The 11th Canadian Conference on Earthquake Engineering
Canadian Association for Earthquake Engineering

CYCLIC SHEAR LOADING RESPONSE OF RELATIVELY HIGH-PLASTIC NATURAL FINE-GRAINED SOIL FROM THE FRASER RIVER DELTA

Achala SOYSA
Research Assistant, University of British Columbia, Canada
achala@civil.ubc.ca

Dharma WIJEWICKREME
Professor of Civil Engineering, University of British Columbia, Canada
dharmaw@civil.ubc.ca

ABSTRACT: The cyclic shear response of relatively high plastic (Plasticity Index = 34) natural fine-grained soil, retrieved from the Fraser River Delta deposits of British Columbia, Canada, using thin-walled sharpened-edge tube sampling was investigated under constant-volume cyclic direct simple shear loading. The response of reconstituted specimens prepared from the same material using slurry deposition was also studied. The development of pore water pressure, shear strain and associated degradation of stiffness with increasing number of loading cycles were observed for initially normally consolidated undisturbed and reconstituted specimens. Normalized cyclic resistance of undisturbed fine-grained soil decreased with increasing initial vertical consolidation stress. Moreover, for a given initial vertical consolidation stress, reconstituted specimens exhibited comparatively weaker cyclic shear response as opposed to undisturbed specimens. Void ratio and effective confining stress characteristics from one-dimensional consolidation testing suggested possible soil 'destructuration' of particle matrix when the specimens are loaded beyond the pre-consolidation stress. This destructuration is likely the reason for the fine-grained soil specimens tested at confining stresses greater than the pre-consolidation stress, as well as reconstituted specimens, to generate comparatively weaker cyclic shear resistance than those of undisturbed specimens.

1. Introduction

A proper understanding of the cyclic shear loading response of soils is an important consideration in the solution of geotechnical earthquake engineering problems. Although the confining stress and void ratio are traditionally considered governing parameters, it has been observed that factors such as microstructure, particle fabric, particle shape, anisotropy etc., also play a key role in influencing the shear response of soils. In the current practice, the cyclic shear resistance against liquefaction of fine-grained soils is often evaluated based on soil index properties - i.e., the plasticity index (PI) and in situ moisture content. Clearly, these indices do not account for the microstructure/fabric and associated parameters. For example, fine-grained material with PI > 7 is generally considered to exhibit “clay-like” behavior. This approach, suggested by Boulanger & Idriss (2004) is based on the limited available knowledge on the cyclic shear response of fine-grained soil, and Boulanger & Idriss (2004) have also cautioned that a threshold based on PI values are only an approximate way of delineating the coarse-grained and fine-grained behavioral patterns. Furthermore, it has also been acknowledged that fine-grained soils that are classified as “clay-like” would still have the potential to experience cyclic softening or cyclic failure.

Plasticity of the fine-grained soil can be depend many mineralogical factors: for example, (Tanaka & Locat, 1999) have indicated that different types of pores identified as inter-aggregate, intra-aggregate, skeletal, and intra-skeletal pores that can trap water could significantly influence the plastic and liquid limits of Osaka bay clay. Accordingly, it has been noted that the physio-chemical properties of fines and presence of microfossils can significantly influence the plasticity and in turn, mechanical behavior of fine-grained soil. Similarly, Hight & Leroueil (2003) have also emphasized the difference in the shear responses, observed for glacio-aqueous clay, alluvial clay, temperate marine clay and tropical marine clay due to their different ageing and depositional and post-deposition environments. Another important parameter is the sensitivity of the fine-grained soil in assessing the cyclic resistance (Idriss & Boulanger, 2008). With the different degrees of destructuration in natural fabric/microstructure of fine-grained soil, characteristics of cyclic shear resistance significantly vary (Hight & Leroueil, 2003; Leroueil & Hight, 2003). Additionally, research works of Leroueil et al. (1979) and Santagata & Germaine (2002) present that destructuring of fine-grained soil result reduction in bulk and shear moduli and undrained shear strength. By comparison of results obtained from cyclic simple shear tests performed on natural silts with those from reconstituted silts, Wijewickreme & Sanin (2008) and Sanin & Wijewickreme (2011) have shown that significant degradation of cyclic shear resistance of silts could occur due to the destructuration.

With this background, it was decided to further investigate the cyclic shear response of fine-grained soils using some natural soils as a part of an extensive and systematic laboratory experimental research program undertaken at the University of British Columbia (UBC). In this regard, a series of cyclic direct simple shear (DSS) tests were performed on relatively high plastic silty clay that was retrieved from Fraser River Delta deposits located in the Lower Mainland of British Columbia, Canada. In this paper, some of the results obtained from the constant-volume cyclic direct simple shear (CDSS) tests are presented, and the effect of effective confining stress and microstructure of relatively high plastic fine-grained soil on the cyclic shear performance of the soil are assessed.

