TECHNICAL UNIVERSITY OF CIVIL ENGINEERING
BUCURESTI

DOCTORAL SCHOOL

THESIS

CORRELATIONS BETWEEN IN SITU INVESTIGATIONS AND GEOTECHNICAL PARAMETERS. VALIDATION BY MONITORING THE BEHAVIOUR OF CONSTRUCTIONS OVER TIME

Summary

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Bucharest, 2023

CUPRINS

1 INTRODUCTION

Today, in situ geotechnical investigation methods are experiencing an upward trend, exponentially, one could say, in their use, especially at national and international level. In practical terms, one can observe an alignment of geotechnical investigation methods in Romania with those present in Western Europe and in the United States. It should be noted that the existing practice in Romania is defined by an affinity for geotechnical laboratory tests, compared to the international situation where laboratory tests were correlated with in-situ investigations. Internationally, in-situ tests are continuously developing, mainly due to the lower investigation costs and the reduced time of investigation and interpretation. However, in-situ tests require either the application of correlations with geotechnical parameters used in calculations or the adaptation of calculation/design methods to the direct use of field test results. Digitization, automation and increased computational power in the construction industry has favored the adoption and use in current practice of correlations between in-situ investigations and geotechnical parameters. The broadening of the spectrum of ground investigation coupled with the harmonization of European construction standards has led to the development of new standards or the updating of existing technical standards.

In the international literature there are numerous correlations between in-situ tests and geotechnical parameters of soils that have been elaborated and developed mainly in Western European countries, the United States and Japan. In the national literature there are correlations determined for specific soils in Romania, but their number is very limited, mainly due to the low use of in situ tests. Also, a good part of them were determined several decades ago. The limited existence of "national" correlations leads to the under- or inappropriate use of field tests and to an excess or lack of caution in establishing characteristic values of geotechnical parameters. In addition, most calculation programs use predefined correlations, most of which are those described in the international literature. In order to avoid possible design errors caused by incorrect use or without proper prior documentation, existing correlations - see chapter 5 - must be validated or adapted to the specific soil types of our country.

Both the current revision of SR EN 1997 and the 2022 edition of NP 074 focus on documenting the correlations used, their justification to ensure the minimum reliability required by these standards. This paper contributes to a better understanding of how to select, interpret and apply correlations between geotechnical soil parameters and in situ test results for soils specific to the Bucharest area. The main novel contribution of the paper is the determination of new correlations between CPT and DMT in situ tests and the usual laboratory tests for Bucharest specific layers. Based on the literature review, an extensive investigation program of the soil was carried out, including in situ tests and geotechnical boreholes. Laboratory tests were carried out on disturbed and undisturbed soil samples obtained from the geotechnical boreholes to determine geotechnical parameters. Parallel analysis of the geotechnical parameters obtained using the newly determined correlations and separately using laboratory tests helped to validate the proposed correlations. Additionally, validation of the proposed correlations was performed by performing back calculations using a finite element calculation program. The validation of the numerical model was possible using the results of the geotechnical monitoring during construction and operation.

2 INVESTIGATION, DESCRIPTION AND CHARACTERISATION OF SOILS

The soil is often characterized as a heterogeneous material, both in terms of depth and horizontal variation of layers' characteristic parameters. Added to this is the possibility of groundwater variation

Geotechnical investigations are required to characterize the soil and determine its physical and mechanical characteristics. Geotechnical investigations consist of methods of determining the physical and mechanical characteristics of geological layer. Depending on the method of determination, geotechnical investigations are classified into field or in- situ tests and geotechnical laboratory tests.

Lack of knowledge or absence of certain information about the soil may lead to inaccurate design geotechnical structures. The latter may result in local or general failure of the built structures. The figure below (Rizkallah & Döbbelin, 1998) presents an analysis of failure cases during excavations, showing that about one in three structural fails occur due to poor investigation of the soil.

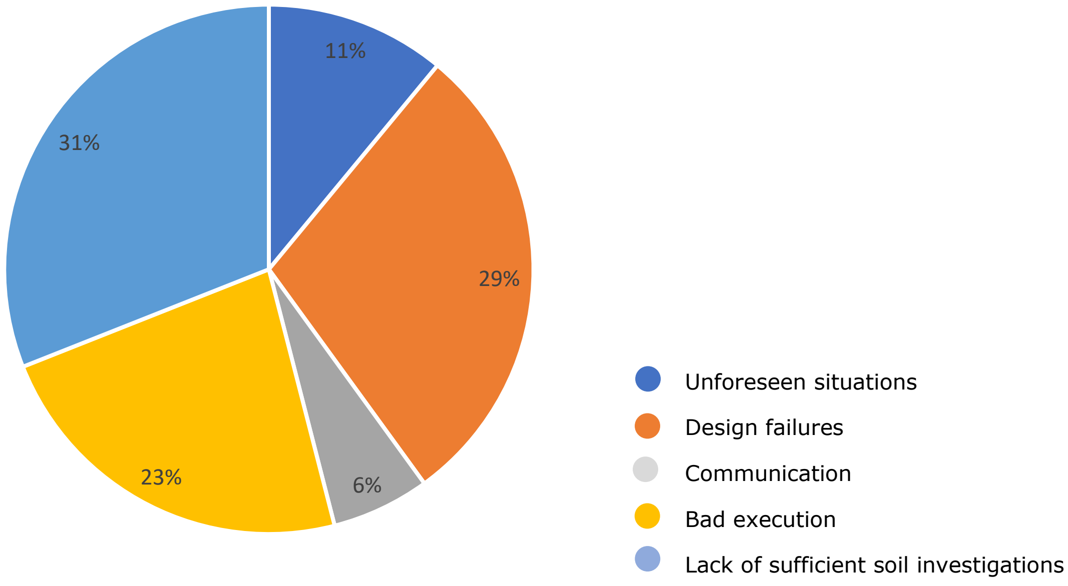

Figure 2-1 Failure of excavation support (Rizkallah & Döbbelin, 1998)

The main ways of investigating the soil are presented below.

As presented above, in situ and/or laboratory tests can be used to determine soil parameters. The methods of determining geotechnical characteristics in situ, depending on the type of test, are divided into direct and indirect. While direct tests obtain information directly from the test, e.g. the sequence of layers during drilling or certain pressures within the soil mass, indirect tests use correlations to determine specific characteristics.

In Chapter 2 of this paper, the main ways of determining the physical and mechanical parameters of soils are presented as part of the review of the existing literature in the field. These modalities are summarized below:

- Boreholes and excavation pits: These allow determination of the sequence of relevant layers, groundwater levels and the collection of disturbed and undisturbed samples that are used for geotechnical laboratory tests.
- Direct field tests: These include static and dynamic plate testing, which allows the determination of the bearing capacity of the soil and the deformation moduli of a soil.
- Indirect in-situ geotechnical investigations: The most common in-situ investigations include dynamic penetration tests and surveys such as SPT and PDU/PDG, CPT/CPTu cone penetration surveys and Menard or Marchetti Pressuremeter tests. Other indirect in situ methods include geophysical tests, field vane test or pocket penetrometer tests.
- Geotechnical laboratory tests: These are necessary for describing soil behavior and include identification and classification tests as well as tests to determine the mechanical behavior of soils.

Using these methods, the physical and mechanical properties of soils can be determined, providing essential information for geotechnical analysis and design.

3 GEOTECHNICAL MONITORING

Due to the inhomogeneous nature of the soil, even in the most fortunate cases where sufficient geotechnical investigation points are available, it cannot be fully characterized. For this reason, tools must be provided during the design process to monitor its behavior over time, so that the plausibility of the design assumptions can be checked during and after construction. Also, when applying the observational design method, as required by SR EN 1997-1:2007, it is mandatory to install a monitoring system.

"The justified uses of instrumentation are so many, and the questions that instruments and observations can answer, so vital, that we should not risk discrediting their value by using them inappropriately or unnecessarily" - Ralph Peck (1984)

Geotechnical monitoring instruments are devices or systems used to monitor deformations, displacements, stresses, etc. in geotechnical projects requiring such monitoring. Geotechnical instruments and monitoring are essential for the successful completion of geotechnical projects. The complexity of geotechnical monitoring varies according to the degree of difficulty of the construction. This can range from simple settlement monitoring for low complexity structures to the use of a wide range of monitoring instruments, devices and software for complex projects such as tunnels, landslides and deep excavations in urban areas.

The general purpose of monitoring is to collect information about the behavior of a material under certain stresses and strains and their variation over time. In the case of geotechnical monitoring, the behavior over time of a soil or rock is observed.

Based on in-situ and geotechnical laboratory tests, the geotechnical parameters of the soil are determined. In the design of geotechnical structures, the geotechnical parameters thus determined are used as input data for determining the soil structure interaction. Thus, forces and deformations are determined in the supporting structures, in the foundation elements or in the soil mass, etc. With the help of these data, the cross-sections of the concrete elements, the cross-section of the reinforcement or the thickness of the backfill and other structural elements are dimensioned.

As mentioned above the structural elements are dimensioned with a certain degree of accuracy. In order to be able to verify the assumptions made underlying the determination of the interaction of the ground with the structure, it is necessary to install geotechnical monitoring instruments to carry out, as appropriate, measurements before, during or after the execution of the designed structures. In geotechnics the most relevant cases are the monitoring of the displacement of deep excavation support, settlement of the soil mass under newly constructed foundations and piles during load tests or during operation.

Monitoring equipment can also be installed directly in the ground for studying displacements, groundwater levels and the variation of long- or short-term stresses. An example is geotechnical monitoring of active and inactive landslides, seismic monitoring or long-term groundwater level monitoring. Other common examples where geotechnical monitoring provides useful information are newly constructed embankments or road foundations.

Chapter 3 discusses the main modalities and tools used in current engineering practice for geotechnical monitoring. The general purpose of geotechnical monitoring is to collect information on the behavior of a material subject to specific stresses and / or strains, in this case a soil or structural element, and the variation of this behavior over time. In geotechnical monitoring, the behavior of a soil or rock formation is monitored over time. Collecting, analyzing and interpreting this information is necessary both during the execution of structures and after their completion in order to assess the correctness of the assumptions made at the design stage. If there are differences from the designed situation, corrective action can be taken in a timely manner, thus reducing the risk associated with unforeseen failures.

4 NUMERICAL MODELLING IN GEOTECHNICAL ENGINEERING

Soil is a complex material with non-linear and anisotropic behavior, which varies over time when subjected to external or internal (self-weight) stresses. Typically, it behaves differently under loading, unloading and reloading. By studying the loading diagrams of a soil, its complex behavior can be observed.

In order to model the behavior of the soil certain assumptions have to be made. These assumptions simulate the soil's behavior and can be express in a mathematical formula. Various safety factors or partial safety coefficients are applied to the results of the investigation in order to cover the differences between the calculation and the actual behavior of the soil.

The important thing in choosing a calculation method is first to determine its complexity. Experience showed that simple methods proved to be effective and have in most cases delivered comprehensive results when it comes to modelling soil's behavior.

With the development of increasingly complex computing technology and software applications, it has also been possible to develop complex numerical methods that are accessible to design teams. With regard to complex numerical methods, it should be noted first of all that no matter how complex the chosen method is, it cannot fully describe the behavior a soil. At the same time, the use of a more complex models does not guarantee a more accurate design. In addition, complex calculation methods implicitly require the allocation of more computational resources, software and finance than simpler methods, without guaranteeing a better result. Their use by inexperienced engineers with limited understanding of the mathematical principals that are used can lead to erroneous modelling and in some cases even to failures.

Chapter 4 provides a brief introduction to numerical modelling in geotechnical engineering. Various numerical methods commonly used in geotechnical engineering are briefly presented, such as the finite element method (FEM), the finite difference method (FDM) and the discrete element method (DEM). Subsequently, the finite element method is detailed by discussing element types, boundary conditions and discretization. The main constitutive models and yield models are then presented.

5 CORRELATIONS BETWEEN GEOTECHNICAL PARAMETERS DETERMINED IN SITU AND IN THE LABORATORY

In situ geotechnical investigations are an increasingly common method for determining the geotechnical parameters used in the design and execution of permanent or temporary structures. With this type of investigation, it is possible to obtain soil parameters faster and with a reduced cost. In most cases the level of soil disturbance during in-situ testing is considered to be lower than when sampling and testing in the laboratory. However, it should be noted that in the laboratory, by using complex tests such as triaxial tests, the initial stress state can be simulated as well as the future stress states specific to the construction time. For example, in-situ information on the collapsibility of the soil exposed to water is impossible, in which case laboratory tests are the only relevant tests. In most cases, a combination of in-situ and laboratory tests leads to a thorough investigation of the foundation ground.

Field investigations usually yield information such as pressures (in the case of Pressuremeter tests), push/pull resistances (in the case of CPT) or a number of blows for a given penetration length of the foundation in case of dynamic probing. This information must then be correlated to obtain geotechnical parameters of a soil, such as density, deformation moduli or shear parameters. The correlations are obtained either through parallel analyses of laboratory results or by performing back calculations based on information obtained from geotechnical monitoring during construction and service time.

Most of the time these correlations are available in the literature or are incorporated in calculation software, but often without knowing under what conditions they have been determined, for what types of soils they are valid and especially what the degree of correlation is.

5.1 CORRELATIONS BETWEEN GEOTECHNICAL PARAMETERS

In this subchapter correlations between different geotechnical parameters obtained both in situ and in the laboratory are presented. Based on several types of tests on the same type of soil, correlations between different geotechnical parameters of soils have been determined over time by different methods and by different authors. These correlations are valid for the soils for which they have been determined and cannot always be extrapolated to other similar soil types without prior validation. The use of these correlations depends largely on detailed knowledge of the conditions of their determination.

According to Figure 5-1 a variation of cone factors can be observed depending on the soil type. Thus, higher values of cone factors are recommended for clayey soils, while silty and sandy soils show lower values of N_c.

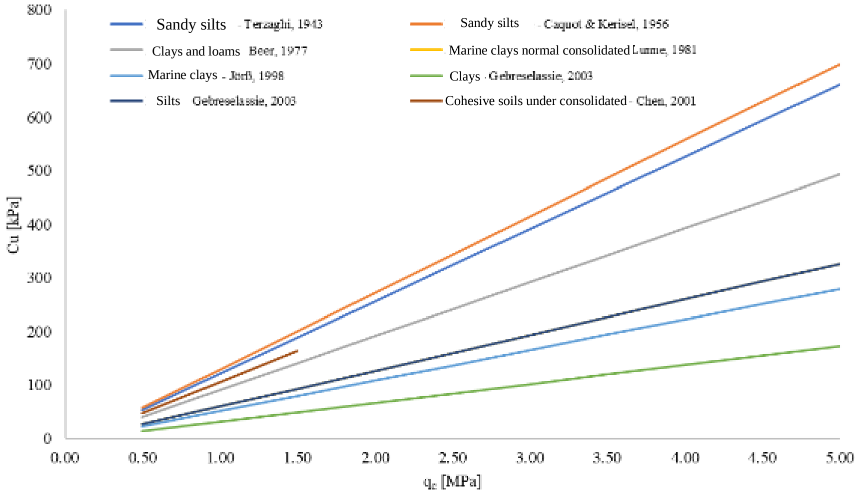

Figure 5-1 Variation c_u as a function of the correlation used

As shown in Figure 5-2, different correlations exist depending on the soil type. The most conservative values are obtained using the correlation proposed by (Hettiarachchi, 2008)while using the correlation proposed by (Hara, Ohta, Niwa, Tanaka, & Banno, 1974) results in less conservative values. Unlike the other correlations (Stroud & Butler, 1975) allows a differentiation between soils of medium to high plasticity and soils of low plasticity.

