Some Observations on the Mechanical Behavior of Natural Fine-grained Soils from Laboratory Direct Simple Shear Testing

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ABSTRACT: The monotonic shear loading response of natural fine-grained soils was investigated using constant-volume direct simple shear tests. Natural soils, retrieved using thin-walled sharpened-edge tube sampling methods from three different locations of the Fraser River Delta of British Columbia, Canada were used for the study. Plasticity indices of the tested samples were 5, 7, and 34. The shear strength of normally consolidated low plastic fine-grained soils was observed to increase with increasing initial consolidation stress, the behavior was found to be stress-history-normalizable. As expected, the constant volume DSS tests on over-consolidated specimens indicated higher shear strength than the normally consolidated specimens. The results from the test program also indicated that the shear strength of high plastic fine-grained soil were significantly higher than those for low plastic silt. Unlike for those observed for natural low plastic silts, the normalized shear strengths of natural high plastic fine-grained soil were noted to decrease with increasing initial consolidation stress.

1 INTRODUCTION

It has been well established that the relative density and effective confining stress are the two prominent factors that govern the shear stress-strain response of sands. As such, the concepts of critical state soil mechanics, which is based upon void ratio and effective confining stress, are commonly adopted in characterizing sand behavior (Schofield and Wroth, 1968). For example, dilative response (tendency to increase volume during shear loading) or contractive response (tendency to decrease volume during shear loading) of sands have been observed to be dependent on the combination of the confining stress and void ratio prevalent during monotonic shear loading. However, factors such as fabric, anisotropy, mineralogy, particle shape, etc., could significantly influence the shear response of soils including fine-grained material such as silt and clay.

Early works by Ladd (1964) observing stress-history-normalizability of clay from isotropically consolidated undrained triaxial tests are well known. Fleming and Duncan (1990) demonstrated that the undrained strength of low-plasticity Alaskan silts can be normalized to consolidation stress with relatively small variations from triaxial tests results. Further, the disturbance of the soil fabric, breaking of cementation, and other inter-particle bonds that give rise to microstructure during the application of loads on fine-grained soil is termed as ‘destructuration’ by Hight and Leroueil (2003) and Ladd and Degroot (2003). The alterations of the soil response to shearing based on the degree of destructuration have been previously noted for clay by Leroueil et al. (1979), Santagata and Germaine (2002), Lunne et al. (2006), and Zapata-Medina et al. (2014). In contrast to the extensive research reported on sand and clay, only limited amount of studies have been undertaken to investigate the shear loading response of silt (Bray and Sancio (2006); Donahue et al. (2007); Guo and Prakash (1999); Hyde et al. (2006); Prakash and Sandoval (1992); Sanin and Wijewickreme (2006).

With this background, a systematic laboratory research program has been undertaken at the University of British Columbia (UBC) for further study of mechanical behavior of different types of natural fine-grained soils. As a part of this program, a series of laboratory tests were performed on natural fine-grained soil samples obtained from three different locations of the Fraser River Delta in Lower Mainland of British Columbia (BC), Canada.

In particular the results of the laboratory testing program carried out on relatively undisturbed natural fine-grained soil using constant volume Direct Simple Shear (DSS) apparatus are presented this paper. The effects of confining stress and over-consolidation ratio (OCR) on monotonic response of these natural soils evaluated based on the data derived from the experimental work is presented.

2 LABORATORY TESTING DETAILS

2.1 *Soil tested*

The soil tested in this study originates from three different locations of the Lower Mainland of BC, Canada. 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 deposits of the Fraser River Delta are Holocene in age and have a maximum known thickness of 305 m (Clague et al. 1996). The subject sites#1, #2 and #3(as shown in Fig. 1) are located on south bank of the North Arm of Fraser River, south bank of NicomeklRiver and south bank of Fraser River respectively.

