Authorized Reprint from
Special Technical Testing Publication 883
Copyright
American Society for Testing and Materials
1916 Race Street, Philadelphia, PA 19103
1985

Richard L. Handy,[1] *John H. Schmertmann,*[2] *and*
Alan J. Lutenegger[3]

Borehole Shear Tests in a Shallow Marine Environment

REFERENCE: Handy, R. L., Schmertmann, J. H., and Lutenegger, A. J., "**Borehole Shear Tests in a Shallow Marine Environment,**" *Strength Testing of Marine Sediments: Laboratory and In-Situ Measurements, ASTM STP 883,* R. C. Chaney and K. R. Demars, Eds., American Society for Testing and Materials, Philadelphia, 1985, pp. 140–153.

ABSTRACT: A wireline adaptation of the borehole shear test (BST) with high-pressure shear plates was one of several in-situ test methods selected to measure strength of overconsolidated phosphatic clays and lime rock at proposed pier locations for the replacement Sunshine Skyway Bridge, Tampa Bay, FL. Although the site's soils ordinarily would be judged too hard for stage BSTs, an on-site decision was made to try it in lieu of single-point testing to save time. Of 18 stage tests attempted in very hard clay, 10 gave satisfactory failure envelopes, 5 gave envelopes indicative of residual shear strength, and 3 gave invalid envelopes attributed to progressive seating of the shear plates. Stage tests in the harder rocks and shell were less successful. The average cohesion from the 10 satisfactory stage tests in the overconsolidated clays was 91 kPa (1900 psf) and the average friction angle ϕ = 25.7°. The BST data proved useful for converting a large number of undrained penetration tests into the effective stress strength parameters needed for design. The additional use of special BSTs using smooth plates also permitted site-specific shear strength corrections for soil against steel piles and casings.

KEY WORDS: field test, shear strength, boreholes, angle of friction, cohesion, borehole shear test, underwater, coefficient of sliding friction

This paper describes the first offshore use of the borehole shear test (BST), which was included as a relatively small part of a comprehensive geotechnical investigation for a large bridge in Florida. Sufficient reference will be made to the larger investigation to show how the BST data fit into the overall picture.

[1]Professor of civil engineering and consultant, Iowa State University, Geotechnical Research Lab, Ames, IA 50011.
[2]Principal, Schmertmann and Crapps, Inc., 4509 N.W. 23rd Ave., Suite 19, Gainsville, FL 32601.
[3]Associate professor of civil engineering and consultant, Clarkson University, Potsdam, NY 13676.

Method

The BST performs a direct shear lengthwise along a borehole, in soil engaged at the sides by two opposed, expanding, sharpened, toothed plates. The test thus determines relationships between a soil's direct shearing strength and applied normal stress [*1*]. Several generations of BST instruments have extended the use of the device from soft sands, silts and clays to hard, overconsolidated clays and shales [*2*], and to rocks ranging from coal to limestone and weathered granite [*3*]. An optional transducer attachment now allows simultaneous monitoring of pore-water pressure during the test [*4*] but was not available at the time of the testing reported herein.

Stage Tests

The use of the BST in soil and in rock has led to two different modes of testing: In soils a sequential stage test is performed whereby the shear head is left in place for successive shears at progressively higher normal pressures, whereas in rocks the shear head is removed, cleaned off, and reinserted for each test point [*2,3*].

Stage testing offers substantial advantages in simplicity, speed, and for testing of individual layers but may produce invalid strength envelopes if the shear plate teeth do not fully engage into the soil or rock [*3*]. A second requirement for stage tests conducted in essentially the same soil is that a sheared soil must reconsolidate so its strength exceeds that of the peripheral unsheared soil, in effect moving the shear plane radially outward into the soil at the side of the borehole [*1,5*]. These requirements would appear to preclude the use of the stage testing mode in very hard soils, but the third author successfully applied this technique for testing soft clay shales in the Denver area.

The feasibility of BST for underwater testing previously was shown by tests conducted in shallow lake sediments by scuba divers [*6*]. Site investigations for the replacement Sunshine Skyway Bridge across Tampa Bay, FL, afforded an opportunity for the first offshore use of BST in holes drilled from a floating platform. The apparatus was modified from rod supported to wireline to reduce hole transit time for the shear head. Furthermore, since testing offshore from a barge precludes testing in one hole while the next is being drilled, an on-site decision was made to try the stage testing mode to save time, even though the Tampa Bay sediments are dominated by hard clays and claystones, harder than had previously been tested by this method.

