

Cyclic Response of a Sand with Thixotropic Pore Fluid

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ABSTRACT

Saturated specimens of Ottawa sand prepared with 0%, 3% and 5% bentonite by dry mass of sand are tested under cyclic loading to investigate the effects of bentonite on the cyclic response. For the same skeleton relative density and cyclic stress ratio (CSR), the cyclic tests on the sand-bentonite mixtures show a significant increase of the number of cycles required for liquefaction compared to the clean sand. This is caused, as observed in resonant column tests, by an increase of the elastic threshold due to the presence of bentonite, which delays the generation of excess pore pressure. Such behavior can be explained by the rheological properties of the pore fluid. Oscillatory tests conducted with a rheometer on bentonite slurries show that for shear strains as large as 1% these materials exhibit elastic behavior with a constant shear modulus. Moreover, due to the thixotropic nature of the bentonite slurries, their storage modulus shows a marked increase with time. This observation is consistent with the increase in the liquefaction resistance of the sand-bentonite mixtures with time also observed in cyclic triaxial experiments.

INTRODUCTION

Liquefaction is an important cause of damage to civil infrastructures during earthquakes. Notable examples are: the collapse of apartment buildings in Niigata, Japan, in 1964; the destruction of the waterfront in Seward, Whittier, and Valdez during the Great Alaskan Earthquake in 1964; the slide at the Lower Van Norman Dam during the San Fernando earthquake in 1971; the damage caused by the earthquake in the Imperial Valley of California in 1979; the failure of a dam in Cerro Negro, Chile, after the 1985 earthquake; the large damage caused to quay walls at Kobe port during the Hyogoken-Nambu earthquake in 1995; the collapse of buildings in Adapazari during the August 1999 earthquake in Turkey; and, most recently, the numerous liquefaction failures caused by the 2007 Pisco, earthquake in Peru.

The sand properties (relative density, stress history, particle size, particle shape and gradation) and the characteristics of the cyclic loading are critical factors that determine the liquefaction susceptibility of a sand deposit (Youd et al., 2001). Observations in the field after an earthquake indicate that the presence of plastic fines increases the resistance to liquefaction (Wang, 1979; Ishihara and Koseki; 1989; Ishihara, 1993; Ishihara, 1996). This observation is the premise for considering the use of bentonite – a highly plastic, widely available clay – as a candidate for treating sand deposits to increase liquefaction resistance.

Preliminary work performed by Haldavnekar et al. (2003) on sand-bentonite mixtures showed that the addition of 5% bentonite to a clean sand specimen increased its liquefaction resistance significantly. The work presented in this paper is the result of additional research performed to evaluate the effects of the addition of bentonite to

clean sand under a broader range of testing conditions, and to gain insight on the mechanisms responsible for the improved resistance to liquefaction of the sand.

The paper presents results of cyclic triaxial and resonant column tests conducted on sand-bentonite mixtures, with 0%, 3% and 5% bentonite by dry mass of the sand. Additionally, it provides results of dynamic oscillatory tests performed on bentonite slurries as a means to gain insight on the behavior of the pore fluid. Our assumption is that the enhanced resistance to cyclic loading of sand-bentonite mixtures is the result of the mechanical properties of the pore fluid: a concentrated bentonite gel, characterized by an elastic response even at large strains. This is supported by additional experimental results that indicate that, due to the thixotropy of the gel, the liquefaction resistance improves with time.

EXPERIMENTAL PROGRAM

Materials

The tests are performed on sand specimens in which the bentonite is mixed with the sand prior to forming the specimen (“wished in place bentonite”) following the procedures presented by Haldavnekar et al. (2003). The sand used for the tests is Ottawa sand C 778, a uniform (coefficient of uniformity, $C_u=1.7$) and clean fine to medium sand, with 2 to 5% finer than the No. 200 sieve. The specific gravity is $G_s = 2.65$, and the maximum and minimum void ratios are $e_{max} = 0.78$ and $e_{min} = 0.48$ (following ASTM standards).

The bentonite used is a Wyoming bentonite (CP-200), commercialized by VOLCLAY, with at least 70% passing the No. 200 sieve, and minimum free swell of 8 ml/g. This commercially available bentonite was selected for its availability in mass quantities for field applications as compared to more purified forms of bentonite. Deionized filtered deaired water was used to flush and saturate the triaxial specimens and to prepare the bentonite slurries.

