COMPRESSIBILITY OF CLAYS DETERMINED FROM IN – SITU TESTS

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ABSTRACT

COMPRESSIBILITY OF CLAYS DETERMINED FROM IN – SITU TESTS

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The objective of this thesis is to establish an understanding regarding the estimations of commonly used geotechnical soil parameters from in situ tests. In order to determine the profile, strength and deformability of soils in a project site, execution of site investigation studies consisting borehole drilling, in - situ and laboratory testing are a must. In – situ testing of soils has become growingly popular during site investigations especially for being practical and economic. Hence, estimations of geotechnical parameters from in – situ test results hold a significant place in the geotechnical design practice.

In the content of this study, reasonability and accuracy of the widely used emprical correlations about the estimation of undrained shear strength and coefficient of volume compressibility of cohesive soils from in – situ tests were evaluated. In addition, direct correlations between some of the in – situ test parameters were also discussed in order to determine the applicability of such relationships.

For this purpose, the data of five extensive site investigation studies performed in mostly cohesive soils from Turkey and Europe were compiled and analyzed in a detailed manner. In – situ testing database is consisted of “ Standard Penetration Test”, “Cone Penetration Test”, “Pressuremeter Test” and “Flat Dilatometer Test”

In – situ test parameters were evaluated together with the results of laboratory tests performed on the samples obtained during site investigation studies. According to the comparison of results with the studies in the literature, a general agreement was observed especially for the cases of similar soil conditions.

Keywords: In–Situ Test, Standard Penetration, Cone Penetration, Pressuremeter, Dilatometer, Undrained Shear Strength, Compressibility

ÖZ

SAHA DENEYLERİNDEN KİLLERİN SIKIŞABİLİRLİĞİNİN BELİRLENMESİ

ÜZELER, Volkan

Yüksek Lisans, İnşaat Mühendisliği Bölümü
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Bu tezin amacı, yaygın şekilde kullanılan geoteknik parametrelerin saha deneyleri sonuçları kullanılarak tahmin edilmesi konusunda bir kavram oluşturmaktır. Bir proje sahasındaki zeminlerin profili, mukavemeti ve şekil değiştirebilirliğinin belirlenmesi için sondaj, saha deneyleri ve laboratuvar deneylerinden oluşan saha araştırma çalışmalarının yapılması zorunludur. Özellikle pratik ve ekonomik olmalarından dolayı, saha deneyleri artan bir şekilde popüler hale gelmektedir. Bu nedenle, saha deneyleri kullanılarak geoteknik parametrellerin tahmin edilmesi geoteknik tasarım alanında önemli bir yer tutmaktadır.

Bu çalışma kapsamında, kohezyonlu zeminlerin drenajsız kayma dayanımı ve sıkışabilirlik katsayısı tahminlerinde yaygın olarak kullanılan ilişkilerin uygulanabilirliği ve doğruluğu değerlendirilmiştir. Buna ek olarak, uygulanabilirliğinin belirlenmesi amacı ile, bazı saha deneylerine ait parametreler arasındaki direk ilişkiler de incelenmiştir.

Bu amaçla, Türkiye ve Avrupa ‘da, çoğunlukla kohezyonlu zeminler içerisinde gerçekleştirilmiş olan beş kapsamlı saha araştırma çalışması verileri derlenerek detaylı bir biçimde incelenmiştir. Saha deneylerine ait veritabanı “Standart Penetrasyon Deneyi”, “Konik Penetrasyon Deneyi”, “Pressiyometre Deneyi” ve “Yassı Dilatometre Deneyi” nden oluşmaktadır.

Saha deneylerine ait parametreler, saha araştırma çalışmaları sırasında elde edilmiş numuneler üzerinde gerçekleştirilen laboratuvar deneyleri ile birlikte değerlendirilmiştir. Elde edilen sonuçların literatürde bulunan çalışmalar ile kıyaslanması sonucunda, özellikle benzer zemin koşulları için, genel bir uyum olduğu gözlemlenmiştir.

Anahtar Kelimeler: Saha Deneyleri, Standart Penetrasyon, Konik Penetrasyon, Pressiyometre, Dilatometre, Drenajsız Kayma Dayanımı, Sıkışabilirlik

To My Parents

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CHAPTER 1

INTRODUCTION

The soil we know today is the result of constant struggle between the internal forces (gravity, earthquakes, volcanoes etc.) and external forces (running water, wind etc.) of the Earth over millions of years. During the geological time scale, soil has been reached a balance and it has become an interface between the life on the surface and the inanimate shell deep below the ground. It both allows the presence of organic and inorganic materials and it is sensitive to the changes in its surrounding.

Understanding the soil behavior is very challenging because it is such a unique material that shows different properties even within a meter. Due to continuous interaction with soil throughout the history, many methods have been developed to comprehend its behavior. However, it still comprises many unknowns so, many people have been researching and analyzing this material continuously to understand and speak the same language with it.

The need to understand the response of soil to both internal and external effects caused the engineers to digitize and model the soil behavior but the uncertainties involved in this process have still remained due to impossibility of researching every location and depth. Thus, the best that an engineer can do is to estimate the behavior of soil as accurate as possible in order to present a safe, economic and durable design.

The key tools to predict the behavior of soils can be described as geotechnical parameters which allow the engineers to define the response of soils in universal terms. Traditionally, engineering properties of soils have been determined by sampling from boreholes and laboratory testing or analyses. Simple in – situ tests and small scale field loading tests have been developed in time where the sampling of the material is too difficult or samples were disturbed severely during the process (Johnston, 1983). In time, laboratory tests have become insufficient to extensively determine the characteristics of the soils as the size of the investigated sites increased. Thus, stages of sampling and transportation of samples were considered to be more time consuming than the testing. On the other hand, development of the new in situ testing equipment and techniques during this period has increased the popularity and the demand to the in – situ tests as a more feasible and practical method. In another words, the laboratory have been brought to the site by in – situ testing.

Since then, many different in situ test methods were developed such as Standard Penetration Test (SPT), Cone Penetration Test (CPT), Pressuremeter Test (PMT) and Flat Dilatometer Test (DMT) regarding the determination of soil properties.

Each in situ test employs a different method and parameters to predict the soil behavior but in contrast to laboratory tests, none of the in – situ tests give the geotechnical parameters directly. Therefore, empirical correlations have been established in order to estimate the geotechnical parameters from in situ test results. In spite of the numerous uncertainties involved in emprical approaches, in situ tests have been commonly preferred for being practical and mostly form the basis of geotechnical design together with laboratory tests.

Standard Penetration Test (SPT) is the most widely performed in situ test among the others. SPT can be roughly defined as the soil resistance to penetration in vertical direction. The measure of the resistance is usually defined as the SPT N number which is obtained by the count of blows to penetrate the soil for a specified depth.

Cone Penetration Test (CPT) has become as popular as SPT, especially in Europe, among the in situ tests due to the possibility of obtaining the continuous soil profile. CPT is commonly performed with a cone shaped probe driven from the surface. In today's geotechnical practice, CPT can be applicable to a very wide range of soils and with the current studies, a reasonable estimation of various geotechnical parameters from CPT data has become possible.

Differently from SPT and CPT, in Pressuremeter Test (PMT) resistance of soil is measured in radial direction by an inflatable rubber membrane and mostly conducted in pre – bored holes. Strength and deformability characteristics of soils can be derived directly from the results which makes the PMT as a valuable in situ test. Furthermore, the applicability in wide range of soils and weathered rocks is one of the main advantages of PMT compared to other tests.

Despite being the most recent in situ test among the others, Flat Dilatometer Test (DMT) has quickly spread around the world. DMT is by insertion of a blade with circular membrane in to the soil and measuring the resistance of soil to the lateral expansion of the membrane. Since DMT is a rather new test, application in the field is limited and the estimations of geotechnical parameters or cross correlations between other in situ tests are mostly on a local scale.

In the scope of this study, a considerable number of in situ test results including SPT, CPT, PMT and DMT were compiled from the site investigation works executed in Turkey and Europe. Measured in situ test parameters in the field were evaluated along with the laboratory tests by the means of emprical correlations to establish the undrained shear strength (c_u) and compressibility characteristics through coefficient of volume compressibility (m_v) of cohesive soils. Additionally, the results of the studies regarding each in situ test were compared with similar literature studies to verify the reasonability of relationships.

Beside the predictions related to undrained shear strength (c_u) and coefficient of volume compressiblity (m_v) parameters of cohesive soils from in situ tests, direct correlations between some of the in situ test parameters were also analyzed because it is important for an engineer to have an idea about the parameters of an in situ test when the other tests are not available for a site.

Although the literature is filled with many correlation studies for various types of in situ tests, reasonability of each study shall be evaluated prior to using in the design. Therefore, accuracy and applicability of the predictions regarding the strength and compressibility characteristics of soils from commonly preffered in situ tests were discussed in the content of this study.

CHAPTER 2

SITE INVESTIGATIONS AND IN – SITU TESTING OF SOILS

2.1. General

Since all of the structural designs interact with soil by a certain level, the interaction mechanism between the soil and the superstructure should be analyzed in a detailed manner. Geotechnical design of a project employs various theoretical and numerical methods to define the soil – structure interaction. During this process, an accurate soil model and parameters should be used to describe the behavior of soil. Thus, for an accurate geotechnical design based on accurate engineering properties and parameters, site investigation studies are indispensable.

2.2. Site Investigations

Site investigations are the first step of a project to obtain necessary information about the subsurface conditions at a specific site. Usually, site investigations regarded as an extra load on the project budget and they are performed on a limited scale. However, the cost of the site investigations usually ranges between 0.5 to 1.0 % of the total construction costs and a proper investigation study provides invaluable data to geotechnical engineers which results in economical designs.

The main objectives of the site investigations should be identification of layering and determination of engineering properties of the soil or rock units in both horizontal and vertical directions.

2.3. Planning and Methodology in Site Investigation Works

Before the planning, all available existing information such as soil data from nearby locations should be obtained. In addition, visual surveying and shallow trial pits on the site may provide valuable information about the soil conditions at a project site.

The most widely used method for soil exploration is boring holes from the surface at pre-determined locations in the site. The main benefit of this method is both in situ testing at boreholes and sampling of soil for laboratory tests are possible at the same time. Locations and depth of the boreholes are generally adjusted according to layout of the structure and the geological conditions at the site.

A typical site investigation study is consists of the stages given in Figure 2.1. As the importance of the project or the risks due to subsurface conditions increase, the extent of the investigation program increases but usually the elements indicated in Figure 2.1 provide sufficient information before a geotechnical design of a project. In any case, the contents of explorations should allow the correlations and extension of the existing database if possible (Bowles, 1997).

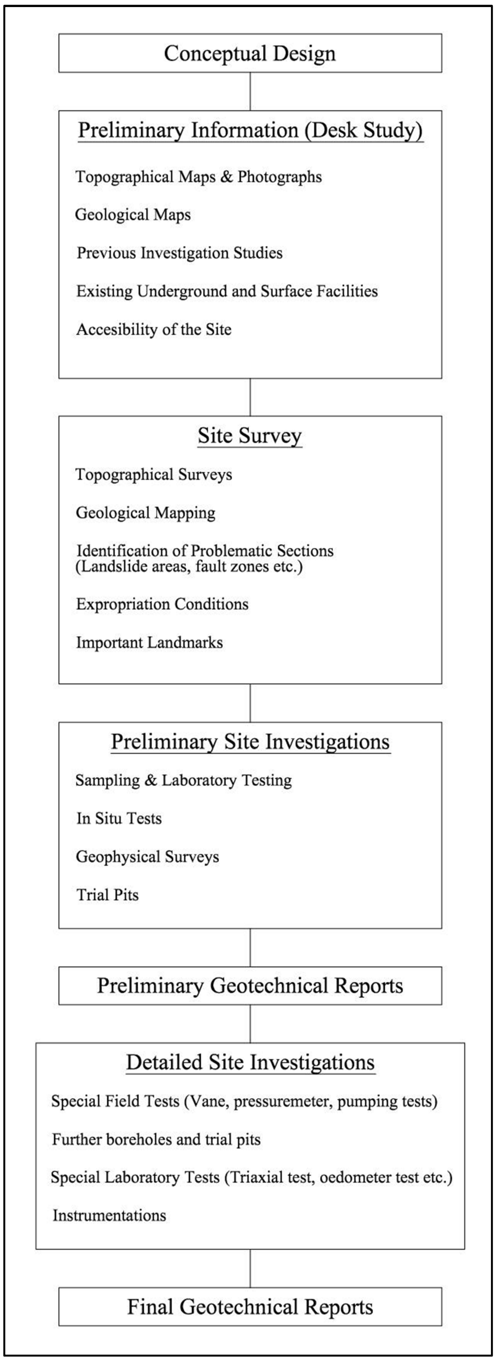

Figure 2.1 Main stages of a typical site investigation study

2.4. Measurement of Soil Properties

2.4.1. General

For any geotechnical site investigation, the ultimate aim is to determine the physical and mechanical characteristics of ground layers. For this purpose in situ tests, laboratory testing or a combination of both can be utilized. Main geotechnical properties sought after in situ and laboratory tests are physical properties (i.e. unit weight, porosity), index properties (i.e. plasticity, grain size distribution), mechanical properties (i.e. shear strength, compressibility) and hydraulic properties (i.e. coefficient of permeability) of the soil.

2.4.2. Laboratory Testing

Direct estimations of engineering properties can be done by various types of laboratory tests on representative soil samples. The quality of the sample can be evaluated with the term "Specimen Quality Designation" (SQD) which was firstly used by Terzaghi et. al. By considering the amount of volumetric strain (ε_{vol}) during laboratory reconsolidation to in situ vertical effective stress (σ_{v0}'), a ranking system was established to define the quality of samples which ranges from A to E defining the best and the worst rank respectively. Terzaghi et. al suggested that, reliable estimations of geotechnical parameters can be obtained by samples with having a SQD rank equal to B better (Degroot et al., 2005). However, there are three major concerns that leave the accuracy of laboratory tests in question:

a. Disturbance:

Disturbance of a soil sample is an unavoidable phenomenon during the borehole drilling especially for cohesive soils. Various equipment and techniques have been invented to sample cohesive soils from boreholes with minimum disturbance. However, there are still some factors that makes impossible to obtain a truly undisturbed sample from a cohesive soil as listed below (Degroot et al., 2005):

- Significant reductions in the effective stress of the sample from the in situ confining stresses due to the bored hole.
- Compression effects ahead of the tube due to friction on the side walls of a sampler and insufficient clearance in the sampler.
- Crushing effects on the sample due to existing damages at the edges of sampler.
- Entrance of the undesired debris and disturbed soil to the sampler from the bottom of the borehole.

- Vibration effects during handling and transportation of a sample from site to laboratory.

- Losses in water content and severely disturbing effects of large temperature changes of the sample due to insufficient protection.

Nature of the soil structure and fabric plays a major role for disturbance in the cohesive soils. Laboratory tests on unfissured, homogenous clays yields more accurate results whereas the results of laboratory tests can be highly misleading for sensitive clays.

In cohesionless soils, on the other hand, even obtaining a sample from the borehole is a very hard task due to the inability of the material to hold itself. There are some freezing or injection techniques to obtain an undisturbed sample from cohesionless soils but the cost of the procedure is usually high and the disturbance effects on the sample is still present at some level.

b. Scale or size of samples:

Size effect concerns on a laboratory sample are mainly due to the macroscopic or fabric structure of the soil. Fabric of a soil volume is defined by the arrangement of all particle groups including all kinds of inclusions, lenses, laminations, organic materials, fissures etc. Since the performance of an engineering structure is mainly governed by the mass behavior of soil, representation of a soil mass with a small sample is a major discussion in geotechnical practice.

c. Boundary conditions:

Laboratory tests are advantageous considering the full control over the boundary and drainage conditions. With some arrangements in the testing equipment and boundary conditions, any type of failure in a soil sample can be simulated which is beneficial for many aspects. However, the boundary conditions and loading pattern set for a laboratory test may not be relevant with in situ conditions.

Because of the mentioned issues about the sampling of soil, methods for estimation of mechanical properties has been commonly preferred by combining in situ tests and empirical correlations with available laboratory test results in geotechnical design practice. During the last decades, improvements in equipment and methods for in situ testing have been developed more rapidly than the traditional laboratory testing techniques. However, laboratory tests are still a significant part of a geotechnical design and if it is used cautiously, laboratory tests complements the correlations obtained from in situ test results.

2.5. In Situ Tests

2.5.1. General

In situ testing can be described as direct or indirect measurement of soil properties under the present stress conditions. They can be performed either from inside of a borehole or directly from the ground surface level. Most of the in situ tests are based on penetration methods which are reasonably fast, repeatable and economic. Commonly used in situ tests in today's geotechnical practice are presented in Figure 2.2.

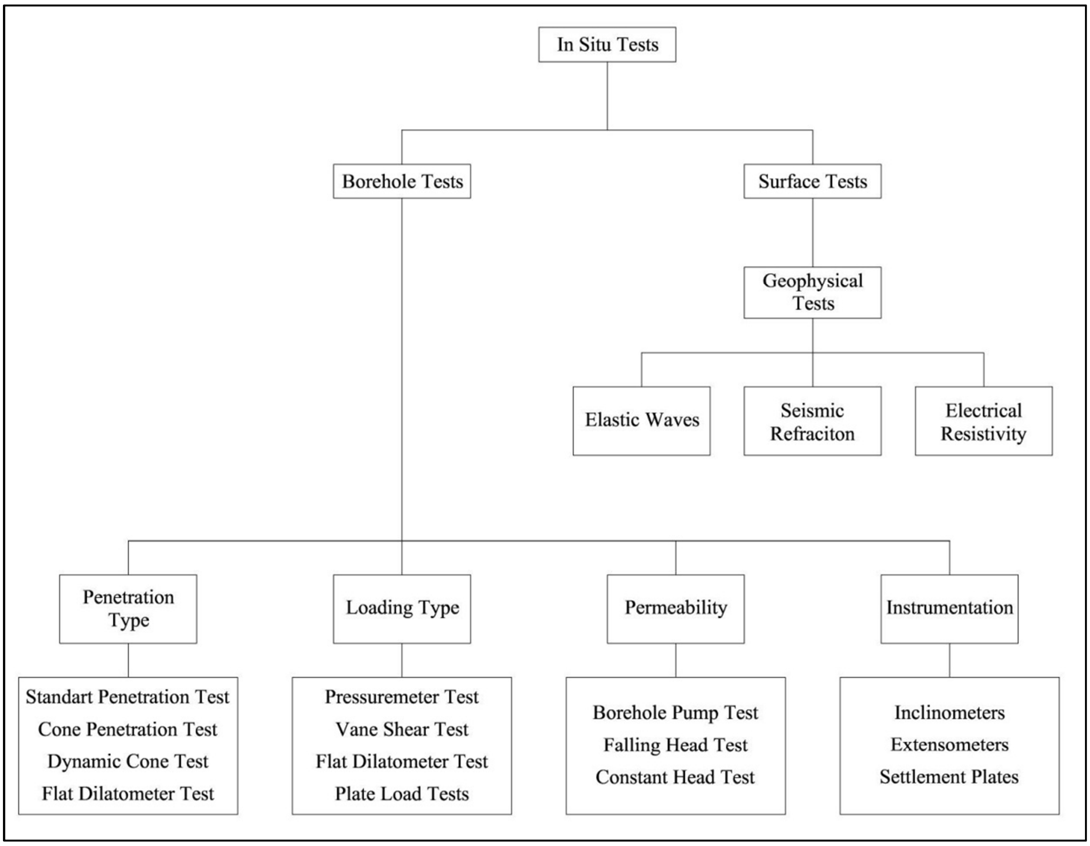

Figure 2.2 Common In situ tests performed during site investigations

2.5.2. Advantages & Disadvantages of In Situ Tests

In situ tests are very practical methods for determination of soil properties with many advantages compared to laboratory tests but they also have limitations some of which can affect the geotechnical design significantly. General advantages and drawbacks of in situ tests compared to laboratory testing are summarized below:

Advantages:

- Larger volume of soil with macro fabric effects (layering, fissures etc.) are represented.
- Continuous record of soil profile can be obtained from some of the tests.
- In situ tests are applicable to any kind of soil.
- Tests are performed under natural environment with current stress states which is essential for the representation of real conditions.
- Most of the in situ tests are economical, practical and less time consuming.
- Repetition of the tests is possible and the test results can be obtained immediately.

Disadvantages:

- Nature of the soil and the index properties cannot be identified at the test depth. (Only in standard penetration test, a disturbed sample is obtained for laboratory testing.)
- Stress and deformation effects are not clear except for pressuremeter test.
- Drainage conditions during the tests cannot be controlled.
- Most of the correlations are based on empirical approaches.
- Each of the tests presents the properties of soil in a different way.
- Inconsistent results are possible for the same type of soils.

Obtained data from in situ tests are usually used for the estimation of geotechnical parameters but there are also many design methods in the literature that use the data of an in situ test directly.

2.5.3. Standard Penetration Test (SPT)

Standard penetration test (SPT) was first introduced in early 1900's by driving an open end pipe into soil during wash boring process and it has become the most extensively used in situ test in site investigation practice.

Originally, the test was used to determine the relative density of granular soils. The idea of the SPT at the beginning was the comparison of blows required to penetrate the tested soil. If the number of blows for a tested location was larger than another location, it was concluded that the denser soil is the one with the largest blow count. Although SPT had been performed only for granular soils in the past, it is executed in almost all kinds of soil today including weak rocks.

2.5.3.1. Equipment and Test Procedure

In 1958 the test method was standardized by ASTM D1586 as follows:

- A standard sampler with dimensions shown in Figure 2.3 is driven into the soil by the energy delivered from a 63.5 kg. weight hammer having a free fall of 760 mm.
- For every 150 mm. penetration of the sampler from the bottom of borehole, number of blow counts are recorded until a total distance of 450 mm. is penetrated.
- Number of blow counts required for the penetration of last 300 mm. is added and it is referred as SPT N value. The number of blow counts recorded during the first 150 mm. is ignored in order to prevent the adverse effects of disturbances during boring process on the test results.
- Procedure is repeated after the drilling to the depth of the next test. (Conventionally test is performed at every 1.0 to 1.5 meters intervals.)

Test is usually stopped on the following conditions:

- 50 or more blows are required for a 150 mm. penetration.
- 100 blows are obtained to drive the required 300 mm.
- 10 successive blows produce no advance.

If any of the listed occasions is encountered during the test, SPT N value for the relevant depth is recorded as "refusal" and indicated with the letter 'R' in borehole logs.

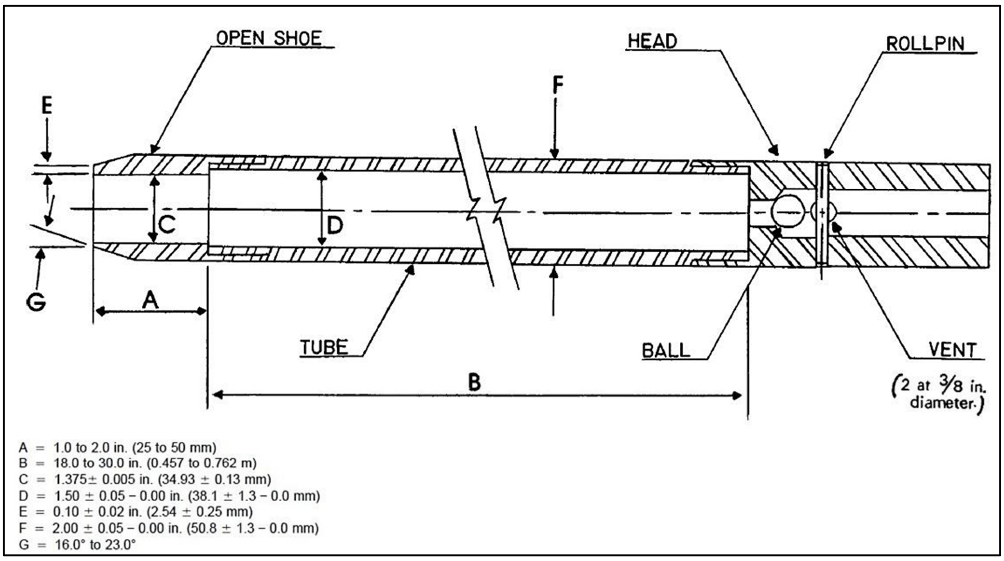

Figure 2.3 Standard split barrel sampler used in SPT (ASTM D1586, 1999)

The reasons for extensive use of SPT in site investigation studies can be related to many factors such as availability of equipment, simplicity of the operation, applicability in wide range of soils and possibility of sampling. For all of its practical aspects, the results of SPT can be dramatically affected by drilling operation, condition or the type of the equipment, capability of the operator, presence of coarse particles, ground water conditions etc.

Because of the variability in equipment and operating conditions, direct use of SPT results for geotechnical designs is not recommended. As a result, many corrections shall be done on the field SPT N values. These corrections can be summarized in an equation form as given below (Canadian Foundation Engineering Manual, 1992).

$$N_{60}= N_F \left\{\frac{E\,R_r}{60}\right\} C_N\, C_R\, C_S\, C_D \qquad \text{(Equation 2.1)}$$

where,

N_{60} = Normalized SPT N value to an energy level of 60 %

ER_r = Rod energy ratio

C_N = Overburden correction factor

C_R = Rod length correction

C_s = Sampler correction

C_D = Borehole diameter correction

N_F = SPT N value measured in the field

In the literature, most researchers (Kovacs et al., 1984, Seed et al., 1984 and Robertson et al., 1983) recommend the SPT N value to be corrected to an energy level of 60 % (Canadian Foundation Engineering Manual, 1992). Bowles (1997) mentions that there are three possible approaches about correction of SPT N value:

- Do nothing on the measured N values.
- Correct only for overburden pressure.
- Apply all of the mentioned corrections.

Since there is not any general agreement on the application of corrections to raw SPT data, many of the correlations with SPT N value only suggests energy corrections, in some cases with overburden corrections. However, overburden correction for fine grained soils is still considered as a controversial issue and not commonly preffered in practice (Sivrikaya & Toğrol, 2007).

In contrast to heavy criticisms about the SPT for being destructive and sensitive to many factors, it still is the most commonly used in situ test in the geotechnical practice and has the largest database.

2.5.3.2. SPT Correlations In Cohesive Soils

Considering the original aim of the test, many of the correlations derived from SPT N values are mainly valid for granular soils. For cohesive soils, on the other hand, it is recommended by many researchers that the correlations shall be compared with laboratory test results otherwise using only SPT N values for cohesive soils could be misleading.

Interpretation of SPT data is mostly performed to estimate undrained shear strength (c_u), coefficient of volume compressibility (m_v) and deformation modulus (E_s) in cohesive soils. The commonly used correlations for each of the mentioned parameters from SPT are presented in the following sections.