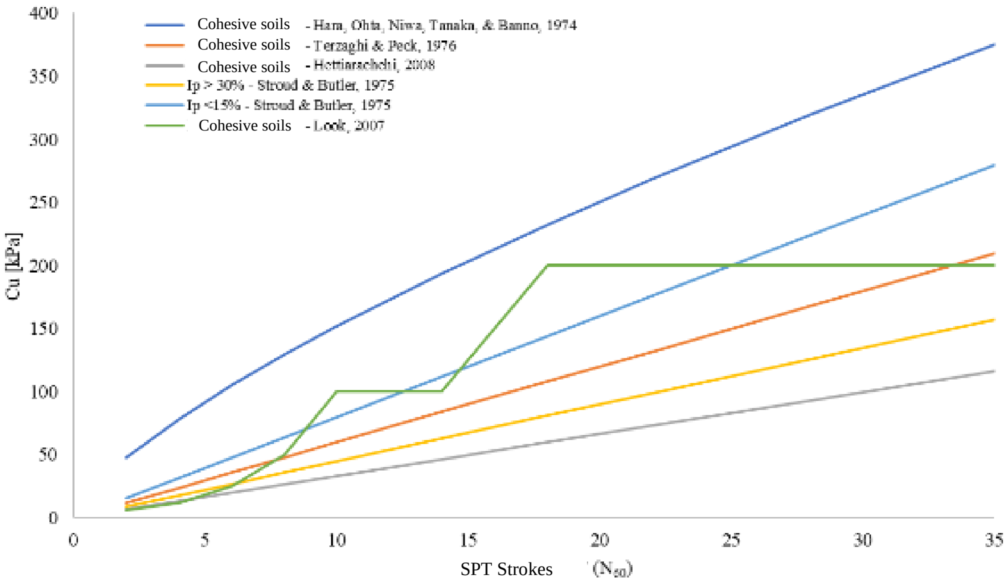

Figure 5-2 Correlation of the number of SPT hits N_{60} with the value of the undamaged shear strength

(Butcher et all, 1995) proposes three correlations for obtaining shear strength using dynamic penetration resistance. The general formula proposed by him gives shear strength values similar to the formula for soft clays. Using the correlation valid for hard clays (c_u > 50 kPa) the values of c_u are significantly higher, which in turn are closer to the values obtained using the correlation proposed by (Langton, 2000). All the above presented correlations are presented in Figure 5-3.

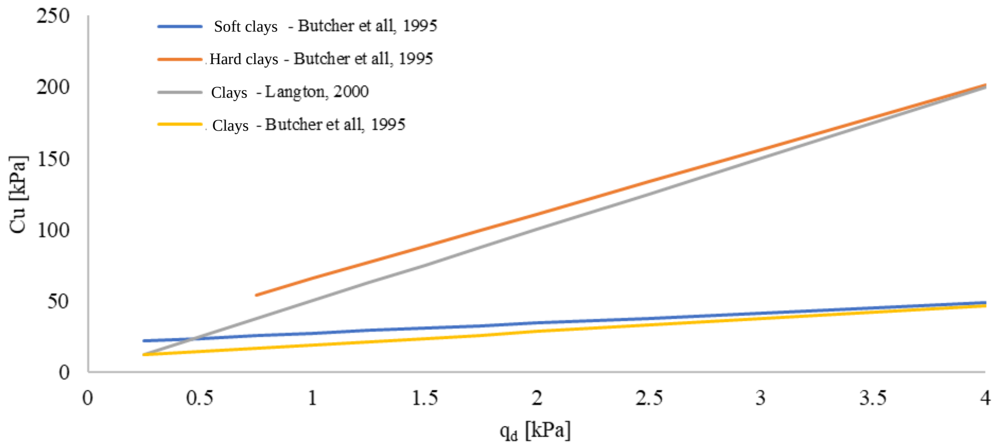

Figure 5-3 Correlation of dynamic penetration resistance on cone q_d and c_u

In his book (Marcu A., 1983) presents correlations for determining the internal friction angle for non-cohesive soils using penetration resistance and varying geological stress. The correlations proposed in (Marcu A., 1983) are presented graphically in Figure 5-4 with solid line together with the values of the friction angle as given using the formula proposed by (Robertson, Campanella, & Wightman, 1983b), with dotted line.

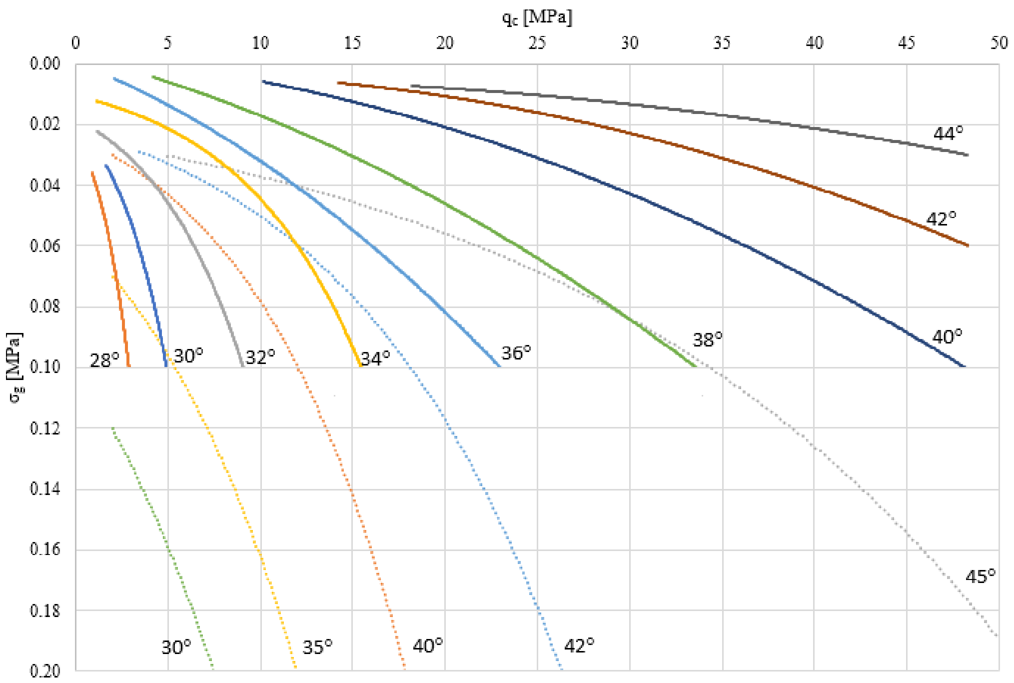

Figure 5-4 Correlation of cone resistance with the angle of internal friction for non-cohesive soils. With solid line the correlation according to (Marcu A. , 1983) and with dotted line the correlation according to (Robertson, Campanella, & Wightman, 1983b)

As presented in Figure 5-4 the correlation shown by (Robertson, Campanella, & Wightman, 1983b) gives fewer comprehensive values compared to the correlation proposed in (Marcu A. , 1983). Also, the correlation extracted from (Marcu A. , 1983) presents only values for geological stress up to 100 kPa (about 5 m depth) while the correlation (Robertson, Campanella, & Wightman, 1983b) provides a more general approach.

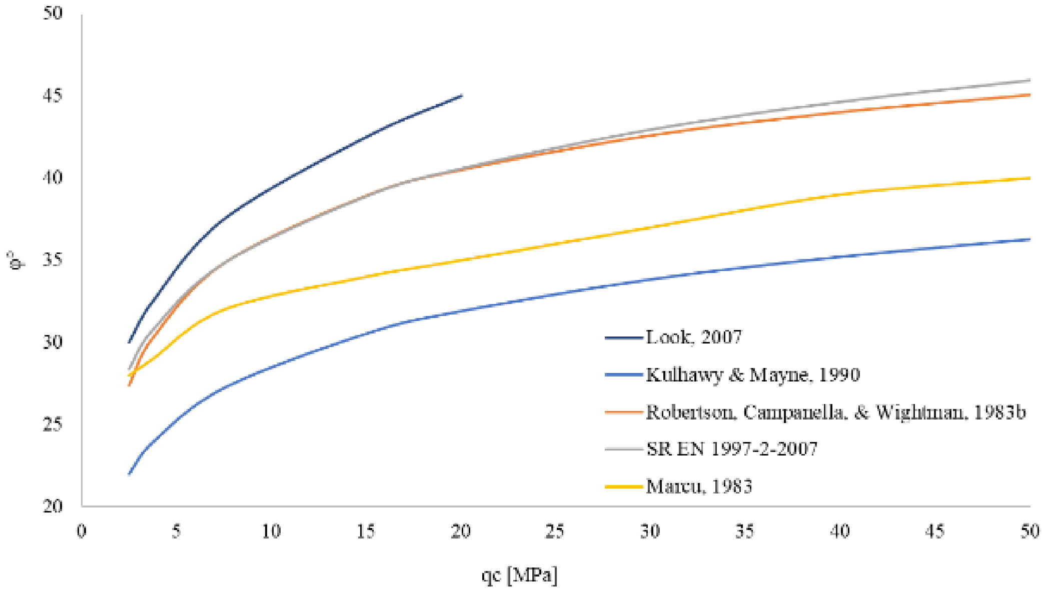

Figure 5-5 Comparison of the internal friction angle values using cone pressure q_c

Figure 5-6 presents the correlation of the number of SPT blows corresponding to a certain internal friction angle in non-cohesive soils.

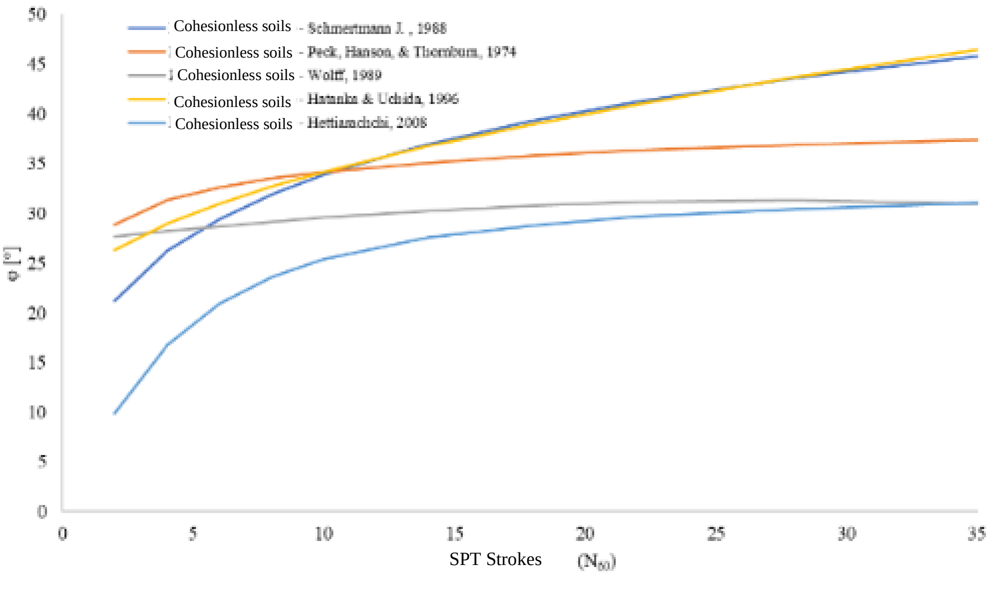

Figure 5-6 Comparison of the results obtained using different correlations for the determination of the internal friction angle using the SPT blows number

In Figure 5-7 Comparison of deformation modulus values obtained using the above-mentioned correlations a comparison of deformation modulus values is presented. As it can be seen in Figure 5-7 a comparison of the values of the deformation moduli obtained by using different correlations according to the soil type is presented.

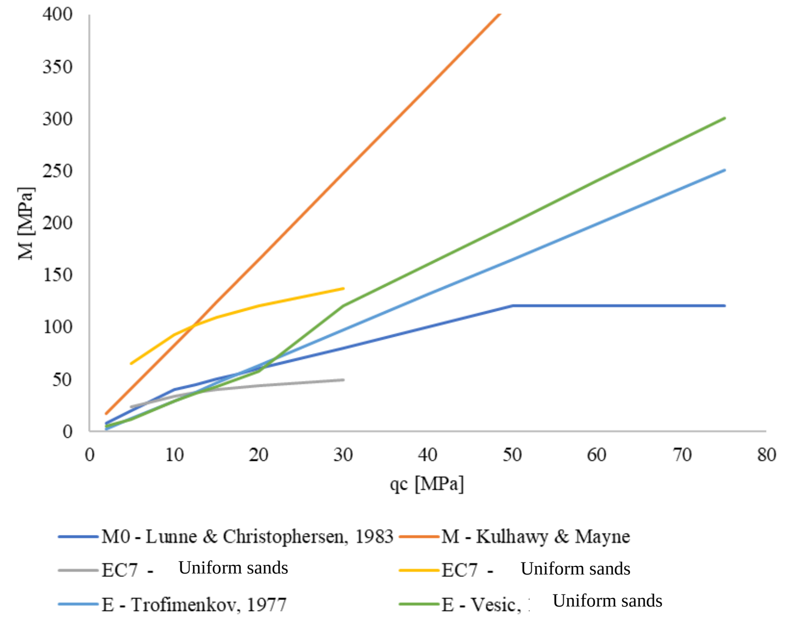

Figure 5-7 Comparison of deformation modulus values obtained using the above-mentioned correlations

Depending on the soil type for which a certain correlation is applied Figure 5-8 presents a comparison between the resulting M values and the number of blows for different types of dynamic tests.

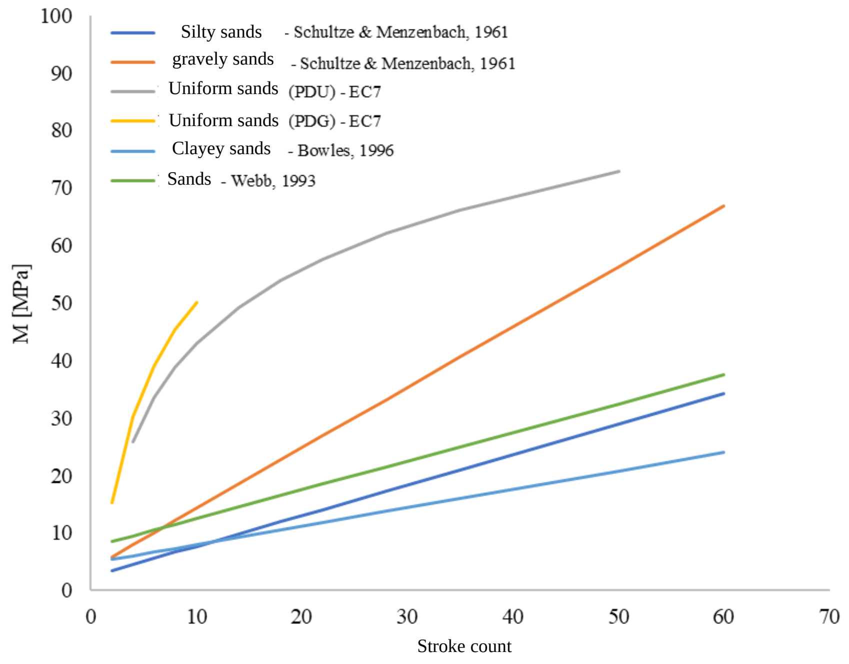

Figure 5-8 Comparison of deformation modulus values obtained using correlations with the number of SPT/PD strokes

In Figure 5-9 a comparison of M modulus values determined using various correlations, determined using cone pressure q_c, for several types of soils is shown.

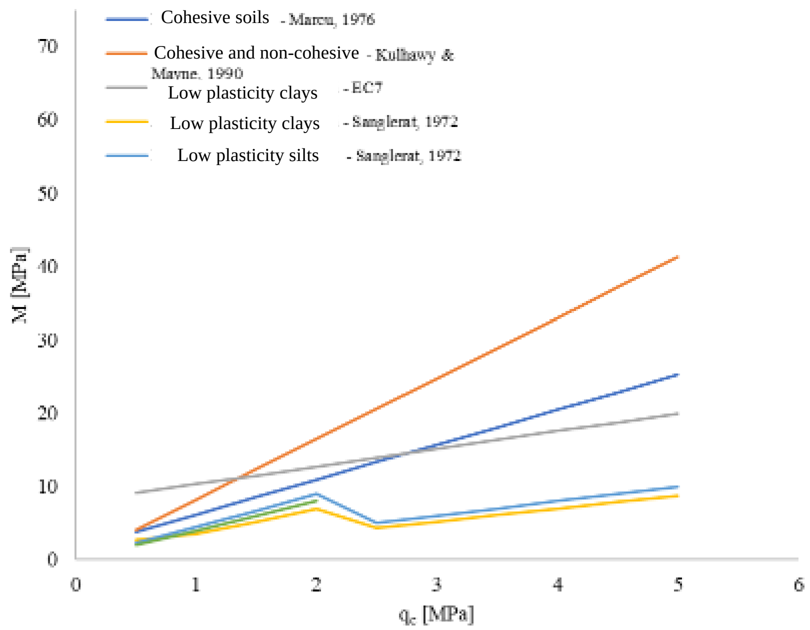

Figure 5-9 Comparison of deformation modulus values obtained using correlations with cone pressure q_c

In Figure 5-10 a comparison of the deformation modulus M obtained using different correlations in the literature for the dynamic probing light obtained for different soil types is shown.

