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COMPARISON OF CYCLIC SHEAR RESPONSE OF THREE NATURAL FINE-GRAINED SOILS HAVING DIFFERENT PLASTICITY

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Abstract

A systematic laboratory experimental research program has been undertaken to further investigate the cyclic shear loading response of natural fine-grained soils. The study findings add to the information base on the observed effects of plasticity index (PI) on the cyclic response of natural soils, which is an important consideration in the current practice for assessing cyclic failure of fine-grained soils. As a part of this study, the shear response of natural fine-grained soil samples retrieved from the Fraser River Delta of the Lower Mainland of British Columbia, Canada was investigated using constant volume cyclic direct simple shear (CDSS) testing. During constant-volume CDSS loading, gradual increment of pore water pressure, shear strain, and associated progressive degradation of stiffness with increasing number of loading cycles were observed for all tested natural undisturbed soil specimens, despite their different plasticity and soil gradation. The potential for excess pore water pressure and shear strain development with number of cycles was noted to decrease with increasing soil plasticity; in turn, the cyclic resistance of natural undisturbed soil specimens was noted to decrease with decreasing soil plasticity.

Keywords: fine-grained soil; plasticity; cyclic shear loading; silt behaviour; liquefaction

1 Introduction

The static and cyclic shear loading response of fine-grained soil has attracted wide attention after the observation of seismic induced ground failures in plastic silts during earthquakes. The type of earthquake-induced failures observed in fine grained soil has been generally different and less common in comparison to those observed in saturated sand. After investigating failures, researchers (e.g., [1] through [3]) have emphasized the need of more improved fundamental understanding of the fine-grained soils during earthquake shaking. In this regard, laboratory element testing of soil that plays a key role in enhancing the knowledge and understanding of the soil behavior becomes important.

In spite of the knowledge gathered so far, there are many facets of the cyclic behavior of fine-grained soil that needs to be understood requiring further laboratory experimental studies. For example, the effects of soil gradation, particle shape, microstructure/fabric, plasticity and anisotropy that significantly influence the shear response of fine-grained soils should be understood, in addition to the influence of confining stress and void ratio that has been traditionally considered as the governing parameters. Moreover, soil indices such as plasticity index (PI) and in-situ water content are often used in current practice as indicators in the performance assessment of fine-grained soils although those indices do not necessarily and directly account for the complex factors such as microstructure/fabric. Therefore, it is of value to establish connectivity between parameters such as PI and the fundamental soil behavior through laboratory testing of natural fine-grained soils.

The morphology, mineralogical composition, physio-chemical properties of finer fraction, micro-organisms and grain size distribution have been identified as some of the many material-related factors that would affect the soil plasticity as related to fines [4]–[6]. The process related factors such as the types of depositional mechanisms, subjected pressures, temperature, and characteristics of water and additives have also been considered to influence soil plasticity [6]. Only limited laboratory experimental investigations have been conducted to study the effect of soil plasticity on the shear loading response [7]–[10]; in most of these studies, the soil samples for testing have been produced by mixing different types of soil with pre-selected grain size distributions, introducing additives to vary the soil plasticity, etc. (e.g., [11]–[14]) as opposed to testing of natural soil samples having different inherited soil plasticity.

This background led to a systematic laboratory research program to be undertaken at the University of British Columbia (UBC) to study the above topic. As a part of this work, a series of laboratory shear tests were performed on natural fine-grained soil samples with three different soil plasticity values, obtained from three different locations of the Fraser River Delta in Lower Mainland of British Columbia, Canada. The cyclic shear loading response of those soils derived from the data obtained from the constant volume Direct Simple Shear (DSS) tests and observations made during the experimental work are presented in this paper.

2 Laboratory Experimental Aspects

The soil tested in this study originates from three different locations of the Lower Mainland of BC, Canada. The subject sites A, B, and C (as shown in Fig. 1) are located on: (i) south bank of the North Arm of Fraser River; (ii) south bank of Nicomekl River; and (iii) south bank of the Fraser River respectively. All these three sites are located in the Fraser River Delta that extends over a distance up to 23 km from a narrow gap in the Pleistocene uplands east of Vancouver, BC, and meets the sea (Strait of Georgia) along a perimeter of about 40 km [15].

