Experimental Investigation of Stress Distribution of Vertically Loaded Short Displacement Pile in Cohesion-less Soil

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Abstract

One can find lot of the methods and theories to determine and interpret the nature of the bearing capacity of displacement piles in cohesion-less soil. The presence of many methods leads to the variety of results and yields that no general and relevant methods have been proposed yet. Many experimental investigations do not fit properly the results of known numerical simulations. These simulations yield that the shear stress near the end of the pile increases significantly, when compared with the rest of pile length. Current investigation presents the specific test results of the short displacement pile under vertical load in cohesion-less soil. The full scale test was performed and the vertical stresses beneath base and shear stresses along the pile have been measured, as a result the shear stress significant increase near the end of the pile was observed. The obtained results can be used for developing the shaft and tip bearing capacity evaluation method for the short displacement piles in cohesion-less soil.

Keywords: displacement pile; shear stress; cohesion-less soil.

Nomenclature

\begin{tabular}{ll}
\(C_p\) & pile perimeter (m) \\
\(F\) & the vertical load applied on top of the pile (kN) \\
\(F_b\) & portion of the load \(F\) transmitted to the pile tip (kN) \\
\(F_s\) & the portion of the load \(F\) transmitted to the soil by the pile shaft (skin) (kN) \\
\(F_u\) & total bearing capacity (kN) \\
\(F_{b,u}\) & tip bearing capacity (kN) \\
\(F_{s,u}\) & shaft capacity (kN) \\
\(f_{s,i}\) & shaft resistance from cone penetration test (kPa) \\
\(l_i\) & length of single layer (m) \\
\(K\) & lateral earth pressure coefficient \\
\(q_{cs,i}\) & single layer cone resistance obtained by cone penetration test (kPa) \\
\(\alpha_s\) & correlation coefficient between \(\tau_{s,i}\) and \(q_{cs,i}\) \\
\(\beta\) & factor depending on the density of sand \\
\(\delta_f\) & interface friction angle
\end{tabular}

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Selection and peer-review under responsibility of the Vilnius Gediminas Technical University

doi:10.1016/j.proeng.2013.04.095
| Symbol | Description |
|--------|-------------|
| $\Delta \sigma_{rd}$ | increment of the radial effective stress occurring during loading process because of the dilation effect in dense soil (kPa) |
| $\sigma_{rc}$ | radial effective stress measured after an installation but before loading (kPa) |
| $\sigma_{r0}$ | vertical effective stress in situ (kPa) |
| $\tau_{s,i}$ | ultimate skin friction along the length $l_i$ (kPa) |
| $\omega$ | correlation coefficient between $\tau_{s,i}$ and $f_{s,i}$ |

1. Introduction

The displacement pile is the oldest type of deep foundation and due its proven efficiency is acknowledged and often employed in geotechnical engineering practice. But one must note that despite the wide usage and long period of experimental, analytical and numerical investigations, the interaction between the soil and the pile was not properly determined, so far. Although one can find a lot of theories, methods and techniques for modeling the skin capacity of displacement pile in cohesion less soil, the obtained results do not fit the tests results properly. When summarizing the findings after the detailed review of references in this field one can conclude about the presence of the essentially different concepts regarding the shaft capacity interpretation. Many experimental results are in conflict with the results of numerical simulations. The principle disagreement is in an interpretation of the distribution of shear stresses along the pile length (perimeter). The numerical analysis of the shaft distribution yields that the shear stress near the end of the pile increases significantly. One can note that proper specified experiments to fix this phenomenon have been not performed. An experiment performed in a descriptive investigation aims to confirm, with relevant accuracy, the analysed numerical simulation results on stress distribution of investigated piles of other researches. Having performed the experiment a significant increment of the shear stress in the last thirty centimetres (approximately pile diameter) from the pile base was recognized. The effect of principle increment of the shear stress unlike of many theoretical statements fits quite good the above mentioned results of numerical simulations. Therefore in the future investigations the assumption, that the displacement pile base (tip) and the shaft capacities are highly correlated, made according current investigation, should be checked.

