Load tests on shaft grouted piles fully instrumented. Measurements and interpretation of the tests results

Essais de chargement sur des pieux injectés au niveau du fût complètement instrumentés. Mesures et interprétation des résultats des tests

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ABSTRACT: The present paper presents the load tests on fully instrumented piles for design and optimization of a piled raft foundation for a building of three basements and 16 stories above ground situated in Bucharest, Romania, on the left bank of Dambovita River. The piles were drilled with bentonite slurry protection and they were shaft post-grouted on the entire pile length for improving their bearing capacity. Supplementary to the load displacement measurements, the test piles were instrumented with strain gauges at each layer change level, displacement transducers and with pressure cell at the pile tip. This instrumentation allowed for advanced interpretations which lead to a more economical and safer solution due to the detailed information regarding the behaviour of the loaded piles.

RÉSUMÉ : L'article présente les essais de chargement réalisés sur des pieux complétement instrumentés utilisés pour le dimensionnement et l'optimisation d'une fondation mixte - radier sur pieux - d'un batiment avec trois niveaux de sous-sol et 16 étages situé à Bucarest, Roumanie, sur la rive gauche de la rivière Dambovita. Les pieux ont été forés sous protection de boue bentonitique et ont été post-injectés au niveau du fût afin d'améliorer leur capacité portante. En plus des mesures de déplacements de la tête des pieux sous chargement, les pieux ont été instrumentés avec des capteurs de déformation à chaque changement de lithologie et avec une cellule de force à la base du pieu. Cette instrumentation a permis une étude complexe sur la capacité portante des pieux et a permis d'adopter une solution plus économique et plus sure pour la fondation.

KEYWORDS: shaft grouted piles, instrumented piles, strain gauges

1 INTRODUCTION

Foundation works often imply a considerable time and cost consumption compared to the value of the project. For this reason, it is highly important to choose optimum and efficient solutions for performing qualitative, safe and feasible, but at the same time economic works.

The design methods for piled foundations are diverse and well documented. However, the complexity of the soil structure interaction phenomena makes the theoretical modelling differ from the experimental results even when using advanced models. Thus, in the case of complex works or of special technology, the norms recommend or even require the performance of pile load tests on site. Even if these tests are very useful for determining the bearing capacity and the rigidity of the loaded piles, they not sufficient for understanding the mechanism of transferring the load from the pile to the surrounding soil. What is more, if an improvement of the pile capacity through shaft and/or base post-grouting is performed, the pile-soil interaction is even more difficult to estimate. In these conditions, the performance of load tests on piles instrumented through their entire length regarding for the lithology becomes essential.

The designed building has three basements and 12 to 16 floors above ground and a surface of about 6600 m^2. The shape of the building is very irregular in plane ("butterfly" shape), but also vertically. The structural solution consisted of reinforced concrete cores and column and flagged slabs. The basement structure is wider than the superstructure consisting of the extension of the vertical elements of the superstructure and also extra reinforced concrete columns and walls.

2 FOUNDATION SYSTEM AND GROUND CONDITIONS

2.1 *Foundation system*

As a result of the structural design, the following piled raft foundation system resulted: general raft 2.0 m thick and 62 piles having 0.8 m and 0.9 m diameter and 35 m depth under the raft under the central core area, and 1.5 m thick raft in rest and 48 piles having 0.8 m diameter and 14 m depth under the raft under the lateral cores of the stairs.

According to NP 123:2010, tests must be performed in the final design phase, in order to assess the bearing capacity of the piles. Thus, through the design, 3 axial compression tests for N2 quality level were performed: tests carried out on piles that are not comprised in the work, in order to determine the bearing capacity and the loading-displacement dependency, according to NP 045-2000. For bearing the reaction at axial loading, 4 piles loaded at pulling forces (anchor piles), and a beam system were used for each test.