Saturated tube soil samples from depth horizons located below the groundwater table were retrieved using conventional mud-rotary drilling test holes put down at the above sites. For this, specially fabricated stainless-steel tubes having an outer diameter of 76.2 mm, with no inside clearance, beveled 5° cutting-edge, and 1.4-mm wall thickness were used. Soil samples retrieved from site A and B found to be classified as silt with sand whereas soil from site C can be described as relatively high plastic clay. Fig. 2 presents the grain size distribution of the soils from the three sites. Atterberg limit tests performed indicated that the plasticity indices of the soils samples from sites A, B, and C are 5, 7, and 34, respectively. Table 1shows the summary of index properties and other related parameters of the tested soil from the three sites. The estimated pre-consolidation stresses through laboratory one-dimensional consolidation testing of specimens prepared from soil samples obtained from the three sites are also indicated in Table 1.

Fig. 1 – Locations of subject sites within the Lower Mainland British Columbia, Canada, over-lain on Map Data © 2016 Google [extract from: https://www.google.ca/maps/@49.1950801,-122.96154,11z]

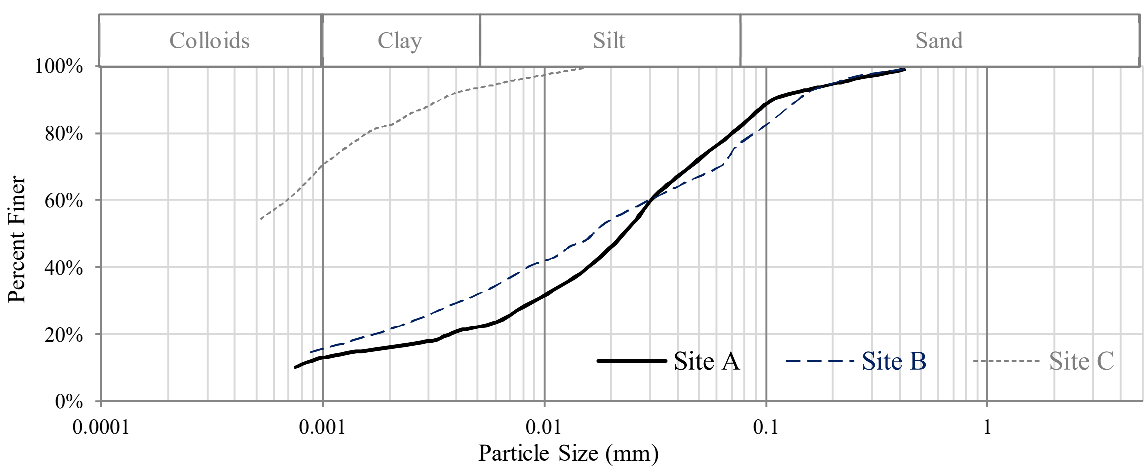

Fig. 2 – Grain size distribution results of soil samples from the three subject sites

Table 1 – Index properties of the tested soils

Parameter	Value(s)		
	Site A	Site B	Site C
Depth level (m)	5~7	4.2~5.5	4.9~6.2
In-situ water content (%)	37~40	37~39	57~65
Pre-consolidation stress (σ_{pc} kPa)	100~125	35~45	75~85
Specific gravity (G_s)	2.70	2.77	2.75
Plastic limit (PL- %)	29	34	42
Liquid limit (LL- %)	34	41	76
Plastic index (PI)	5	7	34
Classification Based on Plasticity Chart	ML	ML	MH

Modified NGI-type (Norwegian Geotechnical Institute type) DSS device [16] at UBC that accommodates a cylindrical soil specimen with a diameter of about 70 mm and a height of about 20 mm placed in a wire-reinforced rubber membrane is used to perform constant-volume DSS tests for this study. A sharpened-edge polished stainless steel ring is pushed vertically downwards on to the extruded soil samples from the thin-walled sample tubes that had a slightly larger diameter of ~73 mm to produce the soil specimens for DSS testing. Then, the top and the bottom sides of the specimens are trimmed using a wire saw, leading to a specimen with a height of about 20 mm having smooth top and bottom surfaces. The trimmed specimen is then carefully placed in the wire-reinforced membrane that laterally confines and enforces an essentially constant radial dimensions and prevents the specimen from localized lateral straining during consolidation and shear deformation.

