In situ and laboratory soil investigations. Correlations between different parameters specific to Bucharest area

Alexandru Poenaru
Technical University of Civil Engineering Bucharest, Bucharest, Romania, alexandru.poenaru13@gmail.com

Alexandra Bilcu, Alexandra Meirosu, Tudor Saidel
SAIDEL Engineering SRL, Bucharest, Romania

Loretta Batali
Technical University of Civil Engineering Bucharest, Bucharest, Romania, loretta.batali@utcb.ro

ABSTRACT: Geotechnical design requires detailed knowledge of the mechanical behavior of foundation ground, as well as the 3-D distribution of the corresponding parameters. The Romanian practice emphasizes mainly the use of laboratory tests, but as a result of the development of advanced soil modeling, it is more and more required to complete and detail the laboratory tests with in situ investigations for all soil types, not only for granular ones as usually done in practice. The paper presents the results and correlations obtained out of in situ investigations such as Cone Penetration Tests (CPT) and Flat Dilatometer Marchetti Tests (DMT), in conjunction with typical laboratory tests (oedometer and direct shear tests) on soils, specific to Bucharest area. The obtained correlations were fine-tuned based on a Case Study in which an excavation was modeled in PLAXIS 2D. The results of the 2D model were compared with the results obtained during the geotechnical monitoring of the excavation during construction. The paper is aimed to confirm and calibrate known correlations and to propose new correlations for specific geotechnical parameters which are commonly used as input data in the current numerical modeling.

Keywords: CPT; DMT; laboratory testing; in situ testing; parameter correlations

1. Introduction

The main purpose of this paper is to determine possible correlations between in situ and laboratory geotechnical investigations for soils typical to Bucharest area. The paper is focused on the correlations between Cone Penetrometer Test (CPT – Fig. 1), oedometer, shear box tests and Flat Dilatometer Marchetti Tests. To achieve those goals geotechnical investigations on multiple sites in Central and West part of Bucharest were performed. With the collected data, correlations were determined as presented in Chapter 5.

Figure 1. CPT Truck used to push the CPT cone and the DMT blade into the ground

In order to calibrate the obtained correlations, back calculations were performed using the geotechnical monitoring data from a recently built structure with 3 underground levels and a reinforced concrete dia-phragm wall as excavation support.

The analysis was performed taking into account that the differences on the measurements during the ge-otechnical monitoring and the results of a FEM model-ing are not entirely dependent on the soil parameters and rather on the constitutive models mathematical modeling errors of a FEM model and the variability of those parameters along the layer.

The FEM modeling was performed using Mohr-Coulomb and Hardening Soil constitutive laws.

The investigated soil layers are the “Bucharest Loam” and “Colentina sands and gravels”, which are typically found in Bucharest. The geological deposits of interest to the present paper and typical to Bucharest area are of Quaternary age (Pleistocene and Holocene). Together with other layers, they cover the entire region, with a thickness of about 300-350 m [1].

At the surface, there are old and new fills, from the low terraces of the Dâmbovița river meadow (2 – 10 m thick). The upper sandy clay complex, "Bucharest Loam", consists of silty-clay deposits with sandy-clay lenses. Those silty-clay deposits are brownish - yellowish, firm, with high compressibility and high to very high plasticity. They can be water sensitive (collapsible), having a usual porosity between 38 to 45%.

The "Colentina sands and gravels" soil layer represents the upper sandy complex (Upper Pleistocene). This layer is composed of fine, silty to medium-coarse sand, with fine gravel, yellowish-grey, with a medium relative density. It is worth noticing that the first aquifer is found in this layer. The aquifer is usually unconfined.

2. In situ and laboratory investigations

To achieve the above presented objectives, field and laboratory investigations were performed on multiple sites in the Central and Western part of Bucharest.

The in-situ investigations program consisted of static electrical penetrometer tests (CPT), as well as geotechnical boreholes with undisturbed soil sampling. As part of the results, CPT test results were used in order to statistically correlate the soil stratigraphy and the geotechnical strength and compressibility parameters (deformation modulus, friction angle and undrained cohesion) for cohesive, as well as for granular soils. The CPT tests were performed using a special CPT Truck, which was able to provide a pushing force of 200 kN. In addition, the CPT cone had a diameter of 44 mm and a length of 30 cm. The CPT tests were performed on the entire depth of the studied soil layers.

All the provisions of EN 1997-1:2004 [2] and EN ISO 22476-1 [3] were respected during the tests.

The DMT tests were performed in 25 cm steps, each step corresponding to a DMT reading, down to a depth of 25 m (Fig. 2). Similarly, the DMT probe was pushed into the ground using the CPT Truck.

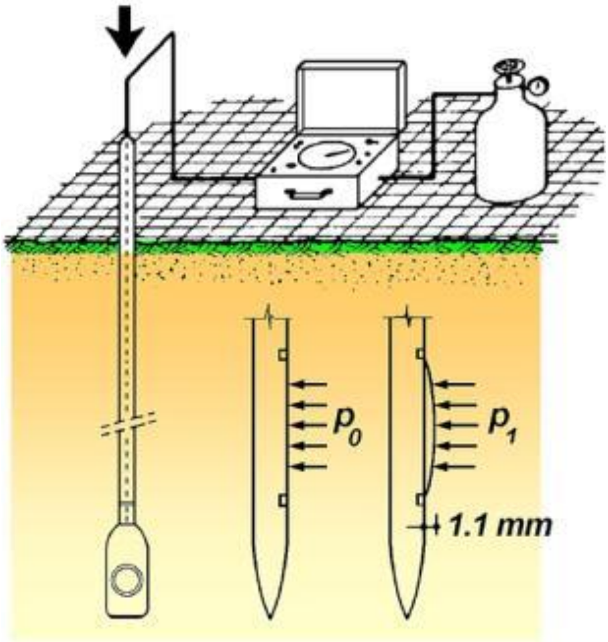

Figure 2. DMT working principle

As part of the investigation program multiple geotechnical boreholes (two for each site) were drilled at least on the entire depth of the studied soil layers. Furthermore, from each borehole disturbed and undisturbed samples were collected in order to be tested in an authorized laboratory. The undisturbed samples had been sampled in thin walled (2 mm thick) stainless steel samplers, manufactured according to ASTM D1587 / D1587M – 15 [4]. The sampling length was approximately 25 to 30 cm. The sampling procedures were those recommended by EN 1997-2:2007 [5]. The laboratory investigations aimed to determine the physical and mechanical parameters of the soil.

The differences between the investigated locations from Central and West Bucharest, in terms of average value of q_c and M_{DMT} parameters, are presented in Fig. 3, 4 and 5.

