THE GEOTECHNICAL CHARACTERISTICS OF MORAINE CLAYS RELATED TO THEIR STRUCTURE

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Abstract. The geotechnical parameters of strength and elasticity have been investigated in the laboratory and in the field. Tests were made on three different moraine clays in southern Sweden. The relationship between water content, fissures, fissure systems, content of coarse grains and the geotechnical parameters was studied. It is shown that the parameters are volume-dependent at low water content. The failure mechanism is found to be quite different at low and at high water contents. A theory is presented to explain why the absolute values of strength and elasticity decrease with increasing water content.

INTRODUCTION

Moraine clays usually provide good support for buildings. Since they cover only a small part of Sweden and since they are good foundation soils, the measurement of the settlement and strength parameters has not been properly studied. Nowadays, when building heights are increasing and when builders are employing construction techniques involving the use of pre-manufactured elements, in which only small differential settlements can be tolerated, the need to know more about moraine clays has greatly increased. My research in the Geotechnical Division at Chalmers University of Technology, Gothenburg, has involved both laboratory and field investigations of moraine clays in southwestern Skåne in Sweden.

The moraine clay has a glacial origin, having been deposited directly by the glacial ice, unmodified since that time, except for normal weathering processes. The moraine clays of southern Sweden consist of unsorted mineral particles formed by the crushing of sedimentary rocks, shales and slates. Sometimes they include soft sediments, which were picked up by the ice drift and mixed into the clay. (A moraine clay is here classified as such, when the clay content is more than 15 per cent by weight.)

Because of the method of deposition, the structure is quite arbitrary and heterogeneous. Of special interest from the geotechnical point of view is the mineral composition, the fissures, the fissure systems, the content of coarse-grained particles and the water content. These factors influence the geotechnical characteristics in such a way that the properties of a small soil sample do not represent a large volume of the deposit. One engineering problem is that only a small volume can be tested in the laboratory, in predicting the settlement and bearing capacity of the soil for a future building. In this article the soil-volume dependence will be related to the above-mentioned factors for the strength and settlement characteristics. Comparisons will be made between undisturbed and compacted specimens. A correlation will be made between laboratory tests and full-scale field tests.

INVESTIGATED MORAINE CLAYS

In this article the results of tests of three different moraine clays in southern Sweden will be presented. Two of the locations are in Lund, one in the southeastern part (herein called Sparta) and one in the northwestern part (called Rehab). The third location (called Tygelsjö) is just south of Tygelsjö, 15 km south of Malmö. These moraine clays are mostly formed of crushed clay-shales which originated from the silurian area in the middle of Skåne and from crushed carbonate rocks. The Lund moraine clays differ in many ways from the Tygelsjö moraine clay (see Table 1), where the usual parameters are given for the three clays. In Fig. 1 the grain-size distributions are presented.

DETERMINATION AND USE OF GEOTECHNICAL PARAMETERS

For those who may not be geotechnically oriented, the geotechnical parameters are briefly reviewed. Moraine clays are over-consolidated, because they have been

Table 1. Properties of the investigated moraine clays.

Property	Dimension	Sparta, Lund	Rehab, Lund	Tygelsjö
Depth	m	1.7	1.9	1.5
Density	t/m^3	2.09	2.07	2.04
Specific gravity	t/m^3	2.67	2.66	2.66
Water content	%	17	16	14
Plasticity index	%	17	11	12
Liquidity index	1	0	0	0
Skempton's activity	1	0.6	0.4	0.6
Clay content	%	29	31	17
Optimum water content	%	16.5[a]	11.5[b]	10.5[a]
Undrained shear strength[c]	kN/m^2	300	470	180
Drained shear parameters[d] c	kN/m^2	40	35	15
ϕ	Degrees	26.5	27	30

[a] Standard proctor compaction. [b] Modified proctor compaction.
[c] Fall-cone test. [d] Drained tri-axial tests.

consolidated earlier under the pressure of the ice sheet. For over-consolidated clays the settlement calculations are fairly easy, because one need only take into account the elastic compression, whereas for normally consolidated clays one also must calculate the long-term settlements caused by pore-water expulsion. Using the theory of elasticity, one can, for example, predict the settlements of a soil loaded through a stiff plate by means of the equation:

$$s = \frac{q\pi D}{4} \cdot \frac{1-\upsilon^2}{E}$$

where s is the settlement, q is the pressure under the plate, D is the plate diameter, υ is Poisson's ratio and E is Young's modulus.

