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Journal of Environmental Protection and Ecology 20, No 1, 135–145 (2019)

Risk assessment

SOIL IMPROVEMENT TO COUNTER LIQUEFACTION USING COLLOIDAL SILICA GROUT INJECTION

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Abstract. Soil liquefaction due to earthquakes is a major reason of damage to buildings and other structures. This study deals with soil improvement against liquefaction by injection of a particular stabiliser, colloidal silica, which is nontoxic and stable. Laboratory experiments were performed to determine the effects of colloidal silica grout injection regarding soil strength and deformations. The experiments involved static and dynamic triaxial tests on untreated and treated soil samples. The variables used in the tests are the relative density (loose – 40%, medium – 60% and dense – 80%), the confining pressure (100 and 300 kPa), and the curing period of silica treated samples (7 and 28 days). The results clearly indicate the significant increase in strength of the soil with colloidal silica injection. Furthermore, the relative increase is the highest in the sand of the lowest relative density which is the most probable candidate for soil improvement. The observations that the increase in the strength of colloidal silica treated sands with curing time is gradual and continuous add to the advantage of this method for use in soil improvement works. By using the dynamic test results, the equivalent Young modulus (or shear modulus) and the hysteretic damping ratio of untreated and treated soils are compared.

Keywords: colloidal silica, earthquake, soil improvement, liquefaction, triaxial tests.

AIMS AND BACKGROUND

Soils may not always have the required engineering properties suitable to support structures during their lifetime, especially under the effect of the natural hazards. For instance, a damaging phenomenon that can occur in the ground during an earthquake is liquefaction. Soil liquefaction causes significant effects on the ground surface and in structures. Damage by liquefaction has been proven by past earthquakes – especially those occurring near urban areas in recent years – such as Tokyo Bay area, Japan in 2011, Christchurch, New Zealand in 2011 and Kocaeli (Izmit), Turkey in 1999. These include the surface eruption of sand and water, large settlements, large amplitude ground movements, a reduction in the bearing capacity, damage to retaining walls, flows affecting houses, permanent horizontal

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deformations – called lateral spreading – on the ground and damage to underground structures[1–4]. Figure 1 shows an example of partially overturned building in Adapazari during the 1999 Kocaeli (Izmit) earthquake because of liquefied and weakened soil beneath. It clearly illustrates that even in soils where a problem is not expected to occur under static conditions, the ground can lose its bearing capacity as a result of liquefaction caused by earthquake loading. Soil improvement practices can be applied and structural measures can be taken, to reduce or eliminate the effects of soil liquefaction. Otherwise, infrastructures and superstructures can be subjected to severe damages during earthquakes[5]. Therefore, numerous studies have been carried out to predict the places where liquefaction may occur before an earthquake, determine the liquefaction areas, and take necessary precautions[5–7].

Different techniques such as dynamic compaction, vibroflotation, stone columns, compression injection, soil replacement, drainage applications, and chemical injection can be used as soil improvement methods. In recent years, passive site remediation such as injection of stabilisers into the soil has been under development as a non-disruptive mitigation of liquefaction risk at locations of existing structures[8]. In this method, the injected substance moves with natural or augmented groundwater flow and fill in the voids between the soil particles. Improvements are observed in the strength and deformation properties of soil after filling the voids in the ground with injection substances to produce a cohesive effect. Some parameters, such as distribution of grains, grain size distribution, chemical characteristics of the ground and the diameter of pore sizes should be taken into consideration when choosing an injection substance. Most recently, colloidal silicates have been widely used because they have very fine grain sizes and their viscosity is close to water[8–10]. As a result, they penetrate into the smallest pores among soil grains like water, their gelation times can be easily adjusted and they increase the cohesion between grains properly. The required hydraulic gradient can be provided by applying injection pressure or low-head injection and extraction wells. The fact that the gelation time can be adjusted with an accelerator material provides ease of application. The important decision with injection of colloidal silica is about the silica concentration and gelation time as it affects the progress of the injected material in the voids between the soil particles and the treatment results on strength and deformation properties of the soil. The silica concentration of the injection material can be controlled by the mixing ratios of primary and auxiliary materials, which affect the strength of the improved ground. The gelation time of the injection material can be controlled by an acid accelerator. It can be extended by decreasing the pH and increasing the acid ratio in the mixture[11]. A pH level between 5 and 6 is necessary for the shortest gelation times[12]. Several studies on the gelation times exist in the literature[13,14]. Gallagher and Mitchell[15] showed that colloidal silica injection provided significant resistance against deformation in loose sand under repeated loadings.

