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TRIAXIAL-VANE TESTS ON A SOFT MARINE CLAY

by K. Tim Law

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Triaxial-vane tests on a soft marine clay

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This paper describes a triaxial-vane apparatus constructed at the Division of Building Research, National Research Council of Canada, Ottawa. A series of tests was conducted on a soft marine clay from South Gloucester, Ontario. Various consolidation pressures were used to study the effect of pressure on vane strength. The test results show that the vane strength is relatively insensitive to a change in vertical consolidation pressure only, but increases steadily with increase in the all-around or horizontal consolidation pressure. This phenomenon is explained using an analysis that incorporates strength anisotropy. The practical implications resulting from this study are also discussed.

On présente la description d'un appareil scissométrique en triaxial construit par la Division de recherches en bâtiment du Conseil national de recherches du Canada, Ottawa. Une série d'essais a été réalisée sur une argile marine molle de Gloucester sud, Ontario. Des pressions de consolidation variables ont été utilisées pour étudier l'effet des contraintes sur la résistance mesurée au scissomètre. Les résultats d'essais montrent que la résistance mesurée au scissomètre est relativement peu sensible aux changements de la pression de consolidation verticale seule, mais qu'elle augmente de façon continue avec une augmentation de la pression de consolidation isotrope ou horizontale. Ce phénomène est expliqué au moyen d'une analyse prenant en compte l'anisotropie de résistance. Les implications pratiques de cette étude sont également commentées.

[Traduit par la revue]

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Introduction

The use of the vane shear test is in a state of uncertainty. Since its introduction in its modern form by Cadling and Odenstad (1950), the vane shear test has been extensively used by engineers to obtain a measurement of the *in situ* undrained shear strength in soft clays. Early experiences with this test showed that it was economical, easy to perform, and gave good assessment of the shear strength as back-calculated from landslides (Cadling and Odenstad 1950; Eden and Hamilton 1957). Recent publications (Eide and Holmberg 1972; Dascal *et al.* 1972) showed, however, that it unacceptably overestimated the shear strength in highly plastic clays. In an effort to resolve this problem, Bjerrum (1972) compiled a number of embankment failures from which a correction curve relating the vane strength to the plasticity index was established. This approach, however, has been strongly criticized by Schmertmann (1975), and more recent case records (Dascal and Tournier 1975) demonstrated that the correction curve needed modification.

In a previous field investigation (Law *et al.* 1977), it was found that the vane strength of clays under fills was essentially the same before and after appreciable compression. Strengths determined from piston samples from the same sites before and after

compression did show an increase. A possible explanation of this apparent anomaly is that the vane strength is dependent on the horizontal stresses.

In order to provide a better understanding of the vane shear test, the test should be conducted in a known stress field with controlled drainage. An apparatus was constructed that made this possible by combining the triaxial cell and the laboratory vane shear machine. This paper describes the apparatus in detail, gives the analysis of the test results, reviews some case histories, and discusses the practical implications.

Description of Apparatus

The apparatus consists of a triaxial cell combined with a Wykeham Farrance laboratory vane machine. The complete setup is shown in Figs. 1 and 2.

The rod (item 1 in Fig. 2) which transmits the torque from the vane machine to the vane (10) passes through the centre of the triaxial piston (3). There are two couplings on the rod. The details of the first coupling (8) are shown in Fig. 3. It consists of a hollow end with an inverted L-shaped slot on the rod and a locking insert fixed on the top of the vane. With this coupling, the vane is locked to the rod when the latter is turned in a clockwise direction. A measurement is thus made of the total torque that

FIG. 1. The triaxial-vane apparatus.

overcomes soil strength around the vane and friction on the rod, which is due to the soil and the BAL-seals¹ (2, 9). As soon as the maximum torque is reached, the vane is released by turning the rod in the reverse direction. Hence, the portion of the torque caused by the friction from the soil and the seals can be measured. Subtracting this from the maximum torque obtained in the first stage gives the torque contributed by the resistance of the soil around the vane. This procedure of eliminating the friction component is different from that of Kenney and Landva (1965) and is believed to be more efficient.

A second coupling (7) is introduced to facilitate the mounting of the soil specimen. It is threaded, permitting the specimen and the cap (6) housing the vane to be mounted in a manner similar to that of an ordinary triaxial test. The piston, with the rest of the rod, is then lowered from the top through the rotating bushing (5). To tighten the coupling, a twist at the top of the rod is applied against the housing in the cap, which is prevented from turning by detachable pins (11) resting against two vertical posts (14) fixed to the collar (15) at the base (16).