Fig. 1 Locations of subject sites in the Lower Mainland BC over-layer with Map data ©2014 Google
Extracted from: https://www.google.ca/maps/@49.1950801,-122.96154,11z

Cone Penetration Tests (CPTs) were performed at these sites (near the planned locations of sampling) prior to the sample collection; the interpreted soil behavior type based on these CPT data suggested the presence of silt and sandy silt in site #1, silt in site #2, and clayey silt and clay in site #3. Saturated samples from depth horizons below the groundwater tables at those site locations were retrieved, and the index properties obtained from laboratory tests are listed in Table 1.Grain size distribution obtained from the sieve analysis and hydrometer analysis for the soil from the above-mentioned sites are shown in Fig. 2. From the results of index tests and grain size distribution analysis, soil from sites #1 and #2 can be classified as silt with sand whereas soil from site #3 can be described as relatively high plastic clay. Estimated preconsolidation stresses for samples from the three sites reveal that soil from the site #1 was slightly over-consolidated whereas soil from site #2 and #3 were normally consolidated.

Table 1 Index properties of the tested soils

Parameter	Value(s)		
	Site		
	#1	#2	#3
Depth level (m)	5~7	4.2~5.5	4.9~6.2
In-situ water content (%)	35~44	38~53	58~69
Preconsolidation 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

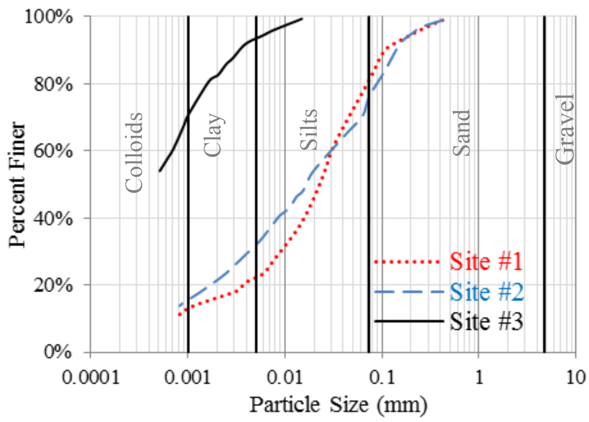

Fig. 2 Grain size distribution results of soil samples from subject sites

2.2 *Soil sample collection*

Relatively undisturbed soil samples were retrieved from test holes put down at the sites using conventional mud-rotary drilling. For this, specially fabricated stainless-steel tubes having an outer diameter of 76.2 mm, with no inside clearance, sharpened 5^0 beveled cutting edge, and 1.4-mm wall thickness were used. The retrieved thin-walled samples were then sealed with rubber expansion plugs and waxed on the ends to preserve the natural water content. The samples were stored in vertical position in a moisture controlled room in the Geotechnical Laboratory of the UBC until extrusion and DSS testing.

2.3 *Specimen preparation*

DSS device at UBC is a modified NGI-type (Norwegian Geotechnical Institute type) device (Bjerrum and Landva, 1966) that accommodates a specimen with a diameter of about 70 mm and a height of about 20 mm placed in a wire-reinforced rubber membrane. The soil specimens for testing were produced by pushing a sharpened-edge polished stainless steel ring vertically downwards on to the extruded soil samples from the thin-walled sample tubes that had a slightly larger diameter of ~73 mm. Then, the top and the bottom sides of the specimens were 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 was then carefully placed in the wire-reinforced membrane that laterally confines and enforces an essentially constant cross-sectional area and prevents the specimen from localized lateral straining during consolidation and shear deformation.

2.4 *Constant-volume Direct Simple Shear testing approach*

A constant-volume condition can be enforced by clamping the top and bottom loading platens of the specimen 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 (Dyvik et al. 1987). 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.

After consolidation to the desired vertical effective consolidation stress (σ'_{vc}), the test specimens were subjected to constant-volume monotonicshear loading. The application of monotonic shear load was controlled by a constant strain rate of about 10 % per hour.

