Vane Shear Strength in Terms of Effective Stresses

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Abstract

Experience with the shear strength determined with the field vane has shown that the measured values of the strength, normalized with the effective overburden stress, s_{uv}/σ'_{vo}, are often lower than 0.1 for normally consolidated quick clays of low plasticity. This undrained shear strength ratio is less than one third of the corresponding undrained strength measured in a triaxial active test in the laboratory. For more plastic clays and overconsolidated clays, however, the difference between field vane and triaxial active shear strength is found to be less. The ratio of the two strengths might even be close to unity in some cases. The explanation for this phenomenon is not that the insertion of the field vane causes disturbance and remoulding in soft, sensitive clays. With a revised failure criterion for soft, contractant clays, it is possible to express the vane strength in terms of effective stresses, and thus to explain the differences in the shear strengths measured by the vane and in the laboratory. It is concluded that the directly measured vane shear strength values do not constitute, in most cases, a representative strength for stability analysis, for instance for embankments and excavations. A method for correcting the vane shear strength for stability analysis in practice is proposed.

Introduction

The main objective of the study is to express the shear strength obtained with the field vane test in terms of effective stresses, and to develop a deeper understanding of the factors that control the undrained shear strength measured by the field vane. The study collected an extensive series of data from field vane and laboratory shear tests and compared the values of undrained shear strengths from each of the different methods of testing. An important part of the study was to make an interpretation of the vane test in light of a recently extended failure criterion for contractant clays (Aas, 1986; Aas and Lacasse 2021a). Once the new understanding was established, the aim of the work was to establish a modified correction factor to apply to the measured vane shear strength for design.

A method was also developed to derive the coefficient of earth pressure at rest in situ, K_0, by combining the measured values of field vane and triaxial active strengths.

Shear strength obtained from the field vane

If one assumes equal values and simultaneous mobilization of shear strength on the cylindrical and the two horizontal end failure surfaces in a vane shear test, the "average" vane shear strength, s_{uV}, for square-ended vanes is given by the expression:

$$s_{uV} = T/(^1/_2 \pi D^2 H + {}^1/_6\ \pi D^3) \tag{1}$$

where T denotes the maximum applied torque and D and H are the vane diameter and height, respectively. If one assumes that the shear strength on the horizontal end surfaces, s_h, differs from the shear strength on the vertical cylindrical surface, s_v, Eq. (1) may be re-written:

$$s_{uV} = (s_v + {}^1/_3\, D/H \cdot s_h)/(1 + {}^1/_3\, D/H) \tag{2}$$

In particular, for a vane with height to diameter ratio, H/D, of 2, the expression for the field vane strength, s_{uV}, becomes:

$$s_{uV} = {}^6/_7\, s_v + {}^1/_7\, s_h \tag{3}$$

For values of s_h/s_v between 1.5 and 2.0, the difference between s_{uV} and s_V is between 7 to 14% only.

In the rest of the paper, the term "vane shear strength" will, for simplicity, be used for the vertical shear strength s_V. The influence of strength anisotropy will be discussed later in the paper. A detailed description of the correct procedures for performing vane tests in Norway is given by Aas *et al*. (1986) and in different standards, including ASTM (2009).

Stress conditions around the vane

Insertion of vane and stresses around the vane

On the basis of pore pressure measurements in connection with vane-triaxial tests, the insertion of the vane creates a substantial increase in pore pressure in the surrounding clay (Kimura and Saitoh, 1983). It is reasonable to believe that, in a saturated clay, this rise in pore pressure should correspond closely to the increase in total horizontal stress, and hence, cause no changes in effective stresses in the clay. Reported measurements of effective stresses in a triaxial specimen before and after the vane insertion (Matsui and Abe, 1981) appear to support this assumption.

Any subsequent dissipation of excess pore pressure will have to be accompanied by a decrease in the total horizontal stress, because the clay, as a result of the specific boundary in situ conditions, cannot undergo any volume changes. Consequently, one should expect rather limited changes in the effective stresses in the clay even in tests where there is a considerable delay between insertion and rotation of the vane (consolidated tests).

Application of torque and zone of shear distortion

Fundamental to any interpretation of the vane test are the assumptions made regarding the interaction, or mode of transfer of stresses, between the vane and the clay when rotating the vane to failure. As a consequence of the axisymmetric geometric conditions of the vane, all four blades have to be deflected and rotate in exactly the same manner. Hence, soil distortion in the sectors between adjacent blades will not contribute to the resisting torque. Matsui and Abe (1981) investigated experimentally and analytically the shear mechanism of the vane shear test in soft clays, and concluded that the normal effective stress in front of and behind the blades remained essentially constant. This means that the point of application of the reaction forces between clay and vane are located at the blade edges, thus causing a concentration of shear distortion in a limited zone circumscribing the vane. Such a concentration of reaction forces at the edges of the vane blades has been confirmed in an *in situ* test with an instrumented vane in

London clay (Menzies and Merryfield, 1980). In addition, the presence of a narrow ring of strained clay next to the edges of the vane blades has also been observed in the laboratory with the help of X-ray photographs (Kimura and Saitoh, 1983).

Failure mechanism in field vane test

Based on the stress conditions and observed distortion around the vane, the failure mechanism in a vane test appears to be very similar to that in a simple shear test, implying uniformly distributed shear stress and strain all over the cylindrical shear zone circumscribing the vane.

