"DETERMINING THE EFFECT OF STAGE TESTING ON THE DIMENSIONLESS PILE SIDE SHEAR SETUP FACTOR"

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SI* (MODERN METRIC) CONVERSION FACTORS					
Property	Symbol	When You Know	Multiply By	To Find	Symbol
APPROXIMATE CONVERSIONS TO SI UNITS					
LENGTH	in	inches	25.4	millimeters	mm
	ft	feet	0.305	meters	m
	yd	yards	0.914	meters	m
	mi	miles	1.61	kilometers	km
AREA	in^2	square inches	645.2	square millimeters	mm^2
	ft^2	square feet	0.093	square meters	m^2
	yd^2	square yards	0.836	square meters	m^2
	ac	acres	0.405	hectares	ha
VOLUME	mi^2	square miles	2.59	square kilometers	km^2
	fl oz	fluid ounces	29.57	milliliters	ml
	gal	gallons	3.785	liters	l
	ft^3	cubic feet	0.028	cubic meters	m^3
	yd^3	cubic yards	0.765	cubic meters	m^3
MASS	oz	ounces	28.35	grams	g
	lb	pounds	0.454	kilograms	kg
	T	short tons (2000lb)	0.907	megagrams	Mg
TEMPERATURE (exact)	°F	Fahrenheit temperature	(°F-32)/1.8	Celsius temperature	°C
IILLUMINATION	fc	foot-candles	10.76	lux	lx
	fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
FORCE	lbf	poundforce	4.45	Newtons	N
PRESSURE	psi	poundforce/square inch	6.89	kiloPascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS					
LENGTH	mm	millimeters	0.039	inches	in
	m	meters	3.28	feet	ft
	m	meters	1.09	yards	yd
	km	kilometers	0.621	miles	mi
AREA	mm^2	square millimeters	0.0016	square inches	in^2
	m^2	square meters	10.764	square feet	ft^2
	m^2	square meters	1.195	square yards	yd^2
	ha	hectares	2.47	acres	ac
	km^2	square kilometers	0.386	square miles	mi^2
VOLUME	ml	milliliters	0.034	fluid ounces	fl oz
	l	liters	0.264	gallons	gal
	m^3	cubic meters	35.71	cubic feet	ft^3
	m^3	cubic meters	1.307	cubic yards	yd^3
MASS	g	grams	0.035	ounces	oz
	kg	kilograms	2.202	pounds	lb
	Mg	megagrams	1.103	short tons (2000lb)	T
TEMPERATURE (exact)	°C	Celsius temperature	1.8°C + 32	Fahrenheit temperature	°F
IILLUMINATION	lx	lux	0.0929	foot-candles	fc
	cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
FORCE	N	Newtons	0.225	poundforce	lbf
PRESSURE	kPa	kiloPascals	0.145	poundforce/square inch	psi

* SI is the symbol for the International System of Units. (Revised August 1992)
Appropriate rounding should be made to comply with Section 4 of ASTM E380

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ABSTRACT

The research work described herein follows previous FDOT research published by Bullock (1999) and by McVay, Schmertmann, Townsend, & Bullock (1999), which investigates the change in pile side shear capacity with time. Many piles exhibit a capacity increase, termed "setup" or "freeze". Although casually observed by many engineers and contractors, research documentation of setup is limited and design methods do not routinely include it. The previous FDOT research investigated setup and determined the "Setup Factor A" for five piles in Florida soils. However, this research was performed by testing the same piles repeatedly, a commonly accepted test method referred to as "staged" testing. The possible effects of staged testing were not determined during the previous work and provided a caveat to the results.

Bullock (1999) also introduced the standard penetration test with torque measurement (SPT-T) as a cost-effective precursor insitu test for pile setup. The research described herein uses the SPT-T to investigate staged testing adjacent to the test pile driven during the previous research at the Seabreeze Bridge in Daytona Beach, Florida. Twelve borings provided SPT-T results in two soil layers: a silty sand and a shelly clay. Staged SPT-T tests were conducted in three of the borings at nominal times of 5, 30, 180, and 1080 minutes after the driving of the SPT sampler, and unstaged tests were conducted in the remaining nine borings at similar times.

Similar to the Vilano Beach sands, the SPT-T results in the Seabreeze sand layer did not exhibit setup, and were not useful for investigating staged testing effects. However, the Seabreeze clay did indicate significant unstaged side shear setup, and the staged tests measured a 150% increase beyond the unstaged side shear. This yields a ratio of $(A_{Unstaged}/A_{Staged}) = 0.4$, which is further supported by test pile data published in the literature. Side shear setup from staged pile tests in clay at both the Seabreeze and Vilano Beach sites also correlated well with staged SPT-T side shear setup, further validating the SPT-T as a setup predictor test.

1. INTRODUCTION

Many types of structures may require support from deep foundations: multi-story buildings, bridges, parking garages, sporting arenas, stadiums, etc. Deep foundation costs, relative to that of the structure, typically range from 5% for some buildings to as much as 30% for some bridges. For economical design, quality control, and quality assurance, engineers routinely test the capacity of deep foundation elements during and/or after their installation, using both static and dynamic methods. For driven piles, these tests often indicate a change in side shear capacity with time after the completion of driving. Engineers commonly refer to an increase in side shear as "setup" or "freeze", and a decrease as "relaxation". Fortunately, relaxation is rarely observed in Florida.

1.1 Pile Setup

Previous research work by the University of Florida (UF) for the Florida Department of Transportation (FDOT), described both in Bullock (1999) and in McVay, Schmertmann, Townsend, & Bullock (1999), investigated side shear setup over as long as three years for bridge piles in Florida. This research sought to develop criteria that would allow geotechnical engineers to include capacity-time effects in the design process, thereby reducing foundation costs. During the UF setup study, the side shear capacity of five, 457 mm, prestressed, concrete piles, driven in a wide range of soil types varying from sand to clay, was measured repeatedly over time. The research conclusions recommended a conservative design pile side shear "setup factor" of 0.20, equivalent to a 20% increase in side shear per log cycle of time relative to the side shear measured (or estimated) 1 day after driving. This conservative factor was in lieu of field testing and included a caveat due to the repeated ("staged") testing of the research piles. (Chapter 2 further describes the derivation of the setup factor.)

1.2 Staged Versus Unstaged Testing

For both practical and economic reasons, the UF research described above used "staged" tests, defined as tests repeated on the same pile at various times after the end of driving (EOD). This is a common and accepted engineering practice, especially for dynamic tests using multiple set checks or redrives. "Unstaged" load tests, or tests performed only once at varying times on separate piles, require many additional piles, and consequently incur greater costs due to increased time, materials, and construction and testing effort. The UF staged tests typically obtained a side shear failure after an axial movement of only 0.1”-0.2", and left the piles unloaded between tests (supporting only their own weight). Although researchers rarely address the effects of staged testing on the measured capacity, a few have indicated that repeated test movements remold and, after drainage, strengthen the adjacent soil. Because of its potentially unconservative impact, the UF setup study recommended additional staged testing research prior to use of the recommended 0.20 setup factor.

1.3 Investigating the Effect of Staged Testing

McVay, Schmertmann, Townsend, & Bullock (1999) also included insitu tests using the cone penetrometer (CPT), the Marchetti Dilatometer (DMT), and the standard penetration test with torque measurement (SPT-T). These tests provided rigorous site characterization and correlation with the observed pile setup behavior at two of the five test pile locations. Both the CPT and the SPT-T measured side shear directly, the CPT in the axial direction and the SPT-T in lateral torsion. Both tests indicated setup behavior, but the SPT-T was viewed as a more practical test. Therefore, to avoid the impractical alternative of repeating the full test program with unstaged test piles, the study recommended using the SPT-T to investigate staged testing effects at one of the other three test pile locations. This effort could also provide additional setup correlation.

1.4 Scope of Work

The research work described herein focuses specifically, and intently, on obtaining SPT-T results adjacent to the Seabreeze test pile driven during the previous UF study near the east abutment of the westbound side of the Seabreeze Bridge in Daytona Beach, Florida. The FDOT provided funds for this work to the UF Civil and Coastal Engineering Department and appointed Mr. Peter Lai as Project Manager. The testing includes twelve SPT borings located with the test pile as a common center. Three of the borings provide staged torque measurements (SPT-T) at geometrically increasing times of 5, 30, 180, and 1080 minutes after the EOD in two soil layers: a silty, fine sand and a shelly clay. The other nine borings supply unstaged tests at the same times. This report summarizes and compares the results, both to determine staged testing effects and to incorporate them as an adjustment to the previous UF setup study.

2. LITERATURE REVIEW

2.1 Pile Side Shear Setup

Pile side shear setup denotes an increase in pile capacity over time, with documented increases to as much as 3 years in both cohesionless and cohesive soils (see UF study by Bullock, 1999, or by McVay, Schmertmann, Townsend, & Bullock, 1999). Many researchers have observed this behavior, but often only as a footnote to other research. In most cases, the observed setup followed an arithmetic linear trend with the logarithm of time.

Skov and Denver (1988) recommend that engineers should consider long-term pile capacity during foundation design, extending beyond estimates obtained from initial driving which are affected by soil remolding and unstable pore-water pressures. They contend that driving resistance provides a reasonable estimate of long-term capacity only for coarse sands and attribute setup increases to the equalization of pore-water pressure (reconsolidation) and the reestablishment of internal bonds in the soil.
Skov and Denver (1988) proposed that the observed time-dependent increase in pile capacity follows a linear trend with the logarithm of the ratio t/t_0, where t is the time elapsed since the end of driving and t_o is a reference time at which increase in capacity begins. They further proposed the use of a dimensionless setup factor, A, representing the semilog-linear slope of this capacity increase as follows:

$$\mathbf{A = \frac{Q/Q_0 - 1}{\log_{10}(t/t_0)}}$$

and	A	=	Pile setup factor, dimensionless (semilog-linear slope)
	Q	=	Pile capacity (force) at time t
	Q_0	=	Pile capacity (force) at time t_0
	t	=	Time elapsed since end of driving
	t_0	=	Initial reference time elapsed since end of driving

Skov and Denver's investigation includes four case histories documenting total pile capacities, but does not separate side shear from tip bearing. They consider both A and t_o correlated to material type and noted that pile capacities from redrives prior to t_0 did not follow the observed semilog-linear trend. Skov and Denver reported A = 0.2 for a sand profile with t_o = 0.5 days, and A = 0.6 for a clay profile with t_o = 1 day. Since the available literature does not currently support tip bearing setup, the use of total capacity (including end bearing) to determine the setup factor, A, may lead to an erroneous, lesser value. The reference time t_0 also affects the value of A by changing the reference capacity, Q_0. To further standardize the setup factor, Bullock (1999) proposed using t_0 = 1 day and limiting the setup factor to reflect the change in side shear (stress or force) only:

$$\mathbf{A = \frac{\tau / \tau_0 - 1}{\log_{10}(t / t_0)} = \frac{\tau A_s / \tau_0 A_s - 1}{\log_{10}(t / t_0)} = \frac{Q_s / Q_{s0} - 1}{\log_{10}(t / t_0)}}$$

and	A	=	Side shear setup factor, dimensionless, semilog-linear slope
	τ, τ_0	=	Pile side shear capacity (stress) at time t or t_0
	Q_s, Q_{s0}	=	Pile side shear capacity (force) at time t or t_0
	A_s	=	Pile side area
	t	=	Time elapsed since end of driving
	t_0	=	Reference time elapsed since end of driving = 1 day

Figure 2.1, from Bullock (1999), shows the semilog-linear trend in side shear capacity measured during the pile research in Florida. Using only the side shear, the UF researchers found whole-pile setup factors of 0.10-0.40 for the variable soil types investigated. Analysis of the embedded strain gages included in the UF tests also provided side shear estimates for specific pile segments (generally < 3m in length). As shown in **Figure 2.2**, the UF study measured a maximum segment setup factor A = 1.60 and most of the segments exceeded A = 0.20.

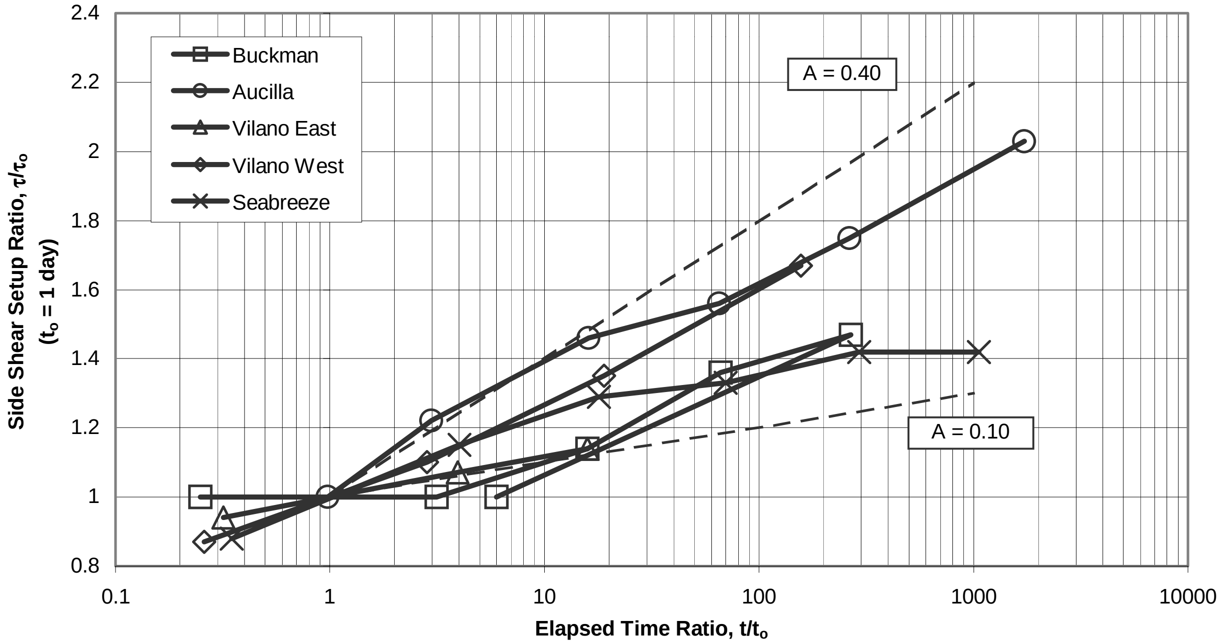


Figure 2.1 Side Shear Setup for Entire Pile (Bullock, 1999)

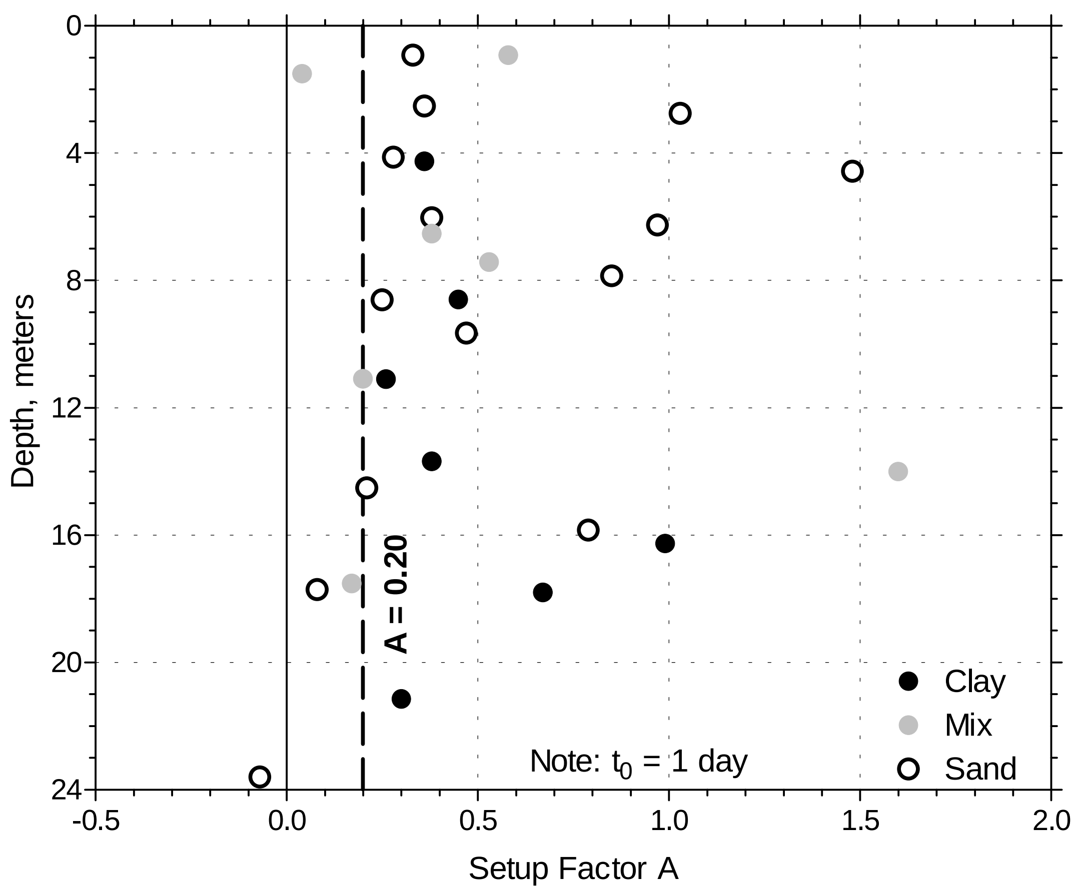


Figure 2.2 Depth Profile of Pile Segment Setup Factors (Bullock, 1999)

Figure 2.2 also shows very little correlation of the A setup factor with either depth or soil type. Contrary to previous explanations based on consolidation drainage, the pile segments monitored during the UF study showed continued setup in both sands and clays long after the dissipation of excess pore pressures and the stabilization of effective stresses.

Although engineers observe setup relatively often, its cause is poorly defined at present. Many, including Soderberg (1962) and Vesic (1977), have hypothesized that radial consolidation of cohesive soils displaced during pile installation increases both the strength and lateral stress around the pile. Schmertmann (1991) proposed that aging might contribute to setup by soil structure changes that increased dilatency and stiffness during "drained dispersion". Chow et al. (1996) suggested that the penetration of the pile through non-cohesive soils might create an unstable ring of soil around the pile, with the ring temporarily supported by an arching effect. Collapse of this arch due to stress relaxation (creep) would subsequently cause an increase in horizontal effective stress on the pile.

Axelsson (1998) performed a study of setup using 235 mm (9.25") square, concrete piles, and 32 mm (1.26") diameter, steel rods driven into a non-cohesive soil. Axelsson described possible setup mechanisms for these piles as shown in **Table 2.1**

Table 2.1 Possible Setup Mechanisms for Rods and Piles in Sand. (Axelsson, 1998)

	Steel Rods	Concrete Piles
Size (Width or Radius)	An increase in dilatency and stiffness (soil aging) has a greater effect on a small rod	Large soil disturbance during installation - strong arching effect and stress relaxation
Surface Roughness	Weak interlocking between soil particles and rod	Strong interlocking between soil particles and pile, leading to large dilation effects during loading
Expected Degree of Setup	Medium	High

Axelsson (1998) estimated pile capacity from dynamic tests on the steel rods and assessed pile capacity using only dynamic tests. Although successful overall, the analysis of a dynamic test requires subtraction of the dynamic capacity component to obtain the static capacity and, therefore, is less definitive than a static test. Dynamic test analyses also typically provide a poorly defined and non-unique estimate of side shear distribution along the pile. Axelsson concluded that the measured setup depended on depth (stress) as well, in direct contradiction to the findings of the UF setup study presented in **Figure 2.2** above, and possibly a result of the dynamic test analysis

Reported values of the setup factor A range from 0.2 to 0.6 (Skov and Denver, 1988), 0.25 to 0.75 (Chow et al., 1996), 0.2 to 0.8 (Axelsson, 1998), and -0.07 to 1.60 (Bullock, 1999). Konrad and Roy (1987) found pile capacity in over-consolidated soft sensitive marine clay to reach 12 times the initial capacity over a period of 25 days. The maximum capacity was reached after excess pore-water pressure had fully dissipated. Bartolomey and Yushkov (1985) found increases in side shear capacity of 80% for a four-pile group and 70% for a nine-pile group after 45 days. Again, the maximum measured value of shear capacity was reached after pore-water pressures had fully dissipated.

2.1.1 Staged Testing

Repeated pile capacity tests are usually staged on the same pile due to economic necessity. Conversely, investigators rarely stage lab tests because of possible effects on the results and the relative simplicity of preparing/obtaining and testing additional samples. Fleming (1952) reported results from staged, undrained, triaxial compression tests in the lab. He staged the tests by stopping the load application at failure, then immediately increasing the lateral confining pressure and continuing the test undrained to another failure point. The Mohr-Coulomb failure envelope constructed from three successive points obtained in this fashion yielded c and ϕ values similar to an envelope of normal tests performed on separate samples. Thus, Fleming did not measure any staging effects when the sample was not allowed to consolidate to the new lateral pressure, but did state that such results might be limited to soils of moderate cohesion. Successive pile tests typically allow time for at least partial consolidation between tests, and therefore staged pile test behavior may not coincide with Fleming's observations.

Kenney and Watson (1961) used Fleming's approach also. They concluded that, for undrained tests, staged testing had little or no effect regardless of the soil's mineral composition or sensitivity. Kenny and Watson also reported no effect on drained strength tests for soils with a stable structure (low sensitivity).

Karlsrud and Haugen (1985) performed laboratory shear tests on both undisturbed and remolded specimens of the more sensitive Haga clay (Sensitivity, S_t = 4.5). They found that the remolded clay had greater drained strength at the same confining pressure. They also field tested small-diameter (153 mm) steel pipe piles jacked into the same Haga clay. These piles exhibited a significant stage testing effect, with 22% greater side shear from the staged tests than unstaged tests at 40 days after the EOD. **Figure 2.3** shows a distinctly greater, semilog-linear rate of capacity increase for the staged tests compared with unstaged tests (data scaled from plot in Karlsrud and Haugen, 1985). The unstaged pile tests included tests in both tension and compression, without discernible difference in measured side shear capacity. The relative stiffness of these piles compared with the clay soil may have minimized this difference. The test data presented in **Figure 2.3** would be more definitive if presented in stress units to eliminate the effect of any differences in penetration length.

The research by Karlsrud and Haugen (1985) led the UF researchers to add a caveat to their staged pile test results, which recommended further investigation of staged testing effects. Of course, to eliminate the uncertain effect of staged testing on the pile setup factor, engineers clearly should perform only unstaged tests. However, because of the significant additional costs of unstaged tests on full-scale piles, a method of estimating the staging effect (either from lab or insitu testing) would prove highly useful. The UF setup study proposed using torque tests on the SPT sampler (SPT-T) for this purpose, and the FDOT provided additional funds for a series of tests at one of the previous pile sites, the results of which are described herein.

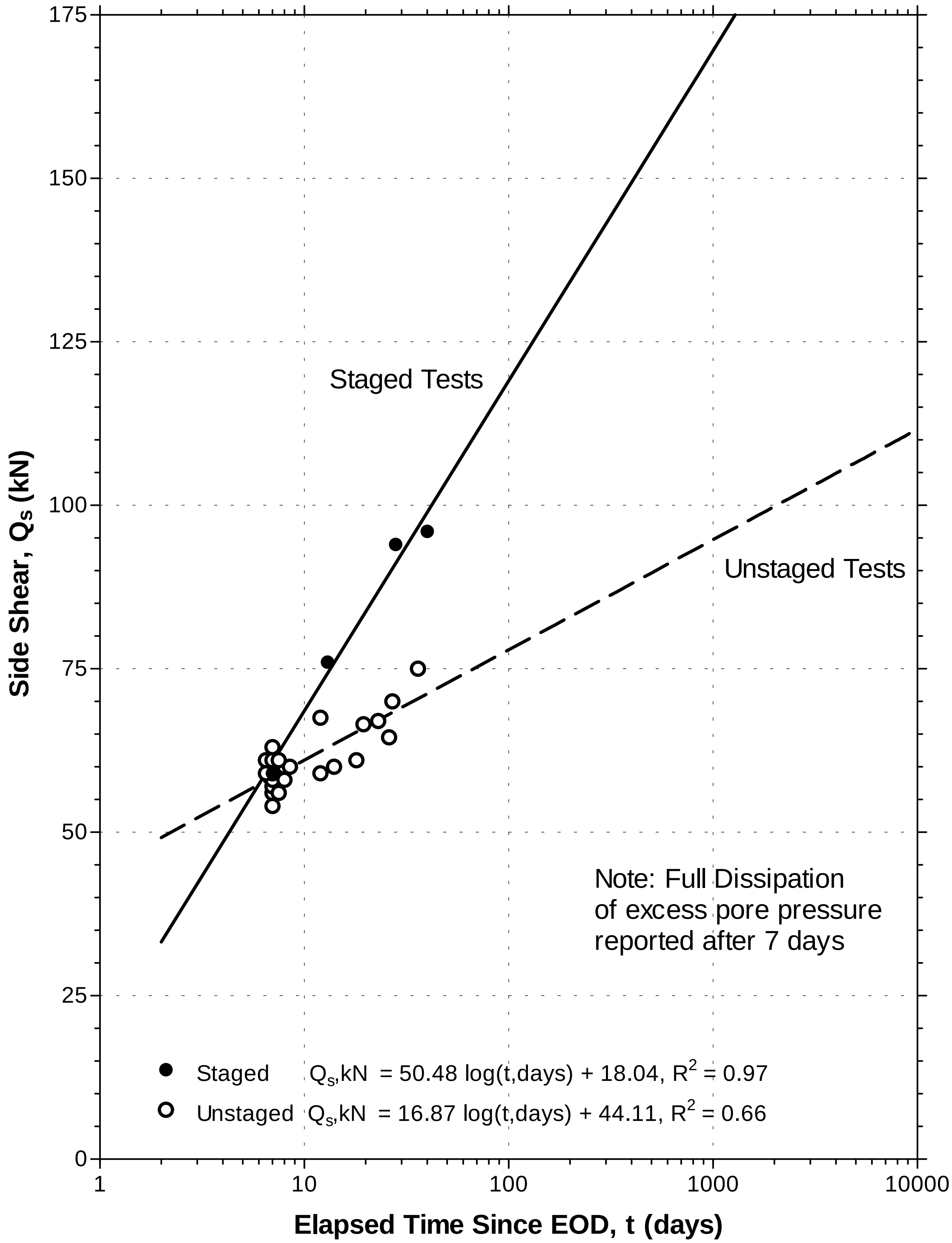


Figure 2.3 Time Effects on Test Pile Capacity in Haga Clay
(data from Karlsrud and Haugen, 1985)

2.2 Standard Penetration Test Using Torque (SPT-T)

2.2.1 Torsional Shear Measurement

Although Ranzine (1988) initially proposed adding torque measurements at the end of the Standard Penetration Test, DeCourt and Filho (1994) actually reported the first such test results. Since the SPT-T measurement occurs after the driving of the sampler, it adds to, and does not interfere with, the traditional SPT. Following the normal 457 mm (18") penetration of the SPT sampler, the SPT hammer is removed and the sampler is turned in place by applying a torsional force at the top of the drill rod string. The driller/operator may measure the applied torque using a calibrated torque wrench, load cell, or gauged section of drill rod. A rotation of 180 degrees is normally adequate to reach the peak value of adhesion, which typically occurs within the first 5^o to 10^o according to Rausche et al. (1996).

The static SPT-T compliments the dynamic SPT measurement of soil strength (bearing plus side shear). Decourt and Filho (1994) recommended the use of a torque ratio, (T/N). For sands, Schmertmann (1979) reported that end bearing at the sampler bottom controls the N blowcount. Hence the T/N ratio should behave in a fashion similar to the cone penetration test (CPT) friction ratio, high for cohesive soils and low for sands.

Lutenegger and Kelley (1998) hypothesize that, although the SPT soil sample is highly disturbed (due to a high area ratio and dynamic penetration), the torsional shear strength measured outside of the sampler during the SPT-T occurs in a partially remolded soil that retains much of its original fabric. In addition, compared with the traditional SPT, test precision should improve significantly due to less operator and equipment variability. Of course, parasitic drill rod friction above the sampler and loss of sampler-soil contact due to wobbling of the sampler or rods during driving may also affect the torque measurement.

2.2.2 Quality Control Using SPT

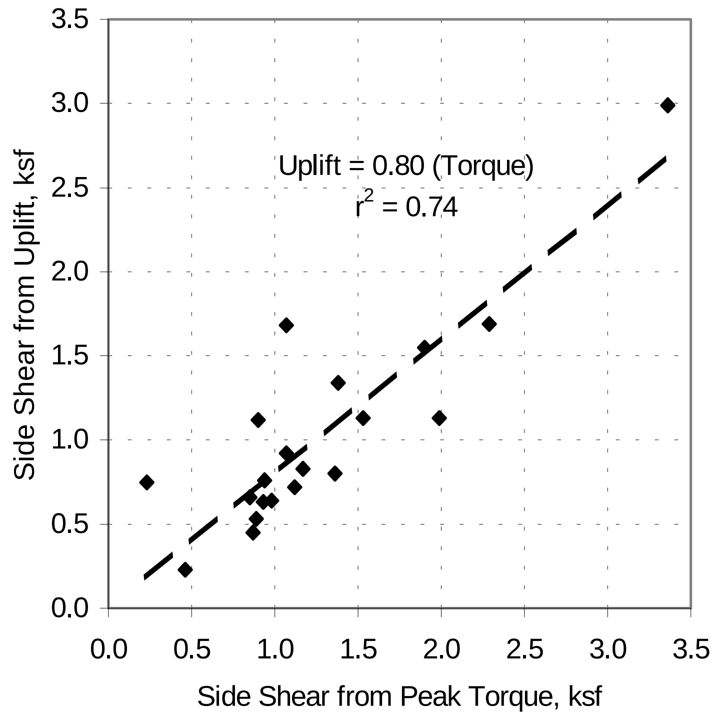
According to Decourt and Filho (1994) the torque ratio (T/N) tends to remain constant for a given soil and given SPT equipment, which makes it an effective tool for quality control. Any deviation from a constant value could indicate undesirable influences on the N value due to operator or equipment problems. The presence of gravel, saprolytic rock, or large shells may also yield inconsistent torque ratios. These larger particles may block the sampler opening and increase the blowcount, leading the engineer to unconservatively estimate greater soil strength or density. However, depending on the particle size, the torsional strength should not change greatly, resulting in a lower torque ratio and more accurate strength estimate.

2.2.3 Site Characterization Using SPT-T

Engineers often depend on the SPT for geotechnical site evaluation because of its ability to penetrate and test a wide variety soils. For instance, the SPT is more likely to obtain usable tests results in a weathered rock profile including soil transition zones, and parent rock than other more sophisticated insitu tests (i.e., Cone Penetration Test, Dilatometer, Pressuremeter, etc.). However, the SPT may provide artificially high N-values in residual and talus soils if pieces of rock prevent soil entry into the sampler, thus masking the presence of softer materials. By adding a 5-minute torque test at the end of the SPT, and without affecting the SPT results, the engineer can obtain additional qualitative and quantitative soil information.

2.2.4 Evaluating Pile Setup Using SPT-T

Rausche et al. (1996) investigated the use of a modified SPT procedure for prediction of the soil damping and quake parameters needed to analyze dynamic pile tests. Their work included uplift (tension) tests after the SPT sampler was driven, followed immediately thereafter by a torque test (SPT-T). Using data from Rausche et al. (1996), **Figure 2.4** shows the peak uplift resistance plotted versus peak torque resistance. Based on their data, the peak uplift is about 80% of the peak torque. Although at present it is unclear whether SPT torque or SPT uplift correlate best with pile side shear, Raushe et al. (1996) concluded that, similar to pile side shear behavior, a semilog-linear relationship existed between side shear on the SPT sampler and elapsed time (see **Figure 2.5**). This observation, the obvious model relationship between the SPT and driven piles, and the work by Bullock (1999) provide credence to the use of the SPT-T for the prediction of pile setup.