The Special Foundations Contractor proposed performing the piles by drilling under the protection of bentonite slurry and with improving the strength and deformability characteristics through shaft grouting in tubes-a-manchette located at 1 m distance interval. Previous studies documented the fact that the improvement technology through shaft grouting lead to an increase of the pile bearing capacity with up to 100% - 200% on the cohesionless layers and with up to 20% - 50% on the cohesive layers (Miller et al. 2013, Plumbridge and Hill 2001).

On the outside of the reinforcement cage, in the cover area of the reinforcement, 4 tubes a manchettes were placed at equal distance, at 1 m interval. Inside the reinforcement cage, in the test piles, steel casings were installed in order to perform the continuity tests by sonic velocity logging method.

At 1-3 days after concrete casting, the shafts of the piles were grouted through the tubes a manchettes, using grouting pressures of 10-33 bars in the toe and lower towards the ground surface, and mean quantities of 5-15 l on each manchette in the cohesive layers, respectively 15-40 l in the cohesionless layers.

2.2 *Site conditions*

The geotechnical investigations consisted in 8 geotechnical boreholes with depth ranging from 27 m to 50 m with disturbed and undisturbed sampling for laboratory tests, SPTs in the boreholes as well as Downhole tests for seismic evaluation of the site and determination of the deformation in the small strain domain. Typical lithology for Bucharest was encountered on site: filling, followed by a thin deposit silty clay at the surface, then Colentina sands with rare gravel, intermediate clay layers and Mostistea fine sand layers with many lens intercalations.

In Table 1, the schematic lithology on site can be followed as well as the characteristic values of the main geotechnical parameters as resulted from the geotechnical report.

Table 1. Lithology and main geotechnical parameters

Layer description	Absolute level	γ	Ic	Dr	E_{oed}	Φ'	c'
	(m ASL)	kN/m³	-	%	MPa	°	kPa
Filling	76.00÷72.00	19.0	-	-	7.0	18	5
Sandy Silty Clay	72.00÷68.00	19.5	0.63	-	9.5	22	15
Clayey Sand with gravel	68.00÷64.00	19.0	-	55	30.9	32	-
Silty Clay	64.00÷61.00	19.6	0.75	-	17	19	40
Fine-medium sand	61.00÷49.00	18.7	-	64	30	34	-
Clay	49.00÷45.00	19.0	0.74	-	17	15	20
Silty sand	45.00÷40.00	18.7	-	64	30	34	-
Heavy Clay	40.00÷26.00	19.5	0.81	-	18	18	40

γ - unit weight at natural moisture content;

Ic - consistency index;

Dr – relative density;

E_{oed} - oedometric deformation modulus for a reference pressure of 200kPa;

Φ' - effective angle of friction;

c' - effective cohesion.

The groundwater level as indicated in the geotechnical report was around 70.50 m ASL for the upper aquifer and around 71.50 m ASL for the lower aquifer.

3 PILE TESTING INSTRUMENTATION AND PROCEDURE

3.1 *Piles instrumentation*

According to NE 045-2000, during pile loading, displacements of the loaded pile and also of the reaction piles were measured through dial gauges having a precision of 0.01 mm mounted on reference beam. The level of the reference beams was checked through topographical measurements at each loading step. The precision required for the topographical measurements was 0.1 mm. Also, horizontal displacements of the test piles were measured during loading tests through similar dial gauges.

In order to determine the loading transfer along the tested piles, the piles were instrumented – additionally related to the requirements of the standards and norms – for determining the stresses and displacements in different sections of the piles, considered as representative.

The test piles were instrumented with vibrating wire (VW) sensors consisting of: 30 concrete embedded strain gauges, a pressure cell at the toe and two displacement sensors measuring the relative displacement from the working platform to the pile head and to the pile toe (Figure 1).

The main technical specifications of the vibrating wire instruments used are given in Table 2.

