Side Shear Setup. II: Results From Florida Test Piles

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Abstract: This paper analyzes the results of a driven pile side shear setup (SSS) test program performed at the University of Florida and described in a companion paper. All pile segments showed SSS, with similar magnitudes in both cohesionless and cohesive soils, and irrespective of depth. Horizontal effective stresses increased after driving, but then stabilized as induced pore pressures dissipated. SSS continued long after pore pressure dissipation due to other aging effects. Pile load tests in all soil types, and standard penetration tests with torque measurement (SPT-T) in cohesive soils, confirmed the approximate semilog-linear time versus SSS behavior and the usefulness of the setup factor A. Staged (repeated) tests significantly increased SPT-T measurements, suggesting a reduction factor of 0.4 for SSS results measured by staged testing. We believe that engineers can now include SSS in routine design, using a minimum setup factor $A=0.1$ for soils similar to those tested, or higher if supported by dynamic or static pile tests, or by SPT-T predictor tests. Examples demonstrate a proposed SSS design method.

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Introduction

This paper presents detailed results from the University of Florida (UF) pile side shear setup (SSS) test program described in the companion paper (Bullock et al. 2005). It also discusses factors affecting SSS, presents a SSS predictor test, and proposes a design procedure for the practical use of SSS (examples in the Appendix).

Expanded Analysis of University of Florida Side Shear Setup Tests

Pile Segment Side Shear Setup

The UF test results in the companion paper (Bullock et al. 2005) showed that the whole-pile side shear capacity, measured by staged (repeated) tests, increased approximately linearly with the logarithm of elapsed time after the end of driving (EOD). The unit side shear calculated for individual pile segments during these tests also followed a semilog-linear trend with time, resulting in segment setup factors, A_i, like those shown in Fig. 1 and Table 1 for the Aucilla River test site (AUC) pile (using least-squares regression and a reference time, $t_0=1$ day). Table 2 provides the A_i setup factors all 28 pile segments, all but one of which showed approximately semilog-linear SSS behavior, irrespective of soil type.

Horizontal Effective Stresses

To investigate the relationship of SSS to horizontal effective stress, 18 of the UF research pile segments also included measurement of the installation-induced pore pressures and total horizontal stresses. Fig. 2 shows the pore pressures and effective horizontal stress measured at the AUC pile. Pore pressures in the sands generally dissipated prior to the first static test. The stiff cohesive soils required less than 1 week for dissipation while the weak clays at the Vilano Bridge West test site (VLW) required 19 to 157 days. In most cases, the horizontal effective stress increased during the pore pressure dissipation and tended to stabilize with complete dissipation. A few of the cohesive soils continued to increase in both total and effective stress after the pore pressures dissipated.

Fig. 3 shows the linear relationship expected between stress (σ_h') and unit side shear strength (f_s) following the EOD. However, after the completion of pore pressure dissipation, f_s continued to increase with little or no change in σ_h' (three examples circled in Fig. 3). The initial trend of the data in Fig. 3 also goes approximately through the origin, suggesting a negligible adhesion component as expected for remolded soil. Fig. 4 shows that the ratio (f_s/σ_h') changes on average from the initial to the final tests. Ignoring adhesion, the pile-soil effective friction angle δ' ($\tan\delta'=f_s/\sigma_h'$) increases from 20.3° to 27.4° (see Fig. 4), reasonable values compared to published literature. This increase of 40% in $\tan\delta'$ may indicate a structural change or "aging" phe-

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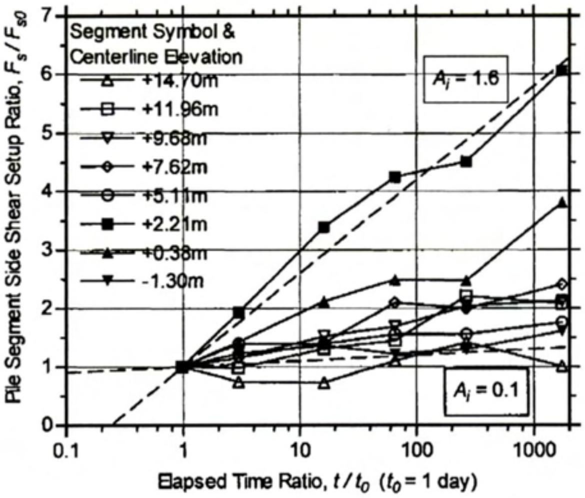

Fig. 1. Aucilla River test site pile segment side shear setup ratios

nomenon in the soil immediately adjacent to the pile. Within the scatter of the data, it appears that all soil types show similar side shear versus effective stress behavior.

Postdissipation Side Shear Behavior

Although the slope of the unit side shear versus the logarithm of time remains relatively constant throughout all of the tests, the pre- and postdissipation relationship between side shear and horizontal effective stress appears to change. Summarized in Table 2, much less effective stress change occurs during the postdissipation increase in side shear. Using the average Δf_s and average $\Delta\sigma_h'$ the ratio $\Delta f_s/\Delta\sigma_h'$(=tan δ) increases from 0.47 (=tan 25°) during dissipation to 1.30 (=tan 53°) after dissipation. Again, these data suggest no obvious trend with soil type, and that much of the postdissipation SSS occurs as a result of something other than increases in horizontal effective stress.

Setup Factor A_i Versus Depth and Soil Type

Fig. 5 presents a depth profile of the pile segment A_i factors from Table 2, essentially all of which come from pile segments below the ground water surface. (Though not shown, postdissipation-only A_i values provide a very similar depth profile.) Disturbance during the tests likely caused the lowest A_i- values (0.04, 0.08, and −0.07), which occurred in sandy soils near the ground surface or at the pile bottom adjacent to the expanding Osterberg cell (O-cell). Based on these data, the A_i factor appears independent of depth over the depth range of 25 m (82 ft). As shown by the A_i value averages for clays, sands, and mixed soils at the bottom of Table 2, at least for the soils investigated in this research, A_i does not seem to depend significantly on soil type. Fig. 5 seems to suggest that in soils to those tested, A_i=0.2 provides a reasonable lower bound for pile segment SSS design. Note that the average A_i=0.51 for all 28 pile segments with a median A_i=0.38.

