STATENS GEOTEKNISKA INSTITUT

SWEDISH GEOTECHNICAL INSTITUTE

No. 23

SÄRTRYCK OCH PRELIMINÄRA RAPPORTER

REPRINTS AND PRELIMINARY REPORTS

Supplement to the "Proceedings" and "Meddelanden" of the Institute

Contributions to the Geotechnical Conference on Shear Strength Properties of Natural Soils and Rocks, Oslo 1967

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STOCKHOLM 1968

FREE DISCUSSION

B. B. BROMS and *H. BENNERMARK* (Sweden):

In this discussion is described a slide which recently took place in a soft sensitive clay. Observations made at this slide have some bearing on the questions discussed in Papers 1/4 by DiBiagio and Aas and 1/1 by Aas.

The slide took place on August 16, 1966, when a 130 m long and 50 m wide area slid into the Säve River close to Gothenburg, in the southwestern part of Sweden. It occurred approximately four hours after the completion of the driving of about 50 spliced wooden piles. The piles were driven along the upper boundary of the slide area as shown in Fig. 1. In this figure is also shown the lateral displacement of the piles and the location of two buildings before and after the slide.

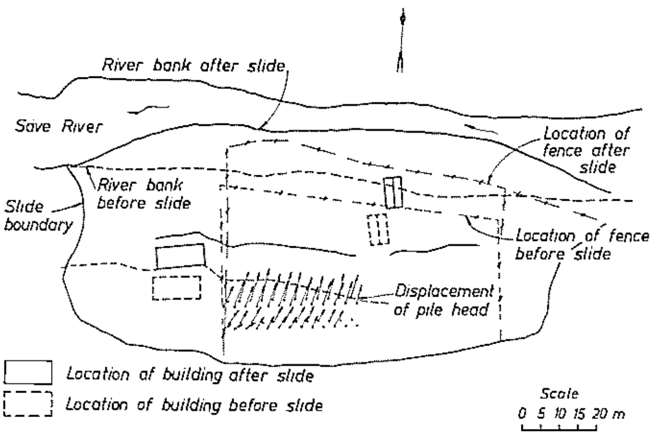

Fig. 1. *Plan over slide area at Säve River 1966.*

A photograph taken a few hours after the slide occurred is shown in Fig. 2. The displaced piles and one of the buildings within the slide area are shown at the centre of the photograph.

Scars of old slides can be seen along the Säve River. These slides have probably been caused by erosion of the river. The Säve River has eroded at the slide area to a depth of about 10 m below the surrounding level area.

The soil conditions were determined from borings within and along the upper boundary of the slide area. The borings show that the soil consists at the ground surface of a dry clay crust with a total thickness of about 1–2 m. Under the dry crust is found a very soft uniform fine-grained clay to a depth of 13 m below the rim of the slide area. (This depth corresponds to 3 m below the water level in the river.) Below this very soft clay is a coarse-grained, medium to soft clay which contains thin sand and silt seams.

The undrained shear strength of the soil was determined by field vane, Swedish fall-cone and unconfined compression tests as shown in Fig. 3. The shear strength increased approximately linearly with depth. The sensitivity of this clay is relatively high (about 25). Since this ratio is less than 50, the clay is not classified as quick, according to the classification systems commonly used in Sweden.

The undrained shear strength was also determined by the NGI direct shear field apparatus. The test results from this apparatus are also shown in Fig. 3. It can be seen that the shear strength determined by this method is higher than by the three other methods. This strength difference indicates that the shear strength is higher along horizontal planes than along corresponding vertical or inclined planes. The clay thus appears to be anisotropic. It was, however, only possible to determine with the NGI apparatus the undrained shear strength to a depth of 4.25 m below the ground surface.

The stability of the slope was analyzed by the $\phi = 0$-method and assuming that the failure surface was cylindrical. The calculated critical failure surface is shown in Fig. 4. This failure surface corresponds to a safety factor of 1.08 and 0.95 with respect to the undrained shear strength as determined by the field vane and the Swedish fall-cone test, respectively. The influence of the piles has been disregarded in the analysis.

The location of the failure surface determined by field vane tests and resistivity measurements coincided closely with the theoretically determined critical failure surface.

The shear strength as determined by the NGI field direct shear apparatus was, however, about 30% larger than that determined by the other methods, as shown in Fig. 3. This shear strength will thus correspond to a safety factor of

Fig. 2. *Photo of slide area.*

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about 1.30, if it is assumed that the ratio of the shear strength by the NGI apparatus and that determined by field vane tests is also representative of the shear strength of the clay below the depth 4.25 m.

It can be seen in Fig. 4 that the stability is primarily governed by the shear strength of the soil in the horizontal direction, that is, the shear strength determined by the NGI field direct shear apparatus. The corresponding factor of safety is close to 1.30.

In Fig. 5 are shown the results from a direct shear test with the NGI apparatus. The test was carried out at a depth of 4.0 m below the ground surface. It can be seen that the shear resistance increased with increasing lateral displacement, reached a maximum whereafter it decreased. A relatively small lateral displacement (5–6 mm) was, however, required to reach the peak resistance.

The pile driving along the upper rim of the slide area caused the soil located between the piles and the river to move downwards towards the river. Calculations indicate that the lateral displacement caused by the pile driving can be sufficient to exceed the peak point of the shear stress-displacement relationship shown in Fig. 5. This displacement can also be sufficient to reduce the undrained shear resistance of the clay by about 20 to 30% and the safety factor to about 1.0. Once the shear strength has been reduced, the weight of the sliding mass is sufficient to cause additional lateral displacements and additional reductions of the shear strength of the soil.

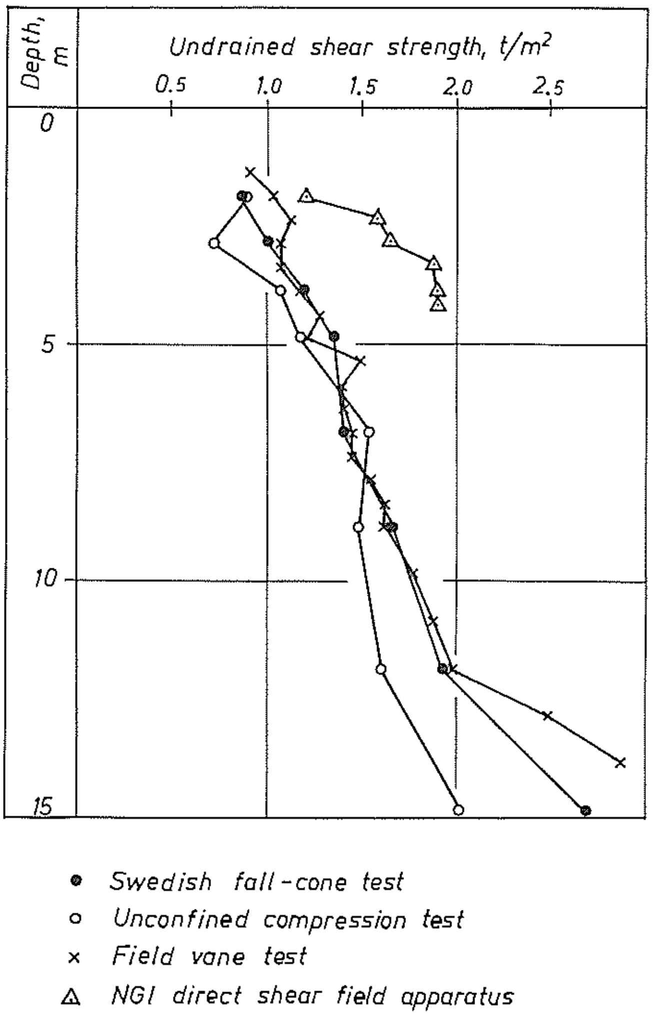

Fig. 3. *Undrained shear strength determined by different methods.*

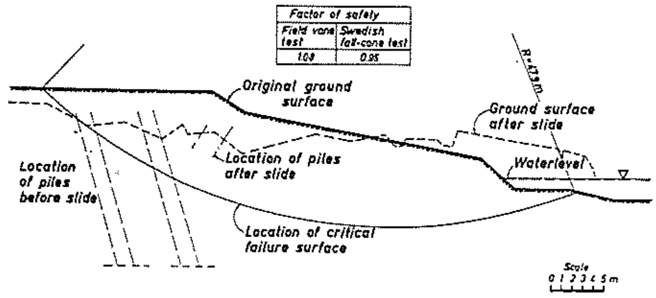

Fig. 4. *Analysis of slide.*

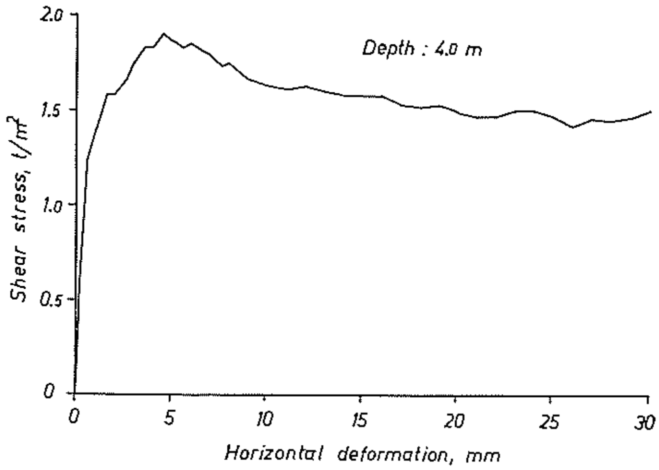

Fig. 5. *Typical result from NGI direct shear field apparatus.*

MR. C.-E. WIESEL (Sweden):

Paper 1/1 describes an investigation where the shear strength anisotropy of some Norwegian clays has been determined with field vane tests. During this investigation vanes having different height/diameter ratios were used. The purpose of this discussion is to present some additional test data from a similar investigation at Lilla Edet, Sweden.

It is shown in Paper 1/1 that the relationship between the failure torque and the undrained shear strengths acting along vertical and horizontal planes can be expressed by Eq. (2) shown in Fig. 1 where

M = failure torque
τ_v = undrained shear strength acting along a vertical plane
τ_h = undrained shear strength acting along a horizontal plane
D = diameter of the failure cylinder
H = height of the failure cylinder

It should be noted, however, that Eq. (2) is based on the following assumptions:

a) the failure surface is a cylinder with a diameter and height equal to the dimensions of the vane,
b) the shear strength is fully mobilized along the failure surface,
c) the shear stresses are uniformly distributed on the failure surface.

It is shown in Paper 1/1 that Eq. (2) is a straight line on a graph where the vertical and horizontal axes represent $2M/\pi D^2H$ and $D/3H$, respectively. This line intersects the vertical axis at τ_v and its inclination is τ_h. The intersection of this line with the negative horizontal axis represents the ratio τ_v/τ_h.

Eq. (2) in Fig. 1 can be transformed into Eq. (3), as shown in the same figure. This eqnation is also a straight line on a graph where the vertical and horizontal axes represent $6M/\pi D^3$ and $3H/D$ respectively. This line intersects the vertical axis at τ_h and has an inclination equal to τ_v. The intersection of this line with the negative horizontal axis corresponds to the ratio τ_h/τ_v.

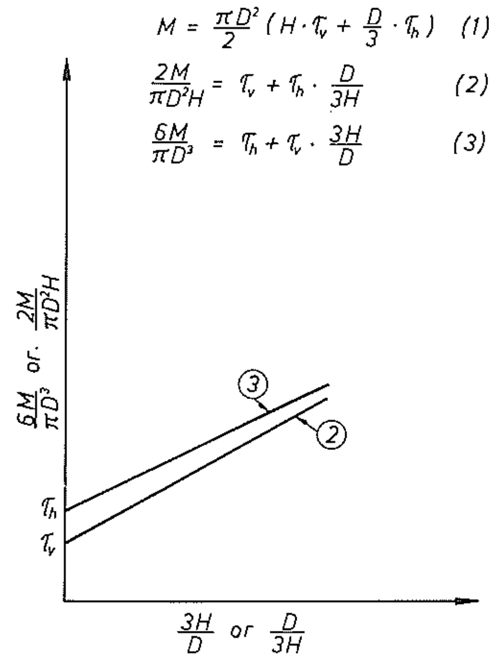

Fig. 1.

The proposed method in Paper 1/1 was used at Lilla Edet, located in the Göta River Valley in the western part of Sweden. The soil at the test site consisted of a normally consolidated or only very slightly overconsolidated marine clay with the following geotechnical properties:

Natural water content, w	80–100%
Liquid limit, w_L	62– 82%
Plastic limit, w_P	30– 36%
Plasticity index, I_P	31– 49%
Sensitivity (cone tests), S_t	12– 59%
Clay content	about 70%

The undrained shear strength as determined by standard vane tests in-situ varied from about 1 t/m² at a dephth of 2 m to about 2.3 t/m² at a depth of 10 m below the ground surface.

The shear strength anisotropy of the clay was determined with vanes with the following six different H/D ratios, namely 0.125; 0.25; 0.5; 1.0; 2.0 and 3.1. Tests were performed for each H/D-ratio in one or two boreholes at one meter intervals between 2 and 10 m below the ground surface.

The results are shown in Fig. 2. Each point represents the average of nine values. It can be seen that the experimentally determined relationships are not linear. Similar results were obtained when each level was studied separately.