2.5 *Test program*

A series of DSS tests were performed on the different types of natural soils retrieved as per above to investigate the monotonic shear response with respect to confining stress and OCR. Table 2 summarizes the tested soil types, water content and initial and post consolidation void ratios of test specimens, test parameters such as σ'_{vc} and OCR. The natural soils from each site were tested to simulate the following three stress history conditions: normally consolidated stress state (OCR =1), OCR of 2 and 4. In the case of OCR = 1, several tests were conducted at different magnitude of consolidation stresses to study the effect of confining stress to the monotonic shear loading response. The estimated preconsolidation stresses for soil specimens from sites #1, #2 and #3 are about 110 kPa, 40 kPa, and 80 kPa, respectively (Table 1); the test specimens were initially consolidated to higher consolidation stresses than these preconsolidation stresses and estimated in-situ stresses in order to make sure that they were in normally consolidated stress state. To investigate the influence due to OCR, specimens from site #1, #2 and #3 were tested at consolidation stress of about 150 kPa, 100 kPa, and 100 kPa respectively after mechanically consolidating to OCR values of 2 and 4.

Table 2. Test Program: Summary of test parameters

Tested Soil	Test ID	WC %	e_i	σ'_{vc}(kPa)	e_c	OCR
Site #1 (PI = 5, Natural silt with sand)	S1-150	44	1.12	147.04	0.99	1
	S1-300	41	1.07	297.20	0.87	1
	S1-600	34	1.02	599.15	0.76	1
	S1-150-2	38	1.02	154.54	0.86	2
	S1-150-4	35	0.95	146.90	0.70	4
Site #2 (PI = 7, Natural silt with sand)	S2-100	47	1.28	101.18	1.06	1
	S2-200	52	1.45	199.88	1.06	1
	S2-400	41	1.21	396.02	0.91	1
	S2-100-2	38	1.12	98.95	0.92	2
	S2-100-4	41	0.99	103.77	0.80	4
Site #3 (PI = 34, Natural clay)	S3-100	69	1.85	102.04	1.77	1
	S3-150	68	1.90	149.60	1.78	1
	S3-200	69	1.97	202.05	1.76	1
	S3-400	63	1.70	398.34	1.14	1
	S3-100-2	66	1.82	104.69	1.65	2
	S3-100-4	68	1.68	104.08	1.16	4

WC : Water content (measured at laboratory, prior to DSS testing)
σ'_{vc} : Vertical effective stress prior to monotonic shear loading
e_i : Initial void ratio (calculated using specific gravity)
e_c : Void ratio after consolidation and before shear loading (calculated using specific gravity)

3 MONOTONIC SHEAR LOADING RESPONSE

The shear stress-strain response of natural soils retrieved from Sites #1, #2 and #3 (that were initially normally consolidated under different consolidation stress levels) observed during the constant-volume monotonic shearing tests are presented in Fig. 3(A).As may be noted, the shear resistance initially increases as the shear strain increases until a maximum shear stress is reached, and then that would remain at almost same magnitude, with the exception of decreases observed at larger strain level during some of the tests. Corresponding excess pore-water pressure development curves are shown in Fig. 3(B). The excess pore-water pressure (Δu) due to shearing is normalized by $\sigma'_{vc}$to define excess pore-water pressure ratio (r_u).Gradual development of excess pore-water pressure can be noted with increasing shear strain. Stress path [Fig. 4(A)] and normalized stress path [Fig. 4(B)] indicated that specimens have deformed in a contractive manner.

The shear stress-strain response and excess pore-water pressure developments of and in natural soils retrieved from Site #1, #2, and #3 (that were subjected to stress state with OCR of 1, 2 and 4 prior to monotonic shearing tests) are presented in Fig. 5(A) and (B). Normalized stress paths that depict the response of over-consolidated specimens are shown in Fig. 6.

3.1 *Effect of confining stress*

When specimens are subjected to higher level of consolidation stresses, their post consolidation void ratios (e_c) become lesser as listed in the Table 1. Having a denser particle arrangement would cause increased shear resistance, and it is observable in Fig. 3(A) and Fig. 4(A)where higher shear resistances are mobilized for the specimens from Site #1 and #2, sheared at higher confining stress levels. From the response of r_u for the test specimens from Site #1 shown in Fig. 3(B), development of Δu seems to be continuous and gradual until it has reached a value of about r_u = 50%. Behavior of natural silt from Site #1are found to be stress-history-normalizable as normalized stress path of those presented in Fig. 4(B) tends to fall within a narrow range.

