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Initial observations on the direct simple shear loading response of sand-silt mixtures with layers

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ABSTRACT: Laboratory experimentation has contributed significantly to the current understanding of the mechanical response of coarse-grained and fine-grained soils, particularly in characterizing such materials with respect to general shear loading. In spite of this, the current understanding on the shear loading response of interbedded sand-silt layers is very limited. In most of the past studies, the experimental studies have been focused on testing uniform sand-silt specimens with homogeneous mixtures, although natural sand and silt deposits exist predominantly in the form of interbedded layers as opposed to homogeneous mixtures. This is largely due to the gravity depositional processes that occur in deltaic and marine settings. The focus of the study presented herein is to compare the monotonic and cyclic shear loading response of interbedded sand-silt specimens with that corresponding to homogeneous sand-silt mixture specimens. A limited number of undrained monotonic and cyclic direct simple shear tests were conducted on normally consolidated, reconstituted specimens of Fraser River sand and silt with pre-selected sand-silt composition, with some specimens simulating silt/sand layering, and the initial observations from this work is presented herein.

1 INTRODUCTION

Investigations on many of ground failures due to earthquakes (Holzer et al. 1999, Youd et al. 2009, Cubrinovski et al. 2017) have reported the existence of interbedded layering sequences of sands, silts, and clays in the stratigraphy of natural soils found in the affected areas. In spite of this, the evaluation of earthquake-induced liquefaction and lateral spreading in the current practice is mainly dependent on the approaches developed in relation to clean sand. Boulanger et al. (2016) reported that these current procedures tend to over-predict liquefaction effects in interbedded sand, silt, and clay deposits. Further, Boulanger et al. (2016) identified the limitation of the triggering correlations for intermediate soils with fines having certain plasticity, and the effects of thin layers in characterizing the soil and the response of soil are some of the many factors that affect the prediction of liquefaction effects in interbedded soil deposits.

In most of the cases, laboratory experimental studies on evaluating cyclic shear resistance of soil have been focused on uniform sand specimens, fine-grained specimens, or uniform sand-fine mixtures. Few researchers (Elgamal et al. 1989; Zeghal et al. 1999; Fiegel & Kutter, 1994; Balakrishnan & Kutter, 1999; Kokusho, 1999; Kokusho & Kojima, 2002; Brennan & Madabhushi, 2005; Özener et al. 2009; and Özener & Özaydın, 2010) have investigated the behaviour of stratified sand-fines during liquefaction by both experimental and numerical approaches. Outcomes of these research studies, based on centrifuge and shake table tests that were performed on multilayer sand-fine soil systems, have identified the potential for redistribution of voids and leading to the generation of thin water films underneath the lesser permeable fine layers, in comparison to higher permeable sand layers. It has been postulated that the entrapped water layers can serve as very low shear strength zones to cause post-liquefaction ground failures such as flow-failures or large lateral displacements.

Yoshimine & Koike (2005), using the outcomes from monotonic and cyclic triaxial tests, have reported that the stratified structure created in sand specimens due to segregation of finer particles could significantly influence the liquefaction resistance; for example, at the same density, the stratified sand showed stiffer and dilative response in comparison to those from counterpart uniform sand. The effects of the silt sandwich in the liquefaction characteristics of stratified deposits were investigated by Jia & Wang (2013) through stain-controlled undrained cyclic triaxial tests on stratified sand samples (interlayered with different thickness of silt). They identified a critical value for the silt thickness, in which the resistance to liquefaction decreased with the increasing of the thickness of silt layer if the thickness of silt layer is less than the critical value.

2 EXPERIMENTAL ASPECTS

2.1 *Material tested*

Fraser River silt (excavated from a site on the south bank of the Fraser River adjacent to the Port Mann Bridge in British Columbia) and Fraser River sand available were used for the experiments. In order to the make soil samples for testing, Fraser River silt was oven dried and then sieved to remove the fine sand portion (coarser than 75 μm). Fraser River silt used in the tests to prepare layered specimens was non-plastic soil with a mean particle diameter of about 18 μm and a specific gravity of 2.72. The finer portion (finer than 75 μm) of Fraser River sand was removed by wet sieving.

