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RESEARCH PAPER

A laboratory and in-situ investigation of silica colloid injection on reducing liquefaction potential in young coastal sediments in the south of the Caspian Sea

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Abstract
In this study, soil samples with different weight percentages of silica colloids were subjected to uniaxial compression strength (UCS) tests in a parking lot in Anzali port (northern Iran) to estimate the suitable amount of these colloids to prevent soil liquefaction (10 wt.%). Next, by performing triaxial cyclic tests, the behavior of native sand and the one stabilized with optimal silica colloidal content (10 wt.%) was studied to ensure the occurrence of liquefaction with this content. Afterward, the soil of the study site was improved through a straightforward method, which includes driving screened pipes and injecting silica nanocolloids into the soil by these pipes with appropriate injection pressure. Key parameters in this method include accurate determination of injection pressure, gelation time, grain-size distribution of soil, and proper distribution of silica colloids at the site. In the next step, the effectiveness of the proposed improvement method was assessed by performing standard penetration tests and measuring the hydraulic conductivity. Overall, the results revealed that the proposed improvement method significantly enhanced the standard penetration number, and the formation of colloidal silica gel lowered the hydraulic conductivity of the study site. For instance, the average permeability in the first layer of unstabilized soil was 2.1 × 10-2 cm/s, which reaches 1.6 × 10-8 cm/s by performing the grouting operation. Finally, the relationship $q_{uF} = 0.68 \times q_{uL}$ was obtained between laboratory and field UCS values of the study site by sampling the soil stabilized in situ and performing UCS tests on them.

Keywords: Silica Colloid; Sand Deposits; Liquefaction; Cyclic Triaxial Strength.

Introduction

To date, several methods have been introduced for reducing liquefaction risk in soils prone to liquefaction. The idea of lowering liquefaction risk by colloidal silica solution was introduced in the 1990s by (Gallagher and Mitchell ,2002). They performed a series of cyclic triaxial tests to investigate the ability of colloidal silica to reduce the liquefaction potential. (Díaz-Rodríguez et al., 2008) conducted a set of shear strength experiments on sandy soils at different relative densities and concluded that silica colloid stabilized sands liquefied under higher cycles of loading. In this study, reaching the corresponding 5% double amplitude strain was considered a criterion for liquefaction initiation. (Ikeno et al., 2011) and (Kawamura et al., 2004) carried out a series of full-scale experiments on the effect of the penetration injection method (PGM) and reported a reduction in liquefaction potential. In another study, (Gallagher et al., 2007) carried out large-scale experiments to explore silica colloids' effect on reducing the liquefaction potential. (Conlee, 2010) performed nanosilica injection with Mandrel injection and Well-Packer

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injection and compared their efficiencies. As shown in Fig. 1, the penetration injection method is conducted through passive and controlled methods. In Japan, more than 800,000 cm^3 of soil have been improved by penetration injection method and silica-based grouts from 1999 to 2016 (Rasouli et al., 2016). In the passive colloidal silica method, silica colloid injection is performed through injection wells on one side of the site and observation wells on the other side (Fig. 1). Besides, in the passive method, the gelation time is controlled to prevent the gelation of the silica colloid before going through the recovery process (Gallagher et al., 2007).

The viscosity of silica colloidal slurry plays a critical role in soil improvement rate. The high viscosity of the solution causes the colloidal silica particles to settle around the injection wells, thereby lowering its effectiveness. The ground slope is also among the major parameters in this process. At high slopes, it is not easy to control the gelation time.(Rasouli et al., 2016) applied a controlled penetration method to inject silica nanocloids into sand deposits to lower their liquefaction potential. Some drilling machines are used in this soil improvement method. The high enough setting time to transfer the silica colloid is among the critical parameters in the gradual stabilization method. In contrast, in the controlled penetration injection method, only a specific radius of soil is improved by reducing and adjusting the setting time. In the present paper, a simple type of controlled penetration injection method has been used to improve the parking site of Bandar Anzali Free Trade Zone on the shores of the Caspian Sea. To this end, pipe-percussion machines (commonly used for micropiling in Iran) were utilized. The advantages of this method include its speed, simplicity, and high controllability on the site. In the next step, the mechanical strength of the samples was investigated following the improvement operation to ensure its efficiency. For this purpose, a series of UCS tests were performed on the samples taken from the study site, the details of which are summarized as the following.

Materials and methods

Geographical and geological characteristics of the site

Our case study is the parking lot of the Bandar Anzali Free Trade Zone on the southwest coast of the Caspian Sea. Fig. 3 presents an aerial photo of the site. Geographical location and access roads to the region are in the range of 49° 00′ to 49° 30′E and 37° 00′ to 37° 30′ N. The most important and best path to access this area is the first-main road of Rasht, which connects Tehran to Karaj, Qazvin, and then Rasht. This site is generally warm and humid in the southern regions of the Caspian Sea.

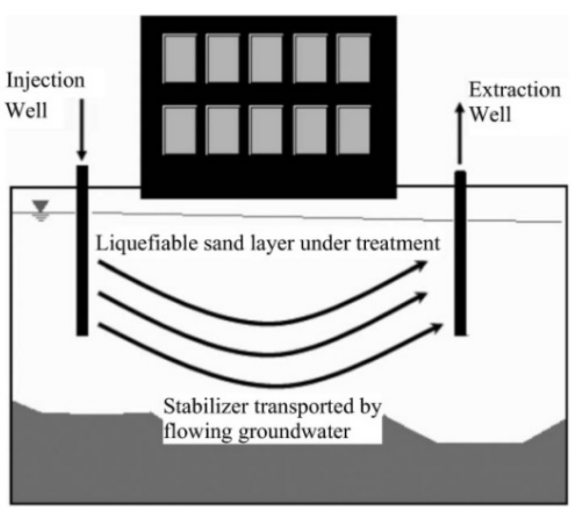


Figure 1. Schematic image of passive injection operation (Gallagher et al., 2007)

The average temperature is about 5 to 9℃ in January and 25 to 26℃ and sometimes above 30℃ in July. The amount of rainfall near Anzali port is about 1,500 mm per year, which declines in the first six months of the year (Lahijani et al., 2009). The temperature differences in off-shore and on-shore in the Caspian Sea region and dependence on the spread of gentle winds create some winds in this part of the Caspian Sea coast. The wind flow direction in this area is from west to east. Caspian Sea water level has started to rise since 1977 such that a water level rise of about 2.4 m has occurred until March 1996 (Kroonenberg et al., 2000).

Geologically, Anzali port is located south of the Caspian Sea. This area includes some parts of the Alborz and Kopet Dag structural zones. The coastal deposits of the Caspian Sea have spread from the north to the middle parts of this region. The rock units are exposed only in the form of small outcrops in the southwest of this area. Precambrian metamorphic rocks, including amphibolite and gneiss, are abundant in the southwestern part. At outmost southwest, a wide range of Paleozoic detrital deposits are seen. These deposits appear in the more northern parts along with Permian limestone. Jurassic deposits exist as limestone and limestone in the south of Anzali (Nogol-e-Sadat, 1991). Tectonics and morphology of the Alborz Mountains on the southern shore of the Caspian Sea are associated with fractures, faults, and thrusts that run parallel to the Caspian Sea coast (Fig. 2).