For the calculation, one thus has to determine Young's modulus and Poisson's ratio. Without large error, Poisson's ratio can be assumed to be 0.3. Young's modulus can be determined in the field by pressiometer tests, and in the laboratory in specimens loaded under uni-axial or tri-axial compression (Fig. 2) or indirectly by oedometer tests. When the tri-axial compression test is performed to evaluate Young's modulus, the lateral pressure is given a value identical with the lateral stress in the field.

To determine the safety against failure, one calculates the short- and long-term bearing capacity of the foundation soil. The safety factor is then the ratio between the calculated capacity and the actual load. When calculating bearing capacity, a failure mechanism is assumed in which the shear strength of the soil is mobilized along slip surfaces. One example of the failure mechanism for a plate loading is given in Fig. 3. Under the condition of equilibrium of the forces, the bearing capacity (q_f) can be deduced.

In the short-term case the undrained shear strength (τ_{fu})has to be determined. This can be done by making fall-cone tests, field-vane tests and uni-axial or undrai-

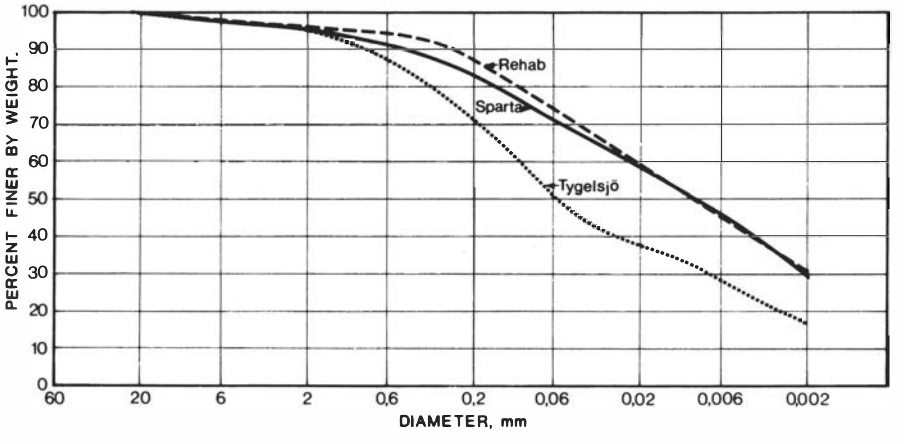

Fig. 1. Grain-size distribution for the investigated moraine clays.

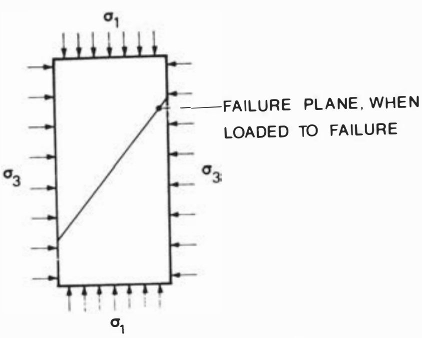

Fig. 2. Specimen under tri-axial compression.

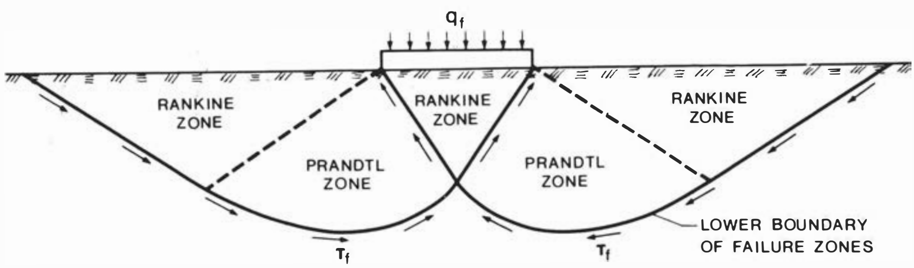

Fig. 3. Soil-failure mechanism caused by plate-loading.

ned tri-axial compression tests. The tests are run so fast that excess pore-water pressure does not dissipate. In the long-term case the drained shear parameters cohesion (c) and angle of friction (ϕ) have to be determined by drained tri-axial compression tests or drained direct-shear tests, run so slowly that no excess pore-water pressures develop. The shear strength in the drained case is determined by Mohr-Coulomb's equation, $\tau_{fd} = c + \sigma \tan \phi$, where σ is the stress normal to the slip surface.