The reason why the determined relationships are not linear may be that one of the assumptions mentioned above is not fulfilled. For example the assumption of cylindrical failure surfaces may not be fulfilled at low H/D-ratios. However, even if the test results with vanes having H/D-ratios lower than 0.5 are excluded from Fig. 2, the remaining points do not correspond to a linear relationship. One additional possible reason why non-linear relationships are obtained may be that the shear strength is not fully mobilized

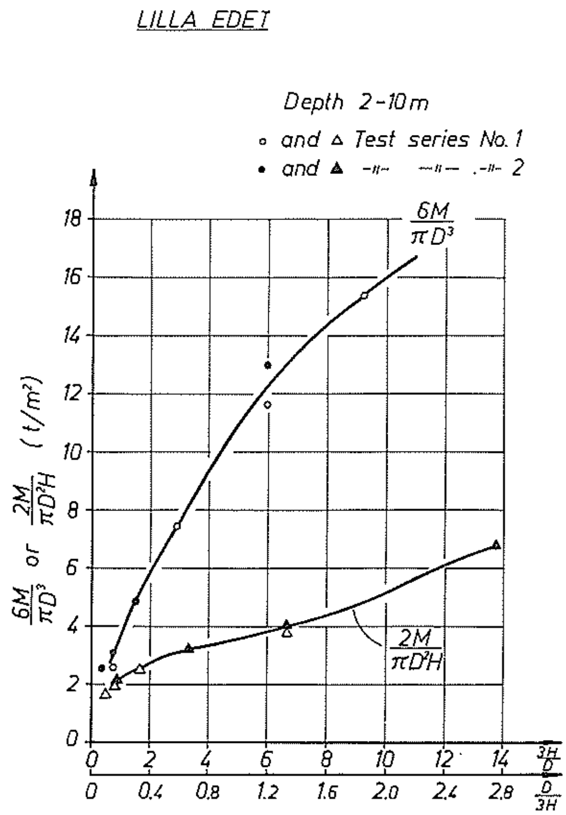

Fig. 2.

at the failure torque. This occurs when the shear strengths τ_v and τ_h are not developed at the same angle of rotation of the vane. An example is shown in Fig. 3. In this case the angle of rotation which corresponds to the maximum torque will depend on the ratio H/D. This means that the shear

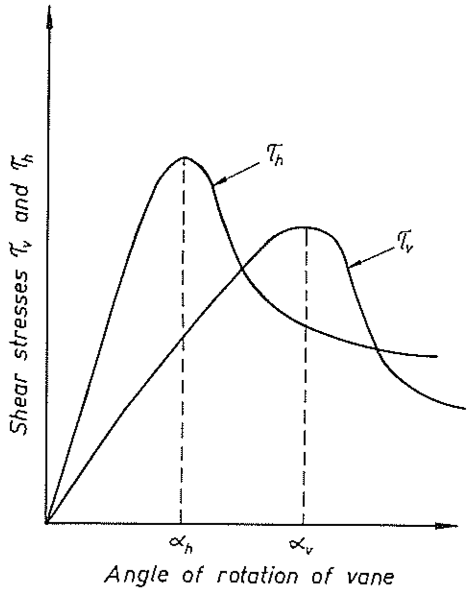

Fig. 3.

stresses acting along the cylindrical failure surface are dependent on the ratio H/D and that the shear strength along the horizontal planes and/or the shear strength along the vertical failure surface is not fully mobilized at failure. The shear stresses τ_v and τ_h in the equations shown in Fig. 1 are in such a case not equal to the shear strengths but equal to those shear stresses which correspond to the angle of rotation at the maximum torque. These stresses will thus be dependent on the H/D-ratio. Another possible reason why nonlinear relationships are obtained may be that the shear stress distribution along the horizontal planes is also dependent on the H/D-ratio.

In conclusion it seems that the proposed method in Paper 1/1 is not suitable for the determination of the anisotropy of the Lilla Edet clay. It may also be questioned whether some of the results shown in Paper 1/1 (Fig. 4) are linear. If the obtained relationships are not linear then the determined values of the anisotropy ratio may not be correct.

Another point of interest is that the experimentally determined relationships can be linear even if the shear strengths along the horizontal and vertical failure surfaces are not fully mobilized at the failure torque. The proposed method can in such a case give incorrect results. Before any conclusions are drawn about shear strength anisotropy it is necessary to verify the assumptions upon which the proposed method is based.

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Contributions to the Geotechnical Conference on Shear Strength Properties of Natural Soils and Rocks, Oslo 1967

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STOCKHOLM 1968

Effective Angle of Friction for a Normally Consolidated Clay

Angle de frottement effectif dans un sol normalement consolidé

by *Rolf Brink*, Consulting Engineer,
Allmänna Ingenjörsbyrån AB, Stockholm, Sweden

Summary

In order to determine the long-term stability of a homogeneous, normally consolidated clay, its shear strength under drained conditions was evaluated. The effective angle of friction was determined by consolidated, drained shear tests and by consolidated undrained triaxial tests. The results obtained by these two methods showed a wide difference, namely 22° and 26° resp. Two methods were used for correction of the triaxial test results with regard to the stress history of the test. Both these methods are based on the 'true' angle of internal friction, which was determined by a series of drained shear tests. With the 15° 'true' angle of friction thus obtained, the triaxial test results were corrected from 26° to 22°, i.e. the same value as that obtained in the direct shear test.

Résumé

Pour calculer la stabilité à long terme, on a cherché la résistance au cisaillement drainé dans un sol normalemenr consolidé. L'angle de frottement effectif a été déterminé par des essais de cisaillement drainés et consolidés et dans des essais triaxiaux consolidés et non drainés. On a obtenu de grandes différences entre les différentes méthodes : 22° et 26° respectivement. Vu l'évolution des contraintes des essais triaxiaux, deux méthodes sont décrites pour la correction des résultats des essais triaxiaux. Les deux méthodes sont construites sur l'angle de frottement interne 'vrai', déterminé dans une série d'essais de cisaillement drainés. A l'aide de l'angle de frottement interne 'vrai', 15°, on a ajusté le résultat des essais triaxiaux de 26° à 22°, c'est-à-dire le même résultat que dans les essais de cisaillement drainés.

Introduction

In connection with the design of a road tunnel under the Göta River at Tingstad in Gothenburg, extensive field and laboratory tests were carried out by the Swedish Geotechnical Institute in co-operation with the Danish Geotechnical Institute in Copenhagen. This tunnel, which is now under construction, will have a length of about 500 m and consists of 90 m long concrete elements which are lowered into a dredged trench in the river. One of the problems in the design of the tunnel was the deep excavation required in the soft clay layers under the river. Also, the final structure should be able to resist swelling and settlement occurring during and after the construction period.

The tunnel was designed to float in the approx. 100 m deep clay layer. An extensive field and laboratory programme was carried out, with the accent on the strength properties of the clay. These investigations are described in more detail in the following.

Undrained Shear Strength

The undrained shear strength was determined in the laboratory on undisturbed soil samples and in-situ with a vane borer.

The undisturbed samples were mainly collected with a foil sampler (Kjellman, Kallstenius and Wager, 1950). With this sampler it is possible to obtain a continuous, undisturbed core with a length of, generally, 25–30 m and a diameter of about 7 cm. A continuous core down to a depth of 90 m from the soil surface was obtained in one of the bore-holes. The results of the laboratory investigation on these samples are shown in Fig. 1.

The topsoil consisted of a somewhat muddy, sandy clay to a depth of 10 m, underlain by a grey to dark-grey clay with sulphide streaks. The undrained shear strength was determined by fall-cone tests and by unconfined compression tests. The results obtained with these test methods are shown in Fig. 1. The shear strength increased linearly down to a depth of 30 m. The increase below this depth was small. Similar tendencies can be found on samples collected by an ordinary piston sampler. The explanation of this phenomenon seems to be the stress alternation in samples obtained at greater depth. The shear strength diagram contains a comparison line which is a low average value from the vane borings. The good agreement between laboratory tests and vane tests even at great depths is due to the relatively low degree of disturbance obtained with the foil sampler.

A summary of the total number of vane tests carried out is given in Fig. 2. The summary shows how homogeneous are the soil layers investigated within the test area, which measured about 1,000 m × 200 m. A constant increase in shear strength with depth is expected, as the pore water pressure is in good agreement with the hydrostatic pressure.

Drained Shear Strength

The drained shear strength was determined by shear tests and triaxial tests. The latter tests were carried out by the Danish Geotechnical Institute.

The direct shear tests were performed as slow, drained tests in a direct shear apparatus constructed by the Swedish

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Geotechnical Institute. The samples had a circular area with a diameter of 6 cm and a height of 1 cm. The tests were of the *Controlled Stress-type* and each load increment (0.05 × normal stress) was acting about half an hour. Each sample was consolidated for 24 hours during the final load increment before the shear test. The average value of the apparent angle of friction obtained in the 12 different series of tests, each containing 4 tests, was 22°, with a maximum difference of only about ±0.5°. The effective cohesion was found to be 0.

The triaxial tests were performed mainly as consolidated, undrained tests. Variations were made (a) with the pore water pressure equal to 0 and varying cell pressure σ_3, and (b) with measuring of the pore pressure and a constant cell pressure σ_3.

The rate of deformation at failure was kept constant and an attempt was made to keep the time from the beginning of the test until failure occurred to 2 hours.

In addition, a number of slow, consolidated drained triaxial tests were carried out (rate of deformation 0.00133 mm/min). The time until failure occurred was 6–9 days. The deformation at failure was 20–30%.

The effective angle of friction varied between 22 and 30° and the effective cohesion between 0 and 0.8 ton/m². If it is assumed that the total number of points are on a straight line, the average value of about 50 tests shows an effective angle of friction of 25.9° and an effective cohesion of 0.41 ton/m².

The difference between the effective angle of friction as determined by drained shear tests and by consolidated, undrained triaxial tests is relatively large (22° and cohesion 0; 25.9° and cohesion 0.4 ton/m²). A more careful investigation of the difference was thus indicated.

Some authors (e.g. Casagrande and Wilson, 1953) have pointed out that consolidated, undrained triaxial tests do not correspond to the failure conditions in the ground. During consolidated, undrained triaxial tests on normally consolidated clays the confining effective stress decreases since the pore pressure increases and *at failure the sample is overconsolidated* with respect to the effective confining pressure. Various methods have been developed for correction of results from the triaxial test for the overconsolidation occurring during shear.

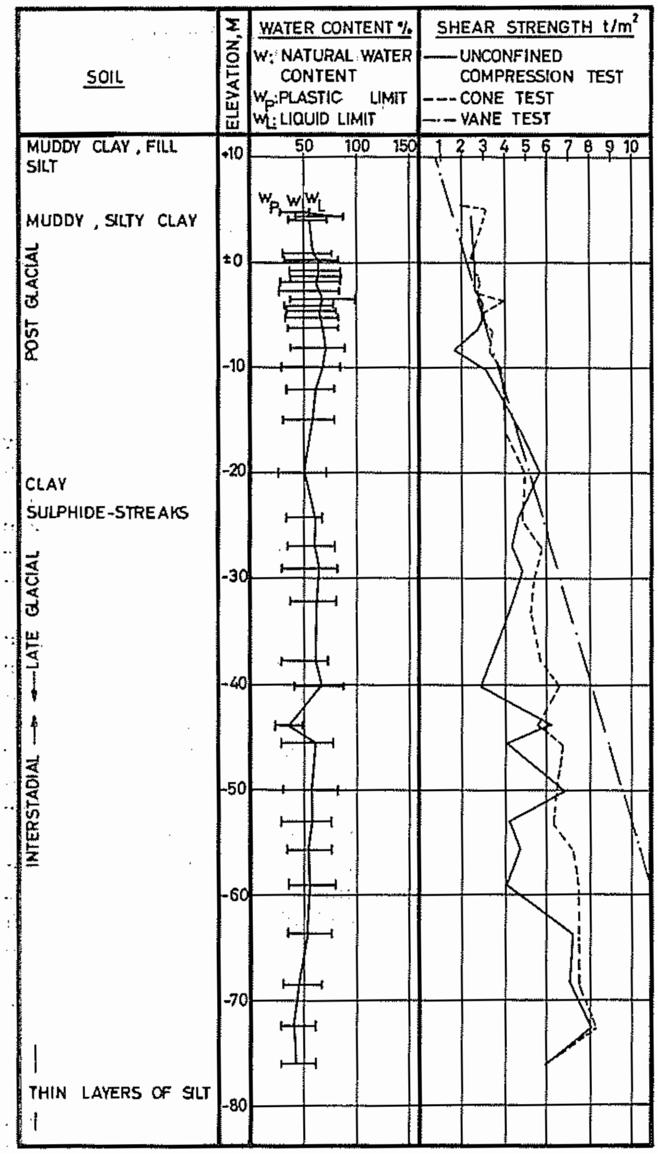

Fig. 1. *Profile of a 90 m deep bore hole.*
Échantillon provenant d'un forage de 90 mètres de profondeur.

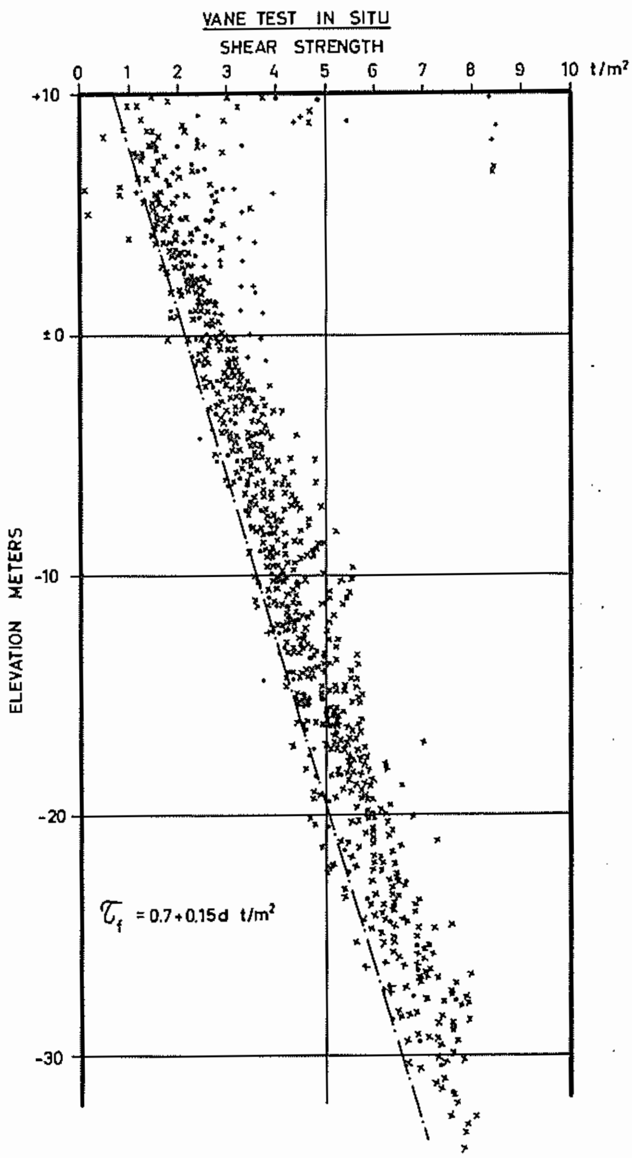

Fig. 2. *Results from vane tests.*
Résultats des essais au scissomètre.