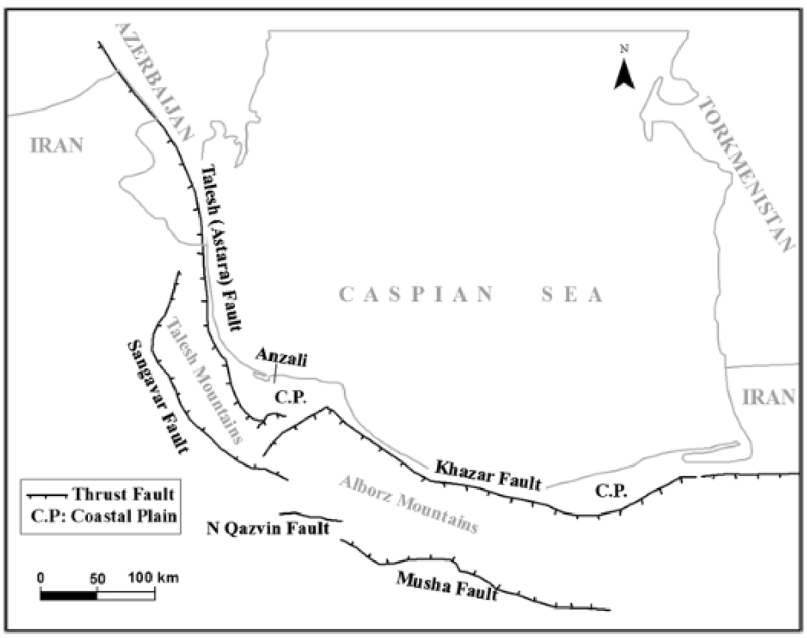


Figure 2. Map of main faults of studied region (Guest et al., 2007)

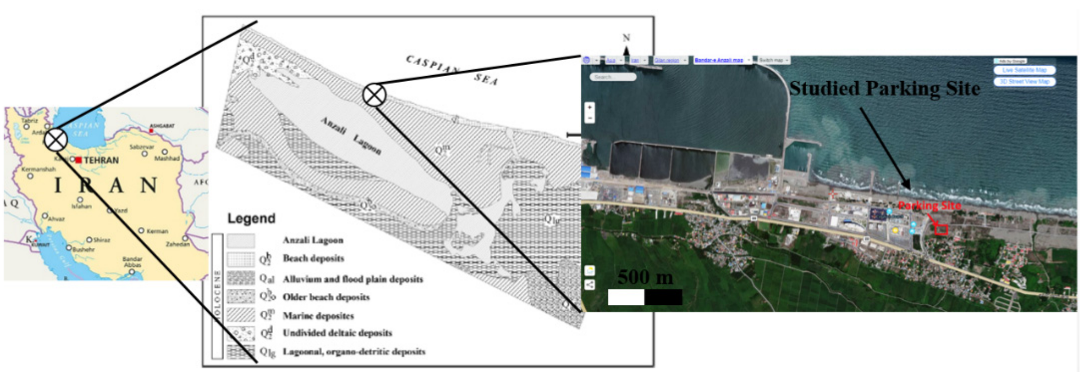


Figure 3. Aerial photo of the parking site of the Bandar Anzali Free Trade Zone

The bed of the Caspian Sea in the north of Anzali port is 100 to 250 m high. Therefore, it cannot be attributed to the accumulation of dunes due to the Caspian submarine currents or the accumulation of Sefidrud and Shafarud sediments. Instead, it is most likely due to volcanic activity in the buried form. In terms of age, the coastal plain of Gilan belongs to the post-Pleistocene. Therefore, the Anzali wetland should have been formed in the late Pleistocene and possibly Miocene. The wetland surface is composed of deltaic sediments, and the wetland itself has been formed by the expansion of coastal sediments as coastal tabs and coastal dams. Faults, especially in the southwest of Bandar Anzali, are mostly northwest-southeast (Guest et al., 2007). Therefore, the Anzali wetland should have been formed in the late Pleistocene and possibly Miocene. The lower surface of the wetland is composed of deltaic sediments, while the wetland itself is formed by the growth of coastal sediments in the form of coastal tabs and coastal dams. Fault, especially in the southwest of Bandar Anzali, is mostly northwest-southeast (Clark et al., 1975).

Geotechnical characteristics of the site

The subsurface geology and soil layering were studied by drilling four (20 m deep) boreholes at the study site. The geological profile of the study site was drawn by studying the cores extracted from drilling, determining the geological material, and performing grain size distribution tests (Fig. 4). Based on these results, three separate geological layers are identified in this place up to a depth of 10 m. These layers are mainly composed of sand and vary from fine-grained to silty sand. According to the Unified Soil Classification System (USCS), soil samples are named SP, SP-SM, and SM. Examining hand samples showed that Anzali sand is of silicate type. Fig. 5 presents the grain size distribution curve of different soil layers in the studied area. In addition, the maximum and minimum density were determined according to ASTM D-4253 and ASTM D-4254, respectively. Table 1 presents the physical characteristics obtained from these experiments for these layers. Based on the grain size distribution curve and the soil's relative density, and the groundwater table's existence at 2 m, it can be inferred that sand deposits up to a depth of 10 m are prone to liquefaction. Also, this result is confirmed using the standard penetration test (SPT) number, presenting which is beyond the scope of this article.

Colloidal silica

Colloid silica is a suspension of silica nanoparticles prepared using saturated silicic acid solutions.

Table 1. Physical characteristics of different soil layers of the study site

	layer1	**layer2**	**layer3**
USCS classification symbol	SP	SP-SM	SM
D_{60}(mm)	0.5	0.43	0.2
D_{50}(mm)	0.42	0.4	0.17
D_{30}(mm)	0.33	0.33	0.1
D_{20}(mm)	0.3	0.31	0.08
D_{10}(mm)	0.5	0.12	0.05
Coefficient of uniformity, C_u	2.5	3.5	4
Coefficient of curvature, C_c	1.09	2.11	0.49
Specific gravity, G_s	2.69	2.69	2.69
Maximum index void ratio, e_{max}	0.89	0.83	0.84
Minimum index void ratio, e_{min}	0.5	0.41	0.4

$C_u=D_{60}/D_{10}$,$C_c=(D_{30})^2/(D_{60}.D_{10})$ where D_{10},D_{30}, D_{50},D_{60} are the grain size that correspond to 10%,20%30%,50%,60% passing.

At low concentrations, colloidal silica has a density close to water density and a particle size of 7 to 100 nm (Iler, 1979). During the colloidal silica preparation, solutions are stabilized against gelation. The colloid silica formed particles have a wide or narrow size distribution depending on the production method and conditions. The silica concentration in the colloid used in this paper is 30 wt.%. This product is produced by Isatis Nano Silica Industries Company located in Yazd. The price of each kilogram of this product is about 0.5 \$. Table 2 shows the specifications of this type of colloid silica.

