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Study of deep foundation performances by frustum confining vessel (FCV)

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Abstract

Physical modeling for study of deep foundations can be performed in simple chambers (1g), calibration chambers (CC), and centrifuge apparatus (ng). These common apparatus face certain limitations and difficulties. Recently, Frustum Confining Vessels (FCV) have been evolved for physical modeling of deep foundations and penetrometers. Shaped as the frustum of a cone, this device applies steady pressure on its bottom and creates a linear stress distribution along its vertical central core. This paper presents the key findings in FCV, as developed in AUT. The FCV has a height of 1200 mm, with top and bottom diameters of 300 and 1300 mm, respectively. By applying bottom pressure up to 600 kPa, the in-situ overburden stress conditions, equivalent up to 40 m soil deposits, become consistent with the embedment depth of commonly used piles. Observations indicated that a linear trend of stress distribution exists, and this device can create overburden stress in the desired control volume along the central core. Moreover, a couple of compressive and tensile load tests were performed on steel model piles driven in sand with a length of 750 mm, and different length to diameter (L/D) ratios between 8-15. Comparison between measured and predicted ultimate capacity of model piles performed in FCV demonstrate a suitable conformity for similar confinement conditions in the field. Therefore, the FCV can be considered as an appropriate approach for the investigation of piling geotechnical behavior, and the examination of construction effects.

Keywords: *Deep foundation, Axial capacity, Physical modeling, Frustum confining vessel (FCV).*

1. Introduction

Deep foundation systems are commonly employed to support high-rise and important structures, guarantee the stability of tall buildings and provide suitable serviceability. The loading on difficult and problematic deposits is what makes their function more significant than others. Deep foundation applications are categorized according to single Shafts (caissons), groups and piled raft foundations.

Due to the critical role foundations have in major projects, the design trend regarding construction and costs focused on the optimum approaches. Therefore, prediction and study of pile analysis, design and performance have been challenging since the beginning of geotechnical engineering practice [1,2,3].

Several attempts and researches have been performed to investigate the determinants for geotechnical and structural capacity, concentrating on the efficiency of deep foundations [4,5]. Issues covered include the soil-pile, pile-pile and pile-soil-cap (raft) interaction. Proper recognition of these and other complex influences can bring about reasonable safety factors [6,7].

Static analysis, full scale pile load testing, and dynamic testing and analysis are commonly used to overcome the uncertainties involved in piling analysis and design. Owing to a few simplifying assumptions and empirical approaches regarding site investigation soil-pile-structure interaction (SPSI), and load or resistance distribution along pile, capacity values have not been provided [1],[8]. The performance of static and dynamic full scale pile load tests are expensive and time consuming. Besides, these monitoring and tests face a major limitation, which is that the capacity is not available until the pile is driven or constructed.

To overcome several difficulties pertaining pile behavior and performance, the experimental physical modeling method has been used to clarify pile-soil interaction. Repeatability, analogy, rapid measurement and cost efficiency are factors that allow physical modeling to be considered an intermediate technique in comparison to element and full scale load testing.

The aim of this paper is to investigate the current approach of physical modeling, and present fundamental factors involved in pile performances. This paper will also

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discuss the development of the Frustum Confining Vessel (FCV) device, along with a well-rounded analysis of the stress field and its relation to actual conditions and piling geotechnical aspects, such as capacity and displacement.

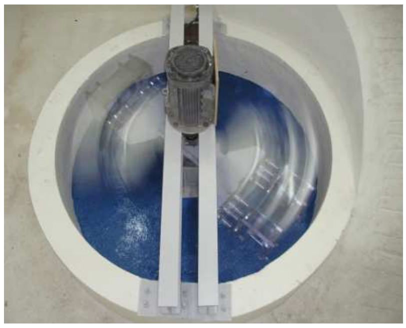
2. Background to Physical Modeling

Physical and small-scale models for the study of deep foundations include simple chambers (1g), calibration chambers (CC), and centrifuge apparatus (ng). The most important limitation of simple chambers is the low stress level in compared to the real condition surrounding the pile in the field. The 1g-stress field is not compatible with the true overburden stress distribution of prototype piles, and a simple correlation cannot be performed. Hettler and Gudehus [9], Franke and Muth [10] took stress distortions into account when analyzing model responses. These procedures, however, are mostly developed to address simple problems under service state or quasi-elastic soil responses, and cannot be applied to load testing problems on 1g model piles.

While, lateral and vertical stress levels in Calibration Chambers (CC) modeling can be increased, creation of constant lateral stresses did not produce realistic stress gradients. This is especially the case in the linear increase of stresses with depth, which generally governs the axially loaded piles. Centrifuge modeling has been evolved to overcome shortcomings, which entails extra costs, accessibility limitations, and rather complication performances. Fig. 1a through 1c illustrate pictures of the simple chamber used for pile modeling in 1g stress fields, the calibration chamber (CC) for modeling CPT and a geotechnical centrifuge apparatus used for physical modeling in ng, respectively.