The rod is turned by a small electric motor which is controlled by a reversing switch. By changing the

FIG. 2. Details of the triaxial apparatus.

motor, a variety of strain rates can be applied. For the current series of tests the angular speed was set at 5°/min.

The vane is similar geometrically to the Norwegian Geotechnical Institute (NGI) field vane which has been extensively used for the past 20 years by the Division of Building Research of the National Research Council of Canada. The area ratio of the blades and the central core to that of the circle circumscribed by the vane is about 10%. The blades are made from 0.28 mm nichrome, 20 mm high and 10 mm in diameter, and can measure strength up to 100 kPa. In its retracted position, the vane is stored in a special housing (12) in the loading cap. At the bottom of the housing, there is a small recess upon which the vane rests during the mounting of the soil specimen.

Anisotropic consolidation can be applied to the soil specimen. This is achieved by putting an appropriate load on the hanger (4).

Just before inserting the vane into the specimen in an undrained test, the back pressure valve at the base is closed while that of the cap is opened. Thus, volume compensation of the vane and the rod can be

¹© 1973. BAL-seal Eng. Co., Justin, CA 92680, U.S.A.

FIG. 3. Details of the coupling between the rod and the vane.

provided through the stainless steel hypodermic tubing (13). When the vane is pushed to the middle of the specimen, the cap valve is closed and a vane shear test can then be conducted.

The original torque measuring device using spiral springs from the Wykeham Farrance vane machine is retained in the present system. Four springs of different stiffnesses are available for soils of varying strengths. With the present vane for use in a soft soil, a precision of 0.5% of the maximum torque can be achieved.

Testing Programme and Results

The soil specimens tested were obtained from the test site in South Gloucester, 21 km southeast of Ottawa, Ont., (from depths between 3.0 and 5.3 m) using the Osterberg 127 mm diameter sampler (Osterberg 1952). The soil is soft Leda clay, the characteristics of which have been studied in a research programme involving a test embankment constructed at the site (Bozozuk and Leonards 1972). Briefly, it is a soft sensitive grey silty marine deposit, lightly overconsolidated, with a plastic limit of 24%, liquid limit of 48%, and natural moisture content of 70%.

Three series of triaxial-vane tests were conducted to study the change of strength under the following conditions: (a) a change of all-around consolidation pressures σ'_c ; (b) a change of horizontal consolidation pressure σ'_h only; (c) a change of vertical consolidation pressure σ'_v only. These consolidation pressures included those that existed before and after

FIG. 4. Measured torque vs. rotation of vane.

FIG. 5. Measured torque vs. all-around consolidation pressure.

the construction of the test embankment. The horizontal components of these stress states were measured by Bozozuk (1974) using the hydraulic fracture technique.

In order to minimize sample variability, four 36.5 mm diameter specimens, 80 mm high, were trimmed from a single section of the Osterberg sample.

The results of all the tests are summarized in Table 1 and Figs. 4-7. Figure 4 shows the typical development of applied torque and of friction torque on the rod versus rotation of the vane. The nonlinear torque-rotation relationship favours the use of a parabolic end shear distribution in the interpretation

TABLE 1. Results of the triaxial-vane tests

Test number	Depth (m)	Consolidation pressure		Measured torque $\times 10^{-3}$ N·m		
		σ'_h (kPa)	σ'_v (kPa)	At failure	Friction	Net
193-15-4A	3.35	34.3	34.3	88.1	14.7	73.4
193-15-4B	3.35	34.3	73.6	85.9	10.2	75.7
193-15-4C	3.35	73.6	73.6	126.6	10.7	115.9
193-17-3A	4.72	37.3	37.3	98.3	11.3	87.0
193-17-3B	4.72	37.3	83.4	101.7	13.6	88.1
193-17-3C	4.72	83.4	83.4	155.9	20.3	135.6
198-2-2A	3.15	40.2	40.2	91.0	12.4	78.6
198-2-2B	3.15	40.2	82.4	98.9	12.4	86.5
198-2-2C	3.15	40.2	98.1	100.0	13.6	86.4
198-2-2D	3.15	40.2	40.2	89.3	10.7	78.6
198-2-4C	3.31	19.6	19.6	83.1	11.3	71.8
198-2-4B	3.31	40.2	40.2	98.3	11.3	87.0
198-2-4A	3.31	82.4	82.4	133.3	16.4	116.9
198-4-5D	5.00	37.3	37.3	116.4	10.2	106.2
198-4-5C	5.00	37.3	88.3	124.3	10.2	114.1
198-4-5B	5.00	88.3	88.3	185.3	16.4	168.9

for strength over the other two more common assumptions, the uniform and the triangular end shear distributions, although the difference in results when using any one of them is generally unimportant. The friction torque as measured averages about 12% of the total torque.