FACTORS GOVERNING THE GEOTECHNICAL CHARACTERISTICS

As the moraine clays were kneaded, deposited and compacted by glacial ice and were not subject to any post-depositional changes, it is possible to study the soil characteristics by using re-compacted samples as successfully as by using undisturbed samples. This offers a cheaper laboratory procedure than taking undisturbed samples for each test. However, the shear strength of natural and laboratory re-compacted samples may be different, especially with small specimen volumes, due to delayed re-ordering of the water phase and of minute mobile particles. These processes may produce time-dependent strength increases in the natural sediments (Pusch and Jacobsson, 1971). Of course, the water content and dry density must be the same in the undisturbed and in the compacted samples.

Volume dependence

Moraine clays have a structure with a more or less arbitrary fissure system, causing planes of weakness. The geotechnical characteristics consequently change with the sample volume. This volume dependence has earlier been pointed out for rock material. For fissured stiff clays, Lo (1970) has stated the strength-size relationship shown in Fig. 4.

The strength (Fig. 4), measured by any type of test on any size of sample, must be bounded at the upper limit by the strength of the intact clay matrix and at the lower limit by the fissure strength. The "intact strength" must be measured on samples free from inhomogeneities. If the clay is highly fissured, it may be difficult to measure any intact strength. a_0, in Fig. 4, is the smallest distance between two fissures. When a sample is larger than this distance, it will contain fissures and the strength will decrease. As the sample size increases, so does the number of fissures and the ordering of their orientation. The strength will thus be decreased in such a way that at large sample sizes the strength asymptotically reaches an operational strength. The probability of the fissures being oriented in such a way that they will form a continuous plane of weakness in one direction is very small, and thus the operational strength will be greater than the fissure strength. From the engineering point of view the problem is to predict the clay field strength. According

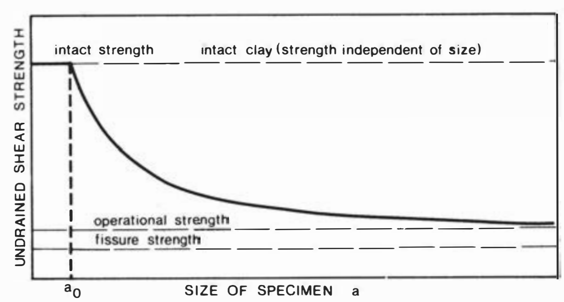

Fig. 4. Diagram of the strength-size relation (Lo, 1970).