Table 2. Specifications of silica colloids used in this research

initial properties of colloidal silica 30% (provided by Isatis nano silica company, Iran)	
Sio_2(%)	(29-31)
Na_2O_2(%)	0.4
Particle Size (nm)	(16-24)
Specific Surface Area(m^2/gr)	(220-330)
Viscosity, Cp	(3-15)
pH	(9-10)

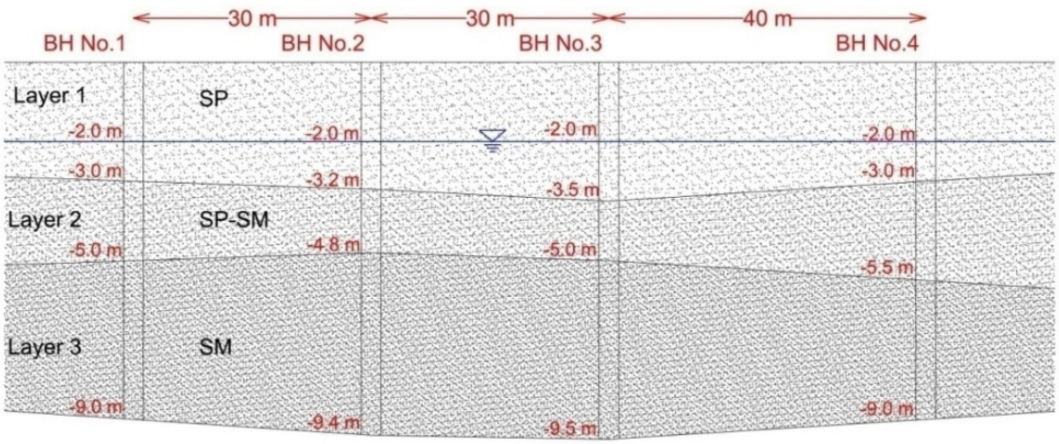


Figure 4. Soil profile drawn based on geotechnical drilling to a depth of 10 m

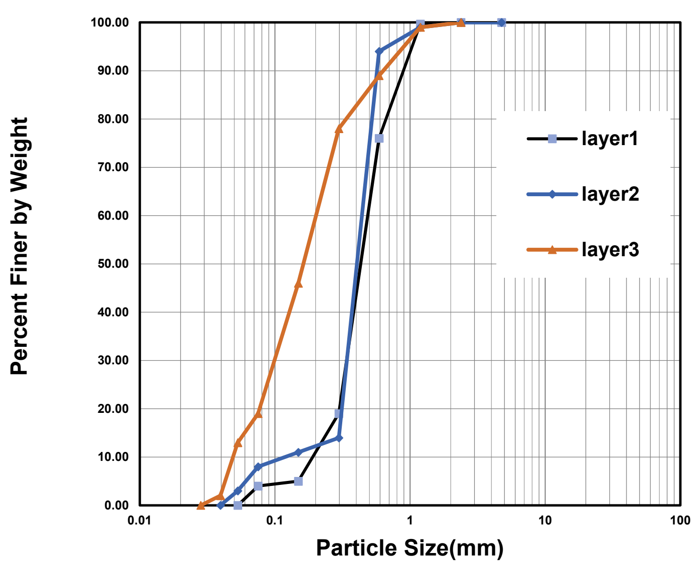


Figure 5. Grain size distribution curve of different soil layers

Schedule of tests and samples' preparation

Experiments

Some laboratory and field tests were performed to determine the geotechnical parameters and effect of the proposed improvement method on soil resistance to liquefaction. For the initial estimation of stabilized soil strength, a soil uniaxial compressive strength (UCS) test was performed according to ASTM D2166. The test was performed using various percentages of colloidal silica. The cyclic triaxial test is among the best laboratory tests to investigate the dynamic behavior of soils and simulate seismic loading (ASTM D5311-92). This test was carried out to estimate the effect of the studied additive on soil improvement. Table 3 shows the laboratory tests program.

Studying the effectiveness of adding silica colloid on the liquefaction resistance of soil layers on a real scale requires performing field experiments. In this respect, the SPT test (ASTM D1586) was performed on soil layers to investigate the initial density of sand deposits in the studied parking area of the Bandar Anzali. These in-situ tests were used to prepare undisturbed soil samples in their natural state. Another basic parameter in the soil liquefaction is the hydraulic conductivity, which was determined using Lefranc permeability (ASTM D-6391-11) performed on the soil layer of the Bandar Anzali Free Trade Zone's parking lot.

Samples preparation

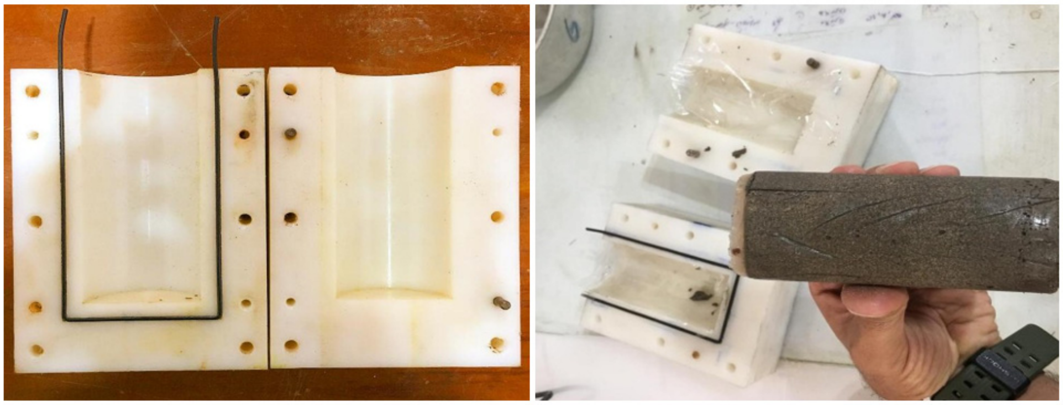
The sampling method is an essential factor in determining and predicting the behavior of natural and improved soils. Due to the sedimentary nature of the study area, the wet sedimentation method was used to prepare a homogeneous sample. In this regard, a sample preparation device is made, as shown in Fig. 6a. Since the samples were tried to be similar to those found in the field, it is imperative to accurately estimate the relative density of sand at the site.