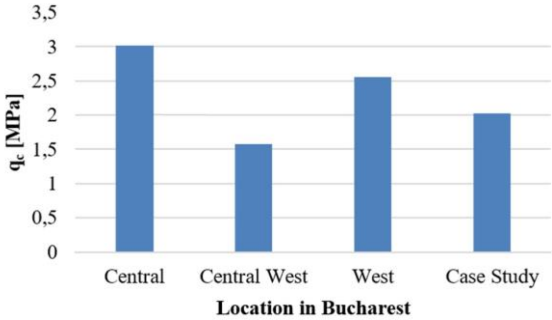

Figure 3. Average q_c variation on the studied locations in case of Bucharest Loam

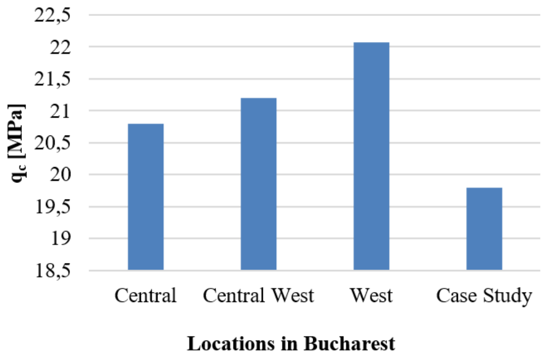

Figure 4. Average q_c variation on the studied locations in case of Colentina Sands and Gravels

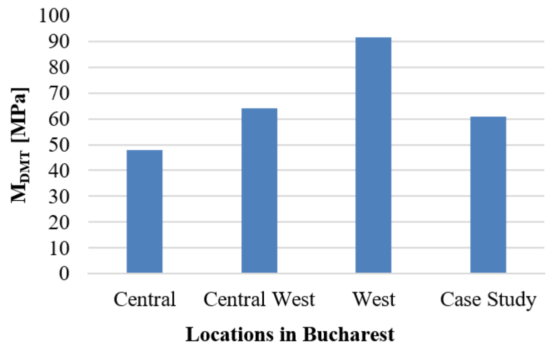

Figure 5. Average M_{DMT} variation on the studied locations in case of Colentina Sands and Gravels

3. Geotechnical monitoring

The instrumentation of geotechnical structures is necessary for the evaluation of displacements and deformations of structures during construction, as well as for the assessment of the performance of new materials and technologies used in the design and construction of geotechnical structures [6].

In order to achieve our goals, the geotechnical works were monitored during and after construction. The geotechnical monitoring consisted in the monitoring of the excavations lateral and vertical deformations with inclinometers and extensometers.

3.1. Extensometer monitoring

The extensometer measurements aim to monitor the deformations of the foundation ground. An extensometer

(Fig. 6) consists of two plastic tubes, generally a rigid inner tube and a flexible outer tube, on which magnetic sensors are mounted. It is drilled to the depth at which the deformation of the ground can be considered as zero. The extensometer column is installed into a drilled borehole, and the annular space between the extensometer and the borehole wall is filled by injecting a self-hardening cement-bentonite-water suspension with a compressibility equivalent to that of the foundation ground.

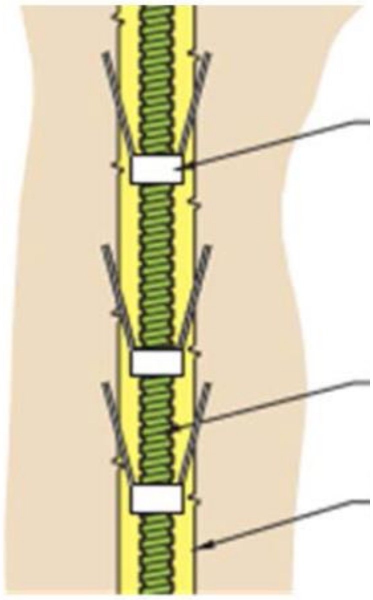

Figure 6. Typical extensometer tube

The measurements for determining the depth-variation of the magnetic rings are performed using an electro-inductive transducer equipped with a millimeter band. Considering the end depth mark as fixed, one can determine the differences between the positions of all the marks between the current measurement cycles and the initial reading.

The extensometer was installed down to a depth of 30 m.

3.2. Inclinometer monitoring

The inclination of the probe is determined digitally with the help of MEMS technology, the equipment allowing the measurement of inclination in 2 orthogonal directions. Thus, tilt values are obtained in the plane determined by the guiding wheels of the probe running down the inclinometric tube - axis "A", and in perpendicular plane to it - axis "B".

The components of the inclinometer monitoring system are the following (Fig. 7):

1. inclinometric column;
2. inclinometric probe;
3. inclinometer cable;
4. datalogger.

After the installation of the inclinometric tubes, the first measurements were performed, which represent the reference "0" measurments. These measurments are performed before any excavation works begin.

The hypothesis underlying the determination of the inclination of the structures using the inclinometer, considers the base of the inclinometric tub to be fixed.

The inclinometer tubes were installed in the excavation protection concrete walls, down to a depth of 25 m.

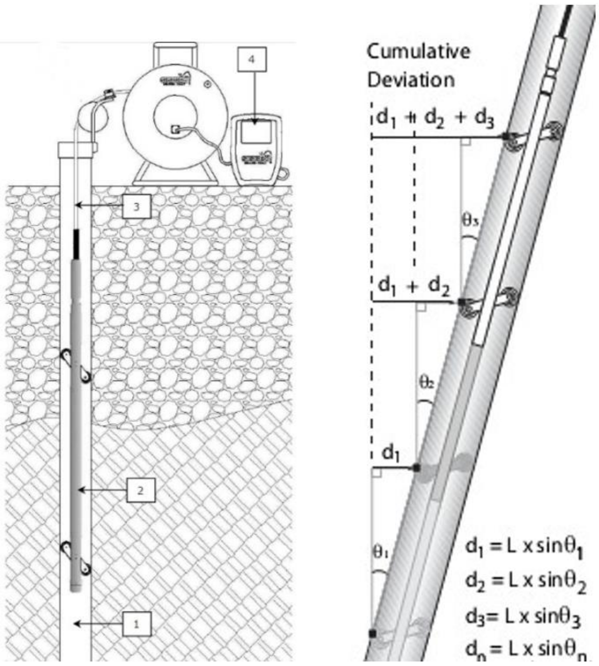

Figure 7. Inclinometer system and working principle

4. FEM Modeling

Soil is a complex material with a non-linear and anisotropic behavior, whose characteristics and parameters vary over time when it is subject to stresses. Usually, the soil behaves differently during loading, unloading and reloading. By reloading, the soil exhibits plastic deformations and its stiffness depends on the stress level. In addition, the soil presents a different stiffness in the domain of small deformations.

