FEATURE ARTICLES

A Soil Bore-Hole Direct-Shear Test Device

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A relatively simple device has been developed for measuring internal friction and cohesion in soils. A hole is bored, and two curved contact plates are expanded inside the hole to apply a pressure normal to its sides (Fig. 1). A shearing stress is then applied by pulling or pushing the expanded device axially along the hole (Fig. 2). The result is essentially a direct shear test on soil at the sides of the bore-hole.

Tests were conducted on various soils ranging from sand to clay. The field test data were then compared to laboratory direct and triaxial shear data on thin-walled tube samples from the same depths in the same holes. This was done to determine what empirical factors, if any, might be required to make

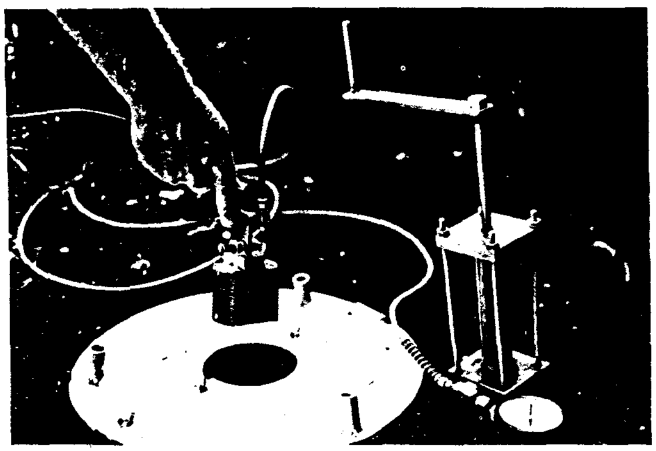

Figure 1. Three-inch bore-hole shear device ready for a test. Pump and gage at right are for expansion of the device inside the hole.

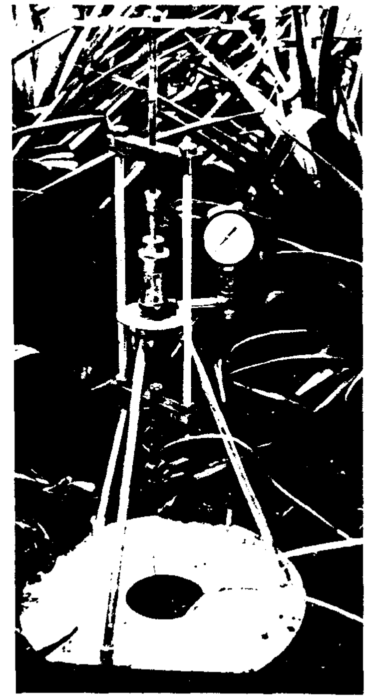

Figure 2. Apparatus assembled for pulling; pulling to obtain ultimate stress is repeated at several expansion pressures for a Mohr-Coulomb failure envelope.

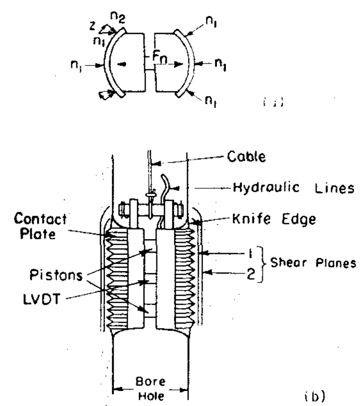

Figure 3. Schematic of bore-hole shear device: (a) circumferential shearing resistance z must be minimized for uniform distribution for contact pressure n_1, and (b) knife edges minimize end resistance.

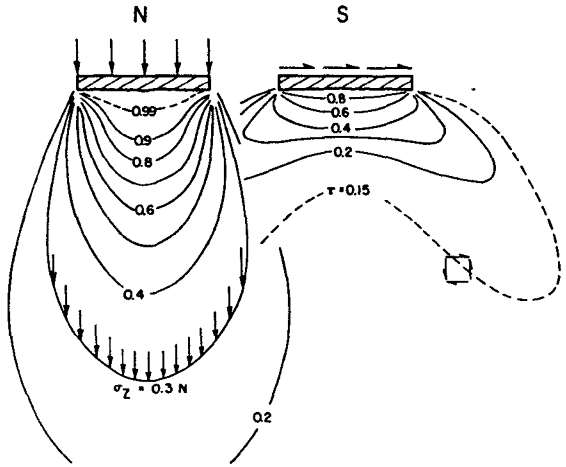

Figure 4. Normal and shearing stress distributions in an elastic medium.

the field test more meaningful. All analyses were made on a total stress basis, since pore pressure measurement did not seem appropriate to the initial feasibility study. However, most soils selected for tests had natural moisture contents well below saturation.