Correction according to Osterman

Osterman (1960) has suggested a graphical method for correction of the results from undrained triaxial tests. The following hypothesis is valid according to this method.

The shear strength in a saturated clay can be separated into two parts. The first part is the true cohesion, which is a function of the water content only, and the other the true friction, expressed by the true angle of internal friction. In a consolidated, undrained triaxial test on a normally consolidated clay – the discussion is valid only for normally consolidated clays and not for slightly or highly overconsolidated clays – the sample is consolidated at a cell pressure that is greater than the normal stress on the actual failure surface at failure of the sample. At failure there will be a friction part corresponding to the normal effective stress on the failure plane at failure and a true cohesion part corresponding to the consolidation pressure. The consolidation pressure is higher than the effective normal stress acting on the failure plane at failure. The graphical correction proposed by Osterman is shown in principle in Fig. 3. It is assumed in this method that the true angle of friction is known. However, as earlier pointed out by Osterman, the adjustment can be carried out without major errors using a roughly estimated value for the true angle of internal friction, preferably chosen according to Gibson (1953).

Correction according to Odenstad

Odenstad (1961) based his formulae for correction of the effective angle of friction on the theory of elasticity. The corrected angle can be determined as shown in Fig. 4.

The effective angles of friction determined by the triaxial tests have been corrected according to the methods proposed by Osterman (1960) and Odenstad (1961). The corrected angle is almost the same and the greatest difference between the two methods is 0.1–0.2°.

'True' Angle of Friction

In order to make a more careful correction of results from the triaxial tests, a series of consolidated, drained shear tests was performed to determine the true angle of internal friction. The tests were performed according to a method proposed by Hvorslev (1937) and Terzaghi (1938). The method involved drained shear tests on normally consolidated and overconsolidated samples. The relation between shear strength and normal stress, and also the relation between shear strength and water content, were determined from normally consolidated tests at different degrees of overconsolidation. At a certain water content there are different shear strength in the different test series. True cohesion is assumed by Hvorslev and Terzaghi to be a function of water content alone, and to be the same in the different test series. The difference in shear strength at a constant water content is thus assumed to be solely a result of differences in the true internal friction.

Four series of shear tests were performed on normally consolidated clay and on clay consolidated at 8, 16 and 25 kg/cm².

The water content was carefully determined after each test. The test results are shown in Figs 5 and 6. Fig. 5 shows the shear strength in relation to the water content at failure. In Fig. 6. the shear strength is plotted against the normal pressure. For the same water content at failure, the difference

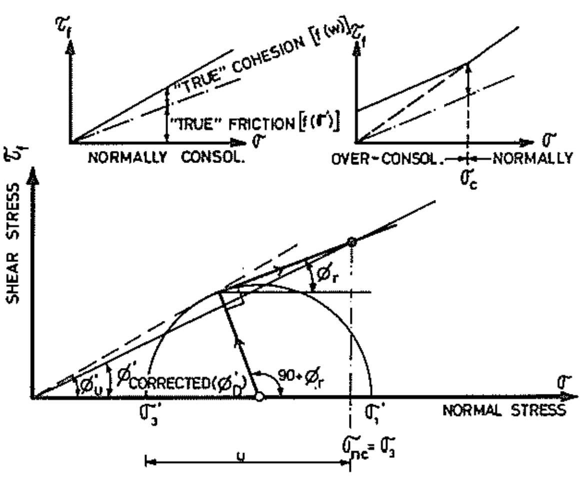

Fig. 3. *Correction according to Osterman.*
Ajustement selon Osterman.

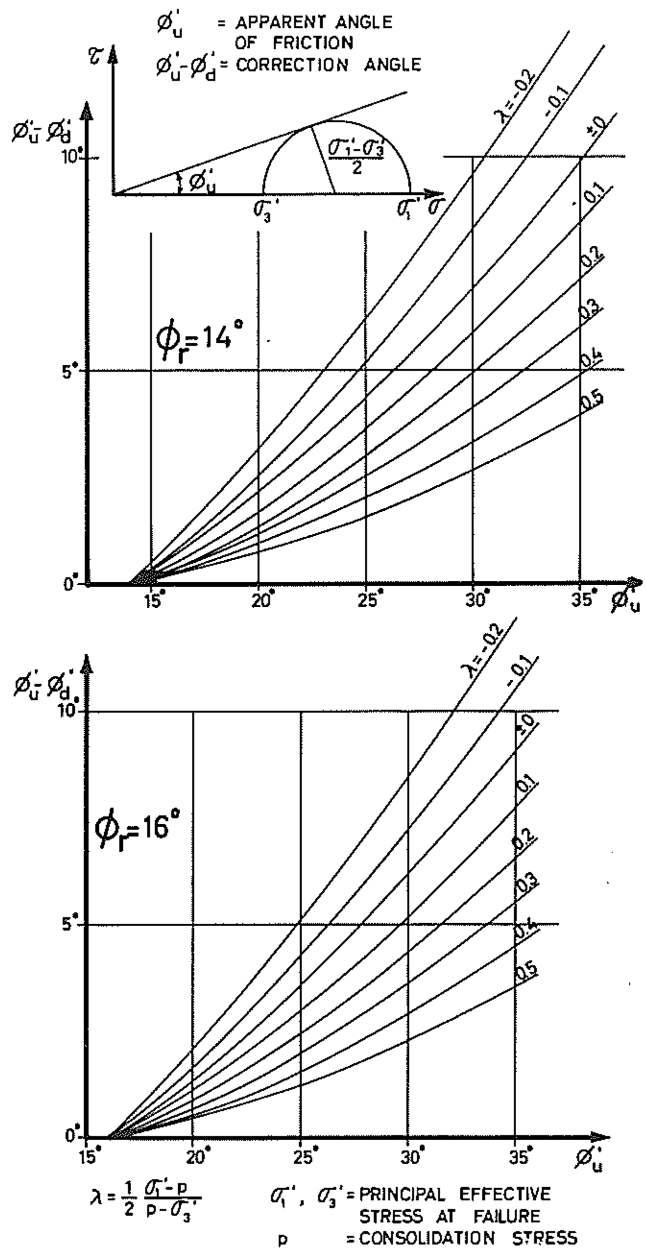

Fig. 4. *Correction according to Odenstad.*
Ajustement selon Odenstad.

in shear strength between normally consolidated and over-consolidated clays can thus be determined, and consequently also the true angle of internal friction. The results are shown in Fig. 7. According to this figure, the true angle of internal friction varies between 13° and 15°.

Gibson (1953) has proposed a failure criterion based on energy conditions. According to this hypothesis it is assumed that failure occurs when the internal work in the sample has reached a maximum. This failure definition considers the internal work caused by volume changes of the sample. Measurements were made of lateral and vertical movements, but these observations were not made with sufficient accuracy to permit analysis according to the Gibson failure hypothesis.

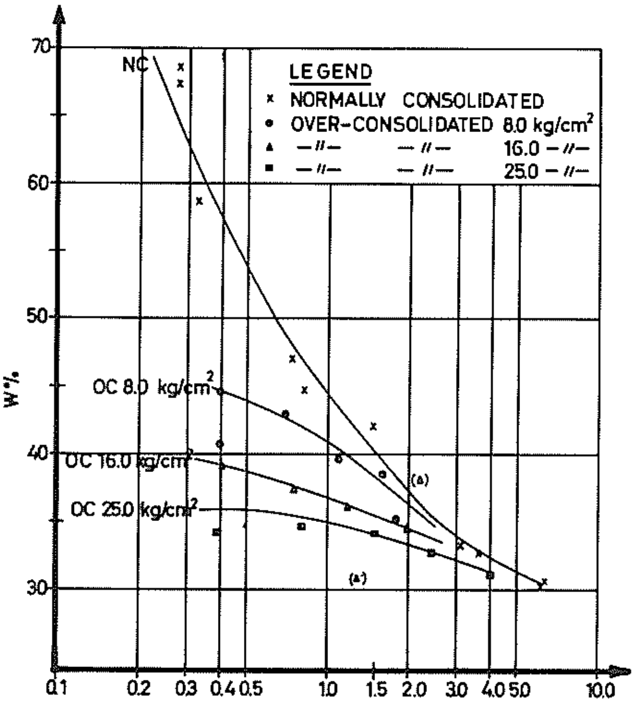

Fig. 5. *Determination of 'true' angle of internal friction: Water content in relation to shear strength.*
Détermination de l'angle de frottement interne 'vrai' : La teneur en eau en fonction de la résistance au cisaillement.

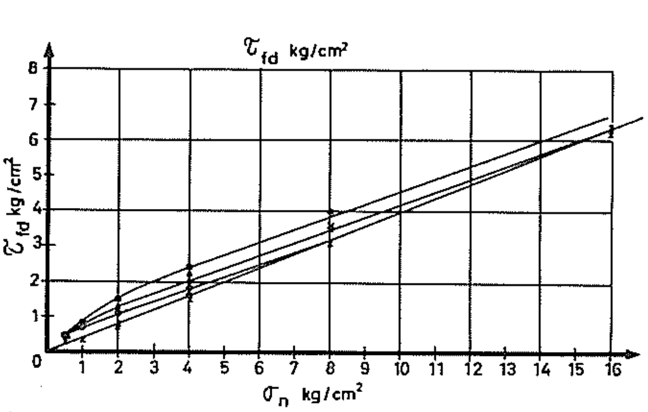

Fig. 6. *Determination of 'true' angle of internal friction: Shear strength in relation to normal stress.*
Détermination de l'angle de frottement interne 'vrai': Résistance au cisaillement en fonction de la contrainte normale.

The computed value of the true angle of friction is therefore probably one or two degrees lower than that determined by the method proposed by Gibson. Gibson reported in his article values of the true angle of internal friction in relation to the plasticity index. According to his summary, the true angle of internal friction for the clay investigated should be 15°—16°.

The Corrected Consolidated Undrained Triaxial Tests

The undrained triaxial tests were corrected with the assumption of a true angle of internal friction of 14°. Fig. 8 shows the thus-corrected effective angle of friction in relation to

$$\frac{\sigma'_1 + \sigma'_3}{2} \Big/ \sigma'_0$$

where σ'_1 and σ'_3 are the effective principal stresses at failure and σ'_0 the assumed greatest consolidation stress in-situ. In the normally consolidated samples where

$$\frac{\sigma_1' + \sigma_3'}{2} \Big/ \sigma'_0 > 1$$

the points are with few exceptions between 20°–22°. This result thus agrees with the results from the drained direct shear tests.

The correction has also been made assuming a true angle of internal friction of 16°, resulting in a one degree higher value for the effective angle of friction.

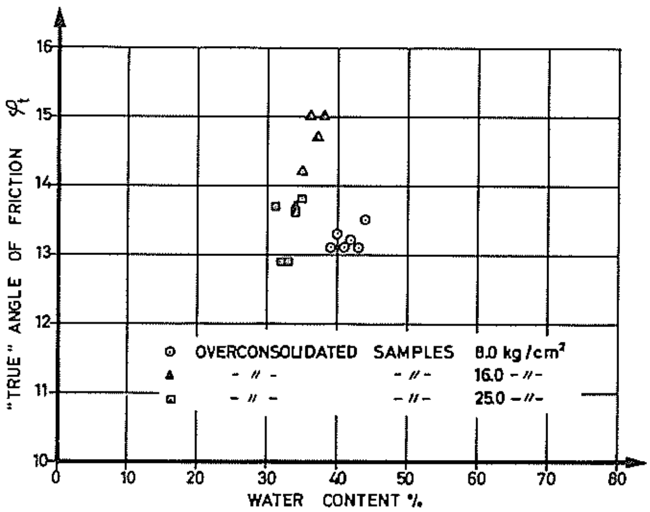

Fig. 7. *'True' angle of internal friction from drained shear tests.*
Angle de frottement interne 'vrai' provenant des essais de cisaillement drainés.

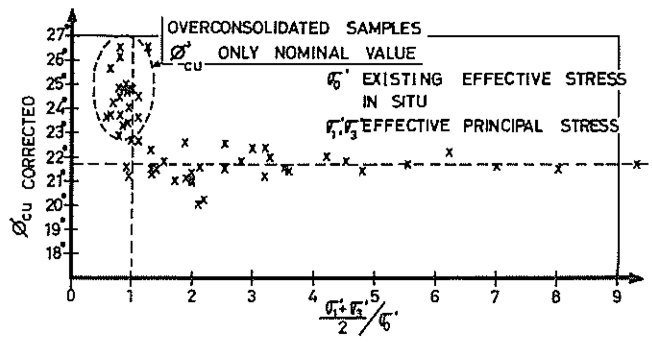

Fig. 8. *Corrected, apparent angle of friction.*
Angle de frottement apparent, ajusté.

Acknowledgement

The above investigations were carried out in the Consulting Dept. of the Swedish Geotechnical Institute. The author is grateful to the late Mr. Sten Odenstad, who was in charge of the geotechnical investigations.

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Shear Strength Parameters and Microstructure Characteristics of a Quick Clay of Extremely High Water Content

Paramètres de la résistance au cisaillement et caractéristiques microstructurales d'une argile fluide à très forte teneur en eau

by *R. Karlsson* and *R. Pusch*[1])
Swedish Geotechnical Institute, Stockholm

Summary

A clay of extremely high sensitivity and extremely high water content was investigated with respect to the strength characteristics. In addition, the constituents and the microstructure of the clay were examined. Microstructural investigations based on electron-microscopy indicate the clay microstructure to be characterized by an extremely open three-dimensional network of particles and particle groups.