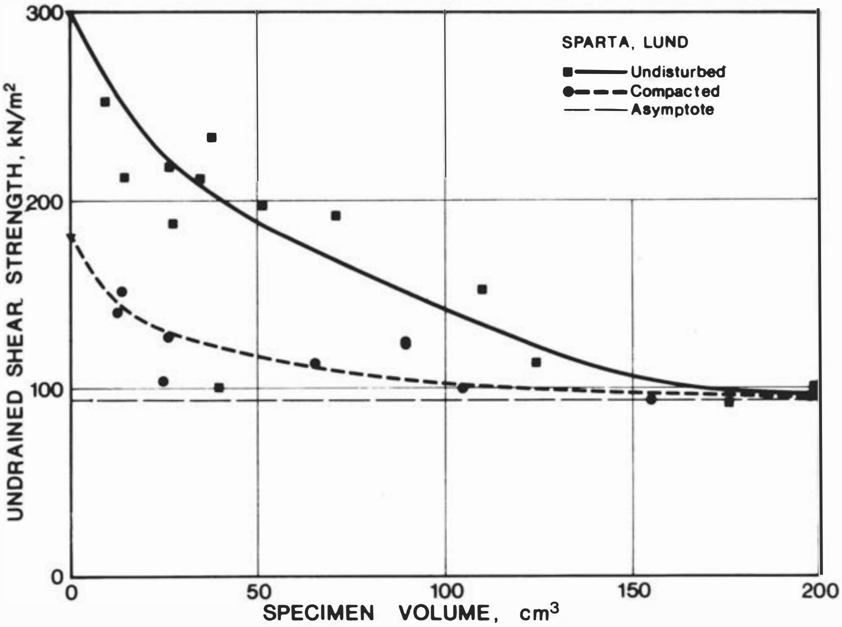

Fig. 5. Undrained shear strength versus specimen volume. Moraine clay from Sparta, Lund.

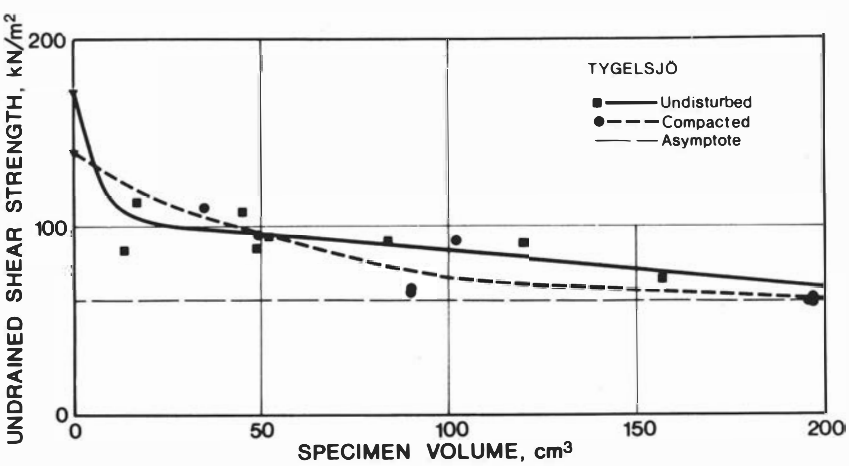

Fig. 6. Undrained shear strength versus specimen volume. Moraine clay from Tygelsjö.

to Lo, this strength is the same as the operational strength. The characteristics of the moraine clays reported herein have a volume dependence similar to that shown in Fig. 4. In Fig. 5 the undrained shear strengths from uni-axial compression tests on undisturbed and compacted samples are plotted against the specimen volume for the moraine clay from Sparta, Lund. Fig. 6 shows the relationship for the moraine clay from Tygelsjö. Results from fall-cone tests are also shown in these figures (denoted by a triangle), assuming the involved volume to be almost zero because of small cone penetration (4—5 mm). From Figs. 5 and 6 the conclusion can be drawn that the undrained strength approaches an asymptotic value, which is the operational value. The scattering of data is large, especially at small volumes, owing to the heterogeneous, fissured structure. Note, however, that the lowest strengths at different volumes quite naturally coincide with the asymptotic value.

Young's modulus from uni-axial compression tests on undisturbed samples has also appeared to be volume-dependent in a manner similar to strength.