2.5.3.2.1. Undrained Shear Strength of Clays (c_u)

The main interested strength parameter of saturated soft clays is undrained shear strength (c_u) because they generally fail in undrained conditions. In laboratory conditions, undrained shear strength is commonly determined by quick, undrained tests such as unconfined compression (UC) or unconsolidated undrained (UU) triaxial tests. One of the earliest correlations between SPT N and c_u was proposed by Terzaghi & Peck (1967) with using the results of UC tests on a wide variety of fine grained soils. A summary of the Terzaghi & Peck's study is given in Table 2.1.

Table 2.1 Approximate ranges of c_u and corresponding SPT N for cohesive soils (Terzaghi & Peck, 1967)

Consistency	**SPT N value**	**Unconfined compression strength (q_u) (kPa)**	**Undrained shear strength ($c_u = q_u / 2$) (kPa)**
Very Soft	0 – 2	0 – 25	0 – 12.5
Soft	2 – 5	25 – 50	12.5 – 25
Medium Stiff	5 – 10	50 – 100	25 – 50
Stiff	10 – 20	100 – 200	50 – 100
Very Stiff	20 – 30	200 – 400	100 – 200
Hard	> 30	> 400	> 200

As a result of the given values in Table 2.1, the relationship between SPT N and c_u can be derived as;

$$c_u \text{ (kPa)} = 6.25 \text{ N} \qquad \text{(Equation 2.2)}$$

After Terzaghi & Peck, many studies have been done in this field by using UC test results. Sowers (1979) proposed a graphical form of estimation with considering the plasticity of the clays as given in Figure 2.4 (Adapted from NAVFAC DM 7.1, 1986).

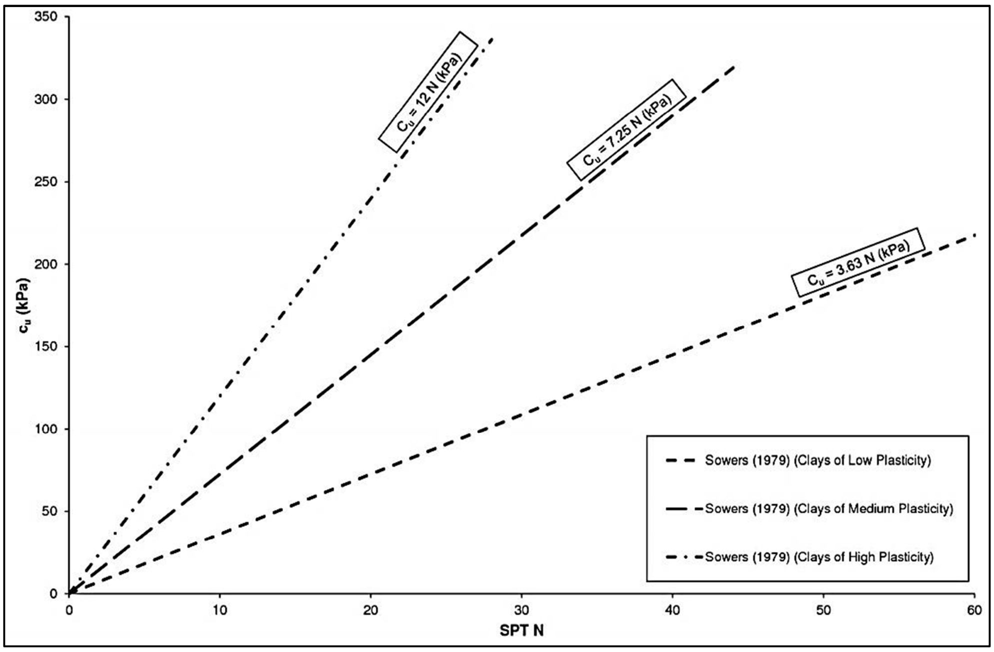

Figure 2.4 Correlation of SPT N vs. c_u
(Sowers, 1979, adapted from NAVFAC DM 7.1, 1986)

From the plot in Figure 2.4, it can be concluded that,

$$c_u \text{ (kPa)} = 3.63 \text{ N (clays of low plasticity)} \qquad \text{(Equation 2.3)}$$

$$c_u \text{ (kPa)} = 7.25 \text{ N (clays of medium plasticity)} \qquad \text{(Equation 2.4)}$$

$$c_u \text{ (kPa)} = 12.0 \text{ N (clays of high plasticity)} \qquad \text{(Equation 2.5)}$$

Sivrikaya & Toğrol (2007) evaluated a large database of SPT N mainly obtained from different sites in Turkey with the results of various laboratory tests such as unconfined compression (UC), triaxial (TA) and field vane tests. In the study of Sivrikaya & Toğrol (2007), fine grained soils are thorougly studied and the results are differentiated according to the type of the laboratory tests. According to the results of the study, the relationships between SPT N_f and c_u are expressed as follows:

$$c_u \text{ (kPa)} = 4.30\ N_f \text{ (UC tests)} \qquad \text{(Equation 2.6)}$$

$$c_u \text{ (kPa)} = 5.10\ N_f \text{ (TA tests)} \qquad \text{(Equation 2.7)}$$

Sivrikaya & Toğrol (2007) clearly stated that the coefficients in the equations are highly dependant on the type of the laboratory test. In addition, it is stated that the scatter of the data have found largest for the UC tests. This result can be mainly related to disturbance and heterogeneity of the sample that influence the behavior. Especially, hard clays can be very sensitive to sampling and testing conditions due to their fissured nature and brittle behavior tendency.

Stroud proposed one of the most popular relationships between SPT N and c_u in 1974. In the study, SPT N data was collected from many sites in United Kingdom together with triaxial tests performed in insensitive stiff and hard clays. The relationship between SPT N value and c_u recommended by Stroud (1974) was:

$$c_u \text{ (kPa)} = f_1\ N_{60} \qquad \text{(Equation 2.8)}$$

Stroud (1974) stated that the factor f_1 in the Equation 2.8 is not a constant value but changes with the plasticity index (PI) of the soil as given in the Figure 2.5.

From the graph given in Figure 2.5, it can be deducted that;

$$c_u \text{ (kPa)} \approx 4.2\ N_{60} \text{ for PI} > 30 \qquad \text{(Equation 2.9)}$$

$$c_u \text{ (kPa)} \approx 4 - 5\ N_{60} \text{ for } 20 < \text{PI} < 30 \qquad \text{(Equation 2.10)}$$

$$c_u \text{ (kPa)} \approx 6 - 7\ N_{60} \text{ for PI} < 20 \qquad \text{(Equation 2.11)}$$

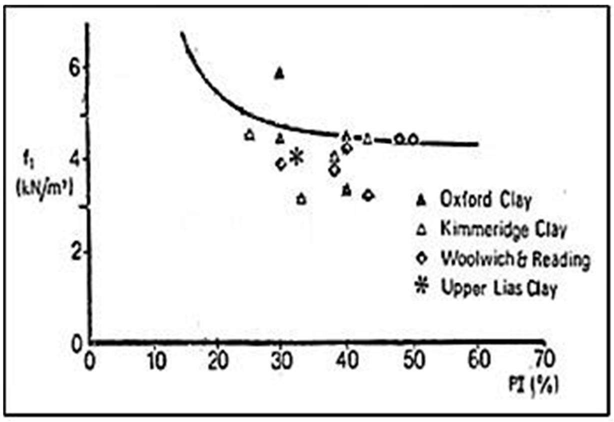

Figure 2.5 Variation of coefficient f_1 with plasticity index (PI) (Stroud, 1974)

As a conclusion of the studies mentioned, it is clear that the relationship between SPT N value and undrained shear strength of cohesive soils takes a general form of an equation as;

$$c_u \text{ (kPa)} = k\,N \qquad \text{(Equation 2.12)}$$

where the value of k depends on site and test conditions (Bowles, 1997).

2.5.3.2.2. Deformation Characteristics of Clays

In any geotechnical design, deformations resulting from the structural stresses are as important as the capacity of the soil to support the structure. Deformation modulus (E_s) thus, is a fundamental parameter that defines the deformability of a soil mass. There are many factors, that affects the E_s like stress history, over consolidation, cementation, anisotropy in cohesive soils.

In situ tests are more commonly used for determination of E_s instead of laboratory tests which are usually expensive and time consuming. However, it is reported by many researchers that the correlations with SPT resulted in considerable scatter in the data and the lack of correlation is expected because SPT N values depends on many variables. Therefore, the proposed correlations are highly recommended to be used for preliminary assessment purposes (Kulhawy & Mayne, 1990). Several suggestions that have been made by Bowles (1997) for fine grained soils are summarized in Table 2.2.

Table 2.2 Estimation of deformation modulus from SPT N (Bowles, 1997)

Soil Type	**E_s by SPT N_{55} (kPa)**
Clayey Sands	320 (N + 15)
Silts, Sandy silts and clayey silts	300 (N + 6)

In addition to estimations from SPT N values, Bowles (1997) also correlated E_s with c_u for clayey soils as given below:

Table 2.3 Estimation of deformation modulus from undrained shear strength (Bowles, 1997)

Soil Type	E_s by c_u (kPa)
Clay and silts with PI > 30 or organic	100 – 500 c_u
Silty or sandy clays with PI < 30 or stiff	500 – 1500 c_u

For an in indirect method of estimating the E_s, one dimensional deformability characteristic of soils can be used. By using the theory of elasticity, deformation modulus of cohesive soils can be estimated as,

$$E_s = M \frac{(1 + \nu)(1 - 2\nu)}{(1 - \nu)} \qquad \text{(Equation 2.13)}$$

where, ν is Poisson's ratio of soil and M is referred as one dimensional deformation modulus (constrained modulus) which can also be defined as;

$$M = \frac{1}{m_v} \qquad \text{(Equation 2.14)}$$

The value m_v in the Equation 2.14 is defined as the coefficient of volume compressibility of soil. Stroud (1974) has proposed a correlation for estimation of m_v from SPT N values as:

$$m_v\ (m^2 / kN) = 1 / (f_2\ N_{60}) \qquad \text{(Equation 2.15)}$$

Coefficient f_2 is also defined as a variable value that changes with plasticity index (PI) of the soil by Stroud (1974). The change in f_2 with PI suggested by Stroud (1974) is illustrated in Figure 2.6.

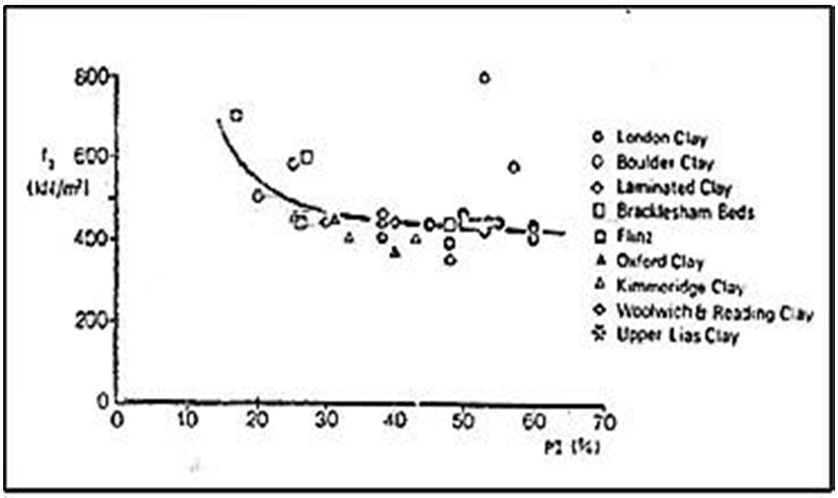

Figure 2.6 Variation of coefficient f_2 with plasticity index (PI) (Stroud, 1974)

The values m_v of a soil can be determined by oedometer tests in laboratory conditions for the effective stresses at the depth of the sample. Hence, a comparison with laboratory tests and correlation proposed by Stroud (1974) is possible.

2.5.4. Cone Penetration Test (CPT)

Cone penetration test (CPT) has become a very popular in situ test in recent years as a logging method and measurement tool of strength and deformability of soils. Early prototypes of CPT equipment consisted of a penetrometer having a cone shaped end and a mechanical pushing system to drive the cone into the soil. The idea behind the CPT was simple: as the cone is pushed to the soil, the force necessary to penetrate was an indication of the strength and density of the soil.

2.5.4.1. Equipment and Test Procedure

Starting from basic mechanical cones, many types of penetrometers are now used in site investigation works such as electric cones, piezocones, seismic cones etc. Aside from special projects, commonly used penetrometer type is electric cone which has a tip of 60° apex angle cone with a diameter of 35.7 mm. (equal to 10 cm^2 cross sectional area) and a frictional sleeve behind the tip to measure the side friction.

A typical CPT probe and data output from the test are given in Figures 2.7 and 2.8. Some additional features can be added to the probe like pore pressure cells, vibration sensors, inclinometers etc. for special studies but the main functions of the cone is the measurement of tip resistance and friction in soils.

CPT is usually performed from the ground surface which is one of the advantages of the test. Pushing of the probe and data collection is generally performed by truck mounted facilities for land applications. Modern CPT equipment allow explorations up to 100 meters in soft soils, but for stiff or coarse grained soils, the depth of penetration can be extended by pre drilling method.

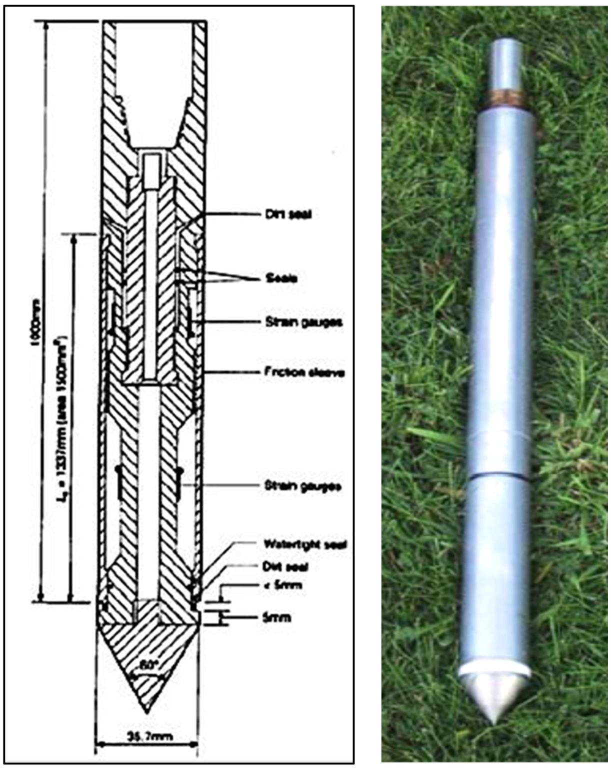

Figure 2.7 Typical CPT probe and its schematic view (Robertson & Cabal, 2010)

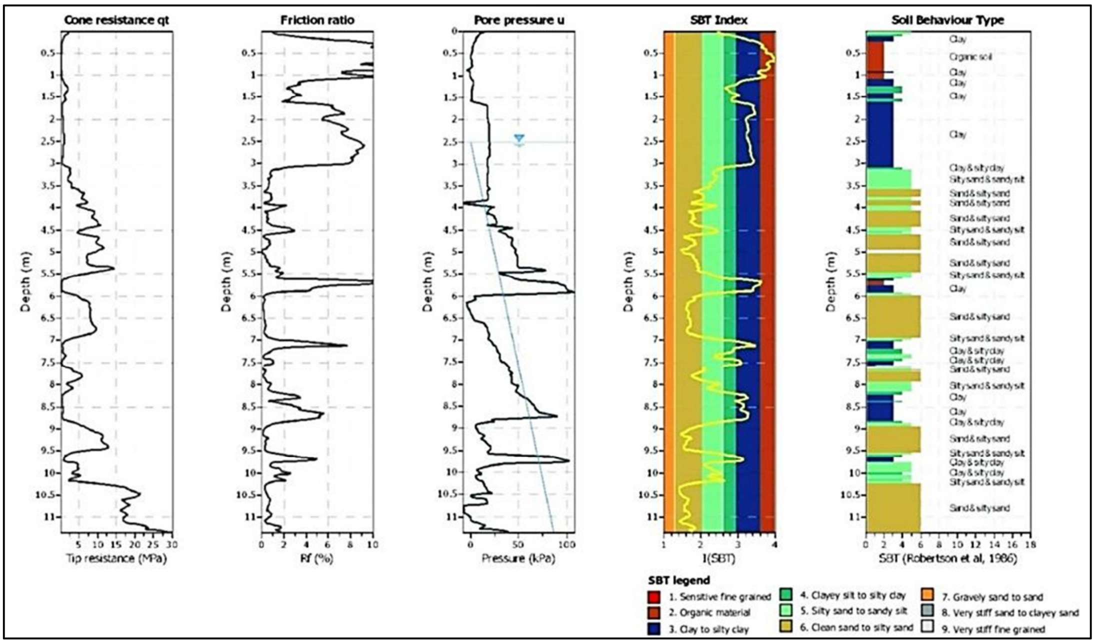

Figure 2.8 Typical CPT data output from a software (Yüksel Proje Uluslararası A.Ş)

Main criteria required for accurate and reliable results from CPT are listed as below:

- **Verticality of the probe during the test:** Verticality of the equipment during the test is one of the key elements of CPT. Pushing equipment shall be positioned appropriately in the field to obtain a pushing thrust as near as possible to vertical. A maximum of 2° of deviation from the original verticality during the test is commonly accepted but a deflection of 5° or more over a meter of penetration causes damages to penetrometer and the rods.

- **Rate of penetration:** Although CPT is a very quick test; rate of penetration should be set carefully to avoid damages. General practice for penetration rate is 2 cm/sec with slight variations depending on the soil conditions. If the test involves pore pressure measurements, rate of penetration has significant effects on the results. Excess pore water pressures starts to dissipate in case of a pause during the test. Therefore, CPT shall be performed as continuously as possible for correct pore water measurements.

- **Interval of readings:** CPT cone provides continuous data about the soil layers. Most standards require the interval of readings to be not more than 20 cm. Modern penetrometers and CPT software provides data collection for every 2 – 5 cm which is satisfactory for the most conditions.

- **Calibration and maintenance of the equipment:** Since CPT is performed by mostly electronic equipment, periodical calibrations and maintenance are essential for healthy test results. Damaged cones should be inspected regularly and replaced if necessary.

Provided that the requirements mentioned above are satisfied, CPT has many advantages as an in situ test. It provides fast and continuous soil profile with related friction and tip resistance values with minimum operator and equipment errors. Despite its wide spread use, initial cost of equipment, lack of sampling, ineffectiveness in coarse grained or hard soils, labor intensive data handling and presentation are major drawbacks of this test (Robertson & Campanella, 1983).

2.5.4.2. CPT Correlations In Cohesive Soils

Cone penetration test is mostly preferred for its capability of soil identification but earliest applications of CPT includes estimation of undrained shear strength and compressibility characteristics of cohesive soils. Most of the time, CPT soundings are performed parallel to bore hole drillings in order to supplement the results of CPT with SPT or laboratory tests.

During the CPT, complex changes in stress and strains are induced around the tip and a valid theoretical solution has not been presented to this problem. Therefore, interpretations of CPT results are mainly based on empirical correlations (Robertson & Campanella, 1983).

2.5.4.2.1. Undrained Shear Strength Of Clays (c_u)

Significant studies and reviews have been made for undrained shear strength (c_u) estimation from CPT data. Estimates of c_u from the tip resistance of CPT (q_c) usually take the form of an equation as given below:

$$q_c = N_k . c_u + \sigma_{v0} \qquad \text{(Equation 2.16)}$$

where,

N_k: cone factor

σ_{v0}: in situ total overburden pressure

N_k factor is generally suggested in the order of 9, if classical bearing capacity theories are applied for the solution. Cavity expansion theories suggest that the N_k value ranges from 7 – 13 whereas steady penetration theory provides a narrow range as 14 – 18 (Kulhawy & Mayne, 1990). Lunne et. al (1997) showed that N_k values can be as low as 6.

Because of the varieties in the theoretical solutions, relationships are generally based on the empirical approaches with a known measured value of c_u in laboratory or other field tests. Some of the proposed N_k factors are summarized by Djoenaidi (1985, cited in Kulhawy & Mayne, 1990) and given in Figure 2.9

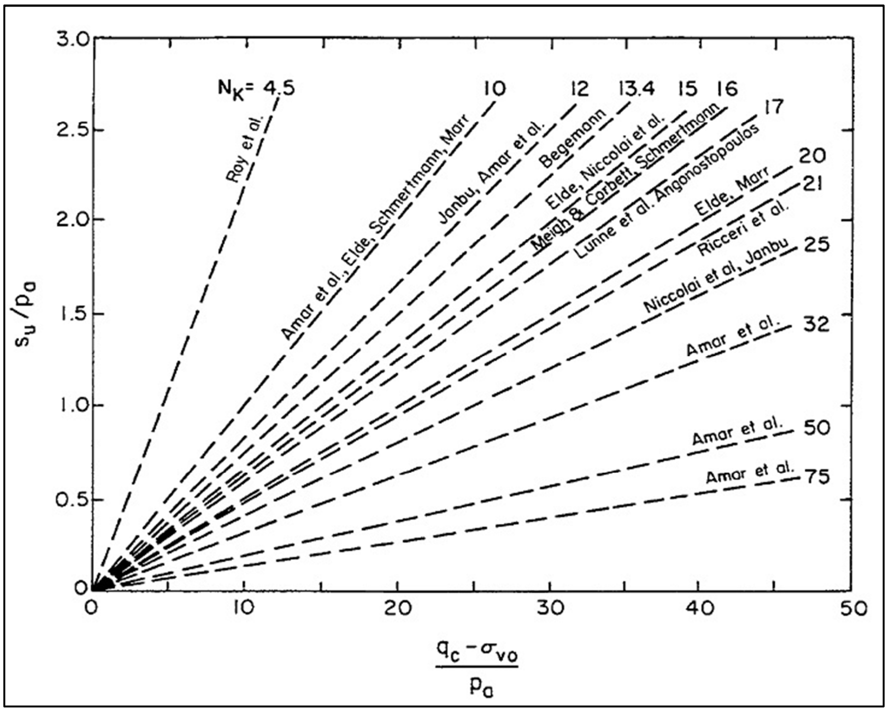

Figure 2.9 Range of N_k factors for CPT data
(Djoenaidi, 1985 cited in Kulhawy & Mayne, 1990)

As it can be seen from the Figure 2.9, N_k values show great variation. The reason for this variation can be related to many factors such as inconsistent reference strengths, mixture of different cone types and effects of disturbance and the effects of fissures on the behavior for stiff over consolidated clays. In addition, pore water pressure corrections for the tip resistance have been extensively discussed in the literature. Aas et al. (1986, cited in Kulhawy & Mayne, 1990) illustrated the effects of pore pressure correction on the q_c values for piezocones with different area ratios as given in Figure 2.10.

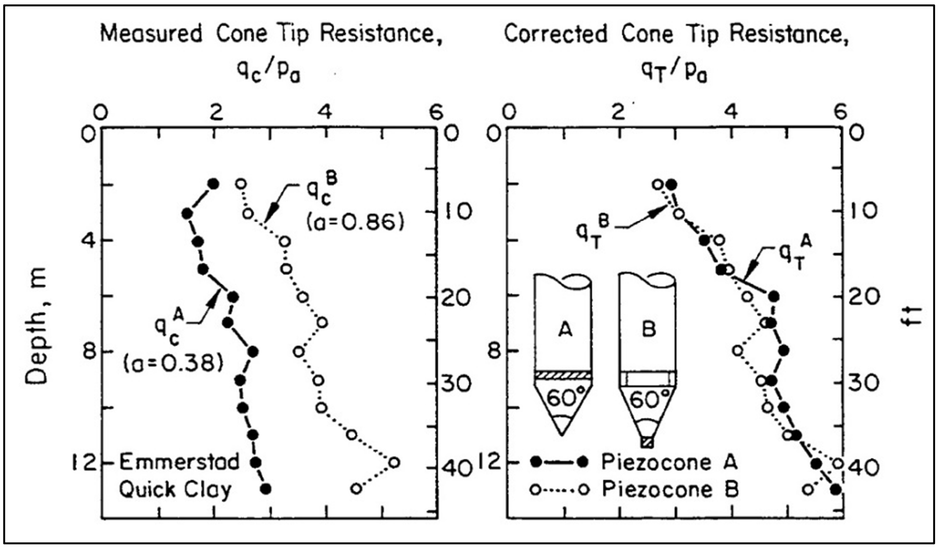

Figure 2.10 Effect of pore water pressure on cone tip resistance
(Aas et al., 1986 cited in Kulhawy & Mayne, 1990)

Another important factor affecting the variation of N_k values is selection of consistent reference strength. Often, field Vane shear test is preferred as a reference to c_u but, it is stated that the field Vane test also requires some corrections itself for the estimation of undrained shear strength. (Kulhawy & Mayne, 1990)

Schmertmann (1975) states that the best procedure for interpretation is making individual correlations for N_k based on c_u measurements for specific soil conditions and CPT procedures (Robertson & Campanella, 1983).

2.5.4.2.2. Compressibility Characteristics Of Clays

Numerous correlations between one dimensional deformation modulus (constrained modulus) and cone tip resistance have been developed most of which are takes the form of following;

$$M = \frac{1}{m_v} = \alpha \, q_c \qquad \text{(Equation 2.17)}$$

Sanglerat (1972) developed an extensive and detailed array of α values for a wide range of tip resistance (q_c) values and soil types which are summarized in Table 2.4

Table 2.4 Range of α values depending on the cone tip resistance values (Sanglerat, 1972)

Soil Type	Cone tip resistance (q_c) (MPa)	α value
Clay of low plasticity (CL)	$q_c < 0.7$	3 - 8
	$0.7 < q_c < 2.0$	2 – 5
	$q_c > 2.0$	1 – 2.5
Silts of low plasticity (ML)	$q_c > 2.0$	3 – 6
	$q_c < 2.0$	1 – 3
Highly plastic silts and clay (MH,CH)	$q_c < 2.0$	2 – 6
Organic silts (OL)	$q_c < 1.2$	2 – 8
Peat and organic clay (Pt, OH)	$q_c < 0.7$	0.4 – 4 (depending on water content)

It can be concluded from the Table 2.4 that, α values gets smaller as tip resistance increases, which is an expected result because, compressibility of a soil decreases with increased strength and effective overburden stress.

In a more recent study, Kaplan et al. (2004) evaluated CPT results with laboratory test data obtained from two different clay sites and proposed a range for a coefficient depending on the cone tip resistance as given in Figure 2.11

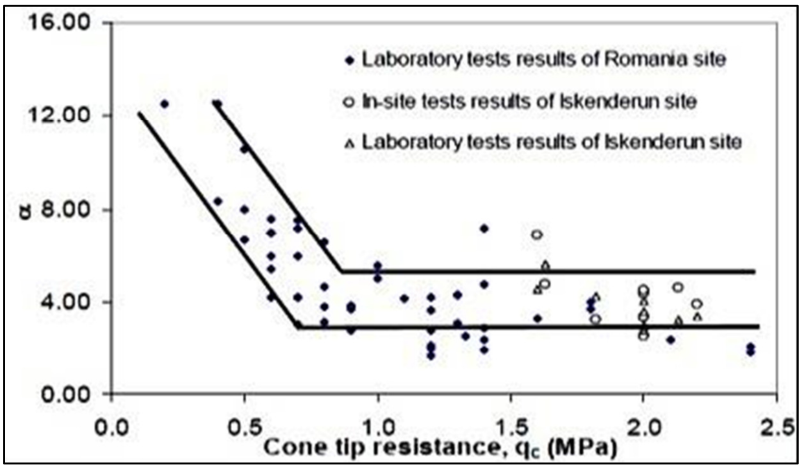

Figure 2.11 Change of α with cone tip resistances (Kaplan et al., 2004)

As a result of this study, it is observed that the α value changes between 4 – 12 and have a tendency to decrease as the tip resistance increases for a tip resistance range of 0 – 0.75 MPa. On the other hand, for tip resistance values greater than 0.75 MPa, α values ranged between 2.7 and 4.7 independent from the increase in the cone tip resistance.

