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Mechanical behaviour of biocemented sand under triaxial consolidated undrained or constant shear drained conditions

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Abstract. Biocementation based on the microbially induced calcite precipitation (MICP) process is a novel soil improvement method. Biocement can improve significantly the properties of soils by binding soil particles to increase the shear strength or filling in the pores to reduce the permeability of soil. In this paper, results of triaxial consolidated undrained (CU) tests and constant shear drained (CSD) tests on biocemented Ottawa sand are presented. In the CU tests, the biocemented sand had more dilative behaviour by showing a higher stress-strain curves and faster pore pressure reducing trends as compared with their untreated counterparts. In the CSD tests, the stress ratio q/p' at which biocemented sand became unstable was higher than that for untreated sands, implying that the biocementation will improve the stability of sand to water infiltration or liquefaction.

Keywords: biocement; microbially induced calcite precipitation; sand; constant shear drained test

1. Introduction

Microorganisms and their activities in soil ecosystems can affect the evolution, physical and mechanical properties of soils. These microbial activities can be controlled and utilized to tackle several geotechnical problems. The adoption of microbial methods in geotechnical engineering has gaining increasing research attention in the last decade. One of the microbial approaches is the microbially induced calcite precipitation (MICP) based soil improvement method (Ivanov and Chu 2008, Dejong *et al.* 2009, Chu *et al.* 2012, Dhami *et al.* 2013, Dejong *et al.* 2013). This method involves the production, precipitation and crystallization of calcium carbonate in soil catalyzed by microorganisms. The produced calcite, the stable form of calcium carbonate crystals, can fill soil pores and bind soil particles, and improve significantly the strength and stiffness of soils. This method is named the biocement method, as the treatment effect using biocement is similar to those of other cementitious materials (Ivanov and Chu 2008). The biocement can be used for many geotechnical and geoenvironmental problems, such as improvement of liquefiable grounds, control of soil erosion, and stabilization of heavy metal contaminants, etc. (Dejong *et al.* 2013, Sidik *et al.* 2014, Kim and Cho 2017). The biocementation effect can also be induced by some other microbial activities or products such as biopolymers (Chang and Cho 2014). In addition to the biocement method, some other microbial activities have also been proposed and tested to solve geotechnical and geoenvironmental problems. Slimes produced by bacteria in soils can reduce the pore volume and increase the viscosity of pore water, whereby the soil permeability can be greatly decreased (Jeon *et al.* 2017, Kim *et al.* 2017). Such a process can be utilized to control water seepage in earth dikes and dams (Blauw *et al.* 2009) or prevent the migration of contaminants in soils (Tang *et al.* 2018a). It is also found that biopolymer produced by bacteria in soil can effectively clog soil pores and reduce soil permeability, which can be used as a leakage or contaminant control method (Kwon *et al.* 2013, Chang *et al.* 2016). Microbial processes can also be adopted to remove heavy metals in contaminated soil grounds (Tang *et al.* 2018b). Biogenic gases in soils can reduce soil permeability and enhance the resistance of soil to liquefaction (He *et al.* 2013, He and Chu 2014).

It has been proven by the laboratory tests that the biocement can be successfully applied to sandy soils. The understanding of the mechanical properties of biocemented sand is required prior to field applications. Results of unconfined compression tests reported by many researchers showed that, the unconfined compression strength of biocemented sand could be as high as several megapascal or even higher, which can satisfy most of the applications in geotechnical engineering (He *et al.* 2016). In direct shear tests, biocemented sands showed increases in both internal friction angles and cohesions, indicating the clear enhancement in the shear strength (Chou *et al.* 2011). In triaxial tests, biocemented sands displayed higher stiffness and dilatancy as compared with untreated sand (Dejong *et al.* 2006, Lin *et al.* 2016, O'Donnell *et al.* 2015, Montoya *et al.* 2015, Liu *et al.* 2019). The improvement of liquefiable ground is one of the promising applications of the biocement method. In the cyclic triaxial tests, the cyclic strength of biocemented sand could be far higher than that of pure sand (Han *et al.* 2016, Sasaki and Kuwano 2016,

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Xiao *et al.* 2018). Centrifugal and 1-g shaking table model tests proved that the biocement treatment could reduce the surface settlement of sandy ground and alleviate the excess pore pressure under the cyclic loads (Dejong *et al.* 2013, Cheng *et al.* 2013).

The mechanical behaviour of sand is strongly affected by its density. The mechanical behaviour of biocemented sand may also be affected by its original density state. In this paper, results of triaxial consolidated undrained tests and constant shear drained tests on biocemented sands with various degrees of treatment and relative densities are presented to study the effect of biocementation on the mechanical behaviour of sand. The results of this study can be used as a database or reference for future designs and constructions related to the use of the biocement.

