SOILS AND FOUNDATIONS Vol. 31, No. 1, 83-94, Mar. 1991
Japanese Society of Soil Mechanics and Foundation Engineering

RECOMPRESSION OF NORMALLY CONSOLIDATED CLAY AFTER CYCLIC LOADING

KAZUYA YASUHARA[i] and KNUT H. ANDERSEN[ii]

ABSTRACT

Cyclic loading may cause settlements in addition to the settlements due to static loads. This paper presents a study of recompression settlements occurring in normally consolidated clay when pore pressure generated by undrained cyclic loading dissipates. The study was based on 18 stress-controlled cyclic direct simple shear tests and 6 conventional consolidation tests on normally consolidated Drammen clay. The specimens were subjected to either one or several series of undrained cyclic loading and drainage. Afterwards, most of the specimens were consolidated further to a vertical stress three times the effective stress at start of cycling.

The study showed that undrained cyclic loading and drainage makes normally consolidated clay more resistant to later undrained cyclic loading.

The recompression index was influenced by cyclic loading, and for normally consolidated Drammen clay the settlements occurring when the cyclically induced pore pressure dissipates, may be calculated by a recompression index which is 1.5 times the recompression index from a conventional consolidation test.

The compression index of specimens which where subjected to undrained cyclic loading and then consolidated past the initial consolidation stresses decreases with increasing cyclically induced pore pressure and increasing cyclic shear strain at the end of cycling.

Key words : compression, normally consolidated clay, repeated load, simple shear test (**IGC :** D 7/D 5)

INTRODUCTION

Cyclic loading may cause settlements in addition to the settlements occurring under static loads. Examples of case histories where cyclic loading causes additional settlements include buildings and other structures subjected to earthquakes (e. g. Zeevaert, 1972 ; Ohmachi et al. 1988), offshore platforms subjected to sea storms (e. g. Eide and Andersen, 1984), foundation ssubjected to machine vibrations, and road pavements subjectde to traffic loads (Yamanouchi and Yasuhara, 1975).

Cyclic loading tends to cause volume reduc-

i) Professor, Department of Civil Engineering, Ibaraki University, 4-12-1 Nakanarusawa, Hitachi, Ibaraki 316.
ii) Section leader, Norwegian Geotechnical Institute, P. O. Box 40 Tåsen, N-0801 Oslo 8, Norway.
Manuscript was received for review on September 4, 1989.
Written discussions on this paper should be submitted before October 1, 1991, to the Japanese Society of Soil Mechanics and Foundation Engineering, Sugayama Bldg. 4 F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101, Japan. Upon request the closing date may be extended one month.

tion in the soil beneath a structure. If the excess pore water pressure cannot dissipate fast enough for the volume reduction to occur, a pore water pressure is generated. For structures on clay the cyclic loading event may have a relatively short duration compared to the time needed for the pore water pressure to dissipate. The conditions will then be undrained during the cyclic loading event, and excess pore pressures will be generated, leading to reduced effective stresses. The reduction in effective stresses together with a local redistribution of static stresses due to cyclic loading, may lead to increased shear strains and settlements of the structure during the undrained period.

During calmer periods between cyclic loading events, the generated pore pressure may get time to dissipate, and the effective stresses increase again. The volume reduction that tended to occur originally will then take place, and additional settlements result.

The cyclic loading and the corresponding unloading and reloading of effective stresses may also change the clay structure and influence the behaviour of the clay during later cyclic loading events.

The settlements that occur under undrained conditions are discussed by Eide and Andersen (1984), and by Andersen (1988). This paper concentrates on the following aspects :

a) The effect of undrained cyclic loading and drainage on behaviour during later undrained cyclic loading.

b) Settlements due to volumetric compression when the cyclically generated pore pressure dissipates.

c) The effect of cyclic loading on settlements due to volumetric compression during consolidation past the effective stresses at the start of cyclic loading.

The study was based on stress-controlled direct simple shear (DSS) laboratory tests on saturated undisturbed normally consolidated clay. In some of the tests the specimens were subjected to undrained cyclic loading, dissipation of the generated pore pressure, and subsequent consolidation past the initial effective stress. Other tests were run as ordinary consolidation tests to provide a reference to evaluate the effect of cyclic loading on the consolidation characteristics.

Cyclic DSS tests on clays followed by drainage have previously been carried out by Andersen et al. (1976), Suzuki (1984) and Ohara and Matsuda (1988). The last two studies used strain controlled conditions.

EXPERIMENTAL PROGRAMME

The tests were run on undisturbed plastic Drammen clay because a large number of cyclic tests have been carried out on this clay previously (Andersen et al., 1980, 1988). Average index properties are : natural water content $w_n=52\%$, liquid limit $w_L=55\%$, plasticity index $I_p=27$, and specific gravity $G_s=2.76$. The tests in the present study were run with the NGI DSS apparatus (Bjerrum and Landva, 1966). The specimens were circular with an area of 35 cm^2 and an initial height of 16 mm. The area of the specimen was kept constant by a wire-reinforced rubber membrane which constrained deformations in the radial direction. At the top and bottom of the specimen filter plates equipped with approximately 1 mm plong needles prevented sliding between the specimen and the filters. The filter and the drainage tubes wers saturated with water with a salinity of 25 g NaC/litre, corresponding to the average in-situ salinity of the pore water of Drammen clay. Cyclic loading was applied as a cyclic horizontal shear stress, τ_h, at the top of the specimen. The cyclic load was sinusoidal with 10 s period. Undrained conditions were simulated by keeping the volume of the specimen constant. As it could be assumed that the area of the sample did not change, constant volume was maintained by automatically adjusting the vertical effective stress, $\sigma^1 v$, on the sample to keep the sample height constant. The vertical effective stress was measured, and the stress paths of static and cyclic DSS tests in a $\tau_h-\sigma' v$ diagram could be established as illustrated in Fig. 1. The pore pressure generated by static