2. Experimental Aspects

2.1. Material Tested

The Fraser River Delta 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 (Luternauer et al., 1993). The subject site of the soil tested in this study is located on the south bank of Fraser River upstream of the confluence with Pitt River. According to Armstrong & Hicock (1980), the site is underlain by overbank marine silt and silty clay. A cone penetration test (CPT) was performed at the location prior to the sample retrieval, and the interpreted soil behaviour based on this CPT data revealed the presence of uniform silt/clay deposit within the depth levels of 5 m to 12 m from the ground surface. Relatively undisturbed tube samples of soil were retrieved from the site using conventional mud-rotary drilling. The sampling tubes were stainless-steel with an outer diameter of 76.2 mm, no inside clearance, sharpened 5° beveled cutting edge, and 1.4-mm wall thickness.

The soil retrieved within the depth levels of 4.4 m to 5.5 m from the ground surface was used for the DSS testing in this study. Ground water table at the borehole used for the sampling was about 1 m below the ground surface. Specific gravity (G_s) of the soil was determined to be 2.75 from laboratory testing. The index properties of the tested soil are listed in Table 1. Particle size distribution analyses conducted for the retrieved soil revealed 10% to 15% of silt and 85% to 90% clay. With the results of Atterberg limit tests on plasticity, soil can be classified as MH in according to the D2487- ASTM (2011).

Table 1 Index properties of the tested soil

Parameter	Value(s)
Depth level (m)	4.9 – 6.2
In-situ water content (%)	58% – 69%
Preconsolidation stress (σ_{pc} kPa)	75 – 85
Specific gravity (G_s)	2.75
Plastic limit (PL- %)	42
Liquid limit (LL- %)	76
Plastic index (PI)	34
Unified soil classification [D2487-11 by ASTM (2011)]	MH

Upon the extrusion of the retrieved soil sample from the stainless steel tubes, a sharpened-edge polished stainless steel ring was pushed vertically downwards into the sample to obtain test specimens with a diameter of about ~70 mm for DSS testing; see next section for details on DSS test apparatus. In addition to the relatively undisturbed specimens, a limited number of reconstituted specimens were also prepared by slurry deposition method for DSS testing. Details related to specimen preparation can be found in Sanin (2010) and Soysa (2015), and they are not repeated herein for brevity.

2.2. Test Apparatus and Procedure

A modified NGI-type [Norwegian Geotechnical Institute type (Bjerrum & Landva, 1966)] direct simple shear (DSS) device at UBC was used as the test apparatus. The apparatus is designed to test specimens with a diameter of about 70 mm and a height of about 20 mm placed in a wire-reinforced rubber membrane. In addition to the very stiff lateral restraint from the steel-wire membrane, top and bottom loading platens can be clamped against vertical movement, thus, imposing a height constraint on the specimen; therefore, a constant-volume condition can be enforced during monotonic and cyclic shear loading application. 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 (Dyvik et al. 1987; Finn et al. 1978). In the test results presented herein, the change of vertical stress during constant-volume shearing is interpreted as the equivalent excess pore-water pressure due to shear loading.

The soil specimens prepared (as described in Section 2.1) were initially consolidated to a pre-determined vertical effective consolidation stress (σ'_{vc}) level. In this, the σ'_{vc} chosen for the test specimens prepared from relatively undisturbed field samples were kept above the estimated field preconsolidation stress to ensure that the specimens are at a normally consolidated (NC) state prior to the application of cyclic shear loading. Then, 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. 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.

2.3. Test Program

A series of constant-volume cyclic DSS tests was performed on relatively undisturbed soil specimens initially consolidated to vertical effective stress levels of 75 kPa, 150 kPa, and 200 kPa. Reconstituted specimens prepared from the same undisturbed samples were subjected to cyclic loading after initial consolidating to a vertical effective stress level of 75 kPa. These tests were conducted to study the influence of initial confining (consolidation) stress and soil particle structure on the cyclic shear response relatively high plastic natural fine-grained soililt and its microstructure on the cyclic shear resistance. The test program undertaken is shown in Table 2.

Natural relatively undisturbed specimens and reconstituted specimens are denoted as NS and RS respectively in the Table 2. The information presented in Table 2 suggests that, at a given vertical effective consolidation stress level, void ratios (e_c) of reconstituted specimens were smaller than those observed for the natural-relatively undisturbed specimens (i.e., Test series NS-75 versus RS-75 test series).

