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Effect of Frequency and Vertical Stress on Cyclic Degradation and Pore Water Pressure in Clay in the NGI Simple Shear Device

Ahmad Reza Mortezaie, M.ASCE[1]; and Mladen Vucetic, M.ASCE[2]

Abstract: The effects of the frequency of cyclic shearing and vertical consolidation stress on the cyclic degradation and cyclic pore water pressure in normally-consolidated kaolinite clay having PI = 28 were investigated with the help of the cyclic strain-controlled Norwegian Geotechnical Institute (NGI) simple shear test. The testing program encompassed three cyclic shear strain amplitudes, $\gamma_c = 0.1, 0.25$, and 0.5%, two vertical effective consolidation stresses, $\sigma'_{vc} = 220$ and 680 kPa, and three frequencies, $f = 0.001, 0.01$, and 0.1 Hz. The results reveal that the cyclic degradation parameter, t, which measures the rate of cyclic degradation with the number of cycles, increases substantially with f and decreases with σ'_{vc}. If f is increased 10 times, t may increase by 20–50%. If σ'_{vc} is increased from 220 to 680 kPa, the t parameter may decrease by 20–38%. The cyclic pore water pressure normalized by σ'_{vc} decreases with f and σ'_{vc}. **DOI: 10.1061/(ASCE)GT.1943-5606.0000922.** © 2013 American Society of Civil Engineers.

CE Database subject headings: Clays; Cyclic loads; Degradation; Pore water; Loading rates; Shear modulus; Water pressure.

Author keywords: Clay; Cyclic loading; Simple shear device; Cyclic degradation; Pore water pressure; Loading frequency; Rate of loading; Secant shear modulus.

Introduction

When fully saturated soils are subjected to cyclic loading in undrained conditions, their stiffness and strength decrease with the number of cycles, N. Such a cyclic degradation is one of the most important phenomena in soil dynamics practice. Fully saturated sands are most susceptible to cyclic degradation. They can completely lose their stiffness and strength due to cyclic loading and liquefy. On the other end of the spectrum are fully saturated clays of high plasticity that under significant cyclic loading can lose only a fraction of their original stiffness and strength. The primary topic of this paper is how the cyclic degradation of clay is affected by the frequency of cyclic loading, f, and the effective vertical consolidation stress, σ'_{vc}. The secondary topic is how the cyclic pore water pressure is affected by f and σ'_{vc}. Knowing the extent of these effects and their trends is important in the evaluation of the dynamic properties of soil for the analyses of cyclic response of soil deposits. One of these properties and the main subject of this paper is the degradation parameter, t, which measures the rate of cyclic degradation with N [see Eqs. (1) and (2)]. The soil response analyses that employ parameter t are rather sensitive to its magnitude, and as shown in this paper, the magnitude of t can be significantly affected by f and σ'_{vc}. The results and trends presented in this paper can

therefore be helpful in the selection of f and σ'_{vc} in the experiments for the evaluation of soil dynamics properties. They can also be useful for the adjustment of available published properties of interest that are not obtained at relevant f and σ'_{vc}.

Cyclic degradation and cyclic pore water pressure can be investigated systematically based on the results of either strain-controlled or stress-controlled cyclic tests. In the cyclic stress-controlled test, the cyclic stress amplitude, τ_c, is maintained constant, while in the cyclic strain-controlled test, the cyclic strain amplitude, γ_c, is maintained constant. Cyclic degradation and pore water pressure changes with N are larger if these cyclic amplitudes are larger, and for a given soil, the cyclic amplitude and N are the most important factors influencing the degradation. Fundamental aspects of the cyclic degradation of clays in cyclic strain-controlled tests, as well as the cyclic stress-controlled tests, are described in the book by Ishihara (1996).

In the investigation described in this paper, the cyclic degradation was tested in the cyclic strain-controlled simple shear constant-volume equivalent-undrained tests conducted in the Norwegian Geotechnical Institute (NGI) type of direct simple shear device (DSS). The NGI-DSS device was originally introduced by Bjerrum and Landva (1966). In each test, a constant cyclic shear strain amplitude, γ_c, was applied in sinusoidal mode, while the variation of the cyclic shear stress, τ_{cN}, and the cyclic pore water pressure, Δu_N, were recorded with N. Here τ_{cN} is the average τ_c in the Nth cycle and Δu_N is the equivalent pore water pressure change, Δu, recorded at the end of the Nth cycle.

Only one soil, a laboratory-prepared kaolinite clay with plasticity index PI = 28 (liquid limit LL = 61 and plastic limit PL = 33), was tested. In the Casagrande plasticity chart, the soil plots practically on the A-line and can be classified as CH-MH, that is, it is between high-plasticity clay and high-plasticity silt. However, for the sake of simplicity and because it is composed of the pure clay minerals, this CH-MH soil is called just clay in this paper. The soil was tested in a series of 16 tests at three different γ_c, under two levels of σ'_{vc} and at

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three levels of f. Prior to the cyclic shearing, all 16 specimens were normally consolidated. Accordingly, the findings presented here are applicable to normally-consolidated soils of similar plasticity and classification which are sheared in the cyclic strain-controlled mode within the ranges of σ'_{vc} and f applied.

Comprehensive studies on the fundamental aspects of cyclic degradation and associated pore water pressure changes in clays began between 1975 and 1980 in connection with the design of foundations of offshore structures subjected to cyclic ocean wave loads. They include, among others, a survey of the cyclic triaxial stress-controlled loading behavior of a number of different soils by Lee and Focht (1976), a detailed investigation of the cyclic triaxial and simple shear stress-controlled behavior of a marine plastic clay (PI = 27) by Andersen (1976) and Andersen et al. (1980), and the investigation and modeling of the cyclic triaxial strain-controlled behavior of a soft high plasticity clay (PI = 49) by Idriss et al. (1976, 1978) and Moriwaki and Doyle (1978). The cyclic strain-controlled behavior studies were followed by investigations of the effects of the type of soil and overconsolidation ratio (OCR) on the magnitude and rate of cyclic degradation and cyclic pore water pressure (e.g., Vucetic and Dobry 1988; Tan and Vucetic 1989; Matasovic and Vucetic 1995), and the studies about the cyclic threshold shear strain for cyclic degradation and cyclic pore water pressure in clays (e.g., Vucetic 1994b; Hsu and Vucetic 2006; Tabata and Vucetic 2010). The results of the cyclic strain-controlled investigations enabled the development of a computer model for the cyclic response of soil deposits (Idriss et al. 1976) and modification of an existing computer model for the seismic response of soil deposits that incorporates the cyclic strain-controlled degradation and pore water pressure change (Matasovic and Vucetic 1993). In the meantime, site response computer models that incorporate the cyclic stress-controlled behavior were developed as well (e.g., Andersen et al. 1988; Andersen and Lauritzsen 1988).

The cyclic strain-controlled studies listed above revealed that, apart from the cyclic strain amplitude and the number of cycles, the cyclic degradation and cyclic pore water pressure in clays are significantly affected by the type of clay and OCR. Clays with higher PI and OCR experience smaller degradation. These studies also revealed that below certain small cyclic shear strain amplitudes, the cyclic degradation and the permanent cyclic pore water pressure change practically do not take place. The corresponding amplitudes are called the cyclic threshold shear strain for cyclic degradation and the cyclic threshold shear strain for cyclic pore water pressure. These threshold amplitudes range approximately between $\gamma_c = 0.01$ and 0.1% and generally increase with PI (Vucetic 1994b). To the writers' knowledge, however, in the studies mentioned in the above paragraph and similar studies on the cyclic strain-controlled behavior of clay, the effects of σ'_{vc} and f on the cyclic degradation and cyclic pore water pressure were not explicitly and systematically investigated. The same is more or less true for the cyclic stress-controlled behavior, in which domain there is also a lack of systematic data on the effects of σ'_{vc} and f. Consequently, the test results and their analysis presented in this paper are a contribution to the body of knowledge on the complex cyclic behavior of clays.