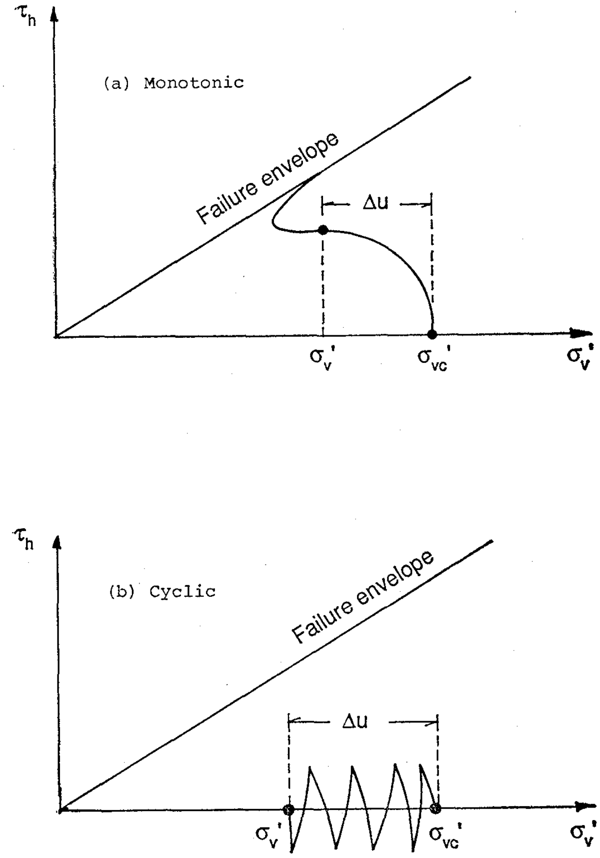

Fig. 1. Effective stress paths and pore pressures in DSS tests under constant volume condition

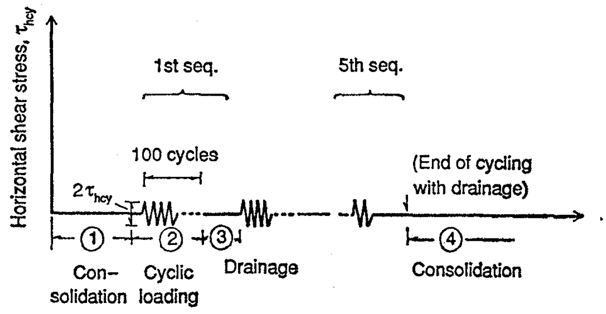

Fig. 2. Loading sequences of Type Bcyclic DSS tests

or cyclic loading can be derived from :

$$\Delta u=\sigma_{vc}'-\sigma_v' \qquad (1)$$

where σ_{vc}' = initial vertical effective stress
σ_v' = current vertical effective stress

The specimens are believed to be fully saturated, but since the tests are run as constant volume tests, full saturation is not critical for the quality of the test results.

The validity of using constant volume conditions to simulate undrained conditions in DSS tests on normally consolidated clays has been shown by Dyvik et al. (1987).

The pore pressure will vary continuously during cyclic loading. This paper concentrates on the permanent pore pressure, which is the pore pressure when the horizontal shear stress passes through zero.

The specimens were consolidated in several increments past their in situ preconsolidation stress ($p'_c \sim 125$ kPa) to a normally consolidated state at a vertical effective stress of $\sigma'_{vc}=392$ kPa for most of tests.

The cyclic shear stress applied is normalized with respect to the horizontal shear stress at failure in undrained static DSS tests, τ_{sf}.

The static tests were run strain-controlled with approximately 4.5% shear strain per hour.

In the cyclic tests the specimens were subjected to either one or five consecutive series of undrained cyclic loading and drainage. The duration of the drainage period was in gen-

Table 1. Testing conditions of DSS tests with undrained cyclic loading and drainage on normally consolidated Drammen clay

Test No.	σ_{vc}' (kPa)	$\tau_{h,cy}$ (kPa)	$\tau_{h,cy}/\tau_{sf}$	N (cycles)
2-11	392	46	0.52	706
2-12	〃	〃	〃	135
2-13	〃	〃	〃	750
2-13-2	〃	〃	〃	1,000
2-14	〃	〃	〃	1,270
2-15	〃	〃	〃	313
2-16	〃	37	0.44	500
2-17	〃	42	0.50	500
2-18	235	27	0.53	650
2-19	196	41	0.96	7
2-20	392	45	0.52	2,000
2-21	〃	〃	〃	74
3-2*	〃	39	0.47	1,000
3-3*	〃	34	0.41	2,100
4-2(B)	〃	56	0.65	500
4-3(B)	〃	42	0.50	500
4-4*(B)	〃	56	0.65	300
4-6(B)	〃	49	0.58	461

* undrained static test is carried out after drainage
(B) denotes type-B test
τ_{sf} is maximum horizontal shear stress in undrained static test

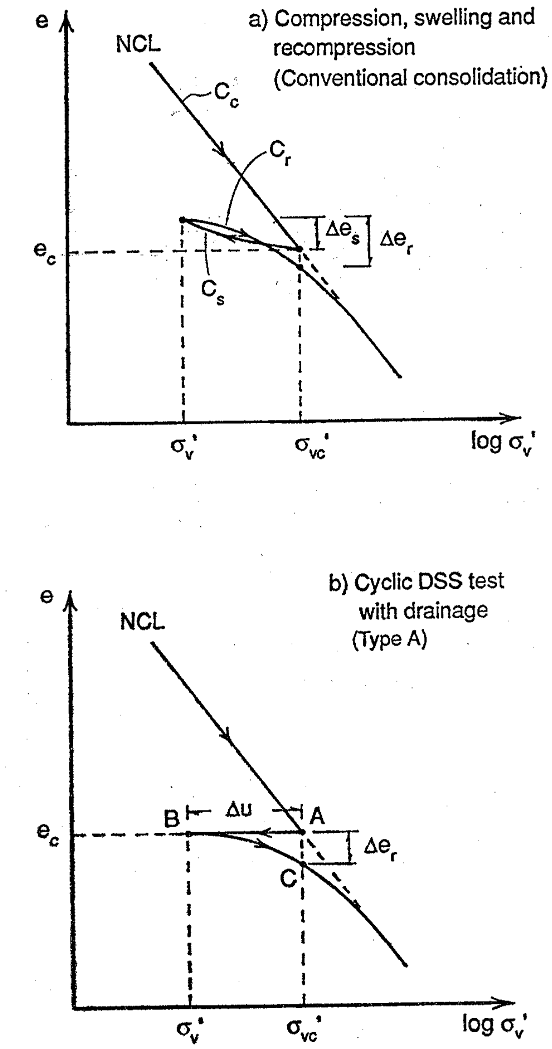

Fig. 3. Schematical illustration of e-log σ_v' relations in oedometer tests and in cyclic DSS tests followed by drainage (Type A)