The relationship between the net torque and the all-around consolidation pressure is shown in Fig. 5. The consolidation pressure σ'_c ranges from below the *in situ* effective horizontal pressure σ'_{h0} to beyond the final vertical effective pressure σ'_{vf} under the test embankment. There is a steady increase of torque with consolidation pressure throughout this pressure range. From σ'_{h0} to σ'_{vf} , for instance, an average increase of torque of about 50% is measured.

Figure 6 shows a similar relationship between

torque and horizontal consolidation pressure at constant vertical consolidation pressure equal to σ'_{vf} . Within the same pressure range, the increase of torque is again about 50%.

A significantly different result is shown in Fig. 7 in which the torque is plotted against the vertical consolidation pressure σ'_v at constant horizontal consolidation pressure equal to σ'_{h0} . In contrast to the previous two series, the torque changes only a small amount with σ'_v . The increase of torque within the same pressure range is only about 5%.

If the conventional strength isotropy is assumed in interpreting the results, the deduced or apparent vane strength will be directly proportional to the torque. This would then lead to the conclusion that within the tested pressure range, the apparent vane

FIG. 6. Measured torque vs. horizontal consolidation pressure at constant vertical consolidation pressure.

FIG. 7. Measured torque vs. vertical consolidation pressure at constant horizontal pressure.

TABLE 2. Definition of strengths and torques

Test type	Consolidation pressure		Symbols of shear strength		Maximum torque
	Vertical direction	Horizontal direction	On vertical plane	On horizontal plane	
1	σ_{h0}'	σ_{h0}'	S_{v0}	S_{h0}	T_a
2	σ_{vf}'	σ_{h0}'	S_{v0}	S_{hf}	T_b
3	σ_{vf}'	σ_{vf}'	S_{vf}	S_{hf}	T_c

NOTE: σ_{h0}' and σ_{vf}' are the *in situ* effective horizontal pressure and final effective vertical pressure under the fill, respectively.

strength is relatively insensitive to an increase in the vertical consolidation pressure only, but is sensitive and increases steadily with an increase in the all-around or the horizontal pressure. A more accurate interpretation is presented in the next section.

Analysis of Test Results

The interpretation of test results is based on the usual assumption that the soil fails along the ends and side of the cylinder circumscribed by the vane. Because a highly complex stress field is induced as a result of the insertion and turning of the vane, the strength referred to hereafter should only be regarded as a measure of the maximum soil resistance on the cylindrical surface under the prevailing stress system. A list of symbols used in this analysis is defined in Table 2.

Strength anisotropy is included in the interpretation. Therefore, the strengths on the vertical and horizontal surfaces, S_v and S_h respectively, may be different. They can be related to the maximum torque (T), assuming a parabolic end shear distribution,

$$[1] \quad T = \frac{\pi D^3}{2} \left(\frac{H}{D} S_v + \frac{3}{10} S_h \right)$$

where H and D are height and diameter of the vane, respectively.

In order to determine the strengths on the two surfaces at various pressures, some of the known behaviour of this material must be noted. Mitchell (1970) shows that strength anisotropy for a Leda clay from Ottawa prevails at low stress range and that it vanishes beyond the preconsolidation pressure P_c . Assuming this is also true in the present case, we obtain

$$[2] \quad S_{vf} = S_{hf} \quad (\text{for } \sigma_{vf}' > p_c)$$

By defining $\alpha = (10H + 3D)/10D$ it can then be shown (Appendix) that

$$[3] \quad S_{v0} = 2(T_b - 3T_c/10\alpha)/[(\alpha - 3/10)\pi D^3]$$

$$[4] \quad S_{h0} = \frac{2}{\pi D^3} \left[\frac{10}{3} (T_a - T_b) + T_c/\alpha \right]$$

$$[5] \quad S_{vf} = 2T_c/(\alpha\pi D^3)$$

With the average values from each test series, the various strengths were calculated and are shown in Table 3. A comparison of Tables 2 and 3 shows that there is a definite strength increase on the horizontal plane with increase in the vertical consolidation pressure σ_v' . This strength increase, however, is associated with a smaller area and a shorter moment arm on the end planes of the cylindrical failure surface. It does not, therefore, contribute significantly to the total torque. Thus, the torque measured and the deduced apparent vane strength based on isotropy are not very sensitive to changes in σ_v' , as has been noted in the previous section.

