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Using Micropiles to Improve the Anzali's Saturated Loose Silty Sand

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***Abstract*—**Today, with the daily advancement of geotechnical engineering on soil improvement and modification of the physical properties and shear strength of soil, it is now possible to construct structures with high-volume and high service load on loose sandy soils. One of such methods is using micropiles, which are mostly used to control asymmetrical subsidence, increase bearing capacity, and prevent soil liquefaction. This study examined the improvement of Anzali's saturated loose silty sand using 192 micropiles with a length of 8 meters and diameter of 75 mm. Bandar-e Anzali is one of Iran's coastal populated cities which are located in a high-seismicity region. The effects of the insertion of micropiles on prevention of liquefaction and improvement of subsidence were examined through comparison of the results of Standard Penetration Test (SPT) and Plate Load Test (PLT) before and after implementation of the micropiles. The results show that the SPT values and the ultimate bearing capacity of silty sand increased after the implementation of the micropiles. Therefore, the installation of micropiles increases the strength of silty sand improving the resistance of soil against liquefaction.

***Keywords*—**Soil improvement, silty sand, micropiles, SPT, PLT, strength.

I. Introduction

IN the event of dynamic loads caused by the earthquake, sandy and saturated soils in the region tend to agglomerate and lose their volume. If the soil fails to be drained quickly, the effective stress is largely reduced due to reduced permeability factor and gradual increase in pore water pressure. Under these conditions, since the shear strength of cohesionless soil is directly related to its effective stress according to equation $\tau = \sigma'\tan\theta'$, the elastic modulus and shear strength of soil may be drastically reduced which may lead to complete disappearance of soil shear strength. This phenomenon is called liquefaction [1]. In order to reduce damage caused by liquefaction during earthquakes, soils susceptible to liquefaction can be consolidated and improved. Methods of sand soil improvement against liquefaction include: replacement of soils susceptible to liquefaction with the appropriate soils, compaction of soil at the site using dynamic and vibroflotation compaction, improvement or injection and using sand or stone drainage [1]. Raising awareness of the good performance of micropiles both technically and economically compared to other similar maintenance structures has led to widespread use of them in land improvement in many regions. Micropiles have been used effectively and significantly in many applications, especially in the resilience of existing foundations, in order to reduce settlements while increasing the load. According to the new research, the existence of structural elements in the soil mass reduces shear deformability. Soil strength increases. Thus conducting studies in the field of using micropiles is necessary [2].

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A. Liquefaction Hazard

Liquefaction or flow generally refers to the loss of saturated soil shear strength due to cyclic stresses caused by factors such as the earthquake. This phenomenon mostly occurs in fine grained saturated sandy soils with uniform particle size, low density and low fine-grain percentage. This type of soils is willing to compact and reduce size if they are under vibration and in case of the absence of drainage and increased pore pressure, the total pressure will be equal to the pore pressure and effective stress will be equal to zero. In this case, the sand does not have any resistance against the shear strength and behaves like a fluid. Among the factors contributing to this phenomenon are soil compaction, soil gradation, the depth of layer and immersive effective stress, intensity and duration of earthquake and the percentage of passing through the filter No200 (fine grain amount and its material) and so on [3], [4]. Granular saturated soil liquefaction is qualitatively understood for many years. There are two main methods for soil liquefaction assessment. The first method is based on Seed and Idriss [5] in which the shear stress amplitude and the number of cycles were considered as a benchmark. Since then, this code has been reviewed and amended several times [6]-[8]. The second method to predict liquefaction potential is based on the strain approach. In this method, the shear strain in the site is compared with the data that relate the cyclic shear strain to the additional pore pressure to determine the liquefaction potential. Strain method is basically different from the stress method. Experimental and field data do not exist in stress method. Nassir and Shokuh [9] introduced the concept of energy on the compaction and liquefaction of cohesionless soil. Since then numerous laboratory studies have been conducted by researchers in the field. It is possible to calculate the energy released by an earthquake. So, it is better to use the energy used to evaluate liquefaction potential instead of using shear stress, strain amplitude and number of equivalent cycles [10]. Towhata and Ishihara, by conducting several cyclic undrained experiments on Toyoura sand, found that there is a unique relationship between shearing and increasing pore pressure and this relationship is independent of strain history [11]. Liang et al., through laboratory studies, showed that in a fixed comprehensive compaction and stress, the energy required for liquefaction is almost fixed regardless of the method of applying stress [12]. Jafarian et al. conducted experiments using a shear torsional hollow cylinder to assess the resistance to liquefaction and additional pore water pressure and proposed a model for additional pore water pressure based on the cumulative wasted energy and also studied the initial static shear stress's effect on the energy leading to liquefaction by conducting experiments [13].