Similar to the response of test specimens from Site #1, shear stress–strain characteristics and stress paths with contractive response can be observed for Site #2. Further, it can be seen from Fig. 4(B) thatmobilized normalized shear stresses at large strain levels for test specimens from Site #1 are similar to those from Site #2.

However, the monotonic shear response observed for the test specimens from Site #3 was found to be different from those observed for Sites #1 and #2. The shear resistance observed for test specimens from Site #3initially increased with increasing shear strain until a peak is reached, then followed by a decrease in shear resistance with further increase in strain level [Fig. 3(A)]. As may be notable from the stress paths in Fig. 4(A), the normally consolidated, high plastic soil specimens from Site #3 deformed in a contractive manner during the monotonic shear loading. It is of interest to note that, unlike the normalized stress path of tests for the specimens from Site #1 and #2, the normalized stress paths for tests for the specimens from Site #3 did not exhibit coincidence as presented in Fig. 4(B). One-dimensional consolidation tests conducted for the soil specimen from Site #3 revealed that when the consolidation stress is increased beyond the estimated preconsolidation stress, significant amount of vertical strain develops that results in notable reduction of void ratio;this observation, suggests possible 'destructuration' in the soil specimen as the confining stress increases, such as that described by Leroueil et al. (1979). In other words, when consolidated to stress levels above the preconsolidation stress level, destructuration of the fabric of the soil specimen would occur, and in turn, the material would behave in a comparatively weaker manner during monotonic shear loading.

3.2 *Effect of OCR*

As a result of higher OCR, the final void ratios of the over-consolidated specimens are lesser, compared to the normally consolidated specimens as indicated in Table 2. The tests performed on silt from Site #1,indicated increase in shear resistance with increasing OCR. The over-consolidated specimens showed negative pore-water pressure development during shear; as shown in Fig. 5(B), greater negative pore-water pressure development during shearing was noted for that are at a higher OCR. Sanin (2010),Seidalinova and Wijewickreme (2013) have also observed similar stress-strain response and trends in development of negative pore-water pressure for low-plastic Fraser River silt and gold mine tailings during DSS tests while Wang and Luna (2011) also presented similar observations for Mississippi river valley silt during triaxial compression tests for over-consolidated specimens.

Normalized stress path responses shown in Fig. 6 indicate contractive deformation of the normally consolidated specimen as opposed to the initial dilative deformation followed by contractive deformation of over-consolidated specimens. The shear resistance is observed to increase with increasing OCR.

Stress-strain, excess pore water pressure and stress path responses observed for the silt from Site #2 are similar to those of Site #1. The results from the monotonic shear testing of relatively high plastic soil from the Site #3 also displayed increase in shear resistance with increasing OCR despite the possibility that the soil fabric would have destructured when the specimens were subjected to higher stresses than the preconsolidation stress during the mechanically over-consolidation process.

Furthermore, the normalized stress paths shown in Fig. 6, indicate that over-consolidated relatively high plastic soil from Site #3 are significantly more dilative in response (increment in effective vertical stress due to shearing) than those from the Sites #1 and #2.

