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Research Note

Study of pile behavior by improvement of confining soils using frustum confining vessel

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Post-grouting.

Abstract. A physical modeling apparatus made at Amirkabir University of Technology (FCV-AUT), called Frustum Confining Vessel, is employed to determine the effects of soil improvement on piles performance. The FCV-AUT is capable of linearly modeling the existing overburden stresses in the field. Three types of Babolsar sand with relative densities between 20% and 70% were examined in these tests. Results of compression and tensile load tests on seven piles, with the embedment length to diameter ratio (D/B) of 9, were used to evaluate the effects of soil improvement methods of soil densification as well as post-grouting on the piles performance. The results show that the increased bearing capacity by soil compaction from loose to medium state is greater than that of the medium to dense state. This increase has been observed to be a function of pile construction methods and pile tip conditions. Also, tip and/or side post-grouting enhanced the performance of bored and concrete precast piles by increasing soil-pile interaction and its bearing capacity. The results of pile load tests carried out on the FCV-AUT have been observed to be well-aligned with the static bearing capacity calculations based on the API method.

1. Introduction

Due to the increased number of construction projects, retrofitting alongside with soil improvement issues have become the center of notice in civil engineering practice since the past two decades. When the site's geotechnical condition is poor, several strategies are suggested such as transferring the project to a suitable location, designing a structure based on poor geotechnical conditions, and using soil improvement methods. The rapid development of different ground improvement methods and their equipment is an indication of consultants and taskmasters' interest toward soil improvement methods. Soil improvement techniques result in the enhancement of the mechanical behavior as well as erosion resistance of soils [1-3].

In projects in which designed piles are not adequate for supporting structural loads, a suitable soil improvement method along with piling can be used to recover the possible defections of pile performance. By reducing the number of piles and construction materials, soil improvement methods can make unusable lands suitable for construction which leads to economic savings [4].

Different soil improvement methods for modification of soils around the bottom and side of piles have been reported by different researchers. The use of cemented soil walls around the piles increases soil stability and resistance to lateral loads and controls settlements of structures [5]. Cement grouting enhances stability and the bearing capacity of deep foundations by increasing the soil and pile interactions [6].

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Post-grouting increases the piles bearing capacity [7]. Cement Deep Soil Mixing (CDSM) increases the pile resistance against lateral loads [8]. Pre-consolidation through preloading in clay soils, shallow densification (road construction works), dynamic compaction, vibro-rotation and vibro-replacement, and explosion are soil improvement methods by which soil density increases [1,9].

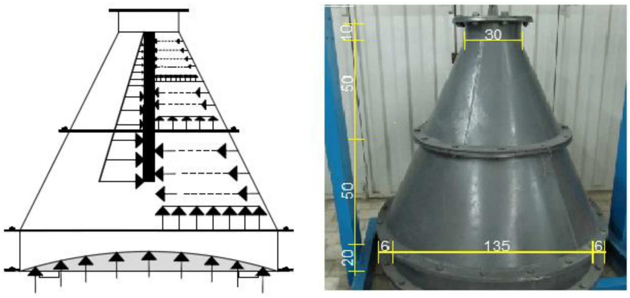
In this experimental study, ground improvement methods, including soil compaction, post-grouting, and pile construction, are investigated via the FCV physical modeling apparatus to evaluate the simultaneous use of piling and ground improvement.

2. Physical modeling of deep foundations

Physical modeling in geotechnical engineering is divided into two main categories of 1g modeling and high stress levels modeling (ng). The ng method is carried out via three apparatuses: Calibration Chambers (CC), geotechnical centrifuges, and Frustum Confining Vessels (FCV) [10,11]. 1g model devices are cheap, but are not able to model the stress gradient acceptably well. Calibration chambers can create high level stresses within the soil, but the stress gradients are uniform throughout the soil specimens. While geotechnical centrifuges are able to well create stress gradients in soil, they are expensive and limited in the world. An FCV apparatus with the ability to create stress gradients acceptably well in depth is much cheaper than are geotechnical centrifuges [10]. Due to the conical shape of the FCV along with the applied pressure at the bottom of the FCV, approximately linearly horizontal and vertical stress distributions are created within the soil similar to those may be found in the field [2,12]. In fact, the horizontal and vertical stresses are zero at the top of the vessel exposed to the open air and increase almost linearly from the zero at the top of the vessel toward the bottom of the device where they are proportional to the applied base pressure. Another noteworthy feature of the FCV is the stress gradient created in the soil which can be reduced or increased by applying the base pressure by means of a hydraulic system [13].

3. FCV-AUT

One FCV apparatus, called FCV-AUT, was built in 2011 in the Amirkabir University of Technology. The height, bottom diameter, and top diameter of the FCV-AUT are 1300 mm,1350 mm, and 300 mm, respectively. Large dimensions of the FCV-AUT would cause the test conditions to be more similar to the field and the pile construction inside the device for convenience. The FCV-AUT consists of a main body



(a) (b)

Figure 1. (a) Stress distribution in FCV. (b) FCV-AUT dimensions (cm).

made of two separable pieces of steel sheet, each of which with a thickness of 10 mm. A 15 mm steel plate is placed at the bottom of the FCV-AUT to which a rubber membrane is attached to apply the base pressure to the soil specimen [10,11] (Figures 1 and 2).