To achieve the constant-volume condition, 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 [17], [18]. 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 relatively undisturbed, saturate soil specimens prepared as described above were initially consolidated to a common vertical effective consolidation stress (σ'_{vc}) of 200 kPa. This σ'_{vc} stress that is significantly greater than the estimated field pre-consolidation stress was chosen to ensure that all the specimens are normally consolidated to a common stress level prior to the application of cyclic shear loading. 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 [$CSR = \tau_{cyc} / \sigma'_{vc}$] would be applied on the specimen in a symmetrical sinusoidal manner at a frequency of 0.1 Hz. The attainment of 3.75% single-amplitude horizontal shear strain (γ) during cyclic shear loading in a DSS specimen was considered as an index to distinguish unacceptable cyclic shear performance, and for the assessment and comparison of cyclic shear resistance.

The details of the series of constant volume cyclic DSS tests performed on the relatively undisturbed soil specimens obtained site A, B, and C, and the corresponding test parameters are summarized in Table 2. Water content of the silt specimens from site A and B were in the range of 37%~40%, whereas relatively high plastic clay specimens from site C indicated water content of 57%~65%. As such, the initial void ratios (e_i) of the consolidated clay specimens from site C were greater than those from site A and B. As the specimens are prepared from relatively undisturbed samples, the observed minor variation of water contents and the initial void ratios amongst the specimens belonging to a particular site can be expected.

Table 2 – Summary of test parameters

Site	Test ID	WC %	e_i	σ'_{vc} (kPa)	e_c	(CSR) τ_{cyc}/σ'_{vo}	$N_{cyc\ 3.75\%}$	$[r_u]_{\gamma=3.75\%}$
Site A	A 200 22	39.68	1.06	199.54	0.94	0.108	36.8	0.79
	A 200 30	37.92	1.01	195.86	0.93	0.151	2.8	0.68
	A 200 36	37.36	0.91	197.18	0.81	0.181	1.2	0.68
	A 200 40	38.86	1.09	196.43	0.92	0.210	0.9	0.52
Site B	B 200 28	38.59	1.13	199.75	0.89	0.141	56.8	0.78
	B 200 30	38.37	1.11	203.37	0.91	0.149	37.8	0.79
	B200 34	37.62	1.08	201.17	0.88	0.169	12.8	0.71
	B 200 38	38.08	1.08	199.39	0.89	0.191	5.8	0.63
Site C	C 200 30	63.96	1.7	200.75	1.47	0.150	139.8	0.66
	C 200 34	64.36	1.61	201.08	1.35	0.169	17.8	0.58
	C 200 36	65.43	1.86	199.78	1.61	0.181	26.8	0.54
	C 200 40	57.55	1.53	201.78	1.32	0.204	6.8	0.46

WC : Water content; σ'_{vc} : Vertical effective stress prior to cyclic loading
e_i : Initial void ratio; e_c : Void ratio after consolidation
CSR: Cyclic stress ratio; $N_{cyc\ (3.75\%)}$: Number of loading cycles for shear strain of 3.75%
$[r_u]_{\gamma=3.75\%)}$: Pore water pressure ratio when the specimen initially reaches shear strain of 3.75%

3 Cyclic Shear Loading Response

The results obtained from the constant volume cyclic DSS tests (see Table 2) are presented below to examine the observed cyclic shear loading response of the tested soils. Typical response, in terms of the accumulation of shear strain and development of pore water pressure during the cyclic loading, stress path, stress-strain response and shear resistance are discussed in following sections.