The specimens prepared using 100% coarse-grained portion of Fraser River sand (as prepared above) were identified as 100C0F (meaning 0% of fine-grained and 100% coarse-grained soil). In a similar way, the specimens prepared using 100% fine-grained soil were identified as 0C100F (meaning 0% of coarse-grained and 100% fine grained soil). Several layered specimens were prepared with different layering arrangements as per below: (a) double layered specimen – one sand layer and one silt layer; (b) triple layered specimen – silt layer sandwiched between two sand layers, and (c) triple layered specimen – sand layer sandwiched between two silt layers. In the cases of sand-silt mixtures and layers, all the specimens possessed 50% of coarse-grained and 50% of fine-grained soil content.

2.2 *Specimen preparation*

Dried and processed soil of ~140 g was used to prepare a specimen. For the double layered specimen, two selected representative portions of fine-grained and coarse-grained soil each containing 70 g were prepared separately. Similarly, for triple layered specimens, 70 g of soil for the sandwiched middle layer, and two portions of 35 g of soil for the bottom and lop layers were prepared separately.

The selected fine-grained portion was transferred into a 300 ml beaker to prepare saturated slurry. De-aired water was added to the soil while mixing until homogeneous slurry was formed. The slurry was kept under vacuum for 24 hours for de-airing purposes. The sample was stirred, re-mixed, and shaken occasionally while it was under vacuum, in order to minimize the entrapped air bubbles inside the slurry. It was allowed to consolidate under its own weight; at this point, the thin-clear water film formed at the top of the slurry was carefully removed by applying suction. Remaining material was stirred once again, thus preparing a slurry mix that is ready to be gently placed in the specimen cavity surrounded by the wire-reinforced membrane. Efforts were made during specimen placement to minimize the formation of entrapped air in the reconstituted specimen.

The selected coarse-grained soil was thoroughly mixed with de-aired water in a 500 ml volumetric flask; then boiled for 30 minutes to remove the entrapped air. After it was cooled down room temperature, the flask was then kept in a vacuum desiccator for at least 24 hours for further removal of entrapped air. The volume flasks were taken out from the vacuum desiccator several times and were shaken vigorously to facilitate the escape of entrapped air bubbles and kept under vacuum.

The coarse-grained portions of the specimens were prepared by water-pluviation, and fine-grained soil was placed using saturated slurry deposition method. The Direct Simple Shear (DSS) test device at the University of British Columbia (UBC), Canada was used for the testing work. The device accommodates a specimen with a diameter of about 70 mm and a height of about 20 mm placed in a wire-reinforced rubber membrane.

As an example, the key steps in preparing one of the specimens identified as 50C50F-(FCF) specimen is shown in Figure 1 (Note: the identification 50C50F stands for the 50% coarse-grained and 50% fine-grained content composition in the specimen, whereas FCF denotes that the specimen has three layers in which coarse-grained layer is sandwiched between two fine-grained soils at the top and bottom boundaries). Figure 1[A] and [D] show the saturated slurry deposition by spooning for the bottom and the top layers respectively; whereas, pluviation of the coarse-grained soil in the water medium and the syphoning the water out after the completion of pluviation as a preparation for the slurry depositing of the top (fine-grained) layer are shown in Figure 1[B] and [C], respectively.

The specimen, which was ready to be tested shown in Figure 1[E], is laterally confined by the wire-reinforced rubber membrane and it enforces an essentially constant cross-sectional area and prevents the specimen from localized lateral straining during consolidation and shear deformation. Therefore, a constant-volume condition can be enforced by restraining the top and bottom loading platens of the specimen against vertical movement to impose a height constraint. It has been shown that the decrease (or increase) of vertical stress in a constant-volume DSS test is essentially equal to the increase (or decrease) of excess pore water pressure in an undrained DSS test where the near constant-volume condition is maintained by not allowing the mass of pore water to change (Dyvik et al. 1987). Therefore, in this test series, change of vertical stress during constant-volume shearing is interpreted as the equivalent excess pore-water pressure due to shear loading.