2. Theoretical background

The vertical load applied on top of the pile is transmitted to the soil stratum by the pile tip and the pile shaft, namely:

$$ F = F_b + F_s $$  \quad (1)

According some research studies the vertical load applied on top of the pile is transmitted to the soil instantly e.g. [1]. However, the other author contends that the load is transmitted progressively, namely: at first through the shaft and only after the shaft is “employed”. id est. then the loading is transmitted via the shaft and through the tip [2-7].

The term ultimate load or bearing capacity of a single pile indicates either the magnitude of an external load for which the settlement of the pile increases continuously with no further increase in load, or at which the settlement begins to increase at a rate far out of proportion to the rate of increase of the load [3]. The shapes of the failure surfaces assumed (proposed) by the different investigators are shown in Fig. 1.

![Failed patterns under deep foundations](image)

Fig. 1. Assumed failure patterns under deep foundations [4]: (a) after Prandtl, Reisson, Caquot, Buisman, Terzaghi; (b) After DeBeer, Jaky, Meyerhof; (c) after Berezantsvev and Yaroshenko, Vesic; (d) after Bishop, Hill and Mott, Skemption, Yassin and Gibson

Frequently in the geotechnical practice it is not easy to determine the ultimate load considering the pile load test graph. Consequently, according to many investigations [1-3], [5], the relative settlement of magnitude 10% of pile diameter can be
accepted, as a relative ultimate load limit, id est. bearing capacity of pile is employed the criterion of ultimate load. Then the bearing capacity is described by the following formula:

\[ F_u = F_{p,u} + F_{s,u} \]  

(2)

According the load portions (bearing capacity values assumed to be transmitted) by the shaft and the tip, the deep foundations are classified to the friction piles and the point the bearing piles, respectively [2-3], [6-7], see Fig. 2.

The skin friction pile carries the main load portion via the pile shaft, that of the point bearing piles via the pile base. The main part of the territory of Lithuania is covered by the glacial origin typically the bearing stratum being located relatively close to the ground surface. Therefore the short piles are the most common type of deep foundations in Lithuania, id est. being in under demand of engineering practice.

The pile bearing capacity can be determined in situ either by the static (most common case) or by the dynamic load tests. Alternatively it can be estimated by processing the field or laboratory test data also. Generally, all pile bearing capacity evaluation methods one can summarize to the three groups, namely: theoretical, semi – empirical and empirical. The evaluation methods via base dominating resistance are most widely described ones and can be easily found in many references. Therefore the aim of the current investigation is the shaft evaluation methods and techniques. Generally, the shaft resistance can be expressed by:

\[ F_{s,u} = C_p \sum_{i=1}^{n} \tau_{s,i} l_i \]  

(3)

According [3] the ultimate skin friction can by expressed via the formula:

\[ \tau_{f,i} = K \sigma_{v0} \tan \delta_f \]  

(4)

Another theoretical approach [8] proposes:

\[ \tau_{f,i} = \beta \sigma_{v0} \]  

(5)

The semi–empirical method developed by in Imperial College of London [9] proposes:

\[ \tau_{f,i} = \sigma_{rf} \tan \delta_f \]

\[ \sigma_{rf} = \sigma_{rc} + \Delta \sigma_{rd} \]  

(6)
The developed pure empirical methods for the evaluating of the ultimate skin friction is based on an assumed correlation factor between the shaft resistance (from the cone penetration test) and the shaft bearing capacity \([1], [10]\). The Lithuanian approach (being already applied for the long period) reads:

\[
\tau_{f,i} = \omega f_{s,i}
\]  

(7)

The other empirical method is based on assumed relation between the cone resistance and the shaft resistance values, obtained from the cone penetration test \([11]\), namely:

\[
\tau_{f,i} = \alpha_s q_{cs,i}
\]  

(8)

The presence of the many methods, including the listed above, leading to the wide range of the shaft friction magnitudes, yields that no general and relevant method has been proposed so far. Therefore the new numerical and experimental investigations have been performed by the researches to study the behaviour of single piles subjected to compression (vertical loading).