3.1 Strain and pore water pressure development

During the application of cyclic shear loading, all test specimens showed gradual accumulation of shear strain and development of pore water pressure. The shear strain and pore water pressure development with respect to the number of loading cycles observed in three specimens having different plasticity (i.e., tests A 200 30, B 200 30, and C 200 30) are presented in Fig. 3. The pore water pressure ratio (r_u) is defined as the ratio of excess pore water pressure (Δu) to the initial vertical effective consolidation stress (σ'_{vc}) prior to cyclic loading. It shows that specimens with PI value of 5, 7, and 34 survived 2, 37, and 139 loading cycles before they attained the single amplitude shear strain level of 3.75%. The results show that the rates of strain accumulation and pore water pressure development during cyclic shear loading decreases with increasing plasticity of the tested specimens.

In Fig. 4, the characteristics of shear strain and pore water pressure are presented with respect to the normalized number of loading cycles - $N_{cyc}/N_{cyc\ (3.75\%)}$ (where the number of loading cycles at a given instance are divided by the number of loading cycles that would take to attain single amplitude shear strain level of 3.75%). As demonstrated during previous research studies, this normalized approach allows comparing the rates of shear strain accumulation and pore water pressure development effectively. As may be noted from Fig. 4, the rate shear strain accumulation of the three test specimens during cyclic loading appears to be similar to each other despite their differences of plasticity. However, some differences in the development of pore water pressure can be observed for the three specimens.

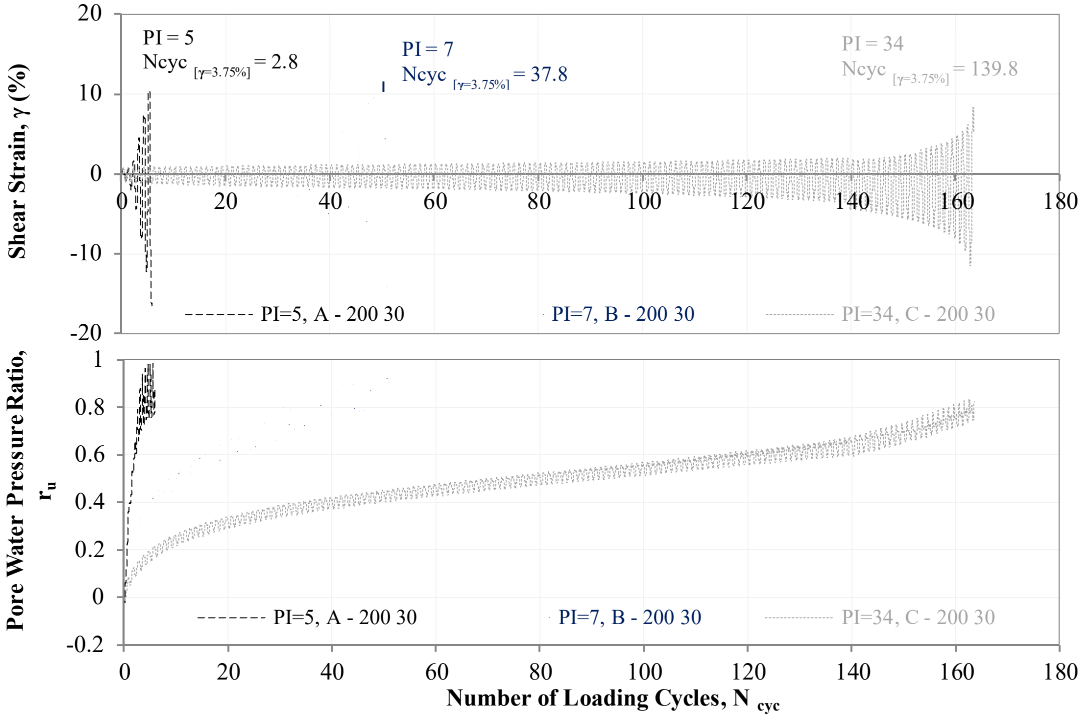

Fig. 3 – Accumulation of shear strain and development of pore water pressure with respect to number of loading cycles during constant-volume cyclic DSS loading with a CSR of about 0.15 for fine-grained specimens (PI = 5, 7, and 34.)

