

EFFECT OF PLASTICITY ON THE LABORATORY CYCLIC SHEAR RESPONSE OF FINE-GRAINED SOILS

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ABSTRACT

The mechanical response of four fine-grained soils having different plasticity is compared using laboratory data from constant volume cyclic direct simple shear (DSS) tests. The data used in the comparison have been derived from specimens prepared from field samples and tested using the same direct simple shear device. Under cyclic DSS loading, fine-grained soils typically exhibit cumulative decrease in effective stress along with progressive degradation of shear stiffness. This “cyclic mobility type” strain development was observed in all of the tests throughout the cyclic loading process regardless of the differences in the plasticity index of the tested materials. The observed behaviour suggests that relatively low-plastic silts are unlikely to experience flow failure under undrained cyclic loading. The build-up of equivalent excess pore water pressure with the increasing number of cycles could still result in significant cyclic shear strains even at moderate levels of cyclic loading given the potential to experience a reasonable number of load cycles. Comparison of the observed cyclic resistance ratio (CRR) for the different materials reveals that the value of CRR increases with increasing plasticity of the soil, supporting previous experimental findings based mainly on data from triaxial tests on re-constituted samples.

Keywords: liquefaction, seismic response, fine-grained soils, direct simple shear testing.

INTRODUCTION

Liquefaction aspects of saturated sands have been the topic of extensive research during the past 35 years, while the behaviour of saturated silty sands and silts with respect to this topic has been studied only on a very limited scale. Saturated fine-grained soils have been noted to be susceptible to earthquake-induced softening and strength reduction as much as relatively clean sands (Boulanger et al. 1998, Bray et al. 2004). Fine-grained soils are commonly found in natural river deposits, and they also originate as a man-made waste product in tailings derived from the processing of ore in the mining industry. There is significant controversy regarding the currently available approaches to assess the earthquake response of fine-grained soils (Youd et al., 2001; Seed et al., 2001), and the current practice relies on empirical criteria for the liquefaction assessment of these soils. As noted during the special session on liquefaction at the 8th NCEE (2006), the commonly used Chinese Criteria (Wang, 1979; Marcuson et al. 1990) to evaluate the liquefaction susceptibility of fine-grained soils is now recommended as unacceptable. Other alternate empirical criteria in this regard have also been proposed. Bray et al. (2004), based on data from the Kocaeli earthquake, have suggested that the presence of active soil minerals in a silt would govern the liquefaction susceptibility, and it could be better delineated based on w_c/LL vs. PI where w_c = water content; LL =Liquid Limit; and PI =Plasticity Index. Boulanger and Idriss (2004) have proposed that soil would behave “sand like” if $PI < 7$, and “clay-like” if $PI > 7$. They essentially suggest that the liquefaction of low-plastic silts can be assessed

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using the methods available for sands. These approaches are relatively new, and there is recognition of the strong need to further understand the cyclic response of silts.

The response of a given soil to cyclic loading is controlled by many parameters such as packing density, microstructure, fabric, level and duration of cyclic loading, confining stress, initial static bias, etc. These parameters have been noted to primarily govern development of excess pore water pressures, stiffness, and strength in a soil mass during the occurrence of an earthquake, in turn, controlling the overall seismic response. While a reasonable amount of research has been undertaken, or is in progress, to understand the performance of natural silty soils (Thevanayagam et al. 2002; Boulanger et al. 1998; Bray et al. 2004; Sanin and Wijewickreme 2006), the available published information on the cyclic shear response of fine-grained soils is limited. Clearly, the current understanding of the behaviour of silts is limited particularly due to the lack of laboratory element test data on the cyclic performance of silts. As such, there is a strong need to conduct laboratory tests on specimens prepared from good quality field samples, preferably using test devices that account for the cyclic rotation of principal stresses that take place during earthquake loading.

With this background, a laboratory research program has been undertaken at the University of British Columbia (UBC) to study the mechanical response of fine-grained soils considering a spectrum of materials ranging from natural silts to mine tailings. This paper presents the effect of soil plasticity on the cyclic shear response from some of the testing conducted as a part of this study.

