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Skin friction of soil displacement threaded piles. What instrumented load test and what design norms say?

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ABSTRACT The design of displacement piles with non-uniform section is not explicitly covered in design norms because of the difficulty in modelling the various phenomena that affect pile behaviour and the limited availability of field results. This paper is a case study on an instrumented static pile test that was carried out on a "Screwsol®" threaded displacement pile in Romania. Based on the geotechnical investigation and the test results, safe bearing capacity will be computed from the test results, and compared to values estimated based on empirical values supplied by the Romanian geotechnical design norm for non-displacement bored piles. Also, the settlements measured in the load test will be analyzed and compared to admissible values. The main objective of the paper is the reasonable estimation of the bearing capacity and load-settlement behaviour of threaded piles for situations where preliminary pile tests are not possible. Also, recommandations for future design norms will be contained.

1 INTRODUCTION

In the field of deep foundations, displacement piles represent a viable solution for many engineering problems and they are commonly used within the foundation system of various structures. Displacement piles are diverse both from the point of view of materials (reinforced concrete, precast concrete or metallic – historically also wooden) and method of installation. They are characterized by little or no removed soil and ensure increased bearing capacity and improved load-displacement behaviour compared to equivalent bored piles, especially for axial loads.

One example of displacement piles is the Screwsol pile, developed by Soletanche Bachy. Although it is a bored pile, the soil is displaced horizontally during the downward rotation of the auger head (Tomlinson and Woodward, 2015). The main feature of the Screwsol pile is the helical flanged shaft (therefore the name "threaded pile"), which is formed during the and concreting phase, performed by high pressure injection while withdrawing the auger head. The reinforcement cage is introduced into the fresh concrete immediately after concreting.

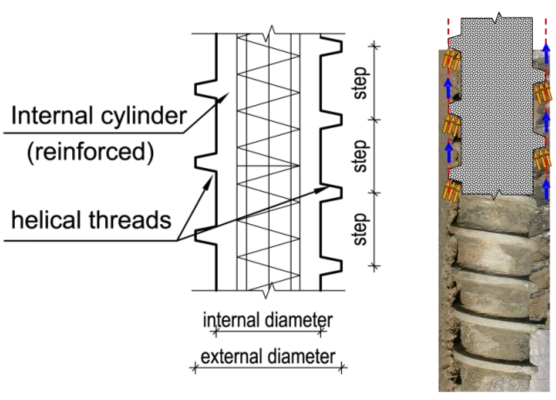

Figure 1. Screwsol pile

Although different versions of threaded piles have been used extensively in recent years across Central and Eastern Europe, there is limited information available on pile behavior, especially regarding ultimate pile capacity and mobilization of toe resistance and skin friction.

This paper presents the results of an instrumented static pile load test conducted in 2013 in Romania on a Screwsol pile. The pile was equipped with strain gages and loaded until failure, with continuous monitoring by the instruments.

The results of the measurements were then processed, leading to an estimate of the actual skin friction values. These results are compared to those obtained using the Romanian design norm NP123-2010.

Conclusions will be formulated regarding the test results and their comparison to the empirical method from the national design norm. Furthermore, general comments will be made regarding the behaviour and efficiency of threadead piles.

2 PRESENTATION OF PILE TEST

The test pile was a Screwsol pile with a diameter of d=330/500 mm (meaning 330 mm cylinder diameter and 500 mm external diameter) and a toe depth of H = 20.95 m from ground level (actual pile length of L = 20.60 m), reinforced on its entire length, in order to provide support for the strain gages and also to protect the pile head from local crushing at high loads. Four reaction piles were used for the load test, executed with the same technology.

The soil investigation revealed relatively homogenous soil conditions, containing stiff to hard sandy clay and silty clay in dry conditions. Mean soil parameters: deformation modulus E_s = 13.5 .. 20.9 MPa (correlated from oedometric modulus), unit weight γ_{nat} = 17.5 .. 20.0 kN/m^3, void ratio e = 0.50 .. 0.65, water content w = 12.0 .. 16.5%, consistency index CI = 0.95 .. 1.00. As for the shear strength parameters are concerned, the method used to obtain them is unclear, so they will be omitted from this analysis.

The static load test was carried out in equal steps of 125 kN, until failure. Application of a load increment was conditioned by stabilization of settlements for the previous step. Additionally, the pile was equipped with 17 extensometers and during the load test the deformation of the concrete was monitored with a frequency of 1 reading / minute.

