

Experiments and elastoplastic analyses on soil disturbance of soft clay subjected to cyclic loading

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Abstract

Although liquefaction of sandy soils is widely recognized as a major cause of seismic damage, recent earthquakes have revealed that soft clayey grounds can also suffer significant damage. To investigate this, we conducted cyclic undrained triaxial compression/extension tests with axial strain control on undisturbed soft clay samples. The results showed that the undrained peak shear strength decreased by up to 25% with increasing number of cyclic loadings, indicating that structural degradation plays a key role. In contrast, remolded samples with a de-structured remolded sample exhibited no such strength reduction. To interpret these observations, we employed an elastoplastic constitutive model incorporating the concept of soil skeleton structure. Numerical simulations successfully reproduced the experimental trends and confirmed that the strength reduction is attributable to structural deterioration. These findings suggest that cyclic loading induces disturbance by degrading the soil structure, leading to a significant reduction in shear strength. This study highlights the importance of considering seismic damage in soft clayey sites, which is often overlooked in practice. Furthermore, the structure-based modeling approach provides a useful framework for evaluating post-earthquake performance and improving seismic design strategies for clayey ground conditions.

Key words: soft clay, soil disturbance, cyclic loading, soil skeleton structure concept

Résumé

Bien que la liquéfaction des sols sableux soit largement reconnue comme une cause majeure de dommages sismiques, des tremblements de terre récents ont révélé que les sols argileux mous peuvent également subir des dommages importants. Pour étudier cette question, nous avons effectué des essais cycliques de compression/extension triaxiale non drainée avec contrôle de la déformation axiale sur des échantillons d'argile molle non perturbés. Les résultats ont montré que la résistance au cisaillement maximale non drainée diminuait jusqu'à 25 % avec l'augmentation du nombre de charges cycliques, ce qui indique que la dégradation structurelle joue un rôle clé. En revanche, les échantillons remodelés avec un échantillon remodelé déstructuré ne présentaient pas une telle réduction de la résistance. Pour interpréter ces observations, nous avons utilisé un modèle constitutif élastoplastique incorporant le concept de structure du squelette du sol. Les simulations numériques ont reproduit avec succès les tendances expérimentales et ont confirmé que la réduction de la résistance est attribuable à la détérioration de la structure. Ces résultats suggèrent que le chargement cyclique induit des perturbations en dégradant la structure du sol, ce qui entraîne une réduction significative de la résistance au cisaillement. Cette étude souligne l'importance de prendre en compte les dommages sismiques dans les sites argileux tendres, ce qui est souvent négligé dans la pratique. En outre, l'approche de modélisation basée sur la structure fournit un cadre utile pour l'évaluation de la performance post-séisme et l'amélioration des stratégies de conception sismique pour les sols argileux.

Mots-clés : argile molle, perturbation du sol, charge cyclique, concept de structure du squelette du sol

1. Introduction

The 1964 Niigata and Alaska earthquakes highlighted liquefaction in sandy ground, leading to a primary research focus on its instability. Past earthquakes have also caused damage to clayey ground, though less frequently than liquefaction-induced damage in sandy soil. During the 1985 Mexico earthquake, 13% of all 5- to 15-story buildings located

within the downtown area underwent severe settlements and tilts, and in one case the structure toppled due to soil foundation shear failure (Mendoza and Romo 1998; Merlos and Romo 2006). Mexico City is in a basin, on land where Lake Tescoco was reclaimed, and its soil profile is composed of extremely soft volcanic ash clay. Similar damage to a building on clayey ground due to a loss of bearing capacity was also

observed in the 1999 Chi-Chi earthquake in Taiwan where low plasticity clay had thickly deposited (Chu et al. 2008). Also, during the 1989 Loma Prieta earthquake, fine-graded clayey ground showed lateral spreading of up to 1.3 m (Boulanger et al. 1998). In addition to instabilities during earthquakes, large deformation after earthquakes have also been reported. Settlement that had once converged before the earthquake accelerated during the earthquake, and long-term deformation was observed in the 1958 Mexico earthquake (Zeevaert 1972), the 1978 off the coast of Miyagi earthquake (Touno 1993; Towhata 2008), the 2007 Chuetsu earthquake (Isobe and Ohtsuka 2013), the 1995 Great Hanshin earthquake (Matsuda 1997; Sato et al. 2018), and the 2011 off the Pacific coast of Tohoku earthquake (Nigorikawa and Asaka 2015; Sento 2020). What these settlement-damaged areas of the grounds have in common is that they were composed of soft clay with high water content.

In response to seismic damage to clayey grounds, many studies have been conducted to understand the dynamic characteristics of clayey soil. Brown et al. (1975), Yasuhara et al. (1982), Gu et al. (2012), Wichtmann and Triantafyllidis (2018), and Khan et al. (2020) conducted cyclic undrained triaxial tests using remolded clay samples, and investigated the effects of confining pressure, overconsolidation ratio, repetition frequency, and stress ratio on the cyclic loading behavior. They concluded that mean effective stress decreases (excess pore water pressure increases) due to cyclic loading even in clayey soil, although not as much as liquefaction of sandy soil. Andersen et al. (1980) and Hyodo et al. (1999) conducted cyclic undrained triaxial tests using undisturbed soft clay samples and showed that the cyclic shear resistance of the undisturbed samples was significantly lower than that of the remolded samples. It is stated that the cementation/bonding between the particles which developed over many years of deposition were disturbed/degraded by cyclic loading, which led to a decrease in cyclic shear resistance. Moreover, Matsui et al. (1992), Yasuhara et al. (1992), Moses et al. (2003), Li et al. (2011), Wang et al. (2013), and Sato et al. (2018) conducted various mechanical tests after cyclic undrained loading and showed that the undrained peak shear strength decreased as the number of repetitions increased. In addition, cyclic loading caused additional consolidation, and the extent of volumetric compression was particularly large for undisturbed samples. However, these studies are limited to presenting experimental observations and do not include reproduction using elastoplastic constitutive models or interpretation based on elastoplastic perspective.

While several constitutive models have been proposed to simulate the undrained cyclic behavior of clay (e.g., Wang et al. 2022; Wichtmann et al. 2022; Du et al. 2023; Yu and Yang 2023), most of these studies have primarily focused on reproducing the cyclic response itself—such as strain accumulation, pore pressure build-up, and stiffness degradation—during loading. In contrast, few studies have addressed the influence of cyclic loading history on the subsequent undrained shear strength, or attempted to interpret the soil disturbance induced during cyclic loading from an elastoplastic perspective. In particular, constitutive models that explicitly incorporate structural degradation

as an internal variable and link it to post-cyclic mechanical behavior remain limited. Many existing approaches rely on empirical or phenomenological formulations, which are not well suited for capturing the transition from cyclic degradation to strength reduction. Therefore, there is a need for a constitutive framework that can consistently interpret both the degradation of soil structure during cyclic loading and its consequences on the subsequent shear strength.

Therefore, the first objective of this study is to conduct cyclic undrained triaxial compression/extension tests on naturally deposited soft clay to investigate the effects of cyclic loading on changes in mechanical behavior. In conventional cyclic triaxial tests, stress-controlled loading often leads to progressive strain accumulation and localized necking, which compromises specimen uniformity and limits the number of cycles that can be applied. To overcome this limitation, strain-controlled loading was adopted in this study, enabling the application of multiple cycles while preserving the element behavior of the specimen. This approach is essential for accurately capturing the gradual degradation of soil structure under cyclic loading. The second objective is to reproduce the experimental results using an elastoplastic constitutive model of soil. Previous studies have extensively investigated the cyclic degradation of structured clays through experimental approaches, few have attempted to interpret these phenomena from an elastoplastic perspective. This study uniquely addresses this gap by employing the SYS Cam-clay model (Asaoka et al. 2002) to simulate and explain the structural degradation induced by cyclic loading, thereby providing a theoretical framework for understanding soil disturbance in naturally deposited soft clay.

2. Soil sample used in the experiment and its fundamental mechanical behavior

The clayey sample used in this study is alluvial clay Ac deposited in Anpachi, Gifu in Japan. Figure 1 shows a cross-sectional view of the site. There is an alluvial sand layer As1 at a depth of 9–15 m (TP –4.0 to –10.0 m), but the clay layer becomes dominant elsewhere. The clay layers can be divided into four layers from Ac1 to Ac4, but each layer has a small SPT *N*-value and is in soft condition. The average *N*-value of each layer is shown in Table 1. One of the important characteristics is that even in deep Ac4 where the overburden pressure is large, average *N*-value is as small as 1.08. Therefore, in this study, this Ac4 was used as an experimental target. Overburden pressure of these samples is approximately 170 kPa. Thin-wall sampling was carefully conducted to minimize disturbance as much as possible; however, some unavoidable disturbance due to stress release and mechanical effects inevitably occurred. In this study, specimens were selected from continuous samples taken at similar depths, and the coefficient of variation in initial specific volume was as low as 0.17, indicating that the variability among specimens was very small.

Fig. 1. Cross-sectional view of the sampling point.

