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Undrained Monotonic and Cyclic Behaviour of a Silty Sand Stabilized with Colloidal Silica

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ABSTRACT

This paper presents a laboratory investigation into passive site stabilization of liquefiable silty sands by means of colloidal silica, CS. The improvement of the mechanical behaviour was examined, by conducting an extensive testing program comprising monotonic and cyclic triaxial tests performed on a quartz silty sand stabilized with CS. The stabilized specimen preparation method adopted in the tests is initially described and then results from the above tests conducted on treated and untreated specimens are presented. It is indicated that stabilization of a silty sand deposit with CS significantly improves both the undrained monotonic and cyclic resistance strength. Surprisingly, the monotonic undrained shear strength of samples that had been cyclically loaded to a double amplitude axial strain of at least 5%, was comparable to the monotonic undrained shear strength of samples that had not been cyclically loaded.

Introduction

Liquefaction of sandy soils is one of the major causes of damage in earth structures and foundations during earthquakes. Over the last decades widespread liquefaction-induced ground deformation and related damage to foundations has occurred as a result of urban expansion and building on liquefaction prone sites. Thus mitigation and prevention of damage due to liquefaction under existing developed sites is one of the main issues of seismic design. To this extent, the technique of passive site stabilization of liquefiable soil under existing structures has been proposed (Gallagher & Mitchell 2002). This method is based on the use of nanomaterials, such as colloidal silica, laponite and bentonite among others, and involves slow injection of the stabilization nanomaterial into the liquefiable soil by means of natural or augmented groundwater flow.

In particular colloidal silica, CS, is an aqueous suspension of microscopic silica particles produced from saturated solutions of silicic acid, H_4SiO_4 (Iler 1979). In dilute solutions, CS has a density and viscosity similar to water and can be made to gel by adjusting the ionic strength or pH of a given solution. This property allows it to be injected or mixed with soil, so that after gelling colloidal silica blocks the void space in the soil and therefore alters its mechanical behaviour. The principal advantages of CS over other potential stabilizers are its excellent durability characteristics, its initial low viscosity and the ability to attain low permeability in grouted soils, long controllable and reproducible gel times, non-toxicity and its low cost.

Previous studies on passive site stabilization by means of CS in laboratory, concerned mainly

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clean sands (Kabashima & Towhata 2000; Gallagher 2000; Gallagher & Mitchell 2002; Mollamahmutoglu & Yilmaz 2010; Vranna & Tika 2015) and very few (Díaz-Rodriguez et al. 2008) have been conducted on silty sands, mainly because of their lower permeability as compared to clean sands.

With increasing application of passive site stabilization, however, there is need to better understand the behaviour of liquefiable sands that contain fines and are stabilized with CS under different loading conditions, as well as to assess the limits of the applicability of this improvement method to these soils. To this extent, a series of monotonic and cyclic tests was performed on a silty sand, stabilized with CS. The effectiveness of the CS stabilization was investigated by conducting also a series of monotonic and cyclic tests on untreated silty sand specimens. The results from the two series of tests are presented and discussed.

Experimental Procedure

Tested Materials

The soil used in this study is a quartz silty sand with non-plastic fines content of f_c = 10%. It has a specific gravity G_s =2.653, maximum and minimum void ratios of e_{max} = 0.682 (ASTM D4254) and e_{min} = 0.414 (ASTM D4253) respectively, a mean diameter D_{50} = 0.30mm and a uniformity coefficient of C_u = 4.13. Its gradation curve lies within the bound gradation curves, suggested for liquefiable soils. Its permeability was estimated within the range from $0.91 \cdot 10^{-4}$ to $1.33 \cdot 10^{-4}$ m/s.