Peak Torque	Peak Uplift
1.07	0.92
3.36	2.99
0.23	0.75
1.99	1.13
2.29	1.69
1.17	0.83
0.89	0.53
1.90	1.55
1.36	0.80
1.07	1.68
1.53	1.13
1.38	1.34
0.94	0.76
0.46	0.23
0.85	0.66
0.98	0.64
0.87	0.45
0.90	1.12
1.12	0.72
0.93	0.63

Figure 2.4 SPT Uplift versus Torque Resistance from Rausche et al., 1996

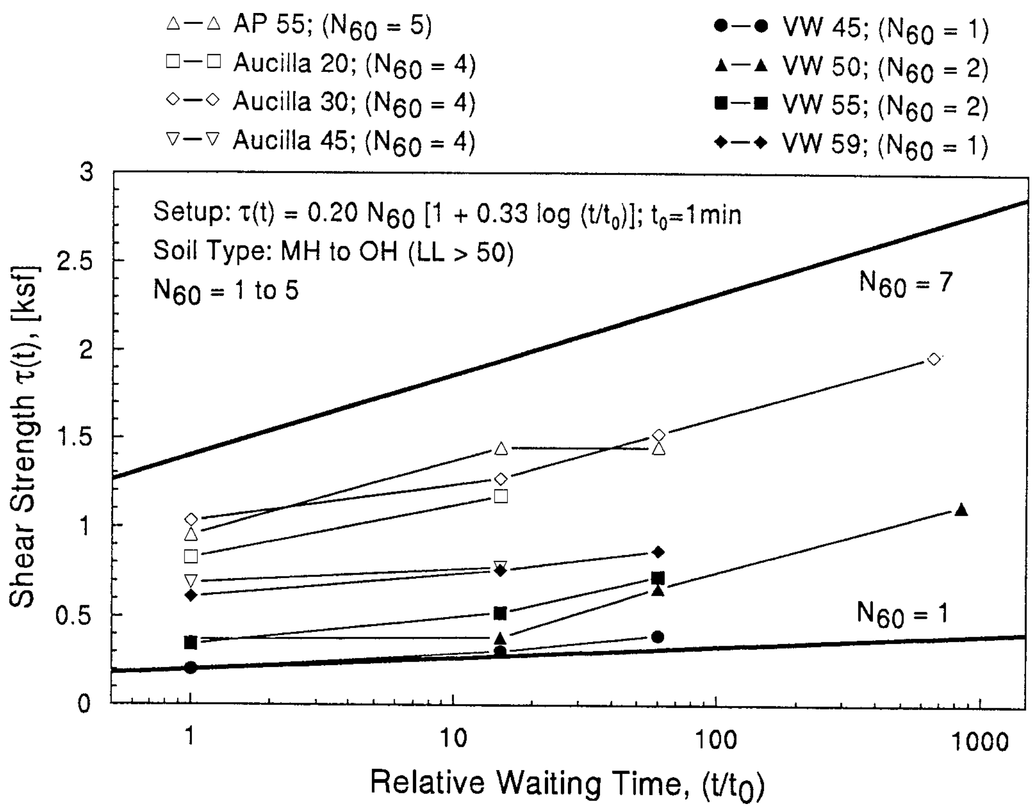


Figure 2.5 SPT Uplift Side Shear vs. Log of Elapsed Time (Rausche et al., 1996)

3. FIELD TESTS

3.1 Site Characteristics

The test site is located near the east abutment of the westbound side of the Seabreeze Bridge in Daytona Beach, FL where a test pile used by Bullock (1999) was driven. **Figure 3.1** shows the site as viewed from the westbound bridge.

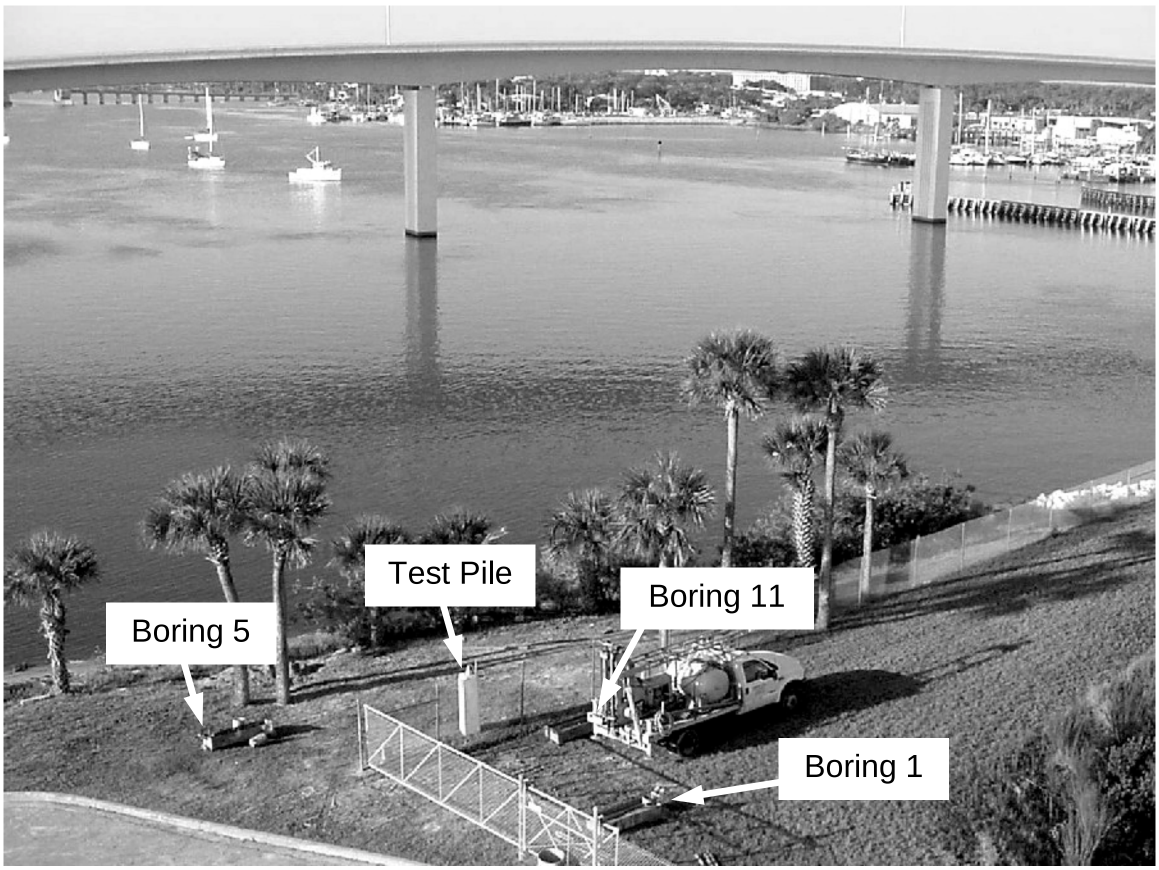


Figure 3.1 Seabreeze Bridge Test Site

The demobilized drill rig is located inside the retention pond fence and is facing west. The 457 mm (18") square concrete test pile can be seen between the drill rig and the far end of the gate on the south side of the retention pond fence about 1.8 m (6') out of the ground. The site is level north of the test pile and slopes down at about a 5% grade to the south. The fence, the grade, and a few of the palm trees created some site access problems for the drill rig. The mud pans seen in **Figure 3.1** are located at boreholes 1, 5, and 11, all of which required an overnight setup period. **Figure 3.3** provides a detailed location plan of the borings.

3.2 Standard Penetration Test

The SPT-T boreholes were spaced 3.3 m (10') apart and were often left open overnight during the SPT-T. Amdrill, Inc. from Orlando, Florida performed the SPT portion of the testing in general accordance with ASTM D 1586-99 using a truck-mounted CME55 drill rig. They drilled the boreholes with a tricone roller bit, AWJ drill rods, and Bentonite drilling mud. To minimize drilling disturbance, support the borehole, and prevent parasitic soil-rod friction during the torque test, Amdrill also installed 89 mm (3.5") diameter steel casing down to approximately 0.3 m (1') above the test depth. The driving energy was visually estimated as approximately 50% of theoretical, based on previous experience with this type of measurement. The split barrel sampler used was 870 mm (34.3") long and 51mm (2") in diameter. To optimize the testing schedule and keep the drill rig busy during the long setup periods, Amdrill provided enough drill rods, samplers, mud pans, and casings to perform tests in three boreholes simultaneously.

SPT torque tests were performed adjacent to the center of only two of the test pile segments, in a silty, fine sand layer at approximately elevation -7.75 m (-25.4') and in a deeper shelly clay layer at approximately -15.75 m (-51.7'). Soil samples were collected from each test depth in all 12 of the borings and stored in 1-gallon Ziploc bags for transportation to the UF laboratory. UF personnel measured the sample recovery (**Figure 3.2**) and recorded it along with the SPT blowcounts on a field log.

The measured torque on the drill rods divided by the sampler radius and soil contact area yields the mobilized torsional shear stress. For tests in the upper sand layer, the contact area was assumed to be the driven sampler penetration of 457 mm (18") times the sampler circumference. The sample recovery length in the upper sand layer averaged 254 mm (10"), or about 56% of the driven penetration. However, the sample recovery length in the lower clay layer averaged about 584 mm (23"), exceeding the apparent length driven. The clay sample recovery in excess of 457 mm probably resulted from unintended penetration of the sampler prior to driving, possibly due to weight of the rod string and hammer. Therefore, for the clay layer, the sample recovery length was assumed more representative of the actual contact area, and it was used in place of the driven penetration to calculate the contact area.

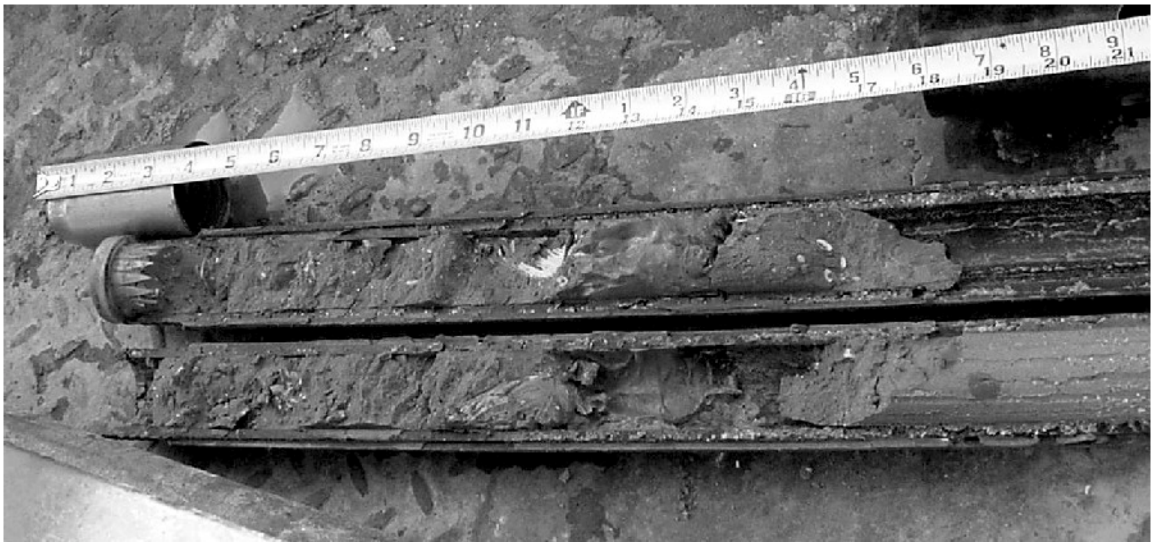


Figure 3.2 Measurement of Sample Recovery.

3.3 Borehole Locations

Figure 3.3 shows a plan view of the triangular borehole pattern used around the Seabreeze test pile during this project. The borehole numbers were designated in clockwise order and do not indicate the sequence of the borings. The different symbols indicate the staging of the tests performed in that borehole. Staged tests locations included torque measurements at approximately 5, 30, 180, and 1080 minutes following the driving of the sampler, while unstaged test locations only included a single measurement at the time specified. Each type of test was performed in three boreholes to account for spatial variability, and each set of three similar boreholes was arranged with the test pile at its geometric center. On several occasions, there was a significant, incidental delay in sampler removal following an unstaged test, and an additional staged test was performed to obtain information about the effect of different staging times on the setup.

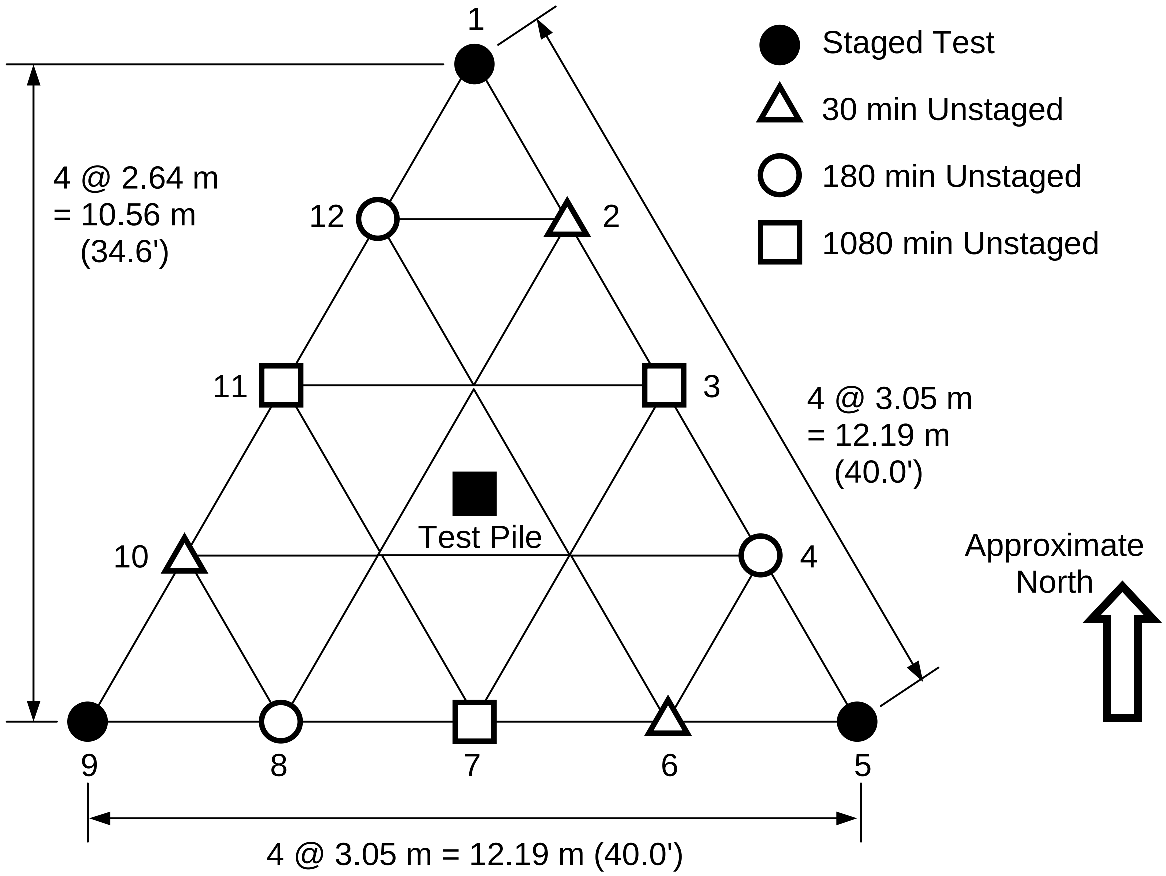


Figure 3.3 SPT-T Borehole Plan View

The 3.3 m (10') lateral spacing between borings was chosen as a compromise between reducing possible soil variability and potential disturbance effects due to stress relief, soil removal, and ground vibrations from adjacent borings. As mentioned above, the advancement of three borings at once helped to compact the testing schedule. A detailed test schedule was developed in advance to complete the work in five days, but required adjustment to six days because of unplanned delays. The testing sequence was planned so that adjacent boreholes were not open simultaneously, further increasing the minimum separation between open boreholes to 6.6 m (20'). Field adjustment of the borehole schedule resulted in one case of open adjacent boreholes, borings 7 and 8, but no cross-hole effects were observed.

3.4 Site Variability

A comparison of the SPT blowcounts and soil sample properties for the two test layers provides a measure of the lateral variability of the SPT-T data at the Seabreeze site. UF graduate student Michael Hicks performed lab tests on the Seabreeze SPT samples at the UF geotechnical laboratory and provided the results included in **Appendix B**. The upper layer, at approximately elevation -7.6 m (-24.6'), consisted of blue-gray silty fine sand. The SPT blowcount at this elevation averaged 12.0 blows per 0.3 m (1'). Sieve analyses from the 12 samples yielded the relatively uniform gradation with the average % passing values shown in **Table 3.1** for this non-plastic, silty sand.

Table 3.1 Silty Sand Layer Variability

Parameter	# of Samples	Average	Standard Deviation	Coefficient of Variation
Elevations (m)	12	-7.6	0.1	-1.5%
N spt	12	12.0	1.6	13.3%
% Passing #20	12	99.9	0.1	0.1%
% Passing #50	12	98.3	1.8	1.8%
%Passing #100	12	72.2	7.7	10.6%
%Passing #200	12	5.5	2.1	37.5%

The lower test layer, at approximately elevation -15.7 m (-51.7'), is composed of blue-gray shelly clay with relatively low plasticity as shown in **Table 3.2**. The SPT blowcount at this elevation averaged 5.6 blows per 0.30 m (1'). SPT soil samples from the three staged test borings yielded an average of 9.5% primarily shell retained on the #200 sieve, of which 75.3% passed the #4 sieve, 46.0% passed the #10 sieve, 18.4% passed the #20 sieve, and 7.7% passed the #40 sieve

Table 3.2 Shelly Clay Layer Variability

Parameter	# of Samples	Average	Standard Deviation	Coefficient of Variation
Elevations (m)	12	-15.7	0.1	-0.7%
N spt	12	5.6	1.4	24.7%
Water content (%)	12	38.2	3.4	8.9%
Liquid Limit	3	39.7	0.8	2.0%
Plastic Limit	3	25.2	1.9	7.7%
Plasticity Index	3	14.5	2.2	15.2%
%Retained #200	3	9.5	3.0	31.9%

Water content, Atterberg limit, and gradation test results for the shelly clay are presented in **Tables B.2 to B.7** in **Appendix B**. Water contents for the 12 samples yielded an average value of 38.2%, very close to the average 39.7% liquid limit. With an average plastic limit of 25.2%, the average plasticity index of 14.5% falls on the "A-line", indicating a CL-ML classification. Results from cone penetration tests by Bullock (1999) indicate a mixture of silty clays and clayey silts at this test elevation. Previous FDOT SPT results (see Bullock, 1999) indicate sandy clay. This soil is referred to herein as shelly clay.

Individual test results are included in **Appendix B**. Overall, the lateral variability between the SPT-T borings near the Seabreeze test pile was minimal and should not significantly affect the SPT-T results.

3.5 Torque Measurement

3.5.1 Torque Cell

The same torque cell used by Bullock (1999) was used for the work described herein. Purchased from Pile Dynamics, Inc., it consisted of an instrumented 2 ft length of AW rod, which the operator threaded onto to the top of the drill rod string. (An AW/AWJ rod sub connected the torque cell to the AWJ drill rods used by Amdrill.) The torque cell contained two complete Wheatstone bridges, one high and one low, separately wired with foil-type strain gage rosettes glued on opposite sides of the rod section. Each rosette included two strain gages, which were arranged at right angles and wired together with the opposing rosette to form a full bridge. **Figure 3.4** shows one bridge of the 2-channel torque cell, along with the drive nut and the sliding T-handle socket wrench used to apply the torque. The sliding handle allowed the operator to keep the rod string centralized in the borehole and work around obstacles while applying torque.

Although any commercial strain meter can be used with this torque cell, a datalogger was used during this research to obtain a time record of the applied torque. UF personnel calibrated the torque cell in the lab just prior to field use by applying a static torque and recording the bridge output with the datalogger. **Appendix A** presents the linear calibration results (r^2 = 0.9999) for both channels. The datalogger program

"TORQ" written for the SPT-T includes the calibration factors, so that the program provides output directly in torque units (N-m).

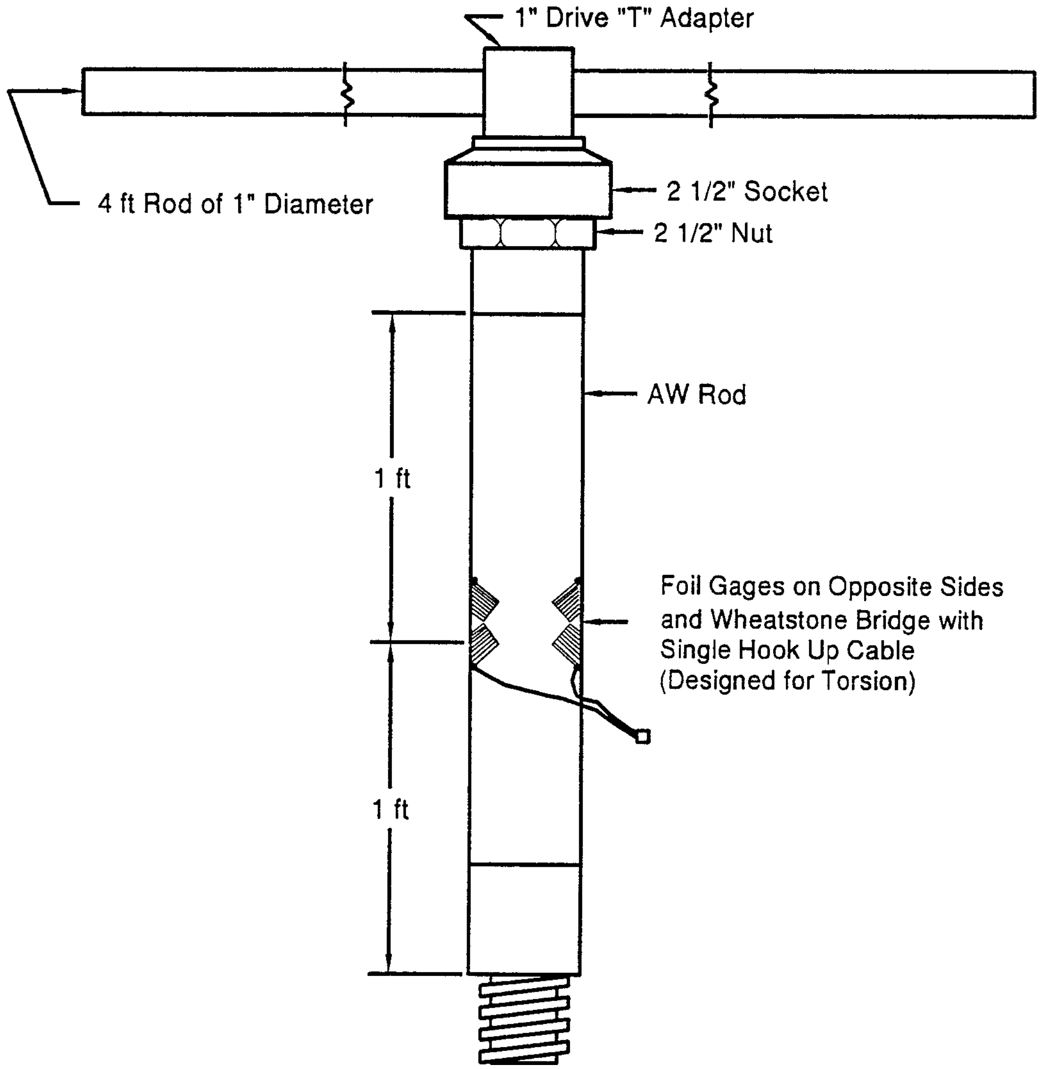


Figure 3.4 Torque Cell Schematic (Rausche et al.,1996)

3.5.2 Datalogger

UF provided a programmable datalogger manufactured by Campbell Scientific, Inc. to monitor the torque cell during this research. The model CR10 datalogger provides a ±2.5 volt excitation circuit with direct mv/v bridge output. However, to increase accuracy, a separate 10 volt power supply (uses two 9 volt transistor batteries) was mounted inside the datalogger enclosure and used for bridge excitation. The datalogger measured differential voltage readings for both the excitation circuit and the two bridge output circuits. The CR10 itself requires a low-amperage, 12 volt power supply, in this case provided by eight D-cell alkaline batteries.

3.5.3 Control Software

The PC208 software manufactured by Campbell Scientific, Inc. provided command control of the CR10 datalogger. This software was used to write a control program and download it to the datalogger. A laptop (or desktop) computer, "talking" to the datalogger through an RS232 serial connection, provided the operator with a real-time interface to monitor and control the datalogger during the SPT-T.

The PC208 software package consists of six separate programs: the "EDLOG" program editor, the "GT" terminal emulator, the "SPLIT" data splitter, the "TELCOM" telecommunications program, the "SMCOM" storage module communications program, and the "WAKETIME" program used to initiate datalogger functions. EDLOG and GT were the only programs required for this research. EDLOG allows the user to create and document programs for the CR10 datalogger. Bullock (1999) used EDLOG to develop the program "TORQ" specifically for data acquisition with the UF torque cell. GT provides computer/datalogger communication for real-time display, data collection, and downloading/uploading of datalogger programs.

An older, DOS-based, Compaq laptop computer (LTE Lite 4/33) was used during the Seabreeze research, but any computer with an RS232 port can fulfill this role. This laptop's internal battery failed during the Seabreeze tests, and it was subsequently powered from the support vehicle battery through a cigarette-lighter adapter cable.

3.6 Standard Penetration Test with Torque

The SPT torque test was performed after driving the sampler, often as soon as possible. Therefore, prior to driving, the torque cell operator normally powered up and prepared the datalogger and laptop for the torque measurement. After removal of the SPT hammer, the operator connected the torque cell to the rod string and hooked up the two

7.6 m (25') long, shielded, 4-conductor cables used to excite and monitor the strain bridge outputs during the test. Although not absolutely required, the driller moved the drill rig away from the borehole to provide additional room for the torque test. **Figure 3.5** shows the support vehicle and torque cell in position for testing. (The support vehicle was required only for the laptop power.)

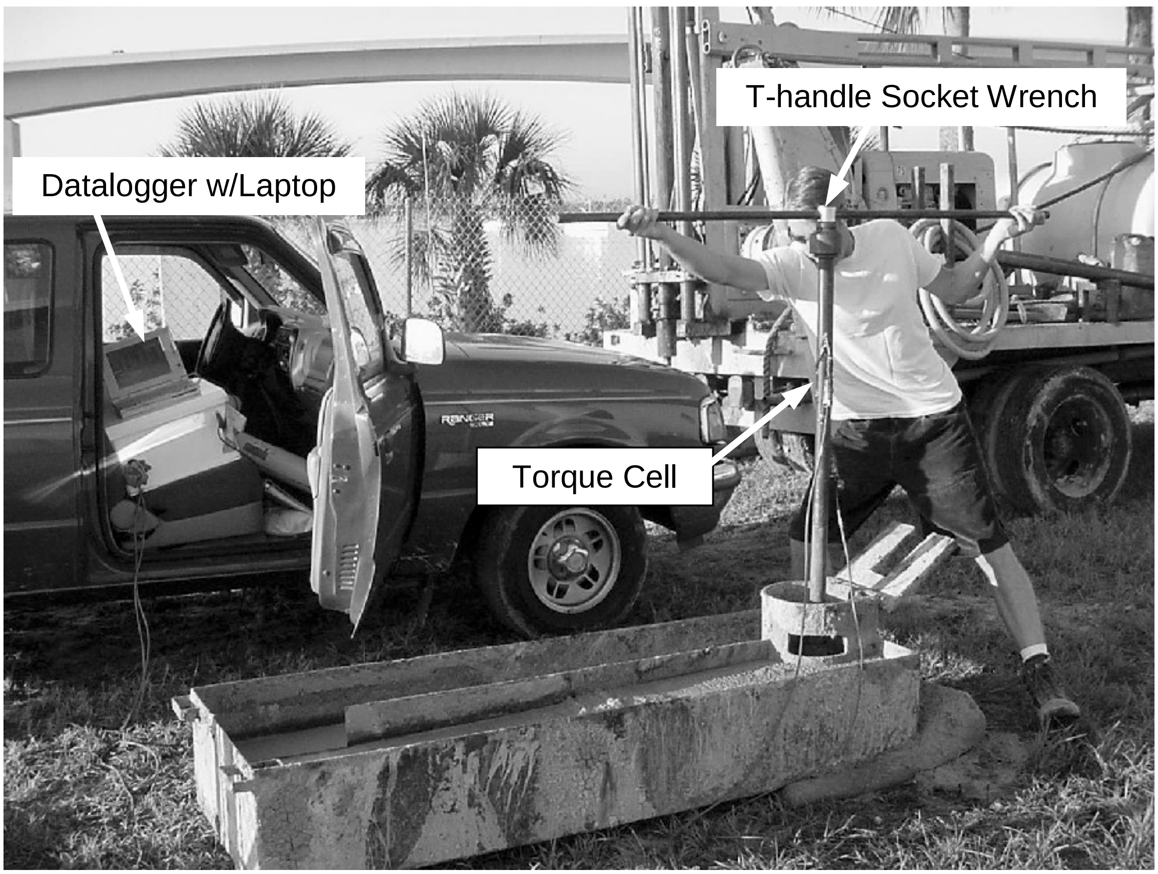


Figure 3.5 SPT-T Equipment

Just prior to the torque test, offset baseline readings were initiated by a laptop command to the datalogger (F2), and repeated until the electronic circuitry stabilized. The "TORQ" program automatically subtracted the final set of measured baselines from the load cell readings. Since these strain bridges did not include temperature compensation, the baselines varied slightly from test to test due to differences in temperature.

After obtaining the baselines, the operator "armed" the data acquisition by transmitting another computer command (F1) to the datalogger, and then activated it by applying torque to the load cell at the designated time of the test. The minimum time required to set up and activate the torque cell prior to testing was about 4 minutes. During the torque test, the operator manually turned the rods through an approximately steady rotation of about 180^{o} over an average time of about 12 seconds (actual time duration ranged 6 to 22 seconds). Each torque bridge was digitized at a rate of approximately 24 samples per second. Data acquisition stopped automatically when the measured torque fell below the threshold value set in "TORQ" (or when the F7 command was transmitted). A final baseline after the test provided confirmation of the initial offset, and the torque cell was then demobilized.

4. SPT-T DATA REDUCTION

4.1 Test Results

UF Professor Paul Bullock and graduate student Michael Hicks performed a total of 49 torque tests at the two target elevations in the 12 borings adjacent to the Seabreeze test pile over a six-day period starting June 18, 2001. The field log of the tests includes rod length, casing length, casing elevation (by survey level), test depth, SPT blowcounts, and the time of day at the end of driving. Although the initially planned five-day schedule was adjusted several times, the testing generally proceeded at an efficient pace. No test problems were encountered, and all of the boreholes remained stable without any indication of collapse or loss of circulation. Initial baselines varied little from final baselines during the Seabreeze tests and did not require adjustment. Loose rod joints interfered with a few of the tests, but did not appear to significantly affect the peak torque reading. A few of the 1080-minute tests exceeded 300 N-m of torque, which approaches the maximum capability of a single operator using the 2 m (6 ft) torque wrench handle.

After the completion of each test, the digitized SPT torque-time record was uploaded to the laptop computer. This datalogger record included a date/time stamp and the digitized torque measurements from each channel in units of N-m. After importing the data into an Excel spreadsheet, the average of the two torque channels was then plotted versus time. **Appendix C** presents plots for all 49 tests.

Table 4.1 summarizes the field data from the 12 borings at the Seabreeze site. The test elevations in **Table 4.1** refer to the elevation at the center of the N SPT value, i.e. at the start of the 0.30 to 0.45 m (12"-18') blowcount.

Table 4.1 Seabreeze SPT-T Results (Amdrill Inc. Drill Crew, 18 Jun 01 - 23 Jun 01, Safety Hammer AWJ Rods, Bentonite Drill Mud, Set BW Casing 30mm above test depth, Sampler Diameter =50.8mm

Boring	Start of Test		Grnd.	Test	Blowcounts			N_{SPT}	Soil Description	Sampler
	Date dd-mmm	Time hhmmss	Elev. m,msl	Elev. m,msl	0.00 - 0.15m	0.15 - 0.30m	0.30 - 0.45m	blows /0.30m		Penetr. mm
1	20-Jun	13:20:47	+2.24	-7.60	6	6	6	12	Bl Gr Si Fn Sand	457 457 457 457
1	22-Jun	15:00:17	+2.24	-15.75	2	3	3	6	Bl Gr Sh Si Clay	533 533 533 533
2	18-Jun	11:57:19	+2.20	-7.56	4	5	6	11	Bl Gr Si Fn Sand	457
2	18-Jun	15:00:26	+2.20	-15.79	2	2	3	5	Bl Gr Sh Si Clay	610 610
3	19-Jun	20:49:08	+2.20	-7.55	6	7	8	15	Bl Gr Si Fn Sand	457 457
3	20-Jun	21:17:29	+2.20	-15.63	2	3	4	7	Bl Gr Sh Si Clay	546
4	18-Jun	13:16:56	+2.11	-7.57	3	6	8	14	Bl Gr Si Fn Sand	457
4	18-Jun	18:39:50	+2.11	-15.87	2	2	3	5	Bl Gr Sh Si Clay	584 584
5	21-Jun	17:43:10	+1.92	-7.83	5	6	7	13	Bl Gr Si Fn Sand	457 457 457 457
5	22-Jun	16:35:03	+1.92	-15.53	2	3	3	6	Bl Gr Sh Si Clay	610 610 610 610
6	19-Jun	12:15:10	+1.84	-7.61	3	5	5	10	Bl Gr Si Fn Sand	457
6	19-Jun	14:46:45	+1.84	-15.69	3	3	2	5	Bl Gr Sh Si Clay	610
7	20-Jun	19:04:37	+1.72	-7.50	4	5	8	13	Bl Gr Si Fn Sand	457 457
7	21-Jun	19:04:37	+1.72	-15.65	2	2	4	6	Bl Gr Sh Si Clay	521
8	21-Jun	10:13:46	+1.68	-7.46	5	4	7	11	Bl Gr Si Fn Sand	457
8	21-Jun	15:02:28	+1.68	-15.54	2	4	3	7	Bl Gr Sh Si Clay	622 622
9	18-Jun	16:49:12	+1.73	-7.57	6	6	7	13	Bl Gr Si Fn Sand	457 457 457 457
9	19-Jun	17:58:52	+1.73	-15.80	2	2	4	6	Bl Gr Sh Si Clay	648 648 648 648
10	20-Jun	14:37:42	+1.92	-7.38	4	5	6	11	Bl Gr Si Fn Sand	457
10	20-Jun	16:25:15	+1.92	-15.76	1	1	1	2	Bl Gr Sh Si Clay	457
11	21-Jun	16:49:44	+2.07	-7.68	4	5	6	11	Bl Gr Si Fn Sand	457
11	22-Jun	14:54:00	+2.07	-15.84	2	3	4	7	Bl Gr Sh Si Clay	660
12	19-Jun	10:18:05	+2.18	-7.57	5	4	6	10	Bl Gr Si Fn Sand	457 457
12	20-Jun	20:10:58	+2.18	-15.65	2	2	3	5	Bl Gr Sh Si Clay	521 521

4.2 Shear Strength

Table 4.2 tabulates the peak torque measured during each test at the Seabreeze site. It also includes the average side shear acting on the sampler at failure calculated from the following equation (Bullock, 1999):

$$\tau = \frac{2T/1000}{\pi(ld^2 + d^3/3)}$$

where τ = average sampler side shear (kPa)
T = peak measured torque (N-m)
l = SPT sampler penetration (typically 0.457 m minimum)
d = outside diameter of SPT sampler (0.0508 m)

The d^3 term in the denominator of this equation is a minor correction for the torsional shear developed in the horizontal plane at the bottom of the sampler.