A constitutive model represents the mathematical modelling of the mechanical behavior of a material. This mathematical model represents a series of equations describing the stress - strain relation. The connection between the equilibrium equations and the compatibility ones are altogether integrated.

In simple terms, the Finite Element Method is a numerical method used to analyze complex physical phenomena, which can be assimilated in two-dimensional (2D) and three-dimensional (3D) space. It is mainly used to calculate engineering data like deformation of solids, fluid flow, heat transfer etc. The analysis using MEF consists practically in the replacement of the real (continuous) domain, in which the physical phenomenon is happening with another domain, or better said a discrete model. The process of assimilating a continuous real model to a discrete one is simply called modelling. The division of the discrete domain into sub-domains, called finite elements, it is called discretization. The application of the MEF, consists in establishing the matrix equation of equilibrium for each finite element. The next step is the introduction of the boundary conditions and the computation of the equations in order to obtain the parameter values in the nodes of each discrete element [7].

The real mechanical behavior of a soil is complex, characterized mainly by a strong anisotropic behavior. Over time, a series of deformation laws were developed to simulate the mechanical behavior of soils like: linear-elastic (Hooke), elasto-plastic (Mohr-Coulomb), hyperbolic (Duncan&Chang) and non-linear elastic (hyper-elastic, hypo-elastic) [8].

To achieve our goals, and to enable us to verify and calibrate our proposed correlations, in the present paper the elasto-plastic model Mohr-Coulomb and Hardening Soil model were used.

The Mohr-Coulomb model is an elastic perfectly plastic model usually used for a first approximation of the mechanical behavior of a soil, therefore using some elementary parameters, to which a failure criterion is added. The main parameters are defined by the deformation modulus (E), internal friction angle (φ) and cohesion(c).

The "Hardening Soil" model solves the limitations of the hyperbolic model. In comparison with the elastic perfectly plastic model (Mohr-Coulomb), the failure surface is not fixed in the space of the principal stresses, but can be extended due to the plastic deformations. It is necessary to comprehend the two types of hardening introduced in this model. The Hardening Soil model implies an elastic behavior of the material during loading and unloading, respectively. Still, the domain of deformations in which the soil behaves perfectly elastic is very small. [9]

5. Correlations between CPT, DMT and laboratory tests

For the scope of this paper, only linear correlations (ax+b) were studied.

A rigorous process for selecting field and laboratory tests was required in order to obtain quality results. The first stage of the actual selection process consisted in choosing the investigation points (geotechnical drilling, CPTs and DMTs) that could be included in a circle with a radius of no more than 3 m.

The second stage consisted in the creation and the in-depth analysis of the geotechnical profiles, for each individual site.

In Stage 3 the borehole samples that suffered significant disturbance were eliminated.

Stage no. 4 consisted of eliminating the laboratory results with implausible results. The plausibility was checked in the following way. Given our previous experience with those kind of soils, for example, for a value of q_c greater than 2 MPa could not correspond a value lower than E_{oed} = 4000 MPa. That means that the laboratory sample was disturbed during the sampling process or during the laboratory instigations.

Stage no. 5 consisted of an iterative and complex process of selection of the remaining laboratory results. For example, in the Bucharest Loam case, some results with significant deviation from the mean were found to come from a single borehole presenting a small sandy lens at a specific level.

The obtained correlations for the "Bucharest Loam" and "Colentina sands and gravels" layers are presented in subchapters 5.1 – 5.5.

For the **Bucharest Loam** layer correlations between q_c (CPT) and the E_{oed} and φ and c parameters were studied. For each of the remaining laboratory samples, after the selection stage 5, a single q_c value was associated. As mentioned in subchapter 2, the soil sample had a length of 25 to 30 cm. Thus, the attributed single value was obtained by averaging all 25 to 30 q_c readings obtained at the corresponding depth of the specific laboratory sample.

The E_{oed} modulus was obtained during a usual incremental loading (IL) oedometer test, between 200 and 300 kPa normal stress. To determine φ and c parameters, direct shear box tests were performed under consolidated - undrained conditions. The specimens were consolidated under the following stresses: 100 kPa, 200 kPa and 300 kPa. Both the oedometer and the direct shear box tests were performed on saturated samples.

For the **Colentina sands and gravels** layer the correlations between q_c (CPT) vs. the M_{DMT} and q_c (CPT) vs. φ (DMT) were determined. The correlation was obtained as following: for each DMT value (a reading every 25 cm) an average value of the 25 q_c readings was attributed, corresponding to the depth of the DMT reading.

For both **Bucharest loam** and **Colentina sands and gravels** layers, graphs were constructed, for each of the parameter combination mentioned above, in which the vertical axis represents the parameter for which a correlation is desired and on the horizontal axis the input parameter.

Using the graphical data, linear regression with a 95% confidence level was used to determine linear correlations between parameters.

In the graphs presented in subchapter 5.1 to 5.5, one can observe with blue diamond-shaped dots the pair of values of the parameters obtained from laboratory and field investigations. With red-squared dots, the values of the parameters from the obtained correlations, presented in the tables below each graph, are presented.

Subsequently, in tables 1 to 5 the obtained correlations are presented. One can see, on the first line of each table the actual correlation and on the second line a "safe" correlation. The so-called "safe" was determined according to NP 122-2010 [10].

The Multiple R and the R square of each correlation have values greater than 0.7, which means that the linear correlations can describe the relation between the studied parameters with sufficient accuracy.

5.1. Correlations between q_c and E_{oed} Bucharest Loam

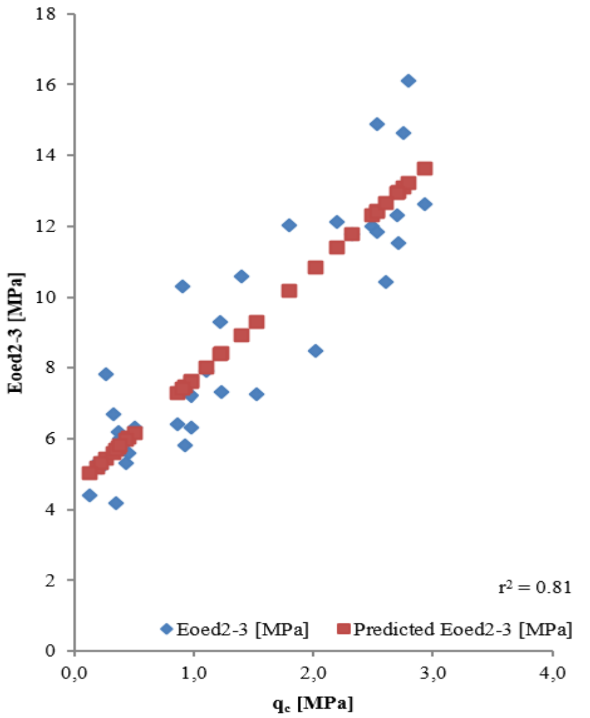

Figure 8. Correlations between q_c and E_{oed} for Bucharest Loam layer

Table 1. Correlations between q_c and E_{oed} for Bucharest Loam layer

Primary correlation	$E_{oed}2\text{-}3 = 3{,}1q_c + 5$ MPa
Secondary correlation	$E_{oed}2\text{-}3 = 3{,}1q_c + 2{,}6$ MPa