Cyclic Degradation and Pore Water Pressure in Cyclic Strain-Controlled Test

In Figs. 1–4 are the results of two cyclic strain-controlled tests conducted at $\gamma_c = 0.5\%$. Figs. 1 and 2 present the results of a test with $\sigma'_{vc} = 691$ kPa and $f = 0.001$ Hz, while Figs. 3 and 4 present the results of a test with $\sigma'_{vc} = 216$ kPa and $f = 0.1$ Hz. In the titles of

Figs. 1 and 2 the label "Hi-Sl" means high vertical stress and low frequency, while the label "Lo-Fa" in Figs. 3 and 4 means low vertical stress and relatively high frequency (see Table 1). In other words, in the first Hi-Sl test σ'_{vc} was high and f was very low, while in the second Lo-Fa test the situation was reversed: σ'_{vc} was relatively low and f was relativity high. The results in Figs. 1–4 are provided to show (1) what the typical test results look like in detail, (2) how the cyclic degradation is manifested, and (3) what the general effects of σ'_{vc} and f might be.

As shown in Figs. 1 and 3, in both tests the cyclic stress amplitude, τ_{cN}, decreased with N indicating the cyclic degradation, while the equivalent pore water pressure, Δu, cyclically increased. The cyclic degradation is also evident from the stress-strain loops in Figs. 2 and 4. The loops clearly exhibit the reduction of the secant shear modulus, $G_{SN} = \tau_{cN}/\gamma_c$, with N. For convenience, in Fig. 4 the modulus G_{SN} normalized to σ'_{vc} is indicated for cycles 1 and 5.

In the cyclic strain-controlled mode, the cyclic degradation with N can be quantified with the degradation index, δ

$$\delta = \frac{G_{SN}}{G_{S1}} = \frac{\tau_{cN}/\gamma_c}{\tau_{c1}/\gamma_c} = \frac{\tau_{cN}}{\tau_{c1}} \qquad (1)$$

and the degradation parameter, t

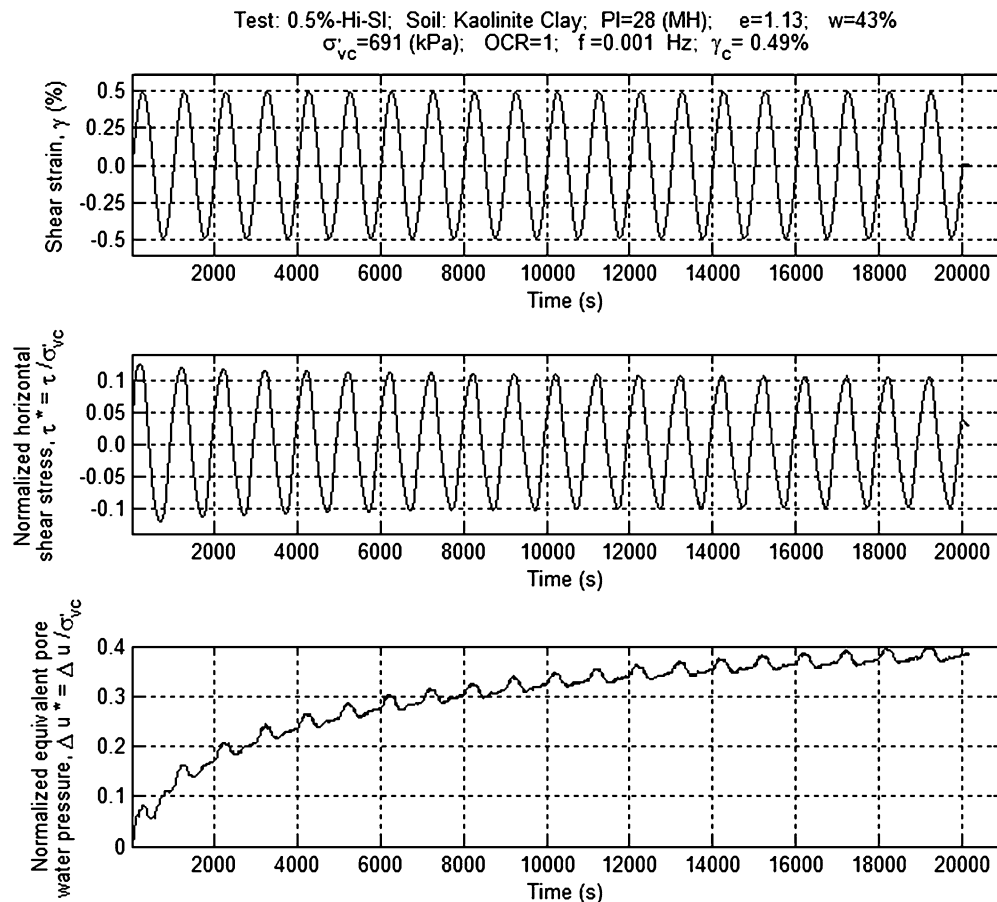$$t = -\frac{\log \delta}{\log N} \qquad (2)$$

The index δ and parameter t were introduced by Idriss et al. (1978) in the context of the evaluation of the cyclic degradation of marine clay deposits underlying offshore structures for oil explorations. Idriss et al. (1978) found that for many clays the relationship δ versus N, in a log-log format is approximately a straight line. The slope of this line is the degradation parameter, t, which for a given γ_c describes the rate of cyclic degradation with N. For overconsolidated clays, δ versus N in a log-log format is also approximately a straight line (Vucetic and Dobry 1988), while for sands it is typically curved (Dobry et al. 1982; Mortezaie 2012).

Note that cyclic degradation in the cyclic stress-controlled mode can also be expressed with a degradation index, δ^*, which also describes the reduction of the secant shear modulus with N. Its basic definition is the same

$$\delta^* = \frac{G_{SN}}{G_{S1}} = \frac{\tau_c/\gamma_{cN}}{\tau_c/\gamma_{c1}} = \frac{\gamma_{c1}}{\gamma_{cN}} \qquad (3)$$

Here, τ_c is constant while the cyclic shear strain amplitude, γ_{cN}, varies with N. The index δ^* decreases with N because γ_{cN} increases with N.

The difference in the pore water pressure change and cyclic degradation between the two tests described in this section is illustrated in Fig. 5. The variation of the pore water pressure with N is displayed as the equivalent pore water pressure recorded at the end of each strain cycle, which is commonly called the cyclic pore water pressure, Δu_N. Furthermore, Δu_N is presented in the normalized form with respect to σ'_{vc}, $\Delta u_N^* = \Delta u_N/\sigma'_{vc}$. While in Fig. 5 the difference in the variation of Δu_N^* with N is relatively small and it cannot be said how σ'_{vc} and f affect it, the difference in the cyclic degradation with N is significant. In the first test with $\sigma'_{vc} = 691$ kPa and $f = 0.001$ Hz, the degradation parameter $t = 0.065$, while in the second test with smaller $\sigma'_{vc} = 216$ kPa and higher $f = 0.1$ Hz, the

Test: 0.5%-Hi-SI; Soil: Kaolinite Clay; PI=28 (MH);   e=1.13;   w=43%
σ'_{vc}=691 (kPa);   OCR=1;   f =0.001 Hz; γ_c= 0.49%