The shear strength characteristics were determined by triaxial tests, direct shear tests and unconfined compression tests. The different types of tests gave different values of certain strength parameters, e.g. ϕ_r and c_r. The differences are said to be due to anisotropic factors owing to the different consolidation and stress conditions prevailing in the triaxial and the direct shear tests. The microstructural conditions involved are also discussed.

Résumé

Une argile de très haute sensibilité et de teneur en eau très élevée a été étudiée en ce qui concerne ses caractéristiques de résistance. En complément, on a examiné les constituants et la microstructure de l'argile. Des examens microstructuraux effectué grâce au microscope éléctronique ont indiqué que la microstructure de l'argile se caractérisait par un réseau tridimensionnel très ouvert de particules et de groupes de particules.

Les caractéristiques de résistance au cisaillement ont été determinées par des essais triaxiaux, des essais de cisaillement direct et des essais de compression simple. Les différents types d'essais ont donné des valeurs différentes de certains des paramètres, par exemple ϕ_r et c_r. On dit que les différences doivent être dues aux facteurs anisotropes par suite, de la consolidation différente et des conditions des contraintes qui prévalent lors des essais triaxiaux et de cisaillement direct. Les conditions microstructurales impliquées font également l'objet de discussions.

Introduction

In connection with an investigation for a new main road in Mölndal (south of Gothenburg) layers of clay were found which had a very high sensitivity and extremely high water content in relation to their liquid limit. These factors caused some construction problems which required a detailed geotechnical investigation[2]) of the clay's shear strength and consolidation properties. Some results from these investigations have been used in this paper. Mineralogical and microstructural studies were made in addition.

Typical geotechnical data of the soil deposits are given in Fig. 1. The clay within the upper four or five metres was formed during postglacial time, while the deeper layers are of late glacial age. At a depth of about 20 m, sand and gravel have been identified. The clay was deposited in a salt (marine) environment and was later leached. Pore pressure measurements (see Fig. 1) have shown that artesian conditions exist. The leaching may be due to the resulting ground water flow. The artesian pressure, which probably developed in connection with the land elevation, caused a reduction of the effective stress in the soil.

[1]) The mineralogical and microstructural contribution is reported by Dr. R. Pusch.

[2]) The field investigations were made by Messrs. Kjessler & Mannerstråle, Gothenburg. Part of the investigations has been sponsored by the Swedish Council for Building Research, Stockholm.

The part of the shear strength investigations reported here includes triaxial tests on samples from a depth of 9 m, direct shear tests on samples from 8 m and unconfined compression tests on samples from 7 to 10 m depth. The tests were performed on undisturbed samples taken with the Swedish Standard Piston Sampler, diam. 50 mm (Kallstenius, 1963).

At 6 to 8 m the soil has a clay content ($< 2\mu$) of 80–90 per cent. The electrical resistivity determined from the pore water was of the order of 300 ohmcm, and the total salt content did not exceed 1–2 gr/l. The pore water analysis showed that the clay has been leached to a high degree and that weathering has occurred (K/Na $\sim$ 0.1).

Mineralogical Composition

X-ray study[3]), which concerned untreated clay material, indicated the presence of illite, quartz, various feldspars and chlorite.

The cation exchange capacity of the hydrogen peroxide-treated, ground, clay-size material was 19–21 meq $(100\ g)^{-1}$ indicating some rock-forming minerals in the clay fraction.

A morphological study of dispersed clay-size particles, based on electron micrographs (Pusch, 1966 a), showed a

[3]) X-ray diffraction patterns and cation exchange capacities were determined in co-operation with the Department of Geology, University of Stockholm.

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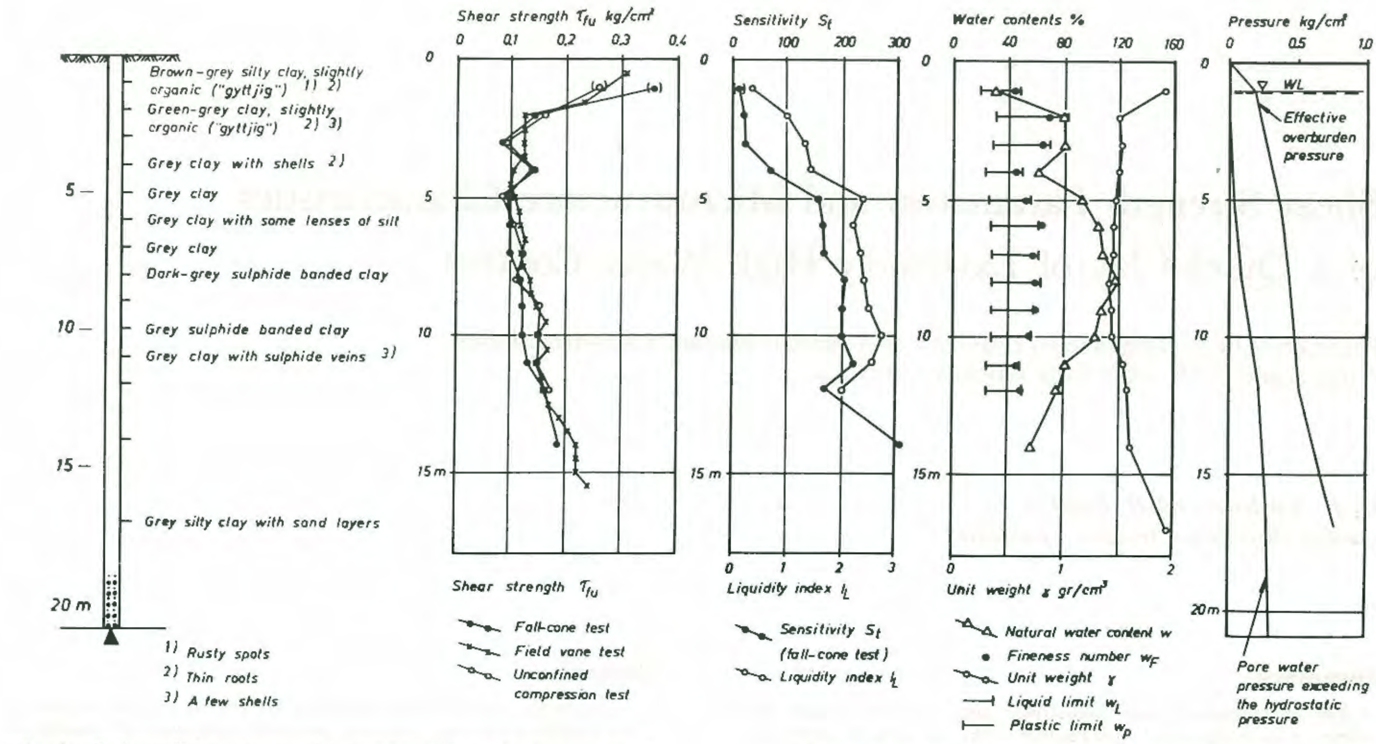


Fig. 1. *Soil conditions at test site at Mölndal, south of Gothenburg.*

Conditions du sol au site d'essai de Mölndal, au sud de Göteborg.

large number of very small particles with a distinct outline and a very small electron penetrability (Fig. 2). This indicates a certain amount of rock-forming minerals also in the finest parts of the clay fraction. This is in accordance with Skempton's (1953) activity value which is of the order of 0.4.

Microstructure

Ultra-thin microtome-cut sections of plastic-treated clay were studied in a Siemens Elmiskop I (Pusch, 1966 b). Earlier investigations have shown that fresh or brackish-water clays have a much more dispersed particle arrangement than salt-water (marine) clays. Both types of clay are characterized by groups or chains of small particles forming links between denser flocs, aggregates or large particles. Also, the median pore diameter is fairly constant, but in the case of salt-water clays the micrographs have shown a certain number of large pores (2–20μ). These large pores mean that a 500 Å thick section of a salt clay has a pore area of 25–65 per cent of the total area, whereas it is 10–25 per cent for a fresh or brackish-water clay. This is in accordance with the permeability as determined from oedometer tests, since the permeability was found to be roughly proportional to the relative pore area.

Fig. 3 shows a micrograph of a the Mölndal clay from 6.5 m depth. This clay seems to be more 'dispersed' than the salt clays previously investigated. Its median pore diameter is somewhat larger but its relative pore area is much larger than those of the fresh and brackish-water clays. The relative pore area of the Mölndal clay is about 60 per cent. However, the Mölndal clay is only slightly more pervious than the fresh and brackish-water clays (2.5×10^{-8} cm/sec and 1.8×10^{-8} cm/sec, respectively). This can perhaps be explained hy the lack of large pores in the Mölndal clay.

Fig. 3 demonstrates that the particle network is very porous and does not consist of large aggregates. The afore-mentioned disperse type of structure is only apparent since very few single particles can be seen. The particles form small clusters at a relatively regular mutual distance, probably caused by the fairly uniform particle size. These clusters may represent the truncated links of a continuous network (Fig. 4). The links probably bear a resemblance to the network formed by fusion of tactoids (Bernal and Fankuchen, 1941, p. 133).

The open arrangement of particles indicates a small value of the internal friction because the effects of macro- and micro-dilatancy are probably small.

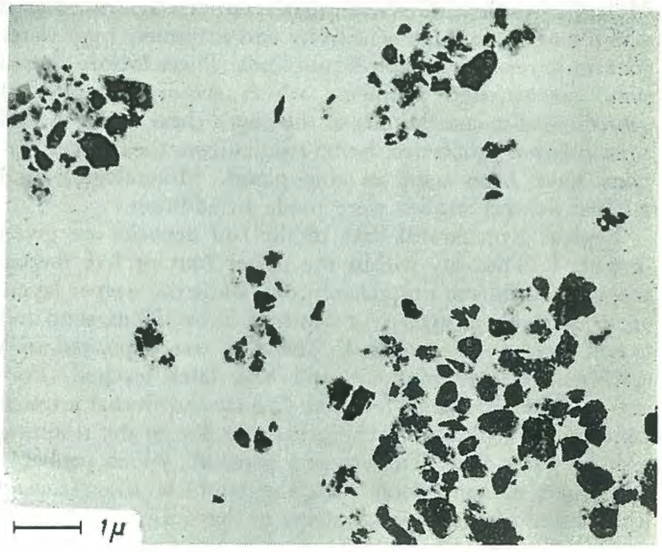


Fig. 2. *Dispersed clay particles of a clay sample from 6.5 m depth. Notice the large number of dense particles. Electronic magnification 7000×.*

Particules dispersées d'argile dans un échantillon prélevé à 6.5 m de profondeur. Noter le grand nombre de particules denses. Grossissement électronique 7000×.

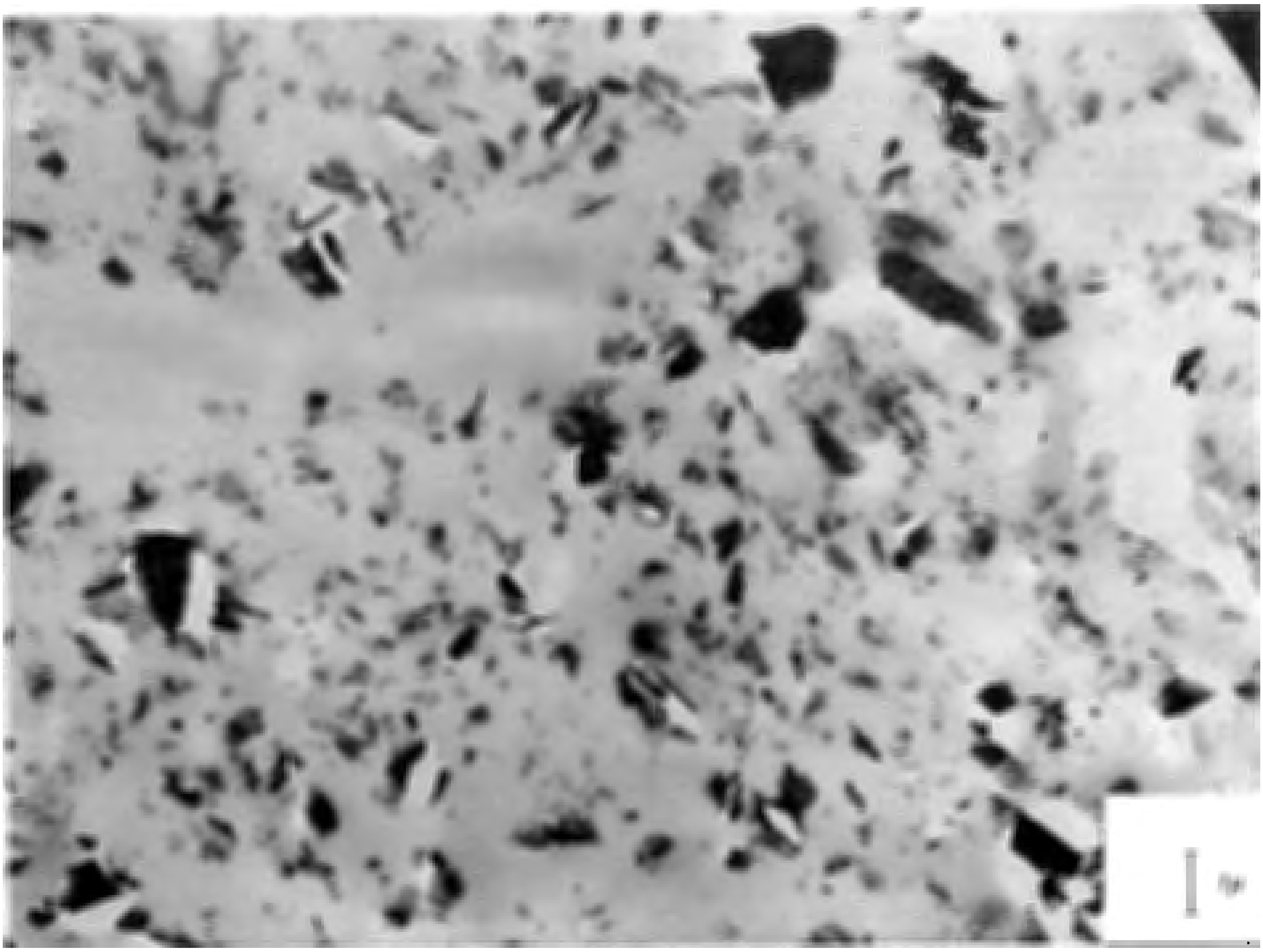


Fig. 3. *Micrograph of an ultra-thin section (thickness about 500 Å) through a clay sample from 6.5 m depth. Electronic magnification 5500 ×. The dark objects represent the solid phase whereas the bright areas represent the pore system. Some mechanical disturbances can be seen adjacent to certain bright spots.*

Micrographie d'une section ultra-mince (épaisseur environ 500 Å) à travers un échantillon d'argile prélevé à 6.5 m de profondeur. Grossissement 5500 ×. Les objets sombres représentent la phase solide tandis que les surfaces brillantes représentent le système interstitiel. On peut déceler quelques perturbations mécaniques à proximité de certains endroits brillants.

