CYCLIC SHEAR LOADING RESPONSE OF SAND-SILT MIXTURES FROM DIRECT SIMPLE SHEAR TESTS

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Abstract

Many of the findings and conclusions derived from the studies on shear loading response of sand and silt mixtures often do not align. Some of the studies have concluded that the presence of fines in sand would increase the material shear resistance against liquefaction; whereas, some other studies have determined that the presence of fines would reduce the shear resistance of sand-silt mixtures. In some of these cases, it has been reported that increasing fine content, up to a threshold value, would initially decrease the shear resistance and then increase with further increase of fine content whereas other results suggest that the presence of fine grains would not affect the strength until a fines content of about 20% has been reached. Some of these differences in research findings arise likely due to: (a) problems in determining the packing relative density in an accurate and consistent way; (b) differences in specimen preparation methods in laboratory testing; (c) differences in loading modes used in testing; and (d) not systematically covering the complete spectrum of fines content ranges. As such, further investigation of shear loading response of sand-silt mixtures in the laboratory is important. With this background, a systematic experimental research program has been undertaken to study the cyclic shear response of soil mixtures. 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. Constant-volume cyclic direct simple shear tests with a sinusoidal loading cycle period of 10 s (frequency of 0.1 Hz) were conducted on normally consolidated (to a vertical effective stress of 100 kPa), reconstituted sand-silt specimens with pre-selected compositions to assess the accumulation of shear strain, development of excess pore-water pressure and derive the cyclic shear resistance. Results and observations on the cyclic shear loading response obtained from the direct simple shear tests are presented. The fine fraction of 0.37 was identified as the transition point where coarse-grain governing soil matrix changes in to fine-grain govern soil matrix as the fine content in the mixture increases. Accordingly, coarse-grain based void ratio was defined for the range of fine-grain content lesser than 0.37 and fine-grain based void ratio was defined for the range of fine-grain content greater than 0.37; and those were identified to be as better indices than global void ratio to describe the observed cyclic shear resistance in the sand -silt mixtures.

Keywords: sand-silt mixtures, cyclic shear resistance, cyclic direct simple shear tests

1. Introduction

The assessment of liquefaction and/or cyclic softening potential of coarse-grained and fine-grained soil mixtures (i.e., sand-silt or sand-clay mixtures) poses significant challenges to the current earthquake geotechnical practice. The methods used in current practice for these assessments are derived mainly focusing on the mechanical response characterized in relation to sand-like and clay-like behavior, but not specifically considering coarse-grained and fine-grained soil mixtures. Additionally, the findings and conclusions derived from many of the laboratory studies on shear loading response of sand and silt mixtures often do not align. Some of the studies have concluded that the presence of fines in sand would increase the material shear loading resistance against liquefaction [1]–[9]. In contrast, some investigators [10]–[14] seem to conclude that the presence, and in particular the increase, of silt in sand would cause its liquefaction resistance to decrease.

Further studies [15]–[20] have shown that the undrained cyclic shear strength of the coarse-fine mixtures decreases with increasing fine content, until a certain threshold fines content (about 20–35% for non-plastic fines) is reached. Moreover, based on their experimental results, Polito and Martin II [18] have 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. Vaid [21] pointed out that depending on the basis of the comparison (i.e., relative density, void ratio or sand skeleton void ratio), different trends of increase (or decrease) in cyclic response would emerge with respect to change in silt content. Similarly, some other investigators [22] and [23] have also stated that the basis of the comparison is the reason for the apparent different trends of increase (or decrease) in soil resistance and strength with respect to change in silt content.

After extensive studies on the effect of fine content towards the liquefaction resistance of sand-non plastic fine mixtures, Thevanayagam [24], [25] produced a frame work for intergranular soil matrix classification, where equivalent inter-granular and equivalent inter-fine, inter-course granular void ratios are introduced as opposed to the global void ratio to be considered in characterizing the shear loading response course-fine mixtures. Deploying this conceptual framework (in which the soil mixture is assumed to be composed of spherical particles having two different diameter values, coarse grains and fine grains), many researchers [26]–[30] have consistently tried to evaluate the shear resistance/ liquefaction resistance of sand-silt mixtures. However, the frame work presented by Thevanayagam [24], [25] does not account for the soil fabric, plasticity, particle shape, gradation and aging effects.

It is quite possible that the conflicting results with respect to the effect of fines content in the performance of silt and mixtures may be arising from the manner in which the test specimens were prepared. For example, the above observed discrepancies on the response of sand-silt mixtures made by previous researchers can likely be due to: (a) problems in determining the packing relative density in an accurate and consistent way; (b) differences in specimen preparation methods in laboratory testing; (c) differences in loading modes used in testing; and (d) not systematically covering the complete spectrum of fines content ranges. The varying and contradicting findings with the notable gaps in the current understanding on the shear loading response of sand-silt mixtures require the need of undertaking more advance research to address the inconsistent findings and in particular, to delineate the effects of soil grain size, distribution of grain size, and packing arrangement in the soil matrix to characterize the shear loading response of sand-silt mixtures.

