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Response of natural fine-grained soils for seismic design practice: A collection of research findings from British Columbia, Canada

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ABSTRACT

This paper summarizes the results from a comprehensive laboratory experimental research program conducted at the University of British Columbia, Canada to study the mechanical behavior of natural fine-grained soils. Constant-volume direct simple shear tests were performed on natural silt samples collected from various locations in British Columbia to investigate the monotonic and cyclic shear loading response of these materials. Influencing factors such as effective confining stress, over-consolidation ratio, initial static shear bias, soil plasticity, and soil fabric/micro-structure on the mechanical behavior of silts were systematically investigated using a series of controlled laboratory tests. The key observations arising from these examinations are presented and discussed to serve as input for the development and/or refinement of design practice approaches for seismic geotechnical design. New directions to be considered to advance the current state of understanding and knowledge on the mechanical behavior of natural fine-grained soils are identified.

1. Introduction

Liquefaction of soils and associated ground movements could cause damage to buildings and infrastructure during earthquakes. For example, it has been reported that the damage costs from a major earthquake in the Metro Vancouver Region of British Columbia (BC), Canada could be as high as $75 billion [1], with significant damage expected to arise from soil liquefaction. Southern Ontario and the St. Lawrence Valley are the other regions in Canada where earthquake-induced soil liquefaction is considered an engineering concern.

The assessment of liquefaction susceptibility of natural fine-grained soils, particularly those with low plasticity, poses significant challenges to the current design practice. This is mainly due to the lack of understanding of the complex stress-strain-strength behavior of fine-grained soils such as low-plastic silt; this knowledge gap, in turn, has led to the use of liquefaction assessment criteria that are primarily based on soil index properties such as plastic limit (PL), liquid limit (LL), plasticity index (PI), and water content. The consideration of silt with PI $<$ 7 as "sand-like" and that with PI $\geq$ 7 as "clay-like" in behavior for the purpose of liquefaction assessment by Idriss and Boulanger [2] is one such example in this regard; the approach proposed by Bray and Sancio [3], again based on index parameters, serves as another example.

Fine-grained silty soils with high levels of saturation are commonly found in natural river deposits, and due to many reasons, most population centers, along with critical infrastructure, industrial plants, etc., are located near river banks on such soil deposits. Experience from recent earthquakes (e.g., Turkey, Kocaeli 1999; New Zealand, Christchurch 2010–2011) suggests that saturated fine-grained soils have the potential for earthquake-induced softening and strength reduction, leading to permanent vertical and lateral ground movements, post-cyclic settlements, foundation sliding, tilting and collapsing of structures [4–6].

In consideration of the above, it is clear that there is a need to undertake additional research on the behavior of fine-grained soils. Geotechnical field and laboratory element-level investigations have a key role to play in advancing the fundamental knowledge in this regard. In recognition of the need, a number of scholars have conducted extensive studies [4,7–12] while appreciating that silt has a complex transitional behavior straddling between the relatively well understood cyclic shear response of clean sand and clay.

In concert with this effort, the first author has undertaken a systematic laboratory program to characterize the earthquake response of natural fine-grained soils collected from a number of soil deposits in BC, Canada. As a major part of this work, the constant-volume element shear response of these soils was investigated using direct simple shear (DSS) test device at the University of British Columbia (UBC), Vancouver, Canada.

In this paper, the factual findings from the above laboratory element testing research, completed over the past 15 years, addressing a range

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List of Symbols

C_c	Compression index
C_r	Recompression index
e	Void ratio
e_c	Void ratio of the specimen after the end of consolidation phase
e_0	Initial void ratio of the specimen
N_{cyc}	Number of loading cycles
$N_{cyc[\gamma=3.75\%]}$	Number of loading cycles to reach the threshold shear strain of $\gamma = 3.75\%$
r_u	Excess pore-water pressure ratio ($= \Delta u/\sigma'_{vc}$)
$r_{u\text{-}max}$	Maximum cyclic pore-water pressure ratio
α	Initial static shear stress bias ($= \tau_{st}/\sigma'_{vc}$)
γ	Shear strain
Δe	Change in void ratio of a laboratory specimen during re-consolidation to in situ vertical effective stress
Δu	Excess pore-water pressure
$\varepsilon_{v\text{-}pc}$	Post-cyclic volumetric strains
σ'_p	Preconsolidation stress
σ'_t	Target consolidation stress
σ'_v	Vertical effective stress
σ'_{vc}	Initial vertical effective stress
τ_{cyc}	Symmetrical sinusoidal horizontal shear stress
τ_{st}	Static shear stress

List of Abbreviations

BC	British Columbia
CSR	Cyclic Stress Ratio
CRR	Cyclic Resistance Ratio
DSS	Direct Simple Shear
LL	Liquid Limit
NC	Normally consolidated
NGI	Norwegian Geotechnical Institute
OC	Over-consolidated
OCR	Over-consolidation Ratio
PI	Plasticity Index
PL	Plastic Limit
Recons	Reconstituted
SA	Single amplitude
UBC	University of British Columbia
Undist.	Undisturbed
1-D	One-dimensional

Table 1
Details of the test program.

Site ID	Geographic location	Material description	Research investigation	
			Monotonic shear response	Cyclic shear response
A	North bank of the South arm of the Fraser River	■ Depth range below ground surface: 5.6–8.7 m ■ Water content: 34–39% ■ LL: ~30%, PL: ~26%, PI: ~4% ■ Sand: ~10%, Silt: ~80%, Clay: ~10% ■ Unified soil classification: ML ■ Specific gravity: 2.69 ■ Estimated preconsolidation stress: ~85–95 kPa	■ Effect of confining stress ■ Effect of static shear bias ■ Effect of plasticity ■ Effect of microstructure/fabric	■ Effect of confining stress ■ Effect of mechanical OCR ■ Effect of static shear bias ■ Effect of plasticity ■ Effect of microstructure/fabric
B	South bank of the North arm of Fraser River	■ Depth range below ground surface: 5.0–7.0 m ■ Water content: 35–44% ■ LL: ~34%, PL: ~29%, PI: ~5% ■ Sand: ~20–35%, Silt: ~70–45%, Clay: ~10–20% ■ Unified soil classification: ML ■ Specific gravity: 2.70 ■ Estimated preconsolidation stress: ~100–125 kPa	■ Effect of mechanical OCR ■ Effect of plasticity	■ Effect of plasticity ■ Effect of microstructure/fabric
C	South bank of the Nicomekl River	■ Depth range below ground surface: 4.2–5.5 m ■ Water content: 38–53% ■ LL: ~41%, PL: ~34%, PI: ~7% ■ Sand: ~20%, Silt: ~65%, Clay: ~15% ■ Unified soil classification: ML ■ Specific gravity: 2.77 ■ Estimated preconsolidation stress: ~35–45 kPa	■ Effect of confining stress ■ Effect of mechanical OCR ■ Effect of plasticity ■ Effect of microstructure/fabric	■ Effect of confining stress ■ Effect of static shear bias ■ Effect of plasticity ■ Effect of microstructure/fabric
D	South bank of the Fraser River prior to the confluence with Pitt River	■ Depth range below ground surface: 4.9–6.2 m ■ Water content: 58–69% ■ LL: ~76%, PL: ~42%, PI: ~34% ■ Sand: ~0%, Silt: ~10%, Clay: ~90% ■ Unified soil classification: MH ■ Specific gravity: 2.75 ■ Estimated preconsolidation stress: ~75–85 kPa	■ Effect of confining stress ■ Effect of plasticity ■ Effect of microstructure/fabric	■ Effect of confining stress ■ Effect of plasticity ■ Effect of microstructure/fabric

LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; OCR: Over-consolidation ratio.

of parameters governing material behavior are presented. In particular, the observed effects of confining stress, over-consolidation ratio, initial static shear bias, soil plasticity, and soil fabric/microstructure on the monotonic/cyclic shear loading response of silts are presented; where possible, rationale have been given to explain the observed behavioral patterns. Although the outcomes herein have arisen only from tests conducted on a selected number of soils, the authors believe that the information has the robustness and sufficient breadth to enhance our understanding of the material behavior while augmenting the laboratory experimental database available for numerical modeling.

The paper initially presents the details related to the soil materials, experimentation methodology, and testing program. This is followed by seven sub-sections presenting results from different aspects of the testing program along with corresponding observations. The emphasis is on the cyclic shear response of fine-grained soils; however, the results from monotonic shear tests are drawn in to illustrate certain behavioral aspects. From a completeness point of view, the findings that have been reported in previous publications by the author(s) are also encompassed in the compilation.

It is important to note that this paper is a revised version of the invited keynote lecture presented by the first author at the 3rd International Conference on Performance-based Design in Earthquake Geotechnical Engineering which was held in Vancouver, British Columbia, Canada on July 16–19, 2017.

2. Geotechnical laboratory element testing program

An experimental research program was undertaken at the UBC geotechnical research laboratory to study the behavior of silts with a major part of the testing conducted using the in-house constant-volume direct simple shear (DSS) device. Most of the tests were performed on specimens prepared from soil retrieved, using specially fabricated thin-walled tube samples, from different locations in the Lower Mainland of BC, Canada. The details about the experimental program with the material description, index properties, and geographic location of various subject sites, and the types of testing conducted are summarized in Table 1 to serve as a key to the investigations undertaken.

The DSS tests were conducted to study the monotonic and cyclic shear as well as post-cyclic consolidation response of natural silts. The cyclic DSS tests are noted to be reasonably representative of the field loading conditions that include continuous rotation of principal stresses that prevail during an earthquake. The response of a given soil during an earthquake is controlled by many parameters such as fabric/microstructure, packing density, confining stresses, initial static shear bias, level and duration of cyclic loading, etc. These parameters have been noted to primarily govern the development of excess pore water pressure, stiffness, and strength in a soil mass during the occurrence of an earthquake and, in turn, to control the overall seismic behavior. The laboratory testing program at UBC was developed with these considerations in mind.

2.1. Direct simple shear test device

The UBC-DSS device is a modified Norwegian Geotechnical Institute (NGI) type device [13]. In this device, a cylindrical soil specimen of ~ 70 mm diameter and ~ 20 mm height is placed in a steel wire-reinforced rubber membrane. The steel wire-reinforced rubber membrane confines the test specimen and prevents it from localized lateral deformations during consolidation and shear loading; as such, the soil specimen would be in a state of zero lateral strain during consolidation and shear loading. If required, a constant-volume condition can be enforced by clamping the top and bottom loading platen of the specimen against vertical movement i.e., imposing a height constraint in addition to the lateral restraint from the steel wire-reinforced membrane. This device has two options with respect to specimen height control; (i) to allow the specimen to deform freely in the vertical direction (e.g., during consolidation); or (ii) to maintain constant specimen height by clamping the vertical loading shaft with the constant height requirement as per ASTM D6528-17 [14] confirmed by direct Linear Variable Displacement Transformer measurement (e.g., during constant-volume loading).

Past researchers [15,16] have shown that the decrease (or increase) in the 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 degree of saturation of the DSS specimens was assessed considering the volume, and moisture content of the tested specimens combined with their respective dry weights. The computed degree of saturation based on this approach indicated that all the tested specimens were in a fully, or very close to, saturated state. There is a potential for non-uniform effective stresses to develop in unsaturated soil specimens due to capillary effects; however, this factor is not a concern since the tested specimens are saturated.