Testing Methods

Tests on sand-bentonite specimens

Two types of tests are conducted on sand-bentonite specimens: cyclic triaxial tests and resonant column tests. Cyclic tests are conducted using a MTS testing system equipped with a 5000 lb hydraulic actuator. External regulators are used to control the cell and pore pressures, and an external load cell was added to measure the cyclic stresses applied. All cyclic tests are performed using a sinusoidal wave form applied at a rate of 1 cycle per second with stress reversal, starting with compression loading. Undrained conditions are chosen to simulate dynamic earthquake loading in the field. Pore pressure generation is monitored with continuous loading cycles at the base of the soil specimen. Different criteria are employed in the literature to identify the number of cycles leading to liquefaction. In this work, the number of cycles to liquefaction is defined as that corresponding to a loss of effective confining stresses to less than 10% of the initial value.

The Drnevich resonant column is used to investigate the small strain behavior of the sand-bentonite specimens. Tests are performed both at very small strains (approximately 10^{-4} %) to determine the initial shear modulus (G_{max}), as well as at higher shear strains (up to 0.01%) to describe the shear modulus degradation behavior.

All resonant column tests presented in this paper are conducted under undrained conditions, while the pore pressure at the base of the soil specimen is measured along with the resonant frequencies and the shear strains.

Similar procedures are used for specimen preparation and for saturation and consolidation of the specimens, for both the cyclic and resonant column tests. The specimen preparation method has been modified from standard procedures (ASTM D 5322 and ASTM D 4015) to accommodate the specific nature of the tested materials. In this method, the sand and the desired percentage of bentonite are dry mixed in an airtight container for 15 minutes and air-pluviated using a funnel in a triaxial split mold (diameter=70 mm, height=200 mm), which allows for preparation of 160 mm high specimens. The mold is then tapped on the sides to achieve the desired skeletal void ratio. All the tests presented in this paper have the same skeletal relative density ($35\% \pm 5\%$). The skeleton void ratio is calculated as the volume of voids and bentonite divided by the volume of the sand; the skeletal relative density is calculated using the skeletal void ratio and e_{\min} and e_{\max} of the clean sand.

Specimens are set under 25 kPa effective confining pressure, and then flushed from bottom to top first with carbon dioxide (CO_2), and then with deionized deaired water. During flushing the gradient is kept low to prevent local liquefaction and bentonite migration (gradient less than 5).

Following flushing and prior to back pressure saturation, specimens are allowed to rest for 72 hours to ensure full hydration and swelling of the bentonite inside the sand pores. The selection of the duration of the rest phase is based on a preliminary study of the swelling time and swelling pressure of sand-bentonite mixtures. This work showed that a minimum time of 36 hours was needed for the bentonite in the pores to hydrate and swell under atmospheric pressure. After the rest period the specimens are back-pressure saturated for 24 hours with a back pressure of 200 to 400 kPa to achieve B-values ≥ 0.95 (based on the recommendations by Black and Lee, 1973), and then consolidated to the desired effective consolidation stress.

The cyclic triaxial specimens are isotropically consolidated in two increments to an effective confining stress of 100 kPa to represent shallow soil (5 to 7 m of depth) field conditions. While the majority of the cyclic specimens are allowed 22 hours of secondary compression at 100 kPa, a limited number of specimens are tested after a longer time to investigate thixotropy effects on the response of the sand-bentonite specimens.

Resonant column tests are performed at three different consolidation stresses: 50, 100, and 193 kPa. The homogeneity of tested specimens was checked to insure consistent void ratio and bentonite content. The void ratio was found to be within $\pm 3\%$ of the target value and the bentonite content was within 0.5% by dry mass of sand throughout the height of the specimen.

Rheological tests on bentonite slurries

The rheological tests are carried out to gain insight on the mechanical properties of the pore fluid that is formed inside the sand specimen as the bentonite hydrates and gels. These tests are performed on bentonite slurries using an Anton Paar Physica MCR301 rheometer, equipped with a 50 mm, 1° angle cone-plate geometry. Bentonite slurries are prepared by mixing bentonite powder (screened to entirely pass

the No. 200 sieve) with deionized filtered water at a concentration of 10% (by mass of the slurry) using a high shear mixer for 20 minutes. The 10% bentonite slurries are equivalent to the pore fluid formed inside a sand specimen with 3% bentonite by dry mass of the sand prepared at a skeletal void ratio equivalent to 35% relative density. Note that this calculation assumes that all the dry bentonite in the specimen fully hydrates and is allowed to swell uniformly within the pore space.