Soil to Steel Friction

The unique BST geometry also allows direct supplemental measurement of soil to steel sliding friction by replacing the sharply toothed BST shear plates with smooth plates. Such tests have been reported with concrete shear plates to represent concrete pile surfaces [7]. In the present sliding friction investigation smooth steel plates were used, bevelled at the ends to reduce end effect.

Soils Tested

A comprehensive discussion of site stratigraphy is beyond the scope of this paper. Briefly, the uppermost layer is a loose sediment, primarily shell and sand, about 6 m thick. This overlies about a 50-m section of the Miocene Hawthorn Formation, consisting of a complex sequence of hard, swelling clays and sandy clays with average standard penetration test (SPT) N values of about 50, claystones with $N > 100$, and calcareous claystones and clayey limestones that usually require coring to obtain samples. BSTs were performed at 1 to 3 m (3 to 10 ft) intervals below the mudline depth of approximately 10 m (30 ft), to a maximum depth of 30 m (100 ft) below sea level. Most tests were conducted in the weaker lime rock and in the hard clay and claystones.

As will be shown, most of the successful stage testing came in the hard clay layers that comprise the weakest materials in this relatively strong Hawthorn formation. This clay had SPT N values typically from 30 to 70, with a coefficience of variation in N of approximately 45%, both horizontally and vertically. Other approximate average geotechnical properties of this highly variable clay are plasticity index (PI) = 65%, liquid limit (LL) = 110%, natural moisture content w = 55%, liquid index (LI) = 0.15, activity = 1.3, undrained shear strength s_u = 500 kPa, sensitivity in unconfined tests about 7, P'_c = 4 MPa, overconsolidation ratio (OCR) = 50, and m (one-dimensional compression modulus) = 100 MPa. It also has an attapulgite-montmorillonite clay mineralogy, weak cementation bonds, a strong swelling tendency, and high in-situ lateral stresses that are discussed later. It will be seen that the relatively high, 4 MPa (600 psi), preconsolidation stresses that were determined from other laboratory and in-situ tests greatly exceed the maximum BST applied normal stress of 1.3 MPa (200 psi).

Borehole shear tests were conducted in 75-mm (3-in.) diameter "Floragell" mudded holes bored with either a diamond-bit core barrel or a drag bit, from a truck-mounted drill supported on a barge that had vertical spuds anchoring it to the seafloor. A 100-mm (4-in.) diameter casing was pushed to a stable position below the mudline and extended up through an open drill well in the barge. The top of this casing was used as the reaction base for pulling the shear head because of its convenience and to avoid barge movement and vibration problems resulting from wind, waves, and tidal fluctuations.

Equipment Modifications for Offshore Use

A conventional BST baseplate dynamometer consisting of calibrated hydraulic support cylinders and a pressure gage for shear readings was supported on two parallel long-stroke hydraulic cylinders to take up cable slack and apply the pulling force, as illustrated schematically in Fig. 1. Excess cable was carried on a reel, and the cable was clamped to the base by a split wedge. After the shear head was lowered to the desired test level, and before it was expanded, the partially buoyed weight of the pulling system was determined from the gage

