Liquefaction due to expansion of a cylindric cavity

Liquéfaction par expansion d'une cavité cylindrique

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SYNOPSIS

Laboratory tests were carried out on solid and hollow cylinder samples with a standard sand to investigate the liquefaction potential at different loading and boundary conditions. The aim of this investigation is to study the deformation behaviour and strength under stress conditions, similar to stress conditions at a pressuremeter test. So far, drained and undrained tests have been carried out at either monotonic, or cyclic loading. Further tests are also being carried out at partly drained conditions most likely to occur in the field.

INTRODUCTION

The liquefaction potential and cyclic behaviour of soils is generally investigated in the laboratory by triaxial or simple shear tests at repeated loading. Attempts were made to estimate the susceptibility of soils to liquefaction in the field from standard penetration tests, and the behaviour of cyclic loading from cyclic screw plate tests and cyclic pressuremeter tests.

The purpose of this investigation is to study the strength and stress-strain behaviour under conditions similar to stress conditions at a pressuremeter test. So far, drained and undrained tests have been carried out on hollow cylinder samples. The samples were loaded by increasing the cavity pressure monotonically, or changing it cyclically. For comparison, conventional triaxial tests have been carried out on solid samples. The test results showed that the deformation and pore pressure development at undrained loading is governed by the characteristic state. The characteristic stress level at zero volume change, or pore pressure change, is followed by a constant value of the stress ratio.

TEST PROCEDURE

The static/dynamic triaxial equipment described by Schwab (1984) was modified to also accept hollow cylinder samples. For this test series a standard quartz sand with a grain-size distribution between 0.3 mm and 0.8 mm and a uniformity coefficient of 1.3 was used. The samples were compacted in moulds at air-dry conditions at a dry density of 1.48 g/cm^3, which is close to the Proctor density at air-dry conditions. The samples were then saturated and a back pressure was applied until a B-coefficient of 0.98 was obtained within one second. After saturation, the samples were consolidated at an effective cell pressure, σ_c', of 50 kPa and then loaded, either stress or strain controlled. At the hollow cylinder tests, the cell pressure corresponded to the external and internal pressure, σ_e' and σ_i', respectively. At the conventional triaxial tests, the samples were loaded axially. The hollow cylinder samples were loaded at constant increase of the cavity volume, ΔV_i, or the cavity stress, $\Delta\sigma_i$ at the static tests and by periodically changing $\pm\Delta V_i$ or $\pm\Delta\sigma_i$ at cyclic tests. The cell pressure was kept constant during loading. Axial load, axial deformation, cavity volume or cavity stress, volume change or pore water pressure were recorded, respectively. The cyclic loading was triangular and the frequency was low, 10 mHz to 50 mHz.

TEST RESULTS

Figs.1a to 1d show typical results of a strain controlled undrained triaxial test of a dense sand. The loading rate was 25 kPa/min. The behaviour of the sample during loading and unloading can be explained by the characteristic state concept (Luong, 1980). Increasing the deviator stress, q= $\sigma_1 - \sigma_3$, the excess pore water pressure, u increased, indicating a compression of the sample. At a further increase of q, Δu became zero and decreased gradually due to dilatancy, Fig.1b. At Δu=0, i.e. at zero volume change, the soil is in the characteristic state (Luong, 1980). When a dense sample is loaded by increasing the deviatoric stress level, η= q/p', the soil passes from a contracting zone (sub-characteristic region) through the characteristic state at Δu=0 and $\eta = \eta_c$ into the dilating zone (sur-characteristic region), Fig.1d. During unloading u increased up to about the maximum value obtained during compression. In this test η_c was found to be 1.03 and η_{max} = 1.45.