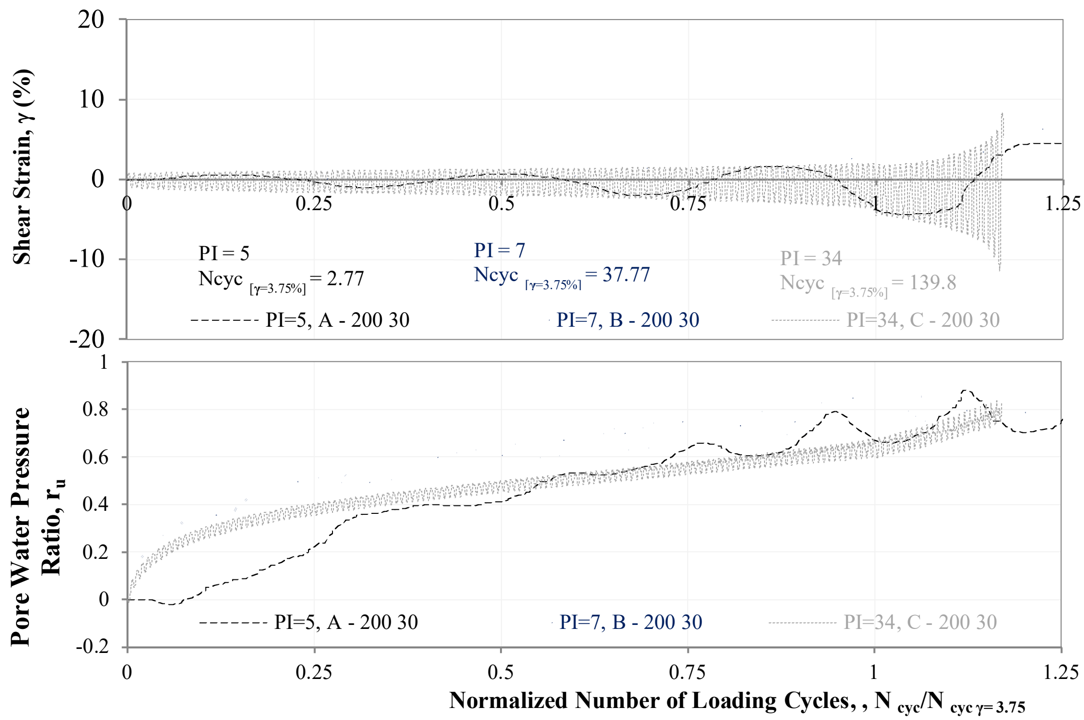

Fig. 4 – Accumulation of shear strain and development of pore water pressure with respect to normalized number of loading cycles during constant-volume cyclic DSS loading with a CSR of about 0.15 for fine-grained specimens (PI = 5, 7, and 34.)

The specimen from site A, having a PI value of 4, indicates a slight negative pore water pressure generation at the early stage of cyclic shear loading; however, after reaching N_{cyc} /$N_{cyc\ \gamma=3.75\%}$ ~ 0.25, the level of r_u is in the range similar to those observed for the other two specimens. Furthermore, the r_u values seems to have a range values of 0.5 to 0.8 at the time of reaching when shear strain (γ) = 3.75% (i.e., N_{cyc} / $N_{cyc\ \gamma=3.75\%}$ = 1). Similar observations were observed for the other tests performed with different CSR values as shown in Table 2 In essence, all test specimens showed a cyclic mobility type gradual strain accumulation associated with the development of pore water pressure despite their difference of plasticity. Flow failure type of strain softening with significant loss of shear stiffness was not observed in anyone of the tests. None of the tests conducted on undisturbed fine-grained soils (i.e., from Site C as indicated in Table 2) achieved r_u=1 even after some of the specimens have reached shear strain of 10%.

3.2 Stress path and stress-strain response

The stress path and stress-strain response obtained from the three tests of A 200 30, B 200 30 and C 200 30 considered above are compared in Fig. 5.