Shear Strength Determination

Triaxial tests

Consolidated-undrained triaxial tests with pore-pressure measurements were made in an apparatus of the Geonor type (Andresen et al., 1957). All samples were consolidated isotropically and tested at a constant strain rate. One test series was performed on normally consolidated samples at pressures between 0.5 and 6.15 kg/cm^2. A second series was performed on overconsolidated samples, consolidated at 6.15 kg/cm^2. The samples were allowed to swell under different reduced pressures before testing. The test results are given in Fig. 5 (see also Table 1).

In Fig. 5 (a) the initial water content, w_n, and the water content at failure, w_f, are plotted against the corresponding stress σ_3. Figs 5 (b) and 5 (c) show the deviator stress at failure, $\frac{1}{2}(\sigma_1-\sigma_3)_f$ plotted against the stress σ_3 and the effective stress σ_3', respectively. The apparent angle of shearing resistance, ϕ_{cu}, and the effective angle of shearing resistance, ϕ'_{cu}, are given in Fig. 5. The true angle of internal friction, ϕ_r, and the true cohesion, c_r, have been evaluated at $w_f=55\%$, according to Hvorslev (1937) and Terzaghi (1938). In addition, the true cohesion, c_r, has been computed as the difference between the value of the shearing resistance, τ_f, and the corresponding friction component at failure. In Fig. 5 (d) the values of c_r are related to

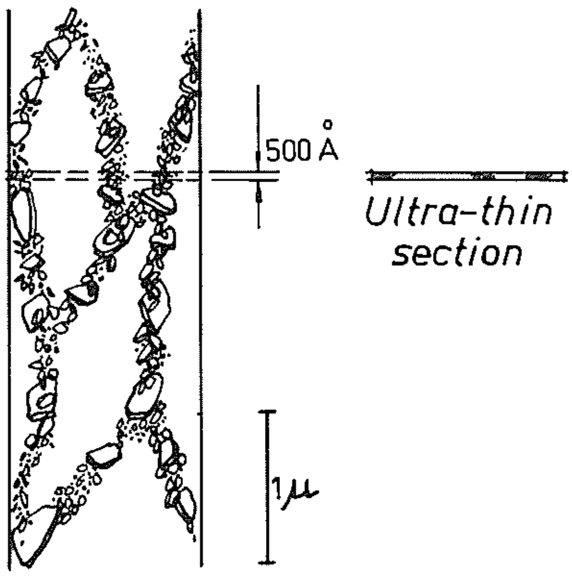


Fig. 4. *Probable mode of aggregation in the three-dimensional network of clay particles in the Mölndal clay.*
Mode probable d'agrégation dans le réseau tridimensionnel de particules d'argile dans l'argile de Mölndal.

the stress σ_3; the $\varkappa$-value is also given in the figure. Fig. 5 (e) shows the relationship between the pore pressure coefficient A_f and the stress σ_3.

The effective angle of shearing resistance, ϕ'_{cu}, obtained from undrained tests is not directly comparable with the effective angle ϕ'_d from drained tests since in the undrained tests the effective stress decreases during shearing and the specimens are thus overconsolidated at failure (see Casagrande and Wilson, 1953, and Osterman, 1960). The value of ϕ'_{cu} has been corrected according to a method suggested by Odenstad (1961). The calculations are given in Fig. 5.

Direct Shear Tests

The direct shear tests were carried out in an improved apparatus of the Kjellman type (Kjellman, 1951) shown in principle in Fig. 6.

In this apparatus the sample is first consolidated in an oedometer ring (Fig. 6 a). After consolidation, the oedometer ring is replaced by a rubber hose which is supported by a series of metal rings (see Fig. 6 b). The metal rings are initially spaced 0.5 mm apart to allow vertical and horizontal movements.

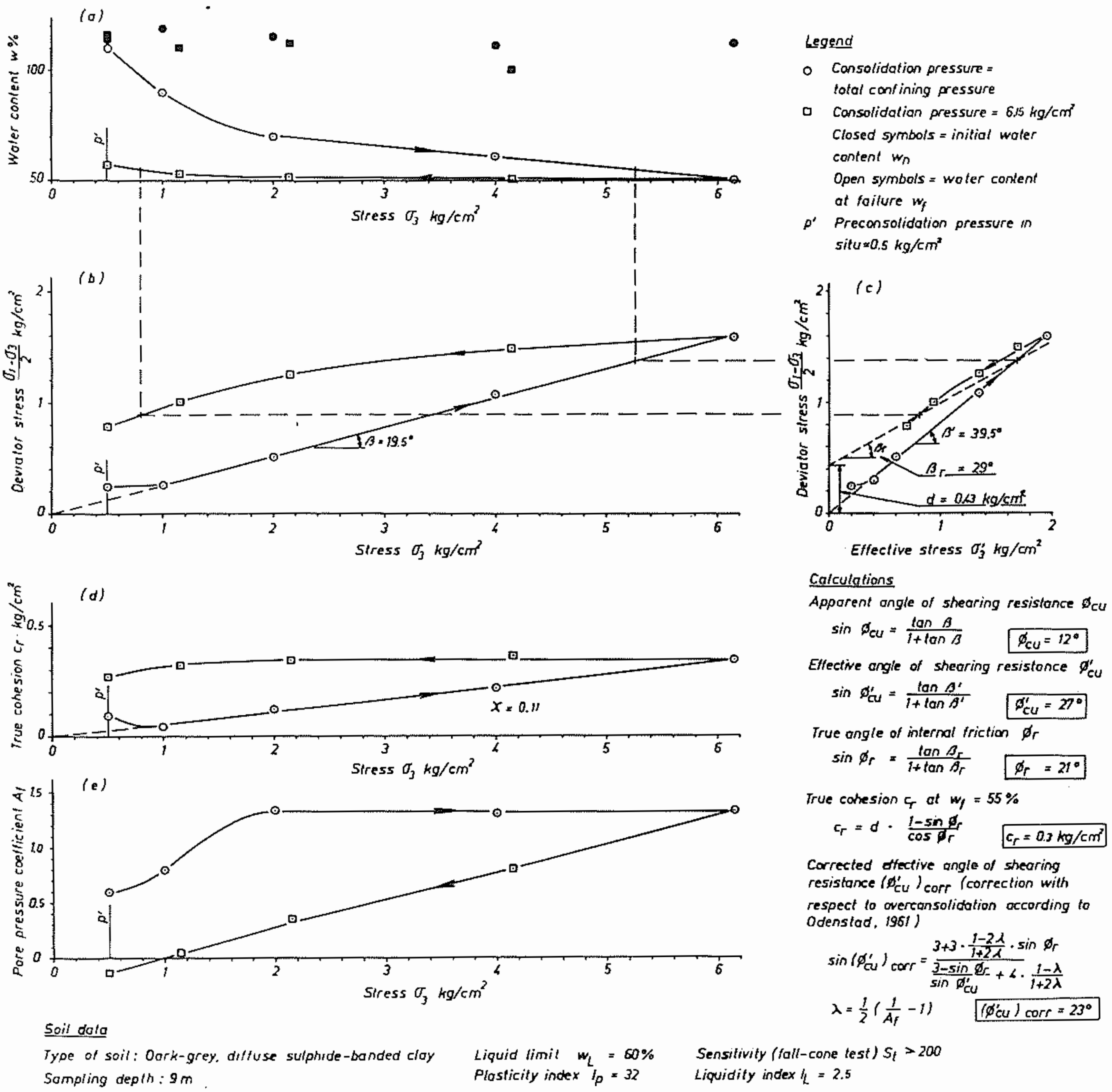


Fig. 5. *Results in connection with triaxial tests and calculated parameters.*
(a) Initial water content and water content at failure versus σ_3.
(b) Deviator stress $(\sigma_1-\sigma_3)/2$ versus σ_3.
(c) Deviator stress $(\sigma_1-\sigma_3)/2$ versus σ_3'.
(d) True cohesion c_r versus σ_3.
(e) Pore pressure coefficient A_f versus σ_3.

Résultats en rapport avec les essais triaxiaux et les paramètres calculés.
(a) Teneur en eau initiale et teneur en eau à la rupture en fonction de σ_3.
(b) Contrainte déviatrice $(\sigma_1-\sigma_3)/2$ en fonction de σ_3.
(c) Contrainte déviatrice $(\sigma_1-\sigma_3)/2$ en fonction de σ_3'.
(d) Cohésion vraie c_r en fonction de σ_3.
(e) Coefficient de pression interstitielle A_f en fonction de σ_3.

Both consolidated-undrained (quick) and drained shear tests were made under controlled stress conditions using samples of 50 mm diameter and 20 mm height. In the undrained tests, the consolidation pressures varied between 0.25 and 8 kg/cm^2. The results are given in Fig. 7 (see also Table 1). In Fig. 7 (a) the water contents w_n and w_f are plotted against the corresponding normal stresses, σ. Fig. 7 (b) shows the relationship between shearing resistance, τ_{fu}, and normal stress σ. The apparent angle of shearing resistance, ϕ_{cu}, is also given in the figure.

One series of drained direct shear tests was made on normally consolidated samples (consolidation pressures 0.25 to 8 kg/cm^2) and four test series on overconsolidated samples, consolidated at 1, 2, 4, and 8 kg/cm^2. The overconsolidated samples were allowed to swell under different reduced pressures. The results are given in Fig. 8. In Fig. 8 (a) the water contents w_n and w_f are plotted against the corresponding normal effective stress σ'. The relationship between the shearing resistance, τ_{fd}, and the normal effective stress, σ', is shown in Fig. 8 (b). The effective angle of shearing resistance, ϕ'_d, is also given. The values of the shearing resistance, τ_{fd}, have been corrected for the volume changes (Gibson, 1953). From the corrected values $(\tau_{fd})_{corr}$, the true angle of internal friction, ϕ_r, and the true cohesion, c_r, are evaluated at $w_f = 50\%$ (Fig. 8 c). In addition, the true cohesion, c_r, has been computed as the difference between the corrected value of the shearing resistance, $(\tau_{fd})_{corr}$, and the friction component at failure. The values of c_r are related to the normal effective stress, σ', in Fig. 8 d, where the value of $\varkappa$ is also given.

The volume-change corrections caused a considerable change in the values of the shearing resistance, especially for the normally consolidated samples. The uncorrected τ_{fd}-values resulted thus in a considerably lower value for ϕ_r and a higher value for c_r than the corrected values ($\phi_r = 6°$ and 18°, and $c_r = 0.55$ kg/cm^2 and 0.85 kg/cm^2, respectively). However, as pointed out by Hvorslev (1953), the corrections according to Gibson's method are somewhat excessive.

Unconfined Compression Tests

The unconfined compression tests were carried out at a constant rate of load increase. The true angle of internal friction, ϕ_r, was evaluated from the inclination of the failure plane (Terzaghi, 1936). The mean value of ϕ_r was 27° with a standard deviation of $\pm 5°$ in a series of 12 tests. The failures were very distinct and occurred at an axial deformation of 1.5 to 2%, indicating a brittle type of failure. The failure planes were not curved.

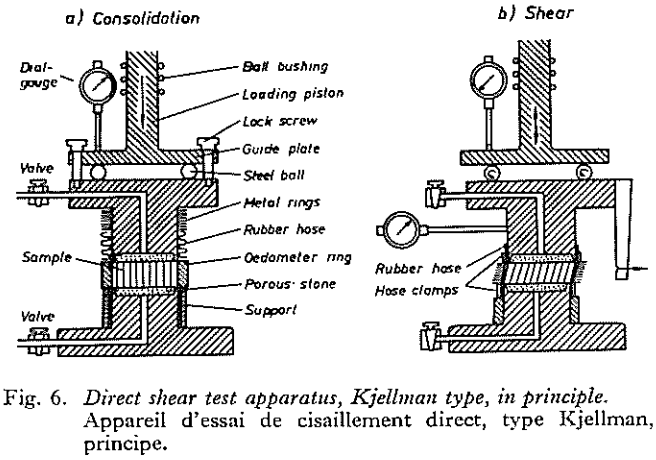


Fig. 6. *Direct shear test apparatus, Kjellman type, in principle.*
Appareil d'essai de cisaillement direct, type Kjellman, principe.

Discussion and General Remarks

The strength and deformation properties as determined from the different tests are shown in Table 1. For comparison, values reported by Gibson (1953) and by Bjerrum and Wu (1960) are also given in the table. The clay from Lilla Edet is of particular interest since it is a quick clay sedimented in the same environment as the Mölndal clay. The Lilla Edet clay, however, has a considerably lower sensitivity and water content than the Mölndal clay.