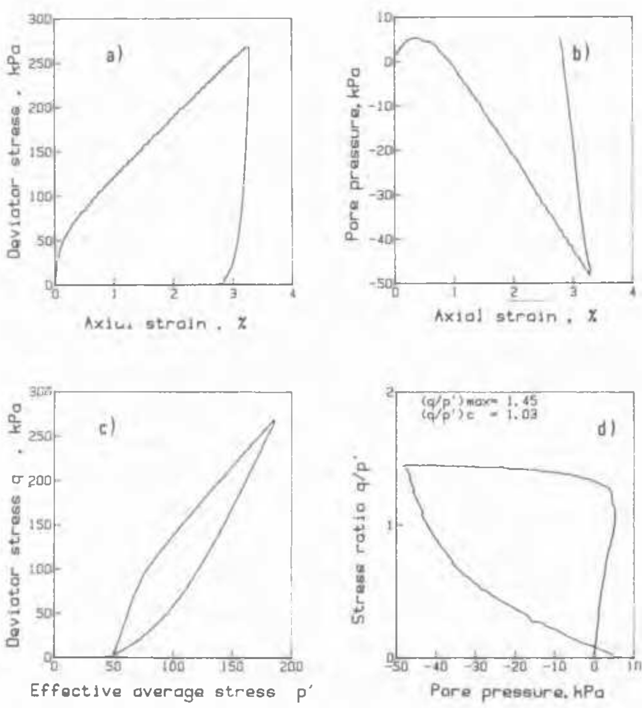

FIG.1 Results of a conventional undrained triaxial test at loading and unloading

Conventional drained and cyclic undrained triaxial tests with q positive, were carried out to determine η_c and η_{max} at different loading conditions. At the cyclic test, q was increased after each cycle. In the compressive zone, u increased during loading and was approximately constant during unloading. Considering the decrease of the mean stress p during unloading a decrease of u could be expected. But as u is constant, unloading is accompanied by positive volumetric strains. With an increasing number of cycles, the pore pressure was built up gradually and increased considerably, partly due to the increase of q and partly due to the decrease of p'. When η exceeded a value of 1.04, which is close to 1.03 obtained at a conventional test, Δu became zero and u thereafter decreased. For η_{max}, a value of 1.42 was found, corresponding approximately to the value obtained at the conventional test. At the drained test, the characteristic state at $\Delta\varepsilon_V$=0 was found at a stress level of η_c= 0.98 and η_{max} of 1.45.

Similar results were found from the tests with the hollow cylinder samples when the cavity pressure, $\Delta\sigma_i$, or the cavity volume, ΔV_i was increased monotonically. The development of u with increasing ΔV_i corresponded to that of conventional tests, cf Fig.2a and 1b. The different zones the soil is passing through during cavity expansion can be visualized on the plot of ΔV_i versus u, Fig.2b. In the con-

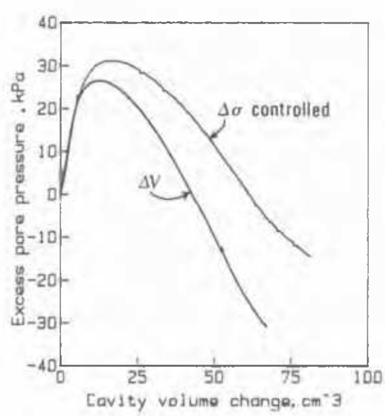

FIG.2a Development of excess pore pressure at a cavity expansion test

tracting zone, u increased with increasing. When approaching the characteristic state, u increased excessively even at constant $\Delta\sigma_i$ at this volume-controlled test. On further loading, the soil starts to dilate and u will decrease subsequently.

A characteristic parameter, similar to η can be defined as $\alpha = \Delta\sigma_i/\sigma_e'$. At this test, α_c was found to be 1.9, when $\Delta u = 0$. Beyond this characteristic state α was constant about 2.0 with decreasing u similar to η at a conventional test. At the cavity expansion test, appreciably higher values of u were obtained and the curve showed a pronounced peak. The different behaviour to the convenional tests is attributed to differences in the stress path and the intermediate principle stress.