Following Kempfert \([12]\) the shaft friction distribution along the pile length is parabolic, with the maximum being reached at the middle of the pile. From the middle to the pile end, shaft friction decreases sequentially. Another research performed by Vesic \([13]\) for the displacement pile subjected by vertical load, reports the results similar to Kempfert statements. The tests of bored piles in silty soil made by \([14]\) were in line with previous research findings. The contrary statement for the skin friction distribution was proposed for the cast-in-situ piles, following the investigations of the \([15], [16]\), it was concluded that the shaft friction increases at the 5 last meters before the pile end. The other investigators introduced a similar study for open-ended piles under cyclic vertical load \([17]\) and the study concluded that the radial stress increases near the pile end.

The numerical study on the pile and multi-layered soil interaction showed smaller shaft shear stress values in the upper part of the pile, and the greater values in the lower part of the pile \([18]\). Another numerical study yielded that the radial stress increased near the pile end \([19]\). In theoretical view the increment of the radial stress increase the friction forces between soil stratum and pile surfaces, id est. the shaft resistance. Note that the above mentioned findings correspond for high level load magnitudes.

3. Experimental section

3.1. Experimental set-up

The test of the model pile was performed at Geotechnical Research Laboratory of Civil Engineering Research Centre of Vilnius Gediminas Technical University. The trench dimensions where the test was performed are: 6.0×6.0×7.0 m. The length of the model pile is 2.25 m, the diameter 0.324 m. The system consisting of four vibrating wire load cells and the Micro-1000 Datalogger (Model 8021) for the measuring of the forces were employed. A principal scheme of the tested model pile is presented in Fig. 3. The hydraulic jack system of 1200 kN capacity was used for inserting the model pile in to the cohesion-less soil is presented in Fig. 4.

![Fig.3. Principle scheme of the model pile](image-url)
3.2. Soil description

The soil of the trench is even graded air-dry sand. The particle size distribution (grading curve) of considered soil is presented in Fig.5. The static penetration test (see Fig. 6) reported the following results: within one and a half meter from a surface is the loose sand (cone resistance varies within bounds of 0.5 and 5.0 MPa; that from 1.5 m to 1.85 m is the middle density sand (cone resistance varies within bounds of 5 and 15 MPa; that of from 1.85 m to 2.3 m is the dense sand of average cone resistance magnitude of 17 MPa; that of from 2.3 m to 4.5 m is the middle density sand of average cone resistance magnitude of 11 MPa.

3.3. Description of the test

The certain numbers of preparatory tests were made to calibrate and verify the reliability of the measurement systems and the model pile construction as well as to improve the loading framework. The conditions and procedures of the main test are described below.

At first stage of the test the model pile was inserted via compressing it to the 0.85 m depth by the hydraulic jacks. Then the pile was unloaded following the conditional force stabilization (id est. the force change during 5 min period is smaller than 1 kN) and the data have been read and stored. After the hydraulic jacks were disconnected from the driving beam system, they were moved by 1.0 m down, and then connected to the driving beam system again. The test was continued (second test stage) the next day. The pile was inserted to the 1.15 m depth of the soil during the second stage of the test. The data have been read and stored again after the conditional stabilization of the load force. The analogous sequence of the test stages was realized for inserting the pile into the soil depths of 1.45 m and 1.7 m, respectively. The test stages are illustrated in the Fig. 6.
4. Results and Discussions

The pile is inserted in soil incrementally, i.e., by stages to reach the certain depth levels. At this stages (they can be treated as “conditional” ultimate states with corresponding ultimate load magnitude) the stress fields at shaft and tip are analysed.

When the pile was inserted into a soil by 0.85 m depth the largest vertical stress got concentrated beneath the centre of the pile base. The maximum stress magnitude of 885 kPa was in the centre, while at the edges of the pile base it reduced up to 484 kPa. The shear stress acting along the side surface of the pile got concentrated nearby the pile tip (in the last 30 cm) reaching the magnitude of 114 kPa, while in the rest surface of pile the stress magnitude was approximately only 2 kPa, see Fig 7a.

When the pile was inserted into a 1.15 m depth, the vertical stress under centre of the pile base has increased and reached 1035 kPa, while on the edges of the pile base it remained barely unchanged of the magnitude 482 kPa. The shear stress nearby the pile tip increased up to 235 kPa, and in the rest of the side surface it remained unchanged (see Fig 7b).