To evaluate and assess the monotonic shear loading response of sand-silt layered specimens, three kinds of layer configurations were considered. Using the specimen identification approach above, constant-volume DSS tests on: (a) a two layered specimen denoted as 50C50F-(FC)-M; and (b) two cases of three layered configurations 50C50F-(FCF)-M and 50C50F-(CFC)-M. It is to be noted that FCF and CFC stand for the specimens in which the middle layer comprises coarser-grained soil and finer-grained soil, respectively. The last letter "M" indicates monotonic testing. The photographs in the middle row of Figure 2 show the arrangements of two layered specimen and the two cases in three layered specimens. The test on homogenous sand-silt specimen is identified as 50C50F-M. For comparison purposes, monotonic shear loading response of sand-silt mixtures are compared with the responses of the parent material of the mixtures, that were observed in homogeneous sand-alone and homogeneous silt-alone specimens denoted as 100C0F-M and 0C100F-M, respectively.

For the cyclic tests, only homogeneous silt mixture and afore-mentioned three cases of layer configuration were considered. All cyclic DSS tests were performed with a CSR of 0.12. The last letter "12" indicates cyclic testing with a stress ratio of 0.12. The test identification approach will be further illustrated as the tests are listed in Table 1 in the next section.

2.3 *Constant-volume direct simple shear tests*

The test specimens were initially consolidated to vertical effective consolidation stress (σ'_{vc}), of 100 kPa approximately and then subjected to constant-volume monotonic shear loading till 20% of

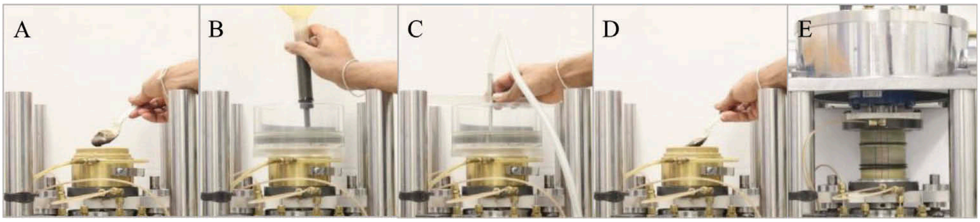

Figure 1. Preparations steps for 50C50F-(FCF) layered specimen- [A] saturated slurry deposition for bottom fine-grained layer; [B] water pluviation of coarse-grained layer in the middle; [C] syphoning the excess water from the reservoir used for water-pluviation; [D] saturated slurry deposition for top fine-grained layer; [E] triple layered specimen that sand sandwiched by two silt layers ready to be tested

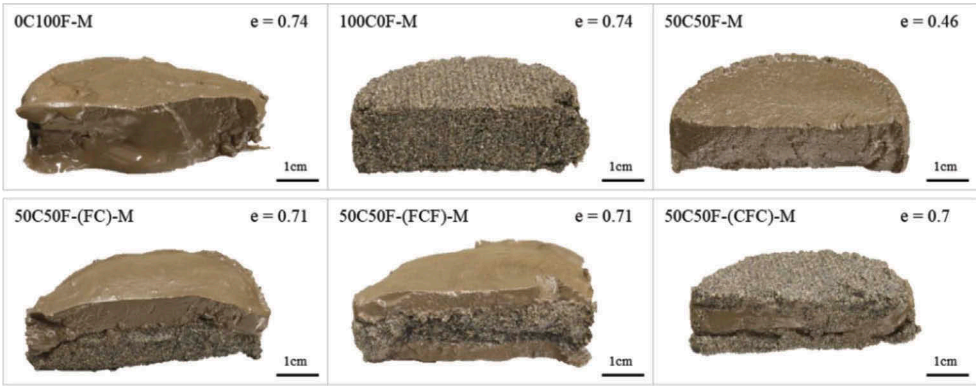

Figure 2. Perspective of the tested specimens after they were taken away from the DSS test device and cut in to half vertically