5.2. Correlations between q_c and tanφ Bucharest Loam

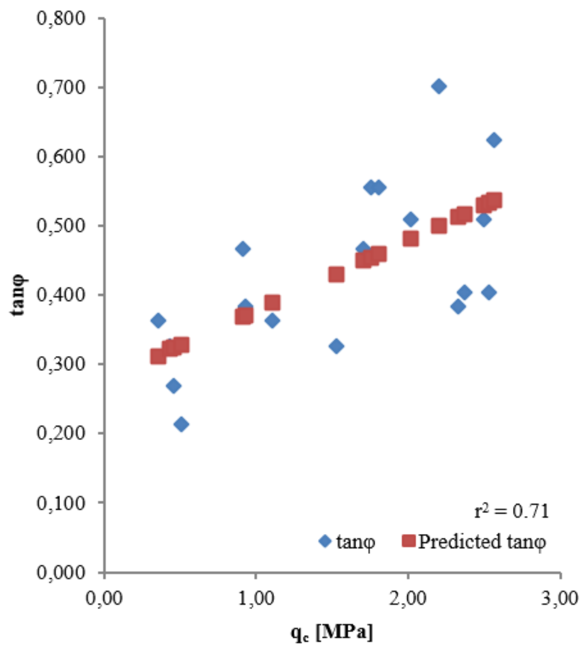

Figure 9. Correlations between q_c and tanφ for Bucharest Loam layer

Table 2. Correlations between q_c and tanφ for Bucharest Loam layer

Primary correlation	$\tan\varphi = 0{,}1q_c + 0{,}27$
Secondary correlation	$\tan\varphi = 0{,}1q_c + 0{,}1$

5.3. Correlations between q_c and c_{cu} Bucharest Loam

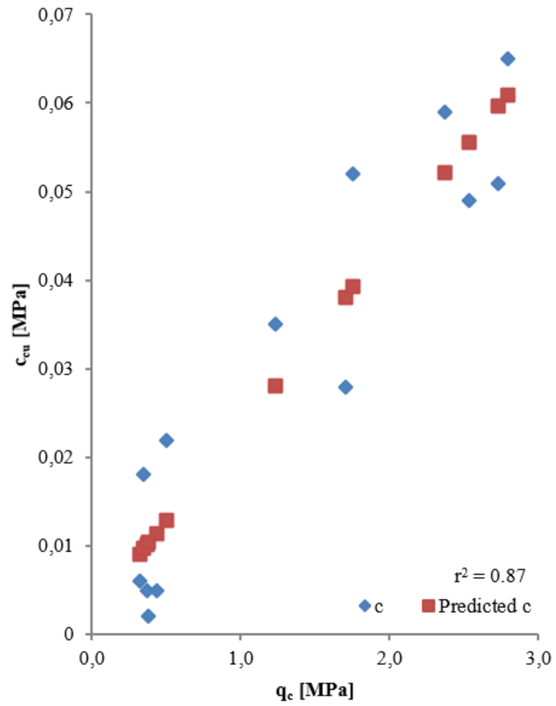

Figure 10. Correlations between c_{cu} and q_c for Bucharest Loam layer

Table 3. Correlations between c_{cu} and q_c for Bucharest Loam layer

Primary correlation	$c_{cu} = 0{,}021q_c + 0.001$ kPa
Secondary correlation	$c_{cu} = 0{,}02q_c$ kPa

5.4. Correlations between q_c and M_{DMT} Colentina sands and gravels

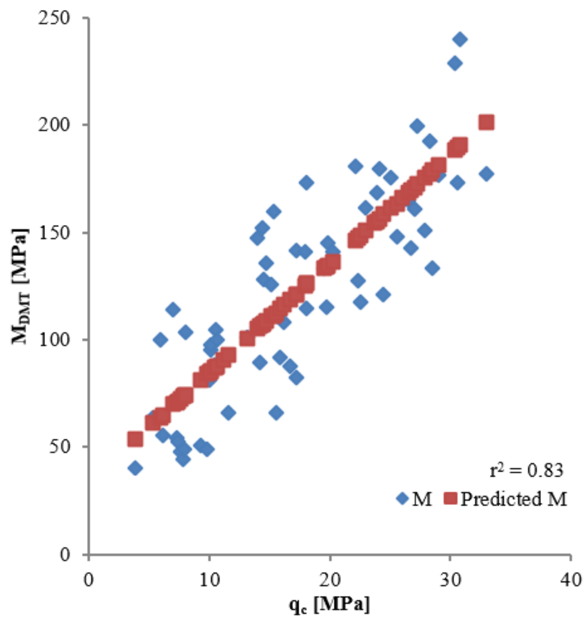

Figure 11. Correlations between q_c and M_{DMT} for Colentina sands layer

Table 4. Correlations between q_c and M_{DMT} for Colentina sands layer

Primary correlation	$M_{DMT} = 5.1\ q_c + 34$ MPa
Secondary correlation	$M_{DMT} = 5.1\ q_c - 8$ MPa