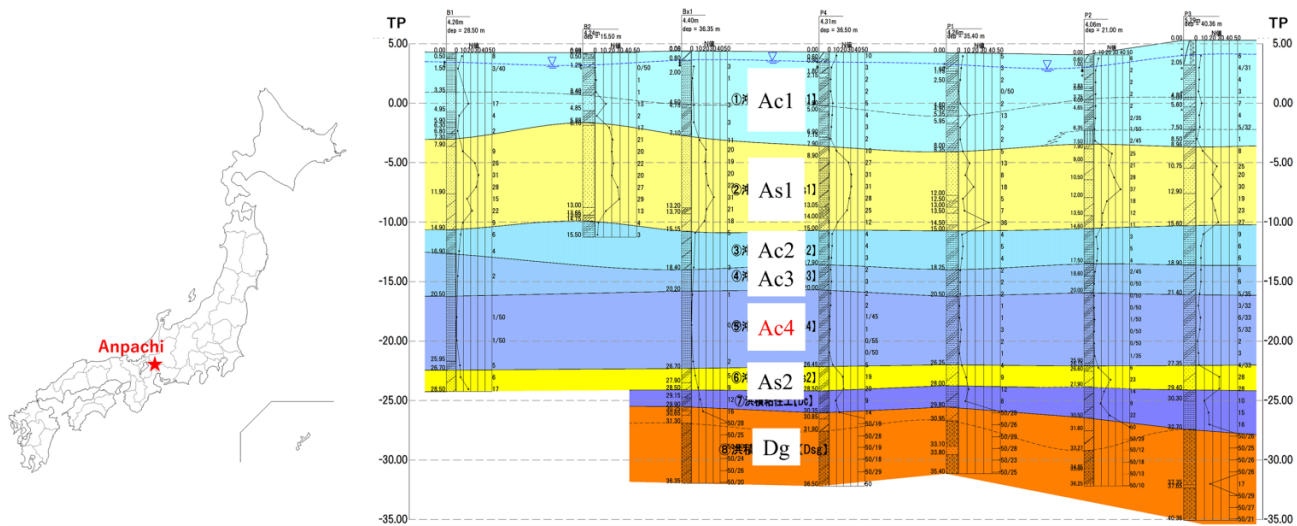


Table 1. Average N-value of each layer.

	TP (m)	Average N-value
Ac1	5.0 to -4.0 m	2.28
Ac2	-10.0 to -13.5 m	4.15
Ac3	-13.5 to -16.0 m	2.53
Ac4	-16.0 to -22.5 m	1.08

Table 2. Physical properties.

Property	Value	Size range
Soil particle density ρ_s (g/cm ³)	2.69	—
Sand content (%)	0.5	>75 μ m
Silt content (%)	44.1	5–75 μ m
Clay content (%)	55.4	<5 μ m
Liquid limit w_L (%)	54.0	—
Plastic limit w_p (%)	25.8	—
Plasticity index I_p	28.2	—
Natural water content w_n (%)	54.4	—
Liquid index I_L	1.01	—

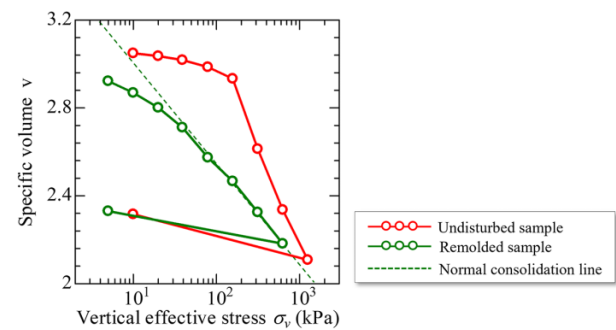
2.1. Physical property

Table 2 shows a list of physical properties of the clay sample. The fine-grained fraction, represented by the combined content of clay and silt, is nearly 100%. The table shows that the liquid index I_L exceeds 1.0. The liquid index is an index that indicates the relative hardness of the sample within a condition of natural water content w_n . An I_L greater than 1.0 indicates that the soil is in a liquid state in terms of its value. Therefore, this clay was deposited in a very soft condition with high water content.

2.2. Compression behavior

Figure 2 presents the experimental results of standard consolidation tests conducted on both undisturbed and remolded samples. The remolded sample was prepared by repeatedly remolding the undisturbed sample at its natural wa-

Fig. 2. Standard consolidation test of remolded and undisturbed samples.



ter content. As a result, the remolded sample exhibits a normally consolidated state from the initial loading stage, allowing for the determination of the normal consolidation line. In contrast, the undisturbed sample has a stress state that lies outside the normal consolidation line, an area considered as an impossible region in classical soil mechanics (Atkinson and Bransby 1987). The degree of bulkiness outside the normal consolidation line is quantitatively defined as “structure” (Asaoka et al. 1998, 2000). In other words, highly structured soil can maintain a larger specific volume under the same vertical stress compared to normally consolidated soil, or it can sustain a higher vertical stress at the same specific volume. This structure is often attributed to cementation or bonding between soil particles. Since undisturbed samples undergo unloading during sampling, they initially exhibit an overconsolidated state. When the vertical stress remains below the overburden pressure (170 kPa), the sample remains in an overconsolidated state, and its compressibility which represented by the slope of the compression curve, is relatively low. However, once the vertical stress exceeds overburden pressure and the sample transitions into a normally consolidated state, its compressibility increases significantly compared to the remolded sample. As the applied vertical stress further increases, bulkiness deteriorates, and compressibil-

Fig. 3. Undrained triaxial compression tests with different confining pressure (without cyclic loading history).

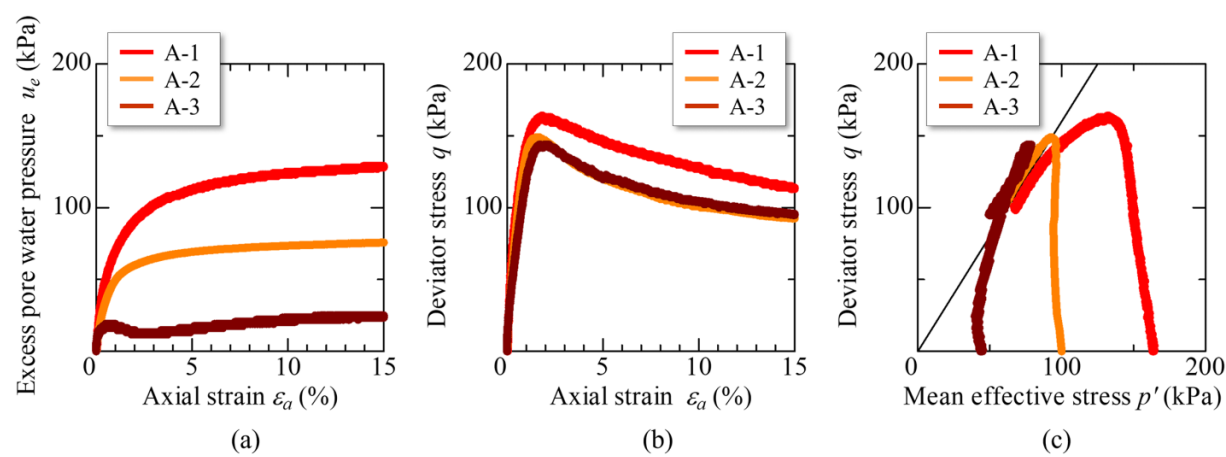


Table 3. Initial condition of the specimen without cyclic loading history.

Test name	Confining pressure (kPa)	Specific volume before consolidation	Specific volume after consolidation
A-1	170	3.01	2.97
A-2	100	3.08	3.02
A-3	45	3.01	2.99

ity gradually decreases with the stress state asymptotically approaching normal consolidation line. The experimental results indicate that the undisturbed sample, which initially exhibited highly developed structure, progressively loses its structure as vertical stress increases, ultimately converging toward the normal consolidation line. The characteristics of soft clay are bulky behavior outside of the normal consolidation line, and greater compressibility compared to remolded samples.

2.3. Monotonic undrained shear behavior

Figure 3 presents the experimental results of monotonic undrained triaxial compression tests using undisturbed samples. The initial conditions of each sample are summarized in Table 3. The confining pressure was set at three levels, 170, 100, and 45 kPa, with the overburden pressure of the sample (170 kPa). Following isotropic consolidation for 24 h, undrained shearing was performed at a constant axial strain rate of 0.007 mm/min. This strain rate was sufficiently slow to ensure element behavior was maintained during shearing, thereby preventing nonuniform distributions of excess pore water pressure within the specimen. The initial specific volume before consolidation exhibited some variation but was approximately 3.0, indicating that the specimens were generally homogeneous. The relationship between deviator stress q and axial strain ϵ_a in Fig. 3b demonstrated that q decreased smoothly after reaching its peak strength in all tests. The relationship between q and mean effective stress p' in Fig. 3a revealed distinct behavior difference based on confining pressure conditions. The specimen in the overconsolidated

state with the lowest confining pressure (A-3: 45 kPa) exhibited “rewinding behavior” (Asaoka et al. 1997; Nakano et al. 2005) characterized by an initial increase in q with a decrease in p' (hardening with plastic compression), followed by an increase in q with an increase in p' (hardening with plastic expansion), and eventually a decrease in q with a decrease in p' (softening with plastic compression). In contrast, specimens in the slightly overconsolidated state (A-2: 100 kPa) and normally consolidated state (A-1: 170 kPa) exhibited “bowing behavior” (Asaoka et al. 1997; Nakano et al. 2005) where q initially decreased with an increase in p' (softening with plastic compression) but subsequently decreased with increase in p' (hardening with plastic compression). Furthermore, when the undisturbed sample exhibited strain softening (decrease of q), excess pore water pressure u_e in Fig. 3a increased smoothly in all cases, indicating the generation of positive excess pore water pressure. This strain softening behavior is attributed to the degradation of the highly developed structure due to shearing, which is a characteristic feature of the naturally deposited soft clay. The undrained peak shear strengths of the A-1 to A-3 samples were nearly identical, as the consolidation pressures applied to these samples were lower or almost equal to the overburden pressure, resulting in similar specific volumes before shearing.