Table 1. Test parameters and summary of test results

Test ID		e	σ'_{vc} (kPa)	$r_{u\text{-}max}$	$S_{u\text{-}peak}/\sigma'_{vc}$	$S_{u\text{-}DSS}/\sigma'_{vc}$	
Monotonic	0C100F-M	0.74	99	0.72	0.18	0.2	
	100C0F-M	0.74	101	0.46	0.21	1.39	
	50C50F-M	0.46	97	0.63	0.18	0.26	
	50C50F-(FC)-M	0.71	97	0.65	0.19	0.26	
	50C50F-(FCF)-M	0.71	99	0.75	0.18	0.18	
	50C50F-(CFC)-M	0.7	100	0.63	0.2	0.28	
					CSR [τ_{cyc}/σ'_{vc}]	$N_{cyc\ [\gamma=3.75\%]}$	$r_{u[\gamma=3.75\%]}$)
Cyclic	50C50F-12	0.46	96	0.95	0.119	9.8	0.75
	50C50F-(FC)-12	0.68	99	0.98	0.121	11.7	0.85
	50C50F-(FCF)-12	0.68	97	0.95	0.124	8.7	0.75
	50C50F-(CFC)-12	0.69	102	0.99	0.116	8.8	0.81

e - post consolidation void ratio prior to the application of shearing CSR - Cyclic Stress Ratio
σ'_{vc} - Vertical consolidation stress prior to the application of shearing $r_{u\text{-}max}$ - Maximum pore water pressure ratio
$S_{u\text{-}peak}/\sigma'_{vc}$ - Normalized peak undrained shear strength before the phase transformation type response
$S_{u\text{-}DSS}/\sigma'_{vc}$ - Normalized undrained shear strength at DSS 20% strain
$N_{cyc\ [\gamma=3.75\%]}$ - Number of uniform loading cycles to reach shear strain of 3.75%
$r_{u\ [\gamma=3.75\%]}$ - Pore-water pressure ratio when shear strain reaches 3.75%

shear strain (γ) was attained with an approximate strain rate of 10% shear strain per hour. For the cyclic DSS tests, symmetrical sinusoidal cycles of shear loading (τ_{cyc}) to impose the desired constant cyclic stress ratio [CSR = τ_{cyc}/σ'_{vc}] amplitude on the specimen was applied at a frequency of 0.1 Hz. As an 'index' for assessing and comparing the cyclic shear strength, the attainment of 3.75% single-amplitude horizontal shear strain in a DSS specimen was considered as a criterion for unacceptable performance and to terminate the cyclic shear loading test phase. At the end of the test, wire-reinforced rubber membrane was carefully removed, and photographs were taken (see Figure 2) for the references of sand-silt layers, after the specimens were cut in to half vertically.

2.4 *Test parameters*

Table 1 provides a summary of the test parameters and test results that are presented and discussed in this paper. As noted in the previous section, the top row of Figure 2 (left to right) shows the photographs of after-test specimens from the monotonically loaded tests conducted with parent 100% silt, 100% sand materials, and the 50%-50% homogeneous mixture of sand and silt, respectively.

Again it is highlighted that, for the cyclic DSS tests, only homogeneous silt mixture and afore-mentioned three cases of layer configuration were considered as listed in Table 1 under cyclic section. All cyclic DSS tests were performed with a CSR of 0.12. Saturated slurry deposition method does not facilitate control in achieving a target void ratio at the time of the slurry placement; therefore, it should be noted that the specimens yield different values of post consolidation void ratio (e) prior to the application of shear loading.