Mayne (1990, cited in Kulhawy & Mayne, 1990) evaluated cone resistance data from 12 different sites and proposed a more general correlation by normalized tip resistance values without any distinction in the soil type. Result of the study with a graphical representation is given in Figure 2.12.

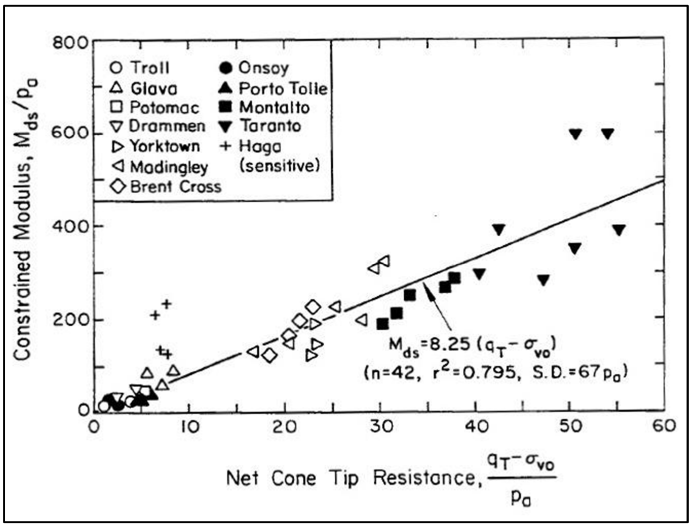

Figure 2.12 Relationship between net cone tip resistance and constrained modulus (Mayne, 1990, cited in Kulhawy & Mayne, 1990)

Bowles (1997), proposed correlations with the cone tip resistance and deformation modulus for clays and sands with fines as given below;

$E_s = 3$ to $6\ q_c$ (for clayey sands) (Equation 2.18)

$E_s = 1$ to $2\ q_c$ (for silty sands) (Equation 2.19)

$E_s = 3$ to $8\ q_c$ (for clays) (Equation 2.20)

From the theory of elasticity, constrained modulus can be related to deformation modulus by the Equations 2.13 and 2.14 as previously mentioned.

The methods presented above provide only a rough estimate of soil compressibility. Furthermore, estimation of a drained behavior parameter like m_v or M from an undrained test is prone to serious error, especially when based on empirical correlations (Robertson & Campanella, 1983).

2.5.5. Pressuremeter Test (PMT)

Invented by Louis Menard in 1954, the pressuremeter test (PMT) has become one of the most fundamental in situ tests preferred in site investigations. The original concept of the PMT created by Menard was inflation of a cylindrical balloon inside a prebored hole to measure the deformation properties of the soil. Following of the invention, pressuremeter test became popular, especially in France, and many studies have been done for both estimation of the geotechnical parameters and design of foundations.

2.5.5.1. Equipment and Test Procedure

The pressuremeter consists of three main parts which are a probe, a monitoring box and tubings for inflation (Figure 2.13).

- **Probe:** A typical Menard type pressuremeter probe includes three separate cells namely as top cell, loading cell and bottom cell. Top and bottom cells are usually called “guard” cells which are filled with gas before the test in order to isolate the loading cell from end condition effects. Load cell is a flexible membrane (usually made from rubber) that is filled with water after the guard cells are inflated.

- **Monitoring box:** Monitoring box is set on the ground preferably close to the borehole. It is used for both controlling the pressure given to the probe and monitoring the volume changes with respect to pressure increase by the dial gauges on it.

- **Tubings:** Tubings on the equipment delivers the gas and water from related tanks to the guard and loading cells on the probe.

Currently, many types of pressuremeters have been developed besides original Menard type pressuremeter such as self boring and pushed in pressuremeters which can be used for different in situ soil conditions.

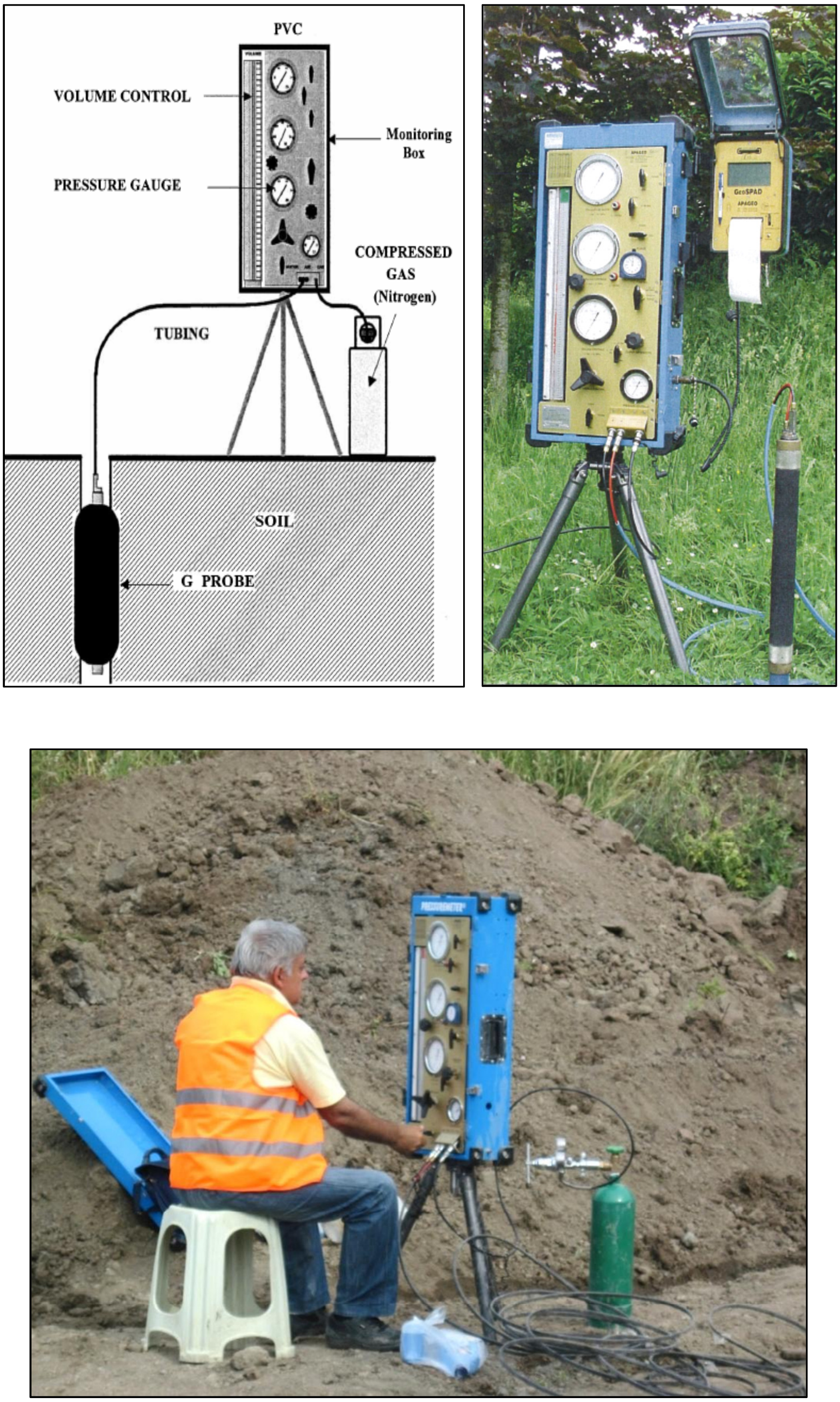

Figure 2.13 Pressuremeter test equipment and application in the field (Apageo, 2006 & Yüksel Proje Uluslararası A.Ş)

PMT is performed either by application of pressures in equal increments (stress controlled) or equal volume increments (strain controlled). In stress controlled tests typically a pressure of 10 kPa is used between the steps and it is hold 1 minute during the stage and the volume of the probe is recorded at 15th, 30th and 60th seconds of each step. Test is usually stopped when the volume of the given liquids is equal to the initial volume of the borehole. In a strain controlled test, on the other hand, volume increment is usually the 2.5 % of the initial volume and the pressure at 15th second is recorded for each increment.

Before the test, two main calibrations namely as volume calibration and pressure calibration are required in order to correct the raw data obtained during the test. The calibrations are explained below:

Volume calibration: Volume calibration is performed for detection of the leaks in the system and making necessary adjustments about system compressibility. Pressuremeter probe is usually placed in a steel tube before the volume calibration and the pressure is increased in steps. For a given pressure, the volume lost is determined since the probe is confined by the tubes. A typical volume calibration curve is given in Figure 2.14

Pressure calibration: Pressure calibration is performed to determine the self-resistance of the rubber membrane to expansion. Before the pressure calibration, probe is taken out from the steel tube and calibration is performed in atmospheric pressure conditions. A typical pressure calibration curve is given in Figure 2.14.

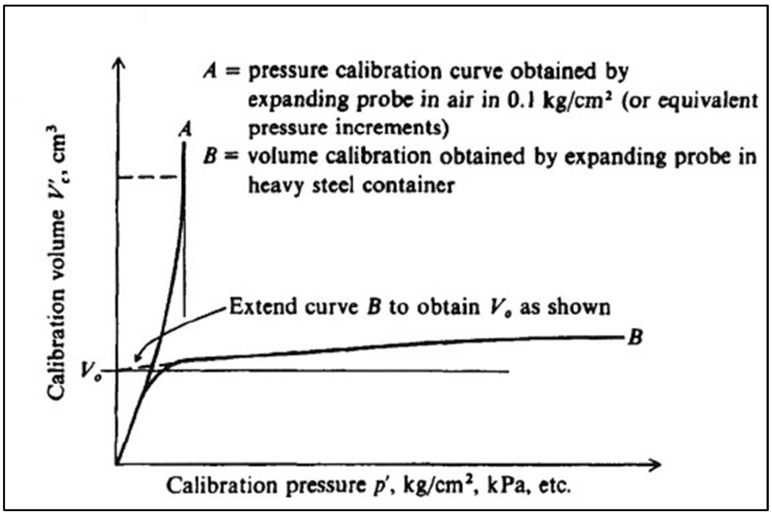

Figure 2.14 Calibration curves obtained before the pressuremeter tests (Bowles, 1997)

Calibrations in the pressuremeter test is essential for obtaining accurate results from the test and if the calibrations are not carried out properly, then the data obtained from the test can be considered as useless.

After the tests, volume changes recorded during the test are plotted against the pressure with considering the necessary corrections based on calibrations. As a result, the corrected pressuremeter graph is obtained as given in Figure 2.15.

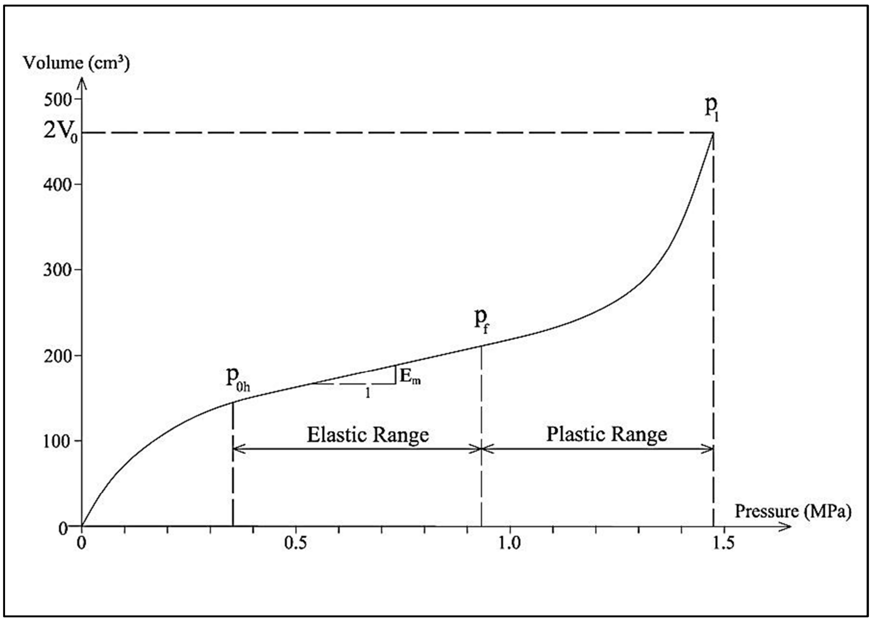

Figure 2.15 Typical corrected pressuremeter test graph

At the beginning of the test, probe expands rapidly inside the borehole without any resistance until the pressure reaches p_{0h} value. At the pressure p_{0h}, it is assumed that the membrane is in full contact with the sides of the borehole. p_{0h} value is often interpreted as total horizontal in situ stress. With further increase in the pressure, the slope of the pressure and volume curve becomes almost constant which is the result of elastic behavior of soil and it is described as elastic range on the graph. After the pressure p_f, permanent deformations and creep occur in the soil and the volumetric expansion increases significantly with the pressure.

The limit pressure p_l, is defined theoretically as the pressure for which an infinite expansion of the probe is expected (Briaud, 1992). As it can be seen from the graph in Figure 2.15; volume tends to increase significantly even with small pressure increases so graph becomes nearly asymptotical to the vertical axis at the pressure p_l. Conventionally, soil is assumed to be failed at the limit pressure (p_l) value at which the volume increase (ΔV) becomes equal to the initial volume of borehole (V_0). The semi log plot of pressure (p) against the inverse of corrected volume reading (1 / V) represents the plastic range with a straight line from which the limit pressure can be identified more easily. The intersection of the straight line with $V = 2V_0$ yields the p_l in the abscissa (Figure 2.16). This method is very useful to determine the limit pressure of hard soils or weak rocks for which the failure is hard to observe.

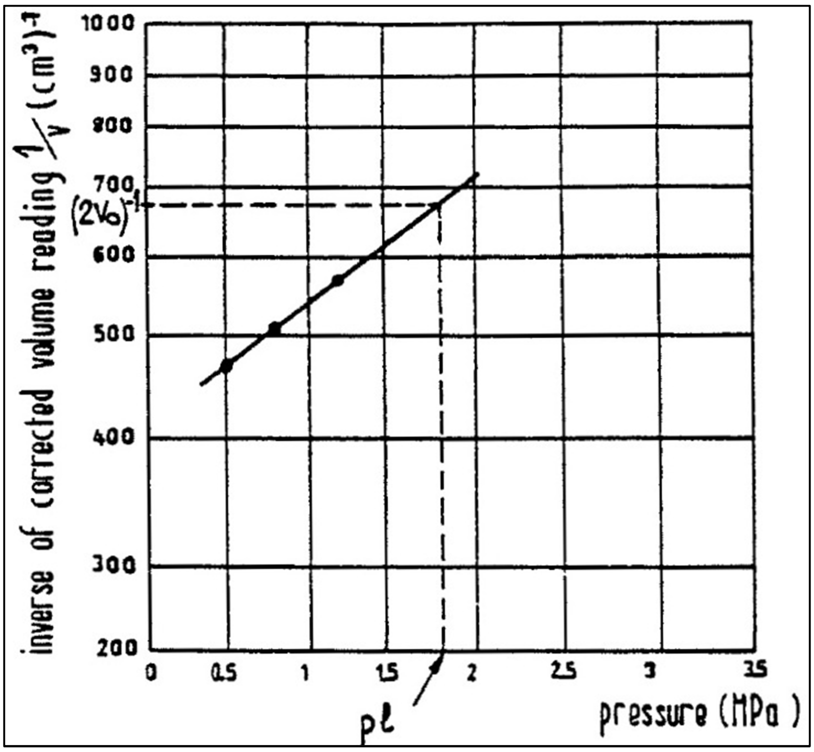

Figure 2.16 Determination of p_l on p vs. 1 / V plot (Amar et al., 1991)

Although limit pressure p_l indicates the fail pressure of the soil, net limit pressure value, p_{ln}, is more commonly used in geotechnical practice and correlations because it is relatively insensitive to the disturbances in the borehole (Briaud, 1992). Net limit pressure is defined as:

$$p_{ln} \text{ (or } p_l^*) = p_l - p_0 \qquad \text{(Equation 2.21)}$$

The value p_0 can either be obtained from the test graph or calculated by the following equation:

$$p_0 = K_0 . (\gamma . z - u_0) + u_0 \qquad \text{(Equation 2.22)}$$

where;

γ = unit weight of the tested soil

z = depth from the ground surface to the test level

u_0 = pore water pressure at the test level

K_0 = coefficient of earth pressure at rest

In theory, p_{oh} value read from the pressuremeter curve and the calculated p_0 value should be equal but in practice it has been found that p_{om} value is very difficult to determine due to the disturbances inside the borehole (Baguelin et al., 1978).

Pressuremeter test is widely used for foundation designs because the method of the test is parallel to the behavior of an actual foundation. It involves well defined boundary and stress – strain relationships different from the other tests like SPT and CPT and it can be performed on any type of soil and rock. It has the advantage of direct comparison of the results with another pressuremeter test due to its standardization and simple operational requirements.

2.5.5.2. Pressuremeter Correlations in Soils

Pressuremeter test results are used for identification of the soils. The difference in the behavior of sands and clays is also distinctive in the pressuremeter curve. The curves of clays generally show a sharp bending near the limit pressure which is a clear indication of failure whereas the bend in the curve is faint for sands. The reason for difference can be related to components of strength. Since sands derive their strength mostly from friction, as the normal forces increase on the particles increase, interlocking between particles become more effective so the strength of the sand increases. On the other hand, in clays, strength is determined by undrained shear strength independent from the applied normal stresses for a rapid test like pressuremeter. As a result, when the shear forces exceed the undrained shear strength, a clear failure is observed on the pressuremeter curve. (Briaud, 1992) General ranges of the net limit pressure and pressuremeter modulus for sands and clays are summarized in Tables 2.5 and 2.6.

Table 2.5 Approximate common values of the pressuremeter parameters for sands (Briaud, 1992)

SANDS				
Soil Type	**Loose**	**Compact**	**Dense**	**Very Dense**
p_{ln} (kPa)	0 – 500	500 – 1500	1500 – 2500	> 2500
E_p (kPa)	0 – 3500	3500 – 12000	12000 – 22500	> 22500

Table 2.6 Approximate common values of the pressuremeter parameters for clays (Briaud, 1992)

CLAYS					
Soil Type	**Soft**	**Medium**	**Stiff**	**Very Stiff**	**Hard**
p_{ln} (kPa)	0 – 200	200 – 400	400 – 800	800 – 1600	> 1600
E_p (kPa)	0 – 2500	2500 – 5000	5000 – 12000	12000 – 25000	> 25000

2.5.5.2.1. Undrained Shear Strength Of Clays (c_u)

Pressuremeter test imposes rapid loads on the tested soil while the drainage of the pore water is usually limited. Thus, it is commonly regarded as a suitable test to estimate the undrained shear strength of cohesive soils. There are different approaches for the estimation of undrained shear strength which can be listed as limit pressure method, yield pressure method, shear curve method etc.

Yield pressure method, uses the yield pressure result and employs the following simple equation for the estimation:

$$c_u = p_y - \sigma_{oh} \qquad \text{(Equation 2.23)}$$

where,

p_y = yield pressure

σ_{0h} = total horizontal stress at rest.

However, yield pressure method is not recommended because yield pressure is generally a large value that may lead to overestimated results (Briaud, 1992). Shear curve method on the other hand, uses a graphical solution for which the entire shear stress and strain graph is derived from the test. This method is also not recommended for being a graphical solution and leading high undrained shear strength estimations.

The limit pressure method is commonly accepted in the practice which uses the theoretical expression as given below (Cassan, 1972 cited in Clarke, 1995):

$$p_l = \sigma_{0h} + c_u \left(1 + Ln \frac{G}{c_u} \right) \qquad \text{(Equation 2.24)}$$

where,

p_l = limit pressure

σ_{0h} = total horizontal stress at rest.

G = shear modulus of the soil

The equation can be rewritten as the following for simplification purposes:

$$c_u = \frac{p_{ln}}{\beta} \qquad \text{(Equation 2.25)}$$

The factor β in Equation 2.25 is referred as pressuremeter constant. It depends on the ratio of shear modulus over the undrained shear strength which varies mainly with the over consolidation ratio of the clay. According to many researchers such as Cassan (1972) and Briaud (1992), β value ranges between 5.5 and 7.5 with an average of 6.5. Baguelin et al. (1978) compiled the published studies in the literature and compared on a graph as given in Figure 2.17

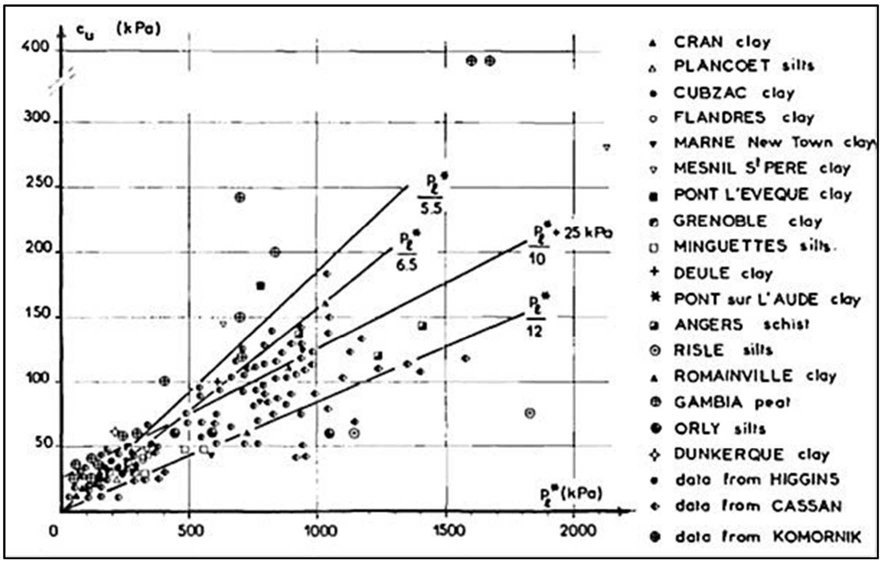

Figure 2.17 p_{ln} vs. c_u correlation studies and proposed β factors in the literature (Baguelin et al., 1978)

Similarly, Clarke (1995) presented a summary of β factors proposed in the literature which are given in Table 2.7.

Table 2.7 Proposed values of the factor β in the literature (Clarke, 1995)

Soil Type	**β factor**	**Source**
All clays	2 – 5	Menard, 1957
Soft to firm clays	5.5	Cassan, 1972, Amar & Jezequel, 1972
Firm to stiff clays	8	
Stiff to very stiff clays	15	
Stiff clays	6.8	Marsland & Randolph, 1977
All clays	5.1	Lukas and LeClerc de Bussy, 1976
Stiff clays	10	Martin & Drahos, 1986

The variation of the factor β can be related to uncertainties involved in the measurement of the σ_{h0}, differences in reference strengths, influence of disturbance and anisotropy. (Clarke, 1995).

Baguelin et al. (1978) presented a nonlinear correlation by comparing a significant number of data given in Figure 2.18.

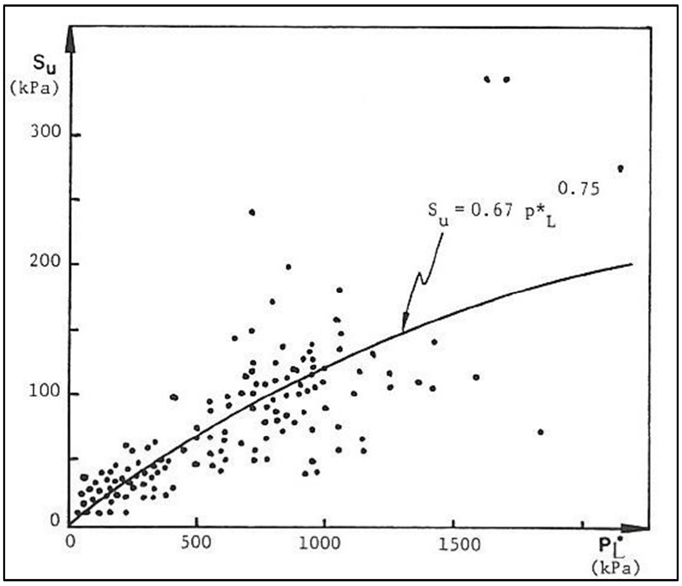

Figure 2.18 Correlation between undrained shear strength and net limit pressure (Baguelin et al., 1978, cited in Briaud, 1992)

2.5.5.2.2. Deformation Modulus Of Soils (E_s)

The slope of the pressuremeter curve in the elastic range is defined as pressuremeter modulus or Menard modulus (E_P or E_M) of the soil. Pressuremeter modulus is commonly used in geotechnical practice for foundation designs because in many cases, the soil or rock shows elastic behavior before the failure conditions.

Expansion of a cylindrical cavity in an infinite elastic medium can be defined from cavity expansion theory (Lame, 1852, cited in Baguelin et al., 1978) as,

$$G = V\ (\Delta P / \Delta V) \qquad \text{(Equation 2.26)}$$

where,

G = shear modulus

V = volume of the cavity

P = pressure in the cavity

Shear modulus can be substituted with Young's modulus by using the equation obtained from theory of elasticity as follows:

$$G = \frac{E_S}{2(1+\nu)} \qquad \text{(Equation 2.27)}$$

The critical parameter in the equation above is the Poisson's ratio (ν) which varies with the type of soil. For practical purposes a value of 0.33 is commonly selected for the Poisson's ratio. However, it is not appropriate to use this value for the undrained behavior of cohesive soils because volume of the soil does not change during the loading. Thus, saturated clay would have a Poisson's ratio of 0.5. Since Menard accepted the ν as 0.33 in his original study, pressuremeter modulus is calculated as:

$$E_M = 2\ (1 + 0.33)\ G \qquad \text{(Equation 2.28)}$$

$$E_M = 2.66\ G \qquad \text{(Equation 2.29)}$$

Even though pressuremeter modulus describes elastic behavior of a soil, it shall be used cautiously for design purposes because of the reasons listed below (Briaud, 1992) :

- Strains on the soil are generally in large ranges which may not be realistic for the real loading conditions.
- Tensile stresses are likely to occur in the circumferential direction during the test. In spite the pressuremeter is a compression test, since the soil is known to be weak under tension; measured modulus is reduced due to tensile stresses.
- Disturbances on the walls of borehole significantly reduce the modulus.
- Aspect ratio (L / D) of the probe have been found to be a factor that can affect the modulus.
- Loading of the soil is relatively fast and in short durations whereas the real superstructure loads act slowly during a larger time span.
- Pressuremeter modulus is a horizontal modulus. For vertical loadings on the soil vertical modulus should be considered which differs from horizontal modulus especially in anisotropic soils.