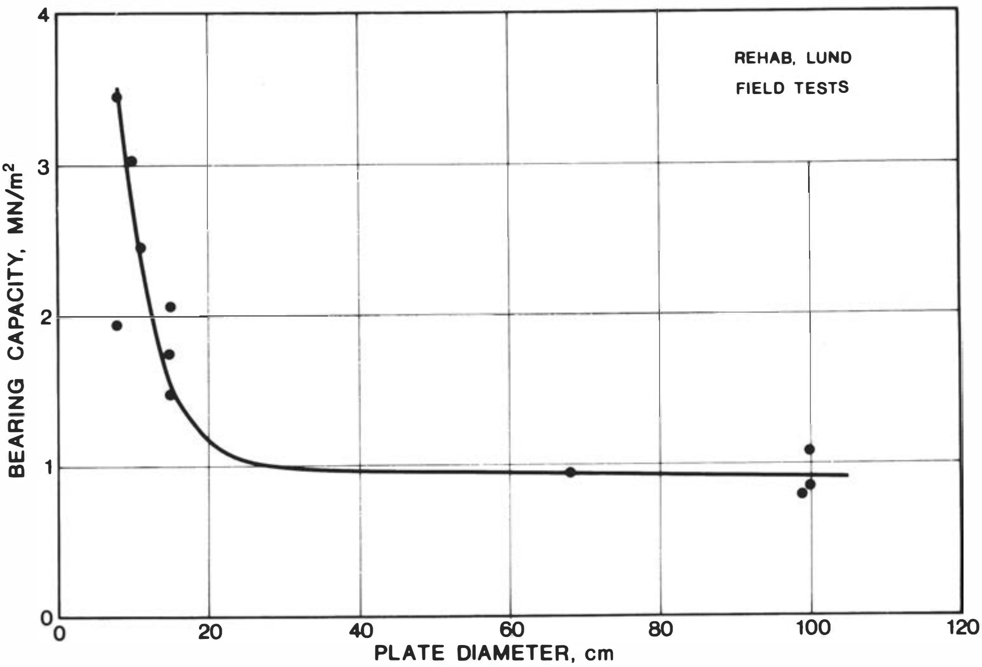

Fig. 7. Plate-bearing capacity versus plate diameter. Field tests at Rehab, Lund.

Field load tests with plate diameters varying between 8 and 100 cm have been performed on an undisturbed clay surface at Rehab, Lund, in order to determine whether the volume dependence is caused by sampling, sample-trimming, etc. The 100-cm plate represents full scale. As the plate diameter increases, the soil volume involved increases (cf. Fig. 2) and thus the measured bearing capacity should decrease. The results (Fig. 7) show the general size dependence. The results of field-vane tests, not presented here, using different vane diameters, show the same relationship. In summary, the investigated moraine clays have a strength-size dependence.

Influence of water content

The characteristics of moraine clays reveal a size dependence similar to that found for rocks and stiff, fissured clays. However, this dependence is also governed by factors other than fissures, such as the water content and content of coarse grains. The water content influences the geotechnical parameters in at least two ways, affecting their absolute value and their volume dependence. The absolute values of strength and elastic parameters decrease with increasing water content.

An investigation has been made on moraine-clay samples compacted by the same effort but with different water contents. Under the given compaction conditions, the dry density will vary, having a maximum at optimum water content. Compaction with a water content lower than optimum is said to be "on the dry side" and with a higher water content "on the wet side". Uni-axial compression tests were made to obtain the undrained shear strength and Young's modulus, and tri-axial compression tests to obtain the drained shear parameters of cohesion and angle of friction. These results are presented in Fig. 8. The parameters are plotted for two different specimen volumes (12—20 and 150—200 cm^3). For comparison, the undisturbed parameters are also given.

The undrained shear strength, cohesion and Young's modulus decrease with increasing water content, although the dry density passes a maximum. The angle of friction does not change. This may be explained by the hypothesis that different microstructures exist with different water contents and for different clay minerals. Unfortunately, no adequate investigation has been published concerning the significance of mineral type, but it is well known that swelling minerals, such as montmorillonite, influence the mechanical properties in a certain qualitative manner at different water contents. Therefore only the microstructure will be considered. Lambe (1958) gave the following hypothesis (Fig. 9) concerning the microstructure of the clay particles when the clay was compacted at different water contents. At low water content (w_A) the particles are flocculated in a disorderly array, caused by smaller

