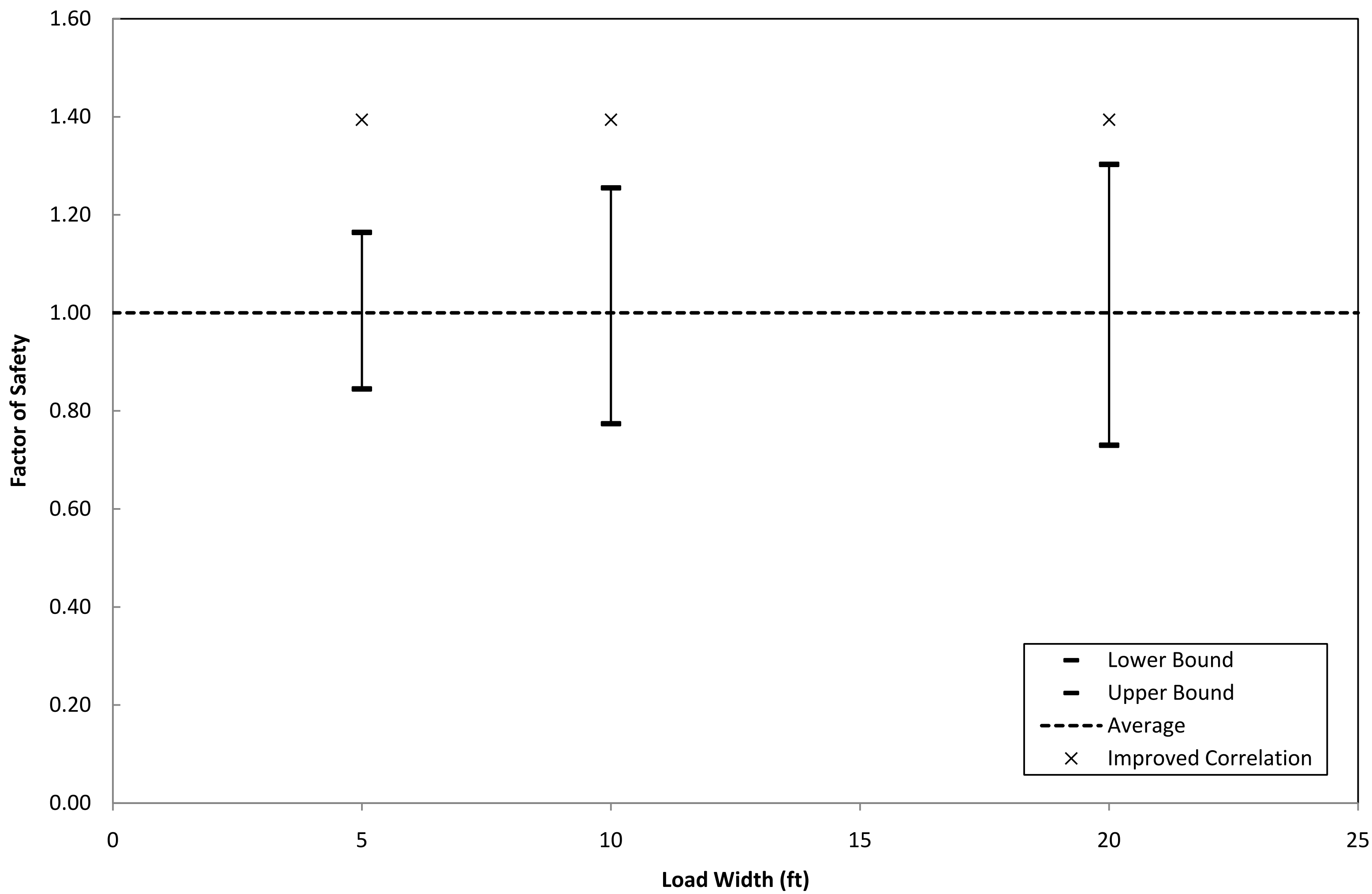


Figure 6.42: Factors of Safety Computed using UTEXAS4 for Site No. 5

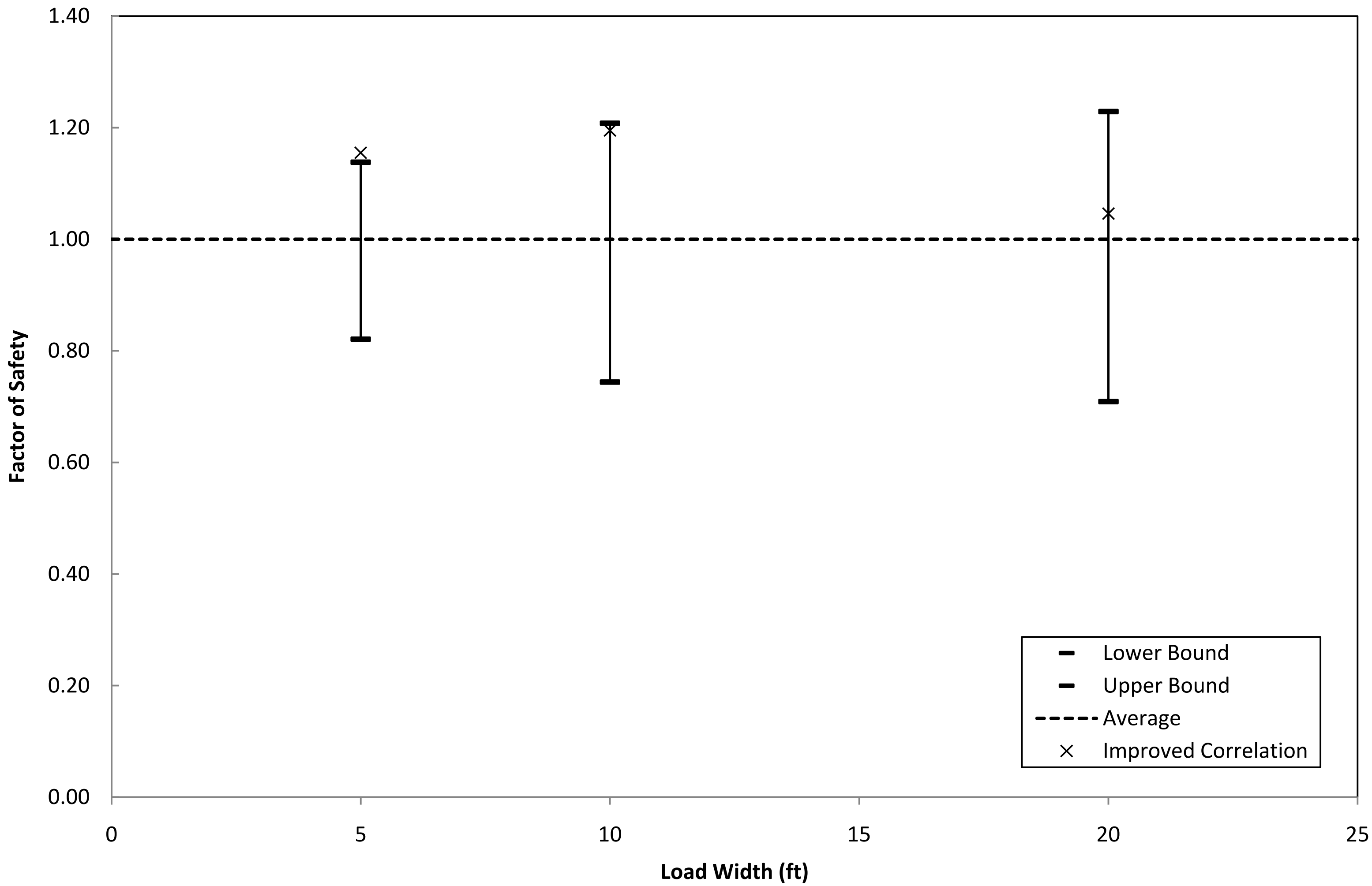


Figure 6.43: Factors of Safety Computed using UTEXAS4 for Site No. 6

Ten of the fifteen factors of safety shown in Figures 6.8 through 6.12 and computed using the shear strength profiles based on the improved correlation fall within the range of values computed using the lower- and upper-bound shear strength profiles; two more factors of safety are within 5 percent of the range. The factors of safety computed for Site No. 5 are the only ones showing a larger deviation.

The average and standard deviation of the factors of safety computed for the lower- and upper-bound strength profiles along with the strength profiles based on the improved correlation are shown in Table 6.7. The factors of safety computed using the lower- and upper- bound shear strength profiles were on average underestimated or overestimated by approximately 25 percent with a standard deviation of approximately 10 percent. On average, the factors of safety computed using the shear strength profiles based on the improved correlation were roughly 1.0 with a standard deviation of 30 percent. The width of the loaded area had little effect on the values shown in Table 6.7.

Table 6.28: Average and Standard Deviation of the Factors of Safety Computed using UTEXAS4

Strength Profile	Average, μ	Standard Deviation, σ
Lower Bound	0.73	0.08
Upper Bound	1.26	0.13
Improved Correlation	1.03	0.30

Chapter 7. Summary, Conclusions and Recommendations

The objective of this project was to develop an improved correlation between Texas Cone Penetrometer blow count and undrained shear strength for soft, clay soils in the upper approximately thirty feet of the ground.

7.1 Summary

Subsurface explorations were carried out by Tolunay-Wong Engineers, Inc. (TWEI) at six sites in the Texas Gulf Coast region where soft soils were expected to exist. Field testing included a series of borings with Texas Cone Penetrometer tests, conventional ("Dutch") piezocone penetration tests, thin-walled tube sampling, and vane shear tests. Laboratory testing was subsequently performed at The University of Texas at Austin on the samples obtained in the field.