B. Micropiles and Their Applications

Micropiles refer to the piles with a dimension of less than 300 mm which are usually associated with lightweight steel reinforcement and cement slurry injection. Experimental observations including earthquake in Kobe in 1995 showed that friction piles behave very well under dynamic loading due to their proper flexibility. Addition to acting as a load bearing and strong element against settlement, micropiles also improve the mechanical properties of the soil due to the injection of cement slurry. History of inventing the micropiles dates back to the early 50's. In this case, along with the issue of performance, potential of fast implementation and economical was very necessary. In such circumstances micropile was invented by an Italian contractor that was extended due to unique features. At first, the micropile was only considered in improving the poor foundation. But with development and implementation of these methods, the scope of its application was extended to other areas of geotechnical engineering. In most projects, according to the conducted geotechnical studies at the sites and identifying the subsurface layers, geotechnical properties of project site are determined. Then through conducting liquefaction analysis at different depths, liquefaction potential of the sites is evaluated in case of earthquake. When liquefaction occurs, huge settlements are imposed on the surface foundation. This event could lead to serious damage to structure and non-structural elements of the building and ultimately destroy it. In general, micropile in technical terms is discussed in two approaches to prevent liquefaction and supply the bearing capacity of piles on the foundation and load transfer to the deeper layers and it is recommended as a desirable option by geotechnical engineers. On the other hand, in designing micropiles, there is no specific method for selecting the design parameters and parameters are selected experimentally that should be optimized in terms of engineering. Micropiles are piles with a diameter of less than 300 mm associated with cement slurry injection in tubes with holes and light steel reinforcement [14]. Various experimental and numerical researches are conducted on micropiles' behavior. Abdollahi et al. addressed flexural stiffness in micropiles with their circular arrangement by numerical methods and Flac 3D software [15]. Takashi et al. studied the slope efficiency using micropiles for improvement purposes on real micropile specimens [16]. Sun et al. proposed a design method for the slope stability using micropiles. The purpose of this study is to find appropriate micropile group to achieve the right shear strength. To reach this goal, a design method is evaluated. This method included parameters such as site selection to install the micropiles, selecting the proper section and length, estimating micropiles' shear capacity, selecting the distance between micropiles and shear strength. The observed results indicate that the slope motion is reduced significantly [17].

Ortega studied the effect of overhead cement on slurry durability used in micropiles. The code is not limited to use a special type of cement. In this study, the overhead cement is used and it is compared with common cement. The results indicate that the overhead cement has a better performance than the ordinary cement [18]. Gorbani conducted three-dimensional element analysis, sensitivity analysis and parametric study of the interaction between soil and micropile. A full 3D finite element seismic model that includes the effect of important parameters on the seismic performance of micropiles was prepared. Then, the parametric study including all earthquake parameters, soil and structure properties and micropile structure on the seismic performance of micropile was studied to obtain the internal forces and displacements as a result of the major earthquake. Then, the least and most important parameters in the internal forces were obtained by the CAM method [19]. Veludo studied micropile compressive strength in connection with the slurry. In this study, the effects of the diameter, length and specific gravity of slurry on the micropiles ultimate strength were addressed. According to studies, strength is increased by reducing diameter and increasing the length [20]. Elaied investigated the footing performance by micropiles in the vicinity of the objects. In this study, a physical model is built and images were taken in succession under their loading and the observations were recorded in the tables and charts. Results showed a significant increase in flexural strength with increasing micropiles depth [21]. Soil liquefaction is one of the most important causes of failure and damage to structures, therefore, in many projects deep foundations are used to pass through the loose layer and reach to strong underlying layer. The piles' performance in liquefactive soils is much more complicated than the piles' performance in non-liquefactive soils. In this case, the pile is subject to dynamic loads of both structure and soil. Also, the strength and hardness of the soil decreases over time due to the non-linear behavior of soil and pore pressure. In these conditions, the piles are highly susceptible to cracking and failure. The impact of liquefaction on piles can be observed on the damage caused by previous earthquakes. In Niigata (1964) earthquake, damage caused by soil liquefaction was considered for the first time. In earthquakes such as the earthquake in Alaska (1964), Loma-Prieta (19898), Hyogoken-nambu (1995) and Bhuj (2001), the impact of the intensification of soil on pile damage is observed. In the last decade, some studies have been conducted on the effect of pile groups on the potential for liquefaction of soils according to previous research. For example, Kagawa conducted a parametric study on the impact of different soils and loading conditions on a single pile in the liquefactive soil

[22]. Klar et al. conducted a parametric study on the impact of flow characteristics on single pile interaction [23]. Experimental results of Kagawa et al. in most cases showed that the additional pore pressure between piles is more than the same amount in the space out of the piles; they proposed that this conclusion (lack of wasting the additional pore pressure) is because the piles reduce the drainage action [24]. On the other hand Sakajo et al. conducted the shaking table test on 36 pile group. The result showed that the existence of the pile will reduce the amount of pore pressure compared to cases where the pile group is not used. This was in contrast to the previous observations and led to the question whether a pile system with a specific configuration can reduce the additional pore pressure or the prevention of drainage [25]. In response to the question, Klar et al. studied the unlimited pile group in liquefactive soil numerically and observed that an optimal distance between the piles minimizes the additional pore pressure. It was also concluded that a specific configuration and shape of pile group could reduce liquefaction potential compared to the case that the piles are not present [26].

