THE INFLUENCE OF INITIAL STATIC SHEAR STRESS ON POST-CYCLIC DEGRADATION OF NON-PLASTIC SILT

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ABSTRACT: Using a sequential procedure of cyclic and post-cyclic direct simple shear (DSS) tests, the strength and stiffness degradation characteristics of non-plastic silt were studied during and immediately after cyclic loading. The post-cyclic degradations of strength and stiffness were considered with special reference to the effects of initial static shear stress (ISSS). The findings obtained from sequential DSS testing on non-plastic silt are: (1) strength and stiffness degradation characteristics of non-plastic silt during post-cyclic loading depend on the increase of the normalized pore pressures generated during cyclic loading, which is more marked with increasing ISSS; (2) degradation of stiffness is particularly sensitive to ISSS; (3) post-cyclic degradation relations of strength and stiffness for non-plastic silt formulated against cyclic-induced excess pore pressure should include the influence of ISSS. The methods we formulated for predicting post-cyclic strength and stiffness take the effect of ISSS into consideration.

Key words: Non-plastic silt, direct simple shear test, cyclic test, monotonic test, constant volume condition, degradation of strength and stiffness, post-cyclic loading

INTRODUCTION

It has been reported that, during recent earthquakes such as Turkey’s 1999 earthquake (JGS 2000a) and the 2000 Tottoriken-Seibu Earthquake (2000b), silty soil liquefaction led to lateral ground flow (Hamada et al. 1999; JGS 2000a; Shimamoto et al. 2001). In both cases, the damaged sites were close to the river or sea and had very high ground water levels, close to the surface.

For example, the silty ground damaged during the 1999 Turkey Earthquake lies in the region between Adapazari and Sapanca near the mouth of the Sakarya River (Aydan et al. 2000; Hamada et al. 1999; JGS 2000a). The city of Adapazari was founded on an alluvial stratum and consists of residual soils transferred from the river about 200 years ago; its ground is constituted of silty clay and non-plastic

Fig. 1 Procedure for the direct simple shear (DSS) test

Fig. 2 Soil test state path

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Note: Discussion on this paper is open until December 25, 2003.
silt. The ground water level was only 1-3m below the surface. The 1999 Turkey Earthquake caused a number of structures in Adapazari to be damaged due to liquefaction. Many buildings settled, tilted or totally collapsed due to liquefaction of silt or silty sand. Vertical displacement of buildings up to 1.1 m was observed. More typically, buildings suffered severe tilting due to loss of bearing capacity in the foundation ground. Such structural incapacitation was particularly severe along the border between liquefied and non-liquefied areas.

During the 2000 Tottori-Seibu Earthquake the most serious damage due to silt liquefaction occurred at the Takenouchi Industrial Complex near seaside (Shimamoto et al. 2001). The non-plastic silt beneath this site had a 2.4 ~ 5.6 uniformity coefficient $U_s$ and was 50-100% fines. Settlement of 0.1 ~ 0.3 m and lateral flow of about 0.2 m were reported there.

This study examines whether silt has a high potential for liquefaction and lateral flow. It also examines how silty soils are affected by initial static shear stress (ISSS), which is counted as an important issue in relation to the stability of silty soils beneath structures and in sloping ground during earthquakes. The NGI type Direct Simple Shear (DSS) tests (Bjerrum and Landva, 1966) were central in determining the cyclic and post-cyclic degradation of non-plastic silt. Specifically, we investigated the following:

1) The degradation of strength and stiffness of non-plastic silt using DSS tests, and
2) The cyclic strength and stiffness characteristics of non-plastic silt, with an emphasis on the effects of ISSS, and
3) Using the results from these cyclic DSS tests, we discovered and herein propose a method for predicting post-cyclic strength and stiffness taking into consideration the effect of ISSS.

