

PhD THESIS

SUMMARY

CONTRIBUTIONS TO IMPROVEMENT OF LARGE DIAMETER PILE RESISTANCE CALCULATION, AXIALLY LOADED, BASED ON IN SITU LOAD TESTS

PhD student:

Dipl.-Eng. Ion RĂILEANU, M.Sc.

Scientific coordinator:

Prof. Univ. Honored Ph.D. Eng. Anatolie MARCU

Corresponding member of the
Academy of Technical Sciences in Romania

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1. Introduction

In the last years in Romania, numerous buildings with many above-ground levels (over 50-100 m high) and industrial constructions have been designed and constructed, which are founded on deep foundation elements. In the design of the foundation systems were considered as deep foundation elements especially large-diameter bored piles, barrettes, end bearing piles (embedded in the rock) and even piles with enlarged base. Under these conditions, both the realization of the load tests on these deep foundation elements and the geotechnical calculation becoming important aspects regarding the assurance of the resistance and stability of the higher and higher structures.



Figure 1. Beginning research for his doctoral thesis, Iasi 2007

1.1. The purpose and main objectives of research

The general objective of the thesis is improving the resistance analyses of foundation piles based on in-situ load tests and detailed analysis of the static load

test procedures to simplify testing instrument piles. The aim of the thesis is at least to achieve the following specific objectives:

1. Understanding and quantifying the processes and parameters of concrete that can influence the results of load test piles instrumentation;
2. Detailing the calculation for determining the transmission of loads to the foundation ground for axially loaded piles which transmit the load both on the shaft and on their base;
3. Detailing interpretation of data collected from instruments used in instrumented load tests for designing of the foundation systems;
4. Comparing the obtained results with the usual values found in the current literature.

The results of this thesis can be used to specific guidelines, norms or regulations such as:

- inclusion of the research results in the new revisions of the Regulation on load testing of piles indicative NP 045-2000 [29], or in the Norm for the geotechnical design of foundations on piles, indicative NP 123-2010 [30];
- implementation in the forthcoming review of the National Annexes to Eurocode 7 [46] of a database on the values of specific parameters determined at national level (NDP - Nationally Determined Parameters) to meet the strategy of CEN (European Committee for Standardization) revision of Eurocode rules.

1.2. The structure of the doctoral thesis

This research paper is structured in seven chapters: Introduction, Current situation on national and international level, Theoretical elements, From strains to efforts for determine the lateral friction resistance and base pressure on test piles, Comparison of in-situ measurement results with calculations of strength and deformability of axially loaded piles, Conclusions and Bibliography.

2. Current situation on national and international level

2.1. Types of load tests/ monitoring works

From the point of view of the types of load tests, two main types can be distinguished, namely the conventional load (top-down load), respectively the Osterberg test (Figure 2) [31].

The usual type test, regulated by the norm NP 045-2000 [29], ASTM D1143M-07 [5] and ISO 2277-1:2018 [24], in which the load is applied by hydraulic jacks on the pile's head, presents numerous limitations [21] related to: limited load capacity, heavy reaction system, danger of accidents etc.

The Osterberg bi-directional test [32] incorporates a hydraulic cylinder (Osterberg cell) located at the base (Figure 4) or in the body of the test pile (Figure 3), so that the limit resistance of the two segments delimited by the hydraulic cell to be approximately equal.

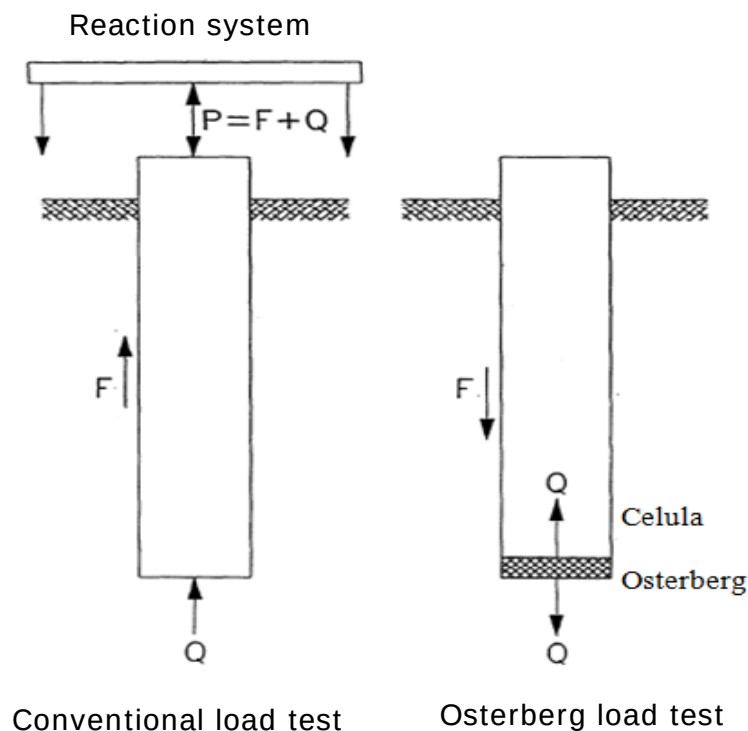


Figure 2. Comparison of loading schemes for the conventional test (classic) and the Osterberg type test

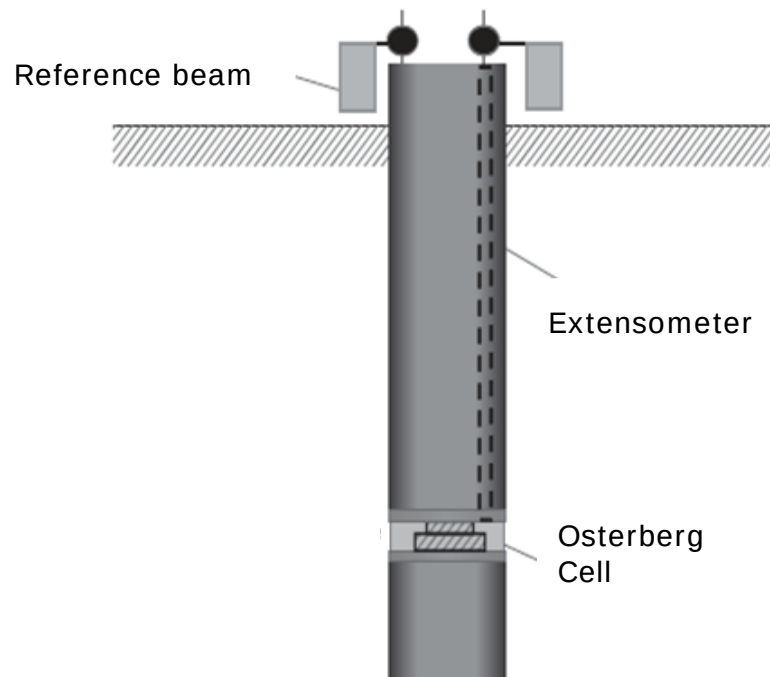


Figure 3. Schematic diagram for the Osterberg test



Figure 4. O-Cell hydraulic cells for Osterberg type test for LT2 (left) and LT1 (right) tests

For this test, the load is applied in increments of 5% to 15% of the maximum pre-evaluated sample load. The value of this load is usually $1.5 \times$ the estimated ultimate resistance.

When testing by the *classical (conventional) method*, a reaction system with anchor piles or drilled anchors is used. Figure 5 shows the reaction system with

anchor piles. In this case the load is applied to the pile's head from top to down and the pile's settlements are measured at the level of his head. [51]

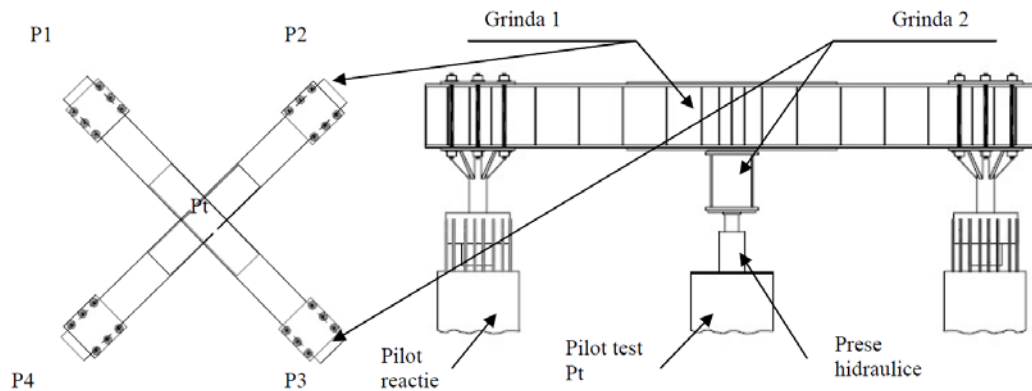


Figure 5. Detail of the reaction system connected with the reinforcement of the reaction piles - the classical (conventional) method for the LT5-LT9 tests

2.2. Test pile measuring / instrumenting devices

Precision measuring devices are required in a test polygon for the instrumented load test piles in order to be able to assess the behavior of the test pile during the test.

For the used equipment and instrumentation during the load test, the conformity manufacturer's certificates stating the accuracy of the records and, where appropriate, the metrological checks and calibration certificates for the devices shall be attached to the test report.

2.2.1. Dial gauges at the level of the pile's head



Figure 6. Dial gauge with magnetic installation system and analog clock (left), with digital display (middle) and electronic transducer (right)

2.2.2. Optical level for checking the stability of the reference beams

According to EN ISO 22477-1: 2018 in the tests there must be a second reading system that can be represented by a precision optical level. On this second system, measurements are made only at the beginning and end of each loading step.



Figure 7. Optical level for checking the stability of the reference beams

2.2.3. Extensometers in the test pile

The extensometers consist of a continuous steel rod with an end anchored in concrete at known depths, protected at concreting by a steel pipe through which the steel rod must be able to slide during loading. The upper end of the steel rod is accessible at the level of the test pile's head, and the relative displacement to the pile's head is measured using a dial gauge.

2.2.4. Extensometer / tasometer

They are installed in the foundation ground to measure the settlement gradients under the pile base, induced by the load applied on the test pile.

2.2.4.1. Tasometric column with anchored rods / extensometer

From a constructive point of view this type of instrument is similar to the extensometer described in chapter 2.2.3 and may consist of one or more anchored rods.

2.2.4.2. Extensometer with flexible tube and magnetic rings

This type of extensometric column consists of two plastic tubes, a fix rigid tube on the inside and a flexible tube on the outside on which magnetic rings are installed. The rings move in solidarity with the foundation ground.

2.2.5. Electronic strain gauge with vibrating wire strain gauges

Vibrating wire strain gauges are used to record the variation of strains, which allow the evaluation of the stress in steel or concrete structures (e.g. steel struts, test piles etc.).

Embedded Strain Gauges

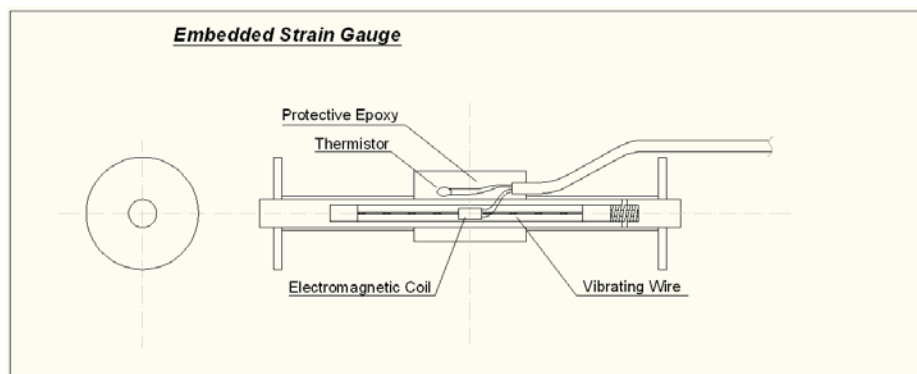


Figure 8. Embedded strain gauges constructive sketch [9] and example of installation on the reinforcing cage of a prefabricated pile (construction site on The Netherlands)

Sister Bars

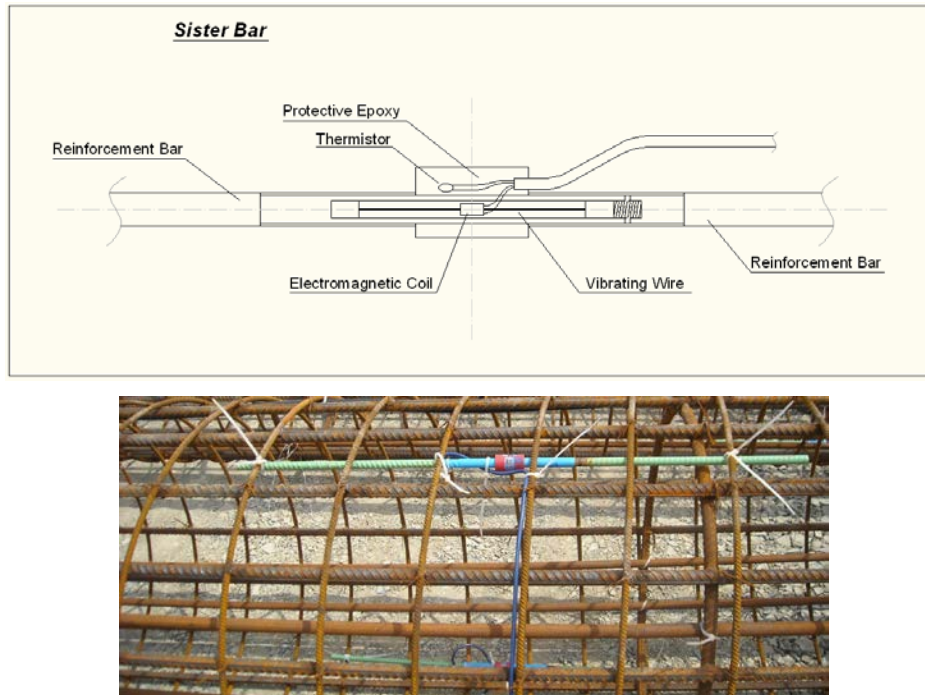


Figure 9. Sister bar constructive sketch [9] and example of installation on the reinforcing cage of a pile (load test site in Constanța, Romania – LT5)

Arc-weldable Strain Gauges

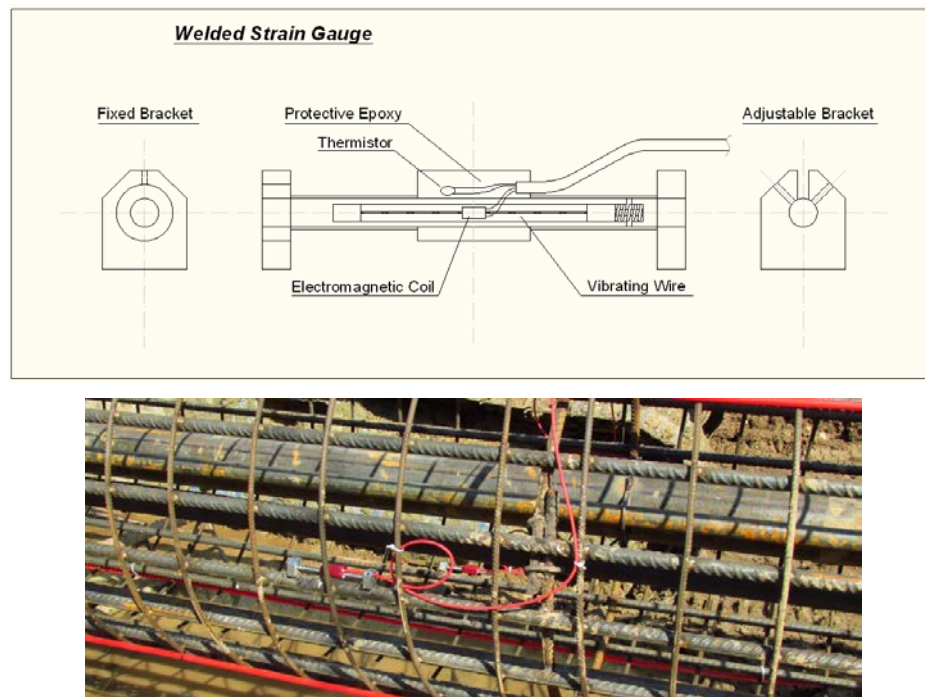


Figure 10. Arc-weldable Strain Gauge constructive sketch [9] and example of installation on the reinforcing cage of a pile (load test in Bucharest, Romania – LT9)

Usually, the measuring range is $\pm 1500 \mu\epsilon$, but it can reach $\pm 5000 \mu\epsilon$, and the measuring accuracy is very high being $\pm 0.1 \dots 1.0\%$ of the measuring range. For example, for a strain gauge with a length of 150 mm the measuring accuracy is 4.5×10^{-4} mm.