C. The Purpose of Study

The aim of this study was to assess the geotechnical properties of saturated silty sand soil improvement with low density to by micropile implementation. For this purpose, a project was conducted in Bandar Anzali in northern Iran as a case study. At first, geotechnical studies, including 3 machine boreholes up to the depth of 20 meters was carried out in the site. Then with the help of software analysis the risk of liquefaction was evaluated at the site. Finally, to improve the geotechnical characteristics of the site micropile implementation was used in the project. Quality improvement was evaluated by PLT.

II. Research Methodology

The case examined in this study is the bed improvement project of the City Center commercial office complex site located in Bandar Anzali- Iran's industrial free trade zone that includes 206 micropiles on which the rectangular and extensive foundation has been implemented with dimensions of 70 × 15 m. Given the importance and sensitivity in designing and implementing optimization consulting, the geotechnical studies of the adjacent site were used for better identification of the soil.

A. Site Properties

1) Site Selection and Basic Geotechnical Properties

Bandar Anzali in Gilan province in northern Iran is located at 49º / 28 '/ 00''N and 37º / 27' / 30''E. of Caspian steppes. Bandar Anzali is limited by Caspian Sea in the north on sands as a narrow strip along the coastline. For this reason, the soil layers properties in the city are the same as the Caspian coast properties. The major components of soil in this area are fine-grained sands. Fig. 1 shows a satellite image of the area under study.

Fig. 1 Satellite image of the site

To determine the physical and mechanical properties such as moisture content, density, grain size distribution and soil compaction, laboratory experiments were conducted and the hardness and strength properties of the site are determined in situ in different parts of the profile and plan. The following tests are generally conducted based on AASHTO and ASTM code recommendations:

A- Laboratory operations:
1. Moisture content determination (AASHTO: T217) (ASTM: D2216)
2. Mechanical grading (under 127 SMB) (ASTM: D2217) (ASTM: D421)
3. Atterberg Limits (ASTM: D4318)
4. Specific gravity

B- Field operations:

C- Impact and SPT according to (ASTM: D1586)

a) Evaluating the Physical and Mechanical Properties of Soil Layers

Structure and soil type, obtained from three machine boreholes drilled by the adviser as well as three machine boreholes drilled by Geotechnical University of Guilan indicate the presence of two distinct layers of soil to a depth of 20 m. The surface soil of the site to a depth of 10 m is formed of a fine-grained sedimentary layer with sand and silt grains that the SPT results for the overhead effect indicate the relatively low density of the layer. (Average number of SPT in the boreholes is 16 impacts for standard penetration of 30 cm). Based on the unified classification, the above layer is in the SM Group. The second observed layer is from the depth of 10 to 15 meters including fine-grained soil of silt and clay with average consistency to fine sand grains; based on the unified classification, the above layer is in the ML Group and the results of standard penetration and the specimens taken from the above layer indicate the medium density of the above layer. Average number of SPT in the depth of 10-15 m is 21 impacts for standard penetration of 30 cm. The standard penetration numbers should be modified for the overhead impact in liquefaction analyses. The modified values are presented in Table I. The results of these experiments show the moisture content of the specimens at the depth of 20 meters in the minimum, maximum and average ranges of 14, 28 and 21%.

The surface soil to a depth of 10 meters has the average grain size of 27% based on the grading results (passed through the filter 200) with the variation range of 6-47% and at the lower depth than 10m it has 47% fine grains with the variation range of 18-60%. Also, the results of tests on 9specimens prepared from various depths have presented the soil dry density as 1.45-1.55 g/cm^2 based on the Table I. Based on the standard density test in accordance with Table I the maximum specific gravity is 1.76-1.92 g/cm^2 with optimum moisture content of 12-14%.

TABLE I
GEOTECHNICAL SOIL TEST RESULTS ON THE PROJECT SITE

Borehole	Specimen depth (m)	Natural specific (gr/cm3)gravity	Moisture (%)	Maximum laboratory specific gravity (gr/cm3)
BH1	4.45	1.75	20	1.88
BH1	8.45	1.81	22	1.92
BH1	12.45	1.77	21	1.8
BH2	8.45	1.9	24	1.88
BH2	14.45	1.87	25	1.8
BH2	4.45	1.8	18	1.76
BH3	2.45	1.74	18	1.78
BH3	8.45	1.83	22	1.89
BH3	14.45	1.87	21	1.91

b) Geotechnical Studies' Results

During two series of geotechnical studies and previous reports, it was reported that the soil is prone to liquefaction at the depths of 5 to 10 meters. Ongoing investigations according to the profile of the soil and the modified results of the N obtained by SPT, the energy applied and the effects of depth, the percentage of fine-grained (filter 200), specific in situ tests and in the laboratory and details of momentum subject to the code on designing buildings against earthquake (2800), analysis of liquefaction potential at the depth of 10m report liquefaction under the basic acceleration as 0.3g as medium. As a whole with respect to the raft foundation of choice under two connected 4-story blocks in dynamic loading conditions such as earthquake with a magnitude of 7.5 and the maximum

acceleration of 0.3g, uniform and non-uniform problems of displacement and settlements and liquefaction potential is in the middle range in terms of damage to the surface and in this case the common aspects are classified as surfaces with high, medium and low intensity in the references.