EXPERIMENTAL DATABASE

The primary database used for this study comprises the results obtained from a series of constant volume cyclic direct simple shear (DSS) tests conducted on fine-grained soils at the geotechnical research laboratory of the University of British Columbia (UBC). Cyclic direct simple shear (DSS) tests are considered to be representative of field conditions during earthquake loading. While other advanced apparatus such as hollow cylinder torsional (HCT) shear device may be considered to simulate seismic loadings, these devices are less attractive due to their lack of availability, experimental complexities, and the associated high costs.

The device used for DSS testing is NGI-type (Bjerrum and Landva 1966), and it allows the testing of a specimen having a diameter of ~70 mm and height of 20 to 25 mm. In DSS tests, as an alternative to suspending the drainage of a saturated specimen, a constant volume condition can be enforced even in a dry soil by constraining the specimen boundaries (diameter and height) against changes. The specimen diameter is constrained against lateral strain using a steel-wire reinforced rubber membrane, and the height constraint is obtained by clamping the top and bottom loading caps against vertical movement. 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 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 (Finn et al. 1978; Dyvik et al. 1987).

The DSS tests had been performed on specimens prepared from field tube samples of two types of natural silts and two types of fine-grained tailings (i.e., Fraser River Delta silt, Kitimat clay, laterite tailings and copper-gold-zinc tailings). The tested soil material type and corresponding material parameters derived from index testing and grain size analyses are summarized in Table 1. Fraser River silt originates from a relatively young, uniform channel-fill silt deposit located within the upper part of the Fraser River Delta of the Province British Columbia, Canada. Kitimat clay is a material of low to medium sensitivity, and it is understood to have been deposited in a marine environment. As described by Wijewickreme et al. (2005a), the copper-gold-zinc tailings were obtained using traditional ore processing methods, and the laterite tailings had been obtained from the processing of limonite ore by a pressure acid leach process. This process involves the solution of nickel and cobalt by sulphuric acid at high temperature and pressure, as described by Chalkley and Toirac (1997). As may be noted from Table 1, the plasticity index (PI) of the four materials considered herein span

between $PI = 2$ for the copper-gold-zinc tailings and $PI = 17$ for the Kitimat clay, in turn, providing reasonable coverage of materials within the region of low plasticity.

Table 1. Summary of Index Properties of Tested Fine-grained Materials

	No. of tests	G_s	Sand %	Silt %	Clay %	LL	PI	$S_u/\sigma'_{vc}{}^\#$	$S_u/\sigma'_{vc}{}^*$	Nominal σ'_{vc} (kPa)	e_i (range)	e_c (range)
Fraser River Silt	5	2.69	10	80	10	31	4	0.30	0.30	100	0.97 - 1.04	0.88 - 0.99
Kitimat Clay	3	2.70	1	54	45	37	17	0.31	0.26	80	0.97 - 1.01	0.89 - 0.96
Laterite Tailings	5	4.10	0	65	35	34	12	n.a.	n.a.	100	1.46 - 1.59	1.32 - 1.45
Copper-Gold-Zinc Tailings	20	3.4 - 4.4	0 - 43	57 - 100	0	19	2	n.a.	n.a.	115 - 460	0.59 - 1.72	0.49 - 1.43

n.a.= Not available; # and * = Values of S_u/σ'_{vc} at the peak and 15% shear strain, respectively, from monotonic tests on undisturbed samples

Sample Disturbance

The two natural silts discussed herein had been obtained using fixed-piston tube sampling conducted in a conventional mud-rotary drill hole. A specially fabricated ~75-mm diameter, 0.9-m long tubes (with no inside clearance, a 5-degree cutting edge, and 1.5 mm wall thickness) were used for the sampling. As noted by Leroueil and Hight (2003), piston sampling using thin, sharp-edged tubes offers a suitable and acceptable means of obtaining relatively undisturbed samples of fine-grained soils. Based on the degree of sample disturbance assessed by measuring the void ratio changes during reconsolidation, Sanin and Wijewickreme (2006) have noted that this method could be used to obtain good quality specimens, and, therefore, suitable for the intended laboratory research work. For the two tailings types, the field samples for DSS testing had been obtained with standard stainless steel Shelby tubes. Shelby tube sampling, in spite of its potential for a relatively higher degree of sample disturbance, is still used as the common sampling tool in geographically distant mine site locations. All the samples used in the tests reported herein had been shipped after securing in light-weight packing (foam-based) material with specific instructions to minimize imparting disturbance.