As a result, pressuremeter modulus can be considered as a relatively low value compared with Young's modulus. Menard (1975) proposed that the pressuremeter modulus should be divided by a correction factor α in order to relate with the Young's modulus (Briaud, 1992). Typical α values proposed by Menard for different types of soil and rock are given in Table 2.8.

Table 2.8 Menard α factors (Briaud, 1992)

Soil type	Peat		Clay		Silt		Sand		Sand and gravel	
	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α	E/p_L^*	α
Over-consolidated			> 16	1	> 14	2/3	> 12	1/2	> 10	1/3
Normally consolidated	For all values	1	9-16	2/3	8-14	1/2	7-12	1/3	6-10	1/4
Weathered and/or remoulded			7-9	1/2		1/2		1/3		1/4
Rock	Extremely fractured α = 1/3			Other α = 1/2			Slightly fractured or extremely weathered α = 2/3			

2.5.5.2.3. Comparison with SPT results

Estimations of pressuremeter parameters, p_{ln} and E_p, from SPT have been studied by many researchers in the past (Briaud, 1992 & Ohya et al., 1982). Attempted correlations have been usually weak because of the differences in the methods and uncertainties involved in the tests. Even so, they are widely used in practice to get an idea about the level of the geotechnical parameters used in the design. Immense database of the SPT makes correlations possible with pressuremeter tests for almost all types of soil. Briaud (1992) suggested rough estimations regarding the pressuremeter modulus and net limit pressure from SPT N value for sands as given below:

$$p_{ln}\ (p_l^*)\ (\text{MPa}) = 0.479\ N \qquad \text{(Equation 2.30)}$$

$$E_M\ (E_0)\ (\text{MPa}) = 0.383\ N \qquad \text{(Equation 2.31)}$$

Briaud mentioned that the scatter in the data is considerably large which makes the correlations essentially useless in design (Figure 2.19, Figure 2.20).

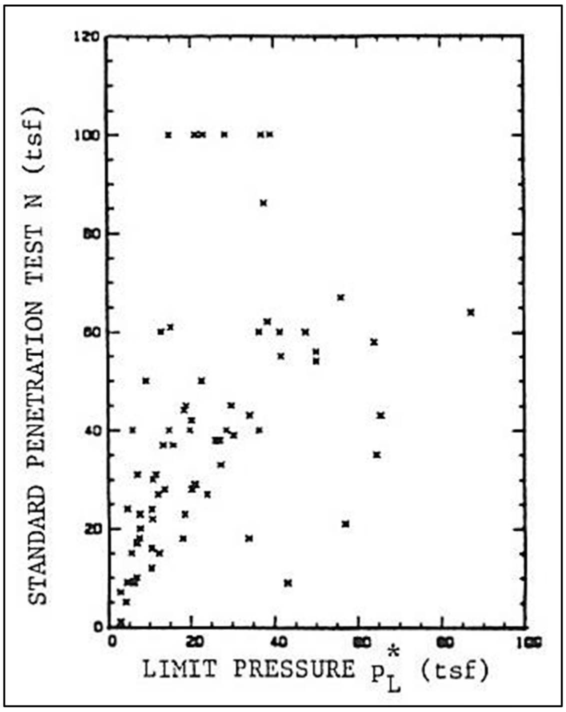

Figure 2.19 Correlation between SPT N and net limit pressure (p_{ln}) (Briaud, 1992)

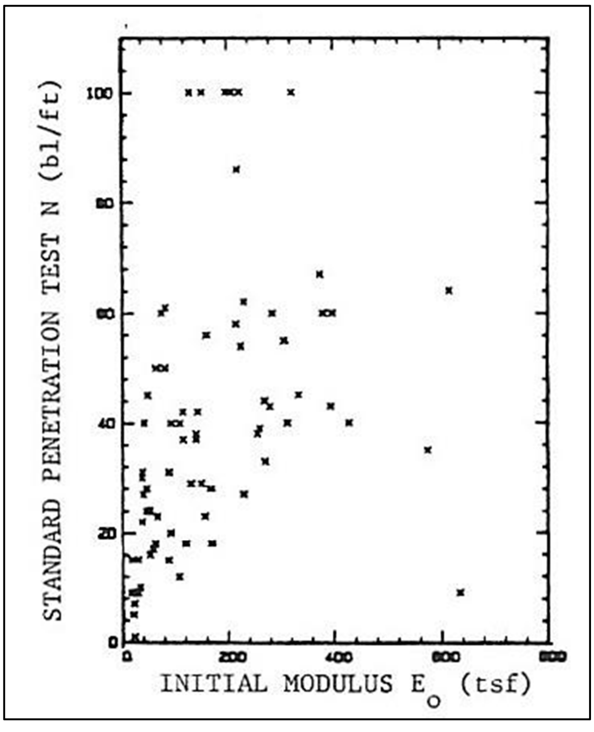

Figure 2.20 Correlation between SPT N and E_p (Briaud, 1992)

Ohya et al. (1982) studied the data obtained from alluvial and dilluvial deposits in Japan. Although the scatter in the data is considerable like the study of Briaud, they found a clear trend for the relationship between SPT N values and pressuremeter modulus (E_p) as given in Figure 2.21 and 2.22.

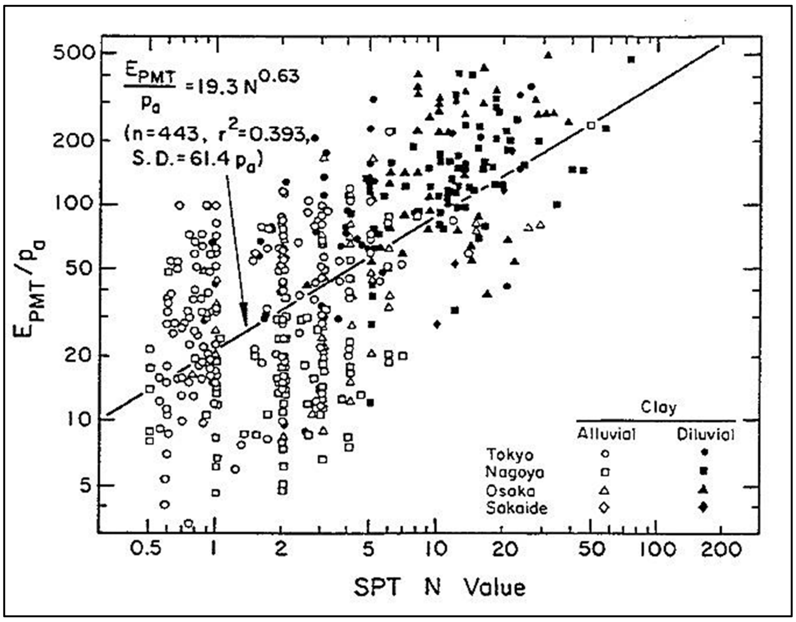

Figure 2.21 Correlation between SPT N and E_p for clays (Ohya et al., 1982)

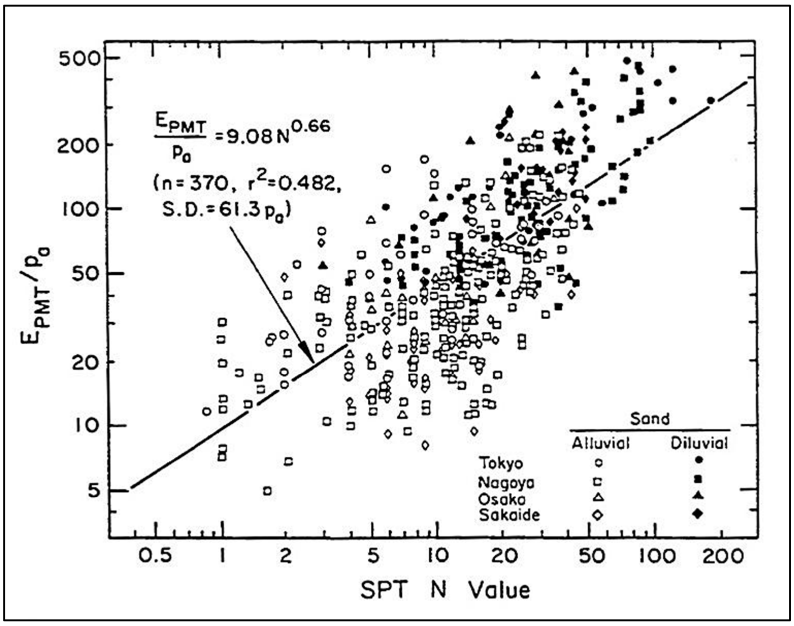

Figure 2.22 Correlation between SPT N and E_p for sands (Ohya et al., 1982)

2.5.6. Flat Dilatometer Test (DMT)

The flat dilatometer test (DMT) was introduced in 1980 by Silvano Marchetti as an alternative in situ test to establish a reliable modulus for the design of laterally loaded piles (Schnaid, 2009). Since then, the test became popular among the practice for being simple, rapid, and economical. Interpretations of the test results are extended to estimate in situ strength and stress history of soils. Comprehensive assessments in the practice were made by Lunne et al. (1989), Lutenegger (1988) and Marchetti himself (1997).

2.5.6.1. Equipment and Test Procedure

Flat dilatometer is 14 mm thick and 95 mm wide steel blade with a 60 mm diameter flexible steel membrane on one of the face of the blade. Dilatometer is generally pushed from the ground surface level to the testing depth with hydraulic pushing equipment like CPT. Generally; blade is pushed to the soil with a constant rate of 1 – 2 cm / sec and test commonly performed for every 20 cm in depth. At the testing level, the steel membrane is inflated by gas and pressure recordings are taken at the starting of the expansion (p_0) and at 1.1 mm deflection of the membrane at center (p_1). A typical dilatometer and its schematic illustration are given in Figure 2.23.

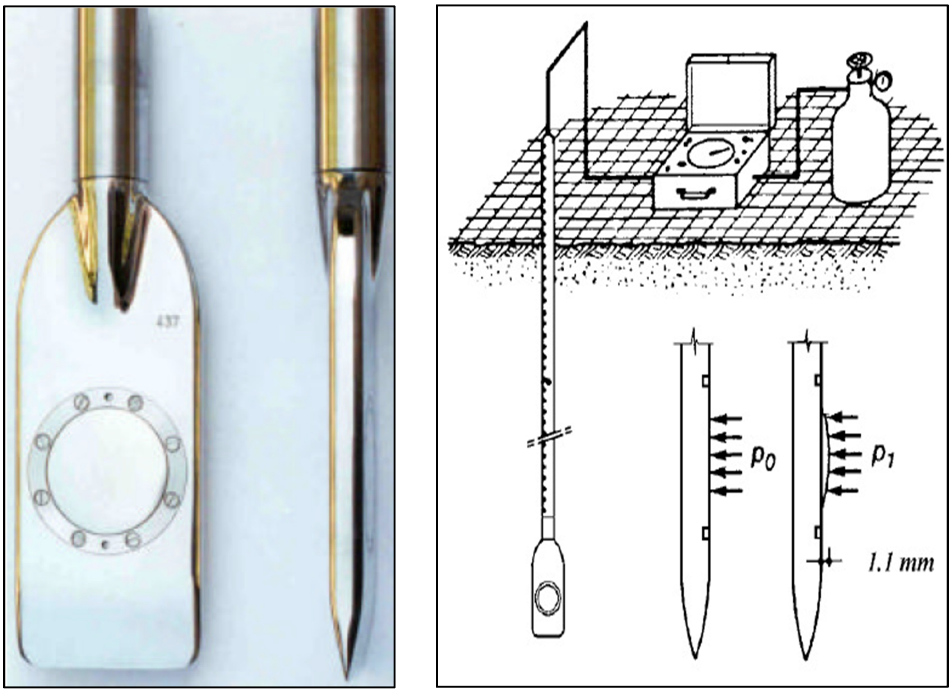

Figure 2.23 Typical flat dilatometer and its schematic illustration (Marchetti et al., 1997)

There are some remarks about the principle of the test indicated by Marchetti (1997) as following:

- Dilatometer test is a deformation controlled test due to imposing of a pre-determined displacement on the soil. Thus, strains are approximately same for al soil types.

- The steel membrane is not a measuring device. The main measuring elements are gauges at the surface level whose accuracy is determined by type or calibrations.

- Although operator effects are limited, qualification of the operator still affects the results.

Regular calibrations for the gauges and the steel membrane are essential operations due to their undeniable effects on the test results (Schnaid, 2009). Stiffness calibration of the membrane consists of two reading corrections related to the membrane stiffness as:

- ΔA = External pressure applied on the membrane in atmospheric pressure to ensure its contact with the soil is perfect.

- ΔB = Internal pressure to expand the center of the membrane by 1.1 mm from its seating position in atmospheric pressure.

ΔA and ΔB values are used for correcting the p_0 and p_1 pressures as following:

$$p_0' = 1.05\,(p_0 - Z_m + \Delta A) - 0.05\,(p_1 - Z_m - \Delta B) \qquad \text{(Equation 2.32)}$$

$$p_1' = (p_1 - Z_m - \Delta B) \qquad \text{(Equation 2.33)}$$

where,

p_0' and p_1' = corrected p_0 and p_1 readings

Z_m = Gauge zero offset under atmospheric pressure.

Parameters obtained from the dilatometer test are defined as the following:

- **Material Index (Id):** Material index is defined as:

$$I_D = \frac{p_1 - p_0}{p_0 - u_0} \qquad \text{(Equation 2.34)}$$

where,

u_0 = hydrostatic pore water pressure

Material index is generally used for identification of soil layers. Marchetti (1997) stated that material index can be interpreted as a "rigidity index" which reflects the mechanical behavior of the soil. However, for a mixture of cohesive and cohesionless soil, it may give misleading results.

- **Horizontal stress index (K_D):** Horizontal stress index is commonly regarded as an amplified value of K_0 by the penetration (Marchetti, 1997). It gives an idea about the stress history of soil with depth. K_D defined as:

$$K_D = \frac{p_0 - u_0}{\sigma_{v0}'} \qquad \text{(Equation 2.35)}$$

where,

$\sigma_{vo'}$ = in situ vertical effective stress

u_0 = hydrostatic pore water pressure

- **Dilatometer Modulus (E_d):** Dilatometer modulus is obtained from p_0 and p_1 pressures by using the theory of elasticity by:

$$\Delta d = \frac{2D\,(p_1 - p_o)}{\pi} \left(\frac{1-\nu^2}{E}\right) \qquad \text{(Equation 2.36)}$$

then for Δd = 1.1 mm and D = 60 mm;

$$E_d = \frac{E}{1-\nu^2} = 34.7\,(p_1 - p_0) \qquad \text{(Equation 2.37)}$$

Marchetti et al. (1997) stated that E_d shall be used with K_D and I_D because the stress history effects are not represented appropriately in the definition and it should not be used directly as the Young's modulus without considering the Poisson's ratio.

2.5.6.2. Flat Dilatometer Correlations

Flat dilatometer test (DMT) results can be interpreted to shear strength and deformation characteristics of cohesive soils. In addition, there are also some studies in the literature that correlate the DMT with other in situ tests to convert the DMT results into commonly used parameters like SPT N or pressuremeter modulus, E_p. In the scope of this study, cross correlations between DMT – SPT and DMT – PMT are investigated rather than the estimations about undrained shear strength and compressibility.

2.5.6.2.1. Correlations with SPT

A generalized correlation between SPT and DMT has not been presented yet. According to Schmertmann & Crapps (1988) the estimation of SPT N from dilatometer test would be a total misuse of the dilatometer data (Marchetti et al., 1997). On the other hand, there are some local correlations present in the literature. Nonetheless, Marchetti et al. (1997) clearly mentions that, such correlations depend on soil type, site specific and should always be confirmed before use in design.

One of the local correlations is proposed by Mayne & Frost (1989) for sandy silts of the Piedmont geologic province in United States with using SPT, dilatometer modulus (E_d) and the secant moduli from back calculation method (Kulhawy & Mayne, 1990) (Figure 2.24).

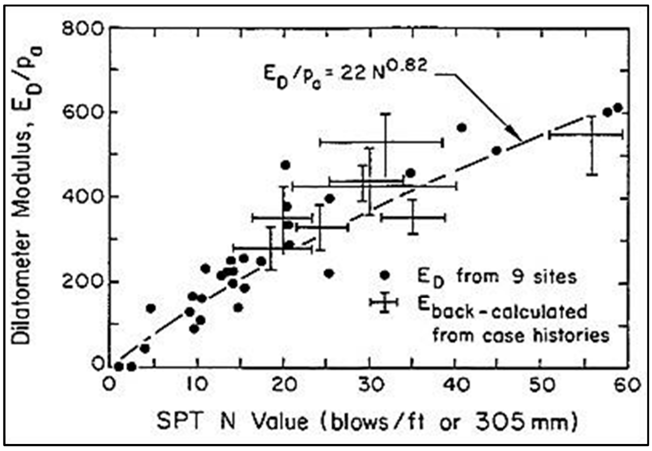

Figure 2.24 Correlation between E_d and SPT N in Piedmont sandy silts (Mayne & Frost, 1989 cited in Kulhawy & Mayne, 1990)

Another study about the correlation of SPT and DMT was performed by Tanaka & Tanaka (1998) by using a database of three sandy soil sites in Japan. According to the result of the study a relatively good correlation is established as given in Figure 2.25.

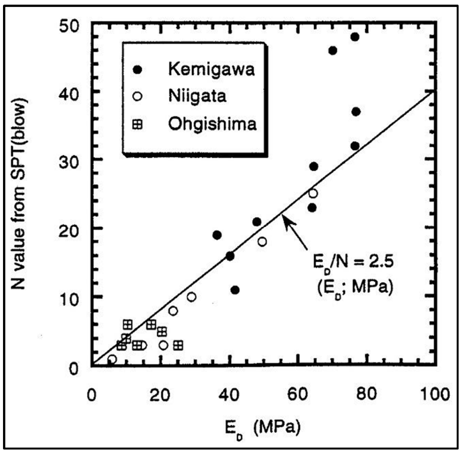

Figure 2.25 Correlation between E_d and SPT N for alluvial sands in Japan (Tanaka & Tanaka, 1998)

2.5.6.2.2. Correlations with Pressuremeter Test

Due to the similarities in the method between pressuremeter test and dilatometer test, some cross correlations exists in the literature. Campanella et al. (1985) reported that the pressure applied during dilatometer test may be enough to reach the yield pressure value in soft soils. Yet, due to duration of the test and location of the membrane on the blade, the pressure applied by dilatometer may actually be less than yield pressure (Lutenegger, 1988). Usually, correlations between pressumeter modulus (E_p) and dilatometer modulus (E_d) are the most interested studies in the literature. Because, the main concern in many designs is about the deformability characteristics of soils rather than the capacity.

There are a limited number of published studies in the literature about the possible relationship of E_d and E_p. Kalteziotis et al. (1991) suggested a general correlation that pressuremeter modulus (E_p) and dilatometer modulus (E_d) can be related approximately as (Marchetti et al., 1997).

$$E_p \approx 0.4\ E_d \qquad \text{(Equation 2.38)}$$

On the other hand, Lutenegger (1988) used the data of Powell & Uglow's study (1986) to compare pressumeter modulus (E_p) and dilatometer modulus (E_d) in clayey soils as given in Figure 2.26. Lutenegger (1988) mentioned that, the scatter of the data is significant which may be the result of borehole disturbances prior to pressuremeter test.

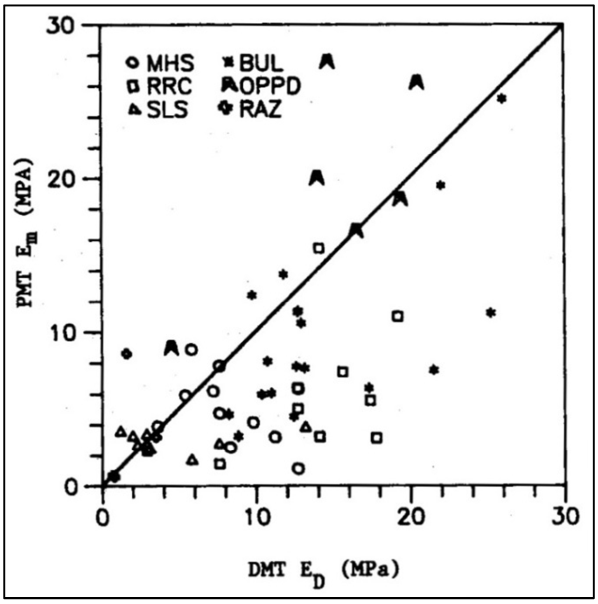

Figure 2.26 Comparison of E_p and E_d in clayey soils (Lutenegger, 1988)

CHAPTER 3

REVIEWED SITE INVESTIGATION STUDIES

3.1. General

In order to study the viability of existing correlations between in situ tests and geotechnical parameters of soils, in situ test data from five different site investigations in Turkey and Europe are compiled along with laboratory test results. Site investigation database is established from the following projects:

- Eurostar Closed Cycle Power Plant Project / Kırklareli / Turkey
- Bursa Ring Road Project / Turkey
- İzmir Port Rehabilitation Project / Turkey
- Subway II Line Project / Warsaw / Poland
- Braila Wastewater Treatment Plant Project / Romania

In the content of site investigation works for the listed projects, a total number of 169 borehole drillings were performed at the project sites. Scope, methodology, selection of in situ and laboratory tests, show differences for each project in the database due to purpose of designs, local practice applications, contract terms and varieties in equipment. SPT, CPT, PMT and DMT results from site investigation works are mainly analyzed throughout the study. Along with the in situ tests, laboratory tests including sieve analyses, Atterberg limits, oedometer, unconfined compression (UC), unconsolidated undrained (UU) and consolidated undrained (CU) triaxial test results are evaluated. Soil types and characteristics of each site vary due to difference in geological formations but in order to deal with a narrower database, analyses are limited to cohesive soils.

Soil profiles of the reviewed projects are mainly composed of inorganic clays of medium to low plasticity (CL) and high plasticity (CH) according to Unified Soil Classification System (USCS). Along with CL and CH type clays, clayey sands (SC), silty sands (SM), poorly graded sands with fines (SP), silts of low plasticity (ML) and silts of high plasticity (MH) are also encountered and some of them are included in the analyses considering the cohesive behavior of these types of soils.

In order to give a general idea about the soil profile, distribution of soil types, plasticity indexes and in situ test results in the entire database are presented in Figures 3.1, 3.2, 3.3, 3.4, 3.5, 3.6 and 3.7 and the ranges are summarized as below.

SPT N	$4 \leq N \leq 88$
Net limit pressure (p_{ln})	73 kPa $\leq p_{ln} \leq 14870$ kPa
Pressuremeter modulus (E_p)	1000 kPa $\leq E_p \leq 197200$ kPa
Cone tip resistance (q_c)	200 kPa $\leq q_c \leq 9800$ kPa
Flat Dilatometer modulus (E_p)	5760 kPa $\leq E_p \leq 99500$ kPa
Soil classification (USCS)	CL – CH, occasionally
	SC, SM, SP,ML and MH
Plasticity index	$6 \leq$ PI (%) ≤ 120
Undrained shear strength (c_u)	13.5 kPa $\leq c_u \leq 511.6$ kPa
Coefficient of volume compressibility (m_v)	$0.2\text{x}10^{-4} \leq m_v \leq 8.1\text{x}10^{-4}$ m^2/kN

Although the soil classifications of each site are similar, in situ and laboratory test results indicate a wide range for the mechanical properties. Detailed soil characterizations for each site are evaluated in the following sections.