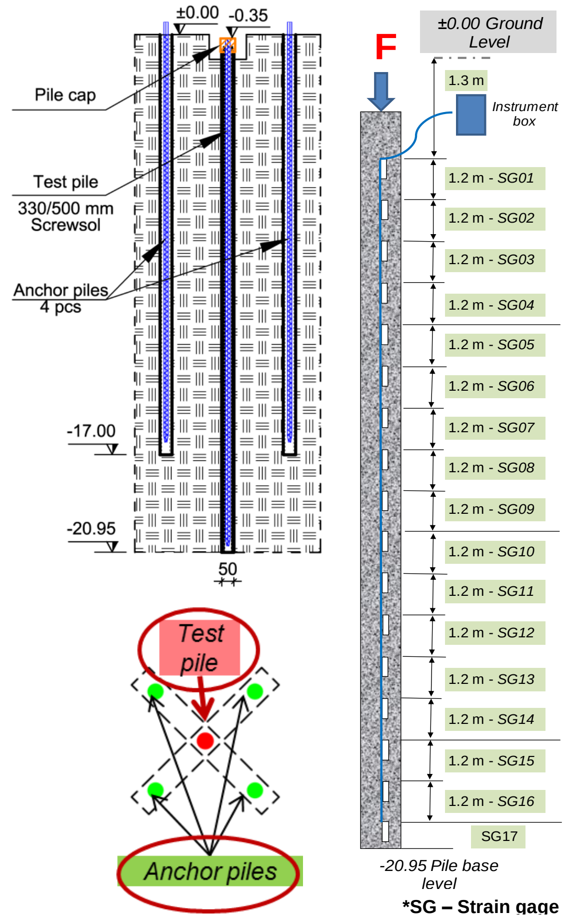

Figure 2. Pile test configuration

Figure 3. Strain gage installed on cage

3 TEST RESULTS

Pile failure occured at the 16th load step (2000 kN), most likely by crushing of the pile head due to eccentrical application of the load. This observation is supported by the load-settlement diagrams of the four individual sensors that measured the settlement of the pile, located at the four corners of the pile.

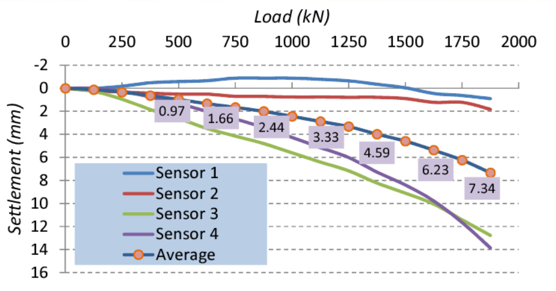

Figure 4. Load-settlement diagrams for sensors

According to fig. 4, a tilting of the pile head occured, with two sensors showing much smaller settlements than the other two (practically no settlement until 1500 kN).

Figure 5. Pile head during load test.

Fig. 5 is an image of the pile head during the load test. It can be observed that the hydraulic pump applying the load is placed slightly to the "left" of the image, with sensors no. 3 and 4 being placed on the left side, further confirming the previously mentioned hypothesis for pile failure.

The clarification of the pile failure mechanism is especially relevant for this case study because analysis of the monitoring results reveal that the pile experienced structural failure, and that geotechnical ultimate capacity (which would manifest in excessive settlements – 0.1 pile diameter) is significantly higher than 1875 kN, which is the last load step for which settlements successfully stabilized.

4 INTERPRETATION OF MONITORING RESULTS

Strain gauge measurements were processed in several steps in order to determine the detailed pile behavior, and to confirm or contradict the preliminary conclusions based on the classical load test results (Li and Ruban, 2009).

The following transformations were used in order to transform raw measurements into pile behaviour characteristics:

- 1: Frequency measurements were first transformed into concrete strain values;
- 2: Pile loads were computed for each level and moment using strain values;
- 3a. Based on the pile loads and pile section geometry effective skin friction values were estimated;
- 3b. Using the same input as in 3a, settlement along pile depth were computed for each load step.

Some details of the transformations will be presented, especially those implying the non-standard cross-section of the Screwsol pile.

The two fundamental relations governing the transformations are the following:

$$P = E \cdot A \cdot \varepsilon \tag{1}$$

Where:

E is the elastic modulus of the pile (kPa);

A is the cross-sectional area (m^2);

ε is the measured strain in the gages (microstrain – dimensionless).

$$q_s = \Delta P \cdot A_l \tag{2}$$

Where:

q_s is the effective skin friction (kPa);

ΔP is the variation of the pile load between two adjacent gages (kN);

A_l is the shaft area between the same two strain gages (m^2).