TESTING PROCEDURE FOR INVESTIGATION OF BOTH CYCLIC AND POST-CYCLIC CHARACTERISTICS

Basic Concept

In previous papers such as those by Yasuhara (Yasuhara et al. 1983, 1992; Yasuhara 1985, 1994a, 1994b), the cyclic triaxial test has been used to investigate changes in strength and stiffness of cohesive soils under cyclic loading conditions. Figure 1 presents the steps in a procedure for obtaining the behaviour of soils both during and after cyclic loading with ISSS $\tau_c$. The procedure shown in Fig. 1 has also been adopted for the simple shear test (Andersen et al. 1976; Vucetic et al. 1998) and the cyclic direct shear test (Yasuhara and Nagano 1995). Cyclic parameters during and after cyclic loading are commonly divided into two categories: strength and stiffness. These characteristics depend on the magnitude of cyclically induced pore pressures and shear strains. For this reason, we investigated, the relation of post-cyclic strength and stiffness characteristics to excess pore pressure in non-plastic silt.

Testing Procedure

Cyclic DSS tests were carried out to investigate post-cyclic degradation of strength and stiffness. In this test, as shown in Fig. 2, a constant specimen height is maintained during both cyclic and post-cyclic loading to hold the constant $\tau_c$ volume condition.

Specimens were initially consolidated under a vertical effective stress $\sigma_{v,NC}$ at point A in Fig. 2. The vertical effective stress $\sigma_{v,NC}$ moved to point B ($\sigma_{v,ay}$) due to the excess pore pressure generated during cyclic loading under constant volume conditions. This means that the difference between the vertical effective stress $\sigma_{v,NC}$ after consolidation and the vertical effective stress after cyclic loading $\sigma_{v,ay}$ should be equal to the excess pore pressure generated by cyclic loading. The ISSS $\tau_c$ was applied after pre-consolidation under drained constant stress conditions. Because of applying this ISSS to each specimen, a small change in the void ratio took place. However, it should be noted that the change in the void ratio was negligibly small. To evaluate the post-cyclic degradation of stiffness, a strain-controlled monotonic test was also performed under constant volume conditions after cyclic loading. This is called the "post-cyclic loading process".

NGI TYPE TESTING DEVICE OF DIRECT SIMPLE SHEAR TEST AND BENDER ELEMENT

Figure 3 shows the NGI-type DSS apparatus with a bender element, which was used at Ibaraki University. In Fig. 3(a), vertical and horizontal stresses up to 5MPa by air pressure and 2MPa by oil pressure can be applied to each specimen. Vertical and horizontal displacement can be measured up to 10mm using strain gauges. Vertical displacement during preconsolidation and shear tests show the average value of results from two gauges attached at the left and right sides of "Top pressure plate" that is located on the top of specimen with 70 mm diameter and 30 mm height. As shown in Fig. 3, gauges 1 and 2 are attached at the top and bottom of pressure plate, respectively, for separate measurements of horizontal displacement. A load or a displacement controller modulates the rate of vertical and horizontal load. Each specimen is contained in a wire-reinforced membrane. In other words, this NGI-type DSS apparatus is capable of performing tests under the $K_0$
condition during preconsolidation. The undrained, or constant volume condition is achieved by keeping the height of specimens constant throughout the shear testing. Thus, excess pore pressures are determined by the difference between the initial vertical stress and that after cyclic shearing.

As shown in Fig. 3(b) the bender element is attached at the center of the top and the bottom of the pressure plate for measuring the velocity of the shear wave during consolidation and at the beginning of shear stages. The shear wave is sent from a transmitter top to a receiver bottom in this device. Maximum stiffness of soil from DSS tests using the bender element can be regarded as a benchmark for reliability of stiffness in soils. However the bender element is used to measure maximum stiffness only during consolidation because it can be broken during the cyclic test when the shear strain becomes large.

**SPECIMEN AND TEST CONDITION**

Soil Properties and Specimen Preparation

Grain size distribution curves for non-plastic silt are shown in Fig. 4. The non-plastic silt used is called DL Clay. The dotted and dashed lines indicate the particle size distribution ranges of feasible and very feasible liquefaction, respectively (JGS 1993). Judging from these ranges of feasibilities, DL Clay has a high liquefaction potential. There is no plasticity index I_p for DL Clay. For DL Clay the density of soil particles p_s was 2.48 g/cm^3 and the liquid limit wL was 25.1%.

All of the specimens were prepared by slurry method. Relative density D_r was used to define the initial condition of the specimens. The density test was performed to determine the maximum and the minimum dry density \( \rho_d_{max} \), \( \rho_d_{min} \) of DL Clay following JJS A 1210 (JGS 2000c) and e_{max} were 1.47 and 0.71, respectively. To investigate the liquefaction of sand in triaxial tests a relative density D_r between 40 or 50% is normally selected (JGS 2000c). Therefore, the dry unit weight of the specimens was 12 kN/m^3, and the relative density D_r was about 42%. The average void ratio of specimen e was 1.16.

