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Summary

PROJECT REPORT

for the Degree of

DOCTOR OF PHILOSOPHY

(Civil Engineering)

**Criteria for verifying the improvement of collapsible soils
as established by specific geotechnical tests**

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1. INTRODUCTION

1.1. General aspects

Collapsible soils (PSU), loess, are unsaturated macro-porous cohesive soils that, when in contact with water, undergo sudden substantial and irreversible changes of the internal structure, reflected by additional settlements (collapse), decrease of the geotechnical parameters values and change of mechanical behaviour. As a result of this specific phenomenon, can occur significant degradations of constructions build on PSU. Thus, soils sensitive to water are classified as difficult foundation soils and when used as such they have to be modified and improved. This category includes loess, loessoid soils and other predominantly silty soils, with high and uneven porosity.

Loess covers about 10% of the Earth's surface. In Romania it is found on 17 – 19% of the territory, mainly in the southeast. Therefore, the need to use this soil for supporting foundations is common.

Different methods can be used to improve PSU depending on the requirements of the project. One method of improvement is dynamic compaction (DC). This method improves the geotechnical characteristics of the soil and consumes the majority of settlement before the execution of constructions. It is used for road and railway platforms, industrial areas, oil tanks, silos, port lands, landfills, etc.

DC consists of improving the ground to an estimated depth by applying repeated impacts with high energy. Soil improvement by DC is achieved by densification through shear stresses induced by high energy blows on the soil surface level. The blows break the bonds between the loess particles in this way restructuring them.

In order to verify the ground improvement are performed specific in situ and laboratory tests. To verify the ground improvement by laboratory tests are required a large number of tests and time. In situ tests are faster and more economical than laboratory tests and provide a semi-continuous profile. Empirical correlations can be used to determine the properties and behaviour of the soil in the studied location.

1.2. Scope of research

Taking in consideration the continuously growing need to build on water sensitive soils and the interest in using DC as a method to improve them, is proposed a more efficient and faster methodology for attesting PSU improvement by dynamic compaction. In this regard, considering my direct involvement for a long period of time at a project located in the south-east of the country, close to the city Constanta, were performed in situ and laboratory tests according to a methodology that was improved during the research and which proved to be appropriate on a real scale.

The present project report was developed in order to contribute at the establishment of criteria for verifying the improvement of water sensitive soils (PSU) and to develop correlations between in situ and laboratory test in order to optimize the certification of their desensitization.

This topic is in line with current preoccupations regarding the development of Eurocode 7, where will be introduced a special chapter for diffused soil improvement - which can also be achieved through DC (EN1997-3 – Chapter 10).

1.3. Research objective

- 1 – Synthesizing the knowledge about PSU behaviour and the technics used to improve it.
- 2 – Development of criteria for attesting the improvement of PSU by DC.
- 3 – Implementing a case study in order to verify the criteria for ground improvement.
- 4 – Elaboration of recommendations for verifying the improvement of water sensitive soil by specific geotechnical tests and by using correlations in accordance with the provisions of Eurocode 7.

1.4. Paper organization

This report is organized in seven chapters. The first part comprises the literature review presenting in detail the characteristics of PSU soils and the method for ground improvement with DC. The second part of this report describes the works executed in the context of a testing program and establishes criteria for certifying the elimination of sensitivity to wetting for PSU deposits.

2. GENERAL CHARACTERISTICS OF WATER SENSITIVE SOILS (PSU) – LOESS

2.1. General presentation

Loess has a metastable structure. In its natural dry phase it behaves as an unsaturated stiff cohesive soil with a rigid skeleton and medium compressibility. Typically it has a high sensitivity to water. On contact with water the chemical bonds between particles soften and there are sudden and irreversible changes of its physical and mechanical properties resulting in additional settlement, which may be triggered under construction loads or even under overburden pressure. Thus loess is considered a difficult foundation soil deposit. In figure 2.1 is presented a loess deposit from Dobrogea.



Figure 2.1: Loess deposit Dobrogea area (personal archive)

2.2. Origins and classification

Loess has been studied since the 19th century. In Romania, the first normative regarding the foundation on water sensitive soils was prepared in 1960. The most widespread theory of the origin of loess is aeolian theory, formulated by Richthofen after studying loess deposits in China. The classification of a PSU is based on the estimation of additional settlement occurred at wetting under its own weight, I_{mg} (NP 125/2010).

2.3. Structure and composition

The loose structure of the loess is given by the shape and character of the surface of the component elements, by their position and by the type of the bonds. Thus, in the structure of loess, there are much larger pores, visible, called macro-pores, whose diameter can reach 1-2 mm (Figure 2.2). Macro-pores are resistant to external actions applied to loess deposits due to the clayey cement that covers their walls. The cement formed in the macro-pores has a higher mechanical strength than the one that can be found in fine pores. Loess is a poly-mineral soil.

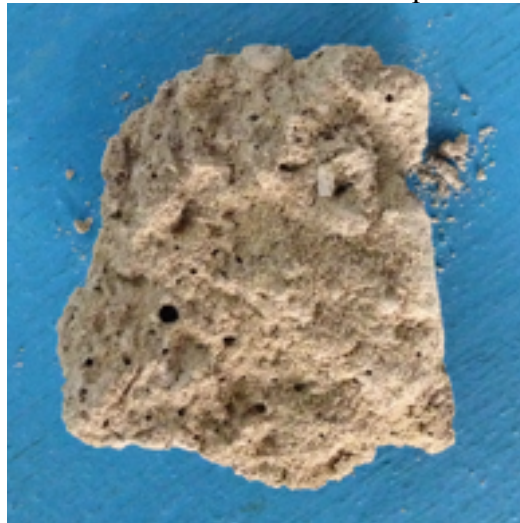


Figure 2.2: Characteristic appearance of undisturbed loess (personal archive)

Loess has a high content of silt particles, and a clay fraction below 30%. The silt particles are placed in the form of honeycombs connected to each other by clay particles and calcium carbonate. Figure 2.3 shows the structures of the loess before and after wetting.

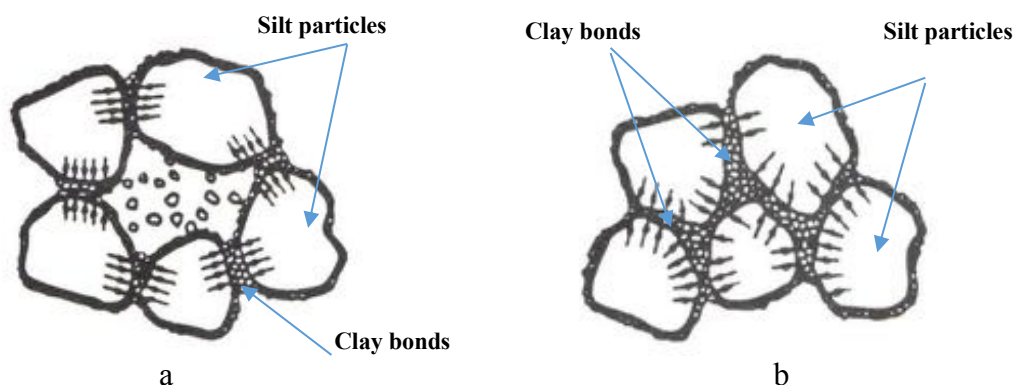


Figure 2.3: Loess structure: a. Natural state; b. After wetting (Donald P. Coduto, 1994)

2.4. Geographic distribution

Loess deposits can be found on all continents. About 10% of the Earth's surface is covered with loess soils (Figure 2.4). The largest loess deposits are in northern China. The thickness of the loess deposits can vary very much depending on the relief. In Romania loessoid deposits cover approximately 40,000 km², representing about 17 - 19% of the territory (Figure 2.5). They are found mainly in the Romanian Plain, in central and southern Dobrogea and in the Moldavian Plateau.

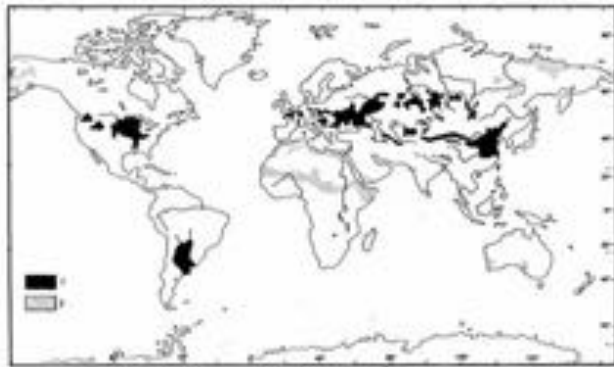


Figure 2.4: Loess deposits (Pecsi, 1990)



Figure 2.5: Loess deposits in Romania (NP 125-2010)

3. SPECIFIC PROPERTIES OF COLLAPSIBLE SOILS (PSU)

3.1. Identification criteria

In order to classify a soil as collapsible (PSU) must be determined the following geotechnical characteristics:

- Chemical - mineralogical composition;
- Physical properties: granulometry, density, plasticity, humidity, porosity;
- Hydraulic properties: permeability in natural state and for different stages of loading;
- Mechanical properties: compressibility in natural state and at saturation, structural resistance, shear strength in natural state and at saturation.

PSU are cohesive soils consisted mostly from silt particles ($d = 0,002 - 0,062$ mm), 50 – 80%. The saturation ratio of loess is small $S_r < 0,80$, and it has high porosity at natural state, $n > 40\%$.

The additional settlement index (i_{m300}) is determined with the oedometer test at 300kPa ($i_{m300} \geq 2 \text{ cm/m} = 2\%$). With the plate test, loaded up to a pressure of 300kPa, are determined the indices η and δ regarding the settlements in natural state and at saturation:

$$\eta = \frac{s_i}{s_n} \geq 5 \text{ și } \delta = s_i - s_n \geq 3 \text{ cm},$$

Where: s_i – settlement at saturation and s_n – settlement at natural state

According to NP 125-2010 loess is classified as collapsible if at least one of the physical and one of the mechanical characteristics mentioned above are valid simultaneously.

3.2. Physical properties

Loess are cohesive soils with silt as dominant fabric (50 – 80%). The clay fraction is found in a small proportion of 5 – 20%. Depending on the proportion of granulometric fabric, loess can be classified as sandy, silty or clayey.

Collapsible loess deposits have a low plasticity index, high porosity (typically over 40% – it can reach 60%) and a correspondingly low unit weight ($\gamma = 11 - 18 \text{ kN/m}^3$) especially when large pores are formed in the vertical direction.

Usually, the plasticity index is low. It is based on the grain size distribution and mineral composition. Soil moisture, typically, in natural state, varies between 6 and 15%.

In general, deposits of loess with a pore index smaller or equal than 0,5 is considered a good foundation soil, and a deposit with a pore index greater or equal than 0,9 requires to be improved. Loessoid deposits with a porosity of smaller than 40% can be considered theoretically insensitive at wetting (Larionov, 1966).

3.3. Hydraulic properties

Loess permeability is high, the permeability coefficient k ranges between 10^{-4} to 10^{-6} m/s. Permeability is influenced by the geotechnical properties of the soil, namely porosity, particle size and structure.

3.4. Mechanical properties

3.4.1. Compressibility

Under natural conditions of humidity, loess behaves, in terms of compressibility, similar to other categories of soils. However, at saturation, additional settlement occurs. In the study of the mechanical characteristics for loess must be examined both situations. The study of their compressibility can be done in the laboratory, with the odometer, or in situ by plate tests or experimental floods.

The oedometric test can be simple or double. The test on a single oedometric sample is performed at natural humidity up to the pressure of 300kPa, when it is saturated. For the double test one of the samples has natural humidity and the other is saturated before the beginning of the test. Parameters obtained in the laboratory by oedometric tests include the oedometric deformation modulus ($E_{200-300}$) in natural state and at saturation, the additional settlement index (i_{m300} , mandatory index for PSU identification) and the structural strength σ_0 .

3.4.2. Shear strength

For loess, the angle of internal friction (φ) has, in general, values of $5^\circ - 25^\circ$.

3.4.3. Cohesion

The cohesion (c) has values of 10 – 30kPa. The fabric of loess consists of silt and sand particles connected to each other by clay particles. The cement formed between particles can contain different amounts of salts, oxides or other simple mineral compounds. They are found in

crystalline form forming bonds between particles or as amorphous films that cover the walls of pores, especially macro-pores. The sensitivity of the loess at wetting is also influenced by the percentage of bonds made up of soluble salts.

3.4.4. *Non-uniformity*

The non-uniformity of the loess can be local (micro-non-uniformity) or can be found on the entire loess deposit. The micro-non-uniformity of the loess leads to large differences of porosity determined in samples taken at very close distances.

3.4.5. *Tensile strength*

Tensile strength varies with humidity and depends on the friability of the loess. At high humidity the tensile strength decreases due to the softening of the material, and at low humidity the resistance decreases because the material becomes brittle.

3.4.6. *Dynamic parameters*

Dynamic parameters can be obtained by in situ and laboratory tests, on samples at natural state and at saturation (by dynamic triaxial apparatus, resonant column, etc.).

3.4.7. *Liquefaction*

The phenomenon of liquefaction of loess can occur only when the degree of saturation $S_r > 0,8$.

3.5. Design aspects

In order to choose the appropriate improvement technologies for loess and the design of the foundation systems it is necessary to evaluate the areas sensitive to water and its classification. Are taken into consideration the settlement at natural state (s_n) and the additional settlement (I_m) that occurs at saturation. The layer that are sensitive to water can be defined based on the structural strength σ_0 . According to NP 125/2010, several deformable areas are distinguished in the foundation soil.

3.6. Conclusions

Given the different behaviour of loess in the natural state and at saturation, this type of soil is included in the category of difficult foundation soils and it is recommended to improve them. The study of loess involves identifying and understanding the causes that lead to changes in physical and mechanical properties due to moisture variations, offering the possibility of limiting the negative effects that these changes in properties can transmit to buildings. The geotechnical design of loess foundation systems must take into account the specific behaviour of this type of soil and provide special construction measures to reduce or eliminate the settlements caused by the presence of water.

