SHEAR STRENGTH OF NORMALLY CONSOLIDATED CLAYS

LA RESISTANCE AU CISAILLEMENT DES ARGILES CONSOLIDEES NORMALEMENT

СОПРОТИВЛЕНИЕ СДВИГУ НОРМАЛЬНО УПЛОТНЕННЫХ ГЛИН

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SYNOPSIS. In the paper are summarized results of an experimental study of normally consolidated clays carried out on samples consolidated at the same pressure as they carried in the field. Traxial and simple shear tests of this type have shown that the clay can sustain a shear stress in addition to the in-situ value, undergoing relatively small deformations provided the shear stress does not exceed a certain critical value. It is this critical shear stress which governs the vertical pressure a clay can carry with small settlements, the so-called quasi-preconsolidation pressure, and it also represents the maximum shear stress which can be mobilized under undrained conditions. The tests have furthermore shown that the critical shear stress is anisotropic, as its value varies with the direction in which the shear stresses are applied, relative to the direction of the in-situ shear stresses. The critical shear stress depends, in addition, on the rate at which the load is applied. Tests with different types of clays have shown that the ratio of critical shear stress and effective overburden pressure is a characteristic parameter of a normally consolidated clay. In general, the critical shear stress ratio increases with the plasticity of the clay.

INTRODUCTION

A few years ago it was discovered that triaxial tests on samples of normally consolidated clays, which prior to shearing were consolidated at the same stresses as they carried in the field, lead to stress-strain properties which are more representative of the behaviour of the clay in the field than those obtained from unconsolidated undrained tests, or obtained from tests on samples consolidated at stresses exceeding the field stresses. Since then a large number of tests of this type, including clays from different localities, have been carried out. In this paper the behaviour of the clays observed in this type of test is described, and the results obtained with different types of clays are compared.

TYPE OF CLAY CONSIDERED

The clays considered in this paper can be classified as normally consolidated, since they never have been subjected to effective stresses which are significantly higher than the present effective stresses in the field. Contrary to the "young" normally consolidated clays, the considered clays are "aged", i.e., they have carried the present effective overburden pressure over a considerable period of time. The secondary consolidation the clays have undergone during this period of aging has resulted in development of a reserve strength and a reserve resistance against a further compression. A characteristic feature of these clays is that in a consolidation test they show a quasi-preconsolidation pressure, p_c, which is higher than the effective overburden pressure, p_o, which the sample carried in the field, and that the ratio p_c/p_o is approximately constant with depth (Bjerrum, 1967).

When a normally consolidated clay of this type is subjected to additional shear stresses, the deformation increases rapidly when a certain critical value of the shear stress is reached, and it is this critical shear stress which governs the shear stress which the clay can carry in the field under undrained conditions. The properties of this critical shear stress are discussed below.

The clays used for the laboratory tests described in this paper were all late glacial or post-glacial normally consolidated clays. A list of these clays along with their index properties can be found in Table I. The samples were mostly taken from depths ranging from 5 to 12 m, which is deep enough to come below the upper dried or weathered crust and shallow enough to permit samples to be taken with a low degree of disturbance. As an example the results of tests carried out with a plastic clay from Drammen will be used. The properties of this clay has been previously described in various publications (Bjerrum, 1967). The general validity of the observed properties will be illustrated by comparing the properties observed on the Drammen clay with those found on clays from other locations.

SAMPLING AND TESTING PROCEDURE

The sampling was done with a 95 mm thin-walled fixed piston sampler which has been described by Berre, Schjetne and Sollie (1969). The field investigations included measurement of pore pressures at various depths by piezometers, so that the vertical effective stresses could be calculated. The horizontal effective stresses in the field were in most cases assumed to be equal to 0.5 p_o. During the study a method was developed for in-situ measurement of the horizontal stresses (Bjerrum and Andersen, 1972). Tests carried out at the sites from which samples had been taken, indicated that a K_o-value of 0.5 (as used in most of the tests) was somewhat too low, especially for clays of high plasticity. A few comparative tests indicated, however that the effect of assuming K_o to be 0.5 instead of 0.6 was small.

The samples were mounted into the triaxial cell, the simple-shear, and the consolidation apparatus, following a procedure described by Landva (1964) and Berre (1969). The triaxial specimens were prepared with a diameter of 8 cm and a height of 13 cm. After being placed in the triaxial cell, the specimens were consolidated under the in-situ stresses, p_o and $K_o p_o$, the stresses being applied in three or four steps. The samples were then left for some days before shearing. The shear test was mostly carried out as so-called constant rate of strain tests. Some tests were carried out by applying the shear stresses in small steps by dead weights. In conventional tests the time to failure was roughly two hours. Two different types of tests were carried out, compression and extension tests. In the compression test the vertical stress was increased and in the extension test the vertical stress was reduced, the horizontal stress being kept constant in both types of tests.

The simple shear device has been described by Bjerrum and Landva (1966). The specimens have a diameter of 8 cm and they were prepared with a height of 1.5 cm. After being placed in the apparatus, the specimen was consolidated under a vertical pressure equal to the effective overburden pressure in the field. The shear test was carried out as constant rate of strain tests; the time to failure was about 4 hours. The shear tests were performed at constant volume, which for saturated clays should correspond to undrained shear tests.

The consolidation tests were carried out with specimens having a diameter of 8 cm and a height of 2 cm. Most tests were performed by applying a load increment once a day.

SAMPLING DISTURBANCE

As a result of the inevitable disturbance during a sampling, any sample undergoes volume change when subjected to the field stresses in laboratory tests. The more disturbance the sample has suffered, the greater the volume change. In Table I are listed volume changes observed at end of consolidation expressed as a percent of the total volume of the samples, $\Delta V_c/V_o$. As seen from the values in the table, the volume change increases from 1.6% in the most plastic clay to 2-4% in clays of low plasticity, which indicates that samples of clays of low plasticity are somewhat more disturbed than samples of highly plastic clays.

During sampling the clay is subjected to shear deformations and thus a distortion of the original structural arrangement. This distortion is greatest in the outer zone next to the sample cylinder and smallest in the central part of the sample. However, after the sampling, the relatively undisturbed central part of the sample may suffer from a swelling, as it absorbes water from the outer, more disturbed zone[1]. The negative pore pressure in the sample is thereby reduced. If the straining, caused by a combined effect of the shear distortion during sampling and the subsequent swelling, exceeds a certain value, the structural arrangement of the central part of the sample will be so much distorted that when the field stresses are applied in a triaxial test, it will not restore its original shape. When the stresses are applied, some contact points will be overstressed and fail, and first after some straining and volume change the structure has obtained a stability sufficient to carry the field stresses again. The deformations connected with sampling are probably smaller and the degree of disturbance for a certain amount of deformation lower, the higher the plasticity of the clay. The reason for this is probably that a plastic clay is more cohesive than a lean clay.