3 MONOTONIC SHEAR LOADING RESPONSE

The stress-strain, pore-water pressure, and stress path response obtained from constant-volume monotonic DSS tests performed on sand, silt, sand-silt mixture and sand-silt layered

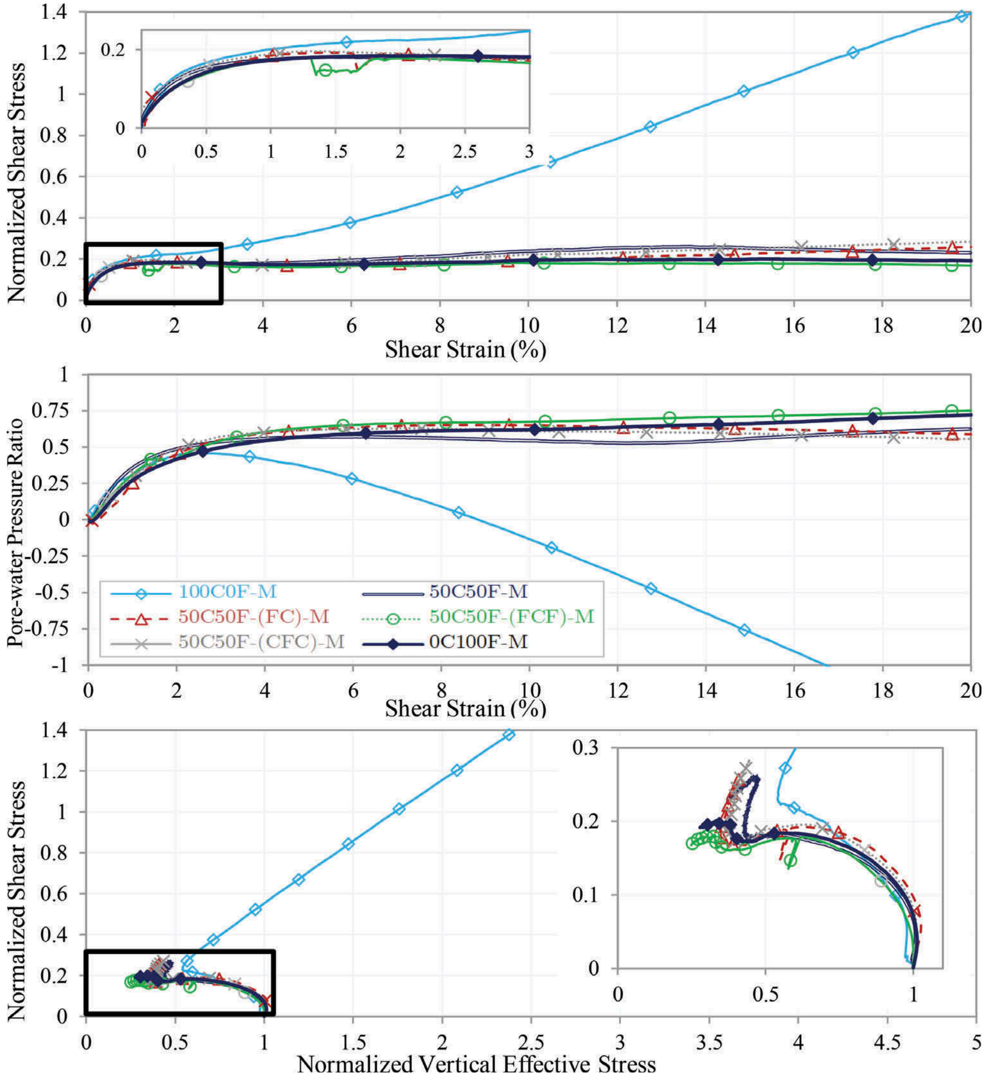

Figure 3. Stress-strain, pore-water pressure and stress path response observed in monotonic DSS tests on layered sand-silt specimens, a homogeneous sand-silt mixture, a homogeneous sand-alone, and a homogeneous silt-alone specimen

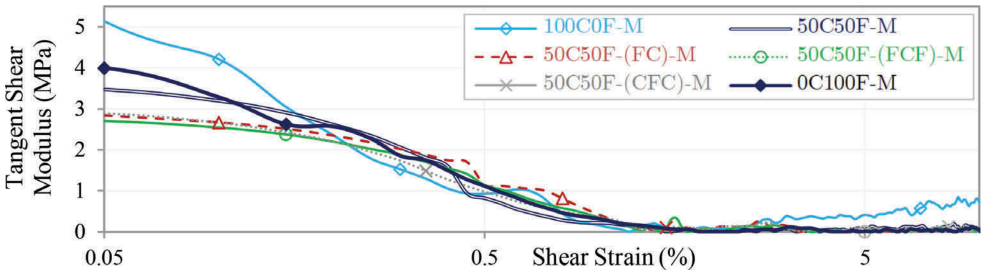

Figure 4. Reduction of tangent shear modulus obtained from constant-volume monotonic DSS tests on sand, silt and sand-silt specimens with layers

specimens are presented in Figure 3. It can be seen that sand-alone (100C0F-M) specimen showed notable strain hardening, dilative tendency with significant development of negative pore-water pressure in constant volume monotonic DSS shearing.