4. GROUND IMPROVEMENT TECHNIQUES FOR LOESS

4.1. Introduction

Considering the development of infrastructure projects and also the urban development and the complexity of the projects, the need of construction on difficult foundation soils is more and more common. The choice of the PSU ground improvement technology depends mainly on the destination of the construction and the constructive solution adopted. Protective measures must be taken into account when choosing the foundation solutions. NP 125 - 2010 provides several instructions for the protective measures.

The improvement of the PSU aims first of all to eliminate the water sensitivity, but also to limit the differential settlements and increase the bearing capacity. The optimal method of ground improvement should take into account technical and economic criteria and environmental protection. Therefore, the research and development of ground improvement technologies is continuous.

4.2. Dynamic Compaction (DC)

4.2.1. General aspects

Dynamic Compaction (DC) is a technique widely used for increasing the bearing capacity and limiting potential settlement of the foundation ground before the execution of the construction. the technique was used to improve loose sands, silts, sandy clays and collapsible soils.

CDI is a ground improvement technology used for hundreds of years but it was engineered by Louis Menard in the 1960s. He presented it in detail in 1975. With the development of crane technology and the requirement of foundation on very large areas (commercial centres, highways) this method started to be widely used.

The main objectives of DC are:

- Increase bearing capacity;
- Limit potential settlement and differential settlement under additional load;
- Fulfil the conditions regarding the verification of the operating limit state (SLE) and the ultimate limit state (SLU);
- Reduce porosity;
- Reduce of voids in the ground;
- Eliminate the collapse potential (where applicable);
- Reduce the liquefaction potential.

Dynamic compaction consists of improving the ground to a calculated depth by repeated high energy impacts. Thus, a weight of 10 – 40 tons is allowed to fall freely and repeatedly from a height of 10 – 30 meters on the same footprint. For each impact point are typically applied 5 to 20 blows. Impacts creates waves. The effect of shear waves is to rearrange the ground particles into a more dense shape. The procedure is performed in 3 – 4 phases. The blows are applied on a designed grid pattern.

The first compaction phases are performed with high energy, having the role of compacting the ground in depth. Greater attention is needed to the design of the execution grid as a very hard surface layer can be created which can prevent the improvement in depth. The last phase of

compaction is called "ironing" and is done with smaller energy. It is applied in order to improve the ground at the surface. The blows are applied from a height of 5 – 6 m on an overlapping grid. The craters formed after the execution of each compaction phase are filled with local or input material. At the end, the ground is levelled and compacted with rolling compactors. Depending on the nature of the soil, can be reached an improvement of 9 – 12 m depth.

4.2.2. Design considerations

4.2.2.1. Applied energy

The compaction energy is determined according to the required compaction depth.

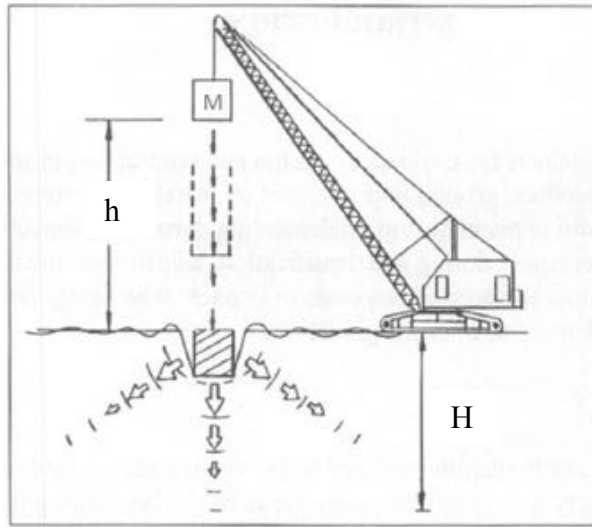


Figure 4.1: Computing the depth required to be compacted (Menard)

Thus, the depth required to be compacted is calculated with the relation (Figure 4.1):

$$H = m * \sqrt{M * h}$$

Where:

H – depth required to be compacted (m);

M – pounder mass (t);

h – drop height (m);

m – empirical coefficient based on soil type

The total compaction energy is determined according to the depth required to be treated and the type of soil. The energy for each compaction phase is calculated as follows:

$$E = N * M * h / (da)$$

Where:

E – applied energy (tm/m²);

N – number of drops at each specific drop point location

M – pounder mass (t);

da – distance between drop points (m)

4.2.2.2. Characteristics of the pounder

The pounder weight can be determined using the relationship:

$$M = H + 2$$

Where:

H – depth required to be compacted;

M – pounder mass in tones

The size of the pounder is very important in order to control the contact pressure at impact.

4.2.2.3. Grid spacing and number of drops

The grid spacing plays a significant role in order to obtain the desired compaction. Because part of the energy is distributed laterally in the ground, the distance during the first compaction phases is chosen depending on the area of influence. The distance between the impact points must be at least equal to the area of influence. Generally, between 5 and 20 blows are applied at each drop point for each phase.

4.2.2.4. Number of compaction phases

It is difficult to determine exactly the number of compaction phases during design because it must be observed the reaction of the ground to the dynamic stresses. However, in general, two deep compaction phases are performed with equal energies. The third phase is executed with lower energy. At the beginning of the work it is indicated to perform an experimental polygon in order to verify the result of compaction by geotechnical tests performed after each phase.

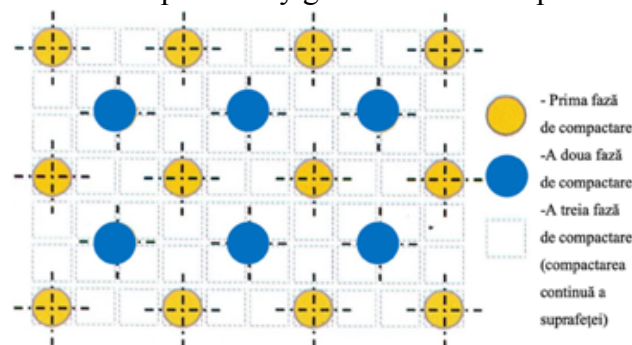


Figure 4.2: Typical compaction phases (Menard)

4.2.3. Soil behaviour during DC

By applying Dynamic Compaction for PSU, the fabric structure is modified by breaking the bonds between the particles, which are rearranged into a more compact shape. This changes the densification state and reduces or eliminates additional settlements due to wetting.

Based on practical experience and studies, were analysed the effects of DC on ground behaviour. The settlement of the foundation ground is faster and higher by the intermittent application of a load than the application of the same permanent load. DC induces considerable settlements for different soil types.

5. CRITERIA FOR VERIFYING GROUND IMPROVEMENT

5.1. Introduction

Verification of the improvement of loessoid soils is done by specific in situ and laboratory tests. The main purpose of the works performed for the improvement aim the elimination of sensitivity to wetting for PSU deposits, reducing or even eliminating the related settlement. A

soil can be classified as water sensitive based on the identification criteria provided in the technical regulations. These criteria were detailed in Chapter 3. In order to verify the ground improvement, are analysed the changes of these parameters after the application of improvement technologies.

In order to consider that loess is no longer collapsible the following criteria were established (Manea et al., 2012):

- Decreasing the additional settlement index $i_{m300} < 2\%$
- Increasing the dry density $\rho_d > 1.6\text{g/cm}^3$
- Reduction of the porosity $n < 40\%$
- Increasing the average odometer modulus $E_{200-300}$ at natural moisture and saturation
- Increasing the average cone resistance $q_c > 2.5\text{MPa}$

In addition, the following conditions must be met also in order to certify the quality of the works (Manea et al., 2012):

- $i_{m300imb} / i_{m300nat} < 1$;
- $\rho_{dimb} / \rho_{dnat} > 1$;
- $n_{imb} / n_{nat} < 1$;
- $E_{200-300imb} / E_{200-300nat} > 1$;
- $E_{200-300imbwsat} / E_{200-300natwsat} > 1$;
- $q_{cmedimb} / q_{cmednat} > 1$.

5.2. Laboratory tests

5.2.1. Parameters determined in the laboratory

The laboratory tests that are performed in order to determine the characteristics of the PSU must show the nature of the material, the composition, the moisture, the state of consistency, the state of compaction, the compressibility and the water sensitivity. In this regard are performed geotechnical borings with disturbed and undisturbed sampling. Due to the nature of the PSU, special attention must be paid to the execution of the geotechnical investigation. Thus, the drilling is done in dry conditions and the samples are taken with a thin-wall tube (Shelby) or large-diameter tubes and monoliths or block samples so as not to change the state. The tests performed include the determination of the following parameters:

- Granulometry;
- Humidity (w);
- Saturation degree (S_r);
- Atterberg limits, plasticity index (I_p) and consistency index (I_c);
- Porosity (n);
- Pore index (e);
- Dry density (ρ_d);
- Additional settlement index (i_{m300}) and deformation modulus ($E_{200-300}$) in natural state and at saturation.

5.2.2. Eodometric test

In order to determine the compressibility of soils and their water sensitivity in the laboratory, according to NP 125/2010, can be used the edometer test. The edometric test can be simple or double (Figure 5.1).

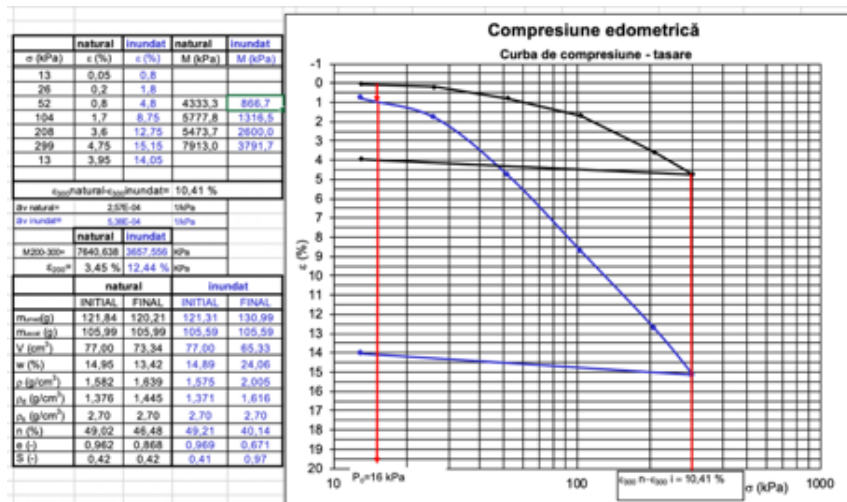


Figure 5.1: Double oedometer test performed during the research (personal archive)

The parameters obtained in the laboratory by edometric tests are:

- edometric deformation modulus ($E_{oed \ i-j}$) for different loading stages (σ) on samples in natural state and at saturation;
- specific additional settlement index for different load steps ($i_{m\sigma}$);
- additional settlement index i_{m300} for $\sigma = 300 \text{ kPa}$ (mandatory index for PSU identification);
- structural strength σ_0 .

The specific additional settlement index is the difference between the settlements obtained on the sample in natural state and the saturated sample under the same pressure by the double edometric test (STAS 8942 / 1-1989 and SREN 1997-2: 2008).

5.3. In situ tests

5.3.1. General methods

In-situ tests are faster and more economical than laboratory testing and provide a semi continuous profile. Also, they provide data in conditions of in situ temperature and without stress relief. Empirical correlations are used to determine soil properties and behaviour. Among the in situ tests that can be used to determine ground conditions is the static cone penetration test (CPT).

5.3.2. Cone penetration test (CPT)

The cone penetration test (CPT) is an in situ test that provides direct measurements that can be used by engineers to determine the properties of the soil based on empirical correlations. The main measurements include uncorrected tip resistance and lateral friction on the sleeve. Some cones can also be used to measure pore water pressure (CPTU).

CPT is being used more and more frequently for projects executed on large areas due to the short execution time (approximately 30 minutes for a depth of 10m) and the much better cost compared to geotechnical borings. CPT tests do not exclude the execution of geotechnical borings, but can considerably reduce their number. Advantages and limitations of the method are described by P.W. Mayne et. al, 2001.

The cone penetration test (CPT) is the most widely used in situ test. It was developed in Western Europe in the 1930s and 1950s. Two types of cone are commonly used, the mechanical cone and the electric cone.

CPT and CPTU have three main applications:

- To assess the geotechnical parameters
- To determine the subsurface strata
- To provide results for geotechnical design

CPT tests are also used to check ground improvement by comparing data from tests performed before and after the improvement. Thus, we can determine:

- The quality of the improvement (increasing the penetration resistance);
- The result of the improvement (estimation of geotechnical parameters by correlations);
- Depth of improvement.

Through empirical correlations, from the CPT/CPTU can be estimated different geotechnical parameters, among which the edometric deformation modulus (E). Even if by CPT tests are not taken soil samples, an approximate soil classification can be obtained based on them.

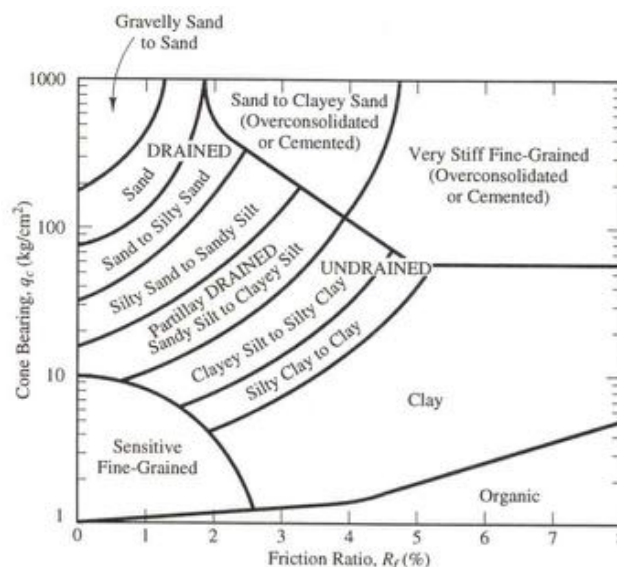


Figure 5.2: Soil classification based on CPT
(Robertson and Campanella, 1983)

CPT has been used internationally also for investigations in loess. Due to the complex nature of loess deposits experience and judgment for the interpretation of the test results are necessary.

