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Lateral stress changes along the pile skin during axial loading in laboratory test

Les essais en laboratoires montrent que la contrainte latérale le long de la surface d'un pieu varie lors d'un chargement axial

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ABSTRACT: The presented paper shows laboratory pile load test in small scale. Skin friction can be calculated based on effective stress in soil σ'_v, coefficient of lateral effective stress K and coefficient of roughness $\tan\delta$ at pile-soil contact surface. The research consider which value of K should be adopted for calculation. Technology of pile affects the state of stress in soil, especially on lateral stress due to cavity expansion theory. The main aim of the research is to measure the lateral stress during pile static load test, and to investigate if we can assume that lateral and vertical stresses are constant. The laboratory test consists of cylinder concrete pile 7 cm in diameter and 25 cm long embedded in cohesionless soil. The additional measurement equipment was used which allows to measure applied load, toe resistance and settlement. Furthermore, on the skin of the pile tactile pressure sensor was installed. The sensor because of its thickness (only 0,1 mm) and flexibility allows to measure of radial stress at the skin of the pile without disturbances on the surrounding soil. Results of laboratory tests indicated that not only lateral stress was changed but also vertical stress in soil. It can be assumed that earth pressure coefficient K changes less that it is usually taken into the calculation.

RÉSUMÉ: Le présent document décrit les essais en laboratoire de chargement de pieu à petite échelle. Le frottement superficiel peut être calculé à partir de la contrainte effective du sol σ'_v, du coefficient de contrainte latérale effective K et du coefficient de rugosité $\tan\delta$ à la surface de contact pieu-sol. Cette étude détermine quelle valeur de K doit être adoptée pour le calcul. La technologie des pieux influe sur l'état de contrainte dans le sol, particulièrement sur la contrainte latérale due à la théorie d'expansion de cavité. L'objectif principal de cette étude est de mesurer la contrainte latérale lors d'essais de chargements statiques et de déterminer s'il est possible de supposer que les contraintes latérales et verticales soient constantes. Les tests en laboratoire consistent à encastrer un pieu de béton cylindrique de 7cm de diamètre et de 25cm de long dans un sol pulvérulent. Les équipements de mesure additionnels ont été utilisés pour mesurer la charge appliquée, la résistance à la pointe et le tassement. Par ailleurs, un capteur de pression tactile a été installé sur la surface du pieu. Grâce à son épaisseur (seulement 0,1mm) et sa flexibilité, le capteur permet de mesurer la contrainte radiale sur la surface du pieu sans les perturbations du sol environnant. Les résultats des essais en laboratoire indiquent que non seulement la contrainte latérale mais aussi la contrainte verticale dans le sol a changé. Il est possible de supposer que le coefficient de la pression terrestre K varie moins que ce qui est habituellement pris en compte dans les calculs

Keywords: pile; skin friction; coefficient of lateral pressure

1 INTRODUCTION

The resistance of the shaft and the toe of the pile as well as the method of mobilizing them together with settlement is one of the most important issues of pile-ground interaction. In many computational approaches, it is believed that the ultimate resistance of the shaft is the result of the friction of the soil against the skin of the pile. The most important thing here is the correct determination of the coefficient of friction of the soil with the shaft surface of the pile and the component perpendicular to the shaft of the stress in the soil. The coefficient of friction of the soil against various materials has been the subject of many studies e.g.(Uesugi and Kishida, 1986). Accepting the correct horizontal stress value for the pile skin friction calculation is still a challenge for engineers. Often, this value is determined on the basis of the vertical component of the effective geostatic stress in the soil σ_v' and earth pressure coefficient *K*. The *K* values given by different authors vary significantly and reach values from 0.2 to 4.0. These values are in the range between the active earth pressure and the passive earth pressure. It should be assumed that the value of the coefficient assumed for calculation will be equals (1) (Rankine, 1857).

$$\frac{1-\sin\varphi}{1+\sin\varphi} \leq K \leq \frac{1+\sin\varphi}{1-\sin\varphi} \qquad (1)$$

Where φ (°) is the angle of internal friction of the soil.