Rheological tests presented in this paper are oscillatory tests measuring the visco-elastic properties of the slurries with increasing shear strain (also referred to as “strain sweeps”). In these tests, the slurry is subjected to an oscillatory shear strain ($\gamma = \gamma_0 \sin[\omega t]$, γ_0 is the maximum applied shear strain) of increasing amplitude at constant frequency (1 Hz in this testing program) while measuring the resulting shear stress τ . The shear stress may be shifted by a phase angle δ with respect to the applied strain. The shear stress can be expressed as the sum of an elastic component, in phase with the applied strain, and a viscous component, out-of-phase, as shown in eq. 1,

$$\tau = \gamma_0 \{G' \sin(\omega t) + G'' \cos(\omega t)\} \quad \text{eq. 1}$$

Where G' and G'' are the storage (elastic) and the loss (viscous) moduli, respectively, and ω is the frequency (e.g. Barnes et al. 1989).

RESULTS

Cyclic Response

Twenty nine cyclic tests are performed, with cyclic stress ratio (CSR) varying between 0.1 and 0.25, on specimens with 0%, 3% and 5% bentonite by mass of sand. All specimens are prepared at the same skeletal void ratio (corresponding to a skeletal relative density of $35\% \pm 5\%$), and consolidated to an effective confining stress of 100 kPa.

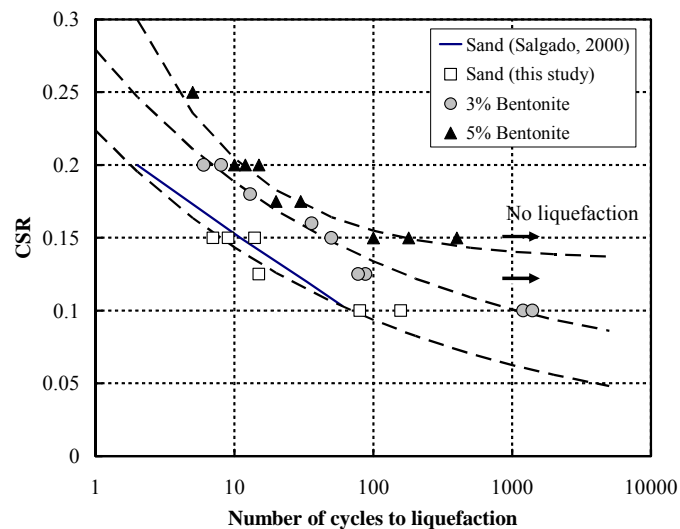


Figure 1 CSR versus number of cycles to liquefaction

Figure 1 shows the number of cycles to liquefaction determined from 27 tests as a function of the applied CSR. The figure also includes the regression proposed by

Salgado et al. (2000) based on tests conducted on the same sand at a slightly higher relative density ($D_r = 40\%$) and using a different cyclic testing apparatus. Note that Salgado et al. (2000) used of a different criterion for determining the number of cycles to liquefaction (5% double strain amplitude criterion). Despite this, the data from the two testing programs are consistent, and the results from the two data sets are within the experimental error for cyclic triaxial tests. This is in agreement with what is reported by other researchers (e.g. Ladd, 1977).

Figure 1 shows that at any CSR, the addition of 3% and 5% bentonite increases the number of cycles to liquefaction by approximately one order of magnitude. For any given CSR, the specimens with 5% bentonite show a greater resistance to liquefaction. For values of $CSR \geq 0.15$, the difference between the results for 3% and 5% bentonite is relatively small. However, the difference between 3% and 5% bentonite increases at lower CSRs. For all the 3% bentonite tests shown in Figure 1 liquefaction is always reached even though in some cases after over 1000 loading cycles. For $CSR=0.125$ and 0.15 , the 5% bentonite specimens (data indicated in Figure 1 by the horizontal arrows) are able to withstand up to 1000 loading cycles without approaching liquefaction.