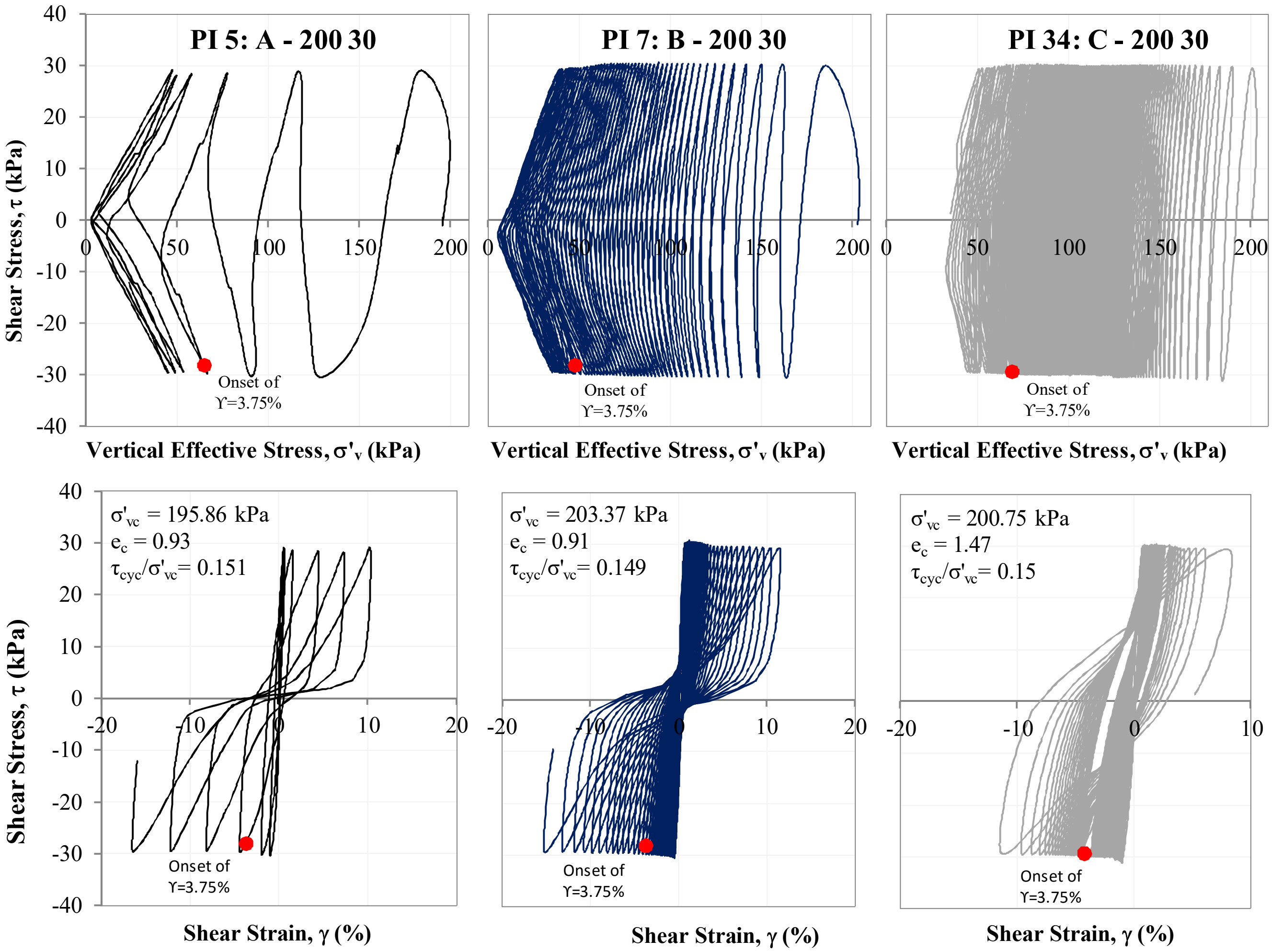

Fig. 5 – Comparison of stress path plots (top) and shear stress-strain plots (bottom) for normally consolidated fine-grained soil specimens (PI = 5, 7, and 34 with CSR ~ 0.15.)