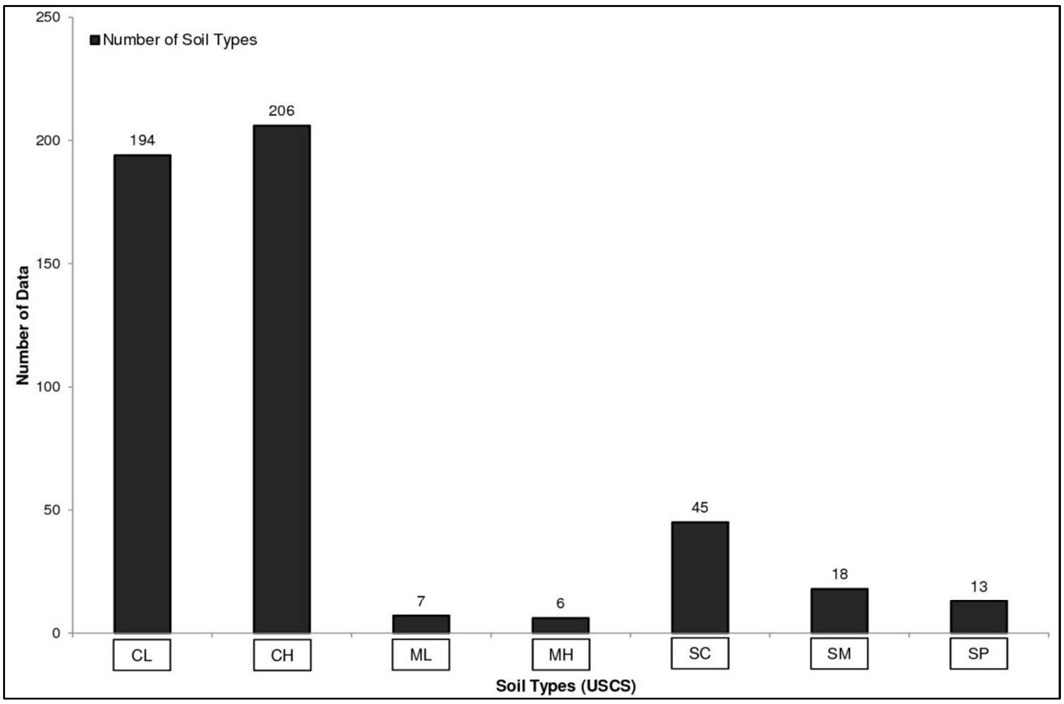

Figure 3.1 Soil type (USCS) distribution of the database

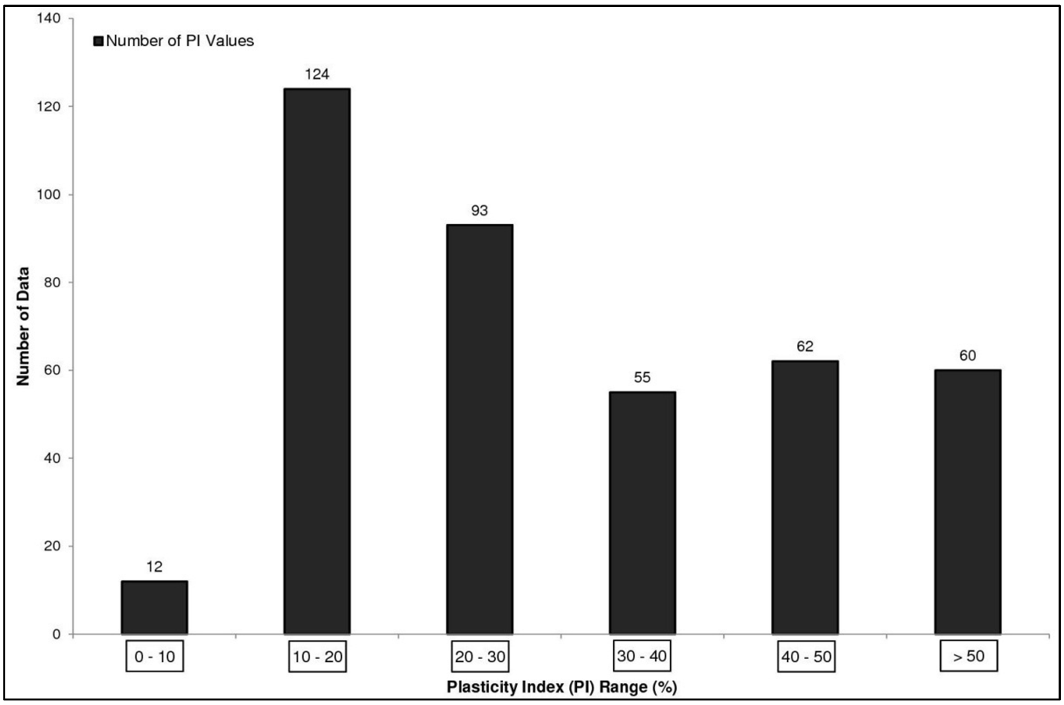

Figure 3.2 Plasticity Index (PI) distribution of the database

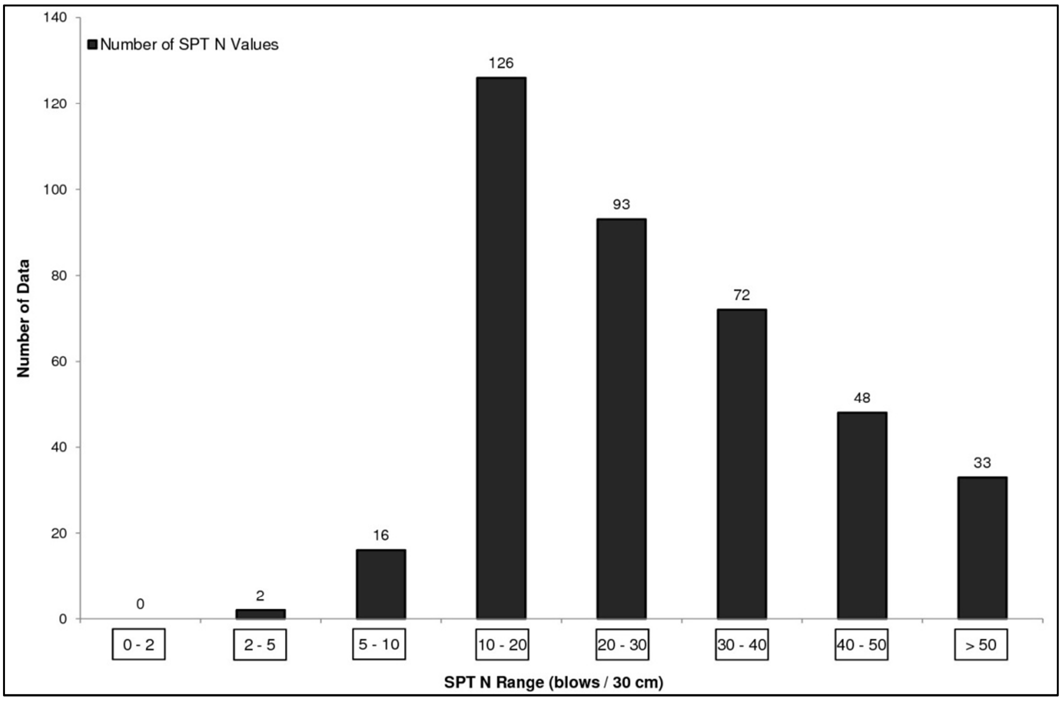

Figure 3.3 SPT N distribution of the database

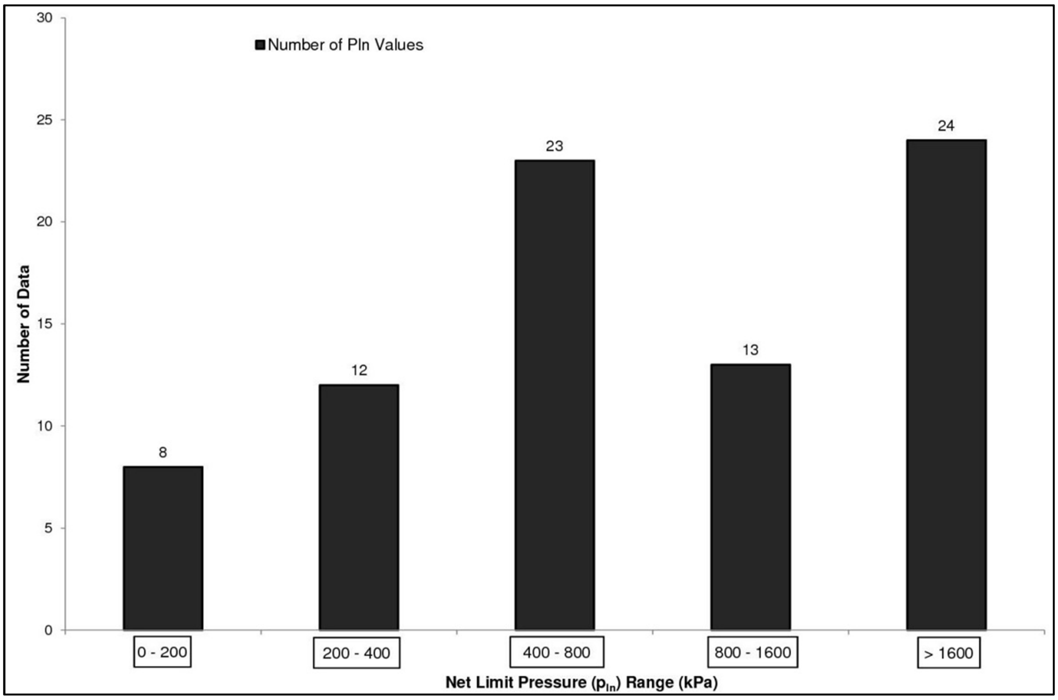

Figure 3.4 Net limit pressure (p_{ln}) distribution of the database

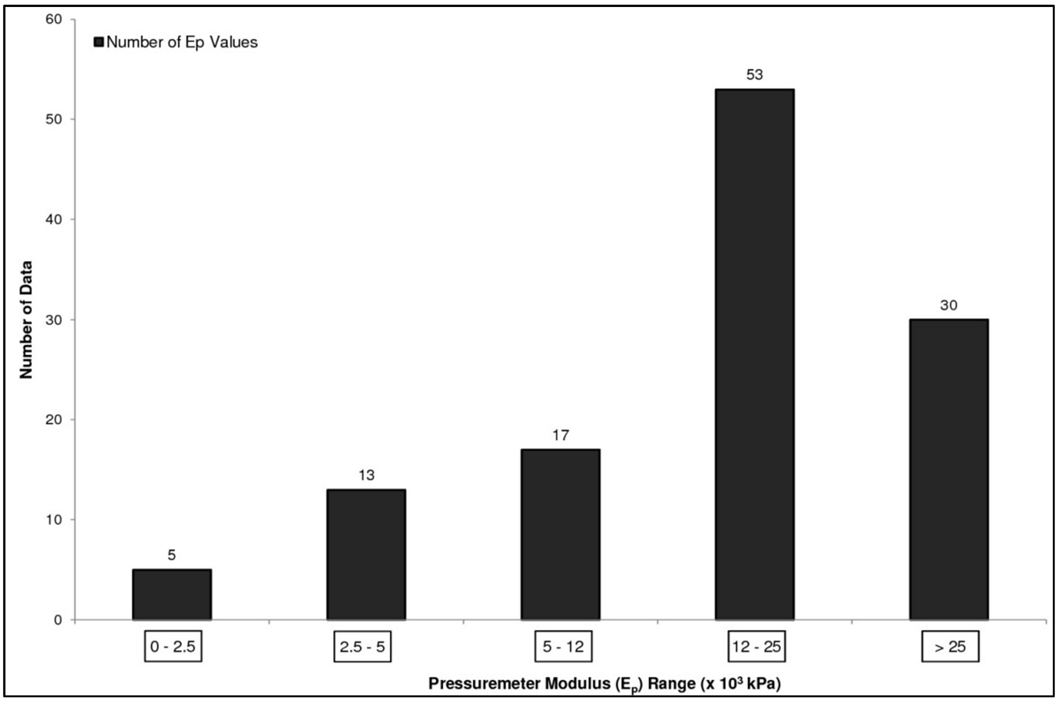

Figure 3.5 Pressuremeter modulus (E_p) distribution of the database

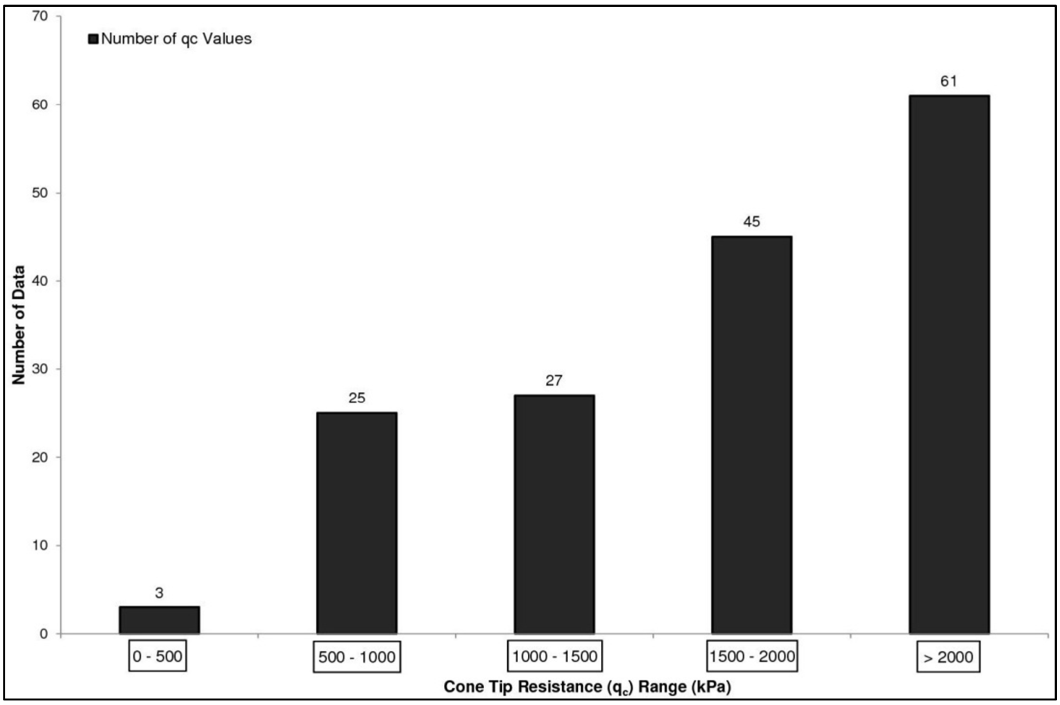

Figure 3.6 Cone tip resistance (q_c) distribution of the database

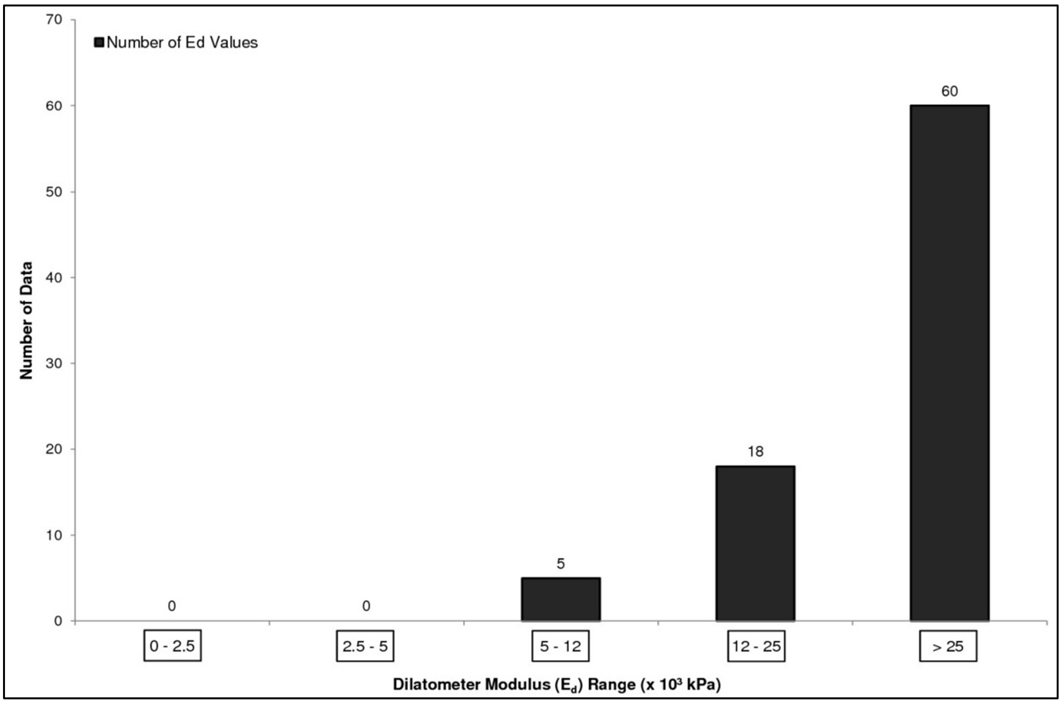

Figure 3.7 Dilatometer modulus (E_d) distribution of the database

3.2. Eurostar Natural Gas Combined Cycle Plant Project, Kırklareli / Turkey

Natural gas combined cycle plant project (CCPP) that is constructed in Kırklareli city, Erikleryurdu village have a capacity of 880 MW with high efficiency technology reducing the emissions. Power plant will be constructed on an area of approximately 150.000 m^2. Project site is at 21 kilometers south of the Kırklareli city and 3.5 kilometers southeast of the Erikleryurdu village. The location of the project is given in Figure 3.8.

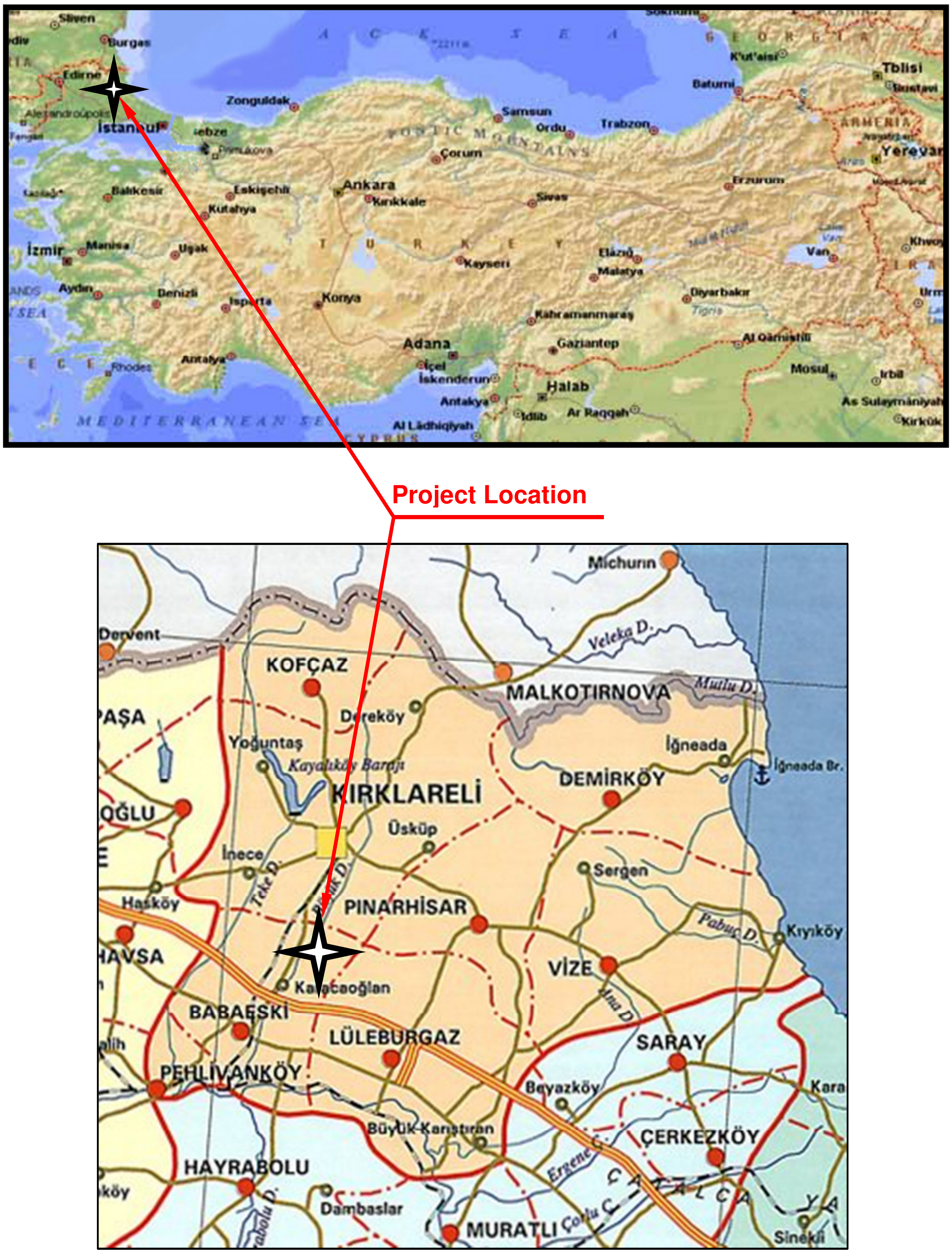

Figure 3.8 Location map of the Eurostar CCPP

Site investigation program for power plant project consisted of 35 borehole drillings with a total depth of 1070 meters and were performed in 2011. Standard penetration tests were carried out in every borehole whereas; pressuremeter tests are conducted only in 15 boreholes out of 35. Disturbed and undisturbed samples were taken during drilling works to be tested in laboratory Field studies were conducted based on the “Technical Specification of Foundation Borings for Structures” published by Turkish Ministry of Public Works.

Standard penetration tests (SPT) were performed systematically at 1.50 meters depth intervals in all boreholes. SPT N values were recorded on borehole logs for each test and disturbed samples from SPT were tested in laboratory to determine the grain size distributions and index properties of soils. Along with SPT, pressuremeter tests (PMT) were performed along the depth of boreholes at every 3.0 meters intervals by a LouisMenard GA type probe.

Based on the site investigations and field surveys, site is determined to be in Pliocene aged Trakya Formation. Materials in the formation fed up Istıranca Massive and developed on Middle – Upper Miocene aged Ergene formation which is alluvial fan – alluvial deposits. Formation starts with little gravelly clayey sand at higher elevations and continued with clay – silt sized units in the plains. The main units in the site consists of brown – reddish brown colored clay or sandy clay type soils. Through the depth, gray colored clays are also observable. Through the clay, clayey sand - silty sand – sand lenses are present. Occasionally, lenses have lateral and vertical connections with each other and thickness of the lenses vary between 1 – 2 meters to 10 – 12 meters. In order to give a visual example of the soil profile in the site, photographs of excavated trial pits are presented in Figures 3.9 and 3.10.

Groundwater level at the site is observed to be about 18.0 meters below the surface. Flow of the groundwater in clayey sand lenses is very slow. However, during the rainy days of the year, surface water can leach in to the ground and can be hold by upper sandy lenses in clay creating a temporary groundwater table.

Alluvial deposits in the project area are mainly formed as CL and SC type soils according to USCS. Stiffness of the clays tends to increase with depth and they are generally stiff to hard in consistency. Clayey sand layers on the other hand, are generally medium dense – dense fine grained sized sands. Fine content of clayey sands are determined to be 15 – 50 % with a representative value of 30 %. Thus, clayey sands (SC) were also considered throughout the analyses.

Figure 3.9 General view of soil layers in the project area (near BH33)

Figure 3.10 General view of soil layer in the project area (near BH31)

Plasticity index values of the clays are generally low with a narrow range of 10 – 30 %. This can be considered as a result of being mixed with non plastic to low plasticity clayey sands. Range distribution of the soil types and plasticity index values for studied cohesive soils are given in Figures 3.11 and 3.12.

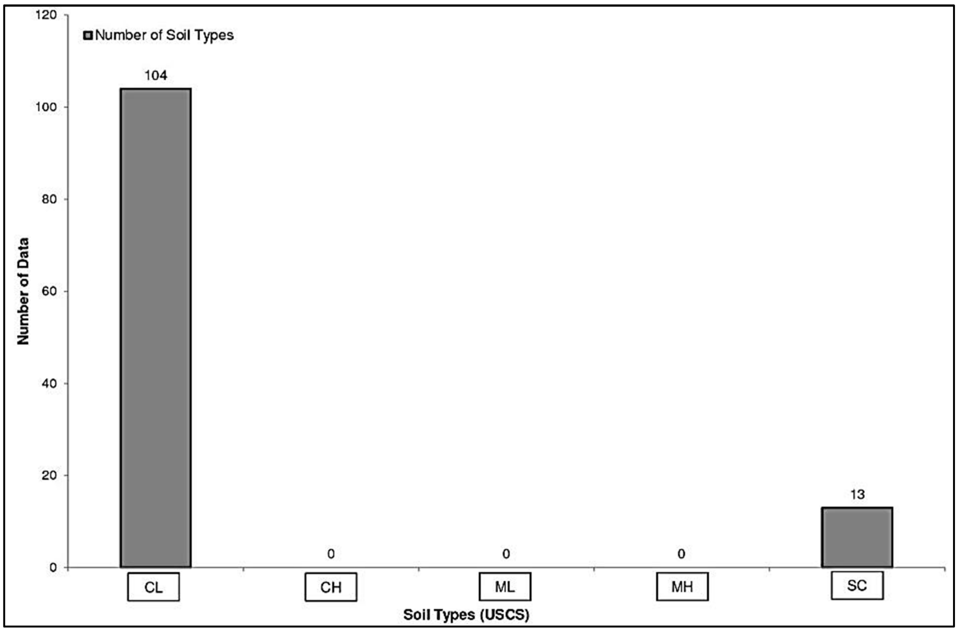

Figure 3.11 Soil type distributions of the alluvial deposits

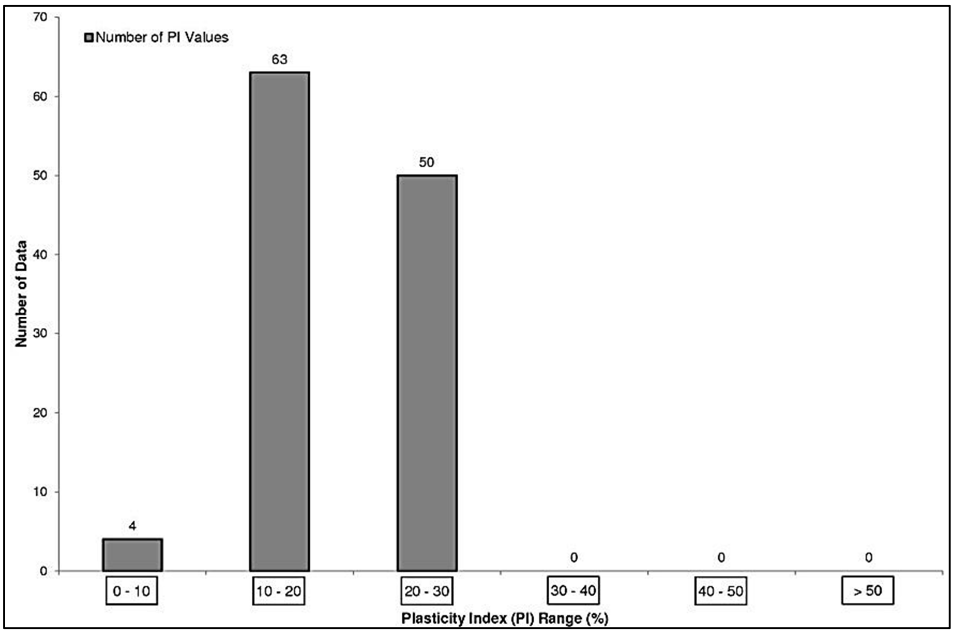

Figure 3.12 Plasticity index (PI) distributions of the alluvial deposits

Laboratory testing part of the project involved unconfined compression (UC), unconsolidated undrained triaxial (UU) and oedometer tests on CL and partly SC type of soils. Undrained shear strength (c_u) and coefficient of volume compressibility (m_v) values are determined from these tests for which the distributions are given in Figure 3.13 and 3.14. Undrained shear strength (c_u) values are mostly larger than 100 kPa and coefficient of volume compressibility (m_v) values vary between 0.5 x 10^{-4} – 2 x 10^{-4} m^2 / kN.