(a)

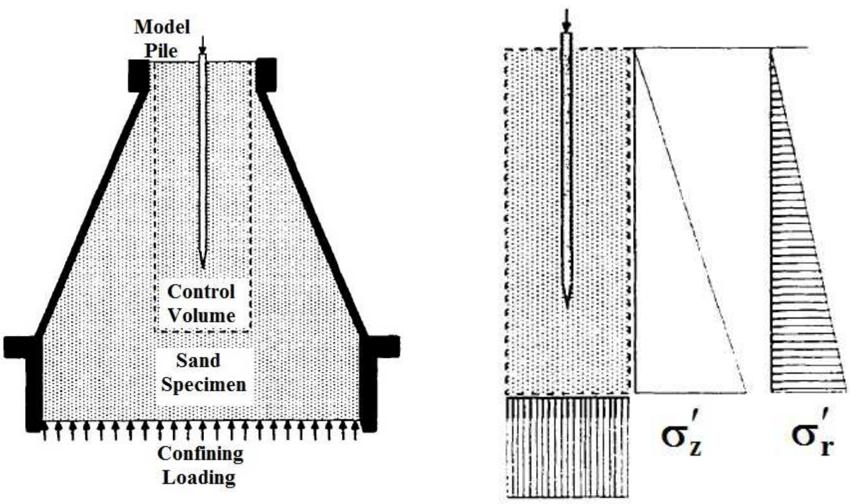
(b)



(c)

Fig. 1 Usual devices for modeling of pile a) simple Chamber at 1g [1] b) Calibration Chamber [2] c) Geotechnical centrifuge at Ng[3]

FCV has been developed for the physical modeling of deep foundations and penetrometers. This device is a frustum chamber that applies steady pressure to its bottom, and creates linear stress distribution along its vertical central line. The confining stress level in soil mass has been controlled by applied pressure to the bottom of the device. Schematic profiles of FCV and an idealized distribution of stresses within a control volume are illustrated in Fig. 2. The purpose of FCV is to produce, within the control volume, a state of stresses similar to those typically found in the field, while simultaneously control the stress levels.



(a) (b)

Fig. 2 a) Schematic of the FCV, b) Idealized distribution of stresses within control volume, [16]

Scaling factors to use with the FCV depend entirely on the degree to which it pressurizes. A model pile of length L_m can either be driven or cast into the control volume under any range of applied hydraulic pressure. The simulated in-situ depth at the toe of the pile determines the length of prototype L_m. The scaling factor can then be obtained from the result of these two quantities. Table 1 displays the particular set of scaling factors relevant to the FCV.

Table 1 FCV Scaling Factors

Dimension	Scale Factor
Length	$\lambda_L = L_p/L_m$
Area	$\lambda_{Area} = \lambda_L^2$
Volume	$\lambda_{Volume} = \lambda_L^3$
Density	$\lambda_\rho = 1$
Mass	$\lambda_M = \lambda_L^3$
Stress	$\lambda_\sigma = 1$
Force	$\lambda_F = \lambda_L^2$

The main advantage of the FCV when compared with other 1g testing environments for piles is the capability to provide more realistic lateral pressures and distribution on the pile's embedment length. The most direct advantage that FCV testing technique offers over the use of centrifuge devices are its relatively low running cost and the simplicity associated with its operation. FCV models also present technical advantages with respect to the volume and size of models. In the case of the centrifuge, the size and weight of payload is limited to the capacity of machinery. Typical geometric ration for model piles in centrifuges are between 1/60 and 1/100, whereas this ratio ranges from 1/10 to 1/30 for FCV models. This small scale modeling in centrifuge has made it difficult to prepare the model, and its inner instrumentation. However, new developments such as miniature sensors have provided partial solutions to this problem. White and Bolton [11]; Zare and et al. [12] developed image processing methods that assist in the interpretation of centrifuge test results. Usually in physical modeling, sand materials have not been scaled, and models are performed with the same sand as used in the prototype. Ovesen [13] presented experimental evidence demonstrating that circular footings embedded in sand with a ratio of footing diameter to a grain size (d/d_g) greater than 30, and the influence of the size on model response can be neglected.

The significant difference between the length of a model pile and the length of the centrifuge rotating arms is that centrifugal acceleration varies linearly along the model pile. Thus, acceleration is controlled only in an average sense, and will cause distortion in the linear stress gradient. Al-Douri et al.[14] discovered that, in small size models the behavior is influenced by near boundary effects. Schnaid and Houlsby [15] reports on the chamber size effects indicate that nearby artificial boundaries are responsible for distortions in the stress distributions on the soil. Moreover, difficulties in the instrumentation of pile-soil model increase when size of the model becomes smaller. While the behavior of deep foundation depends on the construction procedure, it is difficult to model construction effects while it is in centrifuge with small-scale and flying conditions.

Owing to all these considerations related to scale effects, it is suggested that larger FCV models are technically better suited than centrifuge models for the testing of model piles. Nevertheless, centrifuge models still offer the possibility of performing lateral load tests, while FCV models lack any experience in this area. Another limitation of FCV models is that undrained conditions may not be reasonably replicated because, pore pressure cannot be controlled as is the case of centrifuge devices.

3. FCV Developed in AUT

The FCV that is placed at the Amirkabir University of Technology (AUT) has a height of 1000 mm and an inside diameter of 300 and 1300 mm for the top and bottom, respectively. The top flange is 100 mm and bottom flange is 200 mm high; it functions as a transition zone of applied pressure (Fig. 3). The device is made of two pieces, and has better facilities. Sedran et al. [16] demonstrate that the stress distributions along the centerline obtained with membrane loading were smoother than in the case of piston loading. The AUT FCV is equipped with a membrane for applying bottom pressure.

