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# An Estimating Method for Post-cyclic Strength and Stiffness of Fine-grained Soils in Direct Simple Shear Tests

# 직접단순전단시험을 이용한 동적이력 후 세립토의 강도 및 강성 예측법

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## 요 지

반복삼축시험에 의한 반복하중 후 강도 및 강성의 예측법을 이용하여, 세립토에 대해서 직접단순 전단시험에서도 그 방법의 사용 가능성을 확인하여 보았다. 사용한 흙은 실트질 점토, 소성 실트와 비소성 실트이다. 반복삼축시험을 통해서 얻은 강도 및 강성 예측법을 직접단순 전단시험에 맞게 수정하여 시험 결과와 비교하였다. 특히, 세립토의 소성지수와 초기전단응력(ISSS)의 영향이 강조되었다. 연구결과는 (i) 세립토의 액상화강도비는 소성지수의 감소와 초기전단응력의 증가에 따라 감소한다. (ii) 등가강성과 전단변형률의 관계에 미치는 소성지수와 초기전단응력의 영향은 그리 크지 않다. (iii) 정규화한 과잉간극수압의 증가에 따른 강도비의 저하는 세립토의 소성지수가 증가할수록 느리다. (iv) 활성도가 큰 소성실트의 강성은 과잉간극 수압의 증가에 따라 급속히 감소한다. (v) 반복삼축시험 결과를 이용한 반복하중 후 강도 및 강성의 예측법을 이용하여 직접단순 전단시험 결과에 수정한 방법은 시험결과와 잘 어울리는 것으로 나타났다.

## Abstract

Based on an estimating method for post-cyclic strength and stiffness with cyclic triaxial tests proposed by one of the authors, cyclic Direct Simple Shear (DSS) tests were carried out to confirm whether the method can be adapted to DSS test on fine-grained soils: silty clay, plastic silt, and non-plastic silt. Results from cyclic and post-cyclic DSS tests were interpreted by a modified method as adopted for cyclic and post-cyclic triaxial tests. In particular, influence of plasticity index for fine-grained soils and initial static shear stress (ISSS) was emphasised. Findings obtained from the present study are: (i) liquefaction strength ratio of fine-grained soils decreases with decreasing plasticity index and increasing ISSS; (ii) plasticity index and ISSS did not markedly influence relation between equivalent cyclic stiffness and shear strain relations; (iii) the higher the plasticity index of fine-grained soils is, the less the strength ratio decreases with increment of a normalised excess pore water pressure (NEPWP); (iv) stiffness ratio of plastic silt has large activity decrease rapidly with increasing excess pore water pressure; and (v) post-cyclic strength and stiffness results from DSS tests agree well with those predicted by the method modified from a procedure used for triaxial test results.

**Keywords :** Direct simple shear test, Initial static shear stress, Plasticity index, Post-cyclic strength and stiffness, Silt, Strength and stiffness degradation

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# 1. Introduction

According to recent earthquakes, it has been reported that fine-grained soils liquefied and led to lateral flow (Stark and Contreras, 1998; Boulanger et al., 1998; Hamada et al., 1999; JGS, 2000a; Shimamoto et al., 2001). Especially, it should be noted that damaged sites were close to rivers or seas and ground water levels were also very close to the ground surface.

For example, during Loma Prieta Earthquake having occurred in 1989, maximum settlement and lateral deformation that occurred around the Moss Landing Marine Laboratory (MLML) at Moss Landing in California were 0.35 m and 2.1 m, respectively. The MLML was located very close to the Old Salinas River and the Monterey Bay. And the MLML elevation is about 3 m higher than the ground water level of Monterey Bay and the Old Salinas River (Boulanger et al., 1998). Samples from a boiled clayey silt had a plasticity index of 17; it comprised 78% fines less than 75 $\mu$m and 24% fines less than 5 $\mu$m. From the triaxial test, very high excess pore water pressure and large shear strain developed even though liquefaction was not found (Boulanger et al., 1998).

During the 1999 Kocaeli, Turkey Earthquake, the soil type of damaged areas is silt or silty sand in Adapazari (Hamada et al., 1999; JGS, 2000a). The Adapazari City was founded on alluvial stratum; it was composed with residual soils transferred from rivers about 200 years ago. The Adapazari City ground constituted silty clay and non-plastic silt. The ground water level was 1-3 m below surface. Through the 1999 Kocaeli, Turkey Earthquake, the severest damage to structures due to liquefaction was observed in Adapazari City. Many buildings settled, tilted, or totally collapsed due to silt or silty sand liquefaction. Settlement of buildings up to 1.1 m was observed. Typically, buildings suffered severe tilting resulting from loss of bearing capacity of foundation ground.

During the 2000 Tottoriken-seibu Earthquake in Japan, liquefaction damage occurred within silt at the Takenouchi industrial complex (Shimamoto et al., 2001). There existed non-plastic silt that had a 2.4 - 5.6 uniformity coefficient $U_c$ and 50 - 100% fines. It was reported that settlement and lateral flow occurred by about 0.1 - 0.3 m and 0.2 m, respectively.

Through the facts mentioned above, this study attempts to investigate whether fine-grained soil has a very high potential of liquefaction and lateral flow. In particular, the effect of an initial static shear stress (ISSS) is regarded as an important issue in relation to stability of slow slope consisted of fine-grained soil beneath structures. Thus, cyclic and post-cyclic degradation of strength and stiffness for clay, plastic and non-plastic silt have been investigated using NGI-type DSS (Direct Simple Shear) tests (Bjerrum and Landva, 1966). The present paper presents the following:

(1) Degradation of strength and stiffness of fine-grained soils are investigated by DSS tests.
(2) Cyclic strength and stiffness characteristics of fine-grained soils are studied with an emphasis on the influence of plasticity index and initial static shear stress (ISSS).
(3) By modifying the methods previously proposed for triaxial tests, a predictive method of post-cyclic shear behaviour in DSS tests is proposed.

