

Borehole shear tests in stiff London and Gault Clay

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ABSTRACT: The Borehole Shear Test (BST) provides direct measurement of the friction angle and cohesion of soils by performing a series of “direct” shear tests on the sides of a borehole at different applied normal stresses. To evaluate the applicability of the test to stiff clays in the U.K., tests were performed at several depths at two sites; Canon’s Park – London Clay and Madingley – Gault Clay. The results of the tests are presented along with results of laboratory characterization tests on samples obtained from each of the test depths. BST results are compared with published results from laboratory shear strength tests on these two clays. The results demonstrate that the BST can provide good results and is a viable in situ test for characterizing the shear strength of stiff clays in the U.K.

1 INTRODUCTION

The Borehole Shear Test (BST) was introduced in the late 1960s as a rapid and direct method of measuring the in situ shear strength of soils by performing a series of direct shear tests on the sides of a borehole. The test was originally presented by Handy & Fox (1967) and was intended to provide independent measurements of the drained friction angle and cohesion of soils to define the Mohr-Coulomb failure envelope. It is the only test available that is designed to give direct measurements of these parameters in situ. Test results have previously been performed on a wide range of fine-grained soil deposits, including for example loess (Lohnes & Handy 1968); stiff glacial till (Lutenecker et al. 1978); clay shale (Ruenkraitersa & Pimsarn 1982; stiff clay (Miller et al. 1998); sensitive marine clay (Demartinecourt & Bauer 1983); and Piedmont residual soil (Lutenecker & Adams 1999). The results obtained from the test have been found to be particularly useful in slope stability analyses (e.g., Handy 1986; Yang et al. 2006)). In spite of the fact that the BST is now over 40 years old, many geotechnical engineers may be unfamiliar with this test. In very stiff clays there is still some uncertainty regarding the interpretation of test results. To the authors knowledge the tests reported in this paper represent the first tests performed in the U.K.

2 BOREHOLE SHEAR TEST

The BST is conducted by advancing a borehole into the soil to a desired test depth, forming a test cavity of correct size and then lowering in a cylindrical load device that is expanded outward using controlled gas pressure against the sides of the hole. The load mechanism is equipped with plates that have sharp wedge shaped “teeth” that grip the soil. Once the soil along the walls of the hole is gripped, the load mechanism is slowly pulled upward to force a shear failure to take place and a peak shearing force is recorded. This concept is illustrated in schematic diagram of Figure 1. This procedure gives a single measurement of both the applied normal stress and peak shear stress at failure. In this way, the BST simulates, at least in part, the procedure used in laboratory direct shear box tests where normal stress is applied to a specimen in the shear box and then failure is produced by shearing the soil. In the BST, much like in the laboratory shear box, this procedure is repeated a number of times using different values of normal stress until sufficient pairs of data points (normal and peak shear stress) are obtained to define the failure envelope of the soil. In the laboratory, this is typically accomplished using different specimens but in the BST, multi-stage testing is performed and the shearing takes place successively in the same zone of soil.

In multi-stage testing, shearing is induced repeatedly in the same soil mass, at successively higher normal stresses. This is similar to laboratory multi-stage testing and improves test precision, saves time, and encourages dissipation of pore water pressures. This procedure is allowable if the consolidated strength of the soil close to the shear plates exceeds that of the adjacent undisturbed soil, i.e., the previously sheared soil consolidates and builds up a cake of soil that rides with the shear plates. In this way, successive shear surfaces move outward to engage fresh soil. The result is generally an exceptionally linear failure envelope in sands, silts, and soft to medium consistency clays. Less experience has been gained in very firm to stiff clays.

3 TEST SITES

Tests were performed at two UK sites to evaluate the applicability of the BST in stiff heavily overconsolidated fissured clays (London – Canons Park and Gault – Madingley). Both sites have been used extensively in the past and the geotechnical properties of the clays at both sites are well documented (Powell & Uglow 1988; Powell & Quarterman 1988; Powell 1990; Butcher & Lord 1993). The Canons Park site has a layer of reworked London Clay over the upper 3 to 4 meters. Borehole Shear Tests were performed in the weathered upper 8 m at both sites.

