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Shear modulus degradation of liquefiable sand in strain-controlled cyclic simple shear tests

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ABSTRACT: The earthquake-induced soil liquefaction leads to a significant degradation of soil, which may influence the site seismic response and the foundation bearing performance. In this study, the degradation in the shear modulus of liquefiable sand under cyclic loading was investigated by strain-controlled cyclic simple shear tests. These tests were performed on saturated specimens made of uniform fine silica sand with a relative density of 40% under confining pressures of 20, 50, and 100 kPa, respectively. In each test, the specimen was cyclically loaded with a specified shear strain amplitude at a frequency of 1 Hz so that excess pore water pressure (EPWP) gradually increased until liquefaction or the maximum number of loading cycles. The results indicate that the shear modulus decreased with the increase of both the shear strain and the loading cycles, implying the relevancy of this decrease to the EPWP. Therefore, the relationship between the observed shear modulus degradation and the EPWP buildup level was further deduced and compared with existing studies for discussions. These findings are beneficial for a more accurate evaluation of the seismic ground response and the bearing performance of foundations at a liquefiable site during earthquakes.

KEYWORDS: shear modulus degradation, liquefiable sand, strain-controlled cyclic simple shear test, excess pore water pressure.

1 INTRODUCTION

Soil liquefaction is one of the destructive geotechnical disasters during earthquakes. It degrades the strength and stiffness of soil owing to the buildup of excess pore water pressure (EPWP) caused by dynamic actions. Hence, the ground settlement and lateral spreading that are harmful to engineering structures may be caused. The soil degradation also influences the seismic response of the ground and reduces the bearing performance of the foundation. Consequently, the liquefaction-relevant soil degradation should be appropriately considered in the seismic design. Many experimental studies have been made to quantify the soil degradation in the process of liquefaction. However, the results are diverse, and more studies are still needed.

To this end, a series of cyclic simple shear (CSS) tests, which are commonly used for the dynamic properties and liquefaction resistance of soil, were conducted to investigate the variation of soil stiffness with the shear strain and the EPWP under cyclic loading. Although the cyclic triaxial (CTX) test apparatus is more common and widely utilized in practical applications, the cyclic simple shear test is usually considered better representative of in-situ conditions. During each test, the strain-controlled cyclic loading was applied to the fine-sand specimen until it was liquefied or the stop criterion was reached, and the change of the shear modulus and the buildup of EPWP with loading cycles were monitored. The relationship of the shear modulus degradation versus EPWP can thus be deduced and compared with existing studies. These results can be a reference for the effective stress seismic response analysis of the ground and the bearing performance assessment of the foundations at a potentially liquefiable site during earthquakes.

2 METHODOLOGY

2.1 *Test equipment*

The Advanced Dynamic Cyclic Simple Shear (ADVDCSS) testing system of GDS Instruments (gdsinstruments.com) as shown in Figure 1 was used in this study. It uses a stack of low-friction retaining rings to enforce a constant cross-sectional area of the cylindrical soil specimen. Thus, the K-zero condition can be ensured during the consolidation and shearing phases, which simulates the reality better than the condition of isotropic confining pressure in most CTX tests. Another feature making CSS tests more realistic is that the shear loading/deformation is directly imposed on the specimen, whereas CTX tests use indirect axial loading/ deformation to induce shearing.

According to ASTM D8296 (ASTM, 2019), the shearing phase in CSS tests is conducted under a constant volume condition to simulate the undrained condition for fully saturated specimens. Hence, a constant height of the specimen should be kept during the shearing, which is achieved by active height control in the ADVDCSS apparatus via feedback from the axial displacement measurement. This constant height status leads to a varying vertical stress. The change in vertical stress during constant-volume shearing of the CSS test is equal to the change in σ_o' and hence assumed to be equal to the change in pore water pressure (PWP) under undrained conditions (ASTM, 2019).

The shearing under constant volume can be performed on dry or saturated specimens (ASTM, 2019). The ADVDCSS apparatus can saturate specimens in a similar way to normal triaxial testing because it is equipped with a cell (Figure 1).