Undrained shear strength concept for contractant clays

A method is proposed using new ideas concerning the assumption of a fundamental failure criterion valid for undrained loading of soft, contractant clays exposed to a brittle failure. The method can be explained with a triaxial active test on a specimen consolidated under in situ stresses, i.e. σ'_{vo} and $K_0{\cdot}\sigma'_{vo}$, where K_0 is the coefficient of earth pressure at rest.

If a saturated clay specimen is loaded undrained to failure in the triaxial compression (active) mode, the effective stress path is independent of the applied total stress changes. This means that not only the maximum shear stress and the corresponding effective normal stress, but also each of the two effective principal stresses at failure, σ'_{1f} and σ'_{3f}, are entirely defined by the consolidation stresses and stress history.

If such a clay has been subjected to creep due to ageing, weathering or overconsolidation, it can withstand an increase in the effective vertical stress from σ'_{vo} to a higher value σ'_{vE} without undergoing plastic strain. This clay, which will have a higher effective horizontal stress, $K_0{\cdot}\sigma'_{vo}$, compared to the original young clay, does not undergo plastic deformations and is able to reduce σ'_h back to the value for the young clay, $\sigma'_{vo}{\cdot}(1 - \sin\phi'_M)$, where ϕ'_M is the mobilized friction angle (Aas and Lacasse, 2021a).

In addition, the effective horizontal stress may be further reduced, as a result of the mobilization of attraction. Attraction is assumed to be due to net attractive forces acting across the water-films constituting the contact points between clay particles. Attraction acts like a tension reinforcement in the clay. It requires practically no deformations to be activated. In a given clay, the attraction will increase with decreasing distance between clay particles, and is therefore a function of the existing consolidation stress, σ'_{vo}. Consequently, the principal stresses (σ'_1 and σ'_3) at failure in the triaxial specimen become σ'_{vE} and $\sigma'_{vo}{\cdot}(1 - \sin\phi'_M - \chi)$, respectively. The parameter χ denotes the relative material attraction and σ'_{vo} represents the effective stress normal to the attractive force. The two principal, or limiting, stresses result in the following expression for the active undrained shear strength, s_{uA}:

$$s_{uA} = {}^{1}\!/_{2}(\sigma'_{1f} - \sigma'_{3f}) = {}^{1}\!/_{2}\sigma'_{vo}\,(\chi + \sin\phi'_M + \sigma'_{vE}/\sigma'_{vo} - 1) \tag{4}$$

where
- σ'_{1f} = Major effective principal stress at failure
- σ'_{3f} = Minor effective principal stress at failure
- σ'_{vo} = In situ effective overburden stress
- χ = Relative material attraction
- ϕ'_M = Mobilized friction angle
- σ'_{vE} = Mobilized effective vertical stress

Standard field vane test

A very important condition for the interpretation of the field vane test, as indicated above, is that there cannot be any volume change and, hence, no change in the width of the cylindrical shear zone around the vane. As a consequence, the effective radial stress remains constant during a field vane test. Considering a clay that has undergone confined consolidation, this radial effective stress will be equal to $K_0 \cdot \sigma'_{vo}$. This means that an element in the cylindrical shear zone has effective stresses comparable to that of a specimen in an active triaxial test. If the clay element had been consolidated under a vertical stress σ'_{vo}, the lowest possible effective horizontal stress it can sustain without undergoing active failure, is equal to the lower limiting stress (minor principal stress) in active shear, σ'_{3f} . When this limiting value is reached, the clay starts to yield, initiating a structural break-down which rapidly leads to failure along the vertical cylindrical surface circumscribing the vane.

In a quick clay, no adhesion (between the clay and the blades) can be mobilized, irrespective of a possible relative movement between clay and vane blades. This means that the horizontal stresses coincide in directions with the horizontal principal stresses. As a result, because of the conditions of no change in radial effective stress, the intermediate and major principal stress under field vane testing, σ'_{2fV} and σ'_{3fV}, can be expressed as:

$$\sigma'_{2fV} = K_0 \sigma'_{vo} + s_V \tag{5}$$

$$\sigma'_{3fV} = K_0 \sigma'_{vo} - s_V \tag{6}$$

As the values of σ'_{3f} for the clay element in the shearing zone around the vane and in the triaxial specimen have to be equal, one obtains:

$$\sigma'_{3fV} = \sigma'_{vo} (1 - \sin\phi'_M - \chi) \tag{7}$$

and hence $$s_V = \sigma'_{vo}[K_0 - (1 - \chi - \sin\phi'_M)] \tag{8}$$

In the case of a young, normally consolidated quick clay where the coefficient of earth pressure at rest K_0 is equal to $1 - \sin\phi'_M$, the shear strength on the vertical cylindrical surface, s_v, becomes:

$$s_v = \chi \sigma'_{vo} \tag{9}$$

If the clay next to the vane is not completely liquefied, a shear stress equal to the remoulded vane strength, s_v', may be mobilized along the blades of the vane at failure. The radial strain necessary to mobilize this adhesion occurs as a result of a simultaneous contraction and stress relief in the failure zone at the start of yield. The contraction is caused by inward drainage under the high pore pressure gradients in the failure zone. The mobilization of radial adhesion along the vane blade results in a rotation of the horizontal principal stresses and to changes in the principal stresses by approximately $-s_v'$ and $+s_v'$ compared to the above equations. This leads to an approximate expression for the shear strength tangential to the cylindrical plane, s_v:

$$s_v = \sigma'_{vo}[K_o - (1 - \chi - \sin\phi'_M)] + s_v' \tag{10}$$

Most soft clays exhibit anisotropy and sensitivity. With these characteristics, it can be implied that one may, without much error, extend the expression to also be valid for the average vane strength ($s_{uV} = s_v$ and $s_{uV}' = s_v'$):

$$s_{uV} = \sigma'_{vo}[K_0 - (1 - \chi - \sin\phi'_M)] + s_{uV}' \tag{11}$$

Anisotropy in undrained shear strength from field vane tests

By combining Eq. (4) and (11) and replacing $\sigma'_{vE}/\sigma'_{vo}$ with $K_0/(1 - \sin\phi'_M)$ for aged clays (Aas and Lacasse, 2021a), one obtains an expression describing the ratio between the triaxial active and field vane strengths and how this ratio varies with K_0, mobilized friction angle and χ. For simplicity, the term s_{uV}/σ'_{vo} in Equation (12) was given the value $0.1 \cdot s_{uV}/\sigma'_{vo}$, corresponding to a clay sensitivity equal to 10 (with sensitivity with the field vane or the fall cone). The notation is defined under Eq. (4) and K_0 is the coefficient of earth pressure at rest.

$$s_{uA}/s_{uV} = {}^{1}/_{2}[K_0/(1 - \sin\phi'_M) - (1 - \chi - \sin\phi'_M]/\,0.9[K_0 - (1 - \chi - \sin\phi'_M)] \qquad (12)$$

Figure 1 illustrates the relationship between the ratio s_{uA}/s_{uV} and the normalized vane strength s_{uV}/σ'_{vo} for 43 sets of data from 20 different locations. The clay deposits included in this data set have experienced secondary compression, resulting in a value of $\sigma'_{vE}/\sigma'_{vo}$ between 1.05 and 1.30. To understand the large range of variation in s_{uA}/s_{uV} between 1 and 3.2 for s_{uV}/σ'_{vo} between 0.1 and 0.5, one needs to consider the values of $\sigma'_{vE}/\sigma'_{vo}$, χ and $\sin\phi'_M$ (Aas, 1986).

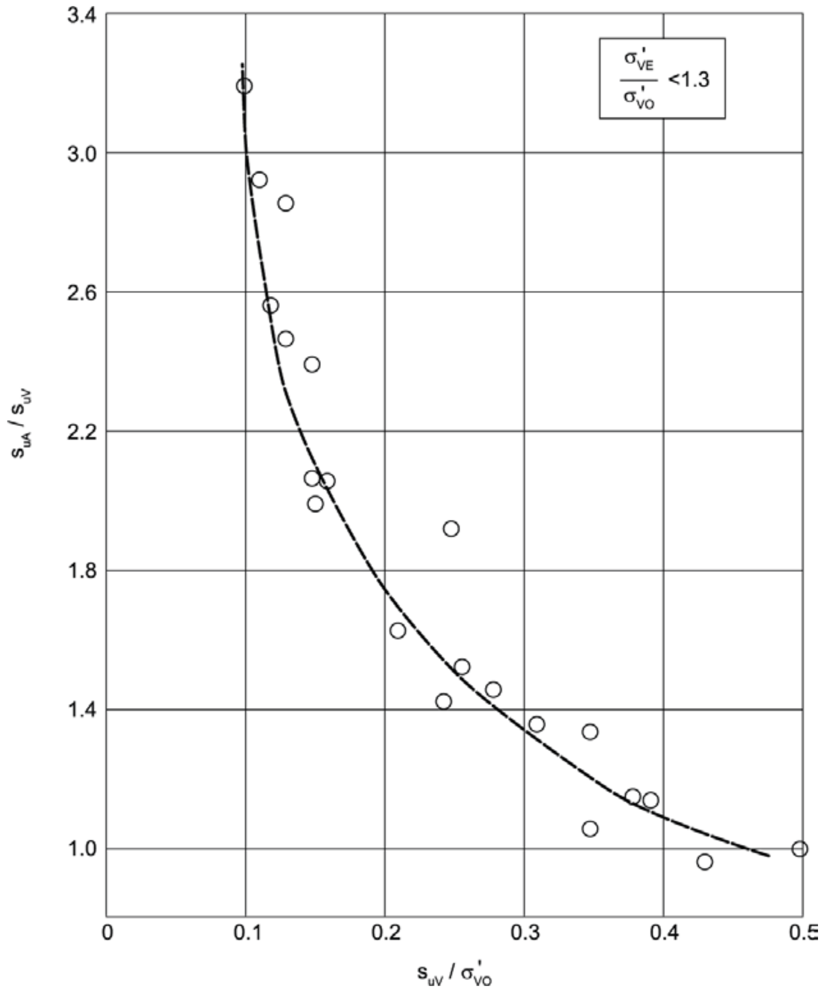

Figure 1. Relationship between the undrained shear strengths obtained from field vane and triaxial actives tests on normally consolidated aged clays (OCR = 1.05 – 1.30)

A value of s_{uV}/σ'_{vo} equal to 0.10 corresponds to a silty, low plasticity clay, exhibiting a high friction angle, a low attraction and a modest effect of ageing (small creep deformations). If one uses values of $\sin\phi'_M = 0.55$, $\chi = 0.10$ and $\sigma'_{vE}/\sigma'_{vo} = 1.05$ in Eq. (12), the resulting ratio s_{uA}/s_{uV}

is 3.2. Correspondingly, for a plastic clay exhibiting a value of s_{uV}/σ'_{vo} equal to 0.4, reasonable values of $\sin\phi'_M$, χ and $\sigma'_{vE}/\sigma'_{vo}$ could be 0.40, 0.30 and 1.30, giving a ratio s_{uA}/s_{uV} of 1.16.