Summary of University of Florida Test Pile Results

The UF research program confirms the approximate semilog-linear time relationship of SSS and extends it to instrumented pile segments as well as the entire pile. Short-term dynamic tests and long-term static tests produced similar SSS behavior (see companion paper—Bullock et al. 2005), apparently with no significant change before and after the dissipation of excess pore pressure. The measured side shear and horizontal effective stresses seemed reasonable, with negligible adhesion at the pile/soil interface and an increase in the interface friction coefficient (tan δ) of 40% during SSS. Much of the postdissipation SSS occurred without significant increase in horizontal effective stress, suggesting a soil-structure change near the pile. All depths had about the same range of pile segment A_i values, with a minimum A_i=0.2 and with no apparent depth dependency. These findings apply to all of the soil types tested, ranging from plastic (plasticity index ≤60%) clays to shelly sands and simplified herein into three categories (Clay "C", Mixed "M," and Sand "S").

Side Shear Setup Predictor Tests

This research also investigated the possibility of using in situ tests to predict SSS factors for design, including staged torque measurements performed on the driven standard penetration test (SPT) sampler, staged measurements of the cone penetrometer sleeve side shear, and staged Marchetti Dilatometer thrust measurements using a load cell immediately above the blade. In sand, all of these tests produced unsuitable negative A values during the necessarily short test duration (<24 h), similar to short-term static test results from the piles. For cohesive soils (C and M), the SPT with torque measurement (SPT-T) proved the most practical.

DeCourt (1989) describes the SPT-T, a simple test that uses a torque wrench at the top of the drill rod string to measure the torsional side shear resistance of the soil against the driven sampler (a 51-mm-diameter model pile driven 457 mm). Rausche et al. (1996) also investigated the use of modified SPT procedures for the prediction of the soil damping and quake parameters needed to analyze dynamic pile tests. Their work at three Florida sites, including AUC and VLW, provided uplift (tension) tests after driving the SPT sampler, and repeated uplift tests (staged) at later times. They concluded that the uplift side shear on the SPT sampler increased linearly with the log of elapsed time. Using t_0=5 min for the Rausche et al. (1996) uplift tests yields the same approximate range of A=0.20 to 0.80 found for driven displacement piles using t_0=1 day.

The UF SSS research included seven-staged series of SPT-Ts in cohesive soils, using rotary methods, drilling mud, and casing in two borings at the VLW site. These tests provided torque measurements of the soil side shear against the SPT sampler at initial times of 4 to 5 min after the EOD, at 60 min, and at final times from 214 to 1,088 min. The SPT-T results showed semilog-linear time versus SSS behavior, similar to the adjacent pile segments. Using the initial SPT-T as a reference (t_0=4 to 5 min), the average value of SPT-T A=0.43 measured over the depth range of 7.6 to 16.2 m (25 ft to 53 ft) compared conservatively with the average of A_i=0.52 from the four adjacent pile segments. Including three additional staged values of SPT-T A obtained in clay by Bullock and Schmertmann (2003) during a related research study at the Seabreeze Bridge (SBZ) site, the overall bias factor (test pile A_i/SPT A) =1.10. Based on these results, we believe that the SPT-T provided an acceptable SSS predictor test for the cohesive soils in the UF research.

Factors Affecting Side Shear Setup Results

Regional Data

This research used only one type and one size of displacement pile, although full-scale and driven with real hammers in field conditions. The instrumentation provided only 28 pile segments

Table 1. University of Florida O-Cell Test Results for Aucilla Pile Segments

Segment C.L. elev., (depth), length, L, f_{s0} at 1 day, (soil type)	Load test No.	Time after end of driving t (days)	Load No.	Pile segment averages: Side shear f_s (kPa)	f_s/f_{s0}	Axial mvmt. (mm)	Effective stress (kPa)
+14.70 m,	1	0.98	L23	8.6	1.00	16.15	
(1.52 m)	2	2.97	L24B	6.4	0.75	17.25	
L=3.04 m	3	16.1	L17	6.3	0.73	11.01	
f_{s0}=8.6 kPa	4	65.1	2L19A	9.5	1.10	9.73	
(M)	5	265	L20	12.3	1.42	9.71	
	6	1727	L19	8.7	1.01	2.48	
+11.96 m,	1	0.98	L23	22.2	1.00	16.18	51.6
(4.26 m)	2	2.97	L24A	21.9	0.98	17.33	55.0
L=2.44 m	3	16.1	L17	29.2	1.31	11.04	65.9
f_{s0}=22.2 kPa	4	65.1	2L19A	32.3	1.45	9.76	96.0
(C)	5	265	L20	49.2	2.21	9.75	124.7
	6	1727	L22	46.3	2.08	11.65	163.8
+9.68 m,	1	0.98	L21	17.9	1.00	1.73	
(6.54 m)	2	2.97	L21	20.8	1.16	1.49	
L=2.13 m	3	16.1	L16	27.3	1.52	3.36	
f_{s0}=17.9 kPa	4	65.1	2L17	30.3	1.69	1.94	
(M)	5	265	L19	36.7	2.04	4.26	
	6	1727	L19	38.2	2.13	2.60	
+7.62 m,	1	0.98	L20	9.9	1.00	1.40	13.6
(8.60 m)	2	2.97	L20	11.5	1.16	1.16	12.7
f_{s0}=9.9 kPa	3	16.1	L15	14.0	1.41	1.40	13.3
L=1.98 m	4	65.1	2L18	20.9	2.10	3.97	22.3
(C)	5	265	L18	19.9	2.00	1.71	20.5
	6	1727	L20	24.0	2.41	5.12	
+5.11 m,	1	0.98	L22	34.2	1.00	4.83	114.4
(11.11 m)	2	2.97	L23	47.7	1.40	16.28	164.7
L=3.05 m	3	16.1	L16	47.4	1.39	3.63	154.4
f_{s0}=34.2 kPa	4	65.1	2L17	53.3	1.56	2.23	158.5
(C)	5	265	L19	53.4	1.56	4.63	
	6	1727	L19	60.2	1.76	2.94	
+2.21 m,	1	0.98	L22	12.7	1.00	5.03	78.6
(14.01 m)	2	2.97	L24A	24.5	1.93	17.83	77.8
L=2.74 m	3	16.1	L17	43.0	3.39	11.65	99.6
f_{s0}=12.7 kPa	4	65.10	2L18	53.9	4.25	4.47	96.3
(M)	5	265	L20	57.4	4.52	10.60	125.0
	6	1727	L22	77.0	6.06	12.47	118.9
+0.38 m,	1	0.98	L22	21.3	1.00	5.18	
(15.84 m)	2	2.97	L23	30.5	1.43	16.72	
L=0.91	3	16.1	L17	45.2	2.12	11.88	
f_{s0}=21.3 kPa	4	65.1	2L19	53.0	2.49	8.04	
(S)	5	265	L19	52.9	2.48	5.29	
	6	1727	L20	81.2	3.81	6.03	
−1.30 m,	1	0.98	L22	84.2	1.00	5.36	125.5
(17.52 m)	2	2.97	L24A	103.6	1.23	18.25	130.7
L=2.44 m	3	16.1	L17	114.3	1.36	12.18	126.1
f_{s0}=84.2 kPa	4	65.1	2L19	102.9	1.22	8.36	120.2
(M)	5	265	L20	109.5	1.30	11.24	126.5
	6	1727	L22	136.0	1.61	13.19	142.6