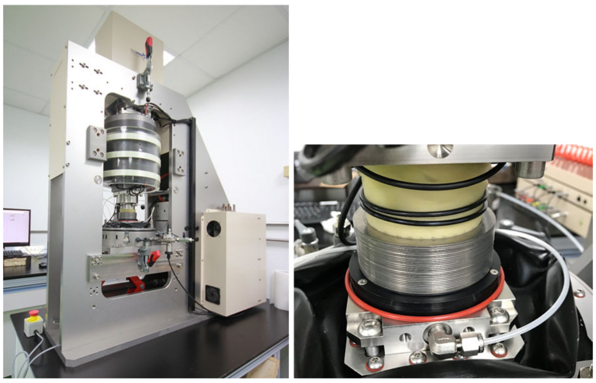

Figure 1. GDS ADVDCSS testing system and stacked retaining rings.

2.2 *Soil specimens*

The soil specimens were composed of Vietnam silica sand with grain sizes of 0.075 to 0.85 mm, a coefficient of uniformity (C_u) of 1.578, and a coefficient of curvature (C_c) of 1.052, which can be categorized as uniform fine sand. Hence, it is considered potentially liquefiable. It also features rapid and easy saturation during the testing, which is beneficial for the execution of tests.

The cylindrical specimens with a diameter of 7 cm and a height of 2 cm were prepared by moist tamping. Each of the specimens was saturated and consolidated at a specified vertical stress to account for the effective overburden pressure (σ_o') under the K-zero condition before shearing. The test results of the specimens with a relative density (D_r) of 40% are presented in this paper.

2.3 *Test procedure*

Each specimen was cyclically loaded with a specified constant shear strain amplitude at a frequency of 1 Hz until it was liquefied or the maximum number of loading cycles (40 was adopted in this study) was reached. The liquefaction criterion is that the EPWP ratio (r_u, the ratio of EPWP to σ_o') exceeds 0.9. This procedure can be regarded as strain-controlled testing for the liquefaction resistance of soil. This study adopted strain-controlled shearing because it induces less intense change of PWP and less distorted stress-strain curves when approaching liquefaction compared with stress-controlled shearing. This helps to more precisely capture the shear modulus degradation and the EPWP buildup during the process of liquefaction. Notably, the buildup of EPWP was represented by the drop in the vertical stress in CSS tests.

2.4 *Test conditions*

Three different vertical stress magnitudes were specified for the consolidation and at the beginning of shearing of the specimens, including 25 kPa, 50 kPa, and 100 kPa. The adopted cyclic shear strain amplitudes (γ_c) included 0.019%, 0.038%, 0,057%, 0.076%, 0.11% and 0.15%. Consequently, the influence of σ_o' and the shear strain level can be examined.

3 TEST RESULTS

3.1 *Shear Modulus vs. shear strain*

Figure 2 demonstrates the relationship of shear stress versus shear strain during strain-controlled shearing. It features the hysteretic loops with decreasing secant slopes. The secant shear modulus (G) for each loading cycle was calculated following ASTM D8296. The G versus γ_c curves before liquefaction for numbers of loading cycles (N) of 1, 2, 4, 6, 10, 15, 26, and 40 under different σ_o' are given in Figure 3. Several of these N values can be related to the number of representative cycles corresponding to different earthquake magnitudes proposed by Seed et al. (1985), as listed in Table 1.

In Figure 3, the degradation in G with increasing γ_c, similar to the typical G degradation curves, can be observed. Moreover, after γ_c exceeded a certain threshold, a larger N led to more degradation in G, and this further degradation with N was magnified as γ_c increased. Since constant-volume shearing in CSS testing is equivalent to undrained shearing, the EPWP buildup is probably responsible for the further degradation in G with N. Notably, the γ_c threshold at which the influence of N on G takes effect became larger at a higher σ_o' and may be relevant to the threshold shear strain of EPWP buildup.

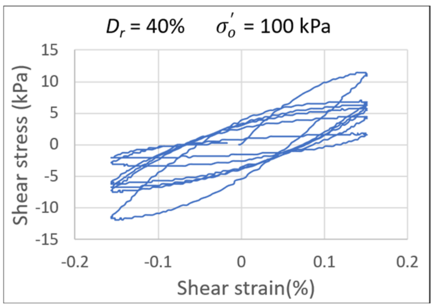

Figure 2. Shear stress vs. shear strain in a strain-controlled CSS test.