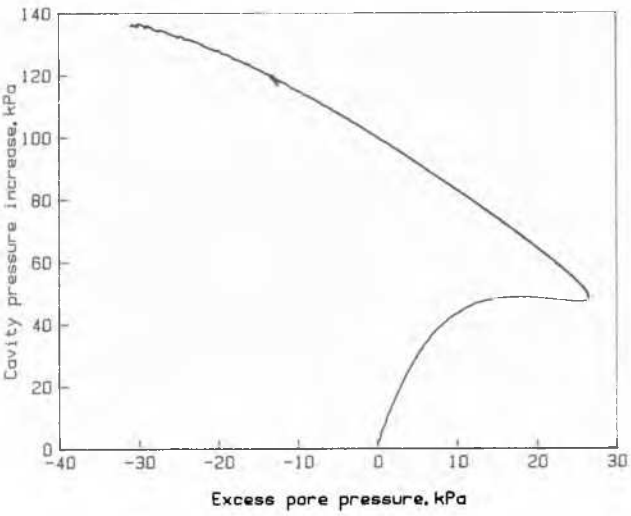

FIG.2b Cavity pressure against pore pressure at a cavity expansion test

LIQUEFACTION TESTS

Typical results of a conventional stress-controlled triaxial liquefaction test are shown in Figs. 3a, b and c. The cell pressure was 250 kPa and the back pressure 200 kPa. A triangular deviatoric alternating stress of Δq_{max}=13 kPa was applied at a frequency of 25 mHz. During the first stage of the test, u increased successively, while the axial strains were moderate, Fig.3a. u_{max} was obtained at $+q_{max}$ and u_{min} at $-q_{max}$. The shape of the pore pressure curve was triangular and the pore pressure amplitude almost constant. However, when u reached about 50% of σ_c' the shape of the pore pressure curve started to distort and the axial strain started to increase.
At u=0.9 σ_c' the pore pressure curve showed two maxima and two minima during one load cycle.

The maxima were reached at q=0, the first during compression from $-q_{max}$ to $+q_{max}$ and the second during extension from $+q_{max}$ to $-q_{max}$. The higher minimum between these two maxima was obtained at $+q_{max}$. The drop of u in this stage is small but still causing some gain in strength. The lower minimum was reached at $-q_{max}$. At this stage of the test, the maxima and minima did not increase or decrease further with the number of load cycles, while the axial strain increased considerably.

The behaviour of the soil at undrained cyclic loading can be explained by the characteristic state concept. As mentioned previously, a soil sample behaves contractive and dilatant at a deviatoric stress level lower and higher than the characteristic stress level η_c, respectively. At the beginning of the test, u is moderate and η is smaller than η_c. The decrease of u at $-q_{max}$ at this stage of the test is attributed to the decrease of the mean stress p below zero.

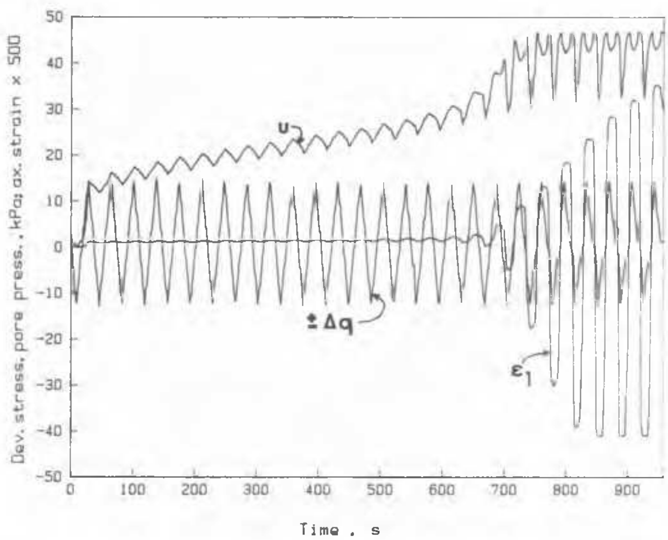

FIG.3a Results of a conventional triaxial stress-controlled liquefaction test

With increasing loading cycles u is increasing and η consequently approaches η_c.