The test specimens had been initially consolidated to a desired vertical effective stress (σ'_{vc}) prior to cyclic loading as indicated in Table 1. For a given specimen, the value of σ'_{vc} selected for testing is larger than the estimated in situ overburden effective stress (i.e., normally consolidated); thus, it is possible that this may have eliminated some of the effects of sample disturbance. During the cyclic loading phase, symmetrical sinusoidal shear pulses were applied at constant cyclic stress ratio τ_{cy}/σ'_{vc} amplitudes, at a frequency of 0.1 Hz.

MECHANICAL RESPONSE UNDER CYCLIC LOADING

Cyclic Stress-strain and Stress-path Response

The typical stress path and stress-strain relationships obtained from the DSS tests conducted on the four types of materials tested are given in Figures 1 through 4. All the specimens show predominantly contractive response during the first quarter to half cycles of loading. In subsequent cyclic loadings, the response gradually changed from contractive to dilative (or experienced phase transformation) during the “loading” (or increasing shear stress) phases. The observed contractive response during “unloading” (or decreasing shear stress) phases suggests significant “plastic deformations” especially after phase transformation. With increasing number of load cycles, the specimens experienced a cumulative increase in excess pore water pressure with associated progressive degradation of shear stiffness. The specimen of Kitimat clay (shown in Figure 2), experienced relatively increased level of degradation of shear stiffness compared to the other tests; this is not unexpected since the shearing was

conducted at a stress ratio very close to its monotonic undrained shear strength (S_u). In a given cycle, the shear stiffness experiences its transient minimum when the applied shear stress is close to zero. The observed cyclic mobility type stress-strain response is similar to the undrained (constant volume) cyclic shear responses previously observed from triaxial tests on natural clayey soils (e.g., Zergoun and Vaid 1994) and dense reconstituted sand (e.g., Sriskandakumar 2004; Kammerer et al. 2002; Wijewickreme et al. 2005b).

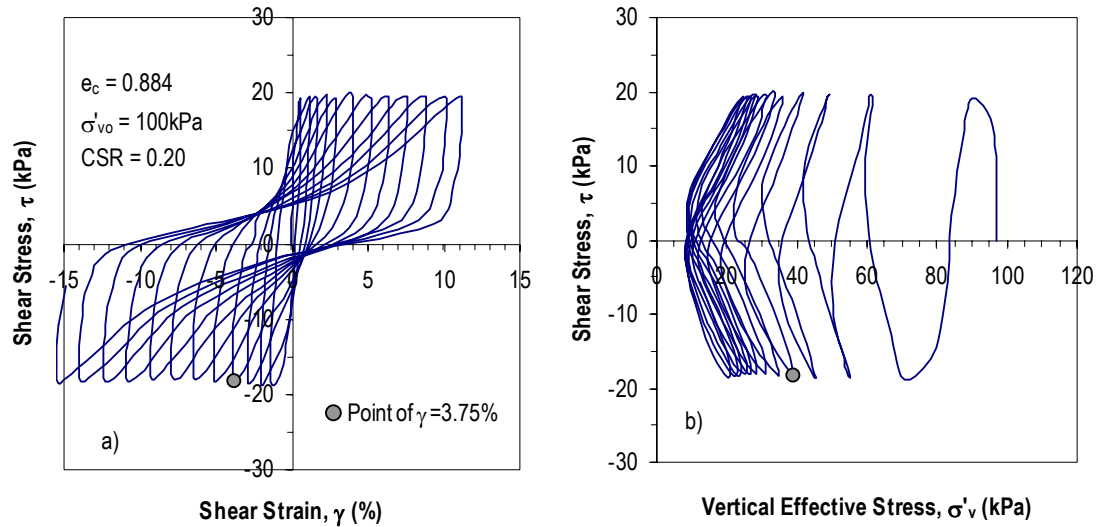


Figure 1. Typical response of Fraser River silt in constant volume cyclic DSS loading: a) stress – strain response; and b) stress path