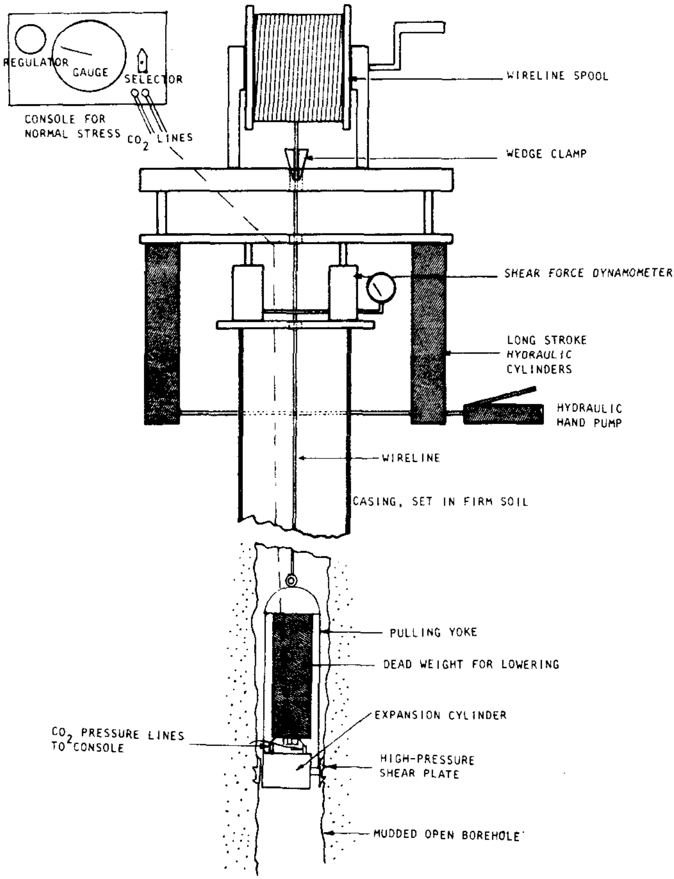

FIG. 1—*Schematic representation of BST modified for over water use.*

reading and subtracted from all subsequent readings to give the net gage pulling force transmitted by shear to the soil. The shear head was not modified except for the addition of a weight to secure a more positive lowering. Expansion pressure was applied by regulated carbon dioxide gas pressure, and a static pore-pressure correction was calculated based on testing depth and density of drilling fluid, and diameter of the opening piston. Details of this calculation are presented in the Appendix.

The high pressure BST shear plates that were used for all soils have separated teeth for testing overconsolidated soils and have been previously described [2]. Normal and shearing stresses were calculated by dividing the respective measured forces by the nominal plate area of 645 mm^2 (1 in.^{2}). To keep total testing time

TABLE 1—*Categorization of Tampa Bay BSTs: shear test category.*[a]

Material	I Shear Failure	II Residual Only	III Progressive Seating
Clay	10	5	3
Gravelly clay	0	0	4
Lime rock	1	0	3
Shell	0	1	1
		SUBTOTALS	
	11	6	11

[a]Total number of shear tests: 28.

within practical limits, the BST manufacturer's recommendation of 5-min consolidating time was followed between successive testing points. The then-available BST equipment did not incorporate a piezometer, so pore pressures were not monitored. However, the results were reviewed so as to estimate the probable pore-pressure behavior, as discussed subsequently.

Results and Interpretations

Thirty-six BST shear envelopes were obtained that include eight of the special smooth-plate tests. In borehole shear testing each data point is reduced and plotted as it is obtained; so each test and test point can be interpreted to ascertain if, for example, proper seating of the shear plate has yet occurred. Later a decision must be made as to whether or not the envelope obtained is a true indicator of the particular soil behavior. This decision is fairly easily accomplished by inspection and recognizing, for example, the physical impossibility of a negative cohesion [*1*]. In order to systematize these interpretations, several categories of tests were defined as shown in Tables 1 and 2 and Fig. 2.

Another aid for interpretation and categorization of BST test results is to continue shearing to obtain both peak and residual shear strengths. The latter are shown by triangles in Fig. 2. Residual strengths are not obtained if the intermittency of shearing caused by rod or cable stretch allows reconsolidation such that

TABLE 2—*Categorization of Tampa Bay BSTS: smooth-plate tests.*[a]

Material	Good Test	Questionable or Unreliable[b]
Clay	3	1
Gravelly clay	0	1
Lime rock	2	...
Shell	0	1
	SUBTOTALS	
	5	3

[a]Total number of smooth-plate tests: 8. Total number of BSTs: 36.
[b]Correlation coefficient < 0.95, perhaps from traversing non-uniform materials.