As η_c is smaller in extension than in compression the characteristic state is first reached during extension. During compression from $-q_{max}$ to $+q_{max}$, the stress path passed from the negative dilating zone through $-\eta_c$ into the contracting zone and u reached a maximum at q=0. Increasing q further, the stress path passed $+\eta_c$ and the sample dilated, causing a decrease of u with a minimum at $+q_{max}$. During extension from $+q_{max}$ to $-q_{max}$, the stress path again passed through $+\eta_c$ into the contracting zone and u increased again to the second u_{max} at q=0. Decreasing q further to $-q_{max}$ the stress path passed through $-\eta_c$ in the dilating zone and u dropped to a minimum at $-q_{max}$. Also in the negative dilating zone an increase of u would be expected. The decrease of the mean stress however, caused a higher decrease of u and thus overwhelms a pore pressure increase due to dilatancy, so that the net pore pressure change in this stage is negative.

At q=0, u is a maximum and, consequently, the effective stress is minimal, causing large strains at low stress level, Fig.3b. The strain was higher in extension than in compression, which can be attributed to the lower effective stress during extension, Fig.3c, and to the gain of strength in the dilating phase of the test at compression.

Similar results were obtained at liquefaction tests with hollow cylinder samples, Figs.4a to 4d. The cell and cavity pressures were 200 kPa and the back pressure 150 kPa. The cyclic cavity pressure $\pm\Delta\sigma_{imax}$ was 22 kPa and was applied triangular at 25 mHz.

As in the case of conventional triaxial tests, u increased gradually during cyclic loading. The corresponding cavity volume change ΔV_i was moderate at this stage. u_{max} was obtained at $+\Delta\sigma_{imax}$ and u_{min} at $-\Delta\sigma_{imax}$. The pore pressure amplitude increased successively and was more pronounced than in the case of con-

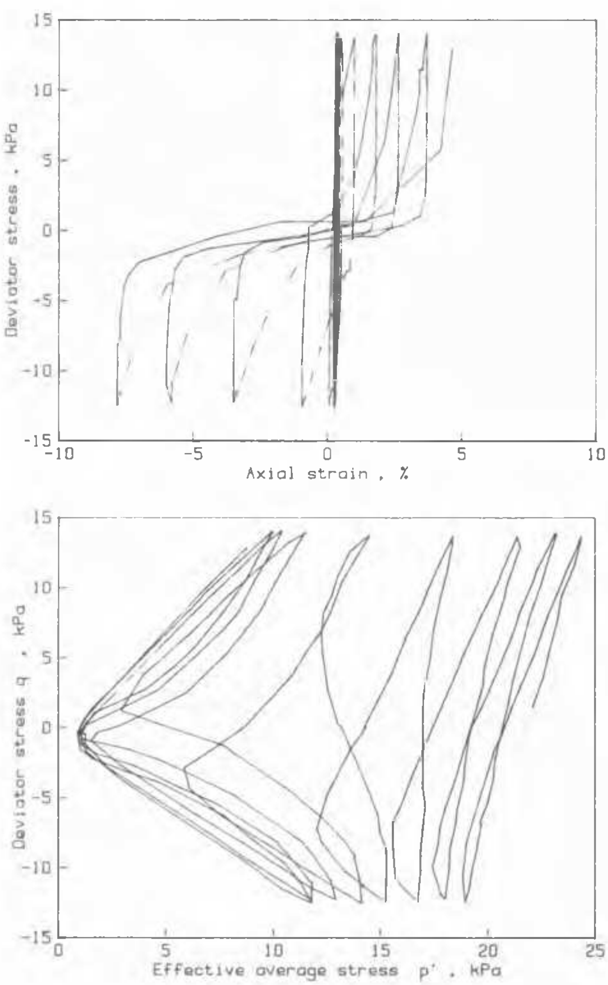

FIG.3b and c Relationship between $q - \varepsilon_1$, and $q - p'$ at triaxial liquefaction test