2.2.6. Thermistors

These types of devices are usually used to monitor the temperature of other sensors (in this case the strain gauges with vibrating wire) so that a temperature correction can be applied [15].

2.2.7. Pressure cells

Pressure cells can be hydraulic, with resistive electrical sensors and with vibrating wire sensors. Depending on the desired pressure range, the pressure / force cells can currently have capacities of up to approx. 4000 kN.

2.2.8. Data-logger

The data-logger device is the center of the entire monitoring system

2.2.9. Inclinator

The inclinometer is a device that can measure the inclination / rotation of a structure (pile, diaphragm wall etc.). Inclinator can be installed in inclined foundation piles to determine the horizontal displacements and bending of the piles when applying axial loads.

3. Theoretical elements

3.1. Concrete properties and deformative processes

3.1.1. Concrete resistance

According to SR EN 1992-1-1: 2004 [45] the resistance classes are based on the characteristic resistance measured on cylindrical samples

$$f_{ck} = f_{ck, cyl} \quad (1)$$

If it is necessary to specify the resistance of the concrete at ages other than 28 days, the following relationships shall be used:

$$f_{ck}(t) = f_{cm}(t) - 8 \text{ (MPa)} \quad \text{for } 3 < t < 28 \text{ days} \quad (2)$$

$$f_{ck}(t) = f_{ck} \quad \text{for } t \geq 28 \text{ days.} \quad (3)$$

unde rezistența medie la vârsta t se evaluează pe baza următoarei relații

$$f_{cm}(t) = \beta_{cc}(t) f_{cm} \quad (4)$$

$$\beta_{cc}(t) = \exp \left\{ s \left[1 - \left(\frac{28}{t} \right)^{\frac{1}{2}} \right] \right\} \quad (5)$$

3.1.2. Modulus of elasticity (Young) E_c of the concrete and stress strain curves

It is known that concrete does not have a linear elastic behavior under loads for compressive stresses and tensile stresses (Figure 11).

The concrete modulus of elasticity depends on the elasticity modulus of its constituents (type and cement quantity, quality and quantity of aggregates).

The evolution of the modulus of elasticity over time can be estimated by:

$$E_{cm}(t) = (f_{cm}(t) / f_{cm})^{0.3} E_{cm} \quad (6)$$

in which $E_{cm}(t)$ and $f_{cm}(t)$ are the values at age t (days) and E_{cm} and f_{cm} values obtained at 28 days. [45]

The stress-strain relationship for nonlinear structural analyzes is

$$\frac{\sigma_c}{f_{cm}} = \frac{k\eta - \eta^2}{1 + (k - 2)\eta} \quad (7)$$

where, $\eta = \epsilon_c / \epsilon_{c1}$

ϵ_{c1} is the deformation at maximum stress, as indicated in table 3.1 of SR EN 1992-1-1: 2004 [45].

$$k = 1,05 E_{cm} |\epsilon_{c1}| / f_{cm}$$

Formula (7) is valid for $0 < |\epsilon_c| < |\epsilon_{cu1}|$ where ϵ_{cu1} is the nominal value of the ultimate deformation.

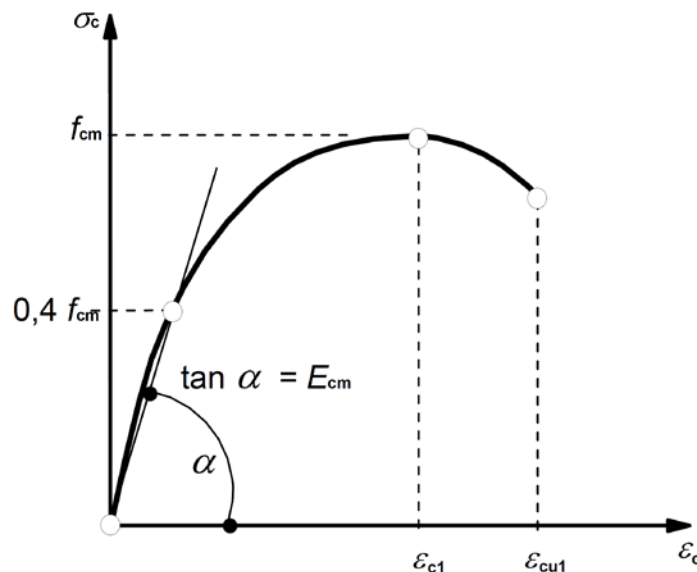


Figure 11. Schematic representation of the stress-strain relationship

3.1.3. Determination of the concrete elasticity modulus in the laboratory

To determine the concrete elasticity modulus in the laboratory it can be used one of the following standards STAS 5585:1971 [49], SR EN 12390-13:2014 [48], ASTM C469:2014 [4] and ISO 1920-10:2010 [23].

3.1.4. Concrete creep and shrinkage

In theory any concrete sample is affected by shrinkage and creep. Concrete shrinkage is a reduction over time in the volume of the concrete sample due to the hydration of the concrete (moisture and chemical-physical changes) [1], and creep is a nonlinear variation of the sample volume at constant stress and temperature. Creep and shrinkage of concrete are related to the physico-chemical processes of hydration of concrete, but the shrinkage of concrete is independent of the effort applied to the sample, while creep depends on the effort applied. [2].

Strain gauges recordings and associated analyzes to understand the behavior of the test pile may be affected by deformations due to concrete shrinkage and creep.

3.1.5. Concrete temperature

Reinforced concrete expands linearly when the temperature varies between 0° C and 60° C. Like any other parameter, the coefficient of thermal expansion α_c

varies depending on the cement / aggregate ratio, the type of aggregates used, humidity, additives used, etc. This coefficient usually varies between $9 \cdot 10^{-6} \text{ }^{\circ} \text{C}^{-1}$ and $13 \cdot 10^{-6} \text{ }^{\circ} \text{C}^{-1}$ [28].

Thus, the following correction must be applied to determine the strains

$$\mu \varepsilon = (L_1 - L_0) + (T_1 - T_0) \cdot (CF_1 - CF_2) \quad (8)$$

3.1.6. Observations on the deformative properties and processes of concrete

When analyzing the test loads, the analytical determination of the modulus of elasticity of the concrete does not provide a high degree of accuracy for determining the variation of the modulus of elasticity with the load. The method of determining the modulus of elasticity of concrete in the laboratory requires a relatively fast loading of samples.

For the analysis of the test pile behavior, it is thus necessary to determine in situ the variation of the concrete modulus of elasticity even during the performance of the load test procedure according to chapter 4.4.

3.2. The theory of elasticity. Basic equations

In this chapter is briefly presented the theory of elasticity and its basic equations.

3.3. Boussinesq problem

This chapter briefly presents the Boussinesq problem.

3.4. Punctual loading

In this chapter is briefly presented the classical solution, described by Boussinesq - the problem of a point load P on an elastic half-space $z > 0$.

3.5. Distributed load

Starting from the Boussinesq theory in this chapter are briefly presented the particular case of uniformly distributed load on a circular surface.

3.6. Mohr-Coulomb elastic-plastic model

In this chapter, the Mohr-Coulomb model is briefly presented.

3.7. Nonlinear Hardening Soil model

This chapter briefly presents the Hardening Soil model, which provides a more realistic description of the stress-strain of soils for stress levels under normal loading conditions (below the level of the ultimate limit state).

4. From strains to efforts for determine the lateral friction resistance and base pressure on test piles

4.1. Test pile instrumentation. General aspects

In this chapter I present some general aspects for the instrumentation of test piles, such as: the number of strain gauges, the optimal distance between them, instrumentation protection measures.

4.2. Transformation of strains recorded in the stresses using the concrete modulus of elasticity determined by prescriptive methods or laboratory tests

From the difference between the forces in two consecutive instrumented sections results the resistance on the pile shaft between the two sections for each loading stage. (9).

$$q_{sm} = \frac{F_i - F_{i-1}}{A_{s,i}} = \frac{E_c \cdot A \cdot (\varepsilon_i - \varepsilon_{i-1})}{A_{s,i}} \quad (9)$$

The measured base pressure is defined by the ratio of the measured stress at the base of the instrumented pile and its area.

$$q_{bm} = \frac{F_b}{A} = \frac{E_c \cdot A \cdot \varepsilon_b}{A} = E \cdot \varepsilon_b \quad (10)$$

4.3. The importance of the variation of the pile's elasticity modulus on the transition from strain to stresses

In this subchapter I presented a particular case in which the variation of the elasticity modulus is important in the variation of basic components of the bearing capacity: lateral friction and base pressure.

4.4. Transformation of recorded strains in the stress using the variable elasticity modulus of the test pile

In the case of reinforced concrete piles, the modulus of elasticity to be considered in the calculation is the combined modulus of reinforcement and concrete, each proportional to the area and its modulus of elasticity.

$$E_{comb} = \frac{E_s A_s + E_c A_c}{A_s + A_c} \quad (11)$$

In order to determine the variable modulus of elasticity of the reinforced concrete section, is established an instrumented level with strain gauges immediately below the pile's head, where the effort in the pile is the same as the force applied. It will result, for each loading stage, respectively state of strains, the reinforced concrete section modulus of elasticity of the loaded test pile [19].

Based on the load tests detailed in Chapter 5, it was observed that the concrete in the pile may be influenced by the creep effects previously described in Chapter 3.1 even if charging is done in a short time. The effect is even more significant when starting the load test at an early age of the concrete. If the creep effects are not corrected in the load analysis after applying the steps to transform the strains into stresses and then into resistance of the pile, negative shaft resistance values may be obtained, which is impossible.

Table 1 presents a comparison of the elasticity modules resulting from this method [19], with values from SR EN 1992-1-1: 2004 according to chapter 3.1.2 or with values obtained in the laboratory according to chapter 3.1.3.

Table 1. Values of the elasticity modulus

	Values from table 3.1 of SR EN 1992-1- 1:2004 (28 days)	Values from Table 3.1 of SR EN 1992-1-1: 2004 and Equation (6) (14 days)	Mean value from two laboratory determinations (14 days)	Values resulted at the pile's head during the load test
E_{cm} (GPa)	36,00	36,02	36,07	-
E_{comb} (GPa)	41,77	41,79	41,84	43,99 ... 29,20

**Determination of the tangent elasticity modulus for the pile
reinforced concrete**

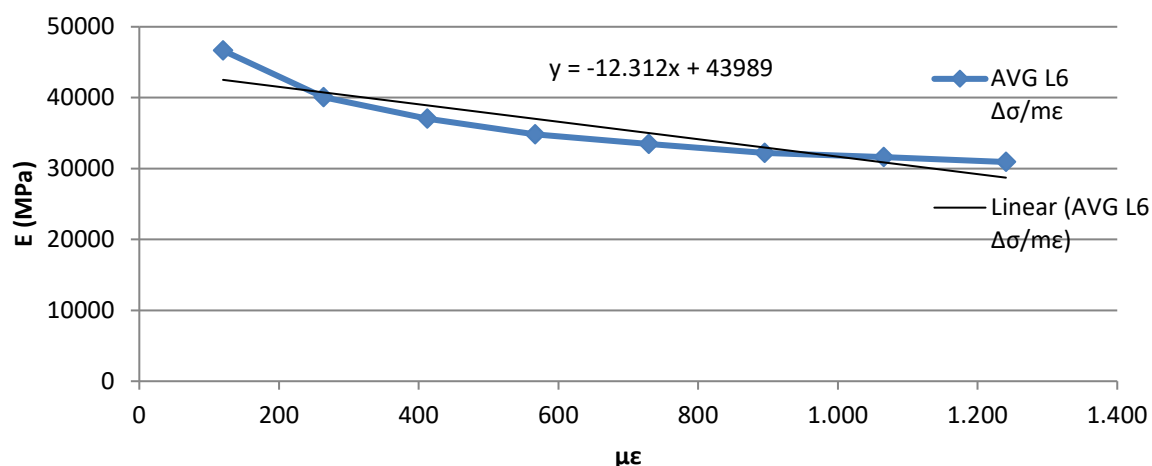


Figure 12. Diagram of the tangent modulus of elasticity for the instrumented section at the head of the LT6 test pile

5. Comparison of in-situ measurement results with calculations of strength and deformability of axially loaded piles

This chapter analyzes the results of the instrumented tests performed on test piles.

5.1. Description of the loading procedure for instrumented test piles and methodology for collecting records

For all analyzed tests the application of the maximum load is done in steps, usually the maximum load is reached with at least 8 loading steps. The unloading of the test pile is performed in at least 4 loading steps.

Especially in the case of instrumented piles for each loading step, it is recommended to hold for at least 60 minutes in order not to interfere with strains generated by creep. The stabilization criterion for the application of the next step is 0.1 mm in a 20 min interval according NP 045:2000 [29] and ISO 22477-1:2018 [24]. In contrast, in the case of Osterberg-type tests, the loading step shall be maintained for at least 30 minutes and until the stabilization of the movements is considered to be <0.05 mm / 10 minutes or 1% of the total settlement in one hour. Maintaining a certain load step will not exceed three hours. [34]

Knowing the impact that can have concrete creep on specific strains and furthermore on transformation process described later on the chap. 4.4 it is highly recommended that the time difference between the applied loading steps does not exceed 3 hours.

5.2. Description of instrumented load tests

During the research I performed tests on nine test piles in different geotechnical conditions. In the execution of which the usual drilling technologies were used for these types of special foundation works.

5.3. Interpretation of pile load tests and comparisons with prescriptive methods

5.3.1. Piles embedded in a layer of semi-rocky rock

Having no other possibility to comply with the norm NP 123:2010 [30], for the calculation of the pile resistance according to the prescriptive method, the pile is considered as end bearing and the formulas indicated in chap. 7.2.4.1 is used

$$R_{c,k} = R_{b,k} = A_b \cdot q_{b,k} \quad (12)$$

The value of the base pressure is calculated with the empirical equation

$$q_{b,k} = \sigma_{cs} \left(\frac{t}{d} + 1,5 \right) \quad (13)$$

During the in situ load test, the pile was loaded up to a force of 12,000 kN, having a settlement of 21.5 mm at its head. The conventional failure of the pile, which is considered to be reached at a settlement of 10% of the diameter of the

pile base, would occur at a settlement value of 60 mm. The axial compression load test was performed 14 days after the pile construction.

In the LT6 test, a pile base pressure value of ≈ 7 MPa was measured (Figure 15), much lower than the above-calculated value of 183 MPa.

However, it can be seen that the measured value does not represent the maximum value of the rock layer resistance present at the base of the pile, the settlement of the pile base being relatively small compared to the diameter of its base.