When the pile was inserted into the 1.45 m depth, the vertical stress beneath the centre of the pile base has increased again reaching the magnitude of 1834 kPa, the stress under the edges increasing also and reaching the magnitude of 888 kPa. The shear stress beside the pile tip has decreased fractionally up to 230 kPa, while in the rest side surface of the pile the stress has increased barely and reached the value of 8 kPa, see Fig 7c.

After the inserting the pile into the 1.70 m depth, the vertical stress under the centre of the pile base and on the sides has increased again and the magnitudes of 3254 kPa and 1876 kPa, respectively. The shear stress beside the pile tip has decreased essentially up to 106 kPa, on the rest of the side surface of the pile the stress increased again reaching the magnitude of 10 kPa, see Fig 7d.
4.1. Interpretation of results and comparison with codified design techniques

The processing of the test results yielded the correlation between the average stress beneath pile tip $\sigma_b$ and the cone resistance $q_{c,ave}$ at considered depths, see Table 1.

Table 1. Correlation ratio $\alpha_p$, for considered depths

| $h$  | $q_{c,ave}$ | $\alpha_p = (\sigma_p/q_{c,ave})$ | $\alpha_p$, EN1997-2 |
|------|-------------|----------------------------------|-----------------------|
| 0.85 | 1250        | 0.48                             | 1.0                   |
| 1.15 | 3625        | 0.18                             | 1.0                   |
| 1.45 | 3950        | 0.30                             | 1.0                   |
| 1.7  | 8475        | 0.27                             | 1.0                   |

The method proposed by [20] has been applied for the averaging of the cone resistance values. It can be found from the graph (see Fig. 8) that the relationship between the normal stress under the pile base and the cone resistance is linear with sufficient accuracy. Note that the correlation ratio for the $\alpha_p$ magnitudes (obtained by processing the test results) varies within the bounds of 0.27 and 0.48 and is quite different from the magnitudes of 1.0 which is specified in EN 1997-2 [11].

![Fig. 8. Graphic representation of correlation between the average stress beneath pile tip $\sigma_b$, and the cone resistance $q_{c,ave}$](image)

The results are reported, having determined the correlation between the average shear stress near the pile tip $\sigma_{s1}$, in the rest of pile surface $\sigma_{s2}$, the averaged shear stress values $\sigma_s$ and the averaged cone resistance $q_{cs,i}$ in an appropriate considered depths. The determined correlation magnitudes are summarized in the Table 2. One can find that the shear stress and cone resistance correlation ratio differs in principle (from 0.4 to 22.8 times.) from the magnitude specified in EN 1997-2 [11].

Table 2. Magnitudes of correlation ratios $\alpha_{s1}$, $\alpha_{s2}$, $\alpha_s$ at considered depths

| $h$  | $\alpha_{s1} = (\sigma_{s1}/q_{cs})$ | $\alpha_{s2} = (\sigma_{s2}/q_{cs})$ | $\alpha_s = (\sigma_s/q_{cs})$ |
|------|----------------------------------|----------------------------------|-----------------|
| 0.85 | 0.228                             | 0.004                             | 0.076           |
| 1.15 | 0.118                             | 0.004                             | 0.066           |
| 1.45 | 0.028                             | 0.009                             | 0.021           |
| 1.7  | 0.016                             | 0.004                             | 0.009           |

5. Conclusions

- Test of short displacement pile under vertical load in cohesion less soil yielded that the shear stress along the pile extremely increases near the pile tip (in the last thirty centimetres, id est. approximately per one diameter height).
- This shear stress increase effect unlike of many theoretical statements fits well the known numerical simulation results presented in above described references.
- It can be stated, that the displacement pile base and shaft capacities are correlated. It can be explained by the reason that the effect shear stress increase occurs due to changes in state of stress in the soil adjacent to the pile tip during installation and loading process.
Acknowledgement

An equipment and infrastructure of Civil Engineering Scientific Research Centre of Vilnius Gediminas Technical University was employed for investigations.

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