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EXPERIMENTAL INVESTIGATION OF STRESSES IN SAND DURING THE INSTALLATION AND LOADING OF THE SHORT DISPLACEMENT PILE

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Abstract. Nowadays it is possible to find many experimental and analytical studies aimed for better prediction of sand soil response during the installation and loading stages of displacement pile. The interaction between the soil and the pile is very complex, this is the reason why it is not exhaustively described, so far. Response of the soil, especially the ultimate state necessitates investigate the nature of soil response via tip and shaft as well as their relation. Qualitative evaluation of the stress state influence on pile behaviour serves for more clear description of the soil ultimate response mechanism. Current investigation presents the results of two specific instrumented piles tests. The 1st type of the tests revealed the shear and normal stresses distribution at particular areas of short displacement pile interface during static vertical load test. The 2nd type of the tests showed the radial stresses increment paths in the soil during the pile installation stage. The performed tests of the short displacement piles results cleared, that during the static load tests the highest shear stresses, on the pile skin, get concentrate near the pile tip and during the installation stage the radial stresses significant increase when pile tip gets near the push in load cells measurement plane.

Keywords: displacement pile, shaft friction, shear stresses, normal stresses, cohesion less soil, bearing capacity.

1. Introduction

The displacement pile is the oldest type of deep foundation and due to its proven efficiency is acknowledged and often employed in geotechnical engineering practice. But one must emphasize that despite the wide and long-term usage in engineering practice this kind of piles, still there are a lot of experimental, analytical and numerical investigations (Ai, Yue 2009; Igoe *et al.* 2011; Krasiński 2014; Said *et al.* 2009; Shelke, Patra 2011; Zhang *et al.* 2011; Zhang *et al.* 2013 and others) where the interaction between the soil and pile is represented differently.

The main part of the territory of Lithuania is covered by the glacial origin soils which lie not far from the ground surface. Therefore the short piles are one of the most common type of deep foundations in Lithuania, and this is the reason why current experimental inquiry is aimed to study this type of deep foundation.

Although a lot of theories, methods and techniques were developed to predict and simulate the behaviour of displacement pile in cohesion-less soil (Baziar *et al.* 2012; Berezantzev *et al.* 1961; Lehane 1992; McClelland 1974; Nottingham, Schmertmann 1975), the obtained results do not fit the tests results properly.

The analysis of above mentioned references related to the principal single pile behaviour perception, including validation of experimental versus numerical simulation, was performed. The references inquiry revealed that there are certain inconsistencies between numerical simulation results and theoretical statements. The main discrepancy was found regarding the shear stress distribution along the pile shaft. Furthermore, no relevant tests were performed to serve for fixing and subsequently for explaining this disagreement.

Therefore two types of specific tests were carried out. The 1st type of tests was aimed to reveal the shear and normal stresses distribution at displacement pile interface and the 2nd type of tests had to explain the origin of determined particular stresses distribution.

2. Theoretical background

It is generally accepted that the vertical load applied on top of the pile is transmitted by the pile tip and the pile shaft (Fig. 1):

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$$F = F_S + F_b = \int_0^L \int_0^{\pi D} \tau_{s.i} dz + \int_0^{0.5D} \int_0^{2\pi} \sigma_{b.i} dz, \quad (1)$$

where F – a vertical load applied on top of the pile; F_S – a portion of the load F transmitted to the ground by the pile shaft (skin); F_b – a portion of the load F transmitted to the pile tip; L – the pile length; D – the pile diameter; $\tau_{s.i}$ – a shear stress acting at skin elementary plot; $\sigma_{b.i}$ – a normal stress acting at base (tip) elementary plot.

It is well known that the load for the bearing stratum is transmitted progressively (during the loading process stage when the load magnitude vary from 0 till its final magnitude F), at first through the shaft and only after the tip is "employed", then load is transmitted via the shaft and the tip.

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 (Terzaghi *et al.* 1996). Frequently, in geotechnical practice it is not easy to determine the ultimate load considering graph of the pile load test. Consequently, the relative settlement of 10% of pile diameter is widely accepted, as a criterion which helps to determine the ultimate load. The bearing capacity is described:

$$F_u = F_{s.u} + F_{b.u}, \quad (2)$$

where F_u – the total bearing capacity or ultimate load; $F_{s.u}$ – a shaft of bearing capacity; $F_{b.u}$ – a tip of bearing capacity.