Varathungarajan (2008) used the results of these field and laboratory tests to develop average, lower- and upper-bound undrained shear strength profiles for each of the sites. Undrained shear strengths were then computed using existing correlations between Texas Cone Penetrometer blow count and undrained shear strength developed by Hamoudi et al. (1974), Duderstadt et al. (1977), and Kim et al. (2007). The validity of these correlations was analyzed in comparison to the strength profiles developed by Varathungarajan (2008) and an improved correlation was developed.

Finally, bearing capacity analyses were performed for the average, lower- and upper-bound undrained shear strength profiles established by Varathungarajan (2008) as well as for the undrained shear strength profiles determined from the Texas Cone Penetrometer blow counts using the improved correlation developed in this study. The computer software UTEXAS4 (Wright 1999) was utilized to carry out the analyses. Factors of safety were then compared to estimate the reliability of the improved correlation.

7.2 Conclusions

The correlations by Kim et al. (2007) tended to produce strengths that ranged from significantly lower to significantly higher than what are believed to be the undrained shear strength. The correlation suggests that as the moisture content increases the strength increases, which seems fundamentally unsound. An improved correlation (Equations 7.1 and 7.2) was developed based in part on the earlier correlations by Hamoudi et al. (1974) and Duderstadt et al. (1977). It was determined that even with a zero ("weight of hammer") blow count, if the soil can support the weight of the cone, it has some strength. Thus, an intercept value of 300 psf was adopted and the following improved equation is proposed:

$$s_u = 300 + 60N \qquad (7.1)$$

where s_u is in psf. Equation 7.1 can also be expressed in the following form independent of the units used:

$$s_u = p_{atmosphere}(0.142 + 0.028N) \qquad (7.2)$$

where $p_{atmosphere}$ is atmospheric pressure.

From the bearing capacity analyses, the factors of safety determined using the improved correlation showed good agreement with factors of safety determined using the strength profiles developed by Varathungarajan (2008); however, in some cases the factor of safety was apparently overestimated by as much as 60 percent.

7.3 Recommendations

The improved correlation is recommended over the existing correlations and is intended primarily for soft, shallow (less than 30 feet) clays with undrained shear strengths of 1200 psf (approximately 0.47 atmospheres) or less (TCP blow counts of 15 or less). Accordingly, this correlation should be used cautiously. In addition, higher design undrained shear strengths might be realized if more extensive field and laboratory testing. Guidance for such further testing can be found in Varathungarajan (2008). Further studies of the improved correlation for very soft soils would be useful, particularly for evaluation of the “intercept” value in Equation 7.2.

Appendix A

Undrained shear strength profiles, including upper- and lower-bounds, were developed for each site based on the results of the laboratory and field tests. Representative undrained shear strength profiles were selected based on judgment and the evaluation and interpretation of results of field and laboratory tests. These profiles are referred to as the *average* undrained shear strength profiles. The coordinates of the lines representing these profiles are presented in tables in this appendix. Elevation ranges where soils were identified as being non-clay and thus no undrained strengths were considered are indicated in these tables. The details of this data and how the strength profiles were established are presented by Varathungarajan (2008). The strength profiles are plotted in Figures 5.1 through 5.6, 5.10 through 5.15, and 6.3 through 6.7.

Table A.1: Site 1 undrained shear strength profile bounds.

Lower Bound		Average Profile		Upper Bound	
El. (ft)	s_u (psf)	El. (ft)	s_u (psf)	El. (ft)	s_u (psf)
+10	800	+10	1200	+10	1600
+4	800	+4	1200	+4	1600
Non-Clay		Non-Clay		Non-Clay	
0	225	0	250	0	400
-6	375	-6	600	-6	900
Non-Clay		Non-Clay		Non-Clay	
-10	1550	-10	1900	-10	2200
-16	2100	-16	2400	-16	2700

Table A.2: Site 2 undrained shear strength profile bounds.

Lower Bound		Average Profile		Upper Bound	
El. (ft)	s_u (psf)	El. (ft)	s_u (psf)	El. (ft)	s_u (psf)
-2	150	-2	275	-2	375
-10	150	-10	275	-10	375
-26	400	-26	550	-26	800

Table A.3: Site 3 undrained shear strength profile bounds.

Lower Bound		Average Profile		Upper Bound	
El. (ft)	s_u (psf)	El. (ft)	s_u (psf)	El. (ft)	s_u (psf)
+31	500	+31	750	+31	900
+13	850	+13	1150	+13	1300

Table A.4: Site 4 undrained shear strength profile bounds.

Lower Bound		Average Profile		Upper Bound	
El. (ft)	s_u (psf)	El. (ft)	s_u (psf)	El. (ft)	s_u (psf)
+10	600	+10	1000	+10	1920
+6	600	+6	1000	+6	1920
Non-Clay		Non-Clay		Non-Clay	
-2	275	-2	430	-2	510
-16	500	-16	700	-16	800

Table A.5: Site 5 undrained shear strength profile bounds.

Lower Bound		Average Profile		Upper Bound	
El. (ft)	s_u (psf)	El. (ft)	s_u (psf)	El. (ft)	s_u (psf)
-2	150	-2	250	-2	350
-27	150	-27	250	-27	350

Table A.6: Site 6 undrained shear strength profile bounds.

Lower Bound		Average Profile		Upper Bound	
El. (ft)	s_u (psf)	El. (ft)	s_u (psf)	El. (ft)	s_u (psf)
-2	160	-2	300	-2	400
-26	400	-26	540	-26	640

Appendix B

Boring logs were prepared by Tolunay-Wong Engineers, Inc. (TWEI) for each of the six sites and are included in this appendix. Two boring logs were prepared for each site, one for the boring where Texas Highway Department cone penetration tests were performed and the other for the boring where thin-walled tube soil samples were taken. In addition, a summary table was prepared for the vane shear tests for each site with the exception of Site No. 3 due to the high strength of soil at this site. These boring logs and summary tables are presented in the following pages.

Site No. 1 – Port Arthur, TX (Site A)