2. Materials and methods

2.1 Testing sand

Ottawa (ASTM Graded) sand was used in this study. It is a poorly-graded quartz sand. The mean size is 0.36 mm. The grain shapes are mostly round. The maximum and minimum void ratios are 0.753 and 0.467, respectively. The particle size distribution is presented in Fig. 1.

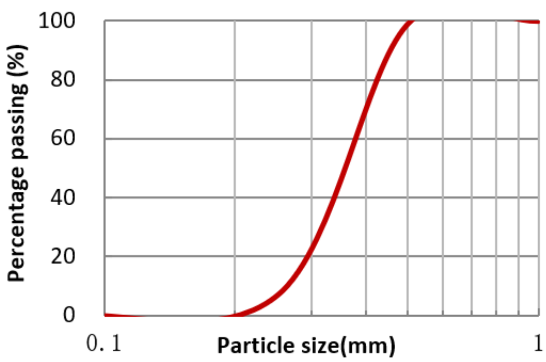


Fig. 1 Particle size distribution of Ottawa (ASTM Graded) sand

2.2 Bacterial cultivation

A strain of *Sporosarcina pasteurii* (CGMCC1.3687 from China General Microbiological Culture Collection Center) was used in this study. It is halotolerant and highly capable of hydrolyzing urea into carbonate and ammonium. The bacteria were cultivated using the liquid medium consisting of: yeast extract 20 g/L, NH_4Cl 10 g/L, $NiCl_2{\cdot}6H_20$ 2.4g/L, $MnSO_4{\cdot}H_2O$ 1g/L, and 2 mol/L NaOH solution for the adjustment of pH to 8.5-9.0. The liquid medium was autoclaved before inoculation. The bacteria were cultivated at 30°C, 100 rpm shaking and aerobic condition in an incubator for 24 hours before harvest. The urease activity of bacteria obtained in this way was 6.6-11.0 mM/min (0.6-1.0 mS/cm/min in terms of the electric conductivity change rate).

2.3 Sample preparation and microbial treatment

In this study, the biocementation effect of sand was achieved through urea hydrolyzing process mediated by ureolytic bacteria. The reaction consists of two steps. In the first step, urea is hydrolyzed into ammonium and carbonate by the bacteria, which also leads to the pH increase in the reaction system. In the second step, calcium carbonate is produced with the presence of calcium source. The reaction is as follows,

$$NH_2\text{-}CO\text{-}NH_2+2H_2O \xrightarrow{\text{ureolytic bacteria}} 2NH_4^{\ +}+CO_3^{\ 2-} \quad (1)$$

$$Ca^{2+}+CO_3^{\ 2-} \rightarrow CaCO_3\downarrow \quad (2)$$

When calcite (the stable form of calcium carbonate) is produced in soil pores, it serves as a cementing agent for soil binding and pore filling.

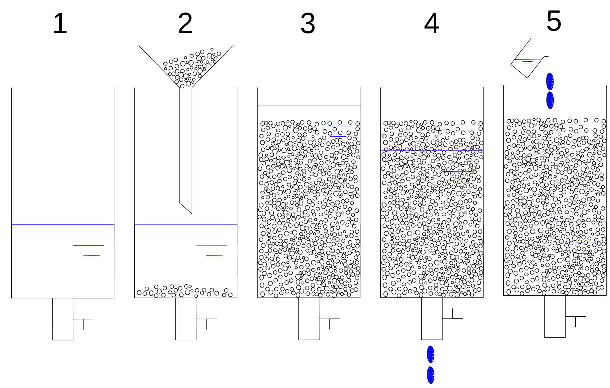


Fig. 2 Procedures of sample preparation

In the biocement treatment, ureolytic bacteria, urea and calcium salt are required. The treatment liquid was obtained by mixing bacterial suspension and urea-calcium chloride solution together at 1:1 volume ratio. The treatment liquid contained 0.5 mol/L equimolar urea and calcium chloride. The soil samples were prepared in cylindrical moulds. The size of the samples was 100 mm in height and 50 mm in diameter. The procedures of sample preparation are as follows (Fig. 2),

1. In the first pass of biocement treatment, the treatment liquid was poured into the mould.
2. Certain amount of dry sand was carefully placed into the mould through a funnel with five equal portions. Each layer was slightly compacted to a designed height in order to obtain a uniform sample.
3. When the sample was formed, redundant treatment liquid above the sand surface was removed. The treatment liquid immersed the sample for 3 days to ensure complete reaction as one treatment. It took around 10 minutes to complete Steps 1-3. The produced $CaCO_3$ in these 10 minutes is minimal.
4. If a sample required 2 or more passes of treatments, after the first pass, used treatment liquid was drained out.
5. New treatment liquid was added and immersed the sample for the subsequent pass of treatment.

