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Imrovement of Loose Sandy Soil Deposits using Micropiles

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Abstract

Micropiles are being used today for a variety of applications. The increased awareness of the sound performance of micropiles especially under seismic conditions has led to the widespread use of this technology for ground treatment. Micropiles have been used effectively in many applications of ground improvement to increase the bearing capacity and reduce the settlements particularly in strengthening the existing foundations. This paper describes a case study in which 350 micropiles of 75 mm diameter and 15-20 m long have been used for the improvement of loose sandy soil deposits. The effect of micropile injection on liquefaction remedation and improving of soil stress-displacement behavior are evaluated using the results of Standard Penetration Tests (SPT) and Plate Load Tests (PLT) on a real site before and after micropile installation. The load-displacement responses of micropiles under tensile axial loading are also presented in this paper. The results show that, by using micropiles, the response of loose silty sand soils under surface loading can be significantly improved and the bearing capacity of loose sandy soil increased considerably. Also, the modulus of subgrade reaction of the soil (K_S) is increased. The SPT values of soil layers are increased after soil improvement with micropile installation. Therefore, the liquefaction resistances of sandy soil are also improved.

Keywords: *micropile, soil improvement, liquefaction remediation, standard penetration test, plate load test*

1. Introduction

Micropiles have been used effectively in many applications of ground improvement to increase the bearing capacity and reduce the settlement particularly in strengthening existing foundations. Design methodology and implementation of this technique are described in FHWA-NHI-05-039 (2005). Micropiles are defined as small-diameter, drilled piles composed of placed or injected grout with some form of steel reinforcement in the center of the grout to resist the bulk of the design load. The central reinforcing element is either a high-strength steel bar or tube that is secured in the grout injected under high pressure to improve bonding with the surrounding soil. Micropiles can be installed through virtually any ground condition and at any inclination.

Most of the current research on micropiles has concentrated on the behavior of a micropile as a structural element (Juran *et al*., 1999; Ousta and Shahrour, 2001; Misra *et al*., 2004, 2007; Russo, 2004; Sadek and Shahrour, 2004 and 2006; Shields, 2007; Alsaleh and Shahrour, 2009). This led to the definition of CASE 1 micropile elements, which are loaded directly and the pile reinforcement resists the majority of the applied load. CASE 2 micropile elements circumscribes and internally reinforces the soil to make a reinforced soil composite that resists the applied load. In this case, micropiles have been used effectively in many applications of ground improvement to increase the bearing capacity and reduce the settlement of foundations. Frictional resistance between the surface of the pile and soil and the associated group/network effects of micropiles are considered as the possible mechanism for improvement. Little research have been carried out recently to evaluate the effectiveness of micropile injection in soil improvement (Mc Manus *et al*., 2004; Babu *et al*., 2004; Han *et al*., 2006).

In the present study, thick loose sandy soil strata with low bearing capacity that found to cover extensive areas in the low altitude strip of coastal land next to the Caspian Sea in the North of Iran is examined. The current solution for foundation design employing spread footings bearing directly on such soils might lead to low admissible pressures associated with significant settlement and liquefaction occurrence during earthquake. Therefore, the micropile injection is presented as an alternative method in this work to improve the bearing capacity and liquefaction remediation of the soil deposit. The paper deals with a case study in which 350 micropiles of 75 mm diameter and 15-20 m long were used to improve the bearing capacity of soil deposit for rehabilitation of the building foundation system. The effect of micropile installation on liquefaction remediation and improvement of soil stress-displacement behavior is evaluated using the results of Standard Penetration Tests (SPT) and Plate Load Tests (PLT) on a real site before and after micropile installation. The load displacement responses of micropiles under

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tensile axial loading are also presented in this paper.

2. Site Characterization

The region under study is located in the low-altitude strip of coastal land next to the Caspian Sea, in the North Alborz region of Iran. The project site is adjacent to buildings of the Khaneh Darya complex of Mahmoud Abad. Numerous boreholes in the Paleo-Quarterner sediments of the coastal strip of the Caspian Sea indicate the existence of a shallow sea (sandstone sediments with seashells and some clay) in the late Cenozoic period. The upper and newest sediments on the site relate to recent period precipitations (river, deltaic, coastal), which cover the region and are generally silt and sand with little consolidation.