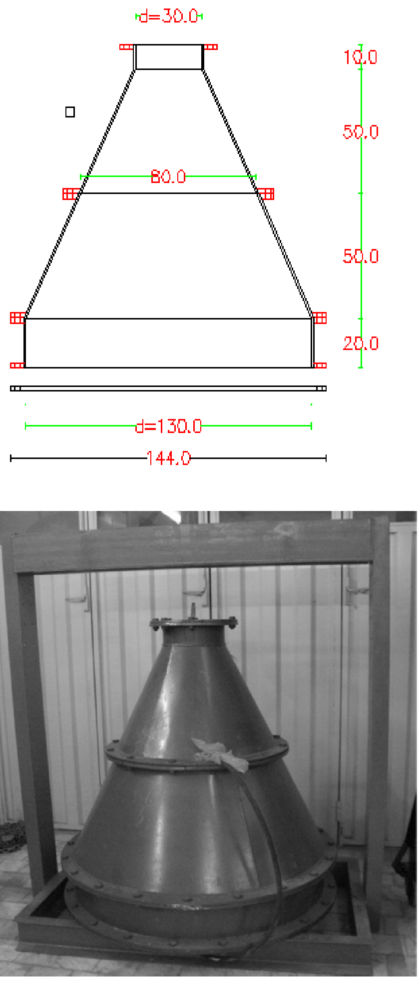


Fig. 3 FCV at AUT and its dimensions (cm)

As illustrated in Fig. 4, this device is divided to four major parts, the frustum body, a bottom pressure system, a loading system, and an instrumentation system. The bottom pressure system includes an air compressor, a regulator and an air- water tank. The compressor capacity is 800 kPa. Stresses acting at the bottom of the FCV are controlled by air regulators. As evident in Fig. 5, the water level in the tank has been identified by a transparent tube. The loading system includes the following, a loading frame, a hydraulic hand pump, and a hydraulic jack. The loading frame is comprised of steel channel sections designed to be suitably rigid, while the base of the loading frame is constructed by welding two1600 mm UNP160 sections with two 900 mm pieces of the same section at the end. Two columns composed by a welded UNP 200 at the base stand approximately 2200 mm tall. A series of holes were pre-drilled into the tops of the legs to provide a range of positions for the reaction beam. The reaction beam, made from two UNP180 sections is lowered into place through use of a gantry crane, and then bolted into place. As presented in Fig. 5, a set of bolt holes drilled into the column of the load frame, provides sufficient height clearance that allow the FCV to be installed on the base, and position the load testing equipment. The second working platform consisted a 1200 mm UNP100 bolted to the columns and two wooden pallets. A bi-direction hydraulic jack, designed for a maximum load of 15 tons and a maximum displacement of 150 mm, was made for applying tension and pressure on the load. A hydraulic hand pump provides the jack’s power, and has a switch valve for tension changes and loading pressures with a capacity to tolerate hydraulic pressures up to 300 bars.

The Instrumentation system includes a Data Acquisition System (DAS) and sensors. DAS has an 8 channel data logger, power pack and computer. Sensors have 10 tons of S-shaped load cells, an LVDT with 50 mm courses and five soil pressure cells with 1000 kPa capacity.