The classical patterns of the pile bearing stratum failure, proposed by the different researches which were summarized by Vesic (1967), are shown in Fig. 2.

Models, which are shown in Fig. 2, are quite conservative and despite the long-term use they do not properly and sufficiently describe the actual pile behaviour. Therefore the failure models improvements and new approaches are always under development. For instance, another approach proposed by Manandhar and Yasufuku (2012) is based on cavity expansion theory and the failure pattern (Fig. 3).

The relevant adoption of the failure pattern is the main framework of the theoretical bearing capacity prediction methods, which is validated experimentally for certain types of soil. For this reason the patterns of pile bearing stratum failure has to be chosen very accurately.

Pile bearing capacity is determined in situ, by the static or dynamic load tests. Alternatively it is estimated according to the field investigation and laboratory test data (indirect method).

Generally, all the pile bearing capacity evaluation methods are classified into three groups: theoretical, semi-empirical and empirical.

According to McClelland (1974) and many other authors the theoretical ultimate skin friction mainly depends on lateral earth pressure coefficient, vertical effective stresses and the surface friction. It is expressed by:

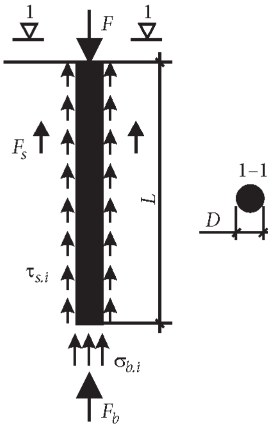

Fig. 1. Transmission of vertical load

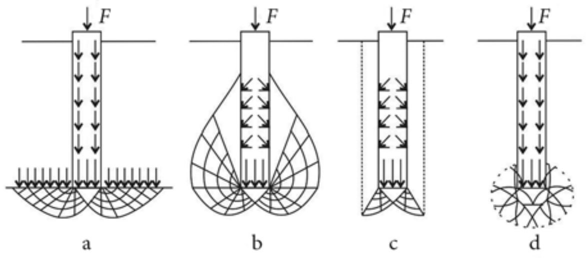

Fig. 2. Assumed failure patterns under deep foundations: a – after Prandtl, Reissner, Caquot, Buisman, Terzaghi; b – after DeBeer, Jaky, Meyerhof; c – after Berezantsev and Yaroshenko, Vesic; d – after Bishop, Hill and Mott, Skemption, Yassin and Gibson

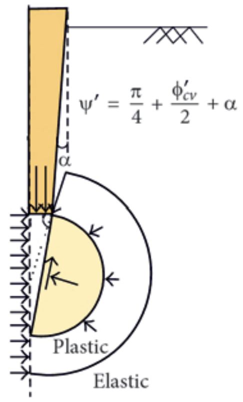

Fig. 3. Pattern of modified failure mechanism around the tapered pile tip in cavity expansion solution

$$\tau_{s.u.i} = K \sigma'_{vo} \mathrm{tg} \delta_s, \quad (3)$$

where $\tau_{s.u.i}$ – an ultimate shear stress; K – a coefficient of lateral earth pressure; σ'_{vo} – vertical effective stress in the soil; δ_s – a coefficient of interface friction.

Up to now, there is no reliable and appropriate lateral earth pressure determination method.

The one of semi-empirical methods was developed in Imperial College of London by Lehane (1992). With this approach, efforts were made to evaluate stress history, but this method is mainly based on cone resistance *qc*. It is well known that cone penetration is indirect method of total soil response evaluation, which actually does not fully describe the stress state of bearing stratum. According to the mentioned method, the ultimate shaft friction is expressing by:

$$\tau_{s.u.i} = \sigma_{rf} \operatorname{tg}\delta_s, \tag{4}$$

$$\sigma_{rf} = \sigma_{rc} + \Delta\sigma_{rd}, \tag{5}$$

where σ_{rf} – the total radial effective stress; σ_{rc} – the radial effective stress measured after pile installation, but before loading; $\Delta\sigma_{rd}$ – an increment of the radial effective stress occurring during loading process because of the dilation effect in dense soil.