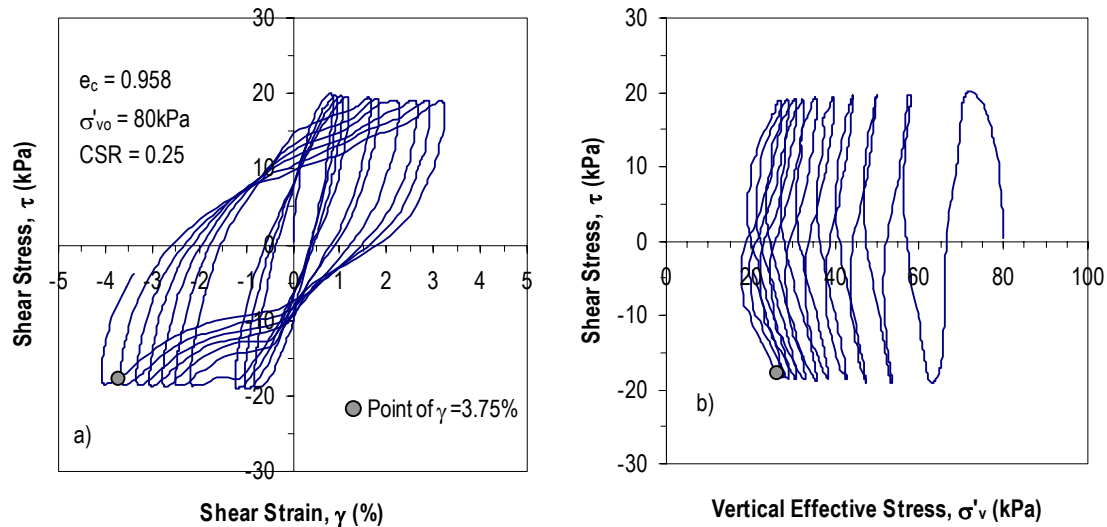


Figure 2. Typical response of Kitimat clay in constant volume cyclic DSS loading: a) Stress – strain response; and b) stress path

The variation of normalized excess pore water pressure ($\Delta u / \sigma'_{vc}$) vs. normalized number of load cycles ($N / N_{\gamma=3.75\%}$), is presented in Figure 5 [Note: N = number of loading cycles; $N_{\gamma=3.75\%}$ = the number of load cycles required to reach a single-amplitude horizontal shear strain $\gamma = 3.75\%$, in a given test. The rationale for using $N_{\gamma=3.75\%}$ is discussed in the next section]. All the tested specimens exhibited gradual increase of Δu with increasing number of loading cycles, the rate of generation of equivalent excess pore water pressure ratio $\Delta u / \sigma'_{vc}$ with N increases with increasing applied CSR. It is

also of interest to note that $(\Delta u / \sigma'_{vc})$ vs. $(N / N_{\gamma=3.75\%})$ seems to follow a narrow band in spite of the significant differences in the plasticity of the tested soils.