The purpose of this study is to determine the change in strength and deformation properties of sandy soil when treated with colloidal silica. The sand sample was obtained from Denizli city which is in earthquake risk zone 1, the highest risk zone of Turkey. Denizli is a typical mid-size city in Turkey with substantial building and infrastructure damage potential, including liquefaction, from expected earthquakes in near future[16,17]. The strengths of the injected soils were determined in the laboratories by static and dynamic tests. The results of the experimental study are evaluated and conclusions are presented herein.

Fig. 1. Partially overturned building in Adapazari, Turkey during the 1999 Kocaeli (Izmit) earthquake because of liquefied and weakened soil beneath

EXPERIMENTAL

Sieve analyses were carried out on differently graded sand samples taken from various quarries around Denizli to determine the sand to be used in this study. As a result of the sieve analyses, it was decided to study the samples within the grading range determined for liquefiable soils. Figure 2 shows that the grain size distribution of the sand soil used in this study falls into the generally accepted range of liquefiable soils. According to grain size distribution, the soil can be classified as clean sand because the fines content is less than 5%.

In this study, clean sand samples and improved samples with colloidal silica were subjected to static and dynamic triaxial tests because triaxial loading is more representative of the loading in the field. Both static and dynamic tests on cylindrical samples were utilised. By using the static tests, the deviator stress difference versus strain relationships corresponding to untreated and treated soils were compared. The level of soil response to earthquake loading depends primarily on the mechanical properties of soil, especially dynamic soil properties. Based on the type of the design or strengthening problem, different soil parameters should be taken into account when considering the seismic response. Many geotechnical earthquake engineering applications require the stiffness and damping properties of soils under dynamic loading. By using the dynamic test results herein, equivalent Young modulus and the hysteretic damping ratio of untreated and treated soils were compared.

The strength and susceptibility of sand to liquefaction depend on the relative density (RD) and confining pressure[18]. Therefore, cylindrical samples were prepared with relative densities of 40, 60 and 80%, and static triaxial pressure tests were carried out at 100 and 300 kPa cell pressures. In dynamic triaxial tests, relative densities of 40 and 60% were used and samples were tested under only a 100 kPa cell pressure because of the capability of the test equipment. The variation of cyclic axial load with time was sinusoidal and the frequency of the cyclic loading was 0.2 Hz. The treated samples for static triaxial tests were cured for 7 and 28 days whereas the samples for dynamic testing were cured only 7 days. The reason why only 7 day cured samples were used for the dynamic tests was the capacity of the equipment. Nevertheless, the tests were sufficient to draw conclusions for the effect of the improvement on the strength and deformation properties of the treated soil. Dynamic triaxial tests were performed by gradually increasing loads according to ASTM D 3999 (Load Controlled Modulus and Damping Test) standard[19]. Dynamic tests were done on clean sand samples and improved sand samples. The latter were loaded for 1000 cycles but no distortion was observed.

During the preparation of clean sand samples, a membrane was fixed on the equipment with O-rings. A porous stone was placed on it first, followed by a sample container called a ‘split mould’. The membrane was flattened using a vacuum. Sand samples with relative densities of 40, 60 and 80% were gradually prepared by pouring the sand from a certain height in 8 layers and tamping in order to achieve a 14 cm height. A porous stone was placed on top. The upper head unit was put in place and a membrane was fixed with O-rings. The sample was maintained by vacuuming up to 30 kPa. The split mould was then removed and the cell was replaced and the unit filled with distilled water. After that, the ambient pressure was set to a desired value and vacuum pumping was finished. The air inside the sample was drawn in by passing carbon dioxide through it more than 45 min; thus, the sample could easily be saturated. Distilled water was passed through the sample, thereby achieving the reverse pressure. The applied cell pressure and reverse pressure were gradually increased. When the B values in clean sand samples was equal to or above 0.97, this operation was terminated. The sample was then consolidated. After this step, loading was applied to the sample to obtain a desired cell pressure. This procedure was repeated for each relative density and cell pressure.

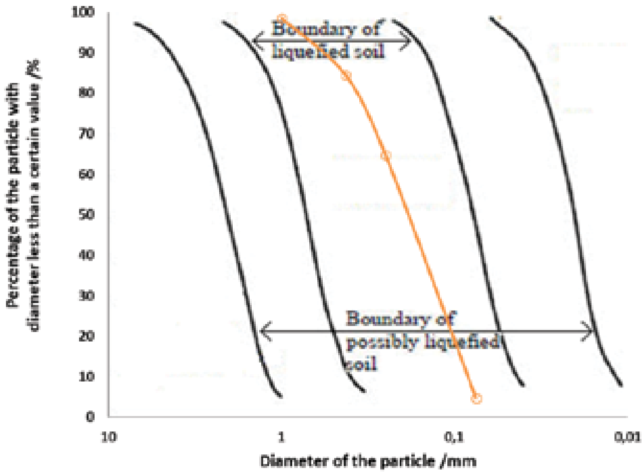


Fig. 2. Grain size distribution of the samples with respect to expected range of liquefiable soils