The soil conditions at the experimental test site were determined from a geotechnical site investigation comprised of both in-situ and laboratory tests. To identify soil layers three boreholes were drilled using a rotary drilling machine to depths of 30 m in specified locations. During drilling operations, standard penetration tests were carried out. The undisturbed and disturbed samples were taken at various depths in order to classify subsurface soils and determine their physical, chemical and mechanical characteristics using laboratory tests. The ground conditions were shown to consist of varying silty sands (SP-SM, SM). Also intermittent layers of silt with limited thickness were observed. Based on the SPT results the upper silty sand layer (0 to 20 meters) is classified as medium to dense and at deeper depths (20 to 30 meter) as dense to very dense. The ground water level was found to be 1m from the ground surface during drilling.The physical and mechanical properties of soil layers are presented in the Table 1.

3. Liquefaction Potential Evaluation

Due to the high ground water level and the existance of loose to very loose sandy soils in surficial depths, evaluation of the liquefaction potential is imperative. For this purpose, the corrected SPT values were evaluated at various depths and then the liquefaction potential was determined based on the NCEER procedure (Youd, 2001).

In the NCEER method, the Seed's recommendations are used to assess the liquefaction potential in sandy soils. Two quantities are required to be determined: 1) Cyclic Strength Ratio (CSR) and 2) Cyclic Resistance Ratio (CRR).

Using Seed and Idriss (1983), CSR is determined from the following equation:

$$CSR = (\tau_{av}/\sigma'_{vo}) = 0.65\,(a_{\max}/g)(\sigma_{v0}/\sigma'_{v0})r_d \qquad (1)$$

Table 1. Mechanical Properties of Soil Layers

Depth (m)	Soil type	γ (kN/m^3)	Fine Content (%)	ϕ^o	$(N_1)_{60}$
0-20	Medium to Dense silty sand	18	7-35	28	17 - 32
20-30	Dense to very dense silty sand	20	12-30	34	22 - 48

Where: $a_{\max}$: Maximum horizontal ground acceleration at ground surface in m/s^2

g: Ground acceleration in m/s^2

r_d: Shear reduction coefficient with depth.

σ'_{v0}: Effective overburden pressure in kPa

σ_{v0}: Total overburden pressure in kPa

τ_{av}: Average shear strength in kPa

The NCEER has recommended the following relations to determine shear stress reduction coefficient with depth (r_d):

$$r_d = 1.0 - 0.00765 * Z \qquad Z < 9.15\text{ m} \qquad (2)$$
$$r_d = 1.174 - 0.0267 * Z \qquad 9.15 < Z < 23\text{ m} \qquad (3)$$

The maximum horizontal ground accelaration of site is taken as: $a_{\max} = 0.35$ g

Determination of CRR based on the SPT results was first proposed by Seed *et al.* (1983). In 1997, this diagram was reviewed by the NCEER and, eventually, Fig. 1 was proposed. Here, based on the data collected from the sites of past earthquakes with/without liquefaction, the base curve for clean sand is given. Using a corrected SPT value $(N_1)_{60}$, CRR is determined from the given curves. In silty sands the effect of fines content must be accounted. For this purpose, the equivalent corrected SPT value is defined as follows:

$$(N_1)_{60cs} = K_s * (N_1)_{60} \qquad (4)$$

where K_s is the fines content correction factor and calculated as:

$$K_s = 1 + [(0.75/30)(FC - 5)] \qquad (5)$$

where, FC : Percent of fine content

The factor of safety against liquefaction is determined with the following equation:

$$FS = CRR/CSR \qquad (6)$$

Results of liquefaction potential evaluation of the site are presented in the Table 2 and Fig. 2. It can be concluded that, in