Notes: Unit side shear is maximum measured during each load test and has been reduced by segment weight (also t_0=1 day).

Table 2. University of Florida Side Shear versus Effective Stress

Test site	Segment		Elapsed time after end of driving (days)			Horizontal effective stress (kPa)			Segment side shear (kPa)			Change During Diss.		Change postdiss.		Setup factor t_0=1 day	
(No. of tests)	Depth (m)	Soil type	Initial t_i	Diss. t_d	Final t_t	Initial σ'_{hi}	Diss. σ'_{hd}	Final σ'_{hf}	Initial f_{si}	Diss. f_{sd}	Final f_{sf}	$\Delta\sigma'_h$ (kPa)	Δf_s (kPa)	$\Delta\sigma'_h$ (kPa)	Δf_s (kPa)	Whole pile A	segment A_i
BKM	0.92	S	0.25	0.25	277				5.6	5.6	12.6				7.0		0.33
(5)	2.75	S	0.25	0.25	277				4.5	4.5	21.2				16.7		1.03
	4.58	S	0.25	0.25	277	4.9	4.9	2.6	0.1	0.1	5.5			−2.3	5.4	0.174	1.48
	6.26	S	0.25	0.25	277	8.3	8.3	15.8	0.7	0.7	4.8			7.5	4.1		0.97
	7.86	S	0.25	0.25	277	11.0	11.0	8.9	3.7	3.7	11.3			−2.1	7.6		0.85
AUC	1.52	M	0.98		265				8.6		12.3						0.04
(6)	4.26	C	0.98	2.97	1727	51.6	55.0	163.8	22.2	21.9	46.3	3.4	−0.3	108.8	24.4		0.36
	6.54	M	0.98		1727				17.9		38.2						0.38
	8.60	C	0.98	2.97	265	13.6	12.7	20.5	9.9	11.5	19.9	−0.9	1.6	7.8	8.4	0.321	0.45
	11.11	C	0.98	2.97	65.1	114	164.7	158.5	34.2	47.7	53.3	53.0	13.5	−6.2	5.6		0.26
	14.01	M	0.98	2.97	1727	78.6	77.8	118.9	12.7	24.5	77.0	−0.8	11.8	41.1	52.5		1.60
	15.84	S	0.98	0.98	1727				21.3	21.3	81.2				59.9		0.79
	17.52	M	0.98	2.97	1727	126	130.7	142.6	84.2	103.6	136.0	5.2	19.4	11.9	32.4		0.17
VLE	0.92	M	0.32		15.9				0.9		2.6						0.58
(3)	2.53	S	0.32	0.32	15.9				8.6		17.3						0.36
	4.13	S	0.32	0.32	15.9	10.0	10.0	33.3	9.7	9.7	15.2			23.3	5.5	0.118	0.28
	6.03	S	0.32	0.32	15.9	52.3	52.3	55.6	31.7	31.7	55.8			3.3	24.1		0.38
	8.62	S	0.32	0.32	15.9	69.6	69.6	334.4	124.6	124.6	183.7			IA	IA		0.25
VLW	7.43	M	0.26	0.26	157	30.7	30.7	46.9	3.3	3.3	9.6			16.2	6.3		0.53
(4)	11.09	M	0.26		157				17.6		20.6						0.20
	13.68	C	0.26	157	157	67.7	45.9	45.9	11.3	27.4	27.4	−21.8	16.1			0.292	0.38
	16.27	C	0.26	18.98	157	84.2	137.7	112.5	14.8	35.9	58.2	53.5	21.1	−25.2	22.3		0.99
	17.72	S	0.26	0.26	157				105.5	105.5	122.5				17.0		0.08
SBZ	9.66	S	0.35	0.35	1058	29.5	29.5	47.7	12.5	12.5	37.0			18.2	24.5		0.47
(6)	14.53	S	0.35	0.35	69.9	46.4	46.4	49.2	15.3	15.3	18.5			2.8	3.2		0.21
	17.81	C	0.35	4.03	1058	81.2	164.6	160.0	29.1	29.5	86.2	83.4	0.4	−4.6	56.7	0.165	0.67
	21.16	C	0.35	4.03	1058	133	226.6	232.4	44.2	86.0	114.5	93.7	41.8	5.8	28.5		0.30
	23.60	S	0.35	0.35	1058				170.4	170.4	111.5				−58.9		−0.07

Note: Diss.=Essentially complete dissipation of excess pore pressure; Blanks indicate no measurement; IA = Instrument anomaly; Choice of final readings affected by availability of effective stress measurements; Segment Setup Factors: (from linear regression) Clay— A_i=0.49 (average of 7, range 0.26 to 0.99, coefficient of variation =53.1%); Mixed— A_i=0.50 (average of 7, range 0.04 to 1.60, coefficient of variation =104.6%); Sand— A_i=0.53 (average of 14, range −0.07 to 1.48, coefficient of variation =81.5%); BKM = Buckman Bridge test site; AUC = Aucilla River test site; VLE = Vilano Bridge East test site; VLW = Vilano Bridge West test site; SBZ = Seabreeze Bridge test site.