Fig. 1. Variation of strain, stress, and pore water pressure with time in a cyclic strain-controlled test on kaolinite clay with $\sigma'_{vc} = 691$ kPa and $f = 0.001$ Hz
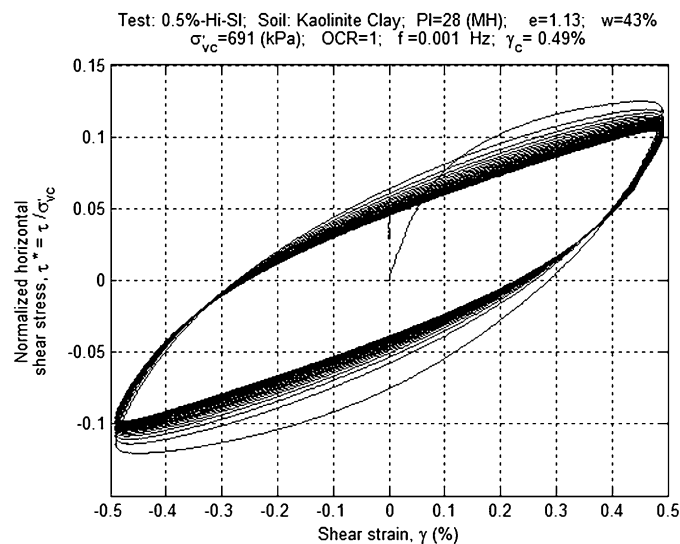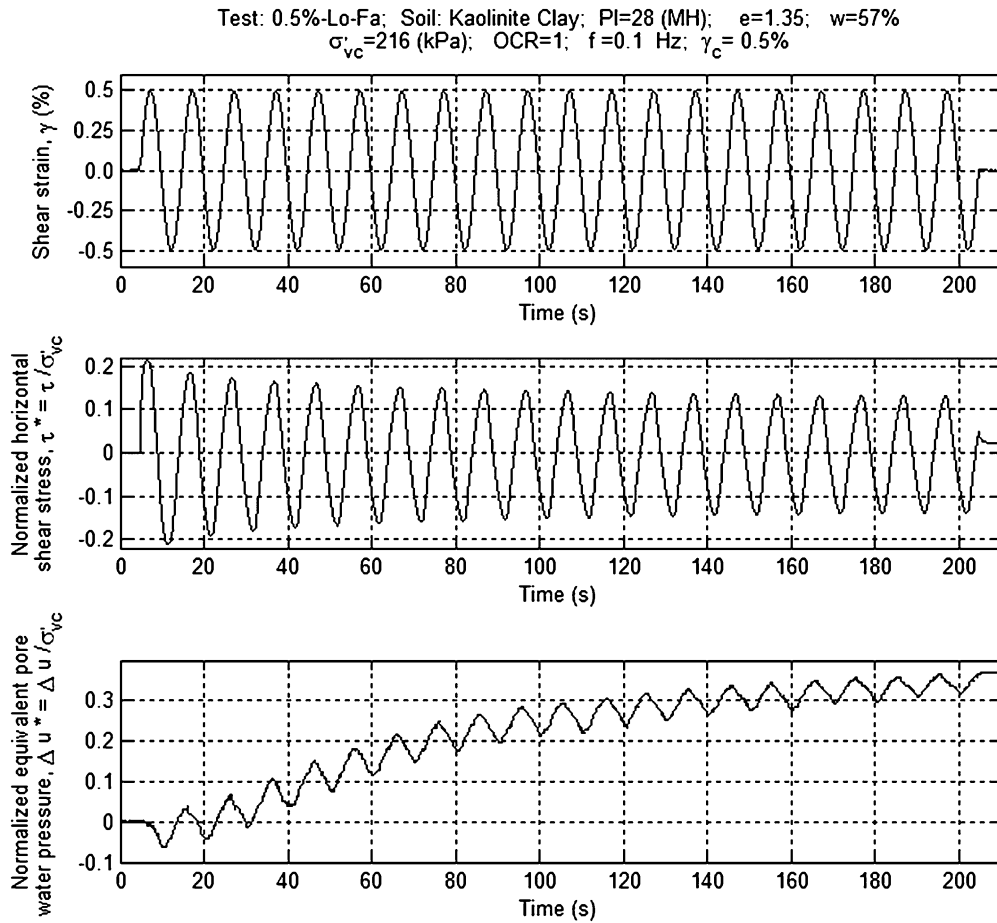

Fig. 2. Stress-strain loops of a cyclic strain-controlled test on kaolinite clay with $\sigma'_{vc} = 691$ kPa and $f = 0.001$ Hz

parameter $t = 0.157$. This indicates that the degradation is substantially affected by either σ'_{vc} or f, or perhaps both of them. In fact, as shown in the following sections, the degradation is considerably larger if σ'_{vc} is smaller and f is higher, and these two particular trends are the primary subject of this paper.

Soil Preparation, Testing Procedure, and Testing Program

The kaolinite clay was prepared in a consolidation tank from the slurry made of a commercially available kaolinite powder. The thin slurry was slowly consolidated into a kaolinite clay cake 180 mm in diameter and about 60 mm high. The consolidation was very slow and resembled a consolidation of natural clay. The cake was cut vertically in three segments. From each segment, a short cylinder specimen, 66.7 mm in diameter and 18 to 19 mm high, was trimmed in the NGI-DSS trimming apparatus. In the same apparatus, the specimen was then placed between the top and bottom caps with built-in porous stones, enclosed tightly in the NGI-DSS wire-reinforced rubber membrane, and secured on its pedestal. The pedestal with the specimen setup was then mounted in the NGI-DSS apparatus where the specimen was consolidated and cyclically sheared. The consolidation was performed in a typical manner in five vertical stress increments for the tests with $\sigma'_{vc} \approx 220$ kPa and seven increments for the tests with $\sigma'_{vc} \approx 680$ kPa. Each increment lasted 2 h, which was more than sufficient for the completion of primary consolidation and some secondary compression. The specimen was left overnight under the last σ'_{vc} increment to consolidate substantially into the secondary compression stage prior to cyclic shearing. It should be mentioned that the vertical compression curves obtained under lateral confinement with the NGI wire-reinforced membrane look very similar to the curves obtained in the standard consolidation device with metal ring confinement (Mortezaie 2012). This means

Fig. 3. Variation of strain, stress, and pore water pressure with time in a cyclic strain-controlled test on kaolinite clay with $\sigma'_{vc} = 216$ kPa and $f = 0.1$ Hz
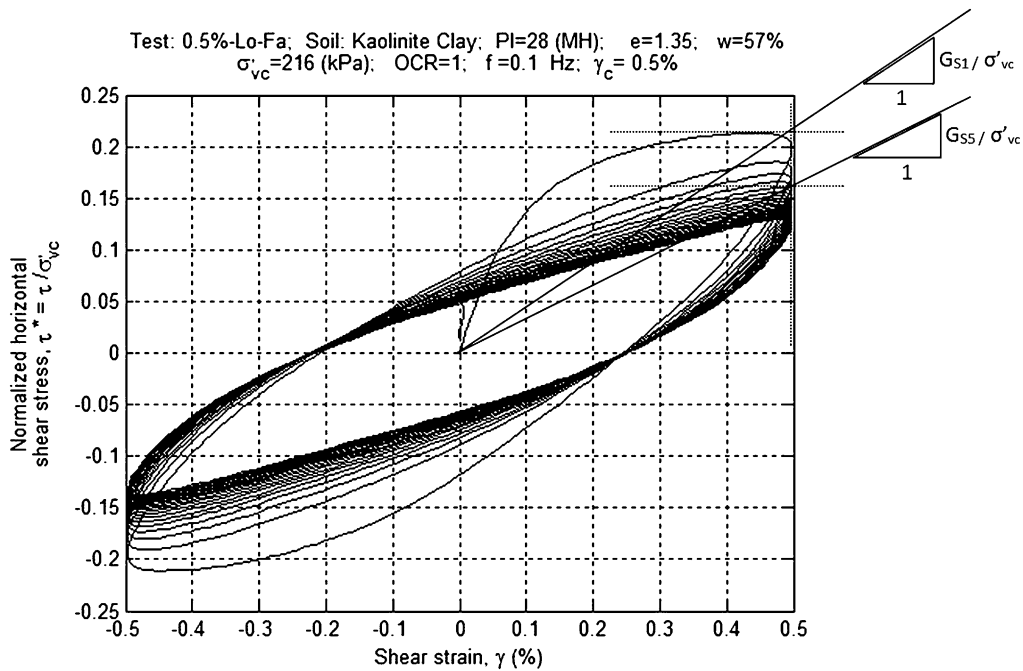
Fig. 4. Stress-strain loops of a cyclic strain-controlled test on kaolinite clay with $\sigma'_{vc} = 216$ kPa and $f = 0.1$ Hz