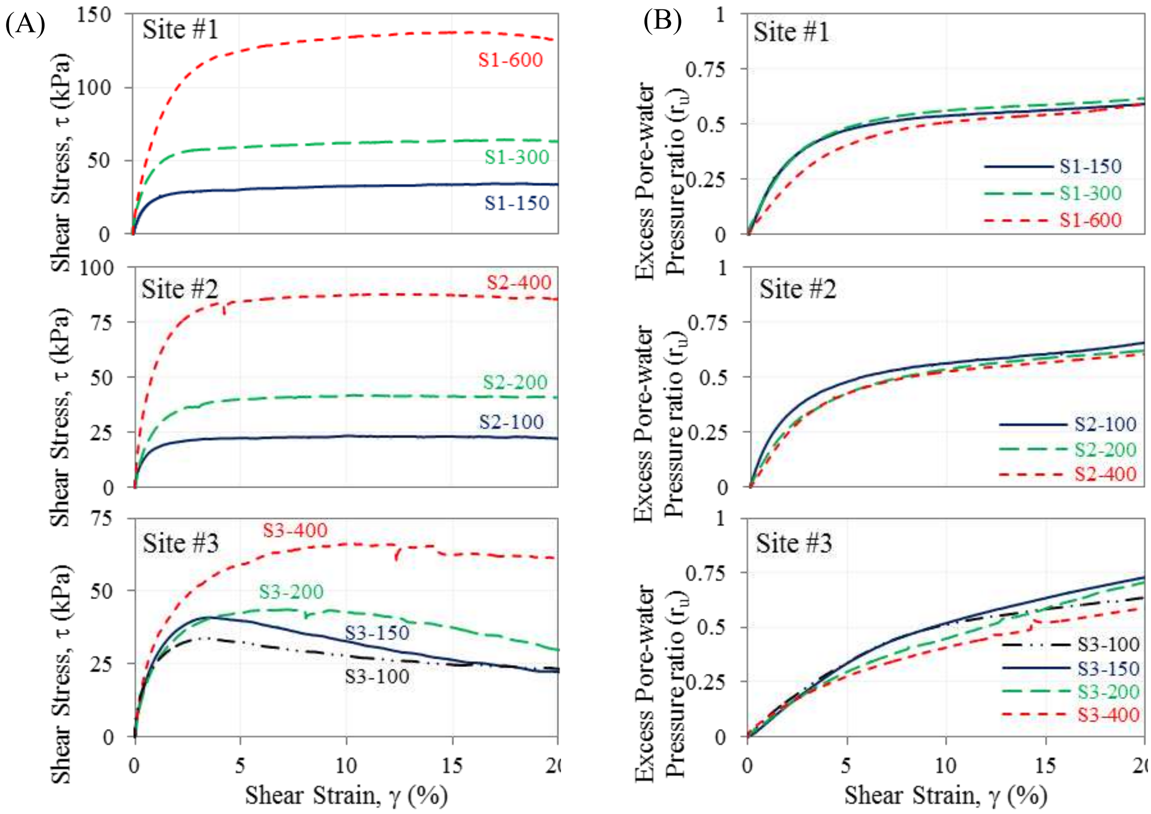

Fig. 3 Shear stress-strain curves (A) and excess pore water pressure development curves (B) of constant-volume monotonic DSS tests on relatively undisturbed specimens of natural silt retrieved from the Site #1, #2 and #3 at normally consolidated stress state at varying confining stress levels