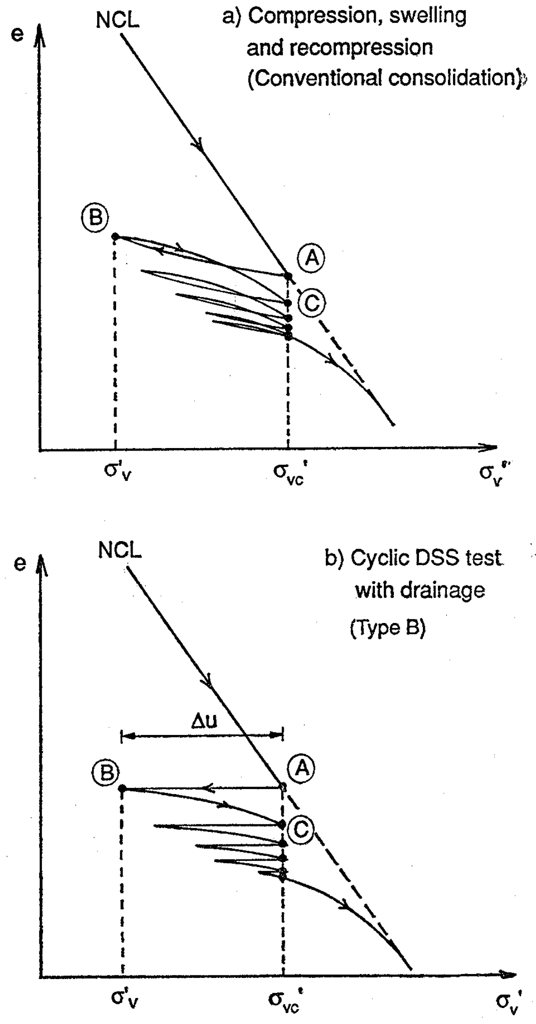

Fig. 4. Schematical illustration of e-log σ_v' relations in oedometer tests and in cyclic DSS tests followed by drainage (Type B)

eral about 60 min. Afthrwards, most of the specimens were consolidated in increments to a vertical effective stress three times the effective stress at start of cyclic loading (392 kPa). The tests were denoted Type A (one series) and Type B (five series). The loading sequence for the Type B tests is illustrated in Fig. 2. The effective stress variations and the volumetric deformations in the two types of cyclic DSS tests with drainage are illustrated schematically in Figs 3 (b) and 4 (b). The testing conditions for the various etsts are specified in Table 1.

The consolidation tests were also run in the DSS apparatus, but without applying any horizontal shear stresses. The purpose was to provide data to compare the consolidation characteristics of specimens with and without cyclic loading. In some of the consolidation tests the specimens were subjected to one unloading/reloading sequence from a vertical effective stress of 392 kPa to compare with the Type A cyclic tests (Fig. 3). In other consolidation tests the specimens were subjected to five unloading/reloading sequences to compare with the Type B cyclic tests (Fig. 4).

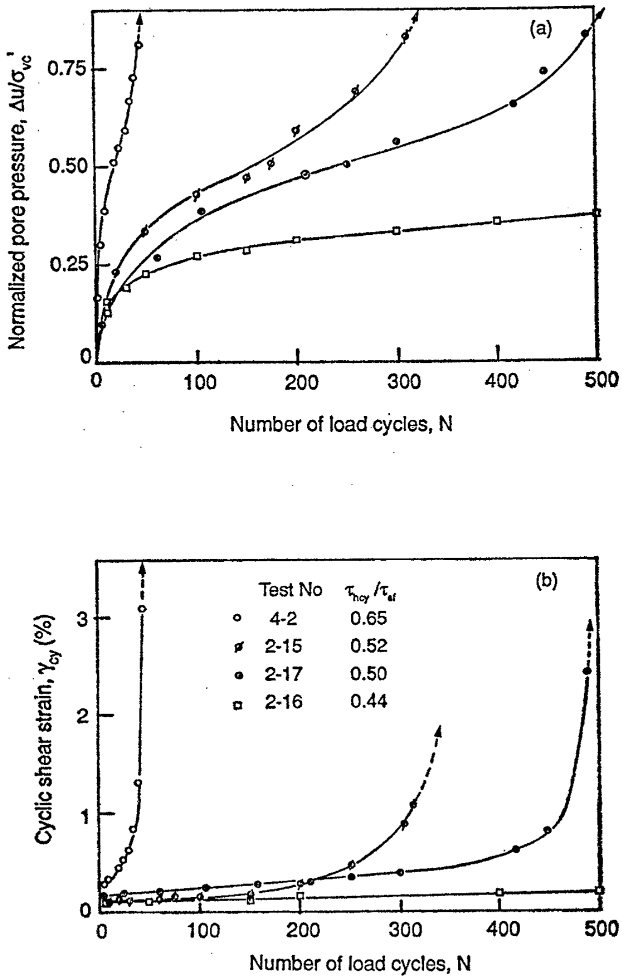

Fig. 5. Variations of pore pressure and cyclic shear strain with number of load cycles (Type A)

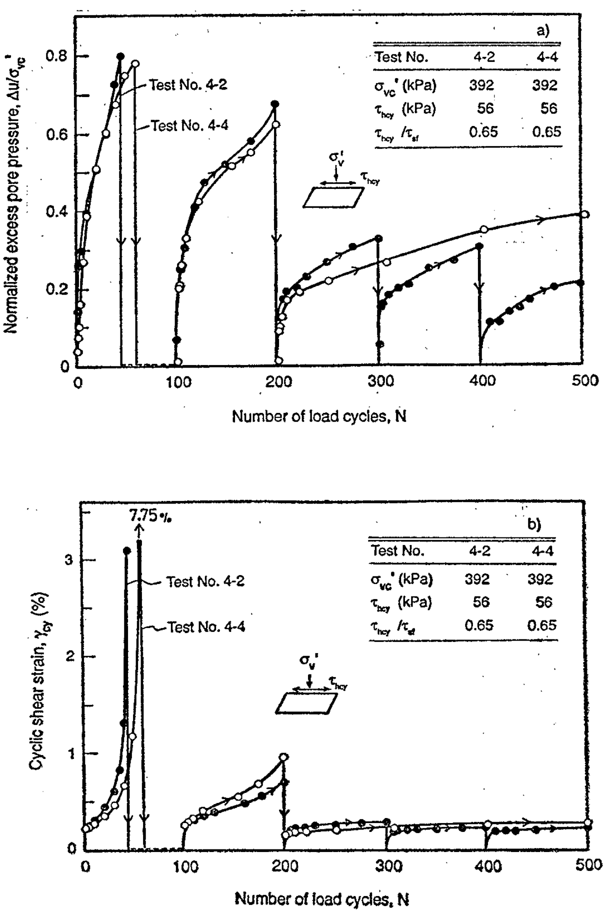

Fig. 6. Results from tests with several series of undrained cyclic loading and drainage (Type B)