5.5. Correlations between q_c and tanφ Colentina sands and gravels

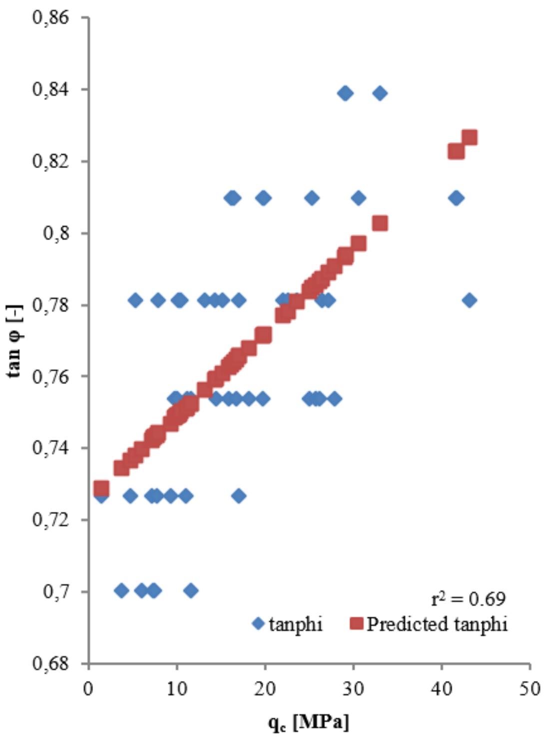

Figure 12. Correlations between q_c and tanφ for Colentina sands layer

Table 5. Correlations between q_c and tanφ for Colentina sands layer

Primary correlation	tanφ = 0.003 q_c + 0.72
Secondary correlation	tanφ = 0.003 q_c + 0.67

6. Case Study Building 3UL+GL+11S

In this chapter, a case study for an office building located in the Central Western part of Bucharest is presented. The building has 3 Underground levels, ground floor and 11 stories. The building has a polygonal shape and a footprint area of approximately 5,000 m^2. The excavation support consisted of diaphragm walls made of reinforced concrete, supported by horizontal steel struts.

In order to calibrate the geotechnical parameters to the parameters required by presented constitutive models, Mohr-Coulomb and Hardening Soil, soil-structure interaction analyzes were carried out.

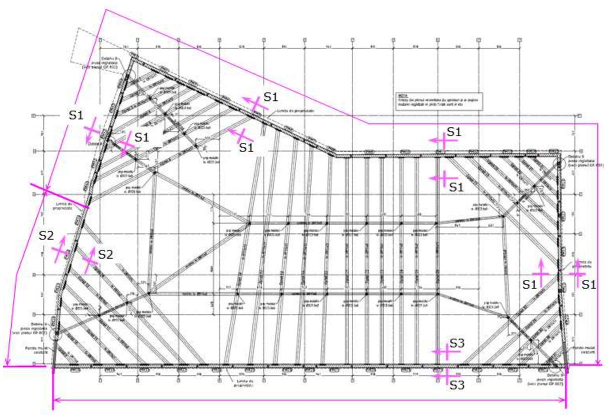

Figure 13. General layout of the excavation

The analysis of the results of the soil-structure interaction involves the calculation and verification of the displacements, deformations and stresses in the foundation elements and throughout the soil. Because only the displacements and deformation were monitored and not the actual stresses, in order to achive a better estimation of the acual loads during the monitoring process, the permanent loads were considered without a safety factor and the variable loads were multiplied by 0.5. This load combination is reffered further as “the long term load combination” or “long term loads”.

The evaluation of the soil-structure interaction was performed under the plane state of deformations, using the finite element method and Plaxis 2D 2017 software.

The geotechnical parameters were provided by a detail investigation campaign, which featured in situ and laboratory investigations. It is worth noting, that those parameters were part of the determined correlations, presented in Fig. 8 -12.

The resulted characteristic parameters were used as input data for the two constitutive models, presented in chapter 4.

Table 6. Input parameters for the Mohr-Coulomb model

Bucharest Loam	Colentina sands and gravels
E_{oed} = 12 MPa	E_{oed} = 40 MPa
φ' = 25°	φ' = 32°
c' = 40 kPa	c' = - kPa
γ = 18.5 kN/m3	γ = 20 kN/m3
ν = 0.35	ν = 0.25

Table 7. Input parameters for the Hardening Soil model

Bucharest Loam	Colentina sands and gravels
E_{50} = 12 MPa	E_{50} = 40 MPa
E_{ur} = 48 MPa	E_{ur} = 120 MPa
φ' = 25°	φ' = 32°
c' = 40 kPa	c' = - kPa
γ = 18.5 kN/m3	γ = 20 kN/m3
ν = 0.35	ν = 0.25
m = 0.5	m = 0.7

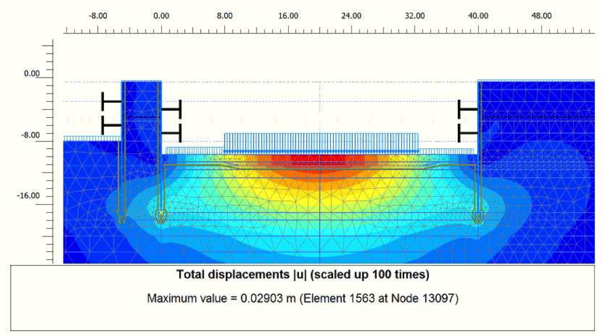

Figure 14. Total displacements of the raft under the long term loads (Mohr Coulomb model)

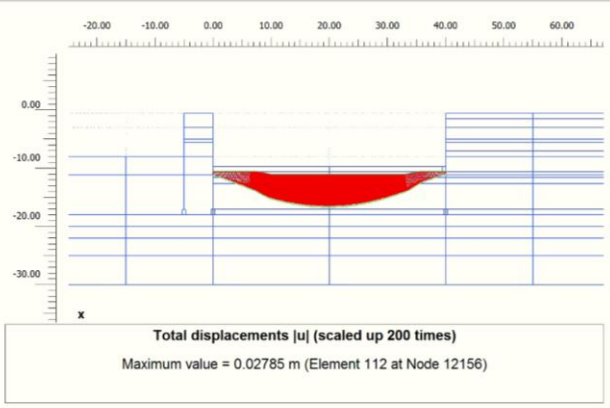

Figure 15. Total vertical displacements of the raft under the long term loads (Mohr Coulomb model)

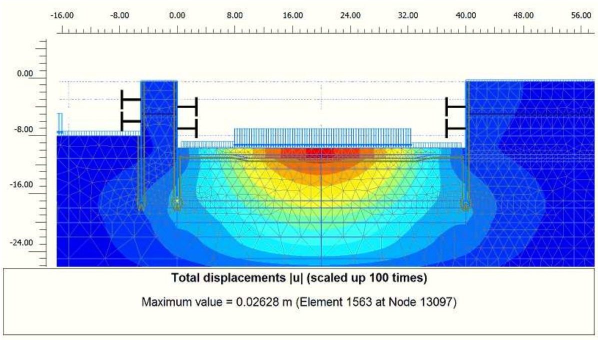

Figure 16. Total displacements of the raft under the long term loads (Hardening Soil model)

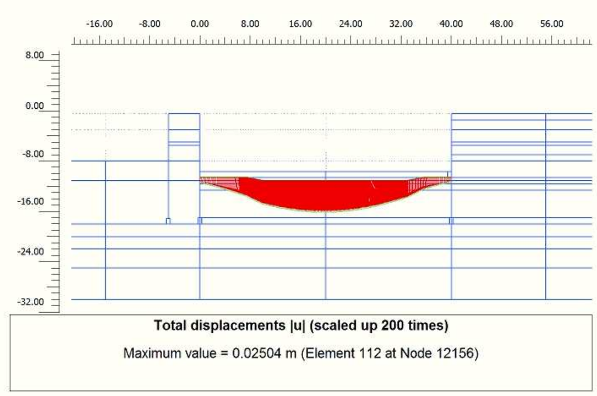

Figure 17. Total vertical displacements of the raft under the long term loads (Hardening Soil model)

In the figures above, the results of the initial calculation in terms of vertical deformations of the ground underneath the raft for long-term loads, obtained by numerical analysis, are presented.

