SOIL SHEAR TESTS BY MEANS OF ROTATING VANES

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SYNOPSIS

This paper describes tests and reports the results of vane shear tests performed in Chicago glacial clays. Shear tests were made to a depth of approximately 50 ft. Vanes at the end of a central shaft were forced into the ground at the bottom of the test hole and the shear strength of the soil was measured in situ by applying a torque to the shaft at a constant rate of 1 deg. per min. and recording torque versus angle of twist. This operation was repeated at intervals of depth. Soil samples were taken between every two vane tests for unconfined compression tests.

In the vane tests it was assumed that the soil sheared on a cylindrical surface whose axis coincided with that of the twisting shaft. The two plane surfaces forming the top and bottom of the cylindrical surface were also assumed to be shear surfaces. On the basis of these assumptions the shear strength of the soil was computed and compared with half the unconfined compressive strength.

Good aggreement was found to exist between the shear strengths obtained by the two methods in the constant strength depth, namely 20 ft. to 45 ft. The upper 20 ft. consisted of silty sand which was unsuitable for this type of test. The soil below 50 ft. was very hard and contained a considerable quantity of gravel. Only one vane test was performed below this level with a smaller vane than was used for the upper soil This was the lowest point to which the test was carried.

Curves of applied moment versus angle of twist showed one discontinuity in an otherwise smooth curve. This was true for nearly all data taken in the 20 to 45 ft range and suggests that at a certain shearing strain the soil structure underwent a sudden change The maximum shear stress occurred beyond this point. This discontinuity is probably due to the tranference of shear strength from an unstable skeleton structure to a constant strength clay matrix assuming the structural character of soft clay as outlined by Terzaghi.

Results of recent tests in Sweden (Carlson 1948) (*2*)[1] and more recently in England (Skempton 1948) (*10*) have indicated that shear strengths of certain clays obtained from laboratory tests differ very considerably from those obtained from shear tests of the soil in place. Shear strengths in place in both these investigations were obtained by rotating blades fastened to the end of a central shaft. The tests further indicated that shear strengths of clays near the surface are substantially the same by either method but for normally loaded clays at greater depths there is considerable difference in the shear values obtained by the two methods.

These discrepancies between vane tests in place and compression tests in the laboratory may be due to a number of causes. No definite conclusions can be drawn, however, until more data is available from locations representing a wide range of soil conditions.

[1] Italicized figures in parentheses refer to the list of references at the end of the paper

It was with this thought in mind that the authors undertook to perform vane tests for a given site in the Chicago area. The Raymond Concrete Pile Company co-operated in this undertaking by making the required borings. The apparatus for measuring the shear values in place was built at Illinois Tech and was fashioned after that used by Skempton.

GENERAL DESCRIPTION OF FIELD TESTS

The site selected for the tests was at Dearborn and 32nd Streets on the campus of the Illinois Institute of Technology in Chicago. This was for reasons of convenience rather than the particular soil conditions. However, it is well known that except for erratic variations in the distances to bed rock south of the Loop, the Chicago area differs very little in the over-all general soil topography. There are, of course, local variations, but the boring log shown in Figure 1 is very similar to a great number of boring logs taken throughout the Chicago area (*7*).

Three wash borings were made in all, spaced approximately ten feet apart.

Hole No 1 was 2½ in. in diameter and was made for the purpose of determining the type of soil, for making penetration tests and obtaining samples for indices, water content and unconfined compression tests It was carried to a depth of 81 ft 8 in at which elevation heavy gravel was encountered The hole was cased to a depth of 20 ft. below the surface and remained open without casing for the remainder of the depth. Samples 1 2 in in diameter were taken at intervals of approximately 5 ft.

