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A comparative study of liquefaction resistance of a clean sand improved by colloidal silica and weak cementation

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ABSTRACT: This paper presents a comparative laboratory study on the improvement of the cyclic response of a liquefiable sand, by means of two treatment techniques; passive stabilization and weak cementation. Colloidal silica solution at concentrations of 6% and 10% and Portland cement added in the amounts of 1%, 3% and 5% by weight of dry soil were used as stabilizing agents for the implementation of passive stabilization and weak cementation, respectively. Results from cyclic triaxial tests on sand treated with the two studied techniques, as well as on the untreated sand for comparison, are presented. The experimental results show significant improvement of the cyclic response of the treated sands and thus indicate the effectiveness of both techniques. However, it is shown that cementation, even at such small cement contents of 1%, results in an improvement equivalent or even greater than that induced by passive stabilization under the studied stabilizer concentrations.

1 INTRODUCTION

In many cases in engineering projects design, improvement of the mechanical behaviour of liquefiable soils is required. At a project site prior to construction, mitigation of liquefaction risk for granular soils may be achieved by a number of soil improvement techniques, based on densification, grouting, or excess pore water pressure dissipation. However, in cases where improvement of soil under existing structures is required, implementation of typical improvement methods is difficult, if not impossible, without the obstruction of operation or the development of structural failures, during the application of dynamic or vibrating techniques.

Passive stabilization is a newly proposed non-desruptive soil improvement technique for the mitigation of liquefaction risk at existing developed sites. According to this technique, a stabilizer is slowly injected at the upgradient side of a developed site, by means of gradual or augmented groundwater flow. The stabilizing material has initially low viscosity, which increases rapidly after delivery at the target location at a controlled time. Thus, soil improvement by this technique applies to the whole structure area of interest, without causing obstructions to its normal operation. The enrichment of the pore fluid with the stabilizer alters the mechanical response of the soil skeleton - pore fluid system, making it less vulnerable to liquefaction. Passive stabilization with the use of colloidal silica as the stabilizing material, was first studied by Gallagher (2000).

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, which enables uniform and effortless infiltration within the subsoil. By adjusting the ionic strength, the pH or both, a given CS solution transforms into a rigid gel after a controllable time, which may range from a few minutes to more than a year (Gallagher 2000). This property allows it to be injected or mixed with soil, so that after gelling, CS coats and bonds the individual particles of the soil and thus alters its mechanical behaviour. Furthermore, gelled CS fills the void space in the soil and therefore blocks water flow (Persoff et al. 1998) and significantly reduces its hydraulic conductivity (Gallagher & Mitchell 2002). 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 (Noll et al. 1992), non-toxicity (Whang 1995, DuPont 1997) and its low cost. In contrast to most chemical grouts, gelled CS does not exhibit syneresis (shrinkage of the external volume of the grout due to loss of water from its pores) (Yonekura & Miwa 1993, Gallagher 2000, Mollamahmutoglu & Yilmaz 2010).

Besides passive stabilization, a number of innovative soil improvement techniques has been proposed in the past few years, whereby stabilization of granular soils is achieved with the use of inorganic materials, biomaterials or bacteria, in order to form cementation bonds among soil particles, thus creating artificially weakly cemented sandy soils (Mitchell & Santamarina 2005, Kim et al. 2014 and others). Moreover, weak cementation is often encountered in natural sand deposits in liquefaction-prone sites, which are characterized by the formation of weak interparticle bonds, due to the presence of, among others, small amounts of silica, hydrous iron oxides and carbonates in the soil skeleton, as a result of weathering, chemical depositions, environmental changes, ageing and others. Due to the fragile nature of such sands, weak cementation is preferably artificially reproduced in the laboratory by the introduction of cementing agents, such as cement, gypsum and lime in sand specimens (Ismail et al. 2002). These agents, acting like cementing bonds at the contact points of grains and also filling the soil voids, significantly influence both the monotonic and cyclic soil behaviour (Saxena et al. 1988, Huang & Airey 1998). Weakly cemented sands reportedly show brittle behaviour and an increase in both shear strength and liquefaction resistance, compared to uncemented sands.