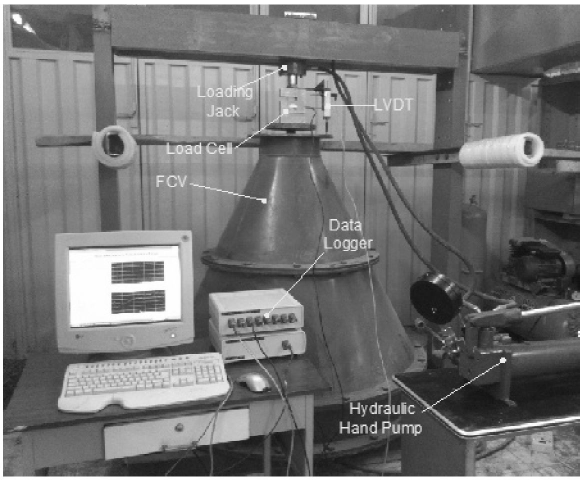


Fig. 4 Overview of FCV at AUT and its accessories

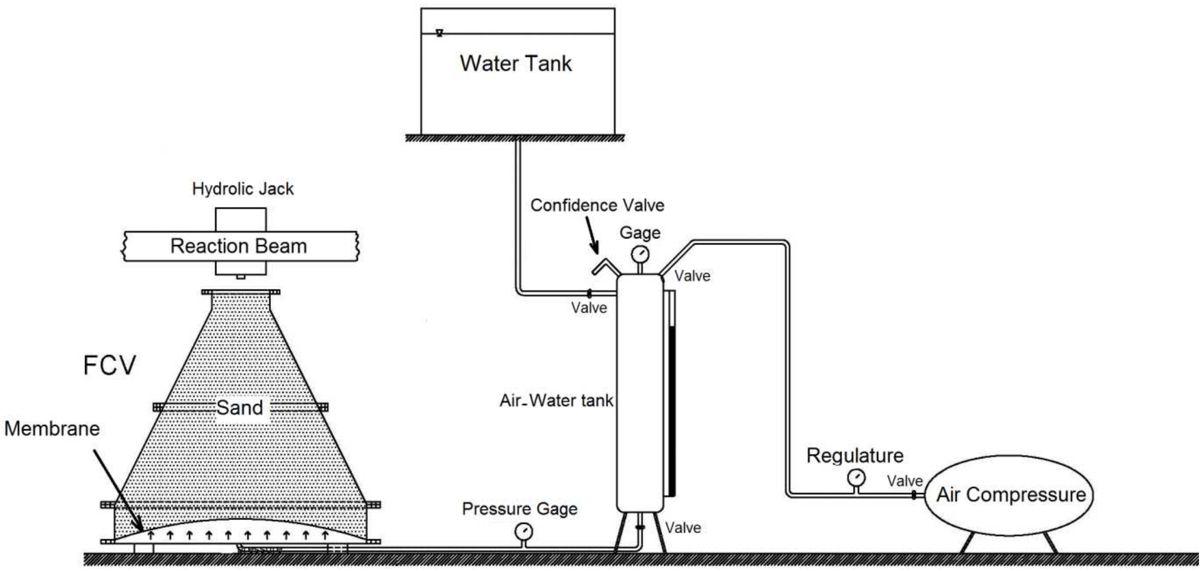


Fig. 5 Schematic profile of FCV at AUT and detailed bottom pressure system

4. Stress Field in FCV

Two series of tests were performed to determine approximate vertical and horizontal stress distributions with depth along the FCV center line.

For purposes of this study, we obtained uniform sand from Babolsar shores located in the northern areas of Mazandaran province in Iran. Fig. 6 illustrates its grain size distribution curve, showing a D_{50} of about 0.18 mm and a coefficient of uniformity, a Cu of 1.67, categorized according to the Unified soil classification System, as SP. The specific gravity G_s of this sand is 2.63. According to density testing, the maximum dry unit weight (γ_{dmax}) and the minimum dry unit weight (γ_{dmin}) for this sand are 17.8 and 15 kN/m^3, respectively.

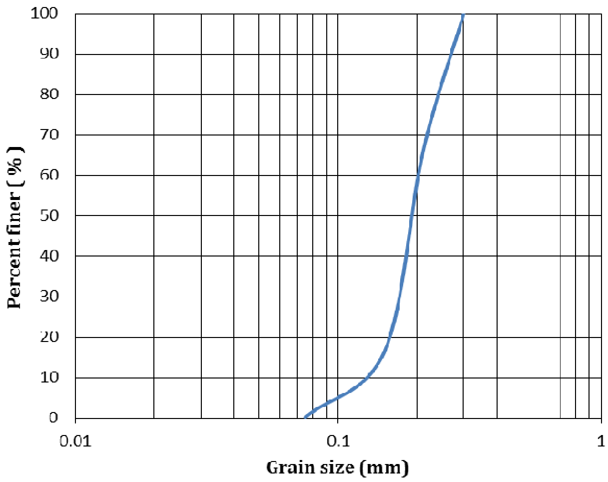


Fig. 6 Grain size distribution of Babolsar sand

Laboratory tests were performed to measure the geotechnical parameters of medium sand that was used in physical modeling tests. Based on results obtained from direct shear tests at stress levels similar to those encountered in the FCV, the internal friction (φ)obtained was 38 degrees. There was no cohesion coefficient (C) to this clean sand.

The following procedure was adopted for each series of stress field calibration tests. First, sand with 4% water content was prepared and placed in the FCV in loose condition by the air raining method. Sand was then placed in each of the 50 mm depth layers and leveled by a wooden pallet. All the models produced by this method showed 30% to 35% relative density, all of which were kinds of loose states.

After that, Soil Pressure Meters (SPM) were placed in specific depths of the FCV center line. The SPMs were used to measure the total soil pressure at different locations of the models, and to estimate the total stress from the applied pressure to the bottom of FCV. They are a load cell type with a 1000 kPa capacity, 70 mm diameter and 15 mm thickness, made in AUT. As shown in Fig. 7a, SPMs installed horizontally at different depths of the FCV are used for measuring the vertical stress distribution in the core area of FCV. Also, as indicated in Fig. 7b, SPMs installed vertically are used for monitoring the horizontal stress distribution in FCV. The positions of both horizontal and vertical SPMs, in the same depth are illustrated in Fig. 8, schematically.