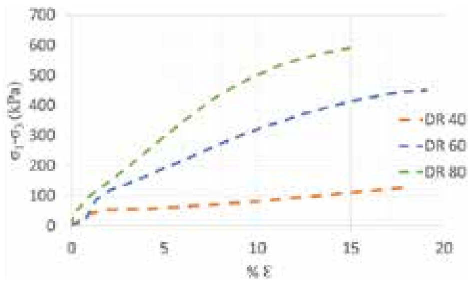
The colloidal silica used as the improvement material includes 40 wt. % of SiO_2. Salt water concentration was used to accelerate the gelation of this material. The ratio of salt water was one part to five parts of the mixture by percentage weight. The gelation time changed from 25 to 30 min, depending on the mixing ratios. A pressure-unit system was designed to inject colloidal silica into the samples after samples had first been saturated with water. The colloidal silica was then injected at the bottom of the samples, and the soil improvements were performed by controlling the colloidal silica emerging from the top. The improved samples were put inside a cup filled with distilled water in an oven at 20°C and cured for 7 and 28 days at a relative humidity of 95%. The cured samples were placed in the triaxial cell. Water was passed through the improved samples for 2 days but the B value did not increase and sufficient permeability could not be obtained. Therefore, tests were carried out without saturating samples.

STATIC TRIAXIAL TEST RESULTS

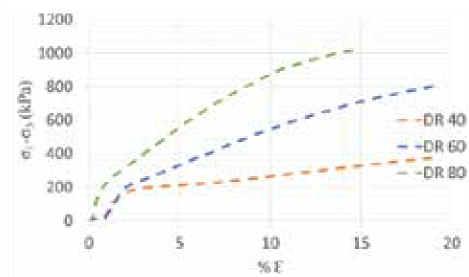
Figures 3*a* and 3*b* show the test results of clean sand samples at a confining pressure of 100 and 300 kPa, respectively. It should be noted that the vertical scales in Figs 3*a* and 3*b* are different. As can be seen in Figs 3*a* and 3*b*, the deviator stress for failure increases as relative density of the samples increases. In essence, the sand samples with 40% relative density has the lowest strength and the shear strength of the samples increases with increasing relative density. Regarding residual pore water pressure behaviour shown in Figs 3*c* and 3*d*, a positive excess pore pressure development was observed in all samples at initial stage. In the 40% loose sample, this initial positive pore pressure increase was significant up to 4% strain levels followed by a slight decrease at higher strains. In the samples with 60 and 80%

(medium and dense sand) relative densities, the positive pore pressure increase was milder followed by negative pore pressure development. The behaviour observed in the residual pore water pressure at 300 kPa follows a similar trend to that at 100 kPa. These observations are in conformity with the results given in the literature for clean sand samples by other researchers[20]. Dilative behaviour was observed at tested samples with relative densities of 60 and 80%. Excess porewater pressure decreased and deviator stress increased starting from low axial strain levels in this investigation – typically on the order of 1–3%. The sample with relative density of 40% strained quickly to approximately 8% axial strain following the exceedance of the maximum deviator stress, indicating an initial liquefaction. After that, dilations started as strains increased. Castro[21] observed and described this type of behaviour as 'limited liquefaction'. Considering that strains at such high values can cause significant damages, samples showing 'limited liquefaction' behaviour can be thought at a state of liquefaction initiation. No static liquefaction was observed at samples with 60 and 80% relative density under the static triaxial loading conditions. The sand at these relative densities show dilative behaviour and there will be no liquefaction potential under static loading conditions. These results indicate that the static liquefaction resistance increases with increasing relative density. Regarding the effect confining pressure, test results in Figs 3*a* and 3*b* show that the deviator stress for failure increases with increasing confining pressure. In other words, the shear strength and the static liquefaction resistance of sands depends on the confining pressure and they increase as the confining pressure increases. Kramer and Seed[18] had similar observations and concluded that the static liquefaction resistance, defined as the shear stress increase under undrained conditions required to initiate liquefaction, was consistently observed to increase with increasing relative density and confining pressure, and to decrease with increasing initial shear stress level.