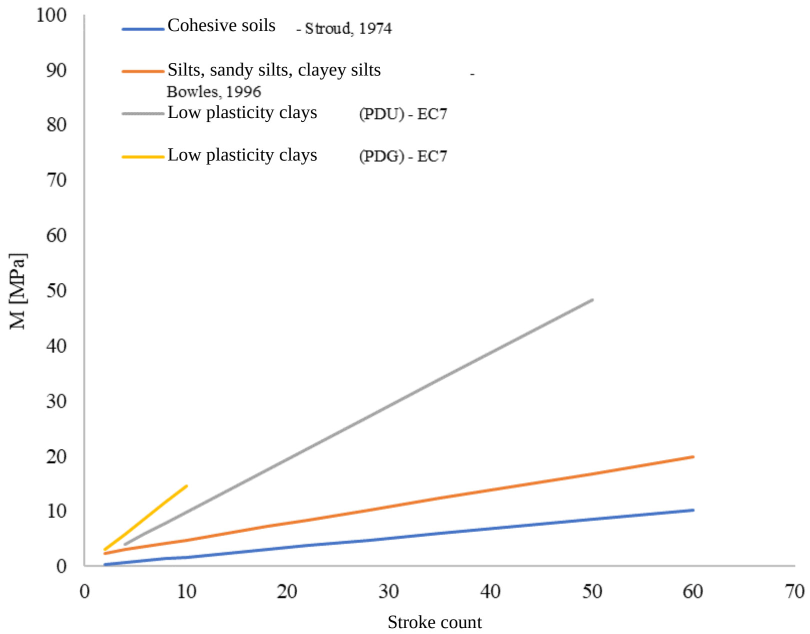

Figure 5-10 Comparison of strain modulus values obtained using correlations with number of blows in dynamic penetration tests

A significant part of the literature survey has been devoted to the investigation of correlations between geotechnical parameters determined both in situ and in the laboratory, according to national and international literature. In this chapter, the common correlations documented in the literature as well as in scientific articles and publications have been summarized and commented. Most of the correlations were presented using variation plots showing simultaneously several specific correlations for determining specific geotechnical parameters. Based in the literature survey new correlations were development. They are presented in chapter 8.

6 EXPERIMENTAL FIELD AND LABORATORY PROGRAMME

The research of the soils in the Bucharest area has been documented since the 19th century. The works of Murgoci (1913) and Protopopescu-Pache (1938) are to be mentioned and are considered as reference papers for the beginning of the last century. These papers synthesize and highlight for the first time the geological boundaries of different soil layers, using names similar to the current ones. The reference documents for the geological description of the soil in the Bucharest area was published by Emil Liteanu in 1952. It completes and deepens the works mentioned above by describing in detail each geological layer. The State Committee of Geology of the Geological Institute of Bucharest published in 1966 the geological map of the Municipality of Bucharest under the coordination of E. Liteanu and G. Murgeanu.

More recent research on the geology of Bucharest include the Geo-Atlas of Bucharest, published in 2007 (Lăcătușu, Popescu, Nicolae, & Enciu, 2008), which focuses more on the chemical aspects of the subsoil. The description of the soil from the geotechnical and geological point of view was only briefly presented. The hydrogeology of the municipality has been studied, including by use of modern computerized methods, among others in the PhD thesis of Dr. Eng. Dumitru Neagu in 2017.

Regarding the geotechnical parameters of the soils in the Bucharest area, they are well known and studied, there are numerous references in the literature and a significant number of scientific papers published in various specialized journals, symposia and national and international conferences.

This chapter briefly presents the sites in Bucharest where the field investigations used to achieve the objectives of this paper were carried out. The experimental field program included geotechnical borehole, static cone penetration tests CPT and Dilatometer Marchetti Tests (DMT).

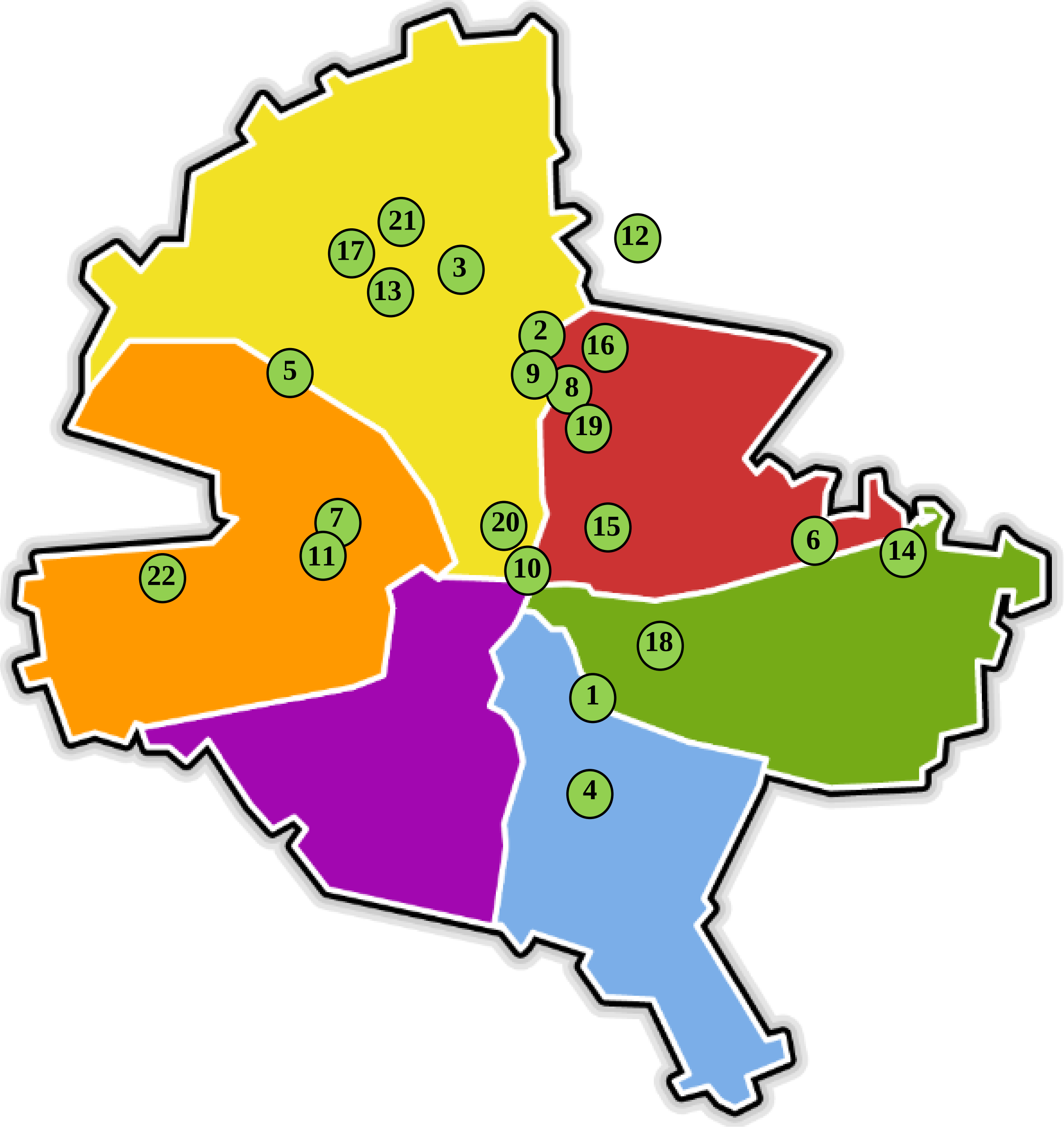

Figure 6-1 Investigated sites in Bucharest

A total of 22 sites located in the north, south, center, west and east of the capital were investigated. Figure 6-1 shows a concentration of sites in the northern part of the capital. This is due to the increased attention that this area has benefited from. The table below summarizes the investigated sites and the investigations carried out for each of them.

Table 6-1 Investigated locations

Location	Area	Address	Drilling/ Survey	CPT	DMT
1.	C-S	Splaiul Unirii 165	7	10	5
2.	N	Calea Floreasca 246	3	2	1
3.	N	Nicolae G. Caranfil 74	2	3	2

Location	Area	Address	Drilling/ Survey	CPT	DMT
4.	S	Strada Povestei 10	3	12	3
5.	NV	Bucharesti Noi Boulevard 25	6	7	2
6.	E	Sos. Vergului 4	2	2	2
7.	C-V	46 Orchids Road	4	3	-
8.	N	164 Barbu Văcărescu Street	4	9	2
9.	N	Calea Floreasca 242-244	6	18	3
10.	C	Tudor Arghezi 1-3	5	3	1
11.	C-V	Sg Constantin Ghercu Street 1b	4	5	5
12.	N	Pipera Boulevard 1/8, Voluntari	13	-	4
13.	N-NV	Menuetului Street No. 8	2	-	-
14.	E	Sos. Vergului 20	2	-	1
15.	C	Logofăt Luca Stroici Street 45	2	-	-
16.	N-NE	Dimitrie Pompeiu Boulevard 2D	1	-	-
17.	N-NV	10 Jiului Street	2	7	1
18.	C-E	321 Mihai Bravu Road	2	-	-
19.	C-N	18 Mircea Eliade Avenue	7	-	-
20.	C	10-18 Mantuleasa Street	2	3	-
21.	NV	Bd. Exhibition no. 2	-	3	1
22.	V	Bd. Preciziei nr. 6	-	3	1

Chapter 6 presents the stratification specific to Bucharest. It is analyzed from a geotechnical and geological point of view, and the main geological and hydrogeological formations are presented in detail. Using this information, as well as the documentation carried out in Chapters 2 and 5, geotechnical investigations were carried out on 22 sites in Bucharest. A total of 78 boreholes were drilled, covering more than 2000 meters of lithological column, with depths ranging from a few meters to more than 70 meters. In addition, 88 CPT cone penetration tests and no less than 34 Dilatometer Marchetti Tests (DMT) tests were carried out. The geotechnical laboratory tests were performed in the Grade II Geotechnical Laboratory of SAIDEL Engineering S.R.L. At the end of the chapter, a detailed geotechnical description of the first 5 geological layers is given, namely the Bucharest Loam, Colentina Gravels, Intermediate Clay Complex, Mostistea Sands and Marl Complex.

7 GEOTECHNICAL CHARACTERISATION OF SOILS

Current geotechnical practice provides the following sequence of steps to determine geotechnical parameters.

The first step is to carry out field and laboratory geotechnical investigations, from which derived values of the geotechnical parameters for the respective soil samples are deduced in accordance with SR EN 1997-1:2004.

The next step is to choose from the multitude of derived values of a geotechnical parameter a value that is representative for the soil being studied and the boundary condition considered in the design. This value is called the characteristic value.

To account for uncertainties affecting the characteristic value of the geotechnical parameters, partial safety factors are applied to the characteristic values, resulting in the design values of the geotechnical parameters.

7.1 DERIVED AND CHARACTERISTIC VALUE OF GEOTECHNICAL PARAMETERS

The process of derivation involves converting the actual results of a test into the value of a geotechnical parameter using correlations, theoretical or empirical relationships.

For example, knowing the measured cone resistance, q_c, one can determine the deformation parameters E_{50} or E_{oed}, or the shear strength parameters φ and c. The correlations are based either on a theoretical relationship between the correlated parameters or on an empirical one - based on experience a relationship between the two parameters is indicated (increasing E_{oed} with increasing q_c).

Another example is the correlation of the undrained shear strength of non-cohesive soils, c_u with the torsional moment recorded with the vane tests. In this case, the value of shear strength depends only on the value of the torsional moment corrected by a form factor, which takes into account the dimensions of the equipment (blades).

The derived value, as finite by (Bond, 2006) of a geotechnical parameter can also be determined directly by laboratory testing (e.g., by triaxial compression), in which case any disturbances resulting from sampling and processing prior to testing should be taken into account when choosing the derived value. The notion of ‘derived value’ as defined by (Bond, 2006), should not be confused with the nationally known derived value (e.g., density is a derived value obtained directly from measurements between two directly determined quantities mass and volume). The notion of derived value defined in (Bond, 2006) is attributed to the value of a parameter chosen by the user based on the experience, field or laboratory tests. By applying statistical methods to several derived values of a geotechnical parameter, its characteristic value can be determined.

As defined in (SR EN 1990, 2004) a characteristic value of a material is the value corresponding to the 5% fraction of a normal distribution of values when a lower value is unfavorable (e.g., compressive strength of concrete), or a value corresponding to the 95% fraction of a normal distribution of values when a higher value is unfavorable.

Caution estimate

Due to the difficulty in choosing characteristic values of geotechnical parameters Eurocode 7 (SR EN 1997) defines the characteristic value as a conservatively estimated value that determines the occurrence of a limit state. The definition of "caution estimate" of a geotechnical parameter is however vague. (Bond, 2006) considers the terms 'estimate' and 'caution' separately and concludes that a caution estimate of a value is in fact an approximate calculation or approximate assessment of a geotechnical parameter, whereby problems are avoided. Further the term ‘causing a limit state to occur’ indicates that a characteristic value must be determined for each limit state. Thus, the characteristic value governing a particular limit state is part of the geotechnical design and cannot be determined beforehand at the stage of geotechnical survey.

In the process of choosing/determining the characteristic value of geotechnical parameters a significant component of engineering judgement is necessary, much greater than for other materials.

In Romania, these elements are currently regulated by the technical standard NP 122-2010 - Technical standard on the determination of characteristic and design values of geotechnical parameters.

Recently, in the 2022 revision of NP 074 - Standard for Geotechnical Documentation for Construction - the elements related to the place of characteristic values in geotechnical documentation have been modified and it is now clearly stated that these must be determined during the Geotechnical Design.

7.2 CHANGES IN THE DETERMINATION OF CHARACTERISTIC GEOTECHNICAL PARAMETERS IN THE REVISION prEN 1997-202x

As presented above, the selection of the characteristic value of a geotechnical parameter in accordance with SR EN 1997-1:2004 and national technical standards, involves several uncertainties, particularly with regard to "caution estimate", the application of statistical methods in any given situation, and engineering judgement being necessary.

Therefore, the revision of Eurocode 7 (prEN 1997-202x), which is in progress but in an advanced stage of approval at the time of writing, has significantly changed the process of selecting the values underlying the determination of the calculation values of geotechnical parameters.

Thus, based on the derived values of geotechnical parameters, included in the Investigation Report (Geotechnical Survey), 2 methods can be applied to determine the representative value (X_{rep}) of a geotechnical parameter:

(1) Selection based on comparable experience and site knowledge of a nominal value of the geotechnical parameter ($X_{nom} = X_{rep}$), which is a caution estimate of the value of the parameter affecting the occurrence of a certain limit state

(2) Selection on the basis of a statistical analysis of a characteristic value of the geotechnical parameter ($X_k = X_{rep}$) affecting the occurrence of a certain limit state with an imposed value of the probability of non-attainment

The representative value is defined as either the nominal value or the characteristic value, affected by a conversion factor (η), which takes into account the effects of scale, humidity, temperature, ageing of materials, anisotropy, stress or strain path. If these effects are included in the derived values (as is usually the case in geotechnical engineering), this conversion factor is 1.

The design value of the geotechnical parameter (X_d) is obtained by assigning to the representative value a partial material coefficient. Whichever of the 2 possible ways of determining the representative value of a geotechnical parameter (based on engineering experience and judgement or on a statistical basis) is applied, the designer must take into account: pre-existing knowledge of the site, uncertainties related to the quantity and quality of geotechnical data, uncertainties due to the spatial variability of the measured properties, and the area of influence of the structure for the considered boundary condition.