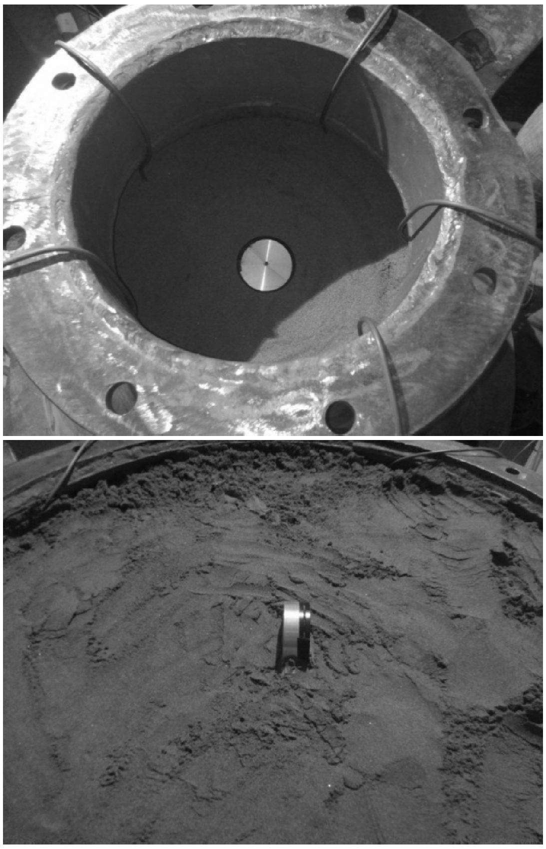
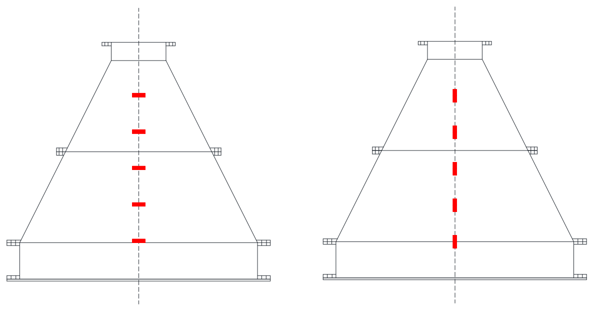


Fig. 7 Soil Pressure Meters installation in FCV, a) horizontally, b) Vertically

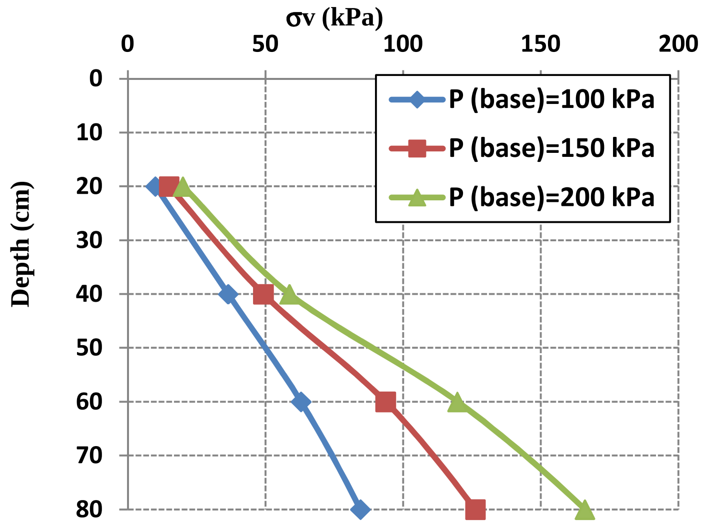


(a) (b)

Fig. 8 Position of soil pressure meters, a) Vertical stress, b) Horizontal stress, distribution tests

After the installation of SPMs and filling sand in FCV, bottom pressure is applied in three stages, 100, 150 and 200 kPa. Bottom pressure is maintained for several minutes in each level until the soil reaches equilibrium.

As indicated previously, the primary objective of the stress field tests was to determine the variation of lateral and vertical stresses with depth that occur along the centerline of FCV. Afterwards, these measurements were used for the calculation of K coefficient as defined, K= $\sigma_{lateral}$/ $\sigma_{vertical}$. Distribution in depth for vertical and lateral stresses and K-coefficient are plotted in Fig. 9. Test results indicate that the linear gradient of both vertical and lateral stresses with depth occur in FCV. Also the k-coefficient is about 0.35, which is close to active lateral stress coefficient (k_a).



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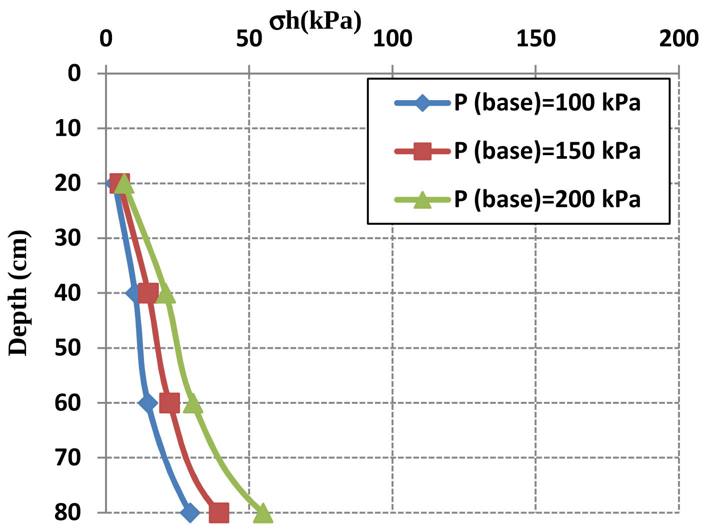

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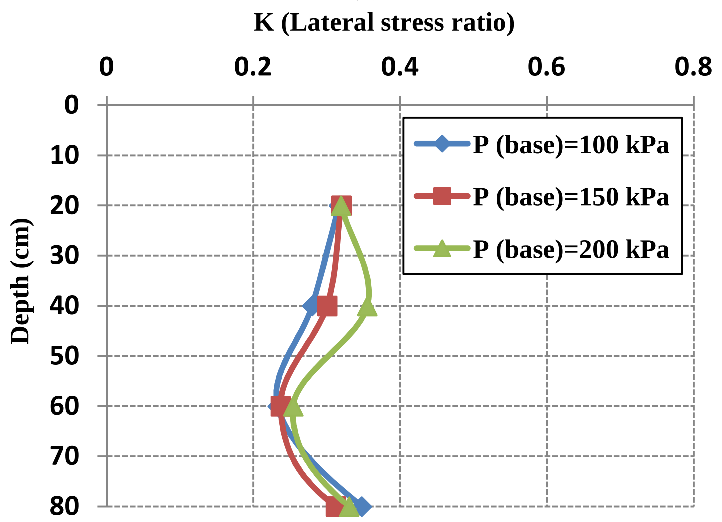