THE CRITICAL SHEAR STRESS

In Fig. 1 is shown a typical result of an undrained triaxial compression test on a sample of the plastic clay in Drammen, maximum shear stress in the sample $\frac{1}{2}(\sigma_a - \sigma_r)$ divided by effective overburden pressure in the field, p_o, being plotted against axial strain. The curve starts at the field shear stress $\tau_o/p_o = \frac{1}{2}(1 - K_o)$ and, as seen, the sample can carry an additional shear stress without substantial straining. When the shear stress ratio reaches a critical value of about 0.37, the deformations start to increase, and the sample fails in undrained shear.

Fig. 1 also shows the results of a drained triaxial compression test, i.e. a test where the vertical stress is increased and the radial stress kept constant. A comparison of the two types of tests shows that the initial parts of the curves are very similar. They rise steeply and reach both of them a critical value of the shear stress at a τ/p_o-ratio somewhat lower than 0.40. An increase in shear stress beyond this value is accompanied by failure of a large number of contact points and thus large deformations. Under undrained conditions the failures of the contact points are accompanied by a further development of pore pressure, a further reduction in average effective normal stress, and thus a reduction in frictional resistance. The critical shear stress represents thus the maximum shear stress which can be mobilized under undrained conditions. Under drained conditions the straining beyond the critical value is accompanied by a tendency for the clay to consolidate. The reduction in volume means an increase in true cohesion. In addition, the average effective normal stress and thus the frictional component will increase. The shear stress can therefore be increased beyond the critical value.

1) This problem has been studied in detail by Schjetne, and the results will be published in a separate paper

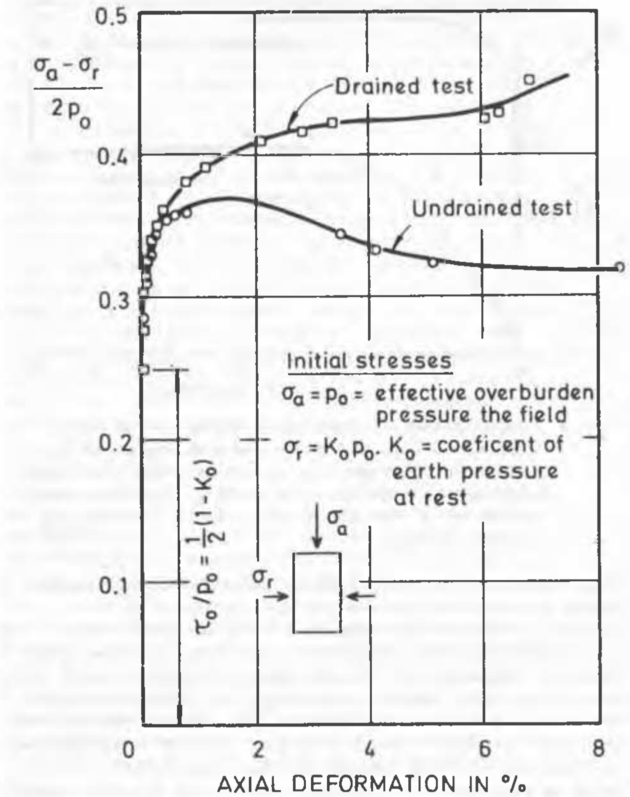

Fig. 1 Results of a drained and an undrained triaxial test on samples of the plastic clay in Drammen.

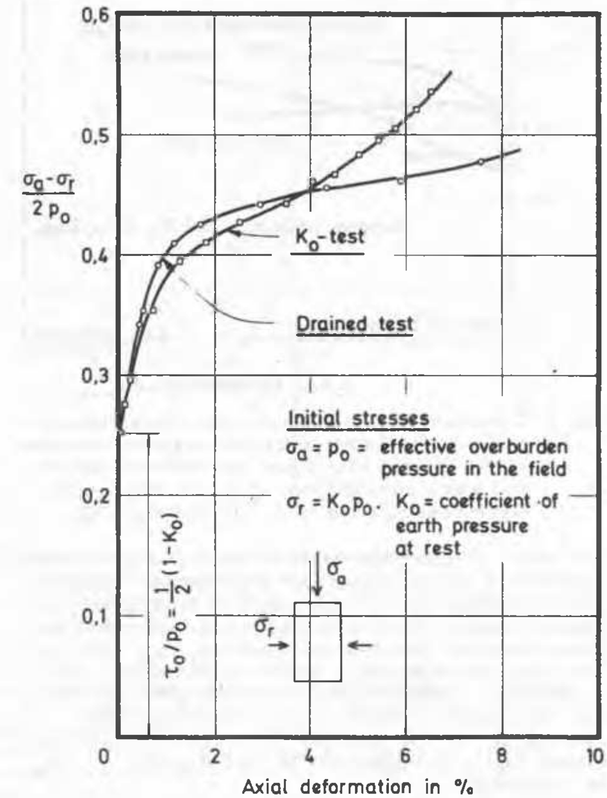

Fig. 2 Results of a K_o-triaxial test and a drained triaxial compression test on two samples of the plastic clay in Drammen.

Fig. 2 shows a comparison between a drained triaxial compression test and a consolidation test carried out with no lateral deformation. The consolidation test in Fig. 2 is labelled "K_o-test", and it was carried out as a drained triaxial test in which the vertical load was increased in steps, and simultaneously the lateral pressure was increased so that the cross-section of the sample remained constant. The result of such a test is very nearly identical to that of a standard oedometer test.

The consolidation curve shows the well known shape with a bend occurring at a stress level somewhat greater than the field stresses. Expressed in terms of shear stresses, the K_o-test shows that the shear stress can be increased from an initial value of 0.25 p_o to about 0.40 p_o before the deformation starts to increase. Expressed in terms of the major principal stress, as usually done, the stress at which the consolidation curve bends corresponds to a value of p_c of about 1.5 p_o. The drained triaxial test shows about the same stress-strain behaviour as the K_o-test with a steep rise and a sharp bend, which is reached at about the same critical shear stress for the two types of tests.

From the test results shown in Figs. 1 and 2 we thus learn that the initial part of the stress-strain curve of an undisturbed plastic clay from Drammen is characterized by a rather steep rise followed by a sharp bend, reached at a relatively small strain. A clay of this type can thus sustain a shear stress in addition to the in-situ value without significant deformations and volume changes, provided it does not exceed a certain critical value. This "critical shear stress" governs the vertical pressure the clay can carry without settlements, the so-called quasi-preconsolidation pressure, or the p_c-value observed in the K_o-test or in standard consolidation tests. The critical shear stress also governs the undrained shear strength which can be mobilized in compression.