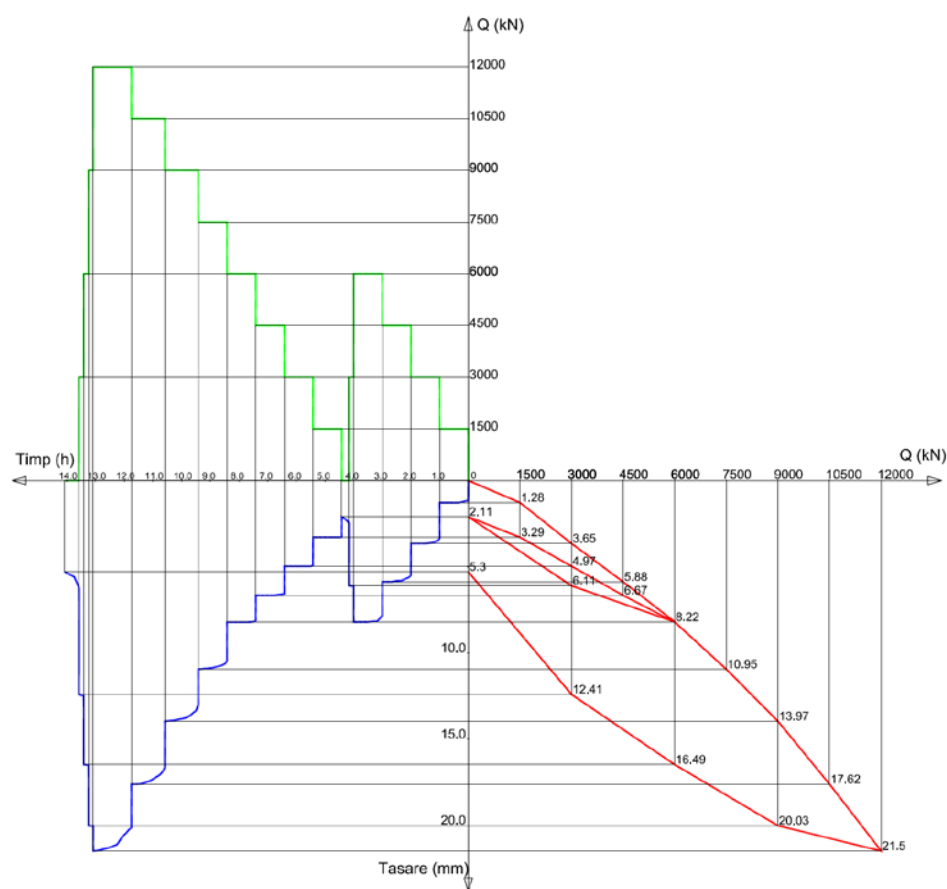


Figure 13. Load-compression-time diagram for the test pile LT6

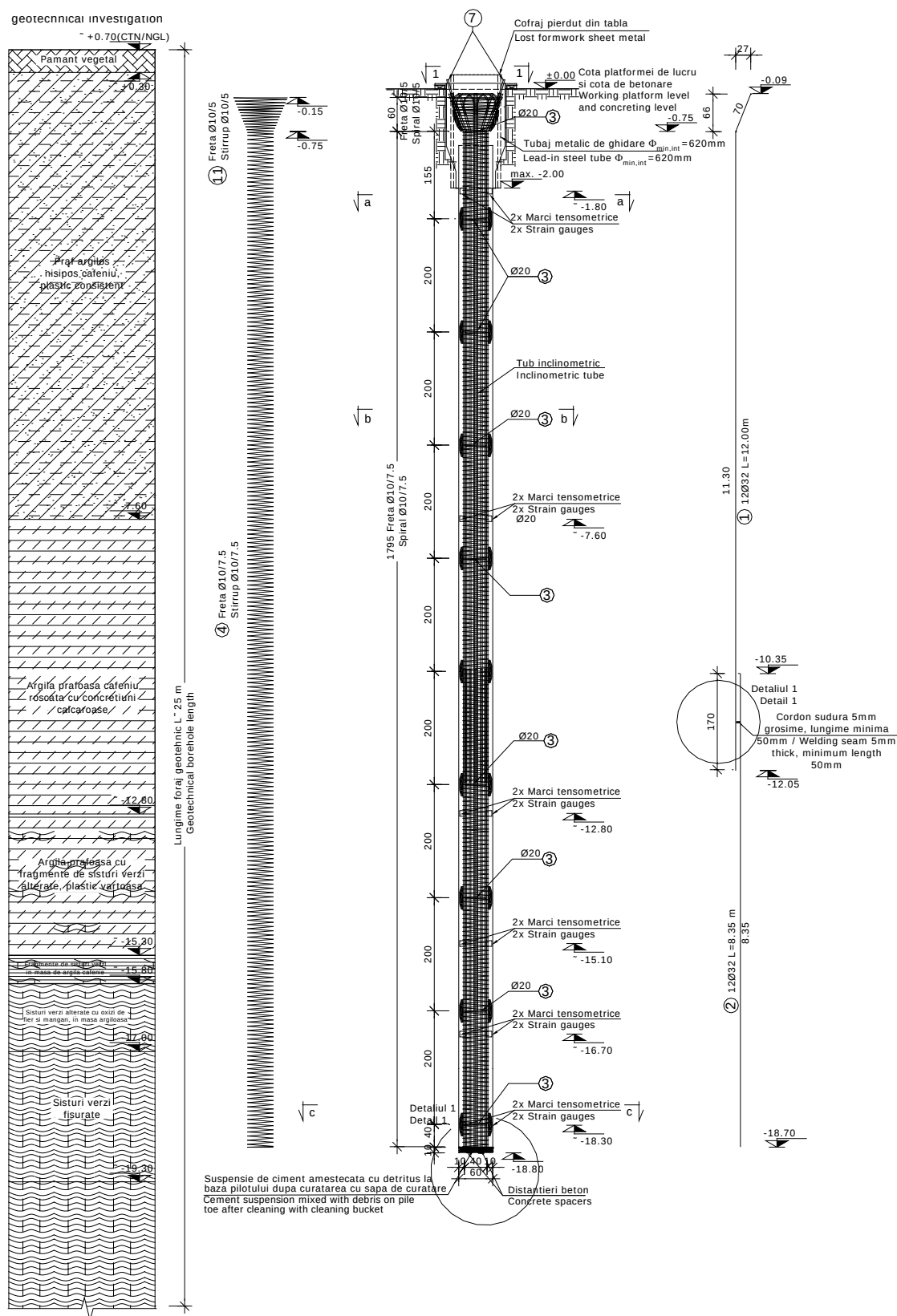


Figure 14. Test pile instrumentation LT6

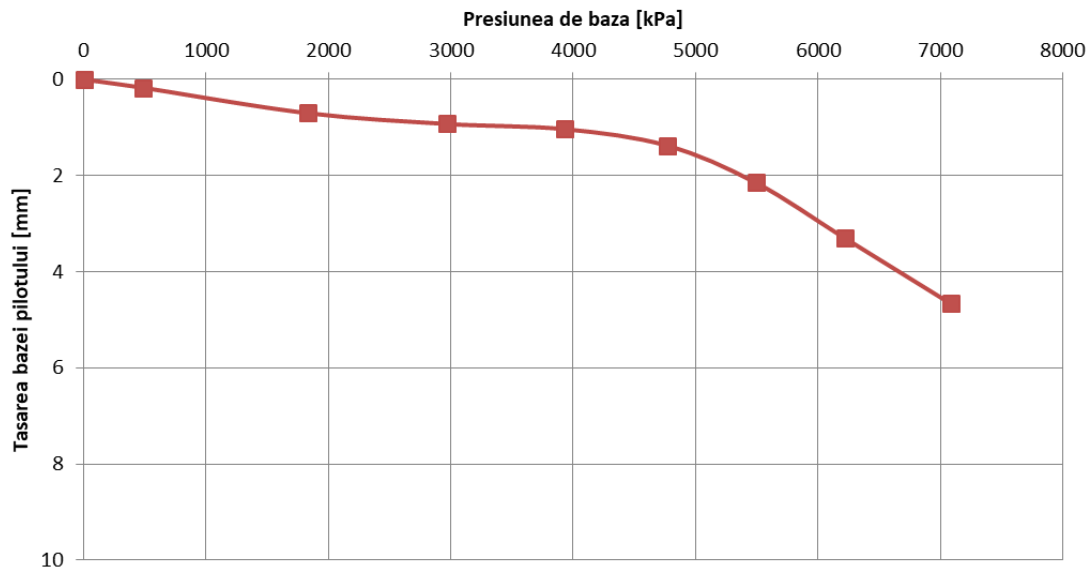


Figure 15. The pile unit base resistance by load test LT6

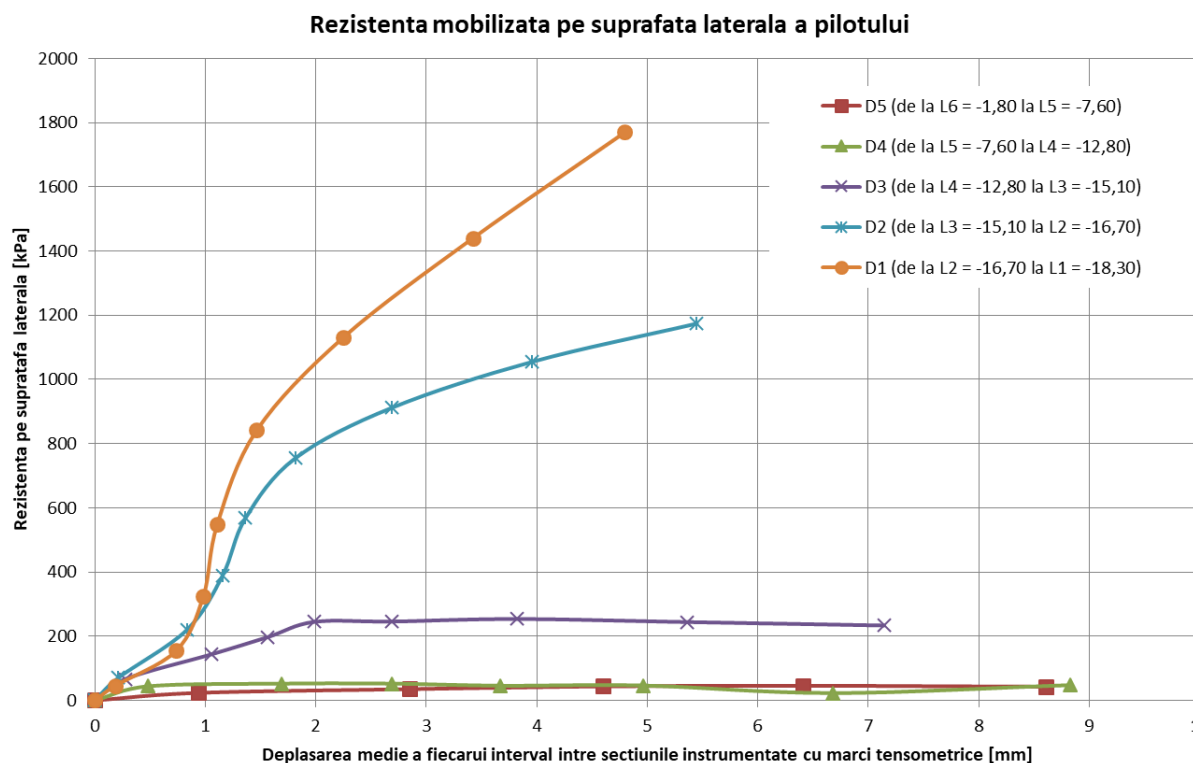


Figure 16. The pile unit shaft resistance LT6

The LT6 pile being an end bearing pile, it is observed that the value of settlement at the pile's head 21,495 mm for the maximum loading step contains

a significant component that represents the pile's shaft deformation of 16,795 mm, and the settlement of the pile base being only 4.7 mm.

From Figure 17 it can be deduced an important conclusion, namely that the elastic deformation of the end bearing pile body is often significant, and the lateral friction on the shaft of the pile in the layers above the rocky layer intervenes in the load-bearing capacity of the pile.

Therefore, in the case of end bearing piles, the percentage of the value of its own deformation at its head must be checked to confirm that the pile behaves like a friction pile and the lateral shaft resistance of the pile can be considered in the bearing capacity calculation.

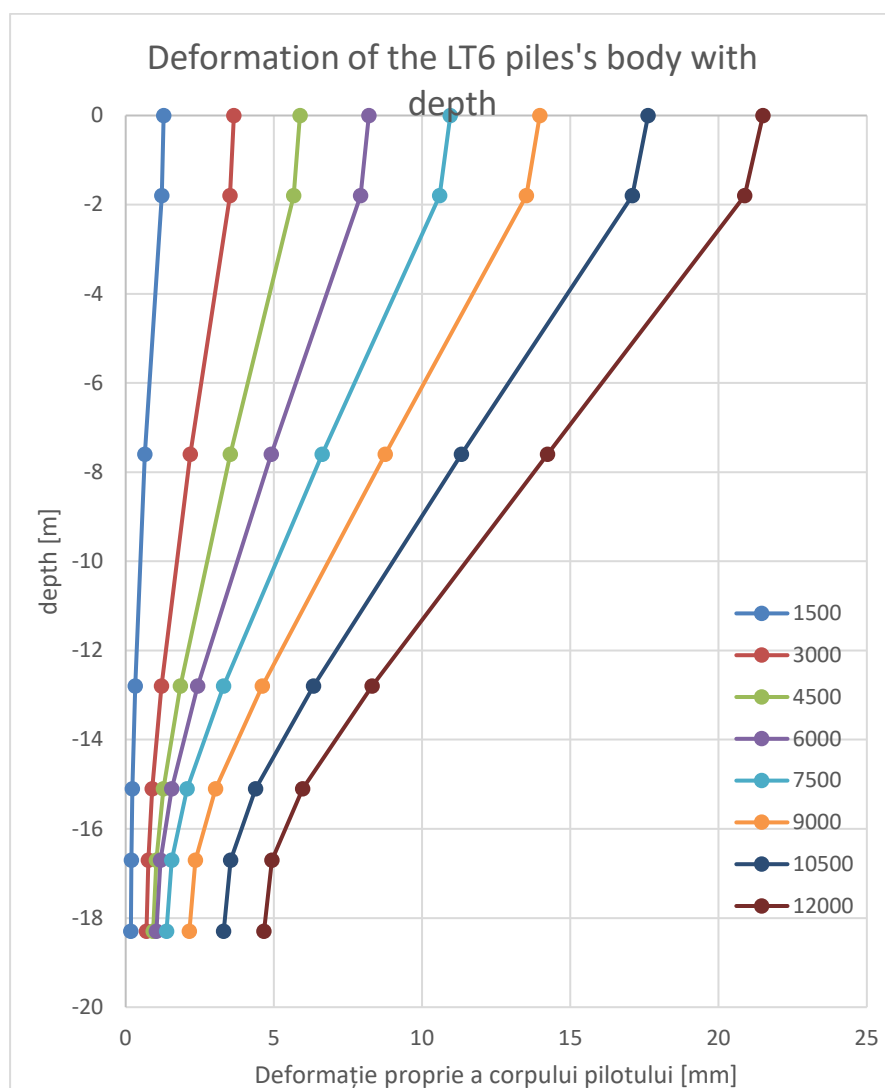


Figure 17. Shaft deformation variation diagram for each loading stage of the LT6 test pile

Under the same geotechnical conditions, I further analyzed the results of 5 instrumented test piles embedded in a rock layer with similar properties. The characteristics of the rock in which the piles are embedded are presented in detail below and summarized in Table 2. The characteristics of the load tests on the instrumented piles can be found in Table 4.