Aas (1965; 1967) measured the ratio of horizontal and vertical shear strength, s_h/s_v,, determined by the field vane. To do this, series of tests were run with different shapes of vanes. Table 1 compiles the ratio of horizontal to vertical vane strengths s_h/s_{uV} for a broad variety of soft clays.

Table 1. Ratio between horizontal and vertical shear strength determined by the field vane.

Clay site	Depth (m)	Plasticity index I_p (%)	Sensitivity S_t	Strength ratio s_u/σ'_{vo}	Field vane strength ratio	
					s_h/s_v	s_h/s_{uV}
Tønsberg	2 – 8	13	30 – 80	0.48	1.0	1.00
Manglerud	4 – 16	8	40 – 70	0.11	1.5	1.40
Lierstranda	4 – 20	6	50 – 150	0.19	2.0	1.75
Fredrikstad	2 – 12	28	7 – 10	0.38	1.0	1.00
Drammen	4 – 14	14	4 – 9	0.17	1.5	1.40
Kelsås	3 – 10	6	20 – 100	0.12	1.8	1.62
Mastemyr	3 – 12	6	10 – 100	0.24	1.5	1.40
Strømsø	3 – 11	30	10	0.33	1.0	1.00
	12 – 15	--	2	0.16	1.2	1.17
Bragernes	5 – 11	--	30 (?)	0.28	1.2	1.17
	11 – 15	--	20 (?)	0.12	1.3	1.25
Skå Edeby	3 – 11	40	11 – 27	0.30	0.7	0.73
Bangkok	3 – 10	85	70	0.57	0.7	0.80

Values of s_h/s_v between 1.5 and 2.0 were reported for lean (I_p < 15%), normally consolidated Norwegian sensitive and quick clays, and a value of 1.0 for the overconsolidated, quick Tønsberg clay. The table also contains earlier unpublished data from a number of other Norwegian locations. Eide and Holmberg (1972) and Wiesel (1973) reported anisotropy ratios equal to 0.77 for Bangkok clay and about 0.7 for Skå-Edeby clay. The upper diagram in Figure 2 shows the ratio of horizontal to vertical shear strength from the field vane (s_h/s_{uV}) versus the normalized vertical vane shear strength (s_{uV}/σ'_{vo}).

Although there are uncertainties about the simultaneous mobilization of shear resistance on the two horizontal vane end surfaces, it appears reasonable to assume that the derived horizontal vane shear strength is close to the direct simple shear strength s_{uD}. To verify this assumption indirectly, the ratio of the direct simple shear strength to the field vane strength from the same database (s_{uD}/s_{uV}) is plotted versus the normalized vane strength (s_{uV}/σ'_{vo}) in the lower diagram in Figure 2. As illustrated in Figure 2, there is a reasonably good agreement between the two diagrams, thus verifying empirically the validity of the assumption, and supporting the method of determining horizontal and vertical shear strength from vane borings.

Determination of in situ value of K_0 from field vane test

Equation (8) can be rewritten in the following way:

$$K_0 = [\sigma'_{3f} + (s_{uV} - s_{uV}')]/\sigma'_{vo} \tag{13}$$

where σ'_{3f} is the minor principal stress at failure in a triaxial active test. This means that the field vane strength can be used in combination with the triaxial test to determine the coefficient of earth pressure at rest K_0. As indicated above, use of the field vane shear strengths s_{uV} and s_{uV}' instead of the actual "vertical" strengths s_v and s'_v in Eq. (13) leads to a maximum error of only a few percent.