In Fig. 3 the drained and undrained stress-strain curves for samples of the plastic clay from Drammen consolidated at the field stresses, are compared with the corresponding curves for samples which before shearing were consolidated at a pressure greater than the field stresses. The last-mentioned curves, which represent the properties of a "young" normally consolidated clay, show no sign of the behaviour observed on the samples consolidated at the field stresses, thus proving that the behaviour characterized by the existence of a critical shear stress reached at small strains is the result of changes which have occurred since the deposition of the clay. From previous studies of the compressibility of the clay it is known that these changes are due to secondary consolidation occurring during and after the deposition of the clay.

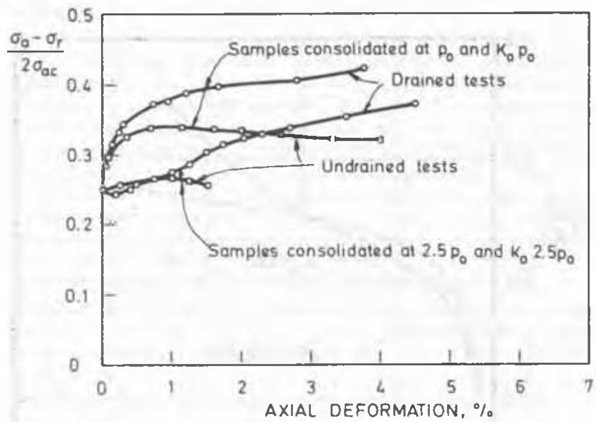

Fig. 3 Comparison of stress-strain curves observed in drained and undrained tests on samples of the plastic clay from Drammen of which (a) were consolidated at p_o and $K_o p_o$, (b) were consolidated at 2.5 p_o and $K_o 2.5\, p_o$.

The critical shear stress thus reflects an increased stability of the structural arrangement of the particles obtained as a consequence of a secondary consolidation. Furthermore, it has been established that the reserve resistance $p_c - p_o$, which a clay has gained during a period of secondary consolidation, increases proportionally with the effective overburden pressure, p_o, which the clay carried during that period. From this can be concluded that in a homogeneous clay deposit, τ_{cr}/p_o is a constant.

Compared with unconfined compression tests, triaxial tests on samples, which before shearing were consolidated at the field stresses, show properties which are believed to be more representative of the field conditions. Fig. 4 illustrates the difference between the two types of tests. This figure shows the results of two undrained triaxial compression tests on two samples of the plastic clay from Drammen, both carried out at the same relatively rapid rate. One of the samples was tested in an unconsolidated condition, the second was consolidated at the field stresses prior to shearing. The shear strength observed at failure in the undrained test on the unconsolidated sample was found to correspond to a τ_{cr}/p_o-value of 0.30, whereas the specimen consolidated at the in-situ stresses showed a value of 0.48. The explanation of this difference is, as mentioned above, that the negative pore pressure set up when the sample was unloaded from the in-situ stresses, has disappeared, partially or completely. The effective normal stresses in the sample are therefore smaller than they would have been had the test represented the conditions in the field. On the other hand, the sample reconsolidated at the field stresses has the correct effective normal stresses and thus the correct frictional contribution to the shear strength. Provided the reduction in water content resulting from the consolidation at the field stresses is not too large, the results of a shear test with such a sample should give a fair representation of the behaviour of the clay in the field.

During the testing programme described in this paper, it was discovered that the time of consolidation also had some influence on the results.

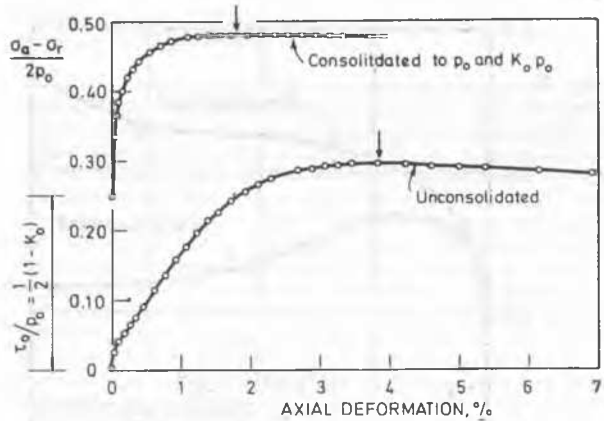

Fig. 4 Comparison of undrained triaxial compression tests carried out on two samples of the plastic clay from Drammen of which (a) was not consolidated before testing, (b) was consolidated at the field stresses, p_o and $K_o p_o$, before testing.

Fig. 5 shows the results of two triaxial compression tests carried out on two parallel samples of the plastic clay from Drammen. Both samples were consolidated at the field stresses, but, whereas the time of consolidation of one of the samples was 4 days, the other sample remained for 70 days before the shear test was carried out.*) A comparison of the two curves shows that the longer time of consolidation has resulted in a more brittle behaviour ; the peak is reached at a smaller strain and the decrease in shear stress after failure is more pronounced. It may be that the test with the long consolidation time to a greater extent has restored its original structural arrangement, and that it therefore is more representative for the field behaviour than the test with the short time of consolidation.

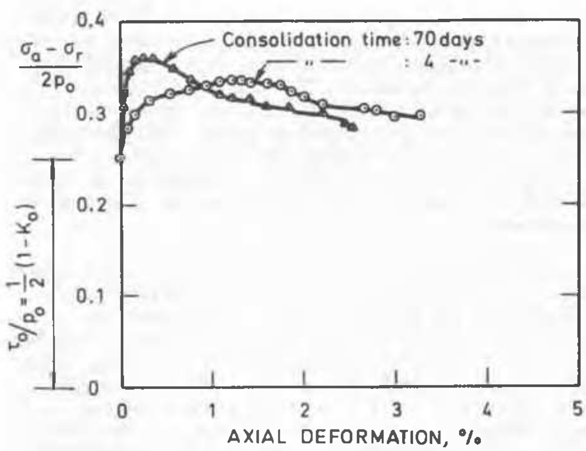

Fig. 5 Comparison of the stress-strain curves observed in undrained triaxial compression tests carried out on two samples of the plastic clay from Drammen of which (a) was consolidated over a period of 4 days, (b) was consolidated over a period of 70 days.

*) Both tests were carried out at a rate of strain of 0.0040 percent per hour.

In the past, the fear has been repeatedly expressed that a reconsolidation of a soft clay to its original stresses would, as a result of sampling disturbance, cause a reduction in water content accompanied by a gain in strength, so that the shear resistance obtained from such a test would exceed the value that can be mobilized in the field. Although it is in principle correct that a reduction in water content is accompanied by a gain in true cohesion, the error thus introduced is small compared with the error involved in testing the unconsolidated clay, i.e. with the effective normal stresses being smaller than those in the field. The most important effect of a sampling disturbance is the destruction of the original structural arrangement of the clay gained during hundreds or thousands of years of secondary consolidation being characterized by the presence of a critical shear strength which is reached at a small strain. A disturbed and reconsolidated sample of a clay with such properties will show a reduction in strength and an increase in strain at failure, and the more disturbed the sample, the more pronounced this effect. This fact is illustrated in Fig. 6, which shows the value of τ_{cr}/p_o observed in a series of undrained compression tests plotted against the volume change the samples underwent when they were consolidated at the field stresses.