After the biocement treatment, the samples were rinses in clean water and then frozen in a confined condition. Frozen samples could be easily installed in the the triaxial apparatus with little sample disturbance.

Table 1 Testing parameters and brief test result

Test No.	Void ratio of host soil after consolidation, e_c'	Relative density of host soil after consolidation, D_r'	Calcite content, *w/w*	Peak stress ratio, $(q/p')_{max}$	Deviatoric stress at 5% axial strain, *q* (kPa)	Pore pressure coefficient *A* at 5% axial strain
CU-30-0	0.675	27.4%	0.00%	1.22	94.6	0.525
CU-30-1	0.655	34.4%	0.74%	1.29	295.0	-0.069
CU-30-2	0.659	32.8%	0.98%	1.27	242.8	-0.047
CU-30-4	0.682	24.9%	1.06%	1.37	533.6	-0.171
CU-50-0	0.629	43.5%	0.00%	1.31	486.9	-0.174
CU-50-1	0.598	54.2%	0.75%	1.39	598.5	-0.171
CU-50-2	0.587	58.0%	1.01%	1.35	748.6	-0.220
CU-50-4	0.613	48.9%	1.22%	1.39	666.0	-0.206
CU-70-0	0.545	72.6%	0.00%	1.41	673.4	-0.177
CU-70-1	0.549	71.3%	0.73%	1.41	804.2	-0.210
CU-70-2	0.525	79.6%	0.96%	1.41	1006.0	-0.243
CU-70-4	0.537	75.5%	1.18%	1.49	1237.5	-0.214
CU-90-0	0.486	93.2%	0.00%	1.51	1032.6	-0.214
CU-90-1	0.500	88.3%	0.98%	1.57	1414.6	-0.211
CU-90-2	0.504	87.0%	1.04%	1.62	1491.7	-0.210
CU-90-4	0.498	89.3%	1.24%	1.57	1475.8	-0.225
CSD-50-0	0.584	59.1%	0.00%	-	-	-
CSD-50-1	0.610	50.1%	0.72%	-	-	-
CSD-50-2	0.603	52.4%	1.08%	-	-	-
CSD-90-0	0.508	85.8%	0.00%	-	-	-

2.4 Triaxial test and constant shear drained test

The triaxial apparatus, TKA-TTS-3S (TKA Co. Ltd.) was used for the triaxial and constant shear drained tests. In the triaxial tests, the sample was installed into the triaxial chamber, and 10 kPa cell pressure was applied to support the sample for 3 hours to allow complete thaw of the sample. Cell pressure and back pressure were increased to 410 kPa and 400 kPa, respectively. The pressure was held for 12 hours to saturate the sample. The saturation process conducted in this way could ensure that pore pressure coefficient *B* was larger than 0.95, indicating a fully saturated state. The cell pressure was increased to 500 kPa and the sample was consolidated at 100 kPa effective confining pressure. After the consolidation, the sample was sheared in an undrained condition and at a strain rate of 0.1 %/min.

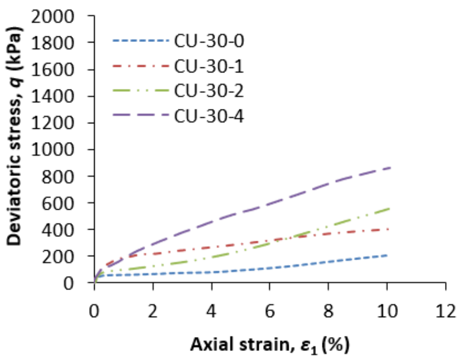
Constant shear drained (CSD) test is probably the most suitable test to simulate the mechanical behaviour of soil slopes in water infiltration conditions or lateral stress relief conditions (Chu *et al.* 2003, Lourenco *et al.* 2011). In CSD tests, the stress path of a sample was controlled in a manner that the mean effective stress *p*' was reduced while the deviatoric stress *q* was kept constant. In this study, CSD tests were carried out by decreasing the cell pressure and the deviatoric stress *q* was maintained constant using a servo system integrated in the triaxial apparatus. In the tests, the samples were firstly consolidated at 150 kPa effective confining pressure by maintaining the cell pressure and back pressure at 550 and 400 kPa, respectively. Then, the stress states of the samples were sheared from (150 kPa, 0 kPa) to (250 kPa, 300 kPa) in the *p*'-*q* plan. From the (250 kPa, 300 kPa) stress state, the deviatoric stress *q* was reduced at 1 kPa/min rate towards (100 kPa, 300 kPa). The axial strain and the volumetric strain were also measured in real time during the tests.

