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 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 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 the 1999 Turkey Earthquake (JGS 2000a) and the 2000 Tottoriken-Seibu Earthquake (2000b), silty soils liquefaction led to lateral ground flow (Hamada et al. 1999; JGS 2000a; Shimamoto et al. 2001). In both cases, the damaged sites were closed 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 silts. 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.

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

Fig. 2 Soil test state path
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 Tottoriken-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 $U$ 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 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, it is investigated as the following:

1) The degradation of strength and stiffness of non-plastic silt were investigated using DSS tests, and
2) The cyclic strength and stiffness characteristics of non-plastic silt were studied with an emphasis on the effects of ISSS, and
3) Using the results from cyclic DSS tests, it is proposed a method for predicting post-cyclic strength and stiffness taking into consideration for 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 $\tau$. 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 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, it is investigated that 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 volume condition.

Specimens were initially consolidated under a vertical effective stress $\sigma_{\text{v,NCI}}$ at point A in Fig. 2. The vertical effective stress $\sigma_{\text{v,NGI}}$ moved to point B ($\sigma_{\text{v,cy}}$) due to the excess pore pressure generated during cyclic loading under constant volume conditions. This means that the difference between the vertical effective stress $\sigma_{\text{v,NGI}}$ after consolidation and the vertical effective stress after cyclic loading $\sigma_{\text{v,cy}}$ should be equal to the excess pore pressure generated by cyclic loading. The ISSS $\tau$ 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 the bender element, which has been in use 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 circular-shaped 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, separately for measurement of horizontal displacement. A load or a displacement controller can modulate 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 $K_o$ condition during preconsolidation. The undrained, or constant volume condition is achieved by keeping the height of specimens constant throughout shear testing. Thus, excess pore pressures are determined by the difference between initial
maximum dry unit weight and ASTM D 4254 (Head 1992), respectively. The limit density of soil particles was determined by relative density $D_r$ and minimum void ratio $e_{min}$. The liquid limit $w_l$ was 19.5%, and the minimum dry unit weight $\rho_{d,min}$ was 25.1%. The vertical stress was $196$ kPa since $\sigma'_{v}$ was set at 196 kPa since $\sigma'_{v}$ was a benchmark for reliability of stiffness in soils. However, the DSS tests using the bender element can be regarded as a benchmark for reliability of stiffness in soils. Therefore, the dry unit weight of the specimens was 12 kN/m$^2$. From these results, maximum and minimum void ratios $e_{max}$ and $e_{min}$ were 1.47 and 0.71, respectively. To investigate liquefaction of sand in the very feasible liquefaction range, the soil was prepared using slurry method. All of the specimens were prepared by slurry method. Relative density $D_r$ was used to define the initial condition of the specimens. To determine the maximum dry density $\rho_{d,max}$ of DL Clay and ASTM D 425 maximum dry unit weight $w_c$ to a water content $w = 10.4$ weight $\rho_{d,max}$ was 10.4.

**Soil Property and Specimen Preparation**

Grain size distribution curves for non-plastic silt and DL Clay following JIS A 1210 (JGS 2000c) are shown in Fig. 4. Non-plastic silt used is called DL Clay. As shown in Fig. 4, the bender element is attached at the center of the top and the bottom of the pressure plate for measuring the velocity of a shear wave during consolidation and at the beginning of shear stages. The dotted and the dashed lines indicate the particle size distribution ranges of feasible and very feasible liquefaction, respectively (JGS 1993). Judging from these ranges of feasibilities, the DL Clay has a high liquefaction potential.

**SPECIMEN AND TEST CONDITION**

**Soil Property and Specimen Preparation**

Grain size distribution ranges of DL Clay are shown in Fig. 4. Non-plastic silt used is called DL Clay. As shown in Fig. 4, the bender element is attached at the center of the top and the bottom of the pressure plate for measuring the velocity of a shear wave during consolidation and at the beginning of shear stages. The dotted and the dashed lines indicate the particle size distribution ranges of feasible and very feasible liquefaction, respectively (JGS 1993). Judging from these ranges of feasibilities, the DL Clay has a high liquefaction potential.

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0, 9.81, and 19.6 kPa were applied under a loading rate of 196.2 kPa/hr before cyclic loading, which did not exceed the peak shear stress $\tau_{\text{peak}}$ (namely equal to shear strength $\tau_f$) 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 $\tau_{\text{f,cy}}$ with stress control was applied to specimens under constant volume conditions, until either 50 cycles were reached or a 10% double amplitude shear strain $\gamma_{\text{DA}}$ 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 for the static test without previous cyclic loading.

### Static Test

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

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

#### Static Test

Fig. 5 Static test results with initial static shear test
Cyclic Test

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

$$ R_{\text{DSS}} = \frac{\tau_{f,\text{cy}}}{\sigma_{\text{vc}}'} $$

where $\tau_{f,\text{cy}}$ is cyclic shear stress, and $\sigma_{\text{vc}}'$ is effective vertical stress at applying an initial confined stress with consolidation pressure as $\sigma_{\text{v},\text{NCi}}$ in Fig. 2. It is seen from Fig. 7 that the cyclic shear strain $\gamma$ increases with increasing ISSS $\tau_s$, while the vertical effective stress $\sigma_{\text{vc}}'$ 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 $\tau_{f,\text{cy}}$ since the ISSS is larger than those in the cases of Fig. 7 (a) and (b).