The *K* coefficient undoubtedly differs depending on the piling technology used. For drilled piles, due to the loosening of the ground near the surface of the pile, the K coefficient will assume values as for an earth pressure in rest or an active earth pressure. In the case of displacement piles, compaction of the soil around the pile surface may result in a passive pressure state. The surface resistance calculated on the basis of the knowledge of horizontal stress is determined for the limit state. In this approach, it is not possible to analyse the pile-to-soil cooperation in the full load range. In order to fully assess the cooperation between the skin of the pile and the soil, two equations should be considered (2):

$$\begin{cases} \tau(s) = G\frac{s}{l} \\ \tau(s) \leq K\sigma_v' \tan\delta_k \end{cases} \qquad (2)$$

Where τ (kPa) is the unit skin friction of the pile; s is the settlement of the pile head (mm), G is a shear modulus of soil (MPa); σ_v' - effective vertical stress in the soil; $\delta_k -$ friction angle at the pile-soil interface (°).

Horizontal stresses in the soil through the use of different pile technologies can be reduced or enlarged in relation to the initial state. The results of analyses based on field and laboratory studies indicate that this increase may be even 10 times the initial value (Vermeer *et al.*, 2007; Basu *et al.*, 2008; Krasiński, 2014; Martinkus *et al.*, 2014; Konkol and Bałachowski, 2017). However, can it be considered that this value remains constant during the entire test trial of the static load of the pile? The paper presents changes of radial stresses on the pile surface caused by pile loading in laboratory conditions.

2 TEST PROCEDURE

2.1 Materials and methods

The tests consisted of test static loads of pile in assumed load levels. In each load step, the value of applied force in the head, the resistance under the pile toe, the displacement of the pile head and the soil horizontal pressure on the pile's skin were measured.

The research involved an instrumented concrete pile with a diameter of 7 cm and a length of 25 cm, equipped with a strain gauge transducer, a force transducer in the head and in the toe. Pile head settlement was measured using an optoelectronic displacement sensor. The piles were examined in non-cohesive soil - medium sand with a known initial density and grain size distribution as shown in Figure 1.

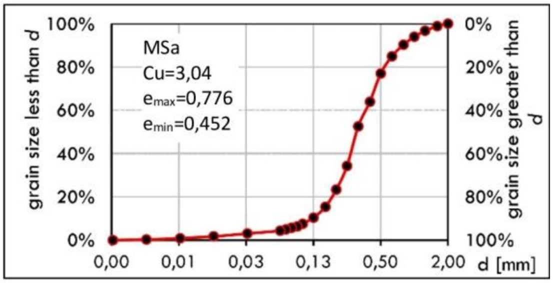

Figure 1. Basic sand properties and grain size distribution

The ground was compacted with layers about 2.5 cm thick. When the soil level in the chamber achieve the target level of the toe of the pile, the pile was placed in the chamber and firmly established to the independent construction. Next filling of the chamber was continued. Soil around the shaft of the pile was compacted layer by layer, until the required pile depth has been reached. The compaction method may have caused the POP value to be added to the initial stress state, i.e. Pre-Overburden-Pressure (Melnikov *et al.*, 2016), which according to calculations was 1.0-1.5 kPa.

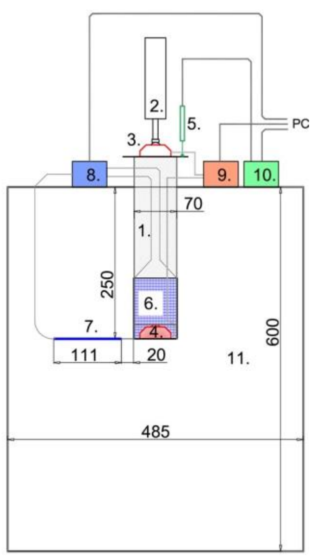

Figure 2. Research stand. 1- concrete pile, 2- load, 3,4- force sensors, 5- settlement sensor, 6,7- tactile stress sensors with PTFE cover, 8,9,10- data loggers, 11- medium sand, (dimensions in mm).