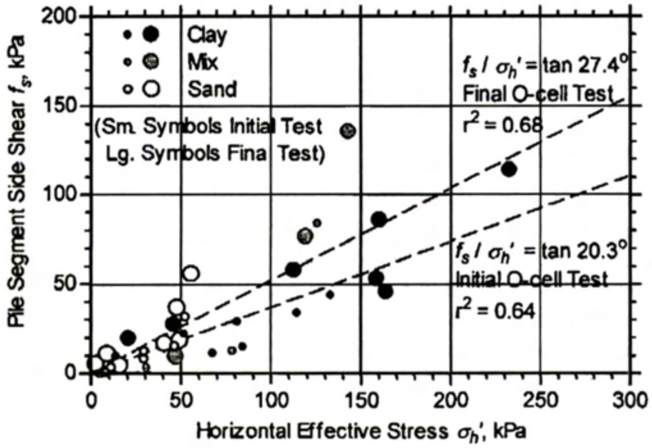

Fig. 4. Initial and final segment side shear (segments with σ_h')

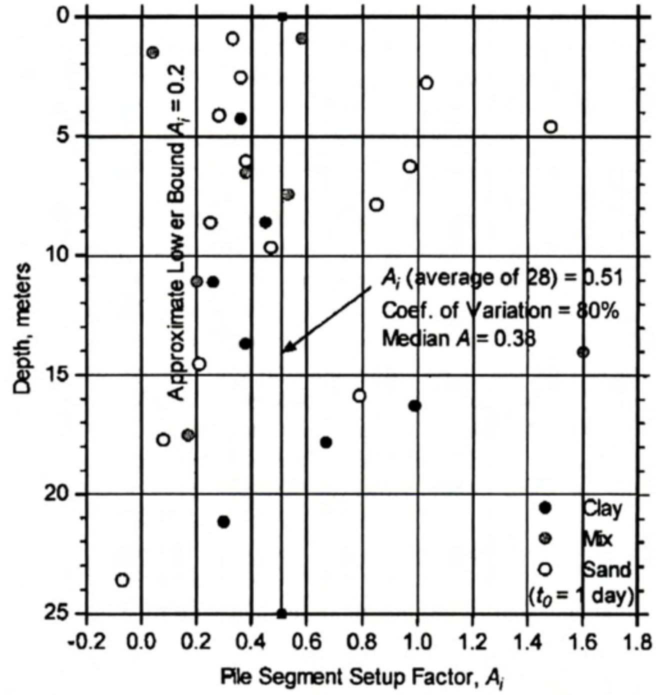

Fig. 5. Setup factor variation with depth (all pile segments)

for the analysis of SSS, 18 of which included horizontal effective stress measurements. The soils included only Florida coastal plain deposits, almost entirely below the groundwater surface. Applying the results obtained to other types of piles and soils, or unsaturated soils, requires appropriate judgment and perhaps additional testing. However, we believe that the UF research, combined with the extensive literature about the setup of driven, displacement piles, makes our subsequent design recommendations widely applicable.

O-Cell Testing

The O-cell test had some key advantages for the UF SSS measurements, including compression loading, the distinct measurement of side shear, easy staged testing over long time periods, and complete unloading of the pile tip to minimize posttest residual side shear. However, it also had some disadvantages for this research, such as the mobilization of pile side shear in the opposite direction from normal pile loading and the lack of an independent load cell to measure the applied force. However, a significant amount of theoretical and practical research on the effect of direction on side shear capacity, mostly with bored not driven piles, has shown little (<10%) or no difference. Side shear direction should have minimal effect on the setup factor A, which represents a relative change in capacity compared to an initial state, and not an absolute measurement. Part of the apparent SSS could have resulted from increasing internal friction in the cell, but the strain measurements give no indication of such an effect. Furthermore, the measured SSS factors seem consistent with those obtained by others using conventional top-load testing with load cells.

Instrument Error

A few instruments performed erratically, or stopped functioning reliably after a period of time, but most appeared to perform