6. RESEARCH FOR ESTABLISHING PSU VERIFICATION CRITERIA

6.1. Introduction

The author's research related to PSU deposits is based on the works performed at the Constanta Bypass project in Dobrogea, an area characterized by the presence of PSU as a foundation ground. The project during the execution of which the author participated directly had as object the construction of approximately 22 km of highway with a width of 26m, 26 bridges and 5 interchanges. Preliminary geotechnical investigations performed revealed the existence of a layer up to 12m thick of loess soils, generally having water sensitivity on the first 6m, placed

on a layer of red clays with calcareous concretions. After depth of about 28 m it is found up to 40 m deep the bedrock consisting of degraded limestone in clay mass. Groundwater was found at depths of over 9 - 12m. The highway starts from Ovidiu to the northern part of the Danube - Black Sea Canal (Figure 6.1.)

Due to the nature of the first soil layer and the project requirements, it was necessary to improve the ground to a depth of 6m. This was done by dynamic compaction (DC).

The purpose for applying DC for this project was:

- to reduce the collapse potential of the loess;
- to reduce differential settlements;
- to improve the bearing capacity of the soil to support the embankments.



Figure 6.1: Project location (Google Maps)

Dynamic compaction was performed on an area of approximately 240,000 m². Was performed an experimental polygon in order to establish the number of required phases, the number of blows for a point, the distance between them, etc. CD was made in three phases on a grid with a network of 7m.

In order to verify the efficiency of the ground improvement by dynamic compaction and to establish improvement criteria and correlations of the physical and mechanical parameters, an extensive testing program was executed. The author was involved during the entire execution period. This program included in situ and laboratory tests performed both before and after ground improvement, using an original methodology.

6.2. Testing program

The testing program includes the performance and analysis of 95 CPTs and 8 geotechnical borings in 4 areas of the project covering a total area of 94,200m². Were analysed a total of 68 undisturbed and 68 disturbed samples.

For each analysed area, several static cone penetration tests (CPT) and laboratory tests were performed as well as disturbed and undisturbed samples were taken (both before and after ground improvement).

The main purpose of the ground improvement works was to eliminate the water sensitivity. In this regard, based on the provisions of NP 125 for PSU characterization, have been established criteria for the verification of ground improved by DC:

- a) Based on laboratory tests:
 - Decreasing the additional settlement index $i_{m300} < 2 \%$;
 - Increasing the dry density $\rho_d > 1,6 \text{ g/cm}^3$;
 - Reduction of the porosity $n < 40 \%$;
 - Determination of the densification $s\%$;
 - Increasing the odometer modulus $E_{oed200-300}$ at natural moisture (w) and saturation (w_{sat}).

The certification of the quality of the works, in addition to fulfilling the above mentioned requirements, implies the satisfaction of the following conditions:

- $i_{m300imb} / i_{m300nat} < 1$, cu $i_{m300imb} < 2 \%$;
- $\rho_{dimb} / \rho_{dnat} > 1$;
- $n_{imb} / n_{nat} < 1$;
- $s > 1\%$;
- $E_{200-300imb} / E_{200-300nat} > 1$;
- $E_{200-300imbwsat} / E_{200-300natwsat} > 1$.

- b) Based on in situ testa (CPT):
 - Increasing the cone tip resistance $q_{cmed} > 2 \text{ MPa}$

The certification of the quality of the works implies the satisfaction of the following condition:

- $q_f / q_i > 1$

Where:

q_i – initial cone tip resistance, before compaction

q_f – final initial cone tip resistance, after compaction

The data processing for the purpose of the research was performed according to an original methodology that included the following stages:

- For an overall characterization of the soil, were made comparisons of the average values of determined parameters to a depth of 6m;
- The composition and nature of the soil were analysed using the methodology based on “footprint and similarity criteria” (NP 126/2010). With this methodology, the soil was analysed in each area before and after the improvement, as well as between treated areas;

- In order to analyse the variation of the cone resistance along the area, were calculated the median and the coefficient of variation (CoV) for results obtained in both natural and modified soil. The coefficient of variation represents the ratio of the standard deviation on the median;
- By establishing correlations based on geotechnical tests between the values determined in the field and those determined in the laboratory, the aim was to optimize the certification of eliminating water sensitivity of PSU.

In this regard were determined and established the following:

- Specific values of the correlation coefficient α of the in situ tests (CPT) with the laboratory tests (E_{oed}) for PSU, $\alpha = E_{oed}/q_c$;
- Correlations between cone tip resistance (q_c) resulting from static cone penetration tests (CPT) and densification (s) resulting from laboratory tests;
- Correlations between the volume reduction on the sensitive layer PSU with a thickness of 6m $\Delta H/H$ and q_c measured in situ, and the densification (s) resulting from the laboratory.

6.2.1. Establishing specific values of the coefficient α for PSU

According to SREN 1997-2 / 2007, in order to establish the edometric deformation modulus used in settlement calculation, can be used the relation:

$$E_{oed} = \alpha * q_c$$

Where:

q_c – cone tip resistance (from CPT)

α – coefficient to be determined based on local experience

Table 6.1 presents orientative values of α from SREN 1997-2 / 2007. It is noted that there is a very wide range of values for the choice of α , even for soils with similar composition. Starting from this relationship, the aim was to establish specific α values for PSU in the studied area.

Pământ	q_c	α
Argilă de plasticitate redusă	$q_c \leq 0,7 \text{ MPa}$ $0,7 < q_c < 2 \text{ MPa}$ $q_c \geq 2 \text{ MPa}$	$3 < \alpha < 8$ $2 < \alpha < 5$ $1 < \alpha < 2,5$
Praf de plasticitate redusă	$q_c < 2 \text{ MPa}$ $q_c \geq 2 \text{ MPa}$	$3 < \alpha < 6$ $1 < \alpha < 2$
Argilă foarte plastică Praf foarte plastic	$q_c < 2 \text{ MPa}$ $q_c > 2 \text{ MPa}$	$2 < \alpha < 6$ $1 < \alpha < 2$
Praf foarte organic	$q_c < 1,2 \text{ MPa}$	$2 < \alpha < 8$
Turbă și argilă foarte organică	$q_c < 0,7 \text{ MPa}$ $50 < w \leq 100$ $100 < w \leq 200$ $w > 300$	$1,5 < \alpha < 4$ $1 < \alpha < 1,5$ $\alpha < 0,4$
Crete:	$2 < q_c \leq 3 \text{ MPa}$ $q_c > 3 \text{ MPa}$	$2 < \alpha < 4$ $1,5 < \alpha < 3$
Nisipuri:	$q_c \leq 5 \text{ MPa}$ $q_c > 10 \text{ MPa}$	$\alpha = 2$ $\alpha = 1,5$

Tabel 6.1: Orientative values for α (SREN 1997-2/2007)

In the laboratory, were performed double edometric tests on samples taken before and after ground improvement in order to determine the $E_{oed200-300}$ edometric modules in natural moisture and at saturation. In situ, were performed CPTs before and after the ground improvement, obtaining the cone tip resistance (q_c).

Furthermore, was determined the coefficient α for each layer before and after the ground improvement. An average value was established both for the natural and improved. Also, were analysed the parameters that influence coefficient (α) part of the equation.

6.2.2. Correlations between cone tip resistance (q_c) and soil densification (s)

Densification is the quantification of the pore index reduction through different soil improvement processes:

$$s = \frac{e_i - e_f}{1 + e_i} (\%)$$

Where:

e_i – initial pore index, in natural state;

e_f – final pore index, after ground improvement

The variation of the in situ test results (q_c) with the variation of the laboratory test results (s) for the PSU layers was analysed as well as the variation of densification in relation to the i_{m300} additional settlement index and structural strength.

6.2.3. Correlations between cone tip resistance (q_c) and ground settlement

During the execution of the dynamic compaction (DC), were monitored the enforced ground settlement after each compaction phase and the total enforced settlement. Based on the total settlement one can calculate the reduction of the equivalent volume $\Delta H/H$ of the ground. Where, ΔH is the settlement induced by DC and H is the depth to which the settlement is related, in this case 6m. The variation of the cone resistance ratio with the average volume reduction for each area was analyzed. Also, the volume reduction measured in situ was compared with the densification obtained in the laboratory.

6.3. Obtained results

According to the methodology presented above, were performed and analysed tests in 4 zones of the project. Covering an area of approximately 94,200m².

6.3.1. Zone 1

Zone 1 has a length of 440m. The compacted area was 35,232m². In this area, were executed two boreholes, with disturbed and undisturbed samples, one on the natural ground and one after improvement by dynamic compaction (DC). In addition were performed 13 CPTs in the natural soil and 14 in the improved soil.

Based on the footprint (annex) the materials in zone 1 are generally characterized by an average relative area (A_r) of 2.5. According to the diagram used for soils classification based on CPT results, the soil is predominantly classified as low moisture clayey silt and sandy silt, in accordance with the classification of NP 125 - 2010, also showing values ranging from sensitive soil with small particles and clay to silty sand.

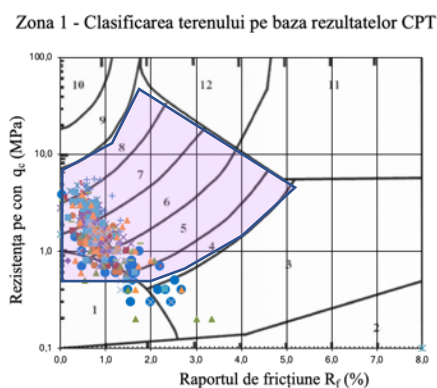


Figure 6.2: Soil classification based on CPT, Zone 1

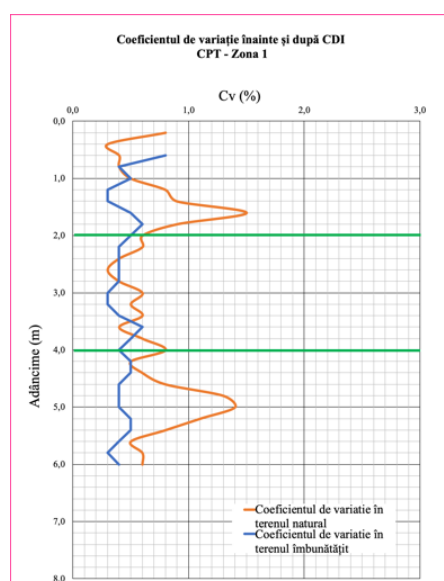


Figure 6.3: Graphical representation of the coefficient of variation (Cov) of the results obtained by CPT, Zone 1

In order to analyse the variation of the resistance on the cone along the area, the median and the coefficient of variation (Cov) were calculated for the results obtained in the natural soil and those obtained after improvement. The coefficient of variation represents the ratio of the standard deviation on the median. The coefficient of variation $Cov \leq 75\%$ indicates a relatively homogeneous series. Thus, for zone 1 it indicates relatively uniform conditions after compaction. Before compaction it shows an increase of variability in the natural soil at depths of 1,5m and 5m. It was possible to process the data with increased attention at those depths.

Based on the analysis of the data obtained by processing laboratory samples, it is found that up to a depth of 6m the soil can be classified as sensitive to moisture according to the identification criteria provided in the technical requirements and detailed in previous chapters. All results are represented in the thesis graph in parallel (before and after improvement).

ZONA 1							
Adâncime (m)	1N - în teren natural						
	i_{m300} (%)	I_c	Medie e	Medie ρ_d (g/cm ³)	Medie n (%)	$M_{200-300}$ natural (KPa)	$M_{200-300}$ saturat (KPa)
0,60			0,55	1,75	35,69		
1,20		0,75	0,55	1,75	35,63		
1,80			0,50	1,78	33,33		
2,40	4,83	0,74	0,90	1,41	47,30	4.830,02	4.081,60
3,00			0,57	1,73	36,34		
3,60	3,44	0,88	0,84	1,46	45,57	8.563,90	4.614,80
4,20			0,53	1,78	34,73		
4,80	4,46	1,00	0,83	1,46	45,15	5.342,10	4.251,10
5,40			0,54	1,73	35,18		
6,00	4,17	0,61	0,86	1,44	45,97	5.054,40	3.364,00
MEDIE	4,23	0,80	0,67	1,63	39,49	5.947,61	4.077,88

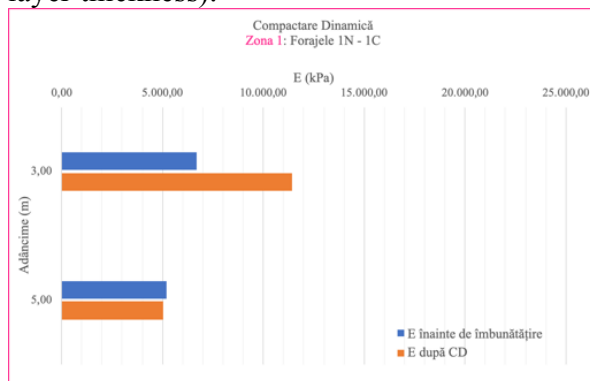
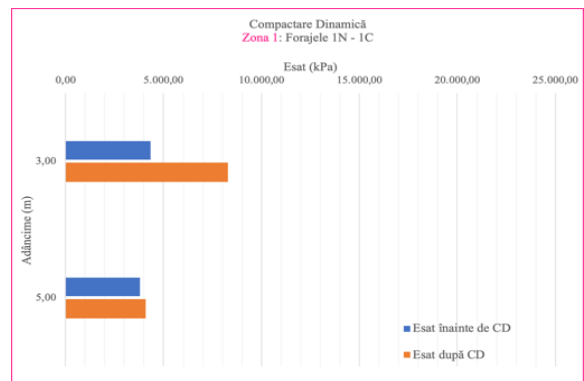
Table 6.1: Parameters obtain in laboratory for natural soil, Zone 1

ZONA 1							
Adâncime (m)	1C - în teren compactat						
	i_{m300} (%)	I_c	Medie e	Medie ρ_d (g/cm ³)	Medie n (%)	$M_{200-300}$ natural (KPa)	$M_{200-300}$ saturat (KPa)
0,60							
1,20		0,87	0,58	1,72	36,77		
1,80			0,57	1,73	36,37		
2,40	0,00	0,83	0,58	1,69	36,85	14.397,00	11.424,00
3,00			0,58	1,72	36,63		
3,60	2,00	0,86	0,71	1,57	41,29	8.463,00	5.129,00
4,20			0,53	1,78	34,69		
4,80	3,23	0,86	0,79	1,50	43,90	5.857,20	4.082,00
5,40			0,64	1,63	39,04		
6,00	0,00	0,76	0,85	1,45	45,76	4.189,50	4.058,10
MEDIE	1,31	0,84	0,65	1,64	39,03	8.226,68	6.173,28

Table 6.2: Parameters obtain in laboratory for compacted soil, Zone 1

Based on the average values, it is observed that the conditions for attesting the quality of the works is satisfied, according to the requirements of chapter 6.2, even if locally (4.8 m) there are values that do not satisfy all the requirements.