In this study, samples from the Ac4 layer were collected using thin-wall samplers with careful attention to minimize disturbance. While complete preservation of in situ structure is difficult to achieve, the mechanical behavior observed in subsequent tests—such as stress states outside the normal consolidation line and strain softening characteristics—suggests that the samples retained a considerable degree of their natural structure.

3. Effect of undrained cyclic loading history on subsequent monotonic undrained shear behavior of soft clay

3.1. Test procedure

Based on the physical and mechanical properties, it was confirmed that the clay sample used in this study was in a

Fig. 4. Cyclic undrained triaxial test under stress control (stress amplitude is constant).

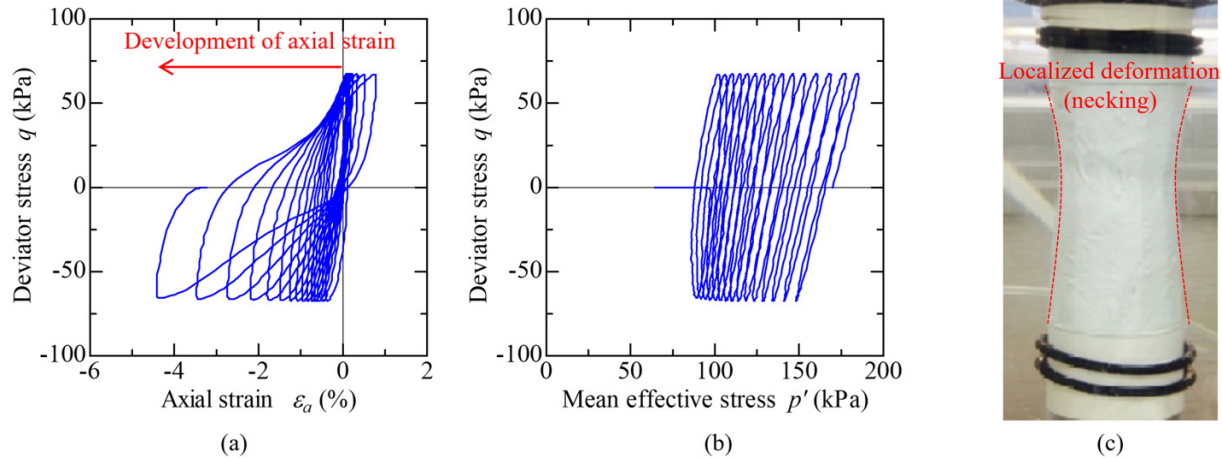
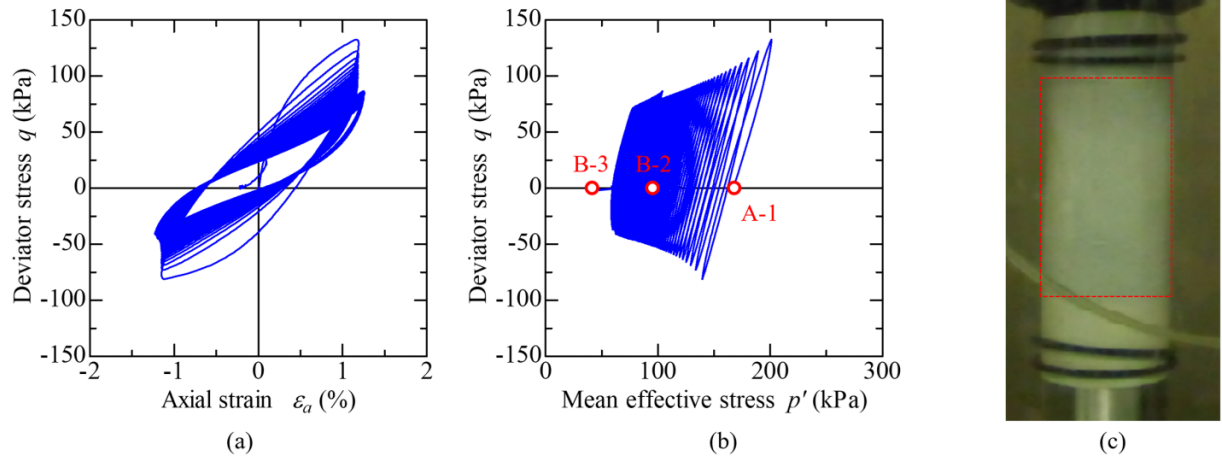


Fig. 5. Cyclic undrained triaxial test under strain control (strain amplitude is constant).



highly structured soft condition. In this chapter, the effect of undrained cyclic loading history on subsequent monotonic undrained shear behavior was investigated using a triaxial test apparatus. In laboratory testing, cyclic loading is typically applied under stress control. However, in stress-controlled tests, axial strain accumulates with repeated loading, eventually leading to localized necking failure of the specimen. This localized deformation compromises the uniformity of the specimen, making it difficult to apply multiple loading cycles while maintaining its element behavior. To address this issue, this study employed strain-controlled cyclic loading, which allows for the application of multiple loading cycles without inducing localized deformation. **Figures 4 and 5** present undrained cyclic shear behavior of clay under stress and strain controls, respectively. Both sets of results demonstrate a reduction in p' along the effective stress path in **Figs. 4 and 5b** and decrease in stiffness in the stress–strain relationship in **Figs. 4 and 5a**, as the number of loading cycles increases. However, in the stress-controlled test, ε_a gradually accumulates in the extension direction and eventually reaches

failure. In contrast, in the strain-controlled test, the specimen experiences a progressive reduction in p' and stiffness without significant development of ε_a . Given these observations, this study adopted a cyclic loading history using strain control to enable the application of multiple loading cycles while preserving the element behavior of the specimen. The specific experimental procedures are outlined below.

- 1) Isotropic consolidation was performed at 170 kPa, which is almost equal to the overburden pressure of the specimen for 24 h.
- 2) Cyclic undrained triaxial compression/extension test with axial strain control was performed with its half amplitude of 1.2%. The loading rate was 14.4 mm/min, which corresponds to one loading cycle taking 20 s. This strain amplitude is relatively large compared to those used in small-strain dynamic tests to obtain cyclic deformation properties, but it is not excessively beyond the range of shear strains that can occur in soft clay during strong seismic events. In fact, shear strains exceeding 1% have been

Fig. 6. Undrained triaxial compression tests with different confining pressure (with cyclic loading history).

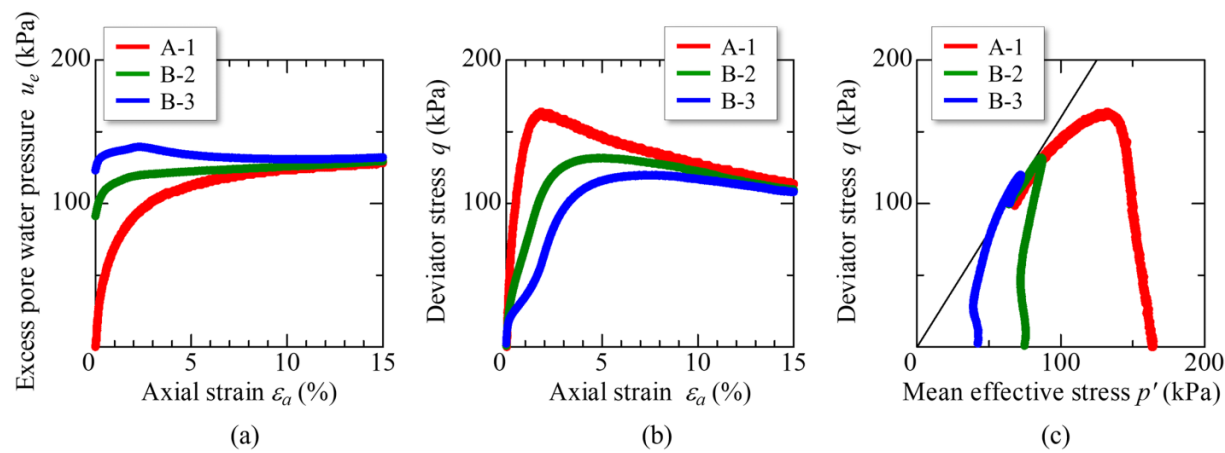


Table 4. Initial condition of the specimen with cyclic loading history.

Test name	Confining pressure (kPa)	Specific volume before consolidation	Specific volume after consolidation
A-1	170	3.01	2.97
B-2	75	3.14	3.01
B-3	45	3.12	3.01

- reported in centrifuge experiments and numerical simulations by Afacan (2014) and ElGhoraiby and Manzari (2015).
- After a set number of cycles, cyclic loading was stopped when residual strain reached zero, and specimens were kept under undrained conditions. When the loading rate is high, there is a concern that the distribution of excess pore water pressure within the specimen may become nonuniform. However, according to Khan et al. (2020), introducing a rest period under undrained conditions after cyclic loading allows the excess pore water pressure to equilibrate throughout the specimen. As a result, the final value of excess pore water pressure becomes independent of the loading rate.
 - After sufficient time has elapsed, monotonic undrained triaxial compression test was carried out at a constant strain rate of 0.007 mm/min.