The system, for supplying the base pressure, consists of an air compressor, an air-water tank, and the rubber membrane. For applying the base pressure, the compressed air of the air compressor is used to push the water of the water-air tank to the space between the rubber membrane and the bottom steel plate. This system is capable of creating maximum pressure up to 600 kPa, which is almost equivalent to the pressure existing at the depth of 30 m of soil [10]. In these tests, the maximum base pressure of 200 kPa is used, which is equivalent to the pressure at the depth of about 10 m of soil.

Load tests on piles are conducted by a manual hydraulic system. This system includes a 15-ton hydraulic jack with a bilateral piston, which enables compressive and tensile loading, as well as a manual hydraulic pump by which the hydraulic power is provided. The load and displacement measurements of the model piles are performed by means of a 10-ton S-shaped load cell and a Linear Vertical Displacement Transducer (LVDT) with a high precision of 50 mm, respectively. The Data Acquisition System (DAS) includes a PC, an eight-channel data logger, and Soil Pressure Meters (SPMs) with the ability to measure pressures up to 1000 kPa (Figure 3).

4. Experimental studies by FCV-AUT

Babolsar sand from north coast of Iran was used for tests conducted in this study. Based on gradation tests in accordance with ASTM-D-6913, parameters of C_c, C_u, and D_{50} were obtained to be 1.22, 1.67, and 0.18 mm, respectively; the soil was classified as SP based on the Unified Soil Classification System. According to ASTM-D-4253 and ASTM-D-4254, maximum and minimum dry unit weights of the sand were 18 and 14.85 kN/m^3, respectively [10].

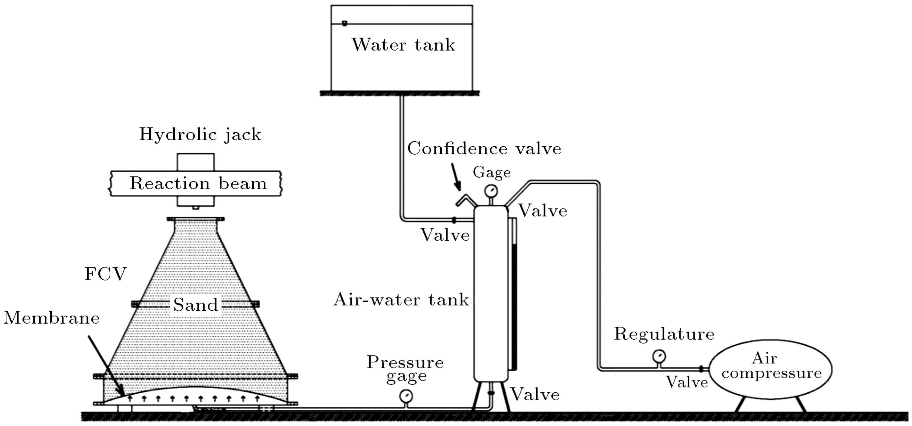


Figure 2. Schematic profile of FCV at AUT and detailed bottom pressure system [16].

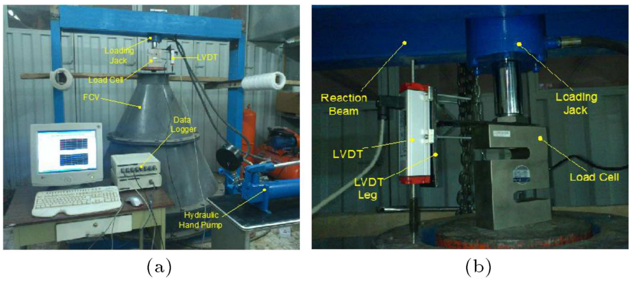


Figure 3. Overview of FCV-AUT and its accessories.

Soils with water content of 4% are used for the experiments as this water content is common in the field. Moreover, laboratory limitations prevent the use of neither saturated soils nor dry soils due to difficulty in the setting up of these tests in both cases. The tests were conducted in three different relative densities of 20 to 25% (loose sand), 45 to 50% (medium sand), and 65 to 70% (dense sand). Medium and dense sands are considered as improved soils in this study. To achieve the desired density, a specific amount of soil was poured into the constant volume of the FCV-AUT (0.57 m^3) uniformly in several layers. Then, hitting the body of the FCV-AUT by a hammer leads the soil inside the device to achieve the desired density uniformly within the apparatus. To control the uniformity of soil, each layer was formed of about 3 to 5 cm thickness. It is worth mentioning that 900 to 1010 kg of soil was used each time to fill the whole volume of FCV-AUT.