As listed in Table 1, 100C0F indicate significantly greater undrained shear strength at ~20% of shear strain in comparison to those of all other specimens. Approximately similar stress-strain responses can be observed for the other specimens; therefore, stress-strain responses in the initial stage are presented separately in the same figure for the clarity.

Although the stress-strain plots in Figure 3 for silt-alone, and homogenous sand-silt mixture specimens showed similar patterns of response, careful observation of those, especially in the strain ranges of 1% to 2%, derived for layered sand-silt specimens, indicated sudden drops of shear stress and later gradual recovery. This may be due to probable local shear or slippage especially at the sand-silt layer interface in the layered specimens (although no direct visual observations are made on this). All three test results that involved with layered sand-silt specimens indicated notable sudden drops in shear stress, following up with gradual recoveries, however, such observations were not found in uniform specimens. Stress-path responses during the initial stage of shearing in all three layered specimens seem to be similar, where all showed contractive type tendencies; however at large strain levels, 50C50F-(FC)-M and 50C50F-(CFC)-M specimens showed slight dilative tendency, while 50C50F-(FCF)-M specimen continued with contractive tendency.

Furthermore, all three layered specimens indicate their overall void ratios to be around 0.7, despite the different arrangement of layers. However, the uniform specimen (50C50F-M) showed the lowest void ratio of 0.46, making it the densest specimen with respect to other specimens; yet, no significant difference could be observed for the shear resistance in comparison to the layered specimens. From these observations, it can perhaps be argued that layered specimens would have indicated greater shear resistance in comparison to that of uniform specimens, if they were tested at same void ratio. However, in the cases of tangent shear moduli derived for the range of shear strain 0.05% to 10% (presented in Figure 4) from the stress-strain plots, all layered specimens show very similar reduction of tangent shear moduli; whereas, the initial tangent shear moduli derived for homogeneous specimen show slightly higher values in comparison to those for layered specimens.

4 CYCLIC SHEAR LOADING RESPONSE

The results for strain accumulation, pore-water pressure development, strain-pore-water pressure, stress-strain, pore-water pressure and stress path response of constant-volume cyclic DSS tests, performed on a homogeneous sand-silt mixture and three types of layered sand-silt specimens with a CSR of 0.12, are presented in Figure 5. Similar to the observations made in monotonic DSS test results, the cyclic responses of both homogeneous sand-silt and layered sand-silt specimens did not show a significant difference, and the cyclic shear resistance of those specimens seems to fall within a very narrow margin with respect to each other. Nevertheless, it should be noted that the homogeneous sand-silt specimen (50C50F-12) had the