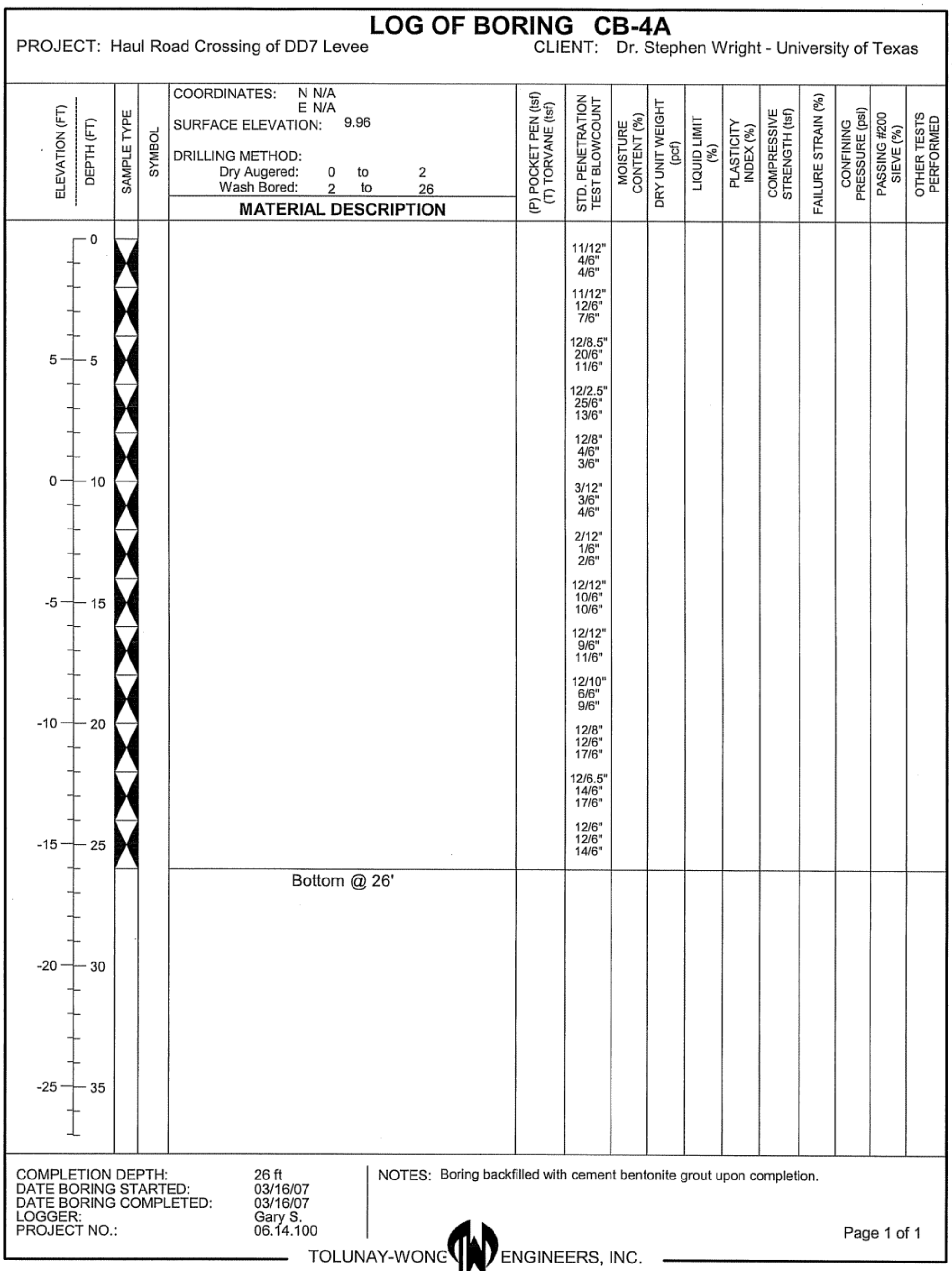


Figure B.1: Boring log with Texas Cone Penetrometer tests for Site No. 1

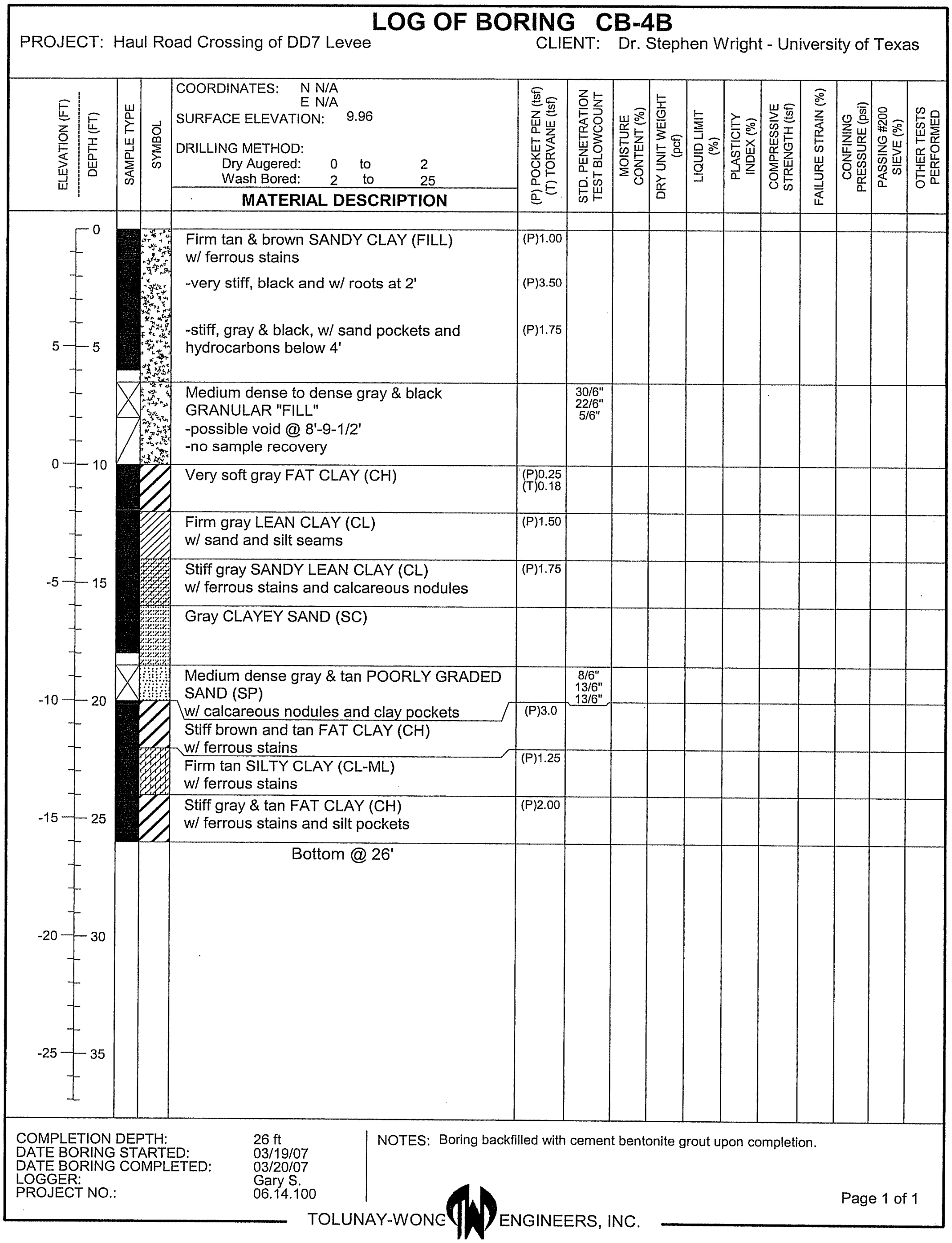


Figure B.2: Boring log with thin-walled tube samples for Site No. 1

Table B.1: Field vane tests for Site No. 1

Project Name: DD7 Haul Road Expansion

Project No. 06.14.100

Client: Total

Technician: Gary S. Gary L.

Drillers: SES

Date: 3/21 & 22/2007

Boring Number	Depth to Bottom of Vane (ft)	Vane Size	Dial Reading @ Initial shear	Dial Reading @ Remold shear	Undrained Shear Strength (psf)Initial Shear	Estimated Undrained Shear Strength (psf) Remold
FB-4	4*	55 mm x 110 mm	69	0	**1441**	**0**
FB-4	11	55 mm x 110 mm	21	8	**439**	**167**
FB-4	17*	55 mm x 110 mm	69	0	**1441**	**0**

* Shear not reached

Notes: Downhole vane testing performed using GEONOR Vane Borer, H-10

Site No. 2 – Port Arthur, TX (Site B)