To measure the stress distribution within the FCV-AUT, soil pressure meters were used in 4 different depths. First, they were placed horizontally to measure vertical stresses. Then, the sensors were set in the vertical direction to measure horizontal stresses approximately at the same location. These tests were repeated for 3 different states of loose, medium, and dense soil states when the applied base pressure of the FCV-AUT was 200 kPa (Figure 4).

As mentioned, tests were conducted on various piles with different materials, cross sections, and construction methods. They included the concrete precast (precast pile placed in drilled hole), bored, concrete driven, steel open-ended, steel close-ended, jacked, H-shaped, tip and shaft post-grouted piles. It is important to mention that all piles were designed and made equal to the volume of a cylindrical pile with a diameter of 90 mm and an embedment depth of 750 mm with the embedment length to diameter ratio (D/B) of 9. These piles are shown in Figure 5.

A 6-kg steel hammer with a drop height of approximately 450 mm was used to drive steel and concrete piles to the soil. A base pressure of 2 bars (200 kPa) was applied to the bottom of the FCV in all tests after the piles reached the desired depth and before pile loading. Post-grouted piles were tested with the grout pressures of 5 and 6 bars for tip post-grouting and 2 bars for shaft post-grouting. Considering the relative density of the soil as well as the vertical and horizontal stresses existing at the pile tip in these tests, the model piles are expected to be about 10 to 14 m long when converted to the real condition existing in the field.

The rapid load test method in accordance with ASTM-D-1143 was used for static loading of piles. First, the bearing capacity of each pile was estimated. Then, in loading stages, loading increments proportional to the ultimate bearing capacity are applied to the pile until the pile head displacement gets fixed. Time intervals and loading steps of these tests were 1 kN in 2 min, 2 kN in 2 min, and 2.5 kN in 2 min for piles in the loose, medium, and dense soil states, respectively.

5. Results

Compressive and tensile load tests were conducted on each of the seven piles in soils with 3 different relative densities of 20 to 25 %, 45 to 50 %, and 65 to 70 % In all

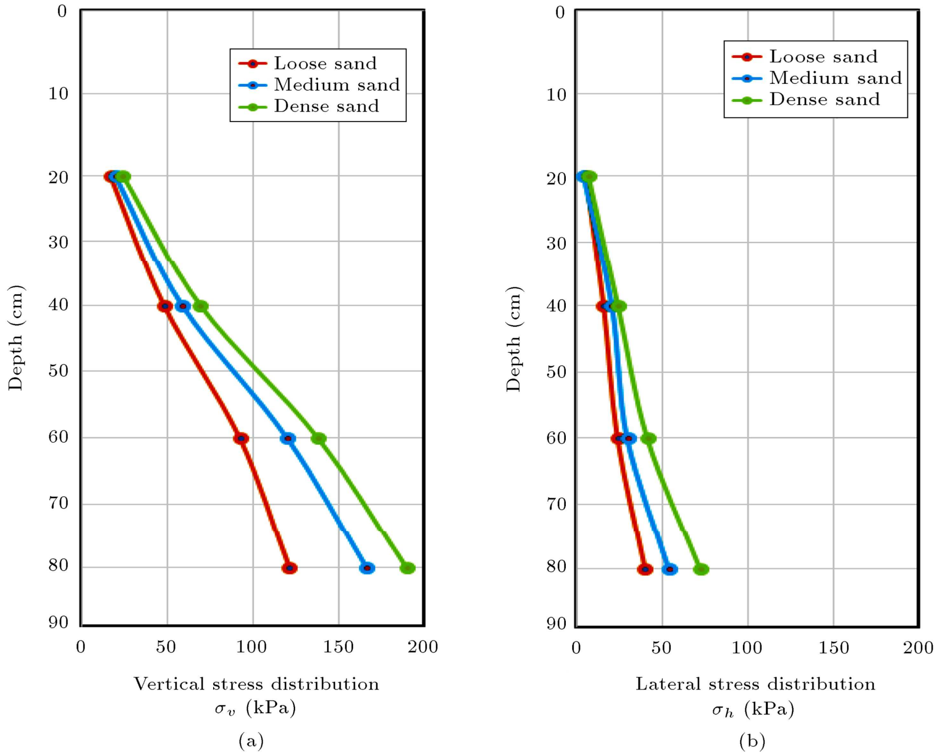


Figure 4. Stress distribution in FCV-AUT: (a) Vertical and (b) lateral.