After determining the various strength values, the rate of change in strength with respect to normal pressure on the vertical and the horizontal planes can be evaluated as $\Delta S_v/\Delta \sigma_h' = 0.33$ and $\Delta S_h/\Delta \sigma_v' = 0.18$. These values are markedly different from each other. The lower rate of strength increase on the horizontal plane probably stems from a higher preconsolidation pressure on this plane. Based on this rate and taking $K_0 = 0.8$ (Bozozuk 1974), the ratio of the vane strength on the horizontal plane to that on the vertical plane is found to be 1.36 under the *in situ* state of stress.

Case Records

Field vane shear tests were conducted in the subsoils of four fills, including the one at South Gloucester (Law *et al.* 1977). The apparent vane strengths were found to be practically the same as those before the embankments were placed. The effective hori-

TABLE 3. Results of calculating strengths on different planes at different stresses

	S_{v0}	S_{h0}	S_{vf}
Strength (kPa)	23.7	30.7	38.7

FIG. 8. Illustration for pressure changes in the subsoil under a fill during and after consolidation: (a) change of total and porewater pressures; (b) change of effective pressures.

horizontal stresses in these cases were estimated using finite element analysis or the hydraulic fracture technique (Bozozuk 1974). No significant increase in horizontal pressure due to the embankments was indicated. The apparent lack of vane strength increase can therefore be correlated to the insignificant horizontal pressure increase. On the other hand, other field evidence and triaxial tests on soils that failed on a plane at about 56° from the horizontal showed that there was a definite strength increase. These observations are in agreement with the analysis of the triaxial-vane test results presented in the foregoing section.

A question may arise as to the applicability of data from axisymmetric tests to the plane strain condition. The difference lies in the change in the intermediate principal stress, a quantity related to the drained Poisson's ratio. Triaxial tests in the appropriate stress range were conducted on the South Gloucester soil. The drained Poisson's ratio was found to be less than 0.1 and the resulting strength gain due to the plane strain condition was less than 3%. This is beyond the precision of the NGI field vane shear test. The laboratory data are therefore applicable to the field situation in this particular case. The soils under the other three fills behaved similarly and it is likely that the same conclusion can be drawn.

Practical Implications

One important implication of this study is that the apparent vane strength obtained from the conventional vane shear test does not reliably reflect strength gains under loading. This apparent vane strength strongly depends on the effective horizontal pressure, which is in turn influenced by a large number of factors related to the soil compressibility and the boundary conditions (Law and Lo 1976). It is then possible that the apparent vane strength may remain practically constant in spite of a definite strength increase on planes other than the vertical. This view is demonstrated by the present test series and by the work of Law *et al.* (1977), which compared increases in the apparent vane and triaxial strengths.

A second implication is that the apparent vane strength may decrease after a structure is built. Consider the stresses generated in the subsoil under a fill during and after its construction (Fig. 8). Steady increases in total pressures, both vertical and horizontal ($\Delta\sigma_v$, $\Delta\sigma_h$), are induced during the construction stage; the excess porewater pressure (Δu) also increases as shown in Fig. 8a. After the construction stops, Δu begins to dissipate while, for the sake of simplicity and not of necessity, $\Delta\sigma_v$ and $\Delta\sigma_h$ remain constant. The changes in effective vertical and horizontal pressures ($\Delta\sigma'_v$, $\Delta\sigma'_h$) can be calculated and are shown in Fig. 8b. $\Delta\sigma'_v$ is positive at all times, whereas $\Delta\sigma'_h$ is negative until Δu has dissipated to the condition that $\Delta u \leq \Delta\sigma_h$. Before this condition is reached, there is a reduction in effective horizontal pressure. A vane test conducted within this period may therefore register a strength reduction. Such an observation has been reported by Stermac *et al.* (1967).