2.2. Soil sampling and assessment of sample quality

Fixed-piston tube sampling conducted in a conventional mud-rotary drill hole was used to obtain undisturbed samples from the identified silt deposits at various locations in the Lower Mainland of BC. Specially fabricated stainless-steel tubes having an outer diameter of 76.2 mm, with no inside clearance, sharpened (5° beveled) cutting edge, and 1.5 mm wall thickness were used for this purpose. The sample quality classification chart developed by Lunne et al. [17] was used to assess the degree of disturbance experienced by silt specimen during sampling and specimen preparation process. In this regard, several one-dimensional (1-D) consolidation tests were performed on the natural silt specimens obtained from various sites. Data from these tests were also used to assess the compressibility characteristics of the soil. According to Lunne et al. [17] criterion, samples could be considered "good-to-fair" in terms of sample disturbance if $\Delta e/e_0 \leq 0.07$, and they are to be assessed as "poor" if $\Delta e/e_0$ is greater than 0.07 (where Δe is the change in the void ratio of a laboratory specimen during reconsolidation to in situ vertical effective stress and e_0 is the initial void ratio of the specimen). The results of the current tests indicated that most silt samples obtained during the sampling process were in a range from "good-to-fair" level of sample disturbance, and therefore, were considered suitable for the intended laboratory research work presented herein. However, it should be noted that the criterion proposed by Lunne et al. [17] was developed for soft low plastic Norwegian clay and caution should be exercised when using this criterion for other low plastic natural soils. Recent work by Krage et al. [18] has also emphasized that the use of the current $\Delta e/e_0$ criterion might be potentially misleading resulting in an apparent increase in sample quality for low-plastic silts. As an alternative, they are considering the use of a normalized $\Delta e/e_0$ by either the compression index (C_c) or the recompression index (C_r) to express sample quality; it is the authors' understanding that the development of a new criterion on this basis is still in progress.

With above assessment in mind, in the following text, the word "undisturbed" will be used to describe when referring to the soil directly obtained from the above described thin-walled stainless-steel tube samples.

2.3. Specimen preparation and DSS test procedures

Upon extrusion, the undisturbed soil specimens were trimmed and carefully placed within the steel wire reinforced rubber membrane with the aid of a polished stainless steel sharpened edge cutting ring. As notable from Table 1, some DSS tests on fine-grained soils presented in this study were conducted on reconstituted specimens. These reconstituted specimens were prepared from a saturated slurry. The approach is essentially identical to that developed by Sanin [19]. The reconstitution technique is well documented in [20] and not repeated herein for brevity.

A typical DSS test involves: (i) consolidating the silt specimens to a selected initial vertical effective stress (σ'_{vc}); (ii) followed by the application of a pre-determined initial static shear stress if it is required to simulate sloping ground situation; and (iii) then subjecting the specimens to cyclic/monotonic shear loading.

For monotonic/cyclic tests, the σ'_{vc} levels were kept slightly above the preconsolidation stress (σ'_p) level inferred from 1-D consolidation testing of undisturbed samples from the field (i.e., to obtain normally consolidated, NC, specimens). After initial consolidation under the applied vertical stress, some of the specimens were subjected to a static shear stress (τ_{st}) to meet a prescribed initial static shear stress bias ($\alpha = \tau_{st} / \sigma'_{vc}$). The τ_{st} was applied in an incremental manner while keeping the specimen in a drained condition; the specimen was allowed to reach equilibrium (i.e., stability in vertical and shear strains) under a given static shear stress increment, prior to application of the subsequent stress increment. Additional monotonic/cyclic DSS tests were also undertaken on specimens that were initially consolidated to a target consolidation stress (σ'_t) and then unloaded to a required σ'_{vc} levels so that a desired mechanical over-consolidation ratio (OCR) was achieved prior to the application of monotonic/cyclic loading.

Constant-volume monotonic DSS tests were conducted using strain-controlled loading with a shear strain rate of approximately 10% shear strain per hour. During cyclic DSS tests, which were conducted using stress controlled loading, symmetrical sinusoidal horizontal shear stress (τ_{cyc}) pulses were applied on the test specimens to achieve the selected levels of constant applied cyclic stress ratio [CSR = (τ_{cyc} / σ'_{vc})] amplitudes, at a frequency of 0.1 Hz. At the end of the cyclic loading phase, in some of the selected tests, the specimens were re-consolidated to the σ'_{vc} levels to assess the post-cyclic settlements in the soil specimen arising from the dissipation of shear-induced pore-water pressure.

3. Test results and discussion

3.1. Typical Response of Sand and Silt to Cyclic Loading

Based on the extensive amount of studies undertaken, typical cyclic shear loading response of coarse-grained soils (i.e., sands) have been well-observed and robustly established. One of the key observations have been that the contractive sands (i.e., relatively loose sands) under cyclic shear loading would exhibit continuous strain accumulation and pore-water pressure development with increasing number of loading cycles; such material would eventually experience abrupt shear stiffness degradation associated with the sudden failure or 'liquefaction' [21]. On the other hand, dilative coarse-grained soils (i.e., relatively dense sands) have been shown to demonstrate a gradual reduction in shear stiffness with increasing number of loading cycles. In a given cycle, the shear stiffness experiences its transient minimum when the applied shear stress is close to zero eventually experiencing zero, or close to zero, transient vertical effective stress conditions at significant strain magnitudes, displaying a 'cyclic-mobility-type' strain development mechanism.

With this backdrop in mind, typical stress-strain, effective stress paths, strain accumulation and pore-water pressure development derived from constant-volume cyclic DSS testing of air-pluviated loose Fraser River sand specimen when loaded with a CSR of 0.1, as reported by Wijewickreme et al. [22], are shown in Fig. 1(I-a and II-a) and Fig. 2. For the first five loading cycles of Fraser River sand, the shear stress versus shear strain response shown in Fig. 1(II-a) shows that the accumulated shear strains are very small (less than 0.1%), and on the 6th cycle, they abruptly develop to very large (10%) strain levels associated with significant reduction of vertical effective stress as shown in Fig. 1(I-a).

For a qualitative comparison, the characteristics obtained from two low-plastic Fraser River silt specimens (PI = 4), obtained from Site A and tested with CSR of 0.1 and 0.17 by Sanin [19] are presented in Fig. 1(I-b, II-b) and 1(I-c, II-c), respectively. With a cyclic loading

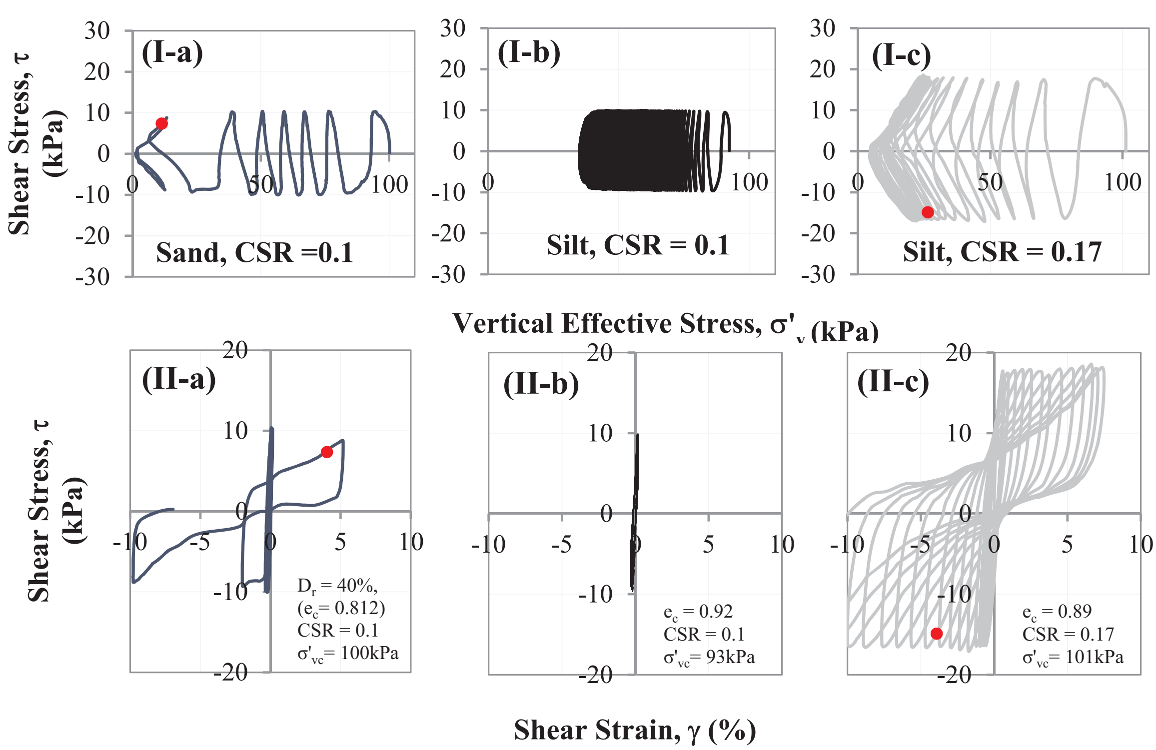

Fig. 1. Typical response without static shear bias - Loose Fraser River sand: (I-a) effective stress path; (II-a) shear stress-strain response (data from [21]); Low-plastic undisturbed Fraser River silt: (I-b and I-c) effective stress path; (II-b and II-c) shear stress-strain (data from [19]). *Note: Red dot indicates the onset of* γ = *3.75%.*

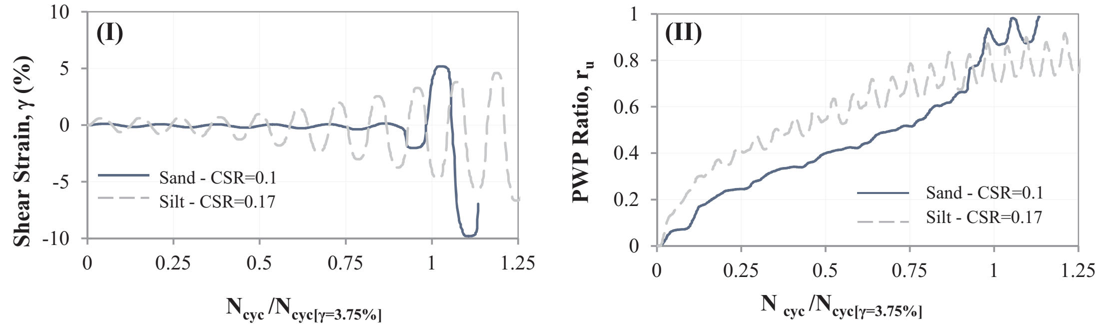

Fig. 2. (I) Typical strain accumulation and (II) pore-water pressure (PWP) development without static shear bias for loose Fraser River sand (data from [22] and low-plastic undisturbed Fraser River silt (data from [19]).

corresponding to CSR of 0.1, the effective stress path [Fig. 1(I-b)] and stress-strain response [Fig. 1(II-b)] shows that the silt specimen develops a non-zero vertical effective stress and insignificant strain levels after 300 loading cycles. When a higher magnitude of cyclic loading (CSR = 0.17) is applied, the respective stress path and stress-strain plots [Fig. 1(I-c) and (II-c)] indicate a gradual shear stiffness reduction and cyclic-mobility type strain accumulation mechanism. After the initial cycles, during the load applications where the magnitude of shear stress is increasing, the response is associated with an increase in vertical effective stress resulting from a dilative tendency.