BEHAVIOUR DURING UNDRAINED CYCLIC LOADING

Typical results of the undrained cyclic part of the Type A tests are shown in Fig. 5. The results from the first series of undrained cyclic loading from the Type B tests are also included. Both the cyclic shear strain (single amplitude) and the pore pressure increase with increasing number of load cycles. The increase in cyclic shear strain is relatively modest in the early part of the tests, but after a certain number of cycles, the cyclic shear strains begin to accelerate and large strains develop in the course of a relatively few additional cycles. The tests were generally stopped at a cyclic shear strain of 3%. If additional cycles had been applied, both the cyclic shear strain and the pore pressure would have increased further.

Typical results of the cyclic part of two Type B tests are shown in Fig. 6. The cyclic horizontal shear stress, τ_{hcy}, was 56 kPa in both tests, corresponding to 65% of the horizontal shear stress at failure in undrained static tests.

Test 4-2 was run by applying five series of undrained cyclic loading with drainage after each series. The cyclic shear strain increased rapidly during the ilrst series of cyclic loading. After 45 cycles the cyclic shear strain was 3%, and a cyclic failure was about to develop. The cycling was therefore stopped to avoid a complete failure, and drainage was permitted. The generated pore pressure after 45 cycles was about 310

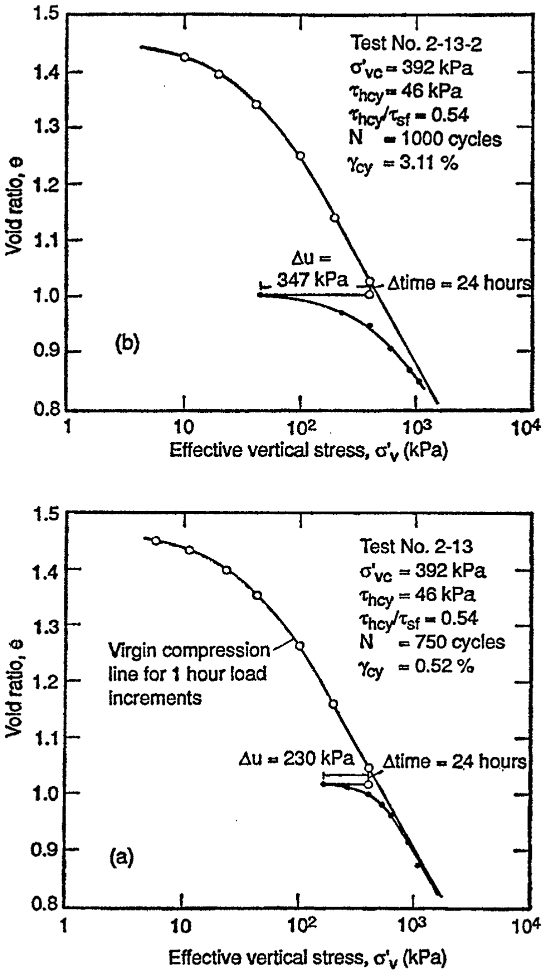

Fig. 7. ***e*-log σ_v' relations in Type A cyclic DSS tests**

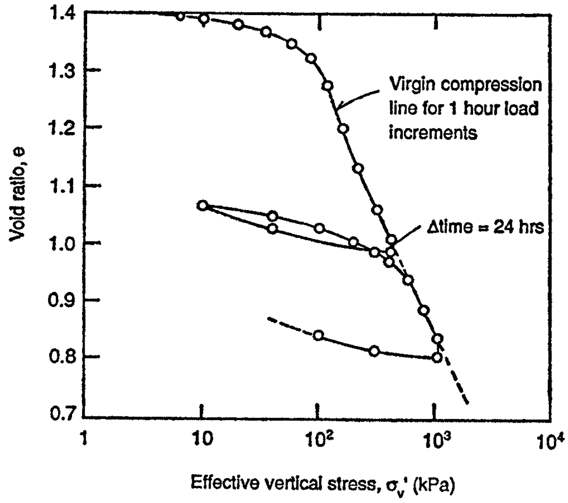

Fig. 8. Typical *e*-log σ_v' relation from consolidation test

kPa, corresponding to 79% of the initial vertical effective stress.

In the following series of undrained cyclic loading both the cyclic shear strains and the generated pore pressure were smaller that in the first series. One hundred cycles could be applied without reaching a cyclic shear strain of 3%, and at the end of the fifth series the cyclic shear strain was only 0.25% and the generated pore pressure only 20% of the initial vertical effective stress. The results of Test 4–2 show that cyclic loading accompanied by drainage makes normally consolidated clay more resistant to later undrained cyclic loading. This tendency was also observed in cyclic triaxial tests on remoulded Kaolinite clay by Matsui et al. (1977). The opposite effect has been observed in overconsolidated clays (Andersen et al., 1976; Yasuhara and Andersen, 1989), where cyclic loading accompanied by drainage made the specimens less resistant to later undrained cyclic loading.

The results from a second test, Test 4–4, are also presented in Fig. 6. In this test the third period contained 300 cycles without drainage. Another difference between the two tests was the duration of the drainage period which was longer in Test 4–4 (about 24 hours) that in Test 4–2 (about 60 minutes). The different drainage periods do not seem to influence the behaviour significantly, and the results of Test 4–4 confirm the conclusions drawn from Test 4–2.

POST-CYCLIC RECOMPRESSION

The pore pressure generated during undrained cyclic loading causes a reduction in the effective stresses in the clay. This corresponds to going from point A to point B in Fig. 3(b). With time, drainage occurs, and the excess pore pressure dissipates. The effective stresses increase again, and the clay will follow a reloading curve from point B to point C. At point C the effective stress has reached the same value as before start of cyclic loading.