# 2. Experimental Program

## 2.1 Testing Apparatus

Using the NGI-type DSS apparatus at Ibaraki University cyclic direct simple shear tests were performed. Vertical and horizontal stresses up to 5 MPa by air pressure and 2 MPa by oil pressure can be applied to each specimen. Vertical and horizontal displacement can be measured up to 10 mm using strain gauges. Each specimen, with 30 mm height and 70 mm diameter, is contained in a wire-reinforced membrane (Song, 2003; Song et al., 2003). The NGI-type DSS apparatus is capable of performing tests under the $K_o$ condition during preconsolidation.

## 2.2 Testing Condition

All specimens were prepared by reconstitution with silty clay, plastic silt and non-plastic silt, respectively. Specimens of Keuper Marl Silty Clay (KM Clay; silty clay) and Arakawa Clayey Silt (Arakawa Silt; plastic silt) were prepared by preconsolidation of slurry. Slurry was controlled to be about twice the liquid limit (LL) before preconsolidation (JGS, 2000b). A large (0.2m in diameter and 0.3m in height) cylindrical vessel was used to preconsolidate slurry under 49 kPa and 69 kPa vertical pressure for Arakawa Silt and KM Clay, respectively. Specimens of DL Clayey Silt (DL Silt; non-plastic silt) had similar relative density $D_r$ of about 42%, because it had no plasticity.

A stress-controlled cyclic shear stress $\tau_{f,cy}$ was applied to specimens with an initial vertical effective consolidation stress $\sigma_{vc}'$ = 98 kPa or 196 kPa under constant volume conditions; this concurs with the fact that the liquefied layer due to earthquakes was located at about 20m maximum depth (JGS, 2000a; Boulanger et al., 1998; Hamada et al., 1999; Shimamoto et al., 2001). Cyclic load frequency was 0.1 Hz with a sinusoidal wave for all tests. ISSS $\tau_s$ was applied up to 29.4 kPa under a constant rate of loading of 196 kPa/hr under a constant stress condition. Table 1 shows other testing conditions. Post-cyclic strength and stiffness of Arakawa Silt and DL Silt were investigated under stress control, but those of KM Clay were tested under both stress and strain control. Especially, post-cyclic induced degradation of strength and stiffness for KM Clay were adapted to the procedure by strain control. Since the post-cyclic induced decreasing strength and stiffness related to increment of excess pore water pressure, it is believed that there is no influence of difference between stress and strain control methods.

It is generally admitted that cyclic-degradation of soils induces instability of earth structures and foundations during earthquakes. To confirm this for fine-grained soils in the laboratory, monotonic DSS tests under identical conditions as cyclic DSS tests were performed after cyclic DSS tests on reconstituted silty soils. Table 1 shows that strain rates in post-cyclic monotonic DSS tests were 0.1%/min or 0.2%/min, respectively, for three soils; the tests continued until shear strain of 20% was attained. Void ratios of specimens are shown in Table 2. It was known that the void ratio of KM Clay after consolidation was the smallest according to the difference of preloading for soils in Table 1.

Table 1. Soil test conditions for cyclic and post-cyclic conditions

| Items | KM Clay | Arakawa Silt | DL Silt |
|---|---|---|---|
| Method of making specimen | Preloading(69 kPa) | Preloading(49 kPa) | Relative density $D_r$ = 42% |
| Soil test | Direct simple shear test | | |
| Test condition | Constant volume condition | | |
| Confining stress | 196 kPa / 98 kPa* | 196 kPa | |
| Rate of application of confining stress | 196 kPa/hr | | |
| Wave form | Sinusoid with 0.1Hz | | |
| Max. number of Load cycles | 50 | 20 | 50 |
| Control method | Stress Control / Strain *Control | Stress control | |
| Peak strain in cyclic loading | Double amplitude shear strain 10% | | |
| Initial static shear stress ratio | 0, 0.05, 0.1, 0.15 / 0* | 0, 0.05, 0.1, 0.15 | 0, 0.05, 0.1 |
| Rate of ISSS | 196 kPa/hr under constant stress condition | | |
| Resting time after cyclic loading | 10 min | 30 min | 10 min |
| Rate of static shear strain after cyclic loading | 0.1 %/min | 0.2 %/min | 0.1 %/min |
| Maximum shear strain in post-cyclic loading | 20% | 20% | 20% |

* This mark is adapted only for post-cyclic test of KM Clay

Table 2. Specimens and soils condition

| Items | KM Clay | | Arakawa Silt | DL Silt |
|---|---|---|---|---|
| Initial Void ratio | 0.85 | 0.83* | 1.20 | 1.16 ($D_r$ = 42%) |
| Void ratio after consolidation | 0.68 | 0.64* | 0.94 | 0.94 ($D_r$ = 70%) |
| Particle Density $\rho_s$ (kN/m³) | 27.4 | | 26.2 | 24.8 |
| Liquid Limit $w_L$ (%) | 38.6 | | 45.1 | 25.1 |
| Plastic Limit $w_P$ (%) | 19.0 | | 27.8 | - |
| Plasticity Index $I_P$ | 19.6 | | 17.3 | - |
| Cohesion c' (kPa) | 0 | | 0 | 0 |
| Friction Angle $\phi'$ (°) | 25 | | 24 | 23 |
| Compression Index $C_c$ | 0.23 | | 0.27 | 0.10 |
| Swelling Index $C_s$ | 0.042 | | 0.046 | 0.003 |

* This mark is adapted only for post-cyclic test of KM Clay

## 2.3 Material Properties

Soil properties of the three soils are summarised in Table 2. Grain size distribution curve is shown in Fig. 1. Figure 2 shows activity of two soils except that of DL Silt. Figure 2 shows that Arakawa Silt has larger activity $A_c$ (= PI / clay content) than KM Clay, even if both have a similar plasticity index. Arakawa Silt is closer to Montmorillonite ( $A_c$ = 7.2) than KM Clay is. It is well known that Montmorillonite has weaker soil structure than Illite ( $A_c$ = 0.75) (Kim, 1998). From these facts, it can be inferred that larger shear strain may occur in Arakawa Silt than in KM Clay under identical severity of cyclic loading.