Table 1. Representative cycle number of the earthquake magnitude (Seed et al., 1985).

Earthquake magnitude	5.25	6	6.75	7.5	8.5
Number of rep. cycles	2-3	5-6	10	15	26

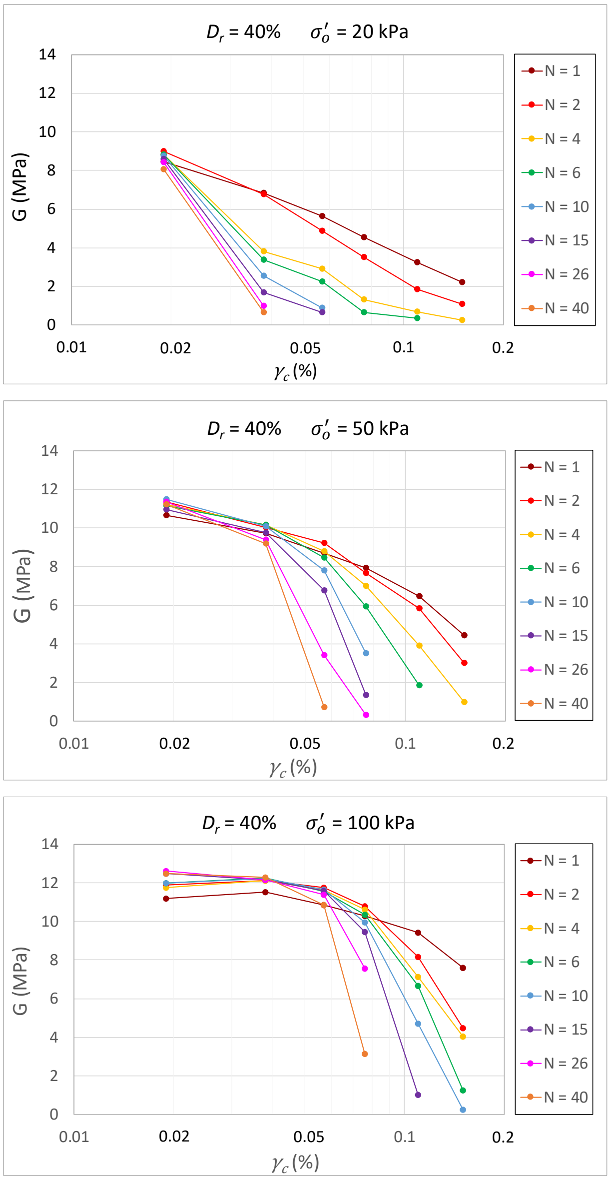

Figure 3. Shear modulus (G) vs. cyclic shear strain amplitude (γ_c).

3.2 *EPWP vs. number of cycles*

Figure 4 shows the EPWP at N = 1, 2, 4, 6, 10, 15, 26, and 40 in terms of the average r_u in a specific cycle. The EPWP built up significantly with increasing N until liquefaction occurred if γ_c exceeded a certain threshold. The γ_c threshold is dependent on σ_o' and is between 0.019% and 0.038%, between 0.038% and 0,057%, and between 0,057% and 0.076% for σ_o' = 25 kPa, 50 kPa, and 100 kPa, respectively. Moreover, lower σ_o' generally led to a sharper increase of EPWP. On the other hand, in the cases of γ_c not above these thresholds, much smaller levels of EPWP buildup were induced, and the specimens were not yet liquefied (maximum r_u = 0.1−0.6) at the end of tests (N = 40).

Because the significant EPWP buildup (presented herein) and the further G degradation (presented in the previous section) with N occurred in similar strain ranges and were both more intensified at larger strains, it is justified that the influence of N on the G versus γ_c curves can be attributed to EPWP buildup.