With this background, a systematic experimental research program was undertaken to study the cyclic shear response of soil mixtures covering the complete range of sand-silt compositions in the mixtures. A part of this study consists of constant-volume cyclic direct simple shear tests conducted on sand-silt mixtures with pre-selected compositions to obtain the cyclic shear resistance for tested materials. In particular, cyclic strain accumulation, pore water pressure development, the trends of cyclic shear resistance of sand-silt mixtures with respect to the sand-silt composition as well as void ratio defined based on different definitions are assessed and compared in this paper.

2. Laboratory Experimental Aspects

2.1 Tested Soil

The parent materials to produce sand-silt mixtures for the experimental study were obtained from the Fraser River Delta of British Columbia, Canada, and they were received in bulk form. The Fraser River sand was wet sieved to remove the portion finer than 75 μm and to obtain the parent coarse-grained material which is identified as 100C0F (i.e., 100% processed Fraser River sand). The Fraser River silt was oven dried and then sieved to remove the sand portion (coarser than 75 μm) to prepare the parent fine-grained material identified as 0C100F (i.e., 100% processed Fraser River silt). 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. The resulting particle size distributions of sand and silt alone and sand-silt mixtures are shown in Fig. 1. Each prepared mixture is identified using a naming-code as shown in the legend of the Fig. 1; for example, 50C50F stands for the specimen with 50% of 100C0F and 50% of 0C100F fraction expressed based on dry weight. Hereinafter in this paper, reference is made to a given mixture through the use of this naming code for convenience and brevity.

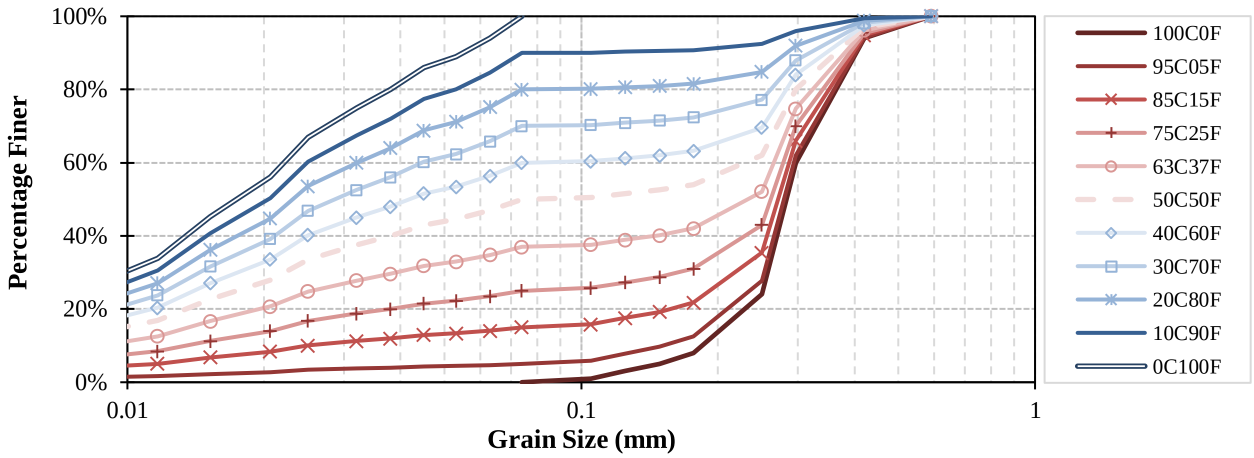


Fig. 1 Particle size distribution of the sand, sand-silt mixtures and silt.