The path BC has similarities with the re-

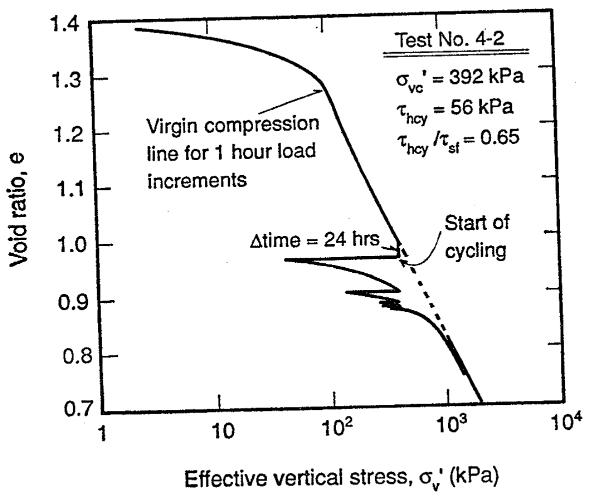

Fig. 9. *e*-**log** σ_v' **relations in Type B cyclic DSS test**

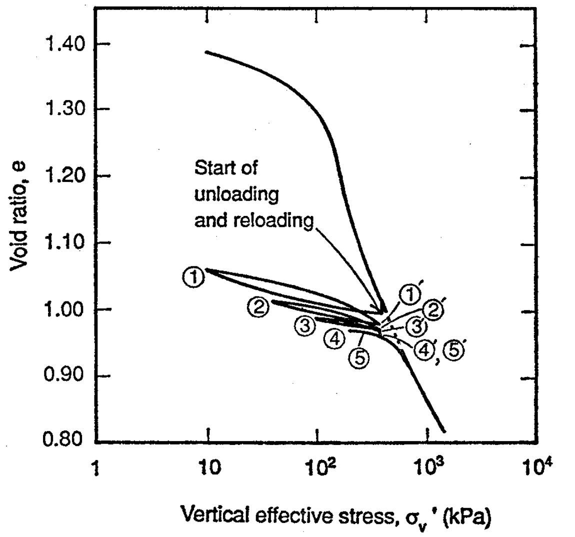

Fig. 10. *e*-**log** σ_v' **relation in consolidation test with five series of unloading and reloading**

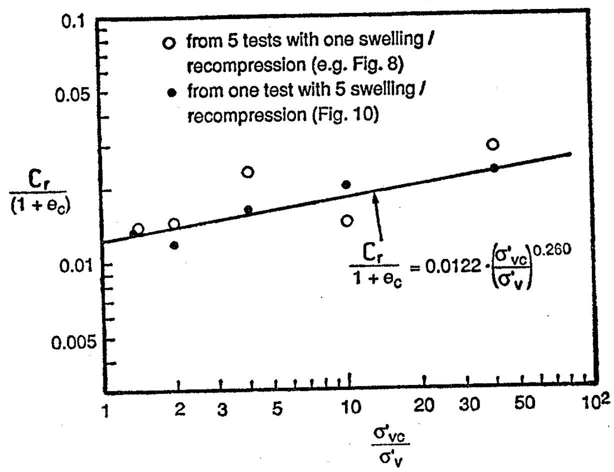

Fig. 11. Recompression index from consolidation tests

compression branch in a conventional oedometer test in which the specimens are subjected to compression, swelling and recompression. This is illustrated in Fig. 3 for the Type A tests and in Fig. 4 for the Type B tests. It therefore seems reasonable to investigate whether conventional oedometer tests with swelling and recompression can be used to predict the settlements due to cyclic loading.

The e-log σ_v' paths for two of the Type A cyclic tests are shown in Fig. 7. The cyclic shear stress has been the same in the two tests, but the number of cycles has been different.

Five consolidation tests with compression, swelling and recompression were performed. The e-log σ_v' path for one of the tests is shown in Fig. 8. This test was unloaded from 392 kPa to 9.8 kPa and reloaded to 1,175 kPa. The four other consolidation tests were unloaded to 39.2, 98, 196, and 295 kPa, respectively.

The e-log σ_v' path for one of the Type B cyclic tests is shown in Fig. 9. The cyclic shear strain and the pore pressure generation during the five undrained cyclic parts of this test were presented in Fig. 6. In Fig. 6 it was shown that the pore pressure generated by cyclic loading became smaller for each series of cyclic loading. This can also be seen in Fig. 9.

A consolidation test with several series of swelling and recompression was also performed to provide comparison with the Type B cyclic test. The result from this consolidation test is shown in Fig. 10. It was attempted to follow the vertical effective stresses measured in the cyclic DSS test in Fig. 9 to enable a direct comparison between the two types of tests.

The recompression index, C_r, from the consolidation tests is shown in Fig. 11. e_c is the void ratio at start of unloading. C_r is

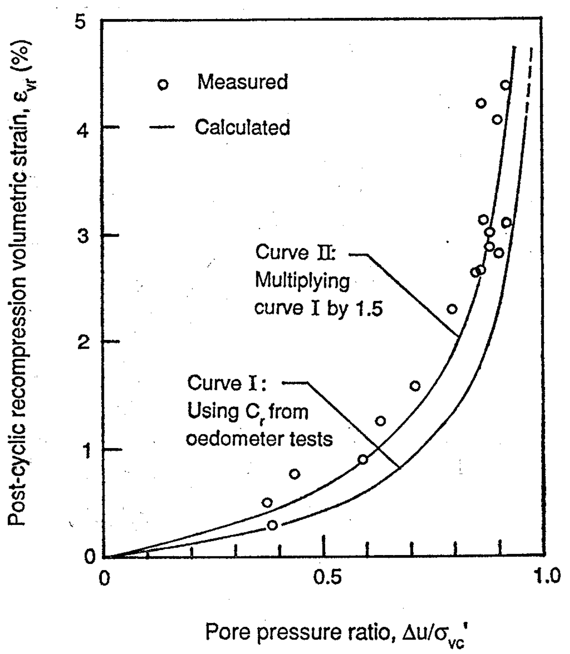

Fig. 12. Comparison between observed and calculated volumetric strain using recompression index C_r. (Tests Type A and first series of tests Type B)