The purpose of the present work is to examine the effectiveness of both passive stabilization and weak cementation as liquefaction mitigation techniques. To this extent, a series of cyclic triaxial tests on a sand treated with both CS and cement was conducted. A comparison between the results from both series of tests is presented and discussed. In addition, the results from treated sands are compared to the corresponding results from the same untreated sand.

2 TESTED MATERIALS

2.1 *Untreated sand*

The soil used in the testing program is a natural quartz clean sand (M31) with grains of variable roundness and sphericity and of relatively uniform frosted texture, indicating its aeolian origin. Its physical properties are presented in Table 1.

2.2 *CS treated sand*

Ludox SM-30 was selected as the stabilizing agent of sand specimens, supplied as a 30% by weight silica water 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. For the studied CS concentrations of 6% and 10%, it was decided to employ gel times equal to 11 and 10 hours respectively, which are adequate for the completion of in-situ CS permeation through the treated soil. This was determined by adjusting the pH value to pH = 6.0, as well as the NaCl concentration of the solution to 0.18N and 0.03N respectively. It is noted that gel time was defined as the elapsed time

Table 1. Physical properties of sand (M31).

e_{min}	e_{max}	γ_{dmin} (kN/m^3)	γ_{dmax} (kN/m^3)	G_s	C_u	d_{50} (mm)	k (m/sec)
0.558	0.805	14.43	16.71	2.655	1.50	0.31	$5.2 \cdot 10^{-4}$

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.

Compressibility tests were conducted on CS = 6% and 10% solutions prior to gelling under pressures ranging from 0kPa to 2000kPa. The measured compressibility, C of both studied CS solutions at a pressure range from 50kPa to 200kPa, was 13.7 to 52.2 times larger than the corresponding distilled water compressibility, C_w, which was measured equal to $4.6 \cdot 10^{-7}$kPa^{-1}.

2.3 *Weakly cemented sand*

White Portland cement (CEM II/A-LL 42.5N) with a specific gravity of 3.103 was used as the cementing agent for the preparation of weakly cemented sand specimens. Its fast gain of strength allowed the adoption of a curing period of 7 days. The specific gravity of the cemented specimens was calculated as a weighted average, based on the proportions of soil and cement in each specimen. The cement content, c.c., defined as the ratio of dry weight of cement and dry total weight, ranged from 1% to 5%. In this work, weakly cemented sands have been defined as sands with an unconfined compression strength less than 640kPa. Even at the highest c.c., the specimens easily disintegrated into individual grains under finger pressure. This c.c. range reflects the actual conditions of many naturally, as well as artificially weakly cemented sand deposits (Sitar et al. 1980).

3 SPECIMEN PREPARATION AND CYCLIC TRIAXIAL TESTING

Cylindrical specimens (height/diameter ≈ 100mm/50mm) of untreated, as well as treated sands with both studied techniques were prepared at various densities, using the undercompaction method, proposed by Ladd (1978). Density of both treated and untreated specimens was controlled by adjusting both the water content (water content ranged from 5% to 13%) and the compactive effort. Saturation was achieved by initially percolating throughout the specimen, from bottom to top, initially carbon dioxide (CO_2) and afterwards de-aired water.

The triaxial testing program consisted of undrained cyclic triaxial tests, performed using a closed-loop automatic cyclic triaxial apparatus (M.T.S. Systems Corporation) (Vranna 2016).

Shearing of untreated and weakly cemented specimens was performed under an effective isotropic stress, p'_0, ranging from 100kPa to 300kPa. For the reasons decribed below, shearing of specimens treated with CS was performed accordingly under a total isotropic stress, p_0, ranging from 100kPa to 300kPa.

3.1 *Untreated sand*

Saturation of untreated specimens was completed by increasing the cell and back pressure consecutively under a small effective stress of 10kPa. In all tests on untreated specimens, a back pressure of 400kPa was used resulting in pore water pressure parameter, B, values greater than 0.95. After saturation, untreated specimens were isotropically consolidated under an effective isotropic stress, p'_0, ranging from 100kPa to 300kPa. A period of time equal to double the consolidation time was allowed before shearing.