Figure 4.1 shows the peak side shear from each of the unstaged torque tests in the silty sand layer plotted against the log of the time elapsed since the EOD, and includes a best-fit "straight" line through the data using the log of the elapsed time and a non-linear least squares regression. All of the semilog-linear trend lines presented herein were determined using the statistical program "Prism" sold by GraphPad Software, Inc. **Figure 4.2** shows the peak side shear data for the staged tests in the silty sand layer, along with a similar best-fit line for each individual set of staged tests (same boring) and an overall best-fit through all of the staged tests. Both of these figures show very little "setup" effect, in agreement with the previous SPT-T results for sands reported by Bullock (1999). **Figure 4.3** presents a comparison of the unstaged and staged trend lines, which shows little effect of the staged testing on the peak side shear.

Figures 4.4-4.6 show plots for the shelly clay layer similar to those described above for the silty sand. Both the staged and unstaged tests in the shelly clay have greater side shear than the silty sand, and also show a significant increase with time. **Figure 4.4** includes an unstaged test at 30 minutes from Boring 10 with a low side shear value that does not fit well with the other data. This particular test also had a significantly lower SPT blowcount than the other borings ($N = 2$ vs. $N_{average} = 5.6$), possibly due to disturbance effects that may also explain the lower side shear. Despite the obvious difference, this test was included in the subsequent analyses with little effect on their results.

Note that both **Figures 4.2 and 4.5** include several staged tests performed at "inconsistent" times different from the scheduled staged tests. The slope of these inconsistent staged test data points, shown with dashed lines, is remarkably similar to that of the scheduled staged test series shown in **Figure 4.5**, even though their initial tests were performed much later than the 5 minute initial time of the scheduled staged tests. (**Figure 4.2** shows little staged setup and is inconclusive.) Despite this similarity, in order to maintain the integrity of the staged versus unstaged comparisons, these inconsistent staged tests are not included in any subsequent analyses of the Seabreeze SPT-T data. However, based on the limited data in **Figure 4.5**, it appears that the change in side shear with the logarithm of elapsed time for a staged test series may be independent of the time of its initial test.

Table 4.2 Seabreeze SPT-T Summary of Peak Side Shear

Boring	Test Elev. m,msl	Elapsed Time, t minutes	Peak Torque, T N-m	Peak Shear, τ_{pk} kPa
1	-7.60	5.35	53.46	28.32
		30.03	51.52	27.29
		180.05	54.43	28.83
		1080.10	64.31	34.07
1	-15.75	5.12	121.14	55.15
		30.12	207.04	94.26
		180.02	310.91	141.55
		1079.75	365.77	166.52
2	-7.56	30.00	67.11	35.55
2	-15.79	30.05	149.52	59.68
		979.12	242.44	96.77
3	-7.55	1062.47	108.06	57.25
		1356.87	101.24	53.63
3	-15.63	1067.15	164.00	72.95
4	-7.57	180.02	60.75	32.18
4	-15.87	171.18	177.27	73.79
		766.72	273.22	113.72
5	-7.83	5.00	79.20	41.96
		29.07	74.09	39.25
		182.82	69.66	36.90
		1082.77	73.71	39.05
5	-15.53	5.07	104.25	41.61
		30.00	174.64	69.70
		180.05	235.02	93.80
		1085.27	261.82	104.50
6	-7.61	29.95	47.05	24.93
6	-15.69	30.12	161.89	64.62
7	-7.50	1079.67	58.90	31.20
		1349.65	61.45	32.55
7	-15.65	980.18	198.31	92.45
8	-7.46	179.22	80.80	42.80
8	-15.54	179.27	158.42	61.96
		990.57	205.24	80.27
9	-7.57	5.33	54.66	28.96
		30.00	53.48	28.33
		180.05	63.30	33.53
		1152.12	55.87	29.60
9	-15.80	5.02	135.33	50.88
		29.65	197.51	74.26
		191.25	240.12	90.27
		1078.50	303.16	113.98
10	-7.38	29.92	70.19	37.18
10	-15.76	30.05	49.16	26.04
11	-7.68	1080.13	76.84	40.71
11	-15.84	1079.95	158.05	58.29
12	-7.57	180.13	62.77	33.25
		502.87	59.39	31.46
12	-15.65	168.20	161.63	75.35
		698.68	253.10	117.99

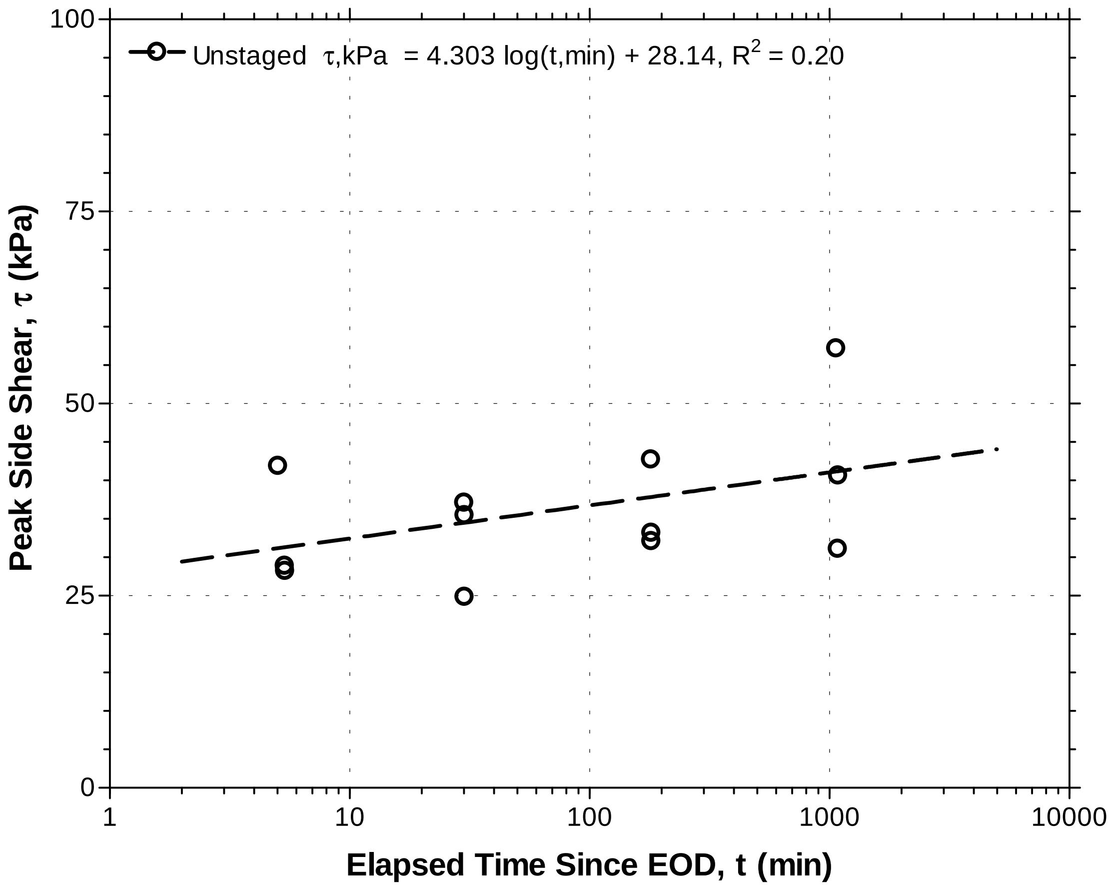


Figure 4.1 SPT-T Unstaged Side Shear, Seabreeze Silty Sand

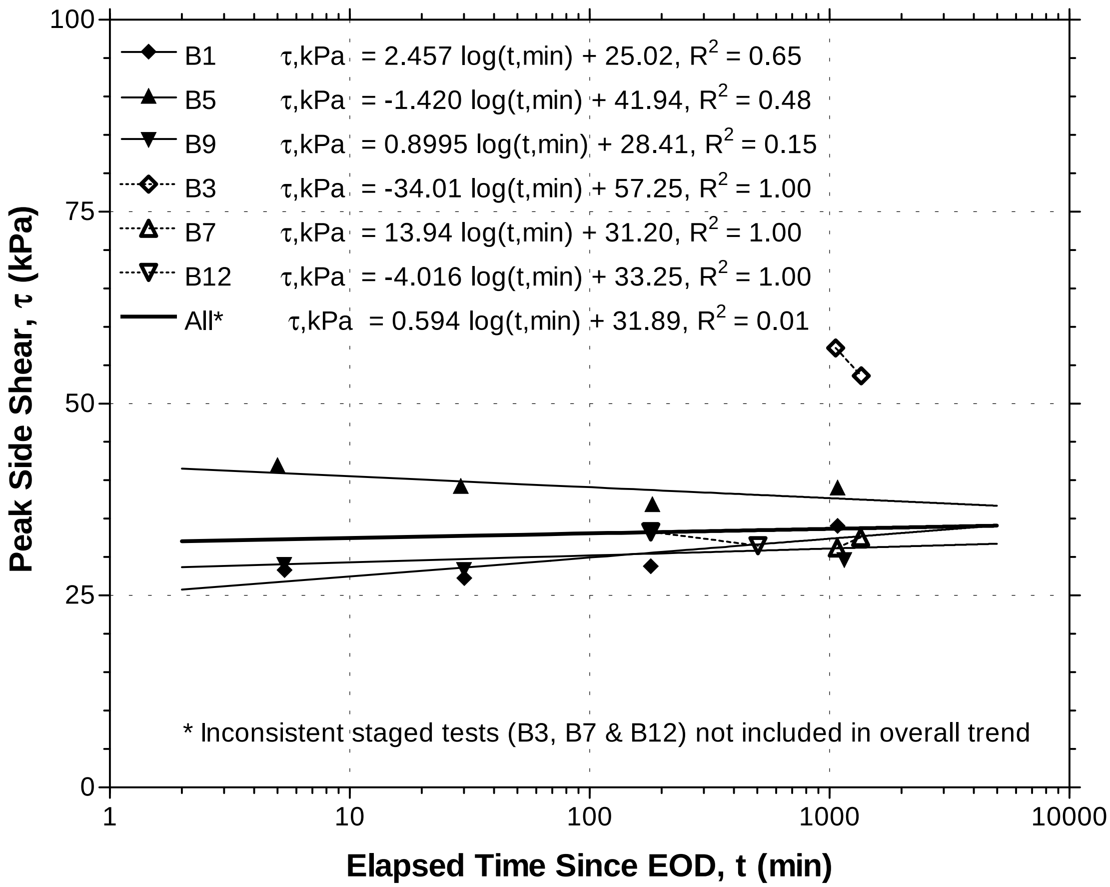


Figure 4.2 SPT-T Staged Side Shear, Seabreeze Silty Sand

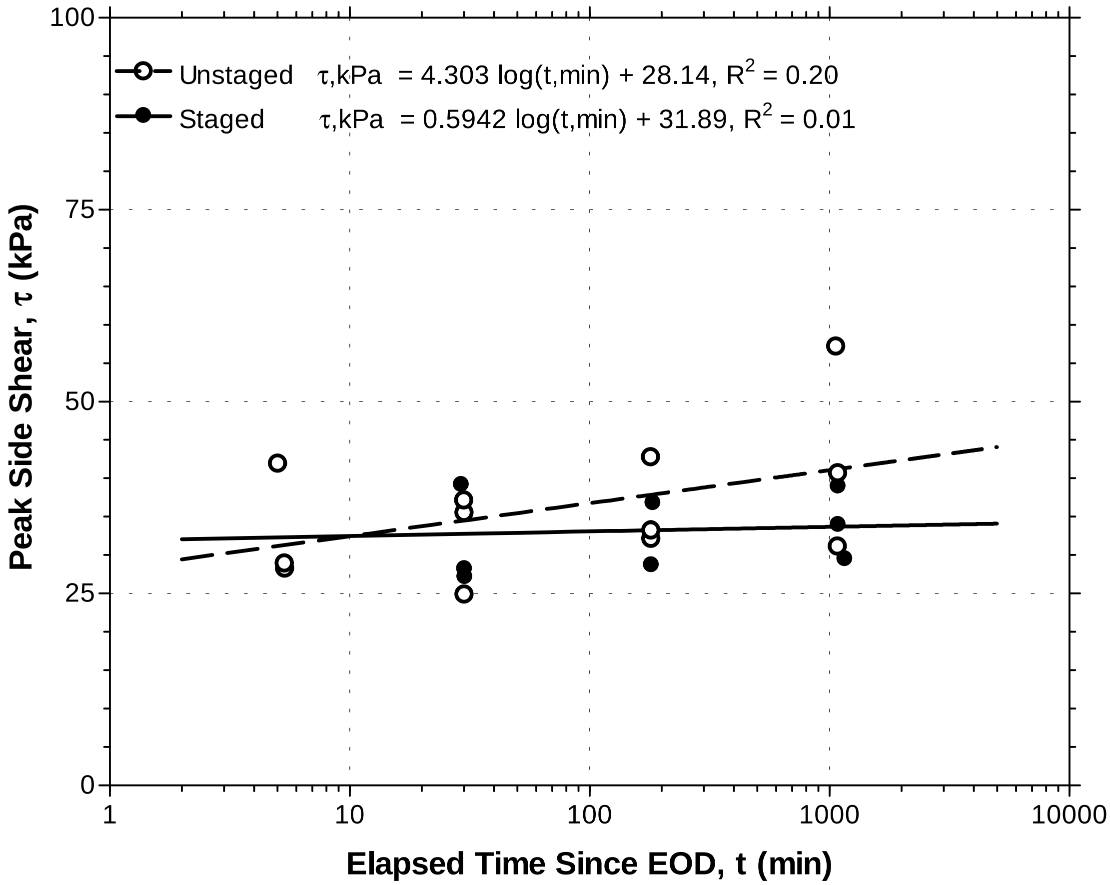


Figure 4.3 SPT-T Staging Comparison, Seabreeze Silty Sand

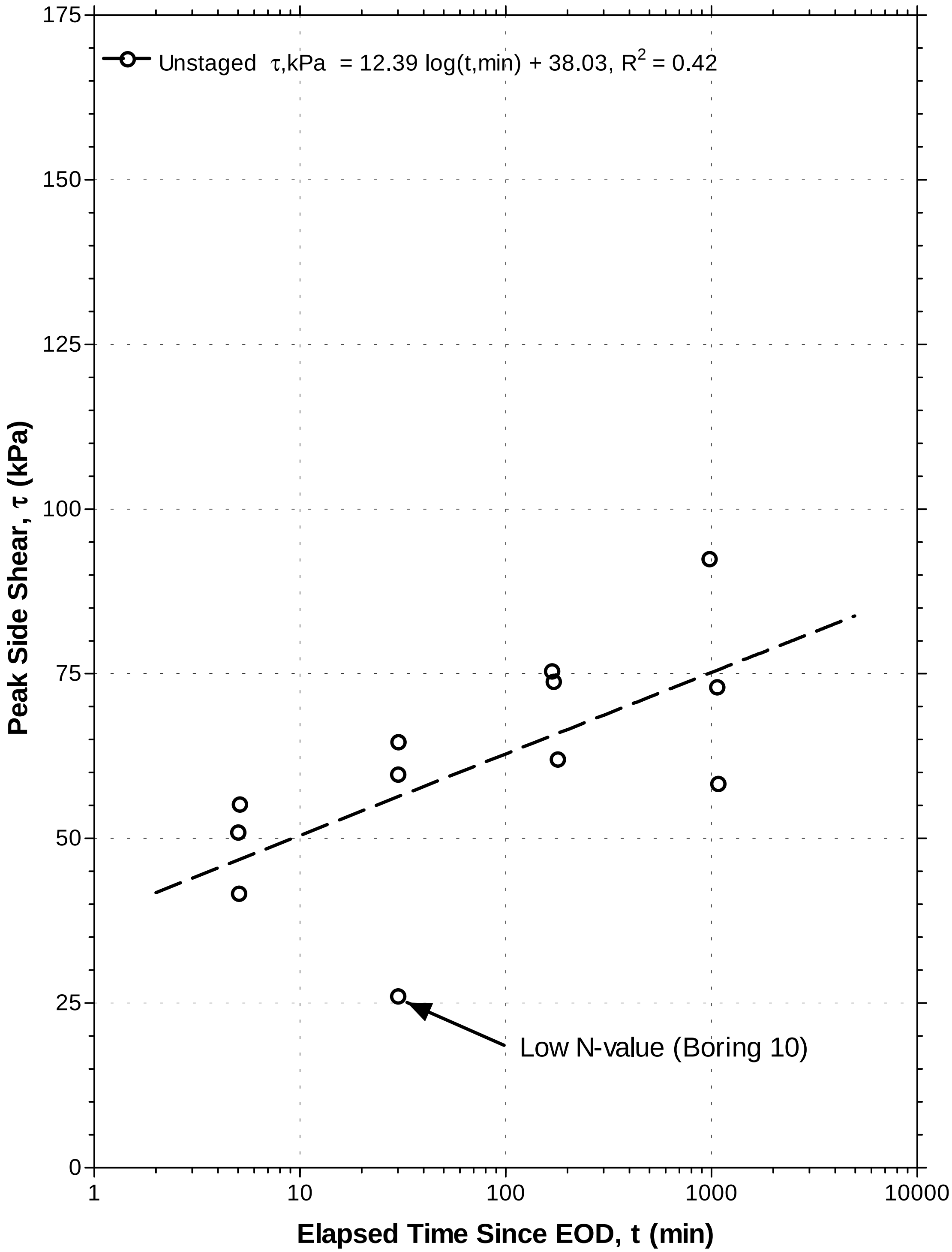


Figure 4.4 SPT-T Unstaged Side Shear, Seabreeze Shelly Clay

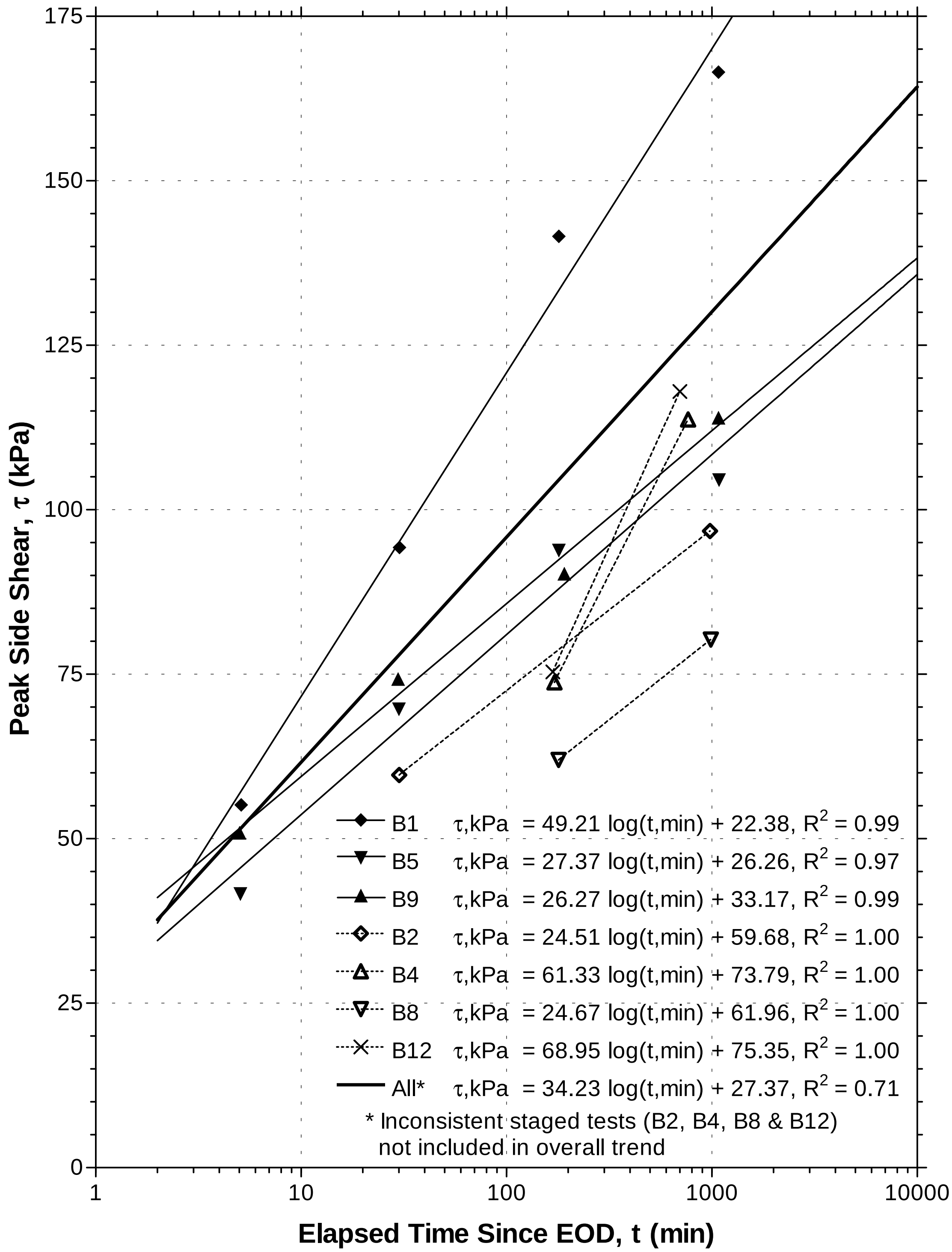


Figure 4.5 SPT-T Staged Side Shear, Seabreeze Shelly Clay

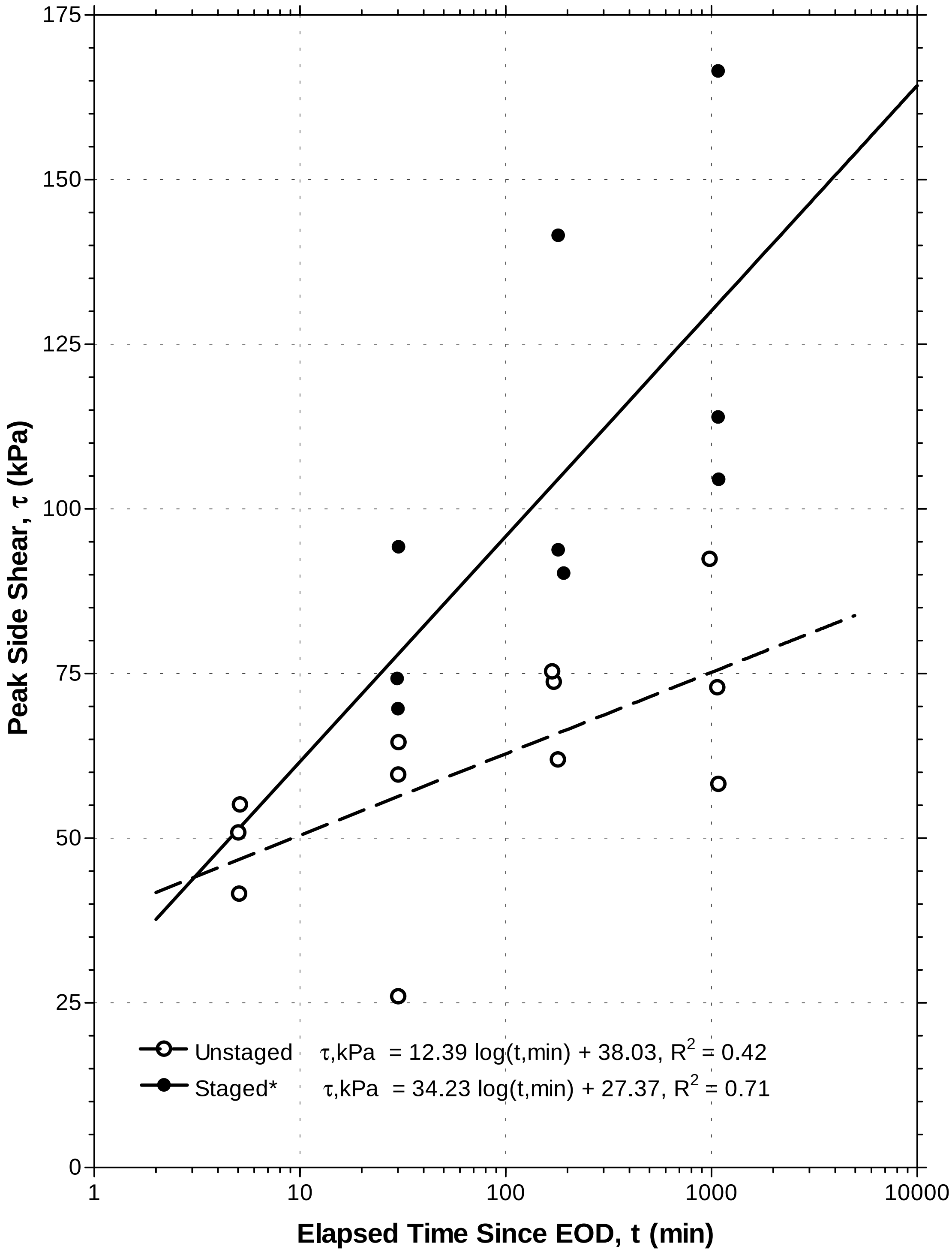


Figure 4.6 SPT-T Staging Comparison, Seabreeze Shelly Clay

5. SETUP FACTORS

5.1 SPT-T Peak Strength Setup Factors

The dimensionless setup factor A described in Section 2.1 is the relative increase in side shear from a reference value, τ_0, per log cycle of time elapsed relative to a corresponding reference time, t_0. Ideally, the reference time should correspond with the start of the semilog-linear increase in side shear, probably affected by factors such as soil type, pile type, penetration rate, etc. However, the difficulty of determining this initial time can be avoided by choosing a standard reference time, practical for testing and sometime after the start of setup effects for most soils.

For the analysis of pile side shear setup, Bullock (1999) chose a convenient reference time of t_{0pile} = 1 day = 1440 min. However, as a reference time for the analysis of the side shear from 14 staged SPT-T's, Bullock (1999) used the time of the first test in each series, or t_{0SPTT} = 3.5-5.1 min, subsequently standardized herein as t_{0SPTT} = 5 min. Although initially arbitrary, this choice provided relatively good agreement between the pile and SPT-T setup factors. The significant difference between the pile and SPT-T reference times for similar setup factors provides evidence of a size scale factor. If, as Vesic (1977) proposed, radial consolidation controls at least the initial stage of pile setup, then for t_{0pile} = 1440 min, by using the axisymmetric consolidation equation for equivalent percentage consolidation (and thus equivalent time factor, T) in the soil around the SPT sampler and the soil around the pile:

$$\mathbf{T_{SPTT} = T_{pile}}, \qquad \frac{\mathbf{c_h t_{0SPTT}}}{\mathbf{(r_{SPTT})^2}} = \frac{\mathbf{c_h t_{0pile}}}{\mathbf{(\bar{r}_{pile})^2}}, \qquad \text{and} \quad \frac{\mathbf{t_{0SPTT}}}{\mathbf{t_{0pile}}} = \left[\frac{\mathbf{r_{SPTT}}}{\mathbf{\bar{r}_{pile}}}\right]^2$$

and with $\mathbf{r_{SPTT}}$ = 25.4 mm and $\mathbf{\bar{r}_{pile}} = \sqrt{\frac{\mathbf{(457\,mm)^2}}{\pi}} = \mathbf{258\,mm}$ (equivalent radius)

then hypothetically $\mathbf{t_{0SPTT}} = \mathbf{1440\,min}\left[\frac{\mathbf{25.4\,mm}}{\mathbf{258\,mm}}\right]^2 = \mathbf{14\,min}$

Though slightly smaller (on a log time scale), the chosen value of t_{0SPTT} = 5 min agrees reasonably well with the above hypothetical value. Of course, Bullock (1999) also documented continued setup long after the completion of consolidation and the dissipation of excess pore pressures. The Seabreeze data provides additional SPT-T results to check the choice of t_{0SPTT} = 5 min.

Tables 5.1 and 5.2 present setup factors determined for the unstaged and staged peak side shear, respectively. For analysis of the unstaged SPT-T side shear, the best-fit log time trend of the side shear ($\tau = a \log_{10}t + b$), shown in **Figures 4.3 and 4.6**, was used to calculate τ_o at t_{0SPTT} = 5 min. The unstaged setup factors shown in **Figures 5.1 and 5.2** for the two different soil types tested at Seabreeze were then determined directly from the slope of the trend line, a, and $\tau_o = (a \log_{10}(t_0) + b)$:

$$\mathbf{A} = \frac{\tau / \tau_0 - 1}{\log_{10}(t/t_0)} = \frac{\tau - \tau_0}{\tau_0 \log_{10}(t/t_0)} = \frac{a\log_{10}(t) + b - (a\log_{10}(t_0) + b)}{\tau_0 \log_{10}(t/t_0)} = \left(\frac{a}{\tau_0}\right)$$

For the staged tests, the best-fit trend line for each series of staged tests was used to estimate τ_o for that series, and then to calculate individual setup factors for each series. The overall staged setup factors shown in **Figures 5.1 and 5.2**, however, were based on the overall trend of all the staged side shear data combined (see **Figures 4.3 and 4.6**), similar to the unstaged data. The unstaged setup factor shown in **Figure 5.1** for the silty sand is small, and the staged setup factor is nearly zero, probably due to disturbance effects. However, **Figure 5.2** for the shelly clay, shows a ratio of 2 in the setup factors of the staged and unstaged tests. Note that, because of the nearly identical test times and semilog-linear behavior, there is little difference between calculating the overall staged setup factors as described above, and fitting a semilog-linear curve through the combined relative side shear and time ratios determined for each series of staged test data.