Notably, the tendency of r_u versus N curve at $\gamma_c = 0.019\%$ for σ'_o = 25 kPa, 50 kPa, in which r_u is below 0.2, is slightly unstable (*i.e.*, neither constant nor monotonically increasing). This implies the assumption that the change in vertical stress during constant-volume shearing is equivalent to the change in PWP may be somewhat inaccurate for a low level of EPWP buildup, and further investigation may be needed.

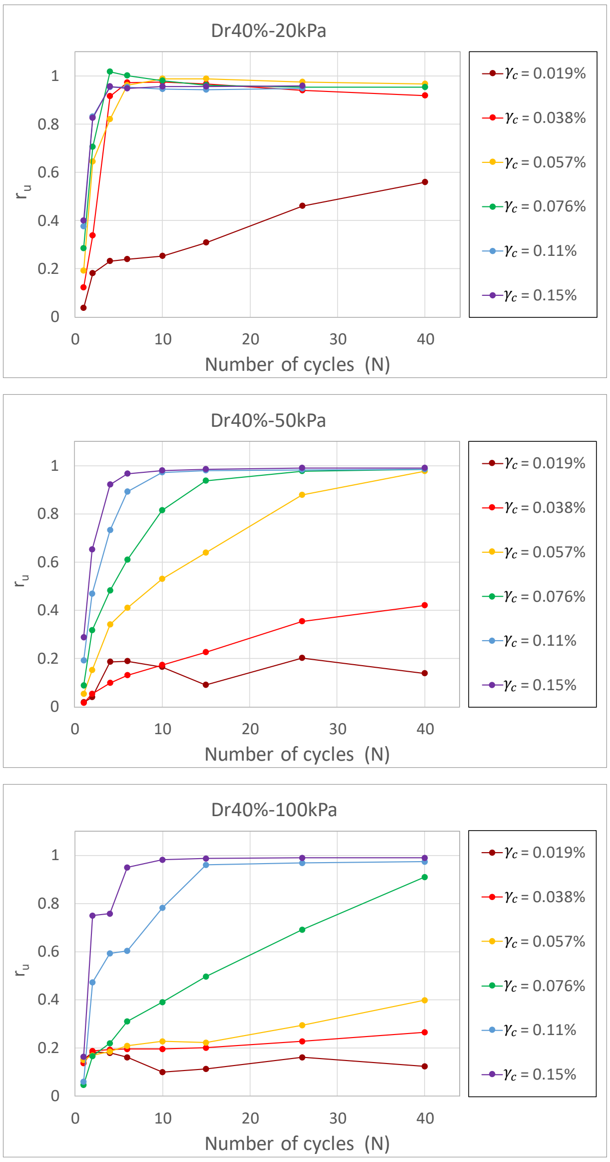

Figure 4. Excess pore water ratio (r_u) vs. number of cycles (N).

4 SOIL DEGRADATION WITH EPWP

4.1 *Shear modulus degradation with EPWP according to the present test results*

Combining Figures 3 and 4, the G versus r_u relationship for each γ_c can be established, as shown in Figure 5. Below the γ_c thresholds of significant EPWP buildup, the variation of G with r_u is trivial because r_u is low and limited in a small range. By contrast, if γ_c exceeded the thresholds, the degradation in G with increasing r_u became evident, and G was substantially reduced for r_u close to 1. Notably, the G value at the first loading cycle (denoted as G_1, corresponding to the initial point of G versus r_u curve) is smaller at a larger γ_c, probably owing to both the strain-dependent nonlinearity of soil and the notable EPWP buildup in the first cycle with a large strain level.

4.2 *Comparison with existing studies*

Different G_1 values of different G versus r_u curves make the comparison of the degradation extent more difficult. Hence, to better quantify the soil degradation caused by EPWP buildup, the degradation index (δ_N) for strain-controlled tests proposed by Vucetic and Mortezaie (2015) was adopted herein:

$$\delta_N = G_N / G_1 \quad (1)$$

where G_N denotes the G value at the N-th loading cycle, and $\delta_N < 1$ means that the soil is degraded compared to its initial state.