Table 2. Specifications of the VW instruments

Specifications	Strain Gauges	Pressure cell	Displacement Transducers
Model	VWS-2100 (Geosense)	VWTPC-4000 (Geosense)	VWDT-5000 (Geosense)
Range	3 000 με	3000 kPa	150 mm
Resolution	1 με	±0.025% FS	±0.025% FS
Accuracy	±0.1…0.5%·FS	±0.1%·FS	±0.1%·FS
Nonlinearity	<0.5%·FS	<0.5%·FS	<0.5%·FS
Frequency range	850-1550 Hz	2000-3500 Hz	1650-2700 Hz

FS – Full Scale of the instrument

The strain gauges were positioned at 10 different levels at layer change and in the middle of the thicker layers of Sandy Silty Clay and Heavy Clay, according to the pile drilling logs at 68.3 (pile head), 64.1, 60.6, 55.0, 49.5, 45.8, 40.9, 36.6, 33.3 and 32.8 m ASL.

The total pressure cells were pre-casted in concrete and placed at the pile toe in order to measure the total pressure at the interface between pile and soil.

The displacement transducers were mounted in a rod extensometer with the head at the working platform and two groutable anchors placed at the pile head and toe.

Figure 1. Picture during the installation of the instrumented piles

All the measurements were carried out with two automatic data loggers Model DT2040 (Geosense/ rst instruments), being able to simultaneously record 40 vibrating wire sensors each, assigning a 1 minute recording interval for all the installed sensors.

3.2 Testing programme

According to the design elaborated in correspondence with NE 045-2000, the provided axial compressive loading for the pile tests was in steps up to the maximum loading Q_{max}=16.5 MN. After performing the first test, it was noticed that the tested pile had a significant bearing capacity backup. So, considering the fact that, after verifying the anchor system (beams and piles), the result was that it is able to bear an 18 MN loading, the second test was continued up to Q_{max}=17 MN, reaching the lift of the loading presses, and the third test was continued up to Q_{max}=18 MN, the designed capacity of the anchor system.

Nine loading cylinders having 2.2 MN capacity each were operated simultaneously through a distributor from a single hydraulic pump (Figure 2). The load was applied and maintained constant manually. At each loading step, displacement was measured at the time intervals prescribed by the norm, respectively 0', 2', 5', 10', 15', 20', 25', 30', 40', 50', 60' until reaching the displacements stabilization criterion: the mean settlement of the pile at 20' interval is lower than 0.1 mm.

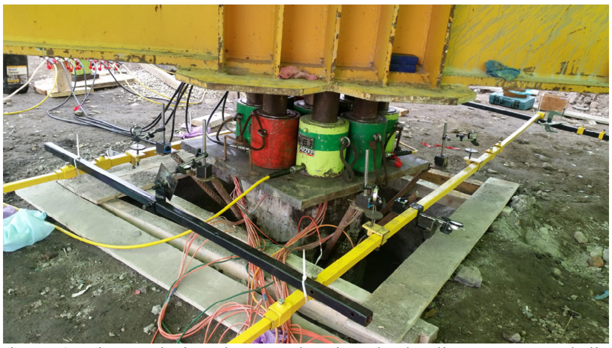

Figure 2. Picture during the test showing the loading system and dial gauges

Failure of the pile is considered when the pile settlement is higher than 10% of the pile diameter or if stabilization of the pile settlement is not reached during 24 hours from applying the last load step.

Based on the data recorded during the tests and on the classic measurements, variation charts of loading depending in time, pile settlement (displacement at the working platform level) in time and settlement (displacement at the working platform level) stabilized with the loading were elaborated.

4 MAIN EXPERIMETAL RESULTS AND INTERPRETATION

4.1 Vertical displacements measurements

For all the three tests performed, the recorded values of the stabilized displacements at the maximum load step were below the threshold value of 1/10 d=80 mm indicated in NP 045-2000 for pile failure.

Taking into account the fact that, out of practical reasons, the measuring equipment and the reference beams were located in an area influenced by the test and anchor piles, the measurements performed on dial gauges, topographical measurements were carried out concomitantly with the displacement measurements on the dial gauges, after stabilization of each load step, on the markers installed on the reference beams during the tests and being related to a point further from the test area considered fixed.