Table 2. RQD and average uniaxial compressive strengths determined in the Construction Materials Laboratory of TUCEB

Pile	RQD	Uniaxial compression strength (N/mm ²)	Remarks	Uniaxial compressive strength by Is50 tests (N/mm ²)	Remarks
LT1	0-40%			20,5	3 tests
LT2	0-19%	48,1	3 tests	40,2	8 tests
LT3	0-20%			72,6	12 tests
LT4	0-40%			27,0	1 test
LT5	0-80%	34,6	1 test	67,2	1 test
LT6	0-18%	28,0	1 test		

The layer of green schist of Neoproterozoic age, more pronounced altered at the top (RQD <25%), is the bedrock or geological foundation with a thickness of several hundred meters.

$$RQD = \frac{\sum \text{Core length} > 10\text{cm length}}{\text{core advancement depth}} \quad (14)$$

Given the degree of degradation (RQD <50%), the strongly wrinkled character of the schistosity planes (in the head of the layer the planes are oriented vertically), this type of rock can be classified as “weak rock”. [7].

The green schist is greenish-gray, it is hard, compact, apparently massive. The rock has a micro-cryptocrystalline structure, mainly granoblastic. The estimated mineral association is [feldspar-quartz-epidote-clinozoite-chlorite].

There is a very slight transformation of the rock in the area of the alteration crust. Films with $\text{Fe} \pm \text{Mn}$ hydroxides are sometimes observed on cracks.

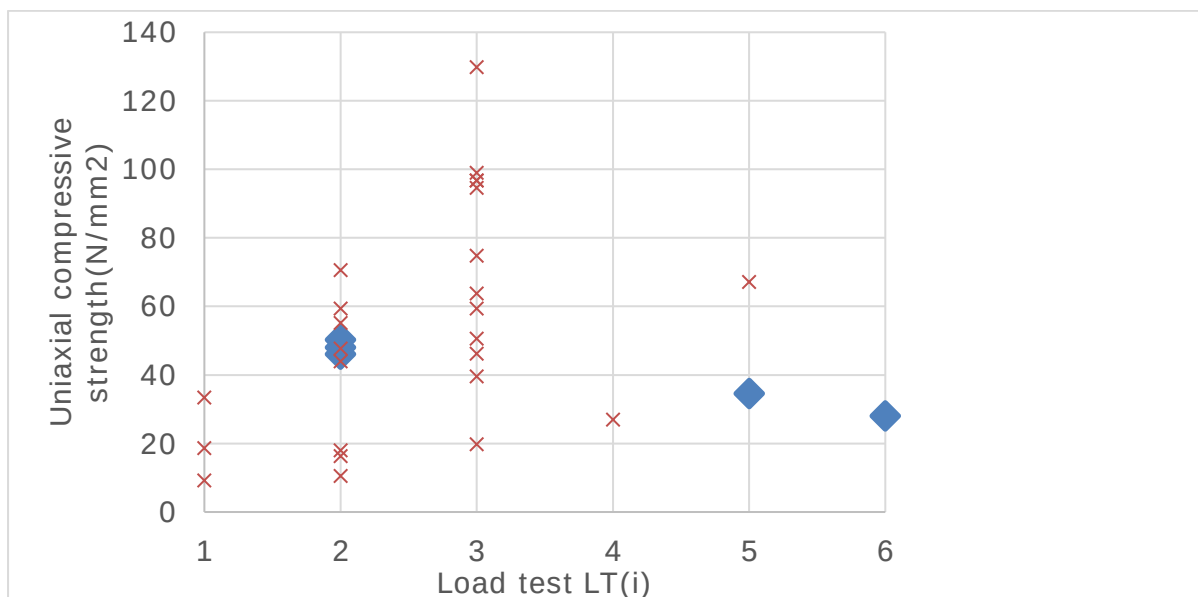


Figure 18. Distribution of uniaxial compressive strength values

Using the same equations from NP123: 2010 [30], presented above for the calculation of the end bearing pile resistance embedded in a rock layer (12), (13) the values presented in Table 3 are obtained.

Table 3. Characteristic base resistance ($R_{b,k}$) for tests LT1... LT5 according to NP 123: 2010 [30]

Pile / test	$q_{b,k}$ [MPa]	A_b [m ²]	$R_{b,k}$ [kN]
LT1	$20,5 \cdot (4,0/1,0 + 1,5) = 112,8$	0,785	88.550
LT2	$40,2 \cdot (4,0/1,0 + 1,5) = 221,1$	0,785	173.485
LT3	$72,6 \cdot (4,0/1,0 + 1,5) = 399,3$	0,785	313.450
LT4	$27,0 \cdot (5,0/1,0 + 1,5) = 175,5$	0,785	137.767
LT5	$34,6 \cdot (2,5/0,6 + 1,5) = 196,2$	0,2826	55.446
LT6	$28,0 \cdot (3,0/0,6 + 1,5) = 182,0$	0,2826	51.430

Table 5 presents and compares the results of tests performed on piles under similar geotechnical conditions (LT1... LT6).

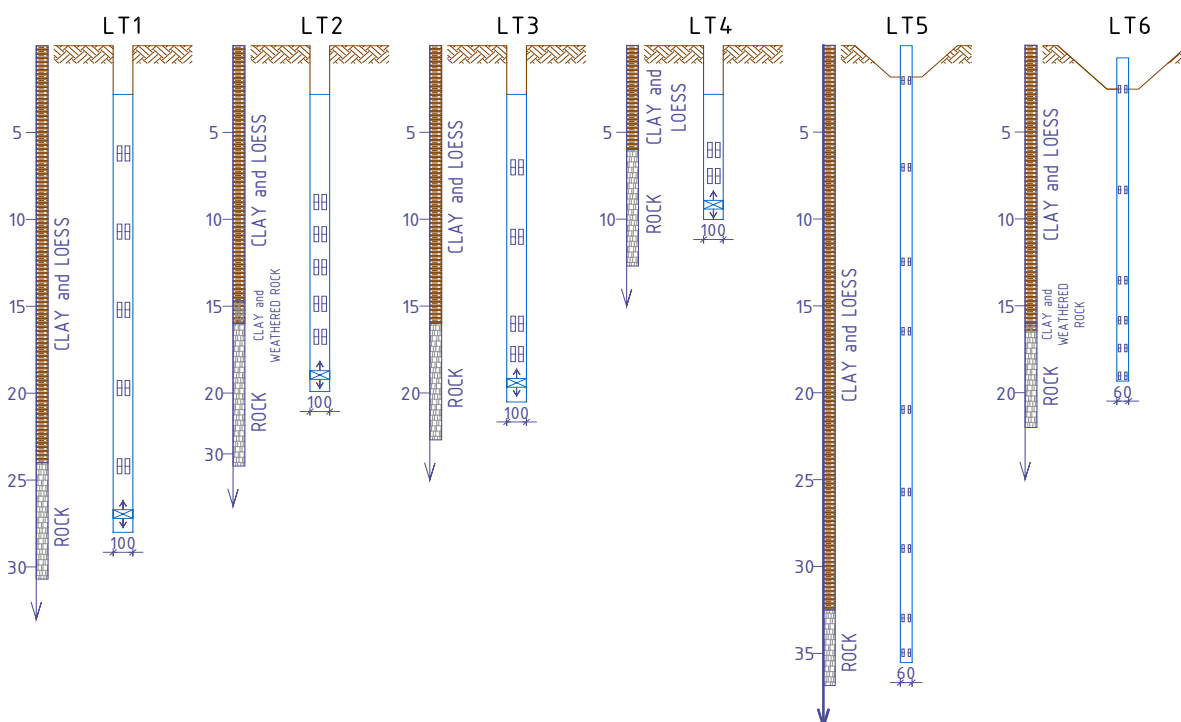


Figure 19. Instrumented test piles embedded in a rocky layer

Table 4. Characteristics of the test end bearing piles

Pile	LT1	LT2	LT3	LT4	LT5	LT6
Pile diameter and length	d=1,0 m L=25,2 m	d=1,0 m L=16,9 m	d=1,0 m L=17,7 m	d=1,0 m L=7,2 m	d=600 mm L=33,5 m	d=600 mm L=18,8 m
Test method	Osterberg	Osterberg	Osterberg	Osterberg	Conventional	Conventional
Duration from pile construction to load test performance	34 days	32 days	22 days	27 days	49 days	14 days

It can be easily observed that the base resistance on all the analyzed piles, embedded in the rock layer (Table 5), is significantly lower than the values calculated according to those presented in Table 3.

Table 5. Test results on test piles with the base embedded in the rock

Pile	LT1	LT2	LT3	LT4	LT5	LT6
Unit resistance on the shaft measured in the rock layer $q_{s,m}$ [kPa]						
Fragments of rock in the clay mass	215	-	-	-	-	250
Weathered rock	520	700	520	560	502	1.175
Compact rock	900	1.100	1.180	690	528	1.775
Base resistance at 5 mm settlement of the pile base $q_{b,m}$ [kPa]						
Base resistance	3.000	7.000	12.000	9.000	*)	7.200
Additional measures to ensure rock concrete contact	Without cementing the base		With cementing the base			

*) Due to the extensive length of the pile (33.5 m) during the load test, the base resistance on the pile was not mobilized for the maximum load of 12 MN applied to the top of the pile.

Figure 20 compares the mobilization curves of the unit base pressure on the embedded in the rock layer for all 6 analyzed piles.

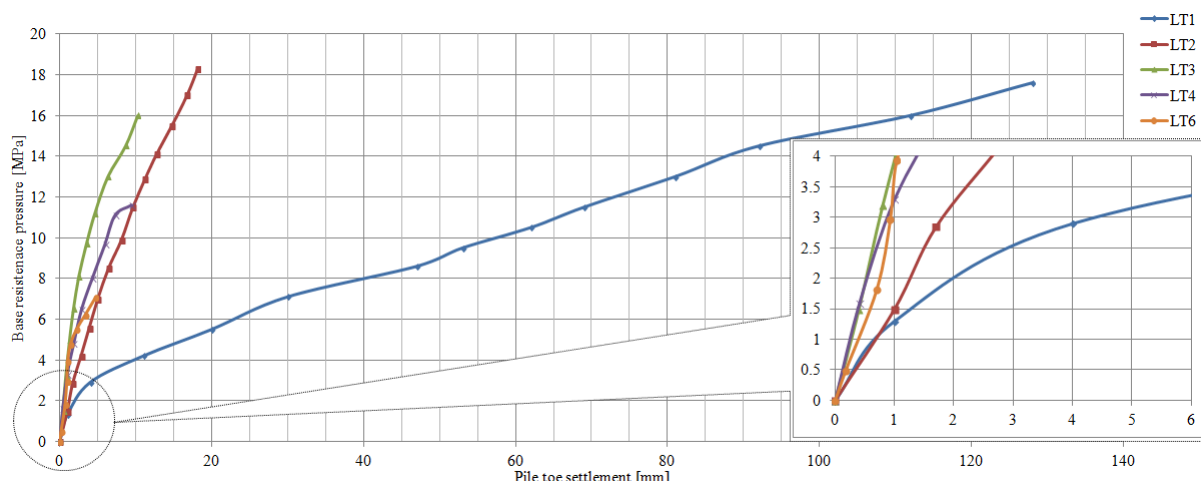


Figure 20. Unit base pressures on piles embedded in the rock and their settlement

Observing the behavior of the LT1 pile, I sought to identify the causes of a 13 cm settlement of the pile base embedded in the "rock".

Knowing the characteristic of the schist from the Dobrogea area (Figure 21), the first analyzed idea as a possible cause of significant settlements was a pronounced state of alteration of the rock.



Figure 21. Rock cracking state - green schist- in the Dobrogea area

The settlement of the end bearing pile base can be influenced by the degree of cracking of the rock layer in which the pile is embedded, in addition to the compressive strength of the rock. Thus, after drilling a pile, I introduced an inspection video camera at the base of the piles to identify the cracking state of the rock layer at the base of the pile.

Surprisingly I discovered at the base of the pile is a significant layer of detritus, as can be seen in Figure 22.

I tried to clean the pile with the special cleaning bucket, but the results were unsatisfactory..

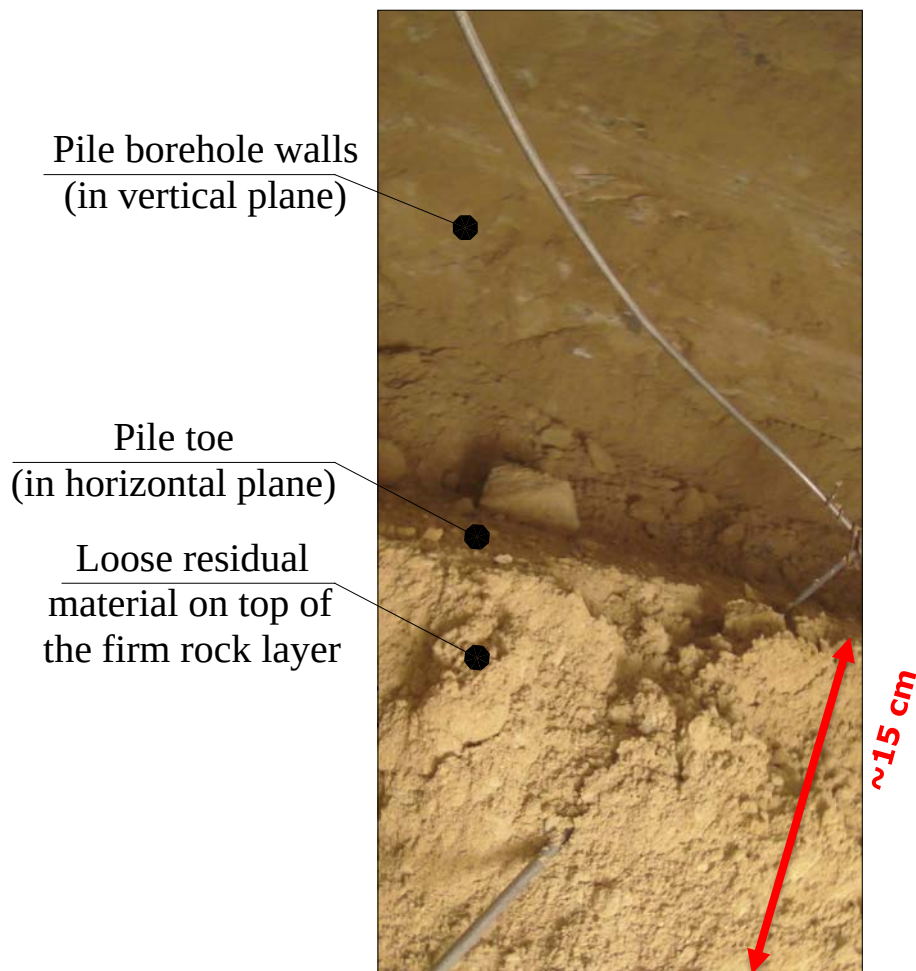


Figure 22. Pile base before cleaning



Figure 23. Pile base after installing the pile reinforcing cage

For the current practice it was used as an additional method to ensure the contact between the pile concrete and the rock layer, cementing the base of the

drilling with a cement suspension with a ratio of $w/c \approx 0.5$ and mixing it with the residual material left on the bottom of the borehole.

On the other hand, as in the case of the LT6 pile, it is highlighted that although the end bearing piles and their base does not settle significantly, at the level of the pile's head there are quantifiable settlements, generated by specific deformations of the pile reinforced concrete body.

In addition, for the LT5 test, although the pile is embedded in the rock, therefore it can be considered a tend bearing pile, in the in situ test, the compressive forces applied to the pile's head were fully taken over by the resistance on the shaft of the pile which crosses both the soil and the rock layers (Figure 24).

Thus, taking into account the significant differences between the use of formula (13) and the results of in situ tests, a situation also reported in [36], when calculating the bearing capacity of piles embedded in rock layers, I propose to use general calculation equation that take into account both components of the pile's strength, namely the pile shaft and pile base resistance.