Table 1. Testing Program of 16 Tests on Normally Consolidated Laboratory-Prepared Kaolinite Clay with PI = 28

		Fastest				Medium				Slowest			
γ_c (%)	σ'_{vc}	$f = 0.1$ Hz	Void ratio, e	σ'_{vc} (kPa)	t	$f = 0.01$ Hz	Void ratio, e	σ'_{vc} (kPa)	t	$f = 0.001$ Hz	Void ratio, e	σ'_{vc} (kPa)	t
0.1	Low	0.1%-Lo-Fa	1.32	219	0.060	0.1%-Lo-Me	1.33	215	0.041	0.1%-Lo-Sl	1.32	220	0.029
	High	0.1%-Hi-Fa	1.07	677	0.042	0.1%-Hi-Me	1.09	691	0.033	0.1%-Hi-Sl	1.08	680	0.022
0.25	Low	0.25%-Lo-Fa	1.34	217	0.109	0.25%-Lo-Me	1.39	217	0.087				
	High	0.25%-Hi-Fa	1.11	684	0.083	0.25%-Hi-Me	1.13	681	0.070				
0.5	Low	0.5%-Lo-Fa	1.35	216	0.157	0.5%-Lo-Me	1.36	217	0.133	0.5%-Lo-Sl	1.40	220	0.092
	High	0.5%-Hi-Fa	1.07	688	0.098	0.5%-Hi-Me	1.12	689	0.082	0.5%-Hi-Sl	1.13	691	0.065
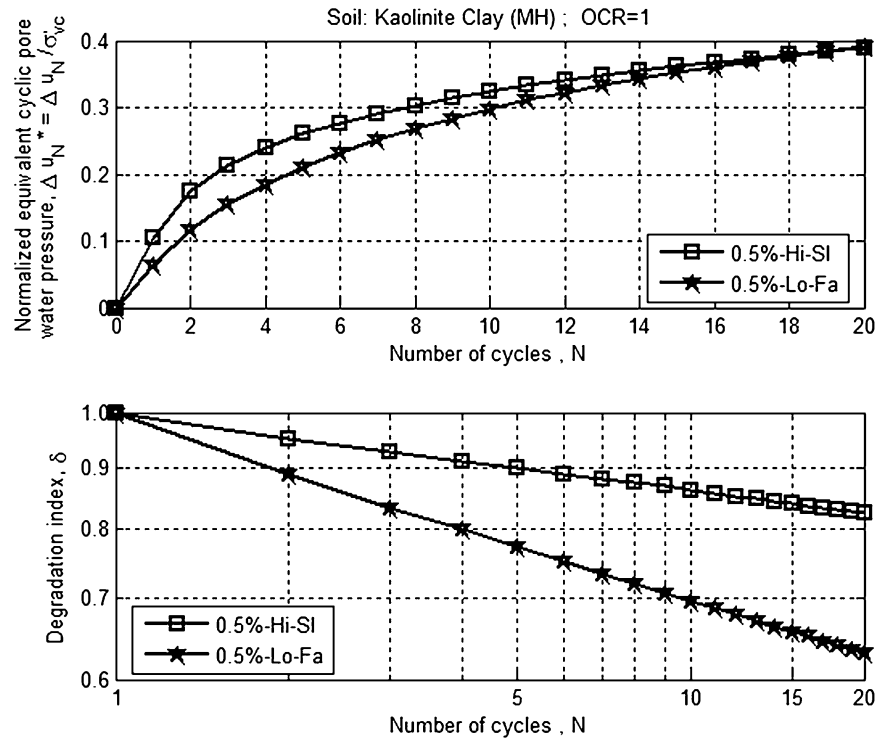
Fig. 5. The difference in cyclic degradation and pore water pressure change in two tests with different σ'_{vc} and f

that the specimens were consolidated essentially under the K_0 condition.

Every test was a typical NGI-DSS constant-volume equivalent-undrained test. Considering that the specimen volume in a truly undrained test on fully saturated soil is constant, in the constant-volume equivalent-undrained DSS test the undrained conditions during cyclic shearing are simulated by maintaining the specimen volume constant. During the shearing, the volume is maintained constant by varying vertical stress while the specimen drains are open and the actual pore water pressure in the specimen is zero. Under such conditions, the change of vertical stress required to maintain constant volume is equivalent to the pore water pressure change, Δu, that would have developed in a truly undrained test (Dyvik et al. 1987). That is why Δu is called here the equivalent pore water pressure. It should be added that in such a test, the vertical stresses applied via the top specimen cap, as well as the horizontal stresses generated by the confinement of the wire-reinforced rubber membrane, are always the effective stresses.

Considering that radial deformations of the specimen confined in a properly selected wire-reinforced rubber membrane are negligible, in the present investigation, the volume of the specimen was maintained constant by keeping the specimen height constant. This is a standard procedure for maintaining constant volume at the NGI

laboratory and other laboratories conducting typical NGI-DSS tests (Iversen 1977). The specimen height was kept constant automatically with the originally provided mechanical closed-loop vertical loading system, which has an electrical motor and a sensitive vertical displacement transducer. Because this system was originally intended for static testing and has a relatively slow response, it cannot keep the specimen height constant during high-frequency cyclic tests. The highest frequency in this investigation had to therefore be carefully selected. After some experimentation, it was concluded that for reliable measurements of the equivalent pore water pressure, the specimen height must be corrected to its constant value at least four times during each straining cycle. This means that the detailed fluctuation of Δu during a single cycle was not perfectly captured, but the constant height was properly maintained on a cycle-to-cycle basis. This also explains why the pattern of the variation of Δu in a single cycle in Fig. 1 is not exactly the same as that in Fig. 3. The highest frequency that satisfies the above criterion for maintaining the specimen height was devised and verified with the help of the NGI-DSS cyclic tests on sand that is not susceptible to the rate of straining and frequency effects. The variation of Δu_N in these tests on sand was practically the same for the wide range of frequencies up to almost 0.5 Hz. At 0.5 Hz and above, however, the

closed loop system could not catch up with the changes of the specimen height due to the cyclic shearing, especially during the first few cycles when the changes are typically the largest. In conclusion, the highest frequency selected was 0.1 Hz (Table 1), which is much lower than the critical frequency of 0.5 Hz.

After the testing, the test records were analyzed and processed to eliminate the false loads and deformations inherent to the NGI-DSS testing, as described in Mortezaie and Vucetic (2012).