During undrained cyclic triaxial testing in untreated specimens, the excess pore water pressure, Δu, builds up as the cyclic axial stress ($\pm\sigma_d$) is applied and approaches the initial confining stress when the state of liquefaction is reached ($\Delta u/p'_0 \geq 0.95$). For a given sand, the rate of increase of excess pore water pressure, its final value and the corresponding double amplitude axial strain, ε_{DA}, depend on the effective confining stress, the density and the applied cyclic stress ratio, CSR = $\sigma_d/2p'_0$ (Papadopoulou 2008). The occurrence of ε_{DA} = 5% is customarily used as a reference point to define the state of cyclic softening or liquefaction of sands (Ishihara 1993). Thus, in order to specify the onset of liquefaction, the number of loading cycles, N, required to reach ε_{DA} = 5%, N_l, is determined by running a series of tests with different CSR values. In view of the typical number of significant load cycles (10 to 20 for an earthquake of a 7.5 magnitude) of actual earthquakes, in this work the onset of cyclic softening and thus the

cyclic resistance ratio, CRR_{15}, for both treated and untreated specimens is considered as the CSR required to produce ε_{DA} = 5% in 15 loading cycles. In fact, for the tested untreated sand, the occurrence of ε_{DA} = 5% practically coincides with the occurrence of $\Delta u/p'_0 \geq 0.95$.

3.2 *CS treated sand*

Treated with CS specimens were formed by thoroughly mixing predetermined quantities of dry sand and water, in order to achieve a uniform consistency. The moist material was placed and compacted in five layers into a cylindrical split mould with a membrane stretched against its wall. Saturation of specimens was performed by following the same procedure as in the untreated sand. After saturation, the CS solution was likewise injected into the specimens from bottom to top, until it filled the sand 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.

After the setting of CS, specimens were left to harden inside the mould for 24hr and then were taken out. The specimens were then placed inside a constant temperature (22°C±1°C) and humidity chamber for a curing time of five times the CS gel time.

It is noted that permeability of treated specimens at a relative density of D_r = 40% after curing and prior to testing was determined by means of triaxial permeability tests. The measured coefficients of permeability, k, were $2.3 \cdot 10^{-9}$m/sec and $2.2 \cdot 10^{-11}$m/sec for CS = 6% and 10% respectively, which both lie within the range of clayey soils.

Saturation of treated specimens after curing and prior to testing was not performed, as it was considered that this would damage the CS bonds, similarly to previous investigations (Gallagher 2000, Koch 2002, Gallagher & Mitchell 2002, Spencer et al. 2008, Mollamahmutoglou & Yilmaz 2010). Besides, saturation is not a process followed during the field operation of passive stabilization. Furthermore, the measurement of any excess pore water pressure was considered unreliable, due to lack of pore water inside the treated specimens and their low permeability. As a result, mean total, p_0, and effective, p'_0, stresses, were considered identical.

3.3 *Weakly cemented sand*

Cemented specimens were formed by thoroughly mixing the relative quantities of dry sand and cement initially and then adding water in order to achieve a uniform consistency. The moist material was placed and compacted in five layers into a cylindrical split mould with a membrane stretched against its wall. Saturation of cemented specimens was performed according to the procedure described above for untreated specimens.

The complete process of mixing, filling the mould and saturation of cemented specimens took approximately 45min to 50min. This time is shorter than initial setting time of 140min of the Portland cement used. The specimens were left to harden inside the mould for 24hr and then were taken out. The specimens were then soaked into water for a curing time of 7days inside a constant temperature (22°C±1°C) and humidity chamber.

After curing and prior to triaxial testing, cemented specimens were saturated by initially percolating de-aired water and then increasing the cell and back pressure consecutively under a small effective stress of 10kPa.