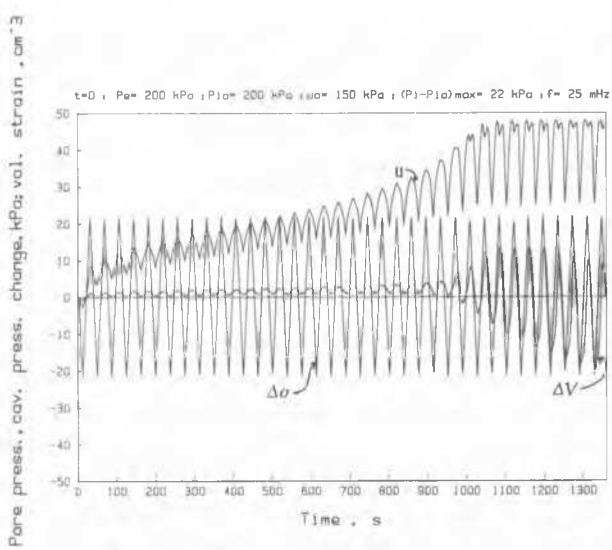

FIG.4a Results of soil liquefaction at stess-controlled cavity expansion

ventional tests. The cavity volume change amplitude also increased and was positive, as might be expected due to the fact that the soil was still in the contracting zone. At the 26th cycle, however, the shape of the pore pressure curve at u_{max} started to distort, indicating a pore pressure decrease at $+\Delta\sigma_{imax}$. At this point, volume change became negative for the first time. At the next cycle, the pore pressure drop at $+\Delta\sigma_{imax}$ can clearly be seen. The two maxima of u were obtained at $\Delta\sigma_i=0$. The shape of the pore pressure curve did not change during further loading, while the volume change amplitude increased considerably. It is interesting to note that $+V_{imax}$ decreased and $-V_{imax}$ increased. This can be explained by the gain of strength due to pore pressure decrease at $+V_{imax}$, as in the case of conventional cyclic triaxial tests, cf Fig.3a.

A volume change $\pm\Delta V_{imax}$ of 4 cm^3 was applied at the tests shown in Figs.4b, c and d. u increased with increasing load cycles, Fig.4b,

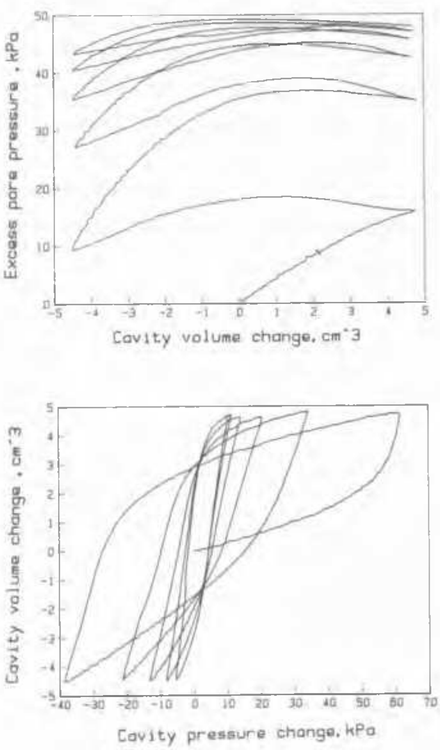

FIG.4b and c Relationship between u - $\pm\Delta V_{imax}$ and $\pm\Delta V_{imax}$ - u at cavity expansion

however, in a progressively decreasing rate. At the first loading u increased almost linearly up to u_{max} at $+V_{imax}$. Decreasing ΔV_i, u increased slightly up to $\Delta V_i=0$ and decreased then gradually to $-u_{max}$ at $-V_{imax}$. During compression, u increased significantly up to $\Delta V_i=0$, then remained constant and decreasing slightly thereafter when ΔV_i approached $+\Delta V_{imax}$. This behaviour can be explained by the characteristic state concept as described earlier.