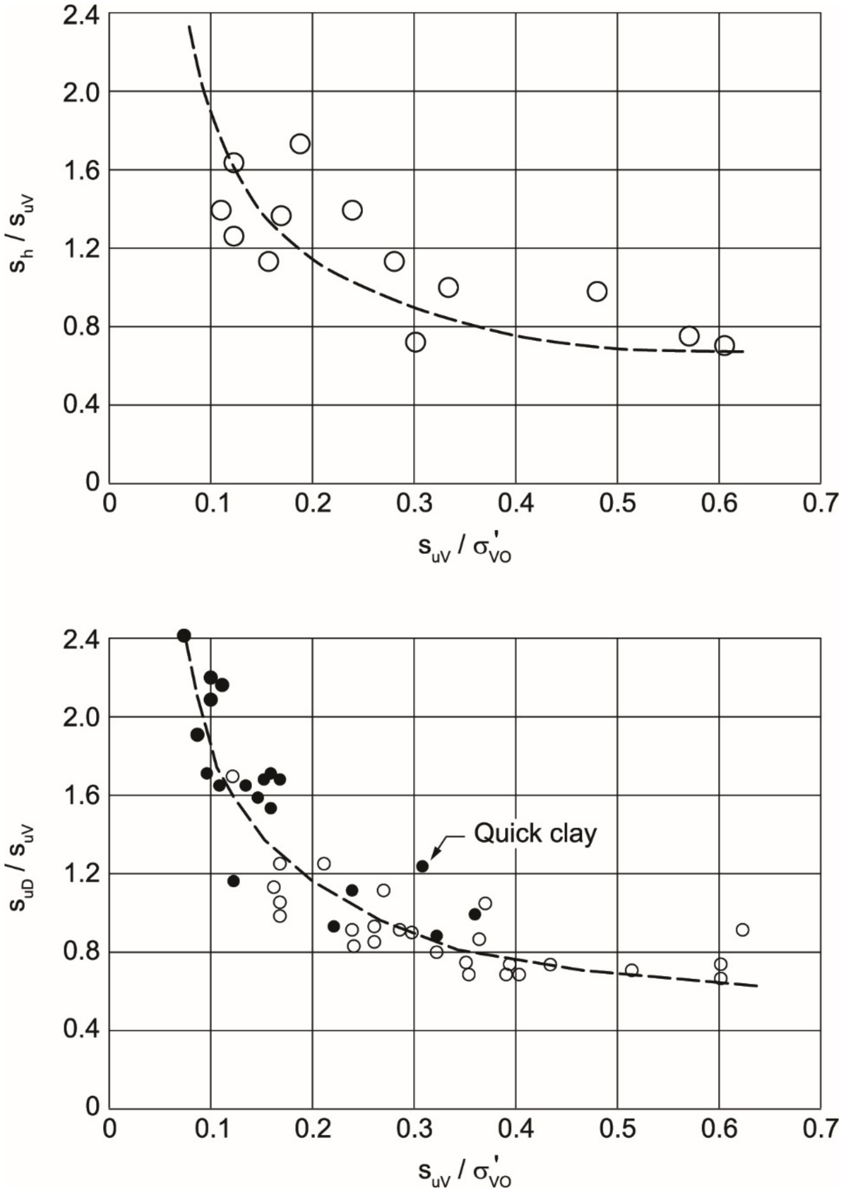

Figure 2. Comparison of horizontal shear strengths derived field vane tests and shear strength from direct simple shear tests.

Figure 3 demonstrates the graphical determination of K_0 on the basis of $s_{uV} - s_{uV}'$ and the effective stress path from an anisotropic consolidated undrained triaxial active test with vertical consolidation stress equal to σ'_{vo}. The two tests shown are for a nearly normally consolidated Ellingsrud quick clay with vane shear strength s_{uV} equal to 6 kPa, and the highly overconsolidated Haga clay with s_{uV} = 41 kPa and s_{uV}'= 9 kPa. The K_0-values for the two clays obtained from the field vane (see also Aas and Lacasse 2021b) were 0.45 and 1.92, respectively. The K_0-values of 0.60 for Ellingsrud and 1.40 for Haga shown on Figure 3, were used to consolidate the specimens in the laboratory, but they were based on an early estimate of K_0 before testing and are not representative of the *in situ* values.

For Ellingsrud clay, three K_0-triaxial tests in the laboratory were run to determine the in situ K_0 and suggested values of 0.45, 0.45 and 0.42 (Aas, 1986), which is very close to the K_0-values obtained with the field vane interpretation in Figure 3.

For the Haga clay, the comparison is more complex because the specimen came from a depth of only 2.2 m. The material was from a desiccated, fissured crust. The *in situ* clay at such shallow depth had also been subjected to many heat/cold, freeze-thaw cycles over the years. The K_0-value of 1.9 obtained from the field vane interpretation agrees well with the result of hydraulic fracturing at the depth of 2 m at the Haga site (Bandis and Lacasse, 1986). On the basis of the correlations by L'Heureux *et al.* (2017) for Norwegian clay, a K_0-value of 1.92 suggests a clay with overconsolidation ratio of 8. The stress history presented by Bandis and Lacasse (1986) gives an overconsolidation ratio of 8 at depth 2 m, based on laboratory tests on Haga clay.

It can be concluded that there is excellent agreement between the field vane-predicted K_0-value and laboratory and *in situ* derived values at both the Ellingsrud and Haga sites.

K_0-values determined as from the filed vane test have been compared with field values based on other *in situ* testing methods at other sites, i.e. hydraulic fracturing (Bjerrum and Andersen, 1972) and self-boring pressuremeter and dilatometer tests (Lacasse and Lunne, 1982a; 1982b; 1982c). The earlier comparisons also show (see Aas *et al.*, 1986) that the field vane approach to determine the K_0-value *in situ* gives a good match with hydraulic fracturing and self-boring pressuremeter, whereas the dilatometer test seemed to overpredict the K_0-values.