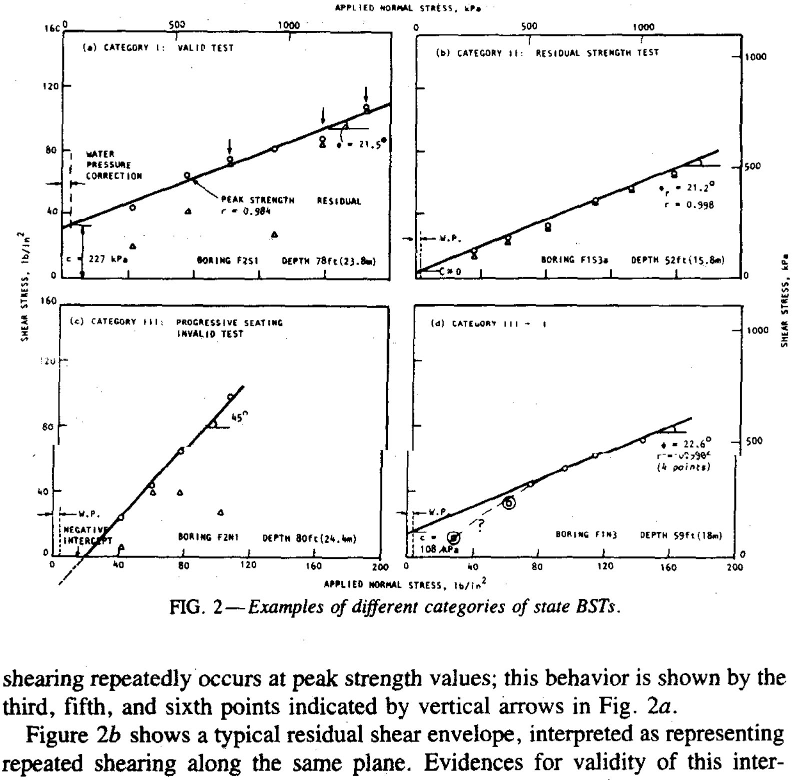

FIG. 2—*Examples of different categories of state BSTs.*

shearing repeatedly occurs at peak strength values; this behavior is shown by the third, fifth, and sixth points indicated by vertical arrows in Fig. 2*a*.

Figure 2*b* shows a typical residual shear envelope, interpreted as representing repeated shearing along the same plane. Evidences for validity of this interpretation include the close agreement between peak and residual values (Fig. 2*b*), as well as the very low but nevertheless positive cohesion intercept. Shear envelopes similar to Fig. 2*b* also were obtained in the smooth-plate friction tests, the slope being indicative of the angle of sliding friction.

Figure 2*c* is representative of a common problem in stage testing hard materials: an excellent linearity that might lead one to incorrectly assume this is a valid shear envelope—except that it almost always gives a negative cohesion intercept. The negative intercept and approximate 45° slope is explained by the progressive seating of the shear plate teeth, causing resistance to drag to be proportional to the applied normal stress. The graph of Fig. 2*c* therefore is an invalid indicator of the soil shearing strength. It is, however, on the low and therefore the safe side, so long as there is no extrapolation beyond the data stress range of the test.

Figure 2*d* shows a hybrid shear envelope commonly obtained in tests in mudded holes; as normal stress is increased, a progressive seating Category III behavior gives way to full seating and a valid Category I shear envelope. This is a fairly common phenomenon and should be expected in any stage BSTs and is a reason why a minimum of five points is used for a test instead of the conventional three in triaxial stage testing. An alternative interpretation of the two-stage BSTs, that the envelope is truly curved, as shown by the dashed line in Fig. 2*d*, again repeats the physical impossibility of a negative cohesion intercept. Because BST shear envelopes are immediately plotted as each point is obtained, an on-site decision is made whether to accept the test or continue it to higher values of normal stress and attempt to overcome progressive seating.

It can be seen in Tables 1 and 2 that of 28 conventional BSTs attempted, only 11, or 39%, were in Category I. However, these eleven are mainly in the hard clay, and of the 18 clay tests attempted, 10, or 56%, are Category I, and 5, or 28%, are Category II, leaving only 3, or 17%, in the unsuccessful Category III. On the other hand, the tests conducted in harder materials that included shell and clay with embedded phosphate nodules gave a success rate of only 2 out of 10, or 20%.

It may be concluded from data in Table 1 that stage testing was an expeditious method for testing the hard clay but was inappropriate for the gravelly clay, lime rock and shell, where the more prolonged single-point rock-testing method would be preferred [*3*].

Use of BST Data for Materials Characterization in Skyway Bridge Design

Table 3 presents the BST results from the Tampa Bay investigations, and Table 4 presents results of the BST sliding friction experiments. The BST strength data reveal a general inverse trend between cohesion and friction angle, shown by the point scatter in Fig. 3. The large double point in Fig. 3 shows the average c' and ϕ' from eight isotropically consolidated, undrained triaxial test series of three specimens each, corrected for pore-water pressure.

The following describes the use made of the BST data to help interpret a large number of other types of in-situ tests and help predict bearing capacities.

An effective stress bearing capacity analysis was used for designing the various drilled shaft and driven pile foundation alternates. The capacities of such foundations depend on the values of effective stress c', angle of internal friction ϕ', lateral stress ratio K', and pore-pressure change Δu assigned to each soil layer, the sequence and thickness of the layers, and the geometry and materials and methods of installation for the shafts or piles. The geotechnical engineers had the results from hundreds of penetration tests of the standard penetration test (SPT) and Marchetti dilatometer test (DMT) types in the hard clays at the main piers for the bridge, and eventually thousands for the entire bridge. Results from many Menard pressuremeter tests (MPMTs), eventually about 130 at the high-level bridge piers, also were available, plus back-analysis results from drilled shaft and driven pile load tests. But, for strength properties all the SPTs, DMTs, MPMTs, and load tests gave only a measure of the undrained shear strength s_u of the hard clays. As discussed subsequently, the relatively few BSTs helped to convert these many s_u values to equivalent c', ϕ', K', and Δu values needed for the effective stress design method.