Using Equation (1), the relationship between δ_N and r_u for each γ_c can be obtained based on the data in Figure 5 and compared with the average of those of Vucetic and Mortezaie (2015) obtained from CSS tests, as shown in Figure 6. The δ_N versus r_u curves exhibit much less divergence than G versus r_u curves for different γ_c and are analogous for different σ'_o. This reveals the effectiveness of δ_N in quantifying the soil degradation induced by various levels of EPWP buildup. Moreover, the results in this study show a similar tendency to Vucetic and Mortezaie (2015). That is, δ_N decreases with increasing EPWP for $r_u > 0.2$–0.3, and its decreasing rate with r_u turns more significant as r_u approaches 1.

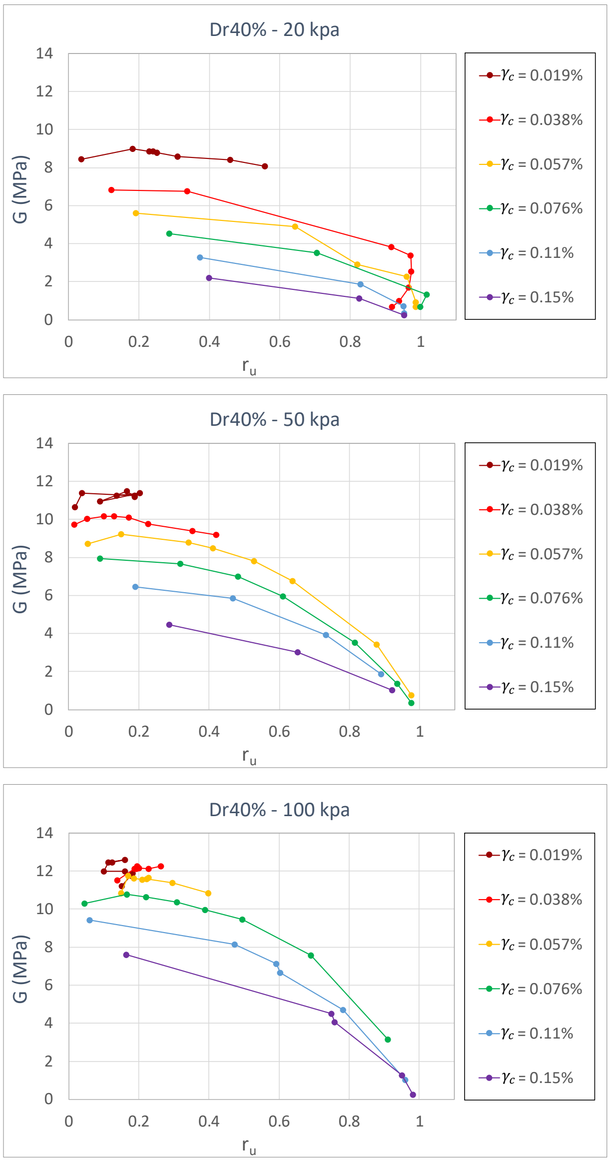

Figure 5. Shear modulus (G) vs. excess pore water ratio (r_u).