Back pressure was applied from bottom to top of the specimen, and a slurry with a water content about twice the liquid limit wL 25.1% was formed. Vertical stress up to 196 kPa was applied to each specimen at a rate of 196 kPa/hr. The vertical stress was kept constant until the primary consolidation of the specimens terminated.

**Test Condition**

The vertical effective stress \( \sigma_{ve} \) was set at 196 kPa since it is known that a liquefied layer normally exists up to about
10 - 25 m below the ground surface. Three levels of ISSS (0, 9.81, and 19.6 kPa) were applied under a loading rate of 196.2 kPa/hr before cyclic loading and did not exceed the peak shear stress $\tau_{\text{peak}}$ (namely equal to shear strength $\tau_i$) of 27 kPa obtained from static tests. The static test was also performed under constant volume conditions in which the shear stress was applied under a constant strain rate of 0.1%/min until reaching a shear strain of 20%. With the same rate of strain the ISSS was applied under constant stress conditions. On the other hand, for cyclic DSS tests, the cyclic shear stress $\tau_{\text{cyc}}$ with stress control was applied to specimens under constant volume conditions, until either 50 cycles were reached or a 10% double amplitude shear strain $\gamma_{DA}$ was attained. The frequency of cyclic loading was 0.1 Hz for all of the tests.

The monotonic test was performed 10 min after the completion of the cyclic loading in order to allow the uniform distribution of excess pore water pressures, and the static shear stress was applied in the same manner as the static test: without previous cyclic loading.

**INFLUENCE OF INITIAL STATIC SHEAR STRESS ON MONOTONIC AND CYCLIC SHEAR STRENGTH AND STIFFNESS**

Static Test

Figure 5 shows the results of static and monotonic tests on specimens with ISSS in terms of the relations of shear stress to shear strain and shear stress to effective vertical stress, respectively. From Fig. 5(a) it can be seen that the higher the ISSS the specimen undergoes, the larger the shear strength that is developed. This is probably caused by the fact that the volume in specimens decreases under the influence of ISSS. In Fig. 5(b) an effective frictional angle $\phi$ equal to 23° was determined. Since the DSS test apparatus is not able to measure pore pressures, they were
calculated using changes in vertical stresses observed during shear under constant volume condition as was previously described. From the results shown in Fig. 5, shear strength and maximum stiffness $G_{max}$ with increasing ISSS can be drawn using the procedure shown in Fig. 6(b). Figure 6(a) shows a definition of post-cyclic shear strength $\tau_c$ and maximum stiffness $G_{max}$; the results are shown in Fig. 6(b).

**Cyclic Test**

Figures 7 and 8 show a typical set of results from a family of cyclic DSS tests on specimens with three different ISSS values. The applied cyclic stress ratio $R_{DSS}$ is about 0.048 in this test. The cyclic stress ratio $R_{DSS}$ is defined by:

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Fig. 8 Relation of normalized pore pressure vs. number of load cycles

\[
R_{\text{NSS}} = \frac{\tau_{\text{c,ey}}}{\sigma'_{\text{ve}}}
\]  (1)

where \(\tau_{\text{c,ey}}\) is cyclic shear stress, and \(\sigma'_{\text{ve}}\) is the effective vertical stress when applying an initial confined stress with consolidation pressure such as \(\sigma_{\text{ve,NG}}\) in Fig. 2. It is seen from Fig. 7 that the cyclic shear strain \(\gamma\) increases with increasing ISSS \(\tau_{\text{s}}\), while the vertical effective stress \(\sigma'_{\text{v}}\) decreases under the same cyclic shear stress. The shear strain in Fig. 7 (c) is larger than that in the other cases shown in Fig. 7 (a) and Fig. 7 (b). The reason for this tendency must be due to the fact that cyclic failure occurred due to the large cyclic shear stress \(\tau_{\text{c,ey}}\) since the ISSS is larger than those in the cases of Fig. 7 (a) and (b).