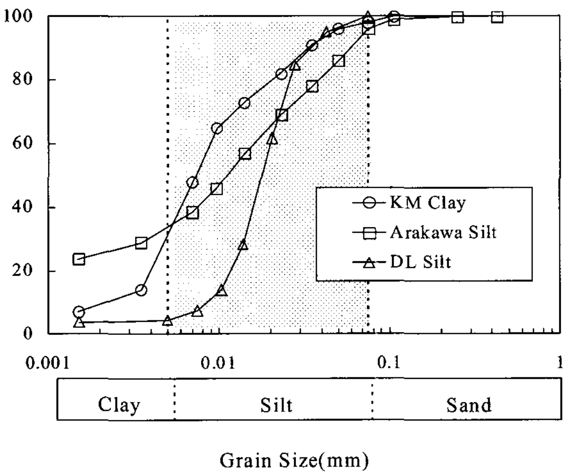



Fig. 1. Grain size distribution curves for fine-grained soils

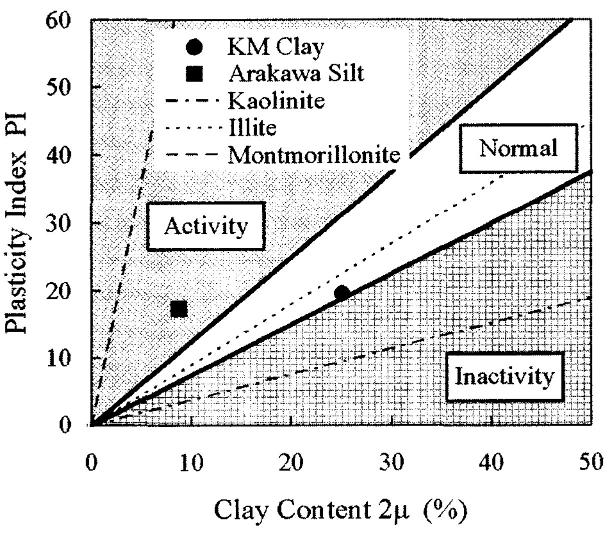
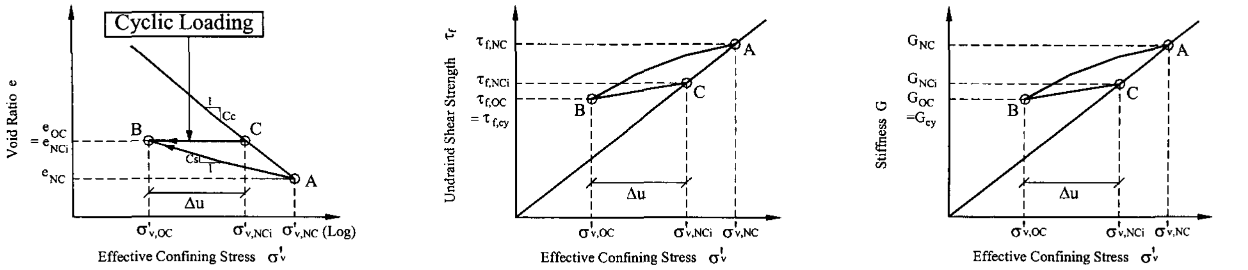



Fig. 2. Activities of fine-grained soils

# 3. Basic Concepts for Soil Tests

A number of researchers have investigated strength and stiffness changes in soils after cyclic loading. Regarding degradation in sand stiffness, for example, Yasuda et al. (1992, 1995) and Nagase et al. (1997) investigated post-cyclic stiffness degradation in Toyoura Sand using both triaxial and torsional shear equipment; they then correlated the change in stiffness of sand with excess pore water pressure generated during cyclic loading. Thereafter, test results were incorporated in a numerical analysis to predict liquefaction-triggered lateral deformation of sand. On the other hand, although comparatively fewer studies have been conducted on post-cyclic behaviour of cohesive soils, Matsui et al. (1980, 1992), Yasuhara (1985, 1994, 1997, 1999), Yasuhara and Toyota (1997) and Yasuhara et al. (1983, 1992, 2000) have




(a) Void Ratio - Effective Confining Stress (b) Strength - Effective Confining Stress (c) Stiffness - Effective Confinging Stress

Fig. 3. Key sketches for void ratio, shear strength and stiffness vs. consolidated stress relations

proposed methods of predicting degradation in strength and stiffness for cohesive soils using cyclic triaxial tests. In this paper, the method for post-cyclic degradation of strength and stiffness for triaxial tests is modified to analyse post-cyclic behaviour in DSS tests.

Figure 3 shows the basic concept of predicting strength and stiffness after cyclic load using excess pore water pressure. The assumption is that if the swelling path (line AB) at unloading in a static state matches decrease (line CB) due to cyclic load, then strength and stiffness after the cyclic load can be predicted without a cyclic test (Yasuhara 1985, 1994, 1997, 1999; Yasuhara et al., 2000). Some specimens were used for monotonic DSS tests to obtain shear strength and stiffness without cyclic loading history; others were tested to investigate post-cyclic strength and stiffness from monotonic loading subsequent to constant volume cyclic DSS tests. As indicated in Fig. 3, pore water pressures $\Delta u$ during DSS tests performed under the constant volume condition can be determined by a difference between before and after vertical effective stress given as $\sigma_v'{}_{,NCi} - \sigma_v'{}_{,OC}$.

## 4. Results of Cyclic Strength and Stiffness

### 4.1 Relation of Shear Stress vs. Shear Strain and Effective Vertical Stress

Figure 4 shows a typical set of stress-stain curves from DSS tests on three soils. In this series of tests, initial static shear stress ratio (ISSSR) is about 0.1 for three soils, and cyclic shear stress ratio CSSR is about 0.07 for DL Silt, and 0.1 for KM Clay and Arakawa Silt. The definition of ISSSR is defined by:

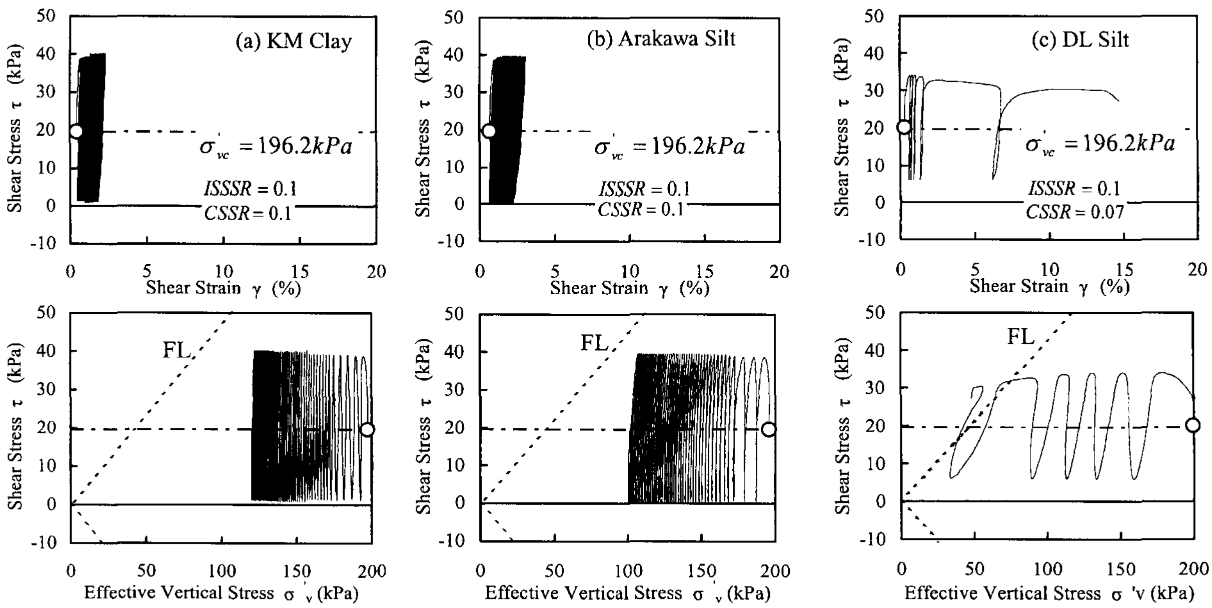



Fig. 4. Typical examples of shear stress vs. shear strain and effective vertical stress of the specimen with ISSS

$$\text{ISSSR} = \frac{\tau_s}{\sigma'_{vc}} \tag{1}$$

where $\tau_s$ is initial static shear stress(ISSS), $\sigma_{vc}'$ is an effective vertical stress. And the definition of cyclic shear stress ratio CSSR is defined by (Song et al., 2003):

$$\text{CSSR} = \frac{\tau_{f,cy}}{\sigma'_{vc}} \tag{2}$$

where $\tau_{f,cy}$ is cyclic shear stress. Therefore, it can be said that the result from cyclic DSS tests on three kinds of soil were carried out under almost identical conditions. Figure 4 confirms that shear strain $\gamma$ of DL Silt is lager than that of other soils under cyclic loading even if the CSSR of DL Slit is smaller. This means that cyclic failure in DL Silt takes place more easily than in the others even under ISSS $\tau_s$ being applied. The shear strain of Arakawa Silt in Fig. 4 is slightly larger than that of KM Clay. Moreover, an increased amount of excess pore water pressure $\Delta u$ that is equivalent to decreased effective vertical stress in Arakawa Silt is larger than that in KM Clay. These tendencies were suggested in Fig. 3.

## 4.2 Cyclic Strength

One possible definition of cyclic strength of fine-grained soil follows the definition of sand liquefaction strength. Liquefaction strength can normally be known from the relation between CSSR and the number of load cycles $N_c$ obtained from a series of cyclic DSS tests with either three or four different cyclic shear stress $\tau_{f,cy}$ levels as presented in Fig. 5. Normally, in cyclic triaxial test the double amplitude of axial strain $\varepsilon_{DA}$ of 5% is used to define cyclic strength of sandy soils. Following the testing method of JGS (JGS, 2000b) for cyclic simple shear test without ISSS, the shear strain $\gamma_{DA}$ of 7.5% was adopted in this study to define cyclic failure. On the contrary, in cases with ISSS, cyclic failure was defined when maximum cyclic shear strain $\gamma_{cy}$ reached 7.5% (Song et al., 2003).

Figure 5 shows the relation of CSSR versus number of load cycles $N_c$ on three soils with different plasticity indexes. It is observed from Fig. 5 that cyclic strength of each soil decreases with ISSSR, irrespective of the plasticity index of soils used in this study. This tendency is the same as observed for cohesive soils by Hyodo et