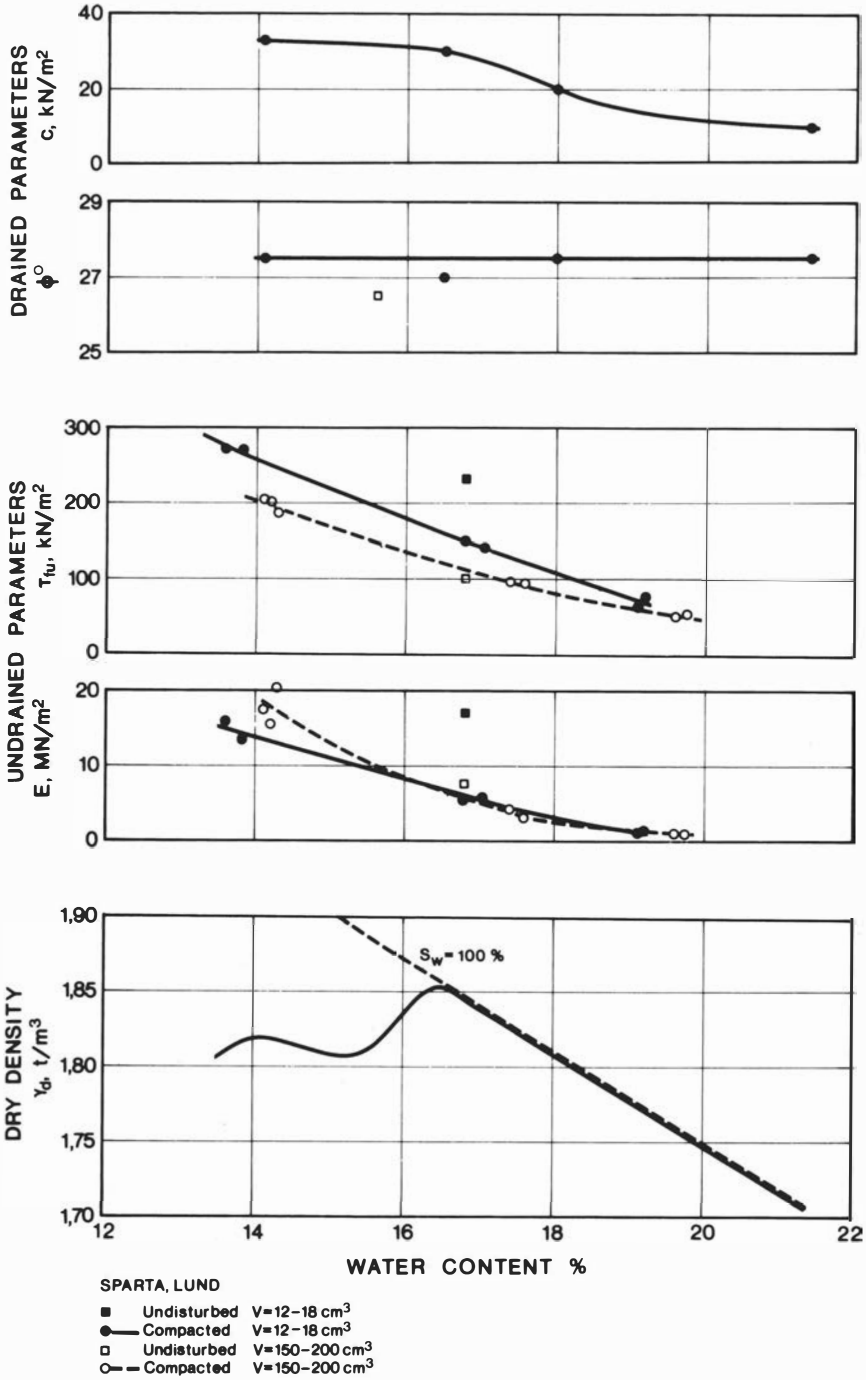

Fig. 8. Strength parameters and Young's modulus at different water contents. The compaction curve is also shown. Results are given both for compacted and undisturbed samples of moraine clay from Sparta, Lund, at two different volumes.

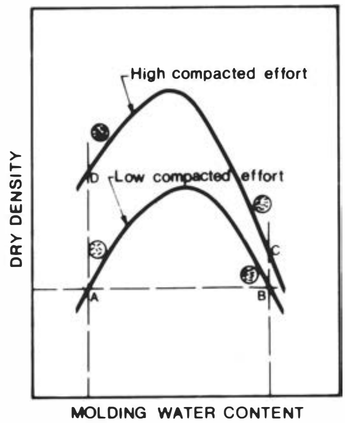

Fig. 9. Effects of compaction on structure (Lambe, 1958).