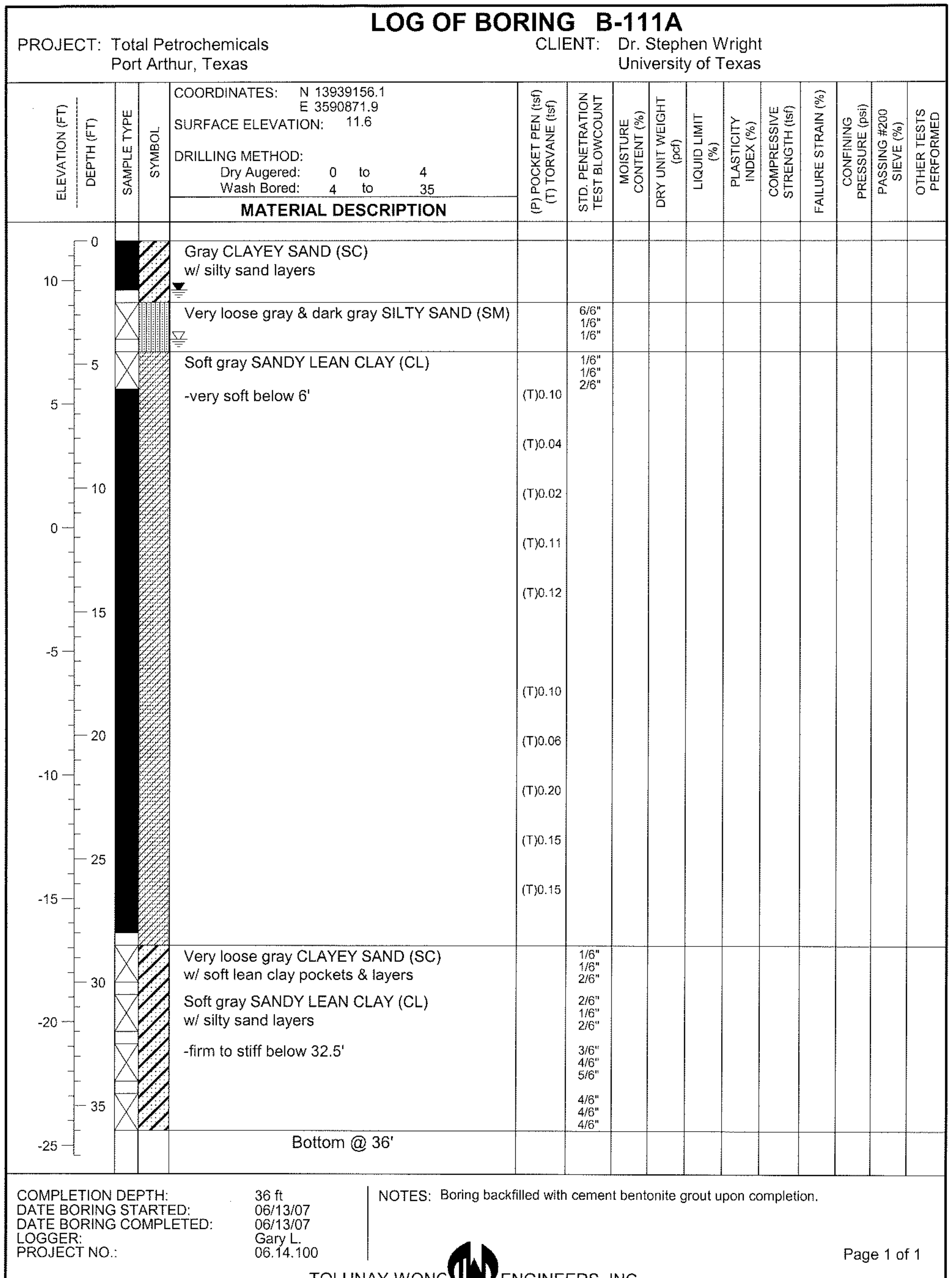


Figure B.3: Boring log with thin-walled tube samples for Site No. 2

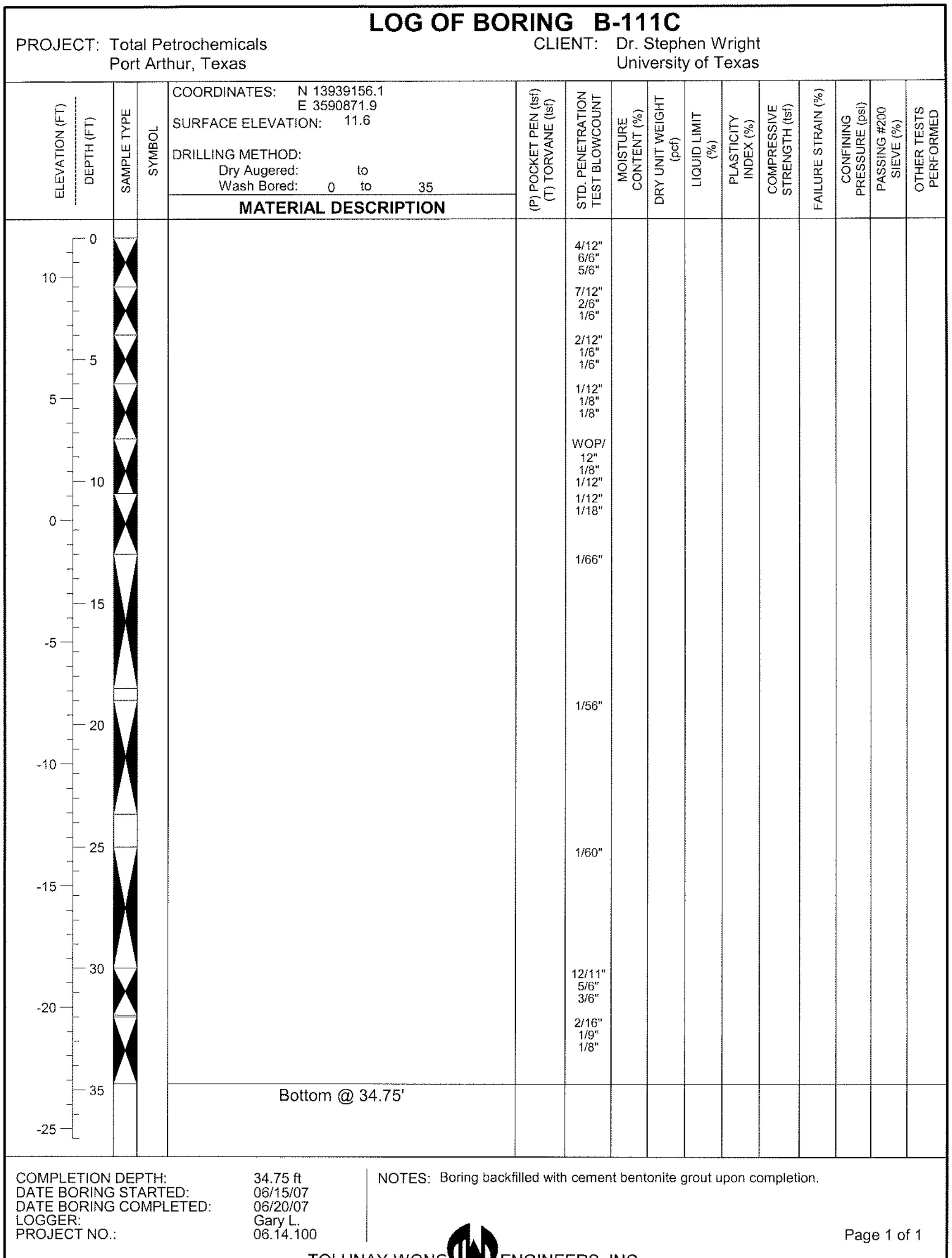


Figure B.4: Boring log with Texas Cone Penetrometer tests for Site No. 2

Table B.2: Field vane tests for Site No. 2

Project Name:	Total Alternate 3 Haul Road	**TWEI Project No.:**	06.14.100	**ASTM Standard:**	D 2573
Client:	Total	**Technician:**	G. Love/R. Lucas	**Coordinates**	N 13939156, E 3590871.9
Drillers:	TWEI	**Date:**	6/6 & 6/7/2007	**Ground Surface El.:**	11.6 ft

Vane Number	Depth to Bottom of Vane (ft)	Vane Size[(1)] (mm)	Dial Reading @ Initial Shear	Dial Reading @ Remolded Shear[(2)]	Uncorrected Undrained Shear Strength - Initial Shear (psf)	Uncorrected Undrained Shear Strength - Remolded (psf)	Estimated or Measured Plasticity Index (PI) [(3)]	Correction Factor (μ)	Corrected Undrained Shear Strength - Initial Shear (psf)	Corrected Undrained Shear Strength - Remolded (psf)
B-111B	6.5	65	20	10	**355**	**167**				
	9.75	65	28	3	**532**	**52**				
	13	65	11	1	**188**	**21**				
	16.25	65	31	4	**553**	**84**				
	19.5	65	34	5	**637**	**94**				
	22.75	65	49	8	**940**	**157**				
	26	65	60	13	**1128**	**209**				
	29.25	65	67	25	**1274**	**449**				

Notes:
(1) Downhole vane testing performed using GEONOR Vane Borer, H-10
(2) After 25 cycles of shear
(3) Bjerrum (1972)

Site No. 3 – Mont Belvieu, TX

LOG OF BORING TD-9A

PROJECT: LDH Terminal
Mont Belvieu, Texas

CLIENT: Dr. Stephen Wright
University of Texas

COORDINATES: N
E
SURFACE ELEVATION:
DRILLING METHOD:
Dry Augered: to
Wash Bored: 0 to 26

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	SYMBOL	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
	0			Firm Gray LEAN CLAY (CL)	(P)1.00										
					(P)1.00										
				-stiff, w/ calcareous deposits below 4'	(P)1.50										
	5			Stiff gray & tan SANDY LEAN CLAY (CL)	(P)1.50										
					(P)2.50										
	10			Stiff gray & tan LEAN CLAY (CL)	(P)2.00										
				Very stiff tan & light gray FAT CLAY (CH)	(P)3.00										
				Tan CLAYEY SAND (SC)	(P)0.50										
	15			Very stiff tan SANDY LEAN CLAY (CL)	(P)2.00										
				Medium dense tan SILTY SAND (SM)		4/6" 6/6" 8/6"									
	20			Tan CLAYEY SAND (SC)	(P)0.50										
				Stiff tan & gray SANDY LEAN CLAY (CL)	(P)1.50										
				-firm below 24'	(P)1.00										
	25			Bottom @ 26'											
	30														
	35														

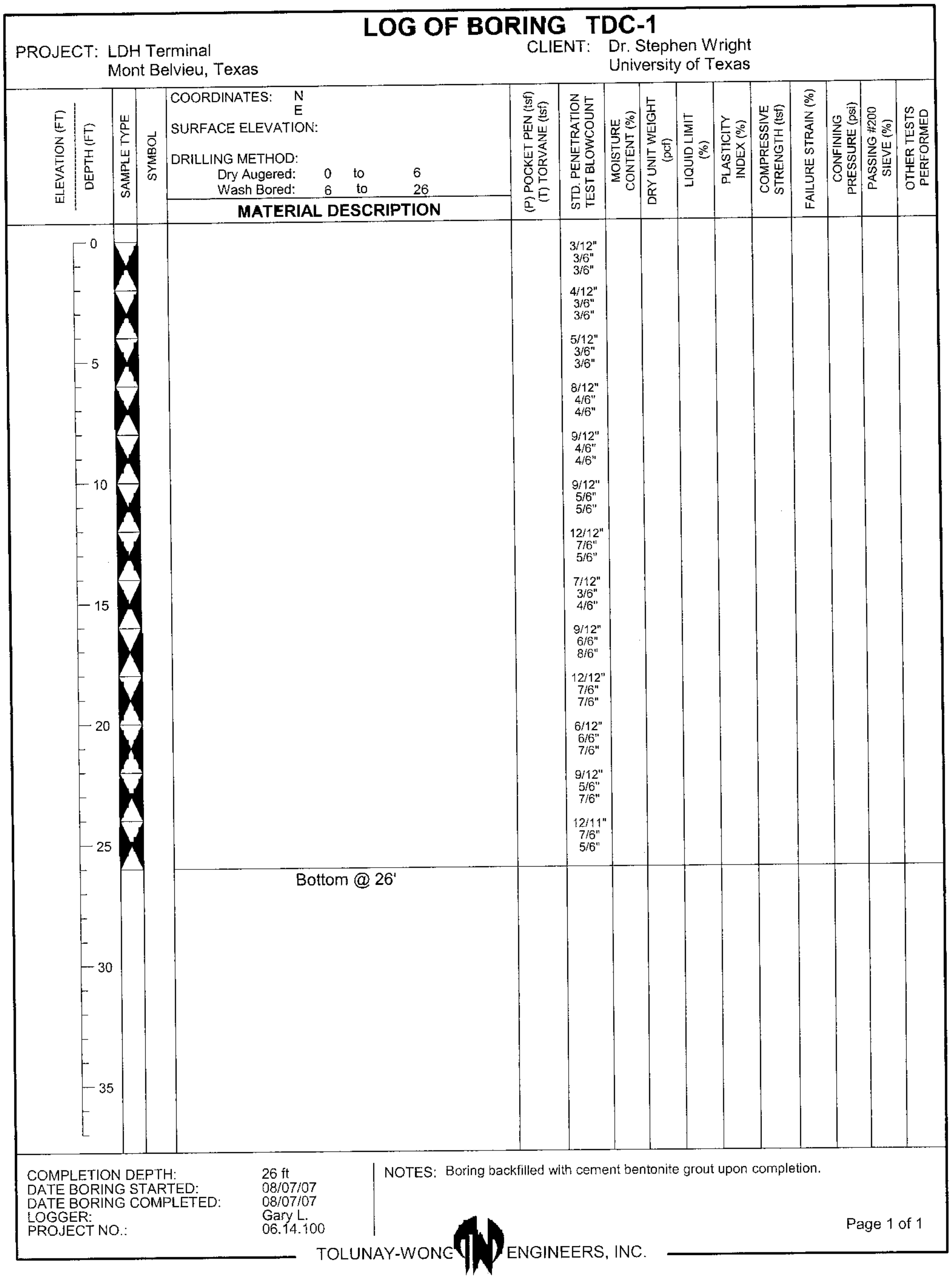
COMPLETION DEPTH:	26 ft
DATE BORING STARTED:	08/09/07
DATE BORING COMPLETED:	08/09/07
LOGGER:	Gary L.
PROJECT NO.:	06.14.100

NOTES: Boring backfilled with cement bentonite grout upon completion.

Page 1 of 1

TOLUNAY-WONG ENGINEERS, INC.

