Use of field vane tests under earth-structures

Utilisation des essais in-situ au scissomètre sous des ouvrages en terre

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SYNOPSIS This paper reviews the results of field vane tests conducted through existing embankments constructed over a variety of soft clays. The shear strengths determined from the vane tests, when compared with the preconstruction values, increased or remained unchanged. Strength increases are associated with large loading areas or relatively thin compressible layers while little or no strength increase is associated with small loading areas or relatively thin compressible layers. In all cases the strength to effective vertical stress ratio under the existing earth-structure is less than that before construction.

Laboratory tests on soils from some of the studied cases have been carried out to help understand field observations. Practical implications are discussed and guidelines are drawn for using field vane tests to estimate strength changes.

INTRODUCTION

The field vane test has been used for 30 years to measure the strength of soft clays. It was introduced to avoid the soil disturbance that occurs when taking samples for laboratory testing. Initial experience with the vane test was satisfactory (Eden and Hamilton, 1956). Bjerrum (1972), however, showed that the vane strength was generally higher than the available strength in the field. He proposed a correction curve expressed in terms of plasticity index, based on actual embankment failure records. These were first-time failures in which the natural soil had not been subjected to any previous man-made loading. In spite of some criticism (Schmertmann, 1975), Bjerrum's method of correction has gained wide acceptance for design and analysis.

There are a growing number of situations in which strength measurements are required after a man-made earth-structure is built. This includes stage construction on weak ground and design review of existing dams. In each situation, a unique loading condition is imposed so that the soil will no longer respond to further load according to first-time loading behaviour. Whether Bjerrum's correction is still applicable is a question that needs to be studied.

This paper summarizes field experience with the vane to measure strength under existing earth-structures. This is followed by a laboratory study on the factors influencing the vane strength. Based on the field and laboratory studies, some guidelines are drawn for using the vane shear test to detect strength changes.

CASE RECORDS

Published results of shear strength changes determined by vane tests under existing earth-structures fall into two separate categories. The first group consists of definite vane strength gains while the second shows zero strength change. The majority of the earth-structures were founded on lightly overconsolidated soils with loads exceeding the preconsolidation pressure throughout or in part of the subsoils. The soil information pertaining to the layers stressed to the normally consolidated state and the earth-structure geometry are summarized in Table I and Table II. The following symbols are used in the tables:

T = time after completion of the earth-structure through which the vane shear test was conducted;
P_c = preconsolidation pressure;
σ'_{vo}, σ'_{vf} = vertical effective pressure before construction and at time T, respectively;
S_{uo}, S_{uf} = vane strength before construction and at time T, respectively;
B, H = total width and height of the earth-structure, respectively;
D = thickness of soft clay.

(a) Cases with vane strength increase

The fills at Rang St-George and Rang du Fleuve, Quebec (Tavenas et al, 1978) were founded on a layer of 12 m thick lacustrine deposits, above a thick layer of Champlain Sea clay. Berms, 2.5 m high and 20 m wide, were placed on both sides of the fills.

Table I Soil information and fill geometry for cases with vane strength increases

No.	Location	Soil Properties W_p (%)	W_l (%)	W_n (%)	p_c/σ'_{vo}	S_{uo}/p_c	S_{uf}/σ'_{vf}	Fill geometry B (m)	H (m)	D (m)	T yrs
1	Rang St. George, Que.	20	70	80	[a]1.2	0.35	0.26	75	7.4	66-86	3
		35	60	--	[b]1.3	0.27	0.24				
2	Rang du Fleuve, Que.	20	70	80	[a]1.2	0.34	0.25	75	5.8	66-86	2.3
3	Matagami, Que.	30	65	100	1.8	0.25	[d]0.16	103	6.5	11.2	6
					1.8	0.28	[e]0.26	103	6.5	11.2	6
4	Rupert, Que.	18	28	30	2.2	0.20	0.21	140	9.1	15.2	5
5	New Liskeard, Ont.	25	60	50	1.5	0.23	0.21	50	2.7	~40	10
6	Lagunillas, Ven.	23	73	65	--	$S_{uo}/=\sigma'_{vo}$ 0.29	0.23	42	10.6	4.3	2
	Ska Edeby, Sweden:										
7	• Area I	20	60	70	1.0	0.28	0.22	[c]70	1.5	10.0	14
8	• Area II	20	60	70	1.0	0.27	0.21	[c]35	1.5	12.0	14
9	• Area III	20	60	70	1.0	0.26	0.24	[c]55	1.5	12.2	14
10	• Area IV	20	60	70	1.0	0.28	0.23	[c]35	1.5	12.6	14