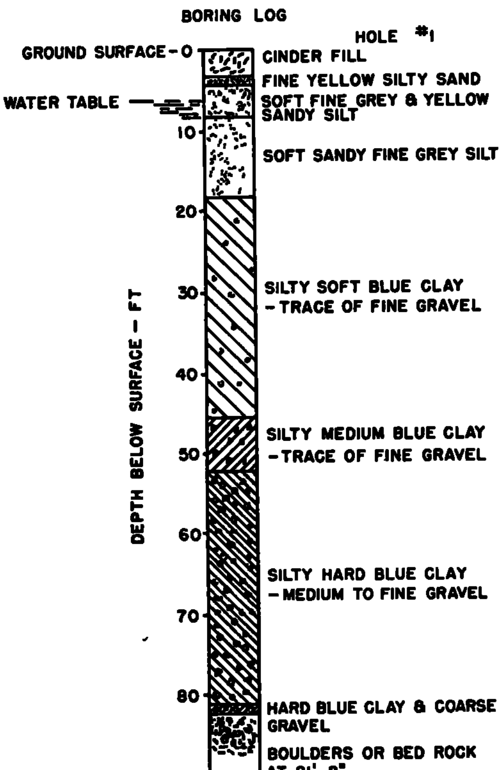

Figure 1. Boring Log

Hole No 2 was 6 in. in diameter and was taken to a depth of 43 ft. only It was abandoned at this level because it was felt that a 4-in. daimeter hole would serve the purpose just as well. The 6-in. hole was lined to about 20 ft from the ground surface and stood unsupported for the remaining 23 ft Samples were taken with a 3-in diameter sampling spoon Vane tests were taken alternately with spoon sampling In every case the vane was driven 2½ ft beyond the bottom of the hole, after which the boring was continued below this point before the next sample was taken. The samples were sealed and set aside for laboratory tests.

Hole No 3 was 4 in in diameter. It was cased down to 20 ft. and remained open for the remainder of the depth It was stopped at 53 ft below the surface This hole was used for shear tests in place only. No appreciable change in soil formation was noticed for the three holes so it was assumed that laboratory test results obtained from samples taken from hole No. 1 were representative of conditions in holes No 2 and 3.

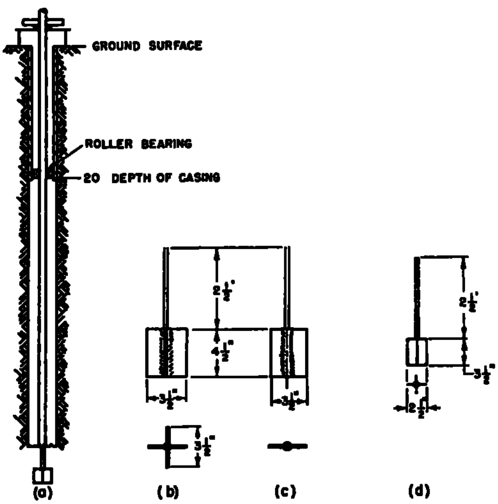

Figure 2. (a) Position of Rod, Bearings and Shear Vanes in Place
(b) Large Four-Blade Shear Device
(c) Large Two-Blade Shear Device
(d) Small Four-Blade Shear Devise

APPARATUS AND TEST PROCEDURE

Details of the principal shear device are shown in Figure 2. Each one consists of four rectangular plates welded to a ½-in. stainless steel rod (approximately 35000 psi. shear yield stress at 0 2 percent set) along the length of one edge of each plate This rod was damaged while forcing the vanes into the soil at 45 ft in hole No. 2 It was cut off at its intersection with the vanes and a 1-in. diameter rod welded on in its place. The plane of each plate makes an angle of 90 deg. with the planes of the others. The weld was made as small as possible and then filed and polished. The lower edges of the vanes were sharpened so as to create as little disturbance as possible when the vanes were forced into the ground. The steel rod extended 2½ ft. above the top of the vanes. At this point it was welded to a fitting (not shown in Fig. 2) which screwed into a 5-ft section of 1½-in. pipe. Other sections of pipe were added as required, as the depth increased A roller bearing was used in the first tests as a guide for the pipe at the bottom of the casing as shown in Fig. 2 (a). A second guide bearing was provided for the shaft about three feet

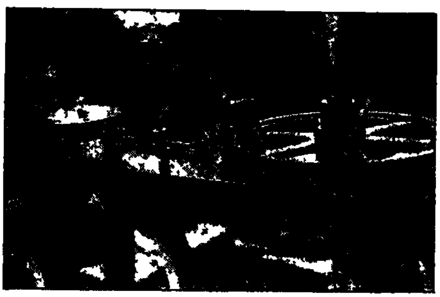

Figure 3. Apparatus for Applying and Measuring Shear Moment

above the ground surface and mounted on a channel. An 18-in diameter torsion wheel was then attached rigidly to the pipe directly above this bearing. The torque to rotate the vanes was applied to this wheel through a flexible wire wrapped around the circumference. This wire was wound on a tennis net winder and the force was measured by a circular spring balance. The entire assembly of the surface apparatus is shown in Figure 3.