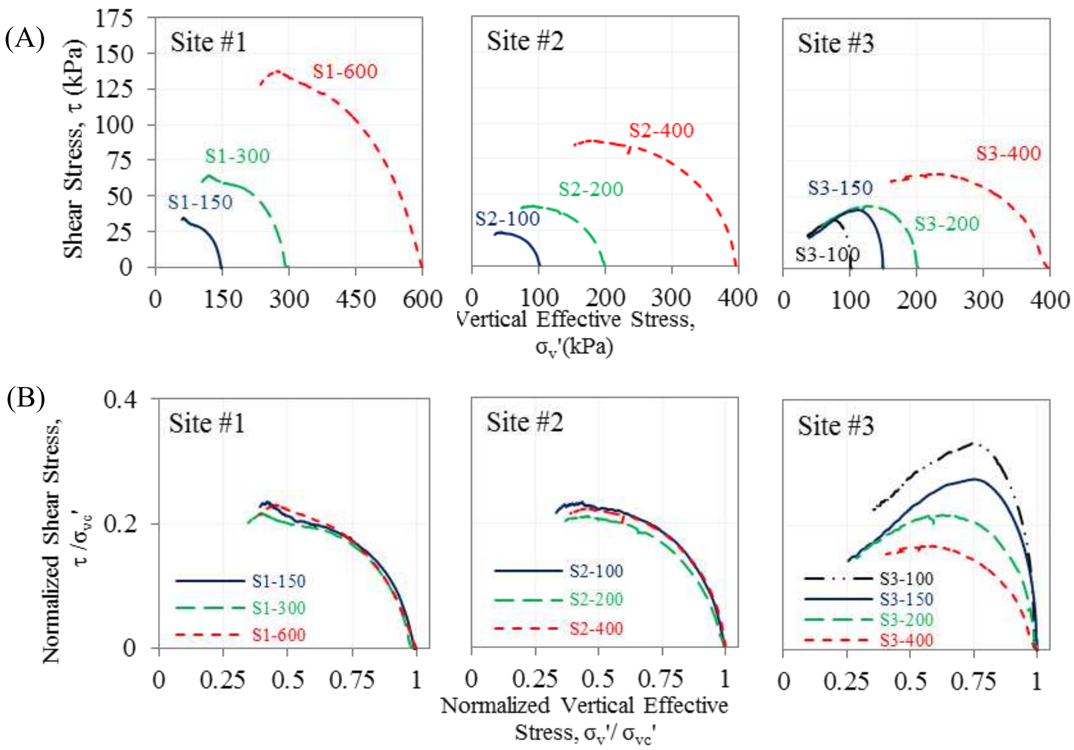

Fig. 4 Stress path curves (A) and normalized stress path curves (B) of constant-volume monotonic DSS tests on relatively undisturbed specimens of natural silt retrieved from the Site #, #2 and #3 at normally consolidated stress state at varying confining stress levels