repulsive (electric) forces between the particles than attractive forces, resulting in a net attraction. As the water content increases, the repulsive forces between the particles increase, thereby permitting the particles to slide into a more orderly array, resulting in a higher dry density, up to a maximum value. Beyond the optimum moisture content (w_B), the degree of particle parallelism increases and the density decreases, because water begins to occupy the space otherwise occupied by solids. To understand the geotechnical properties, one also has to take into account water deficiency. In summary, increasing the water content results in a larger orientation of clay particles and in lower pore-water tensions. As the consequence of the behaviour described by Lambe's theory and as observed in the test results of Seed and Chan (1959), the structure stiffens at reduced water content, resulting in increasing strength and in diminishing compressibility. If the tests are run undrained, these effects are further increased by a lower pore-water over-pressure at lower water contents. With large strains, the initial flocculated structure will be destroyed.

Another explanation has been suggested by Pusch (pers. comm.). The basic thermodynamic idea is that aggregation with concomitant irregular coupling of particles takes place regardless of the water content, in saturated clay. When shearing takes place, fairly small deformations are sufficient to produce local orientation of particles (domains). This orientation brings adjacent particles into parallel positions, which increases the inter-particle repulsion forces between the equally charged basal surfaces. At a high water content, water is therefore sucked up into the shear zone (domain), resulting in an increased water content and a reduced shear strength. At a low water content, however, the majority of the pore-water is adsorbed on mineral surfaces and has a largely reduced mobility. Water suction into the shear zone is therefore prevented and the shear strength is high.

Fig. 8 shows that the difference in undrained shear strength between small (12—20 cm^3) and large (150—200 cm^3) specimen volumes decreases with increasing water content. At high water contents (on the wet side) the difference can be neglected (Figs. 8 and 10). The parameters of the undisturbed samples are the same as those of the compacted samples at large specimen volumes. The parameters of the small undisturbed samples are considerably higher than those of the compacted samples. This is probably due to time-dependent, strength-increasing re-ordering of mobile particles and of the water phase. It cannot, however, be excluded that cementation in the undisturbed clay may have contributed to the strength difference.

An explanation of the varying strength-size relationship at different water contents may be found in the fact that moraine clays are brittle (almost no plastic deformations at failure) at low water contents and ductile (large plastic deformations) at high contents. This means that the failure process, affected differently by coarse grains and fissures, changes with the water content.

Failure processes affected by coarse grains and fissures

Let us assume that a specimen of moraine clay is compressed uni-axially between two stiff plates by a stress q (Fig. 11). As the plates are stiff, the total longitudinal deformation will be the same over the whole surface. However, at different sections (for example,

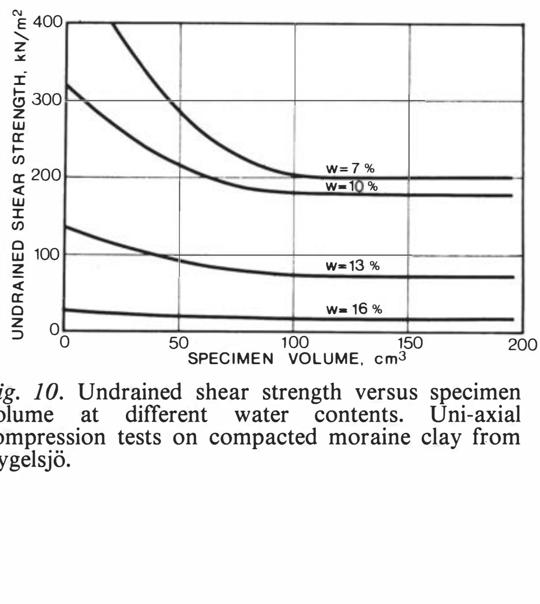

Fig. 10. Undrained shear strength versus specimen volume at different water contents. Uni-axial compression tests on compacted moraine clay from Tygelsjö.