Note: a - lacustrine clay; b - marine clay; c - circular fill; d - under centre; e - under berm

Table II Soil information and fill geometry for cases with no vane strength increase

Location	Soil Properties W_p (%)	W_l (%)	W_n (%)	p_c/σ'_{vo}	S_{uo}/p_c	S_{uf}/σ'_{vf}	Fill geometry B (m)	H (m)	D (m)	T (yrs)
Gloucester, Ont.	20	50	70	1.4	0.37	0.27	20.1	3.65	20.1	8
Boundary Road, Ont.	20	50	70	1.6	0.35	0.28	24.4	4.27	21.3	5
Kars, Ont.	20	40	60	2.3	0.24	0.19	48.2	7.92	16.8	16
Ska-Edeby, Sweden	20	60	70	1.0	0.23	0.15	8.5	1.50	15.2	10

The test embankment near Matagami, Quebec was built to study the behaviour of the underlying lacustrine deposit upon which numerous dykes and dams would be constructed in connection with the James Bay hydroelectric power project. The strength measurement through and outside the fill were reported by Eden and Law (1980).

The test embankment at Rupert, Quebec, was built on a soft marine deposit of low plasticity. The purpose was also to provide design information for the James Bay hydroelectric power project. Two berms were placed on each side of the embankment which reached a maximum height of 9.1 m. A part of the embankment was deliberately failed and documented by Dascal and Tournier (1975).

The approach fill in New Liskeard (Lo and Stermac, 1965; Stermac et al, 1967) was built on a deep deposit of varved clay. The original plan called for a maximum fill height of 10 m. Stage construction was adopted but part of the fill failed at 5.5 m before reaching the intended first stage height of 6.1 m. The excess pore water pressure measured under the rest of the fill remained high and dissipated very slowly over a period of two years. The fill was subsequently lowered to 2.7 m; from this the results shown in Table I were obtained.

The Lagunillas preload, Venezuela (Lambe, 1962 and 1973) was used to improve the ground condition to support heavy process tanks. The subsoil consisted of 5.3 m of silt underlain by 4.3 m soft clay. The full preload was applied in three stages over a period of almost one year. Vane strength gains were detected under the preload at different times after the commencement of construction.

The test field at Ska-Edeby, Sweden, was comprised of four circular loaded areas and one test embankment (Holtz and Lindskog, 1972; Holtz and Broms, 1972). Sand drains of different spacings were installed through three of the circular fills. In 1957 the circular fills were built to a load of 27 kPa except for Area III which had the same load under the berm but 39 kPa under the centre. The test embankment was constructed four years later using materials from unloading the Area III fill to the berm level. In 1971, vane tests were conducted through the fills and the embankment. Strength gains were measured under all the circular fills but no strength change was detected under the embankment although the excess pore pressures under it were dissipated.

(b) Cases with no vane strength increase

The Gloucester test fill (Bozozuk and Leonards, 1972; Law et al, 1977) was built over a soft, highly sensitive Champlain Sea clay deposited in several stages. The top 6 m was stressed to about 40% beyond the preconsolidation pressure. Vane tests through the centre of the fill eight years after construction showed no vane strength increase.

Triaxial undrained tests, however, on samples taken from under the fill at the time of the vane testing showed a definite strength increase. Settlement observations and measurement of the change of moisture content supported the triaxial test results.