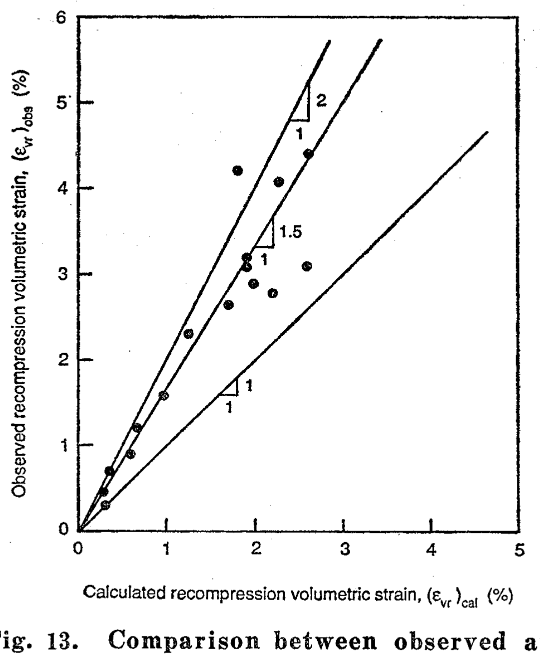

Fig. 13. Comparison between observed and calculated recompression volumetric strain using recompression index C_r (Tests Type A and first series of tests Type B)

calculated as the secant value from point B to point C in Figs. 3 (a) and 4 (a). The recompression index increases with increasing magnitude of unloading, i. e. with increasing value of the ratio σ_{vc}'/σ_v'. There is a tendency for the C_r–value to be slightly smaller in the consolidation test with 5 series of swelling and recompression than in the 5 tests with one series of swelling and recompression in each test.

The recompression volumetric strains in the cyclic DSS tests were calculated by :

$$\varepsilon_{vr}=\frac{\Delta e_{vr}}{1+e_c}=\frac{C_r}{1+e_c}\cdot\log\left(\frac{\sigma_{vc}'}{\sigma_v'}\right)$$
$$=\frac{C_r}{1+e_c}\cdot\log\left(\frac{1}{1-\Delta u/\sigma_{vc}'}\right) \quad (2)$$

where e_c is the void ratio at start of cyclic loading, Δu is the pore pressure due to undrained cyclic loading and C_r is the recompression index from oedometer tests in Fig. 11, determined for the value of σ_{vc}'/σ_v' corresponding to the values of $1/(1-\Delta u/\sigma_{vc}')$ observed in the cyclic DSS tests.

The recompression volumetric strains calculated using C_r from the average line drawn in Fig. 11, are compared to the measured values for the Type A cyclic DSS tests in Fig. 12. The calculated recompression strains are smaller than measured. There is some scatter in the measurements, but reasonable agreement between calculated and measured recompression strains is obtained by multiplying C_r by a factor α.

Thus, instead of Eq. (2), we have :

$$\varepsilon_{vr}\alpha=\frac{C_r}{1+e_c}\cdot\log\left(\frac{1}{1-\Delta u/\sigma_{vc}'}\right) \quad (3)$$

For Drammen clay we adopt the value of $\alpha=1.5$ which was determined from a regression analysis of the results in Fig. 13. The reason why the C_r from the consolidation tests must be corrected by a factor α, may be because the cyclic loading not only generates an excess pore pressure in the clay, but that it also disturbes the clay structure.

Since recompression volumetric strain due to dissipation of cyclically induced pore pressures is considered to be influenced by cyclic shear strains as well as cyclically

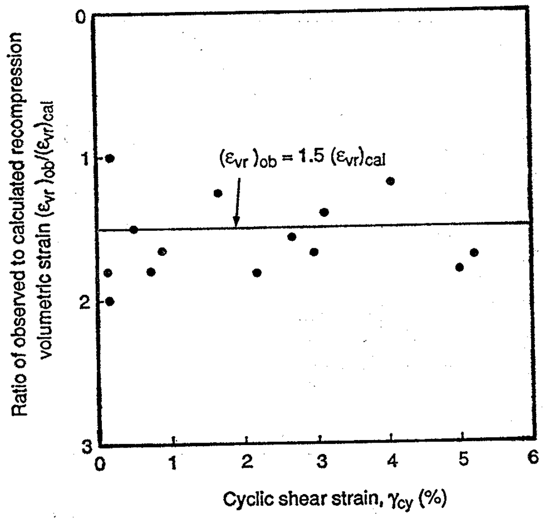

Fig. 14. Ratio of observed and calculated recompression volumetric strains as function of cyclic shear strain

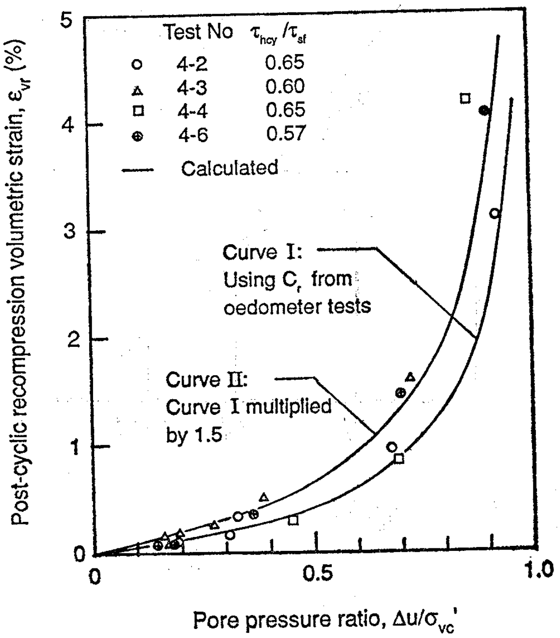

Fig. 15. Comparison between observed and calculated volumetric strain using recompression index C_r. (Test Type B)

induced pore pressures, the ratio of observed to calculated volumetric strains is plotted versus cyclic shear strain in Fig. 14. The results in Fig. 14 indicate that the ratio of $(\varepsilon_{vr})_{ob}/(\varepsilon_{vr})_{cal}$ is almost constant 1.5 independent of the amplitude of cyclic shear strain.