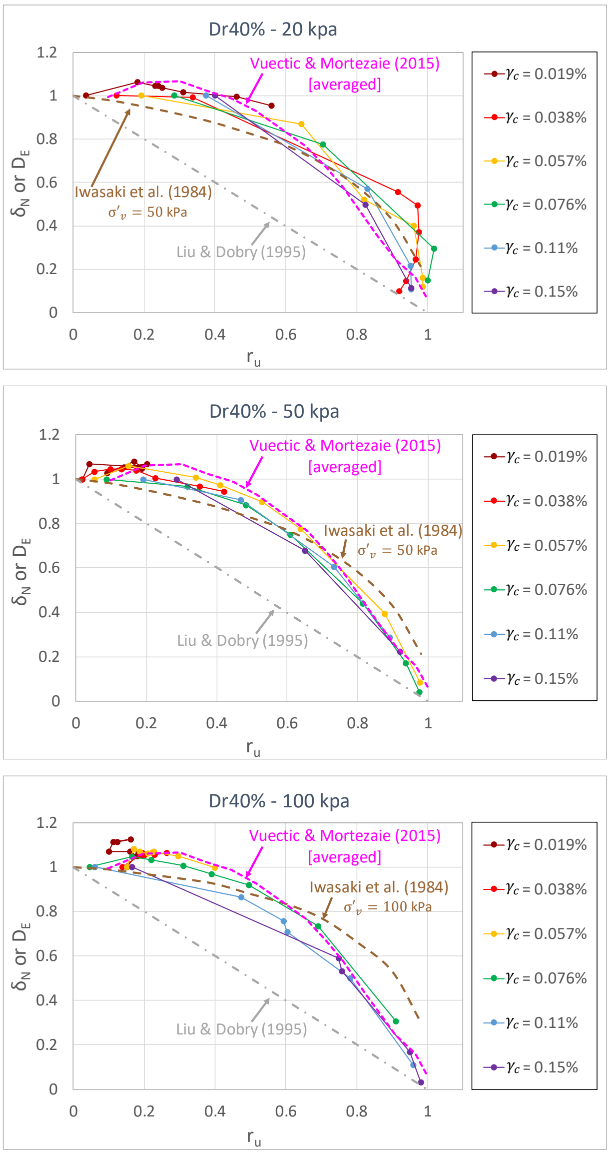

Figure 6. Degradation index (δ_N) vs. excess pore water ratio (r_u)

Another feature reported in Vucetic and Mortezaie (2015), that δ_N slightly increased for small r_u, can also be observed in this study at smaller γ_c. The cause for this, suggested by Vucetic and Mortezaie (2015), is the cyclic stiffening due to the changes at particle contacts, which prevails over the effects of the EPWP buildup at several initial loading cycles.

The reduction factor for subgrade reaction coefficient (D_E) versus r_u proposed by Iwasaki et al. (1984) based on the plate loading tests on saturated sand and by Liu and Dobry (1995) based on the centrifuge model tests of a laterally loaded pile are also included in Figure 6 for comparison [Liu and Dobry (1995) used the term "degradation parameter" (C_u) yet it is replaced by by D_E in this paper for simplicity reason]. The D_E versus r_u curve of Iwasaki et al. (1984) is roughly close to the δ_N versus r_u curves of this study and Vucetic and Mortezaie (2015), indicating that the subgrade reaction coefficient and the shear modulus of soil may have a similar degradation tendency with EPWP buildup. However, the D_E versus r_u curve suggested by Liu and Dobry (1995) is linear and hence shows a large discrepancy from the other curves in Figure 6 except for r_u close to 0 and 1. This is possibly because the horizontal subgrade reaction coefficient (k_H) of a laterally loaded pile might be dependent on the section rigidity of the pile based on the suggestions of JRA (2017) on the estimation of k_H.

In the seismic design practice of foundations, the mixed use of quantification indices for the degradation of liquefiable soil in the shear or elastic modulus (for the analysis based on the elasticity theory) and the subgrade reaction coefficient (for the analysis based on the beam-on-elastic-foundation approach) is common. The above comparison implies that this mixed use may be acceptable for shallow foundations; however, it should be applied with caution for laterally loaded piles.

5 CONCLUSIONS

- The strain-controlled cyclic loading in the CSS tests until the specimen is liquefied (or the stop criterion is met) provides a gradual variation of the shear modulus and the EPWP. This is beneficial for the observation of soil degradation during the process of liquefaction.
- The test results indicate that the EPWP buildup with increasing N is significant if γ_c is larger than a certain threshold, and it subsequently causes further degradation in G other than the strain-dependent nonlinearity of soil. Moreover, G can be substantially reduced when soil is close to liquefaction.
- The index δ_N is demonstrated to effectively quantify the soil degradation induced by various levels of EPWP buildup. The δ_N values obtained in this experimental study decreased as EPWP increases for $r_u > 0.2{-}0.3$, and their decreasing rate with r_u turned more significant as r_u approaches 1. These findings are conformable to Vucetic and Mortezaie (2015).
- According to the comparison presented in this study, the degradation in the subgrade reaction coefficient and the shear modulus of soil induced by EPWP buildup could be discrepant in some situations, so caution is needed in the seismic design practice of foundations in liquefiable soil.

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