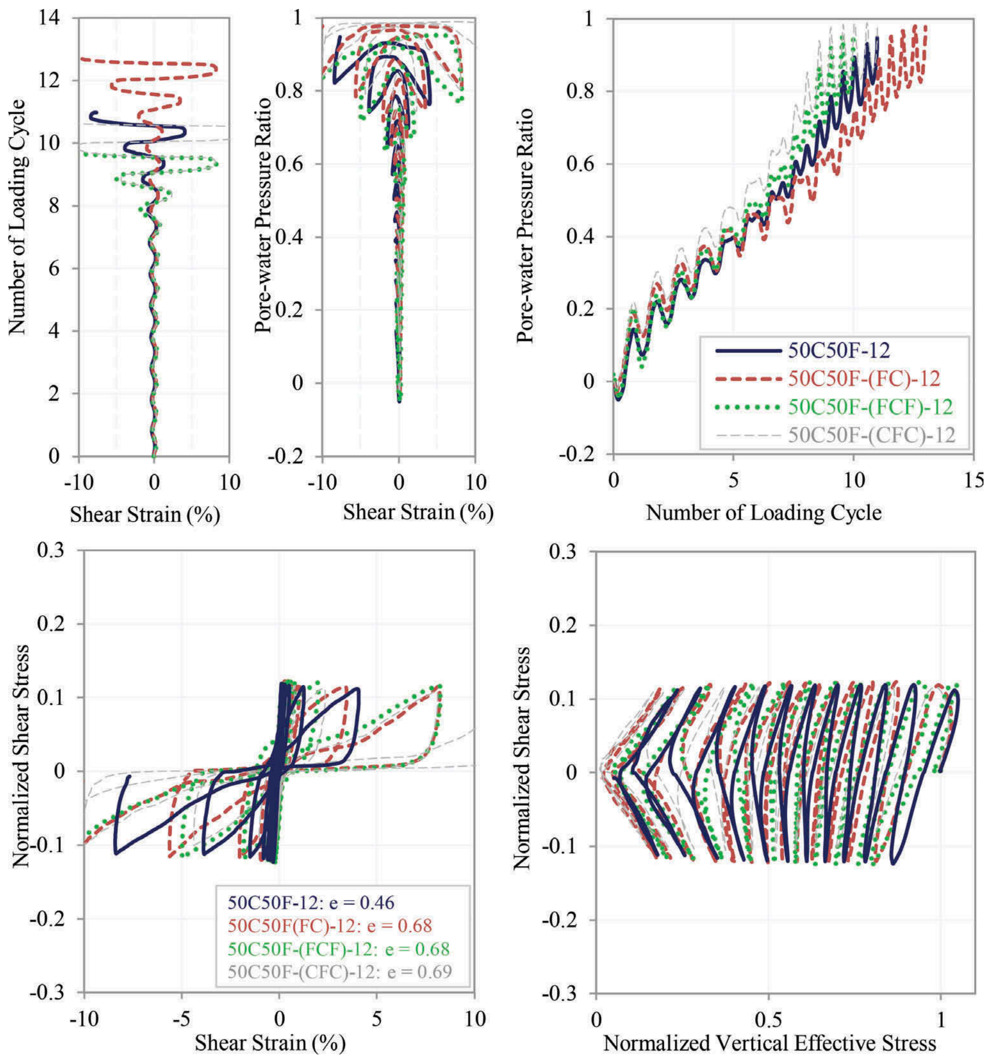

Figure 5. Strain accumulation, pore-water pressure development, strain-pore-water pressure, stress-strain, pore-water pressure and stress path response observed in cyclic DSS tests on layered sand-silt specimens and a homogeneous sand-silt mixture

lowest void ratio, although, the observed cyclic resistance of it is similar to those layered specimens that had much greater void ratios of 0.68 and 0.69.

The degradation of the shear stiffness during cyclic shearing was investigated, by computing the secant shear moduli for respective loading cycles considering the highest positive shear stress in a given loading cycle with the corresponding positive shear strain as shown in Figure 6 for the case of 50C50F-(FC)-12. Figure 7 shows the comparison of the degradation of secant shear moduli of homogeneous and layered sand-silt specimens. For convenient comparison, the number loading cycles were divided by the corresponding number of loading cycles to attain threshold shear strain of 3.75% to derive normalized number of loading cycles.

The comparison of results presented in Figure 7 shows secant shear modulus degradation characteristics of specimens with different layer configurations are very similar to each other. Homogeneous sand-silt mixture specimen showed greater initial secant shear modulus, however the amount of degradation during cyclic loading is also greater meaning higher rate of degradation for the homogeneous specimen.