The edometric modules increased considerably after compaction on the first 4m, reaching a percentage of improvement of 70% for the edometric modules determined in the soil at natural humidity and at 90% for those determined in saturated soil. This aspect highlights the stronger effect of DC in the surface and median area of the improved layer (approximately 2/3 of the layer thickness).

Figure 6.4: Graphical representation of oedometric modulus $E_{200-300}$, Zone 1Figure 6.5: Graphical representation of saturated oedometric modulus $E_{200-300}$, Zone 1

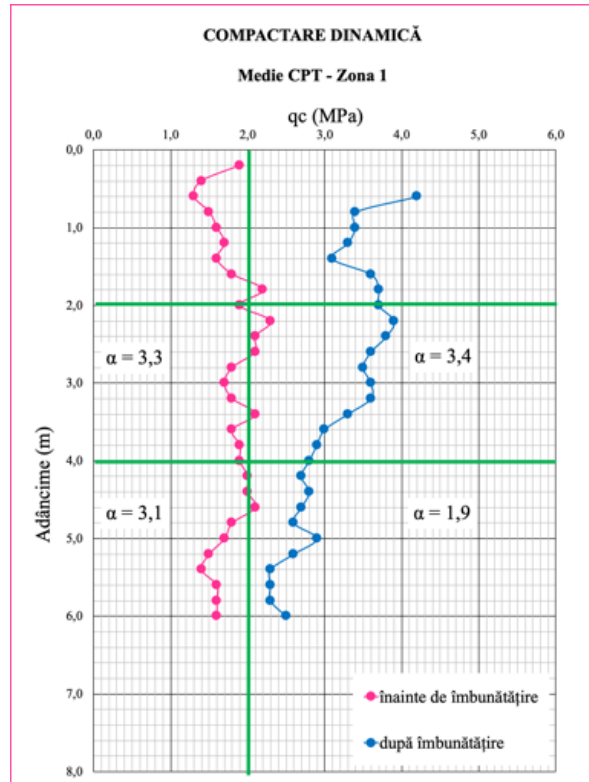


Figure 6.6: Graphical representation of CPT results, Zone 1

From the cone penetration tests (CPT) results the following average values for the tip cone resistance q_c :

Depth (m)	Average q_c natural (MPa)	Average q_c DC (MPa)	% Improvement DC
0 - 2	1,7	3,6	112%
2 - 4	2,0	3,4	70%
4 - 6	1,7	2,6	53%
Average	1,8	3,2	78%

Table 6.3: Average values of q_c (MPa), Zone 1

Based on the edometric modules determined in the laboratory and the cone resistance determined in situ, were calculated the values of α :

Depth (m)	α_{natural}	A_{DC}
2 - 4	3,3	3,4
4 - 6	3,1	1,9
Average	3,2	2,7

Table 6.4: Average values of α , Zone 1

For this area it is observed that the α values do not change significantly in depth nor before and after improvement. There was a decrease in α between 4 and 6 meters depth, after compaction, where although q_c increased $E_{200-300}$ did not have an obvious modification.

It is considered that α for the respective area is quasi-constant. Thus, $\alpha = 3$ was chosen both for the natural and improved soil. Below are the values of the edometric modules calculated with $\alpha = 3$ and the values q_c obtained in the natural and in the improved soil.

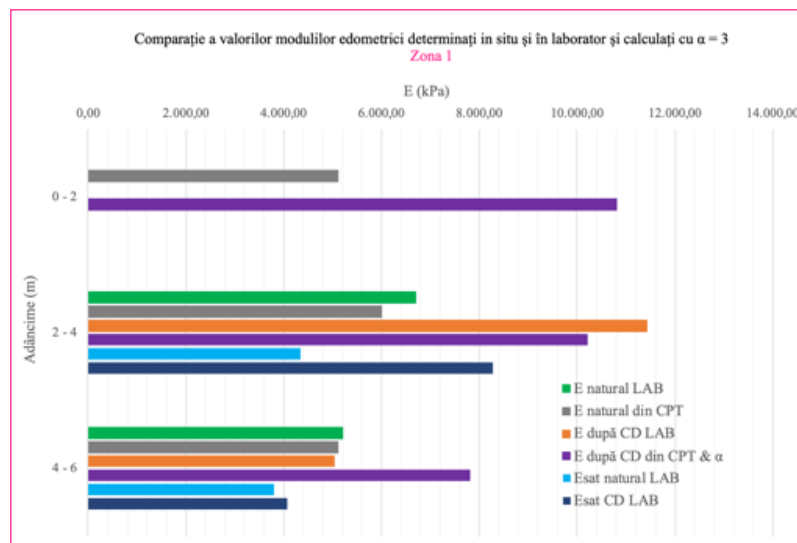


Figure 6.7: Comparison of oedometric modules determined in laboratory and the ones calculated with $\alpha = 3$, Zone 1

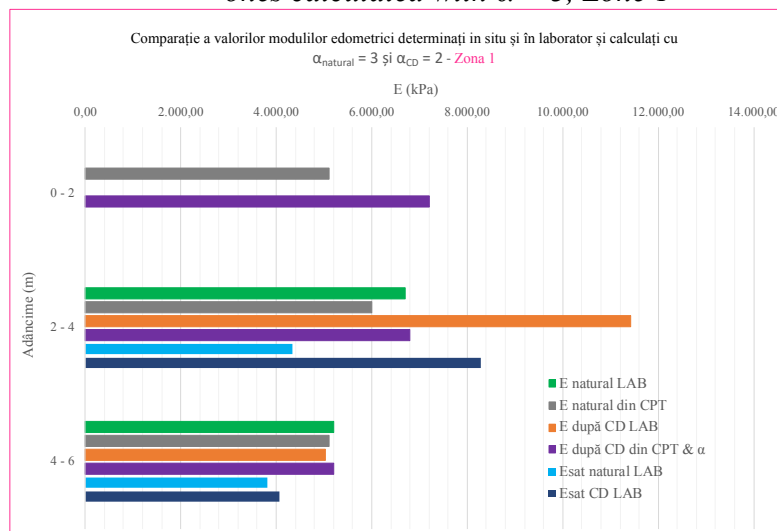


Figura 6.1: Comparison of oedometric modules determined in laboratory and the ones calculated with $\alpha_{\text{natural}} = 3$ și $\alpha_{\text{CD}} = 2$, Zone 1

As there is a decrease of α below 4m after improvement, the edometric modules were also calculated with different values of α , the average values obtained for the natural and for the improved soil are considered $\alpha = 3$ and $\alpha = 2$. From the comparison of these measured and calculated data it is found that for zone 1 it is possible to choose the same value α for the natural land and for the improved soil, respectively $\alpha = 3$.

Based on the pore index (e), was calculated the densification (s) obtained by improvement. For the first 6m resulted an average densification of 1,2%.

The following table, comparatively and punctually, presents the densification, the ratios of additional settlement index and the ratios of the afferent cone tip resistance.

Depth (m)	s (%)	i_{m300f}/i_{m300i}	q_{imed} (MPa)	q_{fmed} (MPa)	q_{fmed}/q_{imed}
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2,40	16,8	0,0	2,1	3,8	1,8
3,60	7,1	0,6	1,8	3,0	1,7
4,80	2,2	0,7	1,8	2,6	1,4
6,00	0,5	0,0	1,6	2,5	1,6

Table 6.5: The values of the parameters obtained on the PSU layers and the average q_c related to them, Zone 1

The variation of the in situ test with the variation of the laboratory test for the PSU layers was analysed by comparing the average values of the parameters resulting from the laboratory with the results of the CPTs performed in this area.

It was observed that where q_f/q_i ratio is higher and i_{m300f}/i_{m300i} ratio is lower the densification is higher. Also, where q_{fmed} is smaller than 3 the densification is smaller.

During the execution of the DC, is monitored the enforced ground settlement after each compaction phase and the total compaction. The average total settlement (ΔH) is 71 cm. If we report the total settlement (ΔH) at a depth (H) of 6m, results a volume reduction $\Delta H/H$ of 11,8%.

The average values of the parameters resulting from the laboratory were correlated with the results of the in situ measurements. Respectively the ratio of cone strength and densification resulting from laboratory tests to the ratio of cone strength and densification resulting from in-situ measurements.

It was observed that the results obtained in the laboratory and those obtained in situ do not correlate, thus it was necessary to repeat the geotechnical boring.

6.3.2. Zone 2

Zone 2 has a length of 510m. The compacted area measured 30.176m². In this area, were executed two boreholes, with disturbed and undisturbed samples, one on the natural ground and one after improvement by dynamic compaction (DC). In addition were performed 21 CPTs in the natural soil and 21 in the improved soil.

Based on the footprint (annex) the materials in zone 2 are generally characterized by an average relative area (A_r) of 2,4. According to the diagram used for soils classification based on CPT results, the soil is predominantly classified as low moisture clayey silt and sandy silt, in accordance with the classification of NP 125 - 2010, also showing values ranging from sensitive soil with small particles and clay to silty sand.

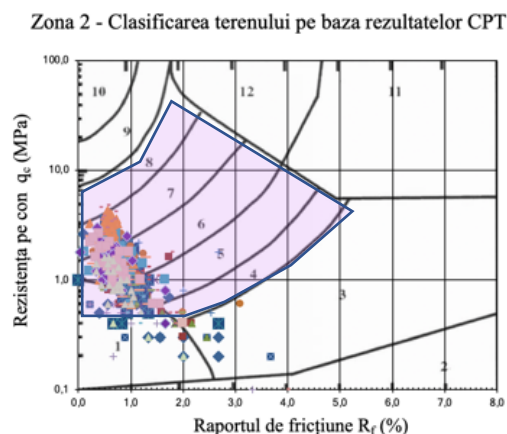


Figure 6.8: Soil classification based on CPT, Zone 2

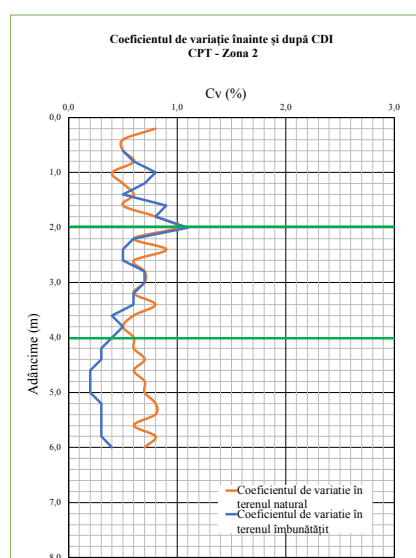


Figure 6.9: Graphical representation of the coefficient of variation (Cov) of the results obtained by CPT, Zone 2

The coefficient of variation Cov indicates a relatively homogeneous series, with an increase of variability de 2m. It was possible to process the data with increased attention at those depths.

ZONA 2							
Adâncime (m)	SPT 2N - în teren natural						
	i_{m300} (%)	I_c	Medie e	Medie ρ_d (g/cm ³)	Medie n (%)	$M_{200-300}$ natural (KPa)	$M_{200-300}$ saturat (KPa)
0,60			0,67	1,63	39,96		
1,50		0,78	0,60	1,71	37,31		
1,80			0,52	1,76	34,20		
2,40	4,72	0,78	0,85	1,45	46,04	7.848,20	3.455,50
3,00			0,65	1,65	39,45		
3,60	2,51	0,78	0,87	1,47	45,84	9.753,50	5.008,30
4,20			0,63	1,67	38,51		
4,80	1,76	0,94	0,61	1,67	37,72	14.548,00	7.035,20
5,40			0,53	1,74	34,78		
6,00	0,50	0,97	0,72	1,56	41,72	12.381,00	9.827,20
MEDIE 6m	2,37	0,85	0,67	1,63	39,55	11.132,68	6.331,55

Table 6.6: Parameters obtain in laboratory for natural soil, Zone 2

ZONA 2							
Adâncime (m)	SPT 2C - în teren compactat						
	i_{m300} (%)	I_c	Medie e	Medie ρ_d (g/cm ³)	Medie n (%)	$M_{200-300}$ natural (KPa)	$M_{200-300}$ saturat (KPa)
0,60			0,48	1,84	32,51		
1,50			0,61	1,69	37,73		
1,80	0,10	0,81	0,52	1,79	34,30	21.462,00	12.683,00
2,40	0,00	0,97	0,49	1,82	32,99	12.440,00	10.769,00
3,00			0,45	1,87	31,12		
3,60	0,00	0,88	0,62	1,68	38,24	15.993,00	11.340,00
4,20	0,00	0,83	0,67	1,63	40,12	12.621,00	14.548,00
4,80	0,00	0,77	0,58	1,72	36,69	17.170,00	13.482,00
5,40			0,51	1,77	33,56		
6,00	2,45	1,00	0,64	1,66	39,06	15.895,00	9.717,00
MEDIE 6m	0,43	0,88	0,56	1,75	35,63	15.930,17	12.089,83

Table 6.7: Parameters obtain in laboratory for compacted soil, Zone 1

Based on the analysis of the data obtained by processing laboratory samples, it is found that to a depth of 6m the soil can be classified as sensitive to moisture according to the identification criteria provided in the technical requirements and detailed in previous chapters. All results are represented in the thesis graph in parallel (before and after improvement).