Fig. 14 to 17 presents the deformation of the slab under long-term loads, in both Mohr-Coulomb and Hardening Soil models. As it can be seen the settlements under the slab are approx. 2.7 cm in case of the Mohr-Coulomb modeling and about 2.5 cm when Hardening Soil model was used.

7. Results of the FEM modeling

7.1. Results for the Mohr-Coulomb model

The choice of an advanced model for the simulation of the behavior of a soil is justified by the fact that an elasto-plastic model is not able to capture the stress-strain non-linearity, the stiffness depending on the stress level and on the inelastic behavior of a soil.

The choice of the initial set of parameters, presented in Tables 6. and 7., is based on the results of the in-situ investigation investigations and the correlations presented in sub-chapter 5.1.

The parameters were subsequently calibrated in order to obtain the so-called “best-estimated values”. The calibration was performed in two steps.

The first step, generically referred to as "Step 1", modeled the interaction of the new structure with the foundation ground, regarding the calculation of the settlements using the long-term loads. This model was helpful in the calibration, mainly of the parameters of the deformation moduli below the foundation ground, at approximately 9-10 m depth (Colentina sands and gravels). The parameters were subsequently changed so that the settlement curve resulted from the FEM model mimic the settlement curve obtained during the geotechnical modeling of the structure.

During the second step, the behavior (lateral displacement) of a reinforced diaphragm wall was modeled, which corresponded to Section 3-3, Fig. 13. Section 3-3 was monitored, during the construction, using an inclinometric column. The model obtained in Step 2 aimed mainly the calibration of the shear parameters and the fine-tuning of the geotechnical parameters that describe how a soil behaves when subjected to deformation.

To achieve the calibration, during the modeling of each of the two steps, multiple sub-steps were performed in which the parameters were varied in a way that the evolution of the results could be monitored.

In case of step 1, as only the deformation moduli were varied during FEM modeling, it was proceeded as described below. The measured settlements of the raft, on the last construction stage is marked in Fig. 18 and 20 with a red line. One can observe the initial modeled settlements, marked in green in Fig. 19 and 21. The calibrated settlement curves are marked in Fig. 18 and 20 with a violet line. Each sub-step corresponded to an increase in the deformation parameter E_{oed} with 5%, from its initial value, for the layers in which, one can observe that the FEM model presented a higher settlement value than the measured one. The parameter was increased, for each soil layer independently, until each FEM modeled settlement point was as close as possible as the measured settlement point.

Similar to what is described in the paragraph above, the modeled displacement curves, corresponding to the displacement of the diaphragm concrete wall, were calibrated to fit the measured displacement curves. The only difference is that all 3 studied parameters (E_{50}, c and φ') where changed, with an increase or decrease of 5% of their initial value. In Fig. 19 and 21, the red curve corresponds to the inclinometer measurements, the green line corresponds to the initial modeled curve and the violet line corresponds to the final calibrated curve.

Using the procedure described above, the calibrated parameters for Mohr Coulomb and Hardening Soil models were obtained.

The results are presented in Tables 8 and 9.

Figure 18. Settlement evolution – Monitoring vs. Mohr-Coulomb model [mm]

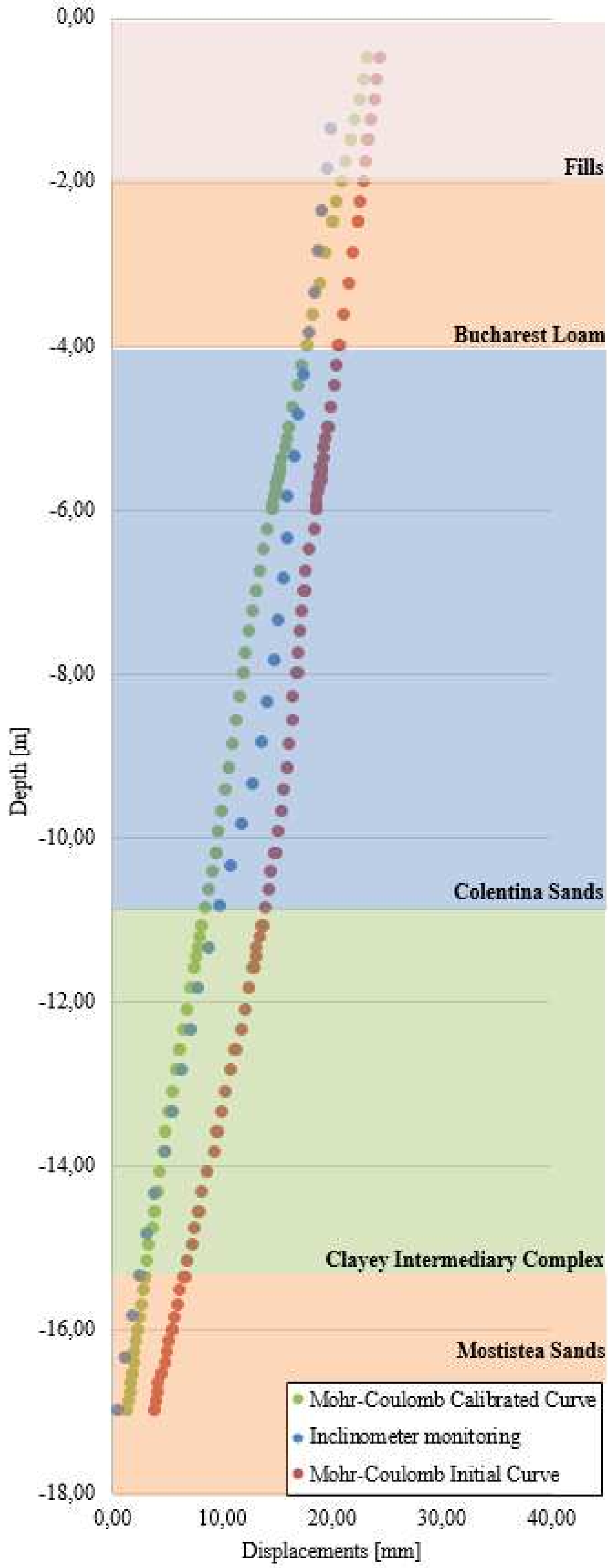

Figure 19. Horizontal displacements – Inclinometer monitoring vs. Mohr-Coulomb model