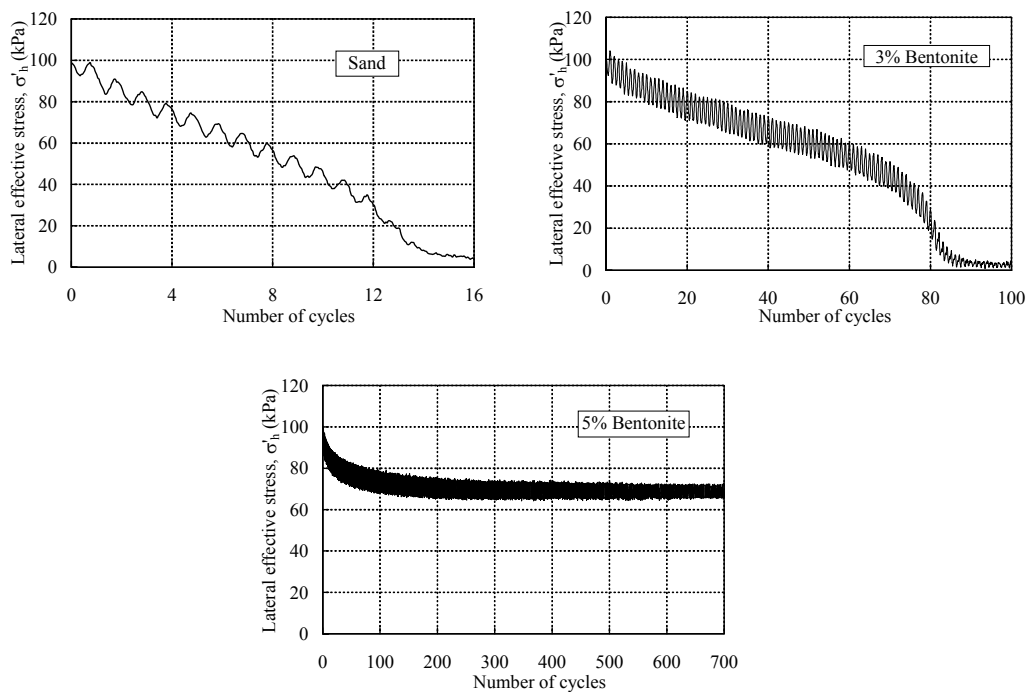


Figure 2 Loss of effective stresses with continuous cyclic loading ($CSR = 0.125$) for 0%, 3% and 5% bentonite specimens

For sands, there is a threshold strain level below which no liquefaction takes place regardless of the number of loading cycles (e.g. Drnevich and Richart 1970). This strain threshold corresponds to a CSR low enough such that there is no pore pressure buildup from one cycle to the next. Figure 1 suggests that in the presence of bentonite, this threshold increases with increasing bentonite content. This is supported by the cyclic stress results shown in Figure 2. The figure shows the change in effective

confining stress with increasing loading cycles for 0%, 3% and 5% bentonite specimens tested at CSR=0.125. As shown in the figure, the 5% bentonite specimen does not liquefy. The pore pressure increases initially with loading cycles but ultimately reaches a steady condition (corresponding to ~40% effective stress loss). The axial strains (not reported here) are also very small (<0.02% double amplitude strain after 1000 cycles, when the test was stopped). The clean sand specimen and the 3% bentonite specimens do reach liquefaction. The rate of excess pore pressure generation in the clean sand specimen is fast and the loss of effective confining pressure is reached after 15 cycles. The 3% bentonite specimen shows a slower rate of excess pore pressure generation, with 50% of the effective confinement stresses lost after approximately 65 cycles. As the confining pressure is reduced, the soil stiffness decreases resulting in the accumulation of strain and the acceleration of the excess pore pressure buildup. The specimen liquefies at 88 cycles. This is the same type of behavior observed during cyclic loading of clean sand at low CSR. Comparison of the response of the clean and 3% bentonite specimens indicates that the presence of bentonite delays the excess pore pressure generation. This limits the loss of stiffness and results in higher cyclic resistance. More details on the effect of bentonite on pore pressure generation and stiffness degradation are presented in the next section.

Small Strain Behavior

Resonant column tests are typically conducted drained at a constant confining stress to evaluate the shear modulus degradation with shear strain. However, the resonant column tests presented in this paper are performed undrained. The goal of these tests is to observe how the linear threshold strain (critical strain) and excess pore pressure generation change with the addition of bentonite.