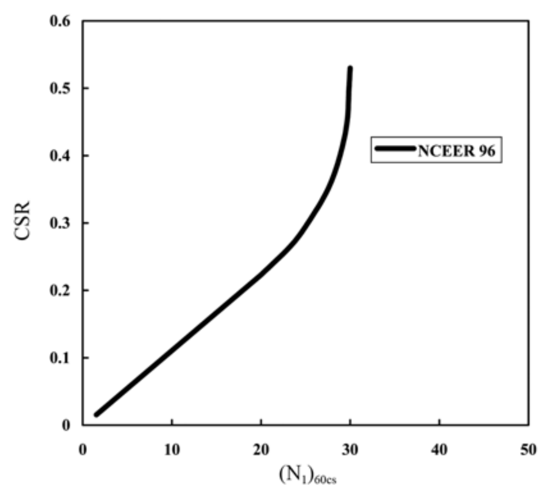

Fig. 1. Cyclic Resistance Ratio (CRR) for Clean Sands Under Level Ground Condition Based on SPT (Recommended by NCEER 96)

Table 2. Liquefaction Potential Evaluation in Boreholes (Before & After Micropile Installation)

Borehole	Z (m)	USCS	σ_{v0} (kPa)	σ'_{v0} (kPa)	CSR	Before micropile installation			After micropile installation		
						$(N_1)_{60CS}$	CRR	F.S	$(N_1)_{60CS}$	CRR	F.S
BH#1	1.5	SP-SM	27	12	0.506	12	0.133	0.262	35	0.567	1.12
	3	SP-SM	54	24	0.501	44	N.L.		53	N.L.	
	6.5	ML	117	52	0.487	24	0.273	0.559	38	0.846	1.73
	7.5	SP-SM	135	60	0.483	35	N.L.		51	N.L.	
	13.5	SP-SM	243	108	0.412	18	0.196	0.473	38	0.846	2.04
	15	SP-SM	270	120	0.389	17	0.189	0.487	35	0.567	1.45
	17	SP-SM	306	136	0.356	28	0.367	1.031	39	1.01	2.83
	19	SP-SM	342	152	0.328	22	0.237	0.724	32	0.413	1.26
	21	SP-SM	378	168	0.306	40	N.L.		54	N.L.	
	23	SP-SM	414	184	0.29	42	N.L.		48	N.L.	
	25	ML	450	200	0.277	30	N.L.		41	N.L.	
	27.5	SM	495	220	0.265	52	N.L.		62	N.L.	
BH#2	1.5	SP-SM	28.2	13.2	0.481	31	N.L.		41	N.L.	
	12	SP-SM	225.6	106	0.416	29	0.407	0.97	35	0.567	1.36
	13	SM	244.4	114	0.402	20	0.219	0.547	36	0.638	1.59
	15	SP-SM	282	132	0.369	14	0.159	0.43	38	0.846	2.29
	17	SM	320	150	0.338	26	0.313	0.925	40	1.253	3.7
	19	SM	357.2	167	0.311	39	N.L.		51	N.L.	
	21	SM	394.8	185	0.291	14	0.149	0.514	32	0.41	1.42
	23	SM	432.4	202	0.275	32	N.L.		42	N.L.	
	25	SM	470	220	0.263	51	N.L.		57	N.L.	
BH#3	3.5	ML	66.5	31.5	0.468	22	0.234	0.498	35	0.567	1.21
	10.5	SM	200	95	0.428	56	N.L.		65	N.L.	
	12	SM	228	108	0.411	18	0.189	0.461	39	1.01	2.456
	13.5	SM	257	122	0.388	16	0.177	0.456	32	0.412	1.06
	15	SM	285	135	0.365	12	0.129	0.355	33	0.46	1.258
	17	SM	323	153	0.334	24	0.273	0.817	34	0.509	1.523
	19	ML	361	171	0.307	36	N.L.		45	N.L.	
	21	SM	399	189	0.287	55	N.L.		61	N.L.	
	23	SM	437	207	0.271	50	N.L.		53	N.L.	
	25	SM	475	225	1.003	56	N.L.		62	N.L.	