In order to measure the radial stresses around the pile, elastic force distribution sensors were applied to the pile, which were mounted on the surface of the pile shaft. These sensors are characterized by high flexibility and thickness of only 0.1 mm. These properties made it possible to minimize the influence of the sensor on distortion of the measurement. The pressure sensors were protected against mechanical damage with a PTFE film with a thickness of 0.07 mm. Figure 2 shows the diagram of the test bench together with the measuring instrument used. More on the preparation of the position was also given in (Meyer and Żarkiewicz, 2018).

In order to verify the correctness of the stress reading on the surface between soil and concrete, the sensor was calibrated in a testing machine. Calibration results indicated that the uniformity of the stress distribution depends on the grain size. For the sand under test, applying the average stress of 53 kPa, the stress distribution is shown in Figure 3. The unit stresses on the surface of the sensor ranged from 0 up to even 120 kPa. Measuring errors of the measured force value with the set force during calibration were in the range of 3 to 5%. Examples of the use of flexible pressure distribution sensors and calibration methods are also presented (Tessari, Sasanakul and Abdoun, 2010).

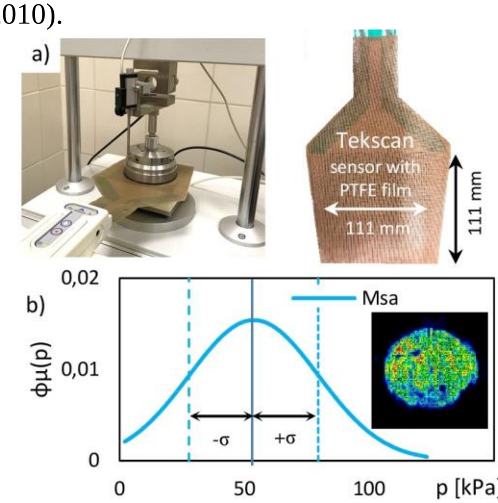

Figure 3. a) Calibration of sensor, b) Pressure distribution of reference load $P = 200\ N, \sigma_{av} = 53\ kPa$

The effect of scale is an issue that still requires research because it still raises many doubts. The main scale parameter in presented research in comparison with field piles is geometry of the pile H/D which equals 3,6. The previous research (Meyer and Żarkiewicz, 2018) indicated that failure mechanism of that pile was more similar to deep foundation than shallow foundations. The grain size of the sand should not be scaled because it would case strongly different behaviour which would be not compared to non-cohesive soil behaviour. The next differences from the field pile is geostatic pressure state which is several times smaller than in field. This could cause a contractive state in soil which is commonly observed in small stress. Analyses indicate that the size of the container, which is almost equal 7 diameters of the pile, should not have a significant impact, but it will also be the subject of further research. Nevertheless, laboratory tests could provide many valuable tips that are difficult to explore in field studies. The main goal is not to compare these tests with natural piles, but to indicate the phenomena that occur in the transmission of the load.

3 RESULTS

3.1 Measurement of vertical stress in soil

During the test static loads, the axial force applied in the pile head, the mobilized resistance under the pile toe, the pile head settlement and the vertical stresses in the soil near to the pile toe were measured according to Figure 4.

Figure 5 shows the results of measurements in relation to time. Visible are the load levels whose time was determined by the stabilization condition of the pile head settlement. The load was increased in any degree, causing the pile toe resistance to increase. The smaller force in the pile's toe was caused by the resistance of the pile's skin. The initial value of the vertical stress equal to σ_v=5,1kPa was the geostatic stress in the level of the pile toe.

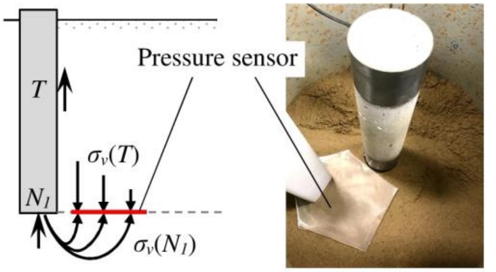

Figure 4. Toe and skin friction influence on vertical stress increasing.