3.2. Monotonic undrained shear behavior after cyclic loading history

Figure 6 presents the experimental results of monotonic undrained triaxial compression tests conducted on specimens with different cyclic loading history at points A-1, B-2, and B-3 in Fig. 5. The initial conditions of each sample are summarized in Table 4. It should be noted that the test results for A-1 correspond to a specimen that was not subjected to cyclic loading (identical to the result shown in Fig. 3). In contrast, B-2 underwent 50 loading cycles, while B-3 was subjected to 300 cycles. p' after cyclic loading is 75 kPa for B-2 and 45 kPa for B-3, respectively, as shown in Fig. 6c. Similarly,

the initial excess pore water pressures of B-2 and B-3 were 95 and 125 kPa, respectively, as shown in Fig. 6a, which were accumulated during the previous stage of cyclic loading. The comparison between specimens with and without cyclic loading history reveals that the initial stiffness during shearing decreases as the cyclic loading history increases, as shown in Fig. 6b. Furthermore, it is evident that the peak shear strength significantly decreases with an increasing number of loading cycles, with up to a 25% reduction observed in undisturbed samples. Since the specific volumes of the specimens are nearly identical, their stress states converge when they reach the residual state. As a result, strain softening behavior becomes less distinct with an increasing number of loading cycles. The sample used in this study is not a “remolded clay” but rather a “naturally deposited undisturbed soft clay with highly developed structure”. We hypothesize that cyclic loading progressively degraded the initially highly developed structure. The cyclic loading history led to (1) a reduction in initial stiffness, and (2) a decrease in undrained peak shear strength. This deterioration in strength and stiffness can be attributed to “soil disturbance” induced by cyclic loading.

4. Elastoplastic interpretation of soil disturbance based on the soil skeleton structure concept

In this chapter, the concept of “soil disturbance” is interpreted from an elastoplastic perspective by reproducing experimental results using an elastoplastic constitutive model for soils. To date, we have been developing an elastoplastic constitutive model, namely super/subloading yield surface Cam-clay model with rotational hardening, abbreviated as SYS Cam-clay model (Asaoka et al. 2000, 2002). This model characterizes the work of soil skeleton structure of the soil in terms of three fundamental components, “structure”, “overconsolidation”, and “anisotropy”, which evolve with ongoing plastic deformation. The SYS Cam-clay model enables description of the mechanical behavior of naturally deposited soil by incorporating the soil skeleton structure concept. Furthermore, this model provides a unified

Table 5. Material constant and initial conditions.

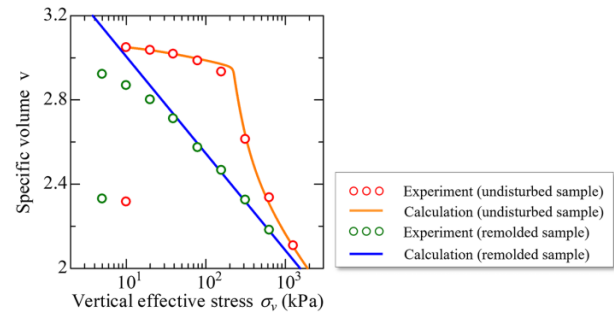
Elastoplastic parameter	
Compression index $\bar{\lambda}$	0.20
Swelling index $\bar{\kappa}$	0.04
Critical state index M	1.70
Intercept of NCL N	2.50
Poisson's ratio ν	0.10
Evolutional parameter	
Degradation parameter of structure a ($b = c = 1.0$)	0.55
Degradation parameter of structure c_s	0.2
Degradation parameter of overconsolidation m	30.0
Evolution parameter of β	0.0
Limit of rotation m_b	0.0
Initial condition	
Specific volume v_0	3.05
Mean effective stress p'_0	10.0
Stress ratio η_0	0.0
Degree of structure $1/R^*_0$	32.3
Degree of overconsolidation $1/R_0$	18.0
Degree of anisotropy ζ_0	0.0

theoretical framework that can describe the mechanical behavior of various soil types, ranging from typical clays to sands, by considering differences in the evolutionary rules governing the soil skeleton structure (Asaoka et al. 2000, 2002; Nakano et al. 2005; Asaoka 2023). The distinction between an undisturbed, naturally deposited soil sample and a remolded sample can be explained by the degree of soil skeleton structure, which develops over an extended period through the natural deposition process. In this study, the experimental results are reproduced using the SYS Cam-clay model. For further details regarding the model, refer to Appendix A.

4.1. Determination of the elastoplastic properties (Noda et al. 2005)

Material constants and initial conditions were determined by reproducing the compressive behavior shown in Fig. 2 and shear behavior shown in Fig. 3, both of which were not subjected to a cyclic loading history. Although different samples were used for each experiment, it is assumed that the degree of soil skeleton structure remained the same at the time of sampling, as all samples were collected from similar locations and depth. To establish a common initial state, it was assumed that $p' = 10$ kPa, corresponding to the stress state at the time of sample removal from the sampler. Based on this assumption, following simulations were conducted to reproduce the experimental results: (1) a standard consolidation test, and (2) an undrained shear under arbitrary isotropic consolidation pressure. The material constants and initial values are summarized in Table 5, and the simulation results are presented in Figs. 7 and 8, respectively. For simplicity, we assume here that there is no anisotropy as $\zeta_0 = 0.0$ in the initial condition. The numerical results successfully reproduced both compressive and shear behavior using a single set of material

Fig. 7. Simulation result of standard consolidation test.



constants and initial values, thereby confirming the validity of the model.

From this point onward, these parameters will be used to simulate monotonic loading behavior, both with and without cyclic loading history, to analyze how the soil skeleton structure changes due to cyclic loading. As in the experiment, cyclic loading was applied under an isotropic stress state of 170 kPa, corresponding to the overburden pressure.

4.2. Reproduction of undrained shear behavior without cyclic loading history

Figure 8 presents the experiment and simulation results of monotonic undrained triaxial compression tests without cyclic loading history. Here, Figs. 8d and 8e show the variation in the degree of structure $1/R^*$ and the degree of overconsolidation $1/R$, respectively. Table 6 summarizes the initial values for each confining pressure. Although the initial condition for all calculations assumes $p' = 10$ kPa, the degree of soil skeleton structure before shearing varies depending on the consolidation pressure. As shown in Table 6, an increase in confining pressure leads to a decrease in the degree of overconsolidation $1/R$, which corresponds to the overconsolidation ratio OCR, gradually approaching the normally consolidated state. However, the degree of structure $1/R^*$ remains nearly unchanged at a highly developed level. ($1/R^* \cong 30$). As observed in Fig. 8, the key characteristics of the experimental results are well reproduced, including (1) the peak shear strength being independent of confining pressure, and (2) strain softening behavior, such as “rewinding behavior” in the overconsolidation state and the “bowing behavior” in the normal consolidation state. During shearing, the degree of overconsolidation $1/R$ rapidly decreases toward a normally consolidated state ($1/R = 1.0$), while the degree of structure $1/R^*$ decreases gradually. This gradual structural degradation during shearing is responsible for the strain softening behavior observed in the experiment.

4.3. Reproduction of undrained shear behavior with cyclic loading history

Figure 9 presents the simulation result of a cyclic undrained triaxial compression/extension test under strain control. Similar to the experimental result, the decrease in p' and the reduction in stiffness are well reproduced as the number of repetitions increases. Focusing on the degree of the

Fig. 8. Simulation results of undrained triaxial compression tests (without cyclic loading history).