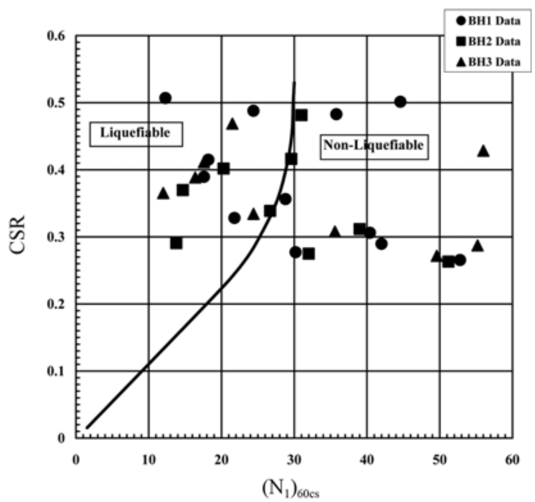

Fig. 2. Liquefaction Potential Evaluation (Before Micropile Installation)

depths less than 20 m, the safety factor is smaller than 1 and the liquefaction is likely to occur due to the existence of loose saturated sandy layers.

4. Improvement of Liquefaction Resistance

Many solutions were been considered in the technical literature (Youd, 2001) to reduce the liquefaction risk and improvement of the soil strength, including:

(i) Increasing the relative density of liquefiable soils by compaction techniques such as dynamic compaction and compaction piles

(ii) In-Situ soil improvement techniques such as micropile installation and grouting.

(iii) Use of deep foundation to transfer a structure's load to underlying compacted layers.

(iv) Replacement of the liquefiable soils with suitable material.

Of the above mentioned solutions the use of micropiles was selected for this project. The micropiles adopted consisted of steel porous tubes with 75 mm outer and 68 mm inner diameter. Micropile reinforcement consists of a single reinforcing bar with surrounding grout (Fig. 3). Standard reinforcing steel conforming to ASTM A615 with yield strength of 420 MPa is used. Diameters of the reinforcing bar were 25 and 28 mm. The micropile installation processes were presented in the Fig. 4.

A cement-water grout mix with a W/C ratio about 0.5 was used in this project. Plasticizer additives are added to cement grout to achieve the required workability. Type “C” standard grout placement techniques (Two-step grouting process) are used to micropile installation (Fig. 4). At first, gravity grouting (Type A) are performed and then after 15 minutes the secondary pressure grouting througth sleeved pipe with 1 MPa pressure were carried out. 300 to 340 lit/meter grout were also used for each micropile installation. Therefore, it seems that the grout volume is ranged between 4 to 4.5 times of the theoretical value (excavated borehole volume), indicative of the very porous nature of the beach deposits. Two types of micropiles with lengths of 10 and 15 meters were installed in 3×3 m grid (Fig. 5). The pile spacing and efficiency was chosen based on the previous experience in the similar projects and also due to in situ trial grouting tests. The grouting pressure for each micropile was considered to be about 1000 to 1200 kPa. Standard penetration tests were performed after micropile installation. Then the liquefaction potential of the soil layer was determined after soil improvement based on the new SPT results. The obtained results are presented in the Table 2 and Fig. 6. The results show that the SPT values of the soil layers were increased after soil improvement with micropile installation. Therefore, the liquefaction resistance of the sandy soil was improved.

Fig. 3. Micropile after Installation

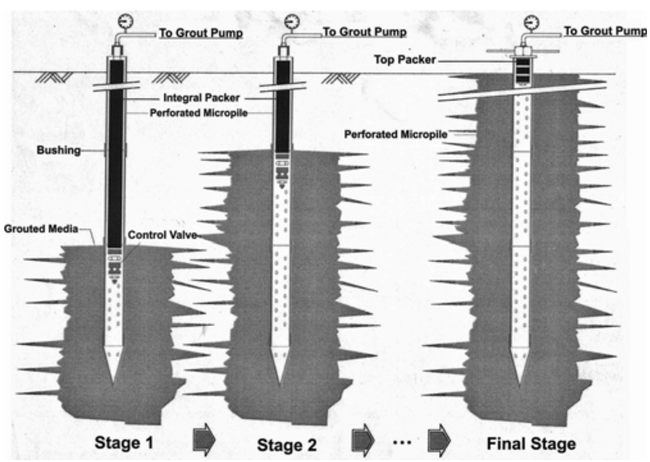

Fig. 4. Micropile Installation Process

Fig. 5. Plan of Micropile Installation (KHANEH DARYA Complex)

Frictional resistance between the surface of the pile, soil and the associated group/network effects of micropiles are considered as the possible mechanism for improvement. The micropiles created as in situ coherent composite reinforced soil system and the engineering behaviour of micropile-reinforced soil is highly dependent on the group and network effects that influence the overal resistance and the shear strength of composite soil micropile system.