In conclusion, when following the procedure outlined above, the net effect of a sample disturbance is a change in properties of the clay in the direction from that of an "aged" to that of a "young" clay, see Fig. 3. Compared with the field behaviour, a disturbance will mean a lower value of the critical shear stress ratio τ_{cr}/p_o and a larger strain at failure.

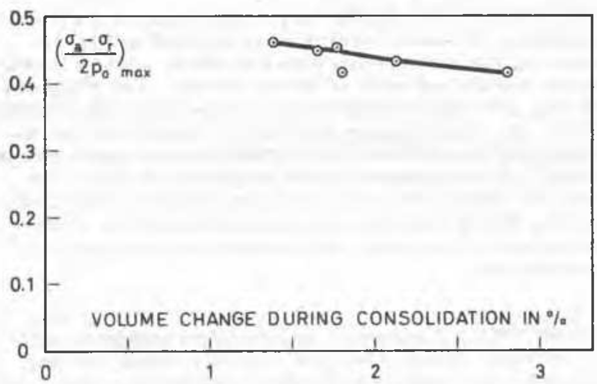

Fig. 6 Relationship between volume change, occurring when consolidating at the field stresses, and critical shear stress observed in undrained compression tests on plastic clay from Drammen.

ANISOTROPY

Fig. 7 shows the results of two undrained triaxial tests carried out at equal rates on two identical samples of the plastic clay from Drammen, after they were consolidated at the field stresses. One of the tests is a compression, the other an extension test carried out by keeping the lateral stress constant and reducing the axial stress. During an extension test, the initial shear stresses are first reduced to zero. The shear stresses are subsequently increased in the opposite direction. As seen from the diagram, the clay in an extension test does not show the same brittle behaviour as observed in compression tests. The stress-strain curve resulting from an extension test is more gently curved and it does not show a pronounced peak. Due to the difficulties involved in defining a failure value from the extension tests which is directly comparable with the peak observed at relatively small strains in compression tests, the value of the so-called τ_{cr} has arbitrarily been selected to be the shear stress at 2% axial strain.

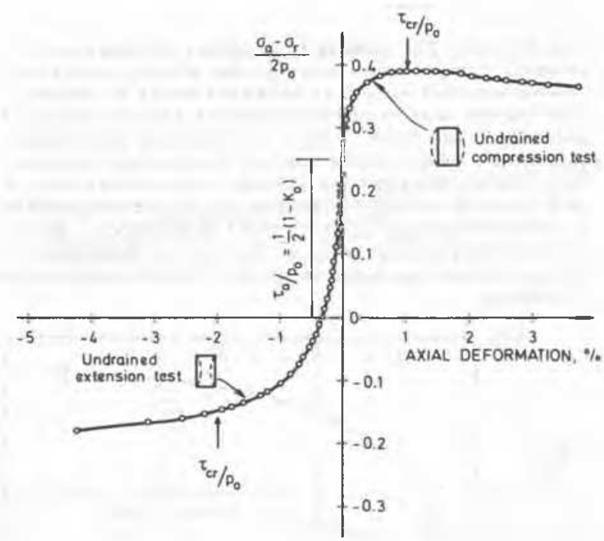

Fig. 7 Stress-strain curves observed in two undrained triaxial tests on samples of the plastic clay from Drammen, (a) compression test, (b) extension test.

With this definition the critical shear stress observed in the extension test in Fig. 7 is as low as about 0.16 p_o compared with the value 0.40 p_o observed in the compression test. The anisotropy expressed as the ratio of τ_{cr}/p_o observed in a compression and an extension test is thus 2.5 for the plastic clay from Drammen.

The pronounced anisotropy of normally consolidated clays obviously reflects that these clays have an anisotropic structure. This fact is understandable, since the clay structure has adapted itself in such a way that it can carry the anisotropic state of in-situ stresses, p_o and $K_o p_o$. If the shear stresses are increased in the same direction as the in-situ stresses were acting, full benefit is made of the shear resistance and especially the frictional resistance already mobilized in the field. A minimum value of the critical shear stress will be found if the shear stresses are applied in such a way that they become reversed relative to the field stresses. With respect to the anisotropy, it should be mentioned that the validity of the above comparison between triaxial compression and extension tests depends on the degree to which the results are influenced by the value of the intermediate principal stress which, in the compression and extension tests, is equal to the minor and the major principal stresses respectively.

EFFECT OF TIME

Fig 8 shows the results of a series of undrained triaxial compression tests on the plastic clay from Drammen carried out at different rates of loading. The fastest test is carried out at a rate of strain of about 35% per hour, which corresponds approximately to the rate usually applied in standard in-situ vane tests, whereas the slowest test is carried out at a rate 25.000 times slower, which corresponds to a rate of loading similar to that in practice. It is immediately observed from the curves that the undrained shear strength is greatly influenced by rate of loading.

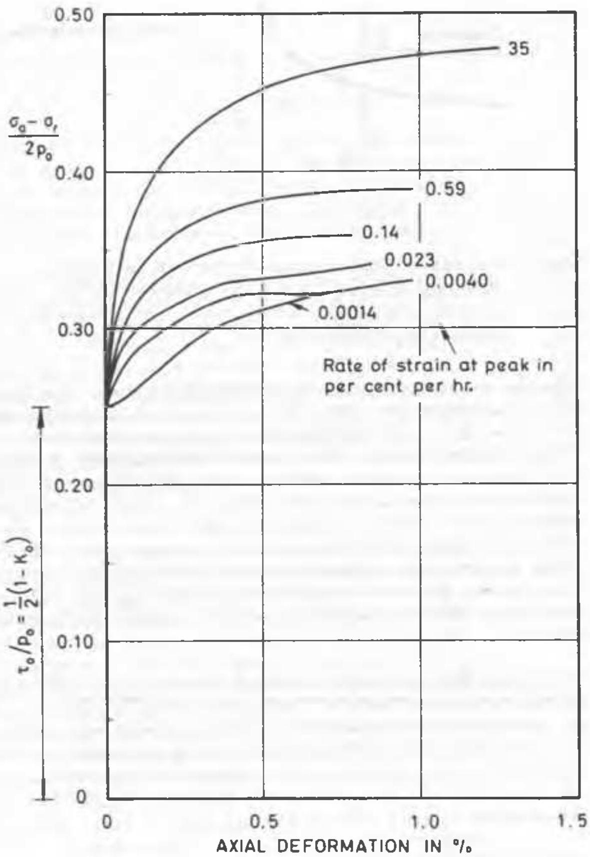

Fig. 8 Results of a series of undrained triaxial tests on samples of plastic clay from Drammen carried out at different rates of strain.