The Boundary Road approach fill (Law et al, 1977) was founded on material similar to that of the Gloucester test fill. Vane tests were conducted through the shoulder of the fill under which the top 8 m was stressed beyond P_c.

The Kars bridge approach fill (Eden and Poorooshasb, 1968; Law et al, 1977) was constructed over 17 m of soft Champlain Sea clay, the top 6 m of which consisted of a weathered crust. Again, no vane strength increase was detected 16 years after construction even though the triaxial tests and moisture change measurements indicated a definite strength increase.

The test embankment at the Ska-Edeby test field was described earlier.

(c) General observations

The following general observations may be made from Tables I and II.

1) For soil stressed to the normally consolidated state under fills, the vane shear strength increase does not depend on plasticity index, overconsolidation ratio, or length of time (less than 16 years) after completion of construction.

2) Vane shear strength increases are found under fills with a wide base (B/H>10) or in thin compressible layers. The final strength ratio, S_{uf}/σ'_{vf}, in these cases are generally smaller than the corresponding values, S_{uo}/P_c or S_{uo}/σ'_{vo} for normally consolidated soil, before construction.

3) No vane shear strength increase is found under fills with a narrow base or in thick compressible layers even though the triaxial tests may indicate the existence of a strength gain.

It appears therefore that the vane shear strength increase is related to the geometry of the fill. In general for a given fill height, the base width has less effect on the vertical stress increase than on the horizontal stress increase. Hence the vane shear strength increase may be strongly affected by the change of horizontal stress. In addition, the relative degree of rate effect for first-time loading and for post first-time loading need to be understood. These aspects were studied in the laboratory and are reported in the next section.

TRIAXIAL-VANE TESTS

The study was carried out using a triaxial-vane machine built by combining the triaxial cell and the laboratory vane shear machine. It enabled different consolidation pressures to be applied on a soil sample in which a vane test could be conducted at different shear rates. A detailed description of the device was given by Law (1979).

The first test series was performed on soils from Gloucester and Matagami. Two consolidation pressures were applied to the soils, one at the in situ pressure and the other beyond the preconsolidation. For each consolidation pressure, tests were run at different rates to study the time effect.

The second test series was conducted on soils from Boundary Road and Matagami. Varying isotropic consolidation pressures (σ'_c) were applied and the vane test was conducted at a constant rate that led to failure in about 15 minutes.

The third series was carried out on the Gloucester soils under isotropic and anisotropic consolidation. The isotropic consolidated tests were similar to the first test series. In the first part of the anisotropic tests, the vertical consolidation pressure (σ'_{vc}) was kept constant while the horizontal consolidation pressure (σ'_{hc}) was varied. In the second part, σ'_{hc} was kept constant while σ'_{vc} pressure was changed.

The results of the first test series on rate effect are shown in Figs. 1 and 2 where the measured strength is plotted against the time to reach failure. The strength is normalized by the value corresponding to a failure time of 10 minutes. There is a general decrease of strength with increase of time to failure or decrease of shear rate. The rate of strength decrease at the in situ pressure and at the normally consolidated state are 11% and 5.5% per log cycle of time, respectively. This difference in rate effect is related to the relative magnitude of the cohesive component of the undrained strength. At the in situ pressure or under first-time loading, the cohesive component is high because of overconsolidation and aging processes since the soil was deposited. At the normally consolidated state or under man-made loading