Table 2. Test Program: Summary of test parameters

Test ID	WC %	e_i	σ'_{vc} (kPa)	e_c	CSR	γ_{max} (%)	$r_{u\text{-}max}$	$N_{cyc\ [\gamma=3.75\%]}$
NS-75 30	63	1.75	76.8	1.70	0.39	25.0	0.76	0.8
NS-75 25	64	1.75	76.0	1.70	0.33	15.9	0.61	7.8
NS-75 20	63	1.91	75.6	1.84	0.26	26.4	0.69	15.8
NS-75 18	62	1.73	75.1	1.67	0.24	22.8	0.74	106.8
NS-75 15	66	1.71	76.2	1.64	0.20	0.9	0.45	NA
NS-150 39	65	1.81	151.8	1.69	0.26	25.0	0.64	8.8
NS-150 36	65	1.81	152.2	1.68	0.24	25.0	0.96	17.8
NS-150 34	65	1.79	153.3	1.66	0.22	25.0	0.96	42.8
NS-150 30	65	1.79	151.2	1.67	0.20	25.0	0.93	113.8
NS-200 40	58	1.53	201.8	1.32	0.20	25.0	1.03	6.8
NS-200 36	65	1.86	199.8	1.61	0.18	25.0	1.03	26.8
NS-200 34	64	1.61	201.1	1.35	0.17	21.1	0.91	17.8
NS-200 30	64	1.70	200.8	1.47	0.15	25.0	0.86	139.8
RS-75 18	88	2.42	70.9	1.41	0.25	17.1	1.03	1.3
RS-75 15	91	2.45	72.3	1.21	0.21	17.1	0.87	7.8
RS-75 10	87	2.36	73.5	1.41	0.15	9.0	0.90	161.8

WC = Average water content of the test specimen computed from the available trimmings from undisturbed specimens and sample portions from reconstituted specimen.
e_i = Initial void ratio (computed value after the application of the seating pressure of about 10 kPa)
σ'_{vc} = Vertical consolidation stress prior to the application of shearing
e_c = Post-consolidation void ratio prior to the application of shearing
CSR [τ_{cyc}/σ'_{vc}] = Cyclic stress ratio, (cyclic shear stress / initial effective vertical stress at the start of cyclic shearing)
γ_{max} (%) = Maximum shear strain during the cyclic shear loading phase.
$r_{u\text{-}max}$ = Maximum pore water pressure ratio during the cyclic shear loading phase.
$N_{cyc\ [\gamma=3.75\%]}$ = Number of uniform loading cycles to reach double amplitude shear strain of 3.75 %

3. Constant-volume Cyclic Direct Simple Shear Loading Response

3.1. Typical cyclic shear loading response

Typical stress path and shear stress-strain response obtained from constant-volume cyclic DSS testing of relatively undisturbed fine-grained soil specimens normally consolidated to σ'_{vc} = 75 kPa, 150 kPa and 200 kPa are presented in Figure 1. Similar behavioral observations derived from the tests conducted reconstituted soil specimens are given in Figure 2. Both natural and reconstituted soil specimens tested herein under constant-volume cyclic DSS loading indicated gradual accumulation of cyclic shear strain with increasing number of loading cycles. This strain accumulation is associated with progressive reduction of effective vertical stress; therefore, development of excess pore water pressure could be interpreted from the test results. While the development of excess pore water pressure seem to have caused the shear stiffness to gradually degrade and lead to cyclic softening, no abrupt loss of shear stiffness or shear modulus was observed in any of these tests. This observed cyclic mobility response suggest that the tested soil is unlikely to undergo flow failures under earthquake loading.

Moreover, the potential for shear strain and pore-water pressure development with number of loading cycles was observed to increase with increasing applied CSR. This can be identified with the decreasing number of loading cycles for the test specimens to reach 3.75% of single amplitude shear strain $N_{cyc\ [\gamma=3.75\%]}$ as CSR is increased (see Table 2). For example, test series performed on natural-relatively undisturbed specimen at normally consolidated stress of about 75 kPa (NS-75 test series), $N_{cyc\ [\gamma=3.75\%]}$ decrease from 107 to 1 when CSR is increased from 0.24 to 0.39. Although these observations are in general agreement for both natural and reconstituted test specimens, the realized cyclic shear resistance (in terms of the variation of CSR versus Ncyc $_{[\gamma=3.75\%]}$) was significantly affected when the test results between relatively undisturbed and reconstituted specimens are compared. Similar significant differences were also noted from the tests that were conducted to investigate the effect of vertical effective consolidation confining stress level. These aspects are presented and discussed in following sections.