Two variations of the 3½- by 4½-in. four blade device were also made Details of all are shown in Figure 2 (b) (c) (d) One has the same dimensions (3½- by 4½-in) but only two vanes, the other has four vanes but is 2½- by 3½-in The purpose of the shear vanes shown in Figure 2 (c) (d) was to determine the effect of vane size, if any, when used in the same soil

The position of the vanes, shaft and bearings for a typical test is shown in Figure 2 (a). The torsion wheel at the top was calibrated in degrees and all tests were performed at the rate of 1 deg. per min. No study of the effect of rate of twisting was undertaken. The rate of 1 deg. per min. was adopted because it was found to be the lowest practical limit. The approximate corresponding rate of strain was used later in the unconfined compression tests. A record was kept of the angle of twist versus

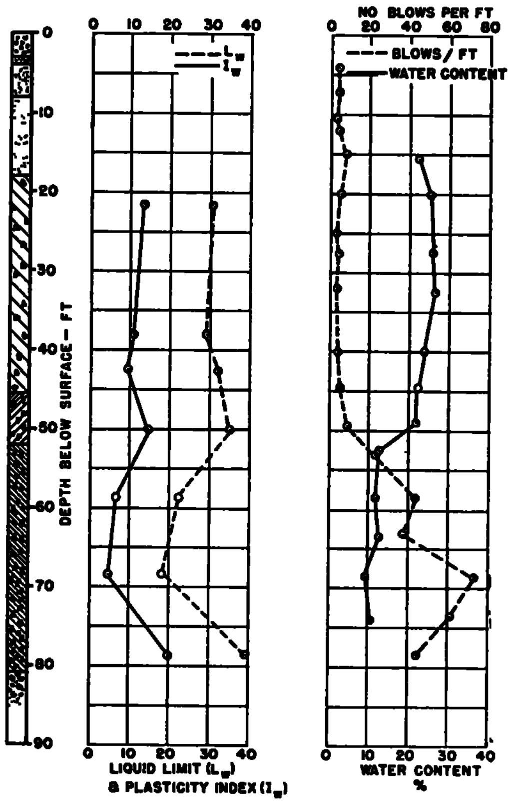

Figure 4. Soil Indices, Water Content and Blows per Ft.

applied torque for all tests except the first two. In all cases the angle of twist of the shaft was deducted. To determine the torque produced by friction in the system several tests were performed using only the rod, shaft and bearing.

It would have been desirable to take vane shear tests all the way down to the top of the rock layer encountered at 81 ft. 8 in. but the device for applying and measuring the torque was not heavy enough to be used in the stiff gravelly clay below approximately the 50-ft. level. In this respect it should be noted that although the larger vanes (3½- by 4½-in.) which seemed best suited for the soils above the 50-ft. level could not be used in the harder clays, data could be obtained below this level using smaller vanes. Tests with these smaller vanes in the upper layers showed that the disturbance area due to pushing the vanes into the soil was so close to the edges of the vanes that no shear failure point could be recognized. Moreover, the remolded shear values were approximately the same as the undisturbed values.

For every shear value obtained in the undisturbed soil a corresponding shear value was obtained for the same soil in the remolded condition. The latter was obtained by forcibly turning the vanes in the soil, then allowing the soil to set for ten minutes and again applying the torque at 1 deg. per min. rotation.

Figure 4 shows soil indices, water content and blows per foot to a depth of 81 ft. 8 in., the limit of boring No. 1. The sampling spoon used in hole No. 1 was 1.2-in. inside diameter by 3 ft. in length. The number of blows per foot plotted in Figure 4 refers to the number of blows per foot of penetration of the sampling spoon using a 140-lb. hammer falling through a distance of 30 in. This is the penetration test used by the Raymond Concrete Pile Company and was carried along as part of the boring log in hole No 1.

SHEAR STRENGTH FORMULAS

Shear values c obtained from unconfined tests were computed by the formula

$$c = \frac{q_u}{2} \qquad (1)$$

Where q_u = unconfined compressive strength. In computing c from the vane tests it was assumed that the shear stress at failure of the soil, at the outer edges of the vanes, was triangular from the center outward along the top and bottom edges. This seems a reasonable assumption since the stress will vary in some manner and will be a maximum at the outer edges where failure first occurs and close to zero at the center. The shear stress along the vertical edges was assumed to be constant and to act on the cylindrical surface of revolution. The value of c is then given by the expression

$$c = \frac{M}{\pi D^2 H \left[\frac{1}{2} + \frac{D}{8H}\right]} \qquad (2)$$

where:

M = Net applied torque
H = Vertical height of vanes
D = Diameter of cylinder of revolution.