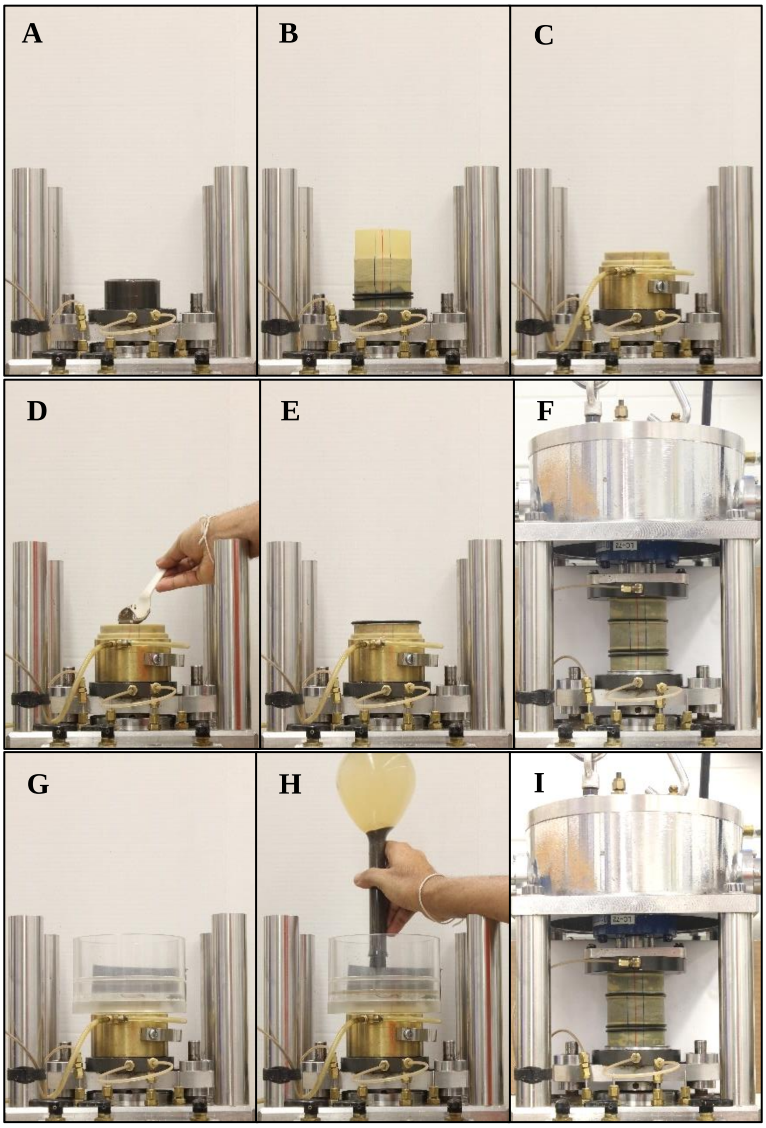
The 100C0F and 0C100F materials essentially comprise the natural gradation of Fraser River sand and silt respectively, after the removal of finer and coarser fraction as mentioned earlier. As such, it is considered to reasonable state that the resulting soil grain size distributions of the mixtures would mimic those of natural soil as much as possible.

2.2 Specimen Preparation

Water pluviation method was used to prepare 100C0F and 95C05F specimens; 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 for the other sand-silt mixtures (i.e., having fines content > 10%) and 100% silt. In order to minimize segregation with fully saturated specimens, appropriate water contents were selected based on visual observations and experience-based judgment when preparing the sand-silt mixtures for slurry preparation.

The following steps, as shown in Fig. 2, were undertaken in preparation of the Direct Simple Shear (DSS) test device to receive the reconstituted specimens: (A) 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; (B) 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; (C) 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).

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 [Fig. 2(D)]. Herein, careful attention was paid to avoid air being entrapped within the specimen during spooning and to achieve a uniform and even top surface.



Specimen Preparation

A – Boiled, then cooled porous plate placed at the base pedestal.

B – Wire-reinforced rubber membrane placed on and sealed to the bottom specimen-base-pedestal using an o-rings

C – Split-mold mounted around the base pedestal; wire-reinforced rubber membrane stretched on to split-mold by application of vacuum, thus forming a cylindrical cavity

Saturated Slurry Deposition Method

D – Saturated slurry deposition on to the cavity created by the split mold.

E – Top o-ring placed on the split mold and ready to seal the specimen with the top platen

F – Test specimen with approximate 20 mm in height and 70 mm in diameter secured at the DSS test device and ready for testing

Water Pluviation Method

G –Reservoir cylinder placed on the split mold and flood with water

H – Sand pluviated in the medium of water by hovering the inverted upright volumetric flask in the water reservoir

I – Test specimen with approximate 20 mm in height and 70 mm in diameter secured at the DSS test device and ready for testing

Fig. 2 Steps in specimen preparation for constant-volume cyclic direct simple shear tests

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 [Fig. 2(G)]. 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 [Fig. 2(H)]. 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.

After placing the sufficient amount of soil material to achieve an initial specimen height of about 20 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 [Fig. 2(F) through Fig. 2 (I)] to the desired initial vertical effective consolidation stress (σ'_{vc}) of 100 kPa.

2.3 Constant-volume Cyclic Direct Simple Shear Tests

The direct simple shear device at University of British Columbia, Canada, which was used for the testing herein, is a modified Marshall-Silver-NGI (Norwegian Geotechnical Institute) type device – initially developed by Silver and Seed [31] - which uses a cylindrical soil specimen and follows the DSS testing methodology as described by Bjerrum and Landva [32]. 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 which is stiff enough to constrain any lateral deformations. Therefore, the soil behavior is essentially in a state of zero lateral strain during consolidation and cyclic 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 top and bottom loading platens of the specimen are clamped against vertical movement, thus imposing a height constraint in addition to the lateral restraint from the steel-wire membrane. This is an alternative to the commonly used approach of maintaining constant-volume by suspending the drainage of a saturated specimen. 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 [33], [34]. 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.