3.2. Effect of confining stress

The effect of confining stress on the cyclic shear performance of relatively undisturbed soil can be assessed by examining the three graphs depicted in Figure 1 for cyclic DSS test performed at three different normally consolidated stresses as 75kPa, 150 kPa and 200 kPa. It is of interest to observe from Table 2 that the initial void ratio prior to consolidation (e_i) for all the relatively undisturbed specimens had values in a close range of 1.7 to 1.8. This suggests that the three specimens were at similar conditions at the starting stage, and hence, the comparison of test results from these specimens after consolidation to different σ'_{vc} levels should provide a reasonable assessment of the effect of initial confining stress. [Note: The value of e_i herein is defined as the void ratio of the specimen after the application of the seating pressure of about 10 kPa just before the main consolidation phase.]

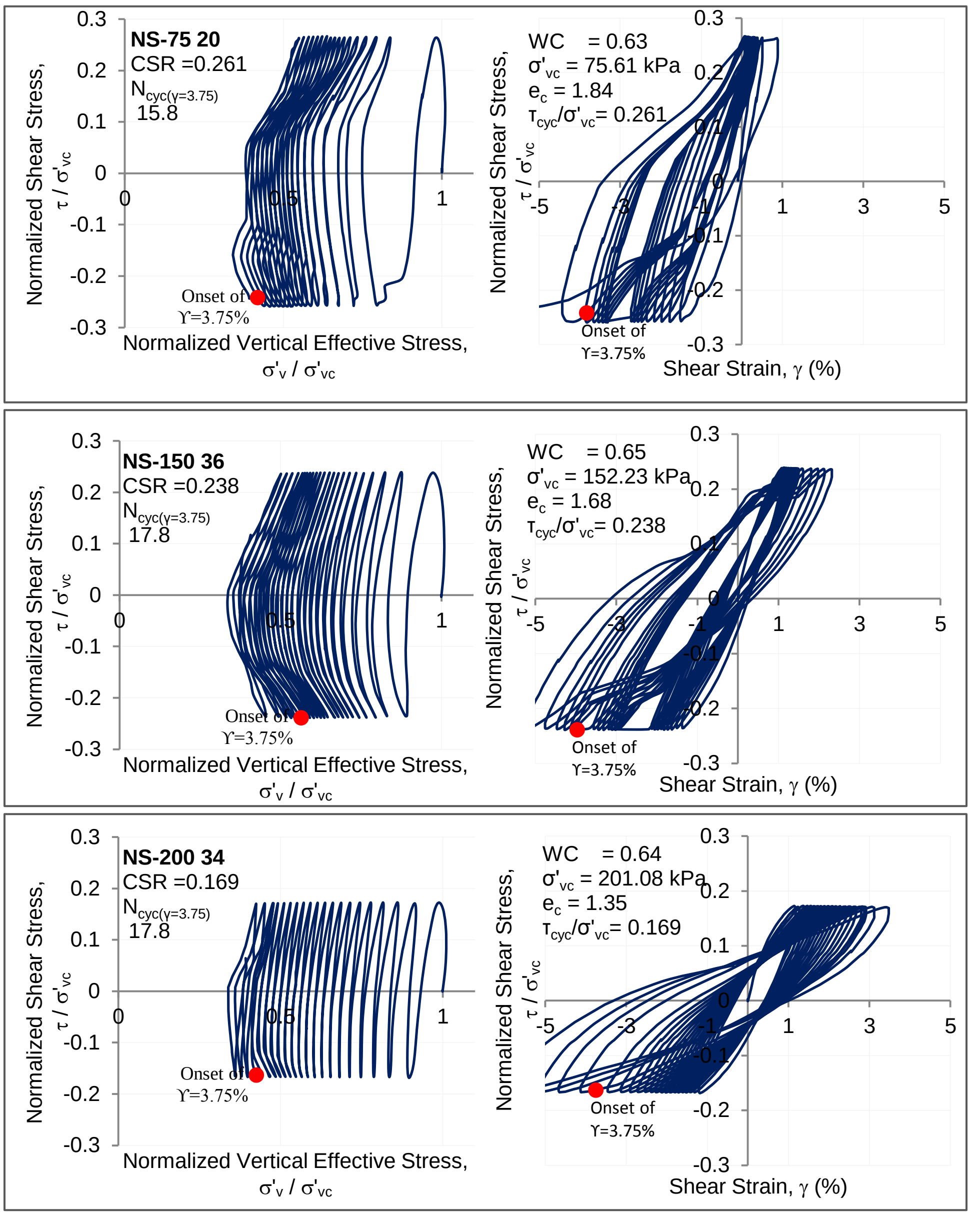

Figure 1 – Stress path and shear stress-strain curves of constant-volume Cyclic DSS test on relatively undisturbed specimen [PI =34], normally consolidated to σ'_{vc} = 75 kPa, 150 kPa and 200 kPa, where $Ncyc_{(\gamma=3.75\%)}$ =15.8, 17.8 and 17.8 for CSRs of 0. 26, 0.24 and0.17 respectively.