Geometrical Considerations

The use of rigid pressure plates meant that to obtain a uniform contact pressure, circumferential shear resistance should be minimized (Fig. 3). Grooves to engage the soil were therefore cut circumferentially around the plates. End resistance of the plates was minimized by sharpening.

Elasticity theory indicates that shearing stress applied to a strip on a soil surface will dissipate more rapidly than an applied normal pressure (Fig. 4).* Therefore shear failure should occur close to the contact plates and in a zone of normally consolidated soil. V-shaped teeth and grooves were cut in the contact plates to engage the soil; hopefully they would also densify and strengthen the soil sufficiently that shear plane would move out away from the plates. This was confirmed in later testing; a compacted soil cake about 0.05 to 0.02 in. thick was found to adhere to the plates after a successful test.

*In addition to stresses shown in Figure 4, application of a shearing stress will superimpose a normal stress that is compressive at the front of the loaded area and tensile at the rear; this effect should balance out so long as the strength envelope is linear. Similarly, application of the normal stress will induce equal and oppositely oriented shearing stresses under opposite ends of the loaded area.

Progressive Relocation of the Shear Plane

Several strengths at several normal pressures are required to determine the Mohr-Coulomb failure envelope. Although we anticipated moving the instrument to untested soil for each new point, tests which were conducted with and without relocating gave closely comparable results. Therefore all test series reported were run without moving the instrument to untested soil. This not only gave a considerable saving in time, but usually gave better failure envelopes by removing the sampling variable. Since all tests in a series are conducted at essentially the same depth in the same hole, they are in essentially the same soil.

The question remains as to why repeated shearing at the same location does not give low values of shearing strength. One possibility is that during or after shearing, the major principal stress (predicatable from the Mohr circle tangent at the test point) causes sufficient compression to "seal" the shear plane and cause it to move outward to a lower stress region. In soils with appreciable internal friction, some test series have been continued to include 10 or more points in the failure envelope, until further testing was prevented by full expansion of the instrument.

Shearing usually results in a slight decrease in expansion pressure, and in some soils each failure occurs as a distinct slip accompanied by an audible crunching noise, suggesting compaction of soil in the shear zone. Replenishing the expansion pressure allows new valid shear failures even at the same expansion pressure, an advantage if check points are needed. In fact, a "remolded" type of test is best performed by continuing shear past failure without replenishing the expansion pressure. Except for the tendency to bow upward after failure, stress-strain curves closely resemble those from laboratory direct shear testing, strain usually being about 0.2 to 0.3 in. at ultimate stress.

From the typical test behavior we infer that after shear failure, the thin layer of soil grains participating in the failure is compacted and added to the dense soil cake adhering to the pressure plates. If this is correct, subsequent shearing should occur at the outer surface of the shear cake, in a zone of normally consolidated but otherwise unaltered soil. The apparent tendency to "heal" the shear zone was much less pronounced in a clay having a low apparent friction angle.

Operation of the Bore-Hole Shear Device

A bore-hole test sequence begins with leveling the ground and boring and logging the hole. As in most soil boring operations, disturbed samples may be taken for correlation to other holes or for moisture content or classification tests. The hole is bored somewhat smaller than the nominal diameter of the shear device in its closed position. (Tests described in this paper were in holes augered 4 in. in diameter for a 5-in. apparatus, or 2.5 in. for a 3-in. apparatus.) The hole should extend several inches or more below the full depth of the investigation. It may be bored out larger above the test zone to facilitate lowering the instrument.

Second, the hole is reamed in the test zone with a sharpened cylindrical tool or hole saw. Cuttings stay in the tool or fall to the bottom and are removed by augering, or if the hole is deep enough, may be left in the bottom.

Third, the base platform is placed on the ground over the hole, and the test apparatus is lowered on a cable, chain, or rod to the shallowest desired test depth.