In case of the Screwsol pile, the most important uncertainty is the choice of the cross-sectional area A (which automatically influences the lateral surface considered in the calculation of skin friction). Also, the displacement of the soil needs to be taken into consideration when computing pile forces and skin frictions. A third uncertainty is the elastic modulus of the combined concrete-steel pile, which can be safely evaluated for design purposes, but for research purposes (such as the back-calculations in this paper) it is difficult to assess.

The typical solution is the calibration of the elastic modulus using relation (1), where E is varied until the pile force at the pile tip level is equal to effective load applied at a certain step, typically the last step before failure. In this case however, due to the number of uncertainties, the elastic modulus was chosen with the realistic value of 26 GPa, and the area was calculated. The area for which computed pile loads matched with effective applied load was the one obtained for the equivalent diameter of 330 mm (exactly the inner diameter of the pile, i.e. the diameter of the reinforced concrete inner cylinder).

The results of the measurements are presented on the following figures (pile force, skin friction, settlement)

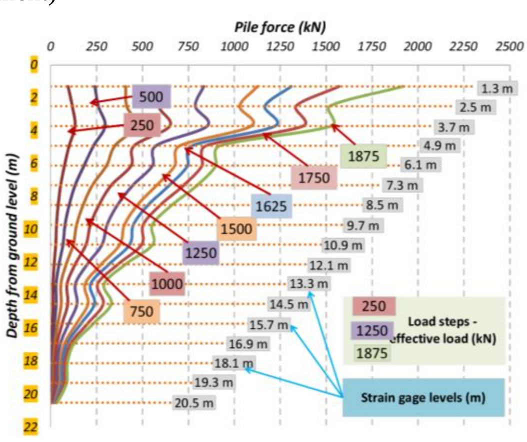

Figure 6. Pile forces for load steps

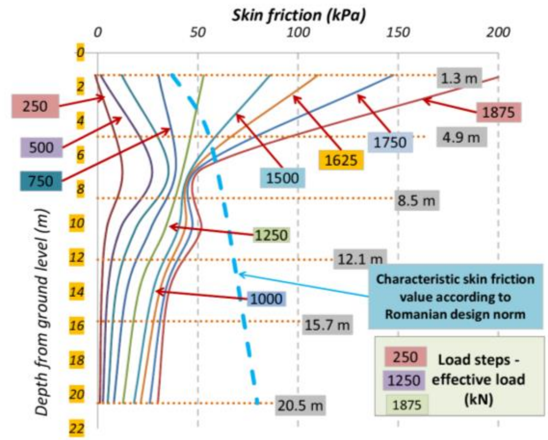

Figure 7. Skin friction diagram

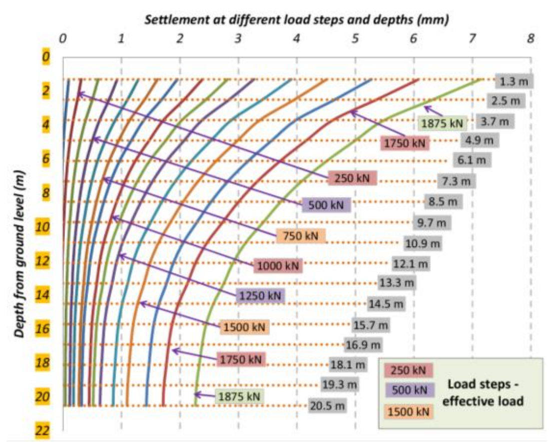

Figure 8. Settlement diagrams

The results presented in fig. 6-8 consistently reveal the pile's tendency to consume the majority of the loads in the upper half of its length. In case of efficiently designed non-displacement piles it is expected that the majority of the loads are transferred through the lower part of the shaft (skin friction) and also by tip resistance. Skin frictions in the upper few meters reach values of 200 kPa and more, compared to a maximum of 80 kPa given by the design norm.

However, in this case there are factors that suggest a different behaviour. Due to the presence of the threads, combined with the displacement of the soil, larger shear stresses may develop in the upper part of the shaft. Equally importantly, the soil is homogenous, even at the pile toe level, while in most design scenarios the soil layers towards the pile toe are significantly stiffer.

5 COMPARISON WITH DESIGN NORM

The comparison of the measurement results with the empirical method from the national design norm is a process which implies several uncertainties due to some limitations in the design norm and also the complex behaviour of the Screwsol pile. In order to attempt a realistic comparison, the following assumptions have been made.