Examination of the void ratios after initial consolidation for the same tests (i.e., see e_c values in Table 2), indicates that e_c value of specimens decreases with increasing σ'_{vc} level; clearly, this is well in accord with the expected soil behaviour where packing density would increase with increasing confining stress. In the same line of thinking, it is arguable that the general expectation would be to observe higher cyclic shear resistance from the specimens having lower e_c values. However, as shown in Figure 2, the experimental results indicated an opposing trend when the cyclic stress ratio (CSR) was plotted against the Ncyc $_{[\gamma=3.75\%]}$ for the range of tests undertaken in this study. As may be noted from Figure 2, cyclic shear resistance of the specimens clearly decreased with increasing σ'_{vc} (i.e., the cyclic shear resistance of the soil specimens deceased with decreasing e_c).

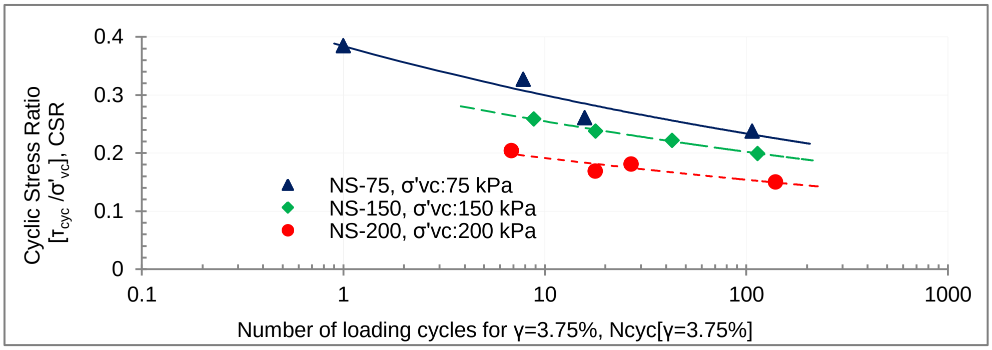

Figure 2 – Cyclic Stress Ratio versus Number of loading cycles to reach γ=3.75% curves from constant- volume CDSS tests on undisturbed specimens at normally consolidated stress levels of 75 kPa, 150 kPa and 200 kPa

The void ratio versus vertical effective consolidation stress characteristics observed during 1-dimentsional consolidation tests for the same relatively undisturbed soil, as presented in Figure 3, seem to provide a rational basis for the above observed unexpected behaviour. The results presented in Figure 3 clearly shows that when the consolidation stress is increased beyond the apparent pre-consolidation stress (σ'_{pc}) or "yield stress", the rate of void ratio reduction with increasing vertical effective stress exhibited by the specimen is significantly large compared tom the same rate at larger stress levels. This highly notable rate of reduction of void ratio has been well explained by a number of researchers, and it has been be attributed to 'destructuration' – i.e., disturbance of the soil fabric, breaking of cementation and other inter-particle bonds that give rise to microstructure (Hight & Leroueil 2003; Ladd & Degroot 2003) in the soil specimen when the confining stress increases. The level of destructuration would increase with increasing amount of σ'_{vc} past σ'_{pc}. As such, with reference to Figure 3, it can be argued that specimens in the test series NS-200 would have had a more destructured particle structure prior to cyclic shear loading than those specimens in the test series NS-150, thus leading to a lower cyclic resistance in the former series than the latter. This argument can similarly be extended when the test series NS-150 is compared with NC-75. In essence, the breakdown of the natural fabric /microstructure and loss of inter-particle bonds seems to be a plausible candidate for explaining the observed reduction in cyclic shear resistance of the soil, when they are tested at higher confining stresses.

3.2.1. Effect of destructuration due to reconstitution

Testing of specimens derived after reconstitution of relative undisturbed soils provided an opportunity to observe the effect of destructuration on the cyclic shear response. Clearly, it is reasonable to state that the level of particle structure destructuration in a reconstituted specimen would be much intense than the destructuration that would occur due to the increase in confining stresses as discussed in the previous section.

With these considerations in mind, three cyclic DSS tests were performed on reconstituted specimens prepared from slurry deposition method (i.e., RS-75 test series listed in Table 2). These reconstituted specimens were normally consolidated to vertical effective stress of 75 kPa, prior to the cyclic shear loading, and the results obtained from these tests are compared with the results from NS-75 test series.

The void ratio (e_c) of the specimen after consolidation to σ'_{vc} of ~75 kPa for the reconstituted specimens was smaller than those observed for the counterpart relatively undisturbed specimens consolidated to the same stress level (listed in Table 2). In other words, the reconstituted specimens after initial consolidation were in relatively denser state just before the commencement of cyclic shear loading. The cyclic shear performance of reconstituted and relatively undisturbed specimens during cyclic loading are presented in Figure 4 for comparison.