Table 5.1 Seabreeze SPT-T Analysis of Unstaged Peak Side Shear

Boring	Test Elev. m,msl	$\tau = a \log_{10}(t) + b$			t/t_o	τ_o	τ/τ_o	A
		a kPa	b kPa	R^2	t_o,min = 5	(LS fit) kPa		
1	-7.60	4.303	28.14	0.199	1.07	31.15	0.909	0.138
1	-15.75	12.390	38.03	0.419	1.02	46.69	1.181	0.265
2	-7.56	4.303	28.140	0.199	6.00	31.15	1.141	0.138
2	-15.79	12.390	38.030	0.419	6.01	46.69	1.278	0.265
3	-7.55	4.303	28.140	0.199	212.49	31.15	1.838	0.138
3	-15.63	12.390	38.030	0.419	213.43	46.69	1.562	0.265
4	-7.57	4.303	28.140	0.199	36.00	31.15	1.033	0.138
4	-15.87	12.390	38.030	0.419	34.24	46.69	1.580	0.265
5	-7.83	4.303	28.140	0.199	1.00	31.15	1.347	0.138
5	-15.53	12.390	38.030	0.419	1.01	46.69	0.891	0.265
6	-7.61	4.303	28.140	0.199	5.99	31.15	0.800	0.138
6	-15.69	12.390	38.030	0.419	6.02	46.69	1.384	0.265
7	-7.50	4.303	28.140	0.199	215.93	31.15	1.002	0.138
7	-15.65	12.390	38.030	0.419	196.04	46.69	1.980	0.265
8	-7.46	4.303	28.140	0.199	35.84	31.15	1.374	0.138
8	-15.54	12.390	38.030	0.419	35.85	46.69	1.327	0.265
9	-7.57	4.303	28.140	0.199	1.07	31.15	0.930	0.138
9	-15.80	12.390	38.030	0.419	1.00	46.69	1.090	0.265
10	-7.38	4.303	28.140	0.199	5.98	31.15	1.194	0.138
10	-15.76	12.390	38.030	0.419	6.01	46.69	0.558	0.265
11	-7.68	4.303	28.140	0.199	216.03	31.15	1.307	0.138
11	-15.84	12.390	38.030	0.419	215.99	46.69	1.248	0.265
12	-7.57	4.303	28.140	0.199	36.03	31.15	1.068	0.138
12	-15.65	12.390	38.030	0.419	33.64	46.69	1.614	0.265

Table 5.2 Seabreeze SPT-T Analysis of Staged Peak Side Shear								
Boring	Test Elev. m,msl	$\tau = a \log_{10}(t) + b$ a kPa	b kPa	R^2	t/t_o t_o, min = 5.00	τ_o (LS fit) kPa	τ/τ_o	A
1	-7.60	2.457	25.02	0.649	1.07	26.74	1.059	0.092
		2.457	25.02	0.649	6.01	26.74	1.021	0.092
		2.457	25.02	0.649	36.01	26.74	1.078	0.092
		2.457	25.02	0.649	216.02	26.74	1.274	0.092
1	-15.75	49.210	22.38	0.987	1.02	56.78	0.971	0.867
		49.210	22.38	0.987	6.02	56.78	1.660	0.867
		49.210	22.38	0.987	36.00	56.78	2.493	0.867
		49.210	22.38	0.987	215.95	56.78	2.933	0.867
2	-7.56							
2	-15.79	24.513	59.68	1.000	6.01	76.81	0.777	0.319
		24.513	59.68	1.000	195.82	76.81	1.260	0.319
3	-7.55	-34.013	57.25	1.000	212.49	33.47	1.710	-1.016
		-34.013	57.25	1.000	271.37	33.47	1.602	-1.016
3	-15.63							
4	-7.57							
4	-15.87	61.332	73.79	1.000	34.24	116.66	0.633	0.526
		61.332	73.79	1.000	153.34	116.66	0.975	0.526
5	-7.83	-1.420	41.94	0.476	1.00	40.95	1.025	-0.035
		-1.420	41.94	0.476	5.81	40.95	0.959	-0.035
		-1.420	41.94	0.476	36.56	40.95	0.901	-0.035
		-1.420	41.94	0.476	216.55	40.95	0.954	-0.035
5	-15.53	27.370	26.26	0.965	1.01	45.39	0.917	0.603
		27.370	26.26	0.965	6.00	45.39	1.536	0.603
		27.370	26.26	0.965	36.01	45.39	2.067	0.603
		27.370	26.26	0.965	217.05	45.39	2.302	0.603
6	-7.61							
6	-15.69							
7	-7.50	13.936	31.20	1.000	215.93	40.94	0.762	0.340
		13.936	31.20	1.000	269.93	40.94	0.795	0.340
7	-15.65							
8	-7.46							
8	-15.54	24.665	61.96	1.000	35.85	79.20	0.782	0.311
		24.665	61.96	1.000	198.11	79.20	1.014	0.311
9	-7.57	0.900	28.41	0.149	1.07	29.04	0.997	0.031
		0.900	28.41	0.149	6.00	29.04	0.976	0.031
		0.900	28.41	0.149	36.01	29.04	1.155	0.031
		0.900	28.41	0.149	230.42	29.04	1.019	0.031
9	-15.80	26.270	33.17	0.993	1.00	51.53	0.987	0.510
		26.270	33.17	0.993	5.93	51.53	1.441	0.510
		26.270	33.17	0.993	38.25	51.53	1.752	0.510
		26.270	33.17	0.993	215.70	51.53	2.212	0.510
10	-7.38							
10	-15.76							
11	-7.68							
11	-15.84							
12	-7.57	-4.016	33.25	1.000	36.03	30.45	1.092	-0.132
		-4.016	33.25	1.000	100.57	30.45	1.033	-0.132
12	-15.65	68.950	75.35	1.000	33.64	123.54	0.610	0.558
		68.950	75.35	1.000	139.74	123.54	0.955	0.558

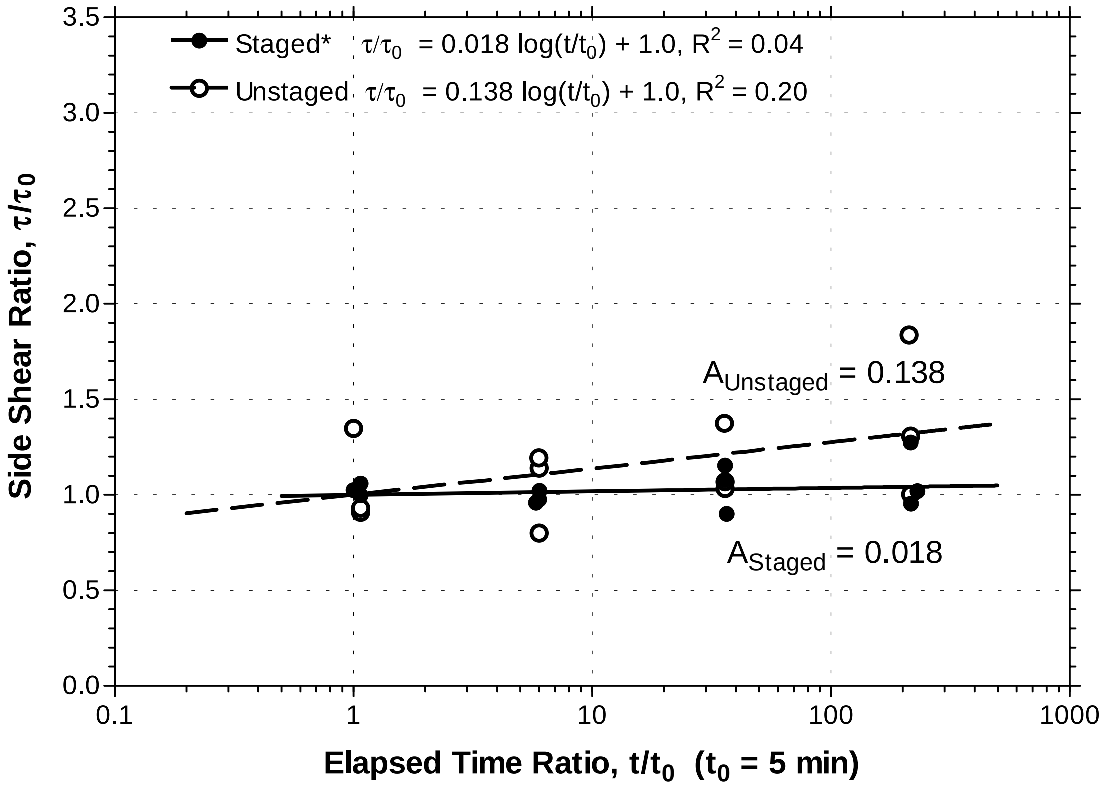


Figure 5.1 SPT-T Setup Factor, Seabreeze Silty Sand

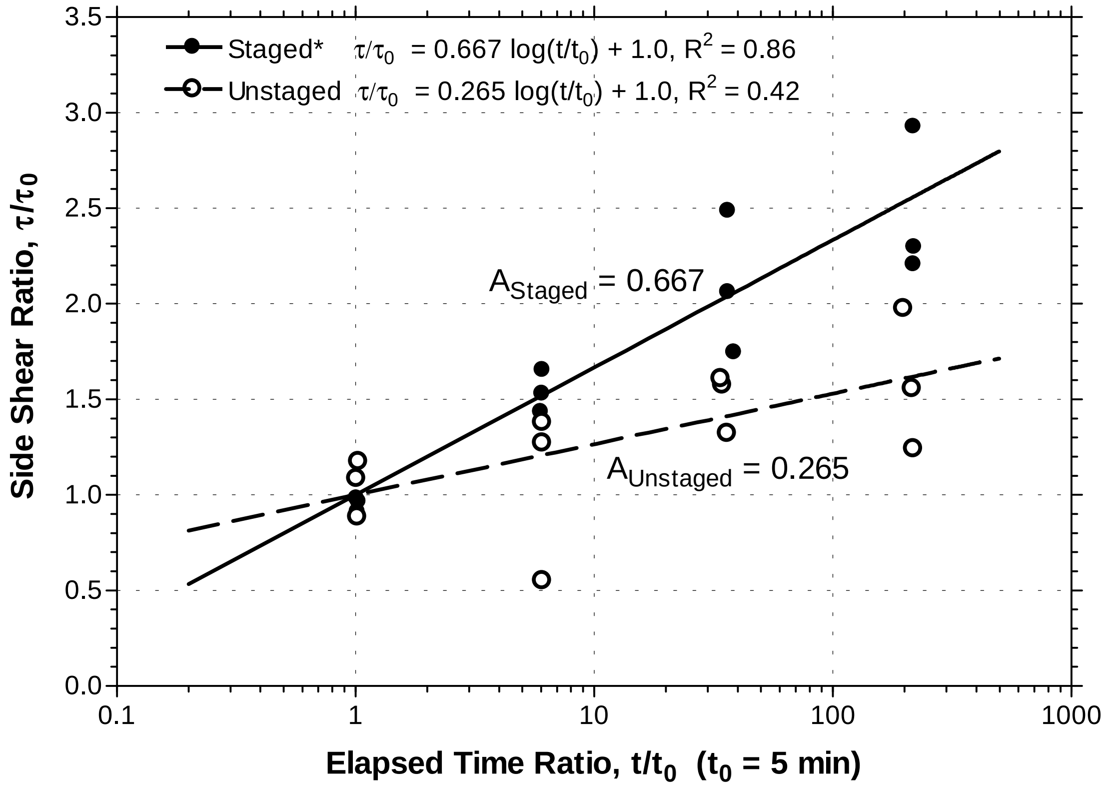


Figure 5.2 SPT-T Setup Factor, Seabreeze Shelly Clay

5.2 Karlsrud and Haugen Setup Factors

In addition to the SPT-T results obtained during this study, comparative setup factors for staged versus unstaged tests can also be calculated from the pile capacity data shown in **Figure 2.3** (from Karlsrud and Haugen, 1985)**. Table 5.3** tabulates the pile side shear force, Q_s, from **Figure 2.3** and calculates the setup factors using a reference time of t_0 = 7 days, the time of the initial tests in both series and also the complete dissipation of excess pore pressure. The resulting ratio of the unstaged to staged setup factors is 0.347 (= 0.289 / 0.832). (Because of test and site variability, the two semilog-linear trend lines actually intersect between 6 and 7 days.)

Since the chosen reference time, t_0, affects the setup factor, **Table 5.4** recalculates setup factors for the data in **Table 5.3** using a t_0 = 1 day, consistent with Bullock (1999). Extrapolating the trend of the unstaged tests backward to a reference time of 1 day provides a reference shear Q_{s0} = 44.11 kN with A = 0.382 for the unstaged tests. This trend line for the unstaged tests will always contain the initial test for any series of staged tests, and therefore, Q_{s0} = 44.11 kN at t_0 = 1 day for the staged tests also. However, the actual staged field tests do not begin at 1 day and the side shear must be increased to add the increment that would have occurred between 1 and 7 days. Based on the limited data in **Figure 4.5**, which shows that the semilog-linear slope of staged SPT-T side shear is not affected by the time of the initial test, it is reasonable to use the same staged test slope (a = 50.48 kN/day) from **Table 5.3** for a staged test series that beginning at 1 day. To provide this same slope, the staged test side shear data in **Table 5.4** are adjusted by adding 26.07 kN so that the reference side shear extrapolated to t_0 = 1 day agrees with the unstaged tests. The resulting ratio of the unstaged to staged setup factors using t_0 = 1 day is (0.382 / 1.144) = 0.334, similar to the previous calculated ratio of 0.347. **Figure 5.3** shows the setup factors determined in the manner described for t_0 = 1 day.

The analyses described above to calculate the ratio of unstaged to staged setup factors are equivalent and have similar results. The small difference between the above results is due to the test and site variability, which result in slightly different Q_{s0} values for the staged and unstaged tests in **Table 5.3**. In both cases the staged and unstaged tests share common reference times and initial shear values as a starting point, and therefore:

$$\frac{\mathbf{A_{unstaged}}}{\mathbf{A_{staged}}} = \frac{\mathbf{a_{unstaged}} \,/\, \mathbf{Q_{s0unstaged}}}{\mathbf{a_{staged}} \,/\, \mathbf{Q_{s0staged}}} = \frac{\mathbf{a_{unstaged}}}{\mathbf{a_{staged}}} \qquad (\textbf{if } \mathbf{Q_{s0unstaged}} = \mathbf{Q_{s0staged}})$$

Table 5.3 Karlsrud and Haugen Analysis of Side Shear, t_0 = 7 days

Test Series	Elapsed Time, t days	Pile Side Shear, Q_s kN	$Q_s = a \log_{10}(t) + b$ a kN	b kN	R^2	t/t_o t_o,day = 7	Q_{s0} (LS fit) kPa	Q_s/Q_{s0}	A
Staged	7.0	59.0	50.48	18.04	0.97	1.00	60.70	0.972	0.832
	13.0	76.0	50.48	18.04	0.97	1.86	60.70	1.252	0.832
	28.0	94.0	50.48	18.04	0.97	4.00	60.70	1.549	0.832
	40.0	96.0	50.48	18.04	0.97	5.71	60.70	1.582	0.832
Unstaged	7.0	54.0	16.87	44.11	0.66	1.00	58.37	0.925	0.289
	7.0	56.0	16.87	44.11	0.66	1.00	58.37	0.959	0.289
	7.0	57.0	16.87	44.11	0.66	1.00	58.37	0.977	0.289
	7.0	58.0	16.87	44.11	0.66	1.00	58.37	0.994	0.289
	7.0	59.0	16.87	44.11	0.66	1.00	58.37	1.011	0.289
	7.5	56.0	16.87	44.11	0.66	1.07	58.37	0.959	0.289
	6.5	59.0	16.87	44.11	0.66	0.93	58.37	1.011	0.289
	8.0	60.0	16.87	44.11	0.66	1.14	58.37	1.028	0.289
	8.5	60.0	16.87	44.11	0.66	1.21	58.37	1.028	0.289
	6.5	61.0	16.87	44.11	0.66	0.93	58.37	1.045	0.289
	7.0	61.0	16.87	44.11	0.66	1.00	58.37	1.045	0.289
	7.5	61.0	16.87	44.11	0.66	1.07	58.37	1.045	0.289
	7.0	63.0	16.87	44.11	0.66	1.00	58.37	1.079	0.289
	12.0	59.0	16.87	44.11	0.66	1.71	58.37	1.011	0.289
	12.0	67.5	16.87	44.11	0.66	1.71	58.37	1.156	0.289
	18.0	61.0	16.87	44.11	0.66	2.57	58.37	1.045	0.289
	19.5	66.5	16.87	44.11	0.66	2.79	58.37	1.139	0.289
	23.0	67.0	16.87	44.11	0.66	3.29	58.37	1.148	0.289
	26.0	64.5	16.87	44.11	0.66	3.71	58.37	1.105	0.289
	27.0	70.0	16.87	44.11	0.66	3.86	58.37	1.199	0.289
	36.0	75.0	16.87	44.11	0.66	5.14	58.37	1.285	0.289
	14.0	60.0	16.87	44.11	0.66	2.00	58.37	1.028	0.289
	8.0	58.0	16.87	44.11	0.66	1.14	58.37	0.994	0.289

Table 5.4 Analysis of Karlsrud and Haugen Side Shear, t_0 = 1 day									
Test Series	Elapsed Time, t days	Pile Side Shear, Q_s kN	$Q_s = a \log_{10}(t) + b$ a kN	b kN	R^2	t/t_o t_o,day = 1	Q_{s0} (LS fit) kPa	Q_s/Q_{s0}	A
Staged Adjusted	7.0	85.1	50.48	44.11	0.97	7.00	44.11	1.929	1.144
	13.0	102.1	50.48	44.11	0.97	13.00	44.11	2.314	1.144
	28.0	120.1	50.48	44.11	0.97	28.00	44.11	2.722	1.144
	40.0	122.1	50.48	44.11	0.97	40.00	44.11	2.767	1.144
Unstaged	7.0	54.0	16.87	44.11	0.66	7.00	44.11	1.224	0.382
	7.0	56.0	16.87	44.11	0.66	7.00	44.11	1.270	0.382
	7.0	57.0	16.87	44.11	0.66	7.00	44.11	1.292	0.382
	7.0	58.0	16.87	44.11	0.66	7.00	44.11	1.315	0.382
	7.0	59.0	16.87	44.11	0.66	7.00	44.11	1.338	0.382
	7.5	56.0	16.87	44.11	0.66	7.50	44.11	1.270	0.382
	6.5	59.0	16.87	44.11	0.66	6.50	44.11	1.338	0.382
	8.0	60.0	16.87	44.11	0.66	8.00	44.11	1.360	0.382
	8.5	60.0	16.87	44.11	0.66	8.50	44.11	1.360	0.382
	6.5	61.0	16.87	44.11	0.66	6.50	44.11	1.383	0.382
	7.0	61.0	16.87	44.11	0.66	7.00	44.11	1.383	0.382
	7.5	61.0	16.87	44.11	0.66	7.50	44.11	1.383	0.382
	7.0	63.0	16.87	44.11	0.66	7.00	44.11	1.428	0.382
	12.0	59.0	16.87	44.11	0.66	12.00	44.11	1.338	0.382
	12.0	67.5	16.87	44.11	0.66	12.00	44.11	1.530	0.382
	18.0	61.0	16.87	44.11	0.66	18.00	44.11	1.383	0.382
	19.5	66.5	16.87	44.11	0.66	19.50	44.11	1.508	0.382
	23.0	67.0	16.87	44.11	0.66	23.00	44.11	1.519	0.382
	26.0	64.5	16.87	44.11	0.66	26.00	44.11	1.462	0.382
	27.0	70.0	16.87	44.11	0.66	27.00	44.11	1.587	0.382
	36.0	75.0	16.87	44.11	0.66	36.00	44.11	1.700	0.382
	14.0	60.0	16.87	44.11	0.66	14.00	44.11	1.360	0.382
	8.0	58.0	16.87	44.11	0.66	8.00	44.11	1.315	0.382

Note: Staged test data adjusted by adding the difference ΔT= 44.11-18.04 = 26.07 kN to the measured side shear. Thus the unstaged and staged tests both start at T_0 = 44.11 kN (t_0 = 1 day), and the staged slope remains the same.

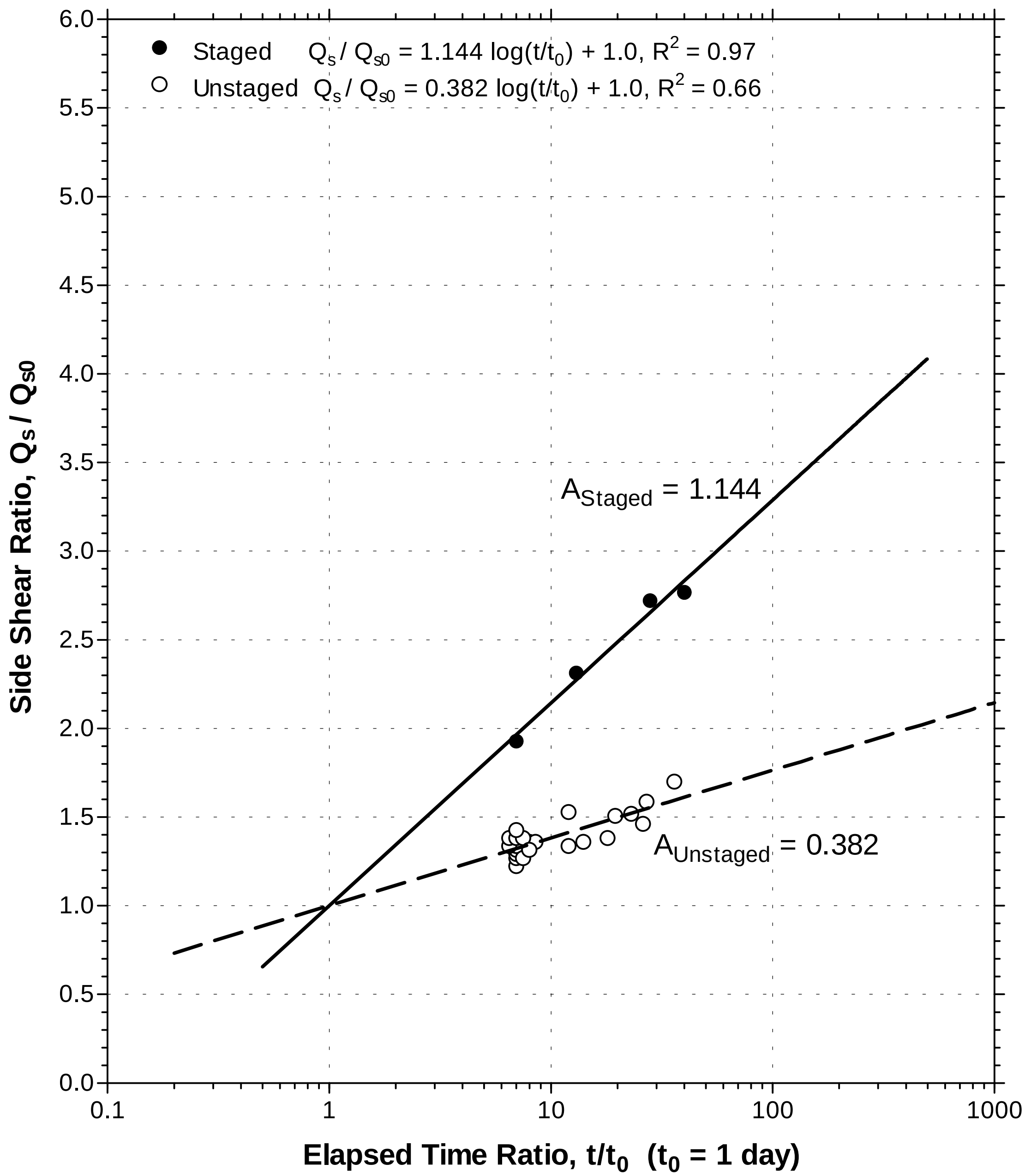


Figure 5.3 Pile Setup Factor, Karlsrud and Haugen (1985) Test Pile

5.3 Staged Versus Unstaged Setup

Table 5.5 summarizes the setup factors for the two layers at the Seabreeze site and the Karlsrud and Haugen test pile. The ratio of unstaged to staged A values for the SPT-T side shear in **Table 5.5** is calculated using both individual staged test series and the overall staged test setup factors for each soil type. The SPT-T staging ratio of 0.397 in the Seabreeze shelly clay layer agrees remarkably well with the staging ratio of 0.334 calculated for the Karlsrud and Haugen test piles. Repeated remolding and ongoing reconsolidation would account for higher strength in the staged A factors. These two sites indicate that staged testing in cohesive soils may increase the measured side shear by approximately 150% over 2-3 log cycles of time, i.e. the staged side shear may be (1 / 0.4) = 2.5 times the unstaged side shear.

The staging ratio in the Seabreeze silty sand layer varies considerably and is highly suspect due to the lack of measured setup. Probably because of disturbance effects, negative staged setup was observed in sands by Bullock (1999) over short time durations (1 to 4 days), both for the test piles and for the SPT-T. The eventual long-term strength gain was attributed to creep and aging affects. The matching pile segment for the silty sand layer has A = 0.509 (see **Table 6.2**). Using the pile segment as representative of the staged A factor, the ratio of unstaged SPT-T/staged pile A would be (0.138/0.509) = 0.271. However, stage testing should have less effect on the structure and drained shear strength of sands than the cohesive soils discussed above. Therefore, a tentative staging ratio of 0.40 for sands would seem conservative.

Based on the results of this research, engineers should consider staged testing effects significant and compensate as appropriate. Figure 5.4 shows the effect of using a staging reduction of 0.40 for all of the measured pile segment setup factors presented in **Figure 2.2**. An unstaged setup factor of 0.10 is shown in **Figure 5.4** as a relatively conservative lower boundary for Florida soils.

Table 5.5 Setup Factor Staging Ratio Summary			
Unstaged A	Staged A		Ratio Unstaged/Staged
Seabreeze Silty Sand Layer (SPT-T)			
0.138	Boring 1	0.092	1.500
	Boring 5	-0.035	-3.943
	Boring 9	0.031	4.452
	Overall	0.018	7.667
Seabreeze Shelly Clay Layer (SPT-T)			
0.265	Boring 1	0.867	0.306
	Boring 5	0.603	0.439
	Boring 9	0.510	0.520
	Overall	0.667	0.397
Karlsrud & Haugen Clay Layer (Pile)			
0.382	1.144		0.334

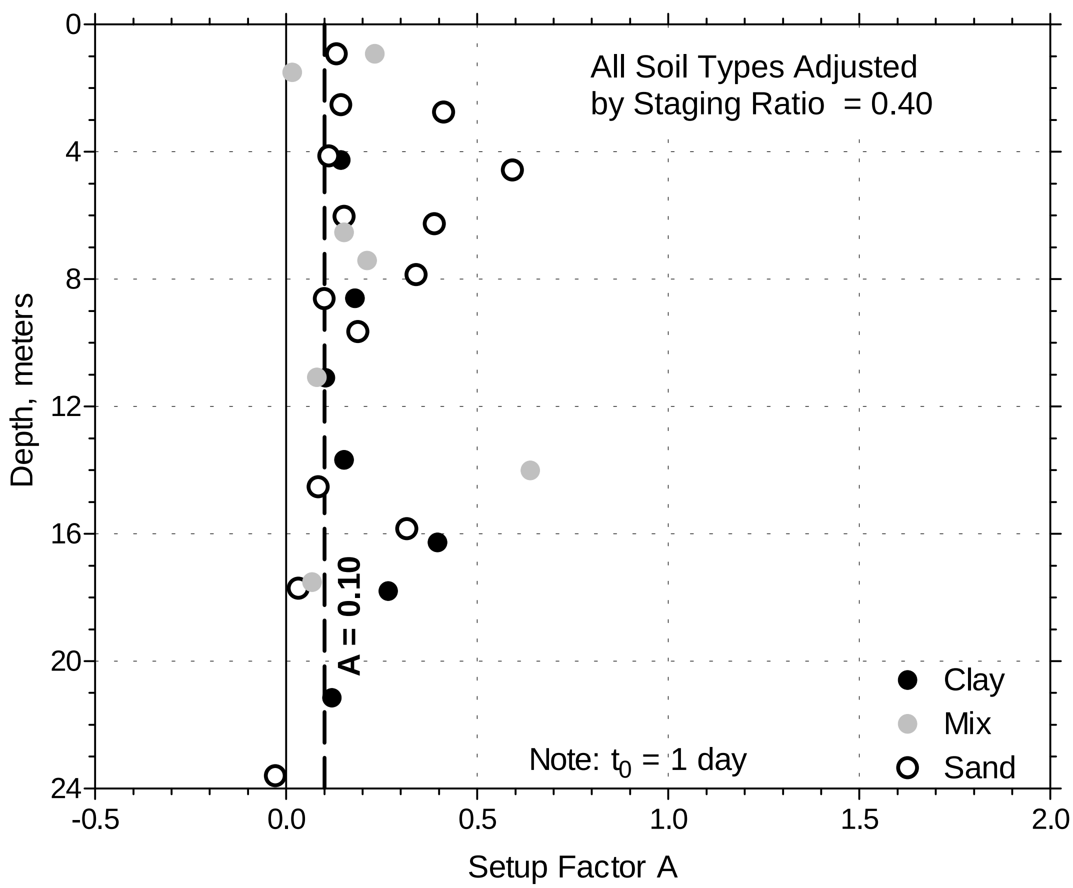


Figure 5.4 Pile Segment Setup Factors from Bullock (1999) Adjusted for Staging

6. SPT-T SETUP VERSUS TEST PILE SIDE SHEAR SETUP

The SPT-T tests reported by Bullock (1999) combined with those performed during this research provide several comparisons between SPT-T and pile side shear, and offer a possible prediction correlation for pile side shear. However, because the side shear changes (generally increasing) with time, any such correlation must include the time behavior of both the SPT-T and the test pile. This chapter investigates the reference side shear and setup factor, which may provide the relationship needed.

6.1 Vilano Beach West SPT-T Results

Although Bullock (1999) performed only staged tests, three pile segments from the Vilano West site in Saint Augustine, FL have SPT-T side shear setup comparisons available. Similar to the silty sand layer at Seabreeze, the SPT-T results from sands at both Vilano East and West piles measured little or no setup behavior. However, as shown in **Table 6.1** and **Figures 6.1 and 6.2**, the SPT-T tests in clay at the Vilano West site exhibit a semilog-linear time trend similar to the Seabreeze site. For consistency with the Seabreeze analyses, these setup factors are calculated using t_0 = 5 minutes.

Table 6.1 Vilano West SPT-T Analysis of Peak Side Shear

Boring	Test Elev. m,msl	Elapsed Time, t min	Peak Shear, τ_{pk} kPa	$\tau = a \log_{10}(t) + b$ a kPa	b kPa	R^2	t/t_o t_o,min = 5	τ_o (LS fit) kPa	τ/τ_o	A
Frz013	-10.03	4.50	39.0	16.290	29.370	0.992	0.90	40.76	0.957	0.400
		60.10	60.3	16.290	29.370	0.992	12.02	40.76	1.480	0.400
		946.70	76.9	16.290	29.370	0.992	189.34	40.76	1.887	0.400
Frz013	-12.47	4.00	17.6	8.195	12.600	0.999	0.80	18.33	0.960	0.447
		62.10	27.1	8.195	12.600	0.999	12.42	18.33	1.479	0.447
		258.70	32.5	8.195	12.600	0.999	51.74	18.33	1.773	0.447
Frz013	-14.90	5.10	19.9	6.584	14.250	0.956	1.02	18.85	1.056	0.349
		60.20	24.1	6.584	14.250	0.956	12.04	18.85	1.278	0.349
		1011.10	34.9	6.584	14.250	0.956	202.22	18.85	1.851	0.349
Frz014	-9.57	4.40	30.3	7.590	25.740	0.996	0.88	31.05	0.976	0.244
		60.60	39.9	7.590	25.740	0.996	12.12	31.05	1.285	0.244
		861.90	47.7	7.590	25.740	0.996	172.38	31.05	1.536	0.244
Frz014	-12.31	4.00	17.0	7.812	11.970	0.986	0.80	17.43	0.975	0.448
		60.00	24.9	7.812	11.970	0.986	12.00	17.43	1.429	0.448
		239.90	31.2	7.812	11.970	0.986	47.98	17.43	1.790	0.448
Frz014	-14.75	4.20	12.3	9.544	7.167	0.986	0.84	13.84	0.889	0.690
		60.00	25.7	9.544	7.167	0.986	12.00	13.84	1.857	0.690
		1087.90	35.4	9.544	7.167	0.986	217.58	13.84	2.558	0.690

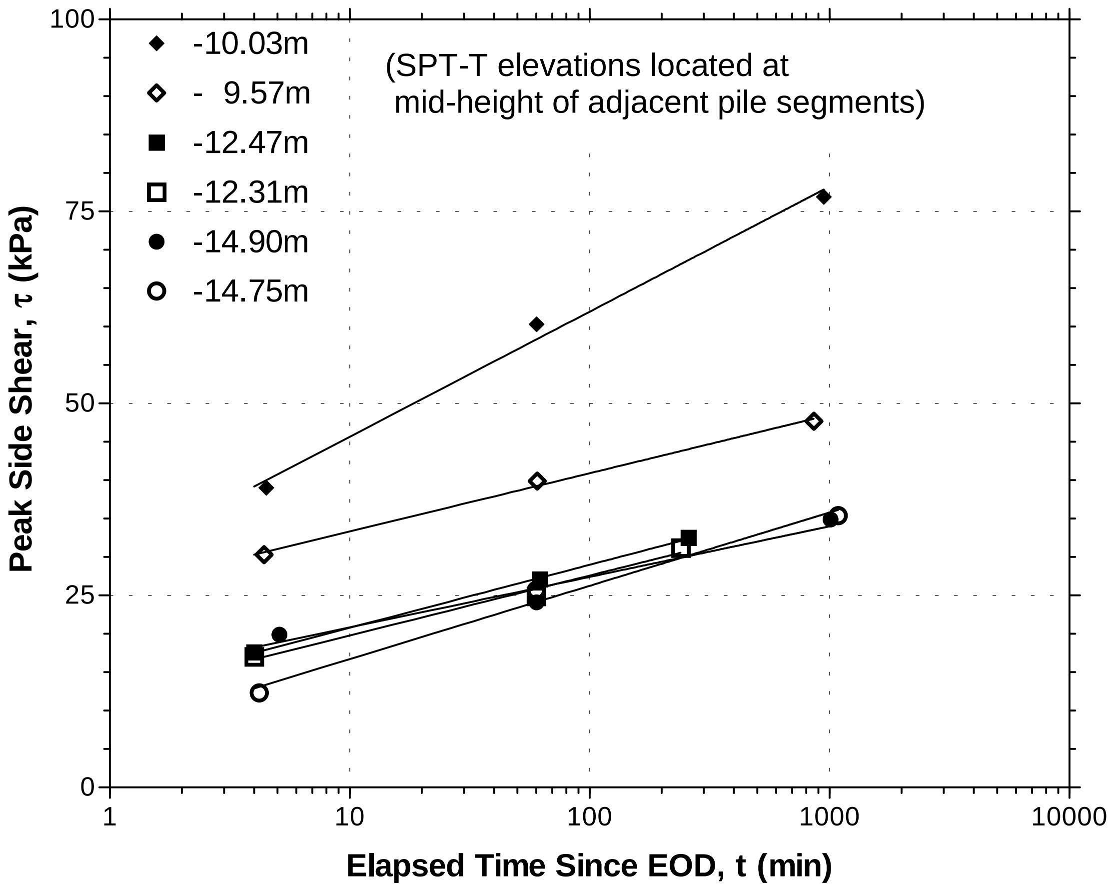


Figure 6.1 SPT-T Staged Side Shear, Vilano West Silty Clay

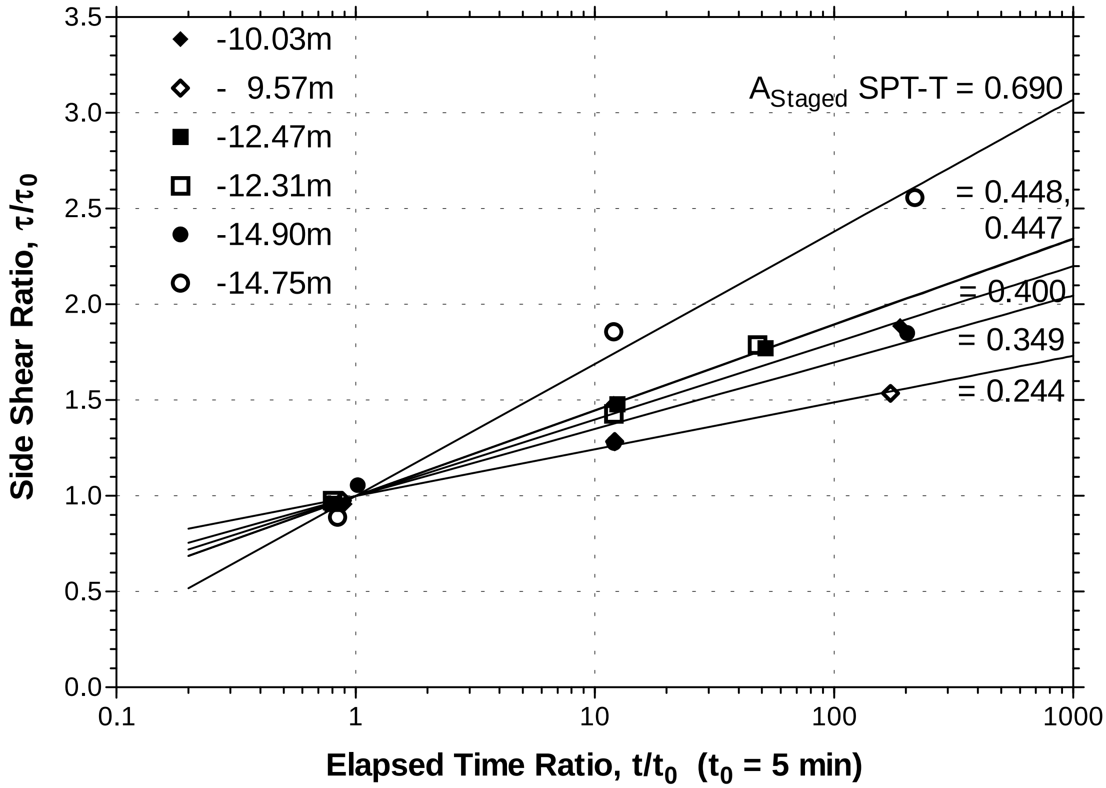


Figure 6.2 SPT-T Staged Setup Factors, Vilano West Silty Clay

6.2 Test Pile Setup Factors

Bullock (1999) reports the maximum side shear measured for each pile segment during each staged test. Then, using the first two values to calculate the side shear at the 1 day reference time, Bullock (1999) calculated and reported a setup factor for each segment. The analyses herein refine this calculation by using a nonlinear least squares regression to determine the overall semilog-linear trend for each pile segment. There was little change in the setup factors using this statistically superior analysis. **Table 6.2** tabulates the setup factors for each segment, along with the segment side shear and reference side shear (1 day). **Figures 6.3-6.6** show the resulting setup factors plotted for the SPT-T side shear versus the adjacent pile segment side shear. The pile segments in **Table 6.2** varied in length from 2.5 to 3.5 m, and the side shear reported was averaged over the segment length. Since the SPT-T length (0.5 m) is much shorter than the matching pile segments, some mismatch between the SPT-T and pile side shear will occur because of axial soil variability.