In the present work, a single-amplitude shear strain (γ) level of 3.75% is considered as the threshold limit for the shear strain that is used to define unacceptable performance. This strain criterion was considered reasonable as single-amplitude γ = 3.75% in DSS specimen is equivalent to 2.5% single-amplitude axial strain in a triaxial specimen, which is also a definition for "liquefaction" [23]. Using this criterion, it can be observed that the silt specimen survived 17 loading cycles of CSR = 0.17 before the shear strain reached the threshold of 3.75%.

The shear strain accumulation and excess pore-water pressure (Δu) development with respect to the number of loading cycles for the above tests are presented in Fig. 2(I) and (II), respectively. As may be noted, in the x-axis, the number of loading cycles were represented by ($N_{cyc}/N_{cyc[\gamma=3.75\%]}$, where N_{cyc} is the actual number of loading cycles applied and $N_{cyc[\gamma=3.75\%]}$ is the number of loading cycles to reach the threshold shear strain of γ = 3.75%). Sudden accumulation of shear strain after a significant number of cycles without much shear strain [as discussed under Fig. 1(II-a)] can be explicitly seen for the sand specimen in Fig. 2(I) when the results are compared with those for the silt specimen.

Pore-water pressure ratio (r_u = ratio of excess pore-pressure, Δu to initial vertical effective stress, σ'_{vc}) reaches 1 in the sand specimen, but not in the silt specimen when the shear strain is about 3.75% [see Fig. 2(II)].

The above results suggest that the response pattern of low-plastic Fraser River silt to cyclic loading can be significantly different in comparison to that for Fraser River sand [Note: the focus herein is to distinguish the differences in the response patterns of sand and silt than to make a direct one-to-one comparison]. There is no sign of strength reduction below the applied cyclic stress ratio of about 0.17; however, the stiffness reduces with each cycle. For example, after 11 cycles, the stiffness is about 20 times softer than the first cycle. These typical results and many other tests [24] have suggested that fine-grained normally consolidated silts and clays of low plasticity can potentially be more resistant to liquefaction than loose sands.

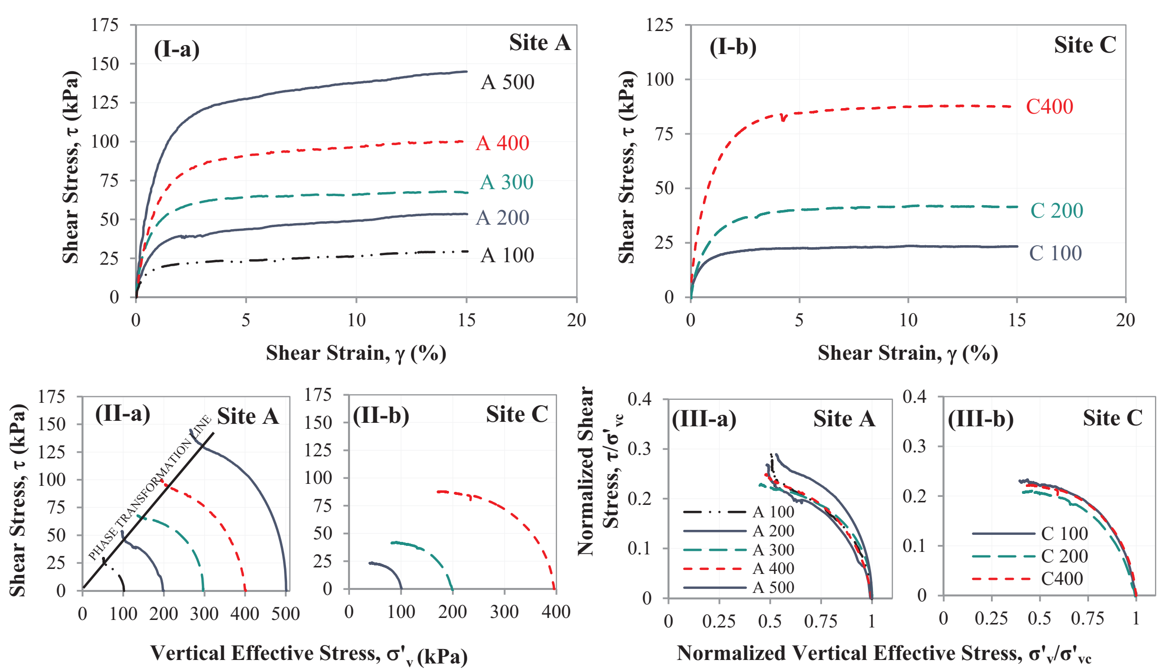

Fig. 3. Constant-volume monotonic DSS test results on relatively undisturbed specimens of natural silt retrieved from the Site A (data from [65]) and Site C (data from [66]) at normally consolidated stress state at varying confining stress levels: (I) shear stress-strain; (II) effective stress path; and (III) normalized effective stress path.

3.2. Effect of initial confining stress

When soil is subjected to a higher consolidation stress levels, the void ratio would decrease as soil particles assume denser packing arrangements. The DSS tests, both monotonic and cyclic loading conducted on soil samples retrieved from Site A and C, provided an opportunity to examine the effect of initial confining stress on the constant-volume monotonic and cyclic response of silts. In general, a denser particle arrangement is expected to cause an increase in shear resistance; this effect is observable from the stress-strain behavior observed from monotonic shear testing for the specimens from Site A, C, and D. First, the results from Site A (PI = 4) and C (PI = 7) are discussed together, and those from Site D (PI = 34) are presented thereafter due to the differences in the observed behavioral patterns.

Test data from a series of monotonic DSS tests on silts obtained from Site A, performed on specimens normally consolidated to σ'_{vc} of ~ 100 kPa (A100) through ~ 500 kPa (A500) are shown in Fig. 3(I-a), 3(II-a), and 3(III-a). In the stress path diagram shown in Fig. 3(II-a), all the specimens from Site A have deformed initially in a contractive manner followed by a dilative response. The line that connects phase transformation points (i.e., the points that soil specimen changes from the contractive tendency to dilative tendency) is shown in the same figure. Clearly, the ability to connect the points along a straight line suggests that the phase transformation seems to have occurred at the same mobilized shear stress ratio (τ/σ'_{vc}). These observations are in accord with the uniqueness of phase transformation line observed for sands by past researchers [25–27]. The observed stress-strain characteristics shown in Fig. 3(I-a) indicate that all the samples from Site A can be considered to have exhibited a behavior of no "strain- softening".

The response of test specimens from Site C is shown in Fig. 3(I-b), 3(II-b), and 3(III-b). As may be noted, the shear resistance initially increases as the shear strain increases until a maximum shear stress is reached. Under further shearing, the shear stress would remain almost at the same magnitude for all the tests conducted at σ'_{vc} of 100 kPa (C100) through 400 kPa (C400). Unlike the response of specimens from Site A, changing from contractive to dilative tendency could not be observed in the stress paths obtained for specimens from Site C [Fig. 3(II-b)]. The behavior of natural undisturbed silt from both Site A and C are found to be stress history-normalizable as notable from normalized stress paths falling within a narrow range as presented in Fig. 3(III-a) and 3(III-b). This indicates that the response of normally consolidated silt is similar to that typically observed for normally consolidated clays [28].

With respect to constant-volume cyclic DSS loading, the specimens that were consolidated to different σ'_{vc} levels initially exhibited a predominantly contractive response. With increasing number of load cycles, excess pore-water pressure cumulatively increased with associated progressive degradation of shear stiffness. Complete data plots in this regard cannot be provided to maintain conciseness, except for those presented in Section 3.1 to highlight typical response. The cyclic-mobility type response was generally observed in all the natural undisturbed silt specimens prepared from both the Site A and C, despite the magnitude of σ'_{vc} and CSR.

In order to assess the cyclic resistance of the silt, the cyclic resistance ratio (CRR) derived considering the number of loading cycles to reach single-amplitude γ = 3.75% from tests conducted on samples from the Site A and C are shown in Fig. 4. From the test results for Site A, it can be noted that cyclic resistance data points for specimens normally consolidated to σ'_{vc} between 85 kPa (in situ stress) and 400 kPa seem to fall on a single trend-line suggesting that cyclic resistance is relatively insensitive to the confining pressure and the associated different initial void ratio due to consolidation. The observed CRR for the silt from Site C also seems to follow the same trend, although the results are available only for a limited σ'_{vc} range between 100 kPa and 200 kPa.

The above observed CRR pattern, indicating independence from the σ'_{vc} value, is also similar to that noted by Zergoun and Vaid [29] for normally consolidated clay in cyclic triaxial tests. Again, this observation is in accord with the typical behavioral frameworks noted for normally consolidated clay (e.g., [28]).

The shear loading response of the relatively high-plastic (PI = 34) soil retrieved from Site D was found to be different from those observed for Site A and C. The results from monotonic shear loading for test specimens from Site D indicated that the shear resistance would initially increase with increasing shear strain until a peak is reached; this behavior is then followed by a decrease in shear resistance with further increase in shear strain level [see Fig. 5(I)].

As may be noted from the stress paths in Fig. 5(II), these normally consolidated, high-plastic soil specimens deformed in a contractive manner during monotonic shear loading. It is also of interest to note that, unlike the normalized stress path of tests for the specimens from Sites A and C, the normalized stress paths for tests on the specimens from Site D did not exhibit a coincidence as presented in Fig. 5(III). The degree of post-peak drop in shear resistance decreased as the level of effective consolidation stress level increased above the pre-

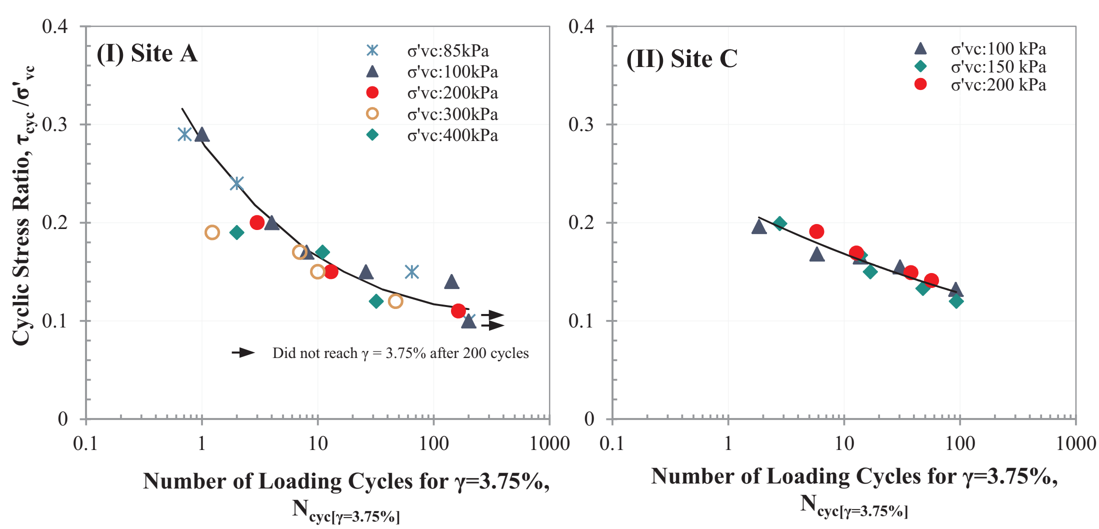

Fig. 4. Cyclic Stress Ratio (CSR) versus number of loading cycles required to reach γ = 3.75% in constant- volume cyclic DSS tests on silts from (I) Site A (after [65]) and (II) Site C (after [20]).