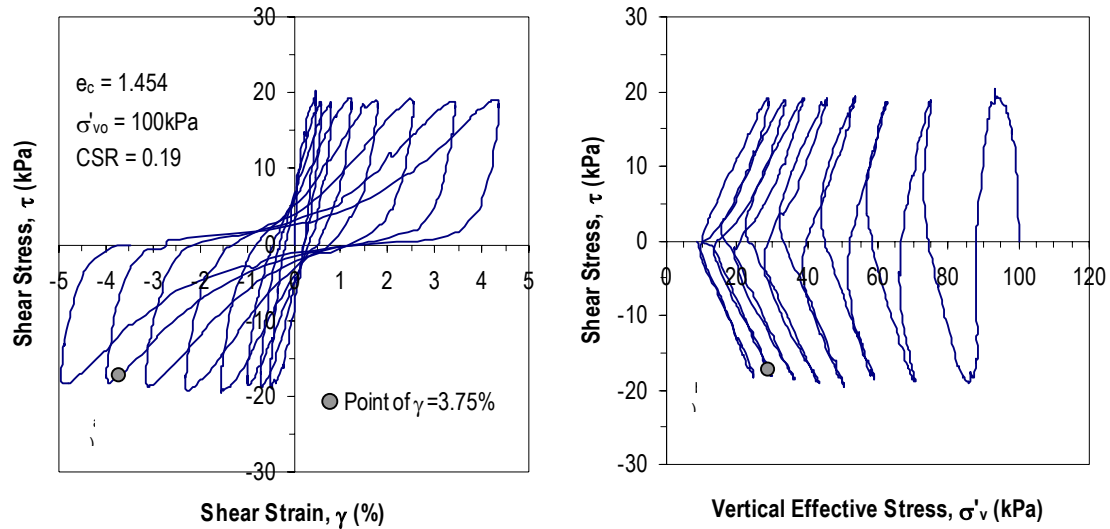


Figure 3. Typical response of Laterite tailings in constant volume cyclic DSS loading: a) Stress – strain response; and b) stress path

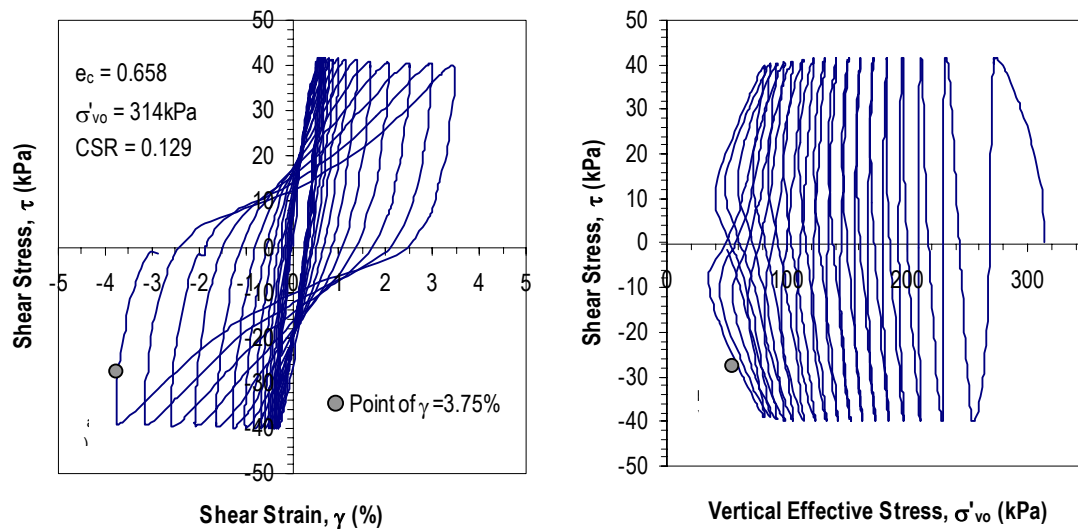


Figure 4. Typical response of Copper-Gold-Zinc tailings in constant volume cyclic DSS loading: a) Stress – strain response; and b) stress path

In an overall sense, “cyclic mobility type” strain development was observed in all of the tests throughout the cyclic loading process although plasticity index of the tested materials were different. Clearly, 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 . In other words, the observed behaviour suggests that the tested silts are unlikely to experience flow failure (i.e., potential for catastrophic failure) under undrained cyclic loading. However, the build-up of equivalent excess pore

water pressure with the increasing number of cycles could result in significant cyclic shear strains even at moderate levels of cyclic loading.