Table 6.2 Test Piles Analysis of Peak Side Shear

Test Pile	Test Elev. m,msl	Elapsed Time, t days	Peak Shear, τ_{pk} kPa	$\tau = a \log_{10}(t) + b$: a kPa	b kPa	R^2	t/t_o t_o,days = 1	τ_o (LS fit) kPa	τ/τ_o	A
Seabreeze	-7.74	0.35	12.5	7.114	13.97	0.964	0.35	13.97	0.895	0.509
		4.03	17.3	7.114	13.97	0.964	4.03	13.97	1.238	0.509
		17.94	22.2	7.114	13.97	0.964	17.94	13.97	1.589	0.509
		69.94	24.4	7.114	13.97	0.964	69.94	13.97	1.747	0.509
		292.90	32.6	7.114	13.97	0.964	292.90	13.97	2.334	0.509
		1057.90	37.0	7.114	13.97	0.964	1057.90	13.97	2.649	0.509
Seabreeze	-15.89	0.35	29.1	19.000	30.69	0.914	0.35	30.69	0.948	0.619
		4.03	29.5	19.000	30.69	0.914	4.03	30.69	0.961	0.619
		17.94	52.5	19.000	30.69	0.914	17.94	30.69	1.711	0.619
		69.94	72.7	19.000	30.69	0.914	69.94	30.69	2.369	0.619
		292.90	80.2	19.000	30.69	0.914	292.90	30.69	2.613	0.619
		1057.90	86.2	19.000	30.69	0.914	1057.90	30.69	2.809	0.619
Vilano West	-9.70	0.26	17.6	4.794	21.30	0.866	0.26	21.30	0.826	0.225
		2.84	25.5	4.794	21.30	0.866	2.84	21.30	1.197	0.225
		18.98	26.3	4.794	21.30	0.866	18.98	21.30	1.235	0.225
Vilano West	-12.29	0.26	11.3	5.494	13.76	0.936	0.26	13.76	0.821	0.399
		2.84	16.3	5.494	13.76	0.936	2.84	13.76	1.185	0.399
		18.98	18.4	5.494	13.76	0.936	18.98	13.76	1.337	0.399
		157.01	27.4	5.494	13.76	0.936	157.01	13.76	1.991	0.399
Vilano West	-14.88	0.26	14.8	15.910	18.53	0.902	0.26	18.53	0.799	0.859
		2.84	18.4	15.910	18.53	0.902	2.84	18.53	0.993	0.859
		18.98	35.9	15.910	18.53	0.902	18.98	18.53	1.937	0.859
		157.01	58.2	15.910	18.53	0.902	157.01	18.53	3.141	0.859

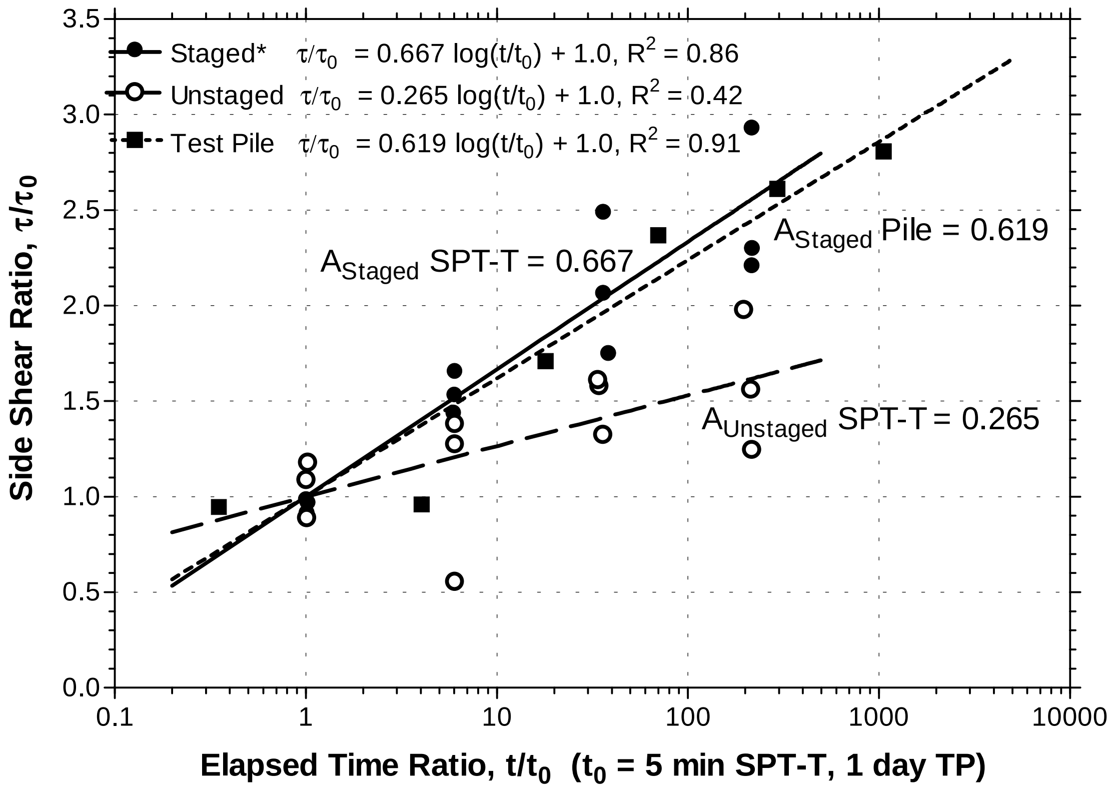


Figure 6.3 SPT-T vs. Pile Setup Factors, Seabreeze Silty Clay

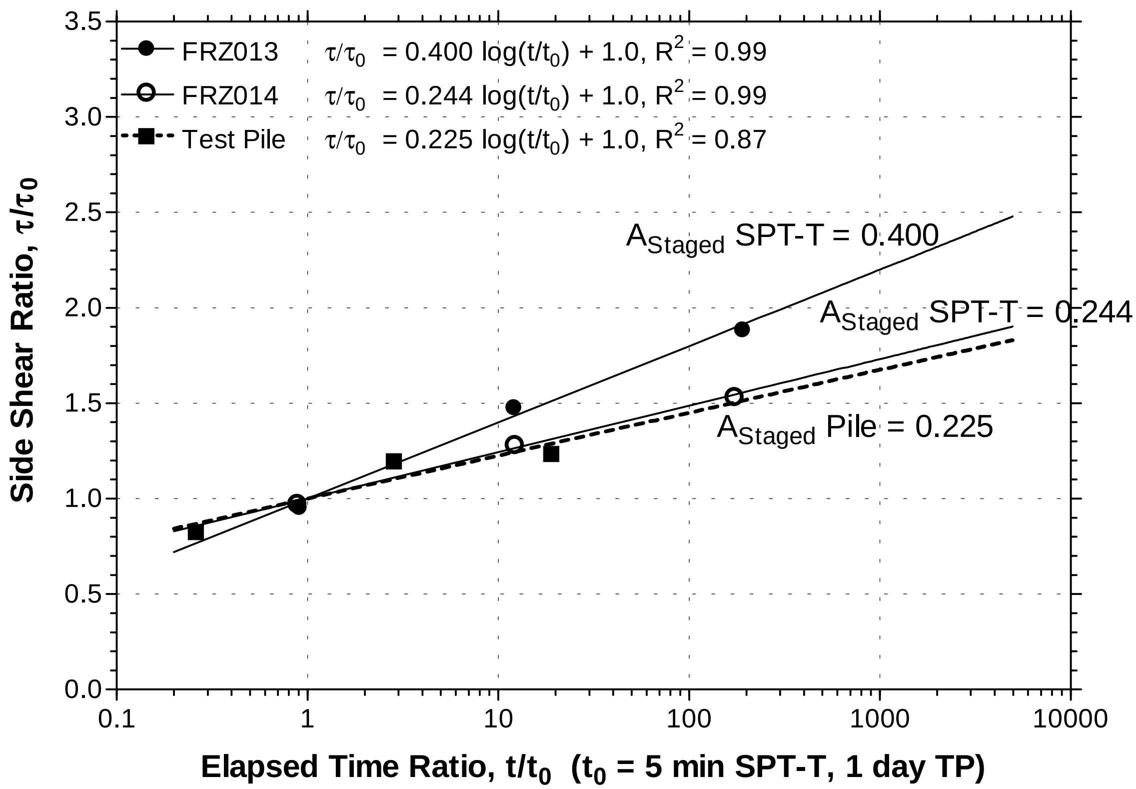


Figure 6.4 SPT-T vs. Pile Setup Factors, Vilano West Silty Clay, Elev. -9.70m

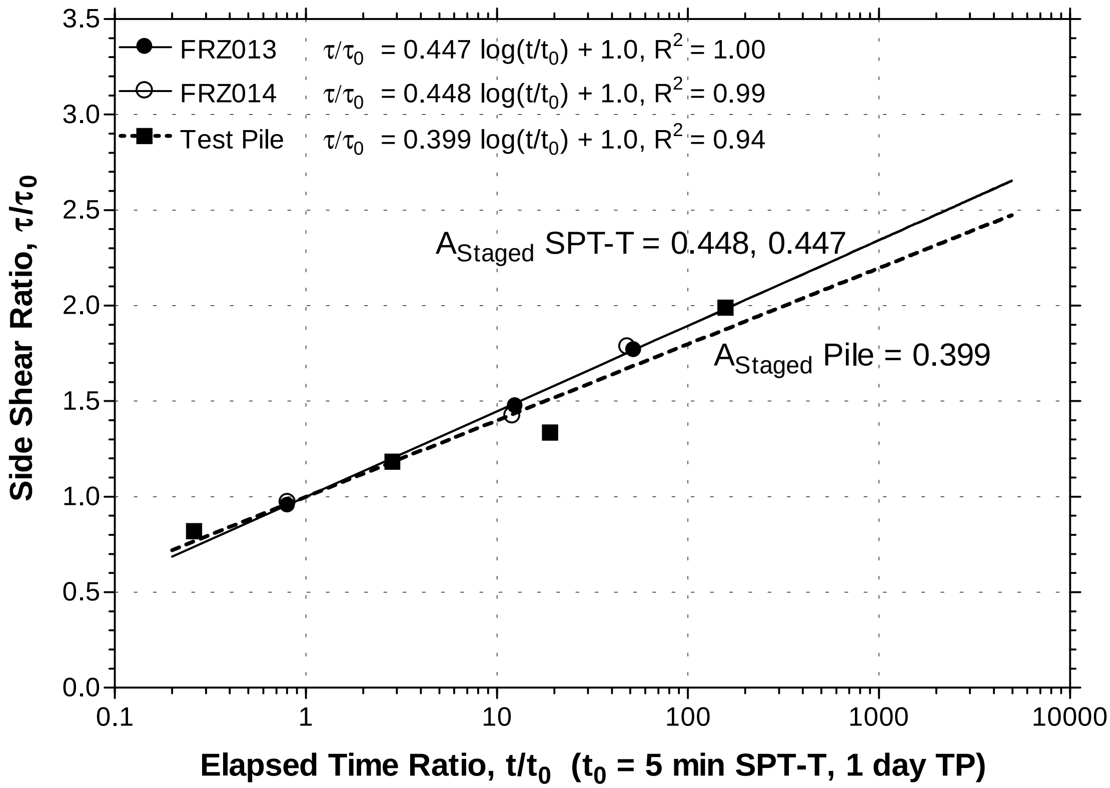


Figure 6.5 SPT-T vs. Pile Setup Factors, Vilano West Silty Clay, Elev. –12.29m

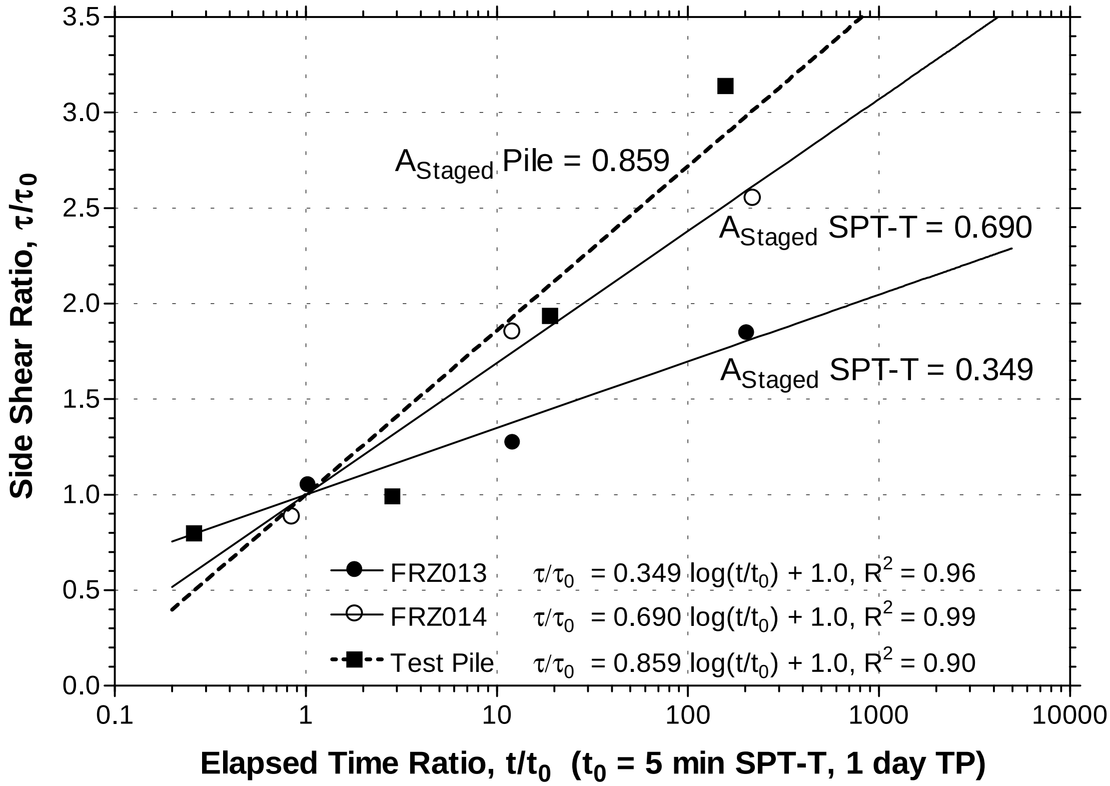


Figure 6.6 SPT-T vs. Pile Setup Factors, Vilano West Silty Clay, Elev. –14.88m

6.3 Comparison of SPT-T and Test Pile Side Shear

Table 6.3 compares the setup factors calculated for each SPT-T test with the adjacent pile segment. For four pile segments, the average bias (measured/predicted) of nine SPT-T setup factor predictions is 1.10 with a coefficient of variation of 52%. The plot in **Figure 6.7** shows these comparisons graphically. The SPT-T continues to show promise as a predictor of setup behavior.

Table 6.4 compares the reference side shear calculated for each SPT-T test at 5 minutes with the adjacent pile segment reference side shear calculated at 1 day. The plot in **Figure 6.8** shows these comparisons graphically. The average bias of these nine SPT-T predictions is 0.77 with a coefficient of variation of 36%. This bias is similar to the comparison of uplift and torsional side shear reported by Rausche et al. (1996) and discussed in Section 2.2.

Based on the above comparisons, the SPT-T over predicts the reference side shear and under predicts the setup factor. Some of this prediction error may be due to site variability as discussed in Section 6.2. However, varying the chosen reference times might improve these comparisons. Decreasing the SPT-T reference time will decrease the predicted reference shear and increase the predicted setup factor. Conversely, increasing the pile reference time will increase the measured reference shear and decrease the measured setup factor. Either change is contrary to "theoretical" ratio of SPT-T to pile reference times calculated in Section 5.1, possibly a result of underestimating the equivalent pile radius. Decreasing the SPT-T reference time to less than 4 minutes is impractical because of the time required after driving to set up the test. Increasing the pile reference time is also somewhat undesirable because of the mathematical convenience of using the 1 day time. In any case, these are relatively minor changes, which can be incorporated later if additional data warrants such a change. The method used herein is adequate for present, and the SPT-T appears to provide a reasonable prediction of pile behavior in cohesive soils. This observation bears verification for other sizes and types of driven piles. At present, previous correlations between SPT N values and pile side shear are more reliable.

Table 6.3 Setup Factor A, Staged SPT-T vs. Test Pile (Cohesive)						
Test Pile			SPT-T (Staged)			
Site	Segment C.L. Elev. m,msl	Setup Factor A_{TP} t_0 = 1 day	Boring	Test Elev. m,msl	Setup Factor A_{SPTT} t_0 = 5 min	Bias A_{TP} / A_{SPTT}
Seabreeze	-15.89	0.619	1	-15.75	0.867	0.71
		0.619	5	-15.53	0.603	1.03
		0.619	9	-15.80	0.510	1.21
Vilano West	-9.70	0.225	FRZ0013	-10.03	0.400	0.56
		0.225	FRZ0014	-9.57	0.244	0.92
Vilano West	-12.29	0.399	FRZ0013	-12.47	0.447	0.89
		0.399	FRZ0014	-12.31	0.448	0.89
Vilano West	-14.88	0.859	FRZ0013	-14.90	0.349	2.46
		0.859	FRZ0014	-14.75	0.690	1.24
				Average Bias		1.10
				Std. Dev. Bias		0.57
				Coef. of Variation		51.7%

Table 6.4 Reference Side Shear (τ_0), Staged SPT-T vs. Test Pile (Cohesive)						
Test Pile			SPT-T (Staged)			
Site	Segment C.L. Elev. m,msl	Reference Side Shear τ_0 (kPa) t_0 = 1 day	Boring	Test Elev. m,msl	Reference Side Shear τ_0 (kPa) t_0 = 5 min	Bias τ_{0TP} / τ_{0SPTT}
Seabreeze	-15.89	30.69	1	-15.75	56.78	0.54
		30.69	5	-15.53	45.39	0.68
		30.69	9	-15.80	51.53	0.60
Vilano West	-9.70	21.30	FRZ0013	-10.03	40.76	0.52
		21.30	FRZ0014	-9.57	31.05	0.69
Vilano West	-12.29	13.76	FRZ0013	-12.47	18.33	0.75
		13.76	FRZ0014	-12.31	17.43	0.79
Vilano West	-14.88	18.53	FRZ0013	-14.90	18.85	0.98
		18.53	FRZ0014	-14.75	13.34	1.39
				Average Bias		0.77
				Std. Dev. Bias		0.28
				Coef. of Variation		35.7%

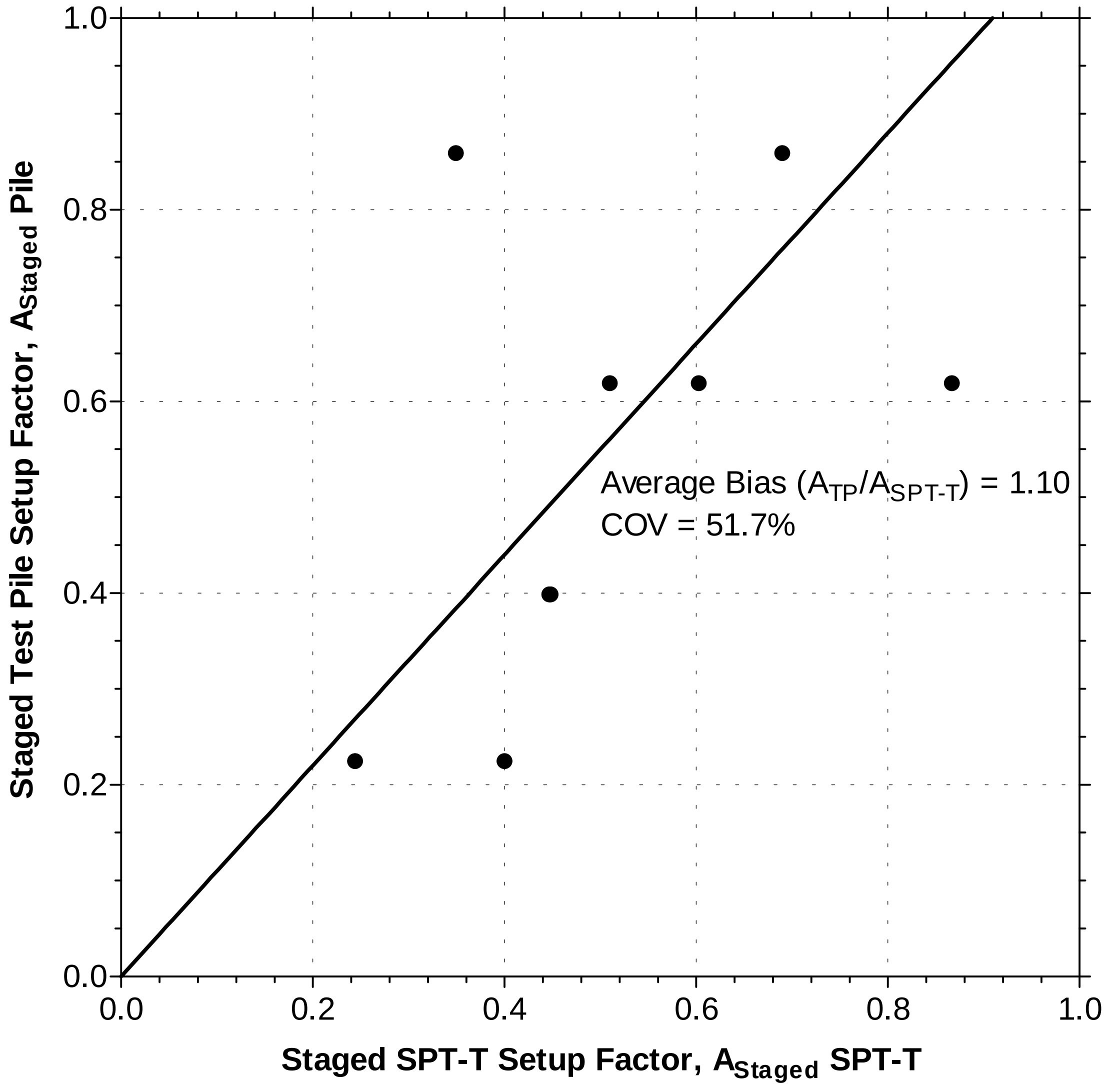


Figure 6.7 Staged Setup Factors, SPT-T vs. Test Pile

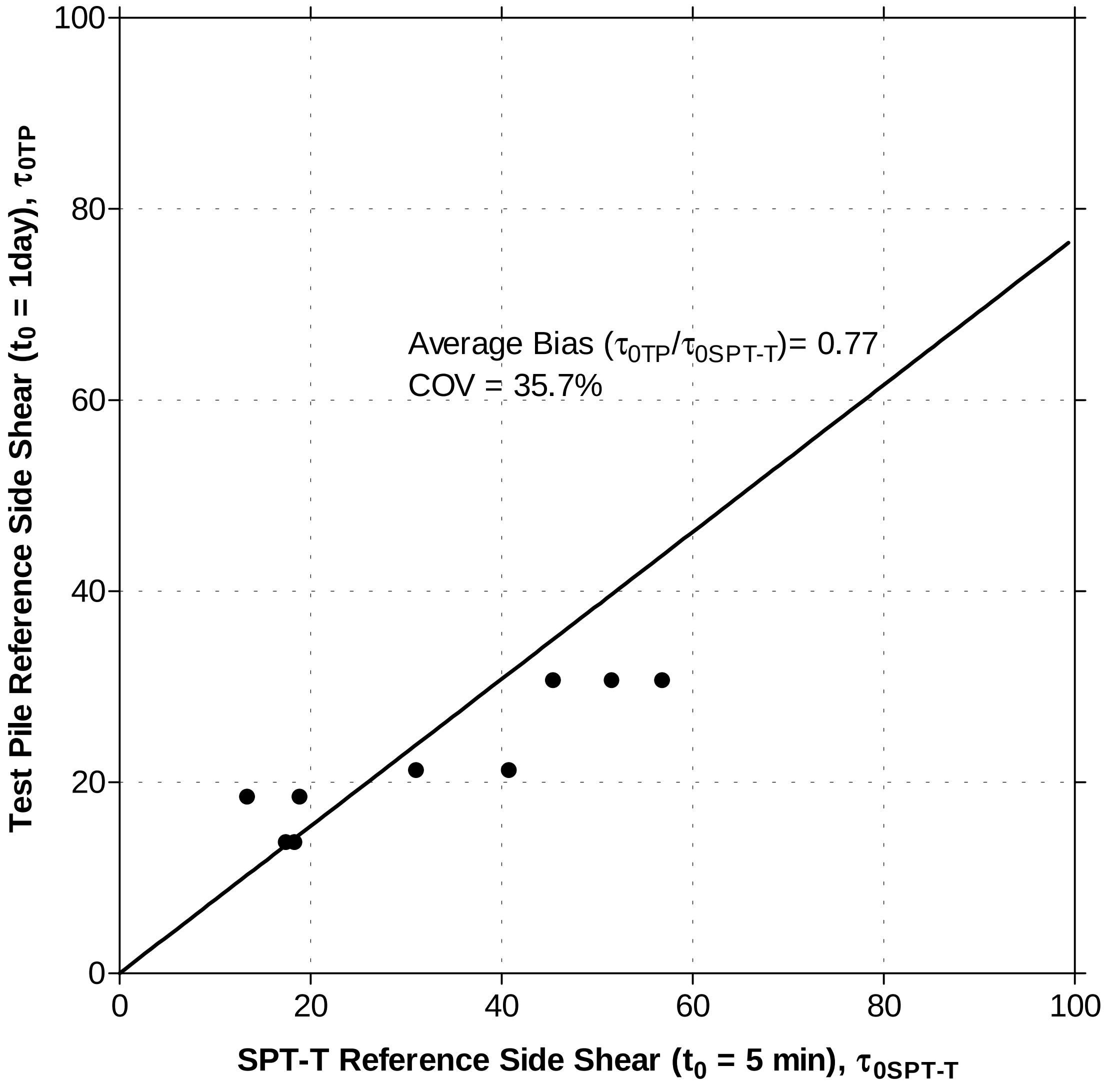


Figure 6.8 Reference Side Shear, SPT-T vs. Test Pile

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The authors present the following conclusions based on the results of this research:

1. SPT-T setup factors compare reasonably well with pile segment ratios in cohesive soil. The SPT-T continues to show significant promise as a predictor test of setup.

2. The SPT-T 5 minute reference time appears reasonable for comparison with pile setup calculated using a reference time of 1 day.

3. The SPT-T setup prediction is not useful for cohesionless soils, at least for a practical test period of less than 24 hours (similar to conclusion in Bullock, 1999).

4. Significant stage testing effects were measured in cohesive soil. We found:

$$(A_{Unstaged} / A_{Staged}) \approx 0.4$$

5. Staged testing effects measured in sands were inconclusive because of the lack of measured setup in the staged tests. The unstaged SPT-T tests showed $A_{Unstaged} = 0.138$. The matching staged test pile segment showed $A_{Staged} = 0.509$, for a ratio of 0.27, which seems too low compared to the ratio measured in cohesive soil.

6. The limited comparisons between pile side shear and SPT-T side shear indicate that for cohesive soils the SPT-T over predicts the pile side shear by a factor of 1.25 (average bias = 0.80).

7.2 Recommendations

Pile side shear setup may potentially have significant economic impact on FDOT bridge foundations. **The authors believe that sufficient research has now been done in Florida, and elsewhere, for the FDOT to make routine practical use of setup in design.** The results and conclusions from this and other projects lead the authors to make the following recommendations:

1. The previous conservative design side shear setup factor of A = 0.2, based on staged tests, and recommended by McVay, Schmertmann, Townsend, & Bullock (1999) for design use without field tests, should be adjusted to A = 0.1 to compensate for stage testing effects. This minimal design setup factor represents a 20% increase in side shear over two log cycles, e.g. from 1 to 100 days. This A = 0.1 setup factor will likely increase when using actual measurements, such as from the SPT-T predictor test, previous site experience, or results from a design phase static and/or dynamic test pile program.

2. A correction factor of ($A_{Unstaged}$ / A_{Staged}) = 0.4 is recommended for both cohesive and cohesionless soils when using staged tests to estimate unstaged pile capacity. Although the staging effects discussed herein are strictly based on static pile tests, Bullock (1999) indicates that dynamic pile tests generally exhibit similar setup behavior. Therefore, pending further dynamic test research, the 0.4 staging correction factor may be used for either static or dynamic staged tests.

3. The current construction practice of performing multiple restrikes on the same pile is a form of staged testing. Engineers may use repeated restrikes with limited penetration (<0.25" each restrike, based on research pile data) during construction to increase the side shear capacity of marginal piles. This technique may increase the side shear capacity of an individual pile beyond that obtained from the initial driving and subsequent unstaged setup.

4. Do not apply staged setup factors measured on one pile to estimate the unstaged setup of adjacent piles without correcting for the staging effect. (To avoid the complication of staging effects, restrikes at different times may be performed on

separate piles driven to similar tip elevations in the same group and then used to estimate directly the unstaged setup behavior of the group.)

5. To increase, probably significantly, the above no-test default A = 0.1 setup factor, perform field tests with the SPT-T for design setup prediction.

6. Although the evidence of setup and staging effects presented herein is adequate for design use, further research comparing staged and unstaged shear strength might prove useful. This work could include simple laboratory tests as well as field tests on either model or full size piles.

7. Perform further research to investigate the effect of pile size and/or type on setup factors.

8. Based on the results from this research and from Bullock (1999) and McVay, Schmertmann, Townsend, & Bullock (1999), **Appendix D** presents recommended procedures for the use of pile setup in design.