Figure B.5: Boring log with thin-walled tube samples for Site No. 3



LOG OF BORING TDC-1

PROJECT: LDH Terminal
Mont Belvieu, Texas

CLIENT: Dr. Stephen Wright
University of Texas

COORDINATES: N
E
SURFACE ELEVATION:
DRILLING METHOD:
Dry Augered: 0 to 6
Wash Bored: 6 to 26

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	SYMBOL	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
	0					3/12" 3/6" 3/6"									
						4/12" 3/6" 3/6"									
	5					5/12" 3/6" 3/6"									
						8/12" 4/6" 4/6"									
						9/12" 4/6" 4/6"									
	10					9/12" 5/6" 5/6"									
						12/12" 7/6" 5/6"									
						7/12" 3/6" 4/6"									
	15					9/12" 6/6" 8/6"									
						12/12" 7/6" 7/6"									
	20					6/12" 6/6" 7/6"									
						9/12" 5/6" 7/6"									
	25					12/11" 7/6" 5/6"									
				Bottom @ 26'											
	30														
	35														

COMPLETION DEPTH: 26 ft
DATE BORING STARTED: 08/07/07
DATE BORING COMPLETED: 08/07/07
LOGGER: Gary L.
PROJECT NO.: 06.14.100

NOTES: Boring backfilled with cement bentonite grout upon completion.

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TOLUNAY-WONG ENGINEERS, INC.

Figure B.6: Boring log with Texas Cone Penetrometer tests for Site No. 2

Site No. 4 – Beaumont, TX

LOG OF BORING B21C

PROJECT: Texas Gasification Project, Beaumont, Texas
CLIENT: Dr. Steven Wright, University of Texas

COORDINATES: N E
SURFACE ELEVATION:
DRILLING METHOD:
Dry Augered: 0 to 6
Wash Bored: 6 to 26

DEPTH (FT)	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
0	Stiff tan & gray SANDY LEAN CLAY "FILL"	(P)1.50										
	Very stiff tan & orange SANDY LEAN CLAY (CL)	(P)3.00										
	-soft & light gray below 4'	(P)0.50										
5												
	Medium dense tan & light gray CLAYEY SAND (SC) w/ lean clay pockets		3/6" 5/6" 10/6"									
	-loose & light gray @ 8.5'-10'		4/6" 4/6" 6/6"									
10	Soft light gray SANDY LEAN CLAY (CL) w/ organics, wood debris		5/6" 4/6" WOH/6"									
	Firm gray FAT CLAY (CH) w/ organics	(P)0.75										
	-soft below 14'	(P)0.50										
15	-very soft & dark gray below 16'	(T)0.12										
		(T)0.13										
20		(T)0.16										
	Very soft light gray SANDY LEAN CLAY (CL)	(T)0.90										
		(T)0.10										
25												
	Bottom @ 26'											
30												
35												

COMPLETION DEPTH: 26 ft
DATE BORING STARTED: 2-20-200
DATE BORING COMPLETED:
LOGGER: J. Parish
PROJECT NO.: 08-14-004

NOTES: Boring backfilled with cement bentonite grout upon completion.

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TOLUNAY-WONG ENGINEERS, INC.

Figure B.7: Boring log with thin-walled tube samples for Site No. 4

LOG OF BORING B21D

CSJ:
Project: Texas Gasification Project
Beaumont, Texas

Client: Dr. Steven Wright
University of Texas

Project No.: 08-14-004
Date: 2-20-2008
Elevation:
Station No.:
Offset:

Dry Augered: 0 **to** 6 **ft**
Wash Bored: 6 **to** 26 **ft**

Free Water at
Water at

Caving at

ELEV/ DEPTH (ft)	SOIL SYMBOLS & SAMPLER SYMBOLS	THD PEN. 1ST 6"	THD PEN. 2ND 6"	DESCRIPTION	Wc (%)	Dens. (pcf)	Qu or UU (psi)	Str. (%)	LL	PI	Pass #200 (%)
0		3/6"	5/6"								
		2/6"	3/6"								
5		7/6"	10/6"								
		10/6"	8/6"								
		6/6"	2/6"								
10		2/6"	1/6"								
		1/6"	2/6"								
15		WOH/ 6"	WOH/ 6"								
		WOH/ 6"	WOH/ 6"								
		1/6"	2/6"								
20		1/6"	1/6"								
		2/6"	2/6"								
25		4/6"	3/6"								
				Boring terminated at 26.0 ft							
30											

Note(s): Boring backfilled with cement bentonite grout upon completion
WOH= Weight of Hammer

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TOLUNAY-WONG ENGINEERS, INC.

Figure B.8: Boring log with thin-walled tube samples for Site No. 4

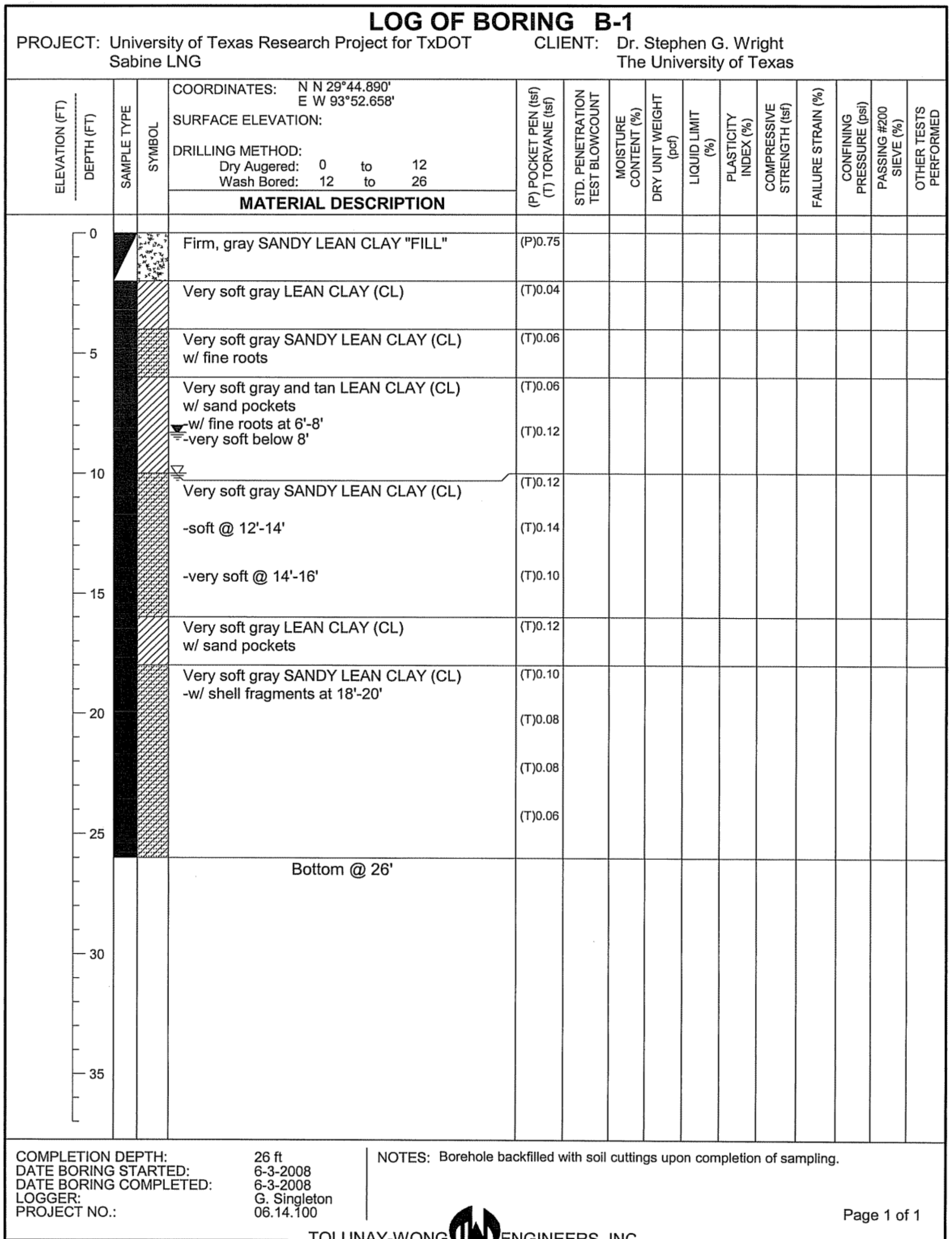
Table B.3: Field vane tests for Site No. 4

Project Name:	Texas Gasification Project	**TWEI Project No.:**	08-14-004	**ASTM Standard:**	D 2573
Client:	Fluor	**Technician:**	G. Siingleton	**Coordinates**	N 13975991 E 3537208
Drillers:	Triangle Resources	**Date:**	3/17/2008	**Ground Surface El.:**	9.0 ft

Vane Number	Depth to Bottom of Vane (ft)	Vane Size(1) (mm)	Dial Reading @ Initial Shear	Dial Reading @ Remolded Shear(2)	Uncorrected Undrained Shear Strength - Initial Shear (psf)	Uncorrected Undrained Shear Strength - Remolded (psf)	Estimated or Measured Plasticity Index (PI) (3)	Correction Factor (μ) (4)	Corrected Undrained Shear Strength - Initial Shear (psf)	Corrected Undrained Shear Strength - Remolded (psf)	Sensitivity, S_T
B-21	13.2	65	39	7	814	125	120	0.54	**440**	**68**	**6.5**
	17.1	65	50	8	1044	157	120	0.54	**564**	**85**	**6.7**
	21.5	65	61	5	1274	104	20	1.00	**1274**	**104**	**12.2**
	25.8 (5)	65	64	(6)	1336	(6)	20	1.00	**1336**	(6)	

Notes:
(1) Downhole vane testing performed using GEONOR Vane Borer, H-10
(2) After 25 cycles of shear
(3) See companion Boring B-21C
(4) Bjerrum (1972)
(5) Unable to push vane shoe below this depth
(6) Unable to obtain remold value

Site No. 5 – Cameron Parish, LA (Site A)