TABLE 3—*Tampa Bay BST results.*

Boring Number	Depth, m (ft)	Test Category[a]	Cohesion, kPa (psf)	ϕ, degrees	Correlation r	Material
F1N3	18.0 (59)	I	108 (2260)	22.6	0.996	clay
	18.9 (62)	I	106 (2220)	25.1	0.994	clay
	19.8 (65)	I	149 (3110)	21.4	0.976	clay
	20.4 (67)	III	...	...	...	gravelly clay
	21.6 (71)	I	64 (1330)	18.1	0.963	clay
	25.0 (82)	III	...	...	...	lime rock
	26.5 (87)	I	56 (1180)	37.0	0.995	clay
	27.7 (91)	I	59 (1240)	27.8	0.968	clay
	31.4 (103)	III	...	...	...	shell
F1S4	14.3 (47)	II	...	11.7	0.991	clay
	16.2 (53)	III	...	...	...	clay
	18.9 (62)	II	...	33.3	0.992	clay
F1S3a	13.7 (45)	I	137 (2850)	19.7	0.991	clay
	15.8 (52)	II	...	22.1	0.998	clay
	19.5 (64)	III	...	...	...	clay
	21.6 (71)	II	...	...	...	gravelly clay
	25.3 (83)	III	...	...	...	gravelly clay
	26.8 (88)	I	115 (2400)	24.4	0.975	clay
F2S1	13.4 (44)	II	...	15.4	0.969	shell
	15.2 (50)	III	...	...	...	gravelly clay
	18.9 (62)	III	...	...	...	clay
	21.3 (70)	I	49 (1030)	29.3	0.995	clay
	23.8 (78)	I	227 (4730)	21.5	0.984	lime rock
F2N1	17.7 (58)	II	...	39.1	0.951	clay
	19.2 (63)	II	...	23.5	0.997	clay
	24.4 (80)	III	...	...	...	lime rock
	25.9 (85)	III	...	...	...	lime rock
	27.4 (90)	I	63 (1310)	31.9	0.992	clay

[a]Categories: (I) valid test; (II) residual ϕ only; and (III) invalid caused by progressive seating.

TABLE 4—*Smooth-plate BST results.*

Boring Number	Depth, m (ft)	Correlation r	c_s, kPa (psf)	ϕ_s, degrees	$c_{s/c}$	$\phi_{s/\phi}$	Material
F1N3	24.4 (80)	0.994	8.7 (181)	22.0	a	a	lime rock
	26.2 (86)	0.997	3.8 (79)	19.3	0.067	0.52	clay
	27.4 (90)	0.992	5.2 (109)	17.0	0.088	0.61	clay
F1S4	14.9 (49)	0.930[b]	21.4 (447)	35.5	a	a	clay
FiS3a	22.2 (73)	0.878[b]	38.7 (809)	23.3	a	a	gravelly clay
F2S1	13.7 (45)	0.983	19.0 (397)	26.2	a	a	shell
	21.9 (72)	0.990	8.3 (174)	24.4	0.169	0.86	clay
	23.5 (77)	0.993	14.6 (305)	31.7	0.064[c]	1.47[c]	lime rock
					CLAY AVERAGES		
					0.11	0.66	

[a] No conventional BST data from this depth
[b] Low correlation coefficient indicative of unsuccessful smooth plate test
[c] Not included in averages