2.5 Measurement of calcite content

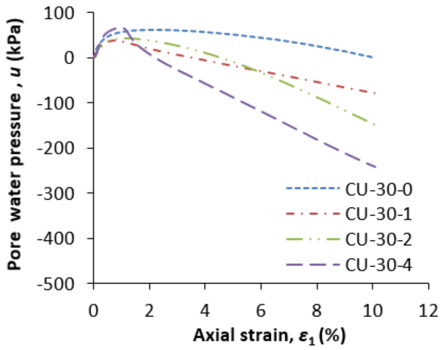
After the triaxial or constant shear drained tests, the calcite content in each sample was determined. Small pieces of soil were taken from the samples. The small pieces of soil were rinsed in de-ionized water to remove soluble calcium and placed in certain amount of acid liquid to dissolve all the calcite. The EDTA titration method was used to determine the concentration of calcium in the acid liquid. The amount of calcium in the acid liquid was correspondent to the calcite content in soil samples.

2.6 Testing programme

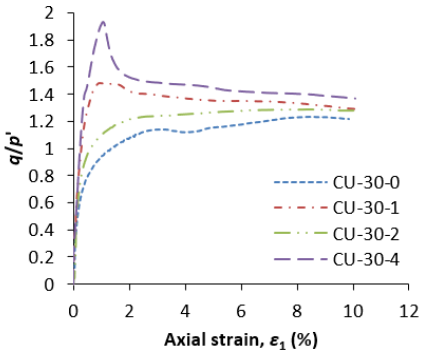
In the triaxial consolidated undrained (CU) tests, there were two testing variables, relative densities (that is, original relative densities of soil) and treatment passes. Relative densities of the test samples ranged from 30% to 90%, and treatment passes ranged from 0 to 4. Total 16 CU tests were carried out. In the CSD tests, 4 tests were carried out and the testing variables also included relative density and treatment passes. A list the test parameters and brief results are given in Table 1. The name of a test consists 3



(a) Stress-strain curves

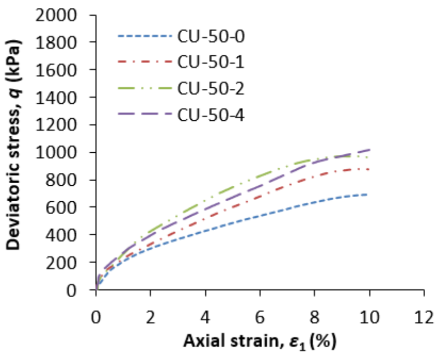


(b) Excess pore pressure curves

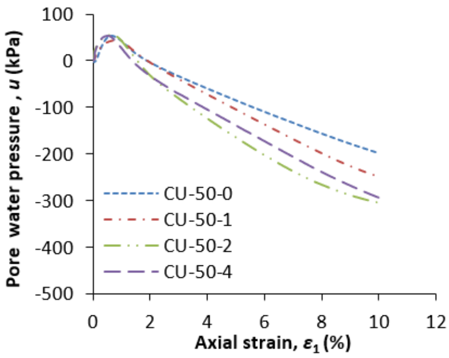


(c) Stress ratio versus axial strain curves

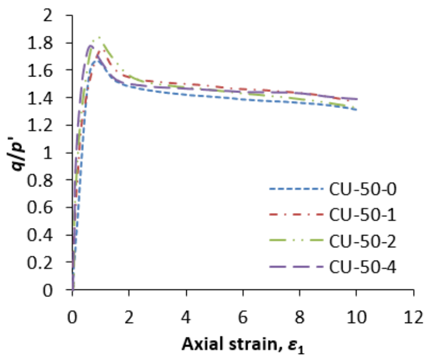
Fig. 2 Results of triaxial CU tests on the samples with 30% original relative density



(a) Stress-strain curves



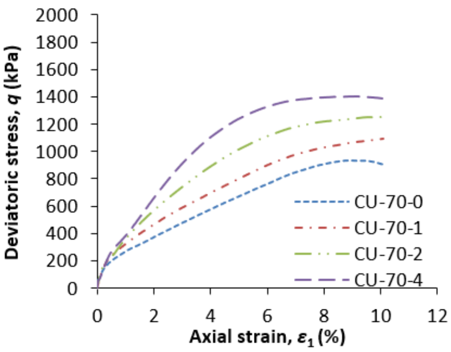
(b) Excess pore pressure curves



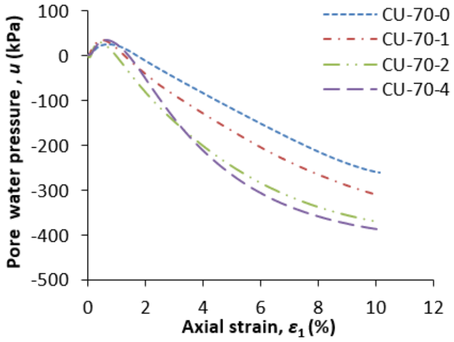
(c) Stress ratio versus axial strain curves