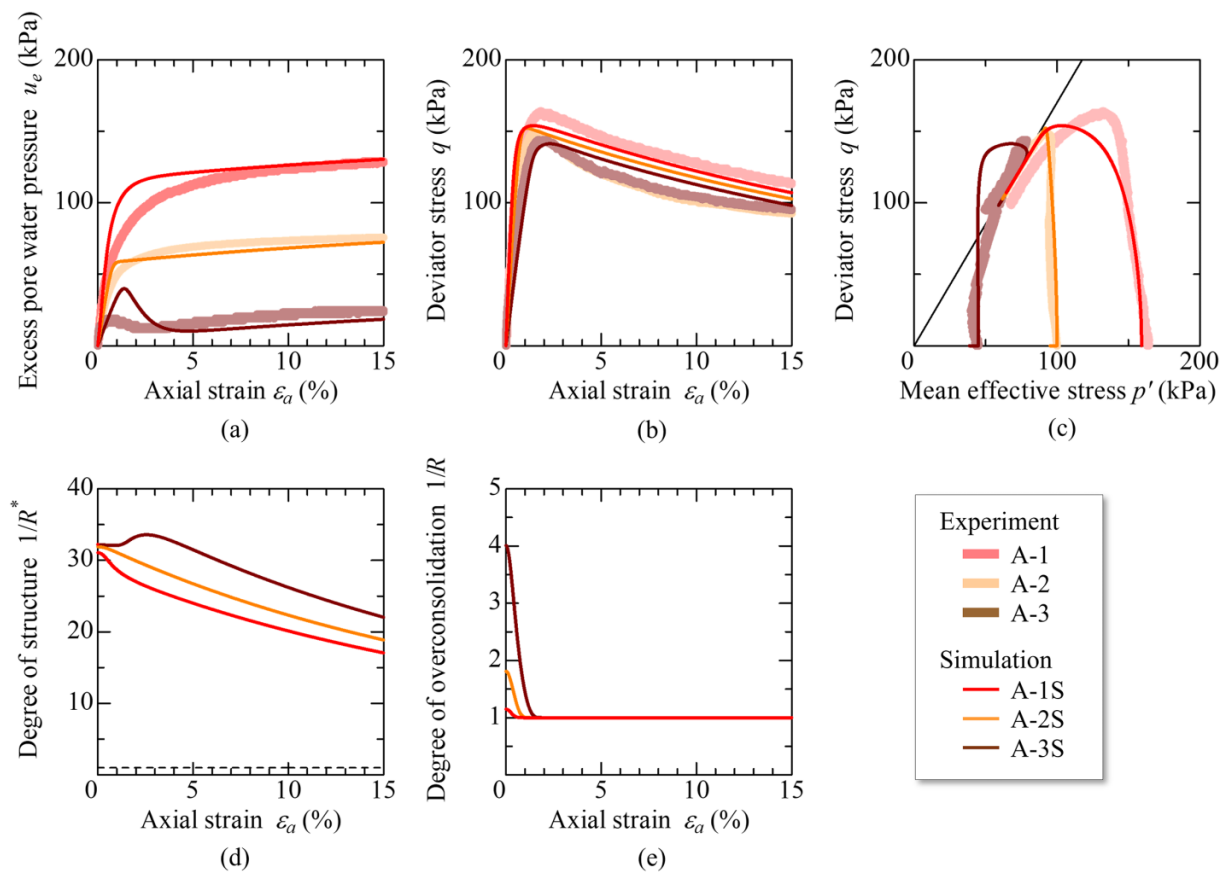


Table 6. Initial values after isotropic consolidation.

	A-1S	A-2S	A-3S
Specific volume v_0	2.93	2.96	2.99
Mean effective stress p'_0	170.0	100.0	45.0
Stress ratio η_0	0.0	0.0	0.0
Degree of structure $1/R^*_0$	31.1	31.9	32.2
Degree of overconsolidation $1/R_0$	1.15	1.81	4.00
Degree of anisotropy ζ_0	0.0	0.0	0.0

structure $1/R^*$ and the degree of overconsolidation $1/R$, $1/R^*$ decreases, representing the degradation of structure, while $1/R$ increases, representing the accumulation of overconsolidation as the number of repetition increases. The results confirm that cyclic loading induces both degradation of structure and accumulation of overconsolidation.

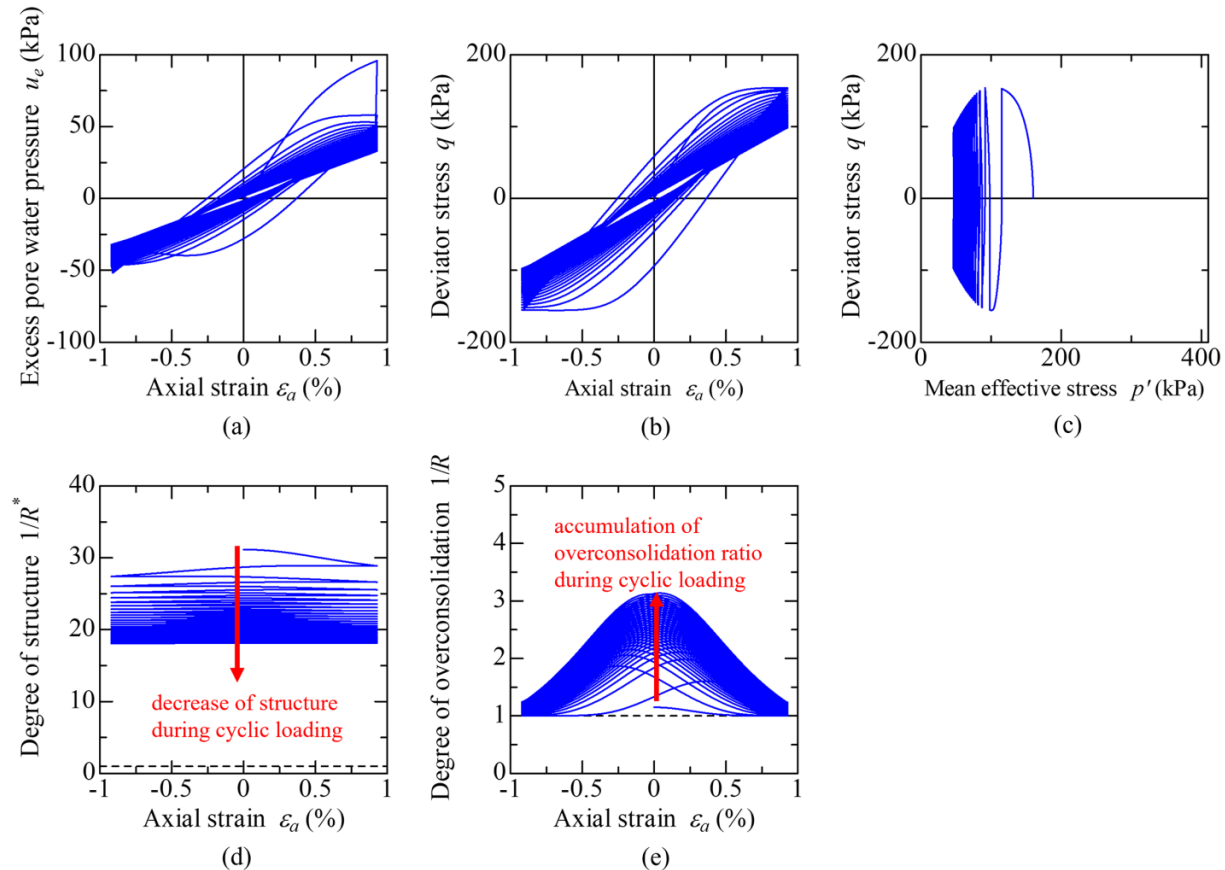
Figure 10 presents the experiment and simulation results of monotonic undrained triaxial tests with different cyclic loading histories. The cyclic undrained shearing was halted after varying number of repetitions, followed by monotonic shearing under undrained conditions. After cyclic loading, the initial excess pore water pressure for B-2S and B-3S were 95 and 125 kPa, respectively, as shown in Fig. 10a, which were accumulated during the previous stage of cyclic loading. The simulation successfully reproduces the experimental findings: (1) a reduction in initial stiffness with increasing cyclic repetitions, and (2) a decrease in undrained peak

shear strength with increasing cyclic repetitions. Although some variation in initial conditions was observed in the experiments, the numerical simulations were conducted using identical initial parameters, varying only the cyclic loading history. This approach allowed us to isolate and confirm the effects of disturbance inferred from the experimental results, independent of specimen variability. Table 7 lists the initial values before undrained shearing for each confining pressure. The trends observed in Fig. 9 continue: $1/R^*$ decreases, indicating structural degradation, and $1/R$ increases, indicating accumulation of overconsolidation ratio with the increase in repetitions. Notably, while isotropic compression resulted in minimal structural degradation, cyclic loading led to significant structural degradation. A direct comparison between A-3S in Table 6 and B-3S in Table 7 reveals the effect of cyclic loading under identical confining pressures and specific volumes. B-3S, which underwent cyclic loading, exhibits a lower overconsolidation ratio than A-3S. This, combined with structural degradation, leads to distinct shear behavior. These results highlight how cyclic loading alters the soil skeleton structure, influencing the subsequent shear response.

4.4. Difference between undisturbed and remolded sample

Due to the limited number of specimens, our experiments were conducted only on undisturbed samples. To analyze the behavior of a de-structured remolded condition, we

Fig. 9. Simulation result of undrained cyclic triaxial test.



virtually assumed this state and simulated both cyclic loading and subsequent monotonic undrained shear behavior in the same manner as for the undisturbed specimen. Figure 11 illustrates the simulation results of a cyclic undrained triaxial compression/extension test under strain control for the remolded sample, which was reproduced by setting only the degree of structure, $1/R_0^*$, to 1.0. Although the remolded sample lacks structure, p' gradually decreases with increasing cyclic loadings due to the accumulation of overconsolidation ratio. This result aligns with previous experimental findings by other researchers, which indicate that p' decreases with cyclic repetitions even in remolded samples (Brown et al. 1975; Yasuhara et al. 1982; Gu et al. 2012; Wichtmann and Triantafyllidis 2018; Khan et al. 2020). Figure 12 compares the decrease in p' due to cyclic loading with and without structure. The p' of the undisturbed sample decreases more significantly with fewer repetitions. This difference attributed to structural degradation during cyclic loading, which resulted in a pronounced negative dilatancy response.