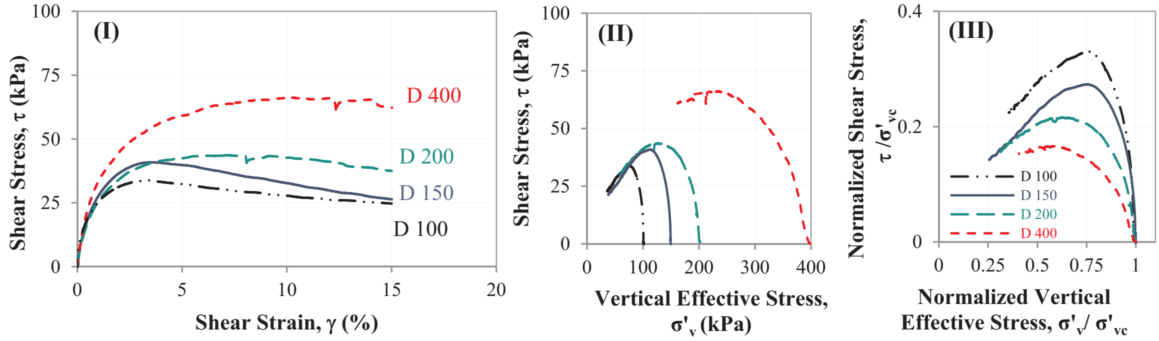

Fig. 5. Constant-volume monotonic DSS test results on relatively undisturbed specimens of natural silt retrieved from the Site D at normally consolidated stress state at varying confining stress levels: (I) shear stress-strain; (II) effective stress path; and (III) normalized effective stress path (after [66]).

consolidation stress.

In addition to the above, 1-D consolidation tests conducted on the same silt from Site D revealed that when the consolidation stress is increased beyond the estimated preconsolidation stress (~ 80kPa), a significant amount of vertical strain develops with a substantial reduction in the void ratio [30]. This observation suggests possible 'destructuration' of the initial particle fabric/microstructure in the soil specimen as the confining stress increases, such as that described by Leroueil et al. [31]. In other words, when specimens are consolidated to stress levels above the preconsolidation stress level, destructuration of the fabric of the soil specimen would occur, and in turn, the material consolidated in that fashion would behave in a comparatively weaker manner during subsequent monotonic shear loading; this observed suppression of the noticeable peak in shear resistance with increase in consolidation effective stress - as noted in the behavior in Fig. 5(II) and (III) - is well in accord with the destructuration inferred above from the 1-D consolidation tests.

The above attributed possible destructuration with increasing effective consolidation stress level for the soils from Site D is further corroborated by the CRR characteristics observed from cyclic DSS tests. As shown in Fig. 6, when CSR versus $N_{cyc[\gamma = 3.75\%]}$ obtained from cyclic tests conducted on specimens from Site D soils initially consolidated to different σ'_{vc} values are plotted together, the CRR clearly seems to reduce with increasing σ'_{vc}; this is much in contrast to the stress normalizability observed for the CRRs obtained for the fine-grained soils from Site A and C. [see Fig. 4].

In an overall sense, the breakdown of the natural fabric/microstructure and loss of inter-particle bonds seems to be a plausible candidate for explaining the above-observed differences in monotonic and cyclic loading characteristics for the soils from Site D. At this point, it would be relevant to highlight the importance of selecting the appropriate consolidation procedures/ recompression techniques, as developed by NGI, in the testing of structured soils [32]. For example, as shown in Fig. 6, the soil specimens consolidated beyond the pre-consolidation stress level indicated a weaker performance simply due to destructuration.

3.3. Effect of over-consolidation ratio

To study the effect of over-consolidation, undisturbed soil samples retrieved from Site A, B, and C were mechanically over-consolidated to pre-determined OCR values prior to the application of constant-volume DSS loading. The results from monotonic DSS loading tests conducted on natural silts from Site B and C and those from cyclic DSS loading tests performed on samples from Site A were available for this paper.

The monotonic shear stress-strain responses of normally consolidated (NC) specimens prepared using the soils from the Site B and C are presented in Fig. 7(I) and labeled as B NC and C NC respectively; the results for the specimens that were mechanically over-consolidated to OCR value of 2 and 4 for the same soil are also overlain on the same figure as OCR 2 and OCR 4. It can be observed that the shear resistance during monotonic shearing increased with increasing OCR.

Normalized stress path responses shown in Fig. 7(II) indicate contractive deformation of the normally consolidated specimen, as opposed to the initial dilative deformation followed by contractive deformation of over-consolidated specimens. As can be seen, the over-consolidated specimens exhibited negative pore-water pressure development during monotonic shear; this negative pore-water pressure development trend increased with increasing OCR. These observations are generally in line with the well-known increase in shear stiffness and strength with increase in OCR for fine-grained soils.

The specimens prepared from Site A were mechanically over-consolidated to values ranging up to 2.1, and Fig. 8 presents the variation of CSR versus $N_{cyc[\gamma = 3.75\%]}$, where the data points have been labeled with respect to the OCRs. Similar to the monotonic response, the increasing OCR has clearly resulted in increasing the CRR characteristics.

3.4. Effect of static shear bias

The term "static shear stress bias" is commonly referred to represent the stress state of an element of soil beneath a sloping ground configuration. The sloping ground would induce finite static shear stresses on the horizontal plane within a soil mass. To simulate the sloping ground conditions during laboratory element testing, the soil specimens are first consolidated with an applied initial static shear stress (τ_{st}). Cyclic or monotonic shear loading is then superimposed on the already existing τ_{st}; in typical laboratory testing, this second loading is applied in the same direction as the applied τ_{st}. Constant-volume monotonic direct simple shear tests were performed on undisturbed specimens prepared

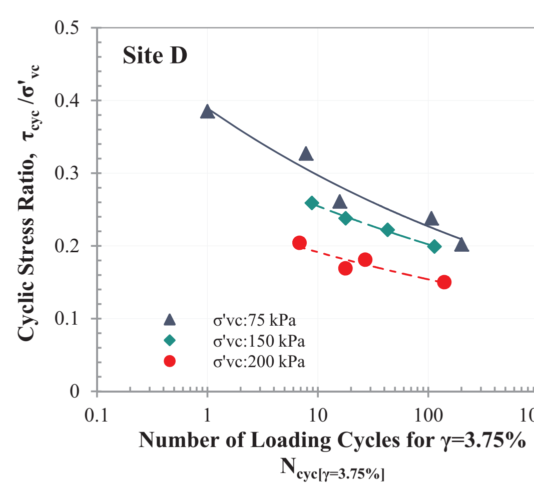

Fig. 6. CSR versus number of loading cycles required to reach γ = 3.75% in constant- volume cyclic DSS tests on silts from Site D (after [30]).

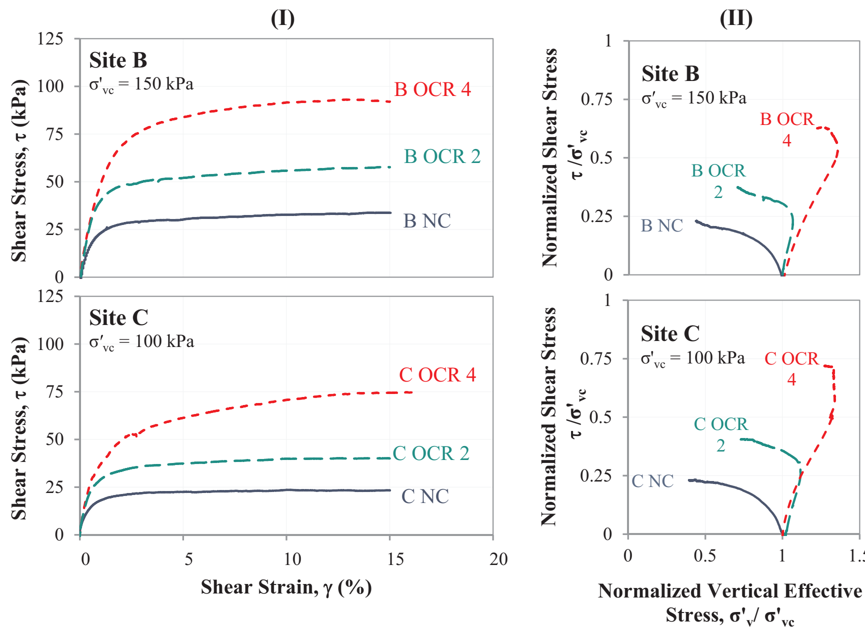

Fig. 7. (I) Shear stress-strain response and (II) normalized stress path response of constant-volume monotonic DSS tests on relatively undisturbed specimens from Site B and C, at confining stress states of NC, OCR 2 and OCR 4 (after [66]).

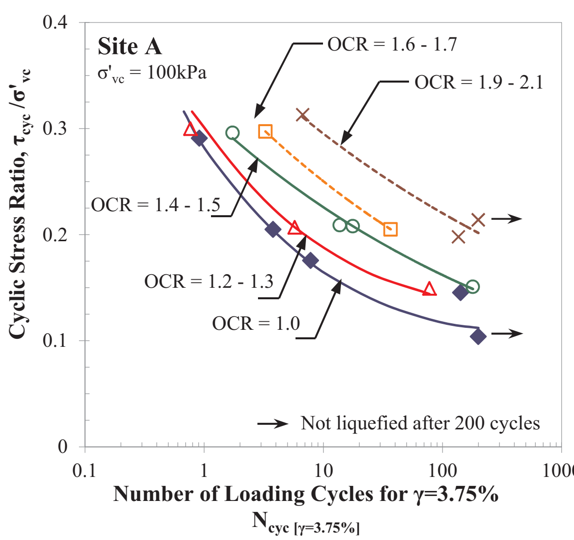

Fig. 8. Cyclic shear loading response: CSR versus $N_{cyc[\gamma=3.75\%]}$ for different OCRs on specimens from Site A (after [24]).

from samples retrieved from Site A. All test specimens were consolidated to the same σ'_{vc} of ~ 100 kPa and a set of selected values of initial static shear stress bias ($\alpha = \tau_{st}/\sigma'_{vc}$) – i.e. α = 0, 0.05, 0.10, and 0.15. The stress-strain and stress path response for these tests are shown in Fig. 9. The plot labeled as A 100 15 in Fig. 9 refers to the test performed on the specimen from site A at σ'_{vc} of ~ 100 kPa with α = 0.15.

The stress-strain response shows a slight increase in the strength with increase in initial static shear bias for the specimens tested at α = 0.05 and 0.10. This trend is noticeable in the results for the specimen tested at α = 0.15 for only up to 5% shear strain. The stress path presented in Fig. 9 indicates that the phase transformation point occurs approximately at the same shear stress ratio as those observed from monotonic tests conducted without static bias. These observations are again in accord with the uniqueness of phase transformation line observed for sands by past researchers [25–27].