The diameter of the surface of cylindrical shear may not always be defined by the diameter D of the vanes. It is possible that the proper value for D in equation (2) may be somewhat larger than the diameter of the cylinder of revolution of the vanes. It seems probable, however, that the diameter of the shear cylinder would not be a constant for all clays but would vary with the degree of plasticity and hardness of the clay, and with the proximity of gravel particles to the shearing edges. The difference between D (as defined for equation (2)) and the true diameter of the surface of shear would probably be small and within the overall accuracy of the test.

The assumption of rectangular sheer instead of triangular shear over the top and bottom edges of the vanes would make a difference in moment of only 4½ percent as noted by Skempton (*10*). Therefore, which ever assumption was used there would be very little difference in the computed values of c.

ACCURACY OF DATA

It is difficult to evaluate the amount of disturbance which the clay underwent as the vanes were forced into place. Carlson (*2*) shows an extracted core in which a four blade vane test had been made. The shear surface formed by the vertical edges of the vanes was found to be cylindrical which justified the original assumption. A comparison of "undisturbed" vane tests with the corresponding remolded tests also provides some clue as to the degree of disturbance as well as the sensitivity of the clay.

Certain precautions were taken in conducting the test so as to cause as little disturbance in the soil structure as possible in inserting the vanes. Where possible the vanes were gently forced into the soil (2½ ft. beyond the bottom of the hole) by hand. This was possible only to a depth of 24 ft. Below this, the vanes were driven into place by means of very light blows of the 140-lb. hammer

The friction in the moving parts could also be a major source of error unless properly accounted for. It was of course desirable to eliminate as much of the friction as possible but at the same time provide sufficient supports to the shaft to insure good alignment.

It was decided to use two bearings, one at the surface and one at the bottom of the casing (Fig. 2 (a)). The surface bearing presented no difficulty. It was machined from brass bushing metal and well greased The lower bearing presented more of a problem since it was below water. A sealed bearing would probably have served the purpose best. However, the bearing could not be rigidly mounted to the casing because the vanes and shaft had to be removed from the hole at 5 ft

TABLE 1

TABLE OF FRICTION MOMENTS

Test No	Depth	Friction Moment	No of Bearings
		in lb	
1	15 ft 3 in	45	2
2	36 ft 4 in	76	1
3	48 ft 10 in	45	1

intervals. If it were free to rotate relative to the casing its value would be doubtful. It was decided, therefore, to use an unsealed loose fitting roller bearing coated with axle grease. The roller bearings were in contact with the shaft and the outside diameter of the bearing was ½ in. smaller than the 4-in. diameter casing. With this arrangement it is very doubtful whether the rollers were of any value at all in reducing the friction, since it is very probable that the shaft and bearing rotated together inside the casing

During the vane tests in the 6-in diameter hole it was noted that there was very little tendency of the shaft to move out of the vertical position during the shearing operation. Consequently it was decided to try a test without the lower bearing There seemed to be so little lateral movement of the shaft that it was decided to eliminate the lower bearing for all tests in the 4-in. hole

To allow for the friction in the system three tests were performed using the shaft and 2½-ft. rod without the vanes. The results of these tests are shown in Table 1.

Although test No. 2 seemed quite a bit out of line, it was decided to use the average of the three values for the frictional moment in all computations Actually the frictional moment was a small proportion of the ultimate shear moment and the difference in using the average of the three values instead of 45 in.