The shaft resistance on the soil layers above the rock in which the pile is embedded can be taken into account, only in the case of checking its mobilization by the occurrence of displacements generated by the pile's own deformation.

These formulas are indicated in SR EN 1997-1: 2004 [46], chap. 7.6.2.2 Ultimate compressive resistance from static load tests and 7.6.2.3 Ultimate compressive resistance from ground test results, NP 123:2010 [30] și [36].

$$R_{c,k} = A_b \cdot q_{b,k} + \sum_i A_{s,i} \cdot q_{s,i,k} \quad (15)$$

Based on the results of the present research presented condensed in Table 5, the covering values in Table 6 can be taken into account for the resistance on the shaft and the base pressure on the piles embedded in a rock layer for a depth of at least 2 diameters and having the rock compression strength $R_c > 20$ MPa.

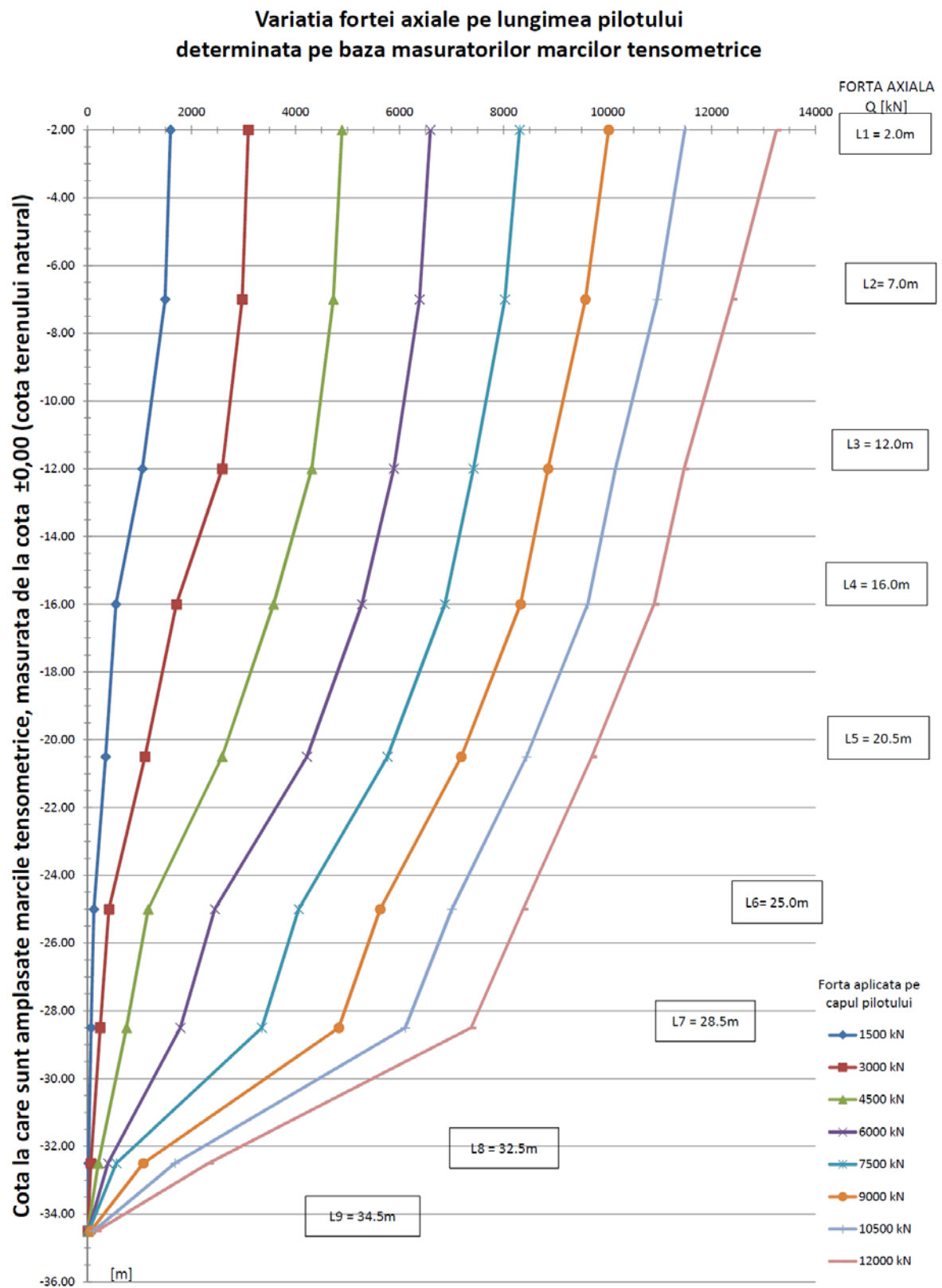


Figure 24. The variation of the effort on the length of the pile (LT5) resulting from the measurements on strain gauges

Table 6. Values of unit shaft resistance and unit base resistance for piles penetrating at least 2 diameters in a rock layer ($R_c > 20$ MPa)

The type of rock	$q_{s,k}$	$q_{b,k}$
	[kPa]	[kPa]
Weathered rock	250	3.000
Compact rock	500	5.000

The values proposed in Table 6 are valid only in the case of ensuring by additional methods the contact between the pile base and the rock, such as the one proposed in the present research.

According to the results of the current research, it is considered that for piles with lengths greater than 10...15 m, the specific deformations of the reinforced concrete element have a significant value to mobilize the resistance on the shaft surface in the soil layers traversed by the pile ($> 2... 5$ mm) if

$$\frac{N}{E \cdot A} > 0,3 \text{ ‰} \quad (16)$$

Knowing that

$$E_{cm} = 22[(f_{cm})/10]^{0,3} \quad (f_{cm} \text{ în MPa}) \quad (17)$$

it can be simply stated that if the stress applied to the pile's head is greater than about 30% of the compressive strength of the pile concrete, the values of specific deformations become significant to mobilize the resistance on the shaft surface in the soil layers traversed by the pile.

5.3.2. Full displacement pile (FDP) in a marly clay complex

The test pile was constructed using FDP (Full Displacement Piles) technology [17], therefore it can be considered a ground improvement pile. The elevation of the pile's head is approx. 2.5 m below the natural ground level.

Table 7. Soil stratification related to the natural ground level LT7

depth (m)	Stratification encountered in pile drilling	I_c
0,00 – 14,5	Filling of cohesive materials	~0,4
below 14,5	Marly clays complex	1,0

The FDP technology being relatively new in Romania, within the norm NP 123:2010 [30] there are no specific values or formulas for calculating the bearing capacity for ground improvement piles constructed on site. By analogy, data for prefabricated friction piles that compact the ground at the time of introduction can be used. Thus, for the calculation of the bearing capacity of the LT7 pile, the data and formulas for friction piles constructed on site, respectively for prefabricated friction piles were used in parallel.

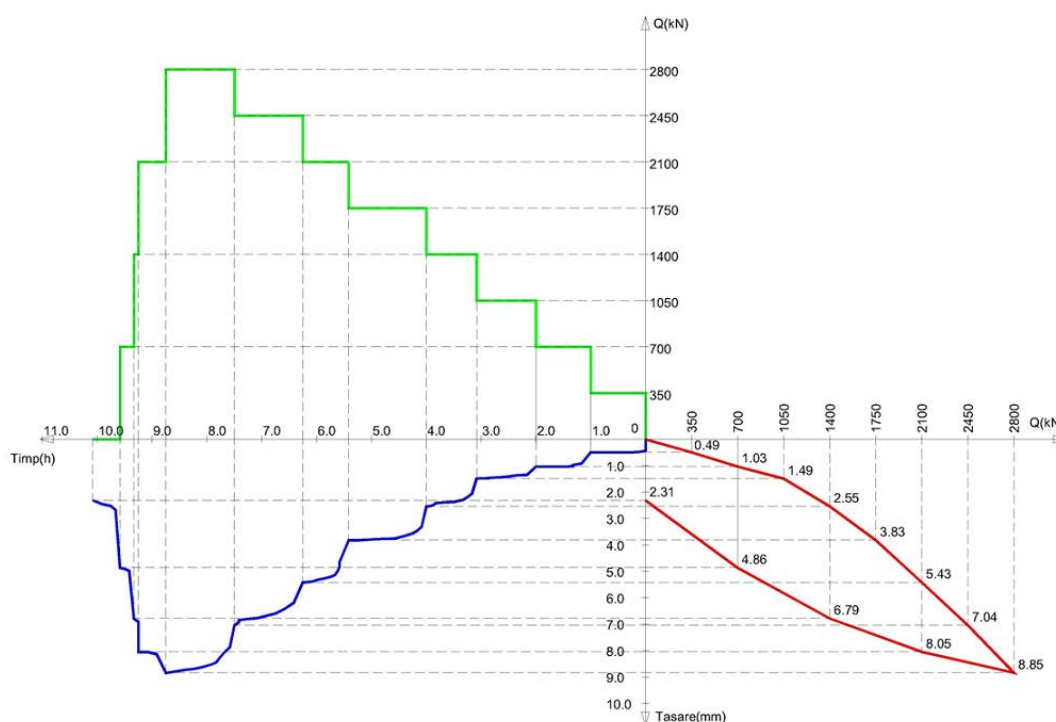


Figure 25. Load time settlement diagram for the pile LT7

Considering the values given in table 9 of NP 123: 2010 [30] for friction bored piles executed on site, a base pressure value of 1,450 kPa can be chosen, and considering the values given in table 5 of the same document [30] for a prefabricated driven and friction piles can choose a value of 11,700 kPa.

A pile base resistance value of $\approx 7,700$ kPa was measured during the load test (Figure 27).

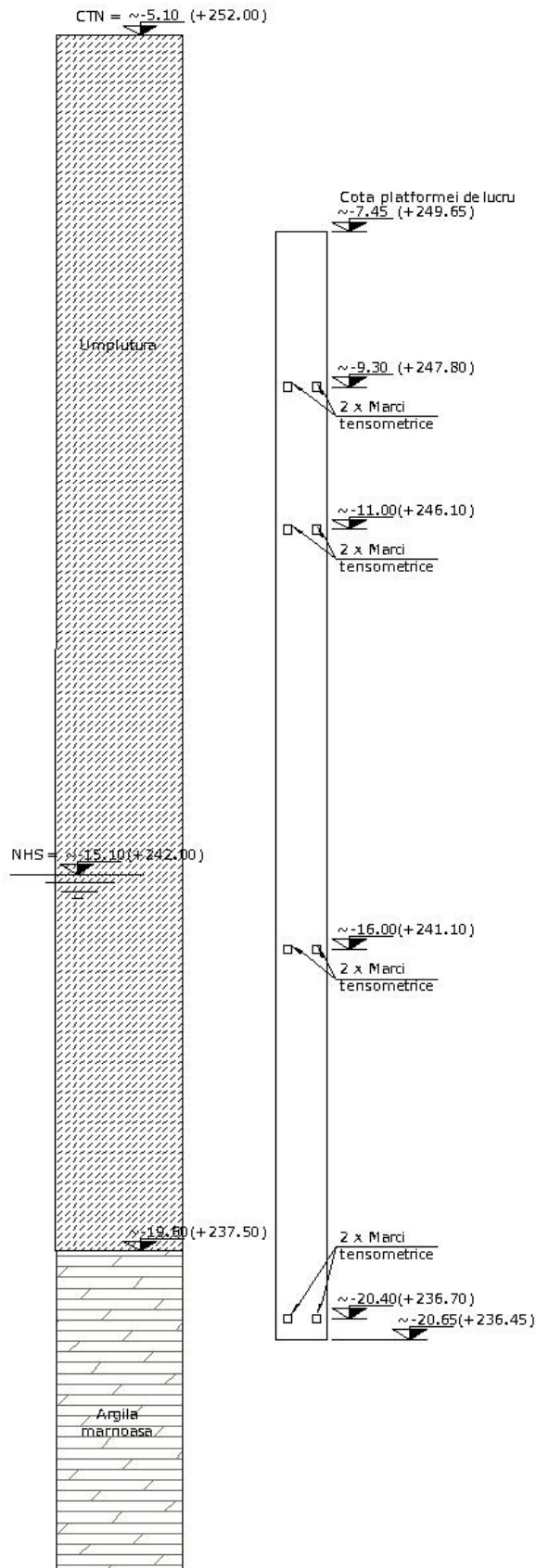


Figure 26. Schematic section of the pile Ø510 mm LT7

Table 8. Characteristic resistance on the pile shaft ($R_{s,k}$), on the pile base ($R_{b,k}$) and total compression ($R_{c,k}$) for the LT7 pile

Pile LT7	$R_{s,k}$ [kN]	$R_{b,k}$ [kN]	$R_{c,k}$ [kN]
Bored pile constructed on site	378	290	668
Driven prefabricated pile		2340	2718

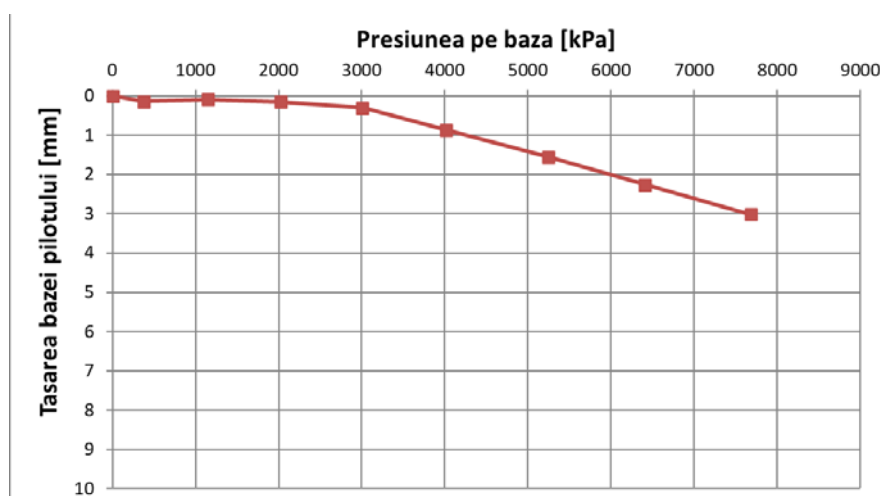


Figure 27. Unit base resistance on the LT7 pile

Figure 28 shows the resistance mobilization curves of the pile shaft.

I presented the differences between the known / standardized values and the values measured by in situ test. However, given the limited number of results available in this research, to propose tabular values or to define partial safety factors for ground improvement piles constructed by FDP technology, in agreement with the TUCEB PhD Student Guide I propose to continue this research in an extended framework based on the methodology proposed in this research for conducting and analyzing tests on test piles.