Table 3. Program of laboratory tests performed in this study

Sample layer	Sample	Fine content(%)	Collodial silica(%)	Gel time	Age of testing	type of test
1	SM4U	4	0	0	0	CT
1	SM4C5T7	4	5	5h	7days	UC
1	SM4C10T7	4	10	5h	7days	CT,UC
1	SM4C15T7	4	15	5h	7days	UC
1	SM8C20T7	4	20	5h	7days	UC
2	SM8U	8	0	0	0	CT
2	SM8C5T7	8	5	5h	7days	UC
2	SM8C10T7	8	10	5h	7days	C,UC
2	SM8C15T7	8	15	5h	7days	UC
2	SM8C20T7	8	20	5h	7days	UC
3	SM19U	19	0	0	0	CT
3	SM19C5T7	19	5	5h	7days	UC
3	SM19C10T7	19	10	5h	7days	C,UC
3	SM19C15T7	19	15	5h	7days	UC
3	SM19C20T7	19	20	5h	7days	UC

Note: M= fine content, C= collodial silica, T= treated sample, U= untreated sample, UC=Unconfined compressive test

CT=Cyclic triaxial test.



(a) (b)

Figure 6. (a) Sample preparation mold and (b) A sample stabilized with colloidal silica

Eqs. (1-3), proposed by (Cubrinovskiet and Ishihara, 1999), was used to estimate the relative density (Dr) of sand layers:

$$D_r = \left[\frac{N(e_{\max} - e_{\min})}{9}\right]\left(\frac{98}{\sigma'_V}\right)^{\frac{1}{2}} \tag{1}$$

$$e_{\max} - e_{\min} = 0.23 + \frac{0.06}{D_{50}} \tag{2}$$

$$D_r = \left[\frac{N\left(O.23 + \frac{0.06}{D_{50}}\right)}{9}\right]\left(\frac{98}{\sigma'_V}\right)^{\frac{1}{2}} \tag{3}$$

where Dr is relative density, e_{max} is the void ratio of coarse-grained soil (cohesionless) in its loosest state, and e_{min} is the void ratio of coarse-grained soil (cohesionless) in its densest state. In this method, the energy ratio of the hammer is assumed to be 78%. As the ratio of hammers used in the parking lot is 60%, to use Eqs. (1) to (3), the answer was multiplied by 60/78. Next, cylindrical specimens with a diameter of 7 cm and a height of 14 cm were prepared by the wet sedimentation method according to the relative density obtained from Eq. (1). In addition, maximum porosity (e_{max}) and minimum porosity (e_{min}) of samples were determined according to standards ASTM D4253 and ASTM D4254, respectively. In the method, first, one-sixth of the height of the mold is filled with silica colloidal. Then, the soil is poured into the mold with a funnel (nozzle diameter = 3.5 mm) from a height of 1 to 3 mm above the surface of the colloidal silica solution. Afterward, the silica colloid solution is increased to a height of 2.6 mm. These steps are repeated until filling the entire mold. Fig. 6a depicts the sample made in this way.

Regarding different soil profiles with depth, it is vital to determine the amount of colloidal silica to achieve the desired resistance to reduce the liquefaction potential. The gelation time is a significant parameter to consider because of applying the injection method at the study site (pushing screened pipes and packing them into the soil followed by applying grout pressure).

Test results and discussion

The effect of silica colloid on UCS

UCS tests were carried out on improved specimens with 0, 5, 10, 15, and 20 wt.% of silica colloids according to ASTM D2166 (Fig. 7). (Persoof et al.,1999), conducting some laboratory studies, showed that cemented sands with UCS < 100 kP are prone to liquefaction in typical earthquakes. Also, (Suaza et al. , 2016), by performing UCS tests on cemented specimens, indicated that stabilized sands with a UCS of 70kP against the cyclic load caused by the earthquake with the maximum ground acceleration of 0.3g do not liquefy. In the present study, a series of UCS tests were performed on the improved samples to determine the appropriate amount of injectable silica colloids to lower the liquefaction potential. The results of these experiments were used to prepare a diagram of changes in UCS with colloidal silica concentration for each soil layer (Fig. 8).

Figure 7. Uniaxial compressive strength (UCS) test on treated specimens

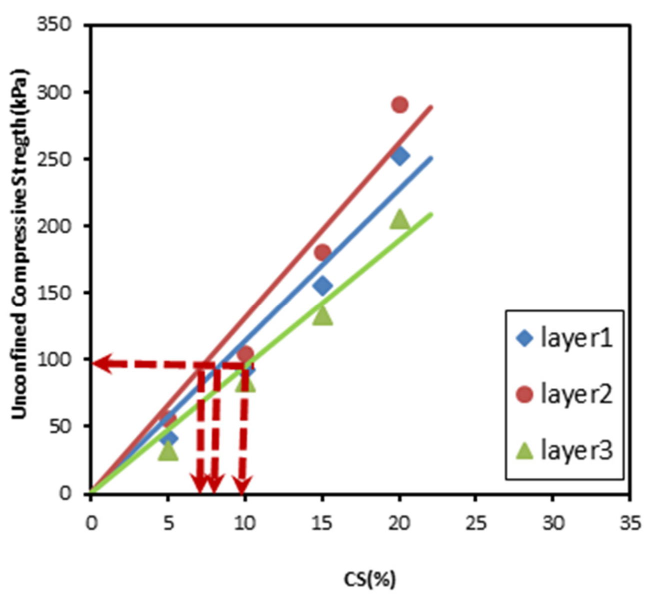


Figure 8. Changes in UCS values with respect to silica colloid percentage

The diagram shows that the UCS increases in all samples with increasing additive concentration. Overall, UCS reaches 100 kPa by adding 10% silica colloid to the third layer, 7.5% to the second layer, and 6.5% to the first layer. As can be seen, in the same silica percentage, samples prepared from the third layer have the lowest UCS due to their higher density. The low UCS of soil samples in the third layer can be attributed to the higher percentage of fine grains in this layer and thus the increase in the distance between sand particles. According to the UCS results, the average amount of injected silica colloid in the Bandar Anzali parking site was considered to be 10 wt.%.

Determining the gelation time

The conversion or gelation time of silica nanocolloids depends on the particles' adsorption rate and mutual reaction. This reaction, in turn, is controlled by several other factors such as particle size and particle surface area. In addition, gelation time depends on the pH and salt concentration of the environment. The gelation time increases with a decrease in the number of particles and an increase in particle size. Generally, the minimum gelation time occurs when 5 <Ph <6. In the case pH is higher than 6 or lower than 5, the gelation time will increase. Besides, the gelation time increases by decreasing the salt concentration in the colloidal silica solution. Overall, the highest gelation time occurs when there is no salt in the solution (Gallagher and Mitchell, 2002). Alkaline solutions cause the particles to ionize and repel each other (Noll et al., 1992). Some studies have reported the gel-turning time to be up to 49 days. However, (Gallagher and Lin , 2005) have reported it to be up to 200 days. In another study, (Persoff et al., 1999), measuring silica colloid-stabilized sand resistance parameters over one year, concluded that the stabilized sand resistance increases continuously during this period.

After performing a series of experiments, (Gallagher et al., 2007) showed that sand stabilized with silica colloid reached its ultimate strength over a processing time equivalent to four times the gelation time. Therefore, the desired resistance can be achieved quickly by reducing the gelation time. In the present research, some experiments with a colloidal silica concentration of 10 wt.% were performed to determine the setting time (Fig. 9). In this project, according to the method of execution and the injection radius, the setting time was 5 h. Based on the test results, the corresponding pH for the setting time of 5 h was estimated to be 3.2.