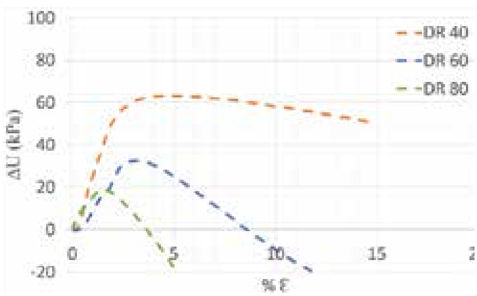
Figures 3*e* and 3*f* show the test results of improved sand samples at cell pressures of 100 and 300 kPa, respectively. The results for both 7 and 28-day-cured samples are presented in the figures. Because pores between grains were filled with colloidal silica in the improved samples, it is difficult to saturate these samples with water, or longer times are required. Similar observations were reported by other researchers[15]. Thus, residual water pressure values were not included for the improved samples. Considering Figs 3*a*–3*b* and 3*e*–3*f*, significant increases in the strength of sand samples after the treatment can be observed. According to the test results for the improved sand samples, higher strength increases were observed in the samples with 40% relative density. The strength increase for these samples was more than four times higher in the samples cured for 7 days. The major reason for this was the higher pore amount and because these pores were almost completely filled with chemicals. When 28-day-cured samples were considered, the increases were even higher. This result reflects the continuous formation of the bonds after gelation. There are examples of this type of observation in literature[8,22]. Yonekura and Miwa[22] reported that the unconfined compressive strength of colloidal silica treated sands increased constantly to reach almost four times the initial strength after 1000 days. They performed the tests at 7, 28, 100, 200, 300, 400, 500, 600, 800 and 1000 days. The treated samples with 7 days curing reached about the same strength levels regardless of the relative density of the sand. However, as number of curing day increases from 7 to 28, the peak deviator stress values differ: peak deviator stress increases with increasing relative density. This difference in deviator stress values decreases at higher strain levels and eventually they somewhat approach each other.



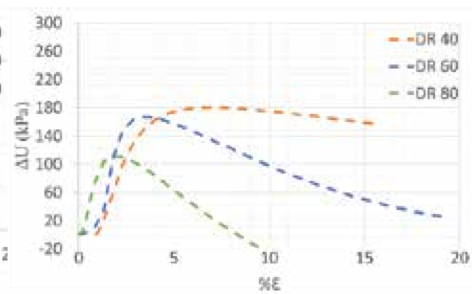
a – $(\sigma_1 - \sigma_3) - \varepsilon$ curves for natural soil at a cell pressure of 100 kPa



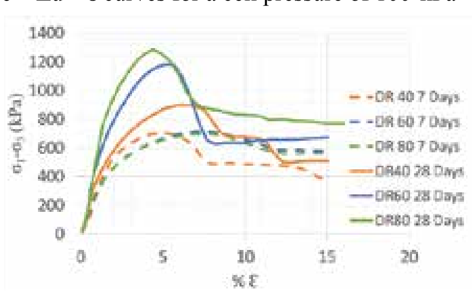
b – $(\sigma_1 - \sigma_3) - \varepsilon$ curves for natural soil at a cell pressure of 300 kPa



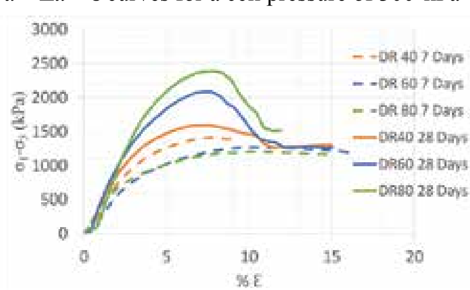
c – $\Delta u - \varepsilon$ curves for a cell pressure of 100 kPa



d – $\Delta u - \varepsilon$ curves for a cell pressure of 300 kPa



e – $(\sigma_1 - \sigma_3) - \varepsilon$ curves for treated soil at a cell pressure of 100 kPa



f – $(\sigma_1 - \sigma_3) - \varepsilon$ curves for treated soil at a cell pressure of 300 kPa

Fig. 3. Test results for natural and treated sand samples at cell pressures of 100 and 300 kPa

The results clearly indicate the significant increase in strength with colloidal silica injection. Furthermore, the relative increase is the highest in the sand of the lowest relative density which is the most probable candidate for improvement considering the loading conditions in the field. The observations that the increase in the strength of colloidal silica treated sands with curing time is gradual and continuous add to the advantage of this method for use in soil improvement works.