Fig. 3 Results of triaxial CU tests on the samples with 50% original relative density

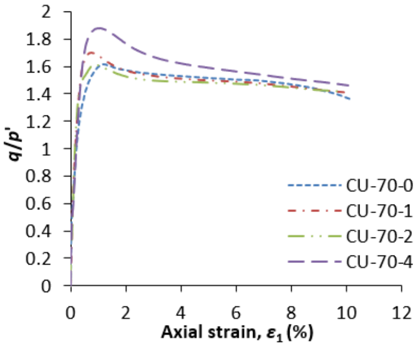
parts: the type of test, the relative density of the host soil and the treatment passes. For example, CU-50-4 means that the test is a triaxial CU test, the relative density of host soil is designed to be 50%, and it receives 4-pass biocement treatments. For the samples with the same designed relative density, although the real relative density of host soil was not exactly the same as the designed value, the compaction efforts applied was the same.



(a) Stress-strain curves

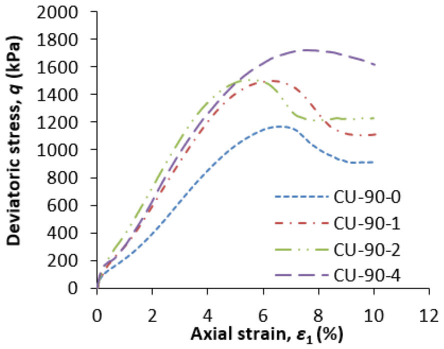


(b) Excess pore pressure curves

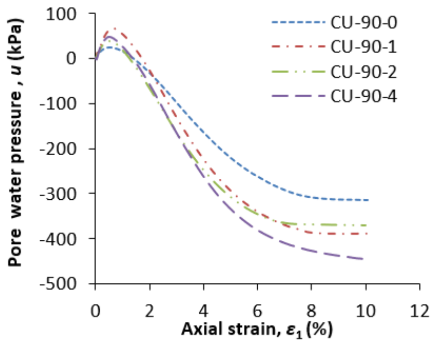


(c) Stress ratio versus axial strain curves

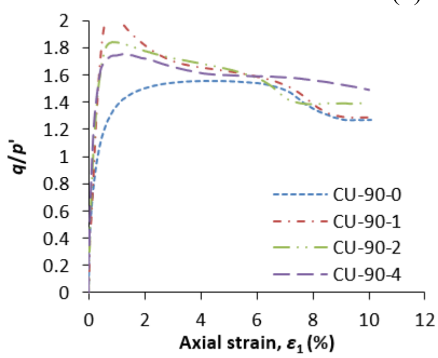
Fig. 4 Results of triaxial CU tests on the samples with 70% original relative density



(a) Stress-strain curves



(b) Excess pore pressure curves



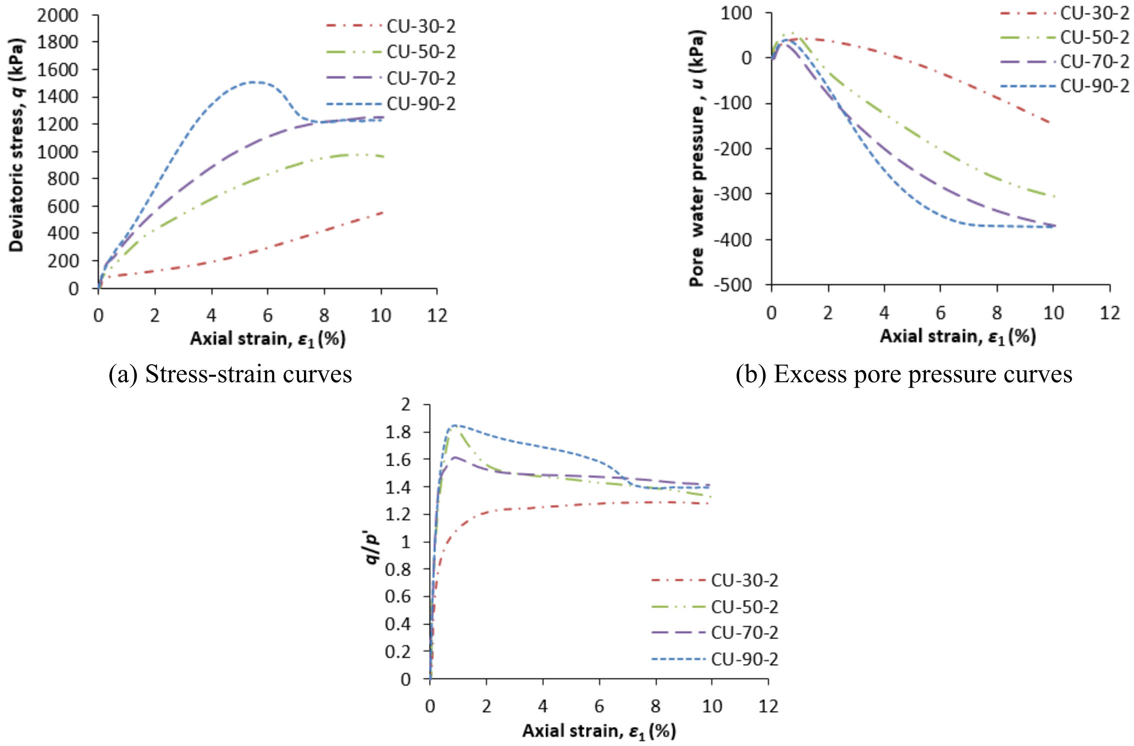
(c) Stress ratio versus axial strain curves