The program of 16 tests is summarized in Table 1. It encompasses three levels of γ_c, 0.1, 0.25, and 0.5%, two levels of σ'_{vc} of approximately 220 and 680 kPa, and three relatively low frequencies, 0.001, 0.01, and 0.1 Hz. The program also included two more tests with $\gamma_c = 0.8\%$ and $f = 0.02$ Hz to verify the repeatability of testing, which has shown to be very good (see Mortezaie 2012). The lower level of $\gamma_c = 0.1\%$ was selected to be somewhat above the cyclic threshold shear strain for cyclic degradation, which for the kaolinite clay is approximately 0.013% (Mortezaie and Vucetic 2012). According to USAEC (1972) the upper level of $\gamma_c = 0.5\%$ is a large cyclic shear strain for seismic response analyses, while according to Andersen et al. (1980) it is a moderate strain for the analyses of the response of the foundations of offshore structures subjected to large ocean wave storms. The range of σ'_{vc} in Table 1 approximately corresponds to the stresses in fully saturated clay deposits between 20 and 70 m depth. The seemingly low frequencies generally cover the range of frequencies of cyclic loads that are applied to the foundations of offshore structures by large ocean wave storms. For example, the highest energy waves in the oil fields of North Sea, including the 100-year design wave, have periods of 15–20 s, which

corresponds to $f = 0.05$ to 0.07 Hz (Schjetne 1976; see also Ellers 1982). Cyclic soil degradation is also an important phenomenon in geotechnical earthquake engineering where the frequencies of seismic shaking are much larger. The frequency of ground accelerations during earthquakes varies widely from earthquake to earthquake and in a given earthquake over different geologic and soil formations, and it ranges generally from 0.5 to 15 Hz. However, the characteristic frequencies of ground velocities are much smaller and those of displacements even smaller, and the frequencies of the ground displacements are similar to the frequencies of cyclic shear strains below the ground surface. In conclusion, the cyclic strain frequencies applied in this investigation are applicable to a range of soil dynamics problems. Furthermore, as shown in the next section, the consistent trends of the cyclic degradation and cyclic pore water pressure over the range of cyclic strain frequencies from 0.001 to 0.1 Hz can be extrapolated to higher frequencies of around 0.5 to 1 Hz. The values of the degradation parameter t in Table 1 are the slopes of straight lines fitted through the log N versus log δ data points for the first 20 cycles of straining.

Effect of Frequency on Cyclic Straining

The effect of frequency, f, on the equivalent normalized cyclic pore water pressure, Δu_N^*, is presented in Fig. 6. The results show that at the same γ_c and σ'_{vc} the pore water pressure Δu_N^* is consistently higher if the frequency is lower. The reason for such a trend is not apparent and should be a subject of future investigation.
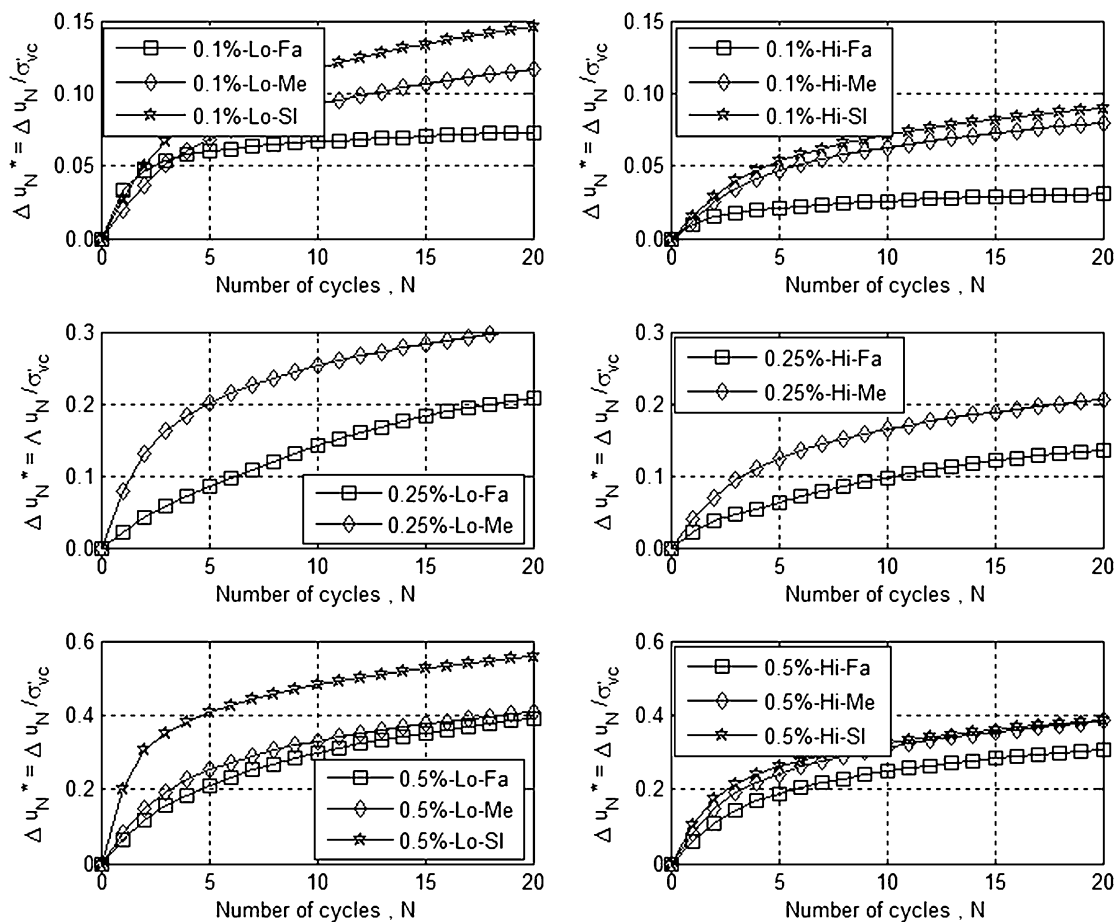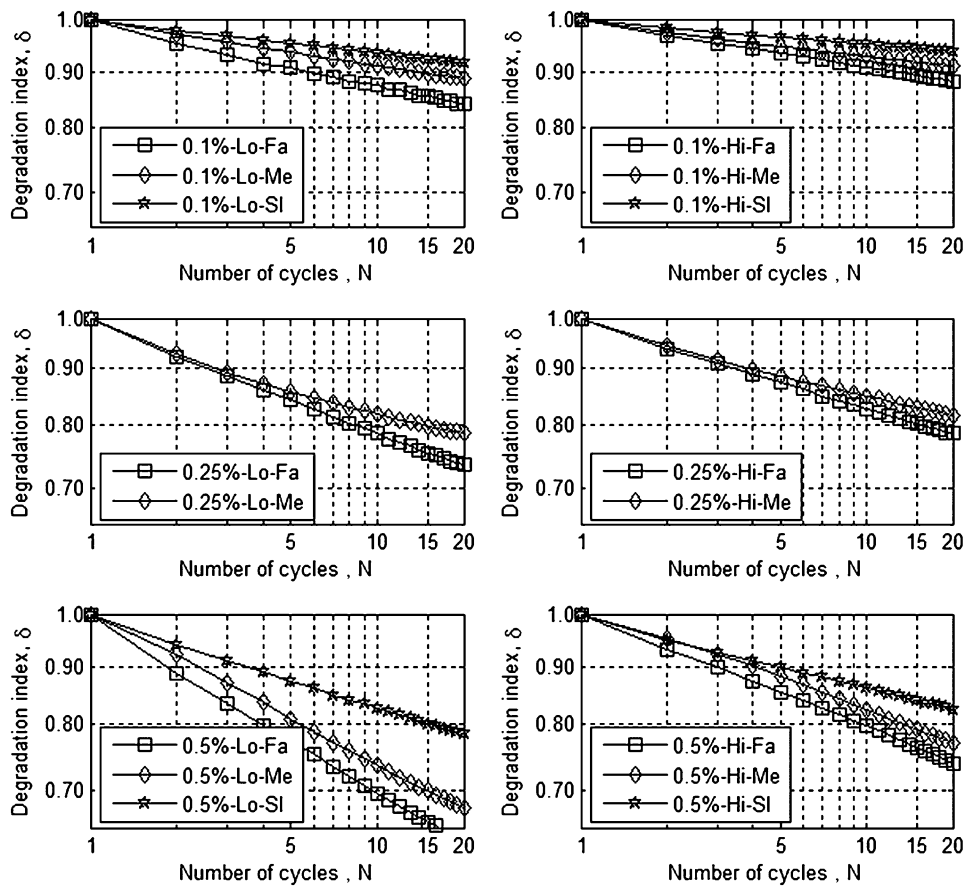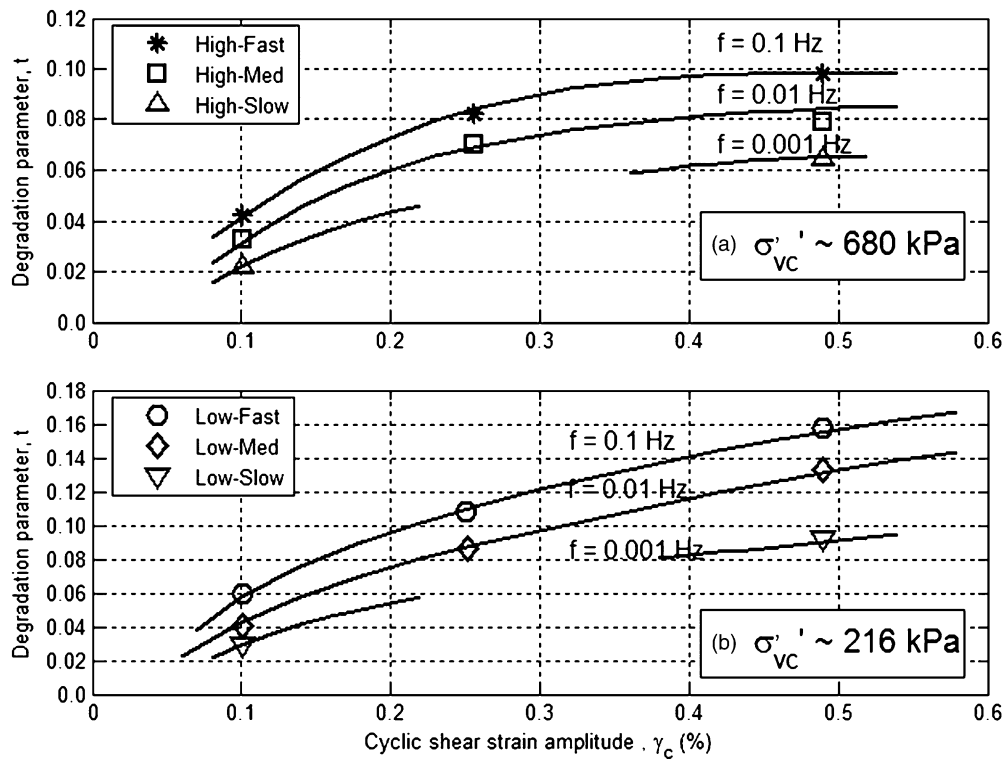