Ludox SM-30 was selected as the stabilizing agent of specimens, supplied as a 30% by weight silica solution with a viscosity of 5.5cP, a pH of 10 and an average particle size of 7nm. Distilled water was added to the initial solution in order to obtain concentrations of 6% and 10% CS. Gel times of the studied solutions were investigated by conducting viscosity measurement tests by means of a rotating Brookfield viscometer. Figure 1 presents typical test results for CS = 10% solutions with the same pH value and different salinity (Fig. 1a) and vice versa (Fig. 1b). It is noted that gel time was defined as the elapsed time for which the tested solution viscosity is equal to η = 3.5cP. Beyond that value, viscosity increases rapidly and eventually the solution transforms into a rigid gel. It was decided to employ a CS gel time equal to 10 and 11 hours for the CS = 10% and 6% solutions, respectively, which was determined by adjusting the pH value to pH = 6.0, as well as the appropriate NaCl concentration of the solutions.

Testing Programme

Cylindrical specimens (height/diameter ≈ 100mm / 50mm) were prepared at various densities, using the undercompaction method, as proposed by Ladd (1978), both for the untreated and treated silty sand. Saturation was achieved by percolating throughout the specimen, first carbon dioxide gas (CO_2) and then de-aired water. Following, the CS solution was likewise injected into the specimens until it filled the soil voids. The procedure was assumed complete when a solution volume equal to four times the soil specimen volume was extracted from the top of the specimen. The viscosity of the CS solution remained low (η < 3.5cP) throughout the specimen percolation process.

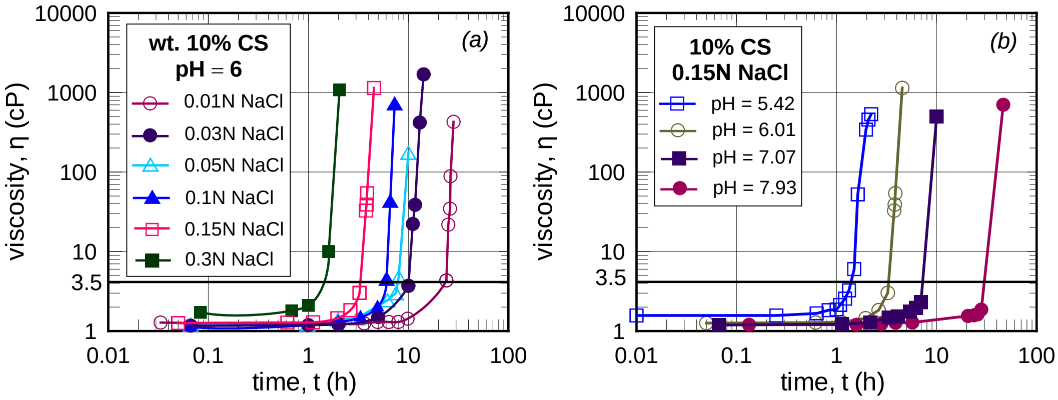

Figure 1. Variation of viscosity, η, with time, t, for CS = 10% solutions with (a) pH = 6.00 and different NaCl concentrations and (b) 0.15N NaCl concentration and different pH values.

After the setting of CS, specimens were placed in a constant temperature and humidity chamber for a curing time of five times the CS gel time (η > 1000cP). Saturation of treated samples prior to testing was not performed, due to the infilling of pore spaces with CS and the possibility of damaging the formed CS bonds. It was assumed, therefore, that total $p = (\sigma_\alpha + 2\sigma_r) / 3$, and effective $p' = (\sigma'_\alpha + 2\sigma'_r) / 3$, confining mean stresses, coincide.

The monotonic testing programme consisted of undrained isotropically consolidated tests on untreated and unconfined compression, as well as undrained consolidated tests on treated silty sand specimens. All types of tests were performed using a closed-loop automatic cyclic triaxial apparatus (M.T.S. Systems Corporation) (Vranna 2015).