(c)

Fig. 9. Stress distribution in FCV, a) Vertical, b) lateral, c) lateral stress ratio (k)

5. Model Pile Tests in FCV

Axial compression and tension tests were conducted to evaluate the performance of the FCV for model piles. Steel tube with different diameters were selected for the modeling of open end driven piles. Details of model piles are given in Table 2. Model driven steel piles have been driven in sand to a depth of 750 mm. Pile embedment depth to diameter ratio of model driven piles (L/D) were 8-15, which model short to medium length piles in prototype. Models were driven into soil by a hammer. Fig. 10 presents model piles that were guided by a short casing to ensure that the piles were centered and driven vertically. After driving the pile, pressure up to 150 kPa is applied to the bottom of FCV. According to Fig. 9a, the bottom pressure equals 150 kPa, while the vertical stress in the depth of pile toe (z=750 mm) is 120 kPa. This stress simulates the actual in-depth stress of 7.5 m for this sand with a density of 16 kN/m^3. In other words, the scale of model driven piles is 1/10.

Table 2 Specifications of model steel driven piles

Model	Outer diameter (mm)	Inner diameter (mm)	Pile embedment (mm)
SDOP1	51	45	750
SDOP2	76	65.5	750
SDOP3	89	80	750

Fig.10. Details of steel model piles driven in FCV with Casing guide

Compressive and tensile loading tests were performed separately on each pile. The vertical load was applied in a stepwise manner by a hydraulic system. A load cell and linear vertical displacement transducers (LVDT) with a high precision of 0.001 mm were mounted on the loading piston to measure the total vertical imposed load and corresponding displacement of pile head (Fig. 12).

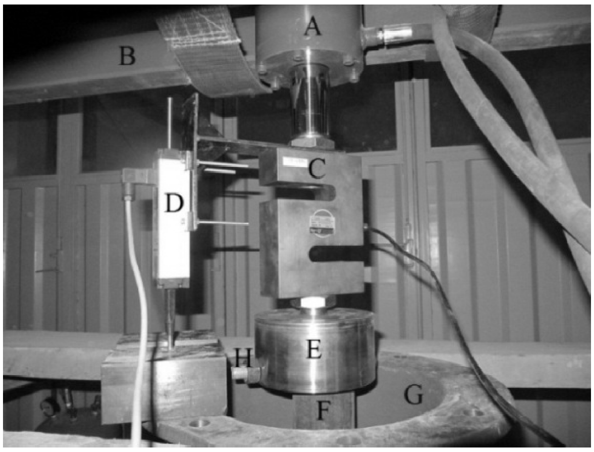

Fig. 12 Loading system apparatus: A) Hydraulic Jack, B) Reaction beam, C) Load Cell, D) LVDT, E) Loading cap, F) Model Pile, and G) FCV

The results of axial compression loading test for the driven steel open-ended model piles are presented in Fig. 13. 150 kPa pressure was applied to the bottom of the FCV in all the tests. As indicated in Fig. 13, all piles reached ultimate capacity in the displacement, up to 15% diameter of pile. Fig. 14 presents the results of the tension loading tests conducted on the model driven pile. As indicated in this figure, the ultimate tension capacity of driven piles occurred in displacement up to 3 mm in pile head.

Modeling of drilled shaft has been performed by a pre-casting concrete pile, which was pre-drilled and installed in FCV. The embedded pile length and diameter is 750 mm and 84 mm, respectively. As illustrated in Fig. 11, a hole is excavated vertically with a diameter larger than the concrete pile. Then, the pile is placed in the hole and positioned vertically at the center of FCV. Afterwards, the void between the pile and hole are filled with sand. In the end, bottom pressure of FCV is applied. For the concrete pile, bottom pressure of FCV was 200 kPa. According to Fig. 9a for bottom pressure equal to 200 kPa, the vertical stress in the depth of pile toe (z=750 mm) is 160 kPa. This stress simulates the actual stress in depth of 10 m for this sand with density 16 kN/m^3. In other words, the scale of model driven piles is 1/13.3.