Past studies on micropile behavior on sands were shown that the network effect of micropiles is significantly influenced by initial sand state. Loose sand when subjected to shear leads to volume contraction (Shu and Muhunthan, 2010). It was assumed

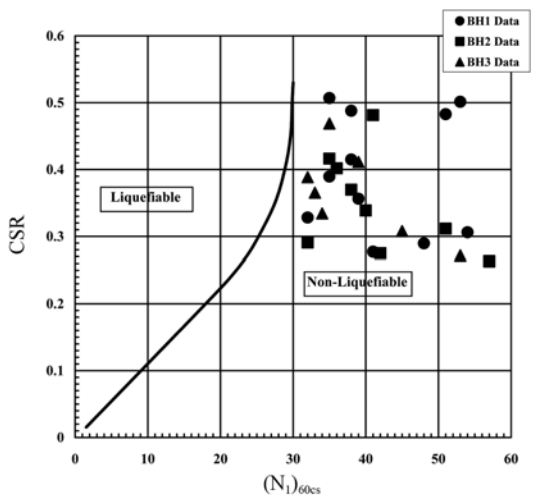

Fig. 6. Liquefaction Potential Evaluation (After Micropile Installation)

that densification of the soil surrounding the micropiles and the corresponding group effect is significant and effective in obtaining the desired level of improvement (Babu *et al*., 2004).

5. Plate Load Test Results

Plate load tests were carried out at the site in two stages including before and after micropile injection. The tests were conducted using 300 mm diameter, 35 mm thickness, rigid circular steel plates. The setup used for conducting the plate load tests was in accordance with the standard ASTM D1194-94 (1998). The load was applied through a system comprising a hydraulic jack, a reaction beam, and a load platform, and measured using a calibrated load cell. Four dial gauges with divisions of 0.01 and 50 mm travel were used for settlement measurement. The gauges were fixed to a reference beam and supported on external rods. The load was applied in cumulative equal increments of not more than one-tenth of the estimated ultimate bearing capacity.

Plate load tests (PLT1, PLT2, and PLT3) were executed out at three points of natural ground. The obtained results are produced in the Table 3. The secant module K_s of the soil recorded was about 1.7 kg/cm^3. This value is very low and related to very loose silty sand. Therefore, the vertical settlement of a shallow foundation constructed on this soil could be very large. The plate load tests performed after micropile injection at the initial places are designated (PLTm1, PLTm2, and PLTm3). The obtained results are shown in the Table 4. The secant K_S modulus of improved soil obtained were 85.3, 30.75 and 93.4 kg/cm^3 for PLTm1, PLTm2 and PLTm3 respectively. The small K_s obtained from PLTm2 is due to low cement infiltration. Based on the obtained results it can be concluded that the K_s module of the soil increased considerably after micropile injection.

Interpretation of testing data uses conventional correlations expressed in terms of applied pressure versus settlement, as shown in Fig. 7 for evaluating the effectiveness of micropile injection in each point of PLT test. As seen previously, the overall response of loose silty sand soils under surface loading was significantly improved by micropile injection. From the results, it can be stated that the bearing capacity of the soil increases considerably and the settlement decreases due to micropile injection.