In the monotonic tests, specimens after isotropic consolidation under p'_0, were subjected to undrained compression at a constant strain rate of 0.1%/min. In the cyclic triaxial tests, a sinusoidally varying axial stress ($\pm\sigma_d$) was applied at a frequency of f = 0.1Hz, under undrained conditions. In this work, the occurrence of double amplitude axial strain, ε_{DA} = 5% is used as a reference point to define cyclic softening of both treated and untreated specimens. For this reason, a series of cyclic triaxial tests with different cyclic stress ratios, $CSR = \sigma_d / 2p'_0$, was carried out in order to determine the number of load cycles, N, required for the development of ε_{DA} = 5% both for the treated and untreated specimens. In view of the typical number of load cycles of actual earthquakes (10 to 20 for an earthquake of M7.5 magnitude), in this work the onset of liquefaction and thus the cyclic resistance ratio, CRR_{15}, is considered as the cyclic stress ratio, $CSR = \sigma_d / 2p'_0$, required to produce ε_{DA} = 5% in 15 load cycles. Confining stresses of either 100kPa or 300kPa were used in the tests.

Test Results

Monotonic Response

Figure 2 presents the Mohr-Coulomb peak shear strength envelopes of the untreated specimens as well as the specimens treated with CS = 6% and 10%, at a relative density, D_r = 31-37%. It is shown that the increase in strength of the treated specimens over that of the untreated is due to the increase in both friction angle, but most mainly in cohesion. This indicates that introduction of CS into the silty sand induces a cohesion factor by infilling the voids and creating bonds among the silty sand grains.

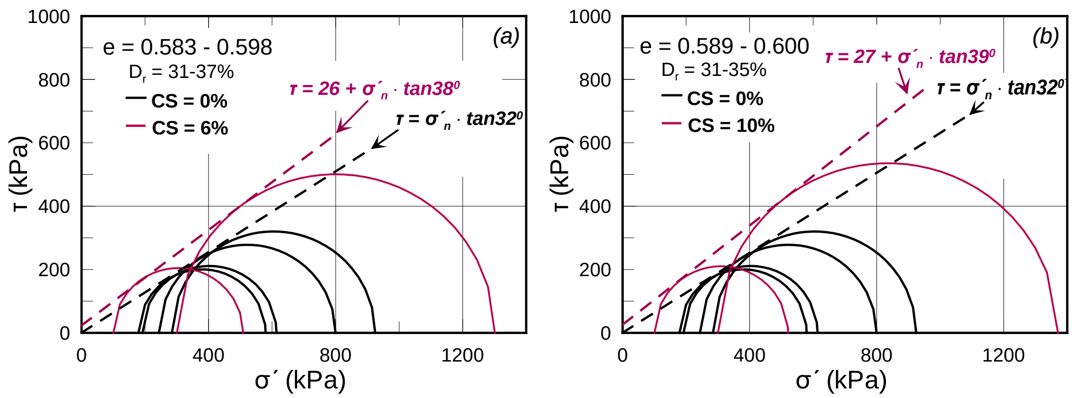

Figure 2. Peak shear strength envelopes for untreated and treated specimens with (a) CS = 6% and (b) CS = 10%.

Figure 3 presents the variation of the undrained shear strength, s_u, with void ratio of untreated and treated with CS = 6% and 10% specimens. The undrained shear strength, $s_{u,max} = q_{u,max} / 2$, of the treated specimens was determined at the point of the maximum deviatoric stress, $q_{u,max}$, which corresponds to an axial strain, ε_α = 2.8 - 16.5%. It is shown that the s_u of treated specimens with either CS = 6% or 10% practically coincides. For comparison reasons, in the case of untreated specimens, the undrained shear strength, s_u at the phase transformation state, $s_{u,PT} = q_{PT}/2$, was used (point of transition from a contractive to a dilative response), since it was mobilized at a comparable range of ε_α = 2.1 - 13.1%. It is noted that the undrained shear strength of the untreated specimens at critical state was determined at larger values of ε_α = 10.4 - 27.9%. It is again shown that the s_u of the treated specimens is considerably larger than the corresponding of the untreated specimens, under the same effective stress. Moreover, the s_u of the treated silty sand increases with increasing confining stress. For the studied density range, it doubles when p'_0 increases from 100kPa to 300kPa.