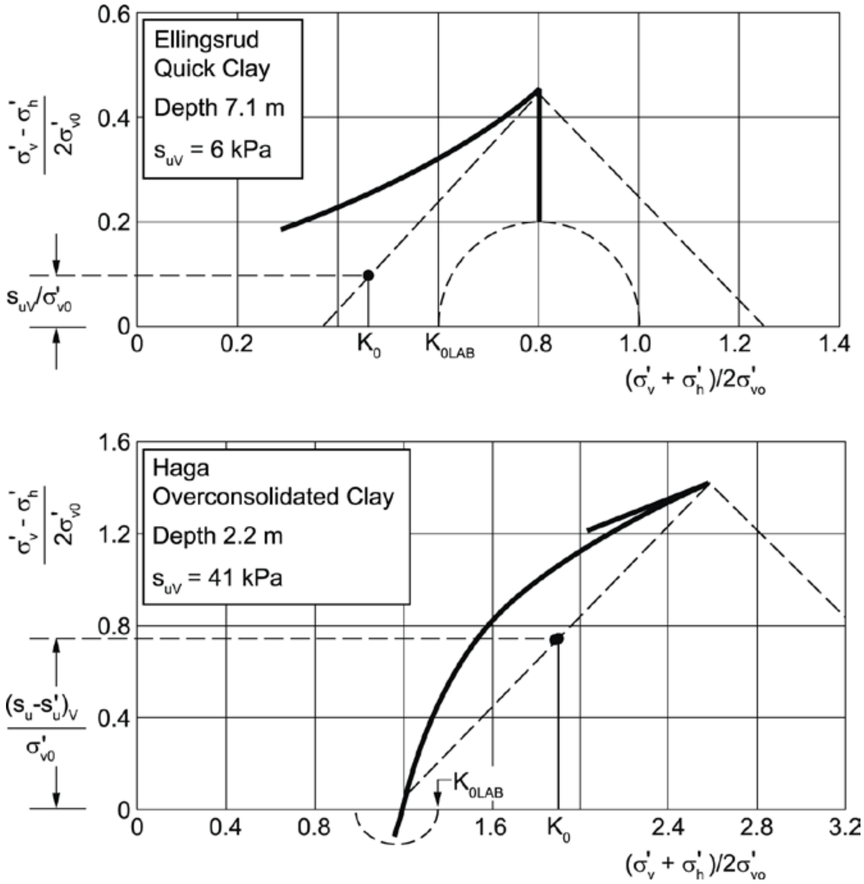

Figure 3. Graphical determination of the in situ value of K_0 from field vane strengths and effective stress paths from undrained triaxial active tests

Correction of vane strength for design analyses

Aas (1979) found that back-calculated factors of safety, based on field vane strength for a number of failed embankments and excavations, increased nearly linearly with the average value of the ratio s_{uV}/σ'_{vo} on the sliding surface. The (uncorrected) calculated factor of safety (FS) was 1.0 for cases where the s_{uV}/σ'_{vo} ratio was equal to about 0.22. The (uncorrected) factor of safety increased to about 2.0 for s_{uV}/σ'_{vo} equal to 0.6, and decreased to 0.6 for s_{uV}/σ'_{vo} of about 0.1. There was therefore need for a correction factor to ensure that the field vane strength gave a safety factor of unity for the failed embankments and excavations.

Aas (1979) also investigated the ratio between field vane strength and the "average laboratory strength", determined as the average of the triaxial active, direct simple shear and triaxial passive strengths, measured on specimen consolidated to the *in situ* stresses. Plotting the ratio s_{uV}/s_{uLAB} against s_{uV}/σ'_{vo}, the points grouped nicely along the same straight line which was fitted through the points in Aas(1979).

The average laboratory shear strength thus calculated might constitute a representative mean value for field strength along a potential failure surface in connection with excavations (base failures), embankments and shallow foundations (Bjerrum, 1972). Hence, Aas *et al.* (1986) presented a diagram where the measured ratio s_{uLAB}/s_{uV} was plotted against s_{uV}/σ'_{vo} for a large number of clays (Fig. 4). Two curves were suggested, one for "normally consolidated", and one for overconsolidated clays. The suggested ratio represented at the time the correction factor by which the measured field vane strength should be multiplied, in order to give the best estimate of average field shear strength to use for design.

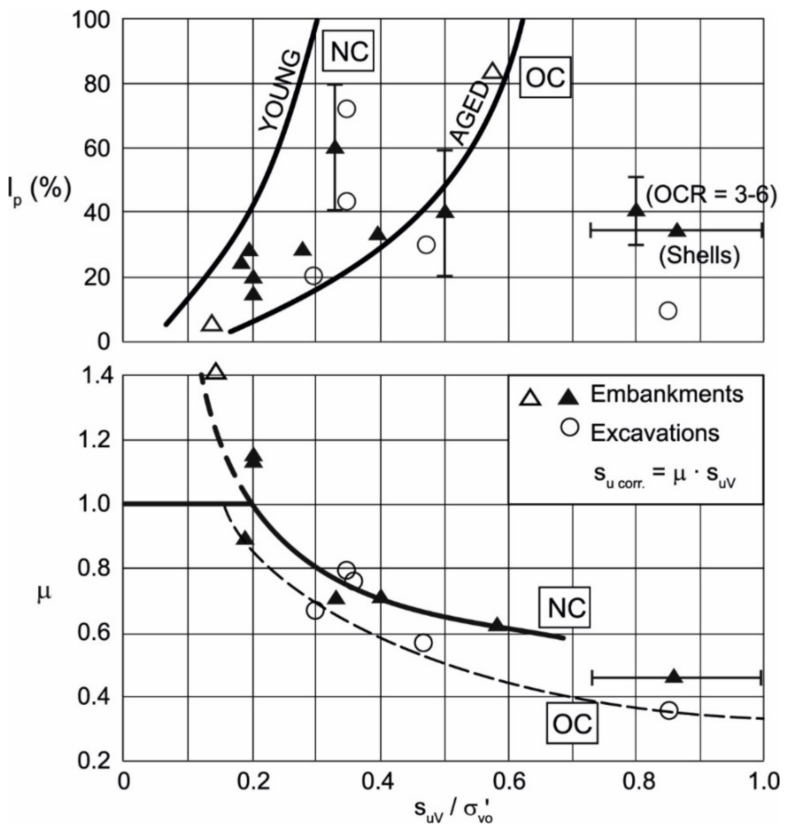

Figure 4. Correction factor for the field vane strength s_{uLAB}/s_{uV} as a function of the normalized field vane strength and the "overconsolidation ratio" (Aas et al. 1986).