Fig. 6. The effect of frequency, f, on the equivalent normalized cyclic pore water pressure, Δu_N^*

Fig. 7. The effect of frequency, f, on cyclic degradation

Fig. 8. The effect of frequency, f, on cyclic degradation

The effect of f on the cyclic degradation is presented in Figs. 7, 8, and 9. The degradation is consistently larger if the frequency is higher. The data in Table 1 reveal that for the 10-fold increase of f (the increase by one order of magnitude) parameter t increases by 20 to 50%. This is a substantial increase and should be taken into consideration in the cyclic degradation analyses. The reason for such a trend is also not apparent and requires further investigation. As
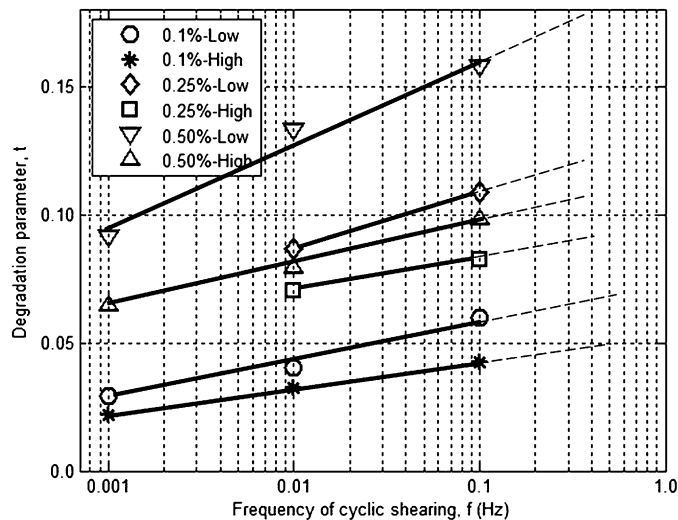
expected, a larger initial secant shear modulus G_{S1} was obtained in tests with higher f due to the effect of the rate of shearing (Mortezaie 2012). According to Eq. (1), larger G_{S1} would result in a smaller degradation if G_{SN} decrements with N are the same in tests with different frequencies. However, a careful examination of the test results revealed that G_{SN} decrements are larger in tests with higher f. In fact, they are so much larger that at higher f the degradation is larger despite the larger G_{S1}.

Fig. 9 reveals that for given γ_c and σ'_{vc} the degradation parameter t increases practically linearly with the logarithm of f. Such a trend is convenient because it allows the extrapolation of parameter t to somewhat lower and higher frequencies. Because the trend is clearly linear over $f = 0.001, 0.01$, and 0.1 Hz, there is no apparent reason why it should not continue upward to at least 0.5 Hz and perhaps even 1.0 Hz. Furthermore, Fig. 8 and the data in Table 1 show that the trend of t with f is not very different at two different σ'_{vc} values, which means that the frequency effect does not depend significantly on σ'_{vc}. For example, if at $\gamma_c = 0.1\%$ and $\sigma'_{vc} \approx 219$ kPa, f increases by two orders of magnitude from 0.001 to 0.1 Hz, t increases by 103% from 0.029 to 0.060. At the same $\gamma_c = 0.1\%$ but larger $\sigma'_{vc} \approx 678$ kPa, t increases over the same range of f by 96% from 0.022 to 0.042. Similarly, if at $\gamma_c = 0.5\%$ and $\sigma'_{vc} \approx 218$ kPa, f increases from 0.001 to 0.1 Hz, t increases by 72% from 0.092 to 0.157. At the same $\gamma_c = 0.5\%$ and larger $\sigma'_{vc} \approx 689$ kPa, t increases by 52% from 0.065 to 0.098.