Figure 3 shows the shear modulus degradation curves for specimens with 0%, 3% and 5% bentonite, all prepared at the same skeletal void ratio, which are tested at 50, 100 and 193 kPa. It is observed, as expected, that the stiffness increases with effective confining stress. Additionally, at any given confining stress, the results show a small drop in the initial stiffness (G_{max}) of the specimens containing bentonite, relative to the data for the clean sand. It is hypothesized that this reduction in G_{max} is a result of the “lubricating” action of bentonite particles at the particle to particle contacts (note that this effect is likely a by-product of the dry mixing specimen preparation procedure). Figure 3 shows that for all specimens, there is stiffness degradation with increasing shear strain. This is in part a result of the reduction in the effective confining stress due to the excess pore pressures generated under undrained conditions (Figure 4).

The shear stiffness data presented in Figure 3 for 50 and 100 kPa confinement are replotted in Figure 4 and normalized by G_{max} . The figure shows that the curves for the bentonite specimens lie above those of the clean sand, with differences increasing with higher shear strains. Figure 4 also presents the excess pore pressures normalized by the initial effective confining stresses. The data indicate that, at the same confining stress, excess pore pressures are generated much earlier in the clean sand specimens. These results are consistent with an extension of the linear range and a smaller

reduction of shear modulus degradation for the specimens treated with bentonite as compared to clean sand specimens.

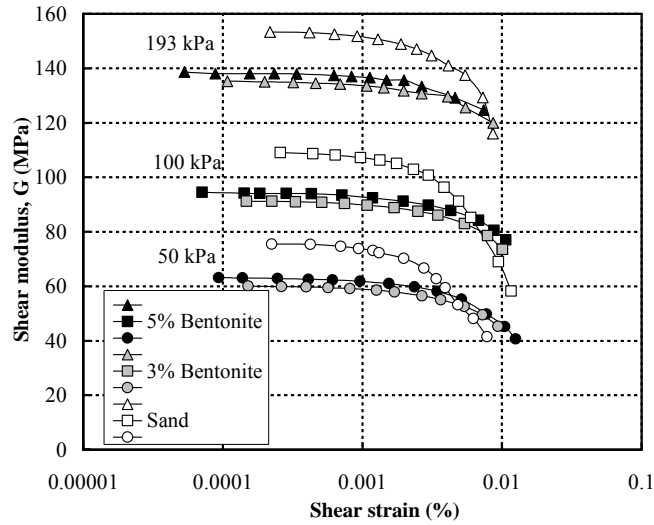


Figure 3 Shear Modulus degradation for 0%, 3% and 5% bentonite at 50, 100 and 193 kPa

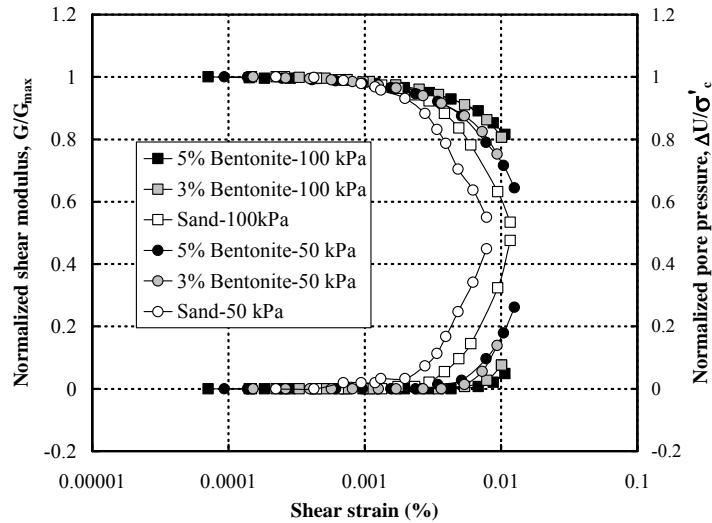


Figure 4 Normalized shear modulus and excess pore pressure for 0%, 3% and 5% bentonite at 50 and 100 kPa

Pore Fluid Rheology

Dynamic oscillatory strain sweep tests are performed to gain insight on the mechanical properties of the pore fluid formed inside specimens prepared with bentonite. Figure 5 shows the results of a strain sweep test conducted at atmospheric pressure on a 10% slurry immediately after mixing.