Fig. 9 summarizes results of a comprehensive study of effect of time on the undrained shear strength of plastic clay from Drammen. In a semi-logarithmic diagram the value of the critical shear stress is plotted against the time to failure. The critical shear stress is expressed relative to the value observed at standard rate of loading with a time to failure of 140 minutes. The test results plotted in the figure include in the first place tests on samples consolidated at the field stresses. But in the second place are also included results of a series of tests on samples consolidated at a pressure well above the field stresses ($2.5\ p_o$ and $K_o\ 2.5\ p_o$, and in the third

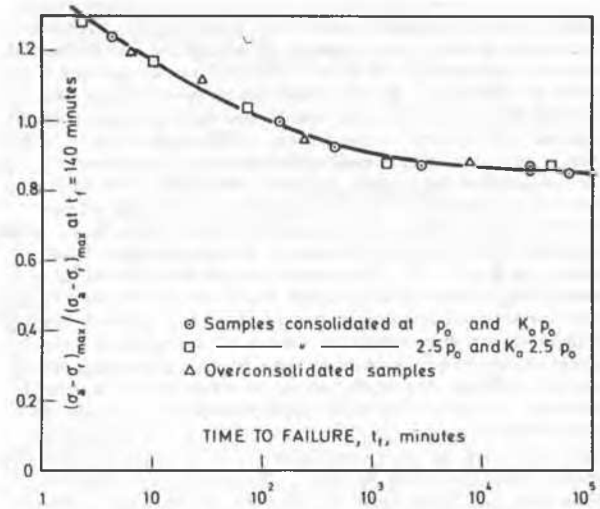

Fig. 9 Relative value of critical shear stress observed in undrained triaxial compression tests on samples on the plastic clay from Drammen, plotted against time to failure.

place were plotted results of a series of overconsolidated samples. As observed all results plot on a single curve which clearly illustrates that the effect of time is most pronounced in the range of loading normally applied in laboratory tests, but decreases as time to failure increases.

A more complete picture of the stress-strain-time behaviour of the plastic clay from Drammen within the range of small strains is presented in Fig. 10, showing the correlation between shear-stress level, shear strain and rate of shear strain. The diagram in Fig. 10a was obtained from undrained compression tests, an the diagram in Fig. 10 shows the corresponding correlation found from drained compression tests*). A comparison of the diagrams in Figs. 10a and 10b show that they are very similar, thus indicating that the stress-strain-time behaviour is approximately the same for drained and undrained conditions.

From such a diagram it is possible roughly to estimate shear strain and rate of shear strain for any stress-time path in a triaxial compression test. For instance, when the clay is subjected to a constant rate of strain test, the behaviour can be observed from the diagram directly; the shear strain increases approximately exponentially with the shear stress, resulting in a sharp bend of the stress-strain curve. The critical shear stress reached when the curve bends increases approximately linearly with the logarithm of the rate of strain applied. Furthermore, the diagram illustrates that the rate of shear strain increases exponentially with an increase in shear stress provided the change in strain is insignificant. Finally, the diagram permits an evaluation of the decreasing rate of straining for a constant shear stress, as the result of a mobilization of additional frictional resistance with further straining.

*) For the drained compression tests the vertical stress was increased and the horizontal stress kept constant.

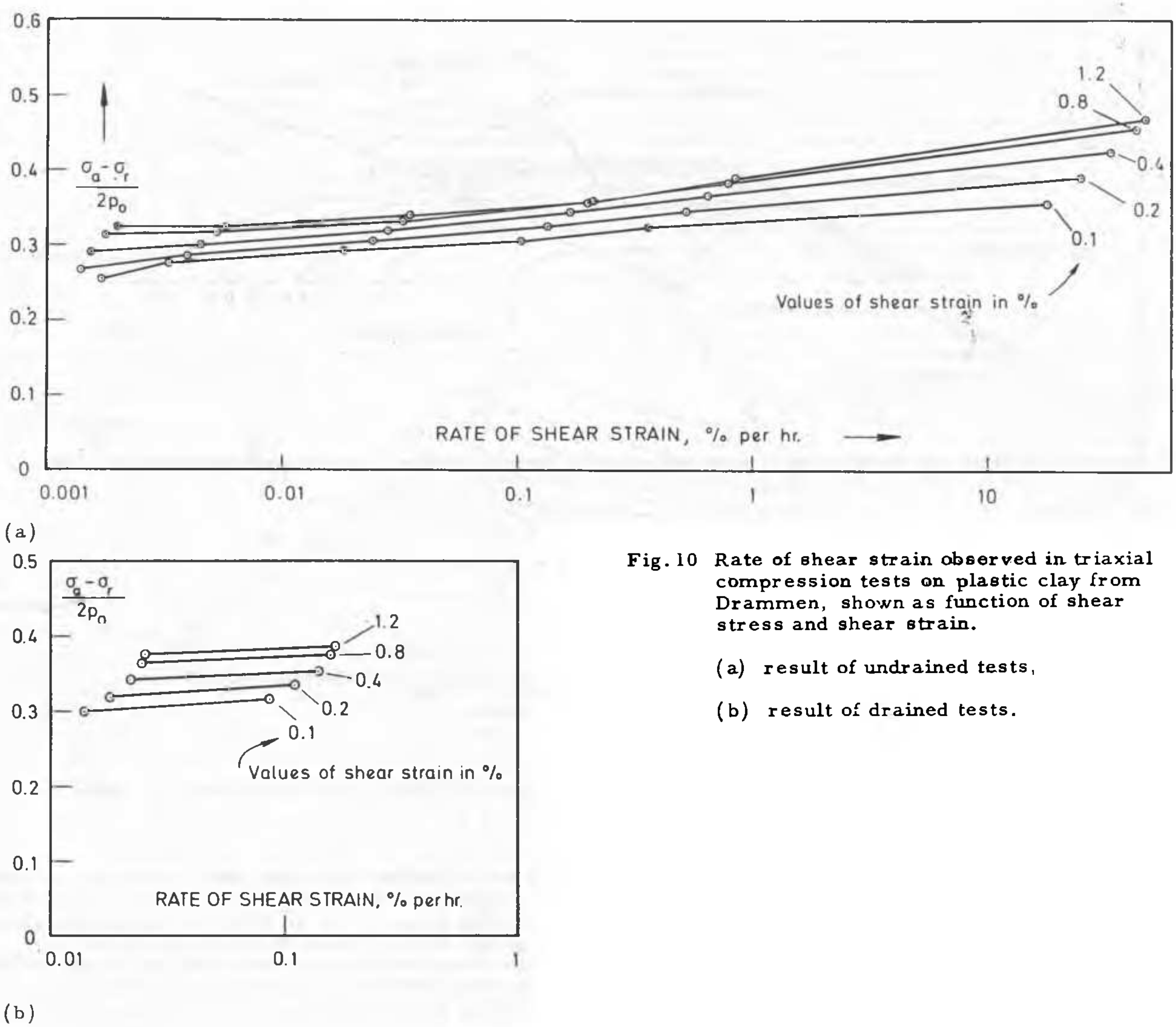

Fig. 10 Rate of shear strain observed in triaxial compression tests on plastic clay from Drammen, shown as function of shear stress and shear strain.