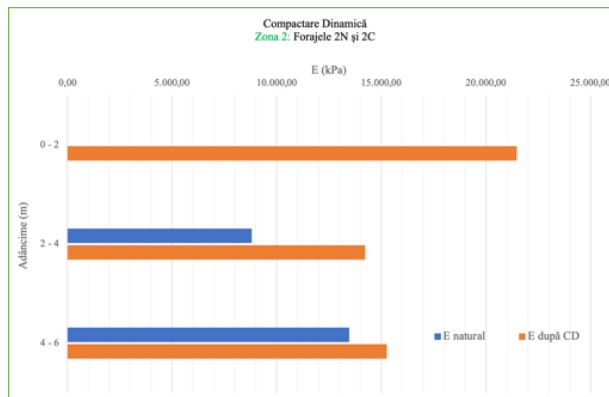


Figure 6.10: Graphical representation of oedometric modulus $E_{200-300}$, Zone 2

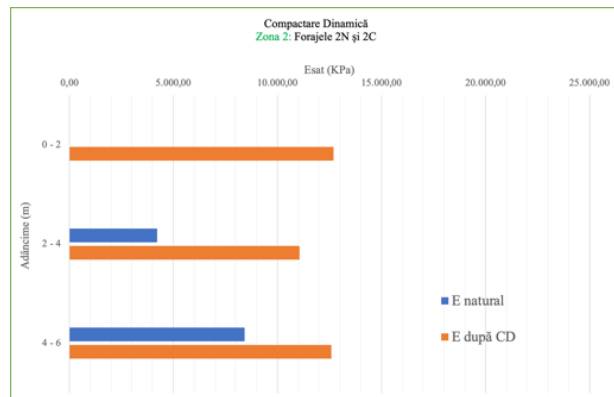


Figure 6.11: Graphical representation of saturated oedometric modulus $E_{200-300}$, Zone 2

The edometric modules increased considerably after compaction on the first 4m, reaching a percentage of improvement of 60% for the edometric modules determined in the soil at natural humidity and at 160% for those determined in saturated soil. This aspect highlights the stronger effect of DC in the surface and median area of the improved layer (approximately 2/3 of the layer thickness).

From the cone penetration tests (CPT) results the following average values for the tip cone resistance q_c :

Depth (m)	Average q_c natural	Average q_c CD/i	% Improvement
0 - 2	1,1	3,9	255%
2 - 4	1,4	3,4	143%
4 - 6	1,7	2,8	65%
Average	1,4	3,4	143%

Table 6.8: Average values of q_c (MPa), Zone 2

Based on the edometric modules determined in the laboratory and the cone resistance determined in situ, were calculated the values of α :

Depth (m)	α_{natural}	A_{DC}
0 - 2	-	5,5
2 - 4	6,3	4,2
4 - 6	7,9	5,4
Average	7,1	5,0

Tabel 6.2: Valori calculate ale lui α , Zona 2

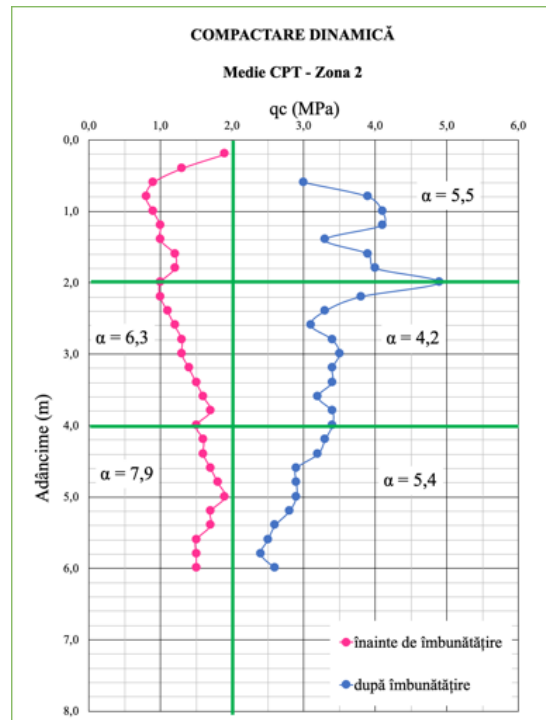


Figure 6.12: Graphical representation of CPT results, Zone 2

For this area it is observed that the α values decrease with about 30 % after improvement. It varies in depth, but does not change significantly. Thus, $\alpha = 7$ is considered for the natural soil and $\alpha = 5$ for the improved soil.

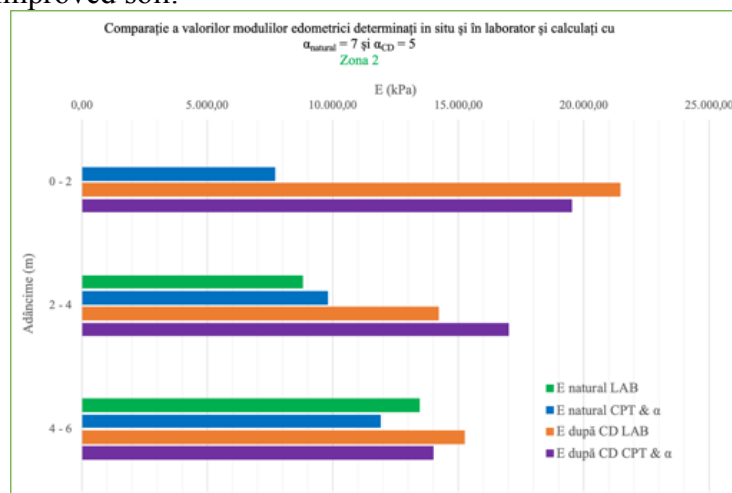


Figure 6.2: Comparison of oedometric modules determined in laboratory and the ones calculated with $\alpha_{\text{natural}} = 7$ and $\alpha_{\text{DC}} = 5$, Zone 1

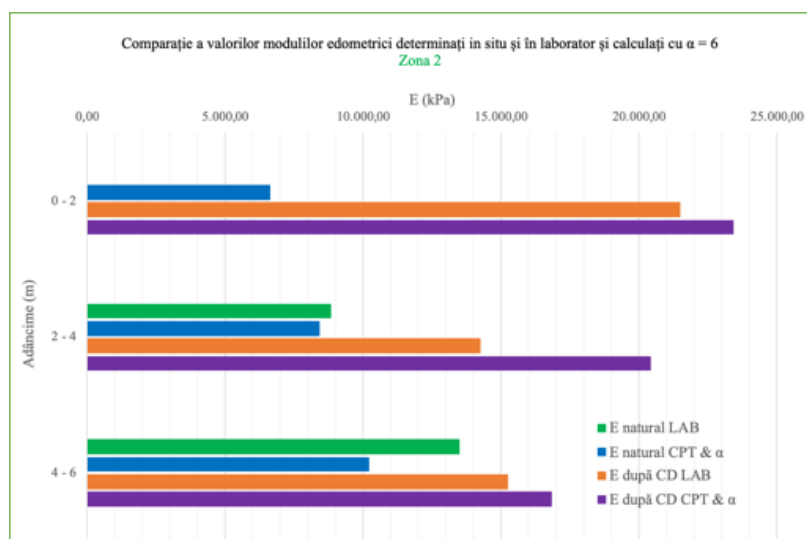


Figure 6.13: Comparison of oedometric modules determined in laboratory and the ones calculated with $\alpha = 6$, Zone 2

From the comparison of these measured and calculated data it is observed that the values of the oedometric modules calculated with the same α value are significantly different from those obtained in the laboratory. Thus, for this area is considered an α value for the natural soil and another value for the improved one (7 and 5 respectively).

The following table, comparatively and punctually, presents the densification, the ratios of additional settlement index and the ratios of the afferent cone tip resistance. Based on the pore index (e), was calculated the densification (s) obtained by improvement. For the first 6m resulted an average densification of 6,6%.

Adâncime (m)	s (%)	i_{m300f}/i_{m300i}	q_{imed} (MPa)	q_{fmed} (MPa)	q_{fmed}/q_{imed}
2,40	19,5	0,0	1,1	3,3	3,0
3,60	13,4	0,0	1,6	3,2	2,0
4,80	1,9	0,0	1,8	2,9	1,6

Table 6.9: The values of the parameters obtained on the PSU layers and the corresponding average q_c , Zone 2

The variation of the in situ test with the variation of the laboratory test for the PSU layers was analysed by comparing the average values of the parameters resulting from the laboratory with the results of the CPTs performed in this area. It was observed that where q_f/q_i ratio is higher and i_{m300f}/i_{m300i} ratio is lower the densification is higher. Also, where q_{fmed} is smaller than 3 the densification is smaller.

The average total settlement (ΔH) is 76 cm. If we report the total settlement (ΔH) at a depth (H) of 6m, results a volume reduction $\Delta H/H$ of 12,7%.

The average values of the parameters resulting from the laboratory were correlated with the results of the in situ measurements. Respectively the ratio of cone strength and densification resulting from laboratory tests to the ratio of cone strength and densification resulting from in-situ measurements.

6.3.3. Zone 3

Zone 3 has a length of 450m. The compacted area measured 22.500m². In this area, were executed two boreholes, with disturbed and undisturbed samples, one on the natural ground and one after improvement by dynamic compaction (DC). In addition were performed 10 CPTs in the natural soil and 10 in the improved soil.

Based on the footprint (annex) the materials in zone 1 are generally characterized by an average relative area (A_r) of 2,5. According to the diagram used for soils classification based on CPT results, the soil is predominantly classified as low moisture clayey silt and sandy silt, in accordance with the classification of NP 125 - 2010, also showing values ranging from sensitive soil with small particles and clay to silty sand.

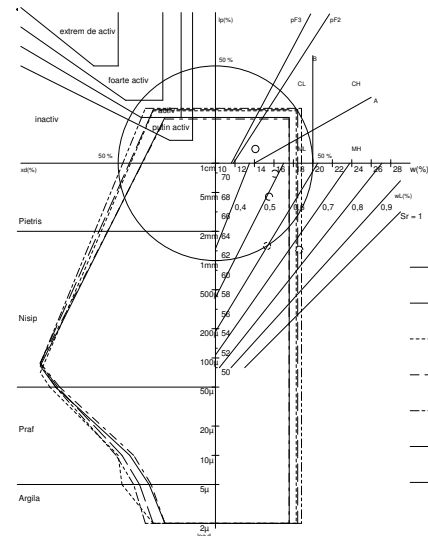
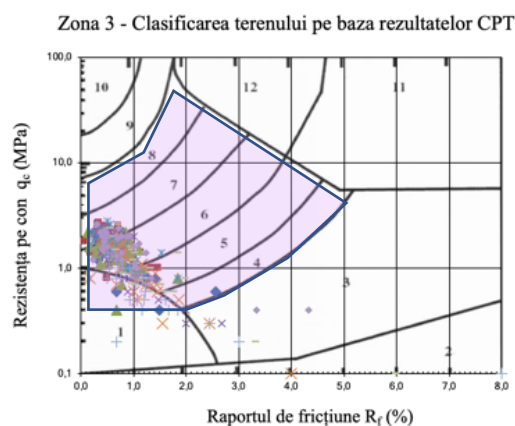


Figure 6.14: Soil classification based on CPT, Zone 3

The Cov coefficient indicates relatively uniform soil conditions, with an increase in variability at the soil surface after compaction. Correlated data processing was possible.