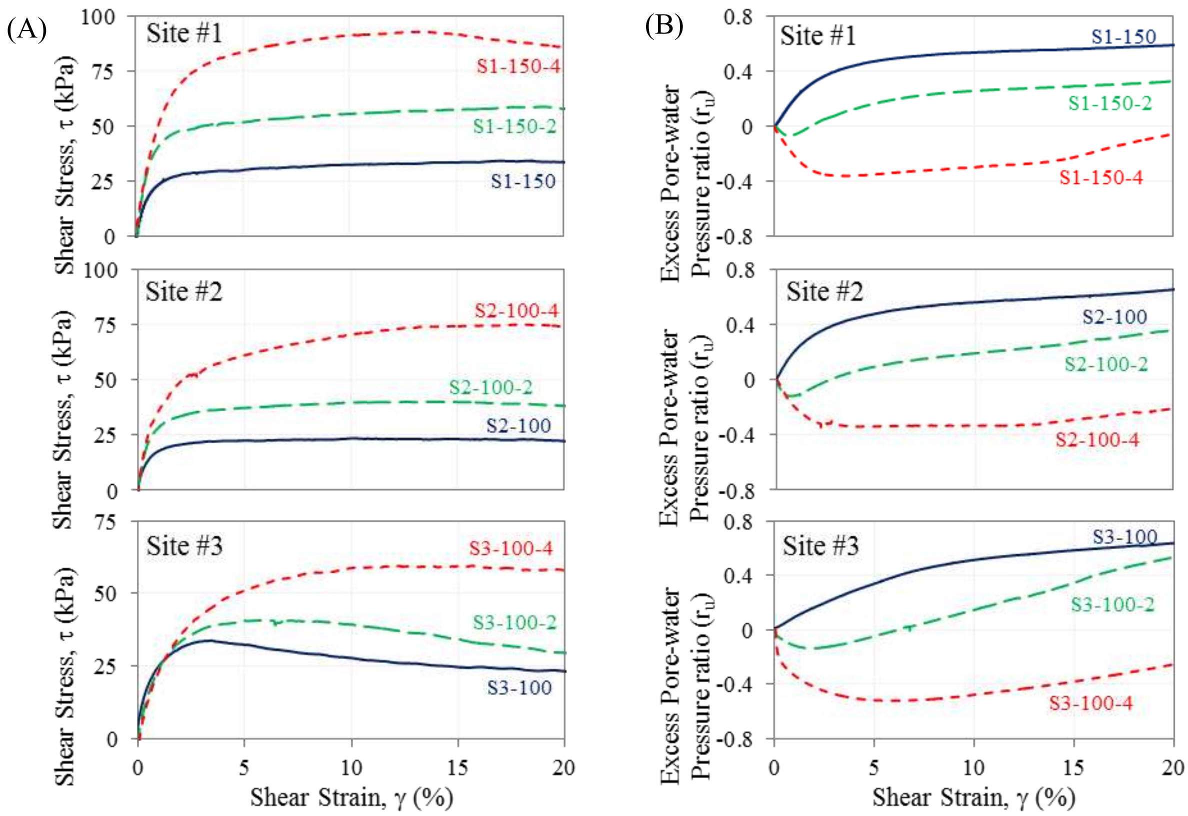

Fig. 5 Shear stress-strain curves (A)and excess pore water pressure development curves (B) of constant-volume monotonic DSS tests on relatively undisturbed specimens of natural silt retrieved from the Site #1, #2 and #3 at confining stress levels of OCR 1, 2 and 4

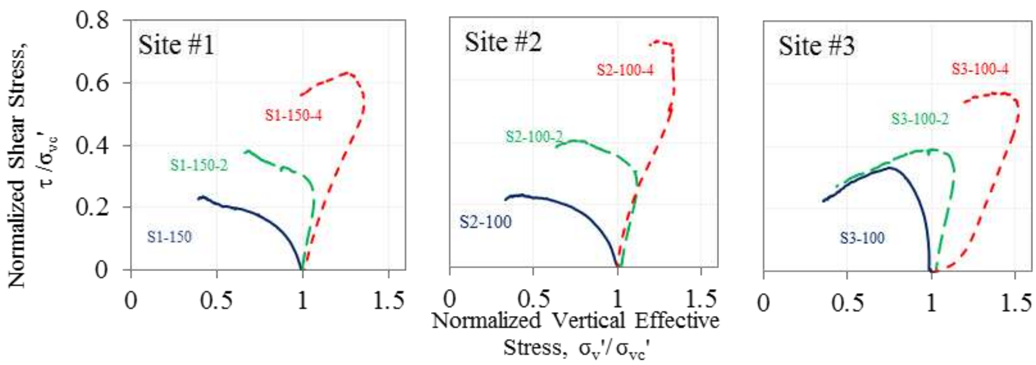

Fig. 6 Normalized stress path curves of constant-volume monotonic DSS tests on relatively undisturbed specimens of natural silt retrieved from the Site #1, #2 and #3 at confining stress levels of OCR 1, 2 and 4

4 SUMMARY AND CONCLUSIONS

The monotonic shear loading response of natural soils obtained from three different sites (i.e., Sites #1, #2 and #3) located in the Lower Mainland of British Columbia, Canada, was investigated using constant-volume DSS testing. The soil obtained from Sites #1, #2 and #3 found to be fine-grained material having plasticity index of 5, 7, and 34 respectively. All test specimens exhibited contractive response during shear when they were tested under initially normally consolidated stress conditions. The constant-volume DSS tests conducted on relatively low plastic silt specimens from Sites #1 and #2, when initially consolidated to stress levels at, or above, the estimated in situ vertical stress displayed monotonic shear responses that are stress-history-normalizable (i.e., normalized stress path and normalized shear strength observed for the normally consolidated silts tested at different consolidation stressestends to fall within a narrow range). In other words, the effect of confining stress on the normalized shear strength was found to be not significant. However, the relatively high plastic fine-grained soil from Site #3 showed that the monotonic shear response and shear strength are significantly influenced by the confining stress. When the relatively high plastic soils were tested at higher confining stresses than the estimated in-situ and preconsolidation stress levels, comparatively weaker response were noted. It appears that possible destructuration occurred when soil from Site #3 is subjected to initial consolidation stress levels exceeding the field preconsolidation stress.The testsconducted on specimens from all three sites indicated that the shear resistance would increase with increasing OCR. As expected, a dilative response during shear loading was observed for over-consolidated specimens. The over-consolidated specimens from Site #3 with higher plasticity resulted in significantly more dilatancy during shear than those observed from the over-consolidated soils from the other two Sites #1 and #2.

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