As shown in Figure 5, for shear strains as large as 1% (linear visco-elastic region), G' remains basically constant. An elastic response is defined by low value of δ and as the region where $G' \gg G''$. Once the shear strain exceeds a critical value

(approximately 1% for the test shown), the storage modulus drops significantly and the behavior of the material becomes increasingly viscous.

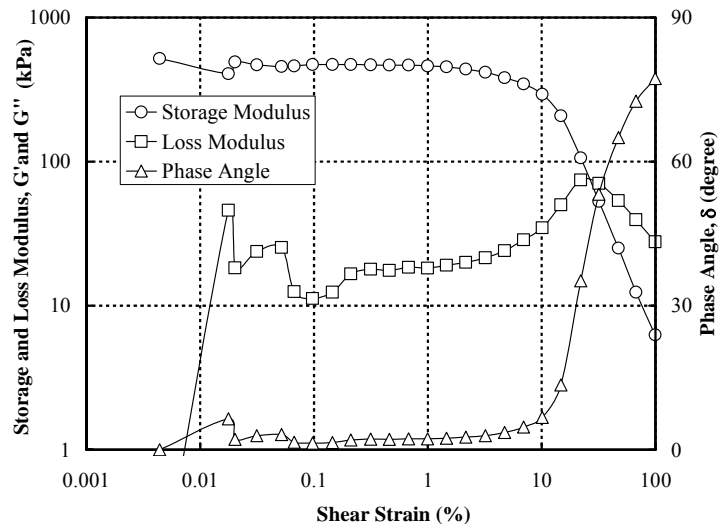


Figure 5 Amplitude sweep test on 10% bentonite slurry

A common occurrence in many colloidal suspensions is the rebuilding of its internal structure after it has been broken by shearing. After the applied shear stress is reduced or completely removed, the internal structure begins to rebuild with time. This time-dependent behavior is referred to as thixotropy (Barnes, 1997).

The thixotropic nature of clay slurries is shown in Figure 6 which plots the storage modulus (G') measured in the initial portion of an oscillatory test as a function of the age of the slurry (up to 1440 hours). The values of G' shown in Figure 6 are obtained by averaging G' values within the linear range. For each ageing time, 4 to 5 strain sweep tests are performed to insure repeatability. The tests show an increase in G' with time (at 600 hours G' is about 2.5 times the value at $t=0$).

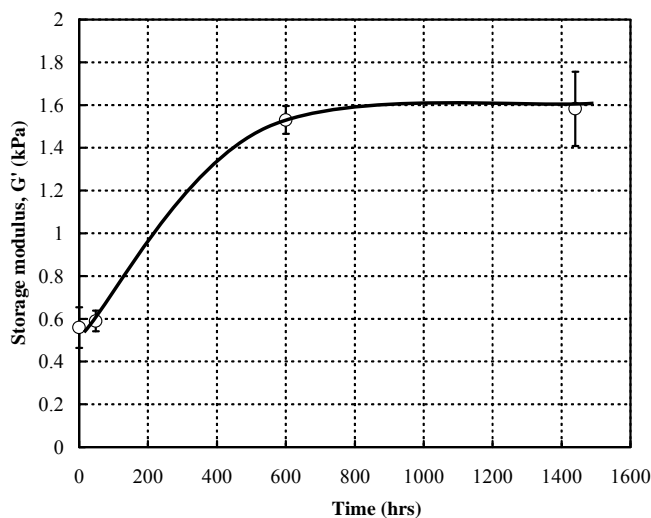


Figure 6 Storage modulus of 10% bentonite slurry versus time after preparation

Thixotropic Effects

Since it is assumed that the pore fluid is responsible for the improved cyclic resistance of sand-bentonite specimens, it is expected that the thixotropic nature of the pore fluid will improve liquefaction resistance with time. Two 3% bentonite cyclic triaxial specimens are prepared as described earlier, but with additional secondary consolidation time: 94 hours and 238 hours, respectively. The specimens are then tested at a $CSR = 0.2$. The number of cycles to liquefaction is presented in Figure 7 along with the results for clean sand and the reference 3% bentonite (22 hours consolidation) at similar CSR . The results show a significant increase in cyclic resistance with time. The specimen with 238 hours of secondary consolidation does not liquefy and shows a pore pressure generation curve similar to that shown in Figure 2 for 5% bentonite.