2) PLT Results

Performing PLT with a simple and inexpensive method is one of the great advantages of this method compared with other approaches such as the implementation of thick or steel concrete piles or sand columns. Due to lower load bearing of the thick and deep micropiles conducting PLT using light jacks will have high accuracy and easy implementation by providing sufficient anchor reaction force. PLT results before the implementation of micropiles are presented in the results and discussion to be compared with the test results after the implementation.

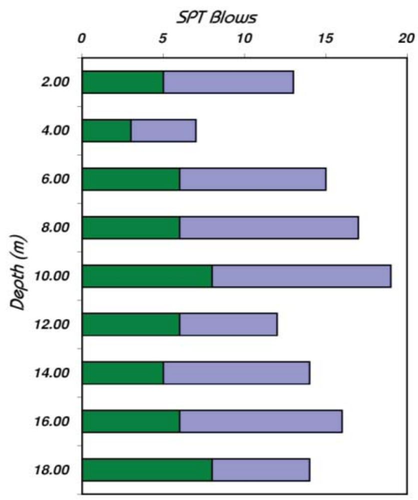
3) SPT Results

SPT is the most common in situ test. This test is performed in accordance with different standards that it is conducted by standard American weights and hammer based on ASTM-D 1586 standard [27]. The results of in situ SPT cannot be used in raw form and in using the results different adjustments such as overhead pressure and energy corrections should be applied. SPT results can be used in designing the surface foundations and identifying different soil geotechnical parameters such as shear strength parameter, soil identification and classification and as an index for measuring the hardness and density of soil. For the site in question SPT is done on each 2m depth of each borehole that the change in the number of impacts (for 30 cm penetration) is drilled for three boreholes.

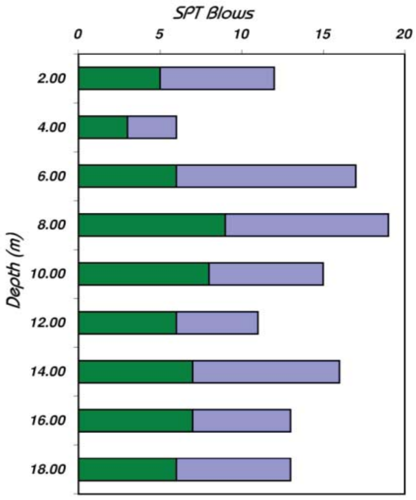
4) Liquefaction Risk Analysis in the Site

For the site under study calculation of liquefaction analysis of different layers were done based on soil profile and the profile of the region through the SPT separately at the site of drilled boreholes. Accordingly, for the severe earthquakes there is risk of liquefaction in great depths, although the safety factor is not low in most depths, it is necessary to take the necessary steps to remove liquefaction at the depths with the safety factor less than 1. Generally, there are different arrangements to eliminate or reduce risks and losses due to liquefaction. Using deep foundations (piles) and load transfer to the underlying layers and non-liquefactive, using soil improvement methods such as soil dynamic compaction to the desired depth, the use of injections to increase the stability of the layers, drainage and holding the groundwater level low, the use of vertical and horizontal drains for fast drainage during earthquake the massive foundations are used to prevent asymmetrical settlements are among such methods.

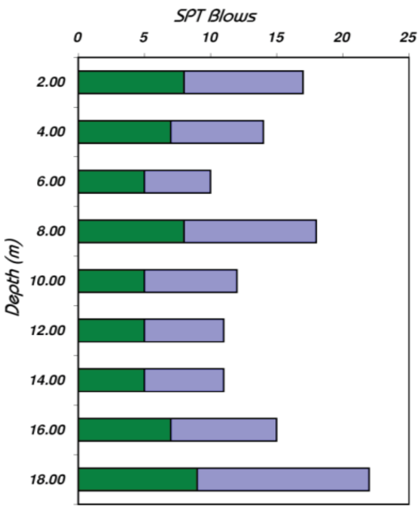
In situ SPT results cannot be used in raw form and different adjustments such as overhead pressure and energy corrections should be applied. In Tables II-IV the modified SPT values are included for different modes.