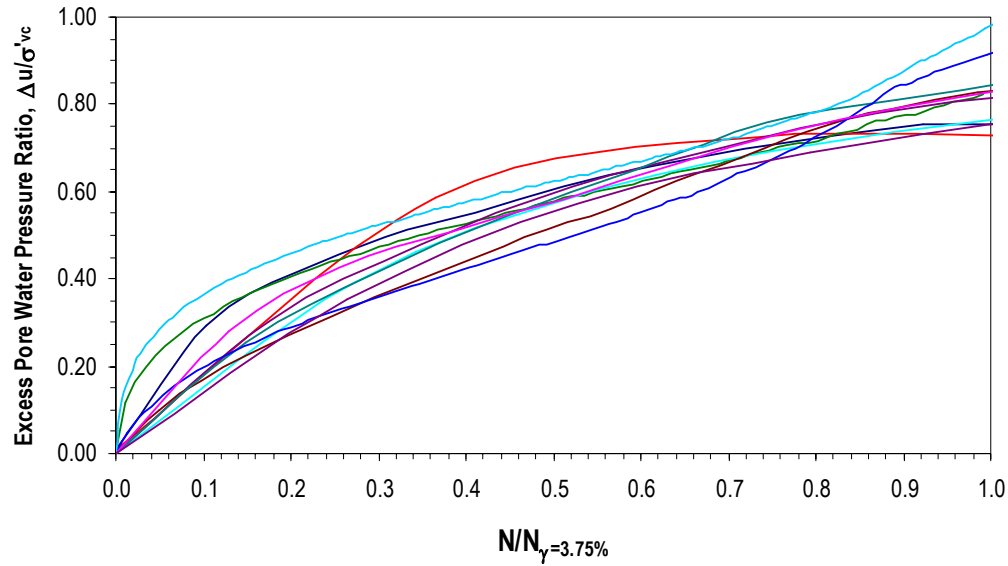


Figure 5. Excess Pore Water Pressure Ratio ($\Delta u/\sigma'_{vc}$) vs. normalized number of load cycles ($N/N_{\gamma=3.75\%}$) for the tested fine-grained materials

Cyclic shear resistance

It is also of interest to examine the cyclic shear resistance by comparing the response observed from DSS testing under different applied cyclic loadings. In order to facilitate this comparison, the number of load cycles required to reach a single-amplitude horizontal shear strain $\gamma = 3.75\%$, in a given constant volume DSS test under a given applied CSR, was defined as $N_{\gamma=3.75\%}$. This $\gamma = 3.75\%$ condition in a DSS specimen is essentially equivalent to reaching a 2.5% single-amplitude axial strain in a triaxial soil specimen. An identical definition has been previously used to assess the cyclic shear resistance of sands by the U.S. National Research Council (NRC 1985), and it also has been adopted in many previous liquefaction studies at UBC.

The variation of cyclic resistance ratio (CRR) vs. $N_{\gamma=3.75\%}$ related to data from the tests on all the material types considered in this paper is presented in Figure 6. Note: As described in Youd et al. (2001), the value of CSR in this interpretation is called CRR since it represents the capacity of the soil to resist cyclic loading. Recognizing that allowance should be made for the expected variability in field samples and experimental scatter, CRR vs. $N_{\gamma=3.75\%}$ relationship for each material can be represented by trend-lines as shown Figure 6. Comparison of the trend-lines reveals that the CRR of the materials seem to generally increase with increasing plasticity of the soil. For each tested material, the value of cyclic resistance ratio to reach $\gamma = 3.75\%$ in 15 cycles (CRR_{15}) was extracted from Figure 6 and then plotted against the corresponding value of PI in Figure 7 to further clarify this observation. Similar trends have been noted by Guo and Prakash (1999) based on review of cyclic triaxial testing data from a number of sources involving testing of undisturbed as well as re-constituted samples and by Ishihara et al. (1981) based on cyclic triaxial testing on undisturbed fine-grained mine tailings. Observation of trends as per Figure 6 and 7 is significant since the data herein have been derived not only from undisturbed samples, but also using the same direct simple shear device that is judged to simulate field loadings during earthquake shaking.