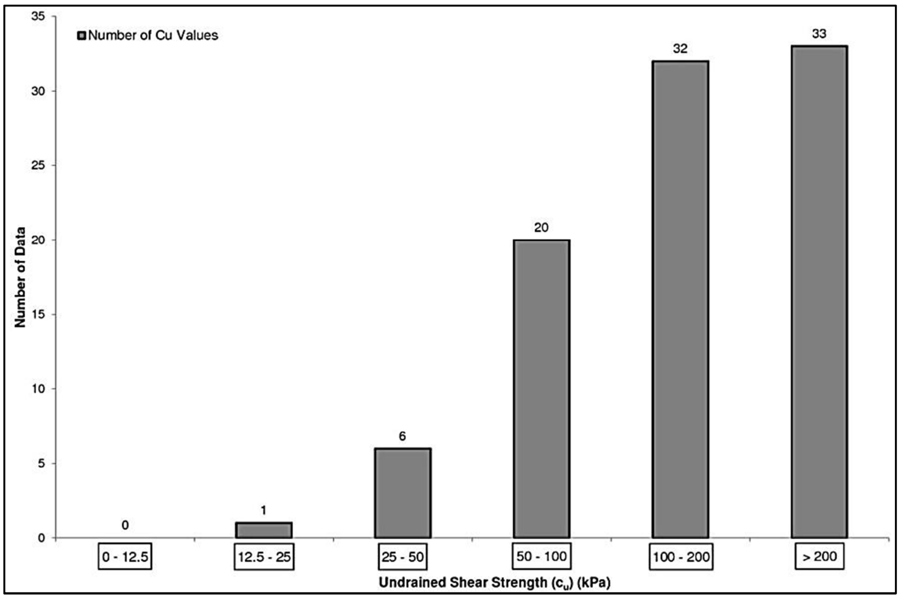

Figure 3.13 Undrained shear strength (c_u) distributions of alluvial deposits

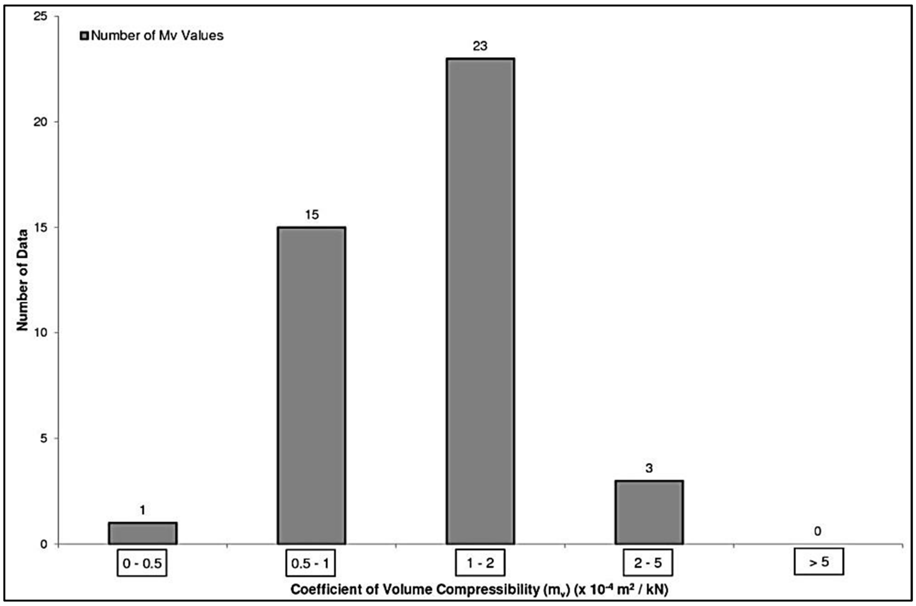

Figure 3.14 Coefficient of volume compressibility (m_v) distributions of alluvial deposits

As for the in situ tests, results of the SPT and pressuremeter tests are presented by distributions of the SPT N, net limit pressure (p_{ln}) and pressuremeter modulus (E_p) in Figures 3.15, 3.16 and 3.17.

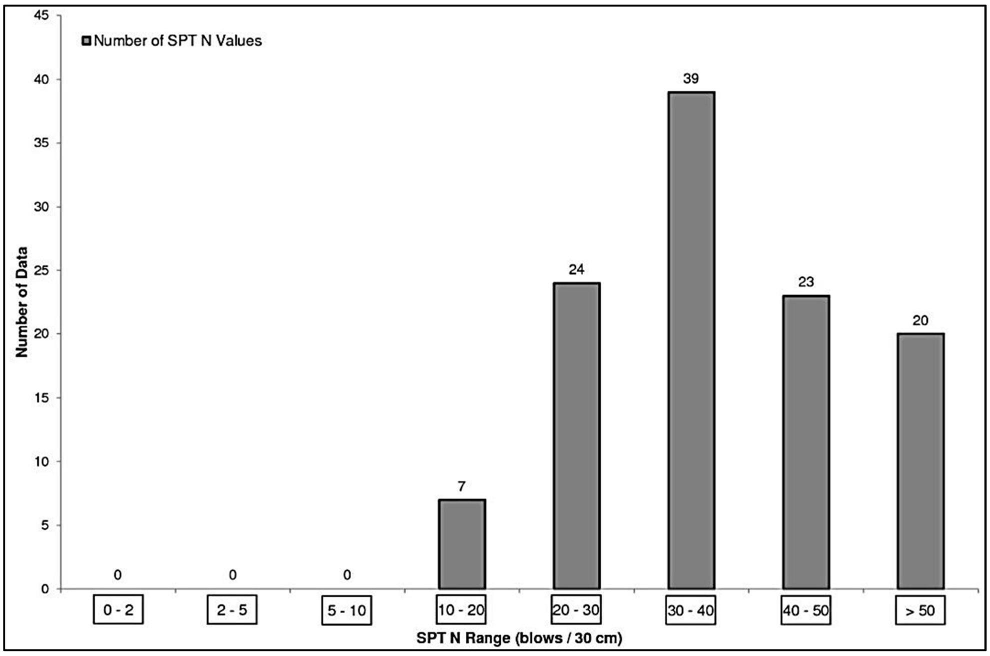

Figure 3.15 SPT N distributions in alluvial deposits

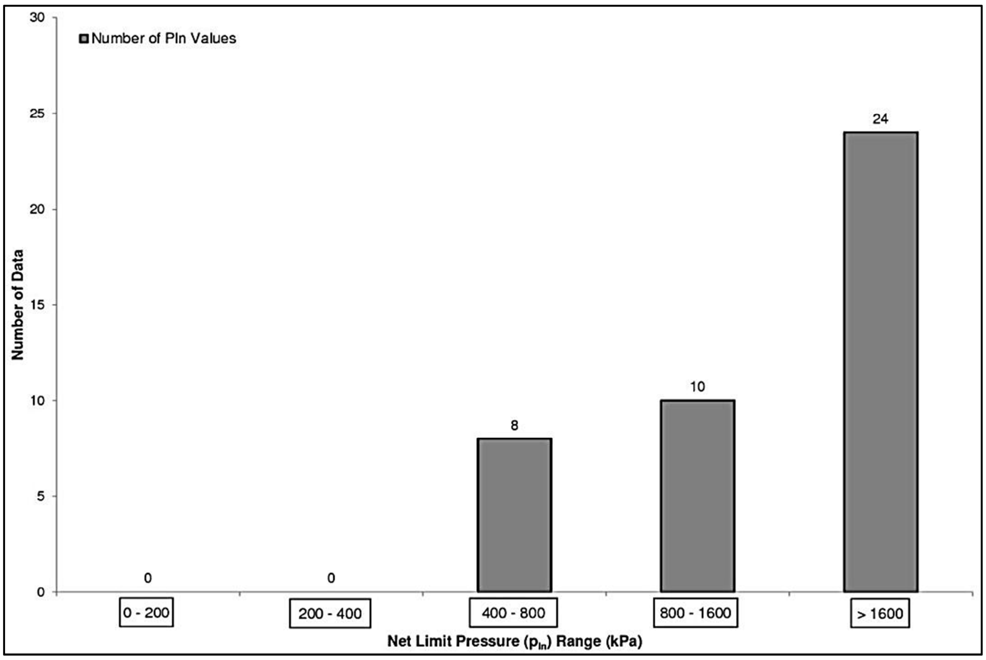

Figure 3.16 Net limit pressure (p_{ln}) distributions in alluvial deposits

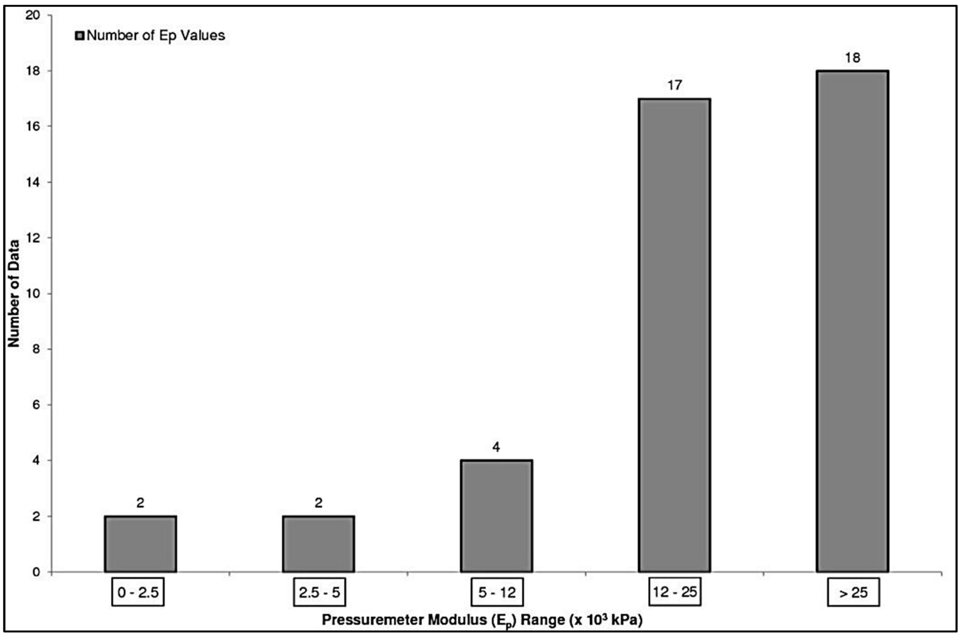

Figure 3.17 Pressuremeter modulus (E_p) distributions in alluvial deposits

SPT N values show a wide variation from 10 – 90. Some refusal (R) values were also recorded throughout the tests. SPT results indicate that the soil profile is consisted of stiff to hard clays in terms of consistency according to the study of Terzaghi & Peck (1976) given in Table 2.1.

On the other hand, net limit pressure (p_{ln}) values are between 400 – 4000 kPa mostly larger than 1600 kPa whereas; pressuremeter modulus (E_p) values ranges between 1500 – 50000 kPa mostly larger than 12000 kPa for the soils. As a result, alluvial deposits can be clearly identified as very stiff to hard by comparing the values in the Table 2.6 given by Briaud (1992).

3.3. Bursa Ring Road Project / Turkey

The aim of Bursa ring road project was to connect the existing İstanbul – İzmir highway with Bursa – Ankara state road by a ring road at the east of Bursa. In the scope of the project, a link road with a total length of 6.34 kilometers was designed from the chainage at 10+325 of existing highway to Samanlı crossing road which leads to Ankara. Planned road will be established on high embankments along the route. Location of the project route is given in Figure 3.18.

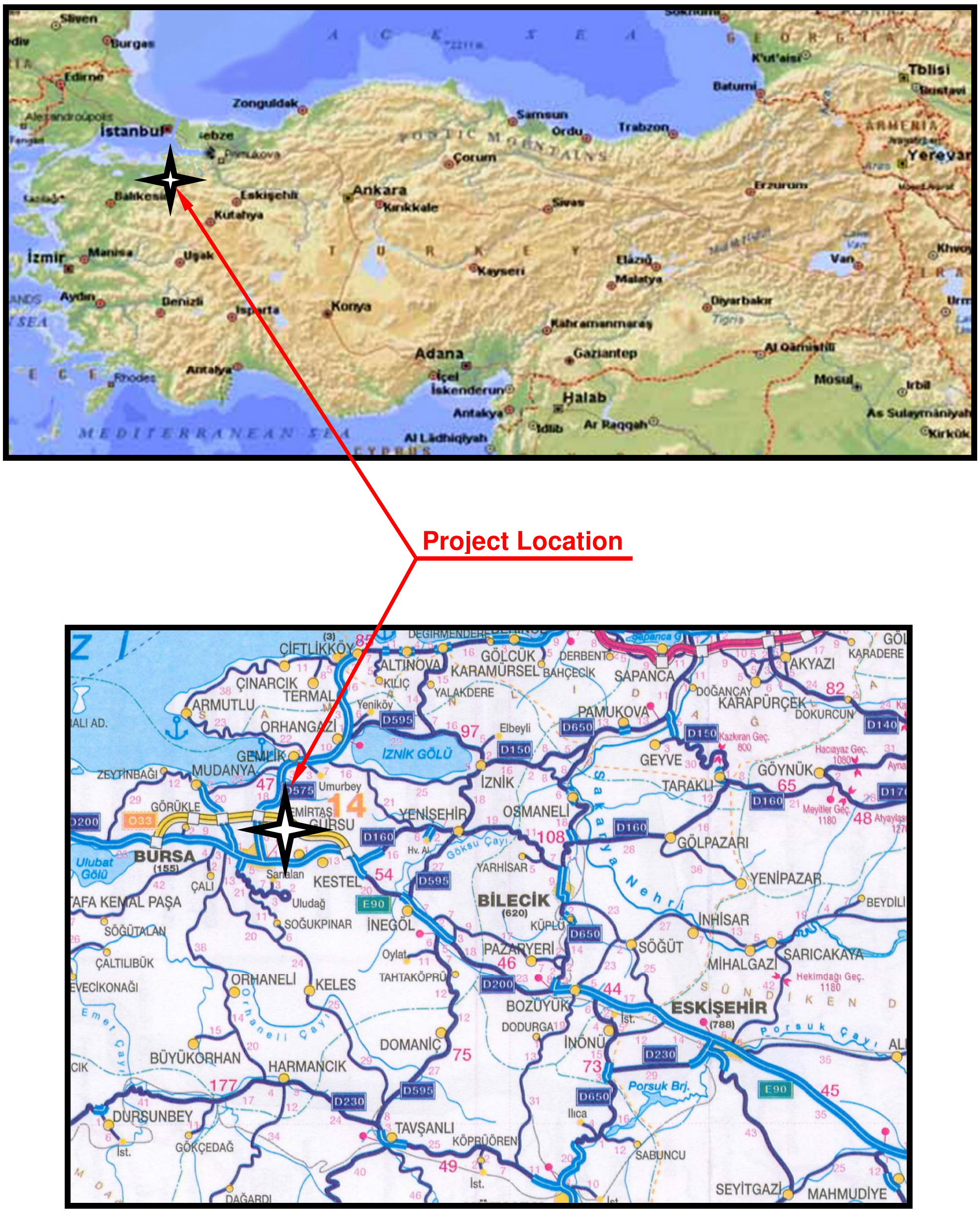

Figure 3.18 Location map of the Bursa ring road project

In order to investigate the soil conditions for the geotechnical design of embankments, site exploration programs were carried out in 2009 and 2010. In the content of site investigation program, 28 borehole drillings of a total length of 830.93 meters are performed together with standard penetration and pressuremeter tests. Disturbed and undisturbed samples were taken from each borehole to be tested in laboratory. Borehole drillings and in situ tests are executed according to “Technical Specifications of Investigation and Engineering Services” published by Turkish General Directorate of Highways in 2005.

Standard penetration tests (SPT) were performed in each borehole at every 1.50 meters depth interval to estimate the consistency of soil layers. Results of SPT were recorded on borehole logs. Representative disturbed samples were obtained during the SPT to determine the grain size distributions and index properties of soils. Beside the SPT, pressuremeter tests (PMT) were also performed along the depth of boreholes with LouisMenard GA type probe.

By using the results of site investigation works and previous geological mappings, main geological formation is identified as alluvial deposits which comprise about 90 % of the route. Alluvial deposits over the Bursa province belong to Quaternary period of Cenezoic era in geological time scale. They are formed mainly by transportation of eroded rock formations through the geological time. Alluvial units covering the area show differences in terms of formation process and they have an active interaction with current tectonics, surface – ground water flow and physical – chemical weathering effects. The streams in the area generally forms meanders due to low graded topography. Therefore, flow energy can only drag fine grained materials (i.e. clays, silts and fine sands) in the suspension which has an important role on the formation of thick fine grained layers during sedimentation. A general view of the project area is given in Figure 3.19.

Ground water flow and aquifer networks are present throughout the alluvial deposits which causes the layers or lenses of well – poorly graded fine materials in various depths and thicknesses. Groundwater level records were kept in boreholes which are determined to be very close to the surface.

Alluvial deposits in this area reach up to 40 meters in depth and can be classified in to two sub categories as fine grained and coarse grained. Fine grained alluviums are generally made of brown to greenish grey, medium to hard clayey silts and silty clays. In addition, some clayey – silty sand layer are observed occasionally.

Coarse grained alluviums at the project site are usually encountered as thin repetitive bands or lenses among fine grained units. They are mostly made of yellow – brown and greenish grey, medium dense to dense silty sand and clayey sands with occasional gravels. Due to difficulties in performance of laboratory unconfined compression, triaxial and oedometer tests, coarse grained alluviums are only analyzed for correlations between in situ tests in the content of this study.

According to USCS, fine grained alluvium deposits are classified mostly as CL and CH type clays. A small percentage of fine grained alluviums can be identified as ML and MH type of silts which were also taken into consideration throughout the analyses.

Figure 3.19 General view of the Bursa ring road project

Fine grained alluviums vary in a wide range in terms of plasticity. Soil layers near to surface generally contain highly plastic clays but with the increase in depth, clays of low to medium plasticity are encountered. Low plasticity clays are generally mixed with silts and sands that have lower fine content values. Atterberg limit test results for fine grained alluviums indicate a wide plasticity index (PI) range of 3 – 44 % with an approximate mean value of 20 %. Range distribution of the soil types and plasticity index values for fine grained alluviums are given in Figure 3.20 and 3.21.

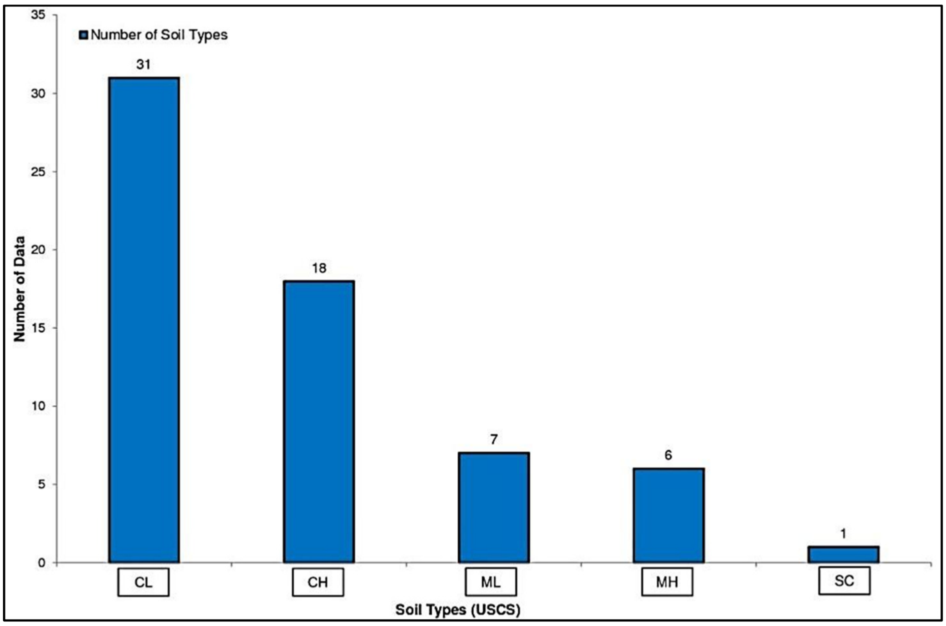

Figure 3.20 Soil type distributions of the fine grained alluviums

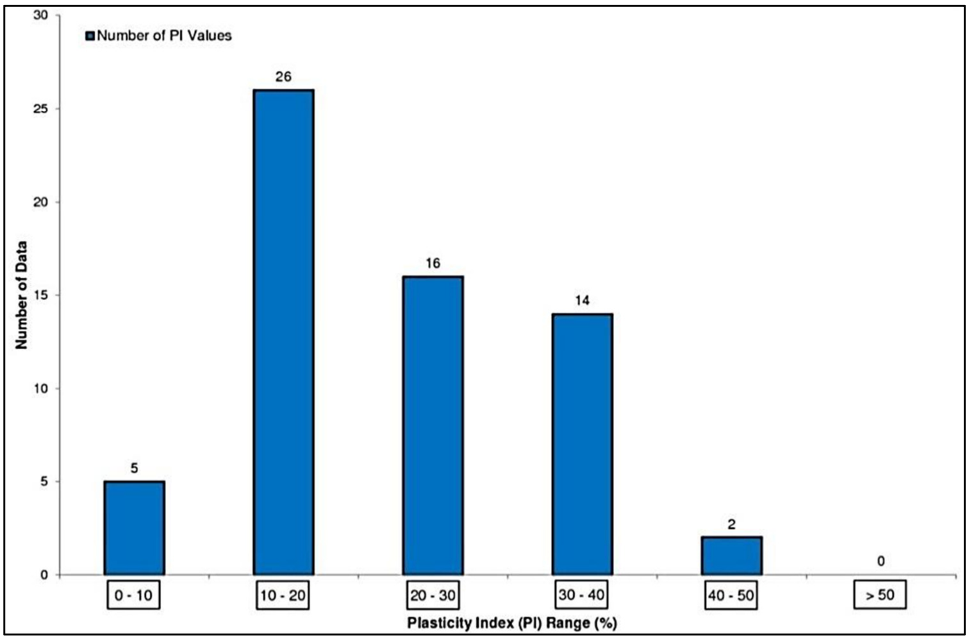

Figure 3.21 Plasticity index (PI) distributions of fine grained alluviums

In the content of laboratory testing, unconfined compression (UC), unconsolidated undrained triaxial (UU) and oedometer tests were conducted to determine the undrained shear strength (c_u) coefficient of volume compressibility (m_v) respectively. Results of the laboratory tests in terms of range distribution are given in Figure 3.22 and 3.23. Undrained shear strength (c_u) values are generally between 25 – 100 kPa and coefficient of volume compressibility (m_v) values are generally in the range of 1 x 10 $^{-4}$ – 5 x 10^{-4} m^2 / kN for fine grained alluviums.

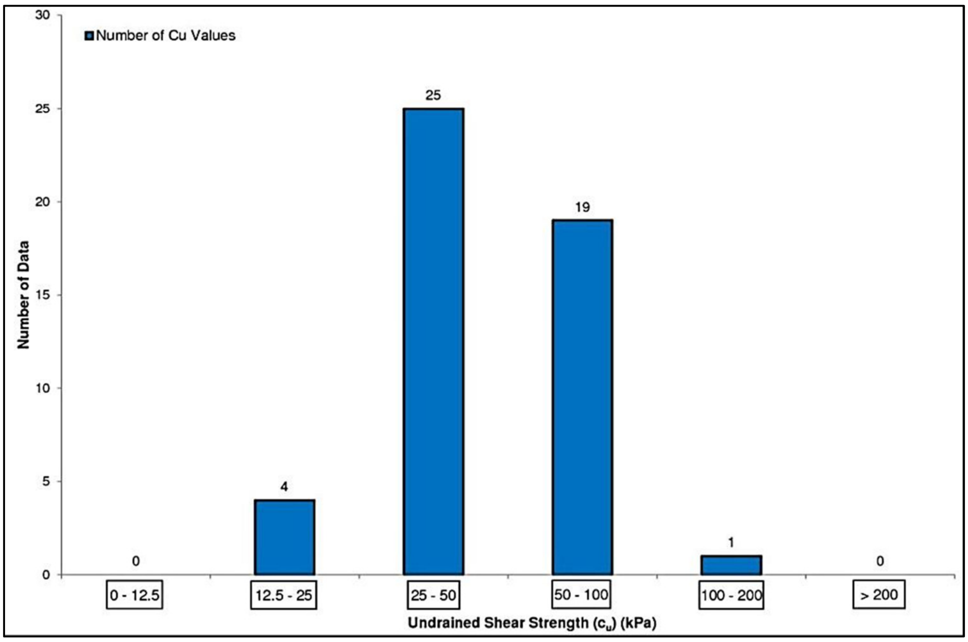

Figure 3.22 Undrained shear strength (c_u) distributions of fine grained alluviums

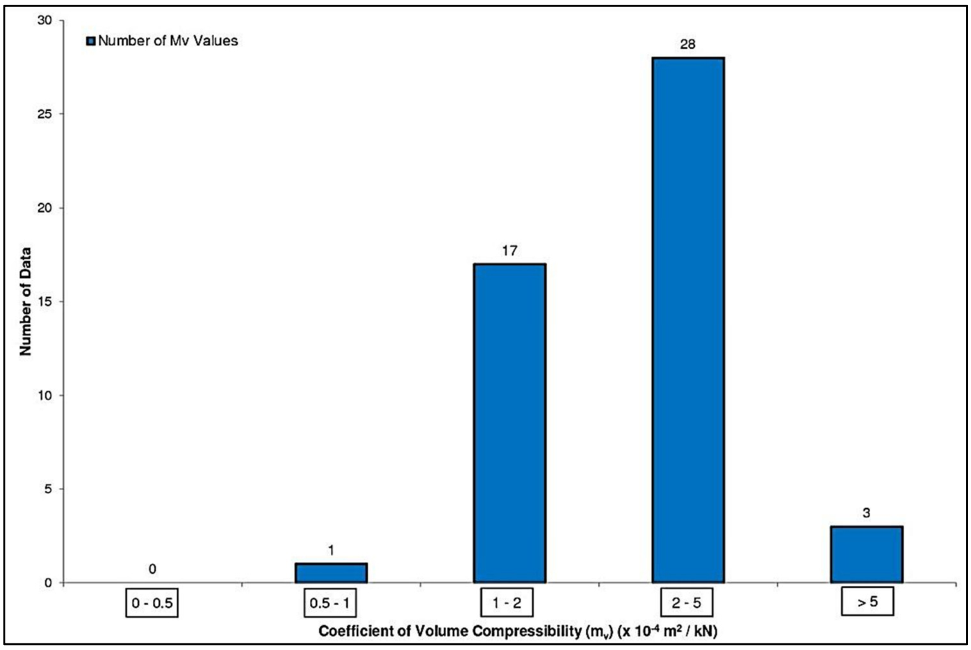

Figure 3.23 Coefficient of volume compressibility (m_v) distributions of fine grained alluviums

On the other hand, in situ test results regarding the SPT N number, net limit pressure (p_{ln}) and pressuremeter modulus (E_p) are presented in Figures 3.24, 3.25 and 3.26