Table 3. Plate Load Test Results on Natural Ground

P (kg)	σ (kg/cm^2)	d (mm)		
		PLT1	PLT2	PLT3
0	0.00	0.0	0.00	0.0
500	0.71	2.5	3.00	3.4
1000	1.41	5.1	5.63	6.1
1500	2.12	7.8	8.25	8.0
2000	2.83	10.6	12.75	12.5
2500	3.54	15.3	16.50	16.0
3000	4.24	20.4	21.00	21.0
3500	4.95	24.4	27.45	26.0
4000	5.66	33.0	37.50	37.5
4500	6.37	48.0	52.5	61.0
5000	7.07	84.0	90.0	81.0

Table 4. Plate Load Test Results after Micropile Installation

P (kg)	σ (kg/cm^2)	d (mm)		
		PLTm1	PLTm2	PLTm3
0	0.00	0.00	0.00	0.00
700	0.99	0.20	0.75	0.14
1400	1.98	0.34	1.15	0.24
2100	2.97	0.47	1.49	0.33
2800	3.96	0.60	1.75	0.42
3500	4.95	0.69	1.98	0.51
4200	5.94	0.80	2.21	0.57
4900	6.93	0.87	2.41	0.65
5600	7.92	0.97	2.65	0.74
6300	8.91	1.05	2.93	0.81
7000	9.90	1.16	3.60	1.08
6300	8.91	1.14	3.59	1.06
5600	7.92	1.11	3.52	1.05
4900	6.93	1.07	3.50	1.01
4200	5.94	1.00	3.46	0.96
3500	4.95	0.93	3.44	0.92
2800	3.96	0.85	3.39	0.87
2100	2.97	0.77	3.28	0.80
1400	1.98	0.60	3.04	0.69
700	0.99	0.42	2.75	0.57
0	0.00	0.12	1.45	0.33

6. Load-Displacement Responses of Micropiles

The load-displacement responses of three micropiles under tension loading are presented in Fig. 8. G14, G27 and H24 are the numbers of three-tested micropile in different places of micropile installation plan in two directions. For example G14 is the micropile in cross of two axes including G and 14. Micropile proof load tests are performed according to FHWA procedure to evaluate the ability of microplie installation to safety withstand in service design loads without excessive structural movement or long-term creep over the structure's service life. Design loads for 10 and 15 meter length micropiles are 7.2 and 10.8 tons respectively. Therefore, the maximum proof test loads were selected as 12 and 18 tons that were equal to 1.67 times of design load. The responses clearly show that the all tested micropiles have elastic behavior until 12 ton vertical load. The maximum vertical displacement is about 1.35 mm and the maximum residual displacement is about 0.13 mm. The results verify that the micropile installation was successful.

As it is shown in Fig. 8, the unloading curves look nonlinear even thought it is expected to be elastic. It is due to the network effect of surrounding soil. Micropile support loads individually as a group or as a network. In cases of groups and networks, the micropiles and the surrounding soil will form a composite block