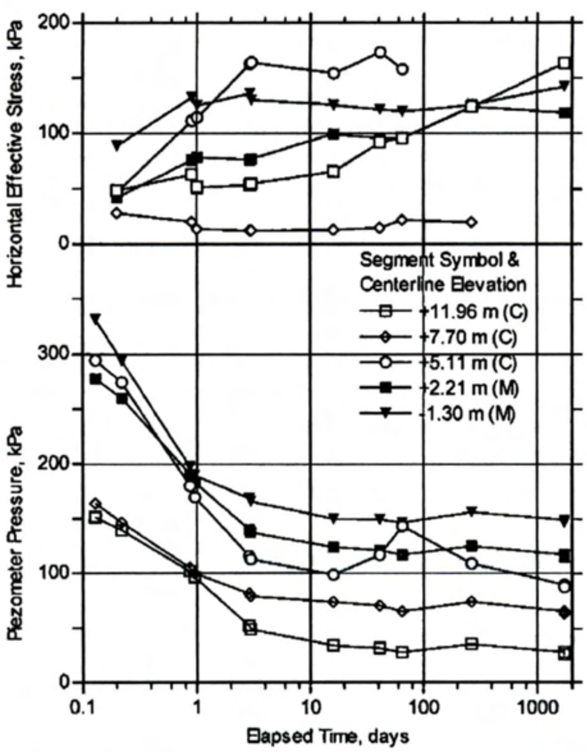

Fig. 2. Aucilla River test site stress and pore pressure

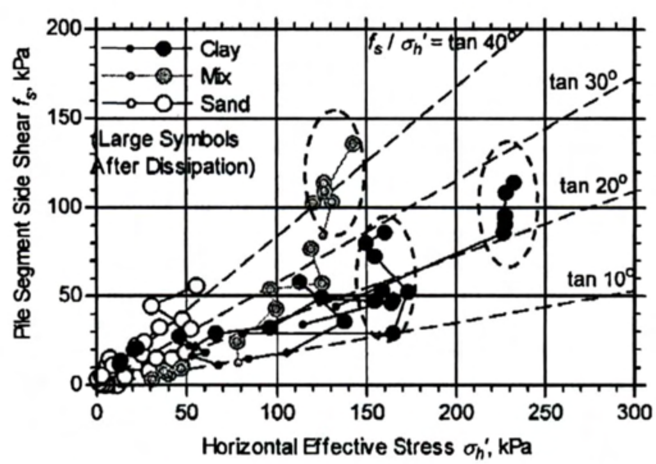

Fig. 3. Effective stress increase (all segments with σ_h')

satisfactorily throughout this research. Random instrument errors may have contributed to the scatter of the data, but no evidence suggests any significant systematic errors.

Horizontal Effective Stress during and after Staged Tests

The σ_h' values reported for this research come from measurements taken before each static test. The datalogger used for the test also recorded pore pressure measurements during and after each test. Each test required approximately 2 h with about 20 load increments to failure and 10 unload increments. In general, the piezometers measured very little pore pressure change during the tests for S soils, and the increase measured in C and M soils dissipated within about 24 h after each test. We did not monitor the manual total stress cells during the tests and cannot report σ_h' during the shear failure. However, Axelsson (1998) reports that before-test σ_h' values measured on displacement piles driven in sand remained relatively constant, but progressively increased during each successive staged test. The soil-structure adjacent to the pile appeared to become more dilatant with time and staged testing, increasing σ_h' during the test and probably also the SSS. An unknown portion of the SSS observed during the UF research may result from similar behavior. In any event, after correcting for staged testing effects when applying test results to unstaged pile capacity, the pile-soil interface changes produce a long-lasting pile capacity increase.

Pile Size and t_0

As discussed in the companion paper (Bullock et al. 2005) determination of the reference time t_0 affects the value of A by changing the reference capacity, Q_0. Comparing the SPT-T and pile segment setup factors empirically, we found that using $t_0=5$ min for the SPT-T setup factor provided reasonable agreement with the 457-mm pile setup factor calculated using $t_0=1$ day $=1{,}440$ minutes. This ratio of 288 $(=1{,}440/5)$ may relate to the diameter of the pile and provide a rational means of correcting setup factors between different displacement pile sizes. The maximum excess pore pressure induced by the penetration of a displacement pile and the maximum soil destructuring should occur near the pile surface, with both diminishing to zero at some radial distance. As the excess pore pressure dissipates in the radial direction, the soil consolidates, a process known to occur over time in approximate proportion to the square of the pile diameter. Assuming that the pile displacement causes a destructuring gradient in the soil similar to a pore pressure gradient, then restructuring (aging) may follow a process similar to consolidation, requiring times in proportion to the square of the pile diameter.

The equivalent diameter of the 457-mm, square UF research piles $=[4\times(457\text{ mm})^2/\pi]^{1/2}=516$ mm. The SPT sampler displaces an annular area of soil with an outside diameter of 51 mm and an inside diameter of 35 mm. Some of this annular area will be forced inside the sampler, and the actual equivalent displacement should fall between these extremes. Using the average sampler wall diameter of 43 mm and the ratio of equivalent diameters squared to correct from the 457-mm pile, the "theoretical" SPT-T $t_0=1{,}440\text{ minutes}\times(43\text{ mm}/516\text{ mm})^2=10$ min. Based on empirical observation, we used $t_0=5$ min to calculate the SPT-T setup factors, the difference from the theoretical $t_0=10$ min having only a minor effect on A. The use of the ratio of the pile diameters squared has some theoretical justification and seems to produce reasonable results. Within the range of normal pile diameters, 305 mm to 610 mm, the above correction to t_0 based on the pile radius has a relatively small effect on A, and $t_0=1$ day seems reasonable for all.

Piles Unloaded

As done for virtually all pile setup research, the piles remained unloaded between tests. But, different SSS may result during construction as the pile load progressively increases. Research by Bea (1960), described in Schmertmann (1976, 1981), showed that cohesive soils loaded, then allowed to creep and age undrained under constant load, increased in both stiffness and shear strength compared to when unloaded. SSS may also increase for partially loaded piles, and unloaded test piles will exhibit conservatively less SSS.

Staged Testing

Almost all SSS research results come from static or dynamic tests "staged" successively on the same pile at increasing times after the EOD, thus including a preshearing effect that might increase or decrease the side shear capacity measured during subsequent stages. Because of the added expense and the complication of site variability, engineers rarely perform "unstaged" tests of separate piles. We could find only two studies that included both staged and unstaged tests. Karlsrud and Haugen (1985) report tests up to 35 days after jacking 153-mm diameter, 5-m long, closed-end, pipe piles into a sensitive (sensitivity =4.5), overconsolidated Norwegian clay. Miller (1994) reported tests of 60-mm diameter, pipe piles up to 417 days after driving or pushing the piles 1.52 m into the drying crust of a sensitive, overconsolidated, Connecticut Valley Varved Clay at the National Geotechnical Experimentation Site in Amherst, Mass. Using $t_0=1$ day, the Karlsrud and Haugen piles provide a ratio of $C_{st}=(A_{\mathrm{Unstaged}}/A_{\mathrm{Staged}})=0.33$ to convert the SSS results from staged to unstaged tests. Again using $t_0=1$ day, our analysis of the Miller (1994) data gives a ratio of $C_{st}=0.33$ to 0.45.