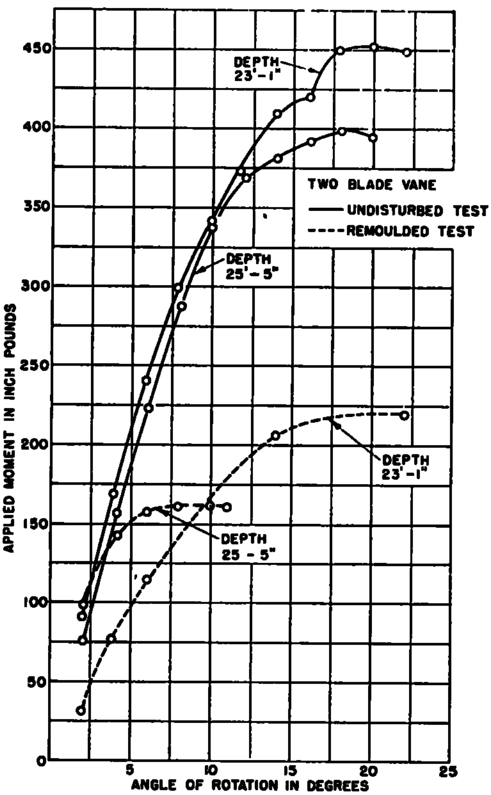

Figure 5. Moment vs. Angle of Rotation for Two-Blade Device

lb would not have made much difference in the computed shear strength

GEOLOGY

The geology of the Chicago area (*1*) (*5*) clearly indicates that all deposits are glacial in origin, except perhaps for the top 10 or 20 ft which must have been deposited in the post-glacial period; probably by flooding. Evidence of the latter is the absence of the distinctive moraine ridges which are very pronounced in neighboring areas. The method of deposition of the soil is also significant. The geology indicates that most of these clays were deposited in standing water. Thus the soil in the 20- to 50-ft. depth is in effect normally loaded just as the strength profile indicates However, the lower hard clays overlying bed rock might have been subjected to considerable consolidation pressure by temporary advances of ice sheets during the recession of the last glacier The water content and the blows per foot (Fig. 4) show a hard clay layer at 50 ft underlain by a somewhat softer clay near the 65-ft. level This would

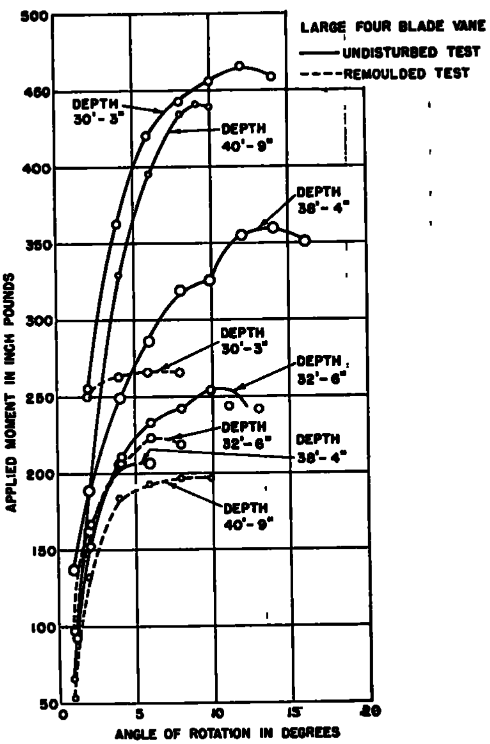

Figure 6. Moment vs. Angle of Rotation for Large Four-Blade Shear Device in Softer Soil

seem to indicate compressive forces other than the overburden pressure.

STRESS-STRAIN CURVES

Figures 5, 6, 7, 8 and 9 show the relation between the net torsional moment and the angle of twist of the vanes for the constant rate of rotation of 1 deg. per min. Figure 5 shows the results of two consecutive tests made with the two-blade shear device (Fig. 2c) in the 4-in. hole. The data shown in Figures 6 and 7 were taken in the same hole at the depths indicated, using the four-blade device (Fig 2b). Figures 7 and 8 are the results of the tests made in the harder clay at the lower depths where the small shear device (Fig. 2d) had to be used

A characteristic common to nearly all these curves in the constant strength region is the discontinuity in the upper portion of the curve. This is more pronounced in some curves than

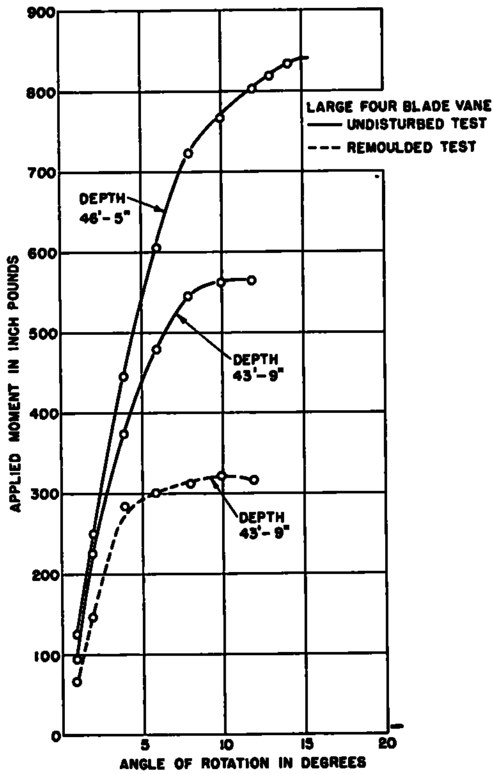

Figure 7. Moment vs. Angle of Rotation for Large Four-Blade Shear Device in Stiffer Soil

in others but the indication seems to be that there is a change in the stress-strain characteristics of the soil as measured by the vane method, just below the ultimate shear strength. The remolded curves show no such discontinuity and are in all cases much lower on the moment scale. The remolded data, however, were all taken when the soil had been allowed to set for ten minutes after being fully disturbed. Remolded data taken after a period of a week or a month would be much more significant.