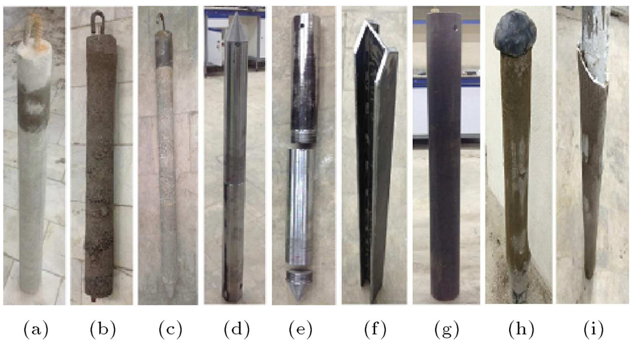


Figure 5. (a) Concrete precast pile. (b) Bored pile. (c) Concrete driven pile. (d) Steel close-end pile. (e) Jacked pile. (f) H-pile. (g) Steel open-end pile. (h) Tip post-grouted pile. (i) Shaft post-grouted pile.

of the tests, the loading continued up to a displacement of 15% of the pile diameter, and the ultimate bearing capacities of piles were calculated based on the Brinch Hansen's method (1963) [14]. The load-displacement curves of piles in compressive and tensile load tests are illustrated in Figures 6 and 7, respectively (the bearing capacity of the concrete precast pile in tension was too negligible, and its graph was not plotted). Based on Figure 6, it may be concluded that since the precast concrete pile has little side interaction with the surrounding soils, the initial slope of the load-displacement curve in the compressive load test for this pile is less than those of other piles. The comparison of concrete bored and driven piles indicates that the bearing capacity of the bored pile is more than that of the driven pile. The fact is that in this experimental study, the conditions of bored piles were ideal, meaning that there is no debris in the hole in time of pile installation, while in real conditions, some debris remain in the pile hole due to hole drilling which results in the lack of side friction of the pile with the surrounding soil as well as reduction of pile's bearing capacity. This fact highlights the effects of installation methods on bored pile performance, and indicates that when there is no debris in the hole, bored piles can have bearing capacities equal to or even greater than those of driven piles. Comparison of the close-ended pile with the jacked pile, installed with a different installation method, shows that the jacked pile had 20% to 30% more bearing capacity compared to the close-ended pile. However, the close-ended pile had more capacity compared to the open-ended pile, especially in the dense state of soil.

Based on Figure 7, it may be observed that in tensile load tests, bored piles have bearing capacities very close to those of the driven piles. For the driven piles, the bearing capacity of the steel open-ended pile is more than 15% that of the H-pile. The comparison of the driven steel close-ended and the jacked pile, tested differently in installation methods, shows that, in tension, the jacked pile is able to carry loads more than 20% that of the driven piles.

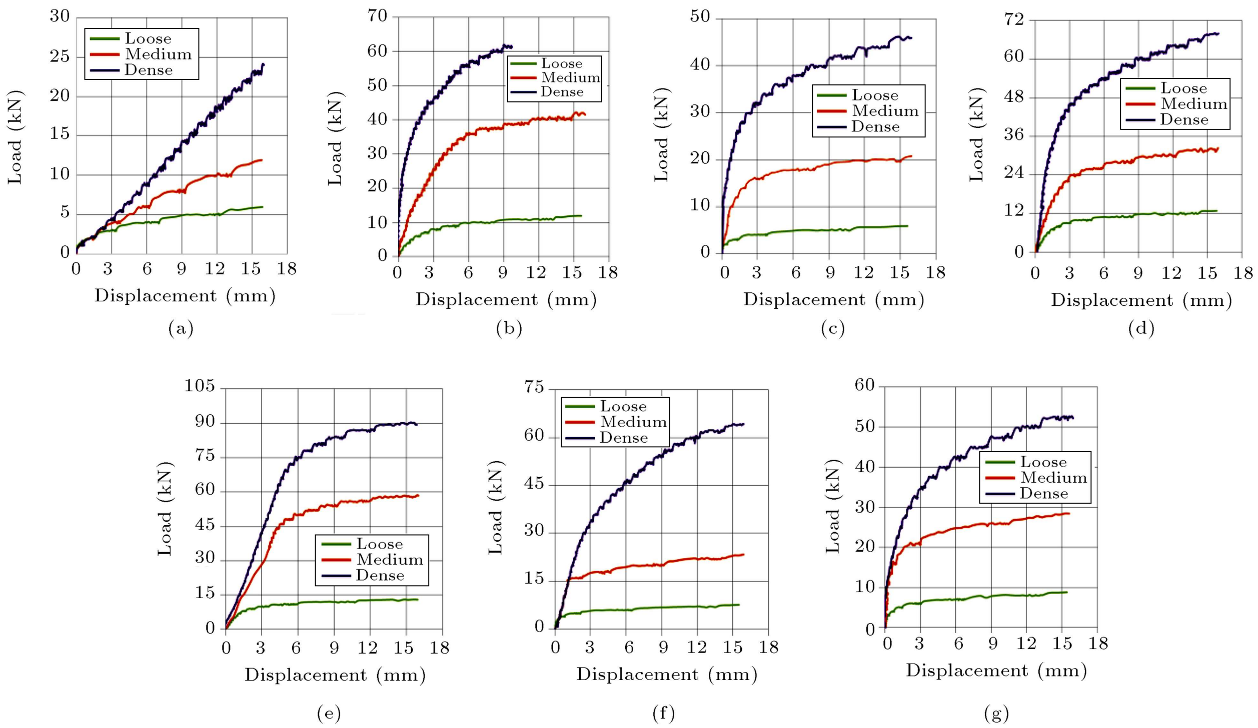


Figure 6. Load-displacement for compression load test of model: (a) Concrete precast pile, (b) bored pile, (c) concrete driven pile, (d) steel close-end pile, (e) jacked pile, (f) H-pile, and (g) steel open-end pile.