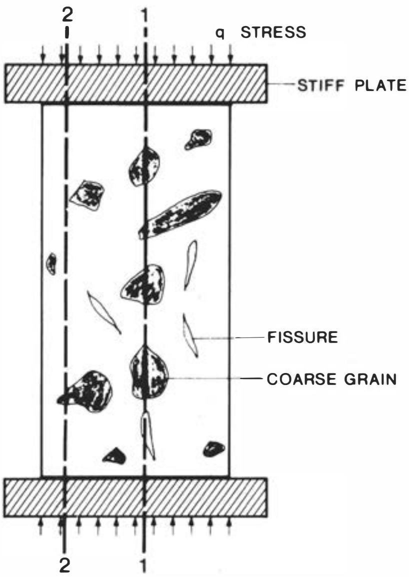

Fig. 11. A specimen of moraine clay uni-axially compressed between two stiff plates.

1–1 and 2–2 in Fig. 11) the coarse grain content is different, and thereby also the stiffness, since a single large particle has a much larger Young's modulus than bulk clay or silt. This produces varying stresses in the sample, the greatest in the stiffest sections. When the applied stress q increases sufficiently, failure is initiated in the fine material at the most stressed sections. If the moraine clay is brittle (low water content) the fracture will proceed through the whole sample, resulting in a macro-fracture. On the other hand, if the moraine clay is ductile (high water content) and with an applied stress q causing fracture in an over-stressed section, the result will be a plastic flow in this over-stressed region, causing stress relief, and only a local micro-fracture develops. In the ductile case, the sample will be able to support a further stress increase, until all the fine material in the sample yields. Fissures in the sample cause stress concentrations at the fissure tips comparable with stress concentrations in sections with a high content of coarse grains. In brittle material, macro-fracture starts at the fissure tips, and in non-brittle material yielding relieves the stress concentration.

Because of the difference in fracture mechanism in brittle and ductile moraine clays, the geotechnical characteristics of moraine clays are only size-dependent at low water contents. The dependence is caused by increasing probability with increasing sample volume that there will be sections with high contents of coarse grains and large fissures critically oriented.

SUMMARY

In this article the geotechnical parameters of strength and Young's modulus, used in designing structures on moraine clay, have been studied. The moraine clays have been investigated in the laboratory with fall-cone tests, uni-axial compression tests and tri-axial compression tests and in the field with load-bearing tests and vane-shear tests. The sample size was varied. It has been shown that the strength (especially the undrained strength) is dependent on the sample volume (the larger the volume, the smaller the value). A hypothesis has been given which requires that one condition for this dependence is that the moraine clay shall be brittle and contain fissures and coarse grains. Macro-fracture will be initiated by stress concentrations at fissure tips and in sections with a high content of coarse grains. The engineering problem is to find the operational values for the whole deposit from the small samples tested in the laboratory. A useful value is obtained if the lowest values from the tests are used, rather than the mean value, which is the usual procedure.

A hypothesis has also been presented to explain why the absolute values of strength and elasticity decrease with increasing water content. Consideration of the soil microstructure suggests that an increasing water content produces a softer structure with lower strength and higher compressibility.

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REFERENCES

Hartlén, J. 1970. Hållfasthetsegenskaper hos några skånska moränleror. SGI särtryck och preliminära rapporter, No. 39, 57–65. Stockholm.

Lambe, T.W. 1958*a*. The structure of compacted clay. *Proc. Am. Soc. Civ. Eng.* 84, SM2:1654:1–34. New York.

Lambe, T.W. 1958 *b*. The engineering behaviour of compacted clay. *Proc. Am. Soc. Civ. Eng.* 84, SM4:1655:1–35. New York.

Lo, K.Y. 1970. The operational strength of fissured clays. *Géotechnique.* 16, 4, 282–304. London.

Lo, K.Y., Adams, J.I. and Seychuk, J.L. 1969. The shear behaviour of a stiff fissured clay. *Proc. 7th Int. Conf. on Soil Mechanics and Found. Engng.* 1, 249–255. Mexico.

Pusch, R. and Jacobsson, A. 1971. Thixotropic action in remoulded quick clay. *Int. Conf. Engng. Geol. Moscow* (in press).

Seed, H.B. and Chan, C.K. 1959. Structure and strength characteristics of compacted clays. *Proc. Am. Soc. Civ. Eng.* 85, SM5:87–128. New York.