Most normally loaded clays consist of a mixture of flaky particles which constitute the clay fraction and coarser particles, more or less equidiminsional in shape which form the coarse fraction. The coarser particles form the skeleton structure and the clay particles the matrix or body of the soil. Terzaghi (*11*)

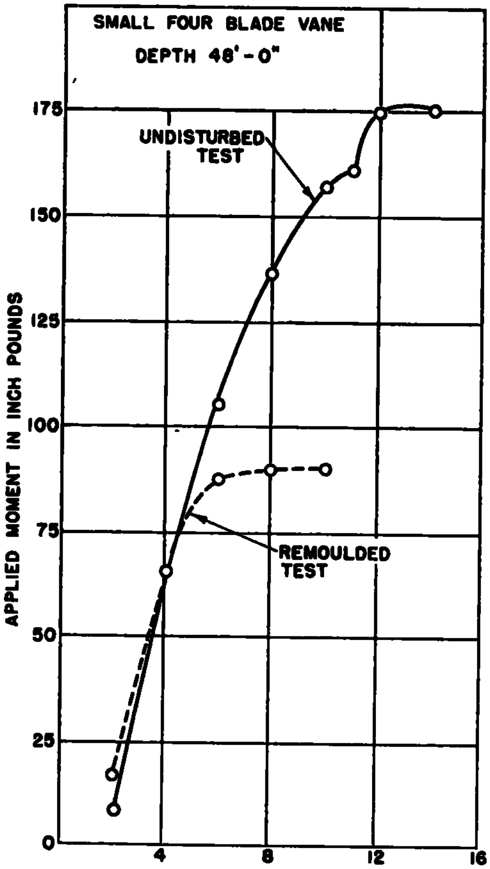

Figure 8. Moment vs. Angle of Rotation for Small Four-Blade Shear Device

suggests that the skeleton structure carries the overburden pressure and the initial shear strength is that of the skeleton If the soil is disturbed the load is transferred to the matrix and the shear strength becomes the shear strength of the matrix which is independent of the normal load and due only to thixotropic hardening. The points of discontinuity in the curves shown in Figures 5, 6, 7 and 8 could represent the shearing strain at which the clay became sufficiently disturbed to cause the collapse of the skeleton. The shear strength would then increase and the ultimate value would be the strength of the matrix. The ultimate value was recorded in every case as the shear strength of the clay and is seen to be approximately constant with depth (Figs. 10 and 11) from 20 ft. to approximately 45 ft.

The shear strengths obtained from unconfined compression tests would be obviously

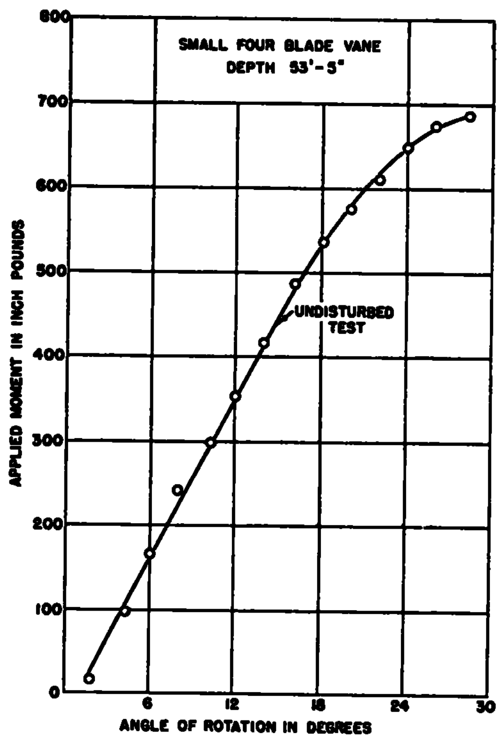

Figure 9. Moment vs. Angle of Rotation for Small Four-Blade Vane at 53-Ft Level

also those of the clay matrix. This would account for the close correlation found between the unconfined compression shear strengths and the vane shear strengths.

Clays which had undergone pre-compression would undoubtedly have been fully disturbed in the process. The only data taken which might be representative of this condition are shown in Figure 9.