It can also be mentioned here that there are other important values of geotechnical parameters, such as the best estimate. This is defined as the estimate of the most probable value of a geotechnical parameter and differs from the representative (nominal or characteristic) value in that it is not a conservative estimate, i.e., it does not include any safety margin. It is used to estimate the most probable behavior of a geotechnical structure, applied when using the observational design method. Best estimate values are also used to check correlations between different geotechnical parameters. It can be determined as:

- The most likely value obtained from a sample of derived data;
- The mean, median or modal value of a sample of derived data, whichever is considered appropriate;
- Most likely value obtained by back analysis based on monitoring results

7.3 CONCLUSIONS

Chapter 7 focuses on the statistical analysis of geotechnical parameters. In this chapter, the necessary steps for calculating characteristic values are defined. Selecting the characteristic value of a geotechnical parameter involves certain uncertainties and requires careful evaluation of statistical methods and engineering judgement. Eurocode 7 (prEN 1997-202x) proposes significant changes in the process of selecting the values used to determine geotechnical parameters. Thus, two methods can be applied to determine the representative value of a geotechnical parameter: selection of a conservative nominal value or selection of a characteristic value by statistical analysis with a specific probability.

8 PROPOSAL OF NEW CORRELATIONS BETWEEN GEOTECHNICAL PARAMETERS SPECIFIC TO SOILS IN THE BUCHAREST AREA

This chapter presents the evaluation of the results obtained from the field and laboratory tests carried out on the analyzed sites and proposes new correlations. The new correlations obtained are, depending on the parameter, linear or logarithmic. The presentation will be made independently for each lithological layer and each parameter.

The lithological layers for which correlations will be presented in the following subchapters are those specific to the Bucharest area. These are commonly referred to as "Bucharest Loam", "Colentina Gravels", "Intermediate Clay Complex" and Mostistea Sands. The geotechnical characteristics of the soils listed above have a significant and decisive influence on the architectural design, structural dimensioning and construction of the above-mentioned structures.

8.1 PROCESS FOR SELECTING RELEVANT TESTS AND SAMPLES

As presented in Chapter 6, the following field and laboratory tests were used to prepare this analysis:

- CPT
- DMT
- Geotechnical drilling and surveys
- Laboratory identification and classification tests
- Laboratory mechanical tests (compressibility in edometer and direct shear)

A rigorous selection process of field and laboratory tests was necessary to obtain quality results. The sample selection steps are detailed below.

<u>Stage 0</u>

Stage 0 consisted of the pre-selection of sites and location. Before starting the actual process of selection of the different laboratory samples or in situ tests, locations with a typical stratification for Bucharest were selected. For example, sites that have undergone significant changes in the recent past were excluded. The following were considered as significant changes: the site has been subject to pollution with hydrocarbons or other liquids \ materials that may affect geotechnical parameters or bearing capacity, terrains on which excavations and fills have been carried out. This screening process resulted in the field and laboratory tests presented in the previous chapters.

<u>Stage 1</u>

The first stage of the actual selection process consisted of choosing investigation points (geotechnical drilling, CPT and DMT tests) that could fit into a circle with a radius of no more than 3 m. The 3 m criterion was chosen because, for technological reasons, it is sometimes not possible to locate points closer than 1.5 m without influencing each other. The distance of 1.5 m was chosen assuming that the test/survey deviates less than **1°/m** from the vertical position. This is also the rejection criterion for DMT equipment and can also be assimilated with the rejection criterion for CPT equipment, if an average survey depth of 25 m and a maximum allowable inclination of 25° are considered**.**

Stage 2

The second stage consisted of drawing geotechnical profiles for each location. The geotechnical profiles included, as a minimum, the geotechnical borehole sheet with stratification description and in situ test plots, as well as tests not covered in this report. This allowed for the removal of close investigations that showed anomalies compared to the other 2 investigation types. As an example, if the laboratory tests corresponding to a geotechnical borehole in the vicinity of the CPT/DMT tests showed significantly lower or significantly higher modulus values than the other boreholes on site, while the CPT/DMT tests showed similar values, the borehole or soil sample was eliminated from the analysis.

Stage 3

In step 3, the remolded laboratory samples were discarded. Remolded samples are defined as undisturbed laboratory samples which, after processing for the test (e.g. sampling from the Shelby tube to the oedometer casing), have undergone minor alterations in the soil state due to the presence of small gravel particles, fossils, etc., which cannot be quantified. Because of the uncertainty provided by these samples, they were removed from the statistical processing.

Stage 4

Step 4 consisted of eliminating laboratory and field tests with implausible values. For example, if a laboratory compressibility test showed an $E_{oed0\text{-}50}$ modulus greater than $E_{oed200\text{-}300}$, the test was eliminated. Another example of samples eliminated due to implausible values are those where the oedometer modulus value $E_{oed\ 200\text{-}300}$ was greater than 30-40 MPa or q_c values greater than 5 MPa for Bucharest Loam. On top of that, the Bucharest Loam layer is a cohesive soil with a consistency from stiff to very stiff for witch high values of the q_c greater than 5 MPa are not typical.

8.2 SUMMARY PRESENTATION OF THE CORRELATIONS OBTAINED

This sub-chapter summarizes in tabular form the new correlations. As it can be seen from Table 8-1 correlations were obtained between cone pressure q_c and the oedometer modulus $E_{oed200\text{-}300}$, q_c and the shear strength parameters, q_c and M_{DMT}, q_c and the undrained shear strength and between M_{DMT} and $E_{oed200\text{-}300}$ respectively. These were obtained for a number of values ranging from 10 to 40. Correlation coefficients between 0.764 and 0.996 were obtained for the correlations determined.

Table 8-1 Correlations for Bucharest Loam

Correlated parameters	Correlation	Correlation coefficient r	Number of values	Standard deviation	Safe correlation
q_c vs. $E_{oed200-300}$	$3476\ q_c + 3456$ kPa	0,870	50	300 kPa	$3476\ q_c + 3156$ kPa
q_c vs. $E^*_{oed200-300}$	$3836\ q_c + 5584$ kPa	0.996	10	122 kPa	$3836\ q_c + 5462$ kPa
q_c vs. τ (pt. σ=50 kPa)	$15.9\ q_c + 22$ kPa	0,832	20	2.58 kPa	$15.9\ q_c + 19$ kPa
q_c vs. τ (pt. σ=100 kPa)	$15.7\ q_c + 40$ kPa	0,828	20	2.57 kPa	$15.7\ q_c + 37$ kPa
q_c vs. $\tan\varphi$	$0.092\ q_c + 0.204$	0,834	20	0,015	$0.092\ q_c + 0.189$
q_c vs. c	$7.8\ q_c + 23$ kPa	0,816	20	1.35 kPa	$7.8\ q_c + 21$ kPa
q_c vs. M_{DMT}	$12.7\ q_c + 3.4$ MPa	0,920	40	0.88 MPa	$12.7\ q_c + 2.5$ MPa
q_c vs. $c_{u,DMT}$	$14,4\ q_c + 30$ kPa	0,837	32	2.11 kPa	$14.4\ q_c + 28$ kPa
M_{DMT} vs $E_{oed200-300}$	$0.17\ M_{DMT} + 4.4$ MPa	0,764	38	1.7 MPa	$0.17\ M_{DMT} + 2.7$ MPa

*for samples with a fine-particle content (<0.063 mm) of more than 90%.

In the case of the Colentina gravel layer, correlations were obtained between q_c and φ, respectively between q_c and M_{DMT}, which are shown in Table 8-2. The number of values for which correlations were determined ranges from 58 to 72. Correlation coefficients between 0.761 and 0.773 were obtained for the correlations determined.

Table 8-2 Correlations for Colentina Gravel

Correlated parameters	Correlation	Correlation coefficient r	Number of values	Standard deviation	Safe correlation
q_c vs. φ	$4.78\ q_c + 25°$	0,773	72	-	$4.78\ q_c + 25°$
q_c vs. M_{DMT}	$3.93\ q_c + 52$ MPa	0,761	58	11 MPa	$3.93\ q_c + 41$ MPa

As can be seen from Table 8-3 correlations were obtained between cone pressure q_c and the oedometer modulus $E_{oed200-300}$, q_c and the shear strength parameters, q_c and M_{DMT}, q_c and the undrained shear strength respectively between M_{DMT} and $E_{oed200-300}$. These were obtained for a number of values ranging from 14 to 67. Correlation coefficients between 0.735 and 0.883 resulted for the determined correlations.

Table 8-3 Correlations obtained for the Intermediate Clay Complex

Correlated parameters	Correlation	Correlation coefficient r	Number of values	Standard deviation	Coverage correlation
q_c vs. $E_{oed200-300}$	$5247\ q_c - 2404$ kPa	0,883	67	980 kPa	$5247\ q_c - 3384$ kPa
q_c vs. τ (pt. σ=200 kPa)	$28.6\ q_c + 28$ kPa	0,838	54	8,5 kPa	$28.6\ q_c + 28$ kPa
q_c vs. τ (pt. σ=300 kPa)	$28.2\ q_c + 60$ kPa	0,735	54	11.7 kPa	$28.2\ q_c + 48$ kPa

q_c vs. tanϕ	0.061 q_c + 0.082	0,813	54	0,019	0.061 q_c + 0.063
q_c vs. c	14.6 q_c + 21 kPa	0,803	54	4,8 kPa	14.6 q_c + 16 kPa
q_c vs. M_{DMT}	20 q_c + 18 MPa	0,884	14	3.2 MPa	20 q_c + 14.8 MPa
q_c vs. $c_{u,DMT}$	52.5 q_c + 22 kPa	0,872	14	8,9 kPa	52.5 q_c + 13.1 kPa
M_{DMT} vs $E_{oed200-300}$	0.15 M_{DMT} + 4.1 MPa	0,850	14	1.7 MPa	0.15 M_{DMT} + 2.4 MPa

The correlations obtained for the Mostistea Sands layer are summarized in Table 8-4. Thus, correlations were determined between q_c and φ, respectively between q_c and M_{DMT}. The number of values for which correlations were determined was 36. Correlation coefficients between 0.789 and 0.829 were obtained for the correlations determined.

Table 8-4 Correlations obtained for the Mostistea Sands layer

Correlated parameters	Correlation	Correlation coefficient r	Number of values	Root mean square deviation	Coverage correlation
q_c vs. φ	4.53 q_c + 25°	0,829	36	-	4.53 q_c + 25°
q_c vs. M_{DMT}	5.31 q_c + 37 MPa	0,789	36	13 MPa	5.31 q_c + 13 MPa

In Chapter 8 of this paper, based on the results of the in situ and laboratory geotechnical investigations presented in Chapter 6, and on the statistical concepts presented in Chapter 7, new correlations between the results of the in situ geotechnical investigations and the specific geotechnical parameters of the soils in Bucharest Municipality are proposed. These correlations are adapted to the specific characteristics of the soils of the Bucharest Loam, Colentina Gravels, Intermediate Clay Complex and Mostistea Sands layers and aim to determine the parameters describing the mechanical behaviors of these soils.

9 VERIFICATION AND VALIDATION OF CORRELATIONS; OBSERVATIONS AND COMMENTS ON PROPOSED CORRELATIONS

Chapter 9 is devoted to the validation of the new correlations proposed in chapter 8. To this end, in order to assess the correlations presented in chapter 8 several methods are proposed. Thus, in the following sub-chapters, the correlations obtained by comparing their results with the correlations presented in the literature are analyzed. Since the field and laboratory tests carried out, as well as the processing methods, are similar to those from which the existing tests in the literature resulted, the new correlations obtained are practically a calibration of the existing correlations for the specific soil conditions in the Bucharest area.

Correlation analysis to determine the angle of internal friction

The figure below shows the theoretical values (linear variation) of the tangent of the internal friction angle for the Bucharest Loam layer that can be attributed to pressure values q_c ranging from 0.5 to 4 MPa using newly proposed correlations and separately the correlation from the literature (Trofimenkov & Vorobkov, 1974). The area hatched in green represents the range corresponding to a 95% confidence level of the tangent of the internal friction angle specific to the layer under study. By analyzing the graph, a similarity can be observed between the proposed correlation (see Chapter 8) and the one known from literature (Trofimenkov & Vorobkov, 1974). The proposed correlation tends to underestimate the values of the tangent of the interior friction angle for q_c values below 1.25 MPa and to overestimate the values of the tangent of the interior friction angle for q_c values above 1.25 MPa compared to the correlation (Trofimenkov & Vorobkov, 1974). Given the information presented in (Marcu A. , 1983) the correlation in the literature (Trofimenkov & Vorobkov, 1974) is altered by a certain safety coefficient, which is however not known, leading to the presented conservative values.

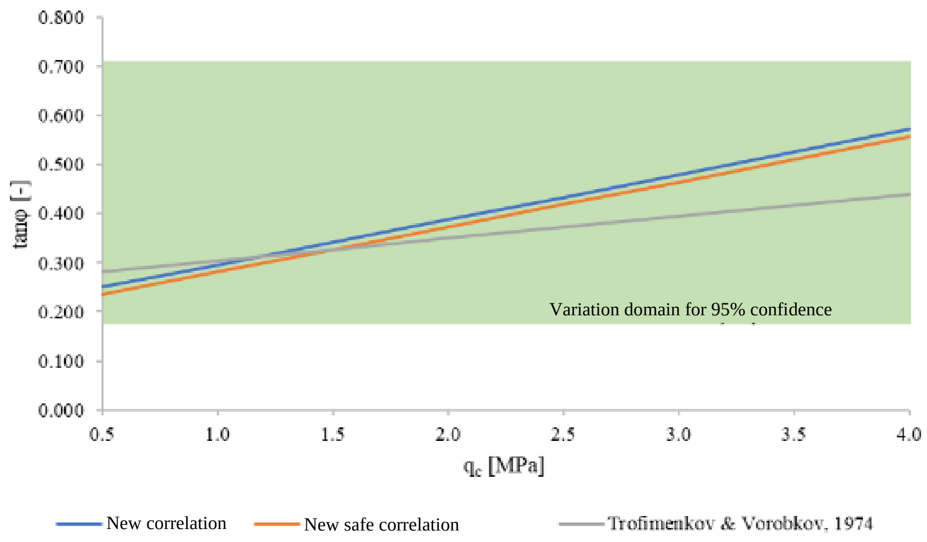

Figure 9-1 Bucharest Loam, results comparison for own correlations and literature correlations for the interior friction angle.

Even compared to the safe correlation presented in Chapter 8 using the correlation (Trofimenkov & Vorobkov, 1974) more conservative results are obtained. However, the new correlation better covers the range of usual values of the analyzed layer. The differences between the new and existing correlations can be explained by the fact that the current correlation in Chapter 8 is optimized for Bucharest Loam.

Figure 9-2 shows the theoretical values of the angle of internal friction for the Colentina Gravel layer that can be attributed to peak pressure values q_c ranging from 5 to 40 MPa using the new correlation obtained in Chapter 8 as well as from the literature (SR EN 1997-2-2007, 2007), (Kulhawy & Mayne, 1990) and (Robertson, Campanella, Gillespie, & Grig, 1986). The area hatched in green represents the range of variation corresponding to a 95% confidence of the internal friction angle of the layer under study. All correlations show logarithmic variation. By analyzing the graphs in Figure 9-2 a good convergence can be observed between all four presented correlations. The correlation proposed by (Kulhawy & Mayne, 1990) tends to underestimate the values of the friction angle giving more comprehensive results especially for q_c values below 15 - 20 MPa. For q_c values above 25 MPa the correlation proposed in this paper gives slightly more reliable results compared to the correlation in Eurocode 7. Differences between the values of the own correlation and those in the literature are attributed to the different soils for which the respective correlations were determined.