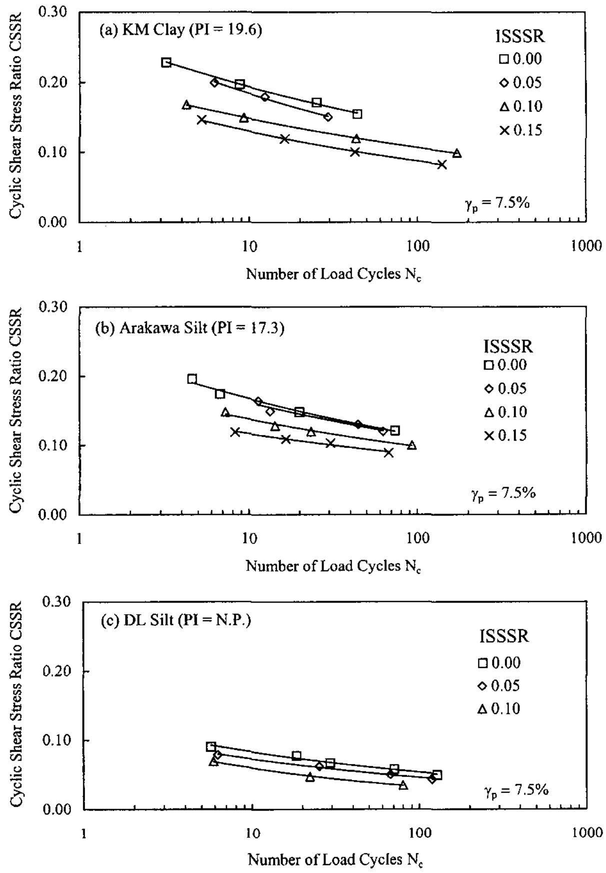



Fig. 5. Relation between cyclic shear stress ratio and number of cycles

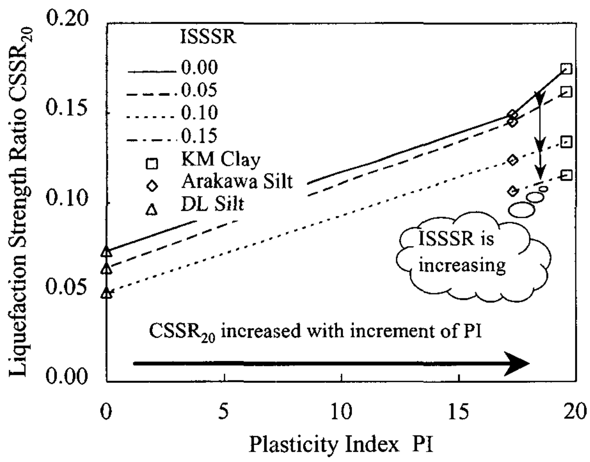



Fig. 6. Relation between liquefaction strength ratio and plasticity index

al. (1994b); a reverse tendency in sands was observed by Vaid and Chern (1983) and Hyodo et al. (1994a). Yet, it is also true that cyclic strength depends on initial specimen conditions, such as density and saturation.

Figure 6 shows that CSSR increases with increasing plasticity index PI. Using results in Fig. 5, Fig. 6 shows the relation between liquefaction strength ratio $CSSR_{20}$ and plasticity index PI; therein, the plasticity index PI of DL Silt is zero even if it is non-plastic silt. Figure 6 indicates that cyclic stress ratio CSSR increases with increasing plasticity index PI.

### 4.3 Cyclic Stiffness

Figure 7 shows relations between equivalent stiffness versus the single amplitude shear strain $\gamma_{SA}$ for three soils; it includes maximum stiffness $G_{max}$ form static DSS test together with results from each cyclic loading step in DSS tests. Note that all results considering ISSS were plotted in Fig. 7. The definition of equivalent stiffness $G_{eq}$ was determined by (Song et al., 2003):

$$G_{eq} = \frac{\tau_{max} - \tau_{min}}{\gamma_{max} - \gamma_{min}} \tag{3}$$

The results in which ISSS equals zero compared with the solid line that is the Hardin-Drnevich model in Fig. 7 are:

$$G_{eq} = \frac{G_{max}}{1 + (\gamma_{SA} / \gamma_r)} \tag{4}$$

where $G_{eq}$ is equivalent stiffness, $G_{max}$ is maximum stiffness, $\gamma_{SA}$ is single amplitude shear strain, and $\gamma_r$ is standard shear strain (Hardin and Drnevich, 1972). Figure 7 shows that equivalent stiffness slightly decreases with increasing ISSS. Equivalent stiffness $G_{eq}$ determined from Eq. (3) could not be obtained within the single amplitude of shear strain $\gamma_{SA}$ in the small strain region (equal to $10^{-5}$ - $10^{-3}$), because DSS test apparatus was useful within middle and large strain regions (over $10^{-4}$) using DSS test (JGS, 2000b). In Fig. 7, it is interesting to note that relations between equivalent stiffness and shear strain

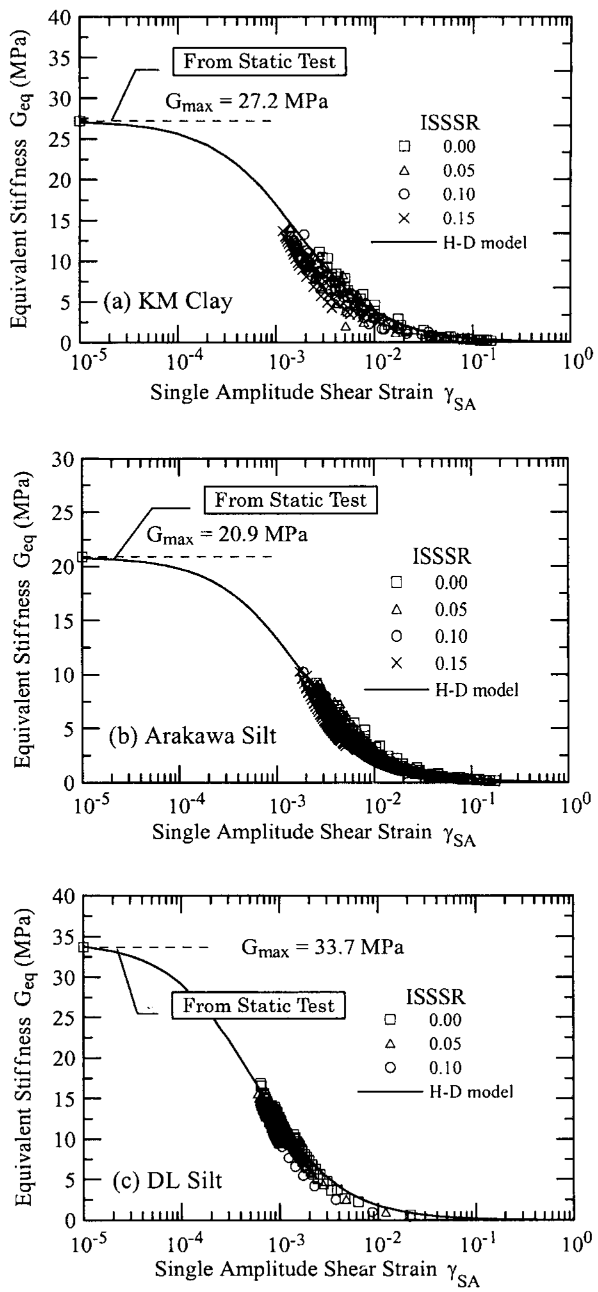



Fig. 7. Equivalent stiffness and cyclic shear strain relations

appear to be less influenced by ISSS. The maximum stiffnesses $G_{max}$ obtained were 27.2 MPa for KM Clay, 20.9 MPa for Arakawa Silt, and 33.7 MPa for DL Silt, respectively, as shown in Fig. 7. Equivalent stiffness corresponding to a certain value of single amplitude shear strain $\gamma_{SA}$ decreases slightly with decreasing plasticity index PI and increasing ISSS.