The typical stress-path and stress-strain response from constant-volume DSS tests performed on undisturbed silt specimens from Sites A and C which were initially consolidated to a σ'_{vc} of ~ 100 kPa with different initial static shear bias and then subjected to cyclic shear loading under similar CSR value (~ 0.15) are shown in Fig. 10(I) and (II). A cyclic-mobility type strain development was observed throughout the cyclic loading process. Strain softening or loss of shear

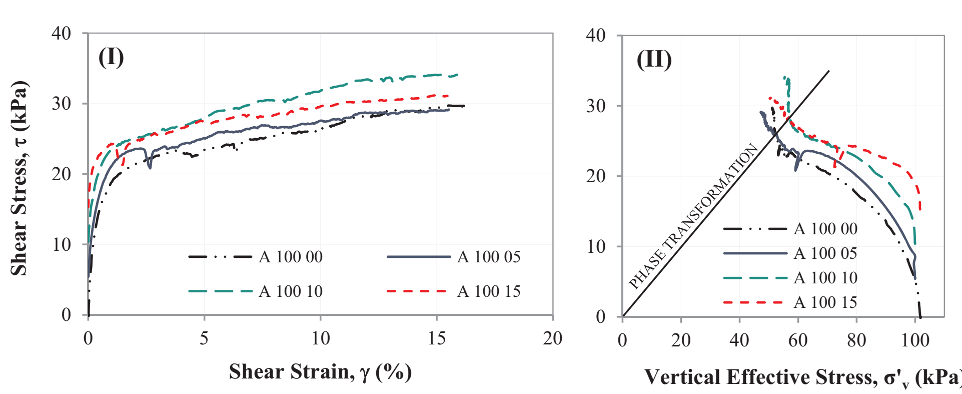

Fig. 9. (I) Shear stress-strain response and (II) stress path response of constant-volume monotonic DSS tests on relatively undisturbed specimens from Site A at varying initial static shear bias (α): σ'_{vc} = 100 kPa; α = 0, 0.05, 0.10, 0.15 (after [19]).

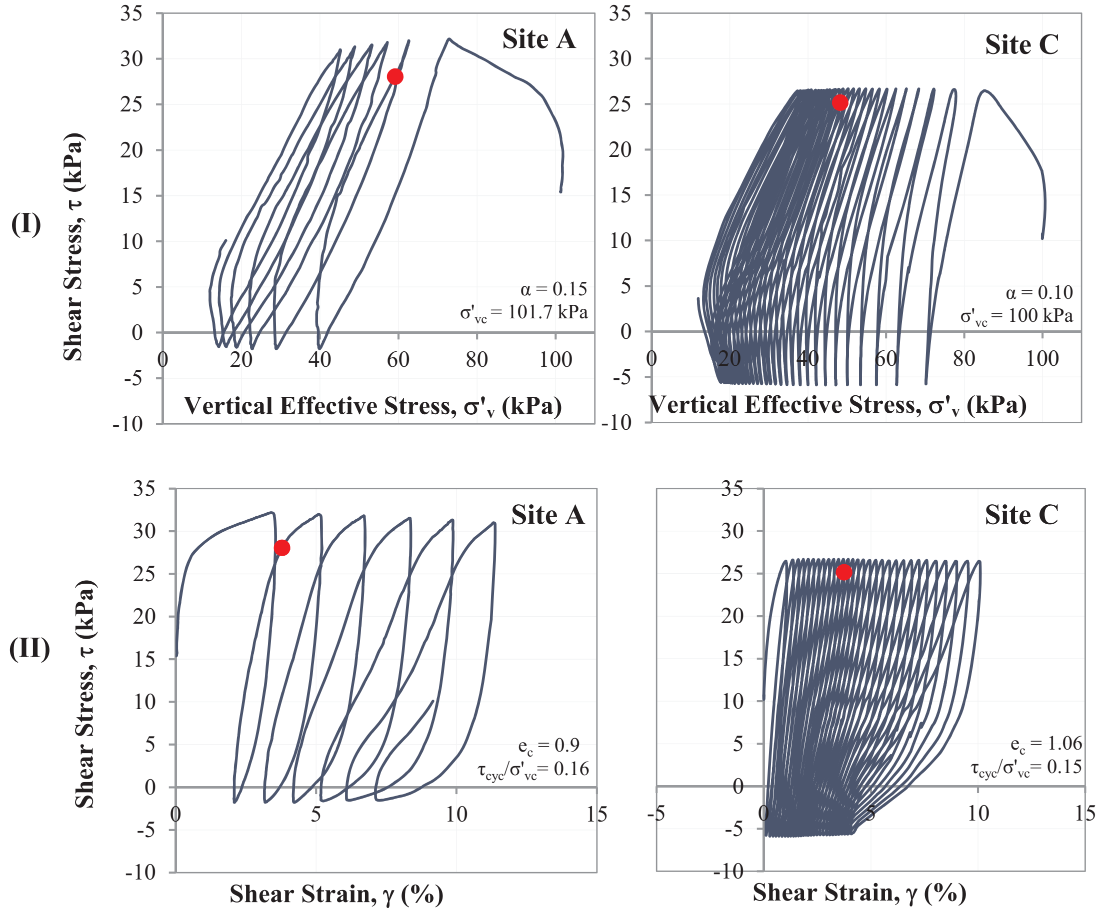

Fig. 10. (I) Typical stress path and (II) stress-strain response of constant-volume cyclic DSS tests with initial static shear bias on relatively undisturbed specimens from Site A (after [19]) and C (after [67]). *Note: Red dot indicates the onset of* $\gamma = 3.75\%$.

strength did not manifest in any of the tests conducted with static shear bias despite the α value and applied CSR level, or the degree of pore-water pressure developed. The development of excess pore-water pressure and accumulated shear strains with increasing number of cycles during the cyclic shear tests with different α and similar CSR values for undisturbed silt specimens from Site A and C are presented in Figs. 11 and 12, respectively. It can be observed that the potential for build-up of equivalent excess pore-water pressure and accumulated shear strains (with increasing number of cycles) seems to increase with increasing initial static shear bias level up to the tested α level of 0.15.

These observations further suggest that the tested silt from Site A and C is unlikely to experience flow failure under cyclic loading (based on tests conducted with $\alpha \le 0.15$). The effect of different values of α on the cyclic shear resistance of the tested silt specimens from Site A and C can be observed by plotting CSR versus $N_{cyc[\gamma = 3.75\%]}$ as shown in Fig. 13. The plot reveals that the CRR of the tested material decreases with increasing α.

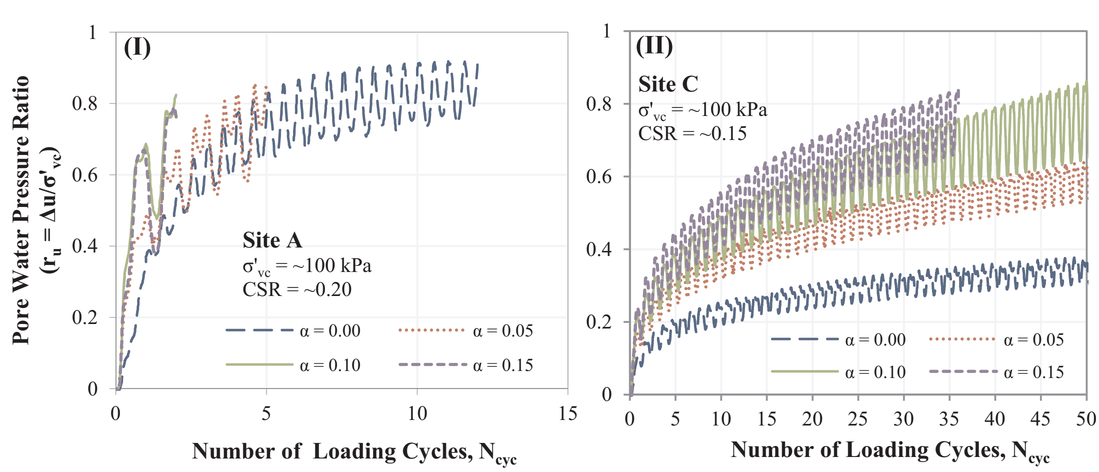

Fig. 11. Excess pore-water pressure development at different initial static shear values and constant CSR values for tested undisturbed specimens: (I) Site A (after [19]); and (II) Site C (after [67]).

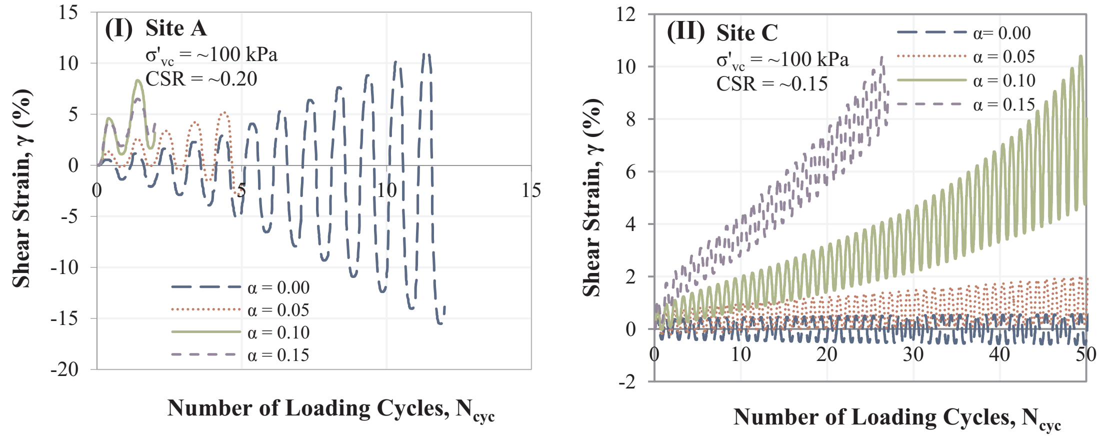

Fig. 12. Accumulation of shear strain at different initial static shear values and constant CSR values for tested undisturbed specimens: (I) Site A (after [19]); and (II) Site C (after [67]).

3.5. Effect of plasticity

It is well known that the soil plasticity (from Atterberg limit testing) is an index that could provide a significant indication of the anticipated shear stress-strain and compressibility characteristics of a given fine-grained soil. For example, the use of soil plasticity index (PI), in the current state of practice, as a parameter to demarcate fine-grained soils either as 'sand-like' or 'clay-like' with respect to their behavioral patterns is a good example in this regard. Both material related factors (such as morphology, mineralogical composition, physio-chemical properties of the finer fraction, moisture content and particle size distribution) and process-related factors (such as depositional mechanisms, subjected pressures, and temperature) could contribute to the soil plasticity of a given soil [33–36]. Knowing that the soil behavior is dependent on many factors (e.g., confining stress, void ratio, fabric, etc.), it is fair to state that the soil plasticity alone would not be sufficient to serve as a distinguishing parameter between 'sand-like' versus 'clay-like' behavioral patterns. With these limitations, testing was undertaken to assess how the PI would affect the behavior of fine-grained soil, particularly with respect to cyclic shear loading.

As notable from Table 1, the soils retrieved from Site A, B, C, and D possess a different level of soil plasticity. In consideration of this, the available data from both monotonic and cyclic shear loading from constant-volume DSS tests were considered to investigate the effect of plasticity. The normalized response of these soils under varying plasticity as derived from selected constant-volume monotonic DSS tests performed on relatively undisturbed normally consolidated silt specimens with plasticity indices of 4, 5, 7, and 34 are shown in Fig. 14. The test specimens were consolidated beyond their estimated pre-consolidation stress to ensure that the all specimens were in a normally consolidated state prior to the shear application.