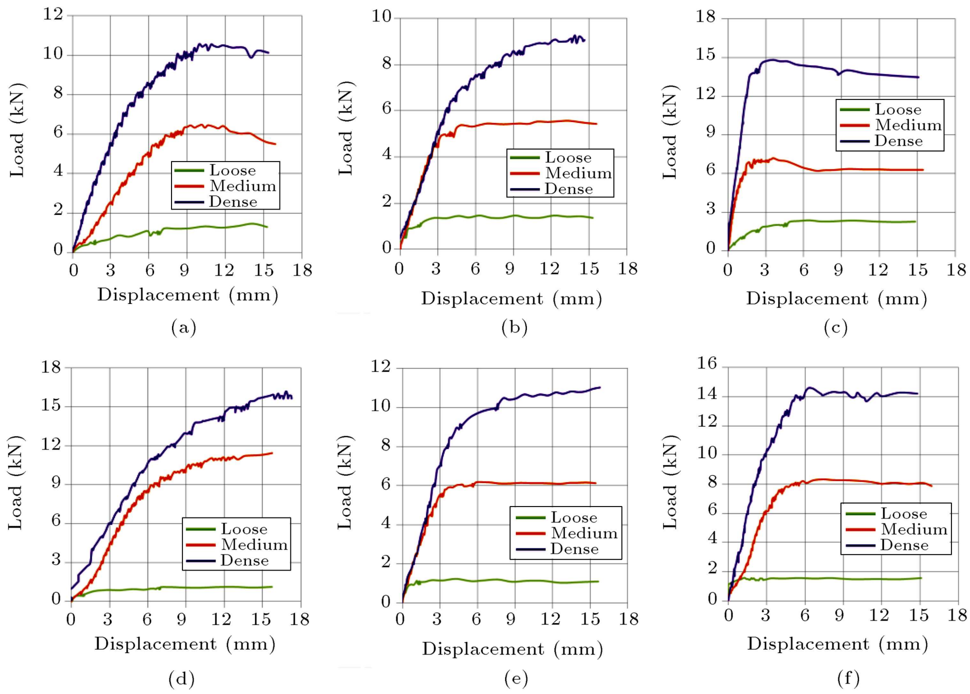


Figure 7. Load-displacement for tensile load test of model: (a) Bored pile, (b) concrete driven pile, (c) steel close-end pile, (d) jacked pile, (e) H-pile, and (f) steel open-end pile.

Figure 8 shows enhancement of the bearing capacity in terms of soil density improvement. Based on this figure, the increase in bearing capacity of piles by soil improvement from the loose to medium soil state is much more than that of the medium to dense state. In general, the increase in bearing capacity of piles has been observed to be more than 100% for most of cases.

Improving the soil density from the loose to medium state has resulted in the following: In com-

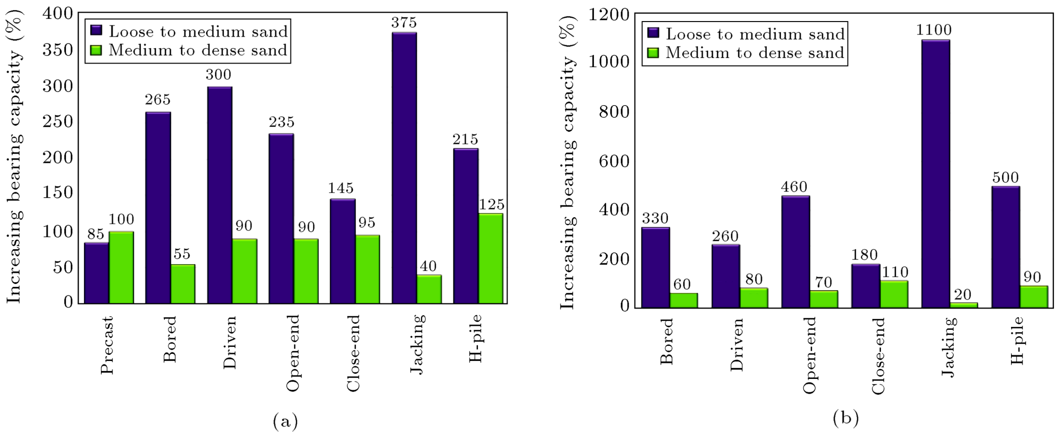


Figure 8. Increasing bearing capacity (%) with increasing of densification in tests: (a) Compression, and (b) tensile.

pressive load tests, the jacked pile and the precast concrete pile showed the most and the least increases in compressive capacity by 375% and 85% increases, respectively. Jacked pile and driven close-end piles of tensile load tests showed the most and the least increases in bearing capacity by more than 1000% and 180% increases, respectively. For soil density improvement from the medium to dense state, the results were as follows: In compressive load tests, the driven H-shaped pile and the jacked pile had the most and the least increases in bearing capacity enhancement by 125% and 40% increases, respectively. In tensile load tests, the close-ended pile and the jacked pile had the most and the least increases in the bearing capacity enhancement by 110% and 20% increases, respectively. Also, based on Figure 8, improving the soil state from the loose to medium affects the tensile capacity of piles more than their compressive capacities.