The difference in the rates of strength increase on different planes as shown earlier will introduce additional strength anisotropy in this lightly over-consolidated soil during loading. It implies that in stage construction on this soil, undrained stability analysis in the second stage should be performed with caution. A single rate of strength increase as used by Ladd and Foott (1974) is not suitable in this case.

Summary and Conclusion

A triaxial-vane apparatus was constructed and used successfully at the Division of Building Research of the National Research Council of Canada. Special couplings were devised to provide sealing and to separate the friction component from the measured torque, thus permitting an evaluation of the apparent vane strength of the soil. Tests were

carried out on specimens from a soft marine deposit near Ottawa. The results show that the apparent vane strength interpreted using strength isotropy is relatively insensitive to a change in vertical consolidation pressure under constant horizontal consolidation pressure, but increases steadily with increase in the all-around or the horizontal consolidation pressure.

An analysis of the results incorporating strength anisotropy indicates a definite strength increase with normal pressure on either the horizontal or the vertical plane. The rate of strength increase on the horizontal plane is significantly smaller than that on the vertical plane, yet both rates are appreciably greater than zero. Such strength increases not reflected in the apparent vane strength under a pressure increment solely in the vertical direction are due to the combined effect of the smaller rate of strength increase on the horizontal plane, shorter moment arm, and smaller area on the end failure plane.

Case histories involving similar materials have been reviewed and were found to give results consistent with those from the laboratory tests. It is therefore concluded that the apparent vane strength obtained from the vane shear test is not a good indicator of strength gain under sustained loading.

Acknowledgements

The design of the triaxial-vane apparatus is the result of a series of discussions with the staff of the Geotechnical Section and the workshop in the Division of Building Research of the National Research Council of Canada. J. B. Bordeleau, technical officer of the Geotechnical Section, carried out a large portion of the tests.

This paper is a contribution from the Division of Building Research, National Research Council of Canada, and is published with the approval of the director of the division.

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Notation

- D, H = diameter and height of the vane respectively
- K_0 = coefficient of earth pressure at rest
- S_h, S_v = vane strengths on the horizontal and the vertical planes respectively
- S_{h0}, S_{v0} = vane strengths on the horizontal and the vertical planes under an all-around consolidation pressure equal to the *in situ* effective horizontal pressure, respectively
- S_{vf}, S_{hf} = vane strengths on the horizontal and the vertical planes under an all-around consolidation pressure equal to the effective vertical pressure under the test embankment, respectively
- T = torque
- u = porewater pressure

$\sigma_c', \sigma_h', \sigma_v'$ = the all-around, horizontal, and vertical consolidation pressures, respectively
 σ_{h0}' = *in situ* effective horizontal pressure
 σ_{vf}' = vertical effective pressure under the test embankment
 Δ = a change.

Appendix

Derivation of S_{v0} , S_{h0} , and S_{vf}

Based on [1] and using symbols in Table 2, one can write

$$[A1] \quad T_a = \frac{\pi D^3}{2} \left(S_{v0} + \frac{3D}{10H} S_{h0} \right)$$

$$[A2] \quad T_b = \frac{\pi D^3}{2} \left(S_{v0} + \frac{3D}{10H} S_{hf} \right)$$

$$[A3] \quad T_c = \frac{\pi D^3}{2} \left(S_{vf} + \frac{3D}{10H} S_{hf} \right)$$

Now from [2]

$$[A4] \quad S_{vf} = S_{hf}$$

Solving the four equations simultaneously one obtains

$$S_{v0} = \frac{2(T_b - 3T_c/10\alpha)}{(\alpha - 3/10)\pi D^3}$$

$$S_{h0} = \frac{2}{\pi D^3} [10(T_a - T_b)/3 + T_c/\alpha]$$

$$S_{vf} = 2T_c/(\alpha\pi D^3)$$

where $\alpha = (10H + 3D)/10D$

The above derivation uses an assumption that needs some clarification. It is assumed that S_{v0} remains unchanged in cases 1 and 2 in Table 2. This is based on the fact that in both cases the normal (horizontal) effective pressures are equal before shearing. Once shearing begins, however, porewater pressures generated in each case will lead to different effective pressures at failure. This aspect was examined during the experiment by measuring the excess porewater pressure. Using the rate of strength increase on the vertical plane, the resulting change in S_{v0} due to the slightly different effective pressure at failure was found to be insignificant. The assumption is therefore acceptable.

A similar procedure was also applied in the case of S_{hf} and the same conclusion was reached.