DYNAMIC TRIAXIAL TEST RESULTS

By using the test result of this study, the equivalent Young modulus was calculated from amplitudes of cyclic deviator stress and cyclic axial strain, and the hysteretic damping ratio was obtained from a hysteresis loop of the relationship between deviator stress and axial strain as defined in the Japanese Geotechnical Society Standards 2000 (Ref. 23). Equivalent Young modulus can be converted to dynamic shear modulus by using different approaches, e.g. the Hardin and Drnevich model[23]. These and similar standards (ASTM 2003) are commonly used by engineers in different parts of the world to determine the stiffness and damping properties of soils from the cyclic triaxial tests. For example, Kataoka et al.[24] used the standards to compare different type of samplers by evaluating physical properties, deformation characteristics and liquefaction strength of samples. Another example is that Yamashita et al.[25] used the same procedure to characterise the undrained cyclic deformation properties of young sand deposits for seismic response analysis. The Equivalent Young modulus and the hysteresis damping ratio for treated and untreated soils were calculated herein by using 5 cycles and 10 cycles of the data as described in the standards. Because the values at 5 and 10 cycles of the data were very close to each other, the values calculated for 10 cycles are presented in Fig. 4.

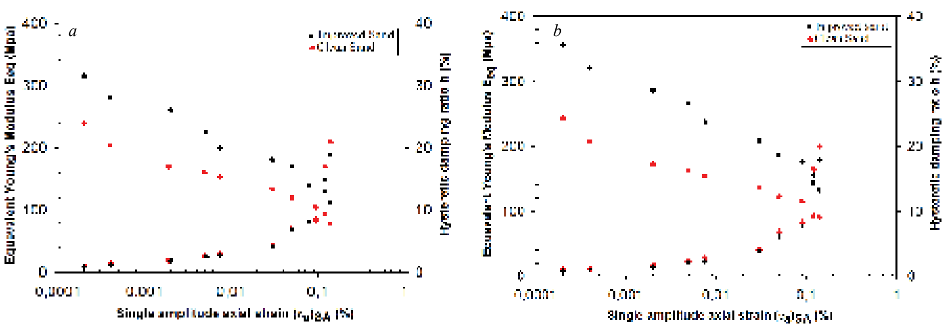


Fig. 4. Equivalent Young modulus versus single amplitude axial strain and hysteretic damping ratio versus single amplitude axial strain curves of the samples with 40% (*a*) and 60% (*b*) RD

Figures 4*a* and *b* show the equivalent Young modulus and the hysteresis damping ratio values obtained for the samples with 40 and 60 % relative densities, respectively. The results indicate that significant increases were observed in the elastic modulus values of the samples improved with colloidal silica. The increase is in the range of 50 to 70%. Also, decreases were seen in the hysteretic damping ratios of the improved samples, but they were not significant.

CONCLUSIONS

Soil improvement may be required at some sites before the new constructions or after the reassessment of existing structures. Among different soil stabilisation methods, this study particularly focuses on improvement by colloidal silica injection into the soil. In this study, the properties of soil samples taken from a nearby quarry in Denizli were improved by using colloidal silica, and the results of the improved samples were compared with those of untreated soil samples. The ease of use of this stabiliser was observed because of its rapid penetration characteristic and adjustable gelation time. Colloidal silica delivery into the voids between soil grains is achieved by both the density of the colloidal silica mixture and the hydraulic gradient. One of the most significant features of colloidal silica is its ability to reach the smallest pores that only water can reach because its density and viscosity is close to water. The required hydraulic gradient can be provided by applying injection pressure or low-head injection and extraction wells. The fact that the gelation time can be adjusted with an accelerator material provides ease of application. Considering static and dynamic triaxial test results herein, colloidal silica injection into the sand soil is very promising against liquefaction. The results clearly indicate the significant increase in strength of the soil with colloidal silica injection. Furthermore, the relative increase is the highest in the sand of the lowest relative density which is the most probable candidate for improvement considering the loading conditions in the field. The observations that the increase in the strength of colloidal silica treated sands with curing time is gradual and continuous add to the advantage of this method for use in soil improvement works. The results indicate that significant increases were observed in the elastic modulus values of the samples improved with colloidal silica. Also, decreases were seen in the hysteretic damping ratios of the improved samples, but they were not significant. The results of this study and other researchers show that colloidal silica grout injection into sand for soil improvement is an effective and promising method against liquefaction.

Acknowledgements. This study was supported by PAUBAP projects with numbers 2012FBE001 and 2014HZL003. Static and dynamic triaxial tests were performed at the soil mechanics laboratory of the Civil Engineering Department of Pamukkale University and geotechnical laboratory of Dokuz Eylul University, respectively. The authors thank to Prof. Gurkan Ozden, Assist. Prof. Ender Basari, Dr. Ramazan Manav, and Assist. Prof. Engin Nacaroglu for their help and support.

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Received 8 January 2019
Revised 13 February 2019

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