SHEAR STRENGTH PROFILE

The shear strength profiles shown in Figures 10 and 11 began at about 20 ft. Material above this level was composed mostly of fill, and fine sand. This accounts for the absence of the hard desiccated top layer which is generally encountered near the surface.

From 20 ft. to approximately 40 ft. the shear strength remained almost constant, showing a tendency to increase near the 40-ft. level. This was true both for the vane tests and the unconfined compression tests. The correlation between the two is excellent in this region and agrees very well with Skempton's (*10*) results. Both the test results and the geology indicate strongly that the soil down to this level is normally loaded.

The shear strength of the clay suddenly increased from 800 lb. per sq. ft. at 40 ft. to 8800 lb. per sq. ft. at 53 ft. This increase correlated very well for both the vane tests and the unconfined compression tests as far as both were taken (Fig. 11). The quantity of gravel became much greater (Fig. 1) at this level too and increased in quantity and size until bed rock was encountered at about 82 ft. Skempton's results (*10*) showed vane shear strengths increasing from 300 lb. per sq. ft. at 40 ft to 800 lb. per sq. ft. at 100 ft. His shear strengths obtained from unconfined compression tests remained approximately constant

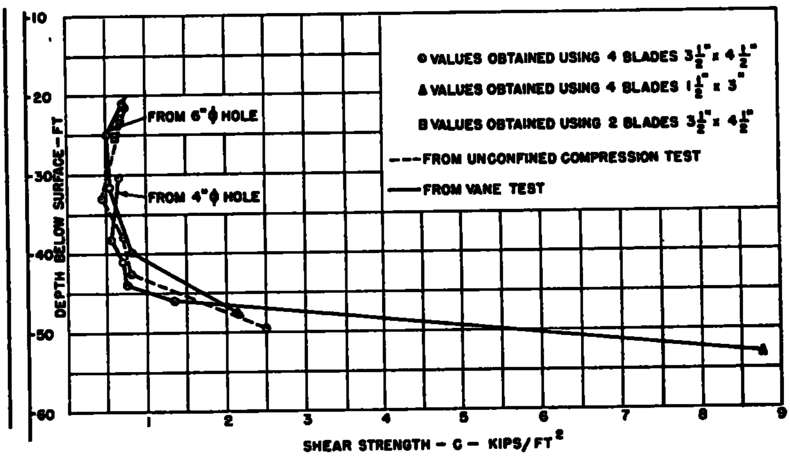

Figure 10. Comparison of Shear Strengths Obtained by Different Vanes and Unconfined Compression Tests

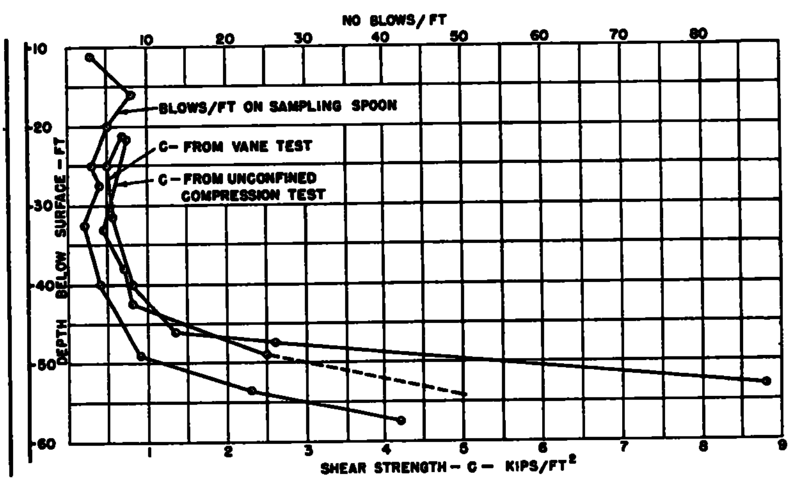

Figure 11. Comparison of Shear Strength Profile Obtained by Vane Tests and Unconfined Compression Tests with Blows per Ft.

throughout this depth. The geology of the Grangemouth area where Skempton's tests were performed as well as his test results indicate a normally loaded soil throughout the entire 100-ft. depth.

DISCUSSION OF RESULTS

The exact stress distribution around the vanes in a plastic soil is probably much more complex than that assumed in the development of equation (2). The shear strength of clay on the horizontal planes may differ from that on vertical planes. The difference in the shear strength on planes of different inclination would depend on the clay structure. Certainly in some clays this difference is significant. The close correlation between unconfined compressive shear strengths and vane shear values in the glacial clay tested, is added evidence of the validity of the unconfined compression test for determining the field shear strength of these clays. Experience has shown (*11*) that tri-axial shear tests in such clays indicate an appeciable angle of friction whereas observations of failures in the field indicate that the soil behaves as if the angle of friction were zero.

