



Article Bi-Directional Static Load Tests of Pile Models

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Abstract: This work examined a new method of bi-directional static load testing for piles, referencing the Osterberg test. Measurements were taken, on a laboratory scale, using six models of piles driven into a box filled with sand. This method allowed for separate measurements of pile base and pile shaft bearing capacities. Based on the results, the total pile bearing capacity and equivalent Q-s diagrams were estimated. The results obtained show that the structure of the equivalent curve according to Osterberg is a good approximation of the standard Q-s curve obtained from load tests, except for loads close to the limit of bearing capacity (those estimates are also complicated by the inapplicability and ambiguity of a definition of the notion of limit bearing capacity); the equivalent pile capacity in the Osterberg method represents, on average, about 80% of the capacity from standard tests.

Keywords: pile testing; axial capacity; laboratory tests

1. Introduction

Static load tests are commonly recognized as the most reliable assessment of pile bearing capacity [1–5]. Preparation of an appropriate retaining structure capable of transmitting the huge loads applied to the pile by a hydraulic cylinder (even 10 or 20 MN) is the most difficult part of the test. Under certain circumstances, an alternative is the Osterberg test, which enables the determination of pile bearing capacity without the need to prepare a retaining structure [6]. Based on two Q–*s* curves from testing (separate tests for upper and lower parts of the pile), an equivalent curve that represents pile behaviour during standard static load tests can be drawn. A large advantage of the Osterberg test is that the pile capacity can be assessed without a retaining structure. Nevertheless, tests conducted on different types of piles and materials have shown that the stress conditions [7] and the interface behaviour [8] are not the same in both kinds of tests. Consequently, the strength characteristics (stress–strain behaviour) on a pile shaft are different for each test.

This paper presents a new method for running bi-directional static load tests for driven piles. The method refers to the concept of the Osterberg test [6,9], but also to the Austrian method of Hayden [10] for micro-piles and to the Slovak method Vuis-P [11]. This new method allows for separate measurements of pile shaft and pile base bearing capacity and requires no retaining structure, in contrast to standard static load tests. There is a lack of papers comparing the results of the standard and bi-directional static load tests [12] and analysing bi-directional static load tests on a laboratory scale.

Although the model examination (including numerical simulations) is not a substitute for full-scale field tests, it may be a source of additional knowledge about pile behaviour and how the behaviour is affected by various factors such as direction of the shaft displacement.

This paper outlines the traditional static load tests and bi-directional tests made on a laboratory scale for models of driven piles. Based on the results from bi-directional tests, the *Q*–*s* equivalent curves were drawn and compared with those from the static load tests. The method shown is directly applicable to models of steel driven piles, but the conclusions could also be useful in analysing other

models of displacement piles. The purpose of this research was to compare the two tests in a simple laboratory model. More comprehensive analysis of laboratory tests and scale effects can be found in numerous papers [13–19].

2. Methodology

2.1. Test Setup

To run tests for model piles, a rigid test box, $B \times L \times H = 1.15$ m $\times 1.25$ m $\times 1.50$ m, was prepared. Internal wall surfaces were lined with PVC foil to protect them against moisture and to reduce side friction and boundary effects. The completed box was filled with sand up to a total height of 120 cm.

Siliceous medium sand from a sand mine near Wroclaw, Poland, was chosen for the study. A particle size distribution curve of the sand is presented in Figure 1, and the sand's parameters are presented in Table 1. The moisture content was considered as insignificant to the test results [20].



Figure 1. A particle size distribution curve of the sand used in this study.

Table 1. Parameters of the sand used to fill the box.

Sand	$ ho_d$ (g/cm ³)	w (%)	d ₅₀ (mm)	$U_c = d_{60}/d_{10}$	w _{opt} (%)	$\rho_{d,\text{opt}}$ (g/cm ³)
	1.66	7.0	0.5	5.2	11.9	1.74

 ρ_d is the dry unit density, $\rho_{d,opt}$ is the maximal dry unit density from the Proctor test; U_c is the coefficient of uniformity; w is the moisture content, w_{opt} is the optimal moisture content from the Proctor test; d₅₀ is the grain size at which 50% of the particles by weight, respectively, are smaller.

The compaction ratio can be calculated as $I_s = 1.66/1.74 = 0.95$. Based on some correlation formulae for sands, the following estimation of the relative density can be derived: $D_r \approx 0.57$, so this is a medium-density sand.