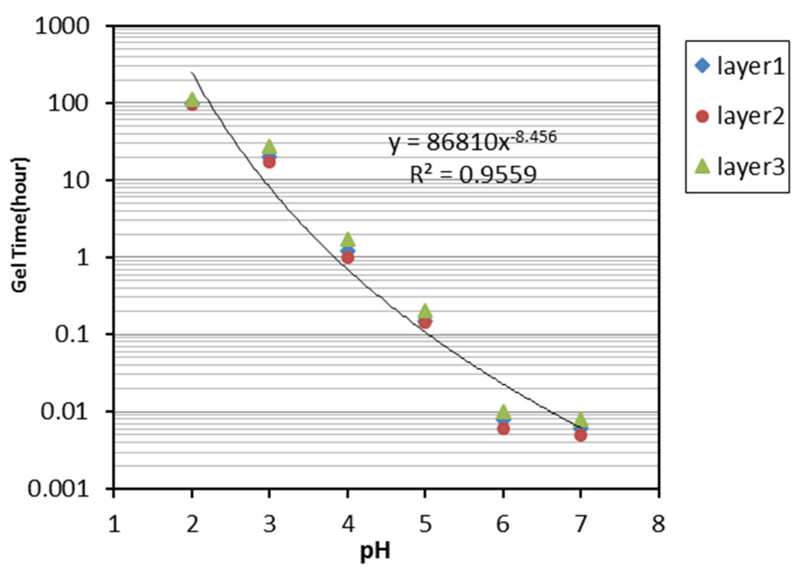


Figure 9. Changes in gelation time with pH

Effect of silica colloid on the dynamic behavior

After determining the optimal amount of colloidal silica additive (10 wt.%) using UCS tests, a cyclic axial test was carried out to ensure its effectiveness in the real conditions of earthquake occurrence and dynamic loading. To this end, a series of cyclic triaxial tests on native and stabilized samples with 10 wt.% of colloidal silica was performed (ASTM D5311-92). Fig. 10a shows how to place the sample in a cyclic triaxial apparatus. Due to filling the voids between the particles with silica colloidal gel, the backpressure was not used to saturate the samples as this pressure destroys the silica colloid bonds. The liquefaction criterion is assumed to reach 5% double amplitude strain in these experiments.

The number of uniform cycles expected from typical earthquakes is between 10 and 20 cycles. In addition, an earthquake with a magnitude of 7.5 is approximately equivalent to 15 uniform load cycles (Seed, 1982). The confining pressure for the corresponding overburden in the first, second, and third layers is 40, 60, and 75 kPa, respectively. Moreover, in this study, the onset of cyclic liquefaction or softening was defined as the cyclic stress ratio (CSR) required to create 5% double amplitude strain in 15 cycles of load application in the cyclic triaxial test. For example, to measure CSR based on 5% double amplitude strain in 15 uniform loading cycles, several cyclic triaxial experiments were performed in each soil layer based on the Dr of the layers (Fig. 10b).

Fig. 11 represents changes of CSR against loading cycles in the third layer. According to Fig. 11, the corresponding CSR for 15 load cycles is 0.23. Fig. 12a illustrates the axial strain changes versus load cycles with Dr = 0.39 and CSR = 0.23. As can be seen, the sample prepared from the third layer reaches 5% double amplitude strain at 13 axial cycles. Furthermore, performing a cyclic triaxial test on samples treated with 10 wt.% silica colloid revealed that none of the samples stabilized up to cycle 100 underwent liquefaction based on 5% double amplitude strain criterion (Fig. 12b).

In-situ implementation of the proposed improvement method

In this study, the injection method was used to increase the resistance of the parking lot of the Bandar Anzali Free Trade Zone against liquefaction. For this purpose, pipe-cutting machines used to perform micropiling in Iran were used, the example of which is shown in Fig. 13 .

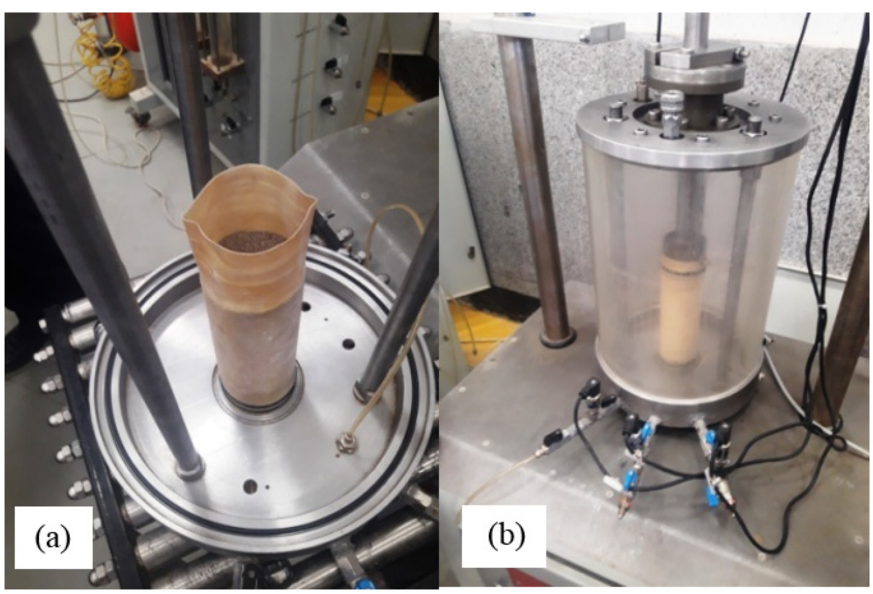


Figure 10. (a) Placing the sample in the test apparatus and (b) Performing the cyclic triaxial testing