Pure empirical ultimate skin friction prediction method is based on correlation between the shaft resistance (from the cone penetration test) and the shaft bearing capacity. This method was proposed by Notingham and Schmertmann (1975):

$$\tau_{s.u.i} = \omega f_{s.i}, \tag{6}$$

where ω – the correlation factor between $\tau_{s.u.i}$ and $f_{s.i}$; $f_{s.i}$ – a shaft resistance determined by cone penetration test.

The other empirical approach is based on relation between the cone resistance and the ultimate shaft resistance, obtained by cone penetration test *(EN 1997-2:2007/AC:2010 Eurocode 7 – Geotechnical Design – Part 2: Ground Investigation and Testing)*:

$$\tau_{s.u.i} = \alpha_s q_{c.s.i}, \tag{7}$$

where α_s – the correlation factor between $\tau_{s.u.i}$ and $q_{c.s.i}$; $q_{c.s.i}$ – a single layer cone resistance determined by cone penetration test.

Berezantzev *et al.* (1961) and other authors agree that theoretical pile tip bearing capacity is expressed by:

$$\sigma_{b.u.i} = N_q \sigma'_{vo}, \tag{8}$$

where $\sigma_{b.u.i}$ – the ultimate normal stress beneath pile tip (base); N_q – a bearing capacity factor which mainly depends on angle of soil inner friction.

Empirical approach which is usually used to predict tip bearing capacity in Lithuania *(EN 1997-2:2007/AC:2010)*:

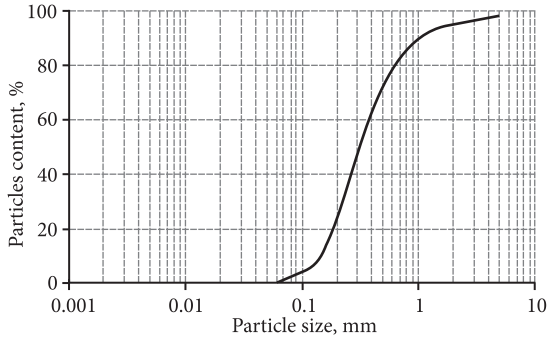

Fig. 4. Soil particle size grading curve

$$\sigma_{b.u.i} = \alpha_b q_{c.b}, \tag{9}$$

where α_b – the correlation factor between $\sigma_{b.u.i}$ and $q_{c.b}$; $q_{c.b}$ – an average cone resistance beneath pile tip determined by cone penetration test, kPa.

The existence of numerous displacement pile bearing capacity prediction methods and techniques (including the listed above) shows that no general and relevant method has been proposed so far. Therefore the new numerical and experimental investigations have been performed by the different researchers to study the behaviour of single piles under vertical load. Following Shelke and Patra (2011) the shaft friction distribution along the pile length is parabolic, the maximum shaft friction occurs at the middle of the pile. The shaft friction decreases sequentially from the middle towards the pile end. The contrary concept of the skin friction distribution was revealed for the cast-in-situ pile, by Zhang *et al.* (2011, 2013). It was concluded that the shaft friction increases at the last 5 m before the pile end. The other researcher introduced a similar study for piles subjected by a cyclic vertical load (Igoe *et al.* 2011). The study concluded that the radial stress increases not far from the pile end.

The numerical study of 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 (Ai, Yue 2009). Another numerical study yielded that the radial stress increase near the pile end (Said *et al.* 2009).

It is relevant to determine the actual ultimate stresses at the pile shaft and under the tip, as well as the stresses acting at adjacent soil in order to fully understand the pile behaviour in sands.

3. Experimental set up and methodology of tests

Two types of specific tests were carried out. The 1st type of tests aimed to reveal the shear and normal stresses distribution at displacement pile interface. The 2nd type of the tests aimed to identify the radial stress patch at the soil. The model piles tests were performed at laboratory pit. The soil volume dimensions are 7.0 m×6.0 m×5.0 m.