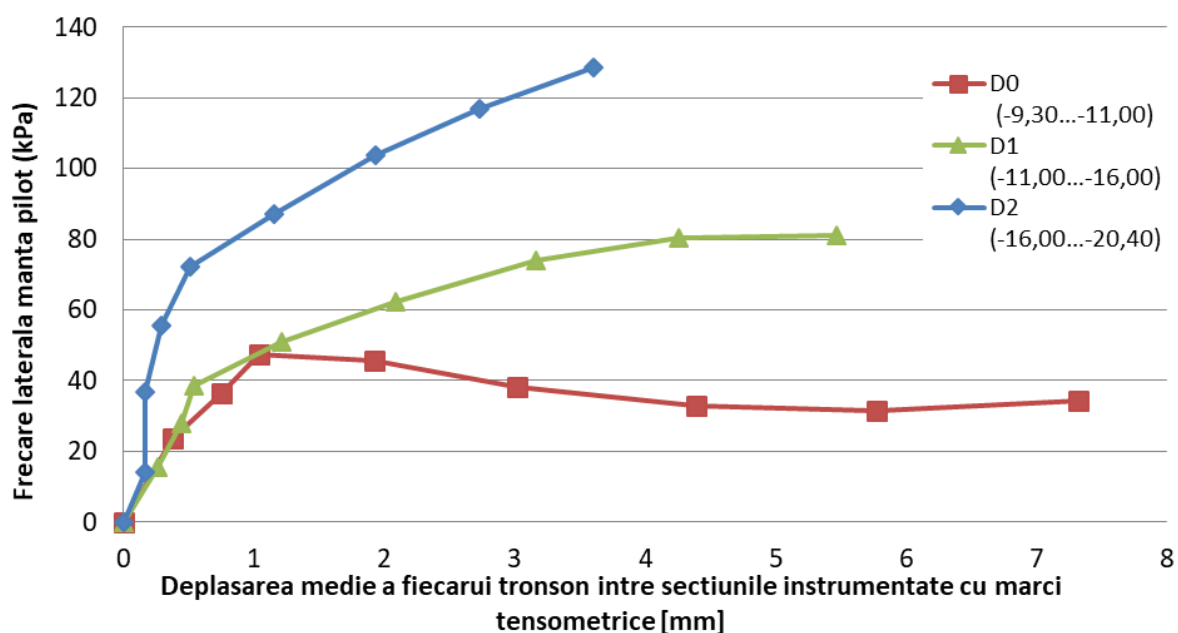


Figure 28. Unit resistances mobilized on the shaft of the LT7 pile determined by strain gauges

Table 9. Unit resistance on the shaft $q_{s,k}$ for the LT7 pile

Depth of elementary layer traversed by the pile	$q_{s,k}$ tab. 6 NP 123:2010 [30]		$q_{s,m}$ maximum measured during the test
	on the elementary layer	weighted average	
[m]	[kPa]	[kPa]	[kPa]
-3,0		9	47 ^{*)}
-5,0	9		
-7,0	11		
-9,0	12	12	81 ^{*)}
-11,0	12		
-13,0	13		
-14,5	14	28	128 ^{*)}
-15,6	72		

^{*)} Values influenced by the improvement of the ground given by the FDP technology.

5.3.3. CFA pile in a sandy layer

Table 10 and Figure 30 show schematically the calculation stratification of the LT8 pile resulting from the geotechnical investigations performed on site [39], respectively the elevation of the instrumented pile. The axial compression test was performed 16 days after the pile construction.

Table 10. Stratification relative to natural ground level $\sim \pm 0.00$ (LT8)

Depth (m)	Stratification	Φ'	I_c
0,00 – 1,00	Fill	-	-
1,00 – 3,00	Loess	-	0,9
3,00 – 10,00	Moistened Loess	-	0,5
10,00 – 18,00	Silty clay with sandy layers	-	0,7
18,00 – 30,00	Fine yellow-gray silty sand	34	-

Pile base level:

$\approx -21,00$ relative to the level ± 0.00 of the construction

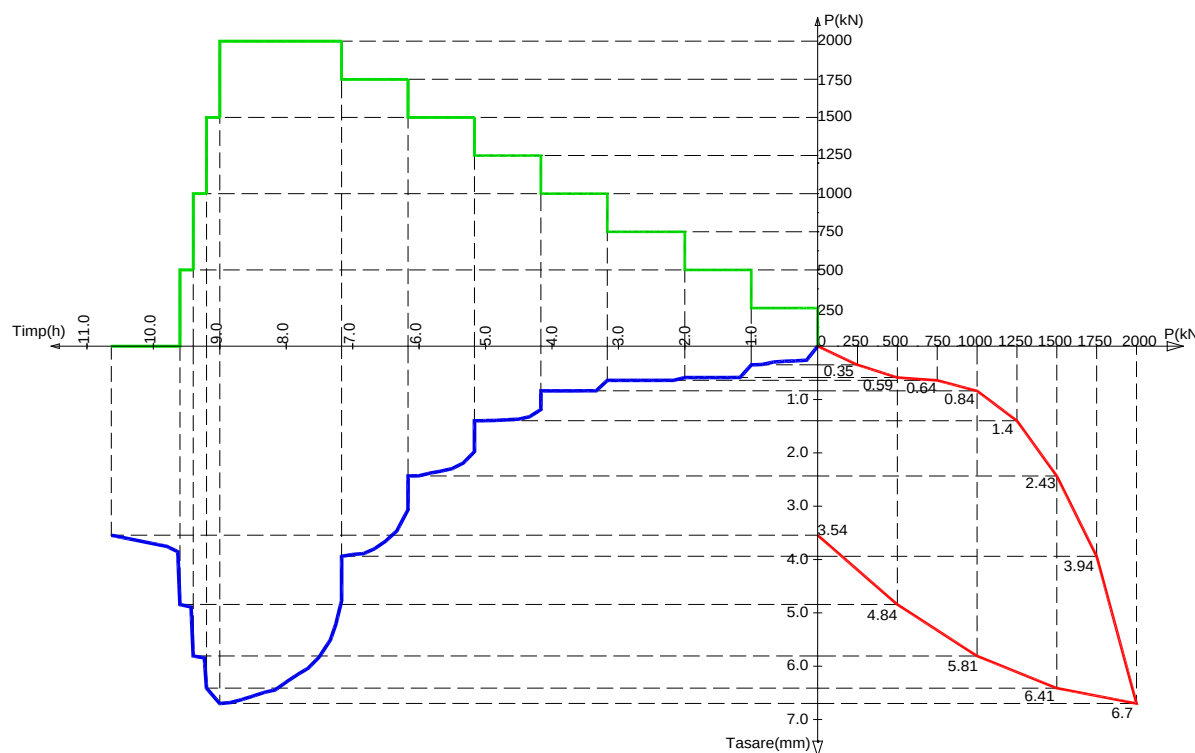


Figure 29. Load settlement time diagram for LT8 pile

For the characteristic value of the pile base pressure ($q_{b,k}$), formula (16) of NP 123:2010 [30] is used for bored piles that are conducted on non-cohesive layers.

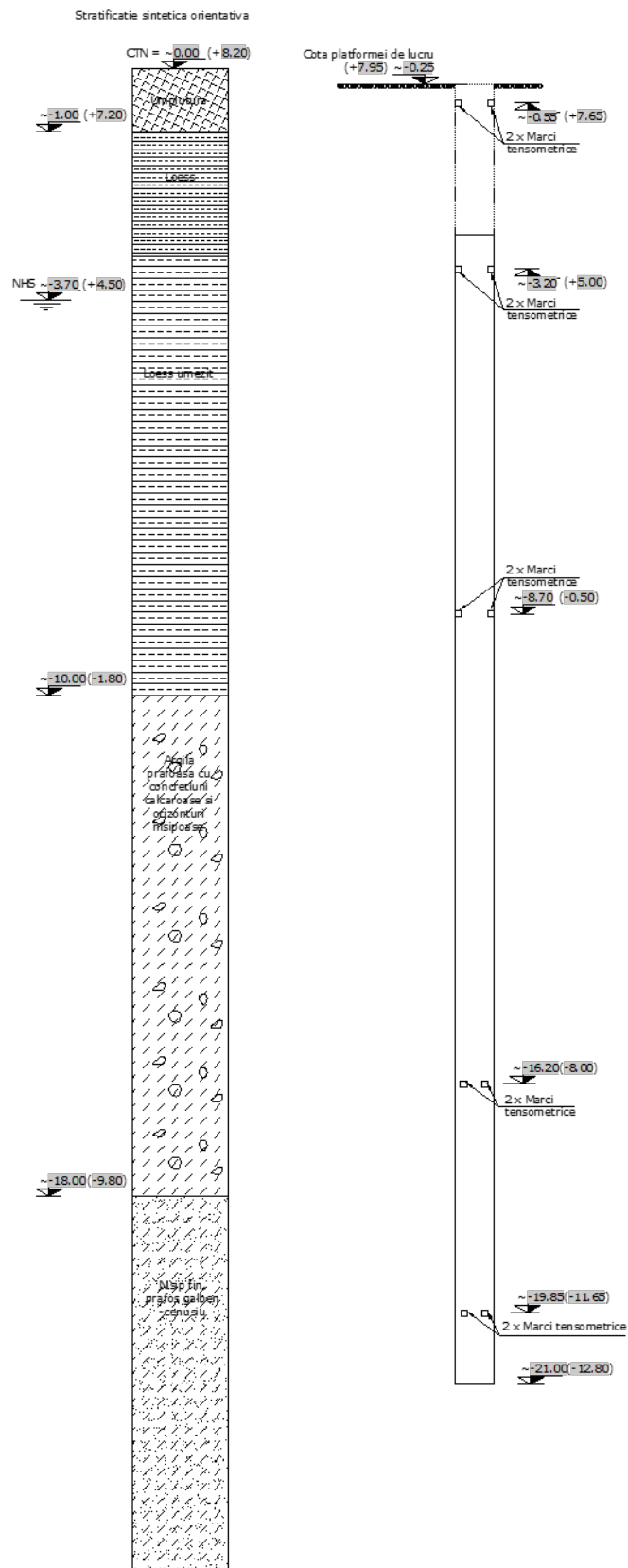


Figure 30. Schematic section of the test pile $\Phi 620$ mm LT8

Thus, results a characteristic base pressure value of 2,445 kPa. During the load test, a mobilized value of the base pressure of ≈ 500 kPa was measured. It is noted that the measured value does not represent the maximum value of the capacity of the base layer of the pile, but only the mobilized value for the applied load level, therefore the test is inconclusive to compare the maximum measured values of base resistance with known / tabular values.

The measured values of the resistance on the lateral surface indicated in Table 11 are extracted from the mobilization graphs of these values presented in Figure 31 and from the effort transfer graphs applied to the foundation ground.

Table 11. Unit resistance on the shaft $q_{s,k}$ for the LT8 pile

Depth of elementary layer traversed by the pile	$q_{s,k}$ tab. 6 NP 123:2010 [30]		$q_{s,m}$ maximum measured during the test
	on the elementary layer	average	
[m]	[kPa]	[kPa]	[kPa]
0,0			
-2,0	15	17,5	25
-4,0	20		
-6,0	24		
-8,0	25	25	19
-10,0	25		
-12,0	46		
-14,0	49	48	34
-16,0	51		
-18,0	54		
-20,0	55	55	77
-21,0	57		

In the case of the LT8 test it might be observed the proximity of the measured values of the resistance on the lateral surface of the piles compared to the tabular values specified in the norm NP 123:2010 [30].

Based on the values of unit base pressure and unit resistance on the shaft extracted from NP 123:2010 [30] and presented above, the characteristic values of the resistance on the shaft, the base resistance and the bearing capacity of the pile LT8 are calculated and are shown in Table 12.

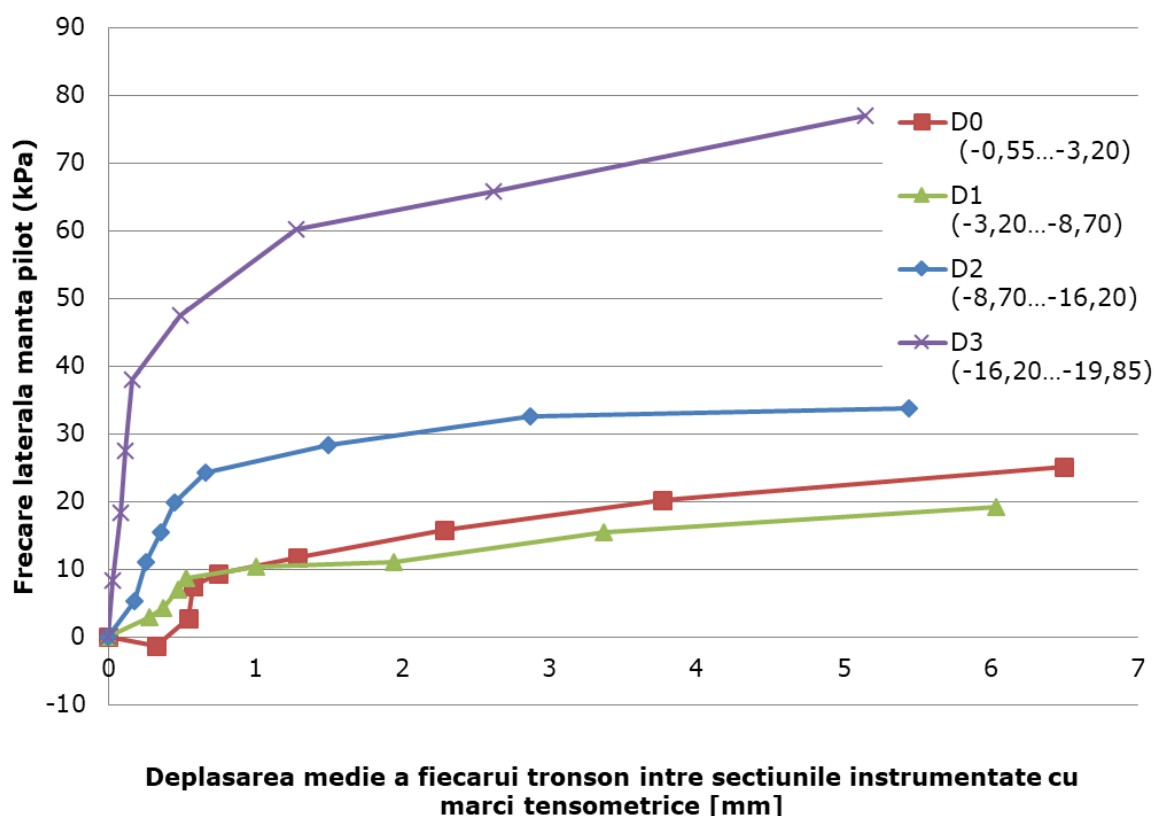


Figure 31. Resistance on the shaft of the LT8 pile

Table 12. Characteristic calculated shaft resistance ($R_{s,k}$), base ($R_{b,k}$) and total bearing capacity ($R_{c,k}$) for LT8 pile

Pile	$R_{s,k}$ [kN]	$R_{b,k}$ [kN]	$R_{c,k}$ [kN]
LT8	1.528	733	2.261

During the load test, the LT8 pile was loaded to a stabilized force of 2,000 kN, with a head settlement of only 6.7 mm. The maximum value of the pile's bearing capacity was not reached during the in-situ load test.

In the previous tables we presented the differences between the known / standardized values and the values measured by the in situ load test. However, given the limited number of results available in this research, for revising tabular values or for defining / revising partial safety factors for CFA piles, in accordance with the TUCB PhD Student Guide I propose to continue this research in an extended framework based on the methodology proposed in this research for conducting and analyzing tests on test piles.

5.3.4. Enlarged base piles performed in Bucharest typical geotechnical conditions

At the national level, there has been a continuous trend towards research and development of technologies for piling as well as detailing the tests on test piles.

The pile with the enlarged base LT9, which I will analyze in detail in this research, had a length of about 32 m, the diameter of the base $d_b = 2,640$ mm and the diameter of the shaft $d = 1,060$ mm. The pile was bored under the protection of polymer suspensions and subsequently after concreting was grouted into the base area through 6 grouting points [37].

According to the prescriptive method mentioned by the national norm [30] for cohesive layers, the determination of the values of the shaft and base resistance of the piles is performed based on the depth and the consistency index I_c . Thus, according to the detailed investigations carried out on the site according to SR EN 1997-2:2007 [47] for the intermediate clay layer in which the LT9 test pile is founded, the consistency index I_c had values between 0.9 and 1.1, and in the calculation, it was considered to have an average value of 1.0.

Under these conditions, for the characteristic value of the pressure based on the piles ($q_{b,k}$) the value of 3,125 kPa was chosen from table 9 - NP 123: 2010 [30], corresponding to a depth of the pile toe of 35 m.