LOG OF BORING B-1

PROJECT: University of Texas Research Project for TxDOT
Sabine LNG

CLIENT: Dr. Stephen G. Wright
The University of Texas

COORDINATES: N N 29°44.890' E W 93°52.658'
SURFACE ELEVATION:
DRILLING METHOD:
Dry Augered: 0 to 12
Wash Bored: 12 to 26

DEPTH (FT)	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
0	Firm, gray SANDY LEAN CLAY "FILL"	(P)0.75										
	Very soft gray LEAN CLAY (CL)	(T)0.04										
5	Very soft gray SANDY LEAN CLAY (CL) w/ fine roots	(T)0.06										
	Very soft gray and tan LEAN CLAY (CL) w/ sand pockets	(T)0.06										
	-w/ fine roots at 6'-8' -very soft below 8'	(T)0.12										
10	Very soft gray SANDY LEAN CLAY (CL)	(T)0.12										
	-soft @ 12'-14'	(T)0.14										
15	-very soft @ 14'-16'	(T)0.10										
	Very soft gray LEAN CLAY (CL) w/ sand pockets	(T)0.12										
	Very soft gray SANDY LEAN CLAY (CL) -w/ shell fragments at 18'-20'	(T)0.10										
20		(T)0.08										
		(T)0.08										
25		(T)0.06										
	Bottom @ 26'											
30												
35												

COMPLETION DEPTH: 26 ft
DATE BORING STARTED: 6-3-2008
DATE BORING COMPLETED: 6-3-2008
LOGGER: G. Singleton
PROJECT NO.: 06.14.100

NOTES: Borehole backfilled with soil cuttings upon completion of sampling.

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TOLUNAY-WONG ENGINEERS, INC.

Figure B.9: Boring log with thin-walled tube samples for Site No. 5

LOG OF BORING TXDOT-B1

Project: University of Texas Research Project for TxDOT
Sabine LNG

Client: Dr. Stephen G. Wright
The University of Texas

Dry Augered: 0 **to** 4 **ft** **Free Water at**
Wash Bored: 4 **to** 26 **ft** **Water at**

Project No.: 06.14.100
Date: 6-3-2008
Elevation:
Station No.:
Offset:
Caving at

ELEV/ DEPTH (ft)	SOIL SYMBOLS & SAMPLER SYMBOLS	THD PEN. 1ST 6"	THD PEN. 2ND 6"	Description	Wc (%)	Dens. (pcf)	Qu or UU (psi)	Str. (%)	LL	PI	#200 (%)
0											
		2/6"	1/6"								
5		1/6"	1/6"								
		1/6"	1/6"								
		1/6"	1/6"								
10		1/6"	2/6"								
		1/6"	1/6"								
15		1/6"	1/6"								
		1/6"	1/6"								
		2/6"	1/6"								
20		1/6"	2/6"								
		3/6"	4/6"								
		4/6"	4/6"								
25											
30											

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TOLUNAY-WONG ENGINEERS, INC.

Figure B.10: Boring log with Texas Cone Penetrometer tests for Site No. 5

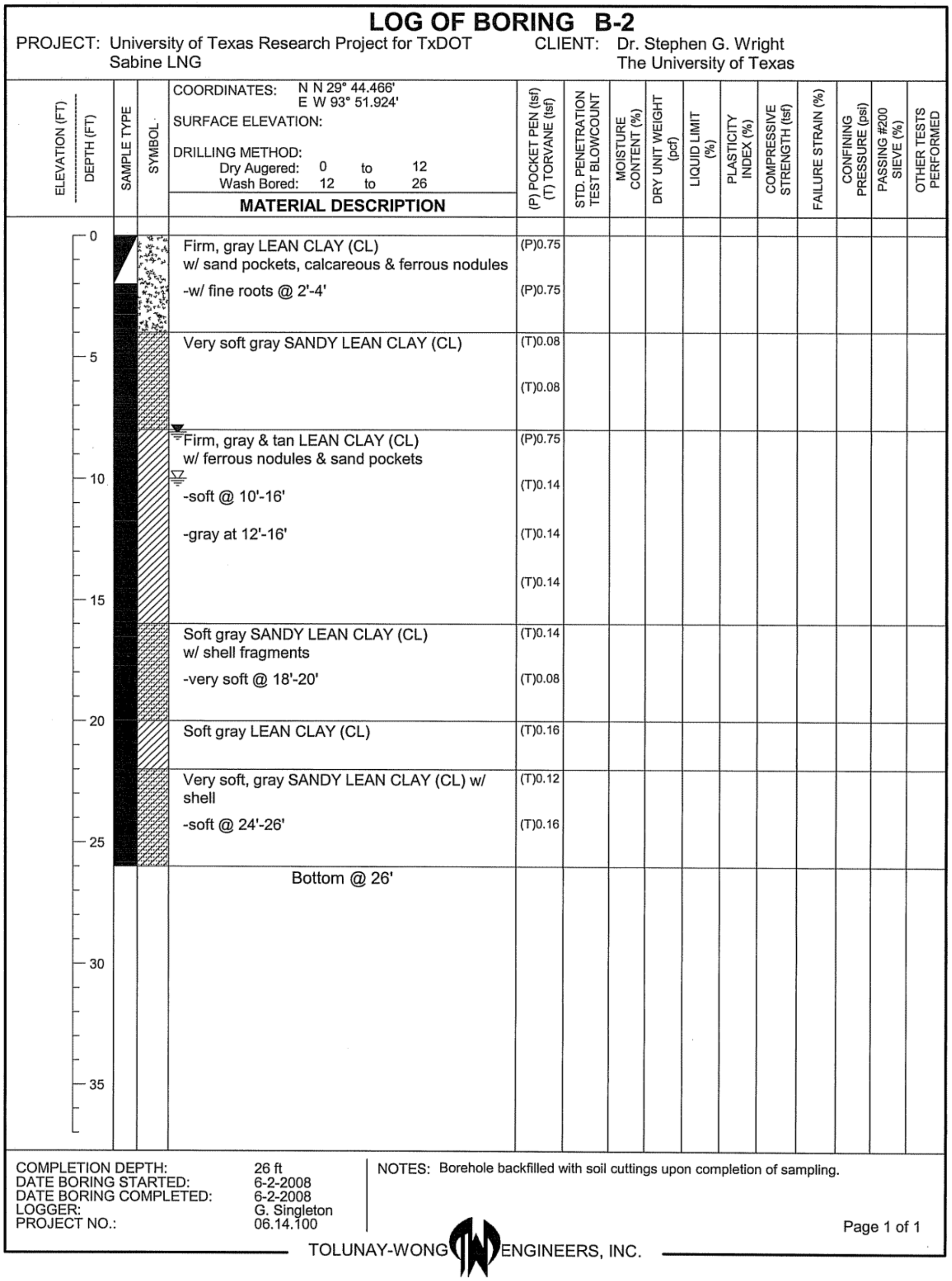
Table B.4: Field vane tests for Site No. 5

Project Name:	University of Texas Research Projec Sabine LNG	TWEI Project No.:	06-14-100	ASTM Standard:	D 2573
Client:	University of Texas	Technician:	G. Singleton	Coordinates	N 29° 44.890' W 93°52.658'
Drillers:	TWEI	Date:	6/3/2008	Ground Surface El.:	

Vane Number	Depth to Bottom of Vane (ft)	Vane Size[1] (mm)	Dial Reading @ Initial Shear	Dial Reading @ Remolded Shear[2]	Uncorrected Undrained Shear Strength - Initial Shear (psf)	Uncorrected Undrained Shear Strength - Remolded (psf)	Estimated or Measured Plasticity Index (PI)	Correction Factor (μ) [4]	Corrected Undrained Shear Strength - Initial Shear (psf)	Corrected Undrained Shear Strength - Remolded (psf)	Sensitivity, S_T
VS-1	3.1	65	17	5	188	52			0	0	
	6.25	65	36	4	397	42			0	0	
	9.6	65	269	6	282	63			0	0	
	12.6	65	25	4	271	42			0	0	
	16.1	65	27	5	292	52			0	0	
	19.25	65	30	6	334	63			0	0	
	22.6	65	26	4	282	42			0	0	
	25.9	65	35	7	397	73			0	0	

Notes:
(1) Downhole vane testing performed using GEONOR Vane Borer, H-10
(2) After 25 cycles of shear
(3) Stopped by operator for health and safety reason (reaching device limits) / test did not reach shear failure
(4) Bjerrum (1972)

Site No. 6 – Cameron Parish, LA (Site B)



LOG OF BORING B-2

PROJECT: University of Texas Research Project for TxDOT Sabine LNG

CLIENT: Dr. Stephen G. Wright, The University of Texas

COORDINATES: N N 29° 44.466' E W 93° 51.924'

SURFACE ELEVATION:

DRILLING METHOD:
Dry Augered: 0 to 12
Wash Bored: 12 to 26

DEPTH (FT)	MATERIAL DESCRIPTION	(P) POCKET PEN (tsf) (T) TORVANE (tsf)	STD. PENETRATION TEST BLOWCOUNT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	COMPRESSIVE STRENGTH (tsf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	PASSING #200 SIEVE (%)	OTHER TESTS PERFORMED
0	Firm, gray LEAN CLAY (CL) w/ sand pockets, calcareous & ferrous nodules	(P)0.75										
	-w/ fine roots @ 2'-4'	(P)0.75										
5	Very soft gray SANDY LEAN CLAY (CL)	(T)0.08										
		(T)0.08										
	Firm, gray & tan LEAN CLAY (CL) w/ ferrous nodules & sand pockets	(P)0.75										
10	-soft @ 10'-16'	(T)0.14										
	-gray at 12'-16'	(T)0.14										
15		(T)0.14										
	Soft gray SANDY LEAN CLAY (CL) w/ shell fragments	(T)0.14										
	-very soft @ 18'-20'	(T)0.08										
20	Soft gray LEAN CLAY (CL)	(T)0.16										
	Very soft, gray SANDY LEAN CLAY (CL) w/ shell	(T)0.12										
25	-soft @ 24'-26'	(T)0.16										
	Bottom @ 26'											

COMPLETION DEPTH: 26 ft
DATE BORING STARTED: 6-2-2008
DATE BORING COMPLETED: 6-2-2008
LOGGER: G. Singleton
PROJECT NO.: 06.14.100

NOTES: Borehole backfilled with soil cuttings upon completion of sampling.