3.1. Soil description

The soil is even graded air-dry sand of mineral composition with dominating quartz (Fig. 4).

The static penetration test (Fig. 5) has reported that, up to 2.8 m from the ground surface, sand is loose and cone resistance varies within bounds of 1.0 MPa and 5.0 MPa.

From 2.8 m to 3.2 m lies medium dense sand, and this layer cone resistance is >5.0 MPa, but <10.0 MPa. At deeper stratums lies dense sand of which cone resistance is >10.0 MPa.

3.2. Description of the 1st test

Hydraulic jack system with 1200 kN capacity was used for inserting the model piles into a certain depth. The length of the 1st steel model pile is 2.25 m, and the diameter is

0.324 m. The system consisting of 4 vibrating wire load cells and the *Micro-1000 Datalogger* (Model 8021) for the measuring of the forces were employed. The main idea of the model pile construction is capability to measure the shear and the normal stresses at particular pile surface areas. In general, with the 1st model pile during installation and loading process are measured shear stresses at two certain pile skin and normal stresses at two particular pile base areas. A principal scheme of the tested model pile is presented in Fig. 6. The certain areas of model pile surface are marked in different hatches.

The certain number of preparation tests were made to calibrate and verify the reliability of the measurement system and the model pile construction, as well as for adjusting a loading framework. The conditions and procedures of the main test are described below.

At 1st stage of the test the model pile was pushed in to 1.1 m depth using the hydraulic jacks. Then the pile was unloaded. After 2 days the static vertical load test was performed and detailed short displacement pile response was obtained. The next day the pile was pushed in to 1.4 m depth, and few days later the static vertical load test was performed again. The both static load tests were carried out in pursuance of special code *(ISO/DIS 22477-1:2005 Geotechical Investigation and Testing – Testing of Geotechnical Structures – Part 1: Pile Load Test by Static Axial Compression).*

3.3. Description of the 2nd test

The 2nd type of pile test was performed using steel closed ended tube. The length of the tube is 1.6 m and the diameter is 0.178 m. The main equipment required for the 2nd type of tests is shown in Fig. 8. For the ground radial stresses measurement were used 6 push in load cells, which were located in horizontal plane 0.5 m from the center of the model pile. The test was carried out in 2 stages.

At 1st stage the load cells with numbers 1, 2 and 6 were pushed in to 0.25 m depth and respectively cells with numbers 3, 4 and 5 were pushed into 0.50 m depth. After few hours the model pile during the 1st stage was pushed in to the soil up to 0.8 m depth.

At 2nd stage the 1st, 2nd and 6th load cells were pushed into 1.0 m depth and 3rd, 4th and 5th load cells were pushed into 1.40 m depth. 2 hours later the model pile was pushed in to the soil up to 1.60 m depth.

At both stages the model pile push in was performed continuously (velocity of the cylinders of the hydraulic jacks is 6.25 mm/s), and the increments of the horizontal stresses were measured at every 80 mm.

4. Result analysis

4.1. The 1st test results

Considering the 1st type test load – settlement curve when model pile was at 1.1 m depth (Fig. 9) it is clearly seen that high level plastic deformations have occurred when vertical load reached 135.29 kN. This load value was adopted as bearing capacity or ultimate load. Accepted ultimate