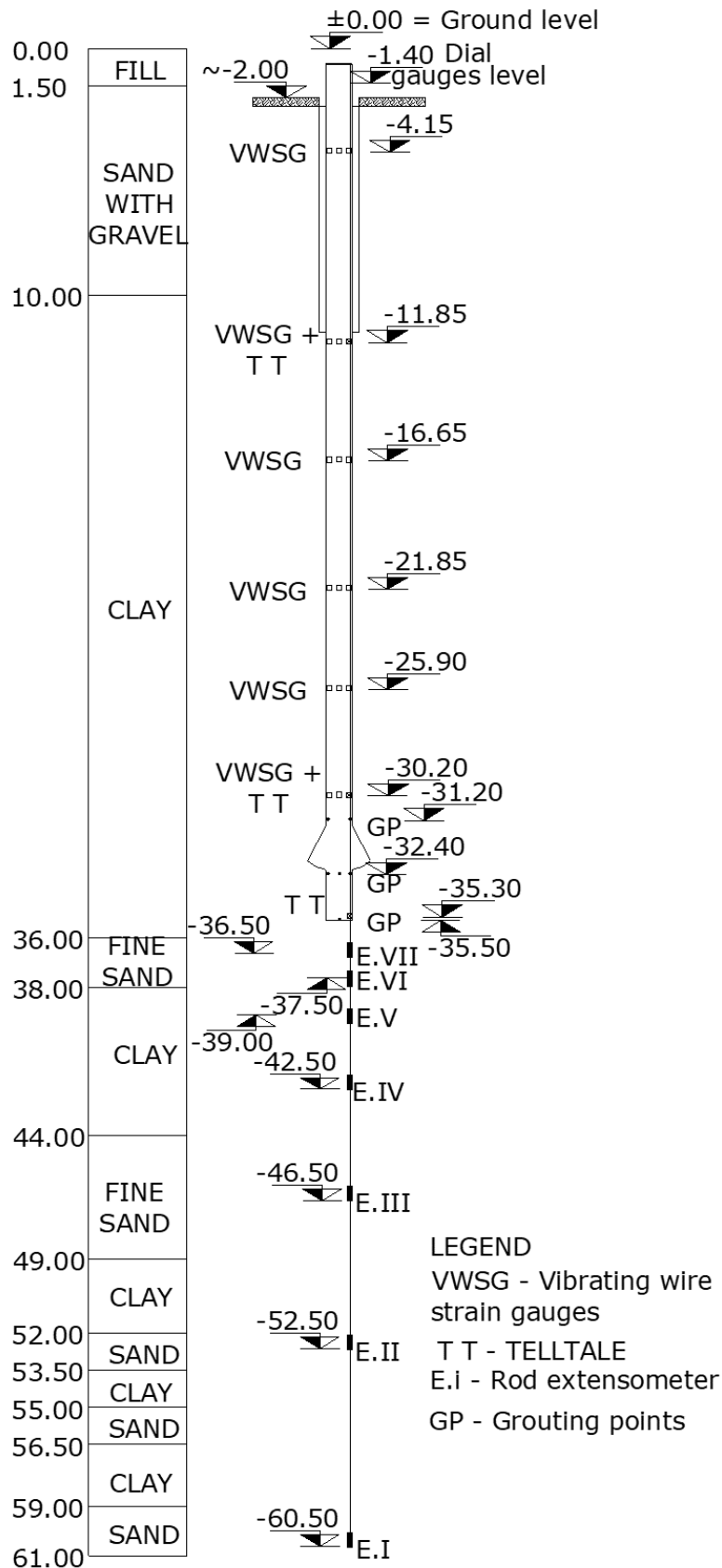


Figure 32. Stratification and instrumentation of the LT9 test pile

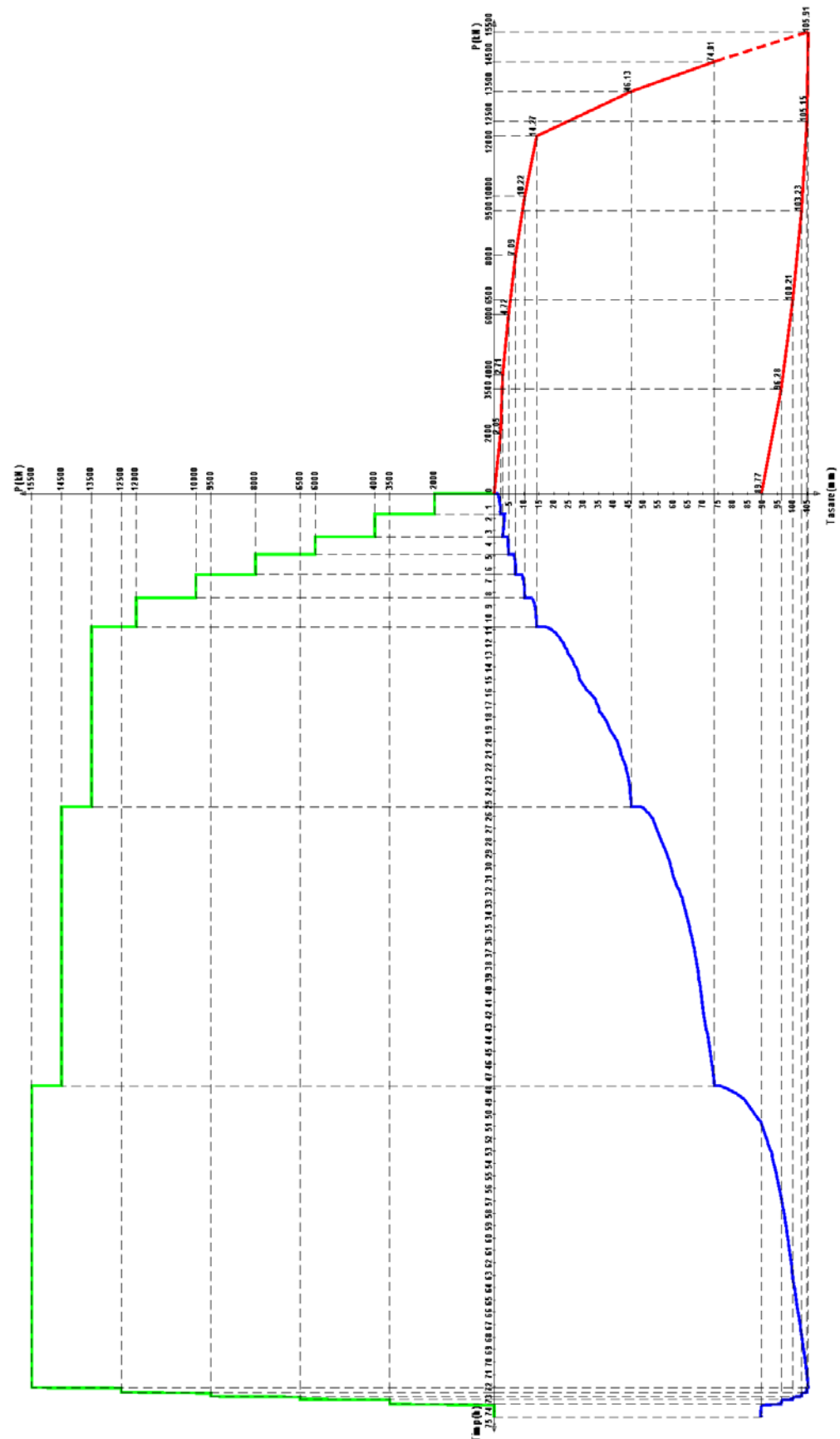


Figure 33. Load settlement time diagram of LT9 test pile

In the instrumented load test, for the LT9 pile a unit base pressure value $q_{b,m} \approx 2.000$ kPa was measured (Figure 35). Please note that the measured value does not represent the maximum value of the bearing capacity of the layer from the enlarged pile toe.

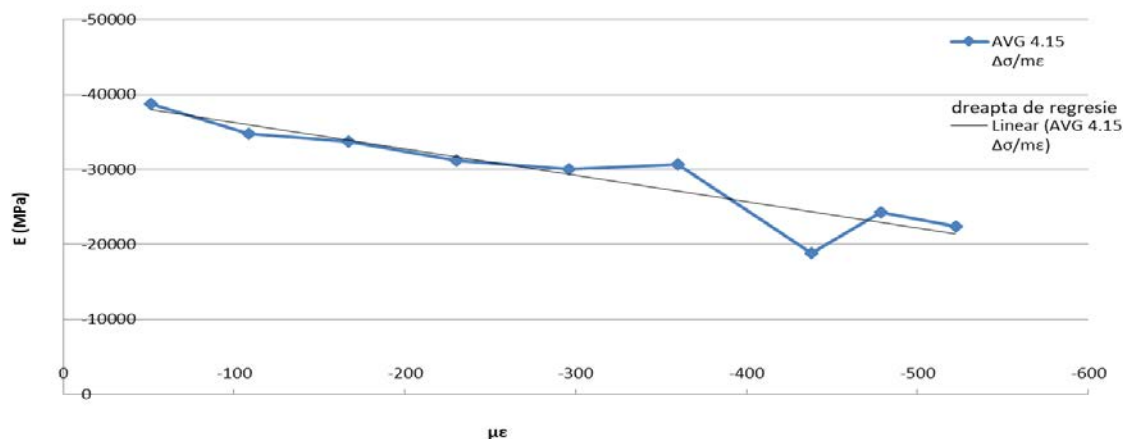


Figure 34. Determination of the deformation modulus for the reinforced concrete of the LT9 test pile

When applying the 13,500 kN loading step, the stabilization condition of 0,1 mm / 20 min was reached after a significantly increased time compared to the previous loading steps or the usual practice, as can be seen in Figure 33. When applying the loading step, the maximum shaft resistance was reached, the applied load increment being taken over by the enlarged base. The extended time for stabilization can be explained by taking into account that the surface area of the base (5.47 m²) is approximately 6 times larger than the area of the pile's shaft (0.88 m²) or of the usual piles, even more so the enlarged base it is founded in a clay layer.

Therefore, in a pile with an enlarged base during the load tests, the phenomenon of consolidation may occur, the surface mobilized under its base being much larger, needing additional time to reach the stabilization condition of 0.1 mm / 20 min.

Also, the extended time of the loading steps produced an imbalance compared to the previous steps in terms of the behavior of the reinforced concrete section, as can be seen even in Figure 34 on the last loading steps. The creep effects of the reinforced concrete section had an increased influence compared to

the unit effort steps. Therefore, it was necessary to adjust the specific deformations with the increment generated by the creep phenomenon.

It is concluded that for the instrumented load tests it is necessary to read the strain gauges at intervals of maximum 20 minutes, I recommend 5 minutes, in order to determine the section of reinforced concrete behavior under loads.

Also, if the time difference between the loading steps is not more than 3 hours then the creep effects of the reinforced concrete section can be neglected, the increase of specific deformations generated by the creep being approximately the same for all steps. This conclusion is valid only if the variation of the concrete elasticity modulus in a section at the head of the test pile according to the chapter 4.4 has been determined.

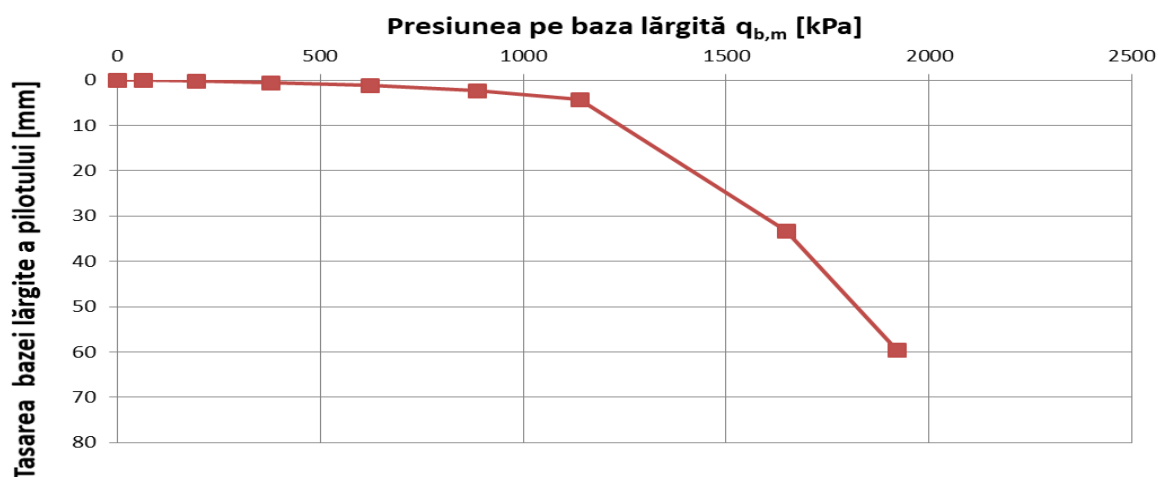


Figure 35. Base resistance mobilized on the enlarged base of the LT9 pile

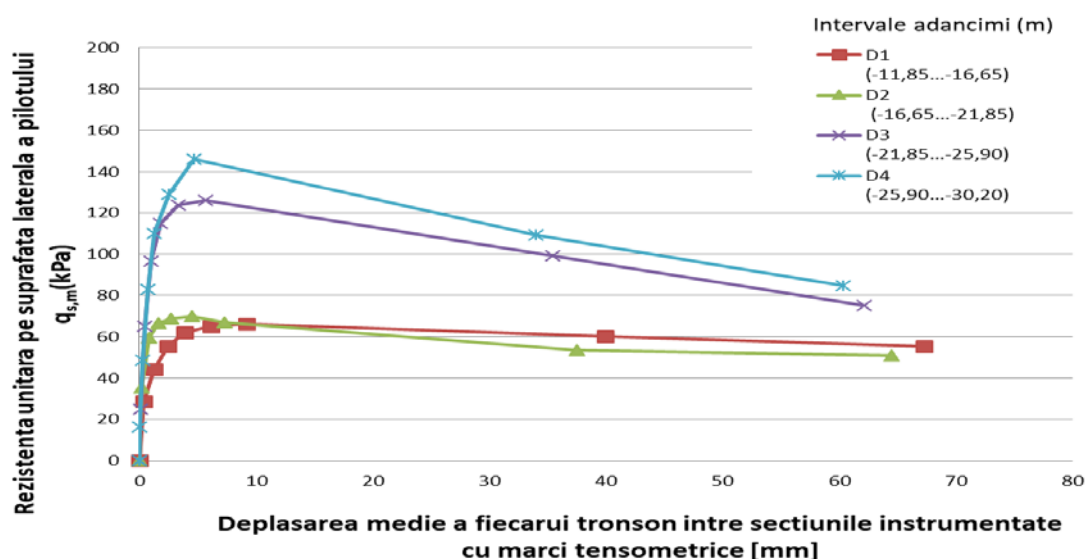


Figure 36. Shaft resistances mobilized on the LT9 pile determined by strain gauges