## 5. Predictive Methods of Strength and Stiffness After Cyclic Loading

### 5.1 Prediction of Post-cyclic Degradation of Strength

Ladd et al. (1977) proposed an empirical equation for the undrained strength ratio between over-consolidated clay and normally consolidated clay measured at DSS tests; it is given as:

$$\left(\frac{\tau_f}{\sigma_v'}\right)_{OC} \Big/ \left(\frac{\tau_f}{\sigma_v'}\right)_{NC} = OCR^{\Lambda_0} \tag{5}$$

where $(\tau_f/\sigma_v')$ is the normalised, undrained shear strength of overconsolidated soils, $(\tau_f/\sigma_v')_{NC}$ is the normalised, undrained shear strength of normally-consolidated soils, $OCR$ is the overconsolidated ratio equal to $\sigma_{v,NC}'/\sigma_{v,OC}'$ as defined in Fig. 3(a) and $\Lambda_0$ is an empirical constant. This modification from a predictive method for strength degradation proposed by Yasuhara (1985) in cyclic triaxial test is:

$$\frac{\tau_{f,cy}}{\tau_{f,NCi}} = \left(1 - \frac{\Delta u}{\sigma_{v,NCi}'}\right)^{1-\frac{\Lambda_0}{1-\lambda}} \tag{6}$$

where $\lambda$ is defined as $C_s/C_c$ ( $C_s$ : swelling index, $C_c$ : compression index).

Equation (6) indicates that post-cyclic degradation of strength is a function of excess pore water pressure normalised by vertical consolidated stress, which is equal to initial vertical stress in normal consolidation (see Fig. 3). Equation (6) proposed by Yasuhara et al. (1992) represents an attempt to prove whether the results from DSS tests could be used to predict post-cyclic strength degradation instead of those from cyclic and post-cyclic triaxial tests.

### 5.2 Post-cyclic Stiffness Degradation

It is well known that decrease in clay stiffness is proportional to the decrease of effective stress and increase in OCR (Ladd et al., 1977). In this respect, it is reasonable to assume that an empirical equation proposed by Worth and Houlsby (1985) for the ratio of normalised stiffness of over-consolidated clay to that of normally consolidated clay should also be valid in DSS tests in terms of the relation:

$$\left(\frac{G}{\sigma_v'}\right)_{OC} \Big/ \left(\frac{G}{\sigma_v'}\right)_{NC} = 1 + C \cdot \ln(OCR) \tag{7}$$

where $(G/\sigma_v')_{OC}$ is the normalised over-consolidated stiffness, $(G/\sigma_v')_{NC}$ is normalised normally consolidated stiffness, OCR is the overconsolidation ratio and C is an empirical constant. Following the same technique proposed by Yasuhara et al. (1992) for post-cyclic degradation in stiffness in DSS tests, Eq. (7) can be rewritten as:

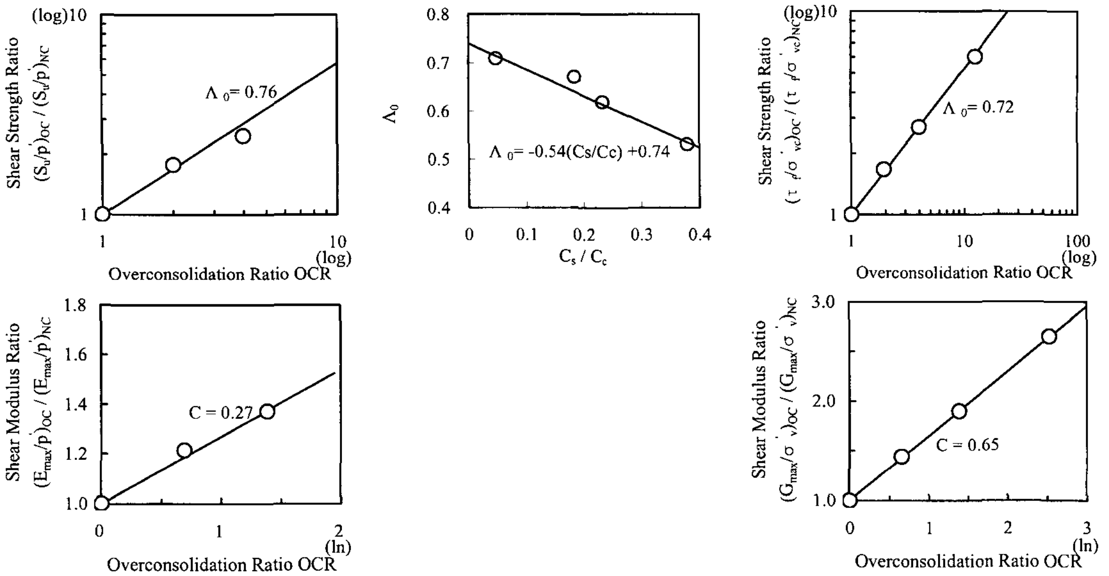
$$\frac{G_{cy}}{G_{NCi}} = \left(1 - \frac{\Delta u}{\sigma_{v,NCi}'}\right)\left\{1 - \frac{C}{1-\lambda}\ln\left(1 - \frac{\Delta u}{\sigma_{v,NCi}'}\right)\right\} \tag{8}$$

Equation (8) indicates that post-cyclic degradation of stiffness as well as post-cyclic strength is a function of excess pore pressure normalised by initial vertical consolidated stress. Subsequently, we discuss whether the same type of relation given by Eq. (8) as proposed for predicting results from post-cyclic triaxial tests is applicable for results of DSS tests (Yasuhara, 1997).