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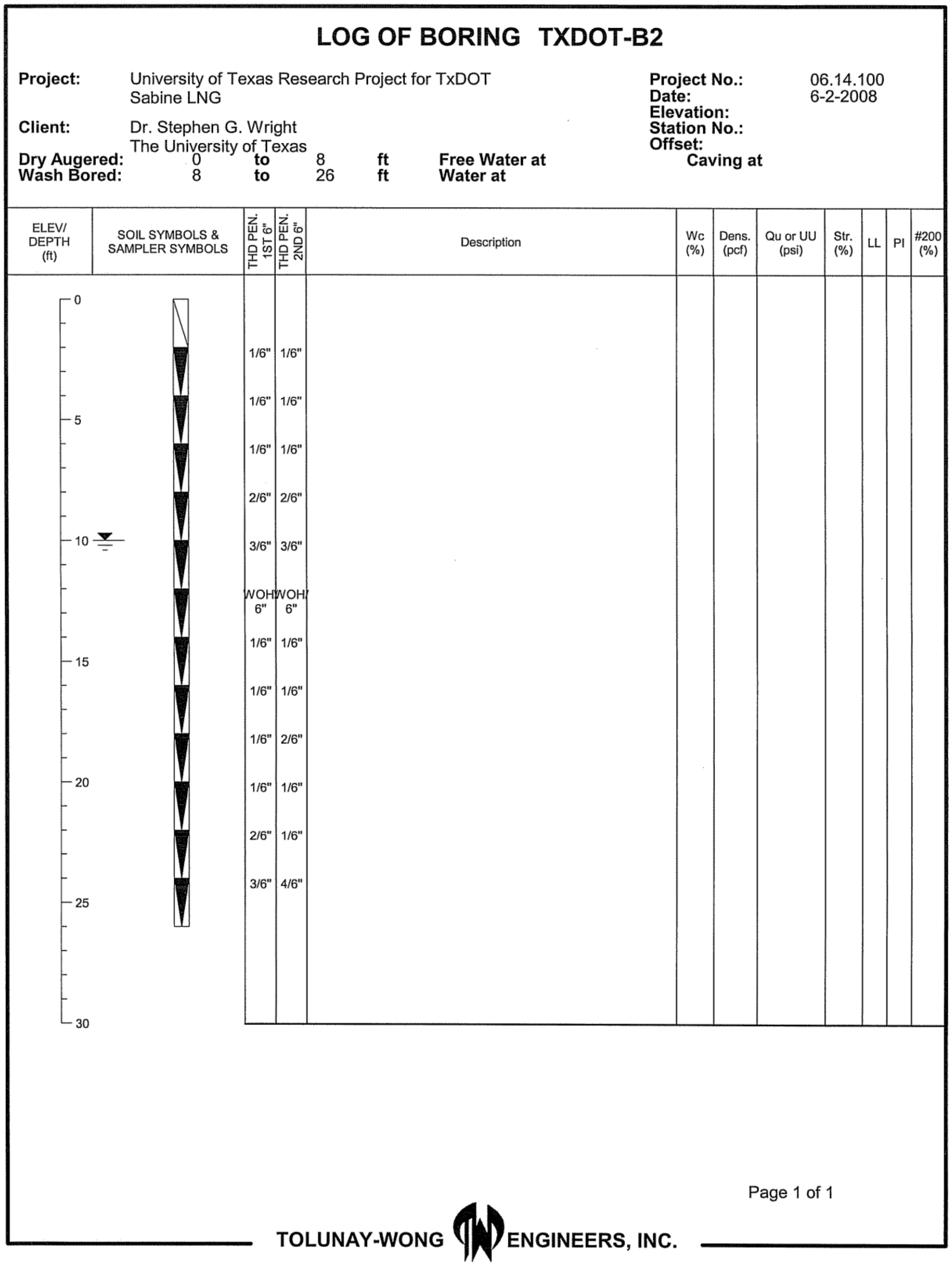
TOLUNAY-WONG ENGINEERS, INC.

Figure B.11: Boring log with thin-walled tube samples for Site No. 6



LOG OF BORING TXDOT-B2

Project: University of Texas Research Project for TxDOT
Sabine LNG

Client: Dr. Stephen G. Wright
The University of Texas

Dry Augered: 0 **to** 8 **ft** **Free Water at**
Wash Bored: 8 **to** 26 **ft** **Water at**

Project No.: 06.14.100
Date: 6-2-2008
Elevation:
Station No.:
Offset:
Caving at

ELEV/ DEPTH (ft)	SOIL SYMBOLS & SAMPLER SYMBOLS	THD PEN. 1ST 6"	THD PEN. 2ND 6"	Description	Wc (%)	Dens. (pcf)	Qu or UU (psi)	Str. (%)	LL	PI	#200 (%)
0											
		1/6"	1/6"								
5		1/6"	1/6"								
		1/6"	1/6"								
		2/6"	2/6"								
10		3/6"	3/6"								
		WOH/ 6"	WOH/ 6"								
15		1/6"	1/6"								
		1/6"	1/6"								
		1/6"	2/6"								
20		1/6"	1/6"								
		2/6"	1/6"								
25		3/6"	4/6"								
30											

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TOLUNAY-WONG ENGINEERS, INC.

Figure B.12: Boring log with Texas Cone Penetrometer tests for Site No. 6

Table B.5: Field vane tests for Site No. 6

Project Name: University of Texas Research Projec Sabine LNG
TWEI Project No.: 06-14-100
ASTM Standard: D 2573

Client: University of Texas
Technician: G. Singleton
Coordinates N 29° 44.466' W 93°511.924'

Drillers: TWEI
Date: 6/2/2008
Ground Surface El.:

Vane Number	Depth to Bottom of Vane (ft)	Vane Size[1] (mm)	Dial Reading @ Initial Shear	Dial Reading @ Remolded Shear[2]	Uncorrected Undrained Shear Strength - Initial Shear (psf)	Uncorrected Undrained Shear Strength - Remolded (psf)	Estimated or Measured Plasticity Index (PI)	Correction Factor (μ) [4]	Corrected Undrained Shear Strength - Initial Shear (psf)	Corrected Undrained Shear Strength - Remolded (psf)	Sensitivity, S_T
VS-2	3.1	65	67	9	793	84			0	0	
	6.25	65	20	6	219	63			0	0	
	9.6	65	13	6	125	63			0	0	
	12.6	65	42	6	459	63			0	0	
	16.1	65	47	3	532	31			0	0	
	19.25	65	52	7	574	84			0	0	
	22.6	65	36	8	407	94			0	0	
	25.9	65	75[3]	--[3]	887	--			0		

Notes:
(1) Downhole vane testing performed using GEONOR Vane Borer, H-10
(2) After 25 cycles of shear
(3) Stopped by operator for health and safety reason (reaching device limits) / test did not reach shear failure
(4) Bjerrum (1972)

Appendix C

In Chapter 2, the minimum undrained shear strength required to support the Texas Cone Penetrometer cone was computed. A strength profile corresponding to the minimum strength required to support the cone was developed for each site and is shown in Figure 5.1 through 5.6 and 5.10 through 5.15. The computations for these strength profiles were made using Equations C.1, C.2, and C.3 and are presented in Tables C.2 to C.7. All constants are shown in Table C.1.

$$\sigma_{v0} = D\gamma_s \qquad \text{(C.1)}$$

$$q_{TCP} = \frac{W_C + W_H + W_{DR} * L_{DR} + \gamma_{DF} * D * (A_C - A_{DR})}{A_C} \qquad \text{(C.2)}$$

$$s_u = \frac{q_{TCP} - \sigma_{v0}}{N} \qquad \text{(C.3)}$$

Table C.1: Constants used in the Computations

Cone Weight, W_C (lbs)	7
Cone Area, A_C (ft^2)	0.049
Drilling Rod Weight, W_{DR} (lbs/ft)	4
Drilling Rod Length, L_{DR} (ft)	Varies
Drilling Rod Area, A_{DR} (ft^2)	0.006
Hammer Weight, W_H (lbs)	170

Table C.2: Minimum undrained shear strengths required to support the Texas Cone Penetrometer cone for Site No. 1

Elevation (ft)	Depth (ft)	Soil Type	Unit Weight, γ_s (pcf)	N	σ_{vo} (psf)	q_{TCP} (psf)	S_u Req'd (psf)
10	0	Sandy Clay Fill	120	15	0	3606	240
9	1				120	3748	242
8	2				240	3891	243
7	3				360	4033	245
6	4				480	4175	246
5	5				600	4318	248
4	6				720	4460	249
3	7	Granular Fill	120	15	840	4602	251
2	8				960	4745	252
1	9				1080	4887	254
0	10				1200	5030	255
-1	11	Fat Clay	120	15	1320	5172	257
-2	12				1440	5314	258
-3	13				1560	5457	260
-4	14	Lean Clay	120	15	1680	5599	261
-5	15				1800	5741	263
-6	16				1920	5884	264
-7	17	Sand	130	15	2050	6026	265
-8	18				2180	6169	266
-9	19				2310	6311	267
-10	20				2440	6453	268
-11	21	Fat Clay	120	15	2560	6596	269
-12	22				2680	6738	271
-13	23				2800	6880	272
-14	24				2920	7023	274
-15	25				3040	7165	275
-16	26				3160	7308	277