Fig. 5 Results of triaxial CU tests on the samples with 90% original relative density

3. Results and discussions

3.1 Triaxial consolidated undrained tests

The results of the triaxial consolidated undrained tests are presented in Figs. 2 to 5. There are four series of tests with the original relative densities of 30%, 50%, 70% and 90%, respectively. Several features can be identified. Both untreated and biocemented sand samples experience strain hardening behavior during undrained shear, as can be seen



(a) Stress-strain curves

(b) Excess pore pressure curves

(c) Stress ratio versus axial strain curves

Fig. 6 Results of triaxial CU tests on the 2-pass treament samples with a various original relative densities

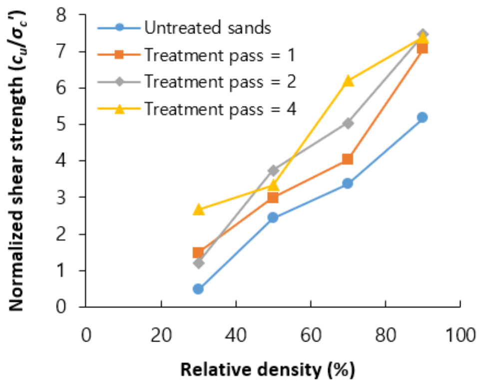


Fig. 7 Normalized shear strength versus relative density curves

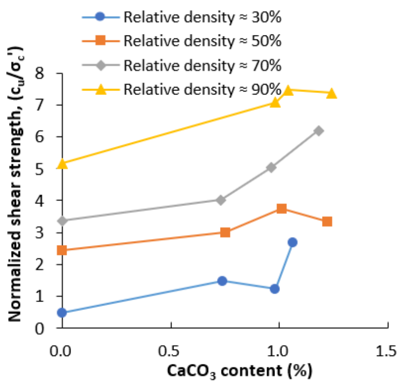


Fig. 9 Normalized shear strength versus $CaCO_3$ content curves

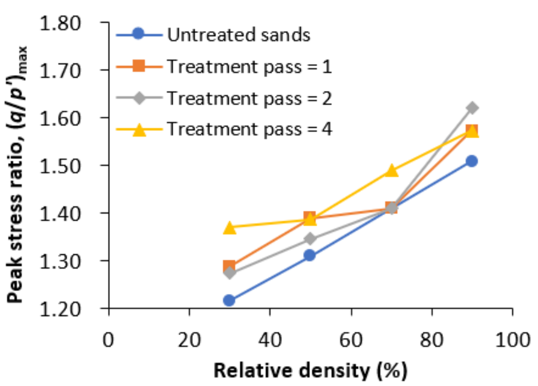


Fig. 8 Peak stress ratio versus relative density curves

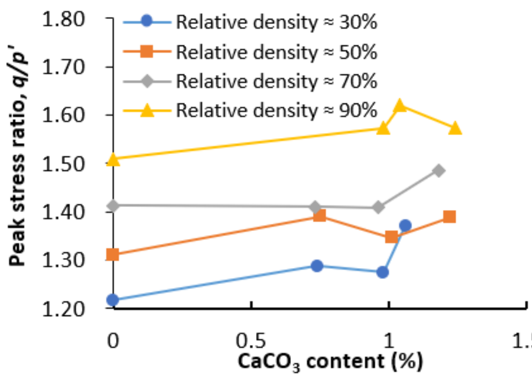


Fig. 10 Peak stress ratio versus $CaCO_3$ content curves

in both stress-strain curves and excess pore pressure curves. However, biocemented sands show more dilative manners as compared with untreated sands. The Biocemented sands have higher stress-strain curves than untreated sands (Figs.