The test specimens were initially consolidated to a common σ'_{vc} of 100 kPa. Upon the completion of consolidation phase, the specimens were subjected to cyclic shear loading in a stress-controlled manner using a computer-controlled pneumatic loading system. The level of cyclic shear stress (τ_{cyc}) was chosen so that a desired constant-amplitude cyclic stress ratio [CSR = τ_{cyc} /σ'_{vc}] would be applied on the specimen in a symmetrical sinusoidal manner at a frequency of 0.1 Hz. The stress-strain pattern-based criterion [35] was considered to determine the number of loading cycles for unacceptable cyclic shear performance, and for the assessment and comparison of cyclic resistance ratio (CRR) [i.e., the term CRR is used when CSR is expressed as a resistance]. The CRR derived from the stress–strain pattern change– based criterion for sands is similar to that derived from the shear strain–based criteria with a threshold shear strain limit of ±3.75%; and for silts it is consistently and slightly lower.

2.4 Test Program

The test parameters used for and results from the constant-volume cyclic DSS tests performed on the soil mixtures shown in Fig. 1. are summarized in Table 1. In addition to the coarse-grained alone (100C0F) and fine-grained alone material (0C100F), 9 types of sand-silt mixtures with different fine-grained fraction (C_F, expressed as a decimal value) such as 0.05, 0.15, 0.25, 0.37, 0.5. 0.6, 0.7, 0.8, and 0.9 were considered in this study. For the evaluation and assessment of shear loading response of sand-silt mixtures, test results obtained for 11 type specimens with different Cyclic Stress Ratios (CSRs) were used.

3. Cyclic Shear Loading Response

The cyclic resistance ratio values of the sand-silt mixtures assessed from the results obtained from the constant-volume cyclic DSS tests are presented in Table 1. In this regard, the observed typical response in terms of development of pore water pressure, stress-strain and stress-path response during the cyclic loading (considering the tests conducted using CSR = 0.09) are discussed in following sections.

3.1 Pore-water Pressure Development

The excess pore water pressure ratio (r_u) is defined as the ratio of excess pore water pressure (Δu) to σ'_{vc} prior to cyclic loading. All test specimens seems to show gradual development of excess pore-water pressure during the application of cyclic shear loading with a CSR of 0.09 as seen in Fig. 3. The rate of development of r_u seem to be very similar for all the test specimens up until r_u reaches about 0.5, where normalized number of loading cycles [the number of loading cycles (N_{cyc}) at a given instance is divided by the number of loading cycles that would take to observe the pattern change of stress-strain loops as an index for unacceptable cyclic shear performance ($N_{cyc[*]}$)] is about 0.6 despite their differences in C_F. Thereafter, the coarse-grained dominant specimens indicate relatively higher rate of r_u development than those observed for fine-grained dominant specimens.

3.2 Stress-strain and Stress-path Response

For comparison purposes, normalized stress-strain and stress-path response derived from the cyclic DSS tests performed on the 11 material types tested with a CSR of 0.09 are presented in Fig. 4. It can be seen that mixtures with different C_F result a wide range for $N_{cyc[*]}$ despite all mixture specimens are cyclically sheared with a CSR of 0.09. Similar observation can be made from Fig. 4, with respect to the trends in development of shear strain and reduction of effective vertical stresses.

3.3 Cyclic Resistance Ratio (CRR)

The test program shown in Table 1 lists all the tests performed with different CSR magnitudes (Note: in Fig. 4, only the tests performed with a CSR of 0.09 was used to present the comparison response with respect to different C_F values). The cyclic shear resistance presented in terms of CRR versus $N_{cyc[*]}$ plots drawn for sand-silt mixture specimens with different C_F values are shown in Fig. 5. It is observable that the 100C0F possess the lowest cyclic shear resistance; in general, the CRR curve for a given material gradually increases as the C_F increases and 0C100F seems to indicate the highest cyclic shear resistance. However, it should be noted that the sand-silt mixture specimens possess different global void ratios (see Fig. 6 and Table 1) as well as the mixtures prepared from both water pluviation and saturated slurry deposition methods can result in different particle structures and the soil fabrics with increasing C_F at a given effective confining stress. Therefore, assessment of the relative trends of CRR of sand-silt mixtures should include good accounting of these factors and void ratio values.