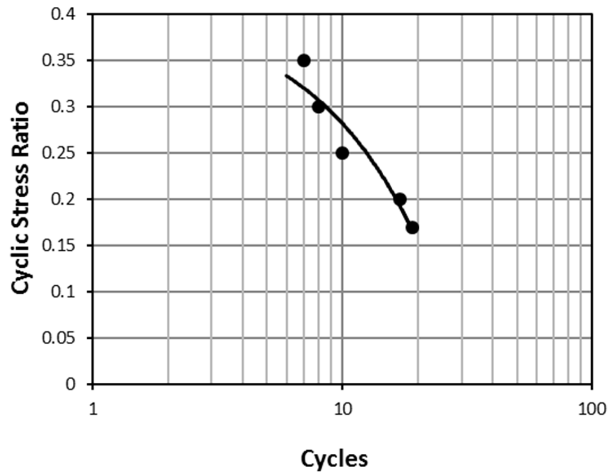
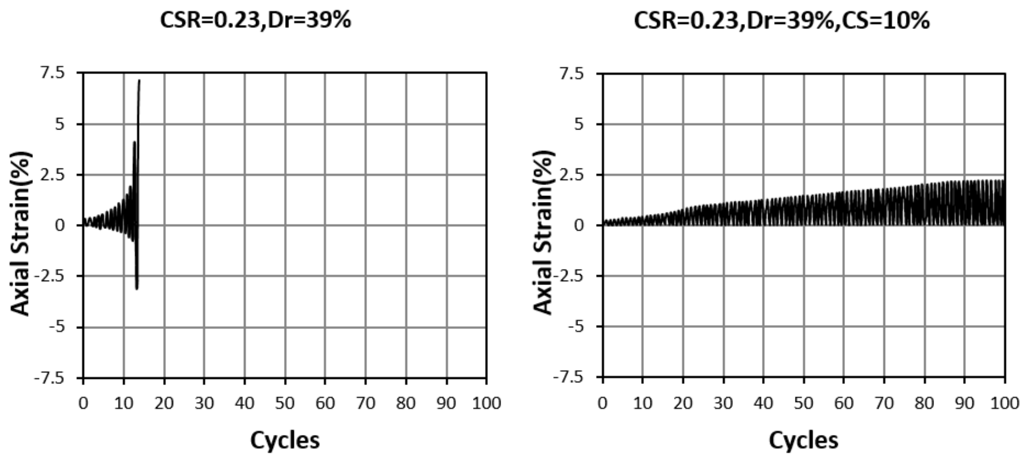


Figure 11. Changes of CSR versus the loading cycle in an untreated sample from the third layer with Dr = 39%



(a) (b)

Figure 12. Axial strain changes versus loading cycle in (a) untreated soil samples and (b) treated soil samples

Figure 13. A typical hydraulic pipe-percussion machine used for micropiling in Iran

The advantages of this method include speed, simplicity, and high controllability of this method on the field operation. Moreover, Fig. 14 illustrates the implementation steps of this improvement method schematically.

In this method, silica colloids were injected using screened pipes that are pushed into the soil shown in Fig. 14a. The height of these pipes is considered 10 m due to the thickness of the liquefaction-prone soil at the study site. Also, the spacing of installed pipes is 2 m, which are packed by a packer. Eventually, they are injected with a pressure of 0.5 MPa. In Fig. 14b, a row of the screened pipes is depicted at the studied site.

Controlling the soil treatment method

Various methods have been recommended to control the effectiveness of the ground improvement method with silica colloidal additive, i.e., SPT and hydraulic conductivity measurements of deposits. Also, UCS and cyclic triaxial tests can be performed on undisturbed soil specimens through sampling undisturbed soil to ensure its liquefaction resistance more accurately. Measuring the silica content is another in situ control method of soil resistance against liquefaction. In addition, using dye such as methyl red can also help investigate the extent of silica colloid penetration. In the present study, soil improvement was controlled by applying SPT and hydraulic conductivity tests of materials, undisturbed soil sampling, UCS, and determination of colloidal silica percentage, details of which are presented in the following.

Standard penetration test (SPT*)*

According to (Gallagher et al., 2007) SPT and shear wave measurements are required to control improvement optimization when using silica nanocolloids. The standard penetration test (SPT) is one of the most widely used and common tests to estimate the geomechanical parameters of granular soils and control the improvement in the world. Therefore, in this site, this test is conducted to control the field performance of the proposed method. The main advantages of this test are its simplicity and no need for high technology, availability around the world, the ability to prepare samples and test simultaneously, the ability to conduct in a wide variety of soils type, extensive database, and also several correlation relationships between SPT test results and main geomechanical parameters of the soils.

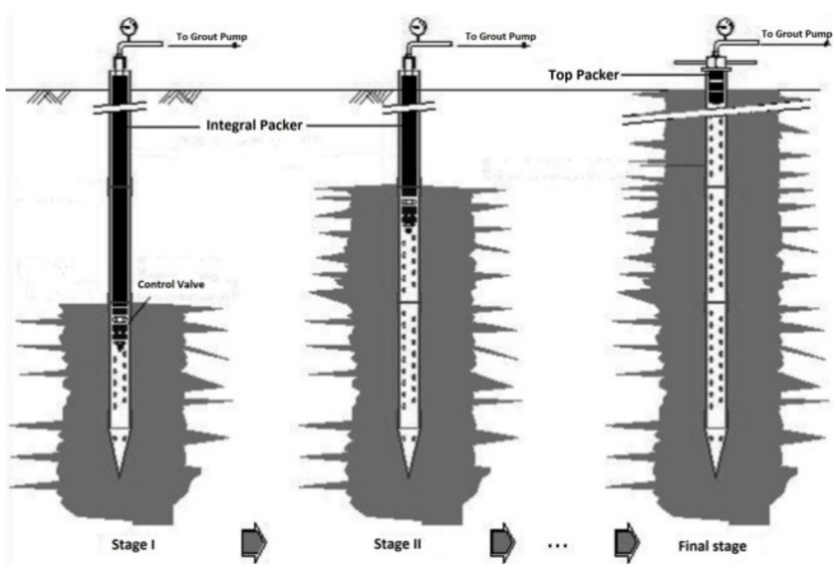
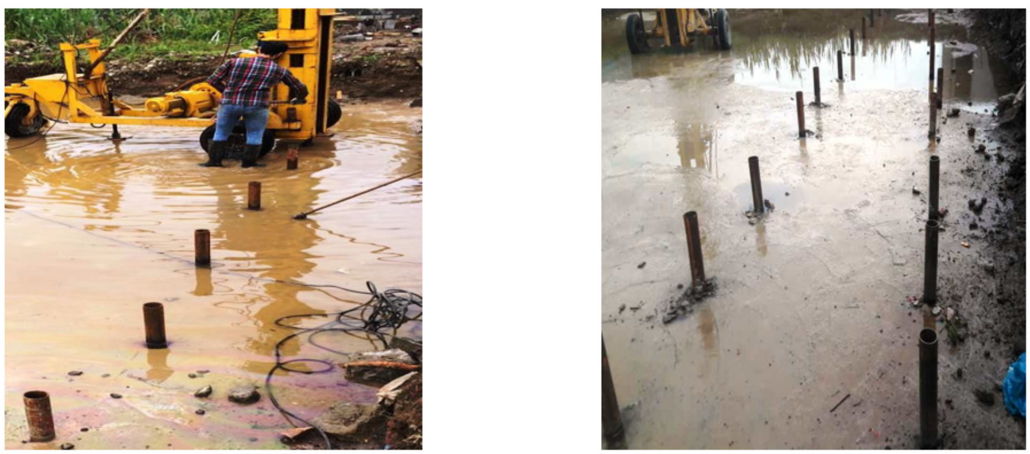


Figure14. Implementation steps of the injection method

In this research, the effectiveness of the proposed improvement method was controlled by drilling 4 boreholes in the parking lot of the Bandar Anzali Free-Trade zone. The radial distance of the boreholes from the injection axis was considered 1 m. Finally, based on the results of these experiments, a graph of SPT number changes in terms of depth was drawn for the study site (see Fig. 16). For ease of comparing the results, the SPT number for native soils is also plotted in these diagrams. As can be noticed, the SPT number in soil layers indicates a significant increase after silica colloid injection operation. However, this increase is less than expected, which can be attributed to the non-uniform distribution of silica colloids in sand deposits and the presence of groundwater flow at the study site. Also, the rate of SPT number increase in silty sands is less than that of clean sand, probably due to the presence of silt particles and subsequent change in soil structure and increasing the distance between sand particles.