(a)



(b)



(c)

Fig. 2 SPT results for (a) BH-1, (b) BH-2, (c) BH-3

Given that the results are raw and need to be corrected to be applied in the calculations, numerical correction of SPT is done as follows:

$$N_{60} = 1.67 E_m C_b C_r N \tag{1}$$

$$(N_1)_{60} = C_N N_{60}$$

According to (1), SPT for different boreholes is corrected as shown in Tables II-IV.

On the other hand, cyclic stress ratio (CSR) is designed based on earthquake profile and total and effective stress of layers is determined as follows:

$$CSR = 0.65\, r_d \left(\frac{\sigma_\circ}{\sigma'_\circ}\right) \left(\frac{a_{\max}}{g}\right) \tag{2}$$

Thus, by comparing CSR with cyclic resistance ratio (CRR) the safety factor can be determined as follows:

$$\Rightarrow FS = \frac{CRR}{CSR}$$

TABLE II
SPT RESULTS AT DIFFERENT LAYERS IN THE BH1

Depth	SPT-N	Em	Cb	Cr	N_{60}	C_N	$(N1)_{60}$
2	13	0.6	1	0.75	9.8	1.67	16.3
4	7	0.6	1	0.85	6	1.38	8.2
6	15	0.6	1	0.95	14.3	1.21	17.2
8	17	0.6	1	0.95	16.2	1.08	17.5
10	19	0.6	1	1	19	0.99	18.9
12	12	0.6	1	1	12	0.92	11.1
14	14	0.6	1	1	14	0.86	12.1
16	16	0.6	1	1	16	0.81	13.1
18	14	0.6	1	1	14	0.77	10.9

TABLE III
SPT RESULTS AT DIFFERENT LAYERS IN THE BH2

Depth	SPT-N	Em	Cb	Cr	N_{60}	C_N	$(N1)_{60}$
2	12	0.6	1	0.75	9	1.67	15
4	6	0.6	1	0.85	5.1	1.38	7.1
6	17	0.6	1	0.95	16.2	1.21	19.5
8	19	0.6	1	0.95	18.1	1.08	19.6
10	15	0.6	1	1	15	0.99	14.9
12	11	0.6	1	1	11	0.92	10.2
14	16	0.6	1	1	16	0.86	13.8
16	13	0.6	1	1	13	0.81	10.6
18	13	0.6	1	1	13	0.77	10.1

TABLE IV
SPT RESULTS AT DIFFERENT LAYERS IN THE BH3

Depth	SPT-N	Em	Cb	Cr	N_{60}	C_N	$(N1)_{60}$
2	17	0.6	1	0.75	12.8	1.67	21.3
4	14	0.6	1	0.85	11.9	1.38	16.5
6	10	0.6	1	0.95	9.5	1.21	11.5
8	18	0.6	1	0.95	17.1	1.08	18.6
10	12	0.6	1	1	12	0.99	11.9
12	11	0.6	1	1	11	0.92	10.2
14	11	0.6	1	1	11	0.86	9.5
16	15	0.6	1	1	15	0.81	12.2
18	22	0.6	1	1	22	0.77	17.1

This comparison for the site shows that although the safety factor in the middle depths will be greater than one, based on the loose condition of the lower depths, existing layers to a depth of about 14 meters are susceptible to liquefaction in earthquakes with maximum acceleration 0.3g. Of course, according to different codes, the average acceleration forms a percentage of the maximum acceleration. Also, an example of liquefaction potential analysis and safety factors' diagram at different depths is shown in Fig. 3.

B. *Micropile Design Details*

1) Designing Micropile [28]

Micropile length: Micropile length should be selected such that the micropile geotechnical bearing capacity is provided through the wall friction resistance.

Micropile cross section: Choosing micropile cross-section has a significant impact on the structural bearing capacity. In order to micropile facilitate implementation and installation in Iran the tubes with the outer diameter of 75 mm (3 inches) and the thickness of 3, (0.16 inches) is used that is the basis for calculations in this project.

a) Micropile Distance

Research has shown that when the micropile is designed to increase bearing capacity, the most effective arrangement is when 25 percent of micropiles are distributed under the foundation. In this study, after determining the number of micropiles all of them will be distributed on the surface to improve the soil profile against liquefaction.

b) Injections Properties

Injection pressure at various stages of injection at different depths is influence by the land type and geotechnical conditions. The maximum injection pressure is limited to 10 atm. The Injection pressure used in the project must be in accordance with the listed items for D mode.

According to geotechnical conditions and the load bearing of the micropiles, the amount of cement could be estimated as the length of micropiles. However, due to the fact that injection should continue until the pressure of 20 to 60 atmospheres, the cement content may be higher than the original estimates. Of course, the amount of cement differs in different soils and it has a direct relationship with the grain size, density, ground water conditions and so on. Water-cement ratio in the slurry injection is recommended as 0.4-0.5. But it varies between 0.5-1.5 according to the implementation conditions. Fig. 10 shows the effect of water-cement ratio on the compressive strength of slurry.

Type of cement: The type of cement in the mortar of cement injection is type I or II Portland cement or type V cement due to chemical conditions according to AASHTO and ASTM recommendations.