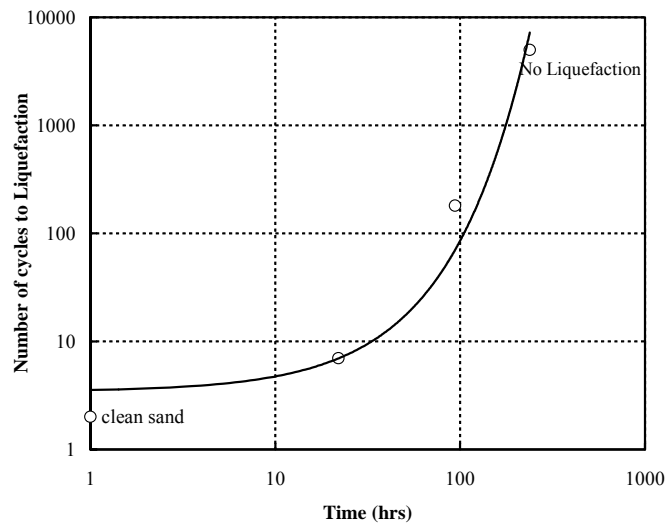


Figure 7 Cyclic resistance of 3% bentonite specimens versus secondary consolidation time at $CSR = 0.2$

CONCLUSIONS

The paper presents the results of cyclic triaxial and resonant column tests performed on dry-mixed, sand-bentonite specimens with 0, 3, and 5% bentonite. In addition, it includes rheological tests conducted on bentonite slurries representing the pore fluid formed inside the sand with 3% bentonite.

The results demonstrate that in the presence of bentonite there is an order of magnitude increase in the number of cycles that produce liquefaction at any given CSR , and an increase in the minimum cyclic loading required to produce liquefaction. This behavior is a result of the delayed generation of excess pore pressures. Resonant column test results show an increase of the strains required to initiate excess pore pressure generation.

The response of the sand-bentonite mixtures is tied to the pore fluid that is formed: a bentonite gel with essentially elastic response up to 1% shear strain. The thixotropic nature of the pore fluid is reflected in the increased cyclic resistance measured over time. In addition to supporting the use of bentonite for mitigating the liquefaction susceptibility of sands, the work provides increased insight into the role played by plastic fines in the cyclic response of natural soils.

ACKNOWLEDGEMENTS

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REFERENCES

- Barnes, H. A., Hutton, J.F., and Walters, K. (1989). "An Introduction to Rheology". *Elsevier Science Publishers*, Amsterdam, The Netherlands, 199 p.
- Barnes, H. A. (1997). "Thixotropy - a review." *Journal of Non-Newtonian Fluid Mechanics*, 70(1-2), pp. 1-33.
- Black, D. K. and Lee, K. L. (1973) "Saturating Laboratory Samples by Back Pressure" *Journal of Soil Mechanics and Foundations Division*, ASCE, Vol. 99, No. SM1, pp. 75-93.
- Drnevich, V.P., and Richart, Jr., F.E. (1970). "Dynamic prestraining of dry sand" *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 96, No. SM2, pp. 435-469.
- Haldavnekar, V., Bobet, A., Santagata, M., and Drnevich, V. (2003). "Soil treatment with a thixotropic fluid: an autoadaptive design for liquefaction prevention." *Proceedings of the 11th International Conference on Soil Dynamics & Earthquake Engineering and 3rd International Conference on Earthquake Geotechnical Engineering*, Vol. II, pp. 553-560.
- Ishihara, K. and Koseki, J. (1989). "Discussion of cyclic shear strength of fines-containing sands." *Earthquake Geotechnical Engineering, 12th International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro, pp. 101-106.
- Ishihara, K. (1993). "Liquefaction and flow failure during earthquakes." *Géotechnique*, Vol. 43, No. 3, pp. 351-415.
- Ishihara, K. (1996). *Soil Behaviour in Earthquake Geotechnics. 1st edition*, Oxford, Clarendon Press, 350 p.
- Ladd, R.S. (1977). "Specimen preparation and cyclic stability of sands." *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 103, No. 6, pp. 535-547.
- Salgado, R., Bandini, P., and Karim, A. (2000). "Shear strength and stiffness of silty sand" *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 126, pp. 451-462.
- Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder Jr., L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soil" *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 127, No. 10, pp. 817-833.
- Wang, W. (1979). "Some findings in soil liquefaction." *Water Conservancy and Hydroelectric Power Scientific Research Institute*, Beijing, China.