Permeability test

One of the important parameters in controlling the improvement of soil layers, especially sand deposits, is their permeability.



(a) (b)

Figure15. (a) Pushing the screened pipes by a hydraulic machine and (b) Pipes implemented at the end of the injection operation

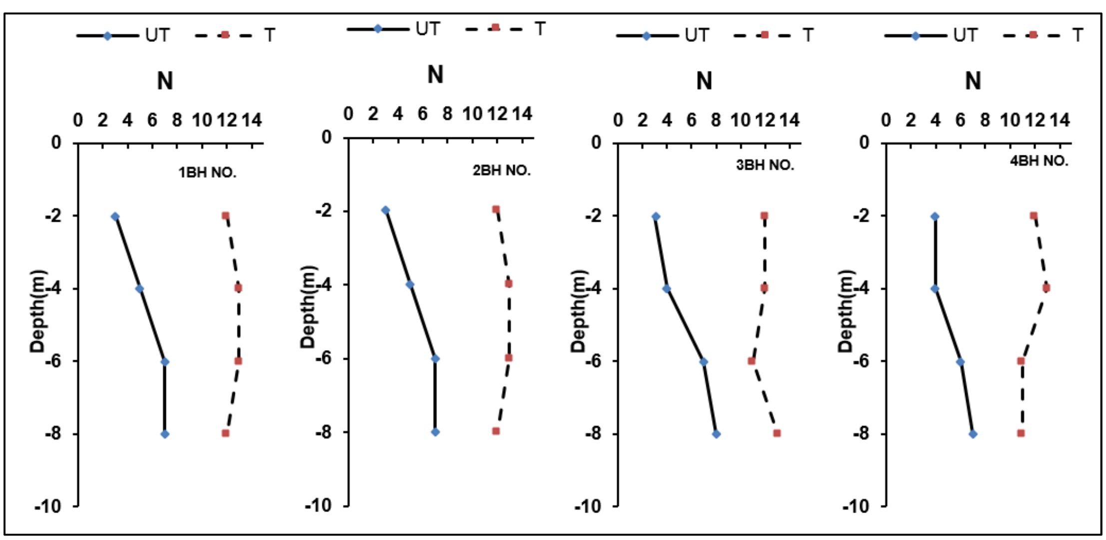


Figure16. SPT results in untreated (UT) sand and treated (T) sand

Performing the Lefranc test allows measuring the permeability changes in soil layers at different depths after the improvement operation. The preparation of the test site and the minimum amount of distribution are very important points in conducting this test. At the study site, the rotary drilling method is used for drilling. Also, the Lefranc test is performed by placing the casing up to the top of the test section and packing the borehole in the upper part (above the test section). In this study, a permeability test with constant load was performed due to the type of soil layers and relatively high permeability coefficient of these deposits to estimate the permeability coefficient in unstabilized soil. In this method, after preparing the test section, water is continuously injected into the borehole with a given pressure until the water level remains constant in the upper point of the test section. Field hydraulic conductivity in different soil layers was determined according to the standard ASTM D-6391-11. The average permeability values in the first to third layers of unstabilized soil were 2.1×10^{-2} cm/s, 2.3×10^{-2} cm/s, and 1.6×10^{-3} cm/s, respectively. Then, by performing the Lefranc test, the permeability of the stabilized layers was determined to be 1.6×10^{-8} cm/s, 1.5×10^{-8} cm/s, and 7×10^{-9} cm/s for these layers, respectively. These values indicate significant reductions compared to those of unstabilized soil. Therefore, it is inferred that the formation of silica colloidal gel between soil layers has reduced soils' permeability.

UCS and silica colloid percentage

An undisturbed sampling of the improved layers was performed to control the effectiveness of the proposed improvement operation. Sampling was done using the Shelby thin wall device. (Gallagher et al., 2007) measure the silica colloid percentage in soil layers by drying the samples containing silica colloids for 2 to 3 days at 200℃. Also, (Rasouli et al., 2016) proposed the novel atomic absorption spectroscopy to measure silica content. In the present research, silica colloid percentage was measured using the method proposed by (Gallagher et al., 2007). The diagrams in Fig. 17 represent variations of soil USC versus a change in the silica percentage in undisturbed samples obtained from the study site. As can be seen, the resistance of the sample increases with increasing the silica colloid percentage. Another important point inferred from these graphs is that the silica colloid percentage in the samples is lower than that of injected silica in the field. This issue can be attributed to the presence of groundwater flows and the non-uniform distribution of silica colloids in the study site compared to the laboratory. As can be seen, the relationship between treated soil resistance and the silica colloid percentage at the study site in different layers is as follows: $q_{Uf} = (7\text{-}8.6) \times (\text{silica content})$. According to the presented relation, UCS of different samples can be obtained by measuring the silica colloid percentage in the samples taken from these layers.

In the following, using the laboratory and field data, the relationship between the average UCS measured in the field and the laboratory for a certain silica colloid percentage was proposed as follows: $q_{uF} = 0.68 \times q_{uL}$
where q_{uF} and q_{uL} are the UCS of undisturbed (field) and disturbed (laboratory) samples, respectively. This relationship, which is extracted using the data obtained from all three layers in the study site (Fig. 18), shows that the actual UCS obtained from the improvement operation by silica colloid injection is about 2.3 of the laboratory test results with the same injection percentage. Therefore, using the mentioned relation, the necessary resistance to prevent the occurrence of liquefaction in the laboratory can be converted to its field value by back analysis and, if necessary, the field injection percentage can be increased. For instance, in the parking site of Bandar Anzali, according to this relation and assuming a UCS of 100 kPa in-situ to prevent the occurrence of liquefaction, the required laboratory resistance is estimated to be 150 kPa.