Figure 9 shows load-displacement curves of the tip post-grouted bored pile and the shaft post-grouted precast concrete pile. To take into account the effect of pile hole drilling on the performance of bored piles and the tip post-grouted pile, about 50 mm thick soil as debris was poured manually to the pile hole before casting the pile. Based on Figure 9, tip post-grouting at the pressures of 5 and 6 bars (500 and 600 kPa) improves compressive bearing capacities of bored piles up to about 25% and 40%, respectively. In tensile load tests, the increase in the bearing capacity by tip post-grouting at the pressure of 6 bars was about 30%. Also, Side post-grouting of precast concrete piles improves its compressive bearing capacity up to 40%. While the precast concrete pile showed little side resistance in tension before grouting, side post-grouting increased its side resistance up to 1.5 kN. The important factor in these tests is controlling the upper displacement of piles during grouting. According to the FHWA (2010) [15], upper displacement should be limited to 6-12 mm; exceeding the limits will result in a decrease in the side, and hence, in the pile total bearing capacity.

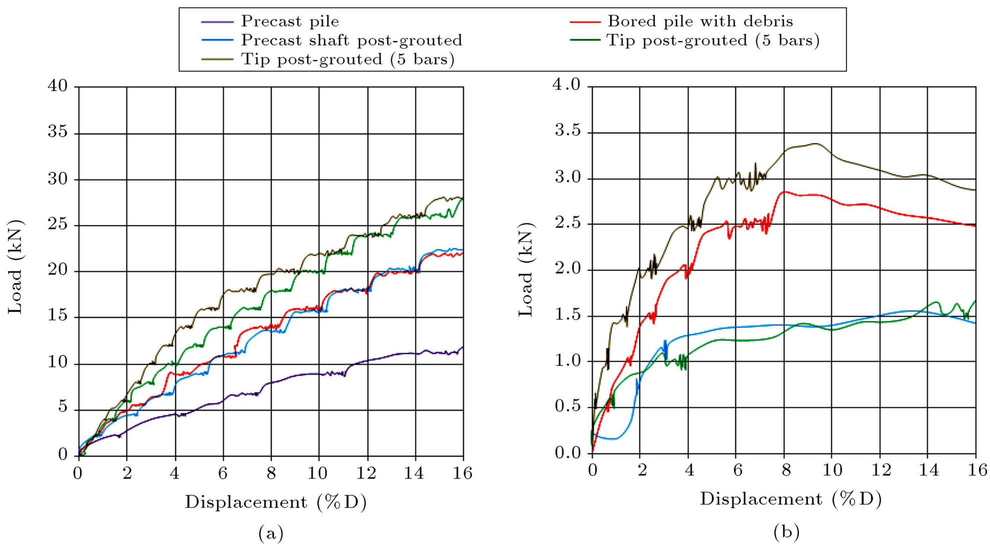


Figure 9. Load-displacement of post-grouted piles: (a) Compression test and (b) tensile test.

Table 1. FCV scaling factors [12].

Dimension	Prototype	FCV model
Length	1	1/N
Mass	1	1/N^3
Force	1	1/N^2
Displacement	1	1/N

Table 2. Pile specifications.

Soil densification	Loose sand	Medium sand	Dense sand
Soil stress at pile tip (kPa)	120	160	180
Soil unit weight (kN/m^3)	16	16.8	17.5
Pile length(m)	7.5	9.5	10.5
Pile diameter(cm)	90	115	125

6. Comparison of test results with API and unified pile design methods

By having the soil stresses as well as the unit weights of the soil, the embedment length of the scaled pile is achievable ($\sigma'_z = \gamma h$). Dividing the scaled pile length by 750 mm, which is the embedment length of all model piles, will yield the scale coefficient factor. Scale factors were applied based on the similarity theory presented by Sedran (1999) (Table 1) [12]. Length, mass, force, and displacement scaling factors were used to relate the model pile to the prototype, as described in Table 2.

To compare the results of loading on tested piles within the FCV-AUT with common static relationships used in practice, the unified pile design [16] as well as American Petroleum Institute (API) methods were used. Based on the unified pile design method [16], tip (r_t) and side (r_s) resistances are calculated according to the following equations:

$$r_t = N_t.\sigma'_z, \tag{1}$$

$$r_s = \beta.\sigma'_z, \tag{2}$$

where β and N_t are bearing capacity coefficients, p'_0 and σ'_z is the effective stress. Based on the American Petroleum Institute (API) method (1990), the side and tip resistances of piles are calculated from Eqs. (3) and (4):

$$r_t = N_q.\sigma'_z, \tag{3}$$

$$r_s = K.p'_0.\tan(\delta), \tag{4}$$

where K is the lateral earth pressure coefficient, p'_0 and σ'_z are the effective stresses at pile tip level, and δ is the friction angle between soil and pile.

