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Monotonic Shear Loading Response of Sand-Silt Mixtures from Direct Simple Shear Tests

Achala SOYSA[a,1] and Dharma WIJEWICKREME[a]
[a] *University of British Columbia, Canada*

Abstract. In an effort to address the inconsistent findings and conflicting conclusions derived from the studies on shear loading response of sand and silt mixtures, a comprehensive experimental research program has been undertaken to investigate the shear loading response of sand-silt mixtures. As a part of the experimental study, constant-volume monotonic direct simple shearing tests were conducted on normally consolidated, reconstituted sand-silt specimens with pre-selected compositions. Sand and silts originating from the Fraser River Delta in British Columbia, Canada was used as the test materials covering the complete range of mixture compositions considered for the study. Results and observations on this experimental work are presented, and specifically, the observed responses of sand-silt mixtures in comparison to those observed from the testing of sand-only and silt-only specimens are discussed.

Keywords. Monotonic shear, direct simple shear test, sand-silt mixtures.

1. Introduction

The findings and conclusions derived from the studies on shear loading response of sand and silt mixtures still remain conflicting despite many researchers have investigated the topic in the last three decades. For instance, there are a significant number of studies [1–10] that have reported that the presence of, and increase in, the silt content in a sand would tend to increase the resistance of sand to liquefaction. The basis for these studies primarily comprised data gathered from laboratory experimental investigations, in situ testing, and field observations during and aftermath of seismic induced ground motions. In contrast to above findings, some investigations based on laboratory experimental studies [11–14] indicate that the presence, and in particular the increase of silt, in sand would cause its liquefaction resistance to decrease.

In addition to the above-mentioned conflicting findings, there are other findings in the available literature that suggests that the increasing fines content, up to a threshold value, would initially decrease the shear resistance of sand, and beyond that threshold the shear resistance would increase with further increase of fine content. For example, Zlatovic and Ishihara [15] based on their tests on Toyoura sand and silt mixtures have observed that, increasing silt contents up to 30%, would result in corresponding increase in contractiveness and decrease in peak and residual strength; whereas, further increase in the silt content would decrease the contractiveness of the material and in turn, would increase its peak and residual strengths. Similarly, some other researchers

[1] Corresponding Author, 6250 Applied Science Lane, Vancouver British Columbia, Canada V6T 1Z4; E-mail: achala@civil.ubc.ca.

[16–18] have observed that the shear resistance would initially decrease with increasing silt content, and thereafter increase with further increase in silt content. Moreover, based on their experimental results, Polito and Martin II [18] suggested that no general statements as to the liquefaction susceptibility of a soil at a specific silt content (e.g., 10 or 30% silt) can be made without knowing the “limiting silt content” of the soil.

It is also noteworthy that some other researchers [19–21] pointed out that the basis of the comparison (i.e. relative density, global void ratio, or sand skeleton void ratio) is the reason for the apparent different trends of increase (or decrease) in soil resistance and strength with respect to change in silt content. Clearly, there is a need to conduct more research to address the inconsistent findings and conflicting conclusions derived from the studies so far on shear loading response of sand and silt mixtures.

With this background, an experimental research program has been undertaken to investigate the monotonic shear loading response of sand-silt mixtures covering the complete range of sand-silt compositions in the mixtures. As a part of the study, undrained monotonic direct simple shear tests were performed with pre-selected sand-silt compositions of soil material. This paper presents the finding from this work. In particular, the effect of sand-silt composition on the peak and large strain shear resistance as well as the strength and stiffness characteristics observed during monotonic direct simple shear (DSS) tests are assessed, compared, and discussed.