(a) result of undrained tests,

(b) result of drained tests.

EFFECT OF A CONSOLIDATION OR SWELLING ON THE STRESS-STRAIN PROPERTIES

It was mentioned above that the critical shear stress observed in triaxial drained, undrained and K_0-tests was nearly identical. This means that in tests carried out at conventional laboratory rates, the initial part of the shear stress-shear strain curves, including the value of the critical shear stress, are practically independent of the average effective normal stress. This finding is further illustrated in Fig. 11, which shows "constant shear strain contours" for drained triaxial tests in a plot of shear stress versus average effective normal stress.

Shear strain, γ, is here defined as the difference between vertical and horizontal strain. Fig. 11 also shows typical effective stress paths for different types of drained tests and paths for an undrained compression and an undrained extension test, both loaded at about the same rate as the drained tests. It is seen that the shear strain contours are nearly horizontal to the right of the stress paths for the undrained tests. This finding also means that the shear strains will not be much influenced by a small drainage for cases where pore pressure dissipation increases the average effective normal stress (consolidation), as is the case for most foundation problems. For cases where pore pressure dissipation decreases the average effective normal stress (swelling), the shear strain clearly tends to increase at constant shear stress for the compression case. For the extension case, shear strain contours have not been determined to the left of the stress path of the undrained test. It is assumed, however, that they will be inclined as for the compression case and that the shear strain will, therefore, increase if a pore pressure dissipation leads to a reduction in the average effective normal stress.

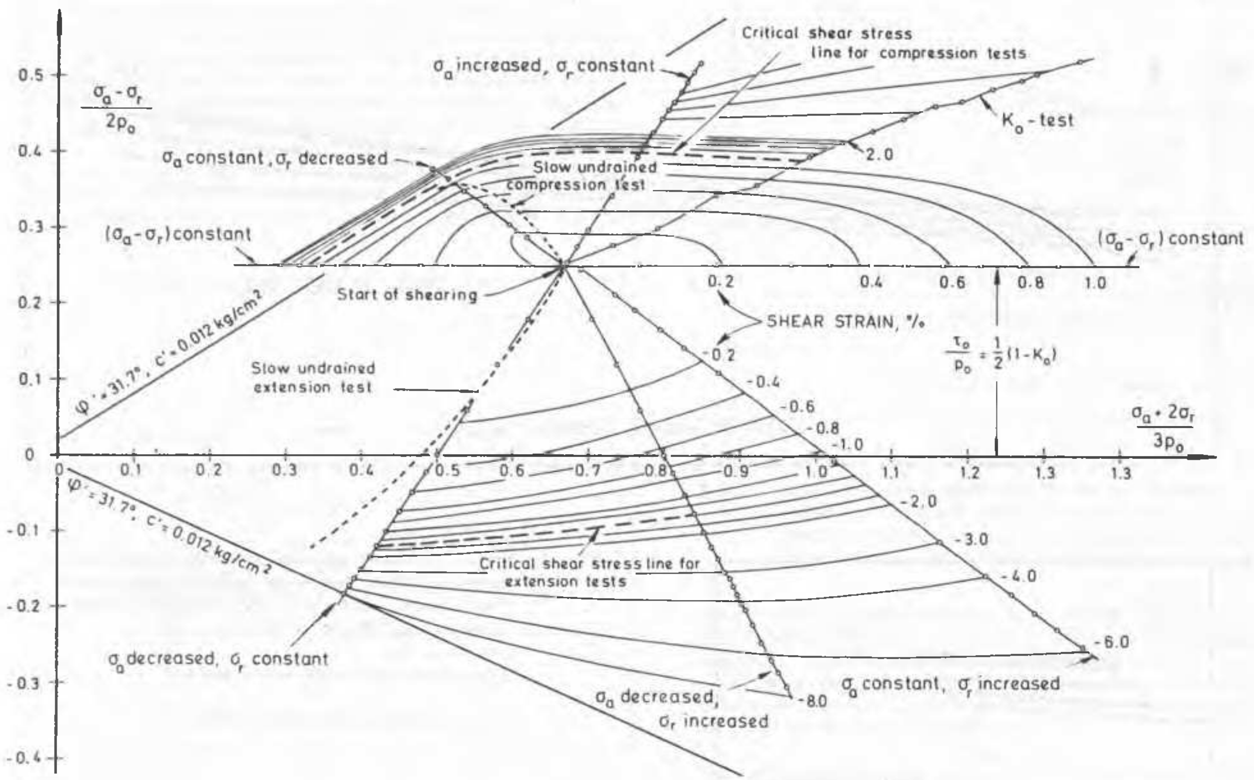

Fig. 11 Shear strain contours observed in drained triaxial tests on plastic clay from Drammen, plotted in a Mohr diagram.

COMPARISON OF DIFFERENT CLAYS

The concepts presented above were discovered so recently that the new parameters are available only from seven different clays. These clays are listed in Table I together with their index properties (columns 2 through 5), and the clays are arranged according to their plasticity indices. (The clays from " Bangkok, east" and " Kimola" are from Thailand and Finland respectively, the others are from Norway). The table lists the main results of the undrained triaxial compression and extension tests (columns 7 through 13). Column 7 gives the value of $K_o = (\sigma_r/\sigma_a)_c$ assumed when the samples prior to shearing were consolidated at the in-situ stresses; column 8 includes the volume changes, $\Delta V_c/V_o$ of the samples when subjected to the field stresses. Columns 9 and 11 give the values of τ_{cr}/p_o observed in compression and extension tests, and for the compression tests are also listed the strain at failure, ϵ_f, in column 10. Columns 12 and 13 give the value of the shear strength parameters φ' and c' in terms of effective stresses observed at ultimate failure.

The table also includes the value of τ_{cr}/p_o observed in simple shear tests (column 14) and values of p_c/p_o and $C_c/(1+e_o)$ found from standard consolidation tests (columns 15 and 16). Columns 17 and 18 give K_o-values determined from K_o-triaxial tests and from tests in-situ, as described by Andersen and Bjerrum (1972). Finally, columns 19 and 20 list the undrained shear strength observed by in-situ vane tests, the values being expressed by the ratio s_u/p_o. Column 19 gives the ratio obtained from the directly observed strength values. In order to permit a comparison with the results of the laboratory tests it is necessary to correct them to the same rate of loading. The resulting values after this correction are presented in column 20.