As can be seen from the table, there are some differences between the different tests and soil types. For example, the values of ϕ_{cu} and $\varkappa$ obtained from the direct shear tests are considerably higher than the triaxial values. Particularly the $\varkappa$-value determined from the direct shear tests is remarkably high. The angle ϕ'_{cu} agrees well with $(\phi'_d)_{corr}$ but on the other hand $(\phi'_{cu})_{corr}$ is somewhat higher than ϕ'_d. The angle ϕ_r determined from direct shear tests is somewhat lower than that from the triaxial tests. The ϕ_r-value obtained from the unconfined compression tests is surprisingly high, which may be caused by anisotropic factors. If this is the case, the angle ϕ_r cannot be determined from the inclination of the failure plane.

The differences in test results between the triaxial and the direct shear tests are probably caused by differences in stress conditions. In the direct shear tests, where the clay specimen is compressed in one direction only, an orientation of clay particles probably takes place. This orientation and the resulting reduced influence of micro-dilatancy, which is a function of the degree of compression, will reduce the shearing resistance parallel to the potential failure plane. For the Mölndal clay, ϕ_r was evaluated at a water content of about 50% while the natural water content was about 120%. The triaxial tests, in which the consolidation was isotropic and the failure plane inclined, consequently gave

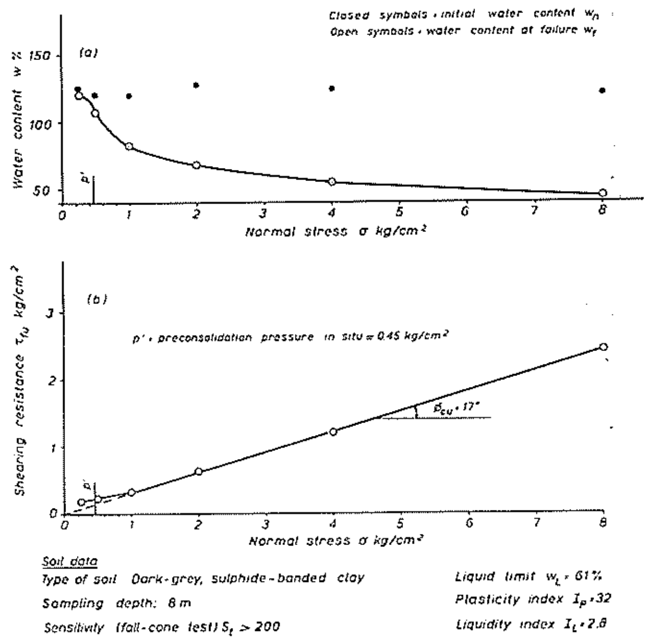
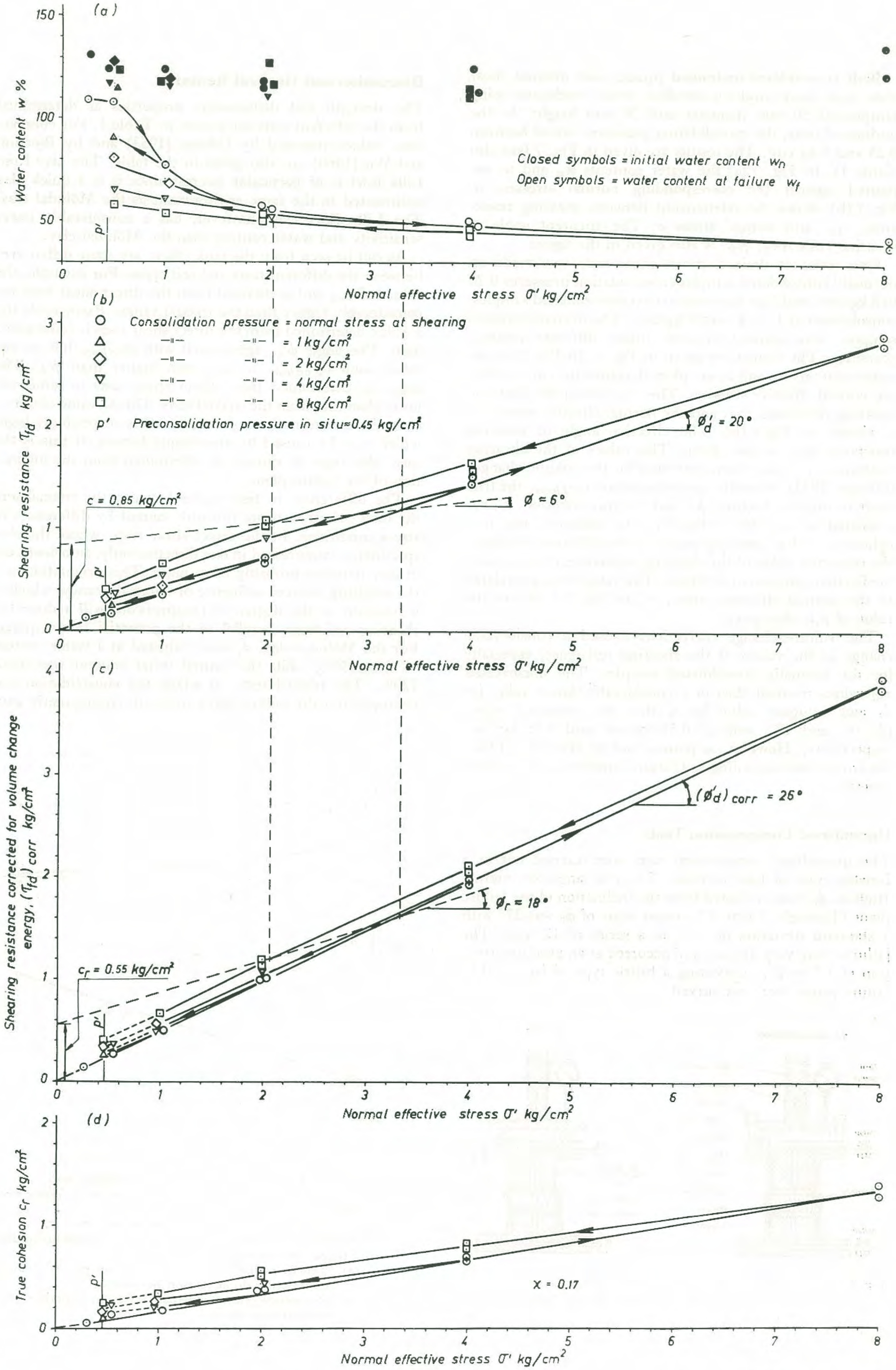


Fig. 7. *Results of consolidated-undrained direct shear tests.*
(a) Initial water content and water content at failure versus normal stress σ.
(b) Shearing resistance τ_{fu} versus normal stress σ.
Résultats en connexion avec les essais de cisaillement direct à teneur en eau constante.
(a) Teneur en eau initiale et teneur en eau à la rupture en fonction de la tension normale σ.
(b) Résistance au cisaillement τ_{fu} en fonction de la tension normale σ.



Soil data: See Fig. 7.

TABLE 1. *Strength parameters of the Mölndal clay and of some other clays, and values of strain at failure of the Mölndal clay.*

Paramètres de résistance de l'argile de Mölndal et de quelques autres argiles, et de valeurs de la déformation à la rupture de l'argile de Mölndal.

Clay and test types	I_p	Strength parameters							Strain at failure	
		ϕ_{cu}	ϕ'_{cu}	$(\phi'_{cu})_{corr}$[1]	ϕ'_d	$(\phi'_d)_{corr}$[2]	ϕ_r	$\varkappa$	ε and γ at $\sigma \leqq p'$	ε and γ at $\sigma > p'$
Mölndal clay	32									
Triaxial tests		12°	27°	23°			21°	0.11	$\varepsilon \approx 3\%$	$\varepsilon = 6$–8%
Direct shear tests, cons. undr.		17°							$\gamma \approx 0.1$ rad.	$\gamma \approx 0.15$ rad.
Direct shear tests, drained					20°	26°	18°	0.17	$\gamma \approx 0.15$ rad.	$\gamma = 0.2$–0.3 rad.
Unconf. compr. tests							27°		$\varepsilon = 1.5$–2%	
Lilla Edet clay (Bjerrum and Wu, 1960)	30–35									
Triaxial tests		14° [3]					20°	0.08		
Unconf. compr. tests							19.5°			
Different types of clays (Gibson, 1953)	32									
Direct shear tests (remoulded clays)		16° [3]			24° [3]		18° [3]			
Unconf. compr. tests							18° [3]			

1) Corrected with respect to over-consolidation.
2) Corrected with respect to volume change.
3) Taken from diagrams.

higher ϕ_r-values than the direct shear tests. However, different stress conditions during shearing may have been of importance.

At 50% water content, the sensitivity was about 15. This decrease of sensitivity indicates a marked change of the microstructure. However, the clay still has a relatively open particle arrangement and a high contractibility.

The water contents at failure, w_f, for normally consolidated samples have been correlated with the corresponding consolidation pressures, σ_c (Fig. 9 a). It can be seen (the two upper curves) that the decrease in water content during consolidation was larger in the direct shear tests than in the triaxial tests. This may be caused by high local shear stresses in the bond regions of the particle network under vertical consolidation. The difference in ϕ_{cu} between the two test types is probably caused by the different stress conditions during consolidation. The figure (lower curve) shows also that there is a large reduction in water content during the shearing process in the drained direct shear tests. This illustrates the high contractibility of the Mölndal clay and indicates a high value of the structural parameter (Skempton and Bisbop, 1954). The instability of the clay structure with deformation is indicated also by the high value of the pore pressure coefficient A_f as shown in Fig. 5 e. Also the difference in $\varkappa$ can be explained by the larger compression of the clay in the direct shear tests than in the triaxial tests.

In Fig. 9 (b) the water content at failure, w_f, has been related for normally consolidated samples to the values of the true cohesion, c_r, obtained from triaxial tests and direct shear tests. The diagram for the triaxial tests shows that there is a pronounced decrease in the true cohesion, c_r, with decreasing water content when the consolidation pressure exceeds the preconsolidation pressure. At a further increased pressure, the true cohesion increases with decreasing water content. For Lilla Edet clay, Bjerrum and Wu have obtained a similar w_f-log c_r relationship. The decrease in true cohesion when the pressure exceeds the preconsolidation pressure, depends on the break-down of cementation bonds, which is in agreement with Bjerrum and Wu's concept. It can be mentioned that the consolidation curves from tests on the Mölndal clay show a pronounced influence of broken cementation bonds when the pressure exceeds the preconsolidation pressure (Leonards and Ramiah, 1960). The w_f-log c_r relationship is uncertain for the direct-shear tests when the preconsolidation pressure is exceeded because of the lack of test values.

Fig. 9 (b) shows also that the values obtained for the true cohesion were higher in the direct sbear tests than in the triaxial tests. This may be related to differences in the clay

Fig. 8. *Results in connection with drained direct shear tests.*
(a) Initial water content and water content at failure versus normal effective stress σ'.
(b) Shearing resistance τ_{fd} versus normal effective stress σ'.
(c) Shearing resistance values corrected for volume change $(\tau_{fd})_{corr}$. versus normal effective stress σ'.
(d) True cohesion c_r versus normal effective stress σ'.

Résultats en connexion avec les essais de cisaillement direct à teneur en eau non-constante.
(a) Teneur en eau initiale et teneur en eau à la rupture en fonction de la contrainte normale effective σ'.
(b) Résistance au cisaillement τ_{fd} en fonction de la contrainte normale effective σ'.
(c) Valeurs de résistance au cisaillement $(\tau_{fd})_{corr}$ corrigées pour changement de volume en fonction de la contrainte normale effective σ'.
(d) Cohésion vraie c_r en fonction de la contrainte normale effective σ'.

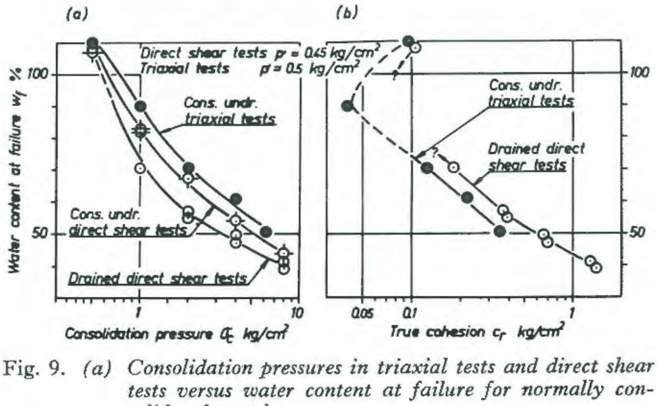


Fig. 9. *(a) Consolidation pressures in triaxial tests and direct shear tests versus water content at failure for normally consolidated samples.*
(b) True cohesion from triaxial tests and direct shear tests versus water content at failure for normally consolidated samples.
(a) Pression de consolidation lors des essais triaxiaux et des essais de cisaillement direct en fonction de la teneur en eau à la rupture pour des échantillons normalement consolidés.
(b) Cohésion vraie lors des essais triaxiaux et des essais de cisaillement direct en fonction de la teneur en eau à la rupture pour des échantillons normalement consolidés.

structure due to the different conditions during the consolidation at the two test types. Thus, in the direct shear test, where the sample is compressed in vertical direction only, a greater reduction of the mean particle distance probably takes place in this direction than in the horizontal direction.

No definite conclusions can be drawn at this stage with regard to the relationship between the microstructure and the physical properties of the Mölndal clay. However, there exists a correlation between the very porous particle network and the high water content. This property is also connected with the difference between the natural water content and the liquid limit, which indicates that a large amount of 'excessive' water has to be removed before the shear strength corresponding to the liquid limit is obtained.

With respect to the extremely open particle arrangement, the true angle of internal friction is high. This can be explained by non-dilatant friction due to van der Waals' forces in the increased areas of particle interaction at increased effective normal stress (Rosenqvist, 1959), and by the high content of rock-forming minerals. In the latter case a certain degree of continuity of these minerals has to be presumed. No such feature can, however, be seen from the structural micrographs.

Additional studies concerning the strength and microstructural properties of the Mölndal clay are in progress at the Swedish Geotechnical Institute.