2. Experimental Aspects

2.1. Tested soil

The sand and silt materials used for the testing program herein originate from the Fraser River Delta of British Columbia, Canada; the materials received in bulk form were processed to obtain the basic sand and silt components required for the experimental study. The sand component was generated by wet sieving to remove the finer portion (finer than 75 μm). With respect to the silt component, the Fraser River silt was oven dried and then sieved to remove the fine sand potion (coarser than 75 μm). The sand and silt were further processed to obtain ingredient sand and silt components required to arrive at specific particle size distributions for the soil specimens; in this regard, sand was sieved through sieve #30, #40, #50, #60, #80, #100, #120, #150 and #200, whereas silt was sieved through sieve #200, #230, #270, and #325 respective portions were kept in storage containers. The calculated portions of sand and silt required for the specific gradations were taken from the respective storage containers, weighed and mixed in order to control the specific gradation of the soil specimens. Sample naming was designated based on their coarse and fine content; for example 100C0F refers to 100% processed Fraser River sand (particles finer than 75 μm were removed), whereas 0C100F denotes to 100% processed Fraser River silt (particles coarser than 75 μm were removed). 50C50F stands for the specimen with 50% of 100C0F and 50% of 0C100F fraction expressed based on dry weight.

The 100C0F and 0C100F materials essentially comprise the natural gradation of Fraser River sand and silt respectively, after the removal of finer and coarser fraction as mentioned earlier. The intent herein was to achieve a mixture gradation, which closely resembles the natural gradation of a soil, when sand is mixed with silt as shown in Figure 1– the intent was to avoid unrealistic soil gradation curves.

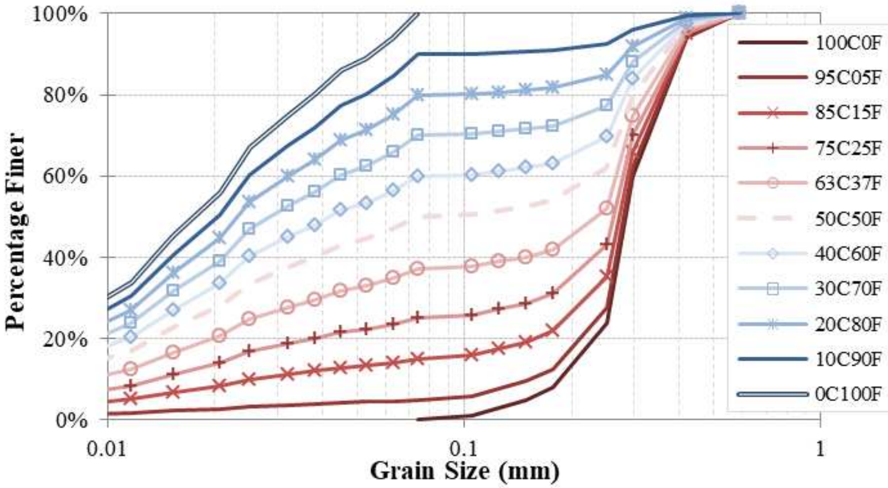

Figure 1. Particle size distribution of the sand, sand-silt mixtures and silt.

2.2. Specimen preparation

100C0F and 95C05F specimens were prepared by water pluviation method; whereas other specimens were prepared by the slurry deposition method. Initial trials on specimen preparation of sand-silt mixtures by water pluviation method revealed that the fine-grained soils would segregate in the specimen when the fines content is greater than 10%. As such, saturated slurry deposition method was used for the preparation of specimens with other sand-silt mixtures and the 100% silt. Suitable water contents were selected based on visual observations and experience-based judgment when preparing the sand-silt mixtures for slurry preparation, in order to ensure minimum amount of segregation with fully saturated specimens.

The following actions were undertaken in preparation of the DSS test device to receive the reconstituted specimens: (i) the porous stones to be used for drainage at the two end platens were initially boiled in de-aired water, cooled to room temperature, and then placed in the DSS device; (ii) the reinforced rubber membrane, which would later enclose the specimen was initially placed on and sealed to the bottom specimen-base-pedestal using an o-ring; (iii) then, a split-mold was mounted around the base pedestal, so that the wire-reinforced rubber membrane could be stretched to line the split-mold thus forming a cylindrical cavity (a vacuum is applied between the mold and the membrane to stretch the membrane and create the sample cavity).