The pile load caused more than threefold increase in the vertical stress component in the soil. The increase in the vertical stress in the soil was caused by resistances on the shaft of the pile, which were then transferred to the surrounding near to the pile toe. However, the results of the analyses indicated that this is not the only reason for the increase of vertical stresses. Only taking into account the resistance of the pile toe gives a full picture of the impact of the load transmitted by the pile on the state of stress in the soil.

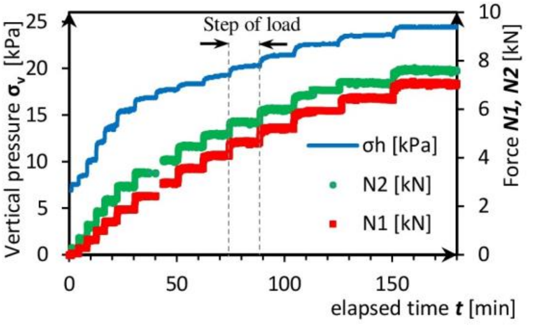

Figure 5. Results of test. N1 – toe resistance, N2– force applied at the head of the pile, σ_v – vertical pressure in the soil at the level of the toe of the pile.

The influence of the resistance of the pile toe is necessary to determine the state of soil pressure in the vicinity of the pile's shaft.

Vertical component of stress in the soil σ_v (kPa) taking into account the influence of the skin friction and toe resistance, in further calculations can be determined in accordance with equation (3).

$$\sigma_v = AN_1 + BT + \sigma_{v0} \tag{3}$$

Where N_1 (kN) is the resistance of the pile toe, T (kN) is the total resistance of the pile's side, A, B are the coefficients of the influence of the toe resistance and skin friction under test equal to 2.186 and 5.36, respectively, σ_{v0} (kPa) is a geostatic stress in the ground.

3.2 Measurement of lateral stress

The piles were equipped with pressure sensors placed on the pile's shaft in accordance to Figure 6.

During the test, the force applied in the pile head, the resistance of the pile toe, settlement of the pile head as well as the lateral stresses on the pile's skin were measured. The results of 5 tests indicated that the ground pressure to the side surface changes during the axial load of the pile. The increase was from 34% to even 315% in relation to the initial value

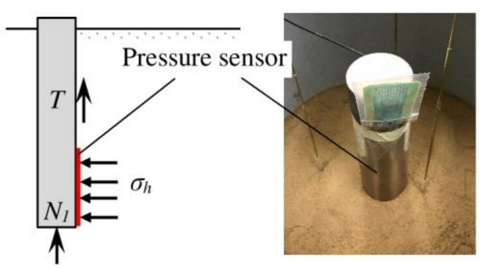

Figure 6. Measurement of lateral pressure on the skin of the pile

The results of tests are shown in Figs. 7 and 8.

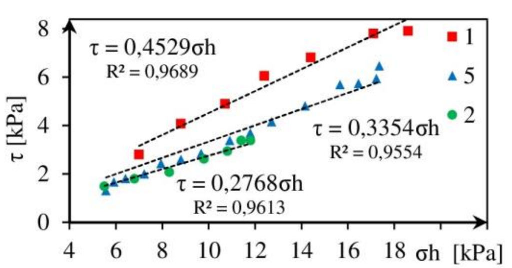

Figure 7. Relationship between resistance on the skin and lateral pressure around the shaft of the pile No 1,2,5.

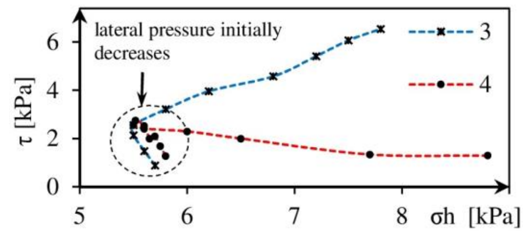

Figure 8. Relationship between resistance on the skin and lateral pressure around the shaft of the pile No 3,4.

The dependences obtained for piles 3 and 4 (Figure 8) differed significantly from piles 1,2,5 (Figure 7). During the analysis of the factors that could have influenced the above results, it was noted that these tests differed between the preparation of the station and the test load of the pile. In the case of a 1, 2, 5 pile, this time was three or four days, and in the case of pile 3 and 4 one day. As in the case of real piles, where the interval between making a pile and its testing has a significant impact on the results obtained. According to the author, due to the use of non-cohesive soils, the reason for this phenomenon is not the dissipation of pore pressure, but the unstable placement of grains, which can be identified with the phenomenon of soil contracting and dilatation.