A number of previous investigations have been performed in which laboratory drained strength parameters have been obtained for London Clay. Table 1 gives a summary of some previously reported values for the drained failure envelopes for weathered London Clay. Bishop et al. (1965) found that the failure envelope of deep samples of London Clay for peak conditions in drained shear showed a change in slope in passing from low stress to high stress. They found that from drained triaxial shear tests the drained parameters changed from $\phi' = 26^\circ$; $c' = 124$ kPa for confining stresses of 70–700 kPa to $\phi' = 10^\circ$; $c' = 758$ kPa for higher confining stresses. They also found essentially no difference in the failure envelopes between vertical and horizontal samples. This change in the failure envelope also appears to occur in drained shear box tests (e.g., Hight et al. 2003). More recently Hight et al. (2007) have shown that different envelopes can be

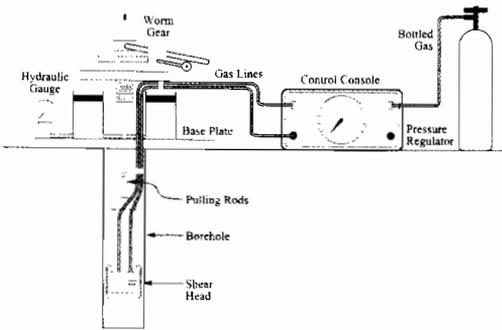


Figure 1. Schematic of the Borehole Shear Test.

Table 1. Previously reported drained failure envelopes for weathered London clay.

Case no.	Location	Φ' (deg.)	c' (kPa)	Reference	
1	Various	20	12	Henkel (1957)	
2	Chingford	19	11	Skempton & DeLony (1957)	
	Sydenham Hill	19	10		
	Uxbridge	21	13		
	Queen Victoria Station	22	10		
	Gresham Street	19	13		
3	Various	20	15	Skempton (1964)	
4	London	26	125	Bishop et al. (1965)	
		10	758		Low confining stress
		10	10		High confining stress
5	Various	20	10	Skempton (1977)	
6	Various	23	50	Hight et al. (2003) blue	
		23	20		Weathered brown peak
		20	0		Weathered brown post peak
7	Various	20	0	Skempton et al. (1969) fissure strength	
8	Heathrow	12	0	Hight et al. (2007) blue clay	
		20	0		lower bound post rupture
		17	0		typical post rupture comp
		17	0		typical post rupture extension
		25	30		upper zone peak
9	Brent Cross	20	20	Atkinson et al. (1991)	
		12	0		Simple shear brown Post peak

assigned to the different zones within the London clay (see Table 1) and also show that orientation can have significant effects of the derived strength parameters.

Table 2 gives some reported effective stress parameters for Gault clay mainly derived from undrained testing with pore water pressure measurements. It is not always clear in the references as to how the values were obtained; the high values from Ng (1992) were obtained from large block samples were as the others were almost certainly derived from tube samples of varying quality. Ng's samples showed pronounced brittleness post peak but the lower bound of his data falls close to other reported values.

4 TEST PROCEDURES

A total of 14 individual tests were conducted over a period of four days, including test drilling. The test procedures recommended by Lutenegeger (1987) were used. Test cavities were obtained using a 76 mm diameter thin-walled tube to minimize borehole wall disturbance. In stiff fine-grained soils this method of creating a test cavity is generally preferred. Smearing of the borehole wall is minimized using a thin-walled sampling tube. A consolidation time of 10 minutes was allowed between the application of normal stress and the beginning of the shear phase. Multi-stage testing was used for all tests. For most tests, six data points representing six different normal stresses were obtained. A least-squares linear regression was performed on each data set in order to interpret the Mohr-Coulomb failure envelope.