From the table it is directly observed that the sampling disturbance as evaluated from the volume change which took place when the samples were consolidated to the field stresses, is greater the less plastic the clay. It can also be read that the lower the plasticity of the clay, the smaller the failure strain, ϵ_f.

The most obvious result of a comparison of the different clays is that the value of the critical shear stress ratio τ_{cr}/p_o increases with the plasticity of the clay, irrespective of whether it is observed in triaxial or in simple shear tests, or whether the tests were carried out as compression or extension tests. This finding is further illuminated by Fig. 12 in which the various values of the ratio τ_{cr}/p_o are plotted against the plasticity index of the clay.

From the table it is furthermore concluded that all normally consolidated clays show a pronounced anisotropy and that the less plastic the clay, the greater is the anisotropy. The ratio of τ_{cr}/p_o observed in compression and extension tests thus

SITE	Depth in metres	INDEX PROPERTIES: Water content w	Liquid limit w_L	Plastic limit w_p	Plasticity index I_p	Sensitivity S_t	TRIAXIAL TESTS: $\left(\frac{\sigma_r}{\sigma_a}\right)_c$	$\frac{\Delta V_c}{V_o}$	Compression τ_{cr}/p_o	Compression ϵ_f	Extension τ_{cr}/p_o	φ' degrees	c' kg/cm²	Simple shear $\frac{\tau_{cr}}{p_o}$	Oedometer $\frac{p_c}{p_o}$	Oedometer $\frac{C_c}{1+e_o}$	K_o-test	K_o In-situ	In-situ vane $\frac{\tau_{cr}}{p_o}$	In-situ vane $\left(\frac{\tau_{cr}}{p_o}\right)_{cor.}$
BANGKOK, east	5.5- 8.7	137	152	64	88	6	0.6-0.9	1.6	0.71	1.60	-0.37	39.8	0.03	0.42	1.8	0.81	0.45	-	0.57	0.44
KIMOLA	7-12	40-60	62	31	31	8	0.50	1.9	0.46	1.20	-0.32	41.5	0	0.36	1.8	0.48	0.45	-	-	-
DRAMMEN (plastic)	6.3- 9.3	52	60	29	31	8	0.50	2.3	0.40	0.93	-0.16	31.7	0.01	0.32	1.5	0.39	0.49	0.61	0.36	0.28
SUNDLAND	5.5- 7.5	59	58	30	28	11	0.43	-	0.40	0.80	-	-	-	0.30	1.3	0.38	0.48	0.55	0.35	0.27
VATERLAND	5-10	40	47	27	20	5	0.50	2.8	0.37	0.80	-0.13	30.0	0	0.28	1.25	0.14	0.51	-	(0.47)	(0.36)
STUDENTER LUNDEN	12-18	33	37	20	17	5	0.60	2.8-4.3	0.32	0.51	-0.09	30.0	0	0.19	-	-	0.51	0.63	0.19	0.15
DRAMMEN (lean)	6.0-12.5	32	33	23	10	8	0.50	1.7-4.2	0.34	0.34	-0.07	30.0	0	0.22	1.3	0.15	0.49	0.64	0.14	0.11
Column number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

Table I. Comparison of strength and stress-strain parameters for different clays.

varies from a value of about 2 in the highly plastic Bangkok clay, to about 5 in the lean clay from Drammen. The correlation between the plasticity index of a normally consolidated clay and its anisotropy is further illustrated in Fig. 12, showing the values of τ_{cr}/p_o observed in compression and extension tests. Although the available test results do not cover the complete range, they certainly indicate that the τ_{cr}/p_o values observed in compression and extension tests are related to the plasticity of the clay and that the anisotropy expressed by their ratio increases for decreasing plasticity index of the clay.

Table I also includes results of constant volume simple shear tests, expressed as the ratio of τ_{cr}/p_o where τ_{cr} is equal to maximum horizontal shear stress. For comparison with the results of the triaxial tests, the values are also plotted against the plasticity index in Fig. 12. As observed, the simple-shear tests show values of τ_{cr}/p_o which are generally close to the average of those measured in triaxial compression and extension tests.

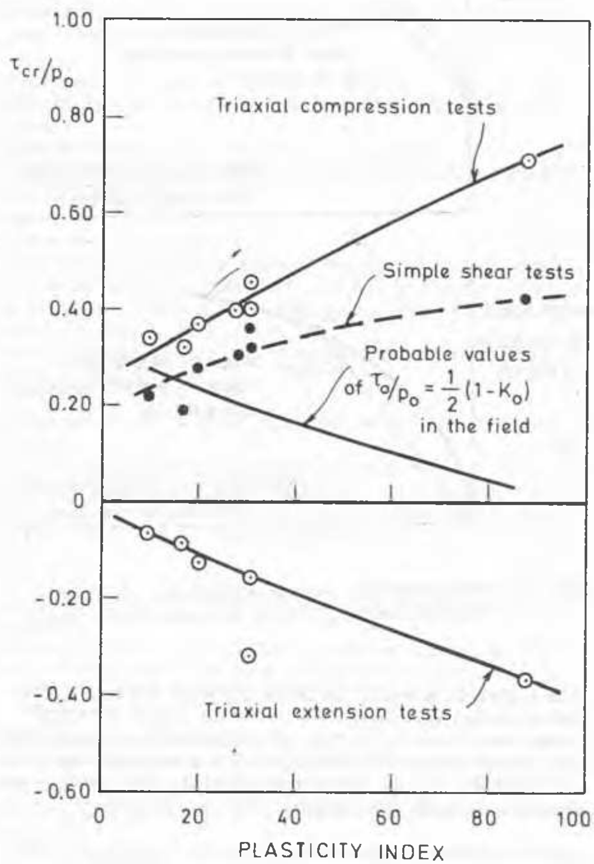

Fig. 12 Correlation between plasticity index and values of τ_{cr}/p_o observed in undrained triaxial compression and extension tests and in simple-shear tests on normally consolidated clays.

A comparison between the corrected vane shear strength and the results of the laboratory tests shows that the vane strength is roughly equal to the average of the values observed in the three types of laboratory tests. It is also observed that the ratios of the shear strength observed in compression and extension tests to the vane strength are of the order of 1.7 and 0.6, thus demonstrating the importance of the anisotropy on the undrained strength of normally consolidated clays.

THE SECANT MODULUS

All tests referred to in this section are undrained. The secant modulus for triaxial tests, E, is defined in Fig. 13a. The secant modulus for simple-shear tests, G, is defined in Fig. 13b. Fig. 14 shows how E varies with stress level below τ_{cr} for a triaxial compression test on plastic clay from Drammen when time to failure was about two hours.