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Ratio c/p' in Relation to Liquid Limit and Plasticity Index, with Special Reference to Swedish Clays

Le rapport c/p' en corrélation avec la limite de liquidité et l'indice de plasticité en se référant spécialement aux argiles suédoises

by *R. Karlsson* and *L. Viberg*
Swedish Geotechnical Institute

Summary

The ratio c/p' has been determined for different types of Swedish Quaternary, sedimentary clays, normally consolidated or nearly normally consolidated. The undrained shear strength, c, has been determined by the field vane. The preconsolidation pressure, p', has been taken as the effective overburden pressure and, in addition, has been determined by oedometer tests. The c/p' ratio has been related to the plasticity index, I_P, and comparisons have been made with the relationship according to Skempton. c/p' has also been related to the liquid limit, w_L, and comparisons have been made with the relationship according to Hansbo. The scattering of the test results and the deviation from Skempton's and Hansbo's curves are great, especially for quick clays. The influencing factors and difficulties in evaluating the two parameters c and p' are discussed.

Résumé

Le rapport c/p' a été déterminé pour différents types d'argiles quaternaires sédimentaires suédoises, normalement consolidées ou presque normalement consolidées. La résistance au cisaillement à teneur en eau constante, c, a été déterminée par essai sur scissomètre de chantier. La pression de préconsolidation, p', a été prise comme pression effective de surcharge et a été, par surcroît, déterminée par des essais oedométriques. Le rapport c/p' a été rattaché à l'indice de plasticité, I_P, et des comparaisons ont été faites avec la relation de Skempton. c/p' a aussi été rattaché à la limite de liquidité, w_L, et des comparaisons ont été faites avec la relation de Hansbo. La dispersion des résultats des essais et la déviation partant des courbes de Skempton et Hansbo sont considérables, spécialement pour les argiles fluides. Les facteurs influants et les difficultés d'évaluer les deux paramètres c et p' font l'objet de discussions.

Introduction

In practical soil mechanics the preconsolidation pressure of clays is often estimated by means of empirical relationships. In Sweden the relationship $c/p' = 0.45 \cdot w_L$ according to Hansbo (1957) is usually used. Sometimes the relationship $c/p' = 0.11 + 0.0037 \cdot I_P$ according to Skempton (1954) is also used. Since, however, in practice these two methods often give uncertain values, especially for quick clays, it has been considered necessary to make a detailed investigation of the methods for different types of Swedish clays. The National Swedish Council for Building Research has sponsored parts of the investigations. The basic material for the work comes from investigations carried out in connection with the consulting activities of the Swedish Geotechnical Institute.

Review of Literature

For purposes of comparison a review of the literature on the subject has been made, mostly dealing with investigations relating to Quaternary sedimentary clays. Because of the restricted distribution of these types of clays, however, there are only a limited number of papers to be found in the international literature on this subject.

Skempton (1954) has based his relationship between c/p' and I_P on various types of clays, but mainly English clays.

Bjerrum (1954) has made comparisons between c/p' and I_P for Norwegian marine clays; quick clays are also included. At $I_P < 15\%$ the values are in the main smaller than those represented by Skempton's curve. In view of this Bjerrum has made an extension of Skempton's curve within the lower region of I_P.

The relationship between c/p' and w_L according to Hansbo (1957) is based on investigations of different types of Swedish sedimentary clays and also on the above-mentioned investigations by Bjerrum. In the case of the Swedish clays, highly sensitive clays are not included.

Kenney (1959) has compiled values by Wu (1958) from investigations on different types of North-American clays, supposed to be of Quaternary origin, and made comparisons between c/p' and I_P. The scattering of the values is great and Kenney is of the opinion that the test methods generally used and the structure of the clays and their geological history are the main factors of influence.

Osterman (1960) has compared values of c/p' ratio with I_P for different types of Swedish clays, besides different types of clays from other countries, for example Mexican clays. The results show great scattering especially for the Mexican clays. Almost all values of c/p' referred to Swedish clays are greater than those represented by Skempton's curve. The values of c/p' for Mexican clays decrease mostly with increasing values of I_P.

Description of Clays Investigated

The investigations have been limited to Quaternary (glacial and post-glacial), sedimentary clays. In most places in Sweden these clays are normally or only slightly overconsolidated, with the exception of the upper part of the clay (dry crust).

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The conditions during the sedimentation of the clays have differed in different regions of Sweden. In the south-western part of the country the conditions were most uniform with constant salt (marine) milieu and in this region the clays can often be fairly homogeneous. However, through land elevation above the sea level, parts of the clays have been exposed to leaching. The sensitivity thus varies greatly and layers of quick clay have often been formed.

In the eastern part of the country the sea water during sedimentation has ranged between fresh and brackish. Due to the seasonal variation of supply of material in connection with the lower salt content in the sea water, the glacial sediments are varved in tbis region. Also in this part of the country the clays are frequently highly sensitive.

Both the glacial and the postglacial clays are in terms of constituent minerals rather similar, the dominating clay mineral being illite. The content of organic matter is usually low in the glacial clays, yet the marine clays in the south-western part of the country sometimes have a rather high content of organic sulphides. The postglacial clays usually have a higher content of organic matter ('gyttja' and organic sulphides). Especially in north-eastern Sweden the postglacial clays have a high content of organic sulphides and are called sulphide clays.

Test Methods Used

The undrained shear strength, c, is determined by field vane tests, represented as c_{vane}. The results obtained have been compared with those from unconfined compression tests and Swedish fall-cone tests (cf. Hansbo, 1957) on undisturbed samples.

The preconsolidation pressure, p', has been taken as the effective overburden pressure, p'_{overb}. In addition, in most cases it has been determined by oedometer tests, p'_{oed}, on undisturbed samples.

The liquid limit has, in certain cases, been determined according to Casagrande's method and is given the symbol w_L. In addition, the plastic limit, w_P, and the plasticity index, I_P, are taken. In most cases, however, the liquid limit is determined by the fall-cone method and is called the fineness number, symbol w_F. The fall-cone method is usually used in Sweden since it is simpler to carry out and more objective than percussion methods. As shown by Karlsson (1961), w_F and w_L agree very well for Swedish soils, when the values are smaller than 100%. For higher values, w_F is somewhat smaller than w_L.

When calculating the c/p' ratio, uniform zones in the clay profiles are considered, i.e. zones with nearly constant liquid limit and – with respect to increasing depth – with almost linearly increasing shear strength. The c/p' ratio is taken as $\Delta c_{vane}/\Delta p'_{overb}$ for the average of the zone and, in addition, as c_{vane}/p'_{overb} for a certain level or – in some cases with large zones – for several levels. At the same levels, when oedometer tests have been carried out, c/p' has been taken as c_{vane}/p'_{oed}. In a few cases it has not been possible to evaluate $\Delta c_{vane}/\Delta p'_{overb}$ because of variations in the soil in the whole borehole, but in these cases c_{vane}/p'_{overb} has always been compared with c_{vane}/p'_{oed}.

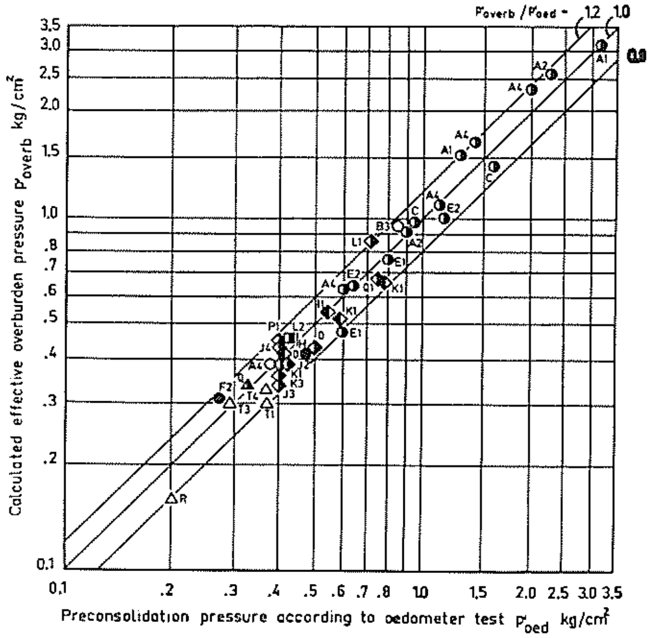


Fig. 1. *Calculated effective overburden pressure p' plotted against preconsolidation pressure p'_{oed} determined by oedometer tests.*
Pression de surcharge effective p' caculée en fonction de la pression de préconsolidation p'_{oed} déterminée par essais oedométriques.

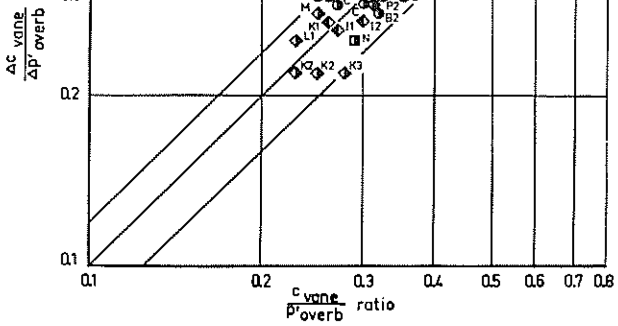
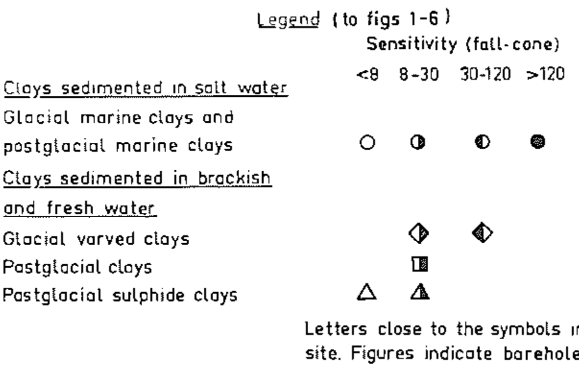


Fig. 2. *Ratio between increment of shear strength Δc_{vane} and increment of effective overburden pressure $\Delta p'_{overb}$ plotted against ratio between shear strength c_{vane} and effective overburden pressure p'_{overb}.*
Rapport entre l'accroissement de la résistance au cisaillement Δc_{vane} et l'accroissement de la pression de surcharge effective $\Delta p'_{overb}$ en fonction de rapport entre la résistance au cisaillement c_{vane} et la pression de surcharge effective p'_{overb}.



Results Obtained

The results are compiled in Figs 1 to 6. In Fig. 1, p'_{overb} has been compared with p'_{oed}, and in Fig. 2, $\Delta c_{vane}/\Delta p'_{overb}$ with c_{vane}/p'_{overb}. As can be observed, the quotient between p'_{overb} and p'_{oed} as well as the quotient between $\Delta c_{vane}/\Delta p'_{overb}$ and c_{vane}/p'_{overb} range from about 0.8 to 1.2. This may be an indication that the clays have been nearly normally consolidated.

In Fig. 3a, the ratio c_{vane}/p'_{overb} is plotted against I_P, and for comparison Skempton's curve is drawn. The divergences of the test values from Skempton's curve have been calculated and are shown diagrammatically in Fig. 3b. In Figs 4a, 5a and 6a respectively, the ratios $\Delta c_{vane}/\Delta p'_{overb}$, c_{vane}/p'_{overb} and c_{vane}/p'_{oed} are plotted against the liquid limit, and for comparison Hansbo's curve is drawn. The divergences of the test values from Hansbo's curve have been calculated and are shown in Figs 4b, 5b and 6b. As appears from the figures the scattering is great, especially for clays with high sensitivity. The values obtained for clays with sensitivity > 30 are situated above Hansbo's as well as Skempton's curves. For quick clays the deviation is in some cases greater than 100%. In the zone of liquid limits between 40 and 100% a curve from the values plotted in the figures should be drawn horizontally rather than inclined as in Hansbo's curve, for example.

Discussion and General Remarks

The value of the c/p' ratio will be dependent, besides the shear strength characteristics of the soil, on the methods used to determine c and p'. The undrained shear strength c is usually determined by field vane tests, and as a rule the height of the vane blades is about double the diameter. This vane mainly gives the vertical shear strength of the soil, whereas the preconsolidation pressure, p', is taken on a horizontal plane. In those cases where the shear strength values differ in the horizontal and vertical directions and where the ratio between the shear strengths varies with the type of soil and with the depth, the value of c/p' (as regards the methods usually used) will be inadequate.

Aas (1965) has made certain studies of the anisotropic conditions in the ground for some Norwegian marine clays, normally consolidated (except one case). The results for the ratio between the shear strength in vertical and horizontal directions vary considerably at different localities. At some localities the layers are almost isotropic with respect to the shear strengths (cf. Bjerrum, Aas and Eide, 1966).

The shear strength characteristics depend on the constituents and the microstructure of the clays. Since the consistency limits are determined on remoulded material, they do not reflect the original structure of the soil. Neither is the influence of variations in the constituents always completely reflected in the liquid limit or the plasticity index. As can be seen, for example, from Casagrande's plasticity chart, soils of different composition (i.e. in terms of type of minerals or type and amount of organic matter) can have the same liquid limit or plasticity index. They can, however, have different shear strength characteristics. This is indicated, for example, by Osterman (1960) in a compilation of values for different types of soils. However, the minerals of the Swedish clays, as mentioned above, are of the same type.