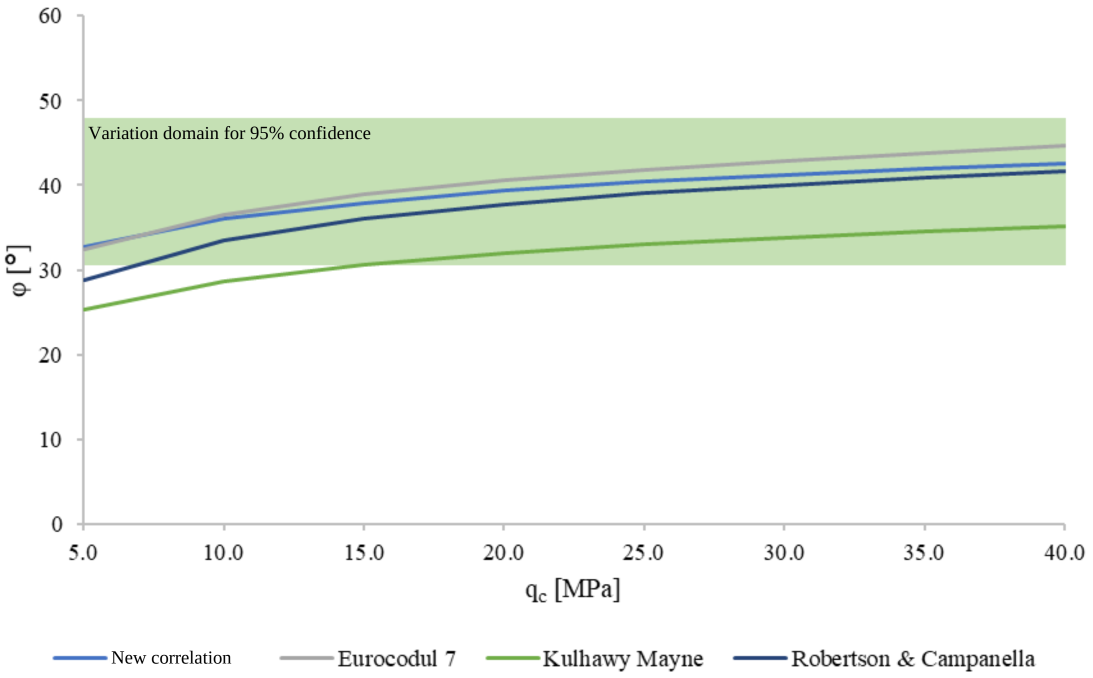

Figure 9-2 Colentina Gravels, results comparison for own correlations and literature correlations for the internal friction angle

In Figure 9-3 theoretical values of the tangent of the internal friction angle for the Intermediate Clay Complex that can be attributed to peak pressure values q_c ranging from 0.5 to 4 MPa are shown using new correlations and from the literature. The area hatched in green represents the range of variation corresponding to a 95% confidence level of the tangent of the internal friction angle specific to the studied layer.

Figure 9-3 shows the differences between the correlation proposed in Chapter 8 and the one known from the literature (Trofimenkov & Vorobkov, 1974). The proposed correlation as well as the safe correlation shown in Figure 9-3 tend to underestimate the values of the friction angle, especially for higher values of the cone resistance q_c. It should be noted, however, that the shear strength of a cohesive soil is influenced by the pair of φ and c values and the way the correlations are determined. In this respect reference is made to the previous chapter.

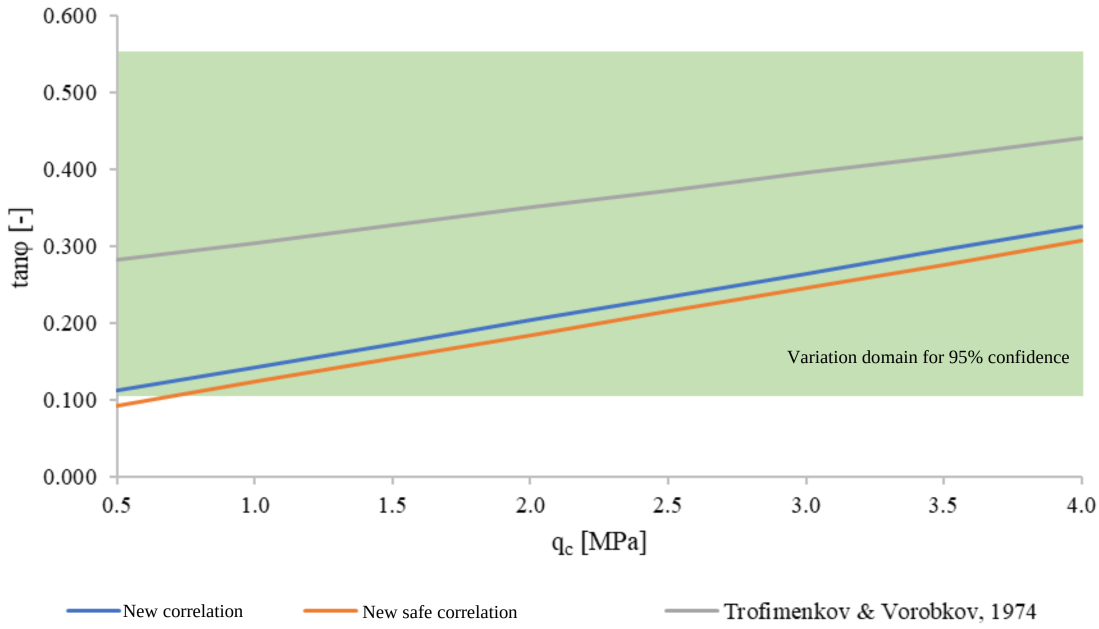

Figure 9-3 Intermediate Clay Complex, results comparison between own and literature correlations for the internal friction angle

In Figure 9-4 theoretical values of the angle of internal friction for the Mostistea Sands layer that can be attributed to peak pressure values q_c ranging from 5 to 40 MPa using own and literature correlations (SR EN 1997-2-2007, 2007), (Kulhawy & Mayne, 1990) and (Robertson, Campanella, Gillespie, & Grig, 1986) are given. The area hatched in green represents the range corresponding to a 95% confidence level of the internal friction angle specific to the Mostistea Sands layer. Similar to the results obtained for the Colentina Gravels all correlations show a logarithmic variation. The own correlation and that from (SR EN 1997-2-2007, 2007) covers better the range of variation with 95% confidence for q_c values below 10 MPa while the two other correlations tend to slightly underestimate the values of the friction angle. As can be seen from the graph for the latter correlations, for φ values lower than 35°, the variation lines are outside the 95% confidence range. The correlation curves in the literature presented by (Kulhawy & Mayne, 1990) and (Robertson, Campanella, Gillespie, & Grig, 1986) tend to underestimate the values of the internal friction angle, for q_c values below 15 - 20 MPa where the differences are up to 10°.

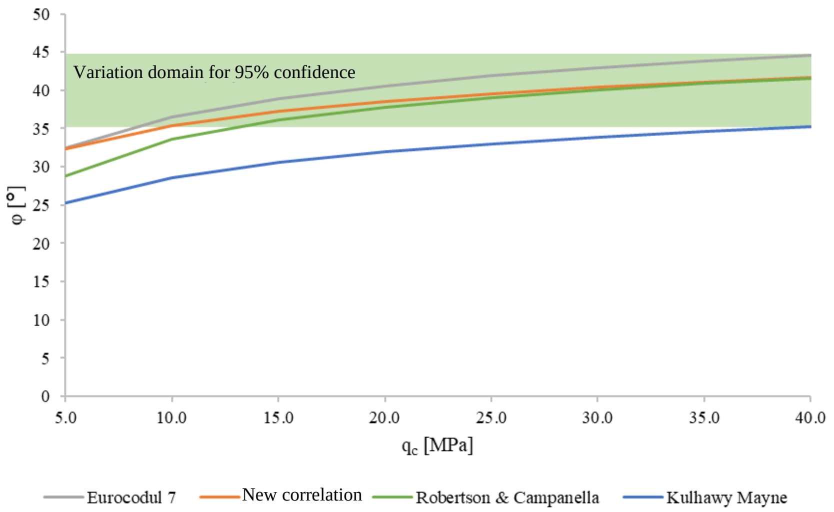

Figure 9-4 Mostistea sands, comparison of results for various own correlations and literature correlations for interior friction angle

Correlation analysis to determine cohesion

In the present paper the new correlation obtained corresponds to the cohesion obtained from the direct shear test CUn type. In Figure 9-5 theoretical cohesion values for the Bucharest Loam layer that can be attributed to peak pressure values q_c ranging from 0.5 to 4 MPa using own and literature correlations are presented.

It should be noted that all correlations underestimate the value of cohesion, which usually leads to more conservative results. This is usually mentioned because combining cohesion with higher friction angle values may lead to a situation where the total shear strength may show too optimistic values. For the range of q_c values below 1 MPa the correlations tend to overestimate the value of cohesion.

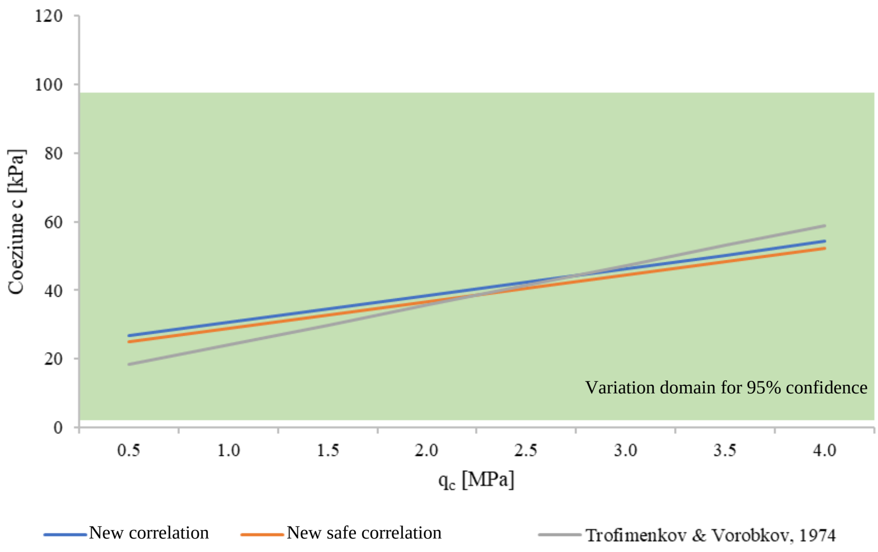

Figure 9-5 Bucharest Loam, results comparison for various own correlations and correlations from literature for cohesion

In Figure 9-6 theoretical cohesion values for the Intermediate Clay Complex that can be attributed to peak pressure values q_c ranging from 0.5 to 6 MPa are shown using own correlations and from literature. Studying the figure below it can be seen how the correlation determined in this paper as well as the one determined by (Trofimenkov & Vorobkov, 1974) fall within the range of variation with 95% confidence for the Intermediate Clay Complex. The own correlations tend to slightly overestimate the cohesion value for q_c values above 5 MPa.

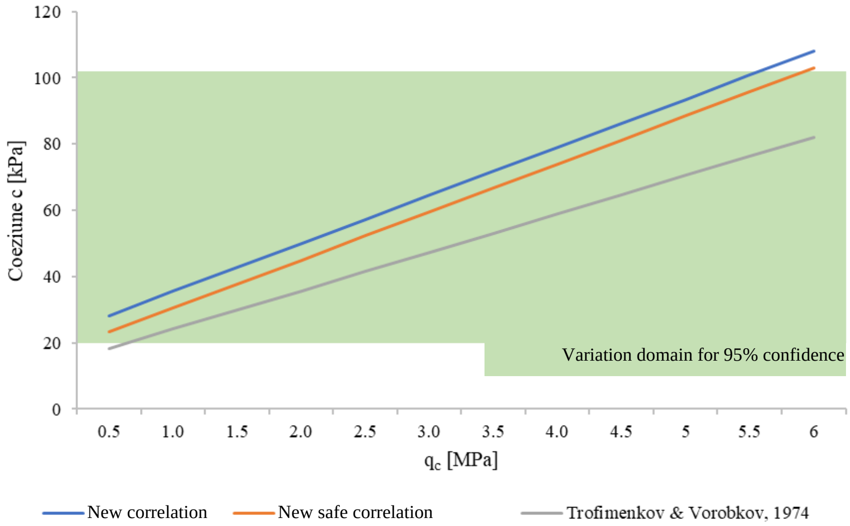

Figure 9-6 Intermediate Clay Complex, results comparison for various own correlations and correlations from literature for cohesion

Correlation analysis for deformation moduli

The Figure below shows the theoretical values (linear variation) of the deformation modulus E for the Bucharest Loam layer that can be attributed to peak pressure values q_c ranging from 0.5 to 4 MPa using the new correlations and literature (Marcu A. , 1983). The area hatched in green represents the range corresponding to a 95% confidence level of the undrained cohesion corresponding to the Bucharest Loam. The analysis of the graph shows a good convergence between the newly proposed correlation and the one known from the literature. (Marcu A. , 1983). Considering the information presented in (Marcu A. , 1983), the correlation in the literature is altered by a safety coefficient, which leads to more conservative values. It should be noted, however, that this correlation better covers the range of values of the deformation moduli determined for the layer in question. This may be due to the determination of the correlation from (Marcu A. , 1983) between q_c and the strain modulus determined with the static plate and not the one determined in the laboratory, as is the one determined for the current correlation.

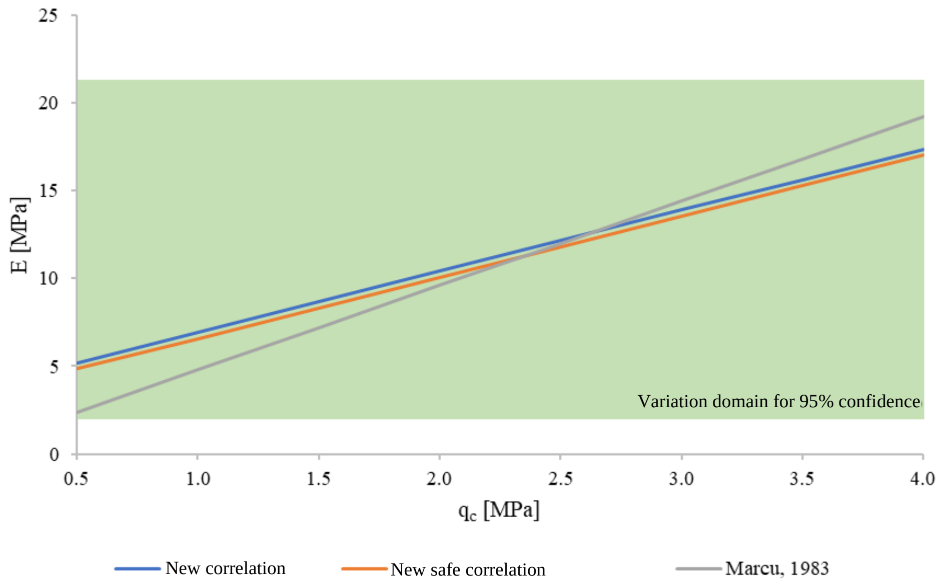

Figure 9-7 Bucharest Loam, comparison of results for different correlations between own and literature correlations for strain moduli $E_{oed200-300}$

In Figure 9-8 the values of the deformation modulus E_{DMT} for the Colentina Gravels layer are shown, which can be attributed to peak pressure values q_c ranging from 5 to 40 MPa using own and literature correlations. The area hatched in green represents the range corresponding to a 95% confidence level of the deformation moduli corresponding to the Colentina Gravels geological unit. Both own and literature correlations show linear variation. All the correlations presented in Figure 9-8 show results close to the values determined in the field for a 95% confidence range of variation.

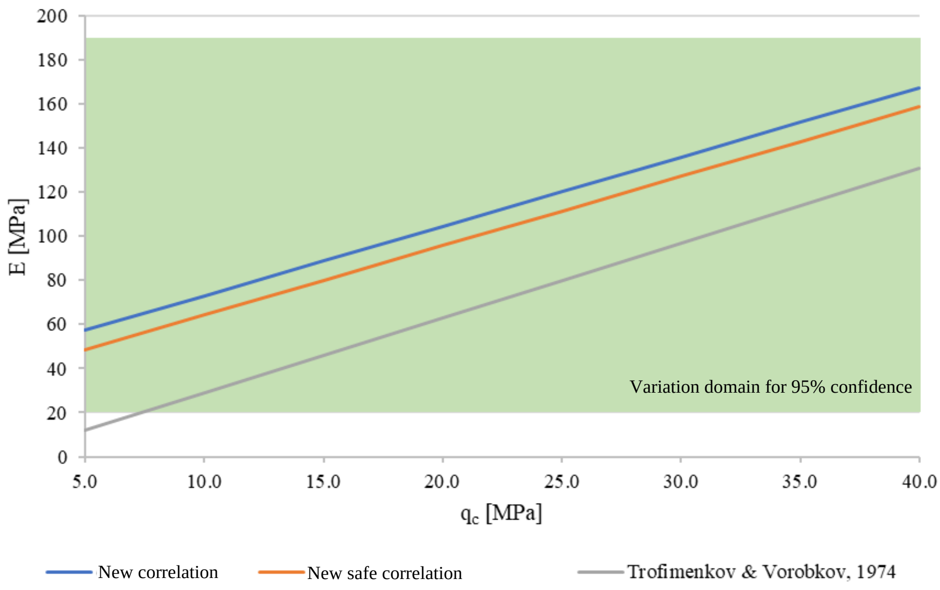

Figure 9-8 Colentina Gravels, results comparison for various new correlations and literature correlations for E variation moduli

Similar to the case of the Bucharest Loam theoretical values (linear variation) of the deformation modulus $E_{oed200-300}$ are presented in Figure 9-9 for the Intermediate Clay Complex layer that can be attributed to peak pressure values q_c ranging from 0.5 to 4 MPa using own and literature correlations. The area shaded in green represents the range corresponding to a 95% confidence level of the undrained cohesion specific to the intermediate clay layer.