Because of the potential effect of a C_{st} stage-testing correction on the UF research, the Florida Department of Transportation (FDOT) provided supplemental funds to perform twelve SPT-T borings centered on the SBZ pile, three borings for staged tests and nine borings for unstaged tests. Bullock and Schmertmann (2003) present the SPT-T results from the SBZ site, obtained from a shelly silty clay layer and a silty sand layer up to 1,080 min after driving the SPT sampler. Using $t_0=5$ min, Fig. 6 shows the SPT-T setup results in the SBZ clay, which gives $C_{st}=(0.265/0.667)=0.40$. Similar to previous SPT-T results in sand, a lack of staged SPT-T setup during the 1,080-minute test period provided an inconclusive measurement of C_{st} for the SBZ silty sand. However, comparing the SPT-T $A_{\mathrm{Unstaged}}=0.14$ with the adjacent pile segment $A_i=0.51$ provided $C_{st}=0.27$. Based on the range of $C_{st}=0.27$ to 0.45 found in the UF research and the above literature examples, we recommend using $C_{st}=0.4$.

Shear Mobilization and Movement Compatibility

The SSS analyses of the UF test results presented herein use the maximum pile segment side shear capacities measured during each static test to calculate the A_i segment setup factors. Because individual pile segments will generally fail at different amounts of axial movement (as during this research), and then decrease toward residual side shear, the sum of the maximum segment side shear values should exceed the whole-pile side shear mobilized at

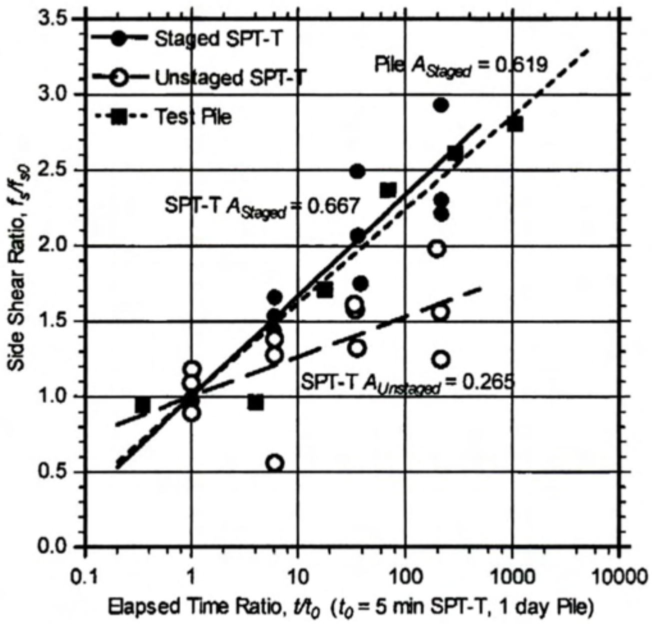

Fig. 6. Standard penetration test with torque versus segment setup factors in Seabreeze Bridge test site clay

the failure load. As shown in Table 2, individual pile segments generally have different setup factors (A_i) than that calculated for the whole-pile (A). The whole-pile A will more closely reflect the setup factors of longer segments, higher capacity segments, and segments that fully mobilize last. For a pile with constant perimeter area, combining the equations presented in the companion paper provides:

$$A = \frac{\Sigma A_i f_{s0i} L_i}{\Sigma f_{s0i} L_i} \text{(sums for all pile segments)} \quad (1)$$

where A=side shear setup factor, whole pile; A_i=side shear setup factor pile segment i; f_{s0i}=unit side shear (stress) at time t_0 for segment i; and L_i=length of pile segment i.

However, the actual whole-pile setup factors shown in Table 2 are approximately one-half of those calculated using Eq. (1), suggesting a correction factor for shear mobilization of $C_{\text{pile}}=0.5$ when calculating A from A_i. Some design methods include movement compatibility explicitly, while many others include it implicitly within the reference database or through conservatism. Direct analysis of field tests should separate whole-pile and pile segment setup factors to avoid misuse of the results.

Group Action

Most production piles are installed and loaded in groups, rather than singly as done in this and most other setup research. Piles driven in groups should more completely destructure the soil between and along the piles, and consequently show greater restructuring (aging) and greater SSS. The research by Camp et al. (1993), described in the companion paper (Bullock et al. 2005) provided the only documentation found of a group effect. They showed an increase in the SSS.

Design Recommendations

Foundation engineers have widely recognized and documented the increase in capacity of driven, displacement piles after installation, sometimes for long periods of time. As this research in northern Florida strongly reinforces, SSS occurs in sand, clay, and mixed cohesive soils. Based on this research and the extensive literature documentation (partially documented in the companion paper), for soils similar to those described herein it seems reasonable to routinely use at least a minimum setup factor in pile design capacity calculations without additional testing. Results from predictor tests and test pile programs may provide larger setup factors. Additional research on the potential effects described above (especially staged testing) may prove useful to further refine the design recommendations provided below:

Side Shear Setup Reference Time t_0

The analyses performed herein use an arbitrary, but practical, reference time of $t_0=1$ day for piles. Assuming that the linear increase of side shear capacity versus the logarithm of elapsed time has begun by 1 day after the EOD (with slope m_Q), then assigning a 1-day reference capacity provides a standard for the comparison of setup factors obtained from different pile-soil combinations. Using this arbitrary reference time of 1 day does not ignore any prior SSS or change the m_Q slope. An engineer may either interpolate or extrapolate the reference capacity from field tests at other times, provided the semilog-linear assumption remains valid. We recommend the continued use of this 1 day reference time in order to develop a reference database.

Side Shear Setup Start and Finish Times

Presumably each pile capacity design method has a research database of full-scale pile tests, and it will already include some SSS up to the average time of the database load tests. Any SSS increase will only apply after this time. The setup factor determined using $t_0=1$ day will apply to the reference capacity extrapolated back to the reference time. We expect an average elapsed time of 1 to 3 weeks after the EOD to perform a load test, but the designer should determine this value. Construction scheduling will most likely define the final time at which the design pile capacity will be required, and thereby control the elapsed time at which to calculate the design side shear including SSS. This research included tests at 1,057 and 1,727 days (at separate sites) showing little change in the A value, but some of the research results herein and some examples in the literature indicate a tendency for A to decrease during later log cycles. Without research or experience indicating otherwise, a maximum SSS development time of 1,000 days seems appropriate.