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APPENDIX A

TORQUE CELL CALIBRATION

Table A.1 Bridge A Torque Cell Calibration Data (1st run)
Calibrated 14 Jun 01 by M. Hicks & P. Bullock

Load #	Applied Weight (lbs)	Applied Weight (oz)	Lever Arm (ft)	Applied Torque (ft-lbs)	Applied Torque (N-m)	CR10 Measurements Msd. (mV/V)	CR10 Measurements Net (mV/V)
Baseline	0	0.00	0.00	0.00	0.00	0.7774	0.0000
Wrench & Bucket	4	6.24	0.34	4.20	5.70	0.7830	0.0056
	2	4.16	1.20				
1	13	13.92	1.20	20.82	28.23	0.8047	0.0273
2	13	10.40	1.20	37.17	50.40	0.8259	0.0486
3	13	10.72	1.20	53.55	72.60	0.8471	0.0697
4	13	10.72	1.20	69.92	94.80	0.8679	0.0905
5	8	13.12	1.20	80.49	109.13	0.8817	0.1043
6	7	0.00	1.20	88.87	120.49	0.8924	0.1150
7	6	13.60	1.20	97.08	131.62	0.9030	0.1257
8	6	13.76	1.20	105.30	142.76	0.9137	0.1363
9	6	14.88	1.20	113.60	154.02	0.9243	0.1469
10	6	14.88	1.20	121.90	165.27	0.9350	0.1576
11	3	6.56	1.20	125.99	170.81	0.9403	0.1629
12	3	6.56	1.20	130.07	176.35	0.9456	0.1682
13	3	6.88	1.20	134.18	181.92	0.9509	0.1735
12	-3	-6.88	1.20	130.07	176.35	0.9454	0.1680
11	-3	-6.56	1.20	125.99	170.81	0.9401	0.1628
10	-3	-6.56	1.20	121.90	165.27	0.9349	0.1575
9	-6	-14.88	1.20	113.60	154.02	0.9242	0.1468
8	-6	-14.88	1.20	105.30	142.76	0.9134	0.1361
7	-6	-13.76	1.20	97.08	131.62	0.9028	0.1254
6	-6	-13.60	1.20	88.87	120.49	0.8922	0.1148
5	-7	0.00	1.20	80.49	109.13	0.8812	0.1038
4	-8	-13.12	1.20	69.92	94.80	0.8674	0.0900
3	-13	-10.72	1.20	53.55	72.60	0.8463	0.0689
2	-13	-10.72	1.20	37.17	50.40	0.8252	0.0478
1	-13	-10.40	1.20	20.82	28.23	0.8039	0.0266
Wrench & Bucket	-13	-13.92	1.20	4.20	5.70	0.7827	0.0053
Baseline	0	0.00	0.00	0.00	0.00	0.7773	0.0000

Table A.2 Bridge A Torque Cell Calibration Data (2nd run)
Calibrated 14 Jun 01 by M. Hicks & P. Bullock

Load #	Weight		Lever Arm	Applied Torque	Applied Torque	CR10 Measurements	
						Msd.	Net
	(lbs)	(oz)	(ft)	(ft-lbs)	(N-m)	(mV/V)	(mV/V)
Baseline	0	0.00	0.00	0.00	0.00	0.7773	0.0000
Wrench & Bucket	4	6.24	0.34	4.20	5.70	0.7827	0.0053
	2	4.16	1.20				
28	13	13.92	1.20	20.82	28.23	0.8048	0.0274
29	13	10.40	1.20	37.17	50.40	0.8256	0.0482
30	13	10.72	1.20	53.55	72.60	0.8466	0.0692
31	13	10.72	1.20	69.92	94.80	0.8678	0.0904
32	8	13.12	1.20	80.49	109.13	0.8814	0.1041
33	7	0.00	1.20	88.87	120.49	0.8924	0.1150
34	6	13.60	1.20	97.08	131.62	0.9030	0.1256
35	6	13.76	1.20	105.30	142.76	0.9137	0.1363
36	6	14.88	1.20	113.60	154.02	0.9243	0.1469
37	6	14.88	1.20	121.90	165.27	0.9349	0.1576
38	3	6.56	1.20	125.99	170.81	0.9421	0.1647
39	3	6.56	1.20	130.07	176.35	0.9456	0.1682
40	3	6.88	1.20	134.18	181.92	0.9509	0.1735
41	-3	-6.88	1.20	130.07	176.35	0.9453	0.1679
42	-3	-6.56	1.20	125.99	170.81	0.9402	0.1628
43	-3	-6.56	1.20	121.90	165.27	0.9349	0.1576
44	-6	-14.88	1.20	113.60	154.02	0.9241	0.1467
45	-6	-14.88	1.20	105.30	142.76	0.9133	0.1359
46	-6	-13.76	1.20	97.08	131.62	0.9027	0.1253
47	-6	-13.60	1.20	88.87	120.49	0.8920	0.1146
48	-7	0.00	1.20	80.49	109.13	0.8811	0.1037
49	-8	-13.12	1.20	69.92	94.80	0.8672	0.0898
50	-13	-10.72	1.20	53.55	72.60	0.8464	0.0690
51	-13	-10.72	1.20	37.17	50.40	0.8252	0.0478
52	-13	-10.40	1.20	20.82	28.23	0.8040	0.0266
Wrench & Bucket	-13	-13.92	1.20	4.20	5.70	0.7827	0.0053
Baseline	0	0.00	0.00	0.00	0.00	0.7774	0.0000

Table A.3 Bridge B Torque Cell Calibration Data (1st run) Calibrated 14 Jun 01 by M. Hicks & P. Bullock							
Load #	Weight		Lever Arm	Applied Torque	Applied Torque	CR10 Measurements	
						Msd.	Net
	(lbs)	(oz)	(ft)	(ft-lbs)	(N-m)	(mV/V)	(mV/V)
Baseline	0	0.00	0.00	0.00	0.00	0.0835	0.0000
Wrench & Bucket	4	6.24	0.34	4.20	5.70	0.0888	0.0053
	2	4.16	1.20				
1	13	13.92	1.20	20.82	28.23	0.1106	0.0272
2	13	10.40	1.20	37.17	50.40	0.1317	0.0482
3	13	10.72	1.20	53.55	72.60	0.1530	0.0696
4	13	10.72	1.20	69.92	94.80	0.1739	0.0905
5	8	13.12	1.20	80.49	109.13	0.1877	0.1042
6	7	0.00	1.20	88.87	120.49	0.1985	0.1150
7	6	13.60	1.20	97.08	131.62	0.2091	0.1257
8	6	13.76	1.20	105.30	142.76	0.2196	0.1362
9	6	14.88	1.20	113.60	154.02	0.2304	0.1470
10	6	14.88	1.20	121.90	165.27	0.2411	0.1576
11	3	6.56	1.20	125.99	170.81	0.2466	0.1632
12	3	6.56	1.20	130.07	176.35	0.2517	0.1682
13	3	6.88	1.20	134.18	181.92	0.2570	0.1735
12	-3	-6.88	1.20	130.07	176.35	0.2517	0.1682
11	-3	-6.56	1.20	125.99	170.81	0.2464	0.1629
10	-3	-6.56	1.20	121.90	165.27	0.2409	0.1575
9	-6	-14.88	1.20	113.60	154.02	0.2301	0.1466
8	-6	-14.88	1.20	105.30	142.76	0.2193	0.1359
7	-6	-13.76	1.20	97.08	131.62	0.2086	0.1251
6	-6	-13.60	1.20	88.87	120.49	0.1980	0.1145
5	-7	0.00	1.20	80.49	109.13	0.1871	0.1036
4	-8	-13.12	1.20	69.92	94.80	0.1732	0.0898
3	-13	-10.72	1.20	53.55	72.60	0.1520	0.0685
2	-13	-10.72	1.20	37.17	50.40	0.1307	0.0472
1	-13	-10.40	1.20	20.82	28.23	0.1094	0.0259
Wrench & Bucket	-13	-13.92	1.20	4.20	5.70	0.0881	0.0046
Baseline	0	0.00	0.00	0.00	0.00	0.0828	-0.0007

Table A.4 Bridge B Torque Cell Calibration Data (2nd run)
Calibrated 14 Jun 01 by M. Hicks & P. Bullock

Load #	Weight (lbs)	Weight (oz)	Lever Arm (ft)	Applied Torque (ft-lbs)	Applied Torque (N-m)	CR10 Measurements Msd. (mV/V)	CR10 Measurements Net (mV/V)
Baseline	0	0.00	0.00	0.00	0.00	0.0828	-0.0007
Wrench & Bucket	4	6.24	0.34	4.20	5.70	0.0881	0.0046
	2	4.16	1.20				
28	13	13.92	1.20	20.82	28.23	0.1099	0.0265
29	13	10.40	1.20	37.17	50.40	0.1312	0.0478
30	13	10.72	1.20	53.55	72.60	0.1526	0.0692
31	13	10.72	1.20	69.92	94.80	0.1738	0.0903
32	8	13.12	1.20	80.49	109.13	0.1874	0.1040
33	7	0.00	1.20	88.87	120.49	0.1983	0.1148
34	6	13.60	1.20	97.08	131.62	0.2090	0.1255
35	6	13.76	1.20	105.30	142.76	0.2197	0.1362
36	6	14.88	1.20	113.60	154.02	0.2304	0.1470
37	6	14.88	1.20	121.90	165.27	0.2411	0.1576
38	3	6.56	1.20	125.99	170.81	0.2464	0.1629
39	3	6.56	1.20	130.07	176.35	0.2517	0.1682
40	3	6.88	1.20	134.18	181.92	0.2570	0.1736
41	-3	-6.88	1.20	130.07	176.35	0.2516	0.1681
42	-3	-6.56	1.20	125.99	170.81	0.2464	0.1629
43	-3	-6.56	1.20	121.90	165.27	0.2409	0.1575
44	-6	-14.88	1.20	113.60	154.02	0.2301	0.1466
45	-6	-14.88	1.20	105.30	142.76	0.2194	0.1359
46	-6	-13.76	1.20	97.08	131.62	0.2087	0.1253
47	-6	-13.60	1.20	88.87	120.49	0.1978	0.1143
48	-7	0.00	1.20	80.49	109.13	0.1870	0.1035
49	-8	-13.12	1.20	69.92	94.80	0.1730	0.0895
50	-13	-10.72	1.20	53.55	72.60	0.1520	0.0685
51	-13	-10.72	1.20	37.17	50.40	0.1307	0.0472
52	-13	-10.40	1.20	20.82	28.23	0.1094	0.0260
Wrench & Bucket	-13	-13.92	1.20	4.20	5.70	0.0881	0.0046
Baseline	0	0.00	0.00	0.00	0.00	0.0828	-0.0007

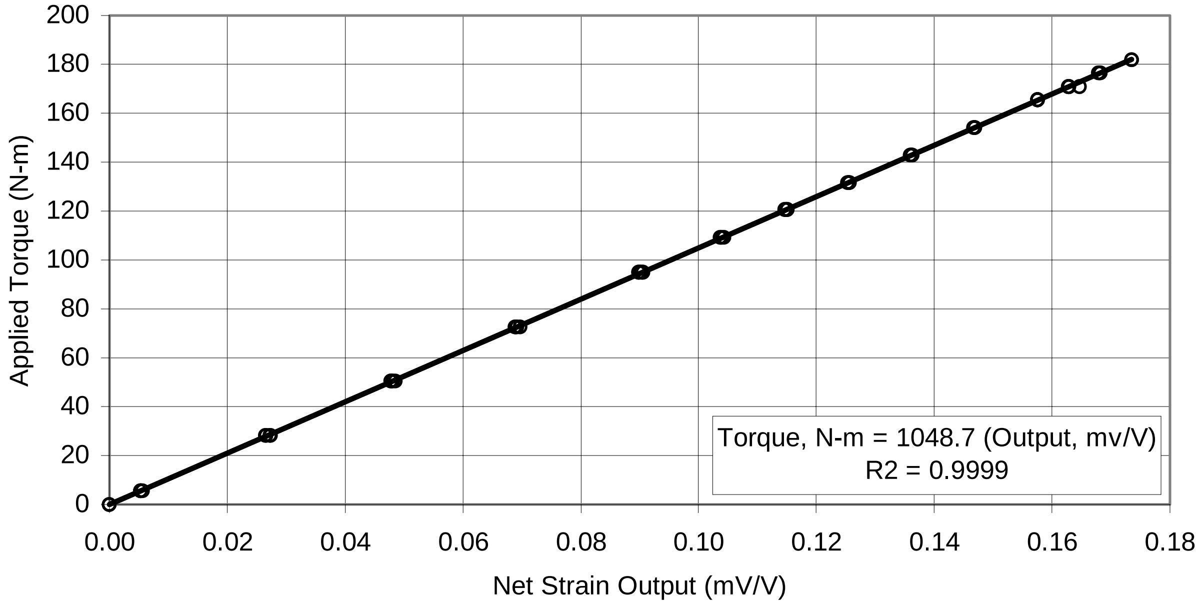

Figure A.1 Torque Cell Calibration (A) (both runs)

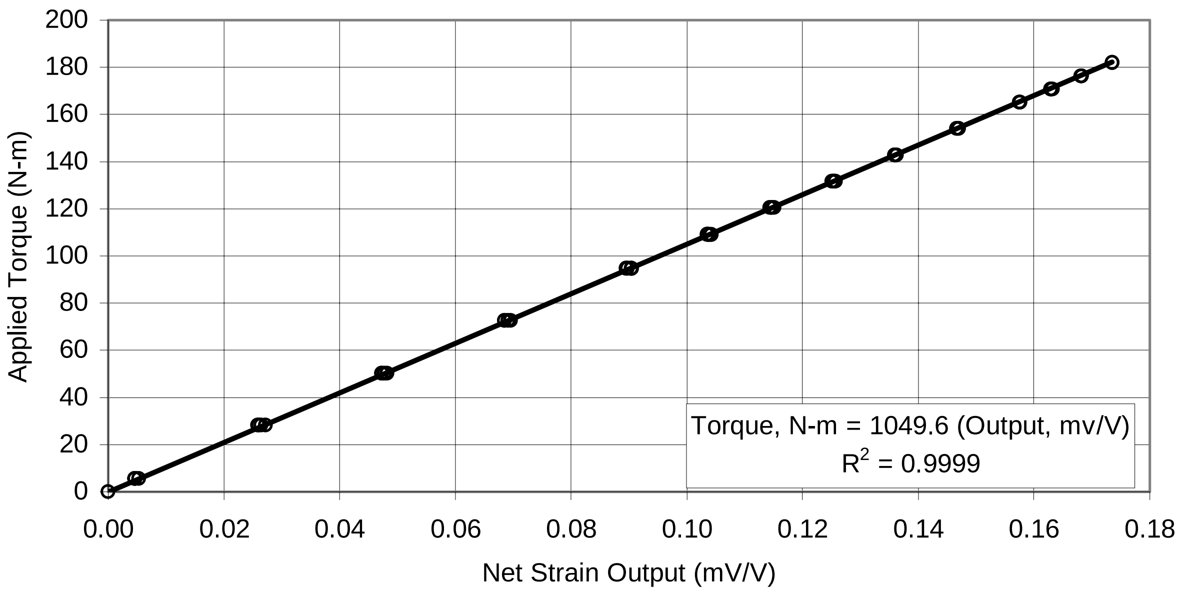

Figure A.2 Torque Cell Calibration (B) (both runs)

APPENDIX B
SEABREEZE LAB TEST RESULTS

Table B.1 Sieve Analyses for Upper Sand Layer						
Boring	Sieve Size	Sample	#20	#50	#100	#200
1	wt + sieve (g)	311.62	389.88	366.23	392.54	446.64
	sieve wt (g)	140.97	389.85	365.27	353.91	325.50
	wt Retained (g)	170.65	0.03	0.96	38.63	121.14
	% Retained		0.02	0.56	22.64	70.99
	% Passing		99.98	99.42	76.78	5.80
2	wt + sieve (g)	264.50	389.91	367.49	400.19	402.57
	sieve wt (g)	132.38	389.85	365.27	353.91	325.50
	wt Retained (g)	132.12	0.06	2.22	46.28	77.07
	% Retained		0.05	1.68	35.03	58.33
	% Passing		99.95	98.27	63.25	4.91
3	wt + sieve (g)	268.12	401.70	365.38	391.18	409.35
	sieve wt (g)	139.46	401.57	364.59	344.16	334.73
	wt Retained (g)	128.66	0.13	0.79	47.02	74.62
	sieve wt (g)		0.10	0.61	36.55	58.00
	% Passing		99.90	99.28	62.74	4.74
4	wt + sieve (g)	276.60	389.94	366.62	389.05	423.98
	sieve wt (g)	136.50	389.85	365.27	353.91	325.50
	wt Retained (g)	140.10	0.09	1.35	35.14	98.48
	% Retained		0.06	0.96	25.08	70.29
	% Passing		99.94	98.97	73.89	3.60
5	wt + sieve (g)	268.46	401.96	375.00	399.31	399.04
	sieve wt (g)	120.85	401.57	364.59	344.16	334.73
	wt Retained (g)	147.61	0.39	10.41	55.15	64.31
	% Retained		0.26	7.05	37.36	43.57
	% Passing		99.74	92.68	55.32	11.75
6	wt + sieve (g)	313.69	389.93	367.26	390.01	455.55
	sieve wt (g)	137.56	389.85	365.27	353.91	325.50
	wt Retained (g)	176.13	0.08	1.99	36.10	130.05
	% Retained		0.05	1.13	20.50	73.84
	% Passing		99.95	98.82	78.33	4.49
7	wt + sieve (g)	272.89	389.88	367.39	382.62	423.51
	sieve wt (g)	138.44	389.85	365.27	353.91	325.50
	wt Retained (g)	134.45	0.03	2.12	28.71	98.01
	% Retained		0.02	1.58	21.35	72.90
	% Passing		99.98	98.40	77.05	4.15
8	wt + sieve (g)	299.26	390.00	367.11	391.01	423.49
	sieve wt (g)	155.12	389.85	365.27	353.91	325.50
	wt Retained (g)	144.14	0.15	1.84	37.10	97.99
	% Retained		0.10	1.28	25.74	67.98
	% Passing		99.90	98.62	72.88	4.90

Table B.1 (continued)						
Boring	Sieve Size	Sample	#20	#50	#100	#200
9	wt + sieve (g)	296.91	390.28	367.72	388.79	436.70
	sieve wt (g)	138.25	389.85	365.27	353.91	325.50
	wt Retained (g)	158.66	0.43	2.45	34.88	111.20
	% Retained		0.27	1.54	21.98	70.09
	% Passing		99.73	98.18	76.20	6.11
10	wt + sieve (g)	293.28	389.87	367.33	388.60	414.70
	sieve wt (g)	160.04	389.85	365.27	353.91	325.50
	wt Retained (g)	133.24	0.02	2.06	34.69	89.20
	% Retained		0.02	1.55	26.04	66.95
	% Passing		99.98	98.44	72.40	5.46
11	wt + sieve (g)	272.90	389.85	366.68	376.97	422.12
	sieve wt	144.97	389.85	365.27	353.91	325.50
	wt Retained (g)	127.93	0.00	1.41	23.06	96.62
	% Retained		0.00	1.10	18.03	75.53
	% Passing		100.00	98.90	80.87	5.35
12	wt + sieve (g)	275.02	389.87	366.54	387.14	429.96
	sieve wt (g)	128.40	389.85	365.27	353.91	325.50
	wt Retained (g)	146.62	0.02	1.27	33.23	104.46
	% Retained		0.01	0.87	22.66	71.25
	% Passing		99.99	99.12	76.46	5.21
Average % Passing			99.92	98.26	72.18	5.54
Standard Deviation			0.09	1.80	7.68	2.08
Coef. of Variation			0.1%	1.8%	10.6%	37.5%

Table B.2 Liquid Limits for Lower Clay

Boring 1	can #	113	611	33	10	64
	can + wet soil (g)	24.76	23.51	30.44	27.91	26.59
	can + dry soil (g)	21.05	20.22	24.92	22.88	22.05
	wt water (g)	3.71	3.29	5.52	5.03	4.54
	wt can (g)	11.21	11.57	10.83	10.81	10.92
	wt dry soil (g)	9.84	8.65	14.09	12.07	11.13
	moisture content (%)	37.70	38.03	39.18	41.67	40.79
	LL blows	48	45	37	21	18
Boring 5	can #	1110	16	4448	36	13
	can + wet soil (g)	24.80	22.92	25.45	30.46	29.30
	can + dry soil (g)	21.27	19.71	21.43	24.81	23.88
	wt water (g)	3.53	3.21	4.02	5.65	5.42
	wt can (g)	11.14	10.90	10.87	10.82	10.84
	wt dry soil (g)	10.13	8.81	10.56	13.99	13.04
	moisture content (%)	34.85	36.44	38.07	40.39	41.56
	LL blows	55	31	23	18	11
Boring 9	can #	26.62	26.37	26.89	27.82	24.46
	can + wet soil (g)	26.62	26.37	26.89	27.82	24.46
	can + dry soil (g)	22.35	22.33	22.31	22.91	20.53
	wt water (g)	4.27	4.04	4.58	4.91	3.93
	wt can (g)	10.82	11.86	10.80	10.90	10.87
	wt dry soil (g)	11.53	10.47	11.51	12.01	9.66
	moisture content (%)	37.03	38.59	39.79	40.88	40.68
	LL blows	50	35	20	17	13

Table B.3 Plastic Limits for Lower Clay				
Boring 1	can #	84	56	3330
	can + wet soil (g)	14.29	17.32	18.52
	can + dry soil (g)	13.55	15.97	17.03
	wt water (g)	0.74	1.35	1.49
	wt can (g)	10.67	10.92	11.39
	wt dry soil (g)	2.88	5.05	5.64
	moisture content (%)	25.69	26.73	26.42
	average (%)	26.28		
Boring 5	can #	2	50	6
	can + wet soil (g)	14.33	19.19	15.04
	can + dry soil (g)	13.64	17.54	14.20
	wt water (g)	0.69	1.65	0.84
	wt can (g)	11.19	11.55	11.00
	wt dry soil (g)	2.45	5.99	3.20
	moisture content (%)	28.16	27.55	26.25
	average (%)	27.32		
Boring 9	can #	803	38	107
	can + wet soil (g)	17.31	16.11	14.83
	can + dry soil (g)	16.32	15.06	14.01
	wt water (g)	0.99	1.05	0.82
	wt can (g)	11.71	10.8	10.66
	wt dry soil (g)	4.61	4.26	3.35
	moisture content (%)	21.48	24.65	24.48
	average (%)	23.53		

Table B.4 Natural Moisture Contents of Lower Clay						
Boring	1	2	3	4	5	6
can #	774	14	107	84	2	105
can + wet soil (g)	31.98	31.92	45.08	33.69	38.69	34.45
can + dry soil (g)	26.37	26.33	36.62	27.56	31.19	28.13
wt water (g)	5.61	5.59	8.46	6.13	7.50	6.32
wt can (g)	11.48	10.90	10.65	10.67	11.19	11.05
wt dry soil (g)	14.89	15.43	25.97	16.89	20.00	17.08
moisture content (%)	37.68	36.23	32.58	36.29	37.50	37.00
Boring	7	8	9	10	11	12
can #	13	554	115	43	6	8
can + wet soil (g)	41.41	37.42	34.18	29.83	43.12	33.24
can + dry soil (g)	33.21	30.20	27.42	23.90	34.57	26.50
wt water (g)	8.20	7.22	6.76	5.93	8.55	6.74
wt can (g)	11.14	11.25	10.87	10.77	10.86	10.90
wt dry soil (g)	22.07	18.95	16.55	13.13	23.71	15.60
moisture content (%)	37.15	38.10	40.85	45.16	36.06	43.21

Table B.5 Atterberg Limits Summary for Lower Clay				
Boring	Natural Moisture %	Liquid Limit %	Plastic Limit %	Plasticity Index %
1	37.68	40.65	26.28	14.37
2	36.23			
3	32.58			
4	36.29			
5	37.50	38.75	27.32	11.43
6	37.00			
7	37.15			
8	38.10			
9	40.85	39.62	23.53	16.09
10	45.16			
11	36.06			
12	43.21	39.62	23.53	16.09
Average	38.15	39.66	25.17	14.49
Std.Dev.	3.41	0.78	1.93	2.20
Coef. of Var.	8.93%	1.96%	7.69%	15.16%

Table B.6 %Retained on #200 for Lower Clay			
Boring	1	5	9
wt. Pan (g)	161.93	153.45	161.19
wt. Pan + sample (g)	612.84	515.87	766.44
wt. sample (g)	450.91	362.42	605.25
moisture content (%)	37.68	37.50	40.85
wt. dry sample (g)	327.51	263.58	429.72
wt. #200 sieve (g)	161.29	153.59	161.99
wt. #200 + sample (g)	203.64	173.08	196.64
wt. #200 sample (g)	42.35	19.49	34.65
% Retained on #200	12.93	7.39	8.06
Average	9.46		
Standard Deviation	3.02		
Coef. of Variation	31.93%		

Table B.7 Sieve Analyses of %Retained on #200 from Lower Clay						
Boring	Sieve Size	#4	#10	#20	#40	Pan(#200)
1	wt + sieve (g)	526.91	477.30	423.26	350.83	377.81
	sieve wt (g)	513.83	466.74	411.62	346.31	375.35
	wt Retained (g)	13.08	10.56	11.64	4.52	2.46
	% Retained	30.95	24.99	27.54	10.70	5.82
	% Passing	69.05	44.06	16.52	5.82	0.00
5	wt + sieve (g)	516.62	473.29	417.55	348.64	377.12
	sieve wt (g)	513.83	466.74	411.62	346.31	375.35
	wt Retained (g)	2.79	6.55	5.93	2.33	1.77
	% Retained	14.40	33.82	30.61	12.03	9.14
	% Passing	85.60	51.78	21.17	9.14	0.00
9	wt + sieve (g)	523.73	476.79	420.17	349.54	378.15
	sieve wt (g)	513.83	466.74	411.62	346.31	375.35
	wt Retained (g)	9.90	10.05	8.55	3.23	2.80
	% Retained	28.67	29.11	24.76	9.35	8.11
	% Passing	71.33	42.22	17.46	8.11	0.00
Average % Passing		75.32	46.02	18.38	7.69	0.00
Standard Deviation		8.97	5.07	2.46	1.70	
Coef. of Variation		11.91%	11.02%	13.37%	22.08%	