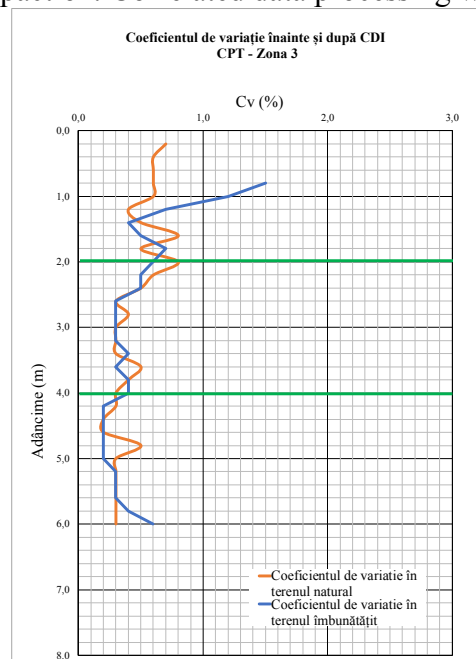


Figure 6.15: Graphical representation of the coefficient of variation (Cov) of the results obtained by CPT, Zone 3

Zona 3							
Adâncime (m)	F13 - în teren natural						
	i_{m300} (%)	I_c	Medie e	Medie ρ_d (g/cm ³)	Medie n (%)	$M_{200-300}$ natural (KPa)	$M_{200-300}$ saturat (KPa)
0,60	10,41	1,00	0,92	1,41	47,91	7.640,60	3.657,60
1,20	9,07	0,97	0,73	1,56	41,81	14.308,00	3.080,10
1,80	8,12	0,97	0,64	1,64	39,08	13.840,00	3.330,90
2,40	7,62	0,96	0,81	1,48	44,59	8.441,60	3.437,90
3,00	2,71	0,91	0,68	1,62	40,46	14.308,00	6.502,30
3,60	2,41	0,97	0,66	1,64	39,65	14.881,00	7.596,00
4,20	6,22	0,94	0,79	1,52	43,98	8.280,30	3.556,10
4,80	8,67	1,00	0,88	1,44	46,58	7.073,50	2.987,00
5,40	4,87	0,94	0,87	1,46	46,44	6.403,90	2.594,40
6,00	2,00	1,00	0,58	1,70	36,79	12.871,00	7.210,80
Medie	6,21	0,97	0,76	1,55	42,73	10.804,79	4.395,31

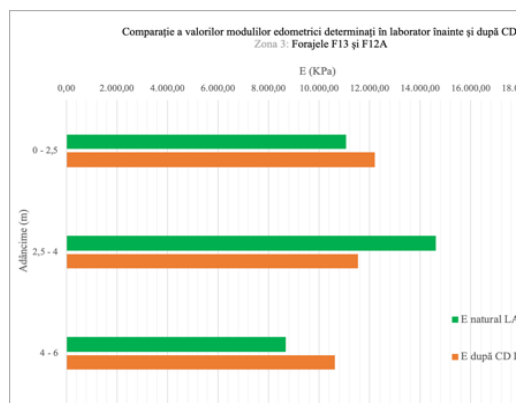
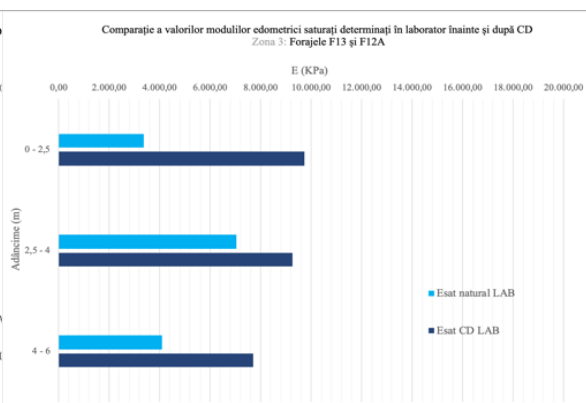
Table 6.10: Parameters obtain in laboratory for natural soil, Zone 3

Zona 3							
Adâncime (m)	F12A - în teren compactat						
	i_{m300} (%)	I_c	Medie e	Medie ρ_d (g/cm ³)	Medie n (%)	$M_{200-300}$ natural (KPa)	$M_{200-300}$ saturat (KPa)
0,60							
1,20							
1,80		0,95	0,61	1,67	37,99	10.957,00	
2,40	0,00	0,98	0,56	1,74	35,84	13.409,00	9.723,30
3,00	0,45	0,79	0,61	1,67	38,06	11.082,70	8.458,00
3,60	0,25	0,80	0,56	1,72	36,07	11.954,00	10.081,00
4,20	0,43	0,76	0,70	1,59	41,00	10.501,00	8.354,80
4,80	0,00	0,76	0,71	1,57	41,54	10.228,00	9.252,70
5,40	1,24	0,83	0,68	1,60	40,35	11.116,00	7.415,90
6,00	2,00	0,86	0,65	1,62	39,48	10.952,00	5.816,60
Medie	0,62	0,84	0,64	1,65	38,79	11.274,96	8.443,19

Table 6.11: Parameters obtain in laboratory for compacted soil, Zone 3

Based on the analysis of the data obtained by processing laboratory samples, it is found that up to a depth of 6m the soil can be classified as sensitive to moisture according to the identification criteria provided in the technical requirements and detailed in previous chapters. All results are represented in the thesis graph in parallel (before and after improvement).

The edometric modules corresponding to the soil at natural humidity do not show significant changes after compaction. At saturation, the modules increase considerably after compaction.

Figure 6.16: Graphical representation of oedometric modulus $E_{200-300}$, Zone 3Figure 6.17: Graphical representation of saturated oedometric modulus $E_{200-300}$, Zone 3

From the cone penetration tests (CPT) results the following average values for the tip cone resistance q_c :

Depth (m)	Average q_c natural	Average q_c DC	% Improvement
0 - 2,5	1,2	3,0	150%
2,5 - 4	1,6	3,5	119%
4 - 6	1,4	3,0	114%
Average	1,4	3,2	129%

Table 6.12: Average values of q_c (MPa), Zone 3

Based on the edometric modules determined in the laboratory and the cone resistance determined in situ, were calculated the values of α :

Depth (m)	α_{natural}	A_{DC}
0 - 2,5	9,2	4,1
2,5 - 4	9,1	3,3
4 - 6	6,2	3,5
Average	8,2	3,6

Table 6.13: Average values of α , Zone 1

For this area it is observed a decrease of α values of more than 50% for the improved soil compared to the value obtained for the natural soil. On depth α natural it changes significantly below the depth of 4m for the natural soil (where it has high compressibility). Based on the average, $\alpha = 8$ is considered for the natural soil and $\alpha = 4$ for the improved soil.

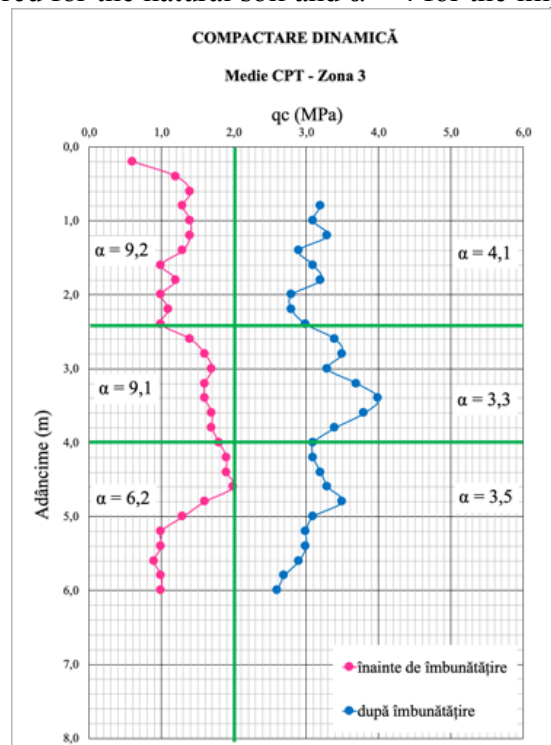


Figure 6.18: Graphical representation of CPT results, Zone 3

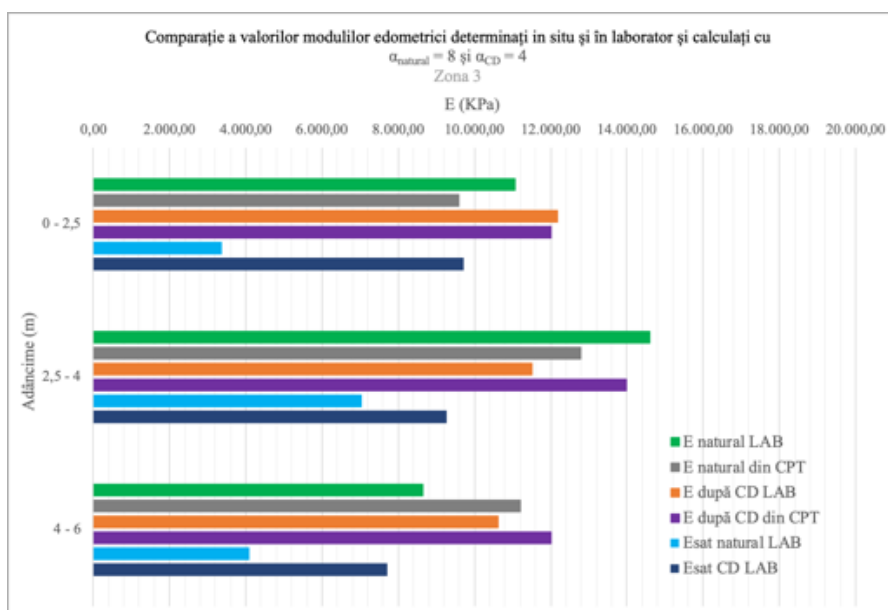


Figura 6.3: Comparison of oedometric modules determined in laboratory and the ones calculated with $\alpha_{\text{natural}} = 8$ and $\alpha_{\text{DC}} = 4$, Zone 3

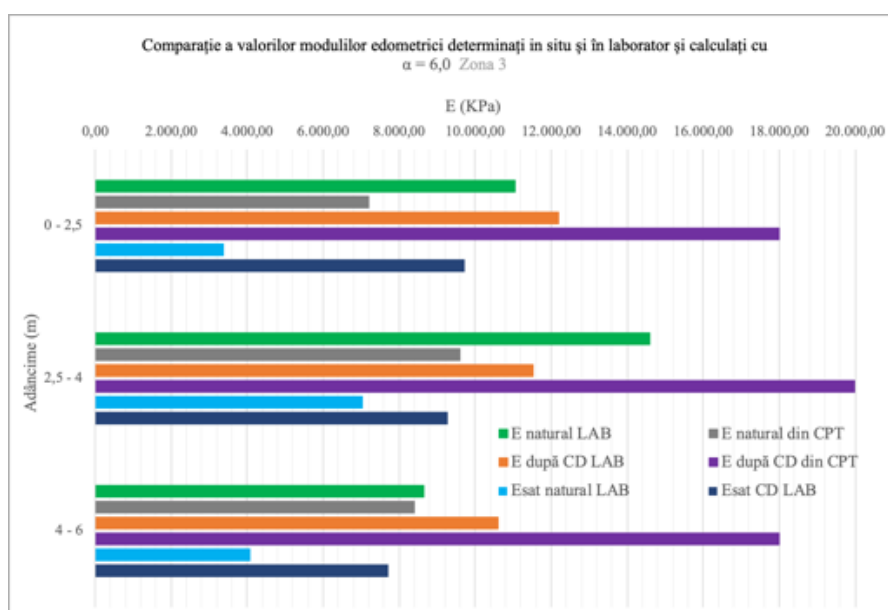


Figure 6.19: Comparison of oedometric modules determined in laboratory and the ones calculated with $\alpha = 6$, Zone 3

From the comparison of these measured and calculated data it is observed that the values of the oedometric modules calculated with the same value of α are significantly different from those obtained in the laboratory. The modules calculated with different values of α , despite the non-uniformity in depth, have values close to those determined in the laboratory. Thus, for this area is considered an α value for the natural soil and another value for the improved one (8 and 4 respectively).

The following table, comparatively and punctually, presents the densification, the ratios of additional settlement index and the ratios of the afferent cone tip resistance. Based on the pore index (e), was calculated the densification (s) obtained by improvement. For the first 6m resulted an average densification of 6,9%.

Depth (m)	s (%)	i_{m300f}/i_{m300i}	q_{imed} (MPa)	q_{fmed} (MPa)	q_{fmed}/q_{imed}
1,80	1,8	-	1,2	3,2	2,7
2,40	13,8	0,0	1,0	3,0	3,0
3,00	4,2	0,2	1,7	3,3	1,9
3,60	6,0	0,1	1,7	3,8	2,2
4,20	5,0	0,1	1,9	3,1	1,6
4,80	9,0	0,0	1,6	3,5	2,2
5,40	10,2	0,2	1,0	3,0	3,0
6,0	-4,4	0,0	1,0	2,6	2,6

Table 6.14: The values of the parameters obtained on the PSU layers and the corresponding average q_c , Zone 3

The variation of the in situ test with the variation of the laboratory test for the PSU layers was analysed by comparing the average values of the parameters resulting from the laboratory with the results of the CPTs performed in this area. It was observed that where q_f/q_i ratio is higher and i_{m300f}/i_{m300i} ratio is lower the densification is higher.

During the execution of the DC, is monitored the enforced ground settlement after each compaction phase and the total compaction. The average total settlement (ΔH) is 83 cm. If we report the total settlement (ΔH) at a depth (H) of 6m, results a volume reduction $\Delta H/H$ of 13,8%.

The average values of the parameters resulting from the laboratory were correlated with the results of the in situ measurements. Respectively the ratio of cone strength and densification resulting from laboratory tests to the ratio of cone strength and densification resulting from in-situ measurements.

6.3.4. Zone 4

Zone 4 has a length of 140m. The compacted area measured 6.300m². In this area, were executed two boreholes, with disturbed and undisturbed samples, one on the natural ground and one after improvement by dynamic compaction (DC). In addition were performed 3 CPTs in the natural soil and 3 in the improved soil.

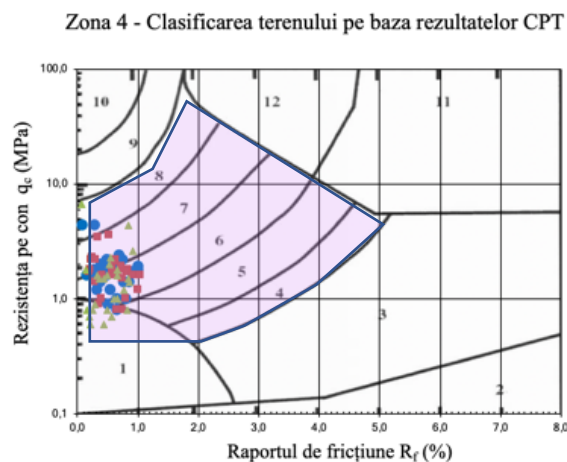


Figure 6.20: Soil classification based on CPT, Zone 4

Based on the footprint (annex) the materials in zone 1 are generally characterized by an average relative area (A_r) of 2.26. According to the diagram used for soils classification based on CPT results, the soil is predominantly classified as low moisture clayey silt and sandy silt, in accordance with the classification of NP 125 - 2010, also showing values ranging from sensitive soil with small particles and clay to silty sand.