For the case of water pluviation, initially, a known weight (about 200 g) of dry soil was placed in a flask. The soil was saturated by boiling with de-aired water in the flask and then cooled to room temperature. After cooling to room temperature, the sample was kept under vacuum until sample reconstitution. The cylindrical cavity was then filled with de-aired water, with a cylindrical extension mounted on the split-mold essentially providing a reservoir of water above the mold-level during water pluviation. The already boiled/cooled saturated soil in the flask was then directly pluviated (deposited) into the membrane-lined, de-aired-water-filled, split-mold cavity prepared as per above. In the water pluviation process, the transfer of soil mass from the flask to the cavity occurs through the water medium by mutual displacement of water with coarse-grain soil under gravity. Once the mold was filled slightly in excess of the required specimen height, the excess coarse-grain soil was removed using a suction nozzle. The suction nozzle was kept at a constant height and traversed over the footprint of the specimen, and this process allows obtaining a final leveled soil surface at the top of the specimen.

In saturated slurry deposition, the wet bulk soil sample for testing was thoroughly stirred to achieve a homogenous paste. The mixture was allowed to settle under its own weight, and excess water on the top surface was siphoned out. The paste was then placed under vacuum until sample reconstitution. The paste prepared as per above was then carefully placed in the cylindrical cavity of DSS test device using a spoon. Herein, careful attention was paid to avoid air being entrapped within the specimen during spooning and to achieve a uniform and even top surface.

After placing the sufficient amount of soil material to achieve an initial specimen height of about 25 mm, the top surface of the specimen was brought to contact with the top pedestal of the test device so that the specimen would be subjected to a relatively small vertical confining stress (i.e., seating load less than 5 kPa). At this point, after placing an o-ring to seal the membrane with the top pedestal and removing the split mold, the specimen was ready for consolidation to the desired vertical effective confining stress (σ'_{vc}) of 100 kPa.

2.3. Constant-volume direct simple shear tests

In the DSS test device, a cylindrical soil sample, 70 mm in diameter and ~20 mm in height, is placed in a reinforced rubber membrane. The reinforced rubber membrane is stiff enough to constrain any lateral deformations and therefore, the soil behavior is essentially in a state of zero lateral strain during consolidation and shear loading. Simple shear tests can be conducted in drained condition or constant volume condition. In constant-volume DSS tests, the constant volume condition is enforced by constraining the sample boundaries (diameter and height) against changes. The sample diameter is already constrained against lateral strain using reinforced rubber membrane, and the height constraint is attained by clamping the vertical movement of the top and bottom loading caps. Dyvik et al. [22] have 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.

The specimens were consolidated in incremental loading steps up to σ'_{vc} of 100 kPa. Upon completion of consolidation, the specimens were subjected to constant volume monotonic loading at a horizontal shear strain rate of about 10% per hour till 20% of shear strain (γ) was attained.

2.4. Test program

A summary of the test parameters and test results of the constant-volume DSS tests performed on the soil mixtures shown in Figure 1 are listed in Table 1. For the evaluation and assessment of monotonic shear loading response of sand-silt mixtures, test results obtained from those 11 specimens with different sand to silt fraction were used.

Table 1. Constant-volume monotonic direct simple shear test program and test results.

Test ID	e_c	σ'_{vc} (kPa)	$S_{u\text{-peak}}/\sigma'_{vc}$	S_u/σ'_{vc}	$r_{u\text{-max}}$	φ'
100C0F M	0.74	101	0.21	1.39	0.459	30°
95C05F M	0.76	101	0.22	0.85	0.461	30°
85C15F M	0.56	100	0.18	0.4	0.582	31°
75C25F M	0.46	96	0.17	0.29	0.644	32°
63C37F M	0.41	99	0.17	0.2	0.682	28°
50C50F M	0.46	97	0.18	0.26	0.625	32°
40C60F M	0.61	96	0.2	0.26	0.64	32°
30C70F M	0.54	101	0.19	0.29	0.57	32°
20C80F M	0.62	102	0.18	0.2	0.701	31°
10C90F M	0.65	100	0.18	0.21	0.662	31°
0C100F M	0.74	99	0.18	0.2	0.721	35°

It should be noted that the neither water pluviation nor saturate slurry deposition methods allow full control over the void ratio at the stage of soil placement in the DSS device; therefore, the variations in the post-consolidation void ratio (e_c) of the specimens listed in Table 1 are to be expected. The observed peak undrained shear strength observed from the specimens before the phase transformation type response or initial strain softening type response ($S_{u\text{-peak}}$), and undrained shear strength (S_u) during the tests at 20% of shear strain were normalized by σ'_{vc}, and the results are listed in Table 1. Based on the interpreted pore water pressure during monotonic shearing, maximum pore water pressure ratio ($r_{u\text{-max}}$) observed in each test is also presented in Table 1. Effective friction angle (ϕ') was derived considering the maximum obliquity state arising from a stress-path plot.