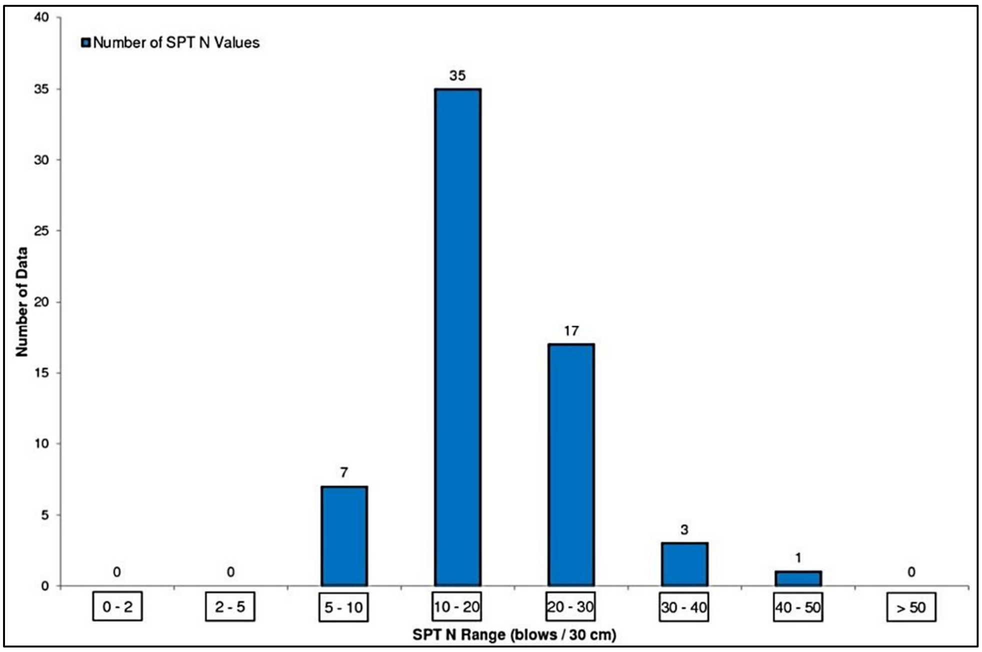

Figure 3.24 SPT N distributions in fine grained alluviums

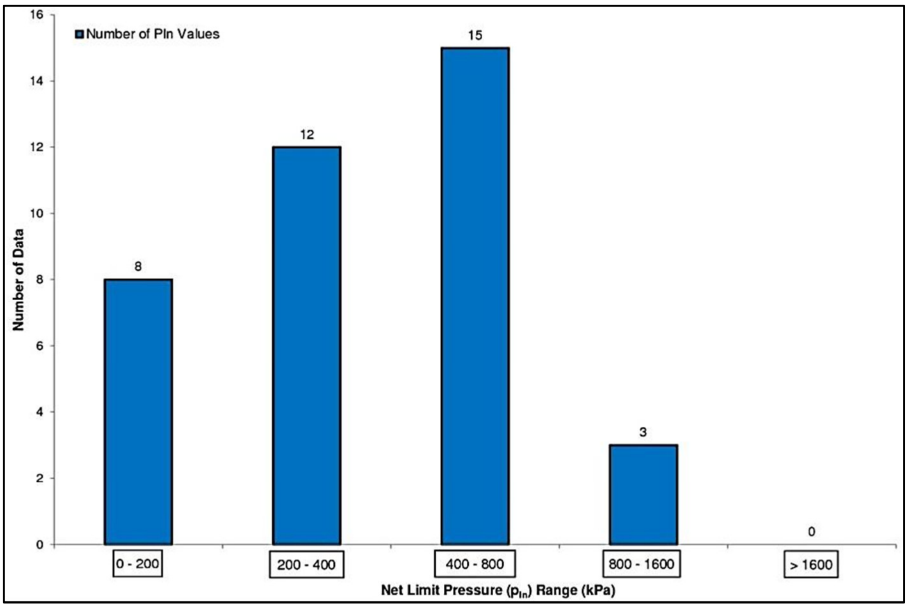

Figure 3.25 Net limit pressure (p_{ln}) distributions in fine grained alluviums

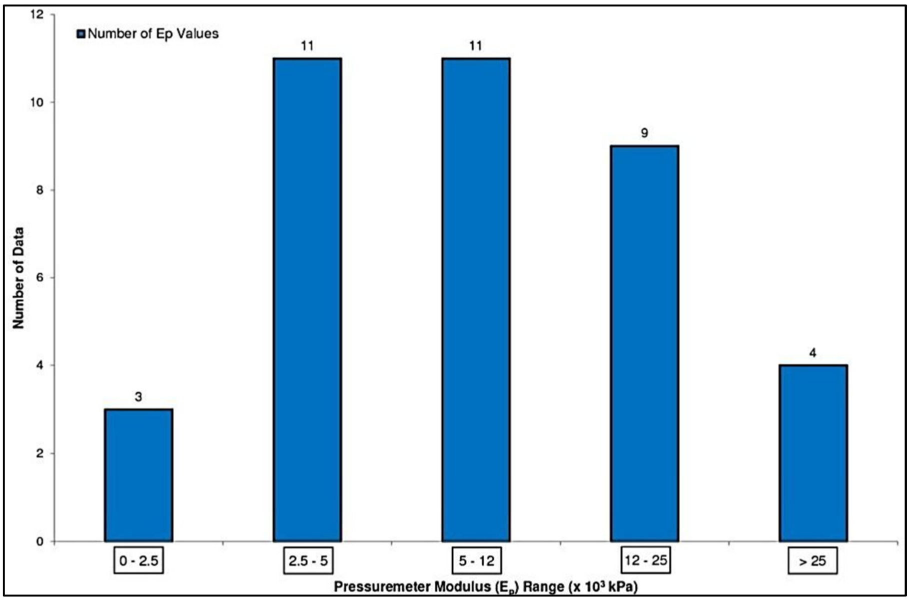

Figure 3.26 Pressuremeter modulus (E_p) distributions in fine grained alluviums

SPT results indicate that the consistency of fine grained alluviums along the project route can be generalized as stiff to very stiff considering the study of Terzaghi & Peck (1976) (Table 2.1). SPT N values are mostly between 10 and 30 where exceptions are also present.

As for the pressuremeter test results, net limit pressure values are mostly accumulated around 200 – 800 kPa range and pressuremeter modulus values change mostly between 2500 – 12000 kPa. According to the Table 2.6 presented by Briaud (1992) approximate consistency of fine grained alluviums can be described as medium to stiff which is compatible with the result from SPT.

3.4. İzmir Port Rehabilitation Project / Turkey

In the scope of the rehabilitation of İzmir Bay and İzmir Port project a site investigation program was undertaken in 2012 to evaluate the soil conditions prior to dredge operation in the bay area, determine the feasibility of dredged material to be used in embankments and identification of soil profile for the geotechnical design of fill sections in the container area. Location of the project is given in the Figure 3.27.

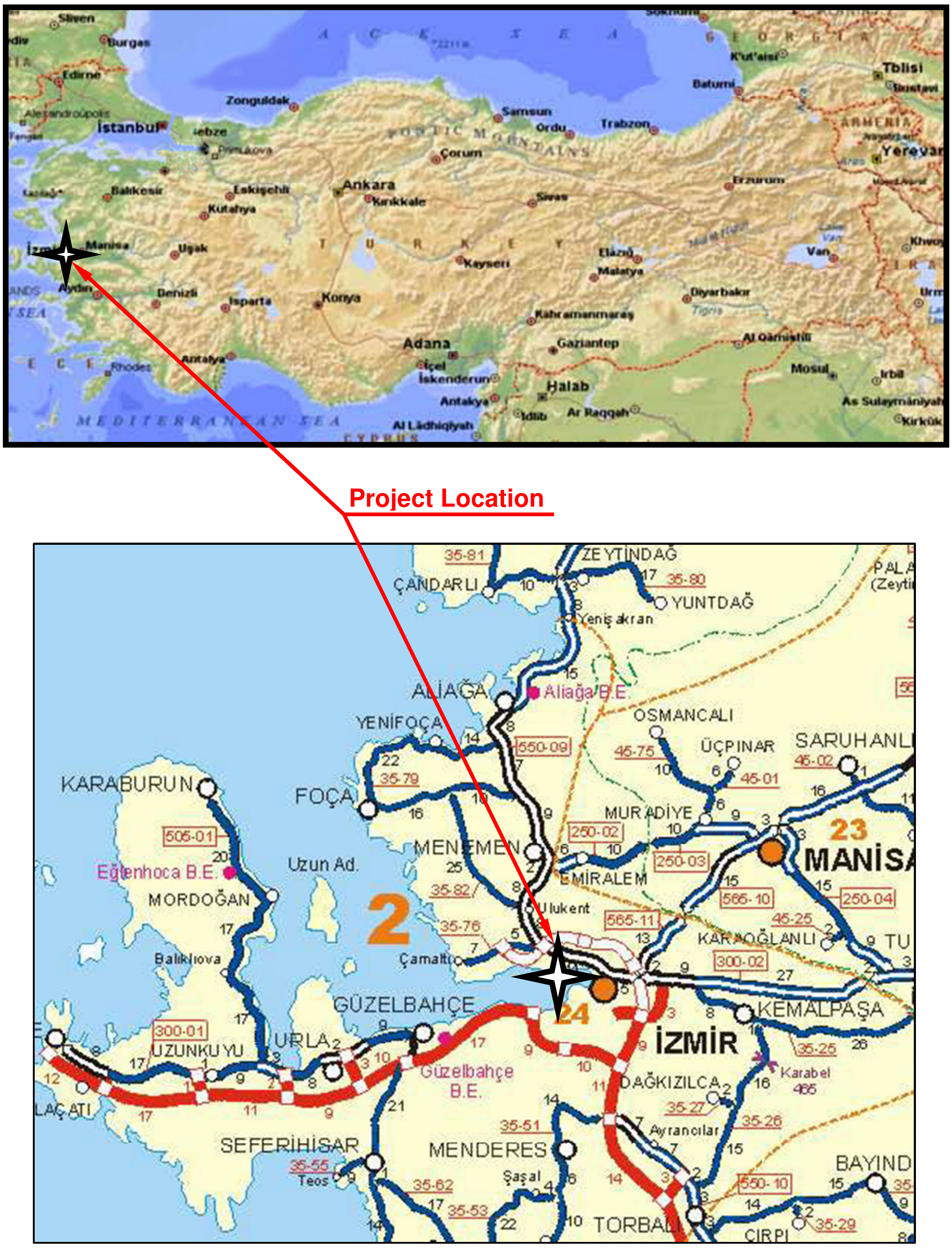

Figure 3.27 Location map of the İzmir Port rehabilitation project

Site investigation program was consisted of borehole drillings inside and nearby locations of the bay area, in situ tests (SPT) and laboratory tests on both disturbed and undisturbed samples. A total number of 85 boreholes were drilled in the project area with a total depth of 2255 meters. Borehole drilling is performed according to "Technical Specification of Foundation Borings for Structures" published by Turkish Ministry of Public Works. Laboratory testing program included triaxial, unconfined compression and oedometer tests aside from the tests to determine the index properties of soil samples. General views from the project site are given in Figure 3.28 and 3.29.

Figure 3.28 General view of the İzmir Port rehabilitation project

Figure 3.29 General view of the İzmir Port rehabilitation project

Geological formations near the project area is mainly comprised of Quaternary aged terrestrial alluviums which overlays the tectonic formations. They are generally made of brown heavily consolidated very stiff to hard clays and are located beneath the normally consolidated alluvial deposits. Alluvial deposits are consisted of various sized materials changing from clays, silts, sands, clayey sands and gravelly clays. They are mostly formed due to transportation of eroded materials by streams to the leveled locations resulting in a wide and deep sedimentation all over the area. Thicknesses of the deposits vary from 25 meters in the Karşıyaka and Balçova plains to 100 meters in middle areas of the Bornova plateau. On the other hand, shallow alluvial deposits in the bay area are generally formed as very soft to soft clays with sand layers and lenses reaching a maximum thickness of 20 meters. Generally medium to stiff clays are present beneath the soft clay layer in the project location.

Results of sieve analyses and Atterberg limit tests indicate the classification of alluviums according to USCS as CH and CL type of clays. Plasticity indexes of these soils changes in a wide range of 10 – 50 % which are mostly accumulated between 40 – 50 % due to dominancy of CH clays in the area. Distributions of the soil types and plasticity index values for alluviums in the studied database are given in Figure 3.30 and 3.31.

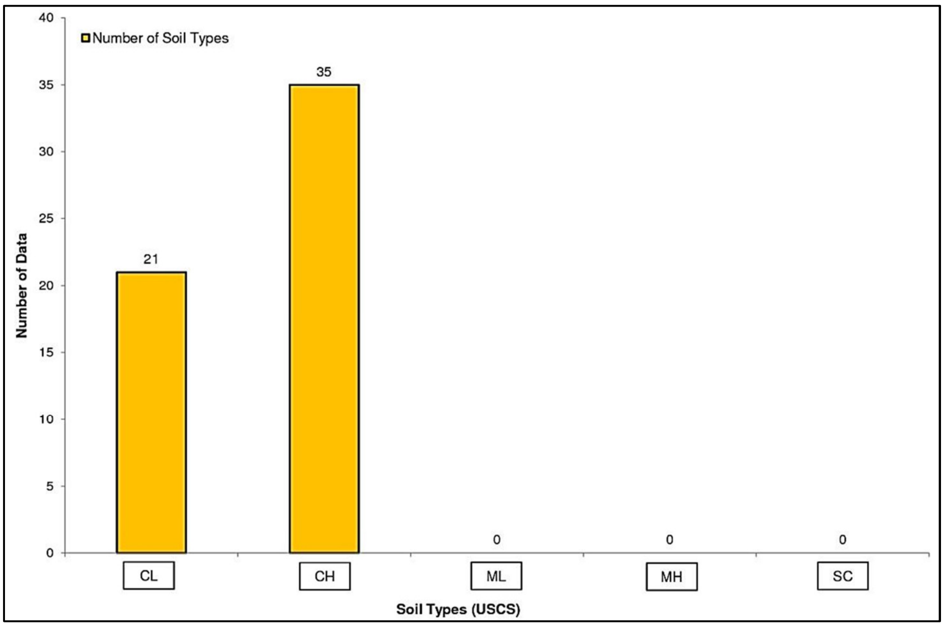

Figure 3.30 Soil type distributions of İzmir alluviums

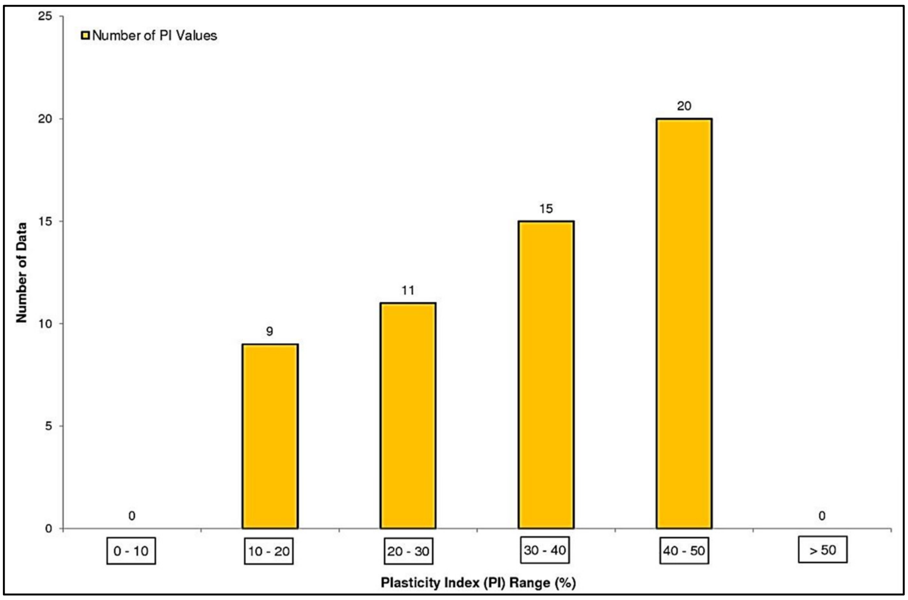

Figure 3.31 Plasticity index (PI) distributions of İzmir alluviums

Laboratory testing program of the İzmir Port Rehabilitation Project included unconfined compression (UC), unconsolidated undrained triaxial (UU) and oedometer tests to assess the undrained shear strength (c_u) and compressibility of soils in terms of coefficient of volume compressibility (m_v). Results of the laboratory tests by distribution are given in Figure 3.32 and 3.33. From the values in Figure 3.32 and 3.33, it can be observed that undrained shear strength (c_u) values are mostly between 50 – 100 kPa and coefficient of volume compressibility (m_v) values are generally in the range of $1 \times 10^{-4} – 2 \times 10^{-4}$ m^2 / kN for İzmir alluviums.

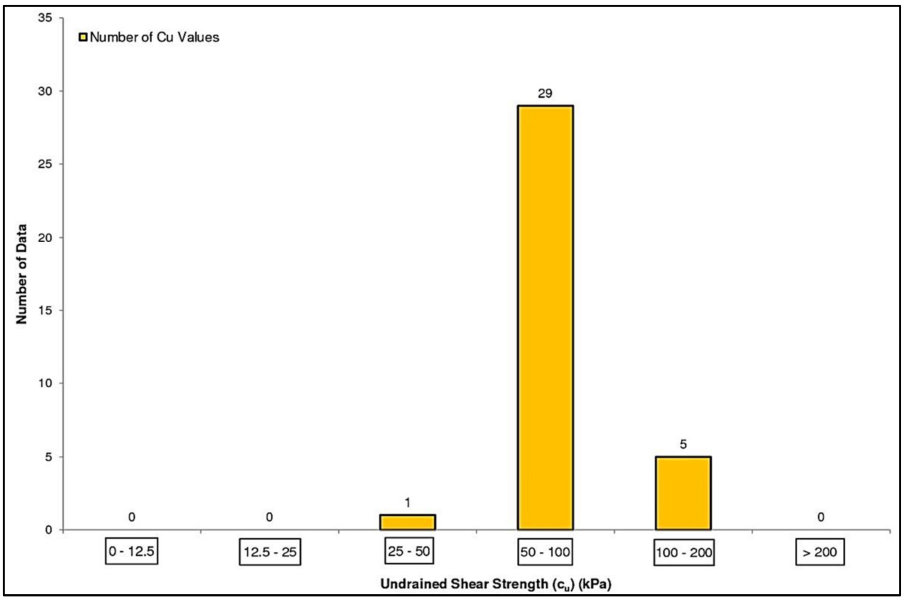

Figure 3.32 Undrained shear strength (c_u) distributions of İzmir alluviums

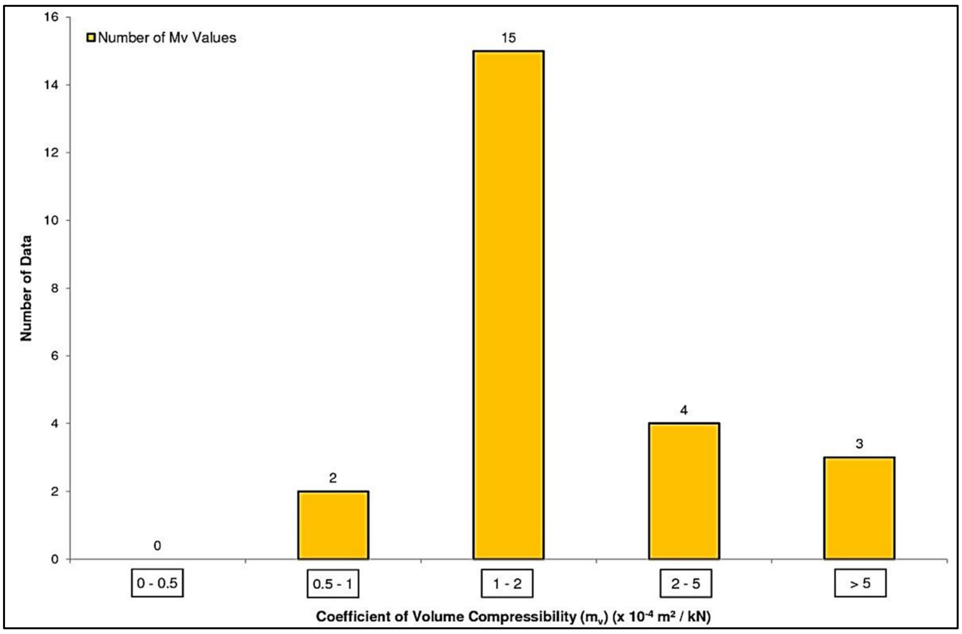

Figure 3.33 Coefficient of volume compressibility (m_v) distributions of fine grained alluviums

In situ testing part of the site investigations is limited to SPT because most of the borehole drillings were performed off the shore in İzmir bay. Recorded SPT N data for the project is given in Figure 3.34. From the plot in Figure 3.34, SPT N values are mostly between 10 and 30 where exceptions are also present. According to the SPT N values, consistency of İzmir alluviums can be clearly generalized as stiff to very stiff.

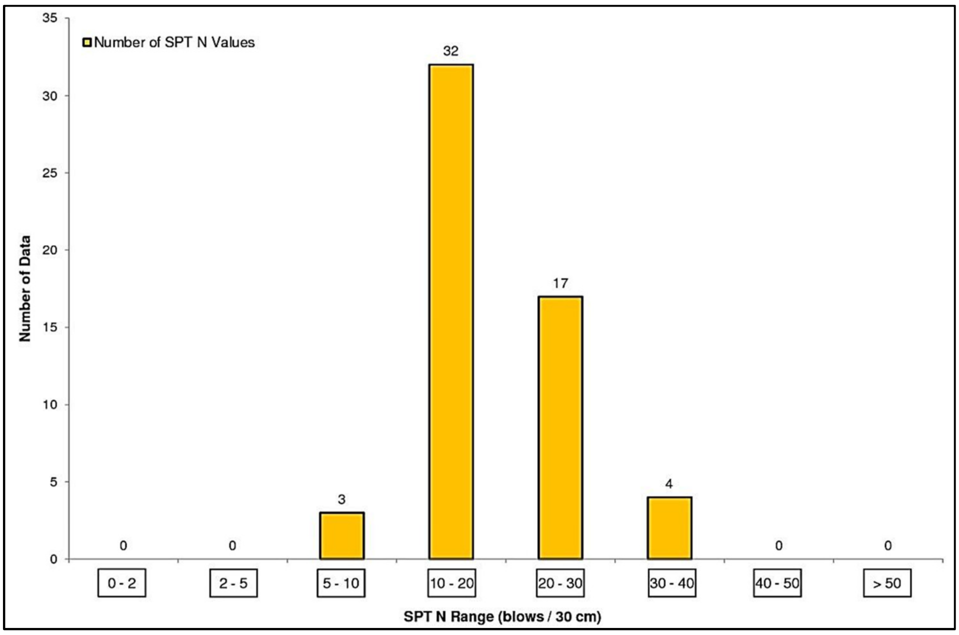

Figure 3.34 SPT N distributions in İzmir alluviums

3.5. Braila Waste Water Treatment Plant Project / Romania

A wastewater treatment plant project (WWTPP) was undertaken as a part of sewer system project. Treatment plant was constructed about 3 kilometers away from the city Braila in Romania. Geotechnical site investigations are executed in both plant area and nearby locations along the pipeline routes. Project location is given in Figure 3.35.

Figure 3.35 Location map of the WWTPP in Braila / Romania

Site investigation program included a total number of 21 borehole drillings. Depths of boreholes are determined considering the presumed influence zone of designed constructions. Drillings are performed according to Romanian Technical Norms in various diameters. Ø 8 – 14” boreholes are made for sampling and in situ testing while the Ø 3” drillings are used for identification purposes. In this study, only the Ø 8 – 14” borehole data was used. Four of the boreholes are used for piezometric measurements as well. Soil samples are obtained as a part of site investigation program and laboratory test are conducted according to Romanian standards. Aside from the sieve analyses and index property tests, unconsolidated undrained (UU), consolidated undrained (CU) trixial tests and oedometer tests are carried out.

In situ test program included cone penetration tests (CPT) and dynamic penetration test heavy (DPH) which is excluded for this study. CPT was performed with a mechanical probe having a cone with an apex angle of 60°, base section area of 10 cm^2 and friction sleeve surface area of 150 cm^2. Pushing of the probe was done with a penetration rate of 2 cm / sec. and data was recorded at every 20 cm interval in the means of cone tip resistance and sleeve friction. 15 CPT are performed with lengths varying between 5.0 – 25.6 meters.

Geomorphologically, Braila city is located on a smooth area. Quaternary formations are developed over the area containing yellow macroporous loess deposits which have potential to collapsing. Since most of the loess deposits are saturated below the depths of 2.0 – 4.0 meters, material can be considered as very compressible. Some of the sites are made up from recent, unconsolidated alluvial deposits containing organic material and peat. The city is in the flood plain zone due to high water levels of Danube River where the ground water level is located at 1.5 – 2.5 meters depth from the surface.

According to the results of laboratory sieve analyses and Atterberg limit tests, USCS classification of soils are determined to be mainly CH type clays with occasional CL levels (Figure 3.36). Loess deposits of CH type clays are highly plastic in general. Plasticity index values obtained from laboratory tests indicate a range of PI between 20 – 78 % which is dominated by the values larger than 50 %. Distributions of plasticity index (PI) values are given in Figure 3.37.

Disturbed and undisturbed samples during borehole drillings were subjected to laboratory testings for the assesment of shear strength and compressibility of soils. Shear strength parameters were determined under unconsolidated undrained triaxial (UU) and consolidated undrained triaxial (CU) from saturated samples. On the other hand, compressibility characteristics of the soils were investigated thoroughly by the oedometer tests on many undisturbed soil samples. Full saturation of the samples is achieved under a back pressure of 312 kPa. Coefficient of volume compressibility (m_v) values were estimated from oedometer test results. Range distributions of laboratory tests are given in Figures 3.38 and 3.39. Undrained shear strength (c_u) values are generally between 50 – 100 kPa and coefficient of volume compressibility (m_v) values are generally in the range of $2 \times 10^{-4} – 5 \times 10^{-4}$ m^2 / kN for loess deposits indicating a high compressibility characteristic.