Figure 8 shows the influence of ISSS on relations between normalized pore pressure NPP and number of load cycles \(N_c\). The normalized pore pressure NPP is defined by:

\[
NPP = \frac{1}{2} + \frac{1}{\pi} \arcsin \left( \frac{N_c}{N_{\text{cl}}} \right) - 1
\]  (3)

where \(\Delta \mu\) is excess pore pressure, and \(\sigma'_{\text{ve}}\) and \(\sigma'_{\text{v}}\) are the effective vertical stress before and after cyclic loading. Figs. 7 and 8 show that at a large ISSS the shear strain \(\gamma\) and the normalized pore pressure NPP build up faster than that with no ISSS. The influence of a small ISSS is not clear.

Figure 9 shows the relations between normalized pore pressure NPP and the number of load cycles \(N_c\) normalized by the number of load cycles \(N_{\text{cl}}\) at liquefaction. The solid line was calculated by following the relation proposed by Seed et al. (1975):

\[
NPP = \frac{1}{2} + \frac{1}{\pi} \arcsin \left( \frac{N_c}{N_{\text{cl}}} \right) - 1
\]

Fig. 10 Relation between cyclic ratio and number of load cycles

\[
NPP = \frac{\Delta \mu}{\sigma'_{\text{ve}}} = \frac{\sigma'_{\text{ve}} - \sigma'_{\text{v}}}{\sigma'_{\text{ve}}}
\]  (2)

where \(\Delta \mu\) is excess pore pressure, and \(\sigma'_{\text{ve}}\) and \(\sigma'_{\text{v}}\) are the effective vertical stress before and after cyclic loading. Figs. 7 and 8 show that at a large ISSS the shear strain \(\gamma\) and the normalized pore pressure NPP build up faster than that with no ISSS. The influence of a small ISSS is not clear.

Fig. 9 shows the relation between normalized pore pressure NPP and the number of load cycles \(N_c\) normalized by the number of load cycles \(N_{\text{cl}}\) at liquefaction. The solid line was calculated by following the relation proposed by Seed et al. (1975):

\[
NPP = \frac{1}{2} + \frac{1}{\pi} \arcsin \left( \frac{N_c}{N_{\text{cl}}} \right) - 1
\]

Fig. 11 Equivalent stiffness vs. single amplitude shear strain

\[
G_{\text{max}} = 35.9 \text{ MPa}
\]

\[
G_{\text{max}} = 31.0 \text{ MPa}
\]

\[
G_{\text{max}} = 28.5 \text{ MPa}
\]

where \(\gamma_{\text{res}}\) is increasing.

\[
\gamma_{\text{res}} = 7.5\%
\]

\[
\tau_{\text{y}} = 0.0 \text{ kPa}
\]

\[
9.8 \text{ kPa}
\]

\[
19.6 \text{ kPa}
\]

\[
0.0 \text{ kPa}
\]

\[
\text{Toyoura Sand}
\]

\[
D_5 = 54\%
\]

\[
\text{DL Clay}
\]

\[
D_5 = 70\%
\]
where \( \alpha \) is a parameter depending on the soil type in this case.

From cyclic DSS tests, it is clear that the relation between normalized pore pressure and number of load cycle ratio \( N_c / N_d \) without ISSS cannot be fitted by Eq. (3). Because Eq. (3) was developed for results without ISSS, results with ISSS are not well matched with Eq. (3) even if the parameter \( \alpha \) is changed. Therefore, a new model is needed to enable prediction of the normalized pore pressure NPP vs. number of load cycle ratio \( N_c / N_d \) in relation to varying with ISSS values.

**CYCLIC-INDUCED DEGRADATION IN STRENGTH AND STIFFNESS**

Figure 10 shows the relation between the cyclic stress ratio \( R_{DSS} \) and the number of cycles \( N \) at peak shear strain \( \gamma_p = 10\% \) in cases with ISSS. The peak shear strain \( \gamma_p \) is taken from the shear stress vs. shear strain relations with and without ISSS. The peak shear strain \( \gamma_p \) is used instead of double shear strain \( \gamma_{DA} \) when there is no relation without ISSS. But the peak shear strain \( \gamma_p \) is employed when there is a relation with ISSS, because it is very difficult to obtain a precise double shear strain \( \gamma_{DA} \) with ISSS (Vaid and Chern 1983; Hyodo et al. 1994). In Fig. 10 it is clear that strength degradation occurs with increasing ISSS \( \tau_r \). In Fig. 10 the cyclic stress ratio \( R_{DSS} \) vs. the number of load cycles \( N_c \) curves for silt are compared with those for Toyoura Sand under constant volume condition using direct shear (DS) tests. Compared with sand, non-plastic silt represents a very weak soil against cyclic loading, even though the relative density of non-plastic silt is larger than that of sand.