TABLE V
THE RESULTS DESIGNING MICROPILES FOR THE PROJECT SITE

	Unit	Value
Casing Thickness	Mm	3
Casing Out. Diameter	Cm	7.6
No. of holes per meter	-	70
Diameter of holes	Mm	0.6
Casing yield stress	Kg/cm^2	2400
Rebar type	-	AIII
Rebar diameter	Mm	28
Rebar yield stress	Kg/cm^2	4200
Grouting pressure	Bar	20-60
Grout strength	Kg/cm^2	400

III. RESULTS AND DISCUSSION

A. PLT Results Using VSS

Loading is started with a loading with pressure 0.5 kg per square centimeter and then the pressure increased at a rate of 1 kg per square centimeter. This process is repeated regularly until the pressure reaches 5.5 kg per square centimeter. Coefficient of VSS is obtained using (3):

$$VSS = \frac{\Delta P}{\Delta d} \times D \tag{3}$$

where ΔP is the increase in loading pressure at each loading in kilograms per square centimeter, D is the circular loading plate diameter in millimeters (usually 200 to 700 square cm) and Δd is the fluctuation associated with the pressure ΔP in millimeters. In Fig. 5, a schematic view of the equipment used to perform PLT is presented. Also, the results of plate loading before and after micropile installation are presented in Figs. 6-8 and Tables VI-VIII.

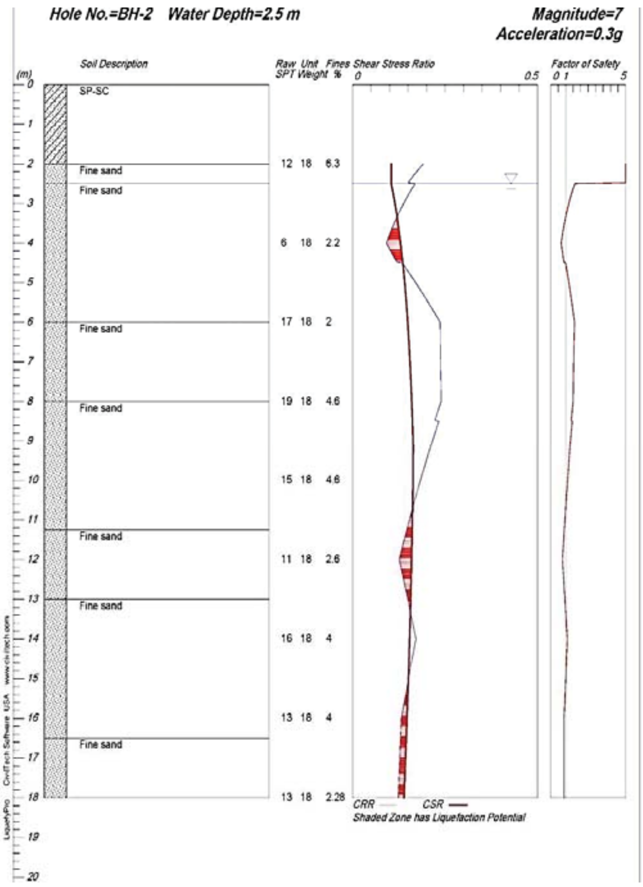


Fig. 3 Liquefaction potential analysis and safety factors' diagram at different depths in BH-2

Fig. 4 Project site after the implementation of micropile

Fig. 6 indicates that by increasing the axial pressure the amount of fluctuation is increased and the slope of changes is linear. Fig. 6 indicates that in the PLT by VSS after the stabilization the rate of rising change is less than the PLT test by VSS before the stabilization thus the pressure-fluctuation pressure in the PLT is greeter in VSS after the stabilization. Also, the maximum fluctuation indicates that after consolidation the level of fluctuation is reduced by half compared to before consolidation. Fig. 7 shows the pressure-fluctuation diagram of plate loading by VSS in the second site before and after consolidation.

Fig. 5 Equipment used to perform PLT

TABLE VI
THE RESULTS OF PLATE LOADING BY VSS IS SITE 1

Load (Kg)	P/A Pressure (Kg/cm^2)	Deflection before Stabilization (mm)	Deflection after Stabilization (mm)
353	0.5	1.5	1.9
1060	1.5	1.92	2.7
1767	2.5	2.2	3.5
2474	3.5	2.62	4.1
3181	4.5	2.85	4.8
3888	5.5	3.1	5.8

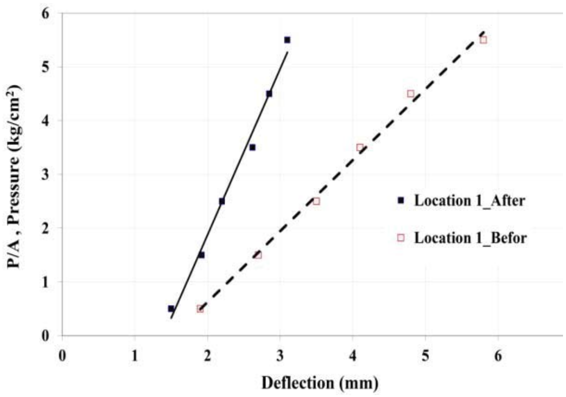


Fig. 6 The results of plate loading by VSS is site 1

According to the Fig. 7, it is clear that by increasing the axial pressure the level of fluctuation is increased and the slope of changes is linear. Fig. 7 shows that in VSS test after the consolidation the rate of fluctuations is less than VSS test before consolidation, thus the slope of pressure-fluctuation diagram in VSS is increased after consolidation.