In highly sensitive clays in particular, the structure in the undisturbed and remoulded state is quite different. Furthermore, the shear strength characteristics for the undisturbed clays are to a large degree due to structural factors: both the arrangement of the particles and the bond between them. According to Osterman (1964) rigid bonds (so-called cementation bonds) between the particles are of great importance. The formation of highly sensitive and quick clays is a com-

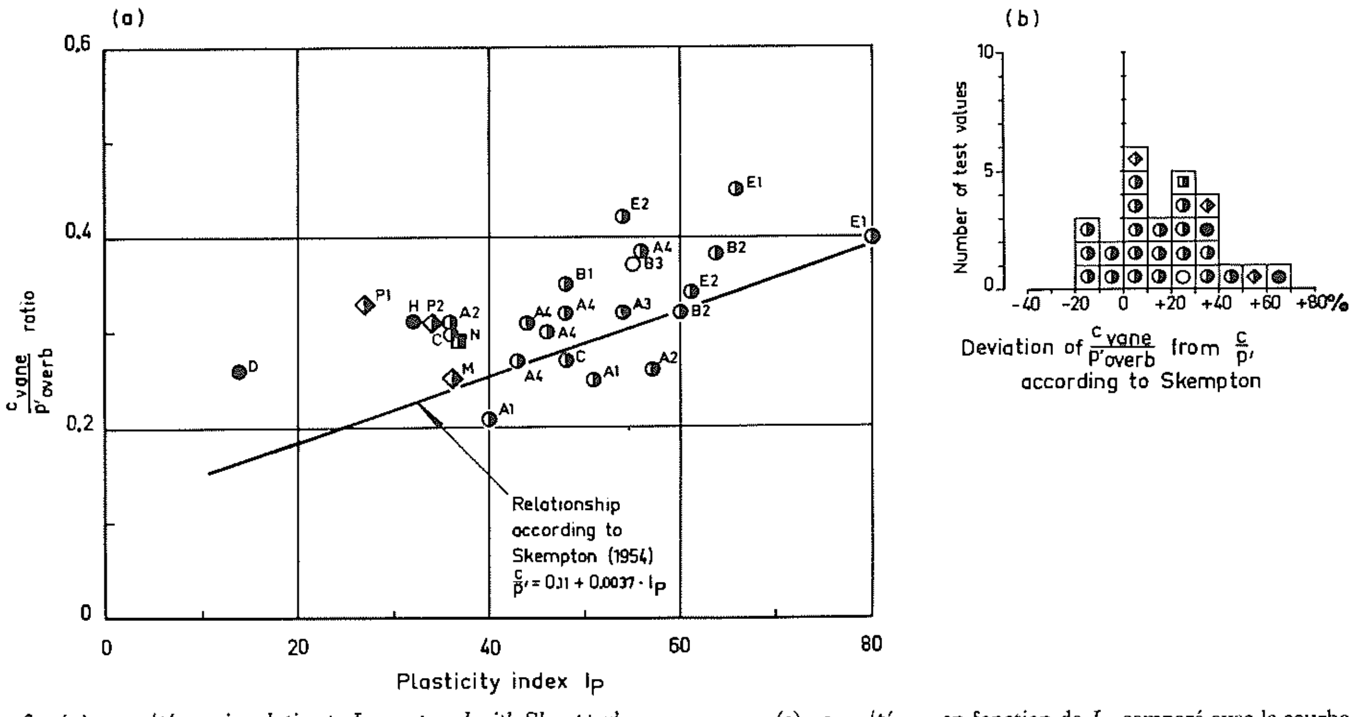


Fig. 3. *(a) c_{vane}/p'_{overb} in relation to I_P compared with Skempton's curve.*
(b) Diagram indicating deviation of obtained values in relation to Skempton's curve.

(a) c_{vane}/p'_{overb} en fonction de I_P comparé avec la courbe de Skempton.
(b) Diagramme indiquant la déviation des valeurs obtenues par rapport à la courbe de Skempton.

plicated colloid-chemical process involving several factors of different nature (cf. Söderblom, 1967). Thus, leaching, weathering and addition of different dispersing agents may cause variations in the shear strength properties, especially with respect to the bonds between the particles. Also of importance is the original composition of the clays and the stress history.

According to Skempton (1954), in salt-sedimented clays the leaching process causes a certain decrease of the undisturbed shear strength while simultaneously this process leads to a reduction of the remoulded shear strength and thus also of the liquid limit. In the present investigations the values of c/p' ratio obtained for highly sensitive Swedisb clays are greater than those represented by the relationships according

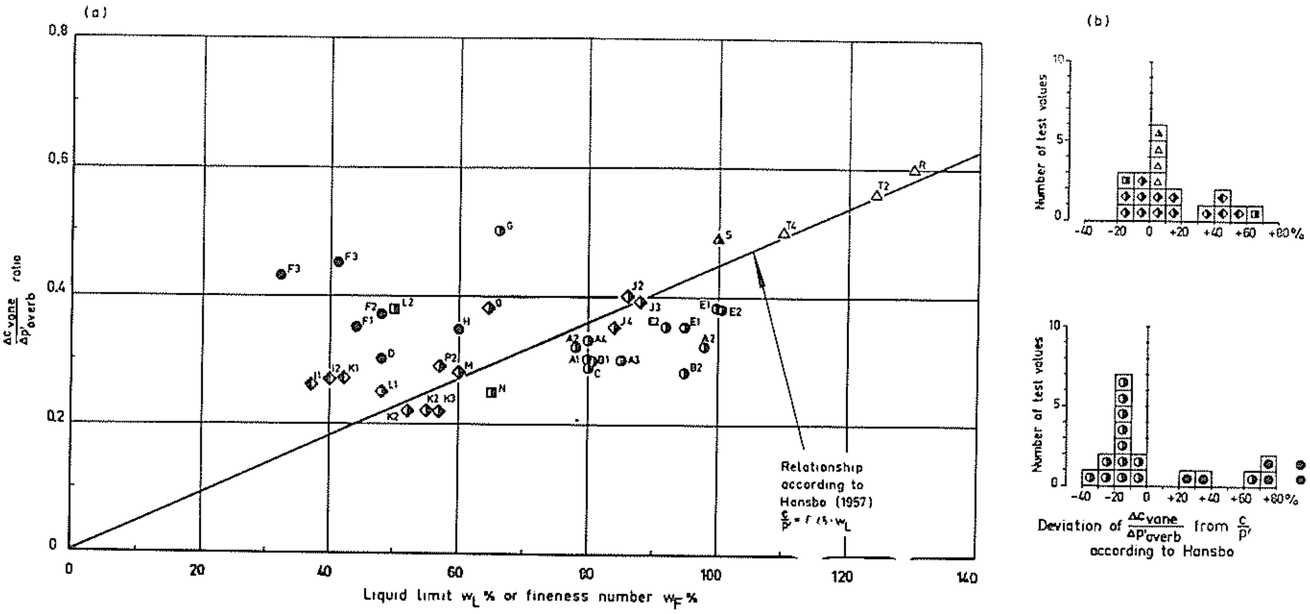


Fig. 4. *(a) $\Delta c_{vane}/\Delta p'_{overb}$ in relation to w_L (or w_F) compared with Hansbo's curve.*
(b) Deviation of obtained values in relation to Hansbo's curve. Upper diagram: Fresh and brackish-water sedimented clays. Lower diagram: Salt-water sedimented clays.

(a) $\Delta c_{vane}/\Delta p'_{overb}$ en fonction de w_L (ou w_F) comparé avec la courbe de Hansbo.
(b) Déviation des valeurs obtenues par rapport à la courbe de Hansbo.
Diagramme supérieur: Argiles sédimentées en eau douce et en eau saumâtre.
Diagramme inférieur: Argiles sédimentées en eau salée.

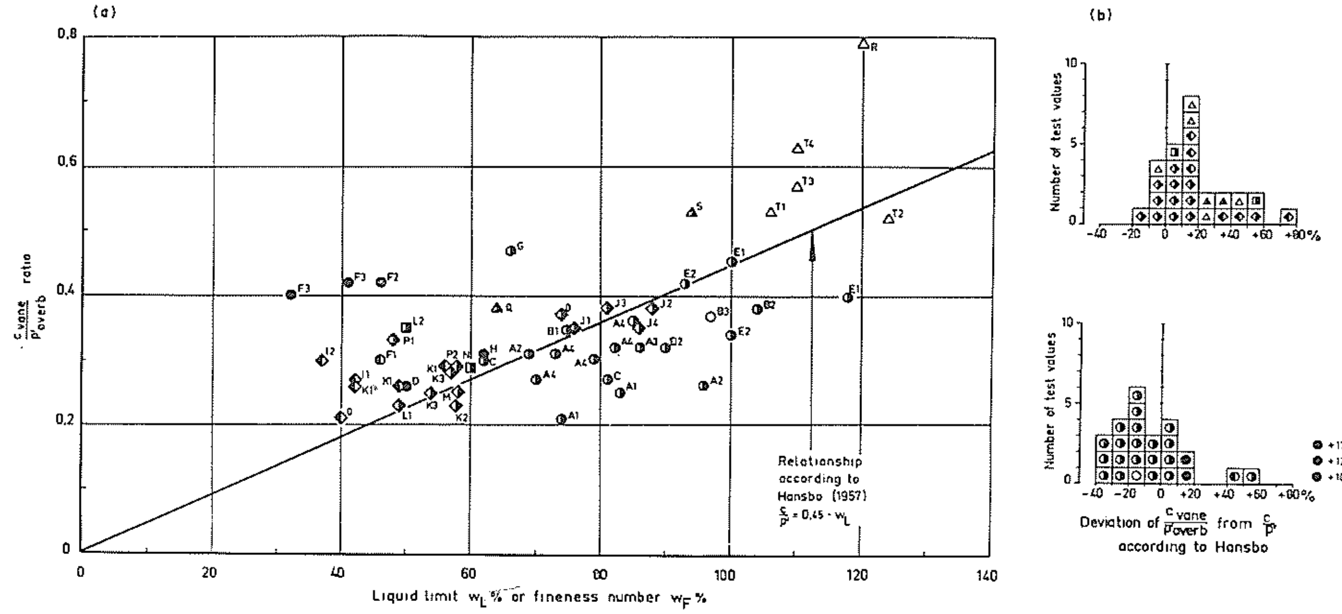


Fig. 5. *(a) c_{vane}/p'_{overb} in relation to w_L (or w_F) compared with Hansbo's curve.*
(b) Deviation of obtained values in relation to Hansbo's curve. Upper diagram: Fresh and brackish-water sedimented clays. Lower diagram: Salt-water sedimented clays.

(a) c_{vane}/p'_{overb} en fonction de w_L (ou w_F) comparé avec la courbe de Hansbo.
(b) Déviation des valeurs obtenues par rapport à la courbe de Hansbo.
Diagramme supérieur: Argiles sédimentées en eau douce et en eau saumâtre.
Diagramme inférieur: Argiles sédimentées en eau salée.

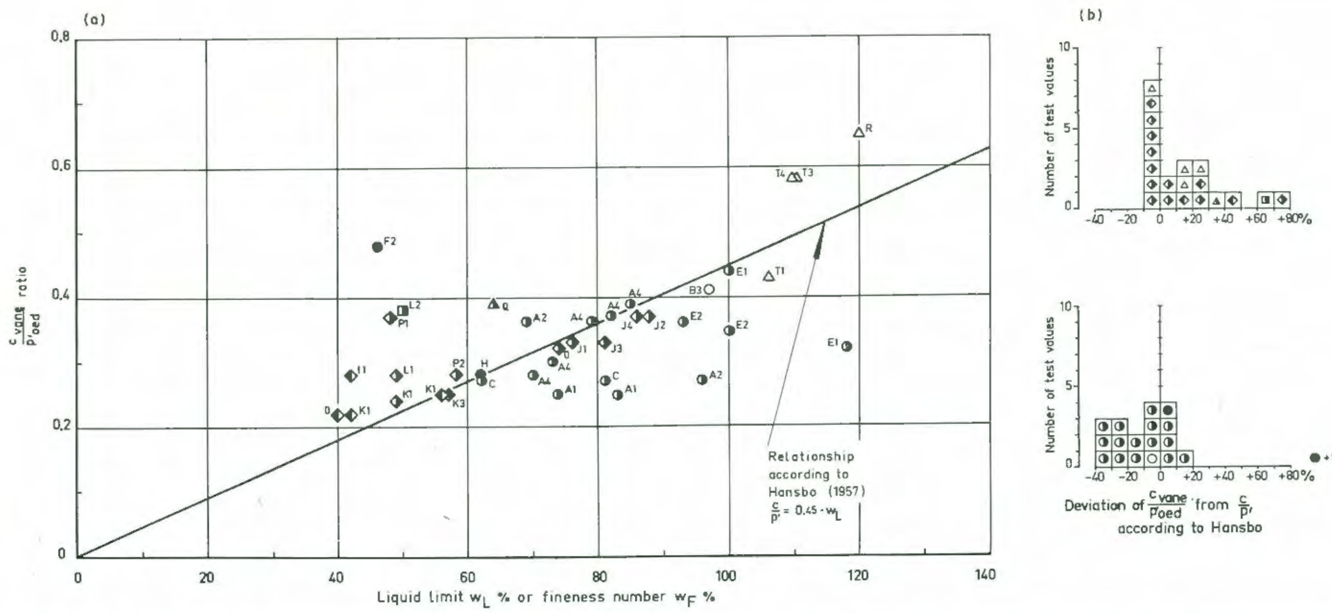


Fig. 6. *(a) c_{vane}/p'_{oed} in relation to w_L (or w_F) compared with Hansbo's curve.*
(b) Deviation of obtained values in relation to Hansbo's curve. Upper diagram: Fresh and brackish-water sedimented clays. Lower diagram: Salt-water sedimented clays.

(a) c_{vane}/p'_{oed} en fonction de w_L (ou w_F) comparé avec la courbe de Hansbo.
(b) Déviation des valeurs obtenues par rapport à la courbe de Hansbo. Diagramme supérieur: Argiles sédimentées en eau douce et en eau saumâtre. Diagramme inférieur: Argiles sédimentées en eau salée.

to Skempton and Hansbo. This indicates a smaller decrease of the undisturbed shear strength by leaching than shown in Skempton's and Hansbo's curves (with respect to the decrease of the liquid limit). On the other hand, Bjerrum's (1954) investigations in relation to Skempton's curve in the main shows the opposite picture for Norwegian clays. These clays are, however, generally coarser than the Swedish clays and have considerably smaller values of I_P.

As a conclusion it may be said that generally there are so many factors involved in the shear strength complex depending, for example, on local variations and geological history, that it is inadequate to speak of a unique relationship between c/p' and I_P or w_L. For a uniform type of clay deposit, however, such relationships may be used, though anisotropic conditions must still be considered.

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