The experiments of Carlson (*2*), and Skempton (*10*), however, very definitely show that in some clays vane tests and unconfined compression tests give quite different results. Carlson (*2*) found this discrepancy to exist at relatively shallow depths in certain clays. In some cases he was able to compare shear strengths computed from slide data with the test values. In all cases reported the vane shear strengths agreed with the slide data whereas the unconfined compression shear values were considerably lower. Skempton found increasing vane shear strengths from approximately 40 ft. to 100 ft. while unconfined compression shear values remained constant with depth.

It is not possible at this time to say that the vane tests give true shear strengths in all clays, but the limited data available seem to indicate that in general, vane tests are probably more reliable than unconfined compression tests If the field shear strength of a clay is determined by the strength of the matrix then it will behave as if the angle of friction is zero and the unconfined compression test should also give the field shear strength. If the fraction of coarser particles in the clay is large enough or if the arrangement of these particles is such as to form a stable structure the soil will have an appreciable angle of friction and the shear strength will increase with normal load. Unconfined compression tests obviously still will give only the strength of the matrix, but the true shear strength could only be determined by tri-axial shear tests or possibly direct shear tests.

There is no reason to believe that vane tests would not also give true shear values under those conditions, indeed the limited data available at present seems to indicate as much. A comparison of quick undrained tri-axial shear tests with vane tests for these clays would be most enlightening, but until this data and more vane test data are available the validity of vane test results under all conditions can not definitely be established.

CONCLUSIONS

The results of the tests described in this paper substantiate Terzaghi's theory (*11*); namely, that shear or tri-axial tests are not reliable methods for determining the shear strength of soft clays. Unconfined compression tests therefore should be used for clays whose maximum shear strength is the strength of the matrix whereas tri-axial shear tests might be expected to give values comparable to vane test values in clays for which the skeleton structure is stable. Tri-axial shear tests should also give reliable shear values in those clays where unconfined compression shears are constant and vane shears increase with depth

It might be possible to distinguish between the two types of clays by means of grain size analyses. Stable skeleton structures of the coarser grains probably depend to a large extent on the fraction of these particle sizes present although the manner of sedimentation is undoubtedly a factor in the stability of the structural arrangement.

Clays with stable skeleton structures may also exist in the upper layers Vane shear tests then would show higher values than unconfined compression shear values

It is believed that accurate data on the shear strengths of clay can be obtained from carefully performed vane tests. On the other hand if laboratory shear tests are to be relied upon, it is most important that the clay structure be fully understood so that the proper shear test may be applied.

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IMPORTANCE OF VOLUME RELATIONSHIPS IN SOIL STABILIZATION

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SYNOPSIS

Stabilized soils in common with other porous construction materials are thermodynamically unstable under conditions of normal climatic exposure This basic instability is not too important, since such materials are not expected to last forever; however, it is important that they do possess an economically justifiable service life Therefore, the rate of deterioration is the really significant fact

Internal deterioration depends on the presence of deteriorating agents within these porous systems. Since water itself is often an important deteriorating agent, or at least a vehicle for other deteriorating agents, the rate of water intake into such systems is of major importance This rate is a function of the porosity and of the size of the pores as well as of the water affinity of the internal surface, and of the surface tension of water

General formulae are derived for the rate of capillary water penetration and tested by means of slaking experiments on clay and sand-clay systems The importance of volume relationships is demonstrated, furthermore, by the results of stabilization tests on synthetic saline soils made with beach sand and containing illite, kaolinite and montmorillonite, respectively, as clay fraction

The formulae, test data, and general considerations presented may serve as a basis for understanding the shortcomings of our stabilization specifications and for placing these specifications on a better scientific basis in order that they may possess validity for foreign climates and soils in addition to their usefulness for our own domestic conditions

Stabilized soils, whether clay-aggregate, soil-bitumen, soil-cement, soil-lime, soil-resin, or other, represent porous systems. Deterioration of such systems can be of a surface character, such as pitting and abrasion, wind and water erosion; or internal, such as.

1. lowering of clay-aggregate adhesion bond,
2. infiltration of water-film at mineral-bitumen interfaces (internal stripping),
3. oxidation or other deterioration of bitumen,
4. destruction of mineral-cement bond by physico-chemical and chemical reactions.

Surface damage can be prevented by proper