Staged Test Reduction

As previously discussed, we recommend a reduction factor of $C_{\text{st}}=(A_{\text{Unstaged}}/A_{\text{Staged}})=0.4$ for all soil types to correct setup A factors measured using staged field tests, including repeated dynamic restrikes, repeated static tests, or repeated SPT-Ts.

Default Minimum A=0.1

The literature review, and the present test results summarized in Table 2 and Fig. 5, suggest the practical use of a default minimum A=0.2 for pile segments. However, Table 2 and Fig. 5 indicate a lower, whole-pile A-value, which combined with the potential errors discussed above (especially stage testing), leads to a more prudent default recommendation of $A = C_{st} C_{pile}(0.2) = 0.4 \times 0.5 \times 0.2 \approx 0.1$ (in soils similar to those tested in northern Florida or otherwise known to develop SSS).

Safety Factor for Side Shear Setup

Given the conservatism included in the above default A value proposed for use without predictor tests and in the proposed stage testing correction for predictor-measured A-values, no further reduction of A seems needed. To avoid calculation errors, a designer should first use the ultimate design side shear from the chosen method, its applicable time, and the assigned A values to calculate the reference side shear (F_{s0}) at the reference time (1 day) before extending it to the chosen design time. Then apply a safety factor (FS) associated with the design method (typically FS=2) to the ultimate side shear calculated for the final design time, which will include the SSS. Load resistance factor design will require additional study of SSS variability to establish appropriate reduction factors (possibly similar to overall side shear factors).

Proposed Side Shear Setup Design Procedure

1. Assign a pile size, length, and type for preliminary design.
2. Prepare an estimate of ultimate unit side shear capacity, $F_{s\ \mathrm{est}}$, for the preliminary design pile using available site information and an acceptable design method (SPT97 or similar). Use $f_{s\ \mathrm{est}}$ 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, $f_{s\ \mathrm{est}}$. 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 the actual time elapsed from the EOD for dynamic tests during restrikes and $t_{\mathrm{est}} \approx 1$ min for dynamic tests at the end of initial driving.)
3. Using a semilog-linear relationship between side shear capacity and log time, calculate the reference side shear capacity, f_{s0}, at a reference time of $t_0 = 1$ day:

$$f_{s0} = \frac{f_{s\ \mathrm{est}}}{1 + A\ \log(t_{\mathrm{est}}/1\ \mathrm{day})}$$

$$(\text{where } A = A_{\mathrm{Unstaged}} = 0.1 \text{ default without tests}) \quad (2)$$

4. Assign a final time, t_f, (after the EOD) to the required ultimate side shear capacity. Depending on the project, t_f=3 to 12 months, or longer. Calculate f_{sf}:

$$f_{sf} = f_{s0}[1 + A\ \log(t_f/1\ \mathrm{day})] \quad (3)$$

5. Multiply the unit side shear values by the pile perimeter area and sum the side shear forces for all layers. (If consistent, the equations above work easily with either unit side shear or side shear force.) Based on the result, change the pile size, length, or type as desired to optimize the design. Also, apply a safety factor as required.
6. Based on the UF research, all soils should have $A \geqslant 0.1$. If possible, perform SPT-T borings and/or a design phase test pile program to determine A for use in the above equations. Perform staged dynamic pile tests at geometrically increasing elapsed times, e.g., 15 min, 3 h, 36 h, etc. Perform staged SPT-Ts at elapsed times of 5 min, 30 min, 3 h, 18 h, etc. Use $t_0 = 1$ day for pile tests and $t_0 = 5$ min for the SPT-T.
7. If A results from staged tests, then multiply it by the factor $C_{st} = 0.4$.
8. Multiply pile segment A_i and SPT-T A by the factor $C_{pile} = 0.5$ for movement compatibility with whole-pile side shear capacity.
9. If possible, confirm the design setup factor A during construction using static tests, dynamic tests, or a combination. If these tests do not accurately separate side shear and end bearing, using the whole-pile capacity to calculate A should produce a smaller A-value (setup expected to increase only the side shear).

Conclusions and Practical Applications

Based on the relevant literature and this UF SSS research, we offer the following general conclusions and recommendations:

1. Using staged tests of unloaded piles, and an accurate measurement of side shear obtained by the O-cell test method, this research demonstrated SSS similar to that observed by others in prior research.
2. All pile segments showed setup, with similar average magnitudes in all soils and at all depths, continuing long after the dissipation of pore pressures, and with postdissipation setup due to aging effects at approximately constant horizontal effective stress. The pile tests (all soil types), and the SPT-T predictor tests (cohesive soils only), confirm the approximately semilog-linear time setup behavior previously observed by others. As done by others, we also expressed SSS using the semilog-linear setup factor A.
3. Engineers now have an adequate research background and experience to include SSS in routine design. For soils similar to those tested in this research and/or known to exhibit SSS, we recommend using a default $A = 0.1$ without performing predictor tests, and higher values when supported by dynamic or static testing of whole piles, or staged SPT-Ts in clay and mixed soils. Reduce A-values measured during staged tests (pile or SPT-T) by the factor $C_{st} = 0.4$. Reduce pile segment A_i and SPT A by the factor $C_{pile} = 0.5$ for movement compatibility with whole-pile side shear capacity (if unknown). If the SPT-T $A_{\mathrm{staged}} \leqslant 0.5$, use the default $A_i = 0.2$ and $A = 0.1$.
4. We propose a conservative method for including SSS in pile capacity design. The Appendix provides some idealized, but realistic, examples to show the methods recommended for including SSS in design. Depending on the percentage of capacity due to side shear, the final design time, and the applicable setup factors, SSS may significantly increase design pile capacity. (The Appendix examples give a 16 to 56% increase.)
5. Dynamic tests during initial driving and subsequent restrikes provide a method, after applying the 0.4 reduction factor for stage testing, by which to check the design A value. Repeated restrikes also allow SSS behavior to occur at the increased rate of staged testing, and may permit the acceptance of a pile that initially does not demonstrate adequate capacity.