3. Monotonic Shear Loading Response

The normalized stress-strain and stress path response obtained from constant-volume monotonic DSS tests performed on sand, sand-silt mixtures and silt specimens are presented in Figure 2. A comparison of normalized shear stress and strain response from the tested 11 specimens with the complete range of sand to silt fraction is shown in Figure 3, whereas stiffness reduction of those with respect to increasing shear strains are illustrated in Figure 4.

3.1. Stress-strain response

Sand specimen (100C0F) indicated a prominent strain hardening type response during a constant-volume DSS test; whereas 95C05F and 85C15F sand-silt mixture specimens also showed strain hardening type responses; however, the degree of strain-hardening response observed from the latter two tests are less prominent than that obtained from the tests 100C0F. The remainder of the specimens considered in the test program do not show any significant strain hardening type responses - other than some "mild" strain hardening type response at the later shearing stages (i.e. beyond γ = 10%). As listed in Table 1, maximum value of normalized undrain shear strength for 100C0F, 95C05F and 85C15F are considerably greater than those for other specimens. However, normalized peak shear stresses prior to the phase transformation or strain softening type response for all of the specimens tested are in the range of 0.17 to 0.22. Further, it can be seen from the normalized shear stress-strain plot in Figure 2 that these peak shear stress values for the test specimens occur at around 2% of shear strain.

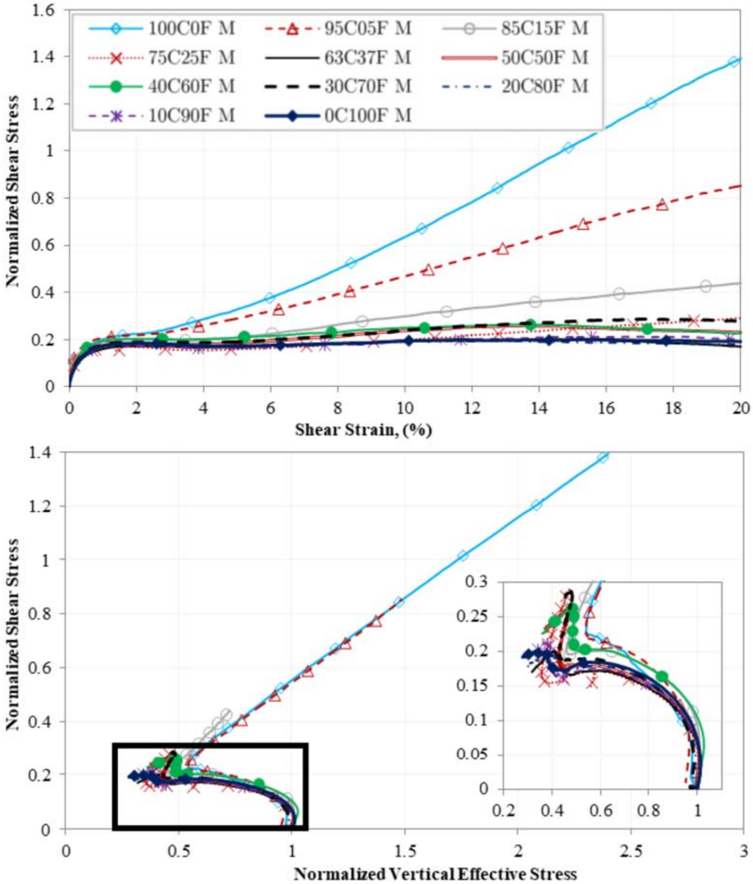

Figure 2. Stress-strain and stress-path response of sand-silt mixtures resulted from constant-volume monotonic DSS tests.