Finally, it should be noted that higher frequency causes simultaneously larger degradation and smaller pore water pressures. Considering that smaller pore water pressure means larger effective stress and potentially stiffer soil, this trend seems counterintuitive.
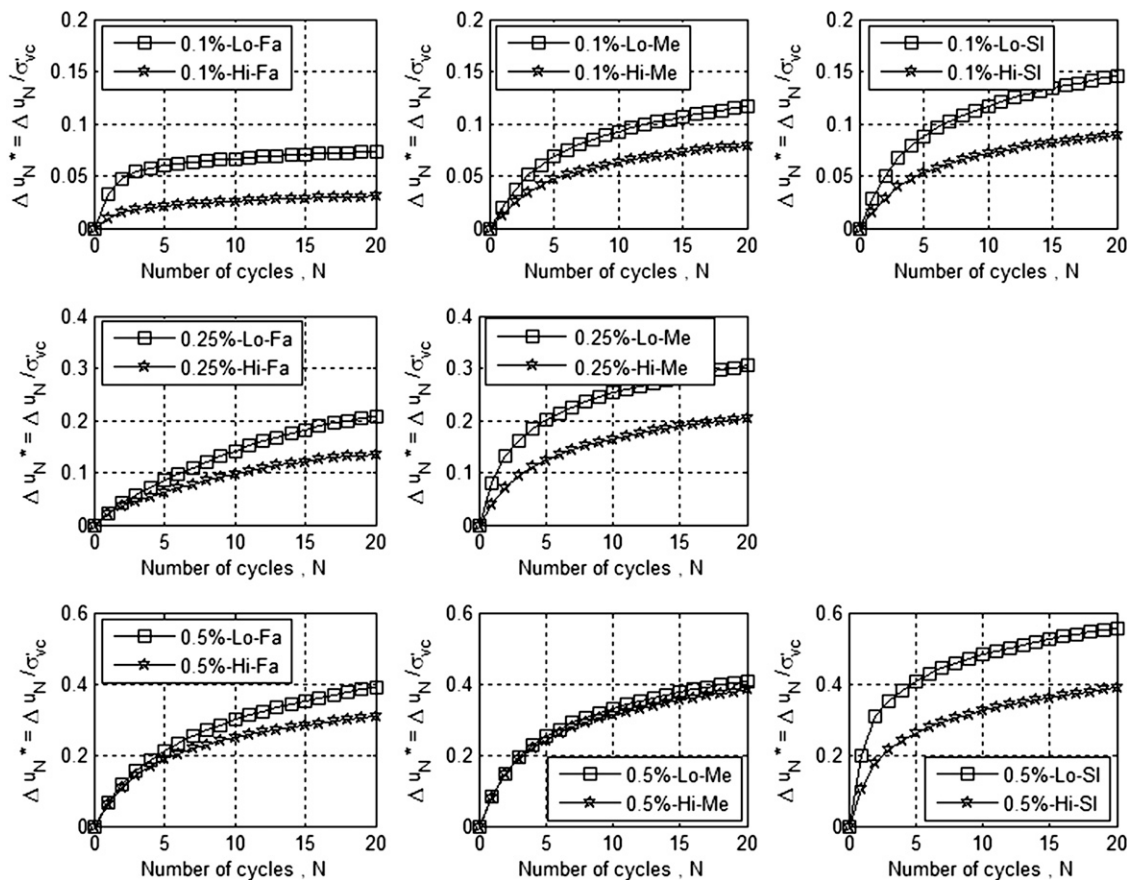

Fig. 9. The effect of frequency, f, on cyclic degradation

Fig. 10. The effect of vertical consolidation stress, σ'_{vc}, on the equivalent normalized cyclic pore water pressure, Δu_N^*

This trend therefore suggests that the cyclic pore water pressure buildup may not be a dominant contributor to the cyclic degradation of normally consolidated clays. In this regard, it is interesting to mention another well-documented aspect of the cyclic behavior of clay that is also counterintuitive. In overconsolidated clays of higher plasticity, with OCR higher than 2, a significant cyclic degradation takes place despite a decrease of pore water pressure with N and associated increases of effective stress (Andersen 1976; Andersen et al. 1980; Vucetic 1988; Tan and Vucetic 1989). The increase of the effective stress would suggest a strengthening of soil instead of its degradation, but that is apparently not the case.

Effect of Vertical Consolidation Stress on Cyclic Straining

The effect of the vertical consolidation stress, σ'_{vc}, on the cyclic pore water pressure, Δu^*_N, is presented in Fig. 10. The results clearly indicate that at lower σ'_{vc} the pressure Δu^*_N is consistently higher. This is most likely because at lower σ'_{vc} the void ratio, e, is larger. At larger e, the soil structure has a larger capacity for volume reduction, which in undrained conditions translates into a potentially larger pore water pressure buildup.

The effect of σ'_{vc} on cyclic degradation is presented in Figs. 11 and 12. It can be seen that the degradation is consistently larger at lower σ'_{vc}. This may in part be due to higher cyclic pore water pressures at lower σ'_{vc}, because a higher pore water pressure causes larger reduction of effective stresses and thus a higher rate of

softening. Since the buildup of cyclic pore water pressure may not be a dominant cause of the cyclic degradation of clays, as already mentioned, the trend shown in Figs. 11 and 12 must also be caused by some other factors which may actually be more important. Consequently, this trend also deserves to be investigated further.

The data in Table 1 show that for the approximately threefold increase of σ'_{vc} from 220 to 680 kPa, the degradation parameter t can decrease by 20–38%. It should be noted that the effect of σ'_{vc} was not observed on high plasticity clays that during static loading exhibit normalized behavior with respect to σ'_{vc} (Vucetic 1988; Vucetic and Dobry 1988) according to the SHANSEP method introduced by Ladd and Foott (1974). As shown by Mortezaie (2012), the kaolinite clay tested does not exhibit normalized behavior during monotonic loading, which somehow may be related to the existence and degree of the effect of σ'_{vc}.

Furthermore, the data in Table 1 reveal that at the same γ_c the trends of t with σ'_{vc} at three different frequencies are not very different. At $\gamma_c = 0.1\%$, parameter t decreases by 20–30% if σ'_{vc} increases from approximately 218 to 683 kPa. At $\gamma_c = 0.25\%$, the decrease is between 20 and 23%, while at $\gamma_c = 0.5\%$, the decrease ranges between 29 and 38%. This means that the effect of σ'_{vc} on t depends relatively little on f.

Discussion and Conclusions

The results presented in this paper are applicable only to the normally-consolidated kaolinite clay and similar clays tested in the
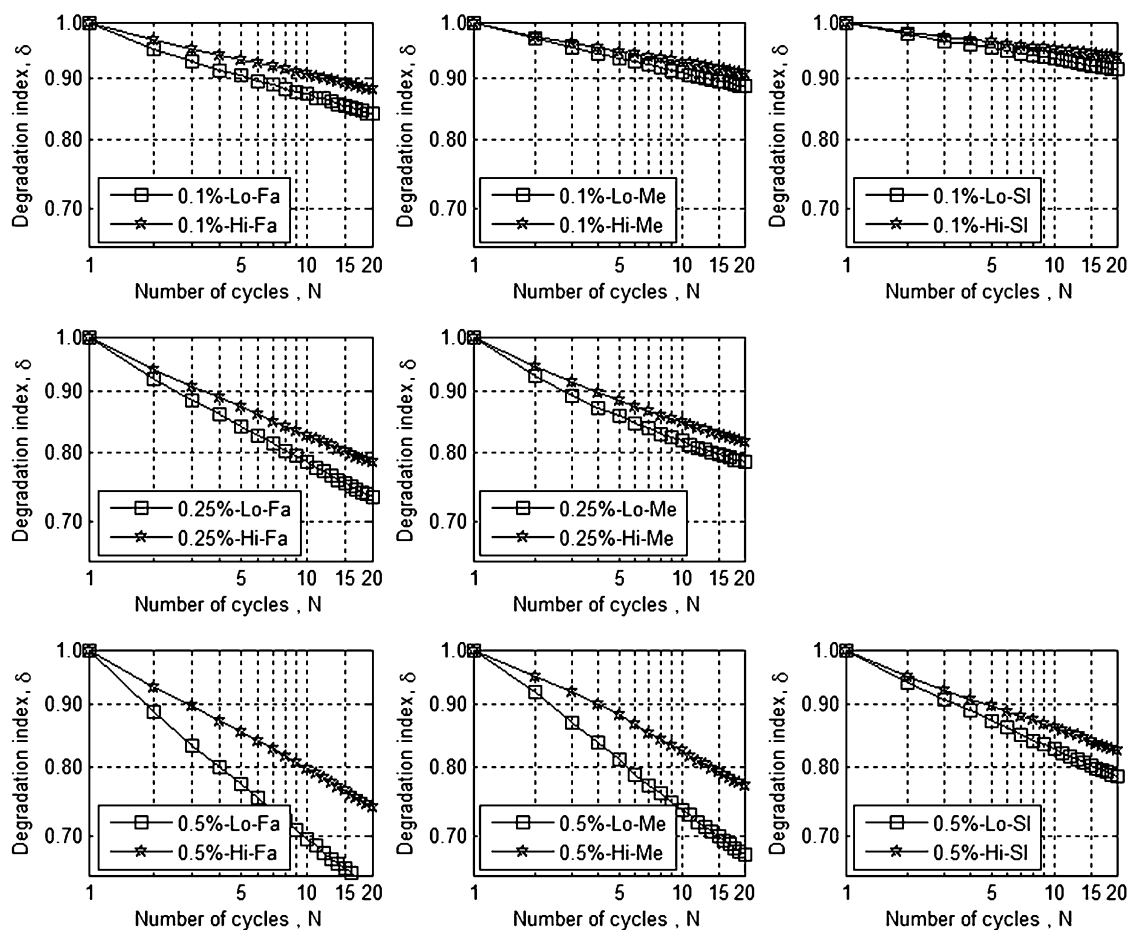