Appendix. Side Shear Setup Design Examples

The following examples illustrate the proposed use of SSS in design calculations. Note that the calculations below do not include end bearing, which is calculated separately and generally not assumed to exhibit setup. Since the total pile capacity generally includes end bearing and side shear, the percent increase of the total capacity will be less than the percent increase of the side shear. The designer should also reduce the side shear capacities by an appropriate safety factor.

Using Default $A=0.1$

Calculate the side shear capacity with setup using a SPT design method for a 24-m-long (78.7 ft), 457-mm (18-in.), square prestressed concrete pile, driven into a two-layer soil profile with 15.2 m (50 ft) of soft clay ($N_{\mathrm{spt}}=6$) over dense sand ($N_{\mathrm{spt}}=30$). The design method provides an ultimate $F_{s1\ \mathrm{est}}$(clay) $=829$ kN (186 kips) and $F_{s2\ \mathrm{est}}$(sand)$=878$ kN (197 kips). Assume that the reference load test database for the method includes movement compatibility from tests performed at about $t_{\mathrm{est}}=1$ week on average. The pile capacity will be mobilized one year after the EOD. No field test data are available. Using Eqs. (2) and (3):

$$\text{Clay:}\quad F_{s01}=\frac{829\text{ kN}}{1+0.1\log(7\text{ days}/1\text{ day})}=764\text{ kN}$$

$$F_{sf1}=764\text{ kN}[1+0.1\log(365\text{ days}/1\text{ day})]=960\text{ kN (216 kips)}$$

$$\text{Sand:}\quad F_{s02}=\frac{878\text{ kN}}{1+0.1\log(7\text{ days}/1\text{ day})}=810\text{ kN}$$

$$F_{sf2}=810\text{ kN}[1+0.1\log(365\text{ days}/1\text{ day})]=1{,}017\text{ (229 kips)}$$

Total: $F_s=1977$ kN with SSS versus 1707 kN without SSS, +16% increase.

Using Standard Penetration Test-Torque A

Staged SPT-Ts indicate $A=0.95$ for the clay layer in the above design example, but no are tests performed in the sand layer (default $A=0.1$). Correct the SPT-T A for staging effects and movement compatibility, and then use Eqs. (2) and (3):

$$\text{Clay:}\quad A_{\mathrm{Unstaged}}=C_{\mathrm{st}}\,C_{\mathrm{pile}}(\text{SPT-T }A_{\mathrm{Staged}})=0.4\times0.5\times0.95=0.19$$

$$F_{s01}=\frac{829\text{ kN}}{1+0.19\log(7\text{ days}/1\text{ day})}=714\text{ kN}$$

$$F_{sf1}=714\text{ kN}[1+0.19\log(365\text{ days}/1\text{ day})]=1{,}062\text{ kN (239 kips)}$$

$$\text{Sand:}\quad F_{s02}=\frac{878\text{ kN}}{1+0.1\log(7\text{ days}/1\text{ day})}=810\text{ kN}$$

$$F_{sf2}=810\text{ kN}[1+0.1\log(365\text{ days}/1\text{ day})]=1{,}017\text{ kN (229 kips)}$$

Total: $F_s=2{,}079$ kN with SSS versus 1,707 kN without SSS, +22% Increase

Using Dynamic Test Pile Measurements (Whole Pile)

Use the same design parameters as above with staged dynamic test pile measurements that indicate $A=0.65$ for the whole pile, $F_{s1\ \mathrm{est}}$(clay)$=815$ kN (183 kips) and $F_{s2\ \mathrm{est}}$(sand) $=864$ kN (194 kips) at 1.8 days. Correct the measured A values for staging effects, and then use (2) and (3):

$$A_{\mathrm{Unstaged}}=0.4\times A_{\mathrm{Staged}}=0.4\times0.65=0.26$$

$$\text{Clay:}\quad F_{s01}=\frac{815\text{ kN}}{1+0.26\log(1.8\text{ days}/1\text{ day})}=764\text{ kN}$$

$$F_{sf1}=764\text{ kN}[1+0.26\log(365\text{ days}/1\text{ day})]=1{,}273\text{ kN (286 kips)}$$

Sand:

$$F_{s02}=\frac{864\text{ kN}}{1+0.26\log(1.8\text{ days}/1\text{ day})}=810\text{ kN}$$

$$F_{sf2}=810\text{ kN}[1+0.26\log(365\text{ days}/1\text{ day})]=1{,}350\text{ kN (303 kips)}$$

Total: $F_s=2{,}623$ kN with SSS versus 1,679 kN without SSS, +56% Increase

Notation

The following symbols are used in this paper:

A = dimensionless setup factor, whole pile (semilog-linear slope);
A_i = dimensionless setup factor, pile segment (semilog-linear slope);
A_s = pile side area;
A_{Staged} = dimensionless setup factor measured using staged tests;
A_{Unstaged} = dimensionless setup factor measured using unstaged tests;
C_{st} = $(A_{\mathrm{Unstaged}}/A_{\mathrm{Staged}})$= ratio of unstaged to staged setup factors;
C_{pile} = reduction factor applied to pile segment A_i or standard penetration test-torque A to correct for movement compatibility;
c_a' = pile-soil adhesion;
F_s, F_{s0} = pile segment side shear (force) at time t or t_0;
f_s, f_{s0} = unit side shear (stress) at time t or t_0;
L = pile segment length;
N_{SPT} = standard penetration test blowcount, blows/0.3 m (blows/ft);
Q, Q_0 = pile capacity at time t or t_0;
Q_s, Q_{s0} = whole-pile side shear (force) at time t or t_0;
t = time elapsed since end of driving;
t_0 = reference time elapsed since end of driving;
δ' = pile-soil drained friction angle; and
ϕ_h' = effective horizontal stress.

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