Following the analysis of the graph, a similarity can be observed between the correlation determined in this paper and the one proposed in (Marcu A. , 1983). In both cases the correlations tend to underestimate the value of the deformation modulus, while the correlation presented by (Marcu A. , 1983) gives a better distribution of strain modulus values. Given the information presented in (Marcu A. , 1983), the correlation in the literature is altered with a certain safety coefficient, which leads to conservative values. Additionally, the differences may also be due to the determination of the correlation in (Marcu A. , 1983) between q_c and the deformation modulus determined with the static plate. In contrast, the new correlation was determined using q_c values from the cone penetration tests and geotechnical parameters determined in the laboratory. It should be noted that for q_c values lower than 0.75 MPa using the safe new correlation will result in negative strain modulus values, which is why its use is recommended only for q_c values higher than 1 MPa.

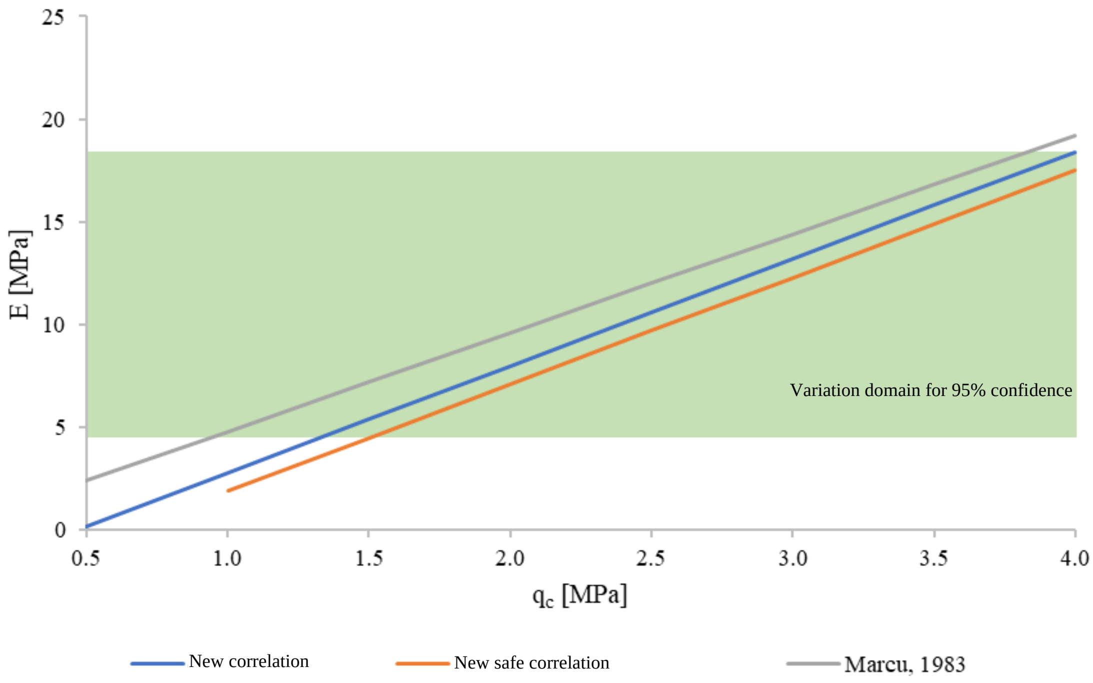

Figure 9-9 Intermediate Clay Complex, results comparison for various new correlations and literature correlations for variation moduli $E_{oed200\text{-}300}$

In Figure 9-10 the values of the deformation modulus E_{DMT} for the Mostistea layer are shown, which can be attributed to peak pressure values q_c ranging from 5 to 40 MPa using own and literature correlations. The area shaded in green represents the range corresponding to a 95% confidence level of the deformation moduli specific to the Mostistea Sands. Both own and literature correlations show linear variation. All the correlations presented above give results close to the values determined in the field for a 95% confidence range of variation. Also, the correlation proposed by (Trofimenkov, Mariupolski, & Pjarnpuu, 1977) generally tends to underestimate values of strain moduli especially for q_c values lower than 15 MPa. The differences between the values of the new correlations and those in the literature are mainly represented by the way in which these correlations were determined as well as the soil for which they were determined.

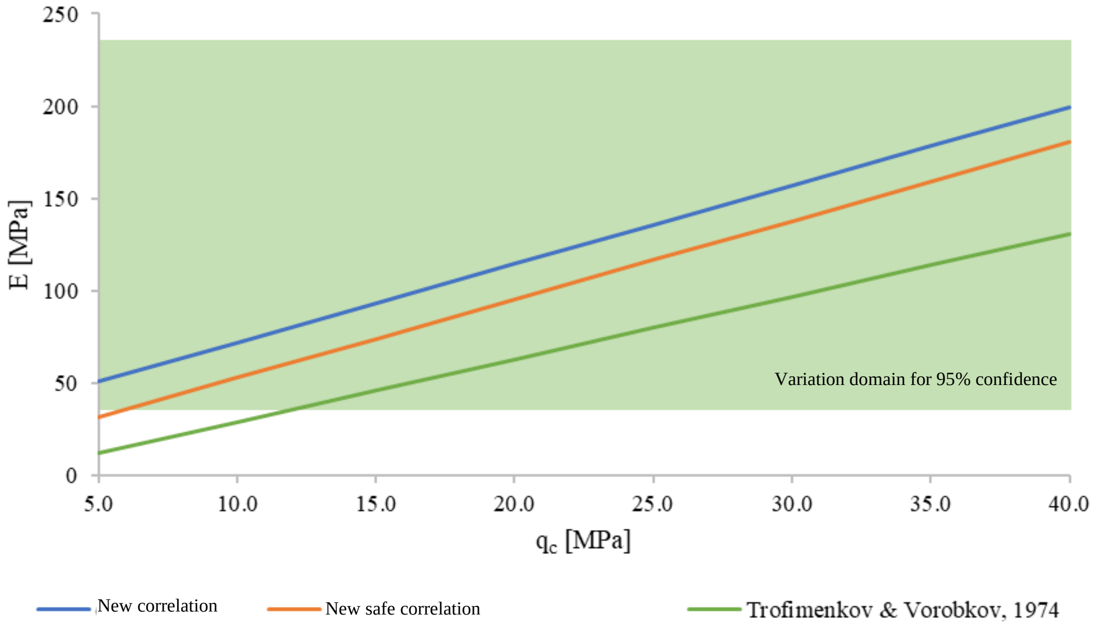

Figure 9-10 Mostistea Sands, results comparison for various new correlations and literature correlations for variation moduli E

9.2 PARALLEL EVALUATION OF GEOTECHNICAL PARAMETERS OBTAINED FROM LABORATORY TESTS AND THOSE OBTAINED FROM FIELD TESTS USING EXISTING CORRELATIONS

The differences between the geotechnical parameters obtained from the laboratory tests in comparison with the geotechnical parameters obtained by applying the correlations obtained in chapter 8 are presented below. The differences will be analyzed for two sites in Bucharest. In the first site the comparison of results will be made for the Bucharest Loam layer while in the second site the comparison will be made for both the Bucharest Loam layer and the Intermediate Clay Complex. The field and laboratory tests carried out on the soils presented in the current subchapter have not been used for the determination of correlations in the chapter 8.

Location 1 - North-West of Bucharest

The first site is located in the north-west of Bucharest, where 8 geotechnical boreholes were drilled, which were doubled by CPT surveys. The stratification is synthetically described below:

- Natural ground elevation to -1,50 m - Fills
- -1,50 to -10,00 m - Bucharest Loam - Composed of clayey silts, sandy clayey silt and silty clays. Color varies from dark grey to brown. The layer is stiff to very stiff.

- -10,00 to -22,00 m - Colentina Gravels
- >-22,00 m - Intermediate Clay Complex

The groundwater level was measured at a depth of approximately 2 m.

As can be seen from Table 9-1 the differences determined according to the previous paragraph range from 2% to 60%, with an average value of approx. 32% for the determination of the oedometer modulus using the correlation determined in chapter 8 and between 1% and 55%, with an average value of approx. 28%, when using the safe value of the determined formula. The values of the oedometer modulus $E_{oed200\text{-}300}$ determined in Chapter 8 are in almost all cases higher than the values determined in the laboratory. Thus, the correlations lead to values with a lower degree of safety than the values determined in the laboratory. The relatively large differences are explained by the fact that the laboratory tests were carried out under saturated conditions due to the fact that in the case of the current location the groundwater level is high (about 2 m). Correlations were determined by correlating CPT tests with laboratory oedometer tests for samples at natural water content. The natural water content of the samples analyzed in this subchapter ranged between 20% and 31% with an average value of ca. 23 - 24% corresponding to a saturation degree between 0.90 and 0.99 with an average value of ca. 0.97. The natural humidity of the samples for which correlations were determined varied between ca. 18% and 25% with an average value of 21% corresponding to a degree of saturation ranging between 0.42 and 1.00, but with an average value of ca. 0.82.

Table 9-1 Comparison of the results of the $E_{oed200\text{-}300}$ oedometer modulus with the oedometer modulus obtained using correlations

Field test results	Laboratory test results	Values obtained by correlation		Differences	
q_c	$E_{Oed200\text{-}300}$	$E_{Oed200\text{-}300}$ initial value	$E_{Oed200\text{-}300}$ safe value	δ compared to the new correlation	δ compared to the safe value of the new correlation
[MPa]	[kPa]	[kPa]	[kPa]	-	-
1,25	5556	7801	7501	40%	35%
2,50	8928	12146	11846	36%	33%
1,85	6897	9887	9587	43%	39%
2,25	7067	11277	10977	60%	55%
1,75	6250	9539	9239	53%	48%
1,17	6667	7523	7223	13%	8%
1,33	6173	8079	7779	31%	26%
1,40	5831	8322	8022	43%	38%
2,75	9860	13015	12715	32%	29%
2,25	9390	11277	10977	20%	17%
1,45	8321	8496	8196	2%	1%
0,95	5450	6758	6458	24%	18%

Field test results	Laboratory test results	Values obtained by correlation		Differences	
q_c	$E_{Oed200\text{-}300}$	$E_{Oed200\text{-}300}$ initial value	$E_{Oed200\text{-}300}$ safe value	δ compared to the new correlation	δ compared to the safe value of the new correlation
1,15	6158	7453	7153	21%	16%
			Average difference	32%	28%

From the analysis of the results presented in Table 9.2, average differences between the determined value of the internal friction angle of ca. -7% can be observed, while for cohesion the average differences reach up to -18%. Analyzing the samples individually, the differences are considerably larger, ranging from +17% to -54% for the internal friction angle and from +55% to -136% for cohesion. It should also be noted that when the value of the internal friction angle varies in a positive direction, i.e., the value determined by correlations is higher than that obtained in the laboratory, the value of cohesion varies in a negative direction, i.e., the value obtained in the laboratory is lower than that obtained by correlations. This phenomenon can be explained by the fact that the shear strength is described by the paired values of φ and c and not by their individual values. The fact that the differences vary in the direction of lower oedometer modulus values in the laboratory tests are explained by the fact that in case of the new correlations, they were determined using the results of shear tests on samples at natural humidity, whereas the results presented below in Table 9.2 are obtained on saturated samples.

Table 9-2 Comparison of results of shear parameters φ and c obtained in the laboratory with parameters φ and c obtained by correlation

Field trial results	Laboratory test results		Coverage values obtained by correlations			
q_c	φ	c	φ		c	
MPa	°	kPa	°	δ compared to the lab values	kPa	δ compared to the safe values
1,25	18	30	18	2%	33	-9%
2,5	27	28	23	13%	43	-52%
1,85	23	40	21	11%	37	6%
1,33	14	55	18	-29%	33	39%
1,4	22	14	18	17%	34	-136%
2,75	16	52	25	-54%	44	15%
2,25	30	21	22	26%	41	-93%
1,45	18	38	19	-4%	34	10%
1,15	12.2	71.5	17	-41%	32	55%
			Average differences	-7%		-18%

Location 2 - North of Bucharest

The second location is located approximately 2 km to the east from the previously investigated site. In the current sub-chapter, the deformation and strength parameters of the Bucharest Loam layer and the Intermediate Clay Complex will be analyzed. Eight geotechnical boreholes were drilled on the site and were duplicated by CPT tests. The stratigraphy is summarized below:

- Natural ground level to -2,00 m - Fills
- -2,00 to -8,00 m - Bucharest Loam - Consisting of silty clays, clayey to clayey and sandy silts of stiff to hard, brown to reddish
- -8,00 to -17,00 m - Colentina gravels
- -17,00 to -27,00 m - Intermediate Clay Complex - composed of clays to silty clays and grey to dark grey clayey silt. The clays can be described as stiff.

The stabilized groundwater level varies between 7 m and 8 m below the natural ground level.

<u>Validation of correlations for the Bucharest loam layer</u>

In Table 9-3 is presented the comparison of the results between the values of the oedometer modulus for the loading step 200-300 kPa obtained in the laboratory and those obtained using the correlations presented in chapter 8 for the Bucharest Loam layer.

Table 9-3 Comparison of the results of the $E_{oed200\text{-}300}$ oedometer modulus with the oedometer modulus obtained by correlations for the Bucharest Loam layer

Field test results	Laboratory test results	Values obtained by correlation		Differences	
q_c	$E_{Oed200\text{-}300}$	$E_{Oed200\text{-}300}$ initial value	$E_{Oed200\text{-}300}$ safe value	δ from the initial value	δ compared to the safe value
[MPa]	[kPa]	[kPa]	[kPa]	-	-
3,81	16129	17603	16613	9%	3%
2,26	10000	9465	8475	-5%	-15%
2,75	12121	12038	11048	-1%	-9%
4,00	16393	18600	17610	13%	7%
1,82	11834	7155	6165	-40%	-48%
2,04	11834	8310	7320	-30%	-38%
2,64	12270	11460	10470	-7%	-15%
1,38	5970	4845	3855	-19%	-35%
			Average difference	-10%	-19%

As can be seen from Table 9-3 the differences determined according to the previous paragraph range from -40% to 13% with an average value of approx. -10% for the determination of the oedometer modulus using the correlation determined in chapter 8 and between -48% and 7% with an average value of approx. -19% when using the coverage value of the formula. The values of the oedometer modulus $E_{oed200\text{-}300}$ determined using the correlations are in almost all cases lower compared to the values determined in the laboratory. Thus, the correlations lead on average to safer values ranging between 10% and 20% compared to the values determined in the laboratory.

Table 9-4 shows the values of the shear parameters φ and c obtained in the laboratory compared to the parameters φ and c obtained using the new correlations presented in chapter 8. In this case only the results obtained using the safe correlations are presented, the difference between the original and the safe correlations are negligible.

Table 9-4 Comparison of results of shear parameters φ and c obtained in the laboratory with parameters φ and c obtained by correlation for the Bucharest Loam layer

Field test results	Laboratory test results		Coverage values obtained by correlations			
q_c	φ	c	φ		c	
MPa	°	kPa	°	δ compared to the safe value	kPa	δ compared to the safe value
1,82	20	31	20	-1%	35	21%
2,26	25	60	22	-12%	39	-32%
2,64	19	23	23	25%	42	88%
2,84	24	29	24	1%	43	54%
2,26	25	16	22	-12%	39	160%
Average differences				0%		58%

From the analysis of the results presented in Table 9-4 average differences between the determined value of the internal friction angle of ca. 0% can be observed while for cohesion the average differences reach up to +58%. Analyzing the samples individually the differences are considerably larger ranging between -12% and 25% for the internal friction angle and between -32% and 160% for cohesion. It should also be noted that although the value of the internal friction angle does not vary on average, the value of cohesion varies in a positive direction, i.e., the value obtained in the laboratory is lower than that obtained by correlation. This phenomenon can be explained by the fact that the shear strength is described by the pair of φ and c values and not by their individual values. Within this unit the materials are quite diverse which is why large variations obtained in the determination of the parameters are to be expected.