APPENDIX C

SEABREEZE TORQUE TEST RESULTS

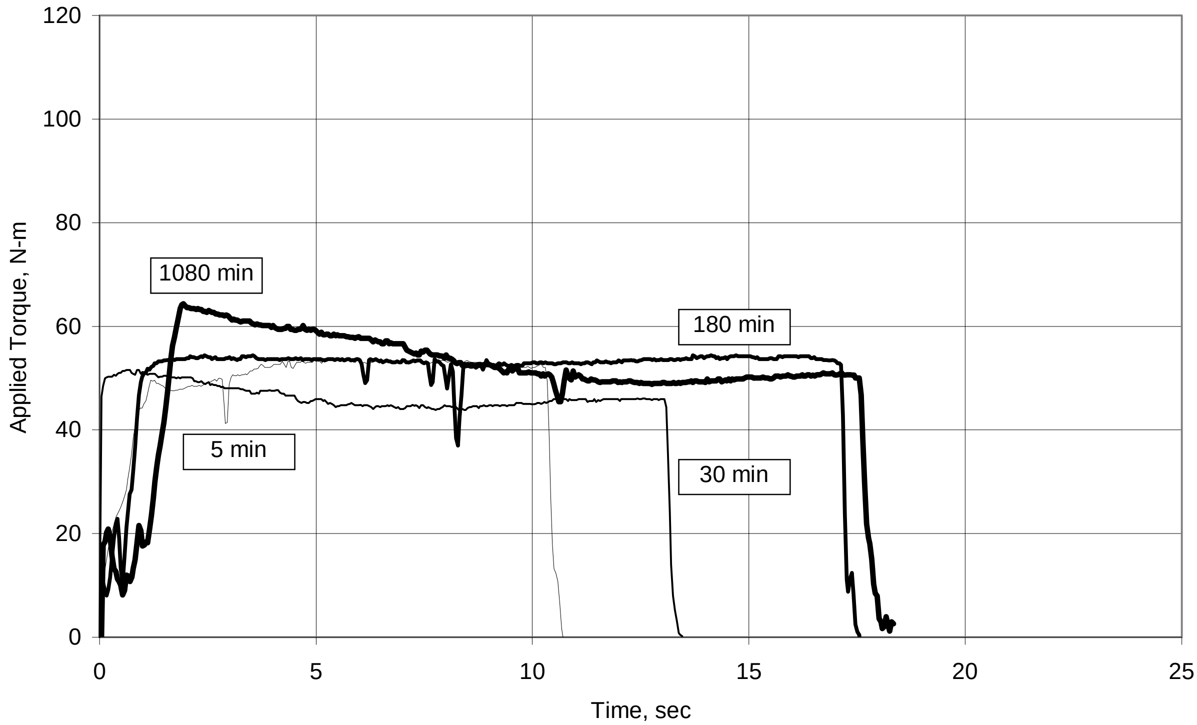

Figure C.1 SPT-T, Boring 1, Staged, Silty Sand Layer

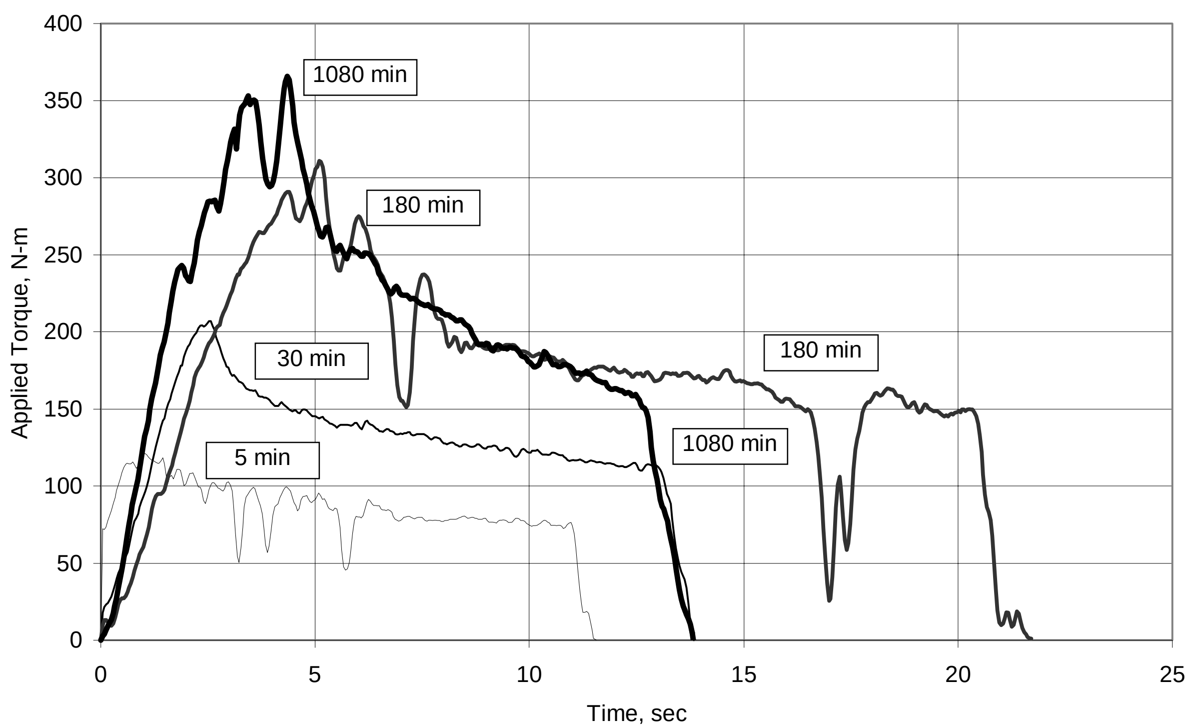

Figure C.2 SPT-T, Boring 1, Staged, Shelly Clay Layer

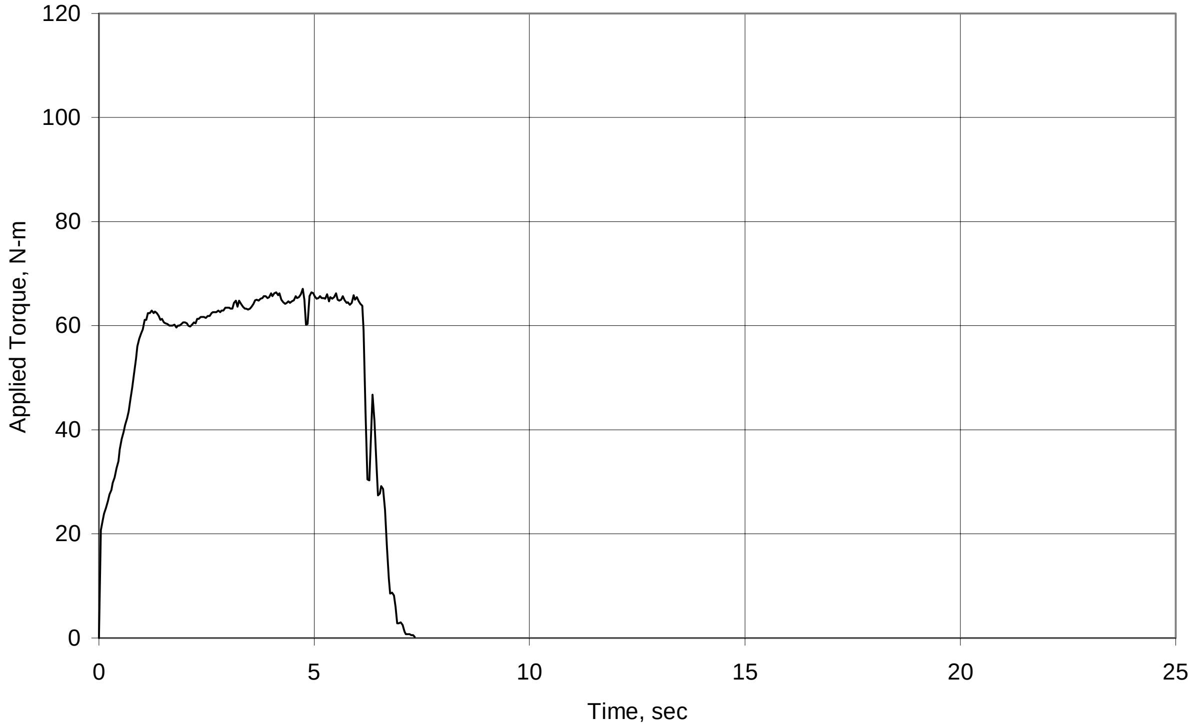

Figure C.3 SPT-T, Boring 2, 30 min Unstaged, Silty Sand Layer

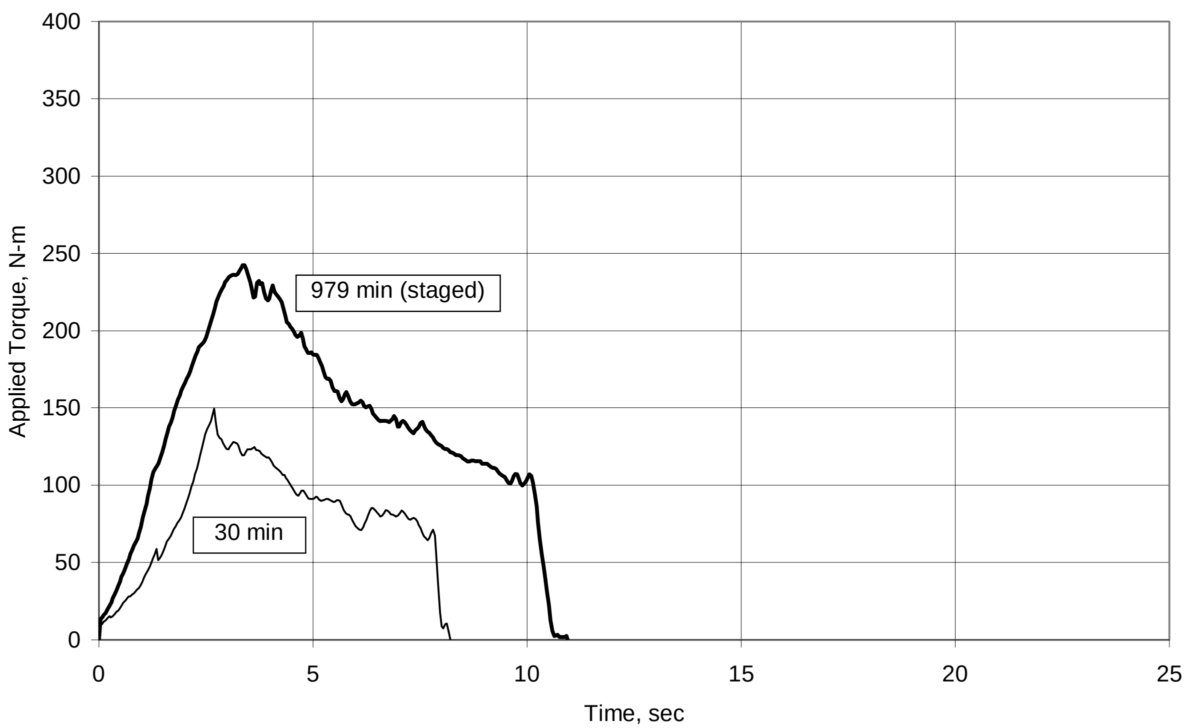

Figure C.4 SPT-T, Boring 2, 30 min Unstaged, Shelly Clay Layer

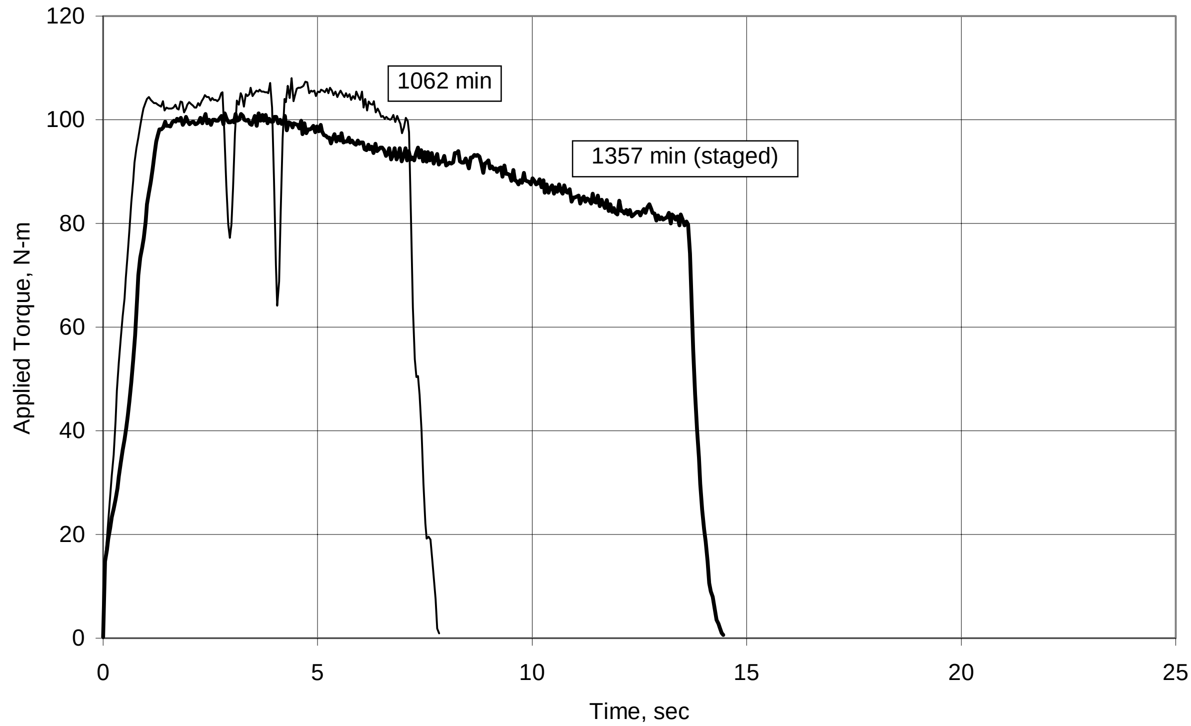

Figure C.5 SPT-T, Boring 3, 1062 min Unstaged, Silty Sand Layer

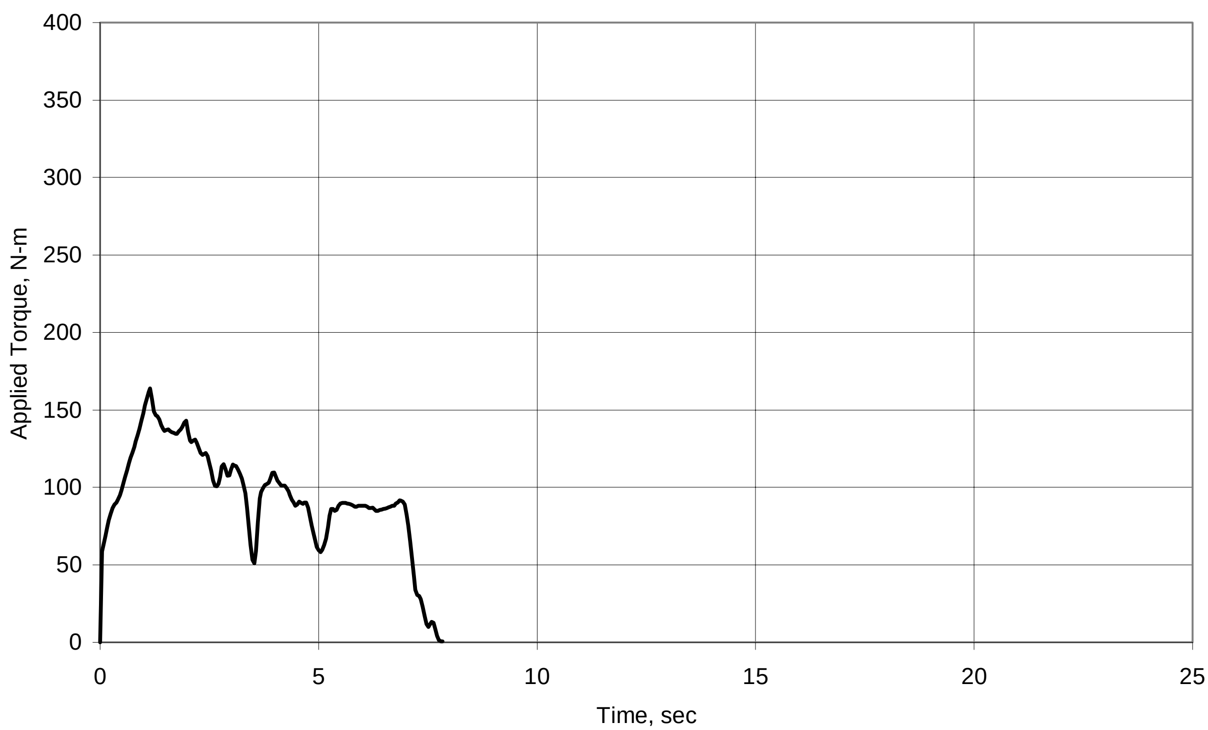

Figure C.6 SPT-T, Boring 3, 1067 min Unstaged, Shelly Clay Layer

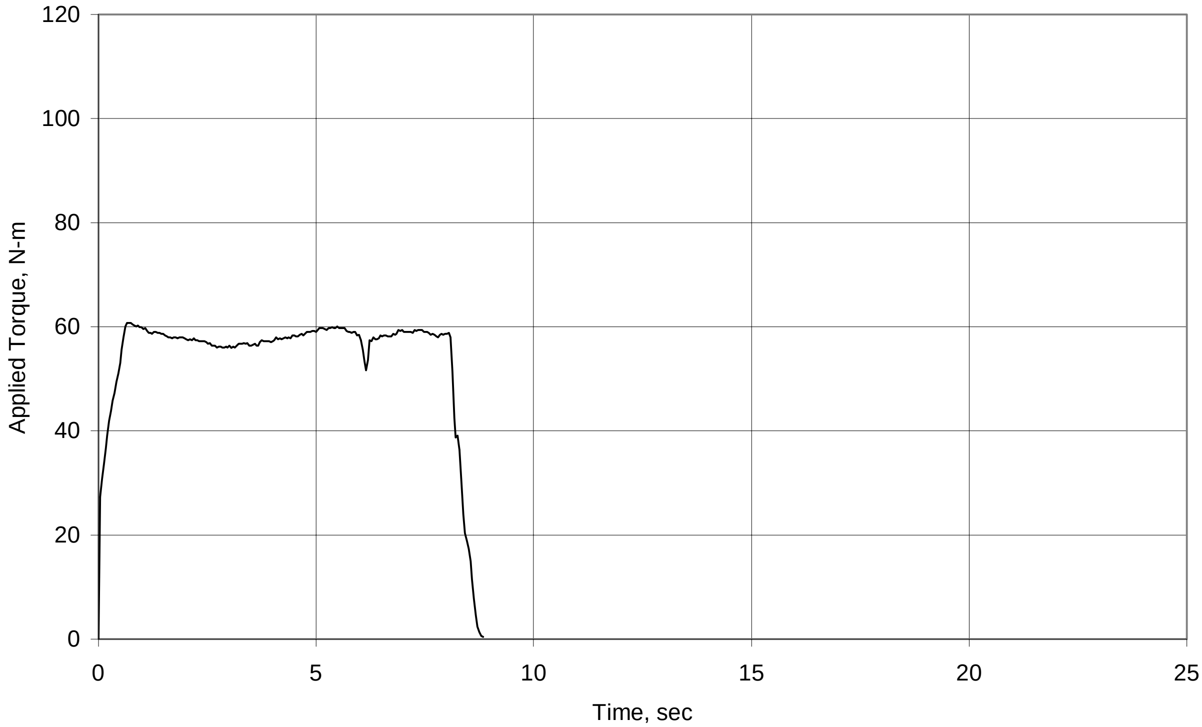

Figure C.7 SPT-T, Boring 4, 180 min Unstaged, Silty Sand Layer

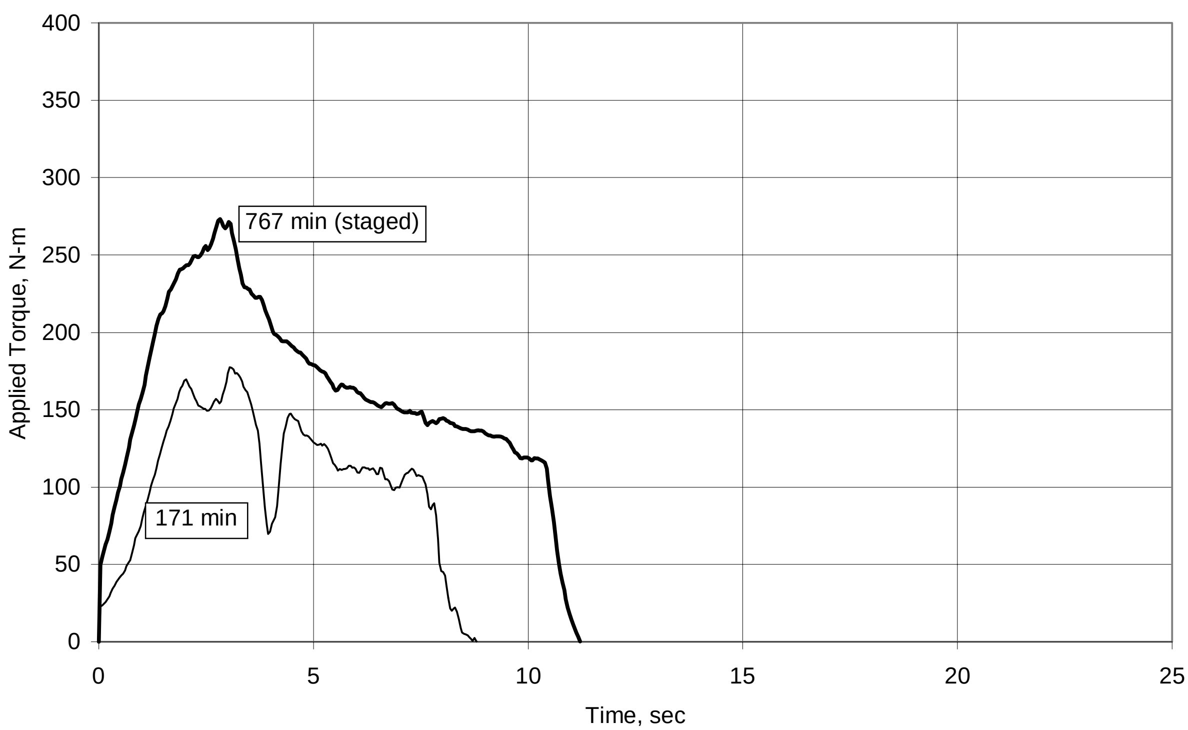

Figure C.8 SPT-T, Boring 4, 171 min Unstaged, Shelly Clay Layer

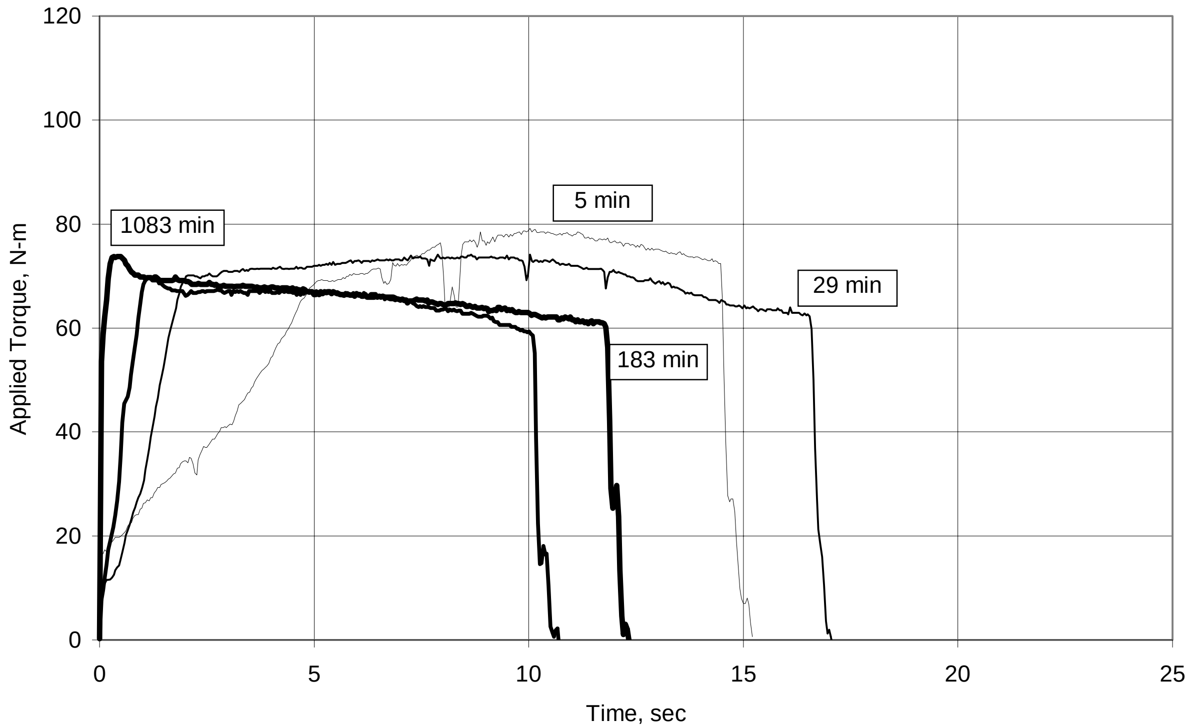

Figure C.9 SPT-T, Boring 5, Staged, Silty Sand Layer

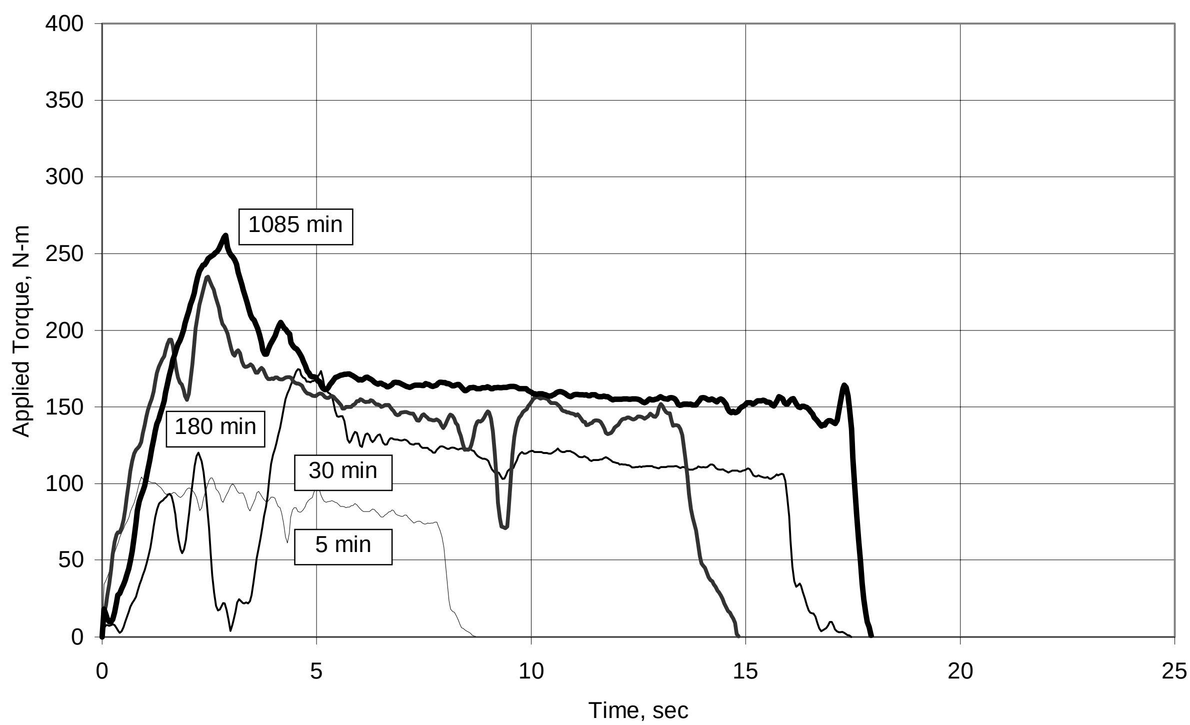

Figure C.10 SPT-T, Boring 5, Staged, Shelly Clay Layer

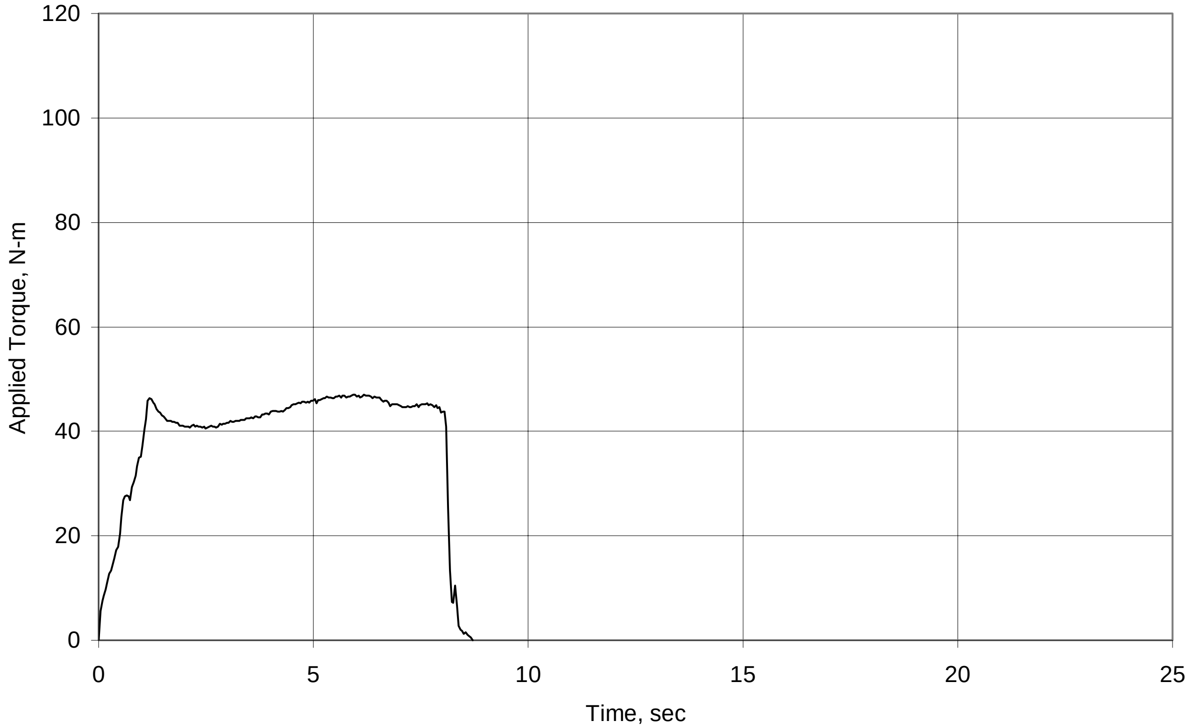

Figure C.11 SPT-T, Boring 6, 30 min Unstaged, Silty Sand Layer

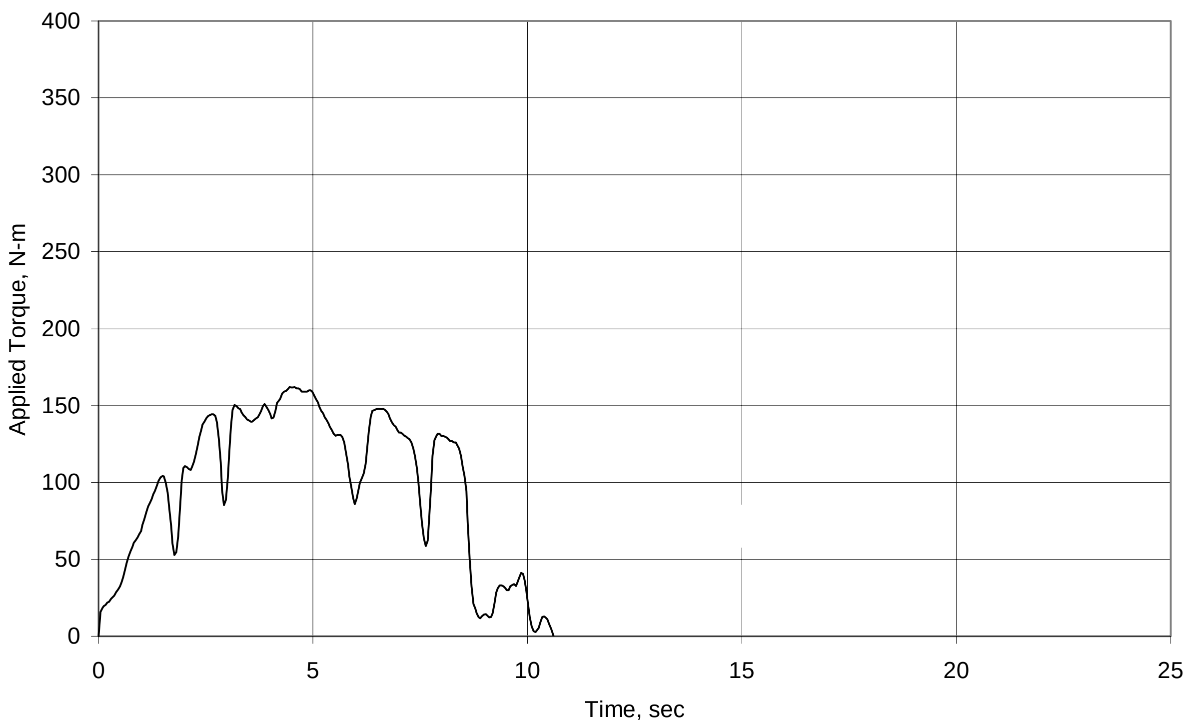

Figure C.12 SPT-T, Boring 6, 30 min Unstaged, Shelly Clay Layer

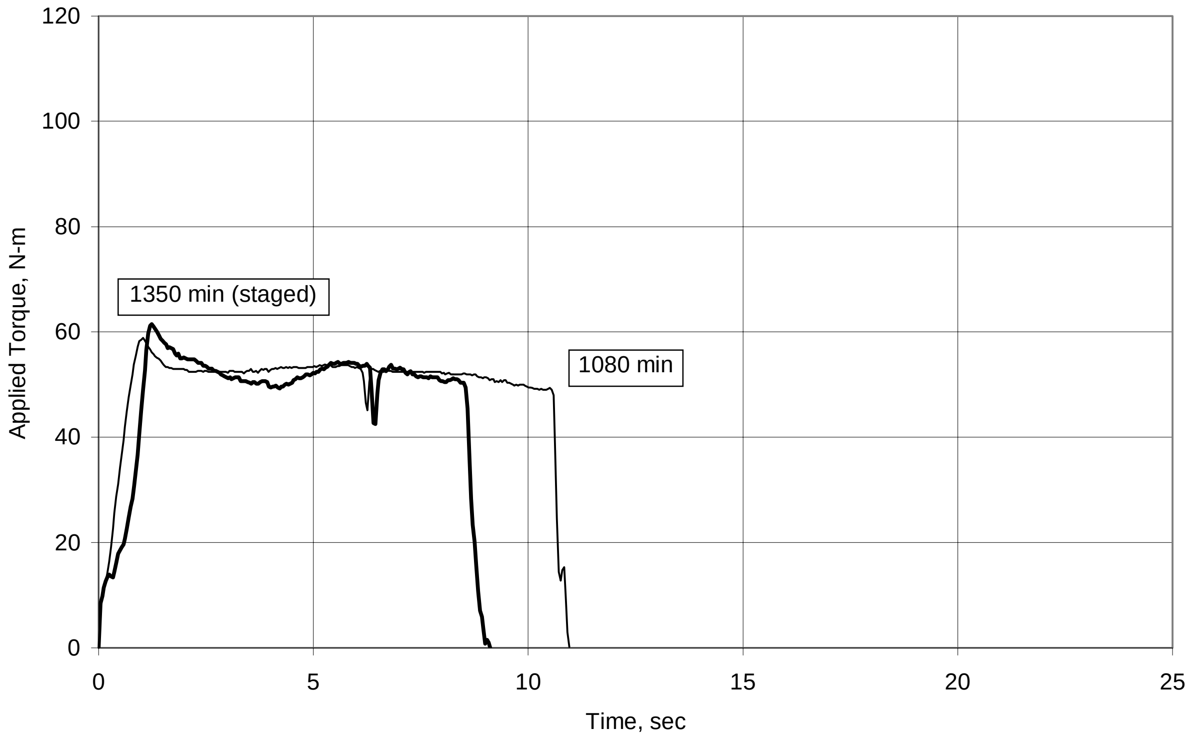

Figure C.13 SPT-T, Boring 7, 1080 min Unstaged , Silty Sand Layer

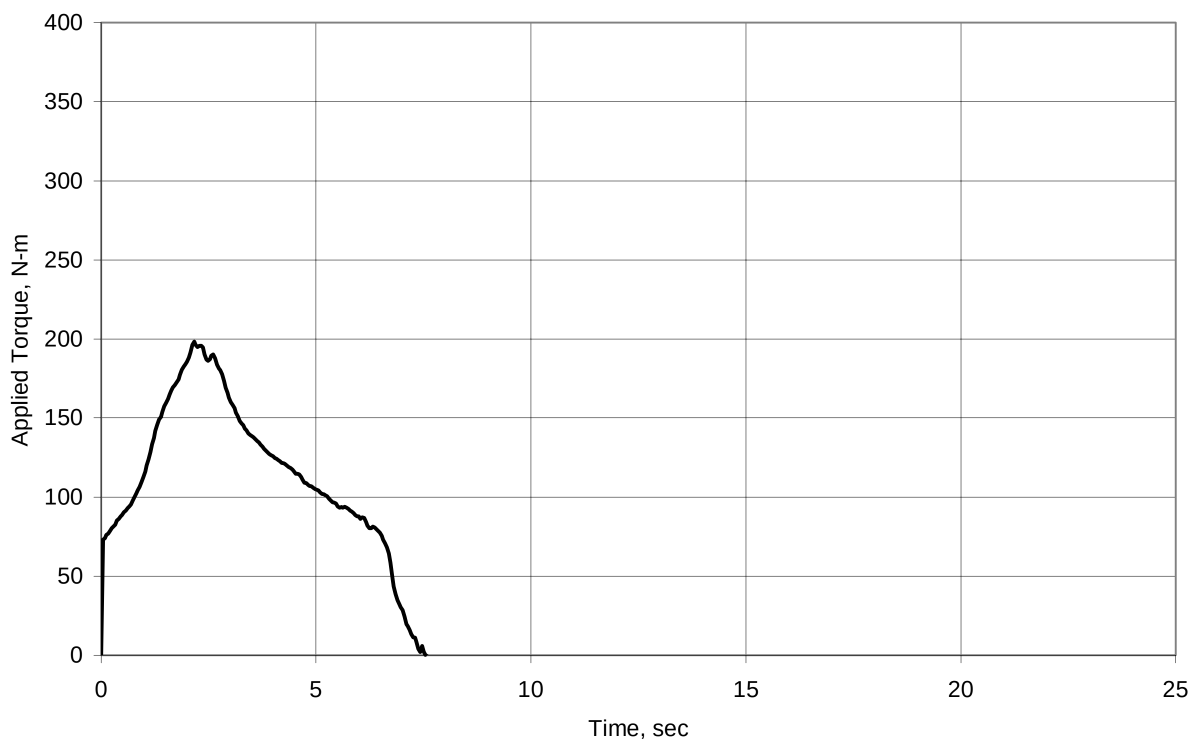

Figure C.14 SPT-T, Boring 7, 980 min Unstaged, Shelly Clay Layer

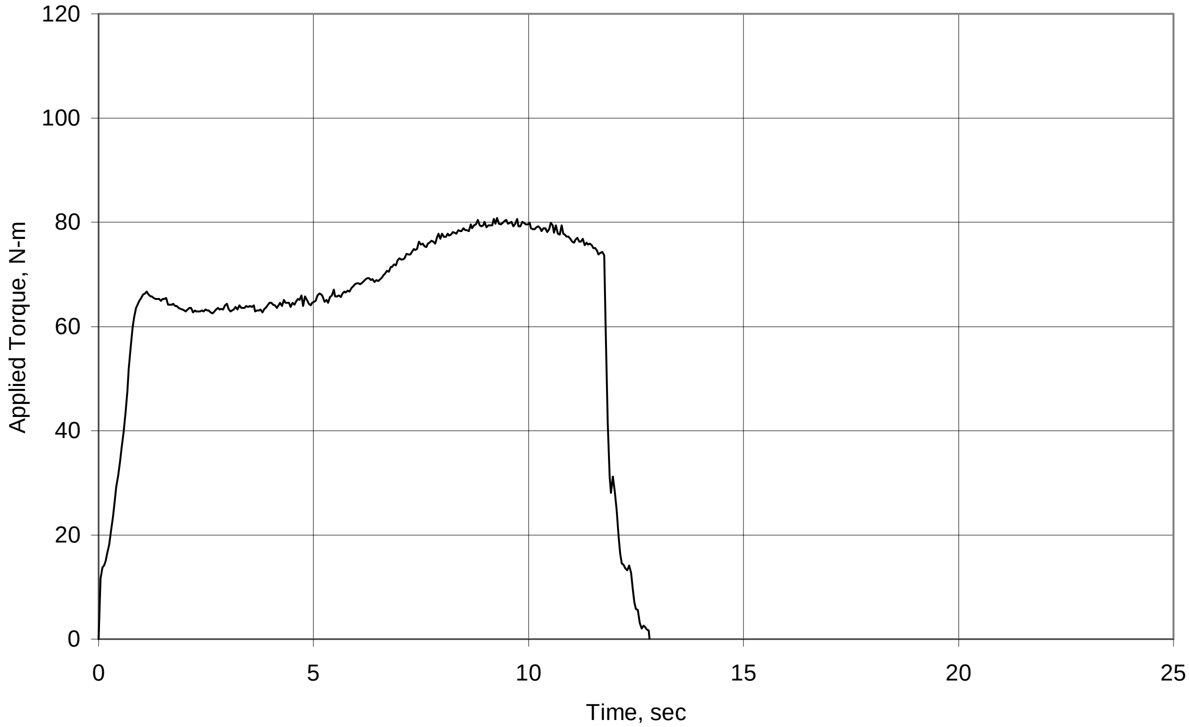

Figure C.15 SPT-T, Boring 8, 179 min Unstaged, Silty Sand Layer

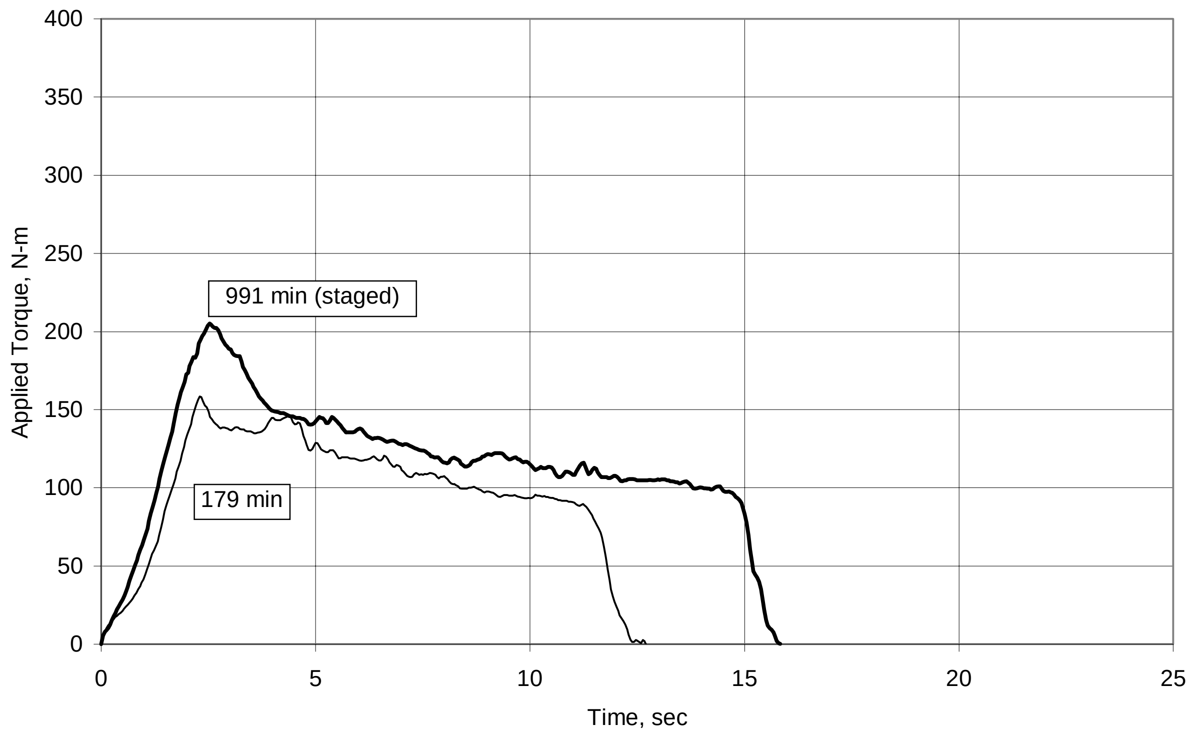

Figure C.16 SPT-T, Boring 8, 179 min Unstaged, Shelly Clay Layer

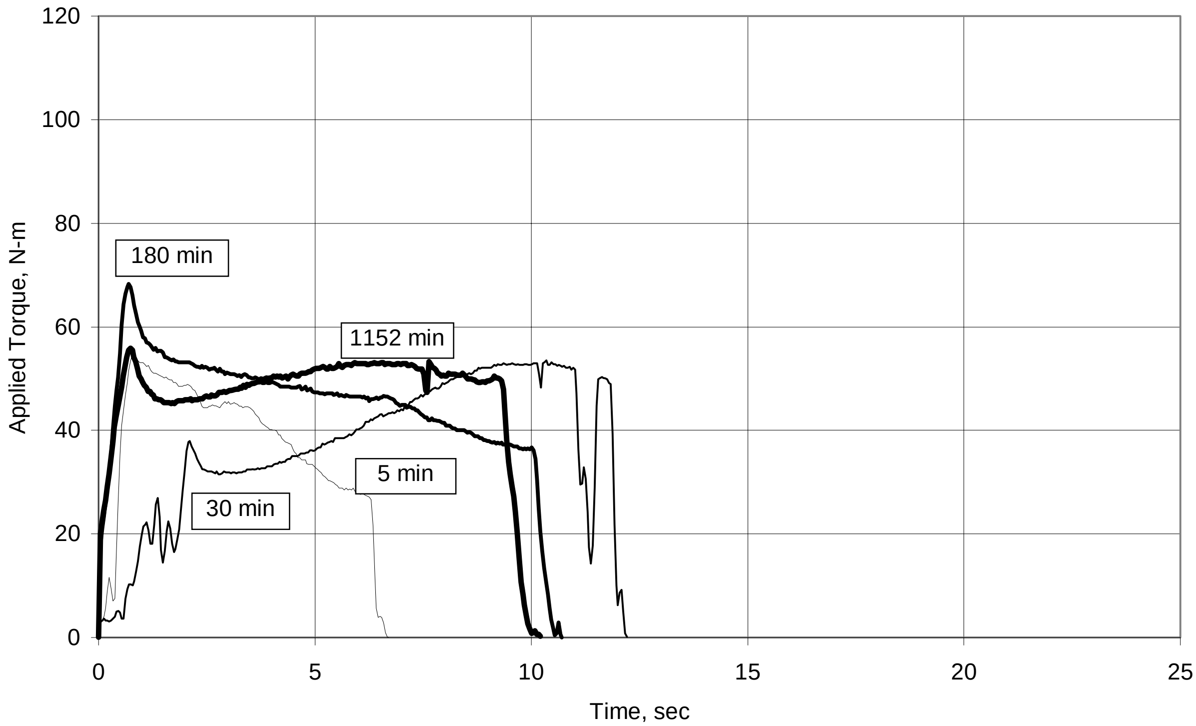

Figure C.17 SPT-T, Boring 9, Staged, Silty Sand Layer

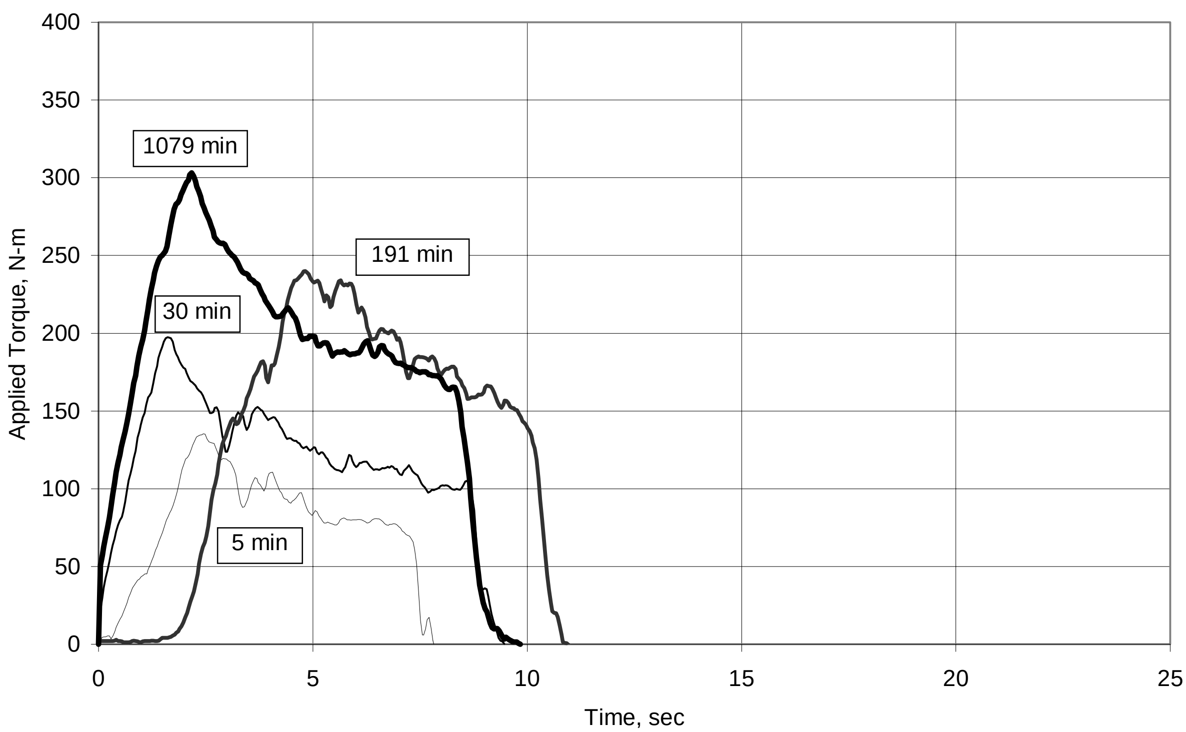

Figure C.18 SPT-T, Boring 9, Staged, Shelly Clay Layer

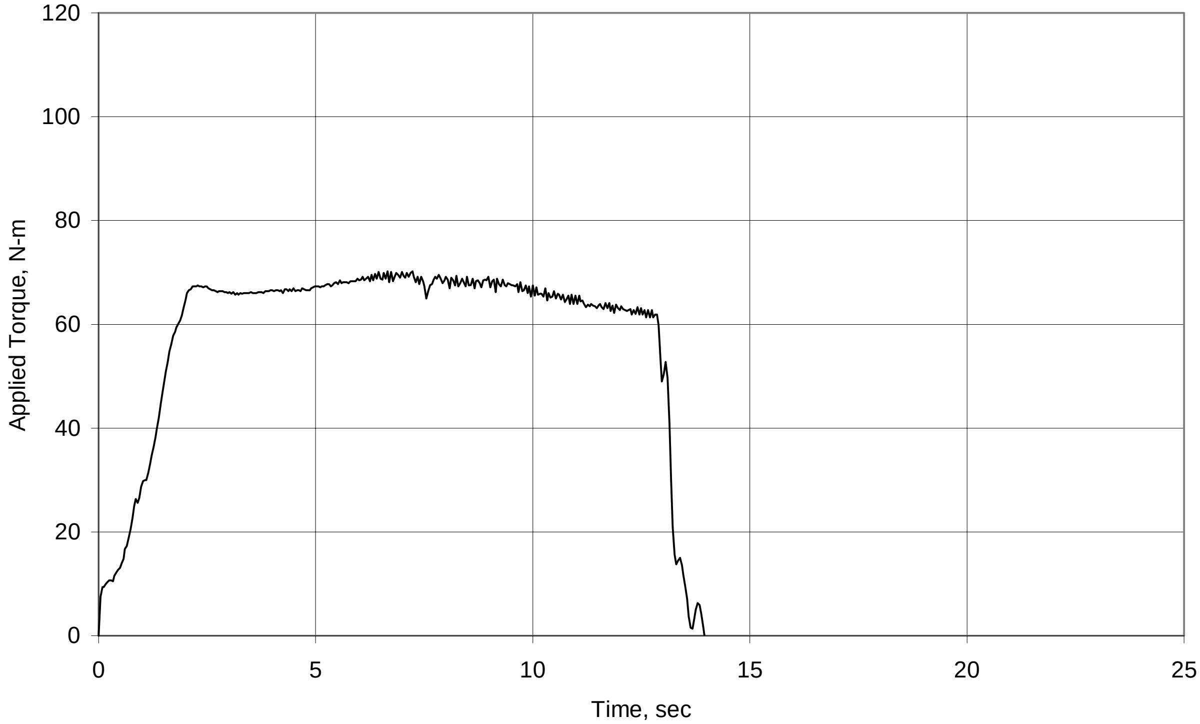

Figure C.19 SPT-T, Boring 10, 30 min Unstaged, Silty Sand Layer

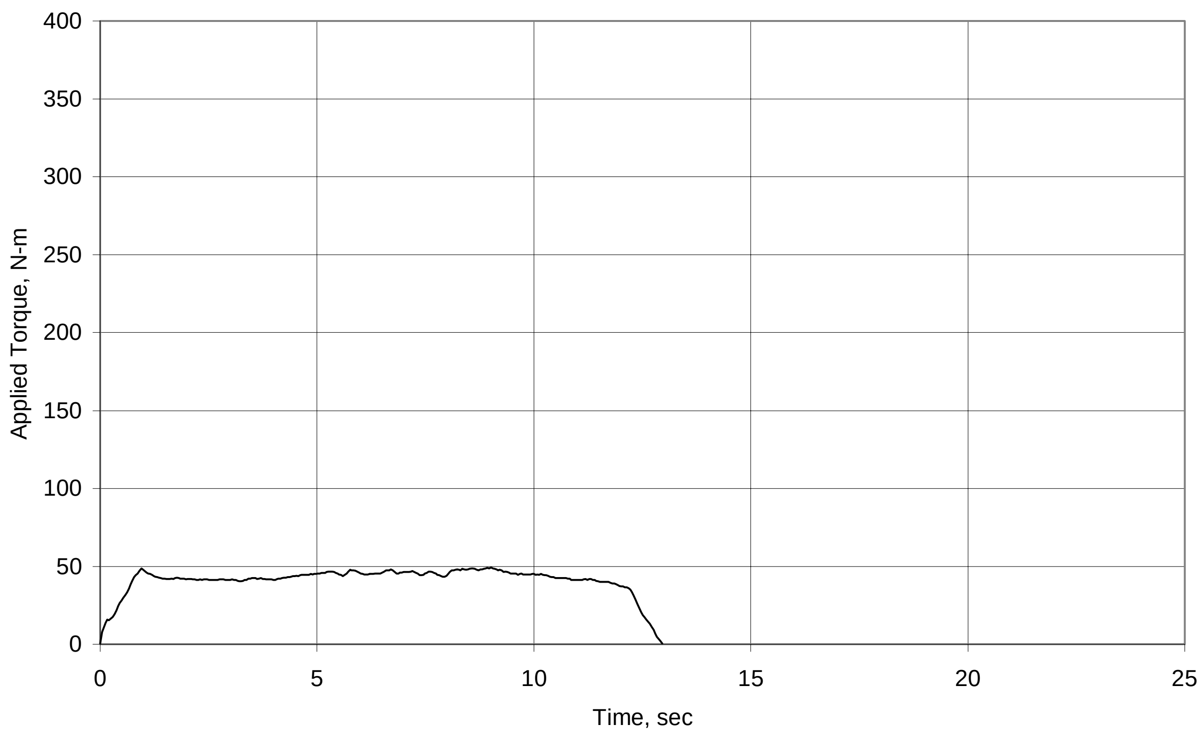

Figure C.20 SPT-T, Boring 10, 30 min Unstaged, Shelly Clay Layer

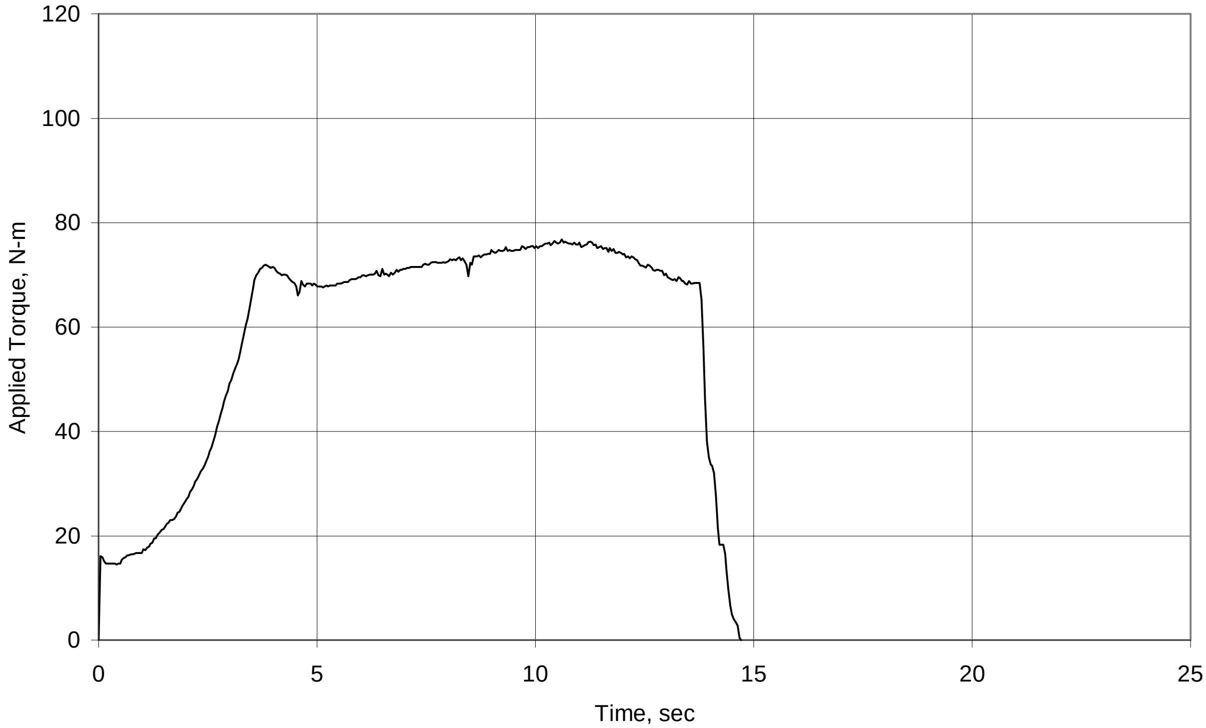

Figure C.21 SPT-T, Boring 11, 1080 min Unstaged, Silty Sand Layer

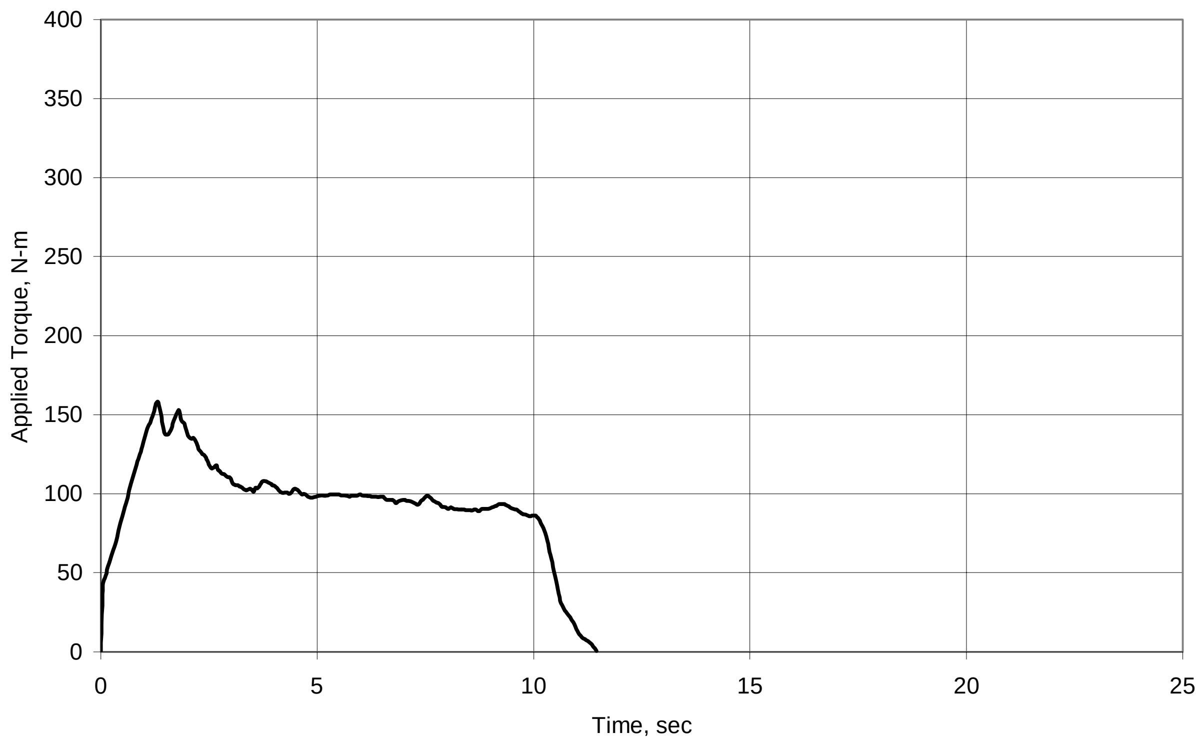

Figure C.22 SPT-T, Boring 11, 1080 min Unstaged, Shelly Clay

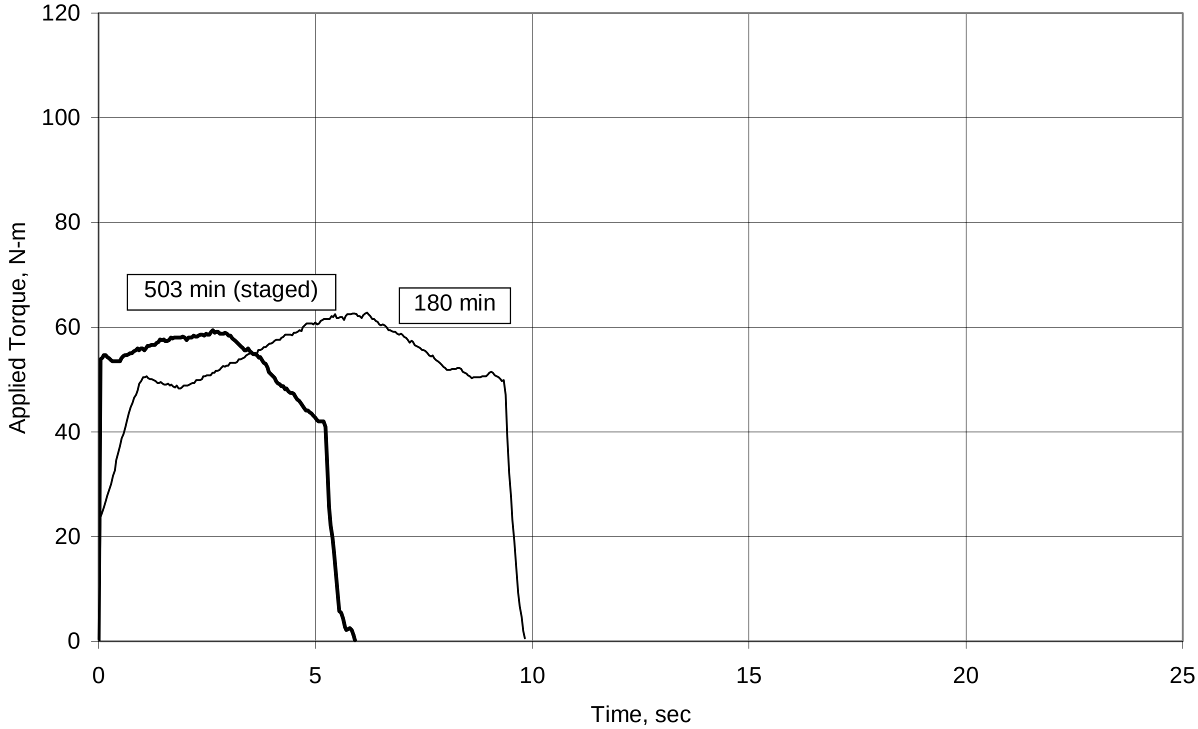

Figure C.23 SPT-T, Boring 12, 180 min Unstaged, Silty Sand Layer

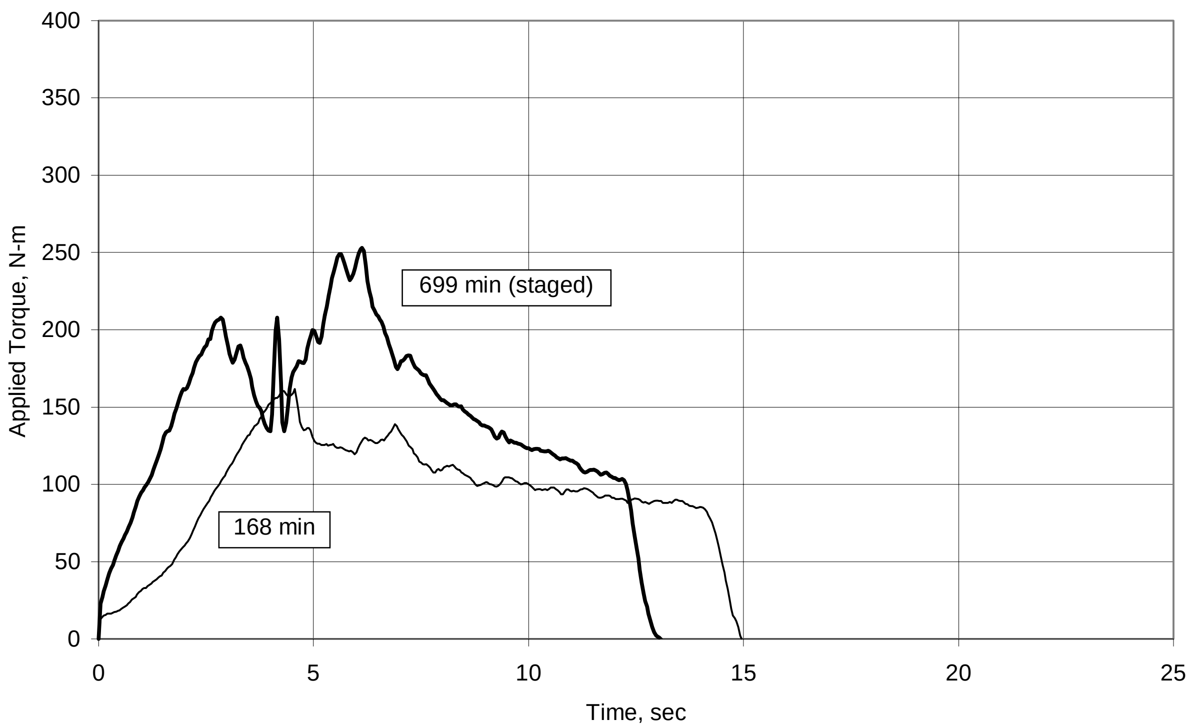

Figure C.24 SPT-T, Boring 12, 168 min Unstaged, Shelly Clay Layer

APPENDIX D

RECOMMENDED PILE SIDE SHEAR DESIGN INCLUDING SETUP

RECOMMENDED PILE SIDE SHEAR DESIGN INCLUDING SETUP

The following design procedure to include pile setup is based on the SPT-T results described in this report and the pile setup research presented in Bullock (1999) and McVay, Schmertmann, Townsend, & Bullock (1999).

1. Assign a pile size, length, and type for preliminary design.
2. Prepare an estimate of ultimate side shear capacity, τ_{est}, for the preliminary design pile using available site information and an approved design method (SPT97 or similar). Use τ for individual layers or the whole pile side shear, and use force or stress units consistently throughout. Then assign a time, t_{est}, associated with the estimated capacity, τ_{est}. (Each design method is calibrated against measured capacities from various field test methods. Static tests normally require about 3-14 days to prepare and perform. Dynamic field tests may be performed at any time, both during initial driving and during restrikes. Use t = 1 minute for dynamic tests at the end of initial driving. Use the actual time elapsed from the end of initial driving for dynamic tests during restrikes.)
3. Assuming a semilog-linear relationship between side shear capacity and log time, calculate the reference side shear capacity, τ_0, at a reference time of t_0 = 1 day using the following equation:

   $$\tau_0 = \frac{\tau_{est}}{1 + A\log(t_{est}/1\,\text{day})}$$ (A = $A_{Unstaged}$ = 0.1 default without tests)

4. Assign a final time, t_f, following pile installation (after EOD) at which the final design side shear capacity will be mobilized. This time may be 3-12 months, or longer, depending on the project. A conservative length of time might be 180 days for a large bridge project. Use the calculated reference side shear τ_0 to calculate the expected ultimate side shear, τ_f, at the final time, t_f:

   $$\tau_f = \tau_0[1 + A\log(t_f/1\,\text{day})]$$ (A = $A_{Unstaged}$ = 0.1 default without tests)

5. Based on the results above, change the pile size, length, or type as desired to optimize the design. Also, apply a safety factor as required.
6. If possible, perform SPT-T borings and/or a design phase test pile program to determine A for use in the above equations. Cohesive soils may have an A value significantly greater than 0.10. Staged dynamic pile tests should be performed at elapsed times in a geometric series, e.g. 15 min, 3 hrs, 36 hrs, etc. Staged SPT-T's should be performed at elapsed times of 5 min, 30min, 3 hrs, 18 hrs, etc. If A is determined from staged tests, then modify it to use an unstaged setup factor in the above equations: $\mathbf{A_{Unstaged} = 0.4\ A_{Staged}}$
7. If possible, confirm the design setup factor A during construction using static tests, dynamic tests, or a combination. Since an accurate allocation of side shear and end bearing is generally difficult to predict from dynamic tests, a conservative calculation of the setup factor uses the whole pile capacity for dynamic tests. This assumes that setup affects both side shear and end bearing and reduces the setup factor.

EXAMPLE SETUP CALCULATIONS

1. Using Default A = 0.1

Calculate the side shear capacity with setup for a 457 mm (18") square prestressed concrete pile, 24 m long (78.7 ft), driven into a 2 layered system, 15.2 m (50 ft) of soft clay (N = 4) over dense sand (N = 30). SPT97 provides ultimate $\boldsymbol{\tau_{clay}}$ = 152 kN (17.1 tons) and $\boldsymbol{\tau_{sand}}$ = 814 kN (91.5 tons). Assume that $\mathbf{t_{est}}$ = 7 days for the SPT97 design method and that pile capacity will be mobilized one year after installation.

Clay:
$$\tau_0 = \frac{\tau_{est}}{1 + A\log(t_{est}/1\,day)} = \frac{152\,kN}{1 + 0.1\log(7\,days/1\,day)} = 140\,kN$$
$$\tau_f = \tau_0[1 + A\log(t_f/1\,day)]$$
$$= 140\,kN[1 + 0.1\log(365\,days/1\,day)] = 176\,kN\ \ (19.8\,tons)$$

Sand:
$$\tau_0 = \frac{\tau_{est}}{1 + A\log(t_{est}/1\,day)} = \frac{814\,kN}{1 + 0.1\log(7\,days/1\,day)} = 751\,kN$$
$$\tau_f = \tau_0[1 + A\log(t_f/1\,day)]$$
$$= 751\,kN[1 + 0.1\log(365\,days/1\,day)] = 943\,kN\ \ (106\,tons)$$

Total: $\boldsymbol{\tau}$ = 1119 kN w/ setup vs. 966 kN w/o setup (125.8 vs. 108.6 tons), *16% Increase*

2. Same as 1. with SPT-T Measurements

Staged SPT-T measurements indicate A = 0.65 in the clay layer, but no tests performed in the sand layer (use A = 0.1).

Clay: $A_{Unstaged} = 0.4\ A_{Staged} = 0.4\ (0.65) = 0.26$
$$\tau_0 = \frac{\tau_{est}}{1 + A\log(t_{est}/1\,day)} = \frac{152\,kN}{1 + 0.26\log(7\,days/1\,day)} = 125\,kN$$
$$\tau_f = \tau_0[1 + A\log(t_f/1\,day)]$$
$$= 125\,kN[1 + 0.26\log(365\,days/1\,day)] = 208\,kN\ \ (23.4\,tons)$$

Sand:
$$\tau_0 = \frac{\tau_{est}}{1 + A\log(t_{est}/1\,day)} = \frac{814\,kN}{1 + 0.1\log(7\,days/1\,day)} = 751\,kN$$
$$\tau_f = \tau_0[1 + A\log(t_f/1\,day)]$$
$$= 751\,kN[1 + 0.1\log(365\,days/1\,day)] = 943\,kN\ \ (106\,tons)$$

Total: $\boldsymbol{\tau}$ = 1152 kN w/ setup vs. 966 kN w/o setup (129.5 vs. 108.6 tons), *19% Increase*

3. <u>Same as 1. with Dynamic and Static Test Pile Measurements</u>

Staged test pile measurements indicate A = 0.65 in the clay layer, and A = 0.40 in the sand layer.

Clay: $A_{Unstaged} = 0.4\ A_{Staged} = 0.4\ (0.65) = 0.26$

$$\tau_0 = \frac{\tau_{est}}{1 + A\log(t_{est}/1\,day)} = \frac{152\,kN}{1 + 0.26\log(7\,days/1\,day)} = 125\,kN$$

$$\tau_f = \tau_0[1 + A\log(t_f/1\,day)]$$

$$= 125\,kN[1 + 0.26\log(365\,days/1\,day)] = 208\,kN\ \ (23.4\,tons)$$

Sand: $A_{Unstaged} = 0.4\ A_{Staged} = 0.4\ (0.40) = 0.16$

$$\tau_0 = \frac{\tau_{est}}{1 + A\log(t_{est}/1\,day)} = \frac{814\,kN}{1 + 0.16\log(7\,days/1\,day)} = 717\,kN$$

$$\tau_f = \tau_0[1 + A\log(t_f/1\,day)]$$

$$= 717\,kN[1 + 0.16\log(365\,days/1\,day)] = 1011kN\ \ (113.6\,tons)$$

Total: $\boldsymbol{\tau}$ = 1219 kN w/ setup vs. 966 kN w/o setup (137.1 vs. 108.6 tons), *26% Increase*

Note that the above example calculations do not include end bearing, which is calculated separately and generally not assumed to exhibit setup. Since the design pile capacity generally includes both end bearing and side shear, the percent increase of the total pile capacity will be less than the percent increase in side shear due to setup. The side shear, including setup, should also be reduced by a safety factor, generally FS = 2.0 for SPT97 analysis.