Cyclic Response

Figure 4 shows the cyclic response of loose untreated and treated specimens, subjected to CSR = 0.30-0.32 under p'_0 = 100kPa. It is indicated that the untreated specimens experience much larger strain in fewer loading cycles, N, than the corresponding treated specimens. Whereas the values of N for ε_{DA} = 1, 2.5 and 5% are very close for the untreated specimen, for the treated specimens with either CS = 6% or 10%, there is a distinct difference between N for ε_{DA} = 2.5 and 5%. For the untreated specimen, as shown in Figure 4b, ε_{DA} increases rapidly and complete liquefaction is reached ($\Delta u/p'_0$ = 100% at ε_{DA} = 4.8%), whereas for the treated specimens, ε_{DA} increases gradually during cyclic loading. It is also indicated that the rate of increase of ε_{DA} with time decreases with increasing CS content. The above pattern of behaviour was also observed at different CSR and densities.

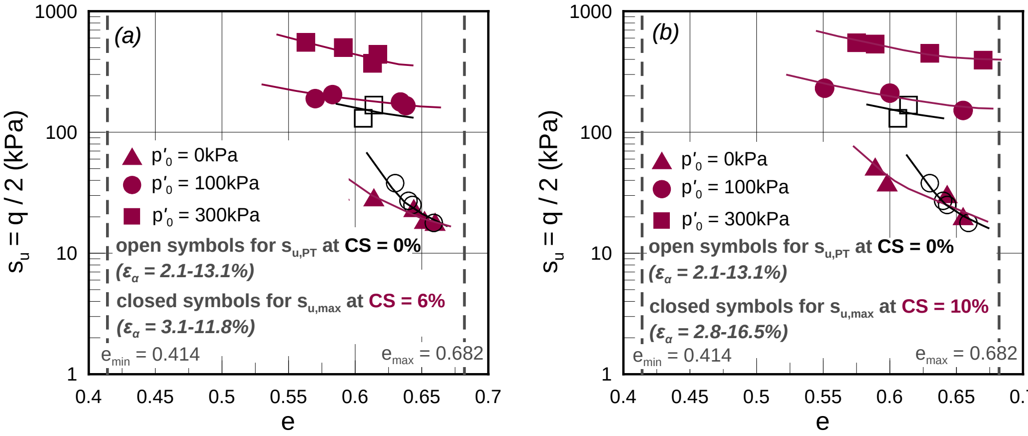

Figure 3. Variation of the undrained shear strength, s_u with void ratio, e for the treated silty sand specimens with CS = 6% (a) and 10% (b), at p'_0 = 0kPa, 100kPa and 300kPa, as well as the untreated specimens, at p'_0 = 100kPa and 300kPa.