Table 8. Calibrated parameter correlation for Mohr-Coulomb model with the best estimated values

Layer	Correlation for Mohr-Coulomb
Bucharest Loam (26.5°)	tanφ' = 0,2 • qc + 0,1
Bucharest Loam (64 kPa)	c' = 0,032 • qc kPa
Bucharest Loam (19 MPa)	E_{50} = 3.1 • qc + 5 MPa
Colentina Sands (34°)	tanφ' = 0,035 • qc
Colentina Sands (50 MPa)	E' = 3 • qc – 8 MPa

7.2. Results for Hardening Soil model

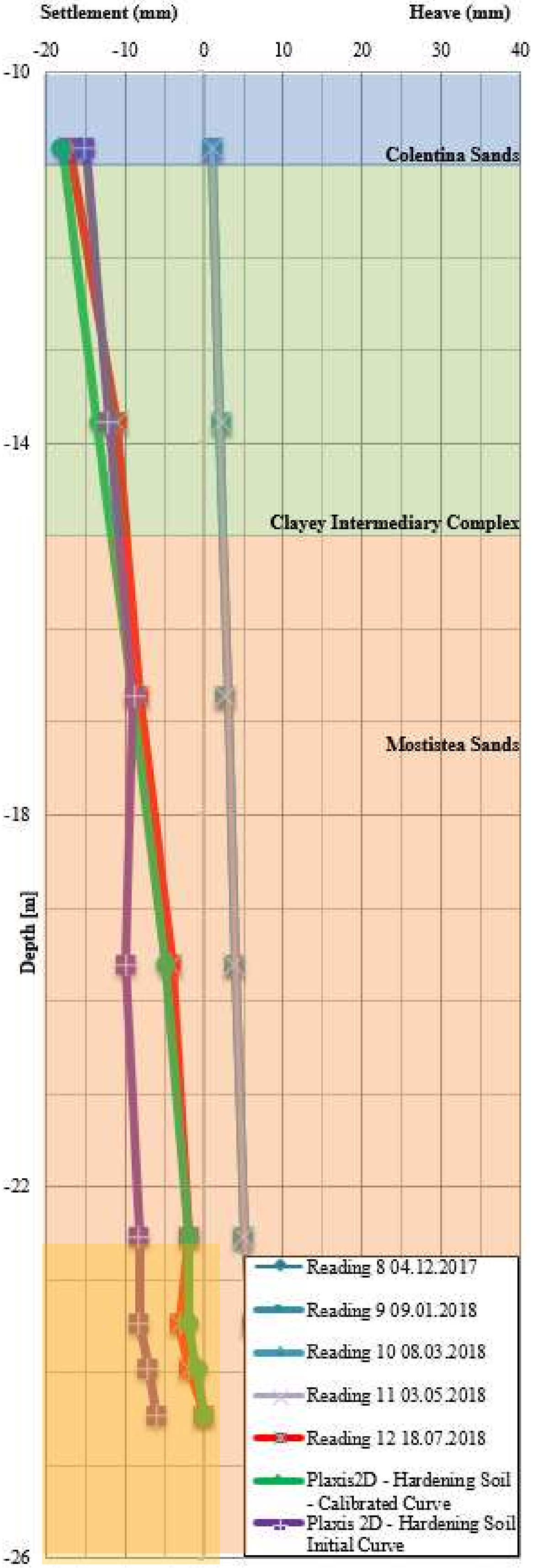

Figure 20. Settlement evolution – Monitoring vs. Hardening Soil model [mm]

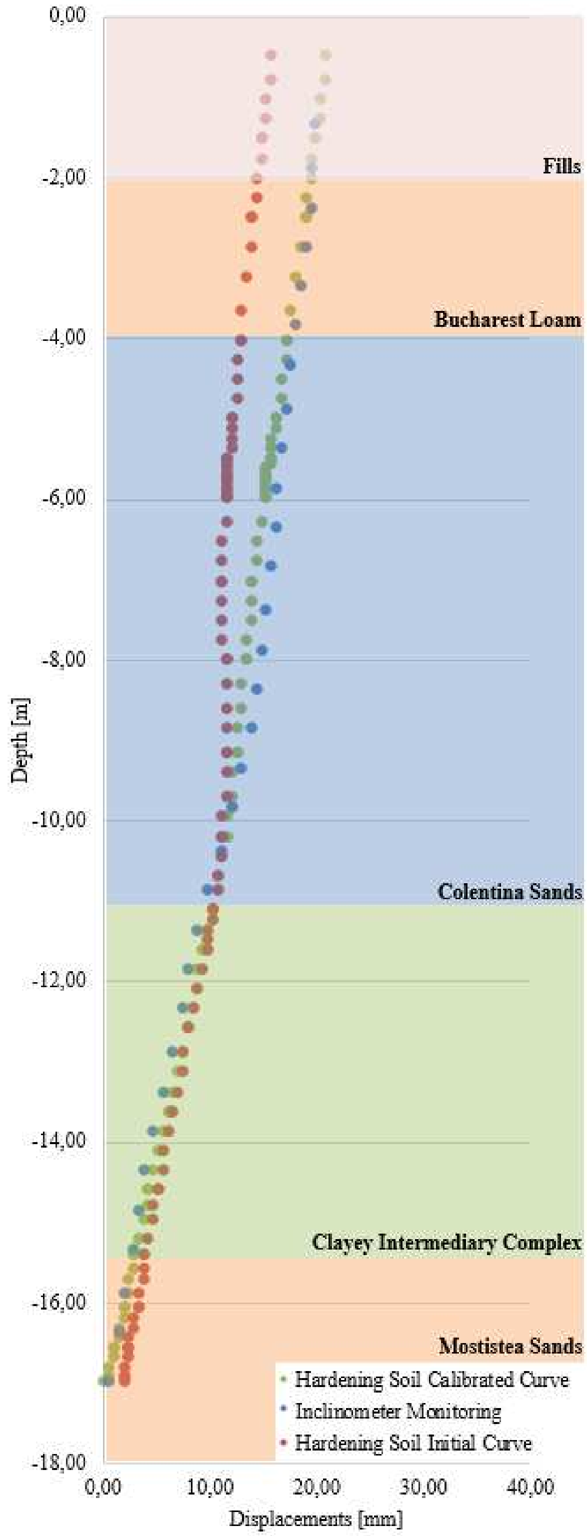

Figure 21. Horizontal displacements – Inclinometer monitoring vs. Mohr-Coulomb model