In the initial stages of shearing, contractive tendency could be observed for the specimens that were subjected to monotonic shear loading; and at significantly large strains, only the specimen with PI = 4 indicated a slight dilative tendency. The specimen with PI = 34 exhibited strain softening after reaching the peak value of undrained shear stress. At large strains, the shear resistance was similar to those observed for other specimens with different plasticity. Based on the post-consolidation void ratio (e_c) as indicated in Fig. 14, it can be seen that specimen with PI = 34, which shows the highest shear resistance also has the highest void ratio - e_c = 1.77 (or lowest density) when compared amongst the specimens that had different plasticity. In a general sense, the results suggest that the increasing plasticity has the tendency to increase undrained shear strength; this is much in accord with the observations by others on this relationship [37–39]. The only exception is the case with the specimen having a PI = 4 that exhibited strain-hardening type response with a dilative tendency at large strain. It is, however, important to note that the variability of the plasticity of natural undisturbed soil samples tested herein and the subjectivity of the plasticity indices derived from the Atterberg limit test should also be taken into account and acknowledged when DSS test results are compared to assess the effect of plasticity.

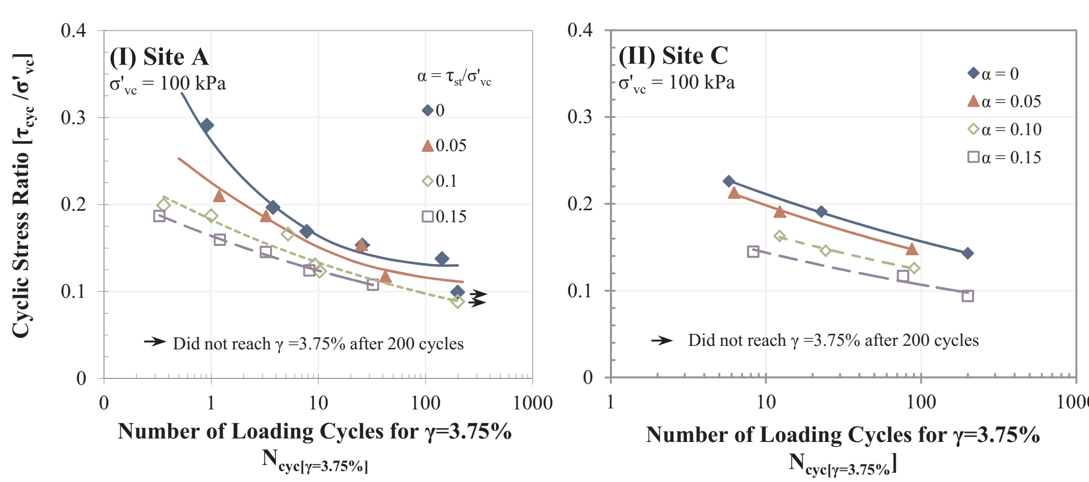

Fig. 13. CSR versus number of loading cycles required to reach γ = 3.75% in from constant-volume cyclic DSS tests performed on undisturbed silt specimens at varying initial static shear bias: (I) Site A (after [19]); and (II) Site C (after [67]).

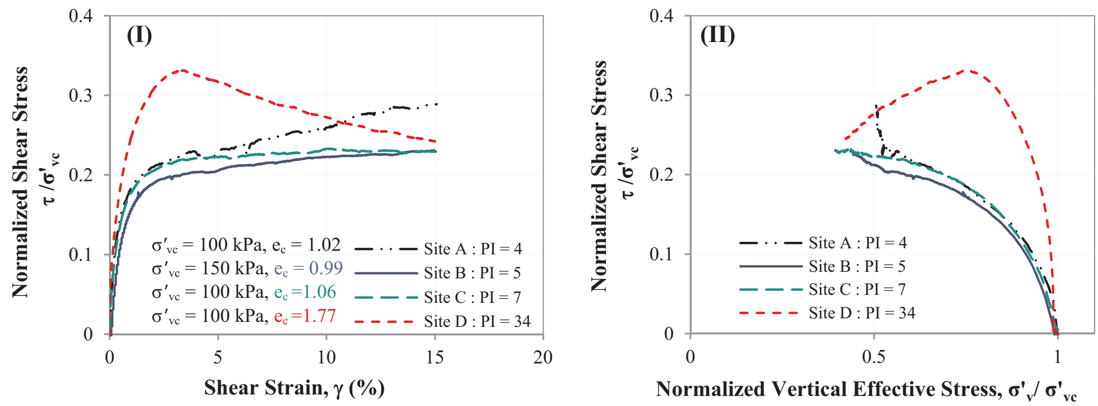

Fig. 14. (I) Normalized shear stress-strain response and (II) normalized stress path response of constant-volume monotonic DSS tests on undisturbed specimens of normally consolidated natural silt with different plasticity indices from Site A, B, C, and D (after [19,20]).

The effect of soil plasticity on cyclic loading can be assessed with respect to the variation of cyclic shear resistance in terms of CSR versus $N_{cyc[\gamma = 3.75\%]}$ obtained for the silt specimens, and the results shown in Fig. 15. Similar results derived by DSS testing of several other fine-grained materials with different plasticity by Sanin [19] are also included in the same figure. While appreciating the scatter of the experimental test results and possible variation of material from field samples, trend lines generated from the experimental results for relatively undisturbed soil specimens shown in Fig. 15 reveals that cyclic shear resistances of the fine-grained materials interpreted from the constant-volume cyclic DSS tests seem to generally increase with increasing soil plasticity.

3.6. Effect of microstructure/fabric

The inherent variability of natural soils, combined with the disturbance and destructuration that occur due to changes in stress state during sampling and specimen preparation, pose significant challenges in obtaining high-quality test samples that could meaningfully represent field soil conditions. It is also important to note that reconstituted specimens considered herein were prepared from slurry deposition method as described by Sanin [19]. As such, the e_c of the reconstituted specimens are likely to be different from those of the counterpart undisturbed specimens retrieved from the field. As shown in Table 2, it is of value to note that all reconstituted specimens for the tests presented in Figs. 16 and 18 had lesser e_c values (i.e., higher packing density) than those possessed by the comparative undisturbed specimens.

Table 2
Comparison of the post-consolidation void ratio (e_c) of undisturbed and reconstituted specimens.

Site	σ'_{vc} (kPa)	Monotonic or CSR	e_c	
			Undisturbed	Reconstituted
A	100	Monotonic	1.02	0.84
	200		0.95	0.81
	300		0.89	0.76
B	150		0.99	0.63
	300		0.87	0.57
	600		0.76	0.53
B	150	CSR = 0.13	0.93	0.65
C	200	CSR = 0.15	0.91	0.67
D	75	CSR = 0.2	1.64	1.21

It is of interest to examine the stress-strain and stress path response observed from constant-volume, monotonic, strain-controlled DSS tests on undisturbed and reconstituted specimens from Site A and Site B as presented in Fig. 16. The stress-strain response of reconstituted specimens from Site A initially exhibited a stiffer response (up to about 1% strain) in comparison to that from the counterpart undisturbed specimens. The peak shear stress (coincident with shear stress at large strain in this instance) could be noted to be greater for the undisturbed

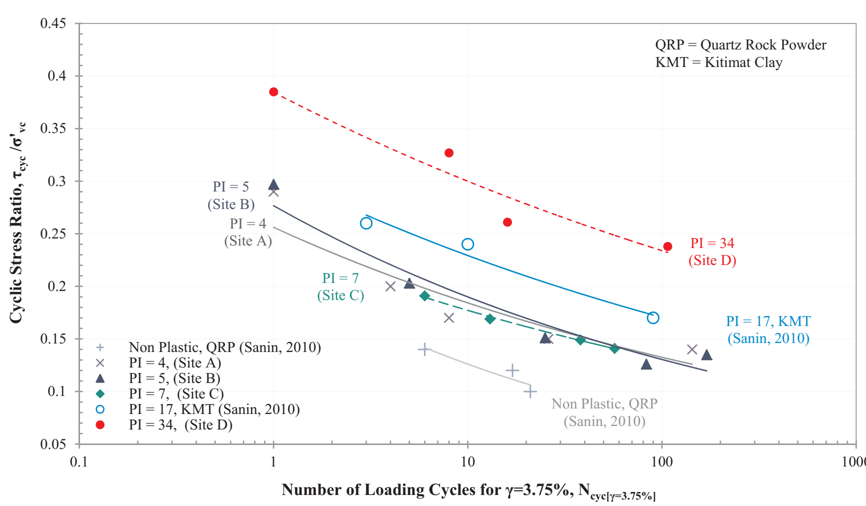

Fig. 15. CSR versus Number of loading cycles to reach γ = 3.75% curves from constant-volume cyclic DSS tests on fine-grained materials with different plasticity (after [61,68]).

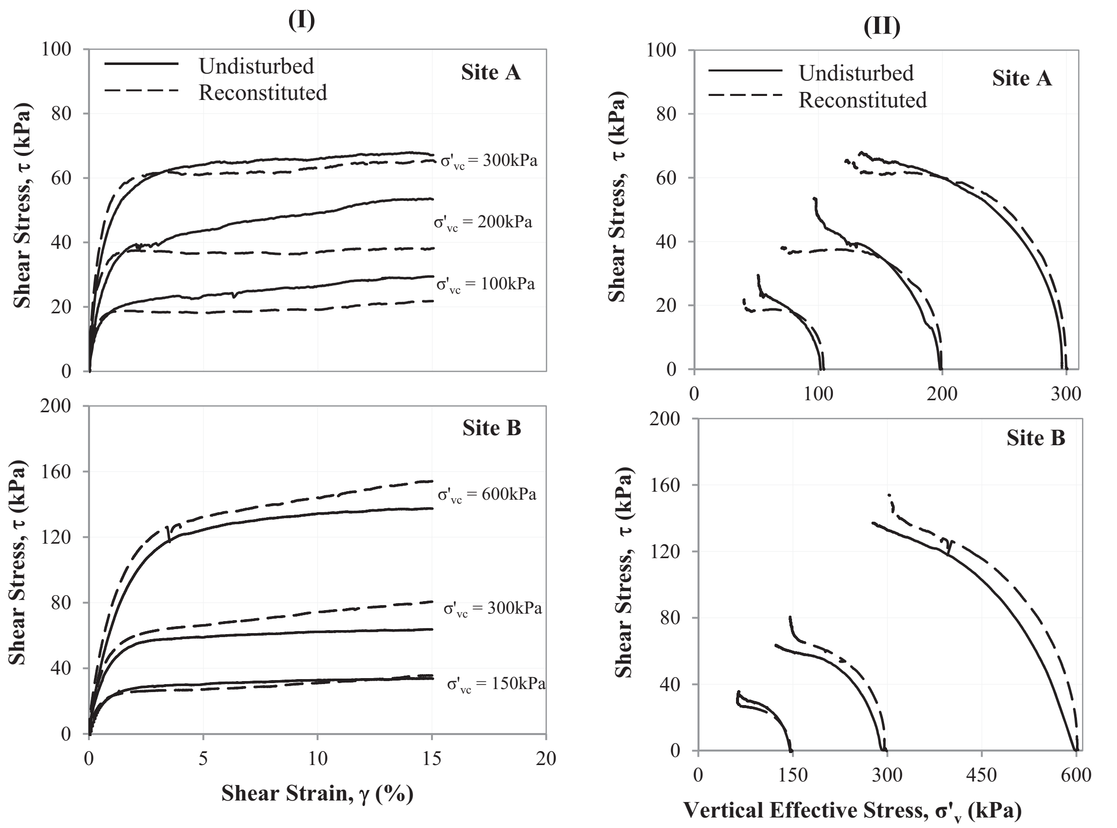

Fig. 16. The results from constant-volume monotonic DSS tests on undisturbed and reconstituted specimens prepared from Site A and B. (I) Shear stress-strain response; (II) Stress path response (after [19,69]).