Fourth, a predetermined expansion pressure is applied hydraulically to the device, and maintained constant until the desired amount of consolidation has

TABLE 2

COMPARISON DATA FROM BORE-HOLE (B.H.), DIRECT SHEAR (D.S.) AND TRIAXIAL (T) TESTS ON THREE SOILS

Soil	No. Test Series	φ, °			c, psi		
		B.H.	D.S.	T	B.H.	D.S.	T
Sand	5	36.9 ± 0.5	36.5 ± 0.3	35.8[b]	0.4 ± 0.3	0.3 ± 0.3	(0.3)[b]
Loam	2	44.8	44.0[c]	–	1.2	0.8[c]	–
Clay	4	5.0 ± 2.6	–	2.5 ± 2.3[c]	10.8 ± 1.0	–	9.0 ± 2.3[c]

[a]The ± entry indicates one standard deviation from the mean.
[b]Only one test performed.
[c]Only one test series.

An unexpected close agreement was obtained between bore-hole, direct shear, and triaxial test data, as given in Table 2. Since only one triaxial test could be performed, c was assumed to be 0.3 psi for the calculation of this φ.

Alluvial Loam. Two field tests were conducted in an alluvial loam selected for properties intermediate between the sand and the clay. Results are shown in Figure 5b, together with five direct shear test points on the same soil. Again the field and lab test results showed unexpectedly close agreement with no need for empirical connection factors (Table 2). Both field and lab tests gave low shear values at high normal pressures, probably due to pore water pressure.

Clay. Four field tests were conducted in a heavy-textured montmorillonite clay, with consolidation times of 10 min for the first point and 5 min for each additional point, and shear by pulling at a rate of 0.46 in./min. Low shear values at normal pressures of less than 10 psi (shown in Fig. 5c) indicated incomplete seating, and only two or three points in each test gave a reasonable shear envelope, after which shearing resistance was sharply reduced, probably due to pore pressure and/or repeated shearing on the same plane. However, because of soil variations, the point pairs obtained without relocating the apparatus gave more consistent data than would have been obtained had the apparatus been relocated for each new test point.

TABLE 3

COMPARISON DATA ON LOESS AT THE LOVELAND TYPE SECTION, WESTERN IOWA

Vert. Dist. from Top of Hill, Ft[a]	φ, °			c, psi		
	B.H.	D.S.	T[c]	B.H.	D.S.[b]	T[c]
0.5	24.0	–	–	0.7	–	–
7	15.2	–	–	1.6	–	–
10	–	24.7	–	–	0.2	–
33	24.3	–	–	(1.7)	–	–
77	25.4	24.1	28.9	1.0	1.3	2.4
99	29.5	–	–	1.4	–	–
131[d]	28.5	24.6	–	4.3	1.8	–

[a]Depths were measured from top of the cut (depth 30 ft) rather than top of the hill (5).
[b]Direct shear data are from Olsen (3).
[c]Triaxial data are from Akiyama (4).
[d]Basal Wisconsin (Farmdale) loess having a higher clay content.

Three of the field tests gave practically identical friction angles between 6.0° and 7.0°, and one made at higher normal pressures gave φ = 0.5°. The three triaxial tests gave φ from 5.0° to -0.6°, depending on which Mohr circles were paired for common tangents. Although the data are closely comparable (Table 2), information on pore pressures is needed. A pore pressure transducer has been added to a new test unit, but data are not available at this writing.

The field tests gave c for the clay from 9.4 to 12.2 psi, averaging 10.8, whereas the triaxial tests gave an average c of 9.0 psi. Since the field tests indicated a slightly higher cohesion than the laboratory tests, we can infer that slippage was negligible between the shear plates and the soil.

Loess. Several series of field tests have been performed at various depths in Wisconsin-age loess, a wind-deposited silt widely recognized for its ability to stand in steep faces.

Systematic laboratory tests were attempted on western Iowa loess in 1958 (3) and 1963 (4), but due to sampling difficulties the results were rather sparse and inconclusive. For comparison, the six complete bore-hole shear test failure envelopes were obtained in only four hours.

Results of the bore-hole, direct shear, and triaxial tests are given in Table 3. The bore-hole tests were possible even in very soft soil, and the lowest friction angle, 15.2°, occurred in a gopher zone. If we disregard this value, the average φ from five bore-hole tests was 26.3°, which may be compared to 24.5° from three series of direct-shear tests and 28.9° from one series of triaxial tests. The cohesion of loess varies depending on the moisture content (2), but data from the three test methods are closely comparable.