5 RESULTS

Table 3 presents a summary of the results of laboratory tests performed on soil trimmings obtained at various test depths at both sites. Soils at both sites are high plasticity clays with similar Atterberg Limits and clay fraction and similar mineralogy (cation exchange

capacity, CEC) however the Gault clay is more calcareous and contains a much higher amount of carbonates than the London clay as indicated in Table 3. Figure 2 shows typical Borehole Shear Test results obtained at

Table 3. Summary of laboratory characterization tests.

Depth (m)	Clay (%)		C.E.C. (meq./100 gm)	Carbonates (%)
	L.L.	P.L.		
Canon's Park – London Clay				
2	71.0	23.7	55.5	25.9
3	70.8	25.9	63.8	22.1
4	68.0	22.3	55.1	21.7
5	–	–	48.0	39.1
6	60.0	27.0	60.4	42.6
7	65.3	21.6	57.6	25.1
Madingley – Gault Clay				
1	75.8	26.9	58.7	27.9
2	76.2	25.5	58.3	27.5
6	66.9	23.3	54.0	45.9
8	–	–	63.4	29.7

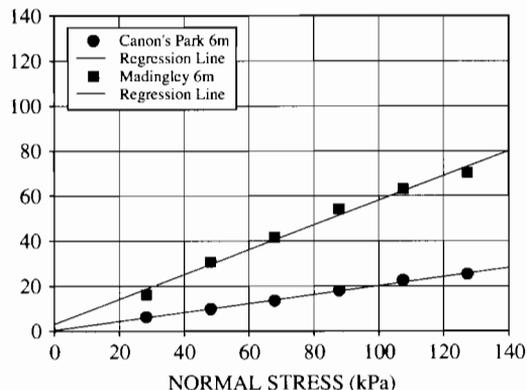


Figure 2. Typical Borehole Shear Test Results at both sites.

Table 2. Previously reported drained failure envelopes for Gault clay.

Case no.	Location	Φ' (deg.)	c' (kPa)	Reference
1	East Anglia	23.5	12.7	Ratnam (2002)
		27.3	23.6	
		24.2	29.9	
		25	22.6	
2	Various	24.5	13	Garrett & Barnes (1984) (design value adopted)
3	Lion Yard	32	3	Ng (1992) compression
4	Lion Yard	34	2	Ng (1992) extension
5	Lion Yard	24	10	Lings et al. (1991) design value
6	M25	29	10	Carder & Barker (2005)
	M25	13.5	1.5	Carder & Barker (2005) (residual)

Table 4. Summary of BST results.

Test no.	Test depth (m)	Friction angle Φ' (deg.)	Cohesion c' (kPa)	Linear regression coefficient R^2
Canon's Park – London Clay				
1*	2	38.5	-13.0	-
2*	3	32.5	-6.5	-
3	4	13.0	1.7	0.977
4	5	11.5	5.5	0.987
5	6	11.0	0.5	0.996
6	7	9.2	2.0	0.995
Madingley – Gault Clay				
7	1	21.0	2.2	0.969
8	2	32.0	0.1	0.995
9	3	26.7	0.0	0.995
10	4	24.7	6.5	0.983
11	5	17.7	15.4	0.974
12	6	27.8	3.3	0.988
13	7	27.2	0.5	0.998
14	8	24.1	2.6	0.999

* Test performed with low normal stresses – results considered invalid

both sites. A summary of the individual tests is given in Table 4. The Borehole Shear Test results generally show very strong linear correlations as indicated by the high linear regression coefficients given in Table 4 which is very typical for this test.