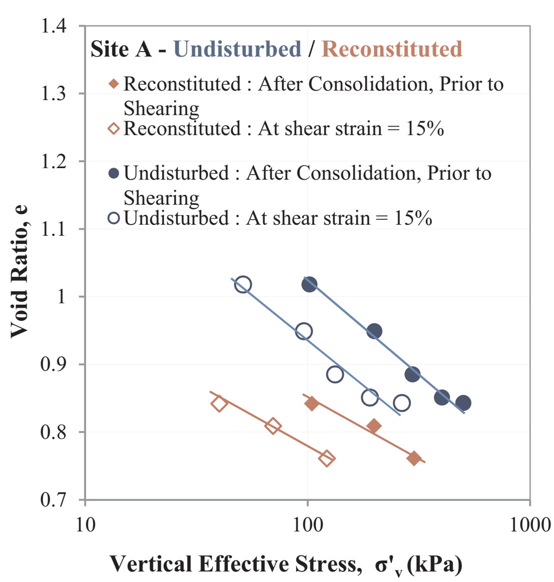

Fig. 17. e-log σ'_v relationships for undisturbed and reconstituted specimens of Fraser River silt after initial consolidation (prior to shearing) compared with those after reaching 15% shear strain in monotonic direct simple shear (after [19]).

specimens although the undisturbed specimens were relatively less dense than the reconstituted specimens. Typically, for a given initial effective confining stress, the general expectation is that comparatively lesser void ratio (denser particle arrangement) would result in an increase in shear stiffness and strength, whereas loss of natural fabric would cause a decrease in shear stiffness and strength. The competitive performance tendencies arising from these factors (i.e., density and fabric) would determine the observed behavior of a given reconstituted specimen. In this case, it appears that the influence of the change in fabric/microstructure due to reconstitution on undrained shear strength has over-shadowed effect from the higher density.

Reconstituted specimens from Site B indicate comparatively stiffer response than the undisturbed specimens. Unlike the results from Site A, the peak and the large strain shear strength of reconstituted specimens from Site B are greater than those of counterpart undisturbed specimens. It appears that in the observed monotonic shear loading response of reconstituted specimens from Site B, the effect from the change in void ratio has masked the influence due to the change in fabric and microstructure.

The e - log σ'_v states derived from the testing of undisturbed and reconstituted specimens at initial consolidation (and prior to shearing) and after reaching a shear strain of about 15% are presented in Fig. 17 – Note: only data from Site A has been presented in the plot for brevity and clarity. The data from undisturbed specimens seem to result in straight lines for the e - log σ'_v states for both the cases of prior to shearing as well as after reaching a shear strain (γ) of about 15% with the two lines paralleling each other; the observations are similar for the corresponding data from reconstituted specimens, thus leading to another pair of parallel straight lines. It is also important to note that the straight lines depicting the state after initial consolidation for the undisturbed and reconstituted specimens are at distinctly different locations in the e - log σ'_v space although the soil is identical; this is much in accord with the observations on an intrinsic consolidation line made by Burland [40]. The observation of distinctly different locations in the e - log σ'_v space between undisturbed and reconstituted specimens is also notable for the straight lines depicting the e - log σ'_v after achieving a ~15% shear strain state.

Careful observation of Fig. 17, while assuming the existence of a

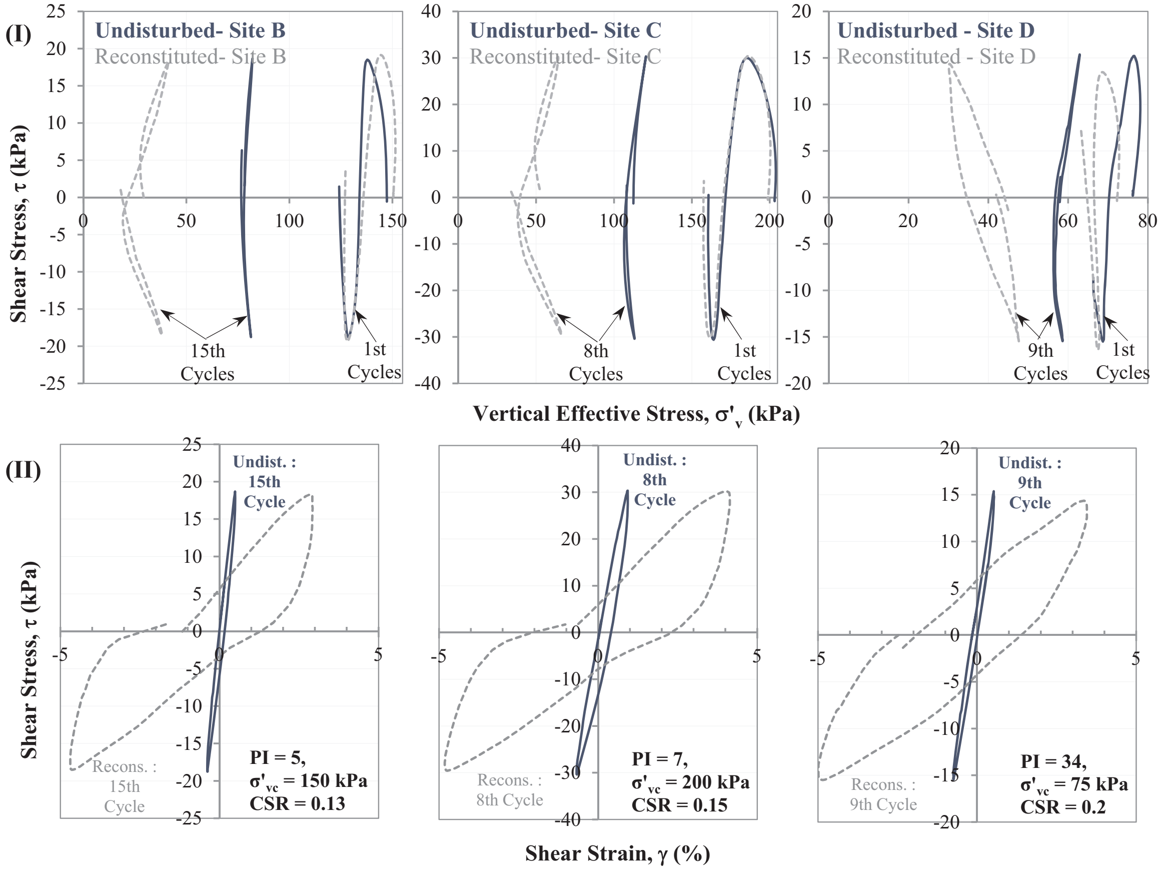

Fig. 18. Comparison of (I) stress path and (II) shear stress-strain curves of selected loading cycles during constant-volume cyclic DSS test on undisturbed and reconstituted specimen under similar normally consolidated σ'$_{vc}$ and CSR from Site B [PI = 5], C [PI = 7], and D [PI = 34] (after [20]).

unique critical state line for a given soil [26], suggest that the e - log σ'$_v$ data points of the reconstituted specimens after reaching γ ~15% would represent scenarios closer to the "critical state" than those data points from undisturbed specimens after reaching γ ~15%. In other words, considering that the critical state corresponds to a unique state at relatively large shear strain condition, it is reasonable to associate large shear strain state of a reconstituted specimen with the material critical state, than that arising from large strain state of the undisturbed specimen. However, particularly when designing for performance with limited deformations in practice, it appears prudent to obtain parameters from the testing of undisturbed specimens considering that the material states arising from reconstituted specimens (with destructuration/fabric-distortion) are expected to be located significantly different from those corresponding to the in situ material states in the real-life problem to be solved.

When test results from cyclic DSS tests performed on undisturbed and reconstituted specimens are compared for the soils from all the Sites B, C, and D as in Fig. 18, it can be noted that the reconstituted

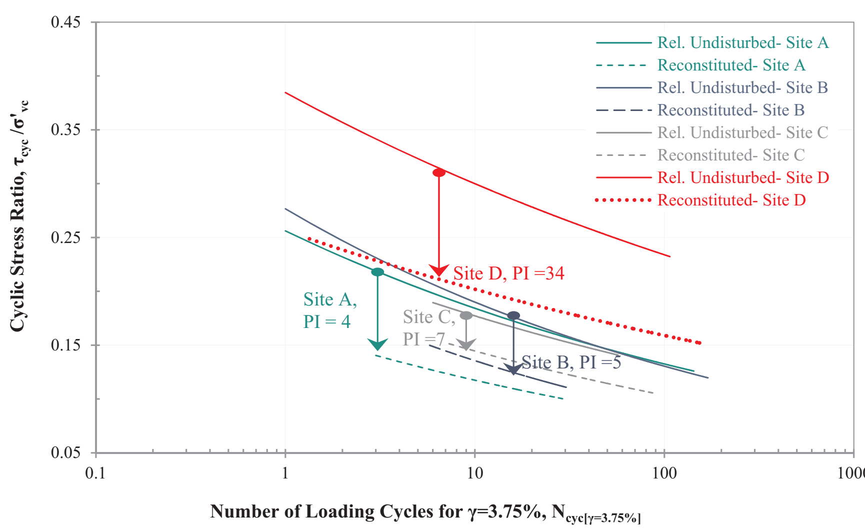

Fig. 19. CSR versus Number of loading cycles to reach γ = 3.75% curves from constant-volume cyclic DSS tests on fine-grained material indicating the reduction of cyclic shear resistance of reconstituted specimen with respect to relatively undisturbed specimen from subject sites (after [20,70]).

specimen displayed an increased degree of stiffness degradation and potential of strain accumulation compared to the relatively undisturbed specimen. These results well highlight the relatively weaker response of reconstituted specimens in comparison to that of undisturbed specimens (despite the comparatively higher density of the reconstituted specimens). It is to be noted that, for the purpose of clarity, selected loading cycles are shown as opposed to plotting the response for a complete set of loading cycles.

The cyclic shear response between undisturbed specimens and reconstituted specimens can be further assessed by comparing the CSR versus $N_{cyc\ [\gamma=3.75\%]}$ plots as shown in Fig. 19. As discussed earlier in evaluating the monotonic shear loading response, the decrease of void ratio and disturbance of natural fabric are two competing factors in governing the cyclic shear resistance of reconstituted material when compared to the cyclic shear resistance of undisturbed specimen. It seems that the influence arising from fabric is predominant in the observed cyclic shear resistance and that it has overshadowed the effect due to the change in the void ratio of the reconstituted specimens.

In an overall sense, the results indicate that the reconstituted specimens generally exhibit higher rates of pore-water pressure development, strain accumulation, and shear stiffness degradation under cyclic loading in comparison to those for undisturbed specimens. These observations suggest that the void ratio and confining stress alone are not the only key variables that would control the shear response of fine-grained soils. Clearly, the role played by the fabric and microstructure in controlling the soil behavior is significant and, it cannot be discounted in predicting the monotonic and cyclic shear loading response of fine-grained materials.