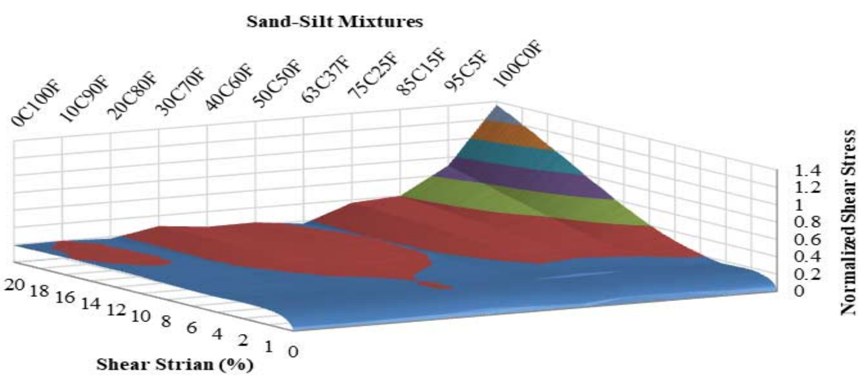

Figure 3. A comparison of normalized shear stress-strain responses of sand-silt mixtures resulted from constant-volume monotonic DSS tests.

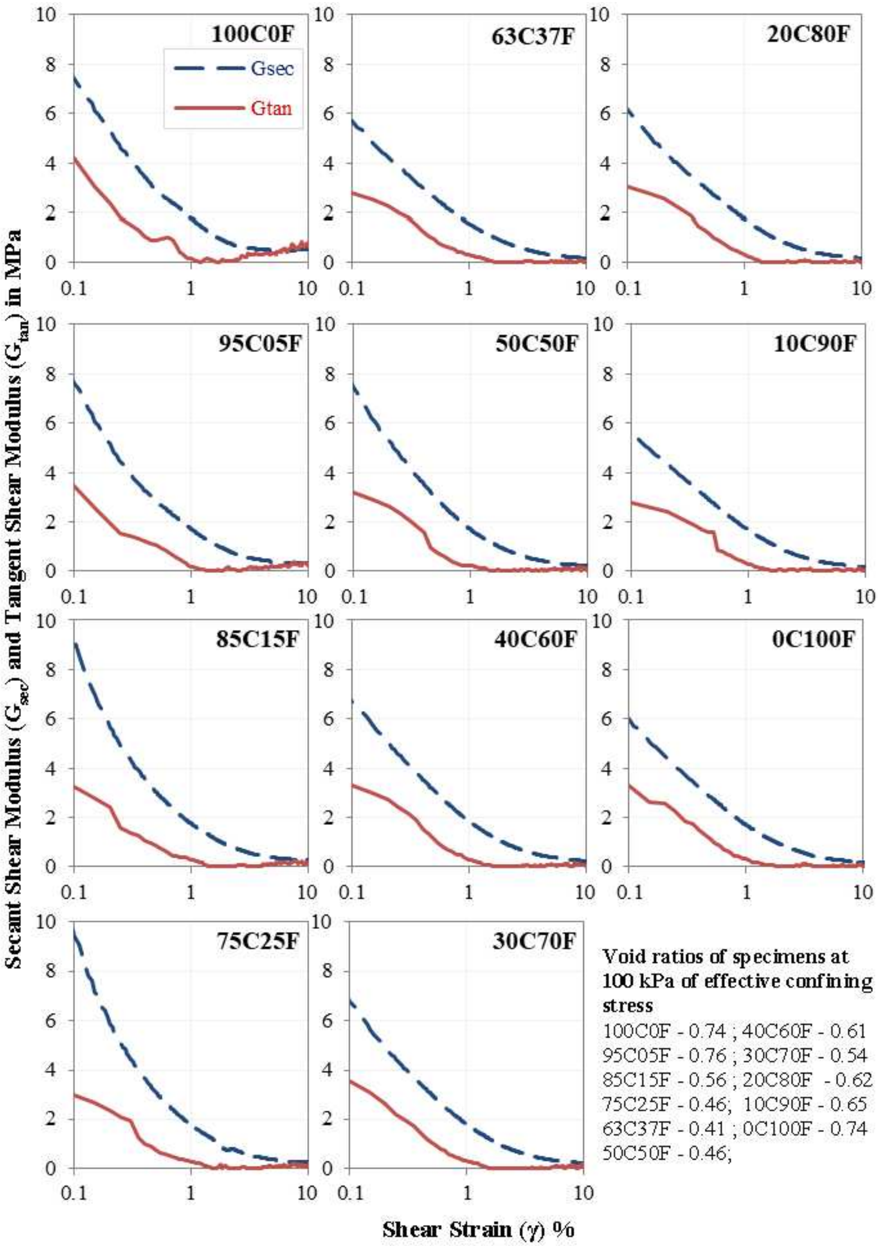

Figure 4. Reductions of secant stiffness and tangent stiffness moduli of sand-silt mixtures derived from constant-volume monotonic DSS tests.

Figure 3 present a clear view on the variation of shear stress development over the complete range of sand to silt fraction in the tested specimens as the shear strain is increased. It should be noted that despite having the lowest e_c in comparison to those of other specimens, 63C37F specimen did not indicate any significant higher shear resistance mobilization - as a result of the increased density.

3.2. Stress-path response

The normalized stress-path plot in Figure 2 indicates that all test specimens showed initial contractive tendency in constant-volume direct simple shear loading. However, after the initial contractive tendency response, stress-paths of 100C0F, 95C05F, 85C15F, 75C25F specimens showed continuous dilative tendency till the termination of the tests; whereas 63C37F, 50C50F, 40C60F, and 30C70F specimens are consist of stress-paths showing initial contractive tendency, then dilative tendency and lastly followed with contractive tendency again. In the cases of 20C80F, 10C90F, 0C100F specimens, contractive tendencies were noted to be continued till the termination of the tests, without any signs of dilative tendencies. Maximum pore water pressure developed during the shearing also indicated the smallest value for coarse-grained 100C0F specimen. The $r_{u\text{-}max}$ value showed a trend of increasing with increasing fine-grained content with greatest value of $r_{u\text{-}max}$ being noted for 0C100F as listed in Table 1.

3.3. Stiffness modulus reduction characteristics