The stress path and stress-strain curves illustrate the changes in contractive and dilative tendencies during cyclic loading, as well as the changes in shear stiffness with increasing number of loading cycles. From the comparison in Fig. 5, it can be seen that the relative rate of degradation of shear modulus of the tested specimens decrease with increasing soil plasticity. Careful observation of the stress path plots shown in Fig. 5 reveals that the specimens with higher soil plasticity experience vertical effective stress reduction at a relatively lower rate with increasing cycles. Furthermore, the relatively high plastic soil exhibited an initially contractive response that gradually changed into a dilative response with increasing cycles of loading. It should also be noted that the comparatively higher plasticity specimen (C 200 30) did not reach zero (or near zero) vertical effective stress level even after application of a large number of loading cycles; whereas, the comparatively lesser plasticity specimen (A 200 30) suffered near zero vertical effective stress condition after application of significant number of cycles.

3.3 Cyclic shear resistance

The variation of cyclic resistance ratio (CRR) versus $N_{cyc\ \gamma=3.75\%}$ for the three tested materials are shown in Fig. 6. It can be noted that the cyclic shear resistance generally increases with the increasing soil plasticity, and as expected, it is in accord with the findings presented above in Fig. 3 thorough 5.

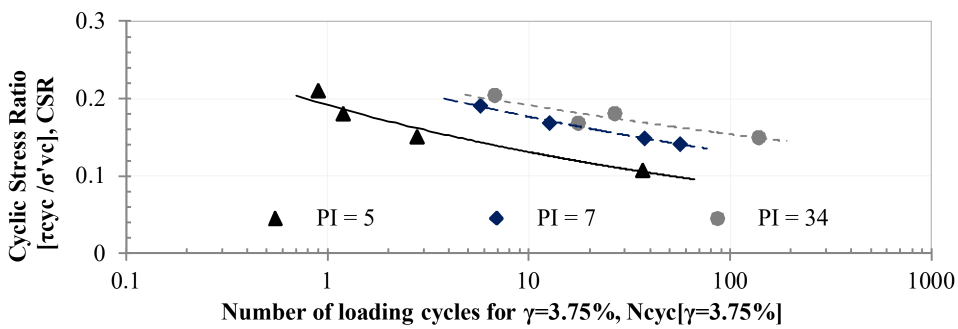

Fig. 6 – Cyclic stress ratio vs. number of loading cycles for shear strain of 3.75% for fine-grained soil specimens from site A, B and C (PI = 5, 7, and 34, respectively)

It is important to note that the tested natural fine-grained soil samples are arising from three different deltaic soil deposits from the Lower Mainland of BC. Therefore, the scatter of the experimental test results due to spatial and random variability should be expected. In spite of this, the experimental results reasonably indicate that cyclic shear resistance tend to increase with increasing soil plasticity, when natural fine-grained specimens having PI of 5, 7, and 34 were tested in constant volume cyclic DSS loading.

4 Summary and Conclusions

Cyclic shear loading response of natural fine-grained soils, having different plasticity, from three different sites located in the Lower Mainland of British Columbia was investigated through a series of constant volume cyclic DSS tests. The soils tested from the three sites had plasticity index (PI) values of 5, 7, and 34. All of the tests were performed on specimens that were initially consolidated to a vertical effective stress of ~200 kPa, thus ensuring that all the specimens were in a normally consolidated state at the time of cyclic shear load application.

Experimental findings from the tested soils indicate that the rate of shear strain accumulation and pore-water pressure distribution decreased with increasing soil plasticity during cyclic shear loading. None of the samples experienced r_u = 1 even when they were at shear strain level of 10%. Furthermore, all the test specimens indicated cyclic mobility type gradual accumulation of shear strain during cyclic shearing despite their difference in soil plasticity and post consolidated void ratios.

The cyclic shear resistance, interpreted based on the number of cycles to reach a shear strain (γ) of 3.75% under a given applied cyclic stress ratio, of the tested natural fine-grained soils (i.e., PI = 5, 7, and 34) were found to increase with increasing PI.

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