4 RESULTS AND DISCUSSION

4.1 *CS treated sand*

Figure 1 compares the cyclic response of loose untreated and treated specimens, subjected to cyclic stress ratio, CSR = 0.23 - 0.25 under p_0, p'_0 = 100kPa. It is indicated that the untreated specimens experience much larger strain in fewer loading cycles, N, than the corresponding treated specimens. In particular, the number of cycles required to reach cyclic softening, N_l,

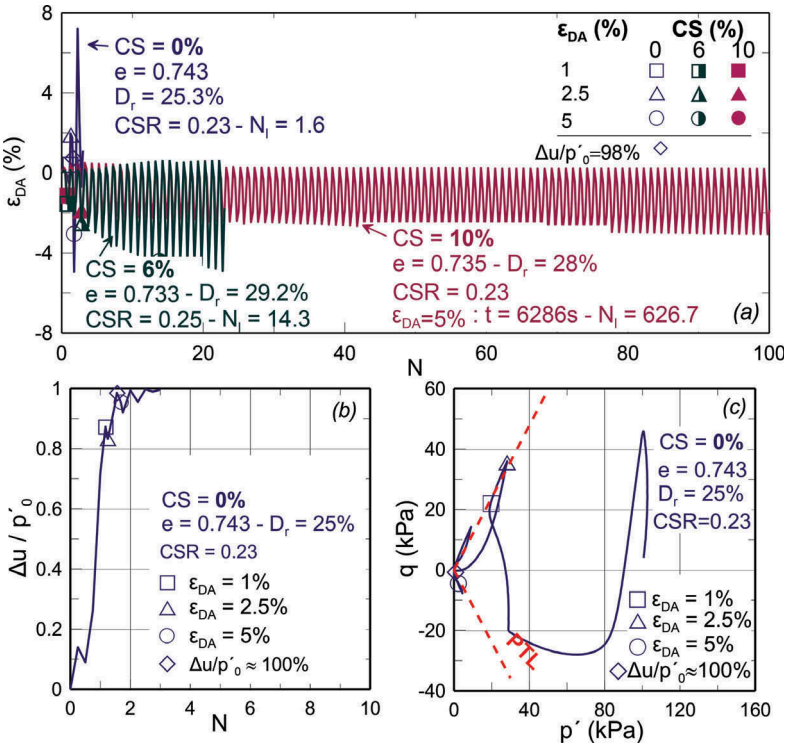

Figure 1. (a) Evolution of double amplitude axial strain, ε_{DA} with number of cycles, N, for treated and untreated sands, for D_r = 25.3 - 29.2% and cyclic stress ratio, CSR = 0.23 under p'_0 = 100kPa. Evolution of (b) normalized excess pore water pressure, $\Delta u/p'_0$, with N and (c) variation of deviatoric stress, q, with mean effective stress, p′, for the untreated sand of Figure 1a.

increases from 1.6 for the untreated specimen, to 14.3 and 626.7 for the treated specimens with CS = 6% and 10% respectively. Furthermore, 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 1b, 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 concentration. The above pattern of behaviour was also observed at different CSR and densities.

Figure 2 presents the variation of CSR with N_l for ε_{DA} = 5%, for treated and untreated specimens, at a loose state under p_0, p'_0 = 100kPa and 300kPa. At p_0, p'_0 = 100kPa, A remarkable increase of N_l is shown for treated specimens, as compared to the corresponding of the untreated. The CS concentration for treated specimens at higher CSR values is shown to be practically insignificant. At p_0, p'_0 = 300kPa (Figure 2b) and at the density range shown, stabilization with CS improves the cyclic response of the sand only at the higher CSR values studied.

4.2 *Weakly cemented sand*

Figure 3 presents the cyclic response of untreated and weakly cemented with c.c. = 1% sands and in particular the evolution of ε_{DA} and $\Delta u/p'_0$ with number of cycles, N and their stress paths, at a given density and CSR, under p'_0 = 100kPa. It is shown that a significantly larger number of cycles is required for the weakly cemented sand to reach liquefaction. This difference