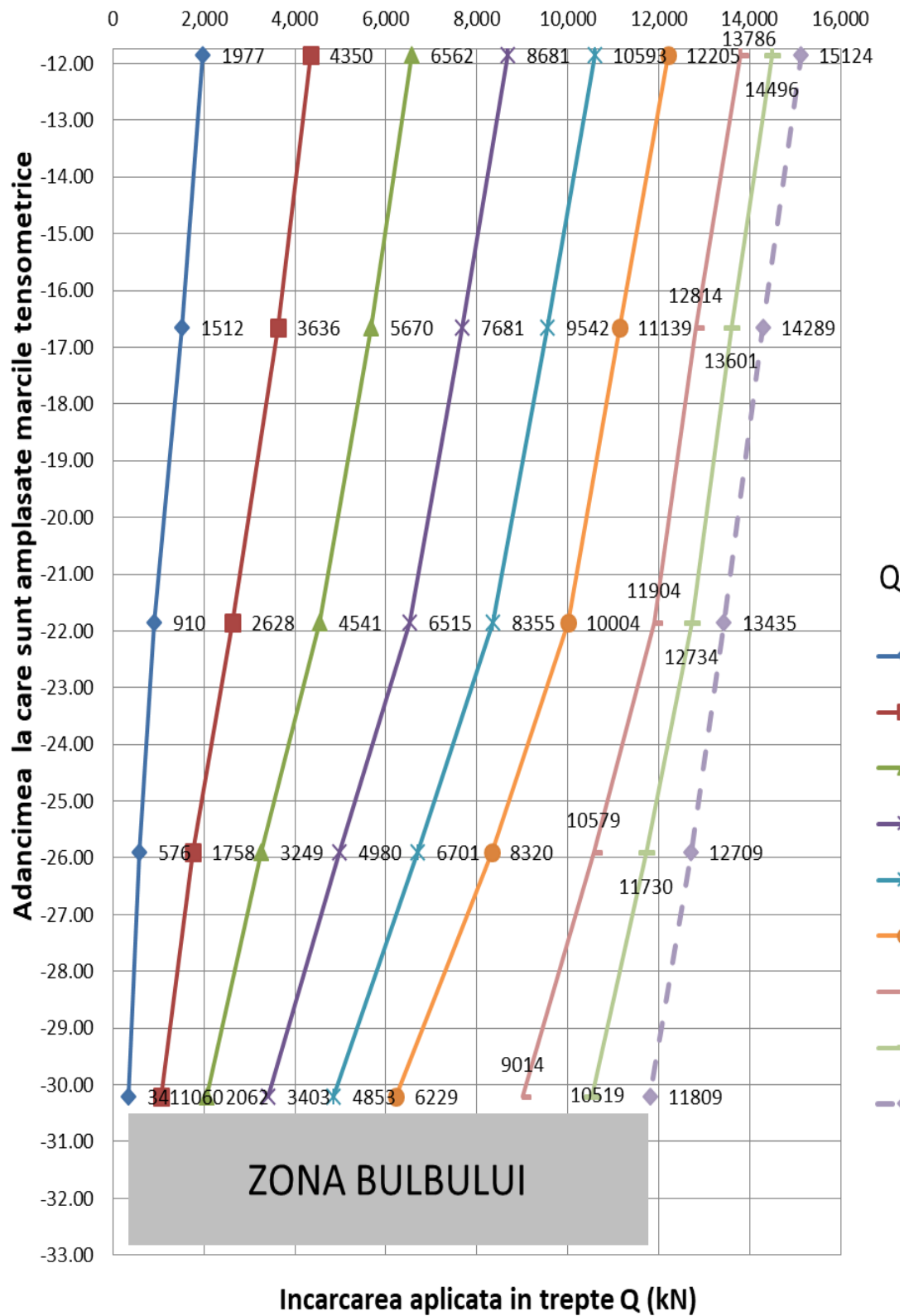


Figure 37. The variation of the stresses on the length of the LT9 pile resulted from the measurements on strain gauges

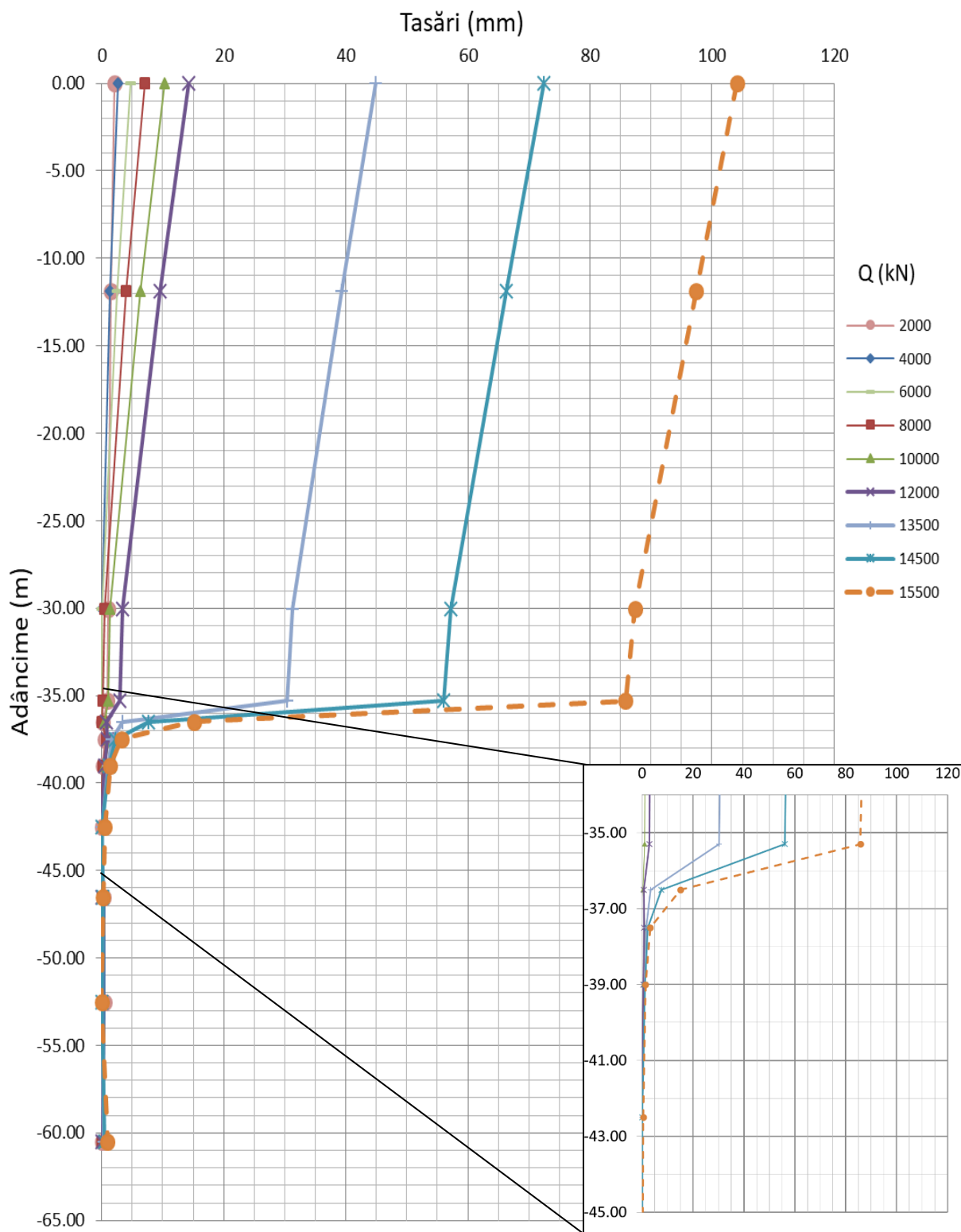


Figure 38. Variation of LT9 test pile and foundation ground settlement from -0.60 to -60.50 for the loading cycle

During the test with the help of rod-extensometers, a shortening of the pile by 18.26 mm for the last loading step was highlighted, which represents 17% of the total recorded settlement. (Figure 38).

Table 13. Unit shaft resistance on $q_{s,k}$ for the LT9 pile

Depth of elementary layer traversed by the pile	$q_{s,k}$ tab. 6 NP123:2010		$q_{s,m}$ maximum measured during the test	Difference between the measured value vs. the value from NP123:2010
	on the elementary layer	average		
[m]	[kPa]	[kPa]	[kPa]	%
-12,0				
-14,0	69	71	66	-7 %
-16,0	72			
-18,0	76			
-20,0	78	78	70	-11 %
-22,0	81			
-24,0	85	85,5	126 ^{*)}	+47 % ^{*)}
-26,0	86			
-28,0	89	90	146 ^{*)}	+62 % ^{*)}
-30,0	91			

^{*)} Values influenced by grouting into the pile's toe area.

For the lower cohesive layers there is an increase of the unit shaft resistance compared to the tabular values in NP 123:2010 [30] due to the grouting process on the shaft up to 62% in the layer where the grouting was made, respectively up to at 47% in the upper layer where the grout has spread.

Following the choice of the values of the base pressure and the resistance on the lateral surface of the piles extracted from the norm NP 123:2010 [30] and presented previously, the characteristic values of the resistance on shaft, the pile resistance and the ultimate bearing capacity of the LT9 pile are calculated (Table 14).

Table 14. Characteristic resistance on the shaft ($R_{s,k}$), on the base ($R_{b,k}$) and total compressive resistance ($R_{c,k}$) for the LT9 pile

Pile	$R_{s,k}$ [kN]	$R_{b,k}$ [kN]	$R_{c,k}$ [kN]
LT9	4.840	17.090	21.930

In view of the LT9 test for piles with an enlarged base with a base area of more than 4 m², in order to avoid side effects in the analysis of the instrumented test, either related to the creep of concrete or the consolidation of the foundation ground or its creep, the waiting time for step stabilization shall be limited to a maximum of 6 hours and a stabilization criterion of 0,5 mm / 20 minutes shall be established which shall be valid for a double number of loading steps, a minimum of 16 steps.

6. Conclusions. Personal contributions. Perspectives on further research

In the introductory chapters a synthesis was made and presented both the equipment generally used in load tests on piles and the specifications they must meet according to the purpose described in the research, namely tests on instrumented piles.

Also, according to the synthesis of the specialized documentation were presented the main deforming processes of concrete and their effects that can influence the analysis of loads on piles, this being the first specific objective proposed at the beginning of the research.

According to the specialized literature, it was presented the procedure for determining, during the axial compression test on instrumented piles, the linear elasticity modulus of the concrete which is variable with the state of specific deformations in the pile.

I contributed to the determination procedure by making some important remarks and observations that I encountered in the instrumented pile tests described in this research and without which the results of the analysis would be strongly flawed.

For load tests on instrumented piles, careful test preparation and real-time collection of recorded data must be performed, and based on this the specialized personnel conducting the load test can make decisions, even during the test, on the steps or the value of the load for its application. Following the in situ load test, a detailed analysis of the data recorded during the test shall be performed in order to be able to perform the interpretation of the data and the transformation from strains into unit stresses. The research also emphasizes the importance of detailing the test procedures of instrumented test piles in order to obtain relevant

results from the test polygons. The waiting time until stabilization on each loading stage proved to be very important due to the influences generated by the creep of concrete, especially for piles with a young concrete. Therefore, if the time difference on each loading step is greater than 3 hours, the contribution of the creep must be quantified in the values of the strain measured in the instrumented load test.

Carrying out the load test until the pile bearing capacity is reached and combining the tests on a natural scale with analyzes based on basic principles of soil mechanics can be determined the behavior of the pile and the transfer mechanism of the loads for the accurate definition of unitary resistances.

In the load tests on piles, deformations and displacements of the pile shaft are measured which are transformed into stresses by means of the variable concrete elasticity modulus during the load test according to the state of stress in its section, according to the methods indicated in this research. The transformation of the measured strains into stresses is extremely important and practically dictates the final result of the instrumented load tests on the piles (shaft and base pressure mobilization curves on the instrumented piles).

This analysis of the test piles also highlights the two components of the bearing capacity of the piles, namely the base pressure and the lateral friction along the entire length of the pile, and allows the transfer of test load results to other foundation piles constructed under same technological and similar geotechnics conditions.

In the research I made comparisons of the measured values with the values resulting from bearing capacity calculations, thus responding to the following specific objective proposed at the beginning of the research.

I grouped into four general categories the nine load tests on instrumented piles, as follows: piles embedded in a rock layer, FDP pile in a marly clay layer, CFA pile in a non-cohesive layer, pile with enlarged base. I detailed the interpretation of the data from the instrumentation used, so I responded to another specific objective of the research.

Following the global analysis of the tests performed and the comparison with the results obtained with usual values from the specialized literature, I made proposals for improving the calculation of the foundation piles resistance.

In the case of end bearing piles embedded in rock, I propose the adoption of a new bearing capacity calculation procedure and propose unit strength values on the shaft and on the base of piles for embedding length in a semi-rocky rock layer, with the compression strength $R_c > 20$ MPa.

In addition, I found that the specific deformations of the end bearing pile shaft can be extremely significant so that it mobilizes the lateral friction on the layers above the embedding in the rock. Based on the measurements performed on six tested instrumented piles, a condition was issued to be able to use the lateral friction both in the soil layers and in the rock layers in which the pile is embedded.

During the research I identified a problem of embedded piles in the rock execution technology and I introduced a practical method for ensuring the contact between the rock layer and the reinforced concrete pile, by introducing a cement suspension on toe of the embedded pile in the rock and mixing with residual material remaining on the toe of borehole. Other methods can also be used, such as grouting the pile toe.

I highlighted the effects of improving the unit resistance on the shaft, in case of grouting on the shaft by at least 50%, as well as in case of using FDP technology.

In the case of piles with an enlarged base, considering the extended surface that is mobilized, I proposed a new criterion for stabilizing the loading steps, simultaneously with the use of an increased number of steps of at least 16 loading steps.

Analyzing the general objective and the specific objectives proposed, it is found that the present research leads to the improvement of the foundation pile resistance calculation.

Personal contributions

Given the complexity of the issues raised by the researched field, many issues were addressed during the thesis, of which the most significant personal contributions to the improvement of the pile resistance calculation refer essentially to the following:

- ✓ Selection of devices and specifications for instrumented load tests;
- ✓ Detailing the factors and behavior of the concrete during the tests;
- ✓ Synthesis of test piles procedures and steps to be followed with emphasis on extremely important details;
- ✓ Presentation of the transition from strains to stresses, including personal remarks on the proposed methodologies;
- ✓ Preparation and performance of nine tests on instrumented piles;
- ✓ Proposal for changing the procedure for calculating the bearing capacity of end bearing piles embedded in semi-rocky layers and the values related to lateral friction and base pressure on the semi-rocky layers of a large part of the sites located in Romania;
- ✓ Identify the significant values of the own deformations of the end bearing piles and propose a condition for consideration and lateral friction;
- ✓ Comparison of the results obtained with values from the specialized literature and national regulations;
- ✓ Identify in the research the need to take additional measures to achieve the piles in order to ensure the contact of rock concrete and proposals that were used in the research;
- ✓ Identification of the effects of improving the unit strength on the shaft when grouting piles and using FDP technology;
- ✓ Propose an improved test procedure for piles with an enlarged base.

As part of this research, on the topic of this doctoral thesis I published six articles at some of the most important national and international conferences, but also national symposia. I mention here the National Conference on Geotechnics and Foundations where I presented improved methods for calculating and

interpreting instrumented tests [34], the XXV AICPS National Conference [37], the 15th edition of the Danube Conference held in Vienna where I published the analysis of the instrumented tests embedded in semi-rocky rock [33], respectively European Conference of Soil Mechanics and Geotechnical Engineering which took place in Edinburgh, in which we presented the results of the instrumented test on a deep pile with an enlarged base [35], and the complete list of published articles can be found in the chapter Bibliography

Perspectives on further research

Given the framework outlined in this research, which details the specifications of the necessary equipment, the methods of testing and analysis can highlight the following perspectives on further research in an extensive research framework at national level.:

- Establishing the values of the unitary resistances of the piles embedded in other types of rock present on the Romanian territory;
- Research into new methods to ensure intimate contact between the pile's concrete and the rock in which it is embedded ză;
- Determining the values of the unit calculation resistances of the piles constructed by the FDP technology in order to include them in the national norms;
- Determination and / or confirmation of tabular values in the current norms for friction piles.;
- Comparison of in situ results with load capacity calculation results resulting from calculations based on CPT tests;
- Revision of partial safety coefficients and model coefficients for the calculation of the bearing capacity of foundation piles according to the execution technology and their type.

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Doctorand,

Dipl.-ing. Ion RĂILEANU, M.Sc.