Figure 13 presents the simulation results of monotonic undrained triaxial tests for the remolded sample, where cyclic loading was stopped after the same number of repetitions as in the undisturbed sample. A direct comparison with Fig. 10 reveals that, in a de-structured state, variations in confining pressure due to cyclic loading history have no effect on peak shear strength. From the perspective of critical state soil mechanics and the Cam-clay model, the shear strength

of remolded soil is determined only by the void ratio. As a result, despite difference in p' before monotonic shear, the shear strength at the critical state remains the same in the remolded sample.

4.5. Elastoplastic interpretation of soil disturbance

Through the numerical simulations of both undisturbed and remolded samples presented in the previous section, two key insights were obtained. The first insight is that the reduction in p' during cyclic loading is more pronounced in the undisturbed sample than in the remolded sample. The second insight is that the undrained peak shear strength of the undisturbed sample decreases with an increasing cyclic loading history, whereas the undrained peak shear strength of the remolded sample remains unchanged regardless of cyclic loading history. In this section, these insights are analyzed based on the soil skeleton structure concept to interpret the nature of “soil disturbance” from an elastoplastic perspective.

Equation 1, which corresponds to eq. A2 in Appendix A, expresses the plastic volumetric strain in the SYS Cam-clay model.

$$(1) \quad \varepsilon_v^p = f(p', \eta^*) + MD \ln R^* - MD \ln R$$

$$(2) \quad \varepsilon_v^p + \varepsilon_v^e = \varepsilon_v = 0$$

Fig. 10. Simulation results of undrained triaxial compression tests (with cyclic loading history).

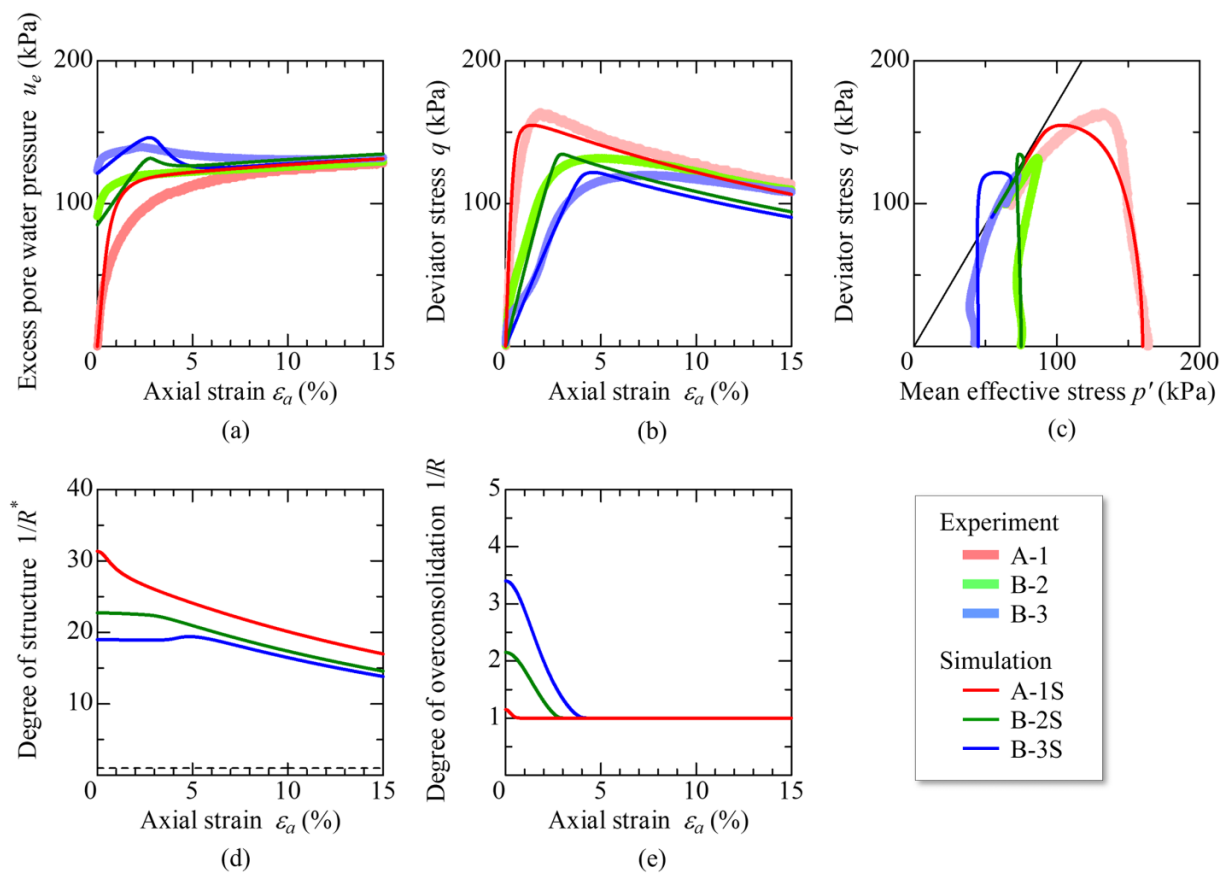


Table 7. Initial values after cyclic loading.

	A-1S	B-2S	B-3S
Specific volume v_0	2.93	2.93	2.93
Mean effective stress p'_0	170.0	75.0	45.0
Stress ratio η_0	0.0	0.0	0.0
Degree of structure $1/R^*_0$	31.1	22.7	18.9
Degree of overconsolidation $1/R_0$	1.15	2.15	3.40
Degree of anisotropy ζ_0	0.0	0.0	0.0

Here, ε_v denotes the volumetric strain, the superscript p represents the plastic component, and the superscript e represents the elastic component. Both components are considered positive in compression. Moreover, f denotes the yield function, M is the critical state index, D is the dilatancy coefficient, R^* indicates the degree of structure, and R indicates the overburden ratio. From this equation, it can be observed that degradation in structure (increase in R^*) results in plastic compression, while degradation in overconsolidation (increase in R) results in plastic expansion. Under cyclic loading, the volumetric strain remains zero due to the undrained condition. In other words, as expressed in eq. 2, the sum of plastic volumetric strain ε_v^p and elastic volume change ε_v^e must always be zero. Here, elastic compression ($\varepsilon_v^e > 0$) results in an increase in p' , and elastic expansion ($\varepsilon_v^e < 0$) results in a decrease in p' . Therefore, it is important to note that plastic

compression ($\varepsilon_v^p > 0$) results in a decrease in p' , and plastic expansion ($\varepsilon_v^p < 0$) results in an increase in p' . These relationships emphasize that structural degradation-induced plastic compression plays a crucial role in the reduction of p' during cyclic loading in undisturbed samples.

In the case of a highly structured specimen, such as soft clay, cyclic loading induces structural degradation, as illustrated in Fig. 9, leading to significant plastic compression. Consequently, as shown in Fig. 12, the reduction in p' during cyclic loading is more pronounced in undisturbed sample compared to remolded sample. Furthermore, when analyzing changes in the overconsolidation ratio, it was observed that overconsolidation ratio gradually accumulates due to cyclic loading. Notably, the accumulation of overconsolidation was more significant in the remolded sample (Fig. 11) than in the undisturbed sample (Fig. 9). This difference arises because, as mentioned earlier, plastic compression in undisturbed sample occurs due to structural degradation, thereby suppressing plastic expansion associated with the accumulation of overconsolidation.

For undisturbed samples, an increase in number of cyclic loadings results in a progressive reduction in peak shear strength. This phenomenon can be attributed to the progressive reduction in structure prior to shearing. Structurally developed soils exhibit bulky behavior compared to a normally consolidated state, meaning they can maintain a larger specific volume under the same stress or sustain a higher stress

Fig. 11. Simulation result of undrained cyclic triaxial test with remolded soil (de-structured material).