Tests similar to the Type A tests have been performed by Ohara and Matsuda(1988) on reconstituted Kaolinite. They interpreted their tests with an expression similar to Eq. (2), but used an index C_{dyn}, instead of the recompression index C_r. They found that the numerical value of C_{dyn} was between the swelling index, C_s, and the virgin compression index, C_c, from conventional oedometer tests. They did not, however, compare C_{dyn} to the recompression index, C_r, which is believed to be more relevant since the dissipation of the cyclically induced pore pressure is a reloading situation.

The recompression volumetric strains for the Type B tests with several series of undrained cyclic loading and drainage are presented in Fig. 15. The recompression volumetric strains during each series of undrained cyclic loading and drainage can also be calculated from Eq. (2) with a recompression index of $1.5 \cdot C_r$, independant of whether the clay has been subjected to undrained cyclic loading and drainage previously.

When the clay is subjected to several series of undrained cyclic loading and drainage, the result from the Type B cyclic DSS tests in Fig. 9 shows however, that the recompression volumetric strains will accumulate and the settlements for each series should be added to get the total recompression volumetric settlement. This is an important difference from conventional consolidation tests with several series of swelling and recompression. The result from the consolidation test in Fig. 10 shows that in the consolidation test, swelling occurs during the unloading part of each series of swelling and recompression. This will reduce the accumulated settlements compared to cyclic tests where the unloading occurs under undrained conditions and no swelling occurs.

COMPRESSIBILITY DURING CONSOLIDATION PAST THE INITIAL CONSOLIDATION RTRESSES

After the pore pressure due to cyclic loading had dissipated, in most of the cyclic

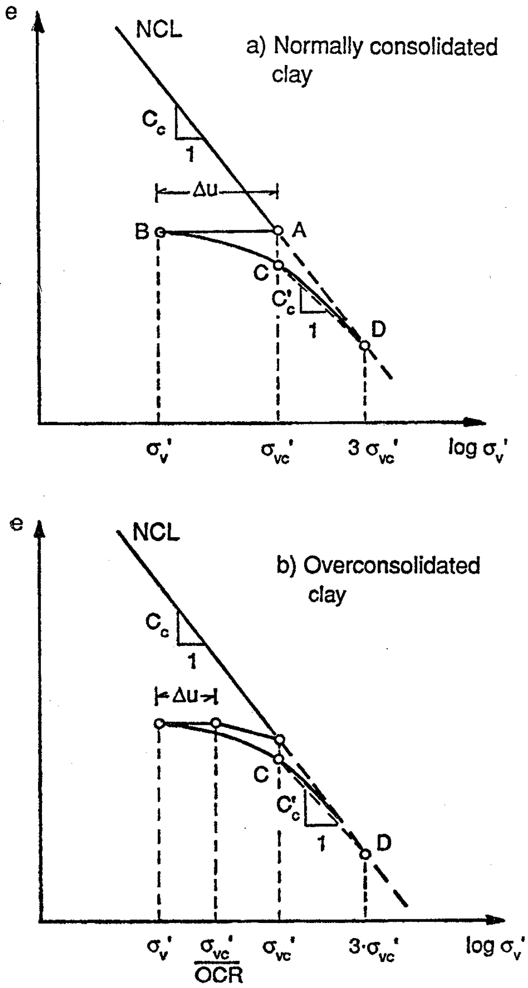

Fig. 16. Definition of compression index after cyclic loading C_c', for consolidation past the initial effective stress

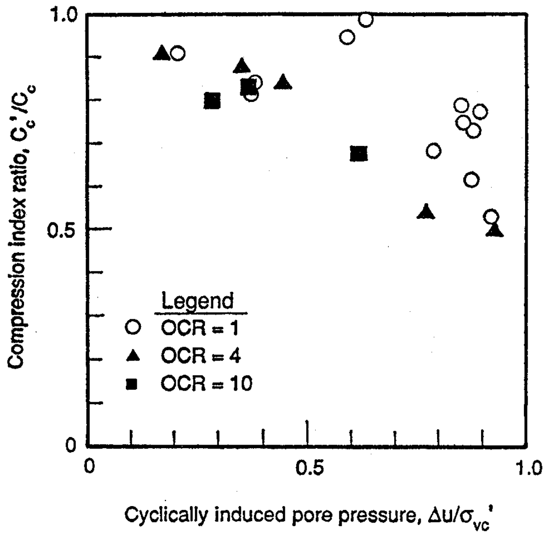

Fig. 17. Compression index ratio as a function of cyclically induced pore pressure

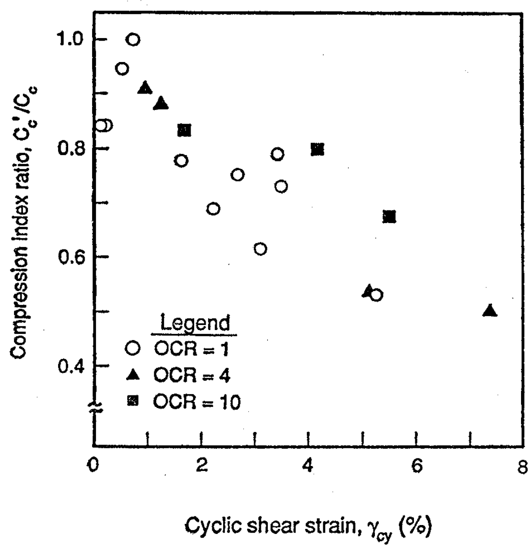

Fig. 18. Compression index ratio as a function of cyclic shear strain

DSS tests the specimens were consolidated to stresses higher that the stresses at start of cyclic loading. This was done to investigate the effect of cyclic loading on the compressibility for vertical loading past the initial effective stresses.

In the consolidation test in which the specimens had not been subjected to cyclic loading (Fig. 8), the recompression curve approached and joined the virgin compression line relatively soon after the vertical stresses passed the vertical stress prior to unloading. In specimens which have been subjected to cyclic loading, however, the recompression curve approaches the virgin compression curve much slower (Fig. 7). This is very similar to stress-compressibility characteristics of undisturbed clay undergoing disturbance due to remoulding (Schmertmann, 1956). The results from the two tests in Fig. 7 also indicate that the recompression line approaches the virgin compression line slower the higher the cyclically induced pore pressure has been. Similar behaviour was observed by Ohara and Matsuda (1988) in strain-controlled cyclic DSS tests on reconstituted

Kaolinite.