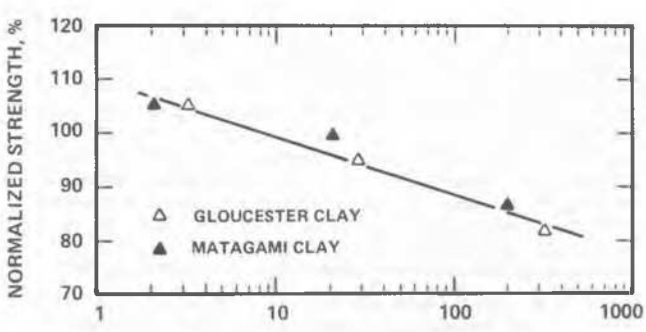

Fig. 1 Vane shear strength variation with time at in situ consolidation pressure

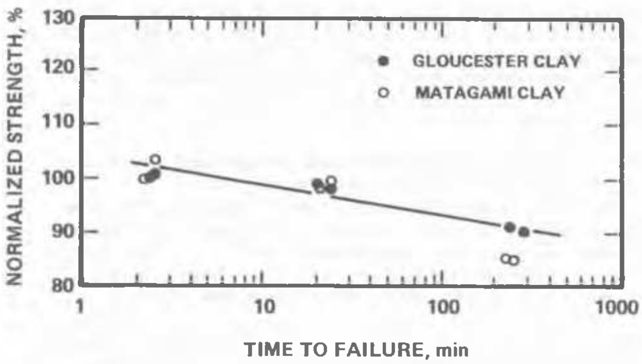

Fig. 2 Vane shear strength variation with time at normally consolidated state

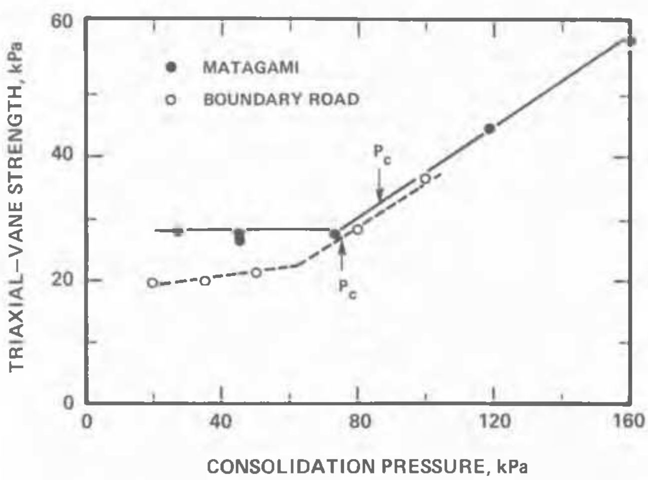

Fig. 3 Triaxial-vane strength versus consolidation pressure

for a limited time, the effects of overconsolidation and aging are removed hence the cohesive component diminishes. Consequently the rate effect, which is mainly associated with the cohesive component, will be reduced at the higher pressure.

The results of the second test series are shown in Fig. 3 where the measured strength is plotted against the isotropic consolidation pressure. Near the in situ pressure, the strength either remains constant or increases slightly with consolidation pressure. At a pressure slightly less than the preconsolidation pressure P_c, the strength starts to increase steadily. This change of rate of strength increase is caused by the destruction of bond or cementation in the clay structure.

The results of the third test series is shown in Fig. 4. As in the second series, the vane

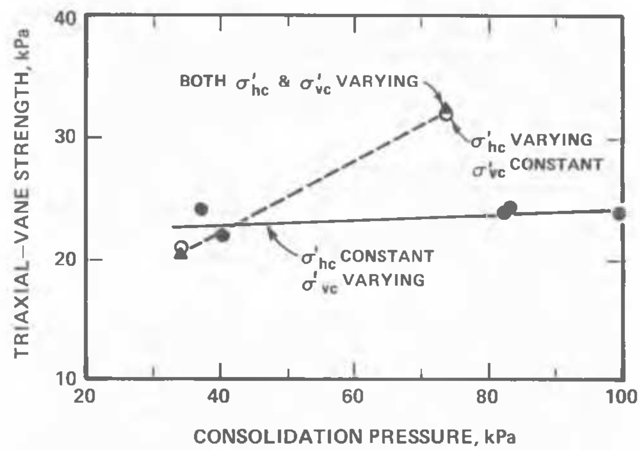

Fig. 4 Variation of triaxial-vane strength with anisotropic consolidation pressure

shear strength increases with σ'_c. The same is also true with σ'_{hc} when σ'_{vc} is kept constant. Under constant σ'_{hc}, however, the vane shear strength hardly increases with σ'_{vc}.