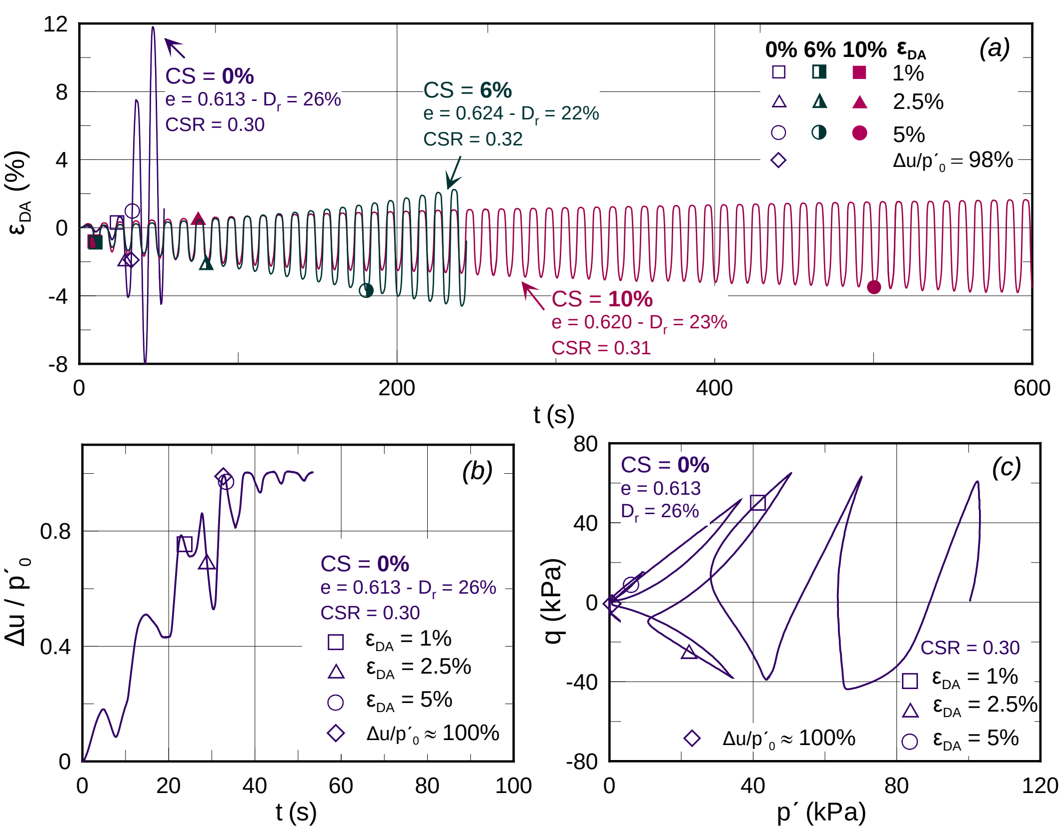

Figure 4. (a) Evolution of double amplitude axial strain, ε_{DA} with time, t, for treated and untreated silty sands, for e = 0.613-0.624 and CSR ≈ 0.31 under p'_0 = 100kPa. Evolution of (b) normalized excess pore water pressure, $\Delta u/p'_0$, with time, t, and (c) variation of deviatoric stress, q, with mean effective stress, p′, for the untreated specimen of Fig. 4a.

Figure 5 presents the variation of cyclic stress ratio, CSR, with N required to reach three levels of ε_{DA} = 1, 2.5 and 5% for the treated specimens with CS = 6% and 10% under p'_0 = 100kPa, at a loose state. It is shown that at relatively high values of CSR, depending on CS content, the N at ε_{DA} = 1% and 2.5% practically coincides, whereas at lower values of CSR, the N value at each ε_{DA} level is distinctive from another. This may indicate that cyclic loading at high CSR values may induce some minor bonding breakage at even ε_{DA} = 1% and therefore acceleration of axial strain.

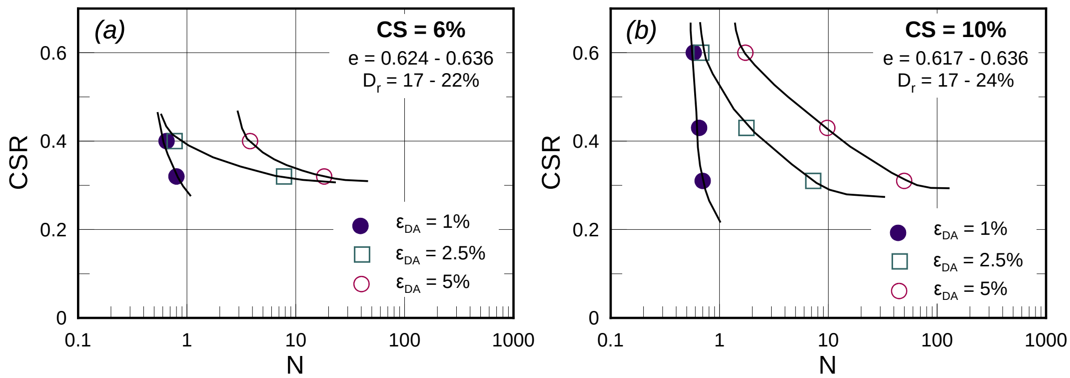

Figure 5. Variation of CSR, with number of cycles N, for treated with (a) CS = 6%, and (b) CS = 10% silty sands, with e = 0.617-0.636 at p'_0 = 100kPa and at different values of ε_{DA}.