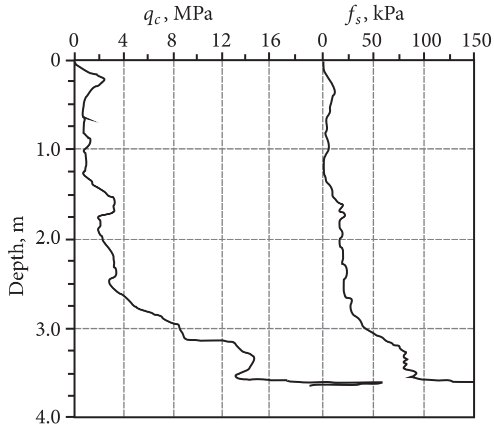

Fig. 5. Results of cone penetration test performed at laboratory pit

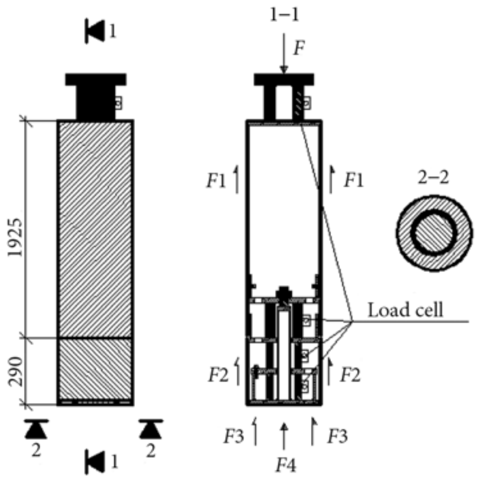

Fig. 6. Principle scheme of the 1st model pile

Fig. 7. Test of the 1st model pile

Fig. 8. Test of the 2nd model pile

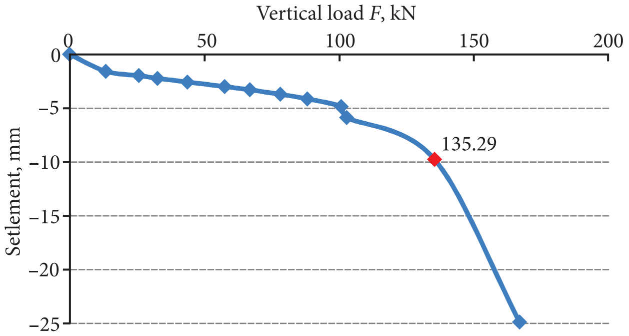

Fig. 9. Load – settlement curve when the 1st model pile was at a depth of 1.1 m

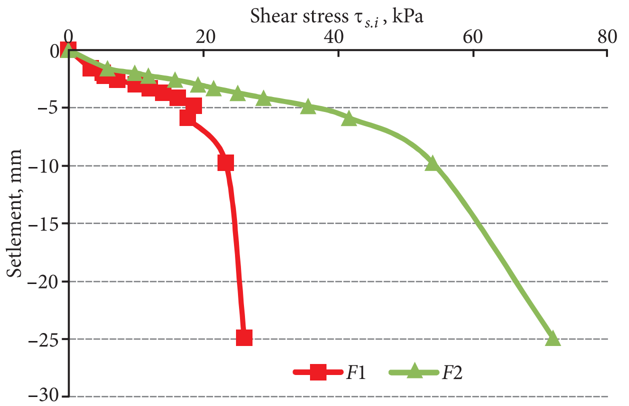

Fig. 10. Distribution of shear stress during static vertical load test when the 1st model pile was at a depth of 1.1 m

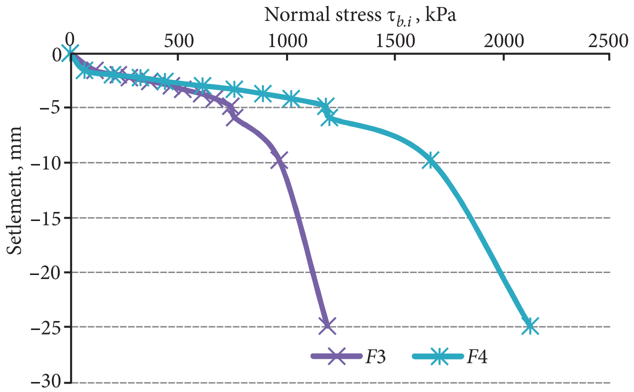

Fig. 11. Distribution of normal stress during static vertical load test when the 1st model pile was at a depth of 1.1 m

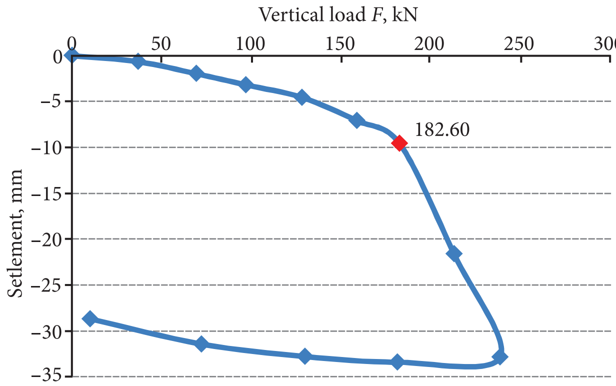

Fig. 12. Load – settlement curve when the 1st model pile was at a depth of 1.4 m

settlement consists of 3.1% of pile diameter and it is obvious that the mentioned value is almost 3 times lower than widely accepted 10% mean.