## 6. Verification of Methods for Post-cyclic Degradation of Strength and Stiffness

### 6.1 Post-cyclic Effective Stress Paths and Stress-strain Relations

As previously described, post-cyclic degradation of strength and stiffness was investigated using constant volume monotonic DSS tests after cyclic loading on the three kinds of soils. Parameters $\Lambda_0$ and $C$ for KM Clay and DL Silt included in Eqs. (6) and (8) were determined by triaxial tests and monotonic DSS tests, respectively. However, because of lack of specimens in this laboratory, parameters $\Lambda_0$ and $C$ for Arakawa Silt could not be found out. Therefore, it is impossible except compelling to fix directly the results from DSS tests on Arakawa Silt. To




(a) KM Clay (b) Arakawa Silt (c) DL Silt

Fig. 8. Methods for determining the parameters $\Lambda_0$ and C

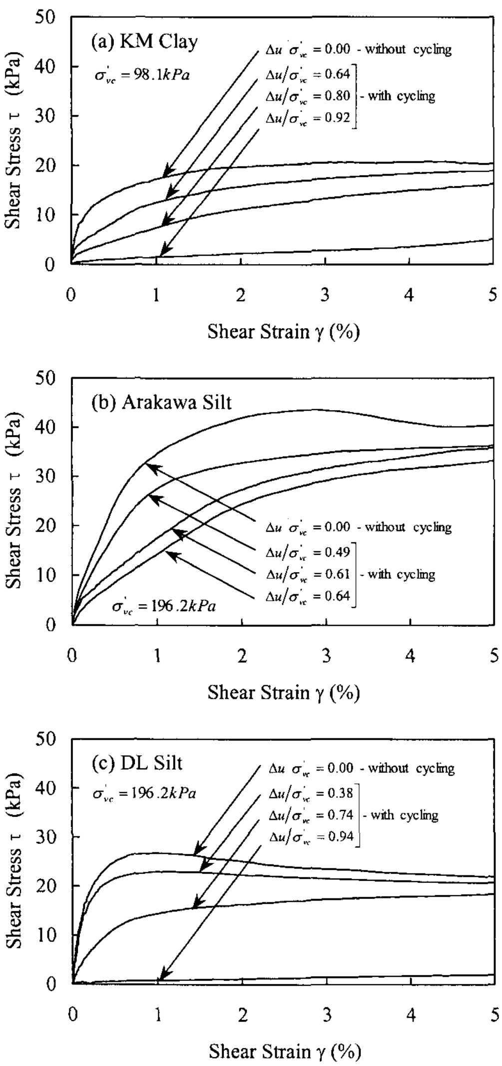



Fig. 9. Post-cyclic shear stress vs. shear strain relations

determine parameter $\Lambda_0$ of Arakawa Silt, test results on parameter $\Lambda_0$ for many kinds of soils from previously published references collated by Mayne (1980) were used. Among them, a suitable value for $\Lambda_0$ was selected from the results on many kinds of clay whose properties, as liquid limit LL, plasticity index PI, frictional angle $\phi'$, compression index $C_c$, and swelling index $C_s$ are similar to those for Arakawa Silt. Following the above-stated method, parameter $\Lambda_0$ of Arakawa Silt was determined, but parameter $C$ of Arakawa Silt could not be obtained except through estimation by fitting results of monotonic tests after cyclic DSS tests. Determination methods for parameters are shown in Fig. 8.

Post-cyclic stress - strain curves on three soils are compared in Fig. 9. The most marked difference in Fig. 9 is that non-plastic silt (DL Silt) loses stiffness even under the small magnitude of excess pore water pressures $\Delta u$ generated by cyclic loading.

## 6.2 Post-cyclic Strength

Figure 10 shows comparisons between a measured and predicted post-cyclic strength for three kinds of soils. Predicted curves were obtained using Eq. (6). Figure 10 indicates that the strength ratio, $\tau_{f,cy}/\tau_{f,NCi}$, decreases with increasing NEPWP $\Delta u/\sigma_{vc}'$. Although this tendency is common with three kinds of soils, the decrease of strength ratio $\tau_{f,cy}/\tau_{f,NCi}$ for DL Silt is most marked, while that for KM Clay is less marked. As shown in Fig. 10, results predicted using Eq. (6) agree well with results measured in DSS tests. However, there is a marked deviation between predicted and measured values for both materials beyond NEPWP $\Delta u/\sigma_{vc}' = 0.8$ for KM Clay, as the shaded area shows. Soil structural deterioration must occur rapidly beyond the region exceeding NEPWP $\Delta u/\sigma_{vc}' = 0.8$; this is nearly equal to 0.8 and is indicated by an arrow in Fig. 10. Results in Fig. 10 also indicate that even if selected parameter $\Lambda_0$ is not directly investigated by soil tests for Arakawa Silt, parameter $\Lambda_0$ estimated by previously extrapolating con ducted testing data (see Fig. 8a) is reliable because of the good agreement between predicted and measured results.

### 6.3 Post-cyclic Stiffness

In Fig. 11, the degradation pattern of stiffness ratio $G_{cy}/G_{NCi}$ with increasing NEPWP $\Delta u/\sigma_{vc}'$ shows the same tendency for KM Clay and DL Silt, but shows a different one for Arakawa Silt. Stiffness ratio $G_{cy}/G_{NCi}$ for Arakawa Silt decreases rapidly in the early stage of NEPWP $\Delta u/\sigma_{vc}'$, but that for KM Clay and DL Silt tends to decrease slowly with increasing NEPWP $\Delta u/\sigma_{vc}'$. One reason for this tendency might be the difference of the applied rate of shear strain in monotonic tests after cyclic tests. Since the applied rate (0.2%/min) of cyclic shear stress for Arakawa Silt was slightly higher than that of the others (0.1%/min), a less-precise predic tion of post-cyclic stiffness $G_{cy}$ for Arakawa Silt might be obtained. A second reason must be the different activity $A_c$ between Arakawa Silt and KM Clay as mentioned in Fig. 2. This accords with the marked decrease of stiffness ratio for Arakawa Silt in contrast with that for KM Clay. Although parameter $C$ of Arakawa Silt was not determined directly by static tests, the predicted line of Arakawa Silt in Fig. 11 matched measured results well. This means that Eq. (8) is applicable to predicting stiffness degradation of soils with and without plasticity index.