With this background, it was considered useful to examine the results with respect to two void ratios: coarse-grains-based void ratio (e_c) and fine-grains-based void ratio (e_f) as defined in the legend of Fig. 6. When fine-grains are introduced to the coarse-grain soil matrix, fine-grains seem to fill the voids in the coarse-grain based microstructure, resulting apparent increase in mass density (or reduction in global void ratio) as shown

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

Test ID	e	C_F	e_c	e_f	σ'_{vc} (kPa)	CSR	$N_{cyc[*]}$	$r_{u[max]}$	$\gamma_{[max]}$
100C0F-13	0.75	0	0.75	-	96	0.134	3	1	10.0
100C0F-11	0.72	0	0.72	-	97	0.112	4	0.97	4.2
100C0F-10	0.70	0	0.70	-	100	0.100	7	0.98	6.1
100C0F-09	0.73	0	0.73	-	93	0.092	22	0.99	6.7
95C05F-13	0.72	0.05	0.81	33.36	98	0.133	8	0.98	6.7
95C05F-11	0.78	0.05	0.87	34.59	97	0.116	10	0.98	5.1
95C05F-10	0.75	0.05	0.84	33.93	89	0.114	10	0.97	5.5
95C05F-09	0.72	0.05	0.81	33.39	95	0.095	24	1	7.3
85C15F-14	0.59	0.15	0.87	9.61	93	0.145	1	0.99	18.2
85C15F-13	0.53	0.15	0.80	9.19	101	0.125	5	0.98	7.0
85C15F-10	0.50	0.15	0.77	9.01	99	0.102	17	0.95	6.0
85C15F-09	0.57	0.15	0.84	9.45	95	0.096	12	0.96	7.5
75C25F-12	0.44	0.25	0.92	4.76	95	0.125	6	0.97	7.3
75C25F-13	0.43	0.25	0.91	4.73	96	0.135	5	0.97	6.8
75C25F-10	0.46	0.25	0.95	4.84	99	0.101	13	0.98	7.6
75C25F-09	0.46	0.25	0.95	4.85	95	0.096	17	0.98	9.7
63C37F-14	0.41	0.37	1.24	2.81	98	0.138	4	0.96	12.0
63C37F-12	0.40	0.37	1.23	2.79	96	0.121	7	0.91	6.9
63C37F-10	0.41	0.37	1.24	2.81	99	0.105	11	0.95	7.1
63C37F-09	0.41	0.37	1.23	2.81	97	0.089	16	0.91	5.7
50C50F-14	0.44	0.5	1.89	1.89	98	0.136	6	0.93	7.3
50C50F-12	0.46	0.5	1.92	1.92	96	0.119	10	0.95	8.4
50C50F-10	0.45	0.5	1.91	1.91	102	0.103	11	0.97	8.9
50C50F-09	0.46	0.5	1.92	1.92	98	0.089	21	0.97	8.2
40C60F-14	0.62	0.6	3.04	1.69	99	0.137	8	0.89	5.8
40C60F-12	0.59	0.6	2.97	1.65	98	0.118	15	0.91	6.1
40C60F-10	0.57	0.6	2.93	1.62	100	0.104	24	0.92	5.1
40C60F-09	0.62	0.6	3.05	1.70	100	0.095	58	0.92	5.1
30C70F-14	0.54	0.7	4.14	1.20	99	0.142	5	0.94	8.2
30C70F-13	0.56	0.7	4.19	1.22	98	0.128	8	0.9	6.2
30C70F-10	0.54	0.7	4.14	1.20	99	0.105	22	0.94	8.9
30C70F-09	0.55	0.7	4.18	1.22	100	0.096	30	0.89	6.5
20C80F-15	0.60	0.8	7.01	1.00	101	0.147	5	1	12.4
20C80F-13	0.63	0.8	7.17	1.04	99	0.135	7	0.88	13.4
20C80F-11	0.61	0.8	7.03	1.01	98	0.109	23	0.97	15.1
20C80F-09	0.61	0.8	7.07	1.02	101	0.095	35	0.93	14.6
10C90F-15	0.64	0.9	15.38	0.82	98	0.153	3	0.92	13.7
10C90F-13	0.65	0.9	15.47	0.83	100	0.131	7	1	14.3
10C90F-11	0.65	0.9	15.53	0.84	98	0.115	16	0.94	15.5
10C90F-09	0.64	0.9	15.41	0.82	100	0.094	39	0.96	13.3
0C100F-16	0.64	1	-	0.64	101	0.160	3	1	20.3
0C100F-14	0.74	1	-	0.74	99	0.144	5	0.91	11.0
0C100F-10	0.65	1	-	0.65	100	0.106	27	0.98	18.0
0C100F-09	0.73	1	-	0.73	99	0.094	39	0.96	14.2

e – Global Void Ratio; e_c – Coarse-grains-based void ratio; e_f – Fine-grains-based void ratio; C_F – Fine-grained fraction; σ'_{vc} – Vertical effective consolidation stress; CSR – Cyclic Stress Ratio; $N_{cyc[*]}$ – Number of loading cycles for stress-strain pattern change; $r_{u[max]}$ – Maximum pore-water pressure ratio; $\gamma_{[max]}$ – Maximum shear strain