At *F*1 area ultimate shear stress mobilized when settlement was 10 mm, and at *F*2 area the ultimate shear stress mobilization did not appear at all (Fig. 10). The magnitude of average shear stresses acting at *F*2 area was three times higher than shear stresses acting at area *F*1.

According to Fig. 11 almost twice larger normal stresses have concentrated at *F*4 area. This effect fits with well-known theoretical statements.

When model pile was at a 1.4 m depth (Fig. 12), the ultimate vertical load was determined 182.60 kN. Accepted ultimate settlement also consists of 3.1% of pile diameter and was in line with test performed at a 1.1 m depth.

At *F*1 area the very small magnitudes of shear stresses have appeared during the test (Fig. 13). At *F*2 area the significant higher ultimate shear stress mobilized when settlement has reached 10 mm. Shear stresses has started to grow again at area *F*2 when settlement reached 22.5 mm.

From Fig. 14 it is obvious that the same effect has appeared as in previous test.

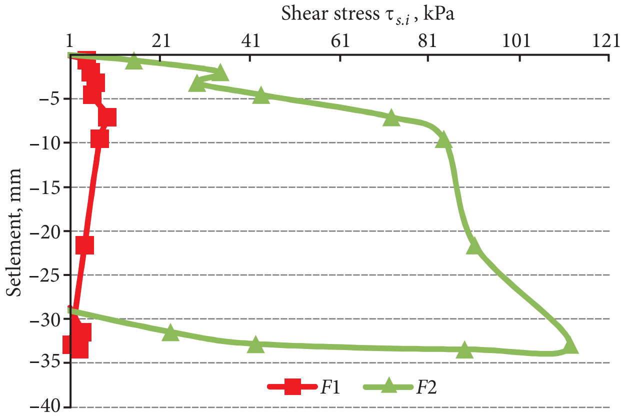

Fig. 13. Distribution of shear stress during static vertical load test when the 1st model pile was at a depth of 1.4 m

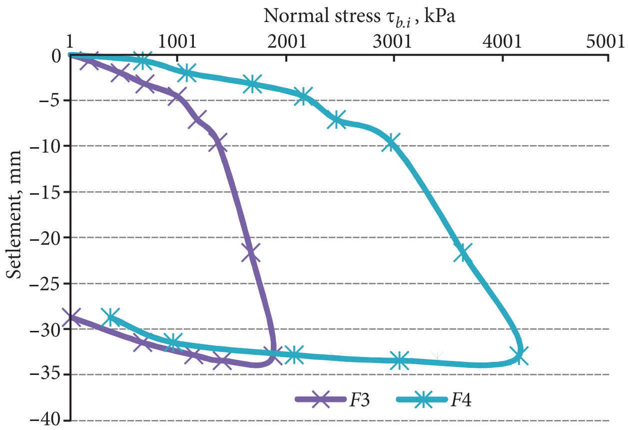

Fig. 14. Distribution of normal stress during static vertical load test when the 1st model pile was at a depth of 1.4 m