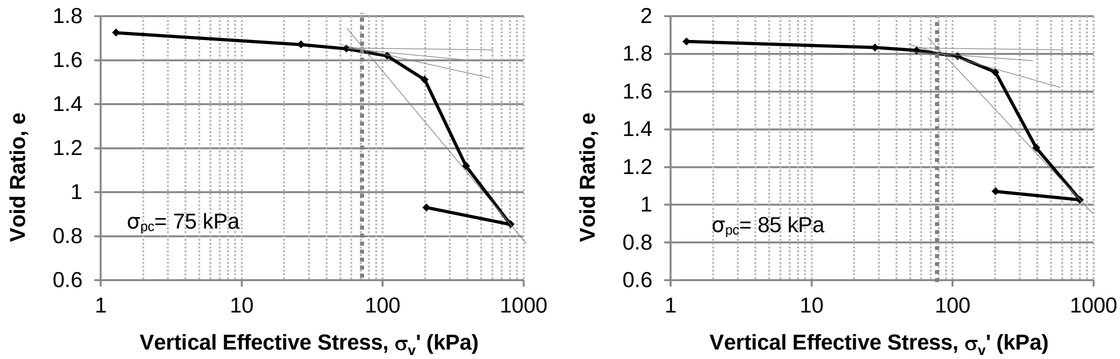

Figure 3 – Void ratio versus vertical effective stress originated form 1-Dimensional consolidation tests performed on relatively undisturbed specimens retrieved from 5 m and 6.1 m depths

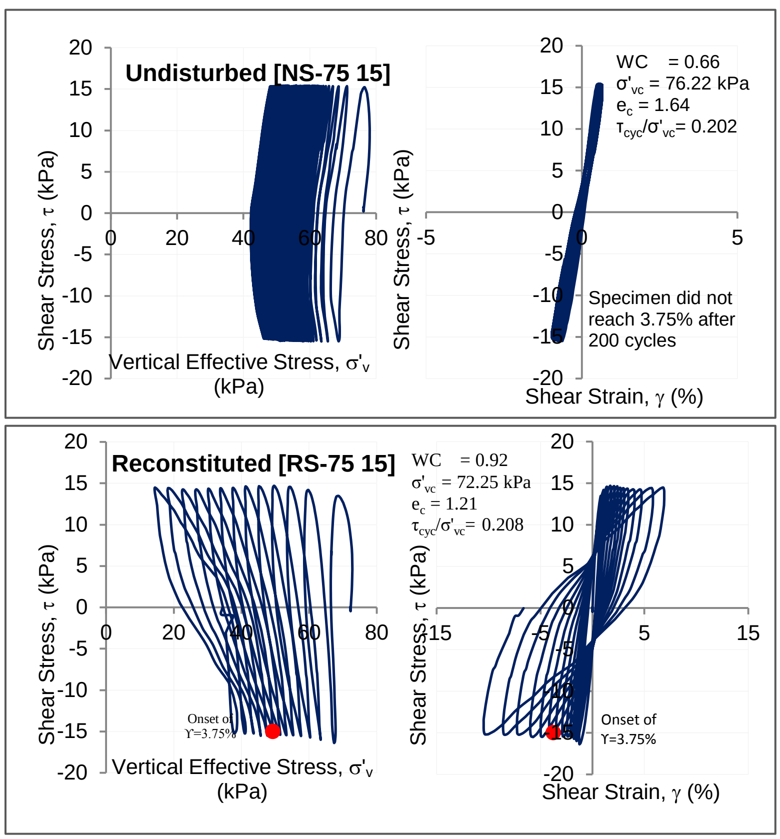

Figure 4 – Comparison of stress path and shear stress-strain curves of constant-volume Cyclic DSS test on relatively undisturbed (NS-75 15) and reconstituted specimen (RS-75 15)

As may be noted, there is a dramatic difference between the cyclic shear response of reconstituted and relatively undisturbed specimens. Despite the relatively denser initial state (relatively lesser e_c) at the time of cyclic shear loading application, as notable from Figure 4, the reconstituted specimens shows faster rate of shear strain accumulation and reduction of vertical effective stress when compared to those of natural-relatively undisturbed specimen. Specifically, the reconstituted specimen with e_c=1.21 reached 3.75% of single amplitude shear strain after the application of 8 loading cycles, whereas natural-relatively undisturbed specimen with e_c = 1.64 did not reach the single amplitude strain limit of 3.75% even after the application of 200 number of loading cycles. The extensive difference between the cyclic shear response of reconstituted and relatively undisturbed specimens is further notable in Figure 5 where the observed cyclic shear resistance – i.e., variation of CSR versus Ncyc $_{[\gamma=3.75\%]}$ - from the reconstituted specimens are compared with those from the testing of relatively undisturbed specimens. Clearly, the breakdown of the particle fabric/microstructure would be the most justifiable explanation for the observed reduction in cyclic shear resistance in the reconstituted soil compared to that of the relatively undisturbed soil.