3.7. Post-cyclic settlements

The volume changes that arise due to the dissipation of excess pore-water pressures induced due to cyclic shear loading is another important consideration since it provides a way of estimating post-cyclic ground settlement and associated failures after the occurrence of earthquakes.

From the test results obtained for specimens from all subject sites, it was notable that post cyclic volumetric strains ($\varepsilon_{v\text{-}pc}$) would generally increase with the maximum cyclic pore water pressure ratio ($r_{u\text{-}max}$) and maximum cyclic shear strain experienced by the specimens during cyclic loading. Based on data from Site A, Wijewickreme and Sanin [41] suggested that there is a reasonably coherent correlation between $\varepsilon_{v\text{-}pc}$ versus $r_{u\text{-}max}$. Fig. 20 presents the results derived from the Site B, C, and D overlain on the correlation by Wijewickreme and Sanin [41]. The observed good agreement suggests that the unique $\varepsilon_{v\text{-}pc}$ versus $r_{u\text{-}max}$ correlation does not seem to be affected by the over-consolidation effects, particle fabric, initial preconsolidation void ratio of the soil, etc. Therefore, the validity of the derived relationship shown in Fig. 20 to estimate the $\varepsilon_{v\text{-}pc}$ subsequent to cyclic loading is now corroborated with additional data from 3 silt sites. Wijewickreme and Sanin [41] also explored the potential for a relationship between $\varepsilon_{v\text{-}pc}$ and maximum cyclic shear strain levels; they found that a meaningful trend between the two parameters could not be derived.

4. Considerations for future research

4.1. Understanding soil particle fabric effects

The observed significant different behavioral display in undisturbed and reconstituted specimen derived from the same material highlights the importance of soil particle fabric/microstructure in governing the response of silt in addition to the traditionally considered effects of void ratio (e) and σ'_{vc}. Significant insights have been gained on the understanding of the mechanical response of sands by studying their fabrics [42–44]. In a similar fashion, the authors believe that it is the right time to examine the particle structure of silts using the currently available sophisticated imaging technologies and help supplement/explain the behavioral patterns observed from mechanical element testing.

High-resolution X-ray computed tomography scan has been effectively used in visualizing grain fabric and obtaining microstructure of fine-grained materials for various applications. The process includes quantifying individual crystals or other discrete objects or void spaces, location, size, shape, orientation, and contact relationships with adjacent objects [45]. While few studies have been conducted to image 3-dimensional fabric microstructure of silt-size soils – e.g., [46,47] on cement paste; [48] on fiber-reinforced polymers – to the authors' knowledge, no studies have been done particularly focusing on the microstructure of silt size particle matrices. There is strong need to focus on examining the particle size, shape, and microstructural arrangement of silt in different fabrics. The first author's research program is currently advancing resin-impregnation procedures for "fixing" silt matrix specimens for this type of imaging work.

4.2. Performance of sand-silt and silt-clay mixtures

In real-life, most soils are encountered as mixtures of sand, silt, and clay. The presence of fines in sand has been noted to increase the CRR [49–51]. Conversely, a number of others [52–54] have concluded that increasing fines would reduce the CRR of sand. Some others suggest that increasing fines, up to a threshold value, would initially decrease and then increase the CRR of sand-silt mixtures [55–57]. These disparities are likely due to: (a) Problems in defining the packing density - e.g., maximum and minimum void ratios are based on methods using dry soil; (b) Use of moist tamping to prepare silt-sand specimens; (c) Lack of attention paid to seismic loading; (d) Not systematically covering the complete fines content spectrum in a given study.

On another front, the effect of mineralogy and plasticity on the CRR has been studied in the past in silt-clay mixtures. These studies have revealed the relationship between the cyclic strength of silt-clay mixtures and the plasticity index (PI) with different limiting values. Some studies [58] indicate that CRR decreases with increasing plasticity whereas others [59–61] suggest an opposing trend. The liquefaction resistance has been reported to decrease with increasing PI up to PI = 4, and increase with increasing PI for PI > 10 [62]. The observed effect of PI on the response of silt-clay mixtures is inconsistent. Considering the wide use of plasticity-based criteria in industry practice to assess the cyclic response, there is a need to study the effect of PI and

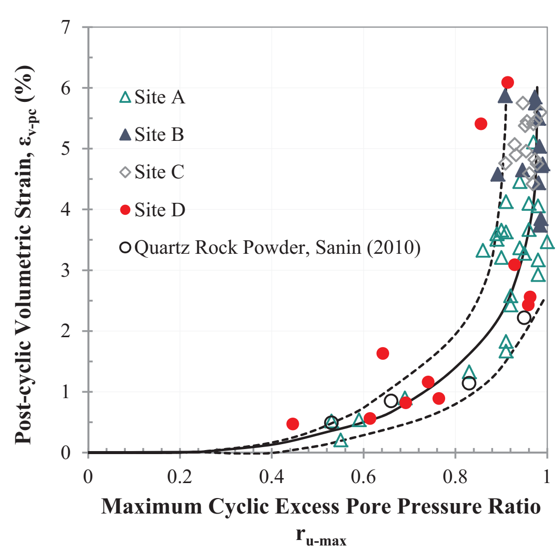

Fig. 20. Characteristics of post-cyclic volumetric strain ($\varepsilon_{v\text{-}pc}$) versus maximum cyclic pore-water pressure ratio ($r_{u\text{-}max}$) during cyclic DSS loading from specimens from Site A, B, C and D, overlain with Quartz Rock Powder (QRP) (data from [19,20]).

clay content on the shear response of silt-clay mixtures via systematic experimentation.

4.3. Performance-based criterion for assessing cyclic shear resistance

It is clear that arriving at a robust definition of liquefaction from an engineering point of view involves a complex process. The current approaches for assessing cyclic shear performance (i.e., liquefaction criteria) are either based on excess pore water pressure ratio (r_u), or 2.5% single-amplitude (SA) axial strain in cyclic triaxial loading or SA shear strain of ± 3.75% in cyclic DSS tests. The definition of onset of "liquefaction" or cyclic failure based purely on an arbitrary cyclic strain is not logical. As opposed to a criterion based on strain alone and/or excess pore water pressure development, it would be more rational to seek for signs of distinguishable changes in the overall stress-strain response pattern as cyclic loading progresses.

Wijewickreme and Soysa [63] have demonstrated that considering the visually observable changes to incremental shear stiffness in stress-strain loop patterns with the progress of cyclic loading can be used to delineate the reaching of unacceptable performance thresholds. Although the work presented by Wijewickreme and Soysa [63] uses a manual approach, the authors believe that there is a clear opportunity for developing computer codes to automate the process.

5. Summary and closure

This paper presents the results from a laboratory experimental research program undertaken to advance the knowledge on the behavior of natural fine-grained soils, and in turn, contribute to developing methods for the assessment of the engineering performance of such geomaterials under earthquake loading conditions. Natural soils originating from the Province of British Columbia, Canada, primarily those retrieved from the Fraser River Delta of the Greater Vancouver Region, were used as test materials in this regard.

All testing work was performed at the advanced geotechnical research laboratory at the University of British Columbia, Canada, with bulk of the work conducted using the in-house direct simple shear (DSS) device. For those instances where relatively undisturbed samples were sought for testing, fine-grained soils were retrieved using piston sampling that employed custom-fabricated, thin-walled, stainless steel tubes (with no inside clearance and a 5-degree cutting edge); as needed, some of the tests were conducted using specimens reconstituted using slurry depositional methods developed in-house. The testing work involved subjecting silt specimens, initially consolidated to selected initial vertical effective stress (σ'_{vc}) levels, to monotonic shear, or cyclic shear with or without applied initial static shear stress bias. The initial σ'_{vc} levels were selected to achieve the desired normally consolidated (NC) or over-consolidated (OC) test specimens prior to monotonic or cyclic shearing as desired.

Under monotonic constant-volume shear loading, the normally consolidated relatively low-plastic natural silt specimens deformed initially in a contractive manner. This behavior is sometimes followed by a dilative response; the phase transformation (from contractive to dilative) occurred essentially at the same mobilized shear stress ratio level. When normalized, the effective stress paths for tests conducted on specimens having different initial stress levels appeared to fall within a narrow range indicating that the response of normally consolidated silt is similar to that typically observed for normally consolidated clays. This indicates that the response of normally consolidated silt is similar to that typically observed for non-sensitive normally consolidated clays [28]. The above normalizability of stress-history was not necessarily observable with certain natural soils (i.e., Site D). It appears that 'destructuration' of the initial particle fabric/microstructure that would occur due to increasing confining stress is the reason for this deviation.

With respect to constant-volume cyclic DSS loading tests conducted on silt specimens, the specimens initially exhibited a predominantly contractive response; however, with increasing number of load cycles, they showed a cumulative increase in excess pore-water pressure with associated progressive degradation of shear stiffness. The cyclic-mobility type response was generally observed in all the natural undisturbed silt specimens prepared from all the sites regardless of the magnitude of σ'_{vc} and CSR. Liquefaction in the form of strain softening accompanied by loss of shear strength did not manifest regardless of the applied CSR value, or the level of r_u. One interpretation of the observed behavior would be to consider that fine-grained soils are unlikely to experience flow failure (i.e., catastrophic failure) under cyclic loading. However, it is important to keep in mind that the reliance on such observations needs to be tempered when faced with conditions such as the presence of fine-grained and coarse-grained soils interlayering or the susceptibility to destructuration.

The cyclic DSS tests conducted on mechanically over-consolidated natural soils indicated that it is possible to have a significant increase in CRR with increasing OCR. These observations are generally in line with the well-known increase in shear stiffness and strength with increase in OCR for fine-grained soils. When shallow foundations are constructed on sites with surficial silts, the soils are often treated with preloading to reduce compressibility, and in turn, foundation settlements. Often, the increase in CRR due to over-consolidation is not accounted in seismic performance evaluations primarily because the assessments are performed using data from in situ geotechnical characterizations conducted prior to preloading. The present results highlight the value of deviating from this practice.

The presence of sloping ground (or the presence of initial static shear stress bias - α) was shown to affect the response of natural fine-grained soils in a significant way. In particular, the CRR of the tested natural soil seems to decrease with increasing α. The current practice has recognized that the effect of initial static shear stress bias is not well understood even for sands. In spite of the knowledge that the liquefaction resistance of loose sand with a static shear bias may be lower than without static shear bias [64], often, no reduction of CRR for this effect is recommended. These observations suggest that this sentiment is true even for the fine-grained soils, and therefore, further studies on this front are clearly needed.

The profound effect of soil plasticity on cyclic loading could be seen from the behaviors on tests conducted on natural soils. In spite of the reality arising from the variability in natural soils, the experimental results from relatively undisturbed soil specimens indicates that the CRR of the fine-grained materials seems to generally increase with increasing soil plasticity.

The paper highlights the need to undertake further research to understand the effects of soil particle fabric/microstructure. The effect of plasticity and clay content on the shear response of silt-clay mixtures is another important consideration. The need to use performance-based criteria such as soil stiffness for assessing cyclic shear resistance, as opposed to criteria based on indirect parameters - such as shear strain alone or excess pore water pressure alone - is also emphasized.

There is strong evidence that silt behavior is complex and not well understood; as such, there is a need to develop robust approaches for silt liquefaction assessment for safe and cost-effective designs. This requires advancing the knowledge of the mechanics of silts under seismic shaking.

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