in Fig. 6.This trend seems to continue till fines fraction (C_F) is about 0.35~0.4 for the sand-silt mixtures considered in this study. As noted by Kuerbis [36], it is arguable that coarse-grains seem to govern the shear loading response of the mixture till that composition; on this basis, e_c is postulated to be a better index to explain the response of mixtures for a C_F range from 0 to 0.37. Further introduction of fine-grains (beyond C_F of 0.37) into the mixture leads to a decrease in mass density (increase in global void ratio); this may be due to the reduction of coarse-grain contacts in the soil matrix - in essence, coarse grains would "float" in in a fine-grain dominated matrix. Since fine-grains would likely govern the shear loading response of the mixture beyond C_F of 0.37, e_f was considered to better index to examine the shear loading response of mixtures having higher C_F of 0.37.

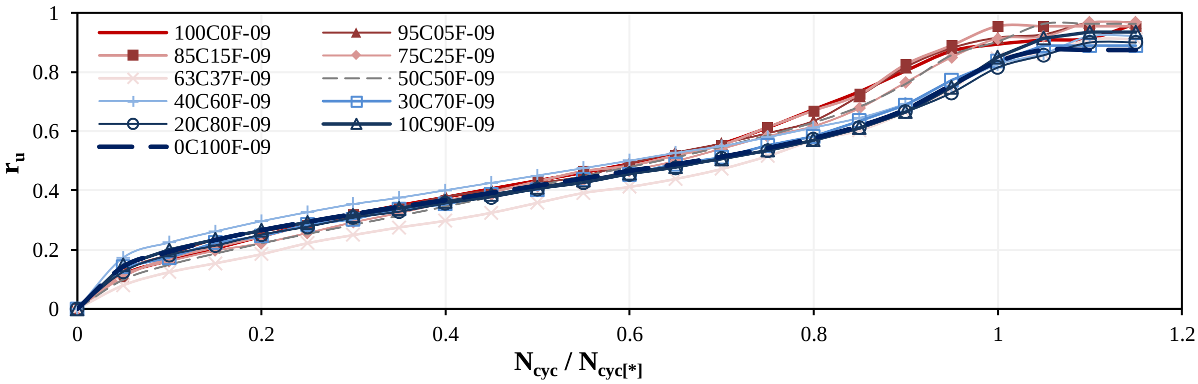


Fig. 3 Excess pore-water pressure development response of reconstituted sand-silt mixtures obtained from constant-volume cyclic DSS tests with a CSR of 0.09 at σ'_{vc} = 100 kPa