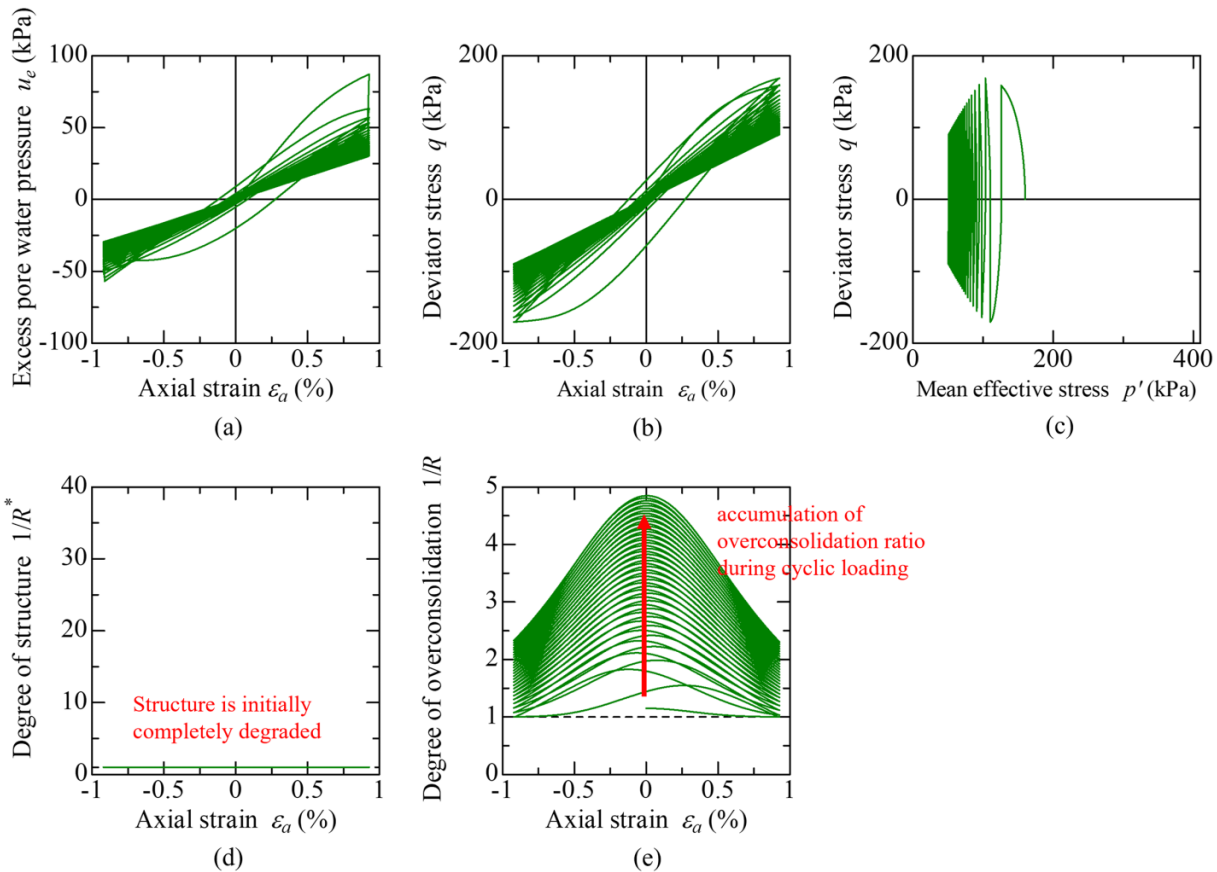
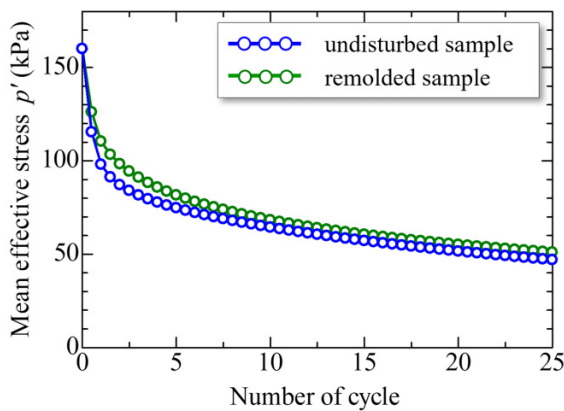


Fig. 12. Comparison of decrease process during cyclic loading in mean effective stress.



at the same specific volume. The degradation of this bulky effect is analogous to the strength reduction observed in sensitive clays when disturbed. For remolded samples, peak shear strength is unaffected by variations in p' caused by cyclic loading.

In summary, even in clayey soil, p' decreases with cyclic loading due to accumulation of overconsolidation, regardless of the presence or absence of a structure. However, when focusing on the undrained shear behavior following

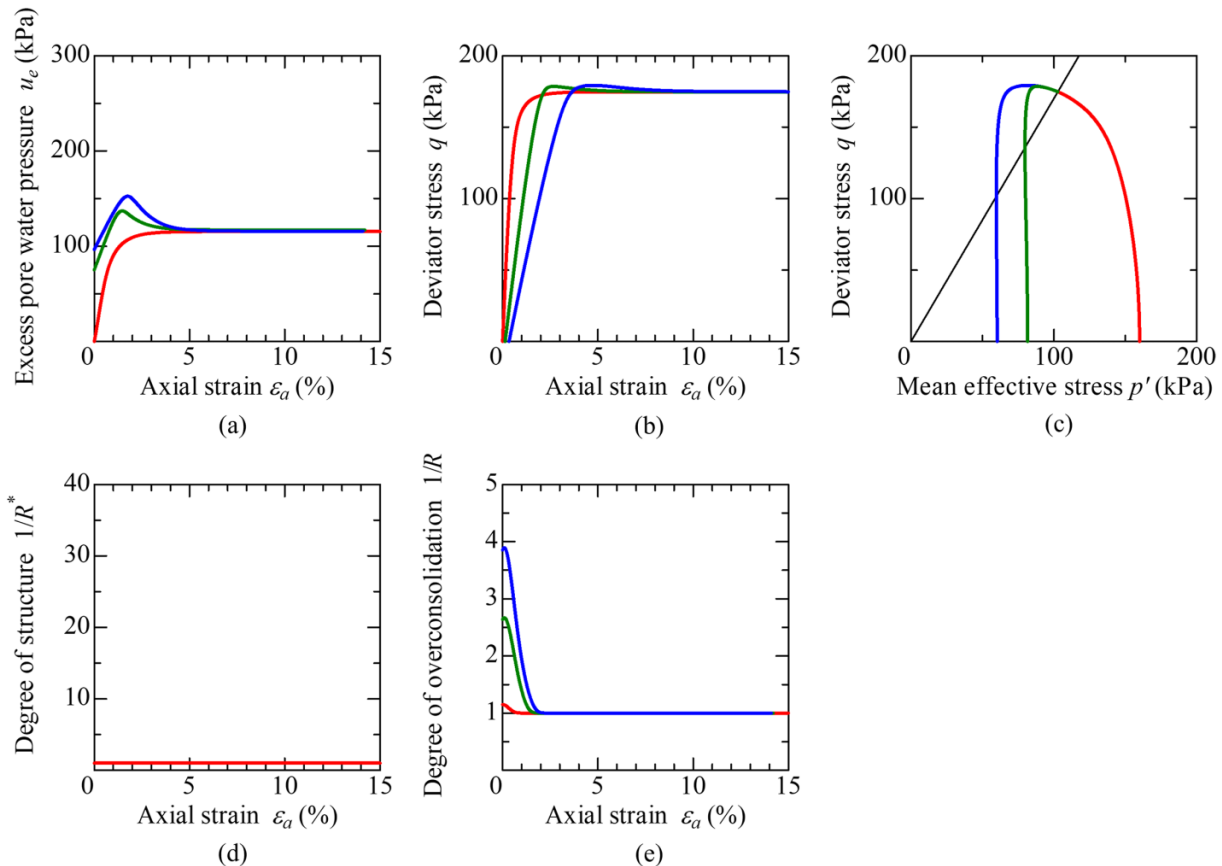
cyclic loading, a distinct difference emerges. In remolded samples with no structure, the cyclic loading history has no impact on the subsequent undrained peak shear strength. Conversely, in highly structured soils, the undrained peak shear strength progressively decreases with increasing cyclic loading history. “Soil disturbance” can be interpreted as structural degradation induced by cyclic loading, which accelerates the reduction in mean effective stress (stiffness degradation) over repeated cycles and ultimately leads to a decrease in shear strength.

Seismic damage does not occur uniformly across all clayey grounds. This study demonstrated that in soft clayey grounds with highly structured conditions, if the soil is disturbed by an earthquake and the structure deteriorates, shear strength decreases, increasing the likelihood of seismic damage. In other words, seismic damage in clayey ground can be attributed to the degradation of highly developed structures, highlighting the necessity and importance of the “structural concept” in evaluating seismic vulnerability.

5. Conclusions

This study investigated the impact of cyclic loading-induced disturbance on the shear behavior of soft clay using triaxial compression tests on undisturbed samples. Additionally, an elastoplastic interpretation of soil disturbance was developed by reproducing the experimental results using an elastoplastic constitutive model that accounts for the work of

Fig. 13. Simulation results of undrained triaxial compression tests with remolded soil (de-structured material).



the soil skeleton structure. The key findings are summarized as follows:

5.1. Experimental findings

Strain-controlled cyclic undrained shear tests followed by monotonic undrained shear tests were conducted on undisturbed soft clay using a triaxial apparatus. Unlike ordinal stress-controlled tests, strain-controlled tests were able to maintain the elementality of the specimen even after multiple cyclic loadings. When the confining pressure was reduced by isotropic unloading, the subsequent peak shear strength remained unchanged. However, when the confining pressure was reduced due to undrained cyclic loading, a decrease in peak shear strength was observed with increasing cyclic loading history.

5.2. Numerical analysis and interpretation based on soil skeleton structure concept

The behavior of soft clay was modeled using the SYS Cam-clay model, which describes the degradation of soil skeleton structure. Numerical simulations were performed to reproduce the experimental observations and to analyze the influence of structure by comparing undisturbed and remolded samples. The results demonstrated that the mean effective stress p' gradually decreased due to the accumulation of overconsolidation induced by undrained cyclic loading in both the undisturbed highly structured

samples and the de-structured remolded samples. However, in the undisturbed samples, the decrease in p' was more pronounced due to plastic compression resulting from structural deterioration. Furthermore, while isotropic unloading did not induce structural deterioration since no shear force was applied, cyclic loading led to progressive structural degradation as the number of loading cycles increased. This confirms that the reduction in peak shear strength observed in undisturbed samples is attributed to structural deterioration caused by cyclic loading. In contrast, in the case of remolded samples, no structural deterioration occurred, and thus the peak shear strength remained unchanged regardless of the cyclic loading history.