ZONA 4						
Adâncime (m)	P4 - în teren natural					
	i_{m300} (%)	I_c	Medie e	Medie ρ_d (g/cm ³)	Medie n (%)	$M_{200-300}$ natural (KPa)
2,50	1,30	0,76	0,76	1,52	44,73	5.131,60
4,50	1,10	0,77	0,76	1,50	43,14	5.820,90
MEDIE	1,20	0,77	0,76	1,51	43,94	5.476,25

Table 6.15: Parameters obtain in laboratory for natural soil, Zone 4

ZONA 4						
Adâncime (m)	F1DC - în teren compactat					
	i_{m300} (%)	I_c	Medie e	Medie ρ_d (g/cm ³)	Medie n (%)	$M_{200-300}$ natural
2,50	0,05	0,86	0,52	1,79	34,19	11.470,60
4,50	0,00	0,80	0,53	1,74	34,76	10.833,30
MEDIE	0,03	0,83	0,53	1,77	34,48	11.151,95

Table 6.16: Parameters obtain in laboratory for compacted soil, Zone 4

The coefficient of variation Cov indicates a relatively homogeneous series, with an increase of variability de 6m. It was possible to process the data with increased attention at those depths.

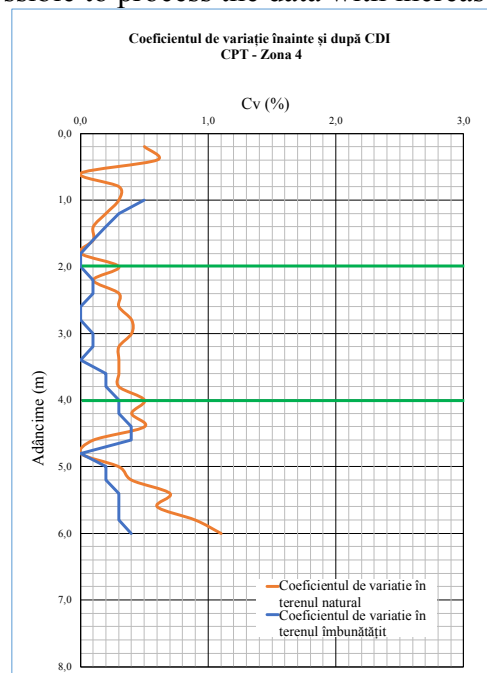


Figura 6.4: Reprezentarea grafică a coeficientului de variație (C_v) a rezultatelor obținute prin teste CPT, Zona 4

Based on the analysis of the data obtained by processing laboratory samples, it is found that up to a depth of 6m the soil can be classified as sensitive to moisture according to the identification criteria provided in the technical requirements and detailed in previous chapters. All results are represented in the thesis graph in parallel (before and after improvement).

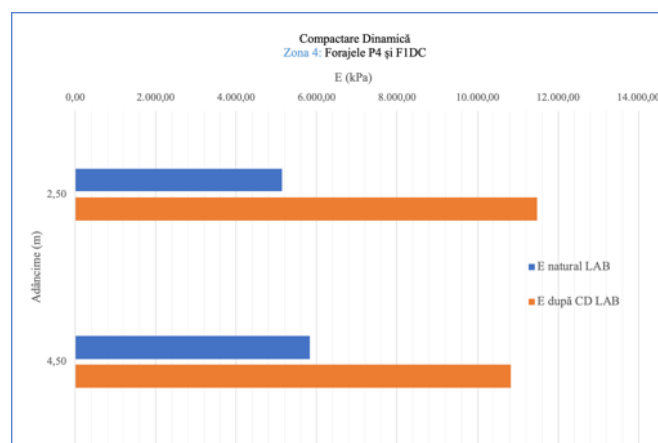


Figure 6.21: Graphical representation of oedometric modulus $E_{200-300}$, Zone 4

The edometric modules increased considerably after compaction reaching a percentage of improvement of 100%.

From the cone penetration tests (CPT) resulted an average value of the tip cone resistance q_c of 1,5MPa and after inmpovement of 3 MPa.

Based on the laboratory determined edometric modules and the cone resistance determined in situ, were calculated the values of coefficient α . Thus, resulted a α value of 3,7 both for the soil in its natural state and for the compacted soil.

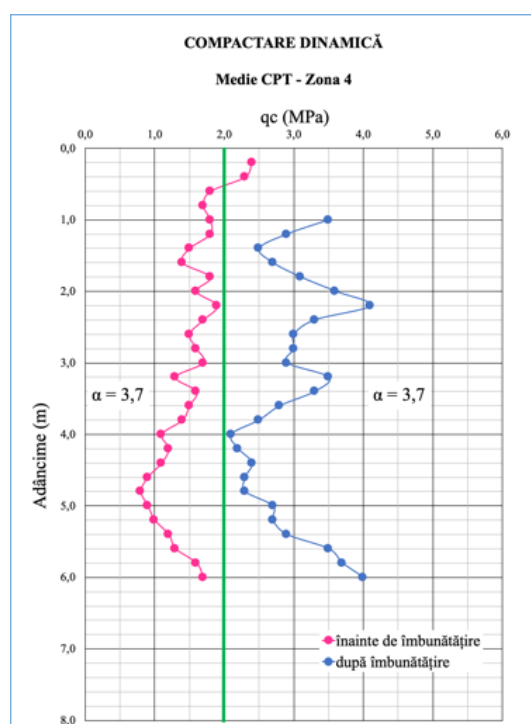


Figura 6.5: Rezentarea grafică a mediei rezultatelor obținute prin teste CPT, Zona 4

The edometric modules were calculated with the same value of α , respectively $\alpha = 4$ for both the natural state and the improved soil.

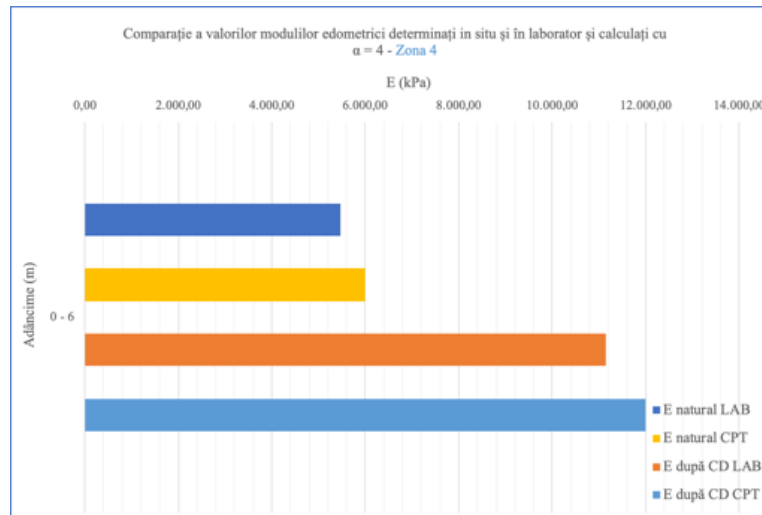


Figure 6.22: Comparison of oedometric modules determined in laboratory and the ones calculated with $\alpha = 4$, Zone 4

Î The following table, comparatively and punctually, presents the densification, the ratios of additional settlement index and the ratios of the afferent cone tip resistance. Based on the pore index (e), was calculated the densification (s) obtained by improvement. For the first 6m resulted an average densification of 13,3%.

Depth (m)	s (%)	i_{m300f}/i_{m300i}	q_{imed} (MPa)	q_{fmed} (MPa)	q_{fmed}/q_{imed}
2,5	13,6	0,0	1,6	3,2	2,0
4,5	13,1	0,0	1,0	2,3	2,3

Tabel 6.3: Valorile parametrilor obținuți pe straturile PSU, Zona 4

It was observed that where q_f/q_i is bigger than 2 the ratio i_{m300f}/i_{m300i} is 0.

The average total settlement (ΔH) is 104 cm. If we report the total settlement (ΔH) at a depth (H) of 6m, results a volume reduction $\Delta H/H$ of 17,3%.

The average values of the parameters resulting from the laboratory were correlated with the results of the in situ measurements. Respectively the ratio of cone strength and densification resulting from laboratory tests to the ratio of cone strength and densification resulting from in-situ measurements.

6.4. Conclusions

To characterize the variation of results for PSU, were made comparisons of the average values of the parameters resulting from the tests performed in all areas studied.

The coefficient α was determined for each area and for each layer, before and after the land improvement. Thus, an average value was established for the natural soil and an average value for the improved one. Also, were analysed the parameters that influence the coefficient α .

It is found that α for PSU in its natural state has values in a wide range, between 3 and 9, and for the land improved by CDI the range is reduced between 3 and 5.

The average cone tip resistance q_c obtained for the natural soil is 1,5 MPa, and after the improvement it doubles, reaching 3 MPa.

Due to the non-uniformity of the PSU, the coefficient α for the natural soil also varies in depth, not only between areas. For an overall analysis, were calculated for each area an average α for the natural soil and one for the improved one. As some deficiencies were observed in the conduct of laboratory tests in zone 1, the coefficient α for this zone was considered only the one obtained up to the depth of 4m.

AREA	q_{cimed} (MPa)	α_i	q_{cfmed} (MPa)	α_f	$q_{cfmed}/$ q_{cimed}
1	2,0	3	3,4	3	1,7
2	1,4	7	3,4	5	2,4
3	1,4	8	3,2	4	2,3
4	1,5	4	3	4	2

Table 6.17: Average values of the coefficient α for each area and related q_c

To certify the value of the average α coefficient considered for each zone, the E_{oed} edometric modules obtained in the laboratory were compared with those calculated based on the values of cone tip resistance and average α . In general, there are no considerable differences between the edometric modules obtained in the laboratory and those obtained with α medium for the natural terrain. Also, there are no considerable differences between the edometric modules obtained in the laboratory and those obtained with average α for the compacted soil. Except for the values obtained for zone 1 below 4 m depth caused by some deficiencies in the conduct of laboratory tests.

Furthermore, is observed a similarity of α and q_c values between zones 1 and 4 and zones 2 and 3, respectively.

Zones 1 and 4 are considered to be less relevant due to a smaller number of samples (zone 4) and deficiencies in the conduct of laboratory tests (zone 1). However, zones 1 and 4 are characterized as follows:

- They have high compressibility at natural humidity and, zone 1, very high compressibility at saturation (in zone 4 $E_{oed200-300}$ was not determined at saturation);
- i_{m300} specific additional settlement index is 4.2% for zone 1 and 1.2% for zone 4, although initially $n > 40\%$;
- The difference between the modules corresponding to the soil at natural humidity and those at saturation is 30% in area 1;
- The ratio of resistance per cone between natural and compacted soil is $q_{cf} / q_{ci} \leq 2$ (1.8 for zone 1 and 2 for zone 4);
- The cone resistance for the natural terrain is $q_c \geq 1.5$ ($q_c = 1.8$ MPa for zone 1 and $q_c = 1.5$ MPa for zone 4);
- These areas correspond to a small coefficient α (3 - 4) for the natural soil and remain similar for the compacted soil (3 - 4).

Zones 2 and 3 are characterized as follows:

- They have medium compressibility at natural humidity and very high compressibility (zone 3) and high (zone 2) at saturation

- i_{m300} specific additional settlement index is 2.4% for zone 2 and 6% for zone 3;
- The difference between the modules corresponding to the land at natural humidity and those at saturation being 40% in zone 2 and 60% in zone 3;
- The ratio of resistance per cone between natural and compacted land is $q_{cf} / q_{ci} \geq 2$ (2.4 for zone 2 and 2.3 for zone 3);
- The cone resistance for the natural terrain is $q_c \leq 1.5$ ($q_c = 1.4\text{MPa}$ for both areas);
- These areas correspond to a high coefficient α (7 - 8) for natural soil and decreases by 30 - 50% for compacted soil (4 - 5).

Summarizing these processed and systematized data, the following conclusions were reached:

- Depending on the ratio of the cone resistance obtained after DC (q_{cf}) and the cone resistance obtained in the natural soil (q_{ci}) the range of variation of the coefficient α for PSU can be characterized as follows:
 - for $q_{cf}/q_{ci} > 2$: $\alpha = 5 - 8$;
 - for $q_{cf}/q_{ci} \leq 2$: $\alpha = 3 - 4$.
- For low values of structural strength σ_0 and high values of the specific additional settlement index i_{m300} the coefficient α has high values. It can be considered that α has higher values the more sensitive the soil is to moisture.
- The cone tip resistance for PSU in the studied areas is approximately 1,5MPa. This q_c value corresponds to the coefficient $\alpha = 3 - 8$.
- After DC, there is a decrease of α of approximately 30 - 50% depending on the specific additional settlement index at i_{m300} and the compressibility of the soil in its natural state.
- The higher the i_{m300} , the more α decreases after improvement. If the initial compressibility is very high α has low values and remains at similar values after DC. Also, where the α values are small, the q_{cf} / q_{ci} ratio is lower.
- Following the improvement of the soil by dynamic compaction, the resistance on cone q_c reaches approximately 3MPa. This value q_c corresponds to $\alpha = 3 - 5$.

The wide range of variation of α on the natural soil is caused by the marked inhomogeneity of the loessoid soils, and the restriction of the α range proves the homogenization of the soil by compaction and the elimination of water sensitivity.

In order to verify the concordance between the laboratory results and those in situ, the averages of the densification obtained in the laboratory and the averages of the volume reduction obtained in situ for each area were compared. For an overall assessment, the cone strength ratio was analysed with the average volume reduction obtained in situ and the densification obtained in the laboratory for all areas. The influence of the i_{m300f}/i_{m300i} additional settlement index ratio was followed.