The sand was gradually poured into the crate, and twenty-centimeter layers were formed, which were subsequently compacted. The control of sand compaction was carried out using a cylinder and a ZORN dynamic plate with accuracy of 0.1 MN/m² (Figure 2). At various locations in the crate, three

measurements were made at three depths, 40, 70, and 100 cm, for cylinder measurement and four depths, 40, 70, 85, and 100 cm, for the dynamic plate. The results of the measurements are summarized in Tables 2 and 3. Both tests showed higher soil density at the middle of the box and lower soil density at the bottom. A constant humidity was kept in the room during measurements.



Figure 2. The cylinder and dynamic plate used for the control of sand compaction.

LP	Sand Layer	Soil Constrained Modulus (E _{vd})	Mean Soil Constrained Modulus (E _{vd,mean})	
	cm	MN/m ²	MN/m ²	
1	40	26.6		
2	40	20.2	23.4	
3	40	23.4		
4	70	26.9		
5	70	17.6	22.9	
6	70	24.1		
7	85	21.5		
8	85	15.9	18.9	
9	85	19.3		
10	100	29.8		
11	100	30.3	28.3	
12	100	24.9		

Table 2. Results of ZORN dynamic plate tests, accuracy 0.1 MN/m².

LP	Sand Layer	Weight	Mean Weight	Volume	Soil Density	Degree of Compaction
	cm	kg	kg	cm ³	g/cm ³	-
1	40	4.18				
2	40	4.19	4.18		1.64	0.94
3	40	4.18				
4	70	4.26		_		
5	70	4.24	4.25	2545	1.67	0.96
6	70	4.25				
7	100	4.30		_		
8	100	4.32	4.31		1.69	0.97
9	100	4.31				

Table 3. Results of soil density tests with cylindrical mould.

To a certain extent, the situation is sensitive to changes in the relative density, D_r , of the sand. In the authors' opinion, a relative density $D_r \sim 60\%$ is representative, i.e., it corresponds to geoengineering practice, because the use of displacement piles is not recommended in very dense sands. Moreover, the paper is focused on the study of relative relations between the shaft and the base bearing capacities, not on the absolute values; in this way, the results and conclusions are less sensitive to the compaction level.

2.2. Pile Models

Six steel pipes, 1.0 m long, with an outside diameter 4.2 cm and wall thickness 0.25 cm, were used as pile models. The layout of the pile models in the box is shown in Figure 3. Pipes used in testing were identified successively as 1, 2, 3, 4, 5, 6, which corresponded to the order they were installed (Figure 3). The pile heads were situated 20 cm above the top of the sand level, so 80 cm of the pipes were embedded in sand, with 40 cm between the end of the pipes and the bottom of the box. This ratio 40 cm/4.2 cm is close to 10; Polish codes of practice for piling works pay special attention to soil conditions within a layer thickness of 5D below the pile base, any deeper mineral layers are not considered in capacity analysis of a single pile. As all the piles were tested separately, not in a group, it was assumed that there were no bottom effects. A similar assumption was made concerning the distance from sandbox's walls. Again, Polish codes require that anchoring piles or the points of support of reference systems for displacement control should be out of the 4D-wide zone around the tested pile (it was more than 6D in the tests in Figure 3).

The pile base included a steel cap, with a diameter equal to an external diameter of the pipe, loosely placed under the pipe during driving (Figure 4a). Additionally, all caps included a threaded bolt, enabling a steel pole to be screwed to them (Figure 4b).

Pile models were driven using a light dynamic penetrometer to the depth of 0.8 m (Figure 5). While piles were driven, the number of necessary blows, N_{10} , to settle the pile by successive 10 cm increments was determined. The results for the successive pile models are given in Figure 6.



Figure 3. Layout of pile models, 0.8 m embedded in the soil, in the box.



Figure 4. (a) Steel bottom of model pile base. (b) Pole placed inside the pipe.



Figure 5. Pile installation with a light dynamic penetrometer.



Figure 6. Diagrams of the N_{10} , number of impacts necessary to achieve 10 cm of displacement, for each pile during installation.

During testing, measurements were taken of the pile model head settlement, s, versus the applied force, Q; thus the *Q*–*s* curve was constructed. The force was applied using a jack and was taken by means of a load cell with 0.02 kN sensitivity. Displacements were measured with two sensors: an electronic displacement sensor, Keyence type GT2, with an accuracy of 0.001 mm, and analogue sensor ensuring displacement accuracy of 0.01 mm.