Secant stiffness and tangent stiffness values were computed from the shear stress-strain curves obtained through the constant-volume direct simple shear tests for the range shear strain levels from 0.1% to 10%, and the results are shown in Figure 4. Tangent shear stiffness values at 0.1% shear strain were in the range of 5.5-9.5 MPa, whereas the range for secant shear stiffness values were 2.8-4.2 MPa. As shown in the Figure 4, the results for the 63C37F specimen, which possessed the lowest e_c among the sand-silt mixture test specimens, indicated the lowest secant and tangent shear stiffness values at the shear strain level of 0.1%. Furthermore, it can be noted that secant shear stiffness reduction curves of all test specimens indicate very similar values beyond 0.5% of shear strain. It should be noted that these comparisons of stiffness reduction of different sand-silt mixture specimens are performed for the specimens at different void ratios, and caution should be exercised in the interpretations.

4. Summary

The monotonic shear loading response of Fraser River sand-silt mixtures were assessed using constant volume DSS testing reconstituted specimen. Sand-silt specimens were prepared by slurry deposition method, whereas sand specimen was prepared by water pluviation method. Key observations from the test results can be identified as below.

- From the sand-silt mixtures that were considered in this study, specimen with 63% of sand and 37% of silt produced the lowest void ratio (densest arrangement) when the specimen consolidated to 100 kPa.
- Sand specimen displayed significant strain-hardening type response with dilative tendency during constant-volume monotonic direct simple shearing, and introduction of silt in to the specimen seems to reduce the strain hardening type response and dilative tendency of the sand-silt mixture specimens. However, it is to be noted that these comparisons were made based on the observed shear loading response of sand-silt mixture specimens those possessed different void ratios.

- When the silt content in the specimen is greater than 25%, the monotonic shear resistance seems does not seem to significantly affect by the sand to silt composition.
- Although some notable variation in tangent shear stiffness values across the various sand-silt mixture specimen could be observed at shear strain level 0.1%, those observable differences seem to be diminished beyond the shear strain level of 0.5%.

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