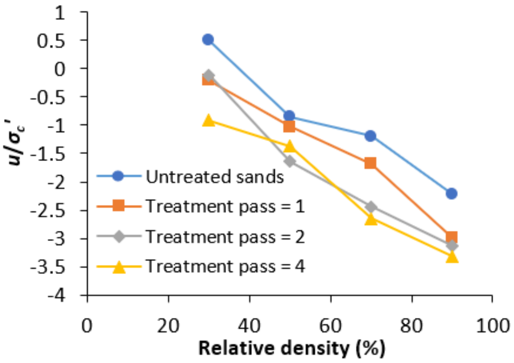


Fig. 11 Normalized pore water pressure versus relative density curves

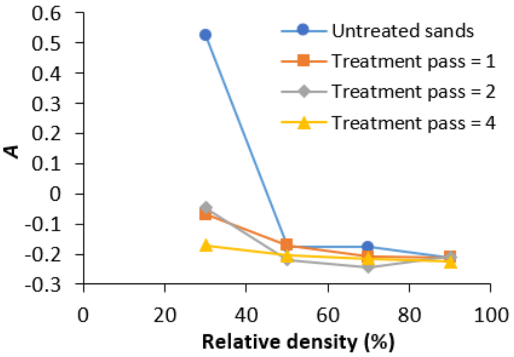


Fig. 12 Pore pressure coefficient *A* versus relative density curves

2(a), 3(a), 4(a) and 5(a)). In addition, biocemented sands show faster excess pore pressure reducing trends than untreated sands (Figs. 2(b), 3(b), 4(b) and 5(b)). As for the effect of treatment passes, the overall trend is that the more the treatment passes are, the more dilative the samples are. Such results clearly demonstrate that the biocement method is effective in improving the mechanical behaviour of sand at various levels of densities. The stress ratio q/p' versus axial strain curves are presented in Figs. 2(c), 3(c), 4(c) and 5(c). The biocemented sands have higher peak stress ratio than untreated sands. For the biocemented sands, the curves reach peak values at a relatively low strain (around 1%), and the curves show gradual reducing trend thereafter. For the untreated sands, this feature is not very clear. In the biocemented sands, more biocementation treatments contribute to the higher peak stress ratio and higher initial stiffness. The biocementation is brittle and the cementation effect gradually degrades after reaching the peak values.

The factor of relative density of original sand also plays an important role in the mechanical behaviour of biocemented sand. It can be seen in Fig. 6 that, with the same two treatment passes, the sand samples with higher relative density are more dilative in stress-strain relationship and excess pore pressure generation. However, the biocement treatment effect is more pronounced at lower relative density. As can be seen in Table 1, compared with untreated sands, sands with two treatment passes have 2.57, 1.54, 1.49 and 1.44 times improvement in the deviatoric stress at 5% axial strain for the samples with 30%, 50%, 70% and 90% relative densities, respectively.

According to the test results, both treatment passes and relative densities of original sands have great influences on the mechanical properties of biocemented sand. Normalized shear strength in relation to relative density and treatment passes are presented in Fig. 7. Here the normalized shear strength is defined as,

$$\frac{c_u}{\sigma_c'} = \frac{q(\text{at 5\% axial strain})}{2\sigma_c'}$$

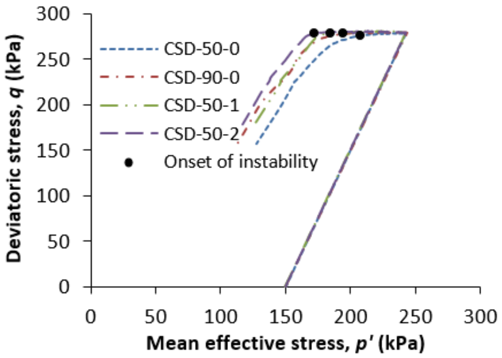
As can be seen, the biocemented sands have much higher shear strength than the untreated sands. In the meantime, with the same treatment pass, higher relative density leads to larger shear strengths. Peak stress ratios $(q/p')_{max}$ are also summarized in Fig. 8 in relation to treatment passes and relative densities. The overall trends are similar to those presented in Fig. 7. Both biocement treatment passes and relative densities have positive effects on the peak stress ratios. The relationships between normalized shear strength and calcite content, and peak stress ratio and calcite content are presented in Figs. 9 and 10, respectively. Both strength and peak stress ratio increase with calcite content at various level of relative densities, indicating the effectiveness of calcite on the mechanical behaviour of sand. Normalized pore water pressures u/σ_c' (at 5% axial strain) in relation to treatment passes and relative densities are presented in Fig. 11, and pore water pressures coefficient A in relation to treatment passes and relative densities are presented in Fig. 12. Both of these two figures reflect the trend that the biocement treatments or increasing the densities leads to a lower pore water pressure generation, indicating more dilative manners.