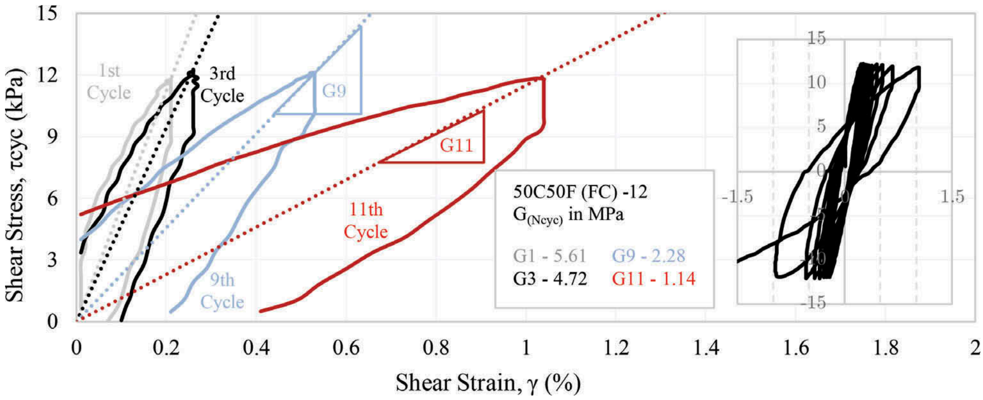

Figure 6. Shear modulus calculation for several loading cycles in 50C50F-(FC)-12 cyclic DSS test

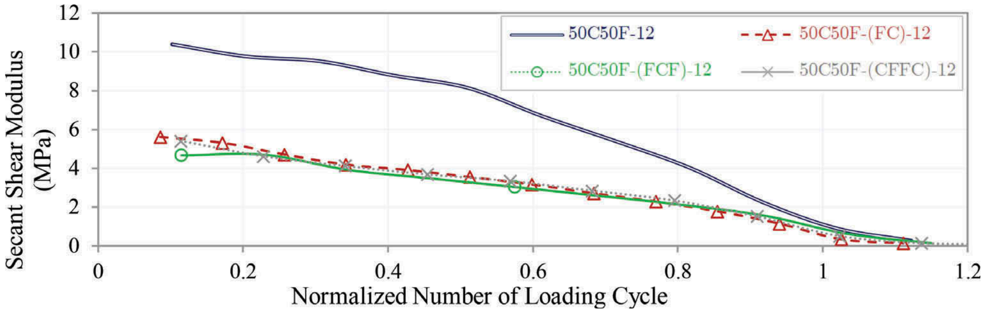

Figure 7. Comparison of shear modulus degradation with respect to normalized number of loading cycles

5 SUMMARY AND CONCLUSIONS

Shear loading response of sand-silt layered specimens with three different layer configurations were assessed by constant volume DSS tests under both monotonic and cyclic loading conditions. For comparison purposes, a homogeneous sand-silt specimen was also tested, and all the specimens had a composition of 50% coarse-grained soil and 50% fine-grained soil. Sand-alone and silt-alone specimens were also tested in monotonic loading to compare the mixture or layered specimens responses with respect to those of parent soil of the mixture/layer, key observations and conclusions can be identified as below

- The configurations with layers considered in the study do not seem to make a significant effect on both the monotonic and cyclic shear loading responses.
- Homogeneous sand-silt specimen, which had an initial lower void ratio compared to those of sand-silt layered specimen with same sand-silt composition, also showed similar monotonic and cyclic shear resistance to those of layered specimens; these observations suggest that homogeneous sand-silt mixture would likely have displayed lesser shear resistance if compared with the sand-silt layered specimens at the same void ratio.
- Sand-silt layered specimens seem to indicate possible local shear and slippage preferably at the sand-silt interface during constant-volume monotonic direct simple shear tests.
- Despite the monotonic and cyclic shear resistance observed from stress-strain plots of layered specimens were found to be very similar to those of homogeneous specimens, initial tangent stiffness (from monotonic stress-strain plots) and initial secant shear stiffness (from cyclic stress-strain plots) of homogeneous sand-silt mixtures were identified to be slightly

greater (as one would expect due to the lower void ratio) than those of sand-silt layered specimens. However, it seems that this slightly greater stiffness values observed in homogeneous specimen did not contribute to increasing the shear resistance of homogeneous mixture in both cases of monotonic and cyclic shear loading.

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