5.3. Concept of soil disturbance

“Soil disturbance” can be understood as the cyclic loading-induced degradation of the structure based on the soil skeleton structure concept. When structured soil undergoes disturbance, in addition to a greater tendency for mean effective stress to decrease during cyclic loading (resulting in reduced stiffness), the subsequent shear strength is significantly decreased.

5.4. Practical implications

These findings suggest that, in addition to sandy soils, soft clayey soils with highly structured conditions may also be vulnerable to significant strength degradation under large

cyclic loading histories. This highlights the need for careful consideration of such soils in seismic design and geotechnical modeling, especially in regions where repeated seismic events may occur.

Future research directions include:

- To further advance the understanding of soil disturbance in soft clays, future studies will investigate clays with varying physical properties, stress conditions, strain amplitudes, and loading rates that better reflect realistic seismic loading scenarios.
- Due to the limited number of soil samples available in this study, laboratory tests using remolded specimens could not be conducted. In future research, experimental validation using remolded samples will be essential. Such tests will enable direct comparison with simulation results and contribute to verifying the applicability of the elastoplastic constitutive model.
- Investigation of the effects of continuous or multi-stage cyclic loading, such as mainshock–aftershock sequences.
- Exploration of long-term loading phenomena, such as creep, which occur under drained conditions.
- Microstructural observations using scanning electron microscopy (SEM) before and after loading are also planned to better understand the mechanisms of structure degradation from a microscopic perspective.
- Applying the knowledge gained on the influence of soil disturbance to initial-boundary value problems for the quantitative evaluation of ground deformation and the development of effective countermeasures for soft clayey grounds under seismic loading.

Article information

History dates

Received: 17 April 2025

Accepted: 14 January 2026

Accepted manuscript online: 16 January 2026

Version of record online: 25 February 2026

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Data availability

Data generated or analyzed during this study are available from the corresponding author upon reasonable request.

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Competing interests

The authors declare there are no competing interests.

Funding information

This research was supported in part by Grants-in-Aid from the Japan Society for the Promotion of Science JSPS “Kakenhi” (grant No. 17H01289).

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Appendix A. Super/subloading yield surface Cam-clay model

Naturally deposited soils, whether clayey or sandy, generally exist in a “structured” and overconsolidated state. To describe the deformation behavior of a soil in this state, we must start from the base of an elastoplastic model of a de-structured/fully remolded soil in a state of normal consolidation. Given that a soil in this de-structured and normally consolidated state still possesses anisotropy, we take for our “base” in this paper the modified Cam-clay model (Roscoe and Burland 1968) with the introduced addition of rotational hardening (Sekiguchi and Ohta 1977), which treats the stress parameter η^* and its evolution rule (Hashiguchi and Chen 1998) as an expression of anisotropy. The degrees of structure and overconsolidation are then introduced and quantified by means of two concepts: the superloading surface for structure (Asaoka et al. 2002) and the subloading surface for overconsolidation (Hashiguchi 1978, 1989; Asaoka et al. 1997). That is to say, the degree of structure is expressed by means of a superloading surface situated on the outside of the Cam-clay normal-yield surface and similar to it (the center of similarity being the origin $p' = q = 0$, and the similarity rate being given by R^* ($0 < R^* \leq 1$), whereas the overconsolidation state is expressed by means of a subloading surface situated on the inside of the superloading surface and again similar to it (center of similarity $p' = q = 0$, similarity rate R ($0 < R \leq 1$); the reciprocal $1/R$ is the overconsolidation ratio). Here, p' is the mean effective stress and q is the shear stress, which are expressed by the effective stress tensor $\mathbf{T}'$ (tension: positive), where $p' = -\frac{1}{3}\text{tr}\mathbf{T}'$ and $q = \sqrt{\frac{3}{2}}\|\mathbf{S}\|$ ($\mathbf{S} = \mathbf{T}' + p'\mathbf{I}$). Here, $\|\cdot\|$ represents the norm of the vector. The closer R^* is to 0, the higher the degree of structure, but with the loss of structure that accompanies progressive plastic deformation, R^* will approach 1 (evolution rule for R^*). Similarly, the closer R is to 0, the more overconsolidated the state of the soil, but as R increases toward 1 with plastic deformation, the state of the soil will also approach normal consolidation (evolution rule for R). It can thus be assumed that the loss of structure with progressive plastic deformation brings a simultaneous release from overconsolidation (a transition to the normally consolidated state), thus eventually reaching $R = 1$ and $R^* = 1$, consistent with the Cam-clay model.

If we start from the Cam-clay eq. A1 below as our base, given that the current effective stress exists on the subloading surface, we first need to adapt relations to the subloading surface through the application of various elastoplastic rules, such as the associated flow rule and Prager’s consistency condition, so as to obtain eq. A2.

The Cam-clay potential:

$$(A1) \quad MD \ln \frac{\tilde{p}}{\tilde{p}_0} + MD \ln \frac{M^2 + \eta^{*2}}{M^2} + \int_0^t J \text{tr} \mathbf{D}^p d\tau = f(\tilde{p}, \eta^*) + \int_0^t J \text{tr} \mathbf{D}^p d\tau = 0$$

The subloading surface:

$$(A2) \quad f(p', \eta^*) + MD \ln R^* - MD \ln R + \int_0^t J \text{tr} \mathbf{D}^p d\tau = 0$$

Here, $D = (\tilde{\lambda} - \tilde{\kappa}) / M / (1 + e_0)$ is the dilatancy coefficient; M , $\tilde{\lambda}$, $\tilde{\kappa}$, and e_0 are the critical state constant, compression index, swelling index, and initial void ratio, respectively; and $J = (1 + e) / (1 + e_0)$ (e is the void ratio at time $t = t$). $-\int_0^t J \text{tr} \mathbf{D}^p d\tau$ (compression: positive) corresponds to the plastic volumetric strain ε_v^p . η^* , which expresses the anisotropy and is obtained from the rotational hardening variable β as follows: $\eta^* = \sqrt{\frac{3}{2}} \|\hat{\eta}\|$, $\hat{\eta} = \eta - \beta$, $\eta = S / p'$, $S = \mathbf{T}' + p' \mathbf{I}$. $\beta = 0$ expresses a state of no anisotropy. In the present paper, the evolution rules for R^* , R , and β are given by the following equations:

Evolution rule for R^* :

$$(A3) \quad \dot{R}^* = J \sqrt{\frac{2}{3}} \|\mathbf{D}_s^p\|, U^* = \frac{a}{D} R^{*b} (1 - R^*)^c$$

Evolution rule for R :

$$(A4) \quad \dot{R} = J U \|\mathbf{D}^p\|, U = -\frac{m}{D} \ln R$$

Evolution rule for β :

$$(A5) \quad \dot{\beta} = J \frac{br}{D} \sqrt{\frac{2}{3}} \|\mathbf{D}_s^p\| \|\hat{\eta}\| \left(m_b \frac{\hat{\eta}}{\|\hat{\eta}\|} - \beta \right)$$

Here, $\mathbf{D}^p$ is the plastic stretching tensor, $\mathbf{D}_s^p$ is the deviator component of $\mathbf{D}^p$. $\dot{\beta}$ in eq. A5 is the Green and Nagdhi's rate of β . The parameter groups for the evolution rules in eqs. A3–A5 all consist of constants, and from their respective functions we may call a , b , and c the degradation indices of structure, m the degradation index of overconsolidation, br the rotational hardening index, and m_b the rotational hardening limit constant (Hashiguchi and Chen 1998).

The associated flow rule and the constitutive equation

Associated flow rule:

$$(A6) \quad \mathbf{D}^p = \lambda \frac{\partial f}{\partial \mathbf{T}'}, \lambda = \frac{\frac{\partial f}{\partial \mathbf{T}'} \cdot \mathbf{T}'}{J \frac{MD}{p(M^2 + \eta^{*2})} (M_s^2 - \eta^2)} > 0, \eta = \frac{q}{p'} = \|\eta\|$$

Constitutive equation:

$$(A7) \quad \dot{\mathbf{T}}' = \mathbf{E} \mathbf{D} - \Lambda \mathbf{E} \frac{\partial f}{\partial \mathbf{T}'}$$

Here, $\mathbf{E}$ is the elastic modulus tensor, $\mathbf{T}'$ is the Green and Nagdhi's rate of $\mathbf{T}'$, and Λ is the expression of the plastic multiplier λ in terms to stretching $\mathbf{D}$. Further, we can establish the relations.

$$(A8) \quad M_s^2 = M_a^2 + br \frac{4 M \eta^{*2}}{M^2 + \eta^{*2}} \left(m_b \eta^* - \sqrt{\frac{3}{2}} \hat{\eta} \cdot \beta \right) - MD \left(\frac{U^*}{R^*} 2 \eta^* + \frac{U}{R} \sqrt{6 \eta^{*2} + \frac{1}{3} (M_a^2 - \eta^2)^2} \right)$$

and

$$(A9) \quad M_a^2 = M^2 + \zeta^2, \zeta = \sqrt{3/2} \|\beta\|$$

The slope M_s of the threshold between hardening and softening $q = M_s p'$, obtained under loading condition $\lambda > 0$, varies according to structural degradation, loss of overconsolidation, and development or loss of anisotropy, as well as with the current stress ratio. Similarly, the slope M_a of the threshold between plastic compression and expansion $q = M_a p'$ varies in response to the development or loss of anisotropy. For further details, the reader is referred to Asaoka et al. (2002).

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