The cavity pressure changes $\mp\Delta\sigma_i$ caused by $\pm\Delta V_i$ decreased with increasing load cycles, as might be expected, Fig.4c. With increasing u, the effective stresses decreased and consequently, the mobilized cavity pressure due to $\pm\Delta V_i$ decreased. A higher cavity pressure was necessary during compression than during extension, due to dilatancy.

The existence of a characteristic state can clearly be seen from Fig. 4d, where the cavity pressure was plotted against the pore water pressure. During the first load cycle in compression, the soil was in the contracting state and u increased. During unloading, u increased negligibly up to $\Delta\sigma_i$ = -25 kPa. Then u decreased significantly. At that point, the soil passed the characteristic state and entered the dilating zone. As $-\alpha_c$ is smaller than $+\alpha_c$, the characteristic state was first reached

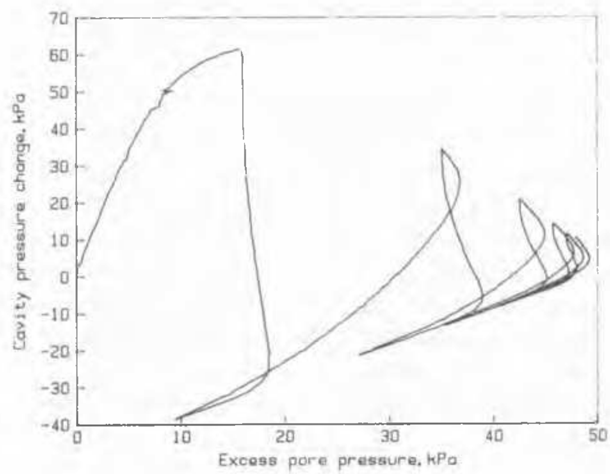

Fig.4d Relationship between $\pm\sigma_{imax}$ and u at cavity expansion

during extension, as during the conventional test. At this test $+\alpha_c$ was reached during compression at the 2nd cycle. In the dilating zone, α was constant as in the case of cavity expansion tests at monotonic loading. It was found from other tests that is independent of the number of load cycles, or loading amplitude.

SUMMARY AND CONCLUSIONS

To study the strength and deformation behaviour of sand at loading conditions similar to the stress conditions at a pressuremeter test, triaxial tests were carried out with hollow cylinder samples. The samples were loaded by changing the cavity pressure, or cavity volume, monotonically or cyclically. For comparison, conventional triaxial tests were carried out. So far, drained and undrained (liquefaction) tests have been performed.

The results have been analysed employing the characteristic state concept. During the conventional tests, it was found that the soil passed from the contracting zone into the dilating zone when the critical stress level, η of about 1.0 was reached. The characteristic stress level was first reached during extension. In the dilating zone, η approached a constant value of about 1.45. Similar results were obtained from the cavity expansion tests. It was found that the parameter $\alpha = \Delta\sigma_i/\sigma_e'$ plays a similar role as η at the conventional tests. Also, the result of the undrained triaxial liquefaction tests indicated the existence of a characteristic state. The pore pressure decreased when the soil entered the dilating zone. At $+q_{max}$, u approached a lower value between two maxima. The maxima occurred at q=0. At $-q_{max}$, u was a minimum. In the dilating zone during extension, no increase of u was obtained as would be expected, due to the decrease of u caused by a decrease of p. Liquefaction occurred at zero deviator stress when u was a maximum.

During the cyclic cavity expansion tests, the parameter α appeared to govern the strength and deformation behaviour, similar to η at conventional tests. Below a critical value of α, the soil is in a contracting zone and above in a dilating zone. After α_c was reached, α approached a constant value. A comparison of conventional triaxial tests and cavity expansion tests has shown that the soil behaves similarly during undrained monotonic and cyclic loading. The deformation behaviour is governed by the characteristic state η_c at conventional tests and α_c at cavity expansion tests.

Theoretical analyses have shown that α_c can be estimated from conventional undrained triaxial tests (Dormieux, 1984).

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