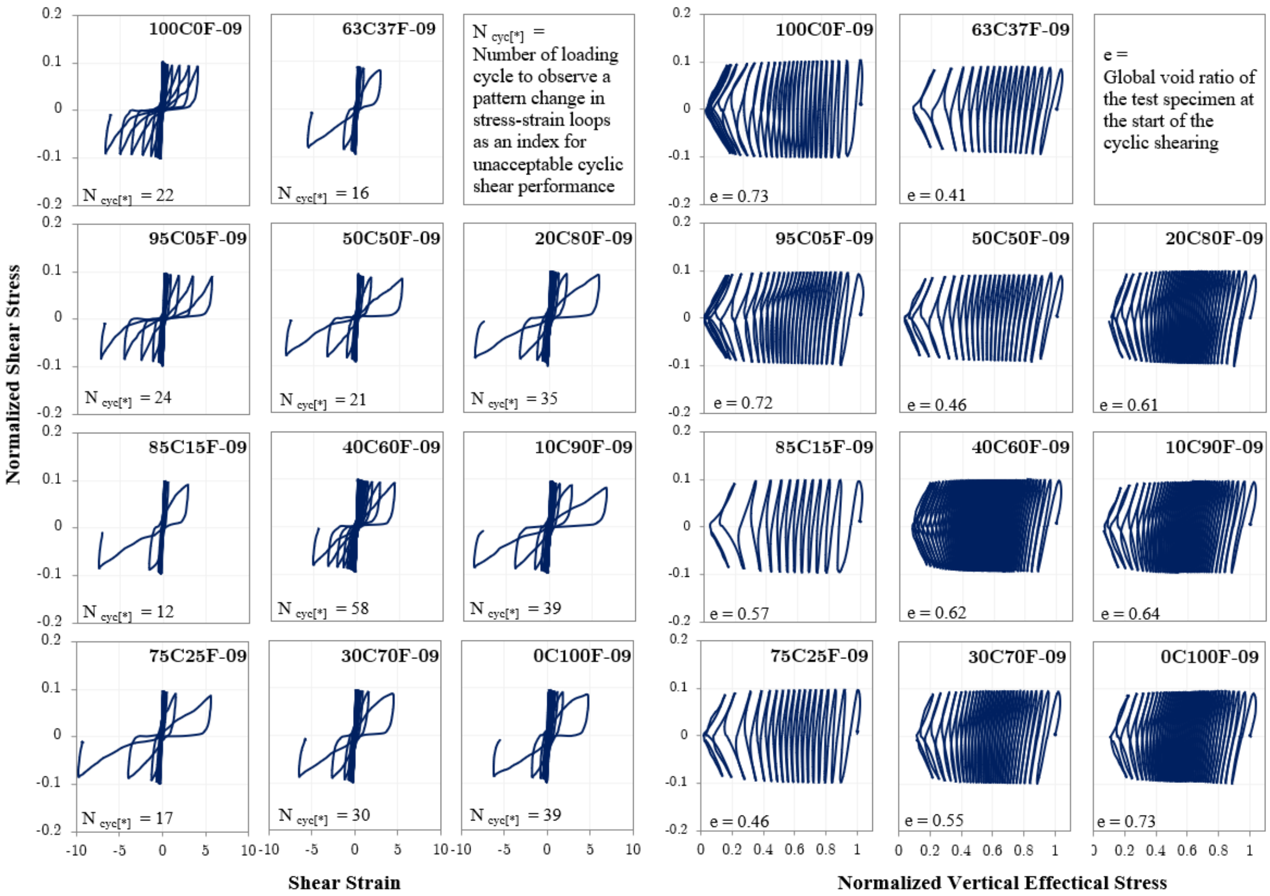


Fig. 4 Normalized shear stress-strain and normalized stress path response of reconstituted sand-silt mixtures obtained from constant-volume cyclic DSS tests with a CSR of 0.09 at σ'_{vc} = 100 kPa

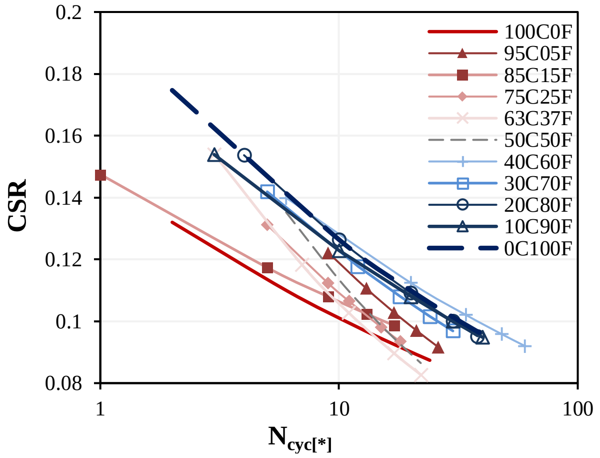


Fig. 5 Cyclic stress ratio vs. number of loading cycles for stress-strain pattern-based criterion

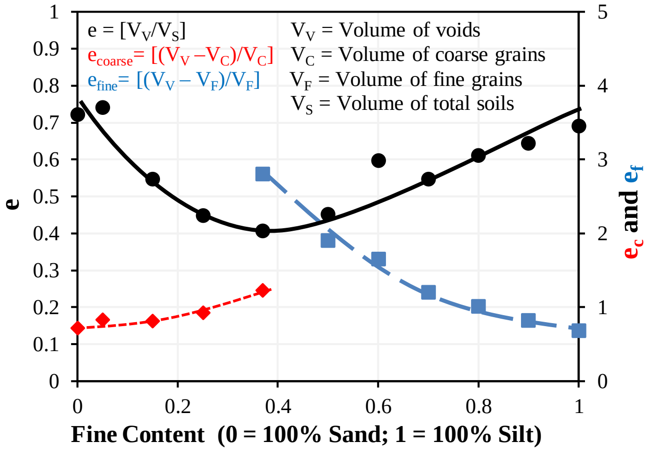


Fig. 6 Variation of global, coarse-grained base and fine-grained void ratios with respect to fine-content of the sand-silt mixtures at σ'_{vc} = 100 kPa

It was observed that some insight on the response of the tested soils could be obtained by examining the experimentally obtained CRR values corresponding to a reference number of loading cycles. In this regard, equivalent N_{cyc} corresponding to a 7.5 magnitude earthquake (defined herein as $N_{cyc\text{-}ref}$) of 15 for sandy soils as indicated in Arango [37], Boulanger and Idriss [38], and $N_{cyc\text{-}ref}$ of 23 for silts as suggested by Verma [39] were considered suitable for the present assessment. Based on this, the CRR_{ref} corresponding to $N_{cyc\text{-}ref}$ = 15 for sand-dominant specimens (i.e., for test specimens having C_F < 0.37), and that corresponding to $N_{cyc\text{-}ref}$ = 23 for silt-dominant specimens (i.e., for test specimens having C_F > 0.37), were extracted from Fig. 5. A plot of these CRR_{ref} values versus e_c is presented in Figure 7; similarly, the CRR_{ref} values plotted versus (e_f) are also included in Figure 7.