Correlation validation for the Intermediate Clay Complex

In Table 9-5 the comparison of the results between the values of the oedometer modulus for loading step 200-300 kPa obtained in the laboratory and those obtained using the correlations presented in chapter 8 for the intermediate clay complex are presented.

Table 9-5 Comparison of the results of the $E_{oed200\text{-}300}$ oedometer modulus with the oedometer modulus obtained by correlation for the Intermediate Clay Complex

Field test results	Laboratory test results	Values obtained by correlation		Differences	
q_c	$E_{Oed200\text{-}300}$	$E_{Oed200\text{-}300}$ initial value	$E_{Oed200\text{-}300}$ Safe value	δ from the initial value	δ compared to the safe value
[MPa]	[kPa]	[kPa]	[kPa]	-	-
3,48	14493	15526	15226	7%	5%
2,89	13245	13478	13178	2%	-1%
1,83	8547	9800	9500	15%	11%
2,09	9569	10702	10402	12%	9%
1,25	8163	7788	7488	-5%	-8%
			Average difference	6%	3%

As can be seen from Table 9-5, the differences determined according to the previous paragraph range from -5% to 15% with an average value of approx. 6% for the determination of the oedometer modulus using the correlation determined in chapter 8 and between -8% and 11% with an average value of approx. 3%, when the coverage value of the determined formula is used. The values of the oedometer modulus $E_{oed200\text{-}300}$ determined using the correlations are in almost all cases slightly higher than the values determined in the laboratory. Thus, correlations lead to less safer values than laboratory determined values. The differences are in this case almost insignificant. The correlations were determined by correlating CPT tests with laboratory oedometer tests for saturated samples.

In Table 9-7 values of the shear parameters φ and c, respectively c_u obtained in the laboratory compared to the same parameters obtained using the correlations presented in chapter 8 are presented. In this case only the results obtained using the safe correlations are presented, the difference between the original and the safe correlation being negligible. The correlations were determined using shear tests performed under saturated conditions similar to the tests presented in Table 9-6.

Table 9-6 Geotechnical shear parameters φ and c obtained in the laboratory and from correlations for the Intermediate Clay Complex

Borehole -	Sample depth m	Sample description	φ °	c kPa	q_c MPa	Shear type -
F1	18	Grey clay	17	46	2,50	CU
F1	22	Sandy clay	23	55	2,62	CU
F1	27	Grey clay	18	75	3,94	CU
F2	17	Clay	17	74	3,42	CU
F7	17	Clay	19	72	4,07	CU
F7	19	Clay	18	85	4,38	CU
FS2	38	Sandy clay dust	13	47	3,35	CU

Table 9-7 Laboratory and correlation-derived undrained shear strength c_u for the Intermediate Clay Complex

Borehole -	Sample depth m	Sample description	C_u kPa	q_c MPa	Shear type -
F1	35	Clay	131	2,00	TXUU
F2	25	Clay sand	176	3,50	TXUU
F4	19	Clay	129	1,75	TXUU
FS2	36	Sandy clay dust	108	2,19	TXUU
FS2	40	Clay	215	3,23	TXUU
FS3	36	Clay	109	1,87	TXUU

Table 9-8 Comparison of results of shear parameters φ and c obtained in the laboratory with parameters φ and c obtained by correlation

Field test results	Laboratory test results		Safe values obtained by correlations			
q_c	φ	c	φ		c	
MPa	°	kPa	°	δ compared to the safe value	kPa	δ compared to the safe value
[MPa]	[kPa]	[kPa]	[kPa]	-	-	
2,50	17	46	12	-28%	53	+14%
2,62	23	55	13	-45%	54	-2%
3,94	18	75	17	-8%	74	-2%
3,42	17	74	15	-13%	66	-10%
4,07	19	72	17	-8%	75	+5%
4,38	18	85	18	2%	80	-6%
3,35	13	47	15	18%	65	+38%
Average differences				-12%		+5%

Table 9-9 Comparison of the results of undrained shear strength c_u obtained in the laboratory with shear strength c_u obtained by correlation

Field test results	Laboratory test results	Safe values obtained by correlations
q_c	c_u	c_u

MPa	kPa	kPa	δ compared to the safe value
2,50	131	118	10%
2,62	176	197	-12%
3,94	129	105	19%
3,42	108	128	-19%
4,07	215	183	15%
4,38	109	111	-2%
Average differences			2%

From the analysis of Table 9.8 and Table 9.9 one can see that for the layer for the Intermediate Clay Complex there are differences between the determined values of the internal friction angle φ and the cohesion c. The average differences for the internal friction angle φ are about 12%. This means that the correlation values differ on average by about 12% from the laboratory determined values. When analyzing the samples individually, the differences can vary considerably, ranging from +45% to -18%. In the case of cohesion, the average differences are up to -5%. In other words, correlation values can be on average up to 5% lower than laboratory values. Shear strength values depend on the pair of φ and c values and not on their individual values. This means that when the internal friction angle has a lower value determined by correlations than that obtained in the laboratory, the cohesion has, on average, a higher value in the laboratory than that determined by correlations. At the same time, the differences between the value of the undrained cohesion c_u determined in the laboratory and that determined using the correlation presented in Chapter 8 range from -19% to 15%, on average taking the value of 2%.

In the light of all the above, it can be considered that the values of the mean shear strength differences determined directly in the laboratory, as well as those determined indirectly by the correlations obtained in chapter 8 lead to similar shear strength results. After the analysis of the second location, it can be noted that for the Bucharest Loam layer the values of geotechnical parameters were 10% to 20% higher by using the new safe correlations compared to the results obtained in the laboratory. Regarding the analysis of shear parameters, the cohesion determined by the correlations determined in Chapter 6 had on average a variation of about 60%, being lower than the value obtained by laboratory determinations. Also, the internal friction angle showed on average a variation between -12% and 25% between the values obtained by the correlations determined in Chapter 6 and those obtained in the laboratory.

For the Intermediate Clay Complex, the differences observed were negligible, ranging between 3-6% for the deformation moduli and between -5% and 12% for the shear strength parameters φ and c. For the undrained shear strength, the values obtained in triaxial and those obtained by in situ parameter correlation using the new correlations have an average variation of about 2%.

10 VALIDATION OF GEOTECHNICAL PARAMETERS BY BACK-CALCULATION

This chapter presents the validation of the proposed correlations obtained in Chapter 8 by back-calculation using finite element calculations.

The case study consists in the numerical modelling of a deep excavation built in the northern of Bucharest using several constitutive models such as Mohr-Coulomb, Hardening Soil and HS-Small. The height regime of the building for which the excavation is 3S+P+23E+Eth. The surface of the excavation is approximately 5,000 m^2 and is polygonal in shape. The results of the geotechnical monitoring are also presented. Based on the execution design and the geotechnical parameters previously determined, a calculation of the interaction of the ground with the structure will be carried out using the finite element method. Based on the obtained results it will be possible to draw useful conclusions on the validity of the determined correlations.

In order to be able to make an assessment of the parameters used in the modelling of the structure, data obtained from geotechnical monitoring were used. By analyzing the geotechnical monitoring data one can observe differences between measured and modelled displacements. Figure 10-5 shows the results of the inclinometer monitoring for inclinometers I_1 - I_2.

For the modelling of the soil-structure interaction, a deep excavation supported using diaphragm walls with a thickness of 80 cm and the base at -17.00 m was considered. The support of the excavation was carried out in a top-down system, using the floor above basement 3 (upper elevation -6.55 m; lower elevation -7.05 m) as the only element for supporting the enclosure walls before the final excavation.

The deep excavation was modeled using several construction stages. The main modelled stages are the following. Figure 10-1 shows the initial site situation where the infrastructure of the neighboring building and its diaphragm wall can be seen. Then follows the stage when the foundation piles and the diaphragm wall are constructed.

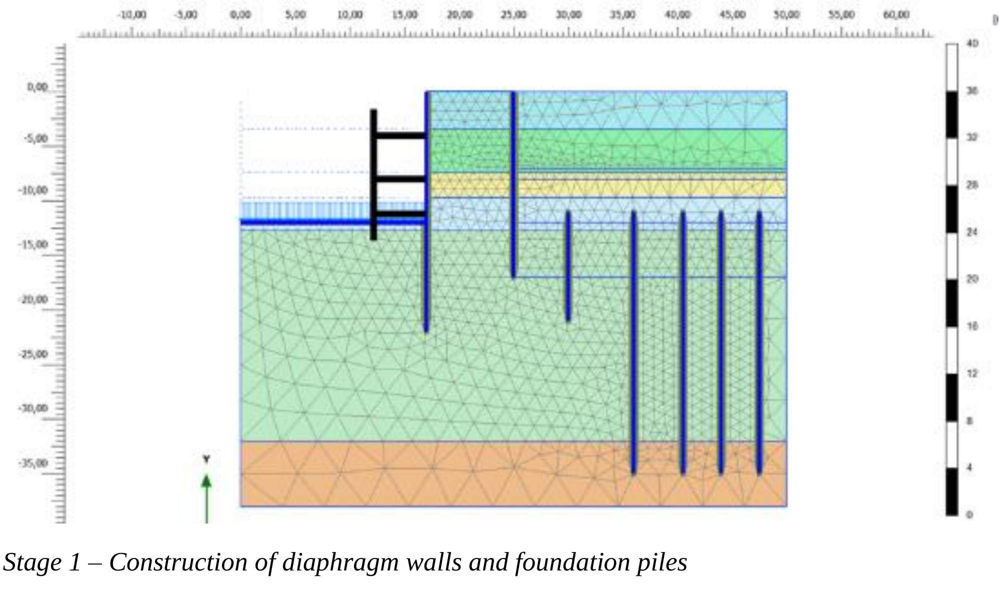

Figure 10-1 Stage 1 – Construction of diaphragm walls and foundation piles

In the next stage, the excavation is carried out down to -7.05 m as the groundwater level is lowered to 50 cm below the excavation level. This execution stage is shown in Figure 10-2.

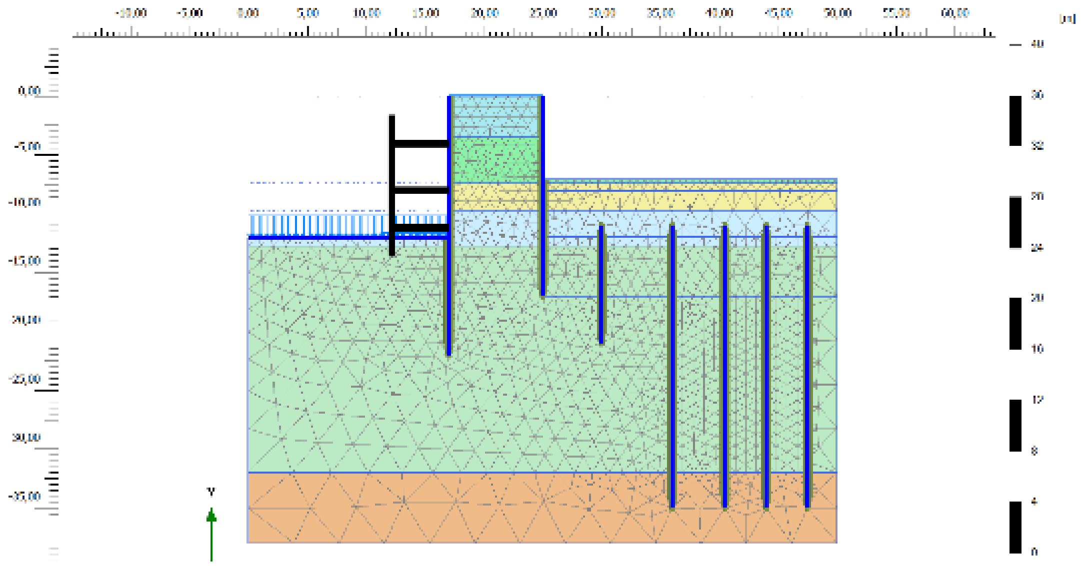

Figure 10-2 Stage 2 – Excavation to -7.05 m with groundwater level lowering to -7.55 m and casting the slab over the 3rd underground level

The last excavation stage down to an elevation of -12.00 m is shown in Figure 10-3. Before excavation the groundwater level is lowered 50 cm below the final excavation level.

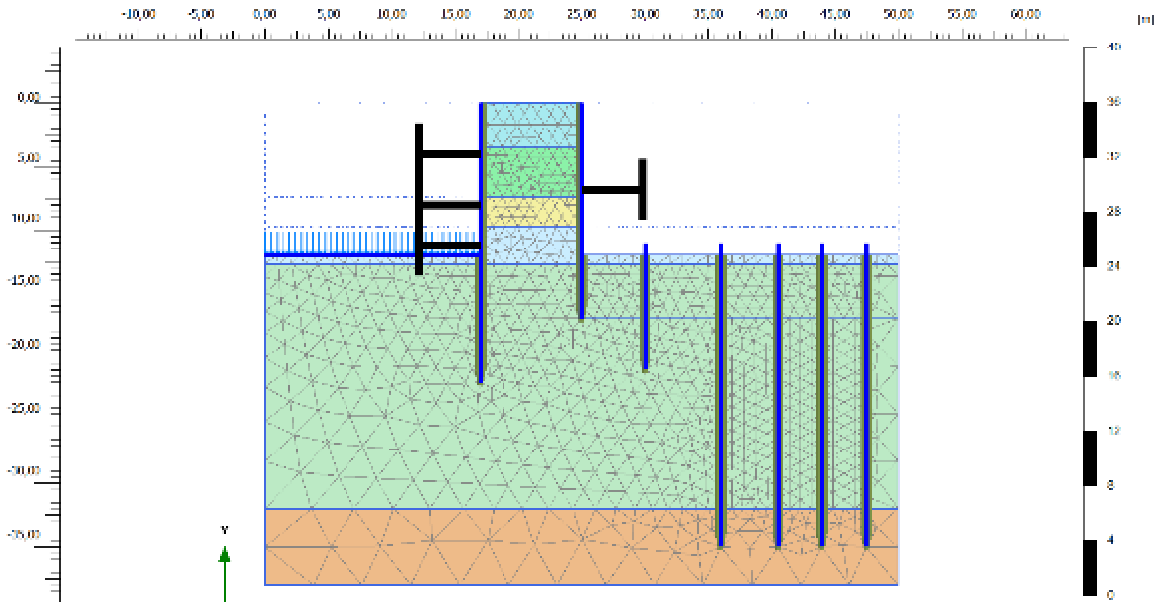

Figure 10-3 Stage 3 – Excavation to -12.00 m with groundwater level lowering to -12.50 m

The last stage of the modelling consisted in pouring the raft and the slab from to -9.55 m. This stage is shown in Figure 10-4.