Later experience with practical design showed that these relationships were essentially valid for low plasticity clays exhibiting a ratio s_{uV}/σ'_{vo} lower than 0.6. However, the relationship for overconsolidated clays has proven to be incomplete and probably valid only for loading times of considerable duration in fissured or dry crust clays.

Figure 5 collates the result of a new study of the relationship s_{uLAB}/s_{uV}, including contractant clays from several locations in Norway and abroad, each exhibiting different degree of ageing or overconsolidation (denoted $\sigma'_{vE}/\sigma'_{vo}$). In Figure 5, the clays are separated into categories with the ratio $\sigma'_{vE}/\sigma'_{vo}$, where σ'_{vE} equals the ultimate vertical failure stress, σ'_{3f}, in an active triaxial test.

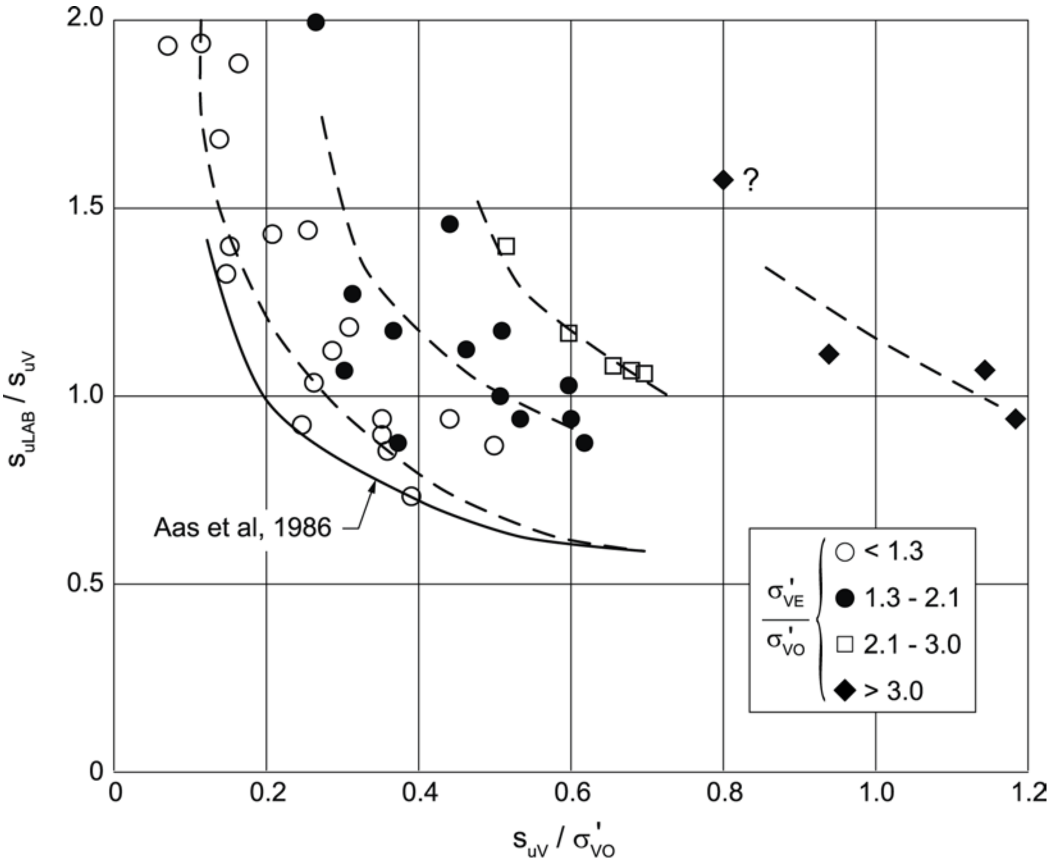

Figure 5. Field vane correction factor s_{uLAB}/s_{uV} as a function of the normalized field vane strength and the overconsolidation ratio.

The curve representing the clays exhibiting $\sigma'_{vE}/\sigma'_{vo} \leq 1.3$ and $s_{uV}/\sigma'_{vo} \leq 0.6$ appears relatively similar to the earlier curve (Aas *et al*, 1986) for normally consolidated clay. However, the diagram in Figure 5 indicates that when the values of $\sigma'_{vE}/\sigma'_{vo}$ increase, higher correction factor s_{uLAB}/s_{uV} are required for clays with the same value of s_{uV}/σ'_{vo}. It is interesting to observe that for clays with an "overconsolidation" ratio exceeding 3, the vane strength seems to be representative for the actual strength to use in design.

Because experience indicates that the direct simple shear strength often lies halfway between the triaxial active and passive strengths, and the simple shear strength may give questionable values in stiffer clays, s_{uLAB} was chosen, in the latest study as $^1/_2\,(s_{uA} + s_{uP})$.

The majority of the correction factors on the shear strength from the field vane test presented in the literature are a function of the plasticity index (e.g., Ladd *et al*. 1977), and in the case of ASTM (2009) and Chandler (1988), as a function of time to failure (mainly for embankments). No account is taken of overconsolidation ratio. Based on Figure 5, this leads to a conservative design in overconsolidated clays.

Summary and conclusions

The interpretation of the field vane test, in the light of a revised failure criterion for contractant clays, makes it possible to explain and understand the values of undrained shear strength measured in this type of test. Deduced theoretical expressions for vane strength and triaxial active strength explain why the ratio between those two shear strengths can vary from less than 0.1 in silty, low plasticity, quick clays to about 1.0 in plastic, aged or weathered clays. In the same way, the study with additional data supports earlier published data for the measured ratio between horizontal and vertical vane strength.