TABLE VII
THE RESULTS OF PLATE LOADING BY VSS IS SITE 2

Load (Kg)	P/A Pressure (Kg/cm^2)	Deflection before Stabilization (mm)	Deflection after Stabilization (mm)
353	0.5	0.8	1.3
1060	1.5	1.1	2.2
1767	2.5	1.36	3.3
2474	3.5	1.7	3.9
3181	4.5	2	4.71
3888	5.5	2.4	5.5

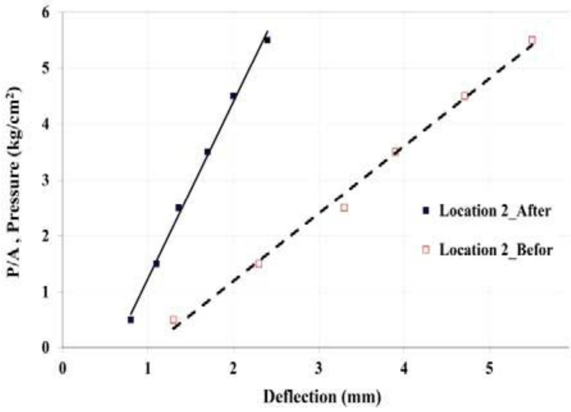


Fig. 7 The results of plate loading by VSS is site 2

TABLE VIII
THE RESULTS OF PLATE LOADING BY VSS IS SITE 3

Load (Kg)	P/A Pressure (Kg/cm^2)	Deflection before Stabilization (mm)	Deflection after Stabilization (mm)
353	0.5	0.88	1.1
1060	1.5	1.23	2
1767	2.5	1.45	3.11
2474	3.5	1.84	3.63
3181	4.5	2.12	4.6
3888	5.5	2.29	5.3

Also, the maximum fluctuation indicates that after consolidation the level of fluctuation is reduced by half compared to before consolidation. Fig. 8 shows the pressure-fluctuation diagram of plate loading by VSS in the third site before and after consolidation.

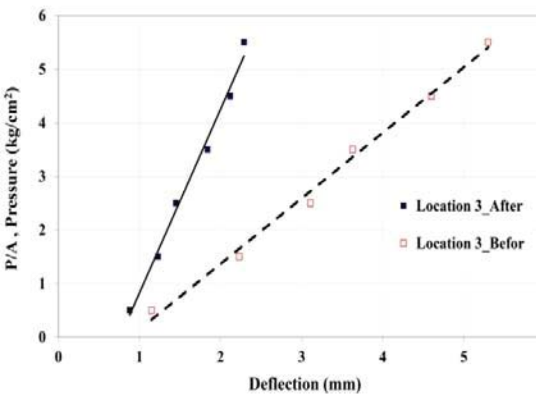


Fig. 8 The results of plate loading by VSS is site 3

According to the Fig. 8, it is clear that by increasing the axial pressure the level of fluctuation is increased and the slope of changes is linear. Fig. 8 shows that in VSS test after the consolidation the rate of fluctuations is less than VSS test before consolidation, thus the slope of pressure-fluctuation diagram in VSS is increased after consolidation. Also, the maximum fluctuation indicates that after consolidation the level of fluctuation is reduced by half after consolidation.

IV. Conclusion

The aim of this study was to assess the geotechnical properties of saturated silty sand soil improvement with low density to by micropile implementation. For this purpose, a project was conducted in Bandar Anzali in northern Iran as a case study. The most important results of the study are as follows:

- ✓ Pressure-fluctuation diagram changes after consolidation is more than before consolidation that indicates improved performance after the consolidation.
- ✓ After consolidation, the level of fluctuation is reduced by half compared to before consolidation.
- ✓ The pressure- fluctuation diagram slope of changes is linear.