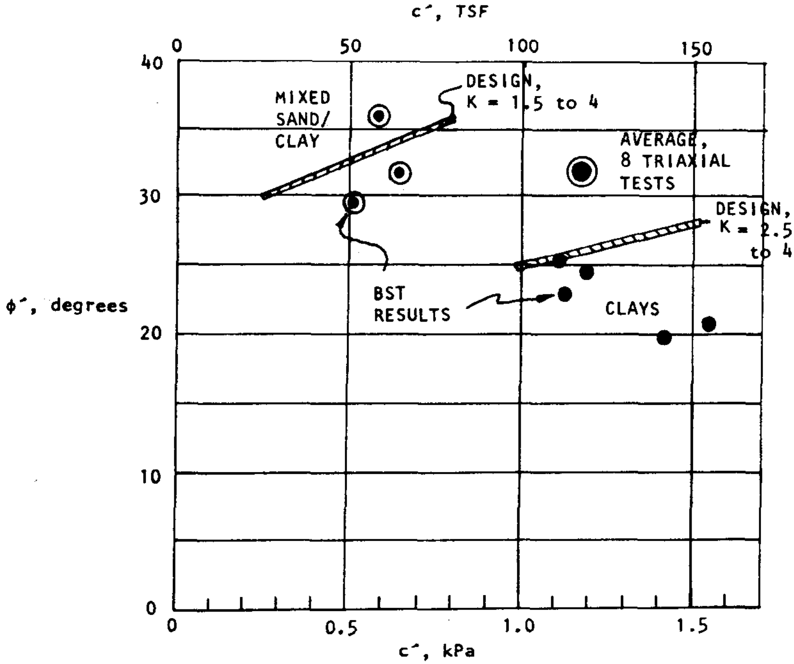

FIG. 3—*Comparative BST, triaxial test, and design values for ϕ and* c.

First, the 10 BSTs in clay gave an average value of about 25° for ϕ, or about what would be expected from fully drained shear tests on undisturbed samples of this attapulgite-montmorillonite clay with its average PI of 65% [*8*]. As noted previously, the stage test method for BSTs in clays does not allow time for significant drainage. However, in the highly overconsolidated, hard clays at the Skyway site the maximum applied normal stress was well below the preconsolidation pressure indicated by consolidation tests, so it appeared unlikely or even impossible that significant positive pore-pressures will develop. The close agreement of the average BST measured undrained ϕ_u with expectations for ϕ' from clay mineralogy, and measured ϕ' from triaxial tests, appeared to confirm that negligible Δu existed at undrained BST shear failure. A parametric study of the effect of various levels of Δu in the subsequent Eq 1 also showed the reasonableness of small values of Δu. The geotechnical engineers therefore subsequently assumed that $\Delta u/\sigma'_v = 0$ when analyzing the BST results where σ'_v is vertical effective stress.

The geotechnical engineers used the equations developed by Durgunoglu and Mitchell [*9*] for the Skyway Bridge foundation bearing capacity analyses. These require knowing the horizontal stress coefficient K or K' when using an effective stress analysis. The BST does not give information about K'. However, the

DMTs and the MPMTs do give such information. From these results, the engineers discovered the hard clays also had high values of K', typically from about 2 to 5. Other evidence also indicated high lateral pressures: for example, piles and shafts developed much higher shear resistances and "freeze" effects than what might ordinarily occur with more ordinary lower K'; in addition, a few research tests with the stepped blade [*10*] gave high K' results; Menard pressuremeter tests gave high initial stress p_o results; and boreholes tended to swell and reduce diameter. Therefore, K' values of from 2 to 5 used in the hard clays were consistent with the findings from the many DMTs and PMTs. It can be shown by use of Mohr circles that such high K' values are possible without passive failure because of the cohesion in these clays resulting from cementation bonding. Similar very high K' values were recently reported in Houston clay [*10*, p. 1415].

All the pieces now came together to allow the interpretation of the great number of s_u data to obtain consistent values of c', ϕ', and K'. Assuming the linear Mohr strength envelope, a vertical axial compression, a horizontal direction for the major principal stress, and expressing s_u in terms of effective stress components gives

$$s_u = c' \tan (45° + \phi'/2) + \sigma_v'/2(K' - \Delta u/\sigma_v')[\tan^2 (45° + \phi'/2) - 1] \quad (1)$$

Substituting the aforementioned values of $(\Delta u/\sigma_v')$ and K', and using the BST results in Table 3 as a guide, allowed solving for c' and ϕ' by trial from all the s_u data.

For the main pier design for the capacity of a single drilled shaft, average values of approximately $c' = 100$ kPa, $\phi' = 26°$, and $K' = 3.7$ eventually were used for the clay layers in the calculations. These compare with the average $c' = 91$ kPa and $\phi' = 25.7°$ from the 10 BSTs in this clay. This demonstrates the strong influence the BST data had in guiding the interpretation of the many more tests of other types. Subsequent pile capacity analysis of results from 13 pile load tests to failure gave a predicted/measured ratio of 1.03. The reader interested in more of the details of the design methods and soil strength values used for the replacement Skyway Bridge foundations should review Ref *11*.[4]

Other Modeling Aspects of BST

Borehole Swell

The BST tests the sides of a borehole after it has remained drilled and open for a period of time. As noted in the previous section of this paper, the clay layers have a very high K', and the clay tends to swell into the borehole. This would tend to weaken the clay and reduce ϕ' or c' or both as indicated by the comparative BST and triaxial data of Fig. 3. But, a similar sequence happens during the construction of a drilled shaft, and the sides of the shaft can swell and weaken before the shaft concrete gets poured, sets, and the structure loads the clay. The BST thus may model in part the effects of shaft construction on the clay strength.