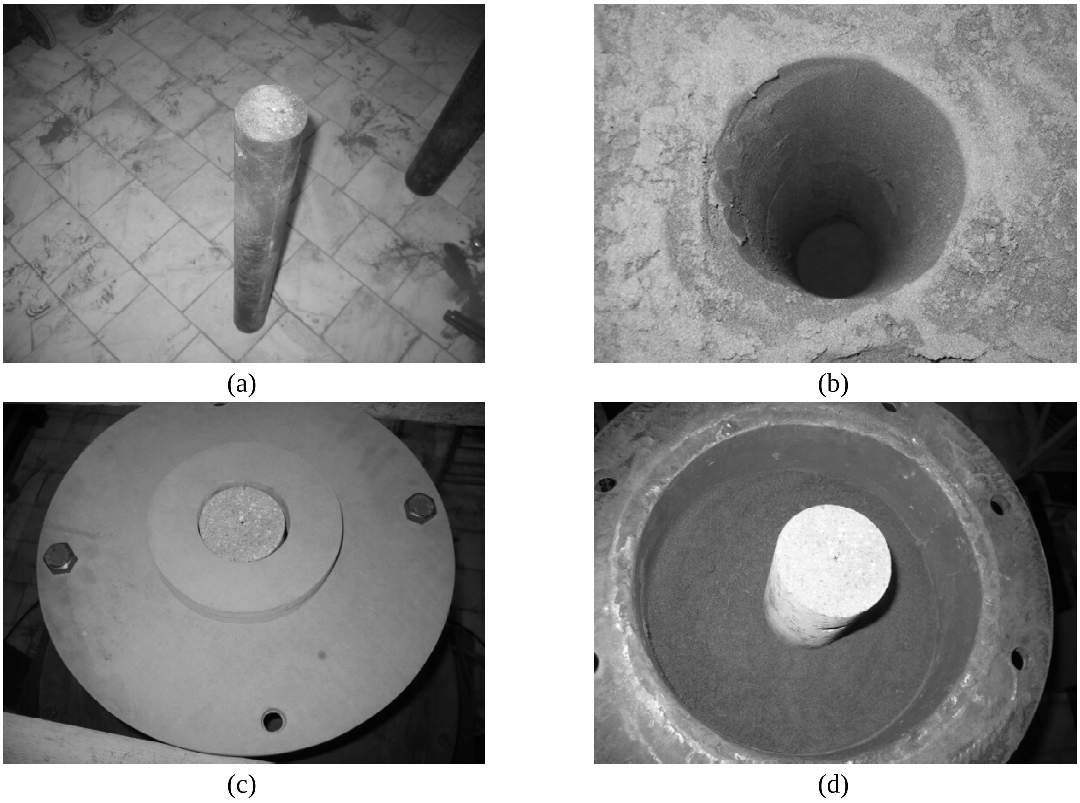

Fig. 11 Sequence of testing on concrete Drilled shaft in FCV: a) pre-casting of model pile, b) Drilling of hole, c) installing of model pile, d) applying bottom pressure of FCV

6. Discussion and Verification

The designer of deep foundation must possess a variety of skills, experiences and considerable knowledge of geotechnical engineering. No single set of simple rules and procedure can cover the variety of conditions that affect deep foundation behavior. The main three aspects that must be considered are structural strength, settlement and bearing capacity. The pile materials employed in this research have had the ability to safely resist all loads imposed during installation and loading test conditions. The pile section and material, regardless of it being steel or concrete were chosen without concern for structural failure. Instead, the significant research effort was focused specifically on the geotechnical performance, i.e. settlement and bearing capacity.

Generally, the actual load-movement response of a single pile is a function of the relative contribution of shaft and toe resistances, the soil condition and the method of pile installation. Since, the aspect of capacity and settlement constantly interact [Fellenius, 1988], hence, in this study they are not separated. Accordingly, the bearing capacity of model piles is realized as the load that fails the soil around the pile. Usually, soil failure occurs as plunging shear failure under the pile toe, accompanied as preceded by direct shear failure along the shaft.

As illustrated in Figs. 13 and 14, the direct axial loading tests whether in tension or compression, experimentally verified the actual pile capacity, as reflected in load-displacement relationships. For tested piles the failure load by plunging is reached when rapid movement occurred under sustained or following slight increase of the load. All of the tested model piles have been failed with a 15% diameter pile displacement. For verification of measured capacity, the Effective Stress Analysis, ESA, has been realized, which governs long-term pile capacity. Accordingly, the bearing capacity or termed ultimate resistance or ultimate load of a pile consists of fully developed shaft resistance (shear resistance generated along the shaft) and toe resistance (generated at the base of the pile) in response to axial load, as follows:

$$Q_u = R_u = R_t + R_s = r_t.A_t + r_s.A_s \qquad (1)$$

Where:
Q_u= bearing capacity of pile;
R_t= toe resistance;
R_s= shaft resistance;
r_t= unit toe resistance;
r_s= unit shaft resistance;
A_t= pile toe cross section;
A_s=pile circumferential area