Brief results of the displacements measurements carried out during the loading tests directly measured on dial gauges (Raw) and corrected through topographical measurements (Corr) are presented in Table 3 and in Figure 3.

Table 3. Vertical displacements measurements results

Test no.	Maximum load	Settlement at maximum load		Settlement after unloading
		Raw	Corr	
Test 1	16.5 MN	37 mm	31 mm	11 mm
Test 2	17 MN	39 mm	39 mm	19 mm
Test 3	18 MN	48 mm	43 mm	27 mm

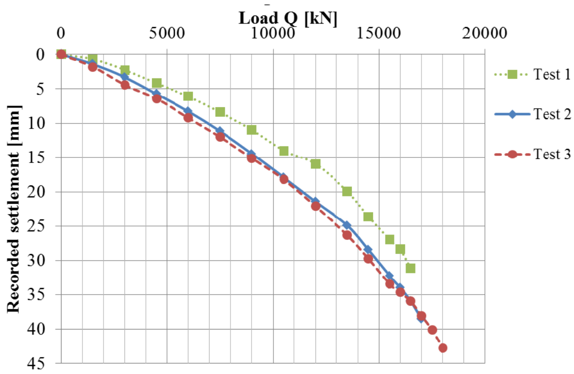

Figure 3. Load displacement curves resulted from the measurements during loading tests

4.1 Results from the VW instruments

Out of the three gauges placed in each section, two were placed opposite to each other, in order to calculate the compressive stress by their average and the third gauge was used for validating the range of the recorded strains. In Figure 4, Figure 5 and Figure 6 is given the mean for the two opposite strain gauges in the case of the third test pile, after excluding the levels with missing gauge of abnormal results.

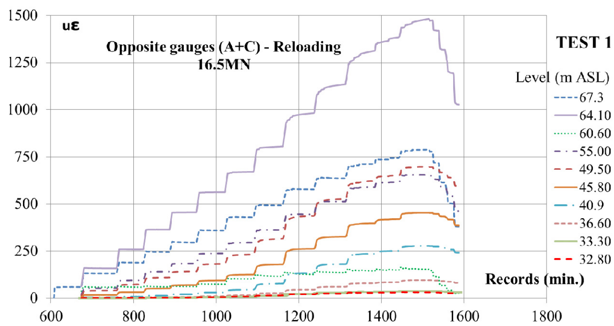

Figure 4. Variation diagrams of the strain recorded through the strain gauges in time (mean of the two opposite strain gauges) – Test 1

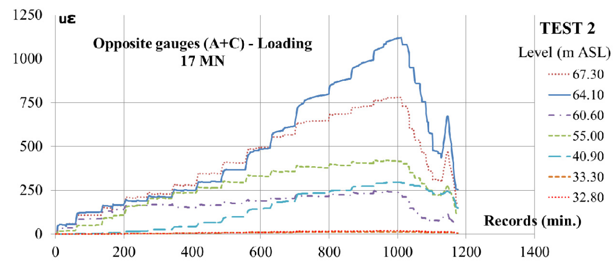

Figure 5. Variation diagrams of the strain recorded through the strain gauges in time (mean of the two opposite strain gauges) – Test 2

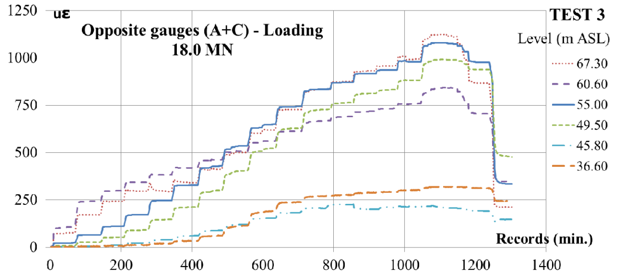

Figure 6. Variation diagrams of the strain recorded through the strain gauges in time (mean of the two opposite strain gauges) – Test 3

Thea measured strain gives the stress in the pile section, by multiplying with the concrete deformation modulus. Since the deformation modulus of the concrete is not constant, but is dependent on the applied loading, E-ε curves were determined and used based on the measurements of the strain gauges located in the first section (level 67.30) where the value of the strain is known (theoretically, equal to the value of the applied force) for different loading steps (Fellenius 2001).