[4]Sonnenfeld, S. L., Schmertmann, J. H., and Williams, R. C., this publication, pp. 515–535.

BST with Smooth Steel Plates

The upper part of the drilled shafts required the installation of steel casing to maintain the shaft hole for the subsequent concrete pour. Therefore a part of the total drilled shaft bearing load capacity would come from the shear of steel against the soil. The same case holds for the shear of any steel piles used separately or as composite tips below concrete piles or shafts. The Mohr shear strength components of the site soils against the relatively smooth steel might be significantly lower than against the rough sides of a poured concrete drilled shaft. This leads to the use of comparative BSTs using smooth steel plates instead of the toothed plate surfaces normally used.

The results from the smooth versus toothed plate BSTs are compared and averaged in Table 3. Six of the eight smooth-plate tests yielded acceptably high correlation coefficients. Of these, three were in clay and three were in lime rock and shell. The smooth-plate tests were of necessity displaced by 0.3 m (1 ft) depth from the locations of the conventional tests, which may explain an errant result from the lime rock. For the three clay tests, the ratio of soil-steel adhesion c_s to soil cohesion c varied from 0.06 to 0.17, averaging 0.11, whereas the comparable sliding versus internal friction ratios ϕ_s/ϕ varied from 0.5 to 0.9, averaging 0.66. For design, the strengths against smooth steel therefore were obtained by reducing c' values to $0.10c'$, and ϕ' values to $0.70\phi'$. The engineers also chose to use these same reduction factors for clay shear strength against the also relatively smooth surfaces of precast concrete piles. This modeling versatility in the BST method thus permitted a site-specific quantitative adjustment for c' and ϕ' instead of depending on empirical relationships of unknown accuracy.

Conclusions

1. The borehole shear test (BST) equipment was successfully adapted for measuring shearing strength in situ and evaluating cohesion c and friction angle ϕ in offshore bottom sediments. All tests were conducted in mudded holes.

2. The stage-testing mode of BSTs was extended for the first time for testing hard clays to even harder claystones, thereby increasing the precision and substantially reducing testing time compared to running a series of single-point tests. In harder materials, such as the lime rock, high-pressure single-point rock testing techniques still are required.

3. The BST data could be fit into three categories: (I) indicate of shear failure at stress levels in accord with the Mohr-Coulomb failure criteria; (II) where filling of the tooth spaces with debris has resulted in only a residual failure envelope indicated by little or no cohesion; and (III) where progressive seating of the shear plate teeth yields an anomalously high friction angle and an apparent negative cohesion. Fitting of data into the three categories thus depends for the

most part on the indicated cohesion, and whether .t is realistic for the material tested, unrealistically low, or negative, the latter being a physical impossibility. Initial test points in many tests fall into Category III until at higher normal stresses plate seating becomes complete, and a valid Category I test results.

4. Of the 18 tests performed in hard clay, 10 gave Category I Mohr-Coulomb shear failure envelopes, 5 gave Category II residual strength envelopes, and 3 gave invalid Category III tests. The data in the first category proved very useful helping to establish the negligible pore-pressure behavior of the hard clay layers, and guiding the interpretation for the clay effective stress strength components from a large number of other types of in-situ shear tests. These components were then applied to the bearing capacity design of the driven pile and drilled shaft foundations for the replacement Sunshine Skyway Bridge in Florida.

5. The use of BSTs with smooth steel plates substituted for the conventional shear plates allowed a site-specific estimate of the reductions in c' and ϕ' that would occur when the site soils shear against relatively smooth surfaces, such as steel piles and the steel casing around a part of the length of drilled shafts.

Acknowledgments

The Florida Department of Transportation (FDOT) has kindly given permission for the authors to use the various data presented herein and obtained from the Sunshine Skyway Bridge Replacement Project. Schmertmann & Crapps, Inc., Gainesville, FL, acting as consultants to Figg & Muller, Inc., Tallahassee, FL, performed the geotechnical engineering for the foundations for all the alternate designs for the project. Figg & Muller, Inc. had the contract from the FDOT for the structural design of the concrete alternate that won the bid competition. FDOT has begun construction of the bridge. Borehole shear tests were conducted by the first and third authors, acting as consultants to Schmertmann and Crapps, Inc.