- The pile is considered as a non-displacement cast-in-place bored pile;
- The tip resistance is calculated using the inner diameter (330 mm), while the shaft resistance is estimated considering an equivalent diameter equal to the outer pile diameter (500 mm);
- For the choice of safety factors used for the calculation of the bearing capacity from field measurement results, the pile will be considered equivalent to a CFA pile.

<u>Bearing capacity from test result</u>

The applicable relation considering a single load test on the project site is:

$$R_{c,d} = \frac{R_{c;m}}{\zeta_1 \cdot \zeta_t} = \frac{1875kN}{1.5 \cdot 1.1} = 1136kN \qquad (3)$$

Where:

$R_{c,d}$: safe bearing capacity, design value (kN);

$R_{c;m}$: ultimate pile capacity from load test, mean value (kN);

ζ_1: safety factor related to the number of tests;

ζ_t: safety factor related to the execution technology and type of bearing capacity (shaft, tip or total);

<u>Bearing capacity from empirical method</u>

Using the empirical method for cast-in-place floating bored piles and the assumptions mentioned above, the bearing capacity is determined bellow:

$$R_{c,d} = \frac{A_b \cdot q_{b,k}}{\zeta_{b,2}} + \frac{U \cdot \sum (q_{s,k_i} \cdot l_i)}{\zeta_{s,2}} \qquad (3)$$

Where:

$R_{c,d}$: bearing capacity, design value (kN);

A_b: cross-sectional area at pile toe (m^2);

$q_{b,k}$: toe resistance, characteristic value (kPa);

U: shaft perimeter (m);

$q_{s,ki}$: unitary skin friction (kPa);

l_i: depth of elementary soil layer (<2 m);

$\zeta_{b,2}$, $\zeta_{s,2}$: safety factors for toe and shaft resistance (dimensionless).

$$R_{c,d} = \frac{0.085m^2 \cdot 1920kPa}{1.2} + \frac{1.57m \cdot 1320kN/m}{1.9} =$$
$$136kN + 1091kN = 1226kN$$

By comparing the two values, the bearing capacity obtained by the empirical method is 8% larger than the one obtained from the field test. However, careful engineering judgement needs to be applied when interpreting this difference.

First of all, the pile settlement at failure was very small (7.34 mm at 1875 kN), which suggests that in case of correct (centric) application of the load, ultimate capacity is significantly higher, probably in the range of 2500-3000 kN (at higher values the compressive stresses in the concrete become excessive and combined with buckling effects lead to structural failure).

Also, when assessing bearing capacity from the pile load test, one of the safety factors (ζ_1) was considered 1.5 because there was only one pile test. In case of 5 or more tests, the safety would become 1.

Considering these two aspects, it can be concluded that for this test, using reasonable judgement, bearing capacity from the pile test is significantly higher than that obtained using the empirical method. Also, considering the monitoring results, it can be concluded that for certain subgrade conditions threaded piles can transmit compressive loads predominantly through skin friction in the upper part, in which case toe resistance should be neglected.

6 CONCLUSIONS

The interpretation of the monitoring results confirms that the pile experienced structural failure and that geotechnical ultimate capacity exceeds the maximum load value. Also, it can be concluded that for this load test skin frictions were only developed in the upper half of the pile, and their shape and value is significantly different than it is indicated in the national design norm for pile foundations: NP123-2010. These results suggest a very good behaviour of threaded piles in stiff clays and confirm the efficiency of these pile types compared to equivalent bored piles.

In order to ensure allow a more realistic design of threaded displacement piles, design norms should be updated based on properly documented case studies and also theoretical considerations.

Some of the author's suggestion for achieving the proper inclusion of threaded (and more generally, cast-in-place displacement piles) are listed below.

Limiting values of skin friction should be increased for certain soil types to account for the lateral soil displacement. Also, recommendations are necessary for choosing the design situations in which toe resistance is neglected.

Another important issue is the effect of the threads in increasing bearing capacity and reducing settlement, which is difficult to quantify, but which has a significant influence. One solution would be the freedom of the Designer to choose the applicable safety factor for shaft resistance, obviously between reasonable limits set by the editors of the norms, based on properly case studies implying instrumented load tests on piles or pile groups.

The third aspect refers to the mobilization diagrams for shaft and toe resistance, which should be specified for displacement piles (including threaded piles) similarly to diagrams for bored piles that are included in some national design norms.

Due to the complex behaviour of threaded piles, these measures must be proceded by several well-documented case studies in order to serve as entry data for future calculation methods and design norms.

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