Tests were initially performed at Canon's Park. The first two tests at depth of 2 m and 3 m were conducted using normal stresses of 10–75 kPa and 30–90 kPa, respectively. These two tests gave very high friction angles and large negative cohesion intercepts and were considered invalid. This behavior may be the result of the shear plates encountering isolated gravel particles at these depths in the reworked London clay or may also be because of the low normal stresses used in these initial tests. All subsequent tests were performed with applied normal stresses in the range of 30 to 130 kPa. The test results obtained in the London Clay generally give lower values of ϕ' and c' than previously reported peak strength envelopes for this range of normal stresses but are actually similar in magnitude to reported values for residual strength envelopes. The results obtained in the Gault Clay are significantly higher than reported residual values and more comparable to peak strength envelopes reported from laboratory tests and are more realistic considering the geologic history of the site and the geotechnical characteristic for this clay as given in Table 3. This upper zone of London Clay at the Canon's Park site may have been significantly reworked over time to resulting in loss of peak strength.

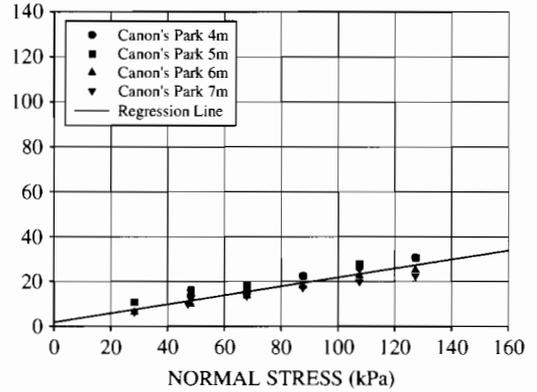


Figure 3. "Global" Results for Canon's Park – London Clay.

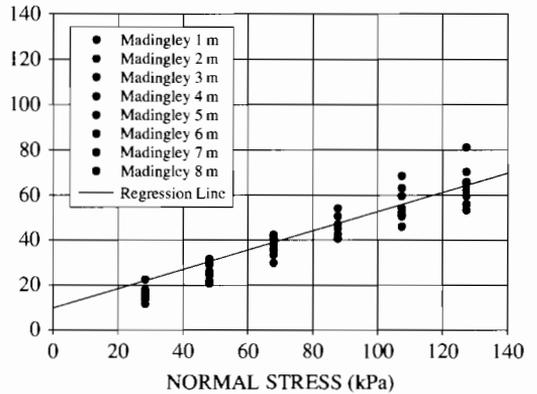


Figure 4. "Global" Results for Madingley – Gault Clay.

6 GLOBAL RESULTS

The test results at each site might also be considered in a "global" context by combining the individual results from all tests into a single population. Considering the relative uniformity of the soils at each site as suggested by the laboratory test results presented in Table 2 this seems reasonable. Figures 3 and 4 give the combined results of the four valid tests performed in the London Clay at Canon's Park and the eight tests performed in the Gault Clay at Madingley. These results suggest "global" failure envelopes of $\phi' = 11.3^\circ$; $c' = 2.4$ kPa for the London Clay ($r^2 = 0.848$) and $\phi' = 25.5^\circ$; $c' = 4.2$ kPa ($r^2 = 0.888$) for the Gault Clay. The results from Canon's Park indicate values closer to residual strength which may be also be suggested by some fissuring of this material while the results from Madingley indicate values closer to reported peak strength which may be indicative of the more massive structure in this deposit. Deeper tests in the less weathered zone may help show any differences in the behavior.

However, the results of tests at both sites provide an initial indication that the test may be applicable to these deposits.

7 SUMMARY

A series of Borehole Shear Tests performed at two sites in the UK in very stiff clays indicates that the test may be used to obtain strength envelopes in typical UK stiff clays. The results obtained at shallow depths in upper weathered London Clay gave lower failure envelopes than expected which may indicate that the clay in this upper zone has undergone significant changes or reworking since deposition to produce residual strength envelopes in this upper zone of the deposit. In Gault clay, the failure envelopes were more in line with what might generally be expected for peak failure envelopes. The test is rapid to perform and no difficulties were experienced during the testing. The soils at both sites are considered very stiff and it is unlikely that the test results are influenced by drilling procedures. In softer materials the disturbance effects at the borehole wall can create difficulty in obtaining reliable results. The tests reported at both sites in this investigation appear to be indicative of the in place strength characteristics of the two deposits within the zones tested.

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