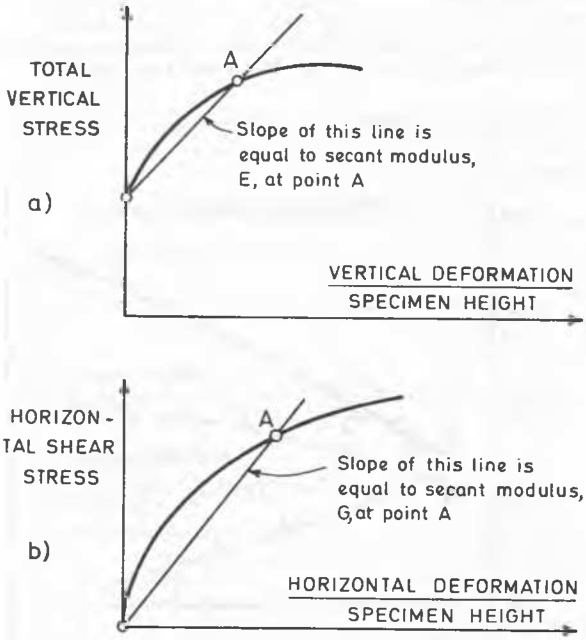

Fig. 13 Definitions of secant modulus; (a) for triaxial tests, (b) for simple-shear tests.

The value of E and G at 50% reserve strength are denoted E_{50} and G_{50}, the reserve shear strength being equal to $(\tau_{cr} - \tau_o)$. In Fig. 15 E_{50}/p_o is plotted versus time to failure for the compression triaxial tests in Fig. 8. It is seen that E_{50}/p_o decreases drastically with increasing time to failure.

Table II gives values of E_{50}/p_o and G_{50}/p_o for the same clays as in Table I. The values are from tests where time to failure was a few hours. It is seen that E_{50}/p_o is about the same for compression and extension triaxial tests. Table II also gives values of E_{50}/G_{50}, the values of E_{50} being taken from triaxial compression tests. This ratio varies from 2.7 to 6.5 with an average value of 4.0. For an isotropically elastic material, the ratio between Young's modulus and modulus of shear is equal to 3, if Poisson's ratio is equal to 0.5.

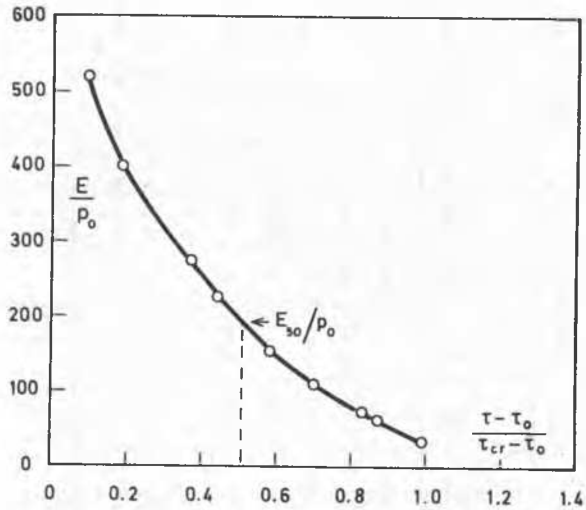

Fig. 14 Variation in secant modulus with stress level for an undrained triaxial compression test.

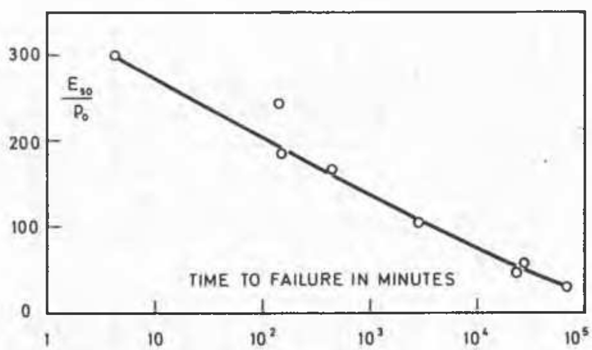

Fig. 15. Variation in secant modulus with rate of loading for undrained triaxial compression tests.

SITE	E_{50}/p_o		G_{50}/p_o	$\frac{E_{50}}{G_{50}}$
	Triax. compr.	Triax. extens.	Simple shear	
BANGKOK, east	160	110	35	4.5
KIMOLA	330	300	105	3.1
DRAMMEN (plastic)	235	210	65	3.8
SUNDLAND	165	-	65	2.6
VATERLAND	335	335	65	5.1
STUDENTERLUNDEN	350	250	55	6.5
DRAMMEN (lean)	260	275	95	2.7

Table II. Values of secant modulus at 50% mobilization of reserve shear strength (undrained tests).

CONCLUSION

The above described experimental study of the properties of normally consolidated clays has demonstrated that a realistic picture of the stress-strain behaviour of this type of clays is only obtained if the tests are carried out with samples consolidated at the same pressures as they carried in the field. Tests of this type have shown that the initial part of the stress-strain curves of such clays is characterized by a rather steep rise followed by a sharp bend reached at a relatively small strain. This means that the clay can sustain a shear stress in addition to the in-situ value without significant deformations, provided it does not exceed a certain critical value. It is this "critical shear stress" which governs the vertical pressure the clay can carry without settlements, the so-called p_c-value observed in standard consolidation tests, and it represents the maximum shear stress which can be mobilized under undrained conditions.

The critical shear stress observed at small strains reflects an increased structural arrangement of the particles obtained as a consequence of the delayed consolidation which has occurred since the clay was deposited. Triaxial compression and extension tests and simple-shear tests have demonstrated that the clays in the range of small strains show the following significant properties:

(1) The ratio of the critical stress to the effective overburden pressure τ_{cr}/p_o observed in compression tests is approximately constant in a homogeneous unweathered clay deposit. The greater the plasticity index of the clay, the higher the value of τ_{cr}/p_o.

(2) The stress-strain property is an anisotropic property, as it varies with the direction in which the shear stresses are applied, relative to the direction of the in-situ stresses. The maximum value of τ_{cr}/p_o is thus obtained when the stresses are increased in such a way that the shear stresses act in the same direction as the in-situ shear stresses; the minimum value is obtained if the shear stresses are applied in the opposite direction.

(3 The stress-strain property depends on the rate at which the stresses are applied. The critical shear stress is thus not a constant, but depends on the rate at which the load is applied.

(4) The initial part of the shear stress-shear strain curve and the critical shear stress are approximately the same for drained compression, undrained compression, and K_o-triaxial tests carried out at conventional laboratory rate of loading.

The above described properties of the normally consolidated clays observed in laboratory tests on samples consolidated at the field stresses are believed to be in general agreement with the behaviour of the clays when they are subjected to a change in stresses in the field. This fact is further discussed in two recent papers (Bjerrum, Clausen and Duncan 1972; Bjerrum, 1972), dealing with the earth pressure problem and the bearing capacity and settlements of soft clays.

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