Figure 11 shows the relation of equivalent stiffness \( G_{eq} \) to single amplitude shear strain \( \gamma_{SA} \) indicating the initial values of \( G_{eq} \) obtained by extrapolation of the Hardin-Drucevich (H-D) model (Hardin and Drucevich 1972). The equivalent stiffness or shear modulus \( G_{eq} \) was determined using (Vucetic et al. 1998):

\[
G_{eq} = \frac{\tau_{\text{max}} - \tau_{\text{min}}}{\gamma'_{\text{max}} - \gamma'_{\text{min}}} \tag{4}
\]

In Fig. 11 all the data for each cyclic loading step in cyclic tests were plotted. Introducing an empirical constant \( \alpha \) to give three different lines modified the Hardin-Drucevich model. The reason for adapting this value was that the Hardin-Drucevich model does not apply to silt but to sand. The modified Hardin-Drucevich model in the present paper is thus given by:

\[
G_{eq} = \frac{G_{\text{max}}}{1 + \alpha (\gamma_{\text{st}} / \gamma_r)} \tag{5}
\]

where \( G_{\text{max}} \) is the maximum equivalent stiffness in static tests, \( \gamma_{SA} \) is single amplitude shear strain, \( \gamma_r \) is standard shear strain, and \( \alpha \) is an empirical constant. Although the constant \( \alpha \) is incorporated, the modified Hardin-Drucevich model matches well with all the results from DSS tests on silt as can be seen in Fig. 11. The maximum stiffness \( G_{\text{max}} \) was determined by static shear tests (refer to Fig. 6). The constant \( \alpha \) was 1.5, 1.6, and 2 for ISSS \( \gamma_r \) values of 0, 9.8, and 19.6 kPa, respectively.

To confirm the reliability of the results from DSS tests, the maximum stiffness \( G_{\text{max}} \) from the binder element was compared with that from the DSS test in Fig. 12. From the tested results in Fig. 12 it is known that the maximum stiffness \( G_{\text{max}} \) from the static DSS test is almost the same as that obtained from the binder element. Maximum stiffness from the binder element can be calculated easily. As shown in Fig. 3(b), the shear wave is sent to a receiver from a transmitter. The elapsed time \( \Delta t \) of the shear wave can be known using the binder element. In Fig. 3(b) a distance \( h \) between the top and the bottom of the binder element can be determined. The maximum stiffness \( G_{\text{max}} \) determined using the binder element is calculated by (Tanizawa et al. 1994):

\[
G_{\text{max}} = \rho v_s^2 \tag{6}
\]

\[
\nu_s = h / \Delta t \tag{7}
\]

where \( \rho \) is soil density, \( v_s \) is shear wave velocity, and \( \Delta t \) is the elapsed time of the shear wave.

**DEGRADATION IN STRENGTH AND STIFFNESS OBSERVED FROM POST-CYCLIC DSS TESTS**

Figure 13 shows a typical set of relations of shear stress versus shear strain and shear stress versus effective vertical
stress in static, cyclic and post-cyclic tests on specimens with ISSS. Cyclic softening is commonly observed in the post-cyclic effective stress paths in Fig. 13. To show the influence of ISSS the results from cyclic DSS tests are compared using almost the same cyclic stress ratio $R_{DSS}$. In Fig. 13 it is clear that the excess pore pressure $\Delta u$ increases with increasing ISSS. Post-cyclic shear strength increases with ISSS, but the larger the ISSS, the smaller the post-cyclic shear strength becomes. Figure 14 demonstrates that post-cyclic shear strength tends to decrease with increasing normalized pore pressure $\Delta u/\sigma_{ve}$ for all cases of ISSS.