Fig. 11. The effect of vertical consolidation stress, σ'_{vc}, on cyclic degradation
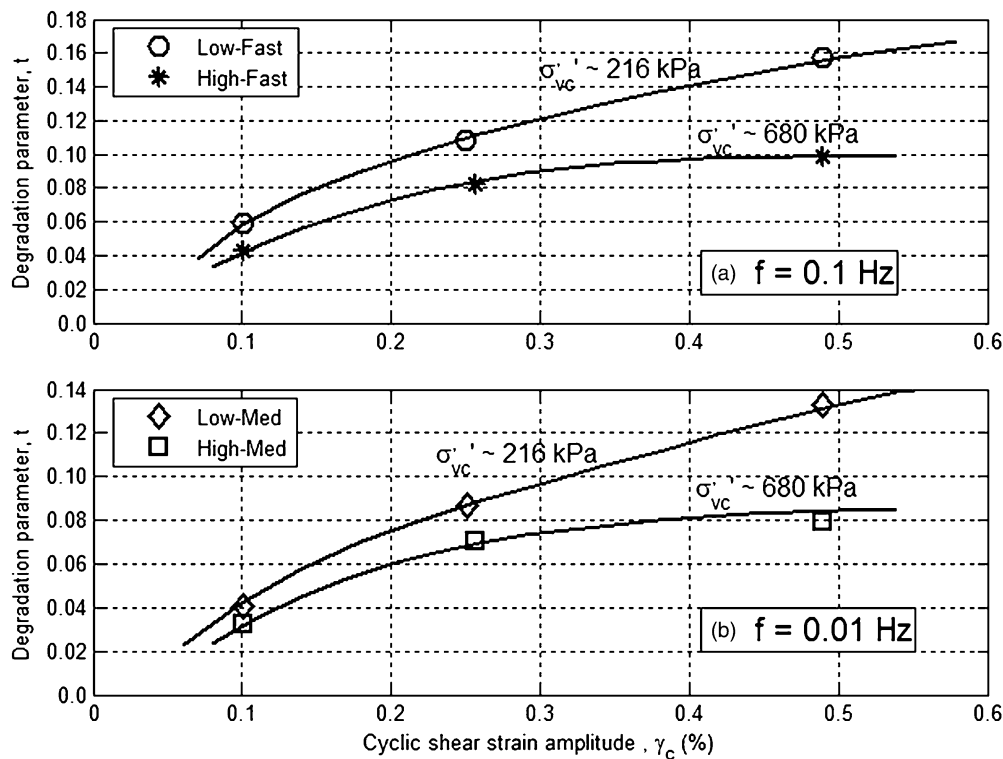
Fig. 12. The effect of vertical consolidation stress, σ'_{vc}, on cyclic degradation

NGI-DSS device in the uniform two-way cyclic strain-controlled mode. The results clearly show that the cyclic degradation and cyclic pore water pressure can be affected substantially by the vertical consolidation stress, σ'_{vc}, and the frequency of cyclic straining, f. The degradation parameter, t, which measures the rate of cyclic degradation with the number of cycles, N, increases with f and decreases with σ'_{vc}. More specifically, this study shows that for the kaolinite clay having PI = 28 and for the range of γ_c between 0.1 and 0.5%, the parameter t may increase by 20–50% if f is 10 times higher. This is a substantial increase and should be taken into consideration in the cyclic degradation analyses. If σ'_{vc} is increased by more than 3 times, from approximately 220 to 680 kPa, t decreases by 20–38%.

The extent of the combined effects of σ'_{vc} and f on the parameter t is illustrated further in Fig. 13 where the data points from the two graphs in Fig. 8 are plotted together. A significant width of the data band indicates that for a proper evaluation of the cyclic degradation, the cyclic tests must be conducted at the frequency and under the vertical stress corresponding quite closely to those in the problem under consideration. The other alternative is to conduct test series at several different frequencies and vertical stresses and develop trends similar to those presented here, and then use these trends to project the data to the desired frequency and vertical stress.

In comparison with other factors affecting the cyclic degradation, these effects of f and σ'_{vc} are relatively significant. They seem as important as the effects of OCR and PI investigated earlier and available in the literature. Previous studies have shown, for example, that when OCR is increased from 1 to 4 for clays having PI between 25 and 55, the degradation parameter t may decrease by 35–40% (Vucetic and Dobry 1988). Furthermore, parameter t corresponding to $\gamma_c \approx 0.25\%$ may decrease in normally-consolidated clays by roughly 70% if PI is increased from 15 to 50 (Vucetic 1994a).

The effects of σ'_{vc} and f on the normalized cyclic pore water pressure, Δu_N^*, are also considerable. However, from a practical
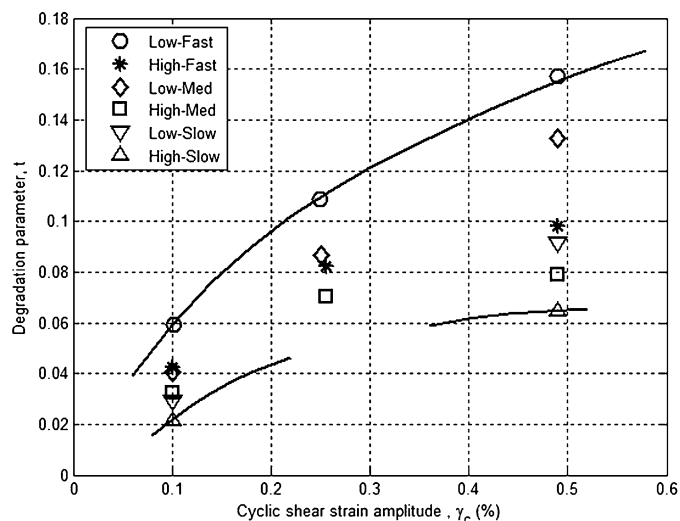

Fig. 13. Combined effects of vertical stress and frequency on cyclic degradation

point of view, the effects on the pore water pressure are less important than the effects on the degradation. What matters the most in soil dynamics analyses is how much of the soil stiffness and strength may be lost due to the cyclic loading, regardless of the pore water pressure development. Still, the pore water pressure change at the end of the cyclic event may be important, because it may cause postcyclic settlement or expansion of the soil and a change in static shear stiffness and strength. If, for example, there is a significant pore water pressure increase at the end of cyclic event, a substantial consolidation may ensue with time and with it considerable

settlement. Due to the higher pore water pressure after cyclic loading, the static secant shear modulus at smaller strains and the ultimate shear strength at large strains will also be lower (Andersen et al. 1980; Singh et al. 1978).

In the end, it must be noted again that from this study it is not clear how much the consolidation stress and frequency may affect the cyclic degradation and cyclic pore water pressure of low plasticity clays, as well as the high plasticity clays, that significantly differ from the kaolinite clay. This can be determined, however, with the help of cyclic testing programs similar to the program described in this paper. Further research to explain on a fundamental level the trends of the cyclic degradation and cyclic pore water pressure with σ'_{vc} and f that seem counterintuitive is also necessary.

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