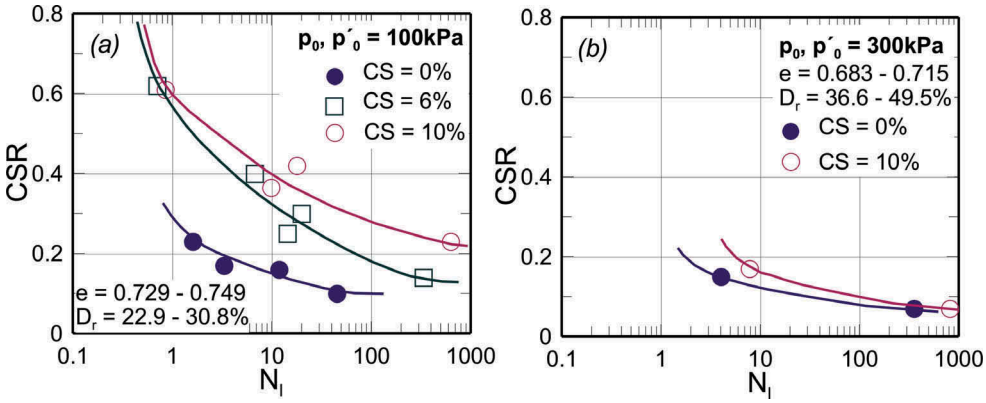

Figure 2. Variation of CSR with N_l for untreated and treated with CS = 6% and 10% sands at (a) p_0, p'_0 = 100kPa and (b) 300kPa.

in behaviour is also depicted in the excess pore water pressure development. For the particular density and CSR, it is indicated that cementation leads to a transition of soil behaviour from flow type liquefaction into cyclic mobility type. In Figures 3(c) and 3(d), the boundary surfaces corresponding to the value of $(q/p')_{max}$, the phase transformation and the critical state are shown. According to the above Figures, for both the uncemented and the cemented with c.c. = 1% sand, the effective stress path moves towards the origin during progressive cyclic loading due to the accumulation of pore pressure, until it reaches the boundary defined by $(q/p')_{max}$.

The effect of cementation and c.c. on the variation of CSR with the number of cycles, N_l, required to reach liquefaction determined at ε_{DA} = 5% for sands at p'_0 = 100kPa is presented in Figure 4. It is shown that increasing c.c. from 0% to 1% and afterwards to 3% results in a significant improvement in cyclic response of the tested sand both at a loose and a medium dense state. At dense state, the results for weakly cemented sands with c.c. = 1% are similar to the corresponding presented by Clough et al. (1989) and Kokusho et al. (2012) for artificially

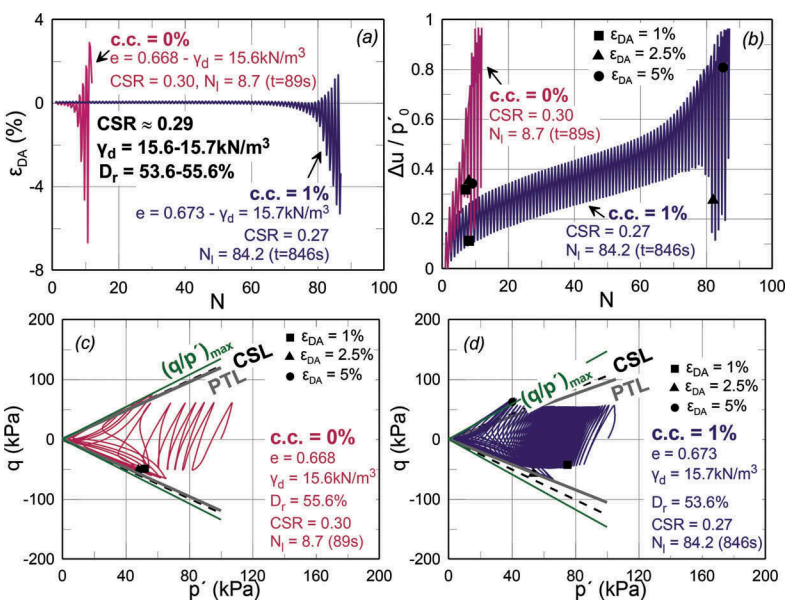

Figure 3. (a) (a), (b) Evolution of ε_{DA} and $\Delta u/p'_0$ with N for (c) uncemented sand and (d) cemented sand with c.c. = 1%, at a given density and CSR at p'_0 = 100kPa. (c, d) Comparison of q - p´ plots for the above sands.