Figure 6a presents the variation of CSR with number N_l for ε_{DA} = 5%, for treated and untreated specimens, at a loose state and under p'_0 = 100kPa. There is a remarkable increase of N_l for loose treated specimens with CS = 6%, as compared to the corresponding of the untreated specimens. Furthermore, the N_l for specimens with CS = 10% is accordingly even higher and approximately double the corresponding of the untreated specimens.

The variation of CRR_{15}, with e, for untreated and treated specimens at p'_0 = 100kPa, is presented in Figure 6b. It is shown that treated silty sands possess at least 50% and 80% higher liquefaction resistance at CS = 6% and 10% respectively, than that of the untreated specimens under p'_0 = 100kPa, for the studied range of densities.

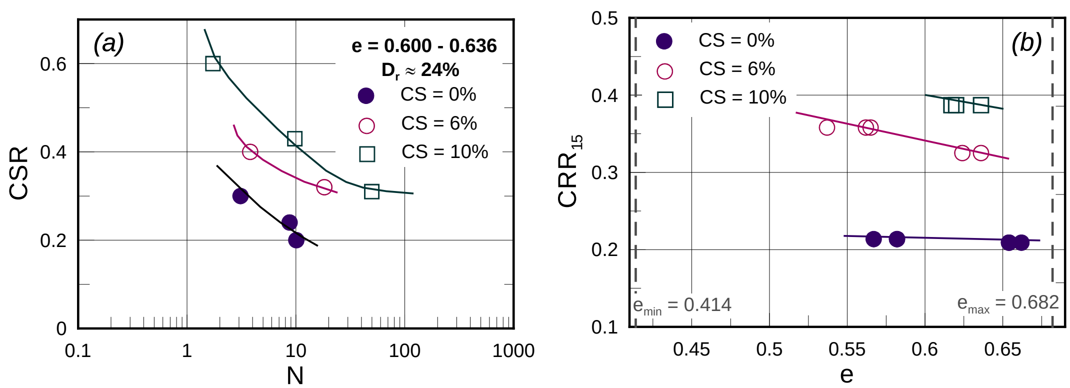

Figure 6. Variation of (a) CSR, with number of cycles N_l required for ε_{DA} = 5%, for specimens with e = 0.600-0.636 and (b) CRR_{15} with void ratio, e, for CS = 0, 6 and 10%, at p'_0 = 100kPa.

To examine the possibility of particle bonding breakage during cyclic loading of the treated specimens, undrained monotonic compression tests were performed on treated specimens after their cyclic loading to at least ε_{DA} = 5%. The number of preceding N ranged from 2.2 to 610 and the preceding accumulated strain, ε_{DA} varied from 5.4 to 6.7%. The test results for this post-cycling shear strength are shown in Figures 7 and 8, in which they are also compared with the corresponding shear strength of treated specimens not subjected previously to cyclic loading. It is shown that the monotonic stress-strain response of the specimens after cyclic loading becomes more ductile up to strains of the order of 15%, as compared to the corresponding response of specimens not subjected previously to cyclic loading and that the loss in shear strength during cyclic loading is insignificant. It may be concluded therefore that no degradation in shear strength of treated specimens subjected to cyclic loading takes place.