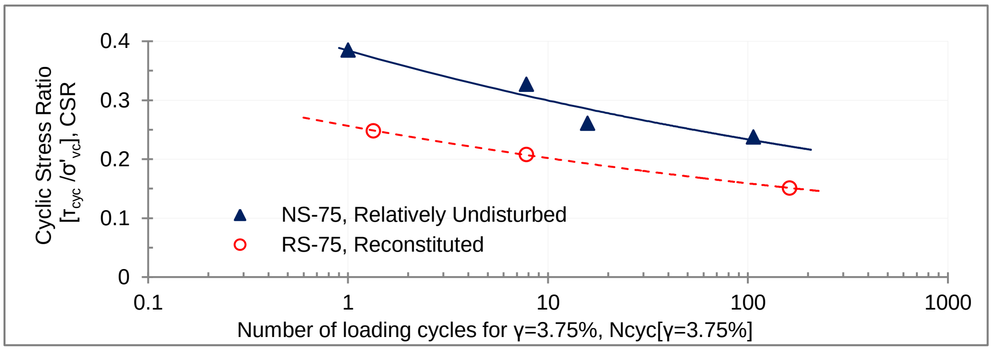

Figure 5 – Comparison of Cyclic Stress Ratio versus Number of loading cycles to reach γ=3.75% curves during constant-volume CDSS test on normally consolidated relatively undisturbed and reconstituted specimen

4. Summary and Conclusions

A series of constant-volume cyclic DSS tests were performed on relatively high plastic (PI=34) natural fine-grained soil retrieved from the Fraser River deltaic deposits in the Lower Mainland of British Columbia. Relatively undisturbed specimens of this natural soil were tested at three different normally consolidated stress conditions of 75 kPa, 150 kPa and 300 kPa, whereas reconstituted specimen prepared from slurry deposition method using the same soil were tested at normally consolidated stress level of 75 kPa. The objective of this series of tests was to study and assess the effect of confining stress and fabric/microstructure changes on the cyclic shear resistance of the soil. The tests on both relatively undisturbed and reconstituted specimens of this relatively high plastic fine-grained soil indicate gradual accumulation of shear strain and progressive development of pore-water pressure during cyclic shearing of . The potential for shear strain and pore-water pressure development with number of loading cycles increased with increasing applied CSR. Only cyclic mobility type behaviour and no liquefaction in the form of abrupt degradation of shear stiffness was observed for both types of specimens.

In the tests conducted on relatively undisturbed specimens, cyclic shear resistance seems to decrease with higher confining stress, despite of the fact that specimens had relatively lower void ratio at higher confining stress. Deformation characteristics observed from 1-dimensional consolidation tests revealed a significant rate of reduction of void ratio with increasing vertical effective stress, when the specimens are loaded beyond the pre-consolidation stress level - implying the possibility of destructuration of the soil fabric/microstructure at higher stresses. The observed tendency for decreasing cyclic shear resistance with increasing effective confining stress is likely due to the increasing destructuration of the soil particle matrix that occurs with increasing effective confining stress.

The reconstituted specimens, consolidated to the desired vertical effective stress resulted in a void ratio smaller (i.e., denser) than the void ratio reached by the relatively undisturbed specimens of the same soil when consolidated to the same vertical effective stress. Despite the above relatively denser state, reconstituted specimens displayed a weaker cyclic shear resistance in comparison to those from the relatively undisturbed specimens. Clearly, the influence of the change in fabric/microstructure on the cyclic shear resistance seem to have clearly been over-by shadowed those effects from changes in void ratio of reconstituted specimens.

5. Acknowledgements

This experimental research work was conducted with the financial support provided by the Natural Sciences and Engineering Research Council of Canada (NSERC) Discovery/Accelerator Supplement Grant (Application ID 429675). For the sampling of undisturbed soils for the laboratory testing, ConeTec Investigations Ltd. of Richmond, B.C. generously provided full funding support. The contribution of technical assistance of the UBC Civil engineering Workshop personnel is sincerely acknowledged.