References

[1] Balkema, A.A., (1997), Hand book on liquefaction Remediation of Reclaimed land, Japan Port and Harbour Research Institute.
[2] Rahimi, A., jafarian, Y., Numerical Study of the Effect of Micropiles Group on Soil Liquefaction Potential, 10th International Congress on Civil Engineering, University of Tabriz, Iran, 2015.
[3] Cernica, J. N., Geotechnical Engineering: Soil Mechanics, 1995.
[4] Jefferies, M., Been, K., Soil Liquefaction: A critical state approach, 2006.
[5] Seed, H. B., and Idriss, I. M. (1971). "Simplified procedure for evaluating soil liquefaction potential." J. Soil Mechanics and Foundations Div., ASCE 97(SM9), 1249–273.
[6] Seed, H. B., and Idriss, I. M. (1982). Ground Motions and Soil Liquefaction during Earthquakes, Earthquake Engineering Research Institute, Oakland, CA, 134 pp.
[7] Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, W. F., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robert son, P. K., Seed, R. B., and Stokoe, K. H. (2001). Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils, J. Geotechnical and Geoenvironmental Eng., ASCE 127 (10), 817–33.
[8] Idriss, I. M., and Boulanger, R. W. (2006). Semi-empirical procedures for evaluating liquefaction potential during earthquakes, J. Soil Dynamics and Earthquake Eng. 26, 115–30.
[9] S. Nemat-Nasser, A. Shokooh, (1979), A unified approach to densification and liquefaction of cohesionless sand in cyclic shearing, Canadian Geotechnical Journal, 1979, 16(4): 659-678.
[10] Day, R.W., Geotechnical Earthquake Engineering Handbook, 2002.
[11] Towhata, I., & Ishihara, K. E. N. J. (1985). Shear work and pore water pressure in undrained shear. Soils and foundations, 25(3), 73-84.
[12] Liang, L. (1995). Development of an energy method for evaluating the liquefaction potential of a soil deposit (Doctoral dissertation, Case Western Reserve University).
[13] Jafarian, Y., Towhata, I., Baziar, M. H., Noorzad, A., & Bahmanpour, A. (2012). Strain energy based evaluation of liquefaction and residual pore water pressure in sands using cyclic torsional shear experiments. Soil Dynamics and Earthquake Engineering, 35, 13-28.
[14] Armour, T., Groneck, P., Keeley, J., Sharma,S., FHWA-SA-97-070 - Micropile Design and Construction Guidelines – Implementation manual: Federal Highway Administration – US Department of Transportation; 2005.
[15] Abdollahi, K., Mortezaei, A., A new expression for determining the bending stiffness of circular micropile groups, Soil Dynamics and Earthquake Engineering 77, pp 58–70, 2015.
[16] Takashi, Y., Yoshinori, N., Yoshinori,O., Efficiency of micro pile for seismic retrofit of foundation system. In: Proceedings of the twelfth world conference on earthquake engineering, Auckland, New Zealand; 2000.
[17] Sun, S., Zhu, B., Wang, J., Design method for stabilization of earth slopes with micropiles, Soils and Foundations 2013; 53 (4) 487–497.
[18] Ortega, J. M., Albaladejo, A., Pastor, J. L., Sanchez, I., Climent, M.A., Influence of using slag cement on the microstructure and durability related properties of cement grouts for micropiles, Volume 38, pp 84–93, 2012.
[19] Ghorbani, H. Hasanzadeh shooiili, E. Ghamari, J. Medzviecka, Comprehensive three-dimensional finite element analysis, parametric study and sensitivity analysis on the seismic performance of soil–micropile superstructure interaction, V 58, pp 21–36, 2014.
[20] Veludo, J., Jlio, E.N.B.S., Dias-da-Costa, D., Compressive strength of micropile-to-grout connections, V 26, Issue 1, pp 172–179, 2012.
[21] Elzoghby, E., Performance of footing with single side micro-piles adjacent to slopes, V 53, Issue 4, pp 903–910, 2014.
[22] Kagawa, T., (1992), “Effect of liquefaction on lateral pile response”, ASCE.Geotech Spec Pub, Piles Under Dynamics Loads, Vol 34, pp.207-223.
[23] Klar, A., Baker, R., Frydman, S., (2003), “Seismic soil–pile interaction in liquefiable soil”, Soil Dynamics and Earthquake Engineering, Vol 24, pp. 551-564.
[24] Kagawa, T., Minowa, C., Abe, A., Oda, S., (1995), “Shaking-table tests on and analyses of piles in liquefying sands”, Earthquake Geotechnical Engineering Proceedings of the First International Conference, ISTokyo, pp. 699-704.
[25] Sakajo, S., Chai, J. C., Nakajima, K., Maeda, M., (1995), “Effect of group pile on liquefaction resistance of sandy ground”, Proc., First Conf. on Earthquake Geotechnical Engineering, IS-Tokyo, pp. 755-760.
[26] Klar, A., Frydman, A., Baker, R., (2004), “Seismic analysis of infinite pile groups in liquefiable soil”, Soil Dynamics and Earthquake Engineering, Vol 24, pp. 565-575.
[27] Annual Book of ASTM Standards (ASTM), 2000.
[28] Micropile Design and Construction, Reference Manual. U.S. Department of Transportation Federal Highway Administration, Publication No. FHWA NHI-05-039, December 2005.