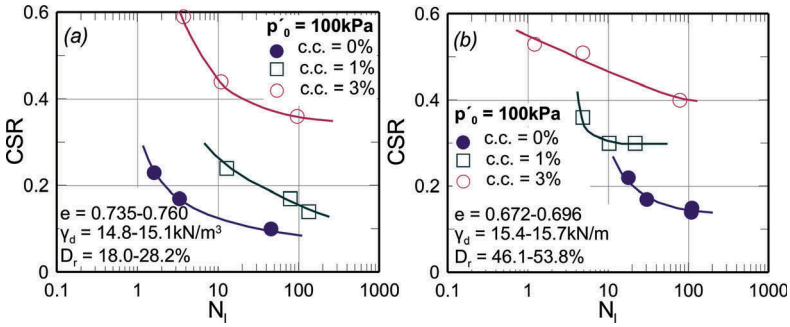

Figure 4. Variation of CSR with N_l required to reach ε_{DA} = 5% for the uncemented sand and the cemented sands with c.c. = 1% and 3%, at p'_0 = 100kPa, at (a) a loose and (b) a medium dense state.

cemented sands treated with Portland cement Type II and I respectively. At loose state, the results for the cemented sand with c.c. = 3% are similar with results presented by Montoya et al. (2013) for biologically treated sands with microbial-induced calcite content of 3%.

4.3 *Comparison between CS treated and weakly cemented sands*

Figure 5 presents the variation of CRR_{15} with void ratio, e for untreated specimens, treated with CS specimens and weakly cemented specimens. At p_0, p'_0 = 100kPa, liquefaction resistance of cemented with c.c. = 1% and 3% specimens is approximately 1.3 - 2 times and 2 - 4 times higher, respectively, than the corresponding of untreated specimens. Similar results are observed at higher at p_0, p'_0 = 300kPa.

Furthermore, for the studied range of densities under p_0, p'_0 = 100kPa, liquefaction resistance of treated specimens with CS = 6% is shown to practically coincide with the corresponding of cemented with c.c. = 1% specimens. Likewise, the liquefaction resistance of treated with CS = 10% specimens is shown to be coincident with that of cemented with c.c. = 3% specimens. With increasing p_0, p'_0 to 300kPa, the effectiveness of weak cementation is more pronounced than that of passive stabilization, since specimens with the smallest c.c. = 1% show a liquefaction resistance equivalent to that of specimens with the highest CS = 10%.

5 CONCLUSIONS

Both studied soil improvement techniques show a remarkable increase of liquefaction resistance of the tested sand. In passive stabilization, CS concentration does not seem to

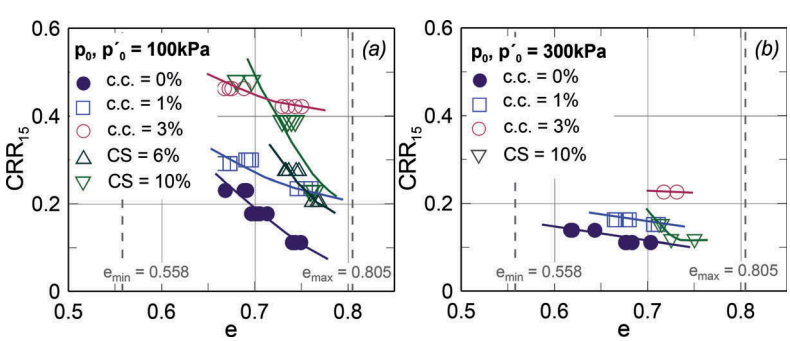

Figure 5. Variation of cyclic resistance ratio, CRR_{15} with void ratio, e for untreated and treated sands at (a) p_0, p'_0 = 100kPa and (b) 300kPa.

significantly affect the cyclic response of the treated sands. On the contrary, in weak cementation, cement content is shown to play a primary role in liquefaction resistance.

Even at such small c.c. as 1%, cementation results in an impressive improvement of cyclic response of the tested sand, equivalent to that induced by passive stabilization.

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