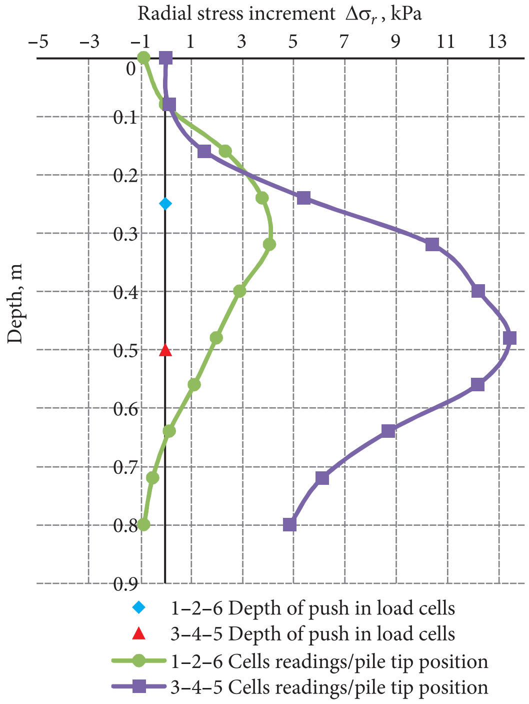

Fig. 15. Results of 2nd model pile test when pile push in was being performed up to 0.8 m depth

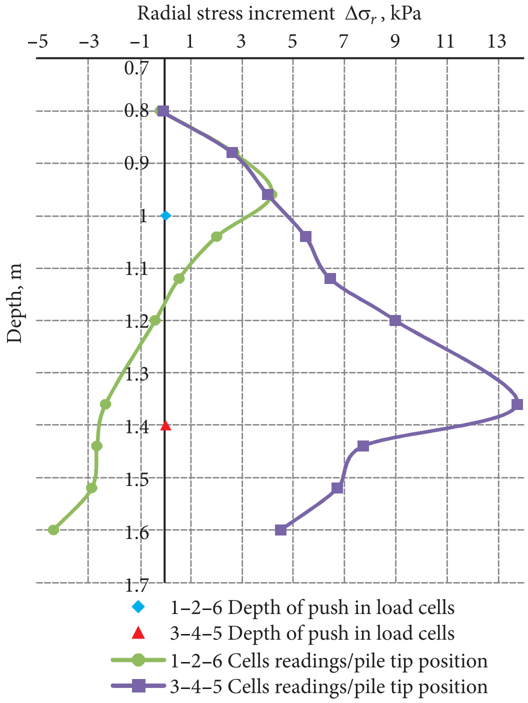

Fig. 16. Results of 2nd model pile test when pile push in was being performed up to 1.6 m depth

4.1. 2nd test results

The radial stress increments patch during the installation of the 2nd model pile is presented in Figs 15–16.

The radial stresses wave is clearly seen which goes to the peak when pile tip gets near the push in load cells measurement plane. These results confirm the statement that the normal stresses beneath the pile base have significant influence on radial stresses increase near the pile tip. This effect appears due to the bearing stratum deformation and failure mechanism.

5. Conclusions, future trends and perspectives

1. The performed static load tests of the short displacement pile determined, that the highest shear stresses on the pile skin get concentrated near the pile tip.

2. Trying to explain the reasons of the effect mentioned in conclusion one, the specific test was performed, which revealed that increment of shear stresses near the pile tip is as a result of increased horizontal (radial) normal stresses, which increases due to bearing stratum deformation and "conditional" failure mechanism. This means that the stress state is one of the governing criteria for describing the failure state of displacement pile.

3. A survey of the main concepts of bearing capacities demonstrates that the empirical approaches are not relevant and/or sufficient, because they are based only on cone static q_c or dynamic q_d penetration test results. These field tests give insufficient information about the initial soil stress state.

4. Analytical and semi-empirical approaches, which are based on failure criteria, involving mechanical properties of the soil in concert with stress state, are used for more accurate prediction of the pile bearing capacity, but merely, if the true values of the failure criteria are determined. Consequently, the determination of these values applying soil field test methods is currently one of the most pressing and difficult geotechnical problems.

5. Research also revealed that the widely accepted 10% of pile diameter conditional settlement, which is widely adopted to determine the ultimate load, does not fit the current investigation. The performed tests yielded that the relative settlement of sand stratum corresponding to the ultimate state (when high level plastic deformations occurred) was 3.1%, even for sufficiently loose sand strata.

6. The obtained results will serve for development of short displacement pile bearing capacity prediction methods, which will take into account the stress state influence.

7. The pile bearing capacity methods applied in geotechnical design should employ not only cone penetration results, but also the push in pressure cell test on purpose to get a better ability on pile bearing capacity prediction.

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