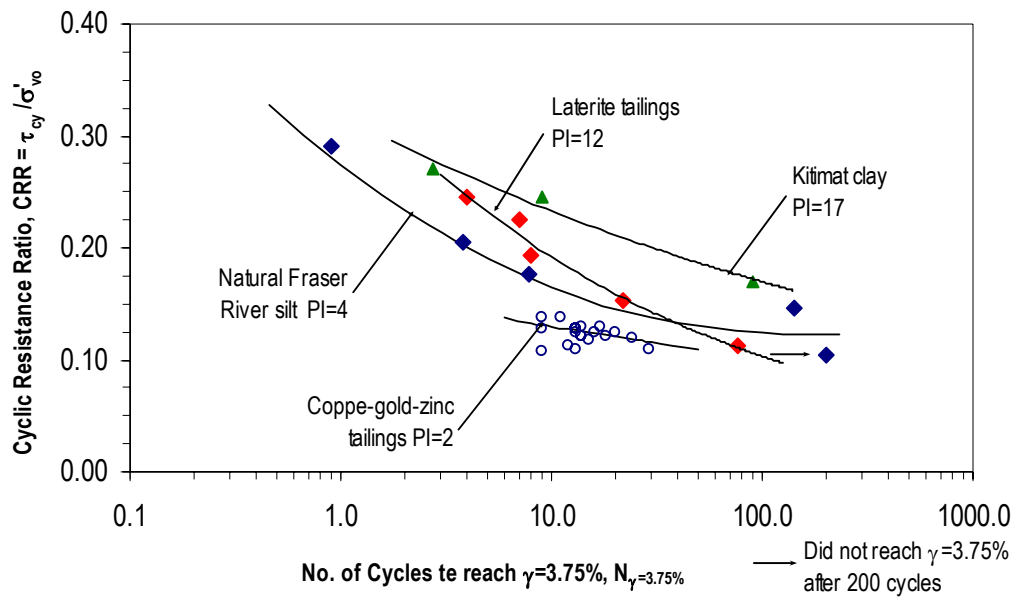


Figure 6. Cyclic resistance ratio vs. number of cycles to reach $\gamma=3.75\%$ for the tested fine-grained materials

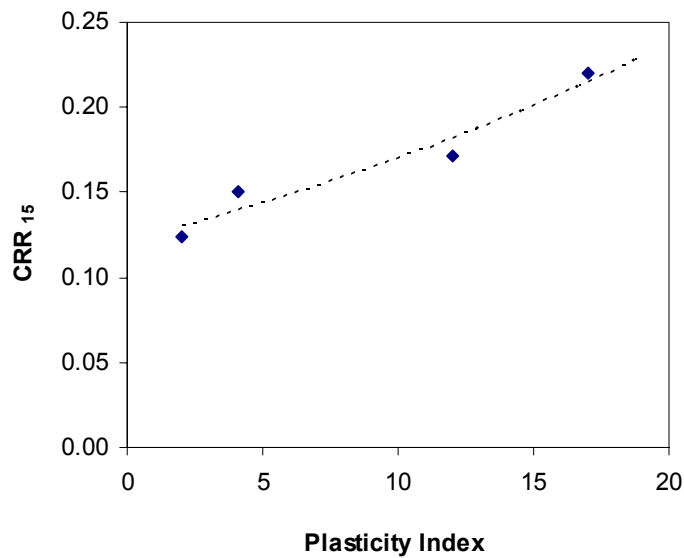


Figure 7. Cyclic resistance ratio to reach $\gamma = 3.75\%$ in 15 cycles (CRR_{15}) vs. Plasticity index (PI) for the tested fine-grained materials

SUMMARY AND CONCLUSIONS

The constant-volume cyclic shear response of four types of fine-grained soils having different levels of plasticity was examined using data from constant volume cyclic direct simple shear (DSS) tests. The intent was to evaluate the effect of soil plasticity on the cyclic shear response while understanding the

individual behaviour of a given soil type. The outcome contributes to enhancing the knowledge on seismic response of fine-grained silts.

Under constant volume cyclic DSS loading, fine-grained soils typically exhibit cumulative decrease in effective stress (or increase in equivalent excess pore water pressure), with increasing number of load cycles, which is also associated with progressive degradation of shear stiffness. The observed cyclic mobility type stress-strain response is conceptually in accord with the responses observed from constant volume cyclic DSS tests on natural silts and dense reconstituted sand.

The cyclic resistance ratio of the materials seems to generally increase with increasing plasticity of the soil. Similar trends have been noted by others based on review of cyclic triaxial testing data from a number of sources involving testing of undisturbed as well as re-constituted samples. Confirmation of these trends herein is significant since the conclusions have been based on data from tests conducted on undisturbed samples, using the same direct simple shear device to test the specimens.

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