Table C.3: Minimum undrained shear strengths required to support the Texas Cone Penetrometer cone for Site No. 2

Elevation (ft)	Depth (ft)	Soil Type	Unit Weight, γ_s	N	σ_{vo} (psf)	q_{TCP} (psf)	S_u Req'd (psf)
3	0	Clayey and Silty Sand	125	15	0	3606	240
2	1				125	3748	242
1	2				250	3891	243
0	3				375	4033	244
-1	4				500	4175	245
-2	5				625	4318	246
-3	6	Fat Clay	110	15	735	4460	248
-4	7				845	4602	250
-5	8				955	4745	253
-6	9				1065	4887	255
-7	10				1175	5030	257
-8	11				1285	5172	259
-9	12				1395	5314	261
-10	13				1505	5457	263
-11	14				1615	5599	266
-12	15				1725	5741	268
-13	16				1835	5884	270
-14	17				1945	6026	272
-15	18				2055	6169	274
-16	19				2165	6311	276
-17	20				2275	6453	279
-18	21				2385	6596	281
-19	22				2495	6738	283
-20	23				2605	6880	285
-21	24				2715	7023	287
-22	25				2825	7165	289
-23	26				2935	7308	292
-24	27				3045	7450	294
-25	28				3155	7592	296
-26	29				3265	7735	298
-27	30	Sandy Clay	120	15	3385	7877	299
-28	31				3505	8019	301
-29	32				3625	8162	302
-30	33				3745	8304	304
-31	34				3865	8446	305
-32	35				3985	8589	307

Table C.4: Minimum undrained shear strengths required to support the Texas Cone Penetrometer cone for Site No. 3

Elevation (ft)	Depth (ft)	Soil Type	Unit Weight, γ_s (pcf)	N	σ_{vo} (psf)	q_{TCP} (psf)	S_u Req'd (psf)
31	0	Lean and Fat Clay	110	15	0	3606	240
30	1				110	3748	243
29	2				220	3891	245
28	3				330	4033	247
27	4				440	4175	249
26	5				550	4318	251
25	6				660	4460	253
24	7				770	4602	255
23	8				880	4745	258
22	9				990	4887	260
21	10				1100	5030	262
20	11				1210	5172	264
19	12				1320	5314	266
18	13				1430	5457	268
17	14				1540	5599	271
16	15				1650	5741	273
15	16				1760	5884	275
14	17				1870	6026	277
13	18				1980	6169	279
12	19	Clayey Sand	110	15	2090	6311	281
11	20				2200	6453	284
10	21				2310	6596	286
9	22				2420	6738	288
8	23				2530	6880	290
7	24				2640	7023	292
6	25				2750	7165	294
5	26				2860	7308	297

Table C.5: Minimum undrained shear strengths required to support the Texas Cone Penetrometer cone for Site No. 4

Elevation (ft)	Depth (ft)	Soil Type	Unit Weight, γ_s (pcf)	N	σ_{vo} (psf)	q_{TCP} (psf)	S_u Req'd (psf)
10	0				0	3606	240
9.5	0.5				60	3677	241
9	1				120	3748	242
8.5	1.5				180	3819	243
8	2	Sandy Clay Fill	120	15	240	3891	243
7.5	2.5				300	3962	244
7	3				360	4033	245
6.5	3.5				420	4104	246
6	4				480	4175	246
5.5	4.5				542.5	4246	247
5	5				605	4318	248
4.5	5.5				667.5	4389	248
4	6				730	4460	249
3.5	6.5				792.5	4531	249
3	7				855	4602	250
2.5	7.5	Sand	125	15	917.5	4674	250
2	8				980	4745	251
1.5	8.5				1042.5	4816	252
1	9				1105	4887	252
0.5	9.5				1167.5	4958	253
0	10				1230	5030	253
-0.5	10.5				1292.5	5101	254
-1	11				1355	5172	254
-1.5	11.5				1395	5243	257
-2	12				1435	5314	259
-2.5	12.5				1475	5385	261
-3	13				1515	5457	263
-3.5	13.5				1555	5528	265
-4	14				1595	5599	267
-4.5	14.5				1635	5670	269
-5	15				1675	5741	271
-5.5	15.5				1715	5813	273
-6	16				1755	5884	275
-6.5	16.5	Organic Clay	80	15	1795	5955	277
-7	17				1835	6026	279
-7.5	17.5				1875	6097	281
-8	18				1915	6169	284
-8.5	18.5				1955	6240	286
-9	19				1995	6311	288
-9.5	19.5				2035	6382	290
-10	20				2075	6453	292
-10.5	20.5				2115	6524	294
-11	21				2155	6596	296
-11.5	21.5				2195	6667	298
-12	22				2235	6738	300
-12.5	22.5				2280	6809	302
-13	23				2325	6880	304
-13.5	23.5				2370	6952	305
-14	24	Sandy Fat Clay	90	15	2415	7023	307
-14.5	24.5				2460	7094	309
-15	25				2505	7165	311
-15.5	25.5				2550	7236	312
-16	26				2595	7308	314

Table C.6: Minimum undrained shear strengths required to support the Texas Cone Penetrometer cone for Site No. 5

Elevation (ft)	Depth (ft)	Soil Type	Unit Weight, γ_s (pcf)	N	σ_{vo} (psf)	q_{TCP} (psf)	S_u Req'd (psf)
0	0	Sandy Clay Fill	105	15	0	3606	240
-0.5	0.5				52.5	3677	242
-1	1				105	3748	243
-1.5	1.5				157.5	3819	244
-2	2				210	3891	245
-2.5	2.5	Fat Clay and Sandy Fat Clay	105	15	262.5	3962	247
-3	3				315	4033	248
-3.5	3.5				367.5	4104	249
-4	4				420	4175	250
-4.5	4.5				472.5	4246	252
-5	5				525	4318	253
-5.5	5.5				577.5	4389	254
-6	6				630	4460	255
-6.5	6.5				682.5	4531	257
-7	7				735	4602	258
-7.5	7.5				787.5	4674	259
-8	8				840	4745	260
-8.5	8.5				892.5	4816	262
-9	9				945	4887	263
-9.5	9.5				997.5	4958	264
-10	10				1050	5030	265
-10.5	10.5				1102.5	5101	267
-11	11				1155	5172	268
-11.5	11.5				1207.5	5243	269
-12	12				1260	5314	270
-12.5	12.5				1312.5	5385	272
-13	13				1365	5457	273
-13.5	13.5				1417.5	5528	274
-14	14				1470	5599	275
-14.5	14.5				1522.5	5670	277
-15	15				1575	5741	278
-15.5	15.5				1627.5	5813	279
-16	16				1680	5884	280
-16.5	16.5				1732.5	5955	281
-17	17				1785	6026	283
-17.5	17.5				1837.5	6097	284
-18	18				1890	6169	285
-18.5	18.5				1942.5	6240	286
-19	19				1995	6311	288
-19.5	19.5				2047.5	6382	289
-20	20				2100	6453	290
-20.5	20.5				2152.5	6524	291
-21	21				2205	6596	293
-21.5	21.5				2257.5	6667	294
-22	22				2310	6738	295
-22.5	22.5				2362.5	6809	296
-23	23				2415	6880	298
-23.5	23.5				2467.5	6952	299
-24	24				2520	7023	300
-24.5	24.5				2572.5	7094	301
-25	25				2625	7165	303
-25.5	25.5				2677.5	7236	304
-26	26				2730	7308	305

Table C.7: Minimum undrained shear strengths required to support the Texas Cone Penetrometer cone for Site No. 6

Elevation (ft)	Depth (ft)	Soil Type	Unit Weight, γ_s (pcf)	N	σ_{vo} (psf)	q_{TCP} (psf)	S_u Req'd (psf)
0	0	Sandy Clay Fill	105	15	0	3606	240
-0.5	0.5				52.5	3677	242
-1	1				105	3748	243
-1.5	1.5				157.5	3819	244
-2	2				210	3891	245
-2.5	2.5	Fat Clay and Sandy Fat Clay	105	15	262.5	3962	247
-3	3				315	4033	248
-3.5	3.5				367.5	4104	249
-4	4				420	4175	250
-4.5	4.5				472.5	4246	252
-5	5				525	4318	253
-5.5	5.5				577.5	4389	254
-6	6				630	4460	255
-6.5	6.5				682.5	4531	257
-7	7				735	4602	258
-7.5	7.5				787.5	4674	259
-8	8				840	4745	260
-8.5	8.5				892.5	4816	262
-9	9				945	4887	263
-9.5	9.5				997.5	4958	264
-10	10				1050	5030	265
-10.5	10.5				1102.5	5101	267
-11	11				1155	5172	268
-11.5	11.5				1207.5	5243	269
-12	12				1260	5314	270
-12.5	12.5				1312.5	5385	272
-13	13				1365	5457	273
-13.5	13.5				1417.5	5528	274
-14	14				1470	5599	275
-14.5	14.5				1522.5	5670	277
-15	15				1575	5741	278
-15.5	15.5				1627.5	5813	279
-16	16				1680	5884	280
-16.5	16.5				1732.5	5955	281
-17	17				1785	6026	283
-17.5	17.5				1837.5	6097	284
-18	18				1890	6169	285
-18.5	18.5				1942.5	6240	286
-19	19				1995	6311	288
-19.5	19.5				2047.5	6382	289
-20	20				2100	6453	290
-20.5	20.5				2152.5	6524	291
-21	21				2205	6596	293
-21.5	21.5				2257.5	6667	294
-22	22				2310	6738	295
-22.5	22.5				2362.5	6809	296
-23	23				2415	6880	298
-23.5	23.5				2467.5	6952	299
-24	24				2520	7023	300
-24.5	24.5				2572.5	7094	301
-25	25				2625	7165	303
-25.5	25.5				2677.5	7236	304
-26	26				2730	7308	305

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