In order to obtain the axial force in a given section of the pile, the stress obtained is multiplied by the pile section area. Also, the area of the section considered in the processing has taken into account the quantity of grouted suspension on the shaft at each level at which strain gauges were installed.

It was observed that the pile was loaded with bending moment also in the upper sections, fact resulted from the horizontal displacement measurements on the dial gauges and by the direct results of the strain gauges measurements. Thus, the processing was performed by averaging the values resulted only from gauges installed opposite to each other, in the same section and at equal distances related to the centre of the section. If one of these gauges was damaged, the resulted values were removed out of the processing.

The change of the strain determines changes in the section stress. However, it is known that residual stresses exist in the monitored elements before their loading with exterior forces (Fellenius 2002). Therefore, the primary values resulted from the primary processing were corrected by considering the residual stresses. So, the envelope curve of the pile loading was constructed (for the loading step of 16.5 MN for all three tested piles), therefore resulting an estimation of the real stresses extracting the residual stresses (Figure 7).

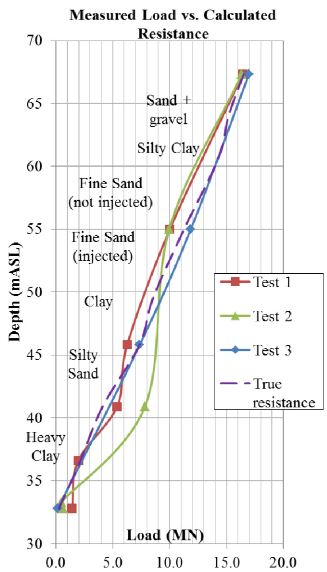

Figure 7. Variation of the axial stress in the tested piles and of the real axial stress as envelope by extracting the residual stresses

5 CONCLUSIONS

The piled foundations solutions for high buildings may be expensive. For this reason, the optimization of the solution by studying the load transfer from the pile to the soil, keeping the same safety level, is very important.

The piles were drilled under the protection of bentonite slurry and with improving the strength and deformability characteristics through shaft grouting leading to more productivity than in the case of drilling with protective tubing. The tests have revealed much higher bearing capacities than those resulted based on the preliminary design through prescriptive methods. This is justified by the fact that the empirical prescriptive methods are, in general, very safe, also that the execution of the piles was correctly performed and in good quality conditions and, additionally, an improvement technology was used by shaft grouting.

For this design, it was provided three loading tests on fully instrumented piles with strain gauges along the pile length, pressure cell at the pile toe and displacement transducers. Pile head displacement was also measured while loading. This allowed to determine the stresses within the piles, evaluating the load transfer to the soil through the entire length of the pile depending on the lithology and depth.

The detailed measurements and the instruments in the test piles which allowed a more realistic processing, as well as the advanced design performed based on it have led to optimizing the dimensions of the foundation piles in relation to the previous design phases. The length of the piles was reduced from 35 m to 22 m and from 14 m to 12m, respectively. The pile diameter was not reduced especially because it would have been difficult to assess the shaft grouting contribution for different diameters. This optimization has as advantage, besides reducing the costs and execution time, keeping deformability conditions considered when dimensioning the structure.

6 ACKNOWLEDGEMENTS

The present study was conducted with great involvement of the design team of geotechnical works and with important help from the structural works POPP & ASOCIATII CONSULTING ENGINEERS, who responded promptly, verifying the study and offering solutions and details to account for the test report. Also, the Special Foundations Contractor OCTAGON CONTRACTING & ENGINEERING gave full support during installation and measurements, as well as for processing all the data. Therefore, we thank all of the above for their support.

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