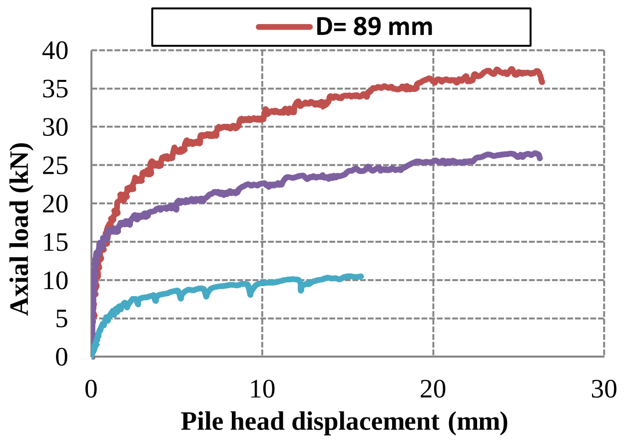

Fig. 13 Load-Displacement for compression load test of model steel driven piles

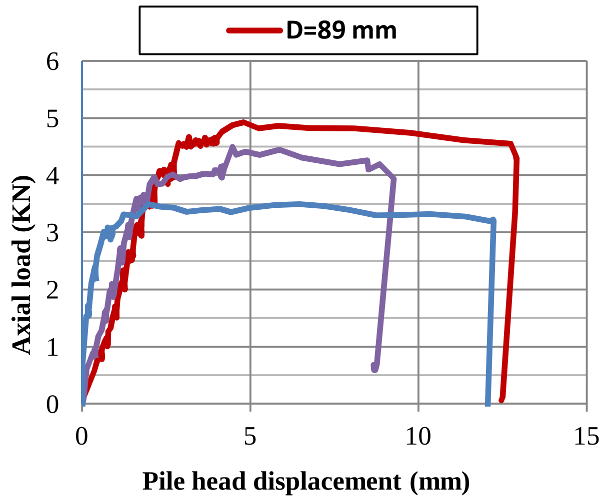

Fig. 14 Load-Displacement for tension load test of model steel driven piles

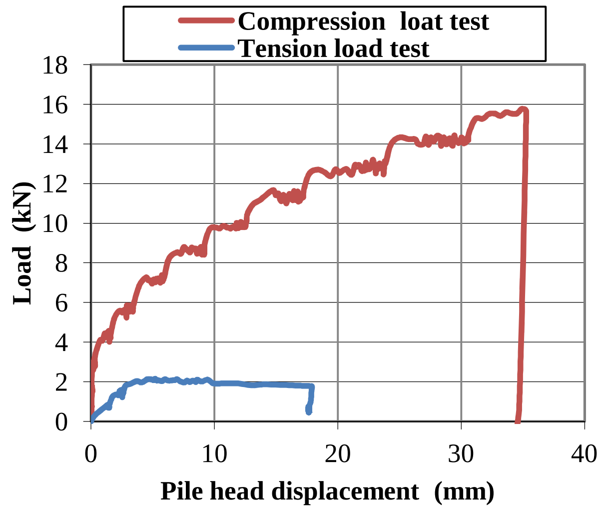

Fig. 15 Load-Displacement for Compression and Tension load test of Concrete model piles

Many theoretical solutions are available for determining the r_t and r_s, which mainly depend on N_q (bearing capacity factor) and K_c (coefficient of construction lateral soil pressure). To avoid doubts about the applicability of the conventional bearing capacity theory, the unified approach as recommended by CFEM, 2006 [17] and Fang, 2001 [18] is realized for static analysis using N_t and β coefficients.

$$r_t = N_t \sigma'_{z=D} \tag{2}$$

$$r_s = \beta \sigma'_v \tag{3}$$

According to the soil type and desired density, the values of β and N_t in eq. 2 and 3, for the driven pile have been chosen, 0.4 and 40, respectively. In Table 3 the calculated axial capacity ($R_{u,calc}$) has been compared to the measured capacity of model test ($R_{u,test}$) for the driven pile.

The comparison demonstrates good agreement between measured and predicted capacities obtained by Effective Stress Analysis (ESA). Results confirm the proper response of model testing, and the consistency of stress gradient in the field and FCV, for usual piles.

Table 3 Analytical and testing results of axial capacity for driven model piles in FCV

Model	r_t (kPa)	r_s (kPa)	R_t (kN)	R_s (kN)	$R_{U,Calc}$ (kN)	$R_{U,test}$ (kN)
SDOP1	4800	48	9.7	2.9	12.7	10
SDOP2	4800	48	21.7	4.4	26.1	23
SDOP3	4800	48	29.9	5.1	34.9	34

7. Conclusions

Among the advantages of FCV in the simulation of an actual stress field, one can point to the conformability and cost efficiency in compared to 1g, Ng and CC apparatus. FCV-AUT has been setup with 1300 mm in height, and 1300 mm in base diameter. Two series of tests with an arrangement of soil pressure sensors along the centerline of FCV have been performed to determine the stress field in FCV-AUT. Also, a set of compression and tension loading tests were conducted, which included six tests on an open-ended steel tube that models driven piles, and two precast concrete piles that model drilled shaft. Experimental and analytical studies denote the following results:

1- Owing to its simplicity, the FCV device presents a practical and economical alternative to centrifuge devices. In addition, most limitations associated with simple 1g and CC devices can be eliminated when model piles are tested in the FCV device.

2- Results of the stress field tests illustrate that FCV could simulate the stress gradient in real cases where full scale piles are performed. Also, comparison between the pile modeling test results in FCV and static analysis prediction has shown good compatibility and agreement.

3-Performing tests on piles with embedment lengths of about 750 to 1000 mm, diameters of 50 to 100 mm and bottom pressures of 150 to 300 kPa in FCV can be representative of prototype piles with 10 to 30 m length, and diameters of 0.5 to 2 m, all of which are practical and common pile dimensions.

4- Effective Stress Analysis (ESA), and the choosing of β and N_t specifically is based on a unified approach that shows good consistency between the predicted and measured bearing capacity for model piles in FCV.

This research presents the preliminarily attempts to use FCV-AUT for the modeling of deep foundations. According to the proper function of the device, the FCV-AUT can be employed for the study of construction effects of different piles, including confinement aspects, boundary conditions, post grouting improvements, and types of installation.

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