3. Testing Program

3.1. Traditional, Compression Static Load Tests

The first stage included static load tests for model piles loaded on their heads; this enabled the determination of the Q-s curve for the whole pile (Figure 7a). Proper struts for the jack were assured by means of a wooden retaining structure transmitting the forces to the rigid ceiling in the room containing the sand-filled box. A cover plate, sensor, and the jack were placed between the pile model and the retaining structure resting on the rigid box (Figure 8a).



Figure 7. Concept of traditional static load tests (**a**) and scheme of bi-directional tests (**b**), 1: uplifted steel pipe, 2: compressed pole of dynamic probe, 3: frame composed by the two flat plates and four threaded bars, 4: steel collar attached to the pile head with three horizontal set bolts 5: load cell, 6: manual jack.



Figure 8. Stands for traditional static load tests (a) and bi-directional tests (b).

Successive loads were applied step by step (by 0.2–0.3 kN increments) after the settlement of the previous step was stabilized. An example of pile head load and displacement vs. time plot is presented in Figure 9. Electronic sensors provided continuous automatic measurements of the force applied and of pipe displacement. The testing was continued until the loss of bearing capacity, i.e., until an uncontrolled increase of settlement was measured even with a small rise of the load.



Figure 9. Example of pile head load and displacement vs. time plot for the traditional static load test.

3.2. Bi-Directional Static Load Tests

After the first stage, described in Section 3.1, the bi-directional tests were performed on the same six piles. In order to run a bi-directional test, a pole of a dynamic probe was placed inside the steel pipe and screwed to the steel bottom (Figure 4). The jack, resting against an appropriate retaining structure as in the standard testing, was placed on the pole (Figures 7b and 8b). A load was applied as in the standard testing, i.e., step by step (by 0.2–0.3 kN increments) after the settlement of the previous step was stabilized. When the load from the jack was increasing, the pole with the cap was pressed into the soil. At the same time, the pipe was pulled out due to the frame composed by the two flat plates and four threaded bars pulled up the collar attached to the pile head via horizontal set bolts.

During testing, the applied force and displacement of the pulled-out pipe were continuously measured, and additionally, the displacement of the pushed pole was read on a continuous basis (Figure 8b). Testing was finished when either base or shaft failed.

4. Bearing Capacity Analysis

4.1. Determining the Bearing Capacity of Pile Base and Shaft

The test results were used to produce diagrams of pile settlement versus applied force for the standard static load tests and of shaft lifting and base settlement versus applied force in the case of the bi-directional tests. The Brinch–Hansen 80% method [21] was used to approximate the test results as it enables a good fitting of Q–*s* curves to the results, and to the estimation of the boundary bearing capacity of the pile base and shaft [22]. A reversed parabolic shape of the Q–*s* curve given by formula (1) is assumed in this method. The shape of the Q–*s* curve and the ultimate capacity Q_{ult} can be determined from the formulae (1) and (2).

$$Q = \frac{\sqrt{s}}{s \times A + B} \tag{1}$$

$$Q_{ult} = \frac{1}{2\sqrt{A \times B}} \tag{2}$$

where A and B are the constants for the Brinch–Hansen method, depending on the system of units adopted.

4.2. Constructing the Equivalent Curve

Based on two Q-s curves (for pile base and shaft) it is possible to draw the so-called equivalent load-settlement diagram, which approximates the traditional static test loads. Several papers have described this problem [6,12] over the years. At the beginning, both curves from the bi-directional test are divided into a series of points. Then, any two points from both curves are selected for the same displacement values and a new point is created. The new point has the same value of displacement, but the ordinate of loads results from summing the loads for the two points selected. This procedure is repeated for successive points until the whole equivalent curve Q-s can be drawn (Figure 10).



Figure 10. Method for constructing exemplary *Q*–*s* curve.

4.3. Results

Results are summarized in Figure 11. Each diagram illustrates the testing performed for particular piles. The results of the whole pile settlement, its base, and those for the lifting of the pile shaft are given in the form of points. Additionally, the Q-s curves approximating the results using the Brinch–Hansen 80% method and the equivalent curve together with an estimated total capacity of the pile are presented in Figure 11. A summary of the capacities obtained is also given in Table 4.



Figure 11. Test results for standard static loads and bi-directional tests including the resultant equivalent curve: (**a**) pile 1. (**b**) pile 2. (**c**) pile 3. (**d**) pile 4. (**e**) pile 5. (**f**) pile 6.