3.2 Constant shear drained tests

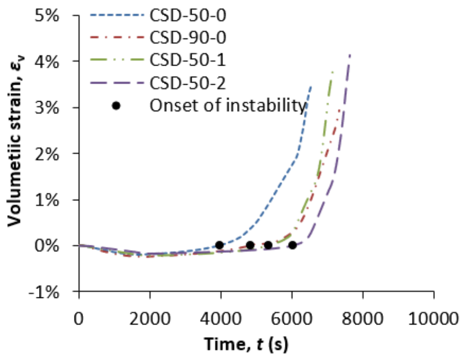
Triaxial compression tests are adopted in most of the industrial and research experiments to obtain the strength and mechanical behaviour of soils. However, the stress path in triaxial compression tests may not properly mimic those in field conditions. Constant shear drained (CSD) tests can reproduce slope failure initiated by increasing pore water pressure or decreasing mean effective stress. Such a stress path in commonly seen in soil slopes in water infiltration conditions or lateral stress relief conditions (Chu *et al.* 2003, Lourenco *et al.* 2011). It is also reported that such a mechanism that leads the instability of soil may account for some slope failure disasters (Chu *et al.* 2003).

In this study, four CSD tests were carried out. Two untreated samples have 50% and 90% relative densities, respectively. Two biocemented samples have 50% relative density and receive 1 and 2 treatment passes, respectively. The results are presented in Fig. 13. Subject to a CSD stress path, soil samples lose their stability at certain stress states. The loss of stability was manifested by a quick reduction in deviatoric stress Fig. 13(a), a sudden increase in both the volumetric strain (Fig. 13(b)) and the axial strain (Fig. 13(c)).

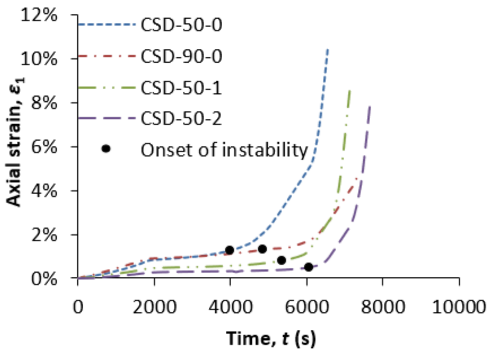
The stress ratio q/p' at the onset of instability is plotted versus calcium carbonate content in Fig. 14. It can be seen that for the untreated sample with D_r = 50%, the sample started to lose its stability at a stress ratio q/p' of 1.33 which



(a) Stress paths



(b) Volumetric strain versus time



(c) Axial strain versus time

Fig. 13 Results of CSD tests

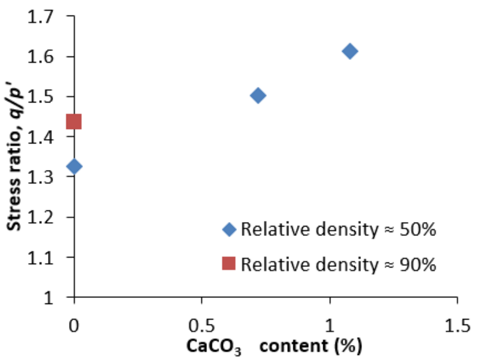


Fig. 14 Stress ratio q/p' against $CaCO_3$ content at the onset of instability

was the lowest. For the untreated sample with D_r = 90%, instability started at a stress ratio q/p' of 1.43. For the two biocemented samples with 1 and 2 numbers of treatments, the onsets of instability were at stress ratios of 1.50 and 1.61, respectively. Although the calcium carbonate contents in the two biocemented samples were relatively low (0.72% and 1.08% as shown in Table 1), the improvement for the stability of the sand was considerable. Therefore, biocementation can be an effective method for improving the stability of granular soil slopes under water infiltration and other similar conditions.

4. Conclusions

In this study, triaxial consolidated undrained (CU) tests and constant shear drained (CSD) tests were carried out on sands with various levels of biocement treatments and relative densities. The following conclusions can be made from this study:

(1) The biocemented sand is more dilative in the triaxial CU tests as indicated by the higher stress-strain curves and faster pore pressure reducing rates as compared with their untreated counterparts. For the biocemented sands, the stress ratio q/p' versus axial strain curves show peak values at relatively low axial strains and decreases after the peaks, which could be due to the degradation of the biocementation effect. In comparison, the q/p' of untreated sand shows smooth variations as the axial strain develops.

(2) For biocemented sand with the same calcium carbonate contents (or number of treatments), the higher the relative density, the more dilative in undrained shear. However, for sand with lower relative density, the degree of improvement is more pronounced in terms of deviatoric stress increasing factors.

(3) Under a CSD condition, biocemented sand can become unstable in a way similar to clean sand. However, the stress ratio q/p' at which instability occurs is higher for biocemented sand, implying that the biocement method could be effective in enhancing the stability of soil slopes under water infiltration conditions.

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