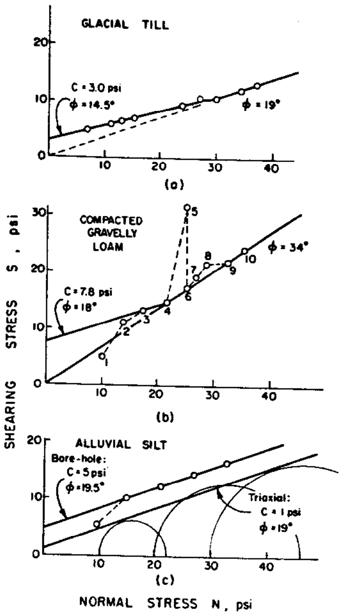

Figure 6. Tests on two highway soils and a damsite silt; triaxial tests on the silt were performed after saturation with water.

Glacial Till. One field test was performed in unoxidized glacial till which presumably had been preloaded under weight of the glacial ice. Results are shown in Figure 6a. A break in the two straight-line segments indicative of preloading occurs at a normal pressure of about 30 psi. The second line segment has a higher φ than the first and extrapolates through the origin, which is characteristic of preloading. Because of the dense, stony nature of the till, laboratory tests were not performed.

Gravelly Compacted Subgrade. Several field tests have been performed in an artificially compacted highway embankment soil composed of glacial till. Difficulty was experienced in augering the holes because of cobbles and gravel, and shear test results were sometimes quite erratic (Fig. 6b). However, by careful evaluation of the test points, useful information could be obtained.

The following system was devised for elimination of erroneous points: the line between two adjacent points cannot extrapolate to a negative cohesion, and cannot have a negative slope, or one of the points must be invalid. Thus in Figure 6b points 1, 5, 7 and 8 may be eliminated. This leaves points 2, 3, 4, 6, 9 and 10. The last four fall in a straight line with the origin, with point 3 indicative of preloading. The shear angles from these points were confirmed in a repeat test performed without gravel problems. The time for both test series (17 points) was less than three hours.

Hawaiian Alluvium. A bore-hole test was conducted at the site of the proposed Kahaluu Dam in Hawaii for comparison with laboratory consolidated-undrained triaxial tests. Texturally this soil is a silt containing 16 percent 5-μ clay. Comparison data are shown in Figure 6c and indicate close agreement of φ values, but a lower laboratory test c which probably relates to presaturation of the laboratory samples.

Conclusions

A new device is described for determining soil frictional parameters by tests conducted inside bored holes. Field trials and comparisons to laboratory triaxial and direct shear data indicate the following:

1. Bore-hole tests in a sand, a loam, several silts and a clay indicate close agreement of c's and φ's with laboratory consolidated-undrained triaxial and/or direct shear test data, and comparable precision.
2. A complete determination of the maximum shear vs. normal stress failure envelope may be made in 15 min to 1 hr, depending on the consolidation time required for a particular soil. The comparable field sampling plus laboratory direct-shear testing time would be a matter of many hours or even days.
3. Erroneous shear values resulting from incomplete seating, bulldozing, remolding, or occurrence of gravel are usually apparent and may be systematically eliminated.
4. Tests of preconsolidated glacial till and an artificially compacted highway embankment showed a dual shear envelope indicative of preloading.
5. Soils too weak for ordinary undisturbed sampling may be tested with the new device, so long as the soil maintains an open hole without the use of drilling mud. However, saturated sandy soils which cave without mud will probably be most economically tested by a dynamic cone penetration or similar test.
6. In soils with appreciable internal friction, a series of ten or more usable points has been obtained in the same test location, the limitation being maximum expansion of the apparatus. One explanation is that the shear planes relocate outward as soil adjacent to the apparatus compresses and gains strength. This tendency is less apparent in soils with low internal friction where only two or three points in a test series may be reliable.
7. Since each test series is performed on essentially the same soil rather than on different extruded and trimmed specimens, soil variability affects entire shear envelopes rather than individual points which make up an envelope, greatly simplifying interpretations. Tests may be precisely located in weak or questionable strata, since the tests are conducted after the hole is bored and logged.

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References

1. Scott, R. F. Principles of Soil Mechanics. Addison-Wesley, Reading, Mass., 1963.
2. ASTM Committee D-18. Procedures for Testing Soils. 4th Ed., 1964.
3. Olson, G. R. Direct Shear and Consolidation Tests of Undisturbed Loess. Unpublished M.S. thesis, Iowa State Univ. Library, Ames, 1958.
4. Akiyama, F. N. Shear Strength Properties of Western Iowa Loess. Unpublished M.S. thesis, Iowa State Univ. Library, Ames, 1963.
5. Daniels, R. B. and Handy, R. L. Suggested Relocation of the Type Locality of the Loveland Loess, Western Iowa. Jour. Geology 67:1, p. 114-119, 1959.