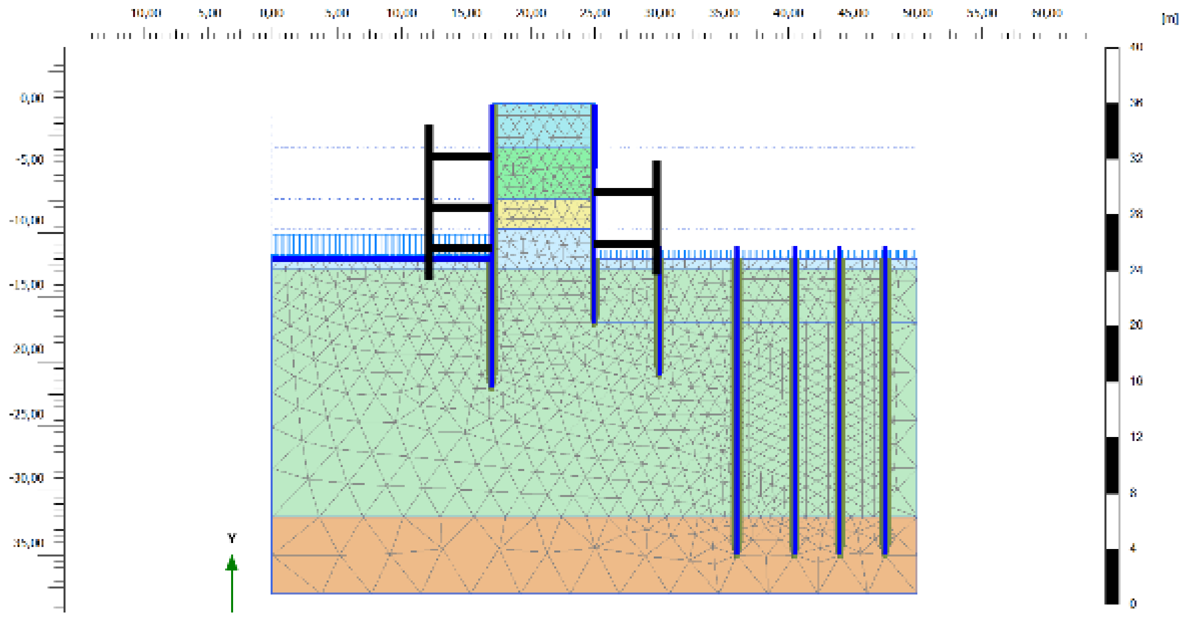

Figure 10-4 Stage 4 – Casting the raft and the slab at -9.55 m

10.1 OBSERVATIONS AND COMMENTS

In this paragraph comments and observations are provided on the base of the numerical modelling results in comparison with the geotechnical monitoring results. All numerical modelling results show higher maximum displacement values in the x-direction than the geotechnical monitoring results. This may indicate, among other things, a conservative estimation of geotechnical parameters. In the numerical modelling presented above the primary correlations proposed in Chapter 8 were used to determine the geotechnical parameters and not their safe values. Safe values are the values determined by the translation of the regression line with the values of the standard deviation. Using the safe value of the correlations in Chapter 8 is supposed to lead to an even more conservative estimate of the displacements. The shape of the displacement diagram of inclinometers I1 and I2 shown in Figure 10-5 is reflected in the numerical modelling results using the Hardening Soil and HS-Small constitutive models. When using the Mohr-Coulomb model, the horizontal displacement diagram shows a significant displacement of the diaphragm wall along its entire length, caused - according to the modelling - by a significant displacement of the excavation base in the x-direction. The displacement of the excavation base is also captured when using Hardening Soil and HS-Small models, but it has a much lower value, and it is close to the monitoring results.

In absolute terms of horizontal displacement, it can be seen that the modelling the excavation using advanced constitutive models, such as HS-Small and Hardening Soil, provides results that almost match the results from the geotechnical monitoring, which is to be expected.

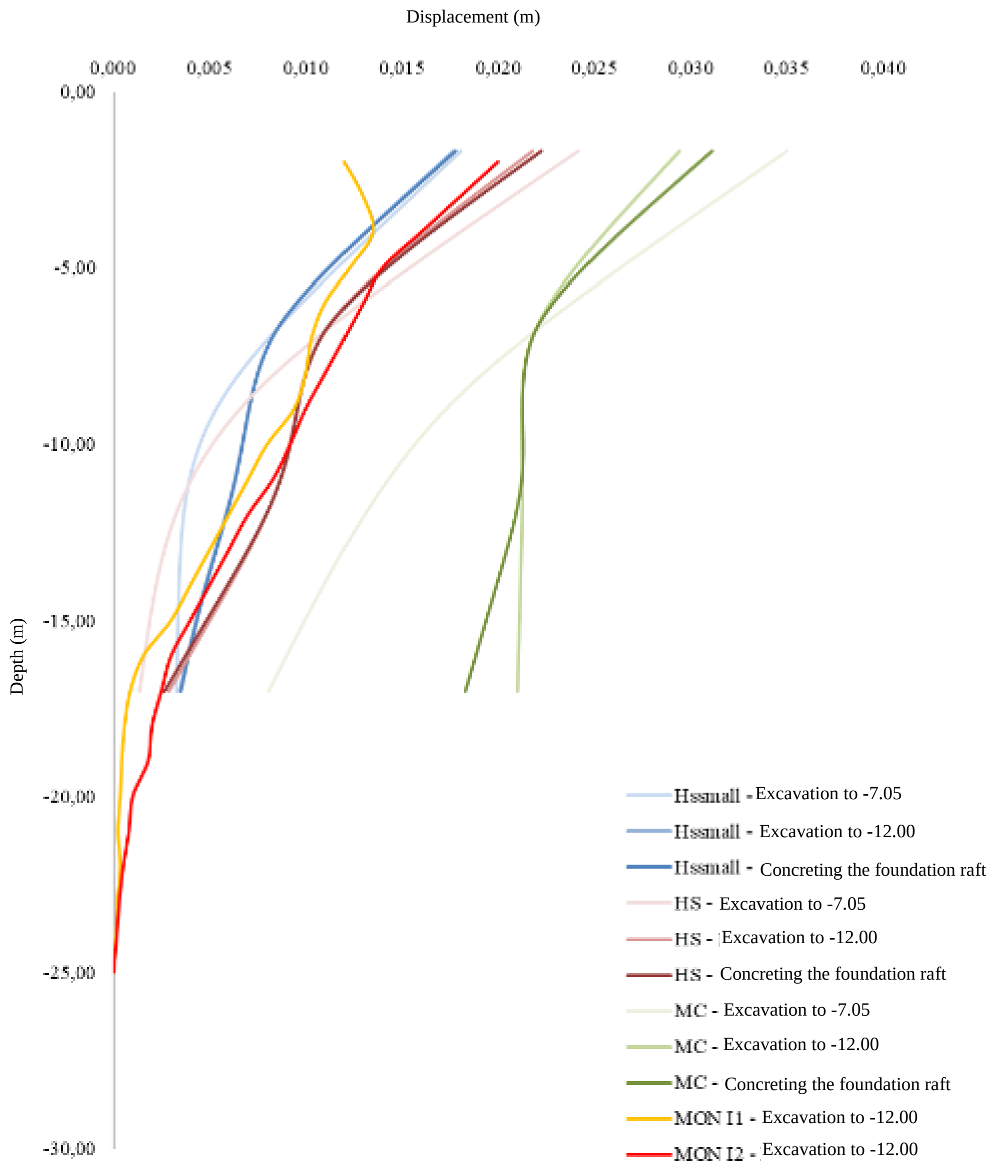

Figure 10-5 Geotechnical monitoring and numerical modelling results - maximum displacements in x-direction (horizontal)

The monitoring results indicate an increase of the horizontal displacement ranging from 0.6 cm to 0.8 cm in Stage 1 and from 1.2 cm to 1.8 cm in Stage 3. Following the displacements resulting from the numerical modelling, it can be seen that the maximum displacement was obtained in Stage 1 of the excavation, before the first slab was poured as a supporting element of the excavation. This phenomenon can be explained by the fact that the software simulates the final phase of an excavation stage with displacements and consolidation processes being consumed. In reality, as the execution is carried out quite quickly, the ground has not yet consumed all its displacements. In the modelling of Stage 2 and Stage 3 of the excavation, a slight return of the horizontal displacement at the top of the diaphragm wall is observed, with an increase of displacement at the bottom of the wall. The displacements at the bottom of the diaphragm wall show values between 0.5 and 2 mm for the Hardening Soil and HS-Small models and 1.8 cm for the Mohr-Coulomb model. The heaving of the excavation base from numerical modelling has values between 0.5 mm and 1.2 cm for the HS-Small and Hardening Soil models and about 2 cm for the Mohr-Coulomb model. The heaving of the excavation base observed during the geotechnical monitoring is similar to the mathematical modelling using the HS-Small constitutive model. Vertical displacements near the diaphragm wall were not monitored. Instead, the measured displacement of the excavation base indicates a heaving of about 2 - 3 cm.

It should be noted that even a detailed numerical modelling cannot capture all the complex phenomena that occur within the soil mass. In addition, the external forces acting on the soil mass as well as on the structural elements within the modelling can only be estimated and their exact value varies more or less constantly. Also, some dynamic loads from traffic on the surrounding street network have been included in the calculation as estimated static cover loads.

In view of the above it can be concluded that mathematical modelling using FEM models can provide good and reliable results for geotechnical design. Also, the results of the modeling using parameters determined with the new correlations reveal similar displacements to the one measured during the geotechnical monitoring of the excavations. Nevertheless, due to the fact that many variables and assumptions are needed to create a FEM model, a calibration of the determined new correlations is not the most suitable by using back calculations.

11 CONCLUSIONS, PERSPECTIVES AND PERSONAL CONTRIBUTIONS

This chapter presents the conclusions of this PhD thesis and proposes recommendations for the determination of geotechnical parameters for soils specific to the Bucharest area based on correlations between in situ and laboratory tests.

In-depth study of the field of in situ investigation and geotechnical monitoring, which is still undervalued in Romania, can lead to advanced knowledge of the estimated behavior of the new structures and to an optimization of their design.

Chapter 2 of this paper describes, as part of the literature survey, the main investigations used to determine the physical and mechanical parameters of soils.

Chapter 3 is devoted to presenting the main geotechnical monitoring methods and tools used in current engineering practice. Collecting, analyzing and interpreting this information is necessary both during the execution of a structure as well as after its completion in order to assess the assumptions made during the design. If deviations from the designed situation occur, corrective measures can be taken and the risk associated with unforeseen failure is reduced.

As part of the literature survey and in order to achieve the objectives of the current research, information on numerical modelling in geotechnical engineering is summarized in Chapter 4. With the increase in computational power and the development of specialized software for calculating the interaction of the foundation soil with the structure, numerical modelling has become the preferred computational method used in current design.

A substantial contribution was dedicated to the research of correlations between in situ investigations and soil parameters determined in the laboratory existing in both national and international literature. The synthesis of this part of the literature survey is presented in Chapter 5.

The development of new correlations according to the proposed objectives requires, among other things, knowledge of the stratification specific to Bucharest. Thus, an experimental research program was proposed which included field and laboratory tests for a number of 22 sites. Within the program, 79 boreholes, 88 CPT cone penetration tests and no less than 34 Marchetti flat dilatation tests (DMT) were carried out. The field and laboratory geotechnical investigations were provided by SAIDEL Engineering S.R.L. company.

Determination of stratification and geotechnical parameters is the starting point for a thorough geotechnical design. This must be complemented by a correct analysis and interpretation of their values. According to the current and future editions of Eurocode 7, the interpretation of field and laboratory test results must be completed by determining the derived values of the geotechnical parameters, followed, at the geotechnical design stage, by the determination of the representative/characteristic values. The application of partial safety factors then leads to the design values used in the design by limit state calculation.

Chapter 7 is devoted to the statistical analysis of geotechnical parameters. The chapter defines the steps required to calculate the characteristic values and gives examples for determining the characteristic value for independently determined variables as well as for correlated variables such as the shear parameters φ and c.

Based on the results of the in situ and laboratory geotechnical investigations presented in Chapter 6 and on the statistical notions presented in Chapter 7, as well as on the bibliographical synthesis summarized in Chapters 2, 3, 4 and 5, new correlations between the results of the in situ geotechnical investigations and the geotechnical parameters of the soils are proposed in Chapter 8, adapted to the specific soils of Bucharest.

In Chapter 9, two case studies evaluated the new correlations. The aim of these was on the one hand to validate the new correlations by comparing their results with the results of existing correlations in the literature. For each parameter and geological layer studied in Chapter 6 a range of variation was obtained. The specific plots for each parameter studied were completed with the range of variation of the laboratory tests corresponding to a 95% confidence level determined in Chapter 6. Differences between the results obtained using the proposed new and existing correlations were analyzed and commented for each geotechnical parameter.

A new validation of the correlations described in Chapter 8 was carried out by means of an analysis in Chapter 9, by evaluating the geotechnical parameters obtained using the proposed new correlations in comparison with the results of geotechnical laboratory tests for two sites in Bucharest. It can be commented that the Bucharest Loam is quite varied in grain size composition and plasticity, while the intermediate clays are more homogeneous.

Chapter 10 presents a numerical modelling in which the deformations of a deep excavation resulting from finite element modelling using geotechnical parameters obtained by applying the proposed correlations and geotechnical monitoring results are compared. A deep excavation in the northern area of Bucharest was modelled using FEM-Software using several constitutive models such as Mohr-Coulomb, Hardening Soil and HS-Small. Based on the obtained results, useful conclusions could be drawn on the validity of the proposed correlations.

Based on the validations, it was concluded that these proposed correlations approximate quite well the representative geotechnical parameters of the main lithological units characterizing the soil in Bucharest area. Thus, they can constitute the starting point of geotechnical databases that allow a wider use of in situ geotechnical tests, knowing that they are limited due to the necessity of correlating their results with the geotechnical parameters that are used in the calculations.

There is also a need to adapt the design methods used nationally to take direct account of the results of in-situ tests, without going through correlations with geotechnical parameters, which introduce additional errors.

11.1 PERSONAL CONTRIBUTIONS

The present paper is addressing complex issues related to the determination of geotechnical properties of soils, a matter which, although common, is not exact, but involves multiple errors and uncertainties that are difficult to quantify and control and that affect the final design outcome.

In the paper, an extensive and up-to-date literature survey has been carried out, the current soil investigations like in situ and laboratory tests have been synthesized, the tools necessary for proper monitoring of geotechnical structures have been presented and detailed, and a synthesis of numerical modeling in geotechnical engineering has been made, with the presentation of yield models and constitutive models used in current engineering practice. The main contribution of the literature review is the research, presentation, synthesis and analysis of existing correlations for different types of cohesive and non-cohesive soils.

In the PhD thesis, data resulting from the investigation of 22 sites in the Bucharest area were used, through which the main lithological units were identified and described from a geological and geotechnical point of view, such as Bucharest Loam, Colentina Gravels, Intermediate Clay Complex, Mostistea Sands. The variation of the geotechnical parameters resulting from the investigations was presented in the form of graphs and tables, elements that are of great use for users who perform statistical analyses for the determination of characteristic values of geotechnical parameters.

As part of the research, this paper presents and analyses how to determine derived, characteristic, representative values of geotechnical parameters, in view of the planned revision of Eurocode 7. Two case studies detail the steps required to determine characteristic values for independent variables and correlated variables such as the shear parameters φ and c. In addition, the differences between the two types of statistical processing for the shear parameters φ and c are presented.

Another personal contribution is the development of a methodology for selecting laboratory and in situ test results that can lead to empirical correlations between two studied parameters. The principles of mathematical statistics with which correlations between two parameters can be determined, verified and validated have also been summarized.

A method for processing and determining correlations for the correlated shear strength parameters φ and c was also proposed in the thesis.

The main personal contribution is the development of new correlations between geotechnical field tests and geotechnical parameters of soils in the study area. Thus, correlations were developed for mechanical behavior parameters using only in situ tests for the Bucharest Loam, Colentina Gravels, Intermediate Clay Complex and Mostistea Sands layers.

The proposed correlations were validated by parallel analysis of their results with the results provided by correlations already existing in the literature. The validation of the correlations was continued by parallel analysis of the results provided by the newly developed correlations and laboratory tests on two sites in Bucharest that were not part of the locations for which the correlations were determined. This analysis allowed the adaptation of the new correlations for the determination of the deformation modulus and the shear parameters for the case of saturated Bucharest Loam. By back analysis of a deep excavation using the finite element program Plaxis 2D, the general validity of the resulting geotechnical parameters using the proposed new correlations was studied by comparing the results of the numerical analysis with the results obtained from the geotechnical monitoring.

The final aim of this research was to verify the existing correlations in the literature for soils specific to the Bucharest area and to develop new correlations adapted to local conditions, as well as to compare the results provided by the new correlations with those existing in the literature. The new correlations and the recommendations for the determination of characteristic values as well as the statistical analysis of geotechnical parameters can be used in current engineering practice.

11.2 POSSIBLE DIRECTIONS FOR THE DEVELOPMENT OF CURRENT RESEARCH

The PhD thesis aimed to contribute to the development of new correlations specific to the Bucharest area. The new correlations are based on data obtained from geotechnical investigations on 22 sites in Bucharest. By extending the database the proposed correlations can be corrected or their accuracy can be increased. By integrating more sites, it is possible to detail the proposed new correlations according to a specific area within Bucharest.

Although the Marl Complex has been identified and described, there have not been sufficient in situ investigations to be able to determine correlations between the results of these investigations and laboratory tests. The development of correlations for this layer and for deeper layers, for example the Fratesti Sands, may constitute research of itself.

Another possible research direction could be to verify and validate the correlations proposed in the current paper for areas bordering Bucharest.

The development of AI-based solutions for analyzing, validating and adapting the proposed correlations may be a direction for development.

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