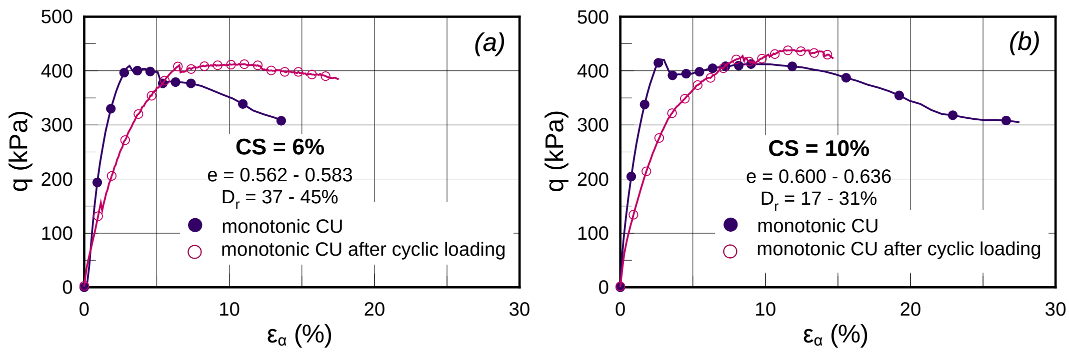

Figure 7. (a) Variation of q with ε_α, for treated silty sands with CS = 6% (a) and 10% (b) at p'_0 = 100kPa, after monotonic loading and after monotonic loading that followed cyclic loading to at least $\varepsilon_{DA} \geq 5\%$.

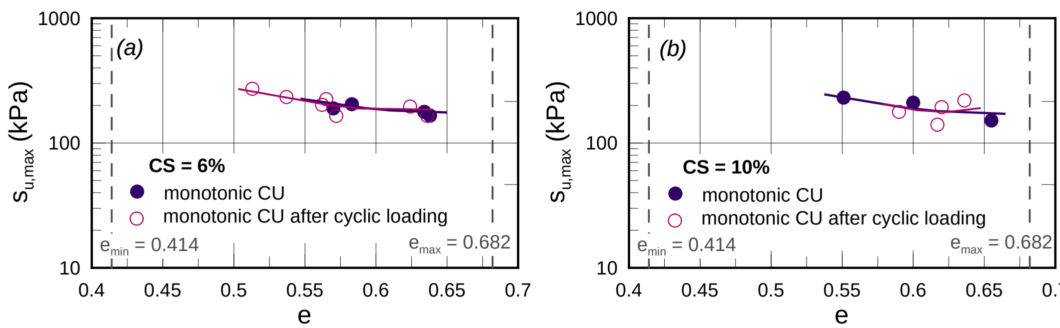

Figure 8. (a) Variation of the undrained shear strength, $s_{u,max}$ with void ratio, e, for treated silty sands with CS = 6% (a) and 10% (b) at p'_0 = 100kPa, after monotonic loading and after monotonic loading that followed cyclic loading to at least $\varepsilon_{DA} \geq 5\%$.

Conclusions

According to the test results, passive site stabilization by CS can be used for silty sands with permeability similar to that of the tested soil, as effectively as it has been done for clean sands. In particular, the following conclusions can be drawn from the work presented:

a) the increase of the Mohr-Coulomb peak shear strength of treated specimens over the corresponding of the untreated specimens, is due to the increase in both friction angle, but most mainly in cohesion.

b) the undrained monotonic shear strength of the treated specimens is significantly larger than the corresponding of untreated specimens at comparable strains. It also increases with increasing effective stress up to 300kPa.

c) for the materials tested at the studied density range, treated silty sands possess at least 50% and 80% higher liquefaction resistance at CS = 6% and 10% respectively, than that of the untreated specimens under $p'_0 = 100$kPa.

d) the treated specimens exhibit increased deformation resistance to cyclic loading, as compared to the untreated specimens, which experience much larger ε_{DA} in fewer cycles.

e) post-cycling undrained monotonic shear strength of treated specimens is not affected by the accumulated strains (at least $\varepsilon_{DA} \approx 5\%$) during the preceding cyclic loading, indicating that no deterioration of silty sand improvement occurs during cyclic loading.

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