Specifications of modeled piles are presented in the Table 3. Using Eqs. (1) to (4), the compressive and tensile bearing capacities of the piles tested within the FCV-AUT are calculated based on the API and the unified pile design methods, presented in Tables 3 and 4, and Figure 10.

The comparison of the compressive and tensile capacities of piles obtained from laboratory load tests with those calculated by API and unified pile design

Table 3. Bearing capacity of compressive tests and comparison by API and unified methods ($\times 10^3$kN).

Pile type	Loose sand			Medium sand			Dense sand		
	Test	API	Unified	Test	API	Unified	Test	API	Unified
Concrete precast	0.6	0.86	1.33	1.86	2.98	4.48	4.31	7.74	9.67
Bored	1.1	1.28	1.78	6.76	4.48	6	12.15	10.70	12.72
Driven	0.55	1.22	2.35	3.72	4.19	9.02	8.23	9.72	22.13
Close-end	1.3	1.75	3.50	5.41	6.07	13.38	12.38	14.35	33.16
Jacking	1.3	1.75	3.50	10.48	6.07	13.38	17.25	14.35	33.16
H-pile	0.7	1.47	2.80	3.72	5	10.73	11.76	9.59	26.84
Open-end	0.8	1.29	2.51	4.56	4.45	9.64	10	10.33	23.57

Table 4. Bearing capacity of compressive tests and comparison by API and unified methods ($\times 10^3$kN).

Pile type	Loose sand			Medium sand			Dense sand		
	Test	API	Unified	Test	API	Unified	Test	API	Unified
Bored	0.15	0.48	0.37	2	1.16	0.95	2.06	1.84	1.61
Driven	0.15	0.55	0.48	0.93	1.33	1.69	1.96	2.11	2.91
Close-end	0.25	0.77	0.68	1.18	1.73	2.18	2.94	2.71	3.69
Jacking	0.10	0.77	0.68	2.03	1.73	2.18	3.13	2.71	3.69
H-Pile	0.10	0.66	0.57	1.01	1.61	2.05	2.25	2.61	2.58
Open-end	0.15	0.51	0.44	1.43	1.32	1.69	2.84	2.13	2.94

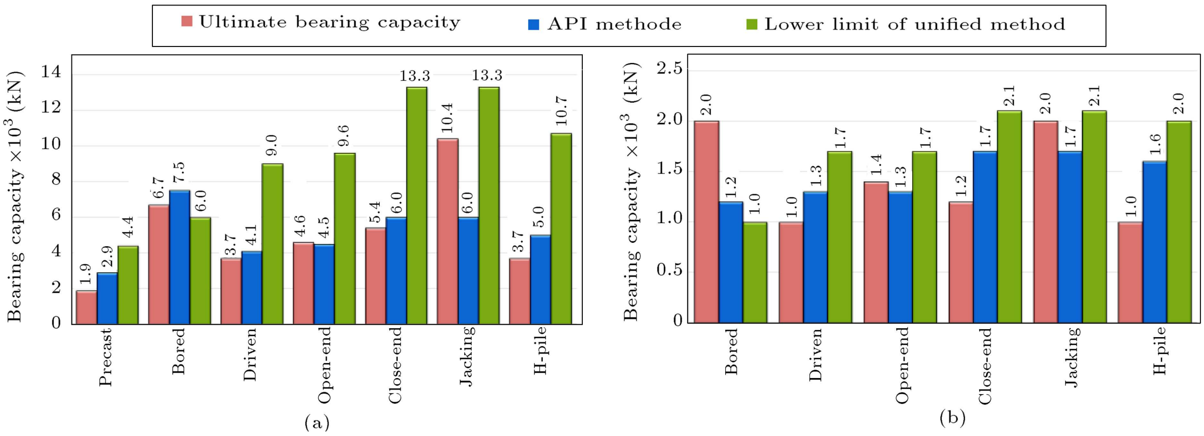


Figure 10. Comparison of bearing capacity of different piles with unified pile design and API method in medium sand: (a) Compression test, and (b) tensile test.

methods indicates that the results of experimental tests conducted on the FCV-AUT are more similar to those calculated by API method. In most compressive load tests, the difference of the bearing capacities between experimental tests and API method's calculations is less than 15%. However, in tensile load tests, this difference is more than 30% for most of the piles.