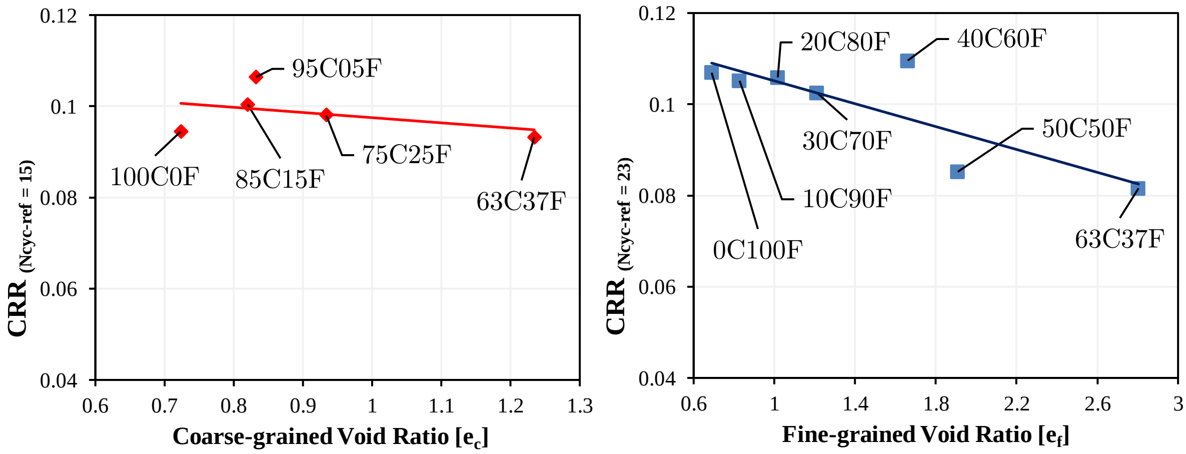


Fig. 7 Tends of cyclic resistance ratio (CRR) derived for number of loading cycles 15 for sand dominant mixtures and for number of loading cycles 23 for silt dominant mixtures against coarse-grained based void ratio (e_c) and fine-grained based void ratio (e_f) respectively.

As may be noted from Fig. 7, the value of CRR_{ref} seem to decrease with increasing in e_c for sand-dominant specimens, and in a consistent manner, the value of CRR_{ref} of silt-dominant specimens would decrease with increasing e_f. It is of interest to note that, if these CRR_{ref} values were plotted with respect to the global void ratio (e), the results would have indicated a decrease in CRR_{ref} with increasing e, then an increase in CRR_{ref} with further increase in e (this graph is not provided due to space limitations). It appears that the use of e_c values to assess the performance of coarse-grains-dominant soil matrices, and the e_f values to examine the fine-grains-dominant matrices, provides a more rational approach - than the use of global void ratio to assess the cyclic response of silt-sand mixtures.

4. Summary and Conclusions

The cyclic shear loading response of Fraser River sand-silt mixtures were assessed using constant volume DSS testing of reconstituted soil specimens. Eleven soil specimens covering wide a range of sand-silt mix ratios [from sand-alone (C_F = 0), to silt-alone (C_F = 1.0) specimens] were tested. Key observations and conclusions from this experimental study are summarized below:

1. From the sand-silt mixtures that were considered in this study, with specimens consolidated to vertical effective stress (σ'_{vc}) = 100 kPa, the specimen with 63% of sand and 37% of silt (C_F = 0.37) produced the lowest void ratio (densest arrangement).
2. A fine fraction of ~0.37 was identified as the transition point where the coarse-grains governing soil matrix changes to fine-grains governing soil matrix, as the fines content in the mixture increases. Accordingly, a coarse-grains based void ratio (e_c) was defined for the mixtures with C_F < 0.37, and fine-grains based void ratio (e_f) was defined for the mixtures with C_F > 0.37.
3. The test data were assessed considering a cyclic stress ratio corresponding to a reference number of loading cycles ($N_{cyc\text{-}ref}$) corresponding to a 7.5 magnitude earthquake, and with respect to e_c, e_f, and conventional global void ratio (e) values.
4. The value of CRR_{ref} was found to decrease with increasing in e_c for sand-dominant specimens, and in a consistent manner, the value of CRR_{ref} was noted to decrease with increasing e_f with respect to silt-dominant specimens. If these CRR_{ref} values were assessed with respect to the global void ratio (e), the results would have indicated a decrease in CRR_{ref} with increasing e, then an increase in CRR_{ref} with further increase in e. The use of e_c to assess the performance of coarse-grains-dominant soil matrices, and the e_f values for the fine-grains-dominant matrices, provides a more rational approach, than the e value, to assess the cyclic response of silt-sand mixtures.

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