A more systematic interpretation of the effect of cyclic loading on the compressibility of Drammen clay for loading past the effective stresses at start of cyclic loading is presented in Figs. 17 and 18. These figures show the ratio between the compression index after cyclic loading, C_c', and the compression index in tests without cyclic loading, C_c. The value of C_c' depends on how far beyond σ_{vc}' the specimen is loaded. To have a well defined value of C_c' to work with, it was decided to use a value of C_c' determined from the secant between point C at σ_{vc}' and point D at $3\cdot\sigma_{vc}'$ (Fig. 16). C_c is the virgin compression index in the same stress range. In specific practical cases, one should keep in mind that the use of the secant value of C_c' from σ_{vc}' to $3\cdot\sigma_{vc}'$ may overestimate the settlements if the actual loading stops at an effective stress lower than $3\cdot\sigma_{vc}'$.

The ratio C_c'/C_c is plotted as a function of the pore pressure generated by cyclic loading in Fig. 17, and as a function of the cyclic shear strain at the end of cyclic loading in Fig. 18. The results from some tests on overconsolidated Drammen clay (Yasuhara and Andersen, 1989) are also included as reference.

The results in Figs. 17 and 18 clearly show that the compressibility for consolidation past the effective stresses at start of cyclic loading is influenced by cyclic loading.

The ratio of the compression indices, C_c'/C_c, decreases with increasing pore pressure (Fig. 17). There is a correlation between the cyclically induced pore pressure and the cyclic shear strain at the end of cyclic loading. It is therefore logical that C_c'/C_c decreases with increasing cyclic shear strain too (Fig. 18).

CONCLUSIONS

1. Undrained cyclic loading and drainage may make normally consolidated clay more resistant to later undrained cyclic loading.

2. Recompression settlements occur when cyclically induced pore pressures dissipate. The recompression index is influenced by the cyclic loading, and for normally consolidated Drammen clay the recompression settlements may be calculated by using a recompression index which is 1.5 times the recompression index from a conventional consolidation test. If the clay is subjected to several series of undrained cyclic loading and drainage, the total recompression settlements are the sum of the recompression settlements for all the series of undrained cyclic loading and drainage. This is different from a conventional consolidation test with several series of unloading and reloading, because swelling occurs during unloading in a conventional consolidation test. Due to this swelling, the total settlement will in a conventional consolidation test be smaller than the sum of the recompression settlements for the various reloading series.

3. The compression index for consolidation past the initial consolidation stresses is influenced by cyclic loading. The compression index decreases with increasing values of the cyclically induced pore pressure and with increasing values of the cyclic shear strain at the end of cyclic loading.

ACKNOWLEDGEMENT

The authors are grateful to the Royal Norwegian Council for Scientific and Industrial Research (NTNF) for providing support to the first author during his stay at the Norwegian Geotechnical Institute (NGI) in 1986 and 1988. The work reported in this paper forms part of the testing programme in which he was involved during his stay.

REFERENCES

1) Andersen, K. H., Brown, S. F., Foss, I., Pool, J. H. and Rosenbrand, F. W. (1976) : "Effect of cyclic loading on clay behaviour," Proc. of Conf. Design and Construction of Offshore Structures. Institution of Civil Engineers, London, pp. 75-79.

2) Andersen, K. H., Pool, J. H., Brown, S. F. and Rosenbrand, W. F. (1980) : "Cyclic and static laboratory tests on Drammen clay," Journal

of the Geotechnical Engineering Division, ASCE, vol. 106, No. GT 5, Proc. Paper 15434, pp. 499-529.

3) Andersen, K. H. (1988) : "Properties of soft clay under static and cyclic loading," Proc., Intern. Conf. on Engineering Problems of Regional Soils, August 1988, Beijing, China.
4) Bjerrum, L. and Landva, A. (1966) : "Direct simple shear tests on a Norwegian quick clay," Géotechnique, Vol. 16, No. 1, pp. 1-20.
5) Dyvik, R., Berre, T., Lacasse, S. and Raadim, B. (1987) : "Comparison of truly undrained and constant volume direct shear tests," Géotechnique, Vol. 37, No. 1, pp. 3-10.
6) Eide, O. and Andersen, K. H. (1984) : "Foundation engineering for gravity structures in the northern North Sea," Invited lecture. Int. Conf. on Case Histories in Géotech. Eng., St. Louis, Mo. Proc. Vol. IV, pp. 1627-1678.
7) Hyodo, M. et al. (1986) : "Deformation analysis of saturated clay deposit under low embankment highway subjected to traffic loading," Proc. 31 th Symp. of JSSMFE "Settlement Prediction and its Countermeasures in Soft Grounds," pp. 27-32 (in Japanese).
8) Matsui, T. et al. (1978) : "Effects of dynamic stress history on mechanical characteristics of saturated clay," Proc. JSCE, No. 257, pp. 41-51 (in Japanese).
9) Ohara, S. and Matsuda, H. (1988) : "Study on the settlement of saturated clay layer induced by cyclic shear," Soils and Foundations, Vol. 28, No. 3, pp. 103-113.
10) Ohmachi, T. et al. (1988) : "Damage due to the 1985 Mexico Earthquake and the ground conditions," Soils and Foundations, Vol. 28, No. 3, pp. 149-159.
11) Schmertmann, J. H. (1956) : "The undisturbed consolidation behaviour of clay," Trans. ASCE, vol. 120, pp. 1201-1233.
12) Suzuki, T. (1985) : "Settlement of saturated clays under dynamic stress history," Journal of the Japan Society of Engineering Geology, Vol. 25, No. 3, pp. 21-31 (in Japanese).
13) Yamanouchi, T. and Yasuhara, K. (1975) : "Settlement of clay subgrades after opening to traffic," Proc. 2 nd Australia and New Zealand Conf. on Geomechanics, pp. 115-120.
14) Yasuhara, K. and Andersen, K. H. (1987) : "Clay behaviour under long-term cyclic loading," Proc. 22 th Conf. JSSMFE, Vol. 1, pp. 635-638.
15) Yasuhara, K. and Andersen, K. H. (1989) : "Cyclically induced recompression behaviour of overconsolidated clay," Proc. 12 th ICSMFE, Vol. 1, pp. 485-488.
16) Zeevaert, L. (1972) : Foundation Engineering for Difficult Subsoil Conditions. N. Y., Van Nostrand Co. Ltd., pp. 250-255.