7. Conclusions

The effects of soil improvement on performance of seven different piles have been studied via the FCV-AUT which is a physical modeling apparatus that can create almost linear stress distributions similar to those occurring in the field. The model piles were tested in the loose, medium and dense states of Babolsar sand. The effects of soil improvement methods of soil compaction and post-grouting as well as pile construction method on the performance of deep foundations, investigated in this study, have the following results:

- Soil densifications around piles increase piles' bearing capacities. This increase from the loose to medium state is greater than that from the medium to dense state. In compressive load tests, the bearing capacity of driven concrete piles has increased more than that of the bored piles about 15% by soil improvement from the loose to medium state and 60% from the medium to dense state. In comparison to the close-ended types, open-ended steel piles are observed to have nearly 60% more increase in the compressive bearing capacity;
- Tip and side post-groutings of the bored and concrete precast piles increased the bearing capacity by improving the soil-pile interaction, which means that the soil-pile interaction has an important role in increased bearing capacity. Increased bearing capacity for tip and side post-groutings has been observed to be up to about 35% and 30%, respectively;
- By comparing the bearing capacity of piles obtained from laboratory load tests with those calculated from the API and unified pile design methods, it was observed that in the compressive and tensile load tests, experimental results are close to those of API method;
- The high ability of the FCV-AUT in modeling stresses up to 30m depth of soil as well as its lower operation cost compared to the centrifuge modeling will make it a suitable option for modeling different types of deep soil elements, such as piles, stone columns, jet-grouting, etc.

References

1. Moseley, M.P. and Kirsch, K., *Ground Improvement*, 2nd Edition, Spon Press Publication, 430 pages, London (2004).
2. Mullins, G., Dapp, S., Fredrerick, E. and Wagner, R. *Pressure Grouting Drilled Shaft Tips-Phase I final report*, Florida Department of Transportation, Fla (2001).
3. Sieradzki, M.P., Bartlett, W., Patton, B.W., Sereno, D.J. and Wehrlen, P. *Deep Dynamic Compaction: Practical and Cost-Effective Ground Improvement at the Port of Long Beach*, American Society of Civil Engineering, ASCE, Ports 2013; pp. 630-639 (2013).
4. Pinto, A., Pereira, A. and Cardoso, D. "Ground improvement solutions at Sana Vasco da Gama Royal Hotel", *Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering*, pp. 2180-2184 (2009).
5. Yamashita, K., Hamada, J., Onimaru, S. and Higashino, M. "Seismic behavior of piled raft with ground improvement supporting a base-isolated building on soft ground in Tokyo", *Soils and Foundation*, **52**(5), pp. 1000-1015 (2012).
6. Dapp, S.D., Muchard, M. and Brown, D.A. "Experience with base grouted drilled shafts in the southeastern United States", Deep Foundations Institute,

10th International Conference on Piling and Deep Foundations, Amsterdam (2006).

7. Mullins, G. and Winters, D. *Post Grouting Drilled Shaft Tips– Phase II Final Report.* Florida Department of Transportation, Fla (2004).
8. Liu, Ch., Soltani, H., Pinilla, J.D., Muraleetharan, K.K., Cerato, A.B. and Miller, G.A. "Centrifuge investigation of seismic behavior of pile foundations in soft clays", *Geo-Frontiers 2011*, pp. 585-594 (2011).
9. Federal Highway Administration (FHWA); *Ground Improvement Technical Summaries,* **1 & 2** (2001).
10. Zare, M. and Eslami, A. "Study of deep foundations performance by Frustum Confining Vessel (FCV)", *International Journal of Civil Engineering* (IJCE), **12**(4), pp. 271-280, Tehran, Iran (2014).
11. Zarrabi, M. and Eslami, A. "Behavior of piles under different installation effects by physical modeling", *International Journal of Geomechanics*, **16**(5), p. 04016014, USA (2016).
12. Sedran, G. "Experimental and analytical study of a frustum confining vessel", Doctoral Thesis, McMaster University (1999).
13. Horvath, R.G. and Stolle, D. "Frustum for testing model piles",*Canadian Geotechnical Journal*, **33**(3), pp. 499-504 (1996).
14. Brinch Hansen, J. " Discussion on hyperbolic stress-strain response, cohesive soils", American Society of Civil Engineering, ASCE, Journal of Soil Mechanics and Foundation Engineering, **89**(4), pp. 241-242 (1963).
15. Federal Highway Administration (FHWA); *Drilled Shafts: Construction Procedures and LRFD Design Methods.*, FHWA-NHI-10-016 (2010).
16. *CFEM; Canadian Foundation Engineering Manual*, Bitech Publishers Ltd. British Colombia (2006).

Biographies

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Mohammad Zarrabi received his MS degree in Geotechnical Engineering from Amirkabir University of Technology, Tehran, Iran, in 2014. He is currently a PhD candidate in Geotechnical Engineering at Polytechnic of Montreal. His main research interests include constitutive modeling, geotechnical earthquake engineering, deep and shallow foundations, physical and numerical modeling and soil modification.

Javad Khazaei received his BS degree in Civil Engineering from Bu-Ali Sina University of Hamedan in 2001, his MS and PhD degress in Geotechnical Engineering, respectively, from Buali Sina University in 2004, and Amirkabir University of Technology in 2016. His field of research interest is mainly related to the areas of piles, FCV physical modeling, helical piles and soil improvement.