APPENDIX

Fluid Pressure Correction to Applied Normal Stress

a. Frontal shear plate area	$6.45\ (10)^{-4}\ \mathrm{m}^2$	$0.00694\ \mathrm{ft}^2$
b. Seawater unit weight	$\times\ 10.05\ \mathrm{kN/m}^3$	$\times\ 64\ \mathrm{lb/ft}^3$
c. Closing force per unit depth	$6.48\ (10)^{-3}\ \mathrm{kN/m}$	$0.444\ \mathrm{lb/ft}$
d. Back-plate area	$5.18\ (10)^{-4}\ \mathrm{m}^2$	$0.00558\ \mathrm{ft}^2$
e. Mud unit weight	$\times\ 10.68\ \mathrm{kN/m}^3$	$\times\ 68\ \mathrm{lb/ft}^3$
f. Opening force per unit depth	$5.53\ (10)^{-3}\ \mathrm{kN/m}$	$0.380\ \mathrm{lb/ft}$
g. Net force per unit depth	$0.95\ (10)^{-3}\ \mathrm{kN/m}$	$0.065\ \mathrm{lb/ft}$
h. Nominal plate area	$6.45\ (10)^{-4}\ \mathrm{m}^2$	$1.0\ \mathrm{in.}^2$
i. Depth correction to σ_n	$-\ 1.47\ \mathrm{kN/m^2/m}$	$-\ 0.065\ \mathrm{psi/ft}$
	$-\ 1.47\ \mathrm{kPa/m}$	

Calculations: $c = a \times b; f = d \times e; g = c - f; i = g \div h.$

References

[1] Handy, R. L., and Fox, N. S., "A Soil Borehole Direct Shear Test Device," *Highway Research News*, No. 27, Spring 1967, pp. 42–51.

[2] Lutenegger, A. J., Remmes, B. D., and Handy, R. L., "Borehole Shear Test for Stiff Soil," Journal of the Geotechnical Engineering Division, *Proceedings of the American Society of Civil Engineers*, Vol. 104, No. GT11, Nov. 1978, pp. 1403–1407.

[3] Handy, R. L., Pitt, J. M., Engle, L. E., and Klockow, D. E., "Rock Borehole Shear Test," *Proceedings of the 17th U.S. Symposium on Rock Mechanics*, Paper 4B6, 1976.

[4] Demartinecourt, J. P. and Bauer, G. E., "The Modified Borehole Shear Device," *Geotechnical Testing Journal*, Vol. 6, No. 1, March 1983, pp. 24–29.

[5] Schmertmann, J. H., "The Borehole Shear Test," *ASCE Specialty Conference on In Situ Measurement of Soil Properties*, Vol. 2, American Society of Civil Engineers, New York, 1975, pp. 57–138.

[6] Easton, C. N., Lohnes, R. A., and Handy, R. L., "In-place c-ϕ Shear Strength Test for Subaqueous Sediments," *Proceedings of the International Symposium on the Engineering Properties of Sea-Floor Soils and Their Identification*, UNESCO, Seattle, WA, 1971, pp. 26–36.

[7] Audibert, J. and Aggarwal, D., *Study to Investigate the Effects of Skin Friction on the Performance of Drilled Shafts in Cohesive Soils, Vol. 1, Field Investigation*, U.S. Army Waterways Experiment Station Technical Report GL-82-1, Vicksburg, MS, 1982.

[8] Kenney, T. C., Discussion, *Proceedings of the American Society of Civil Engineers*, Vol. 85, No. SM3, p. 73; reproduced in Lambe, T. W. and Whitman, R. V., *Soil Mechanics*, John Wiley and Sons, Inc., New York, 1969, p. 307.

[9] Durgunoglu, H. T. and Mitchell, J. K., "Static Penetration Resistance of Soils—Analysis," *ASCE Specialty Conference on In Situ Measurement of Soil Properties*, Vol. 1, American Society of Civil Engineers, New York, 1975, pp. 151–171.

[10] Handy, R. L., Remmes, B., Moldt, S., Lutenegger, A., and Trott, G., "In Situ Stress Determination by Iowa Stepped Blade," *Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers*, Vol. 108, No. GT11, Nov. 1982, pp. 1405–1422.

[11] Schmertmann, J. H. and Crapps, D. K., "Use of In Situ Penetration Tests to Aid Pile Design and Installation," *Proceedings*, GEO-PILE '83 Conference, Associated Pile and Fitting Corp., April 27–29, Hollywood, FL, pp. 27–47.