D:1.	R_b	R_s	$R_{c,e}$	R_c	$R_{c,e}/R_c$
Pile	kN	kN	kN	kN	-
1	1.61	2.16	3.77	4.69	0.80
2	1.56	2.11	3.67	5.22	0.70
3	2.54	3.08	5.60	6.07	0.92
4	1.80	2.82	4.38	5.32	0.82
5	3.22	4.62	7.78	8.66	0.89
6	3.29	5.66	8.10	9.49	0.85

Table 4. Test results.

 R_b : pile base bearing capacity from bi-directional test. R_s : pile shaft bearing capacity from bi-directional test. R_c : total pile bearing capacity. $R_{c,e}$: equivalent bearing capacity of total pile.

5. Discussion

Conducting Osterberg tests pointed out that the bi-directional tests provide the most useful results when the base capacity is close to the pile shaft capacity, which does not happen often. Then, the equivalent bearing capacity from such tests corresponds approximately to the static compression capacity of the pile [6]. In this study for all pile models tested, it was found that the shaft capacity was higher than that of the base. This was anticipated due to the results of the pile installation, when smaller numbers of blows were measured on the lower parts of the piles. Nonetheless, in each case, when the base reached its capacity, the pile shaft was in an elastic–plastic state close to capacity loss. Hence, it was also possible to determine the ultimate bearing capacity of the shaft using the extrapolation with the Brinch–Hansen 80% method.

The capacity results of the pile models differed to some extent. A clear connection between capacity measured and local soil compaction was observed, i.e., larger capacity was found for the pile models featuring higher resistances while they were driven with a light dynamic probe (piles no. 5 and 6 in the central part of the box, Figure 3). The method underestimates, more or less, the bearing capacity, so one can conclude that the shaft pushed into the soil mobilizes the resistance more effectively than the one being pulled-out of the subsoil.

Problems arise when transferring the laboratory test results to full-scale foundation piles. The size of grain highly affects results of scale effects. More conclusive, constructive results could be obtained when finer graining soil is used in model testing, e.g., in [23], or in tests with a geotechnical centrifuge [17] or hydraulic gradient [24].

The differences in capacities obtained for particular piles may be the result of other operating methods for the bases and shafts in the two tests. In the traditional compression test, the shaft operates during penetration, while in bi-directional test, the shaft operates during pulling out. The ratio of the shaft capacity of the pile driven in vs. that of the pile pulled out depends on, but is not limited to, soil conditions, pile installation method, pile stiffness, and other factors. De Nicola and Randolph [25] demonstrated in an example of a parametric study for piles in the elastic-and-ideally-plastic model of Coulomb Mohr, that the ratio between capacity of the pulled out shaft and that of the driven in shaft is maintained within 0.7-0.85. The results found in this paper can also be assessed in a similar range. Note that a 75% reduction of *s* in the upper *Q*–*s* curve in Figure 10 will also cause a correction of the equivalent *Q*–*s* curve in Figure 11; in particular, the real *Q*–*s* curves in Figure 11 and the equivalent ones will converge.

6. Summary and Conclusions

The guiding principle of bi-directional static test loads is to run separate measurements of the forces transferred by the pile base and pile shaft without the need of constructing an expensive retaining structure. The results obtained were approximated and analysed with the Brinch–Hansen 80% method. This allowed us to assess the bearing capacities and to determine Q–s curves of shafts and bases

separately for each pile. Moreover, bi-directional test results were used to draw the equivalent *Q*–*s* curves according to the Osterberg method and to determine the total capacity of each pile.

The results showed that the proposed method can be suitable for determining the pile capacity of the pile, which was also confirmed by field tests and numerical simulations [26]. However, in true scale pile tests, results can be difficult to predict due to different layered soil conditions.

The problems that can appear in relation to the bi-directional method are similar to those in the Osterberg test: the bearing capacity of the whole pile is in fact determined by the capacity of the weaker pile element (shaft or base). Hence, it is necessary to perform an earlier assessment of pile capacity in a given soil. In general, the equivalent bearing capacity according to the Osterberg method appeared to be about 20% lower than the standard total capacity.

Operating conditions of a pile base are also different depending on the type of test carried out. In the traditional compression test, loading of the shaft can increase the bearing capacity of the base. In the bi-directional test, pulling out the shaft can cause a reduction of stresses at the pile base level, and in consequence reduces the base bearing capacity.

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