Table 9. Calibrated parameter correlation for Hardening Soil model with the best estimated values

Layer	Correlation for Hardening Soil
Bucharest Loam (20°)	tanφ' = 0,13 • q_c + 0,1
Bucharest Loam (56 kPa)	c' = 0,028 • q_c kPa
Bucharest Loam (17 MPa)	E_{50} = 3,5 • q_c + 3
Colentina Sands (38.5°)	tanφ' = 0,015 • q_c + 0,5
Colentina Sands (42 MPa)	E_{50} = 2,2 • q_c - 5 MPa

8. Conclusions

The present paper presents the calibrations of some correlations for the typical first two soil layers in Bucharest area.

The first chapter is a brief introduction on the theme of the paper and it gives details about the main objectives. The evaluation of correlations between different geotechnical parameters was performed for two lithological layers. Starting from the surface "Bucharest Loam" and "Colentina sands and gravels" layers were analyzed. These layers were chosen because they significantly influence most of the geotechnical works, such as direct foundations, deep foundations, shoring works and tunnels as well.

An exemplary advanced geotechnical modeling was performed to simulate the soil behavior using two constituent models, Mohr-Coulomb and Hardening Soil. To verify and calibrate the correlations the method of back-calculation was used, using the data provided by the geotechnical monitoring of the structures during and after construction. The instruments used to monitor the structures, presented in this report were inclinometers and extensometers.

The second chapter is focused on a brief presentation of the geotechnical investigations program. The in situ investigations consisted in static cone penetration tests (CPT) and Flat Dilatometer Marchetti Tests (DMT) tests.

Together with the in situ investigations, geotechnical laboratory tests were conducted to determine the physical and mechanical parameters of the soil.

The third chapter briefly presents the aforementioned monitoring instruments and the underlying technology.

In Chapter 4, the constitutive models used for determining the correlations are briefly presented. They are based on complex equations that try to describe the behavior of the soil as accurately as possible. Choosing a specific model for the design of a geotechnical structure must be performed with great care, since the complex models do not necessarily give better results.

According to the graphs from Chapter 2, one can say that the q_c parameters for the Bucharest Loam located in the central part of Bucharest are 50% higher than the average, while for the Colentina Sands and Gravels, the West area presents q_c values 10% higher than the average. It is worth noting that regarding M_{DMT} values, they tend to follow the same trend as the CPT values. Nevertheless, both q_c and M_{DMT} determined values of the presented Case Study are about average, as it can be seen in Fig. 3, 4 and 5.

Regarding the shear strength parameters (φ and c), obtained from the proposed correlations, specific to the "Bucharest Loam" layer, the following conclusions can be drawn. The differences between the values obtained using the proposed correlations and the correlations in the literature are approx. 40% for the internal friction angle and maximum 30% for cohesion. The proposed correlation for the internal friction angle tends to underestimate the determined value of the parameter with approx. 30%, while the correlations in the literature presented in EN 1997-2:2007, tend to overestimate the parameter with approx. 10%. In case of cohesion, the values obtained from the correlations, proposed in the paper differ from the correlations in the literature with a maximum of 25% (the correlation between q_c obtained in-situ from CPT and c_{cu} obtained from laboratory tests).

Chapter 6 presents a Case Study, in which the previously determined parameters (Table 6 and 7) were used as input data, with their characteristic value for FEM back-calculations. The aim of the FEM back-calculation was to obtain plausible correlations which can lead to the "best-estimated values" for the studied parameters (Table 8 and 9). Those back-calculations were necessary to verify the FEM results by comparing them to the curves and graphs resulted from the geotechnical monitoring. In terms of total settlements and deformations, the differences between the two models was relatively small, with a total settlement under the raft of 2.7 cm in case of the Mohr-Coulomb model and 2.5 cm for Hardening Soil model, or a difference of 8%. It should be noted, that with regards to the lateral deformations measured using an inclinometer, as presented in Fig. 18 to 21, one can state that the FEM model which used the Hardening Soil model provides results which fit better to the measured ones.

It is worth mentioning that the resulted correlations obtained using advanced constitutive models generate deformations relatively closer to those obtained directly from the geotechnical modeling. This can be related to the fact that, advanced soil models like Hardening Soil tend to better simulate the soil behavior.

Additional case studies are programed to be analyzed in order to confirm, validate and eventually correct the correlations presented in this article. Based on the obtained results, useful conclusions can be drawn about the validity of the used correlations and, eventually, the correlations will be adjusted.

References

[1] Liteanu, E. "Geologia Bucurestiului" (Geology of Bucharest), St.Tehn.Econ., E, 1, Bucharest, Romania, 1951 (in Romanian)

[2] EN 1997-1:2004 Eurocode 7: Geotechnical design - Part 1: General rules, European Committee for Standardization, Brussels, 2004

[3] EN ISO 22476-1 Geotechnical investigation and testing — Field testing — Part 1: Electrical cone and piezocone penetration test, International Organization for Standardization, 2012

[4] ASTM D1587 / D1587M – 15 Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes, ASTM International, 2015

[5] EN 1997-2:2007 Eurocode 7: Geotechnical design Part 2: Ground investigation and testing, European Committee for Standardization, Brussels, 2007

[6] Hashash, Y. M. A.; Finno, R. J. "Development of new integrated tools for predicting, monitoring, and control-ling ground movements due to excavations". Practice Peri-odical on Structural Design and Construction, v. 13, p. 4, 2008

[7] Diaconu, D., "Bazele calculului automat al structurilor cu metoda elementului finit" (The basis of automatic calculation of structures with the Finite Element Method), Ph.D. Thesis, 2016, (in Romanian)

[8] Lo Presti, D., Pallara, O., Fioravante, V., and Jamiolkowski, M.. "Assessement of quasi-linear models for sands". In "Pre-failure Deformation of Geomaterials", pages 363–372, London, 1998

[9] Finno, R. and Calvello, M. "Supported excavations: the observational method and inverse modeling" J Geotech Geoenv Engng ASCE, 131(7):826–836, 2005

[10] NP122:2010 "Normativ privind determinarea valorilor caracteristice si de calcul ale parametrilor geotehnici" (Norm regarding the determination of the characteristic values and of calculation of the geotechnical parameters), ASRO, 2010, (in Romanian)