The study has led to a method to determine the value of the coefficient of earth pressure at rest by combining the results of field vane and triaxial active tests. Theoretically derived values have been found to agree fairly well with recorded values in situ with the self-boring pressuremeter and hydraulic fracturing tests.

An additional study included contractant clays from several locations in Norway and abroad, each exhibiting different degree of ageing or overconsolidation ratio up to about 1.3. The result confirm earlier findings that for soft clays with field vane strength ratio of $s_{uV}/\sigma'_{vo} \leq 0.6$, the ratio s_{uLAB}/s_{uV} gives a representative measure for the correction factor by which the measured field vane shear strength should be multiplied to give a representative field strength for stability analysis. So, for normally consolidated clays, the same correction factor as earlier published applies. A new correction factor is suggested for clays with apparent overconsolidation higher than 1.3.

References

Aas, G. (1965). A study of the effect of vane shape and rate of strain on the measured values of in situ shear strength of clays. ICSMFE. 6, Montreal, **1**: 141–145.

Aas, G. (1967). Vane tests for investigation of anisotropy of undrained shear strength of clays. Geotechnical Conference on Shear Strength Properties of Natural Soils and Rocks. Oslo 1967. Proc. **1**: 3–8.

Aas, G. (1979). "Vurdering av korttidsstabilitet i leire på basis av udrenert skjærfasthet". NGM-79, Helsingborg, pp. 585 – 596. (in Norwegian).

Aas, G. (1986). In situ site investigation techniques and interpretation for offshore practice. Recommended interpretation of vane tests. NGI Report 40019-24. Oslo 1986-09-08. 87 pp.

Aas, G. and Lacasse, S. (2021a). Shear Strength of Soft Clay in Terms of Effective Stresses. 1st paper in this NGI publication.

Aas, G. and Lacasse, S. (2021b). K_0 as a function of changes in stress and strain conditions during consolidation, unloading and reloading. 2nd paper in this NGI publication.

Aas, G., Lacasse, S., Lunne, T. and Høeg, K. (1986). Use of in Situ Tests for Foundation Design on Clay. ASCE Conf. in Situ '86, Blacksburg, Virginia, USA. pp. 1–30.

ASTM (2009). STP 1014 on Vane Shear Strength Testing in Soils. *Annual Book of ASTM Standards*, 04.09.

Bandis, C. and Lacasse, S. (1986). In situ site investigation techniques and interpretation for offshore practice. Interpretation of self-boring and push-in pressuremeter tests in Haga clay. NGI Report 40019-09. Oslo 1986-09-08. 109 pp.

Bjerrum, L. (1972). Embankments on soft ground, SOA Report. ASCE. Spec. Conf. Performance of Earth and Earth-Supported Structures. Lafayette, IN, USA. **2**: 1–54.

Bjerrum, L. and Andersen, K.H. (1972). In situ measurements of lateral pressures in clay. Proc. 5th International Conference on Soil Mechanics and Foundation Engineering, Tokyo. **1**: 11–20.

Chandler, R.J. (1988). The in-situ measurement of the undrained shear strength of clays using the field vane: SOA paper. Vane Shear Strength Testing in Soils: Field and Laboratory Studies, ASTM, 1014, 13–44.

Eide, O. and Holmberg, S. (1972). Test fills to failure on the soft Bangkok clay. ASCE. Spec. Conf. Performance of Earth and Earth-Supported Structures. Lafayette, IN, USA. **1**: 159–180.

Kimura, T. and Saitoh, K. (1983). The influence of strain rate on pore pressures in consolidated undrained triaxial tests on cohesive soils. *Soils and Foundations.* **23**(1): 80 – 90.

L'Heureux, J.S., Ozkul, Z., Lacasse, S., D'Ignazio, M. and Lunne, T. (2017). A revised look at the coefficient of earth pressure at rest for Norwegian Clays. Fjellsprengningsteknikk - bergmekanikk - geoteknikk. Oslo 2017. Ch. 35.

Lacasse, S. and Lunne, T. (1982a). In situ horizontal stress from pressuremeter tests. Proc. Symp. Pressuremeter and its Marine Applications, Paris. 187–208.

Lacasse, S. and Lunne. T. (1982b). Penetration tests in two Norwegian clays. ESOPT II, Amsterdam. 661–669.

Lacasse, S. and Lunne, T. (1982c). Dilatometer tests in two soft marine clays. ASCE. Conf. Updating Subsurface Sampling of Soils and Rocks and In Situ Testing, Santa Barbara, CA. 369–379.

Ladd, C.C., Foott, R., Ishihara, K., Schlosser, F. and Poulos, H.G. (1977). Stress-deformation and strength characteristics. SOA Report. Proc. 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo. **2**: 421-94.

Matsui, T. and Abe, N. (1981). Shear mechanisms of vane test in soft clays. *Soils and Foundations*. **21**(4): 69–80.

Menzies, B.K. and Merrifield, C.M. (1980). Measurements of shear stress distribution on the edges of a shear vane blade. *Geotechnique*. **30:** (314–318).

Wiesel, C (1973). Some factors influencing in situ vane test results. Proc. 8th International Conference on Soil Mechanics and Foundation Engineering. Moscow, **1**: (475–478).