Post-cyclic degradation of strength

Figure 15 shows the change in strength characteristics with and without ISSS $\tau_s$ after cyclic loading. In every case post-cyclic strength $\tau_{c,\text{cy}}$ is normalized by the static shear strength $\tau_{\text{nci}}$ without cyclic loading obtained from the static DSS test (see Fig. 6). In Fig. 15 it is known that the shear strength $\tau_{c,\text{cy}}$ increases with increasing ISSS $\tau_s$.

In Fig. 15(a), the decrease of the strength ratio $\tau_{c,\text{cy}} / \tau_{\text{nci}}$ with increasing cyclic load ratio $R_{DSS}$ becomes rapid at a certain value of the cyclic stress ratio $R_{DSS}$. As ISSS $\tau_s$ increases, the marked decrease in the strength ratio $\tau_{c,\text{cy}} / \tau_{\text{nci}}$ starts at a lower value of the cyclic stress ratio $R_{DSS}$. Figure 15(b) shows that the strength ratio $\tau_{c,\text{cy}} / \tau_{\text{nci}}$ also decreases with increasing normalized pore pressure $\Delta u/\sigma_{ve}$. Using the results from post-cyclic monotonic DSS tests, the post-cyclic degradation of strength for non-plastic silt is formulated as:
Influence of initial static shear stress on post-cyclic degradation

\[
\frac{\tau_{f,\sigma}}{\tau_{f,Nc}} = \left(1 - A_i \left(\frac{\Delta u}{\sigma_{\nu,Nc}}\right)\right)^{0.2} \exp\left(1.53 - \frac{\tau_s}{\sigma_{\nu,Nc}}\right)
\]

where \(A_i\) is a parameter to determine the decreasing tendency of shear strength ratio \(\tau_{f,\sigma} / \tau_{f,Nc}\) and the second order in Eq. (7) expresses the influence of ISSS \(\tau_s\) from Fig. 6. The parameter \(A_i\) depends on ISSS \(\tau_s\), thus:

\[
A_i = 1.07 \exp\left(0.63 \frac{\tau_s}{\sigma_{\nu,Nc}}\right)
\]

This is slightly different from the proposal by Yasuhara (1985, 1994b) and Yasuhara et al. (1992) for predicting post-cyclic undrained strength of cohesive soils using triaxial tests.

Post-cyclic degradation of stiffness

Figure 16 shows the change in stiffness characteristics with and without ISSS \(\tau_s\) after cyclic loading. In every case post-cyclic stiffness \(G_{\rho}\) is also normalized by the stiffness \(G_{\rho,NC}\) obtained from the static test (Fig. 6). As well as Fig.15(a), Fig. 16(a) indicates that the slope of the degradation of the stiffness ratio \(G_{\rho,\infty} / G_{\rho,NC}\) with increasing cyclic load ratio \(R_{DSS}\) rapidly increases at a certain value of the cyclic stress ratio \(R_{DSS}\). As ISSS \(\tau_s\) increases, the rapid decrease in the stiffness ratio \(G_{\rho,\infty} / G_{\rho,NC}\) is observed at a lower value of the cyclic load ratio \(R_{DSS}\). From Fig. 16(b) showing the relation between the stiffness ratio \(G_{\rho,\infty} / G_{\rho,NC}\) and normalized pore water pressure \(\Delta u / \sigma_{\nu,NC}\), it is also clear that the stiffness ratio \(G_{\rho,\infty} / G_{\rho,NC}\) decreases with increasing normalized pore pressure \(\Delta u / \sigma_{\nu,NC}\). Using the test results, the post-cyclic degradation of stiffness for non-plastic silt is estimated as:

![Fig. 14 Post-cyclic shear stress vs. shear strain](image)

![Fig. 15 Post-cyclic induced degradation of strength](image)

(a) Shear strength ratio vs. cyclic stress ratio  (b) Shear strength ratio vs. normalized pore pressure
The following are the main conclusions derived from the present study:

1) Cyclic and post-cyclic degradation of non-plastic silt is very sensitive to the application of ISSS. Degradation in stiffness is more sensitive to ISSS than that in strength.

2) According to results from cyclic DSS tests on non-plastic silt, the decrease in both the strength and stiffness with increasing normalized pore pressures becomes more marked at increased ISSS.

3) Using the results from post-cyclic DSS tests, post-cyclic degradation relations for strength and stiffness of non-plastic silt are formulated against cyclically induced excess pore pressure. The effect of ISSS is included in these proposed relations.

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