Area	E (tm/m ²)	s (%)	$\Delta H/H$	i_{m300f}/i_{m300i}	q_f/q_i
1	358	1,2	11,8	0,3	1,8
2	400	6,6	12,7	0,2	2,4
3	358	6,9	13,8	0,1	2,3
4	426	13,3	17,3	0,0	2,0

Table 6.18: Average values obtained for each area

It is observed that where the q_f/q_i ratio is higher and the i_{m300f}/i_{m300i} ratio is lower the densification is higher. The increase in densification reflects desensitization. The results obtained in situ have the same tendency as those obtained in the laboratory.

7. FINAL CONCLUSIONS AND RECOMMENDATIONS

7.1. General conclusions

The present project report was developed in order to contribute at the establishment of criteria for verifying the improvement of water sensitive soils (PSU) and to develop correlations between in situ and laboratory test in order to optimize the certification methodology for the elimination of sensitivity to wetting.

This topic is in line with current preoccupations regarding the development of Eurocode 7, where will be introduced a special chapter for diffused soil improvement - which can also be achieved through DC (EN1997-3 – Chapter 10).

In this regard, in order to control the efficiency of the application of such improvement technologies based on specific mechanical behaviour parameters, it is recommended either direct measurements (laboratory tests) or indirect determinations by in situ tests and obtaining the values of the parameters by correlations, topic addressed in the personal research. The conclusions are being summarized below.

In order to study the PSU behaviour and establish correlations based on geotechnical tests between the values determined in situ and those determined in the laboratory, the author developed a special testing program in 4 zones within a project located in the southeast of the country, near Constanta. According to the project requirements, it was necessary to improve the ground on the first 6m. For this purpose, was used a diffuse improvement technology, namely dynamic compaction (DC). The main purpose of the executed works is to eliminate the sensitivity to water.

In order to verify the ground improvement quality are performed specific in situ and laboratory tests. To verify the ground improvement by laboratory tests, especially in the case of large earthworks volumes, are required a large number of tests and time. In situ tests are faster and more economical than laboratory tests and provide a semi-continuous profile. Empirical correlations can be used to determine the properties and behaviour of the soil in the studied location.

In this regard, based on author's research conducted in order to develop the present project report I have developed an efficient and fast methodology for certifying the results for ground improvement by DC for PSU, based on correlations established and verified by the measured processing data in laboratory and in situ. The research and elaboration of the thesis was performed in stages.

Through specific laboratory and in situ tests performed before and after the ground improvement by DC were established, in a first phase, on the basis of specific technical national and international norms, the requirements for the parameters of the improved soil and the quality requirements.

In the following stages of the research, the main aim was to establish a correlation coefficient between in situ and laboratory tests for PSU. According to SREN 1997-2 / 2007, can be used a correlation coefficient (α) between in situ tests (CPT) and compression tests performed in the laboratory (E_{oed}). In the research, the coefficient α was determined for each area and for each layer, before and after the ground improvement. Was analysed its variation on depth and was established an average value for the natural soil and an average value for the improved one. Were analysed also the parameters that influence the coefficient α .

Thus, based on the results obtained and validated by in situ and laboratory tests, the author proposes a methodology for attesting the ground improvement of PSU by DC that includes the following steps for project, used in the figures 7.1 and 7.2:

- a) Execution of an experimental polygon in order establish the coefficient α on site for the natural soil and for the improved soil by:
 - Execution on the basis of a specific program of laboratory tests for the characterization of PSU (physical and mechanical properties)
 - Execution in parallel of the static cone penetration tests (CPTs)
 - Establishing the variation of the specific coefficient α
- b) Knowing the α values in the polygon for PSU, to determine the deformability characteristics (E_{oed}) after the ground improvement, to certify the quality of the works will be performed only in-situ CPT tests (without the need for other laboratory tests) using the criteria proposed in the project report.

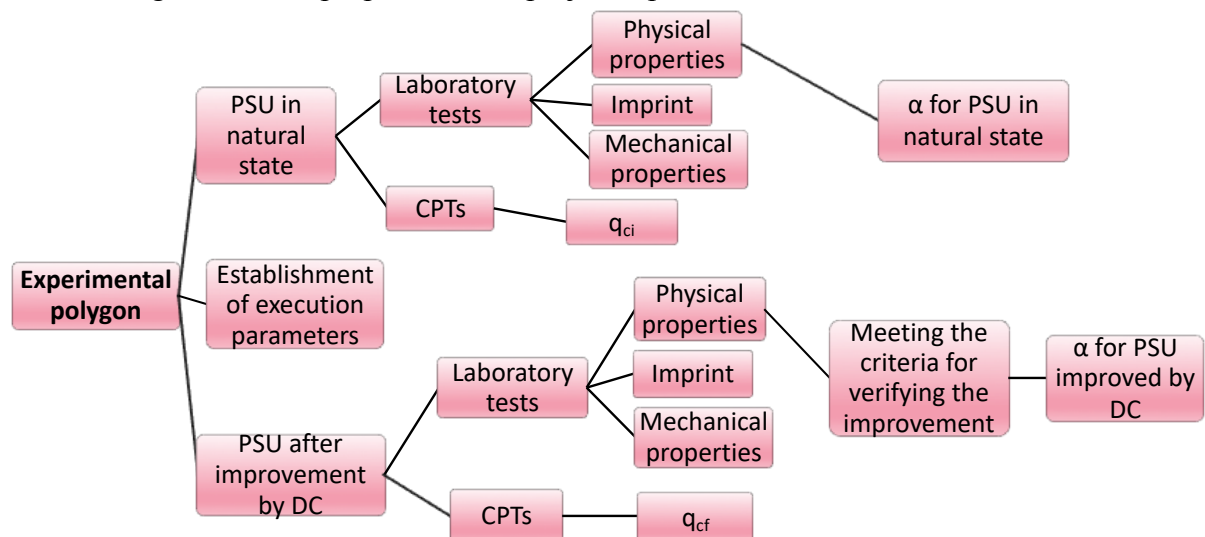


Figure 7.1: Methodology for determining the specific coefficient α - during the testing polygon

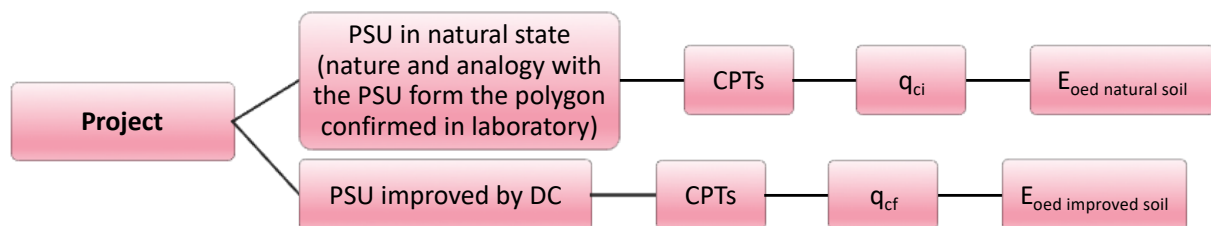


Figure 7.2: Methodology for certifying the quality of DC for PSU - during the execution of the project

Thus, through the performed testing program and the statistical processing of the data resulted that the ground improvement was efficient. The improved soil became is no longer sensitive to water and it has a lower deformability than the natural soil reflected by increases of the edometric modules. These changes in compressibility varies from one area to the other, depending on the initial compressibility and local variations in the nature of the materials and the humidity. Correlations of mechanical parameters were established in order to optimize the certification of eliminating the water sensitivity for PSU.

Based on the CPTs performed in the studied location, can be recommended for preliminary research the following correlations applicable to PSU similar to those analysed (according to the relative area and the analogy coefficient):

- For $q_c < 1,5$: $\alpha = 6 - 8$
- For $2 > q_c \geq 1,5$: $\alpha = 3 - 5$

Based on the CPTs performed in the studied location, can be recommended for preliminary research the following correlations applicable to PSU similar to those analysed improved by DC (according to the relative area and the analogy coefficient):

- For $q_c \geq 3 \text{ MPa}$: $\alpha = 3 - 4$

Thus, in the same spirit as Eurocode 7, by refining the values obtained by research, I defined the alpha values for PSU in its natural state and improved by DC.

The author considers that these results can be exploited in the national normative regarding PSU (NP125/2010) for design and execution of projects on water sensitivity soils. In this regard, it is proposed to introduce in this normative specific for PSU from Romania the performance of CPTs in order to use correlations with the coefficient α to determine the compressibility of the ground according to Eurocode 7. This coefficient α for PSU can have the values obtained by the present research.

At the same time, the author considers that a table like the one from SR EN 1997-2 / 2007 (Annex D, Table D.2) adding the values obtained for PSU can be introduced for information, noting that the values obtained fit well in the existing table for cohesive soils.

Soil	q_c	α
Low plasticity clay	$q_c \leq 0,7 \text{ MPa}$	$3 < \alpha < 8$
	$0,7 < q_c < 2 \text{ MPa}$	$2 < \alpha < 5$
	$q_c \geq 2 \text{ MPa}$	$1 < \alpha < 2,5$
Water sensitive soil (PSU)	$q_c < 1,5 \text{ MPa}$	$6 < \alpha < 8$
	$q_c \geq 1,5 \text{ MPa}$	$3 < \alpha < 5$
Water sensitive soil (PSU) improved by DC	$q_c \geq 3 \text{ MPa}$	$3 < \alpha < 4$
Low plasticity silt	$q_c < 2 \text{ MPa}$	$3 < \alpha < 6$
	$q_c \geq 2 \text{ MPa}$	$1 < \alpha < 2$
Clay with high plasticity	$q_c < 2 \text{ MPa}$	$2 < \alpha < 6$
Silt with high plasticity	$q_c > 2 \text{ MPa}$	$1 < \alpha < 2$

Organic silt	$q_c < 1,2 \text{ MPa}$	$2 < \alpha < 8$
Organic peat and clay	$q_c < 0,7 \text{ MPa}$	
	$50 < w \leq 100$	$1,5 < \alpha < 4$
	$100 < w \leq 200$	$1 < \alpha < 1,5$
	$w > 300$	$\alpha < 0,4$
Crete	$2 < q_c \leq 3 \text{ MPa}$	$2 < \alpha < 4$
	$q_c > 3 \text{ MPa}$	$1,5 < \alpha < 3$
Sand	$q_c \leq 5 \text{ MPa}$	$\alpha = 2$
	$q_c > 10 \text{ MPa}$	$\alpha = 1,5$

Table 7.1: Proposal to complete the table with coefficient α for PSU (SREN 1997-2/2007)

7.2. Personal contributions and future research directions

The main personal contributions included in the present thesis are:

1. State-of-the-art literature review regarding the characteristics and behavior of water sensitive soils (PSU).
2. Bibliographic synthesis on ground improvement by DC and its effects on PSU.
3. Establishment of verification criteria for the control of PSU improvement.
4. Implementation of a case study, execution of in situ and laboratory tests and interpretation of the results.
5. Identification of the parameters that influence the correlation coefficient of the in situ test results (CPT) with the laboratory test results (E_{oed}) α for PSU.
6. Establishment of the variation of the coefficient α for PSU in the natural state and after the ground improvement by DC, in accordance with Eurocode 7.
7. Proposing an efficient methodology for attesting DC works on water sensitive soils.
8. Formulation of practical recommendations regarding the verification of the improvement of water sensitive soils by dynamic compaction using the criteria established and attested by laboratory and field tests.
9. Proposal of specific PSU values of the coefficient α that correlates the results of in situ and laboratory tests with the possibility of introducing these values in the national technical norm.

All these elements were used and exploited as some of the results were obtained at prestigious scientific events in the country and abroad as follows:

1. Sanda Manea, Romeo Ciortan, George Tsitsas, Monica Dumitru, *Verificarea îmbunătățirii caracteristicilor geotehnice a pământurilor sensibile la umezire prin compactare dinamică intensivă*, A XII-a Conferința Națională de Geotehnică și Fundații, Iași, România, 2012.
2. Monica Dumitru, *Geotechnical testing for certification of loess improvement by dynamic compaction*, Fifth International Young Geotechnical Engineering Conference - 5iYGEC'13, Franța, Paris, 2013.
3. Ciortan R., Manea S., Tsitsas G., Dumitru M. *Compactare dinamică – soluție de îmbunătățire a terenului pentru proiecte sustenabile în România*. Conferință ASTR, Brașov, România, 2013.
4. Ciortan R., Manea S., Tsitsas G., Dumitru M. *Tehnologie ecologică privind consolidarea de adâncime a terenului de fundare*. Conferință ASTR, Brașov, România, 2013.
5. Tsitsas G., Ciortan R., V., Konstantakos D., Dumitru, M. *Technical and economic aspects of prevalent ground improvement techniques in romanian waterfront projects*. Proceedings of the PIANC World Congress San Francisco, USA, 2014.
6. Tsitsas, G., Dimitriadi, V., Zekkos, D., Dumitru, M., Ciortan, R., Manea, S. *Dynamic Compaction of Collapsible Soil – Case Study from a Motorway Project in Romania*. Proceedings of the XVI European Conference on Soil Mechanics and Geotechnical Engineering, Edinburg, Vol. 3, pp 1487-1492, Scotland, 2015.
7. Ciortan R., Manea S., Tsitsas (Dumitru) M., Mirițoiu R. *Lucrări de umplutură pentru refacerea profilului unei faleze și controlul compactării acestora*. A XIII-a Conferința Națională de Geotehnică și Fundații, Cluj-Napoca, România, 2016.

Regarding the future research directions in the issue of PSU behaviour as foundation ground, it is proposed to continue the research to establish and certify such correlations for discrete ground improvement technologies applied for water sensitive soil, as well as to establish a correlation with the degree of densification.

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