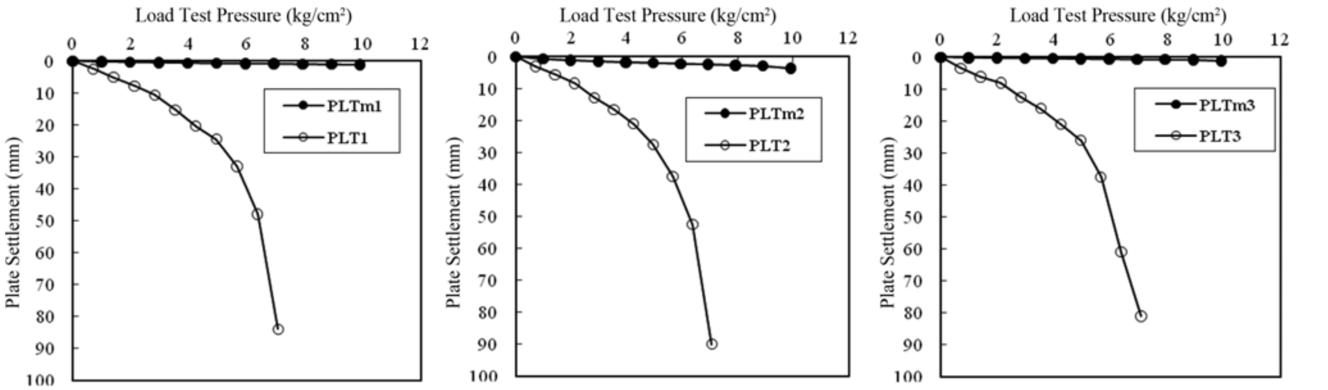

Fig. 7. Compare of Pressure versus Settlement Curves for Plate Load Test

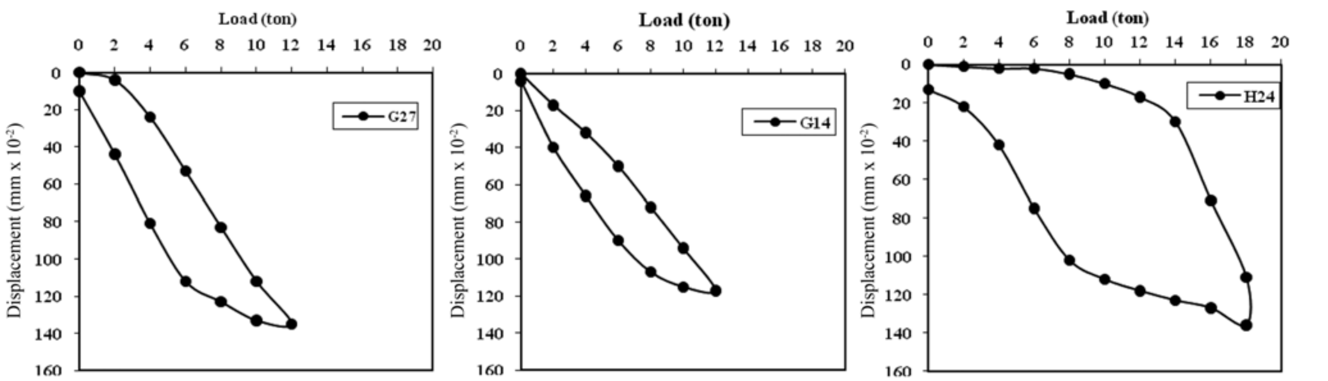

Fig. 8. Load-Displacement Response in Micropile Loading Tests (G14, G27 and H24)

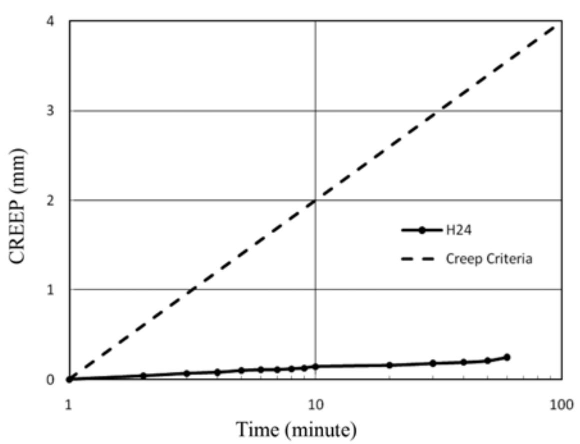

Fig. 9. Displacement Creep vs. Time Curve (H24 Micropile)

to resist the applied soil (Muhunthan, 2005). The nonlinear behavior of the soil has a significant influence on the response of the micropile to loading. Micropile in highly nonlinear soil has a larger modulus reduction and larger settlement than the linear soil (Mared and Muhunthan, 2005).

The creep test was performed in the H24 micropile at 18 ton load step with 60 minute hold time and the obtained results were presented in Fig. 9. As it is shown the creep settlement is lower than the acceptance criteria.

7. Conclusions

This field study was conducted to investigate the effectiveness of micropile installation on load transfer mechanisms and the deformation behavior of loose silty sand subjected to additional surface loading. The following observations and conclusions were made regarding the results of Standard Penetration Test (SPT) and six plate-loading tests (300 mm diameter, 35 mm thick) carried out directly on the loose silty sand stratum before and after micropile injection.

1. The response of loose silty sand soils under surface loading can be significantly improved by micropile installation.
2. The bearing capacity of loose sandy soil increased considerably and the ground settlement also decreased due to micropile installation.
3. Based on the recorded results it can be concluded that the stiffness K_S of soil is increased considerably after micropile installation.
4. The SPT values of soil layers are increased after soil improvement with micropile installation. Therefore, the liquefaction resistances of loose silty sand soil are also improved.

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