While it is expected that the vane shear strength should increase with σ'_c or σ'_{hc}, the lack of increase with σ'_{vc} can be explained by the fact that the bulk of the resistance to a standard vane comes from the vertical face of the cylinder circumscribed by the vane rotation. The strength on this vertical face is dependent on σ'_{hc}. When σ'_{hc} is kept constant, the overall vane shear strength therefore hardly changes despite an increase in σ'_{vc}. This probably explains the lack of a vane shear strength increase noted in the above records in which the base width is relatively narrow, a condition leading to only a small increase of horizontal stress.

DISCUSSION

(a) Anisotropy of vane shear strength

Many soft clays display undrained strength anisotropy. If S_h and S_v are the strengths on the horizontal and vertical failure surfaces, respectively, the standard vane, with a height-to-diameter ratio of 2, measures an overall strength S_u given by:

$$S_u = 0.86\ S_v + 0.14\ S_h$$

or

$$S_u/S_v = 0.86 + 0.14\ S_h/S_v \qquad (1)$$

where S_h/S_v is an anisotropy ratio.

Based on Bjerrum's work (1973) on first-time loading, this ratio varies from 1.0 to 2.0 depending on plasticity. It will be higher after the first-time loading because the ratio of vertical effective stress increase to horizontal effective stress increase is greater (hence higher S_h/S_v) than before. It can also be deduced from the same work (Bjerrum, 1973), that higher S_h/S_v leads to larger underestimation of the strength available in the field.

(b) Bjerrum's (1972) reduction factor

Bjerrum's reduction factor μ was based on three processes involved in first-time loading failures: rate effect, strength anisotropy, and progressive failure. The effects of these processes are changed substantially after first-time loading. As shown earlier, the rate effect will be reduced; anisotropy leads to a greater strength underestimation and progressive failure will be less significant as more plastic behaviour prevails at the normally consolidated state. All these processes together tend to bring the vane shear strength closer to the field strength. The μ factor, therefore, will be too severe to apply directly to the vane shear strength measured some time after construction.

(c) Horizontal stress increase under earth-structures

Calculations were carried out using the elastic analysis of Poulos and Davis (1974) for the post first-time loading condition. The results were expressed in terms of R where R is the ratio of the horizontal stress increase under a fill to that under a truly one-dimensional state. For fills with a wide base (B/H>10) on a thin compressible layer (D/B<0.25), R was about 0.9. Decreasing D/B did not significantly change this value. Increasing D/B decreased the value of R, but at depths between the surface to two times the fill height, R remained closed to 0.9. For a narrow base (B/H<6) on a thick compressible layer, R was less than 0.6.

(d) Limits of strength changes under fills with a wide base

Upper and lower limits can now be established for S_{uf}/σ'_{vf} for design with post first-time loading conditions. Figure 5 illustrates the change of vane strength under one-dimensional consolidation. The initial vane strength S_{uo} is assumed constant from σ'_{vo} to P_c. Beyond P_c the effects of overconsolidation and aging are obliterated and the strength starts to rise with σ'_v at a rate $S_{uf}/\sigma'_{vf} = S_{uo}/P_c$. In the field situation, however, S_{uf}/σ'_{uf} is always smaller than S_{uo}/P_c because the one-dimensional condition is seldom completely reached, hence $S_{uf}/\sigma'_{vf} < S_{uo}/P_c$. This is substantiated by the data in Table I.

The lowest possible limit for S_{uf}/σ'_{vf} corresponds to the condition where Bjerrum's reduction factor and the influence of horizontal stress are both operative. This is given by $S_{uf}/\sigma'_{vf} > 0.9\ \mu\ S_{uo}/P_c$ where the factor 0.9 denotes that the effective horizontal stress increase under fills with a wide base is about 90% of that under one-dimensional consolidation.