During the test, values of horizontal stresses were obtained which, in relation to the initial value of the geostatic stress, would indicate the state of passive pressure. However, it should be borne in mind that the vertical stresses do not remain constant and change with the pile load. The maximum soil resistance is related to the critical state, and thus the frictional resistance should be calculated for increased radial stresses. Unfortunately, to determine the increase of these stresses together with the load of the pile, it is necessary to correctly identify the pile's relationship with the soil, the interaction of the toe and the pile's skin.

The earth pressure coefficient determined in laboratory tests can be expressed in the form (4):

$$K(s) = \frac{\sigma_h(s)}{\sigma_v(s)} \tag{4}$$

Where $\sigma_h(s)$ (kPa) is the horizontal component of the stress in the ground, $\sigma_v(s)$ (kPa) is a vertical component of the stress in the soil as a function of settlement of the pile.
Figures 9 and 10 show the change of pressure stresses σ_h obtained directly from the measurement as well as the average unit skin stress of the pile τ determined in accordance with Equation 5. These changes relate to the settlement of the head of two selected piles.

$$\tau(s) = \frac{N_2(s)-N_1(s)}{\pi DH} \qquad (5)$$

Where N_2 (kN) is the axial force is applied to the pile head, N_1 (kN) is the resistance under the pile toe, D, H (m) diameter and pile length equal to 0.07 m and 0.25 m, respectively.

For the analysis, piles were differing in the way of mobilizing resistances of the skin with settlement were selected. Pile No. 1 (Figure 9) showed a smooth increase in the shaft resistance, which was accompanied by an increase in pressure stresses.

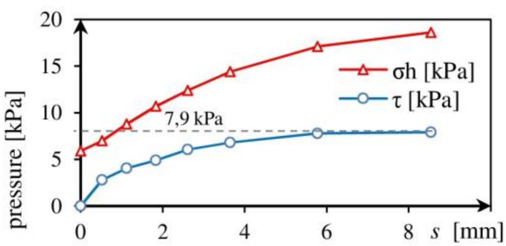

Figure 9. Lateral pressure and skin friction changes according to settlement of the pile no 1.

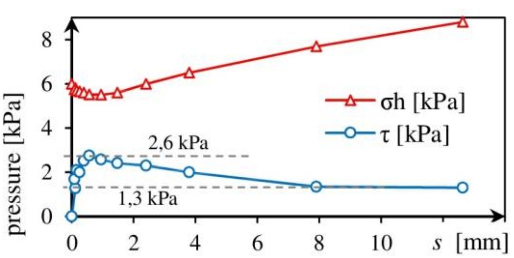

Figure 10. Lateral pressure and skin friction changes according to settlement of the pile no 4.

If in calculating the skin friction, take into account only the soil friction under initial stresses, then the mobilized shear stresses on the pile's skin should not exceed 2.5 kPa. The study achieved values of over 7 kPa. This was due to the increase in pressure stress. During the study of pile no. 4, presented in Figure 10, the increase of resistance of the pile's shaft to the value of 2.6 kPa was observed. At further loading, reduction occurred. The explanation of this phenomenon may be the achievement of critical shear stress in the contact zone between the soil and the pile's skin. Analysing the change in stress pressing pile no. 4, one can notice a completely different character of changes than in the case of pile 1. First, the pressure stress was reduced, but after exceeding the values of settlements at which the maximum unit skin friction was reached, the compressive stresses began to grow. The main difference between the pile behaviour presented on Figure 9 and 10 is the mechanism of horizontal pressure mobilization. The small decreasing in horizontal pressure which was pointed in Figure 10 could case flip the grain on the shaft of the pile. Despite the increase in stresses, slip still occurred.