6. References

Armstrong, J. E., & Hicock, S. R. (1980). Surficial Geology, New Westminster, West of Sixth Meridian, British Columbia. Geological Survey of Canada, “A” Series Map 1484A. Retrieved from http://ftp2.cits.rncan.gc.ca/pub/geott/ess_pubs/108/108874/gscmap-a_1484a_e_1980_mn01.pdf

ASTM. D2487 - 11, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (2011). ASTM International, West Conshohocken, PA. doi:10.1520/D2487-11.

Bjerrum, L., & Landva, A. (1966). Direct simple-shear tests on a Norwegian quick clay. *Géotechnique*, *16*(21), 1–20. doi:10.1680/geot.1966.16.1.1

Boulanger, R. W., & Idriss, I. M. (2004). *Evaluating the potential for liquefaction or cyclic failure of silts and clays*. Center for Geotecnical Modelling, Department of Civil & Environmnetal Engineering, Univeristy of California, Davis. Retrieved from http://citeseerx.ist.psu.edu/viewdoc/download?doi=10.1.1.132.3827&rep=rep1&type=pdf

Dyvik, R., Berre, T., Lacasse, S., & Raadim, B. (1987). Comparison of truly undrained and constant volume direct simple shear tests. *Géotechnique*, *37*(1), 3–10.

Finn, W. D. L., Vaid, Y. P., & Bhatia, S. K. (1978). Constant volume simple shear testing. In *Proceedings of the 2nd International Conference on Microzonation for Safer Construction-Research and Application* (pp. 839–851). San Francisco, California.

Hight, D. W., & Leroueil, S. (2003). Characterisation of soils for engineering purposes. In T. S. Tan, K. K. Phoon, D. W. Hight, & S. Leroueil (Eds.), *Characterisation and Engineering Properties of Natural Soils: Proceedings of the International Workshop, Singapore, 2-4 December 2002* (pp. 255–360). Lisse ; Exton, PA : Balkema.

Idriss, I. M., & Boulanger, R. W. (2008). *Soil liquefaction during earthquakes*. Earthquake Engineers Research Instiute.

Ladd, C. C., & Degroot, D. J. (2003). Recommended Practice for Soft Ground Site Characterization : Arthur Casagrande Lecture Práctica. In *12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering* (pp. 3–57). Boston, MA.

Leroueil, S., & Hight, D. W. (2003). Behaviour and properties of natural soils and soft rocks. In T. S. Tan, K. K. Phoon, D. W. Hight, & S. Leroueil (Eds.), *Characterisation and Engineering Properties of Natural Soils: Proceedings of the International Workshop, Singapore, 2-4 December 2002* (pp. 29–255). Lisse ; Exton, PA : Balkema.

Leroueil, S., Tavenas, F., Brucy, F., La Rochelle, P., & Roy, M. (1979). Behaviour of destructured natural clays. *Journal of Geotechnical Engineering*, *105*(6), 759–778.

Luternauer, J. L., Clague, J. J., Hunter, J. A. M., Pullan, S. E., Roberts, M. C., Woeller, D. J., … Hart, B. S. (1993). *The Fraser River Delta, British Columbia: architecture, geological dynamics and human impact, Canada. Reprinted from Deltas of the World. Proceedings of the Eighth symposium on Coastal and Ocean Management, New Orleans, Louisiyana*.

Sanin, M. V. (2010). *Cyclic Shear Loading Response of Fraser River Delta Silt*. PhD Thesis, The University of British Columbia. Retrieved from http://elk.library.ubc.ca/handle/2429/30064

Sanin, M. V., & Wijewickreme, D. (2011). Cyclic shear response of undisturbed and reconstituted Fraser River Silt. In *14th Pan-American Conference on Soil Mechanics and Geotechnical Engineering 64th Canadian Geotechnical Conference*. Toronto, Ontario.

Santagata, M. C., & Germaine, J. T. (2002). Sampling disturbance effects in normally consolidated clays. *Journal of Geotechnical and Geoenvironmental Engineering*, *128*(12), 997–1006. Retrieved from http://ascelibrary.org/doi/abs/10.1061/(ASCE)1090-0241(2002)128:12(997)

Soysa, A. (2015). *Monotonic and cyclic shear loading response of natural silts*. M.ASc. Thesis, The University of British Columbia. Retrieved from http://hdl.handle.net/2429/52356

Tanaka, H., & Locat, J. (1999). A microstructural investigation of Osaka Bay clay: the impact of microfossils on its mechanical behaviour. *Canadian Geotechnical Journal*, *36*(3), 493–508. doi:10.1139/t99-009

Wijewickreme, D., & Sanin, M. V. (2008). Cyclic Shear Response of Undisturbed and Reconstituted Low-Plastic Fraser River Silt. In *Geotechnical Earthquake Engineering and Soil Dynamics IV* (pp. 1–10). Sacramento, California: American Society of Civil Engineers. doi:10.1061/40975(318)88