The range for S_{uf}/σ'_{vf} applicable for designing stage constructions is therefore given by

$$\frac{S_{uo}}{P_c} > \frac{S_{uf}}{\sigma'_{vf}} > 0.9\ \mu\ \frac{S_{uo}}{P_c} \qquad (2)$$

If S_{uf}/σ'_{vo} falls within the limits, no correction is required. A comparison of S_{uf}/σ'_{vf} and $0.9\ \mu\ S_{uo}/P_c$ based on data from Table I is shown in Fig. 6. For most cases,

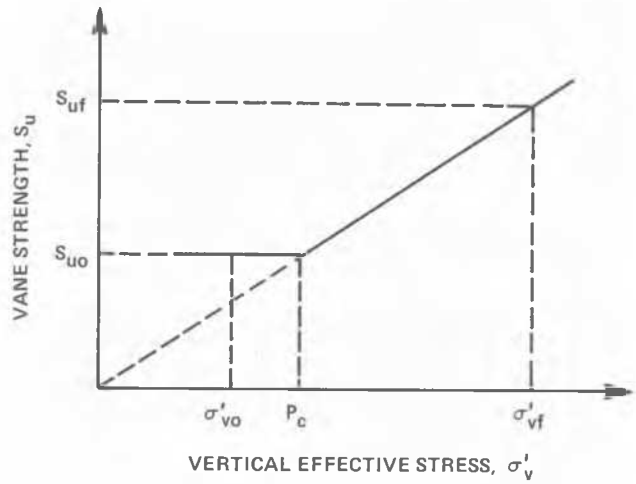

Fig. 5 Idealized strength variation with vertical effective stress under one-dimensional consolidation condition

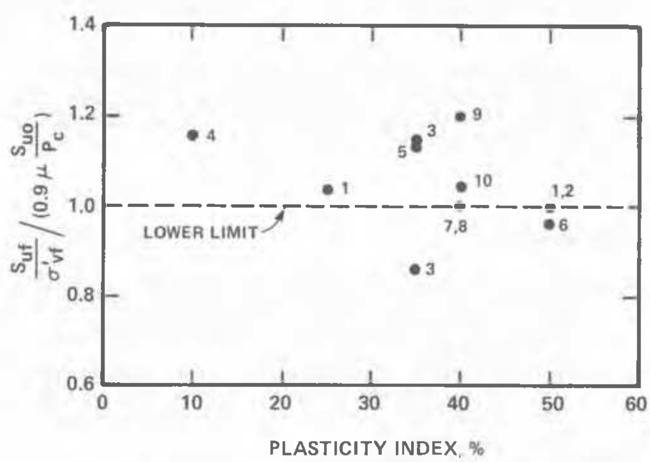

Fig. 6 Measured strength ratio, S_{uf}/σ'_{vf}, under fill compared with theoretical lower limit. Note: numbers refer to cases listed in Table I

including under the berm of Matagami fill, Expression (2) appears to apply except under the centerline of Matagami fill. There, soil variability may be the explanation.

SUMMARY AND CONCLUSIONS

The use of the field vane test under existing earth-structures has been reviewed in this paper. The case records show that the geometry of the structure is an important factor affecting the interpretation of the standard vane with a height-to-diameter ratio of 2. Cases with a narrow base ($B/H < 6$) founded over a thick compressible layer ($H/B > 1$) show no vane shear strength increase even if other evidence may indicate an increase. Cases with a wide base ($B/H > 10$) show apparent vane shear strength increases with a strength ratio, S_{uf}/σ'_{vf}, less than the initial value, S_{uo}/P_c. The geometry factor suggests that the horizontal stress increase imposed under the structure is of great importance and this is confirmed by the triaxial-vane test results.

The analysis of the field and laboratory data leads to the following suggestions for using the field vane test beneath structures:

1) The field vane test is not reliable for cases with a narrow base at least for a period of 16 years after construction.

2) The field vane test may be used for cases with a wide base. The measured strength need not be reduced by Bjerrum's (1972) reduction factor μ and the strength ratio S_{uf}/σ'_{vf} should fall within the limits of Expression (2).

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