3.3 Earth pressure coefficient K

Taking into account the vertical stress calculated on the basis of equation (4) for each pile, the coefficient K was determined, which is shown in Figure 11, depending on the settlement of the pile. This analysis shows that the K coefficient changes with the settlement of the pile. It was assumed that in the initial state, i.e. before loading the pile, the horizontal stresses were equal to vertical stresses. This was indicated by the results of stress measurements before the test. With the increase in settlement of the pile head, a slow increase in the K coefficient was observed from 1.0 to 1.2 for the pile No. 1, and for the pile No. 4 the coefficient K decreased to 0.4 and stabilized at a constant level.

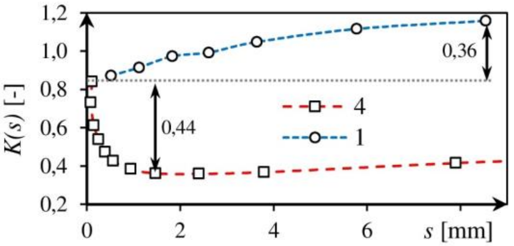

Figure 11. Lateral pressure and skin friction changes according to settlement of the pile no 4 and pile no 1.

When calculating the load capacity of a pile, dependence (6) is often used (Han *et al.*, 2017)

$$\tau_f = q_{sL} = K\sigma'_{v0}\tan\delta \tag{6}$$

Where τ_f, q_{sL} (kPa) are the unit limit shaft resistance, σ'_{v0} (kPa) is a vertical component of the initial stress in the ground, δ is the friction angle on the pile-soil contact surface, *K* is the soil pressure coefficient defined as the ratio of mobilized horizontal stress to the initial vertical stress in the soil (Han *et al.*, 2017), (Loukidis and Salgado, 2008), (Lashkari, 2013).

Many authors indicate that radial stresses around the side surface σ'_h grow with settling (Boulon and Foray, 1986; Fioravante, 2002; Lehane and White, 2005; Lashkari, 2013; Flynn and McCabe, 2016) according to equation (7), and the main cause of this increase is the phenomenon of soil's dilation during shear.

$$\sigma'_h = \sigma'_{h0} + \Delta\sigma_h{}' \tag{7}$$

Where σ'_{h0} (kPa) is a radial stress in the ground before loading, $\Delta\sigma_h{}'$ (kPa) is a change in radial stress caused by soil resistance on the pile's skin.

The question arises: whether the change in the radial stress in the vicinity of the pile surface can be expressed by the earth pressure coefficient? The earth pressure coefficient in the formula (6) is a parameter expressing the state of destruction determined on the basis of the initial state. It assumes that only radial stresses change. The *K* coefficient obtained from the retrospective analysis often exceeds the limit values as for passive pressure (Krasiński, 2003; Loukidis and Salgado, 2008). Laboratory tests which were carried out indicate that load transfer mechanism changes the state of stress in the ground. So not only a horizontal component, but also a vertical component in the soil is changed. Taking into account the changes in both components indicates that the earth pressure coefficient changes in a much smaller range than it results from the definition of the ground pressure coefficient *K* used in the formula (6). But then it should be assumed that *K* coefficient relates to the changed component of vertical stress in the soil. When examining the load capacity of a pile, not only the limit values of the skin friction and the pile toe resistance are important, but also the mobilization of these resistances with settling (Meyerhof, 1976). Taking into account these factors gives more clear model of the pile's interaction with the soil.

4 CONCLUSION

Research has shown that the skin friction *T(s)* changes both the horizontal component and the vertical component of the stress in the soil.

The determined soil pressure coefficient *K* as the ratio of the horizontal component to the vertical component of the stress in the soil mobilized at a given settlement of the pile was from 0.4 to 1.4.

The results of laboratory tests indicated that the change in the state of stress in the soil is caused by the simultaneous interaction of the resistance of the toe and the pile's skin.

The character of changes in soil pressure coefficient *K* is strictly dependent on the phenomenon of dilatancy or soil contractancy, depending on the initial porosity index and soil condition and stress in soil (Sawicki and Świdziński, 2007; Peng, Ng and Zheng, 2014)

A great influence on the change in the *K* coefficient has the time between the execution and the test of the pile. In the short time interval, the reduction of the *K* coefficient was observed, and in the long run the increase of the *K* coefficient in relation to the initial value.

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