Figure 8 shows an influence of ISSS on relations between normalized pore pressure $NPP$ and number of load cycle $N_c$. The normalized pore pressure $NPP$ is defined by:

$$ NPP = \frac{\Delta u}{\sigma_{\text{vc}}'} = \frac{\sigma_{\text{vc}} - \sigma_{\text{vc}}'}{\sigma_{\text{vc}}'} $$

where $\Delta u$ is excess pore pressure, and $\sigma_{\text{vc}}'$ and $\sigma_{\text{vc}}$ are the effective vertical stress before and after cyclic loading. Through Figs. 7 and 8 it can be seen that the influence of ISSS is not clear at the small ISSS, but in the case of large ISSS the shear strain $\gamma$ and the normalized pore pressure $NPP$ build up faster than that without ISSS.

Figure 9 shows the relations between normalized pore pressured $NPP$ and number of load cycles $N_c$ normalized by the number of load cycles $N_{cl}$ at liquefaction. The solid line is the fitting of the Seed model.

$$ \gamma = \frac{1}{2} \arcsin \left( \frac{N_c}{N_{cl}} \right)^{1/2} $$

$$ \alpha = 0.75 $$

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for silt are compared with those for Toyoura Sand in which tests under the constant volume condition was conducted by using direct shear (DS) tests. Comparing with sand, non-plastic silt belongs to a very weak soil against cyclic loading even if the relative density of non-plastic silt is larger than that of sand.

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

$$G_{eq} = \frac{\tau_{max} - \tau_{min}}{\gamma_{max} - \gamma_{min}}$$  \hspace{1cm} (4)

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

$$(r_{SA} G_{eq} G_{max}) = 1 + \alpha(r_{SA}/r_{s})$$  \hspace{1cm} (5)

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

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

$$G_{max} = \rho v_s^2$$  \hspace{1cm} (6)

Fig. 12  Comparison between $G_{max}$ from DSS test and bender element

$$NPP = \frac{1}{2} + \frac{1}{\pi} \arcsin \left\{ 2 \left( \frac{N_c}{N_{cl}} \right)^{\gamma} - 1 \right\} \hspace{1cm} (3)$$

where $\alpha$ is 0.75 as a parameter is depending on the soil type. From cyclic DSS tests, it is clear that the relation between normalized pore pressure and number of load cycle ratio $N_c/N_{cl}$ obtained without ISSS can be fitted by Eq. (3). Because Eq. (3) was developed for the results without ISSS, results with ISSS are not well matched with Eq. (3) even if the parameter is changed for the cases with ISSS. Therefore, a new model is needed to enable prediction of the normalized pore pressure $NPP$ vs. number of load cycle ratio $N_c/N_{cl}$ relation with ISSS.

CYCLIC-INDUCED DEGRADATION IN STRENGTH AND STIFFNESS

Figure 10 shows relation between cyclic stress ratio $R_{DSS}$ and number of cycles $N$ at peak shear strain $\gamma_p = 10\%$ in the cases with ISSS. The peak shear strain $\gamma_p$ is taken from shear stress vs. shear strain relations with and without ISSS $\tau_s$. The peak shear strain $\gamma_p$ is used instead of a double shear strain $\gamma_{DA}$ with ISSS (Vaid and Chern 1983; Hyodo et al. 1994). In Fig. 10 it is known that strength degradation happens with increasing ISSS $\tau_s$. The cyclic stress ratio $R_{DSS}$ vs. number of load cycle $N_c$ curves calculated by following the relation proposed by Seed et al. (1975) as:
Influence of initial static shear stress on post-cyclic degradation

\[ v_s = \frac{h}{\Delta t} \]  \hspace{1cm} (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 between shear stress versus shear strain and 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 similar cyclic stress ratio \( R_{DSS} \). In Fig. 13 it is known that the excess pore pressure \( \Delta u \) is increased with increasing ISSS. The post-cyclic shear strength is increased with ISSS, but the larger ISSS is the smaller post-cyclic shear strength becomes. Figure 14 demonstrates that post-cyclic shear strength tends to decrease with increasing normalized pore pressure \( \Delta u/\sigma'_v \) for the 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 \( \tau_{f,cy} \) is normalized by the static shear strength \( \tau_{f,Nci} \) without cyclic loading obtained from the static DSS test (see Fig. 6). In Fig. 15 it is known that the shear strength \( \tau_{f,cy} \) is increased with increasing ISSS \( \tau_s \).