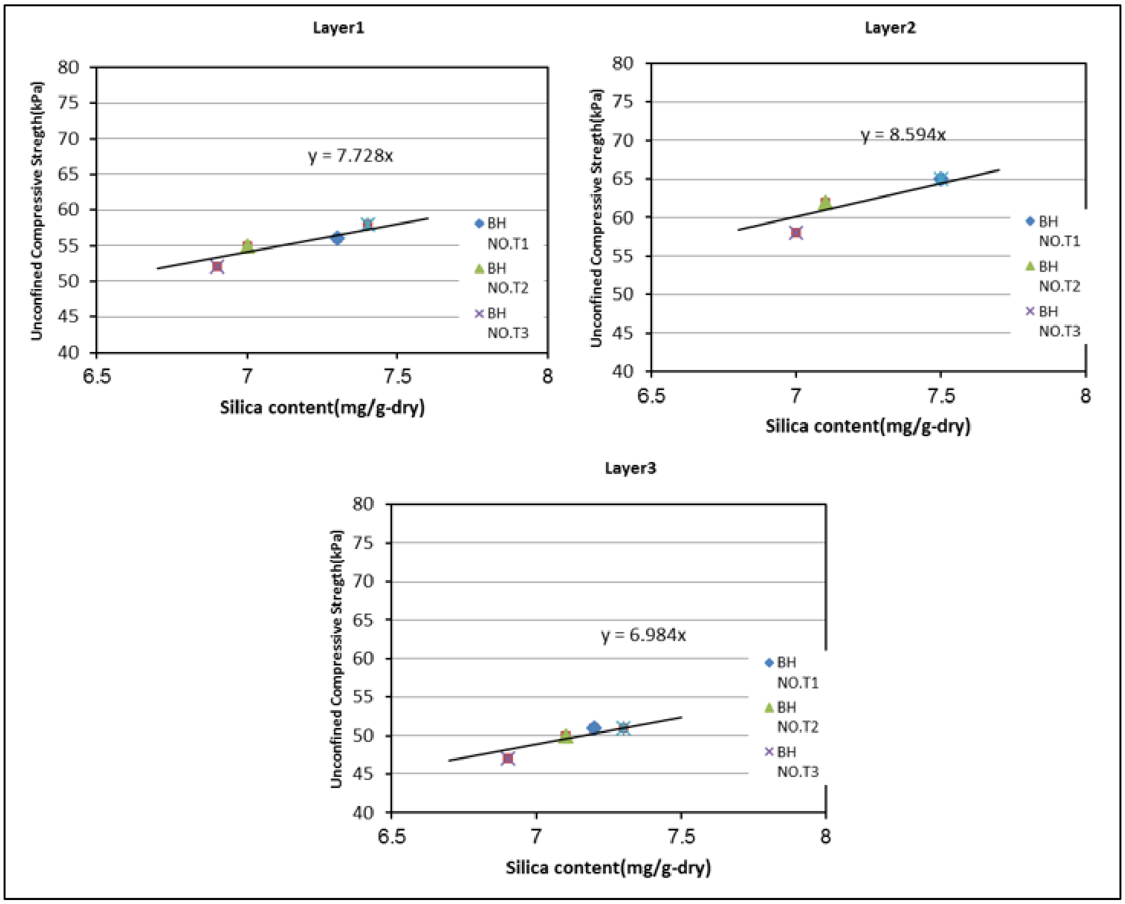


Figure17. Changes in UCS values against the silica colloid percentage at the study site

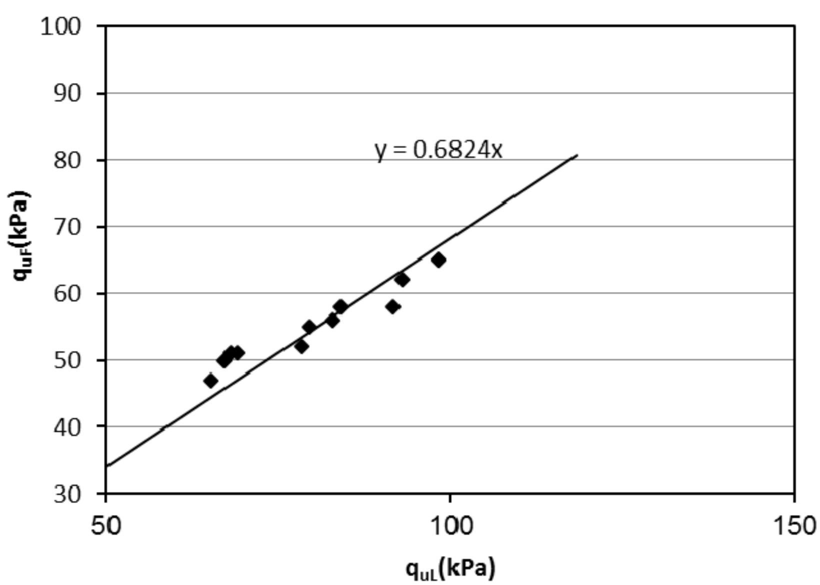


Figure18. Changes in field UCS (q_{uF}) versus laboratory UCS (q_{uL}) for a certain silica colloid percentage

Conclusion

In this paper, the existing engineering methods and tools were used in an innovative way to inject silica colloids into loose sand deposits in a construction site on the southern shores of the

Caspian Sea. For this purpose, the subsurface geological condition of the study site was carefully identified by conducting several geotechnical boreholes, and undisturbed samples were prepared for laboratory experiments. The results showed that shallow sediments are composed of three separate layers, the predominant material of which is sand, and these sediments are prone to liquefaction up to a depth of 10 m. Therefore, the natural relative density of soil layers was determined accurately by performing the SPT. In addition, the static and dynamic behavior of natural and stabilized soils were tested separately by making samples by the wet sedimentation method in silica colloidal solution with relative density in situ. Afterward, the behavior of the samples under dynamic load was studied by conducting a series of cyclic triaxial tests. The results revealed that all soil samples taken from three separate layers, in uniform load cycles of less than 15 cycles (i.e., equivalent to an earthquake with a magnitude of 7.5), reach 5% double amplitude strain, suggesting the occurrence of liquefaction. Also, the silica colloid percentage required to obtain a UCS of 100 kPa was examined as a criterion for non-liquefaction of soil in soil layers. According to the obtained results, adding 10 wt.% silica colloid to the prepared sand samples provides a UCS above 100 kPa. Then, silica colloid was injected into the soil layers by piling the screened pipes and packing these pipes at 2 m intervals with an injection pressure of 0.5 MPa. Ten days after the end of the injection operation, undisturbed sampling was performed, and SPT and hydraulic conductivity tests were carried out in the soil layers. The main results of these tests are as follows:

An SPT experiment was performed to control the effectiveness of the proposed on-site improvement method. The results showed a significant increase in the SPT number. Therefore, this test seems to be a suitable tool to evaluate the strength of silica colloidal soil.

The permeability testing results in three separate layers showed that their permeability was significantly reduced, suggesting the formation of soil silica gel and thus its reduced permeability. Therefore, determining the hydraulic conductivity of soil seems to be another suitable measure to control the improvement of sand stabilized with silica colloid.

By measuring the silica percentage and uniaxial strength on the samples obtained from the soil layers stabilized at the site, a relationship was established between the silica percentage and the UCS of the soil samples. Using this relationship and measuring the silica percentage in the soil, the UCS of the samples can be predicted.

According to the relationship $q_{uF} = 0.68 \times q_{uL}$ between the in-situ and laboratory UCS, the resistance required to prevent liquefaction in the laboratory can be easily calculated using the field resistance required based on the proposed 100 kPa criterion. Then, according to the UCS of the laboratory obtained from this relationship and the diagram, the percentage of injected silica at the site to prevent the occurrence of liquefaction was easily calculated.

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