### 6.4 The Relation Between Post-cyclic Degradation in Strength and Stiffness

Combining Eq. (6) with Eq. (8), we can correlate the post-cyclic stiffness ratio $G_{cy}/G_{NCi}$ with the post-cyclic stress ratio $\tau_{f,cy}/\tau_{f,NCi}$.

$$\frac{G_{cy}}{G_{NCi}} = \left(\frac{\tau_{f,cy}}{\tau_{f,NCi}}\right)^{\frac{1-\lambda}{1-\lambda-\Lambda_0}} \left\{1 - \frac{C}{1-\lambda-\Lambda_0}\ln\left(\frac{\tau_{f,cy}}{\tau_{f,NCi}}\right)\right\} \quad (9)$$

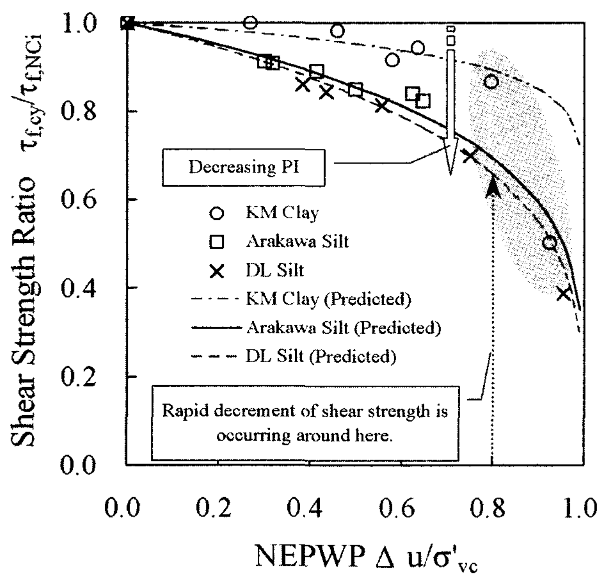



Fig. 10. Post-cyclic undrained shear strength ratio vs. normalized excess pore pressure relations

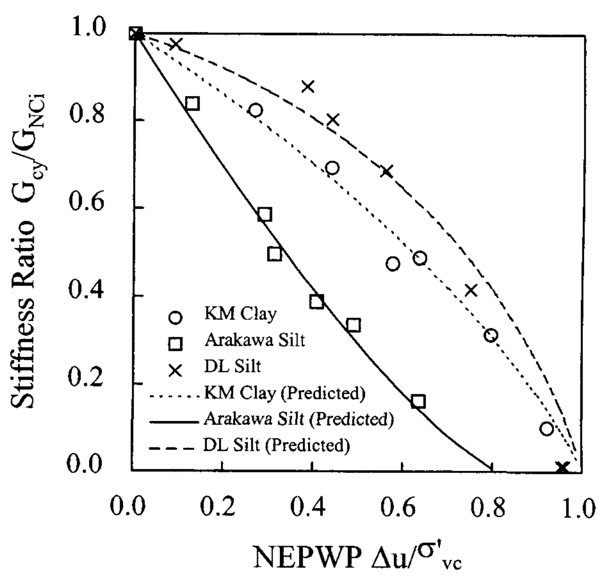



Fig. 11. Relation between post-cyclic shear modulus ratio and normalized excess pore pressure

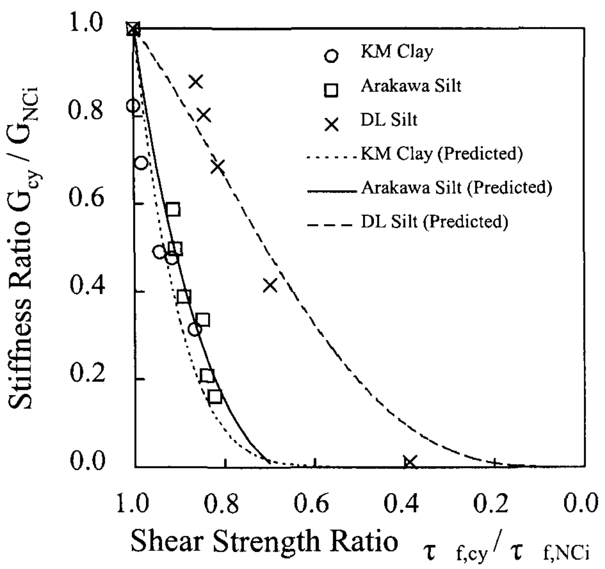



Fig. 12. Post-cyclic stiffness ratio vs. shear strength ratio relations

Figure 12 compares measured data with results predicted by Eq. (9). According to Fig. 12, decrease of stiffness ratio $G_{cy}/G_{NCi}$ with decreasing strength ratio $\tau_{f,cy}/\tau_{f,NCi}$ for plastic silts is smaller than that for DL Silt. However, predicted results generally agree well with measured results for all silty soils used in this study.

## 7. Conclusions

Strength and stiffness characteristics of normally consolidated, reconstituted fine-grained soils during and after cyclic loadings were investigated using NGI-type direct simple shear (DSS) testing apparatus. Among the three soils used in this study, one is silty clay; others are plastic silt and non-plastic silt. The influence of plasticity index and initial static shear stress (ISSS) on cyclic and post-cyclic behaviour of silty soils is particularly emphasized. The following results were obtained by investigation:

(1) Cyclic strength of fine-grained soils decreases with decreasing plasticity index of soils and with increasing ISSS. The tendency of fine-grained soils differs completely from that of sand in cyclic triaxial test, but is quite similar to that of clay, as reported by Hyodo et al. (1994a, 1994b).

(2) Equivalent stiffness vs. shear strain relations in the medium to large strain regions are not influenced markedly by plasticity index and ISSS; however, equivalent stiffness corresponding to a certain value of single amplitude shear strain $\gamma_{SA}$ decreases slightly with decreasing plasticity index PI and increasing ISSS.

(3) Post-cyclic shear strength decrement in plastic silt is more marked than that in clay corresponding to the same magnitude of NEPWP $\Delta u/\sigma_{vc}'$.

(4) Both post-cyclic strength and stiffness can be predicted as a function of NEPWP $\Delta u/\sigma_{vc}'$ except for the region in the vicinity of which cyclic failure of specimens is nearly reached. Yasuhara (1985, 1997) previously proposed it in terms of investigation of cyclic and post-cyclic triaxial tests on cohesive soils.

(5) It is known that the relation between post-cyclic strength and stiffness in DSS tests can be predicted by a modified method of the basic procedure proposed by Yasuhara (1985, 1997) for post-cyclic triaxial tests.

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