Fig. 13 A set of static, cyclic and post-cyclic test results with ISSS
In Fig. 15(a), decrease of the strength ratio $\tau_{f,cy} / \tau_{f,NCi}$ with increasing cyclic load ratio $R_{DSS}$ becomes rapid at a certain value of the cyclic stress ratio $R_{DSS}$. With increasing ISSS $\tau_s$, the more marked decrease in the strength ratio $\tau_{f,cy} / \tau_{f,NCi}$ starts at a lower value of the cyclic stress ratio $R_{DSS}$. Figure 15(b) shows that the strength ratio $\tau_{f,cy} / \tau_{f,NCi}$ also decreases with increasing normalized pore pressure $\Delta u/\sigma_{c}$. Using the results from post-cyclic monotonic DSS tests, post-cyclic degradation of strength for non-plastic silt is formulated into:

$$\frac{\tau_{f,cy}}{\tau_{f,NCi}} = \left( 1 - A_1 \left( \frac{\Delta u}{\sigma_{c,NCi}} \right) \right)^{0.2} \exp \left( 1.53 \frac{\tau_s}{\sigma_{c,NCi}} \right)$$

(8)

where $A_1$ is a parameter to determine decreasing tendency of shear strength ratio $\tau_{f,cy} / \tau_{f,NCi}$, and the second order in Eq. (7) is expressed influence of ISSS $\tau_s$ from Fig. 6. The parameter $A_1$ is depended on ISSS $\tau_s$ like:

$$A_1 = 1.07 \exp \left( \frac{0.63 \tau_s}{\sigma_{c,NCi}} \right)$$

(9)

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 $G_{cy}$ is also normalized by the stiffness $G_{NCi}$ obtained from static test (Fig. 6). As well as Fig.15(a), Fig. 16(a) indicates that the slope of the degradation of the stiffness ratio $G_{max,cy} / G_{max,NCi}$ with

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

![Fig. 15 Post-cyclic induced degradation of strength](image)
Influence of initial static shear stress on post-cyclic degradation

increasing cyclic load ratio \( R_{DSS} \), rapidly increases at a certain value of the cyclic load ratio \( R_{DSS} \). With increasing ISSS \( \tau_s \), the more rapid decrease in the stiffness ratio \( \frac{G_{\text{max, cy}}}{G_{\text{max, NCi}}} \) is observed at a lower value of the cyclic load ratio \( R_{DSS} \). From Fig. 16(b) showing the stiffness ratio \( \frac{G_{\text{max, cy}}}{G_{\text{max, NCi}}} \) and normalized pore water pressure \( \frac{\Delta u}{\sigma_{v,NCi}'} \) relations, it is also clear that the stiffness ratio \( \frac{G_{\text{max, cy}}}{G_{\text{max, NCi}}} \) decreases with increasing normalized pore pressure \( \frac{\Delta u}{\sigma_{v,NCi}'} \). Using results tested the post-cyclic degradation of stiffness for non-plastic silt is estimated as:

\[
\frac{G_{v}}{G_{NG}} = \left(1 - A_2 \left(\frac{\Delta u}{\sigma_{v,NCi}'}\right)^{\frac{1}{3}}\right) \exp \left(-2.3 \frac{\tau_s}{\sigma_{v,NCi}'}\right) \tag{10}
\]

where \( A_2 \) is a parameter to determine decreasing tendency of stiffness ratio \( \frac{G_{\text{max, cy}}}{G_{\text{max, NCi}}} \), and the second order in Eq. (9) shows an influence of ISSS \( \tau_s \) formulated from Fig. 6 and is given as:

\[
A_2 = 1.04 \exp \left(5.23 \frac{\tau_s}{\sigma_{v,NCi}'}\right) \tag{11}
\]

As well as Eq. (7), Eq(9) is also different from the relation proposed by Yasuhara et al. (1994a, 1997, 1998). However, it is the same form as the relation proposed by Shimabukuro et al. (2000), which is valid for post-cyclic stiffness prediction of non-plastic silt in triaxial tests.

By comparing Figs. 15 with 16 it is concluded that post-cyclic degradation of strength and stiffness shows similar tendencies with each other. Also, the tendency of the strength ratio \( \frac{\tau_{c, cy}}{\tau_{c, NCi}} \) and the stiffness ratio \( \frac{G_{\text{max, cy}}}{G_{\text{max, NCi}}} \) tends to decrease similarly when those are plotted against the cyclic load ratio \( R_{DSS} \) and the normalized pore pressure \( \Delta u / \sigma_{v,NCi}' \).

CONCLUSION

In this paper, the post-cyclic strength and stiffness degradation of non-plastic silt was investigated using direct simple shear (DSS) tests. It was found that the strength and stiffness of non-plastic silt after liquefaction almost disappeared in the cases without initial static shear stress (ISSS). On the other hand, even if it was very difficult to determine the shear strength and stiffness of non-plastic silt with ISSS after cyclic loading, it was carefully investigated. 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. In particular, degradation in stiffness is more sensitive to ISSS than that in strength.

2) Both the strength and the stiffness to the normalized pore pressure decrease markedly with increasing ISSS.

3) Through the results from cyclic DSS tests on non-plastic silt, characteristics of post-cyclic degradation in strength and stiffness should depend on the fact that the generation of pore pressures during cyclic loading is marked with increasing ISSS.

4) Using the results from post-cyclic DSS tests, post-cyclic degradation relations for strength and stiffness for 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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