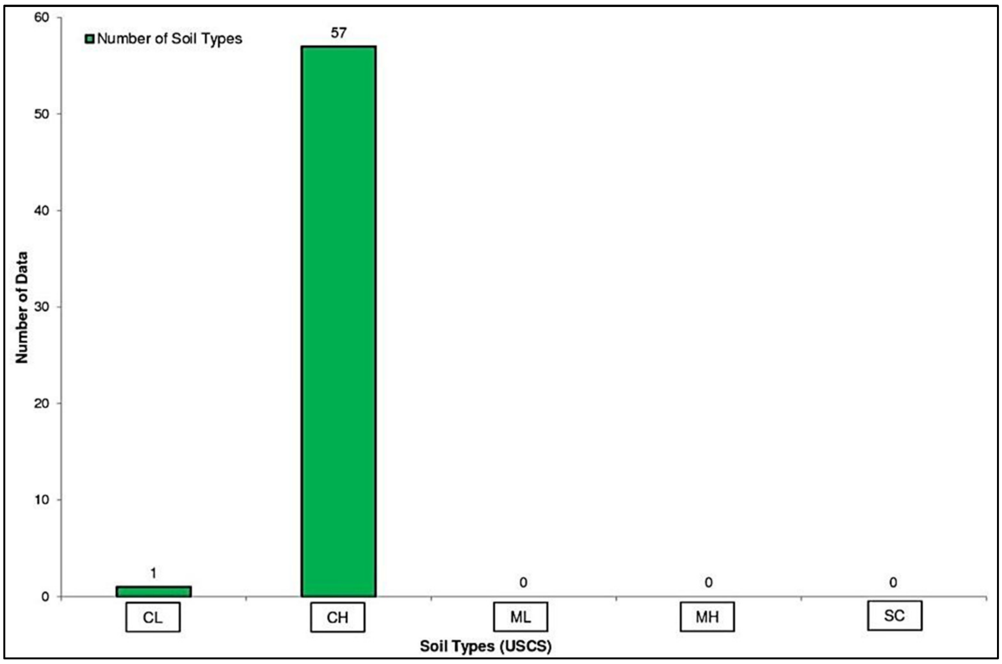

Figure 3.36 Soil type distributions of the loess deposits

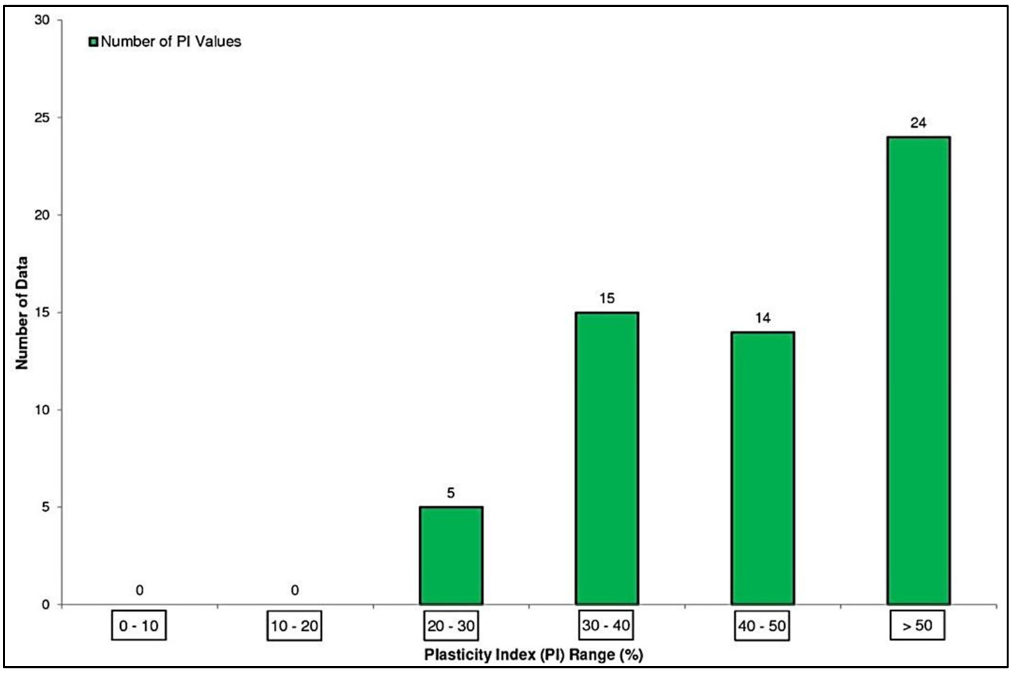

Figure 3.37 Plasticity index (PI) distributions of loess deposits

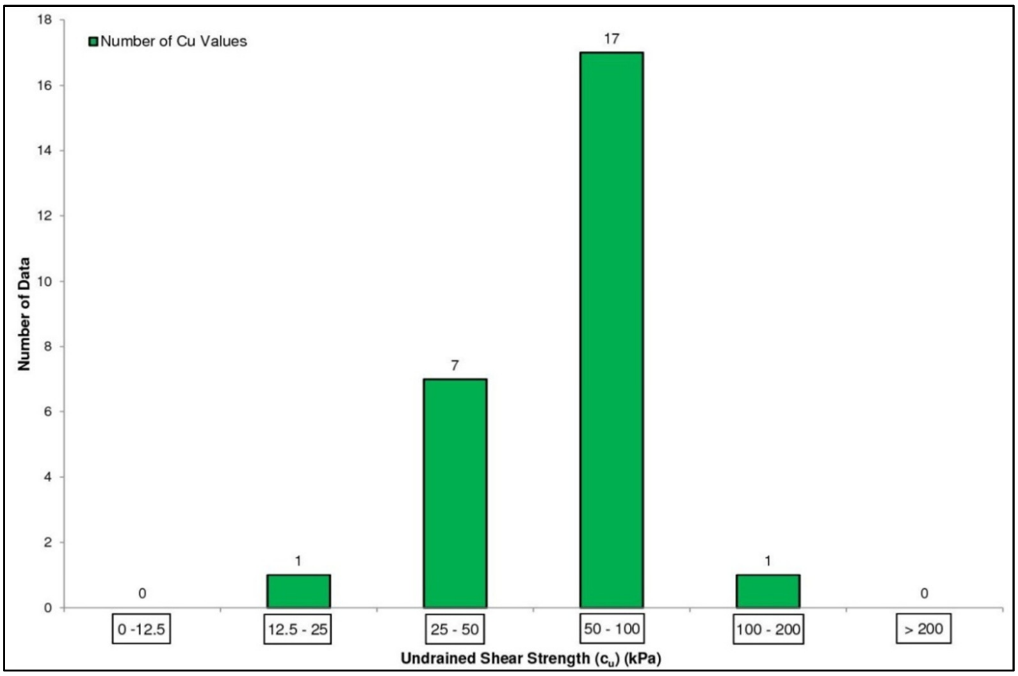

Figure 3.38 Undrained shear strength (c_u) distributions of loess deposits

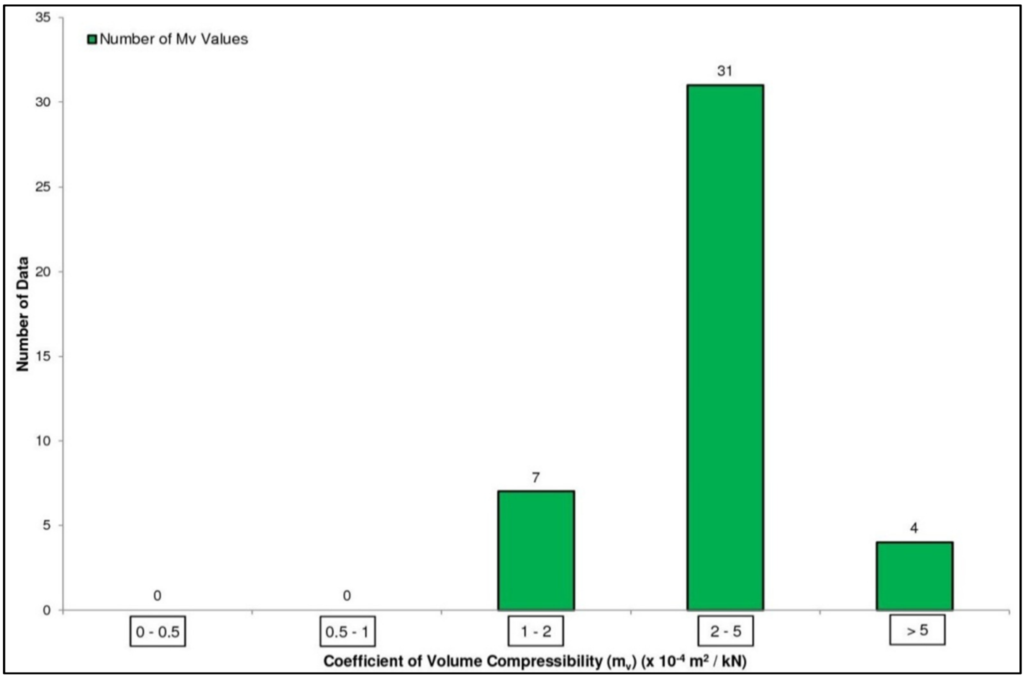

Figure 3.39 Coefficient of volume compressibility (m_v) distributions of loess deposits

As for the in situ tests, CPT logs are evaluated and the derived ranges of cone tip resistance (q_c) values are given in Figure 3.40. CPT cone tip resistance values are found to vary mostly between 700 – 2000 kPa.

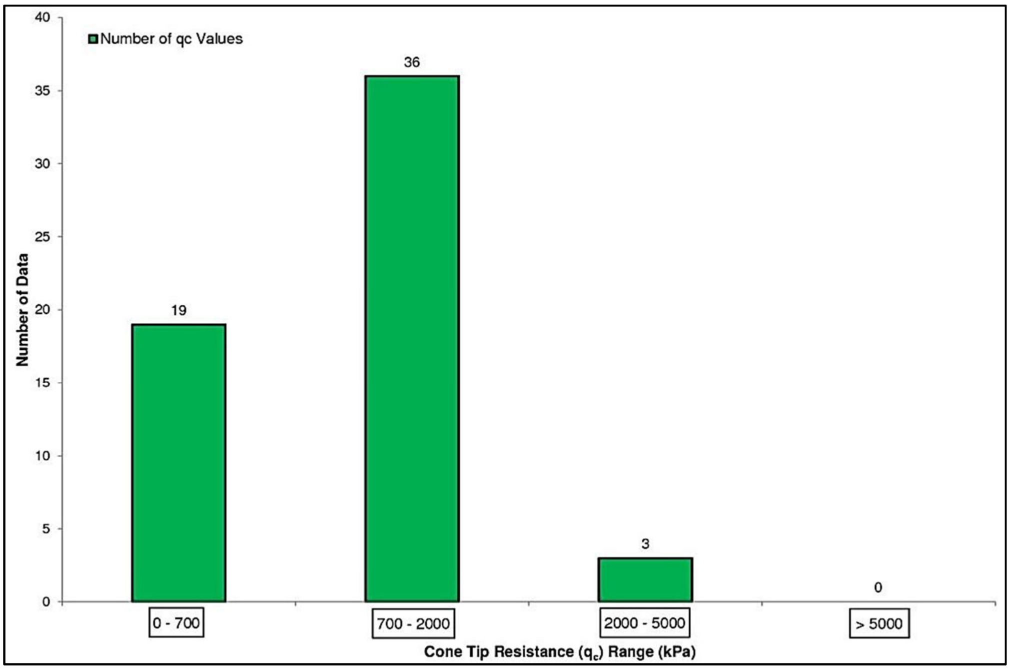

Figure 3.40 CPT cone tip resistance (q_c) distributions in loess deposits

3.6. Warsaw Subway II Line Project / Poland

A Geological and geotechnical investigation program was executed in the scope of subway line project consisting 19 stations and 16 tunnels in Warsaw city. Total length of the subway II line will be equal to approximately 21 kilometers with side tracks. It has been planned to cross the city from west to east and connect the left and right side of the Vistula River. The location of the project is mapped on the Figure 3.41.

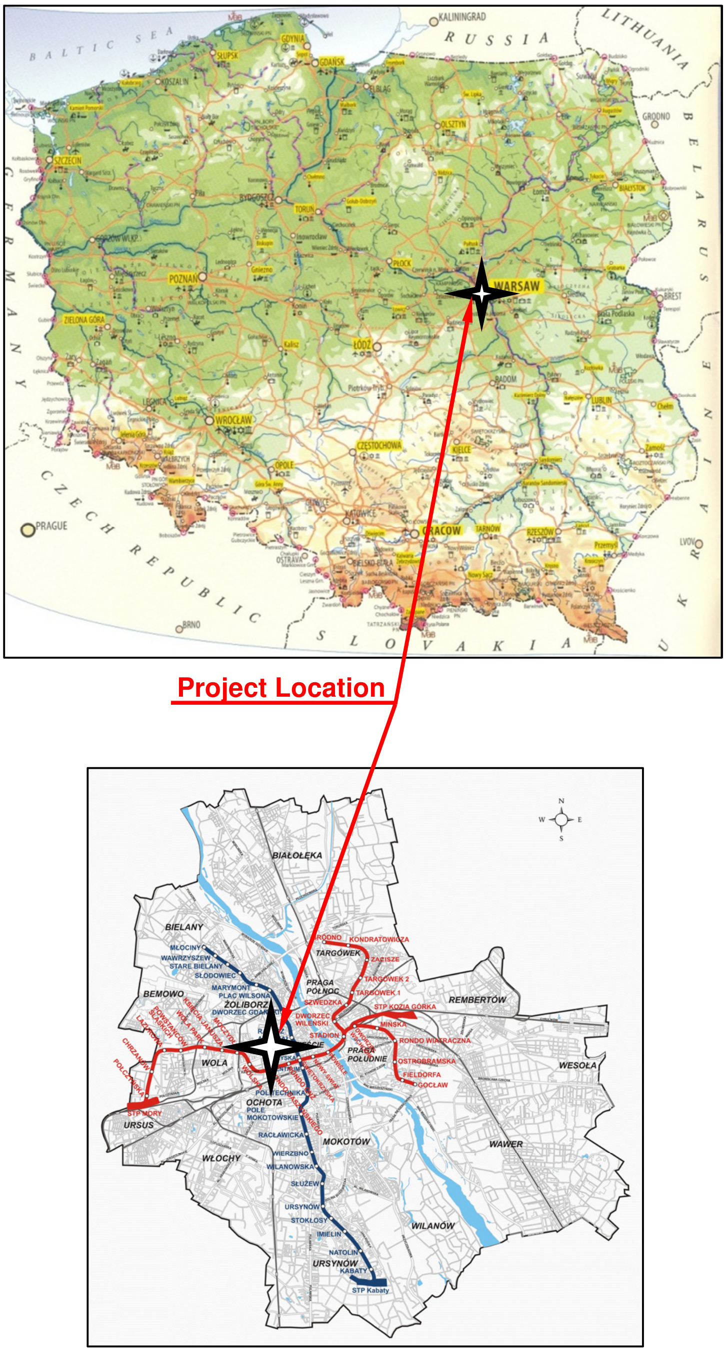

Figure 3.41 Location map of the subway II line project / Warsaw

Site investigation program in 2003 for the subway II line included the explorations of subsurface conditions for subway stations S8, S9, S10, S11, S14 and tunnels T8, T9, T10. Later, investigations were expanded for stations S12, S13 and tunnels T11, T12 and T13 in 2007. In the extent of site exploration programs, 90 borehole drillings were performed in station areas and 66 borehole drillings were performed along the tunnel alignments with a total number of 156 boreholes. Along with borehole drillings, a wide range of in situ tests were carried out including SPT, CPT, PMT and DMT. Along with the SPT in most of the boreholes, 71 CPT, 25 DMT and 10 PMT were performed in station areas whereas 61 CPT were conducted through the tunnel route besides SPT. Locations of in situ tests were determined so that the continuity of the data regarding the soil profile is achieved and cross correlation between boreholes and in situ tests are possible.

CPT was conducted with both electric cone by Geotech and mechanical cone by Begemann. The tests were stopped when the side friction and tip resistance values exceeded 50 MPa and 200 kN respectively.

Dilatometer tests were carried out in the station areas for the evaluation of settlements as well as the structural design of station. First readings were taken at depth of 2.0 meters up to impenetrable layers with an interval of 20 cm. Methodology of the tests were compatible to the Marchetti ‘s (1980) guideline.

During the drilling works, disturbed soil samples were taken and subjected to for grain size distribution and index properties tests in laboratory. Undisturbed samples on the other hand, were acquired for selected cohesive soil layers by thin walled samplers.

From the geological point of view, the studied area lies within the Warsaw Basin, composed of Upper Cretaceous deposits, developed as marls and marly high plastic clays below approximately 250 meter from ground surface. This Crateceous Basin is filled with deposits of Tertiary aged Oligocene (sands, sands with gravel, slimes and highly plastic clays), Miocene (sands, slimes and highly plastic clays with inter beddings of brown coal) and Pliocene (grey, greenish – grey high plasticity silty clays inter bedded with fine silty sands) deposits. Oligocene and Miocene units are consistent with the sedimentation processes whereas Pliocene units show irregularities due to intense erosion and glaciotectonic effects in Quaternary period. The variations in the surface of Pliocene units result in highly variable thicknesses during sedimentation of the Quaternary deposits.

The central part of the subway II line is in interaction with two main groundwater reservoirs which are linked with the moraine plateau and valley of Vistula River. In the plateau area, four different aquifers are encountered. The aquifers close to ground surface form a network by sandy deposits of variable thicknesses with a stabilized water level of 6.0 meters from surface. On the other hand, in the river valley area, two main aquifer horizons are identified. First aquifer horizon is located approximately 5 – 7 meters from the surface in sand and gravel deposits and has a direct connection with river water. The second of the aquifers is encountered in Pliocene high plasticity clays by the means of sand and silt layers or lenses between depths of 7 – 28 meters from surface. These pockets of water are considered to have no hydraulic connections among themselves.

According to the results of laboratory tests, main soil profile is comprised of CH and CL type of clays. CH clays have plasticity index values mostly larger than 40 % whereas CL type clays have a plasticity index range of 10 – 30 %. In addition to CH and CL clays, SC type of sands was determined mainly due to presence of sand layers of lenses inside the clay formation. Distributions of soil types according to USCS and plasticity index (PI) values are given in Figure 3.42 and 3.43.

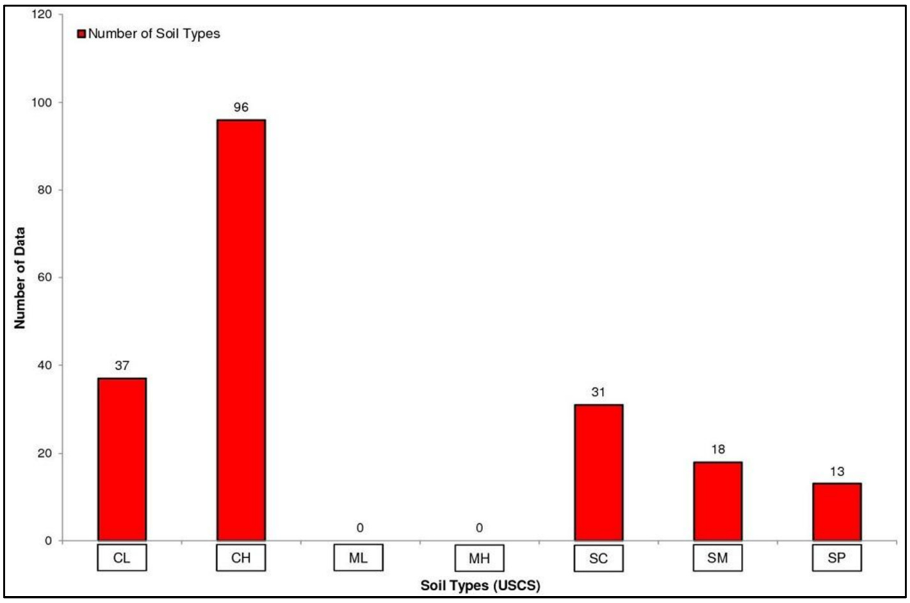

Figure 3.42 Soil type distributions of the Pliocene & Quaternary deposits

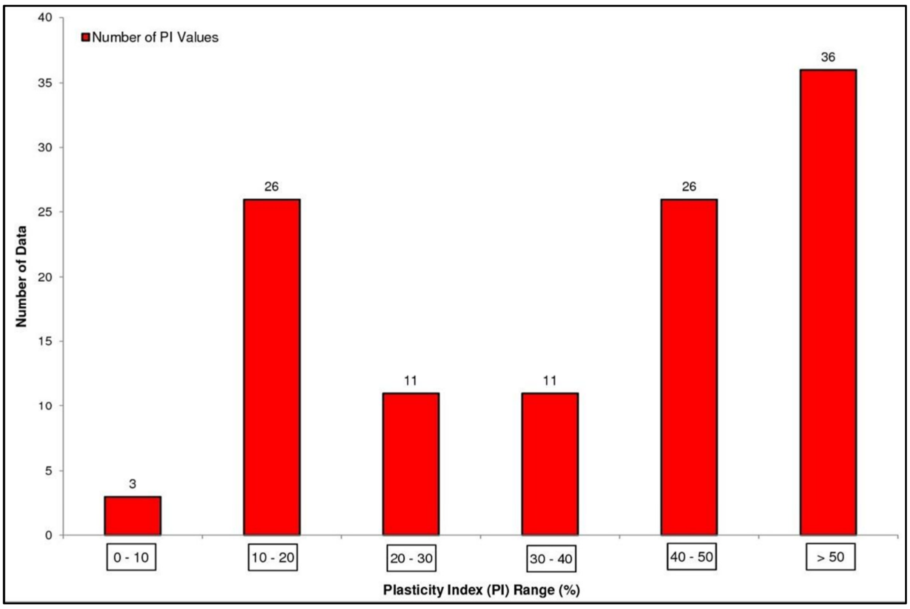

Figure 3.43 Plasticity index (PI) distributions of Pliocene & Quaternary deposits

Shear strength and compressibility characteristics of the CH and CL clays are determined by laboratory oedometer and triaxial tests. Oedometer tests were conducted on fully saturated undisturbed samples under the effective stresses of in situ conditions. In triaxial tests, fully saturation of samples were achieved by flooding water in to the system and application of back pressures of about 300 – 400 kPa. After then, samples were consolidated isotropically and sheared by strain controlled method. The results of undrained shear strength (c_u) from triaxial tests and coefficient of volume compressibility (m_v) values from oedometer tests are given in Figures 3.44 and 3.45 in terms of ranges.

Undrained shear strength (c_u) of the units are observed to be vary mainly between 50 – 100 kPa and the coefficient of volume compressibility (m_v) values are mostly accumulated between the ranges of 0 – 0.5 x 10^{-4} m^2 / kN and 0.5 x 10 $^{-4}$ – 1 x 10^{-4} m^2 / kN. From the m_v ranges in Figure 3.45, it can be concluded that the compressibility of clays are small under in situ stresses.

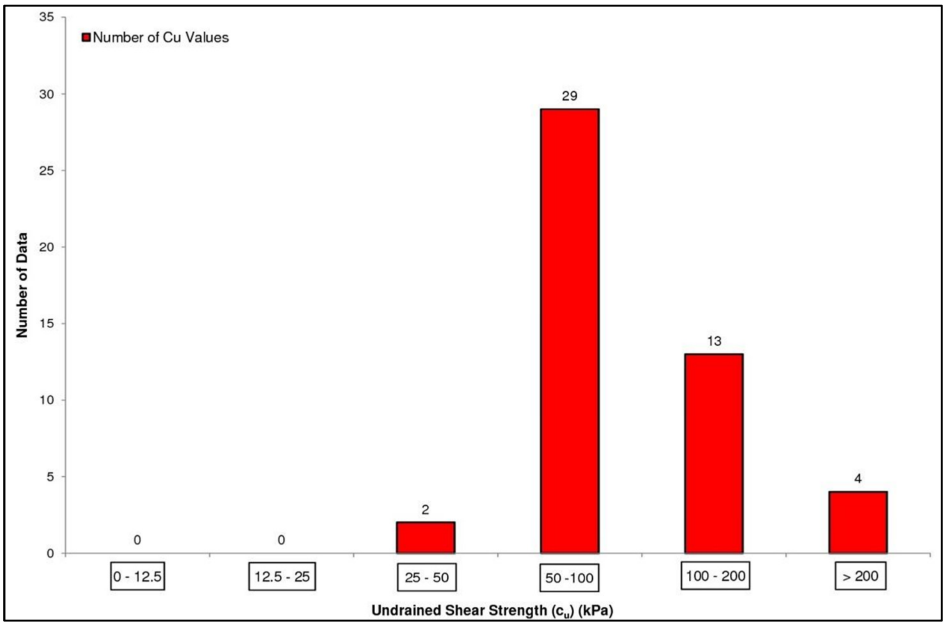

Figure 3.44 Undrained shear strength (c_u) distribution of Pliocene & Quaternary deposits

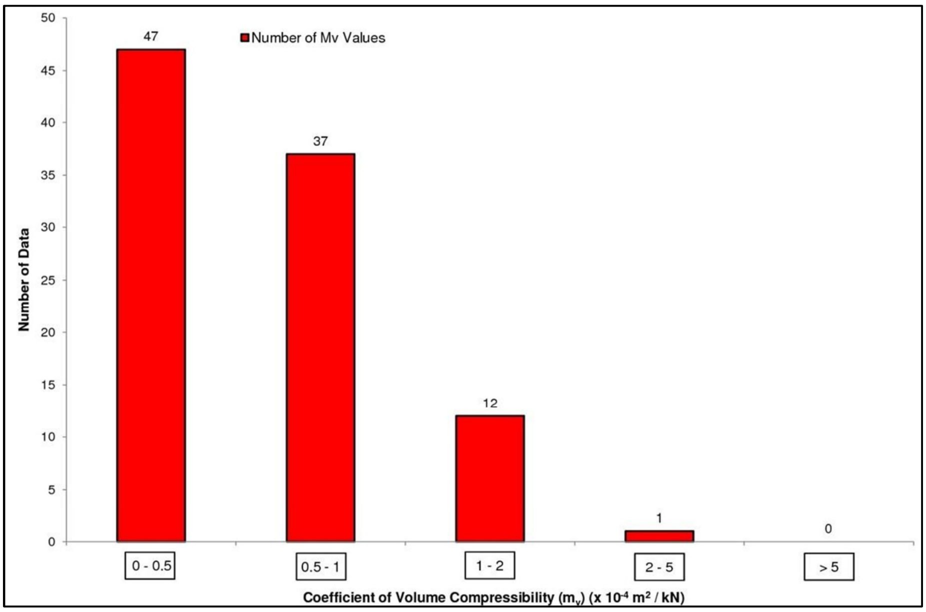

Figure 3.45 Coefficient of volume compressibility (m_v) distribution of Pliocene & Quaternary deposits

SPT N values obtained in Pliocene deposits generally show an inconsistent distribution with depth due to irregular layering of soils. Up to 20.0 meters depth from the surface, SPT N is found to be in the range of 10 – 55 with a rough average of 25. This wide range in the SPT N numbers for the first 20.0 meters can be related to presence of sandy – gravelly layers or lenses in clay body. After 20.0 meters, range of N values reduces to 25 - 60 with an average of roughly 40. Distribution of SPT N numbers throughout the project is given in Figure 3.46.

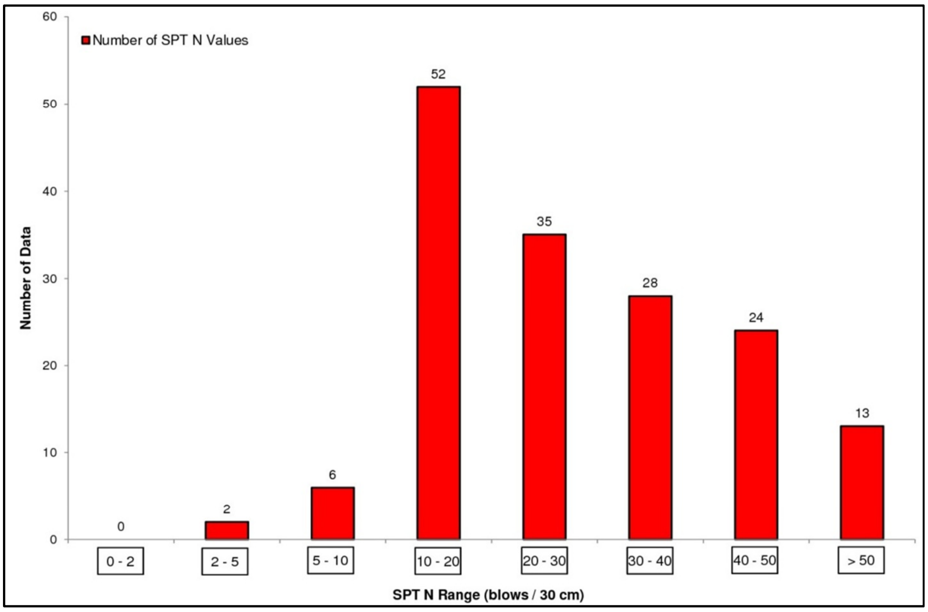

Figure 3.46 SPT N distributions in Pliocene & Quaternary deposits

On the other hand, CPT cone tip resistance (q_c) values for the general soil profile are observed to vary between 2400 and 9800 kPa with an average of approximately 5000 kPa. Distributions of CPT cone tip resistance (q_c) according to number of data is given in Figure 3.47.

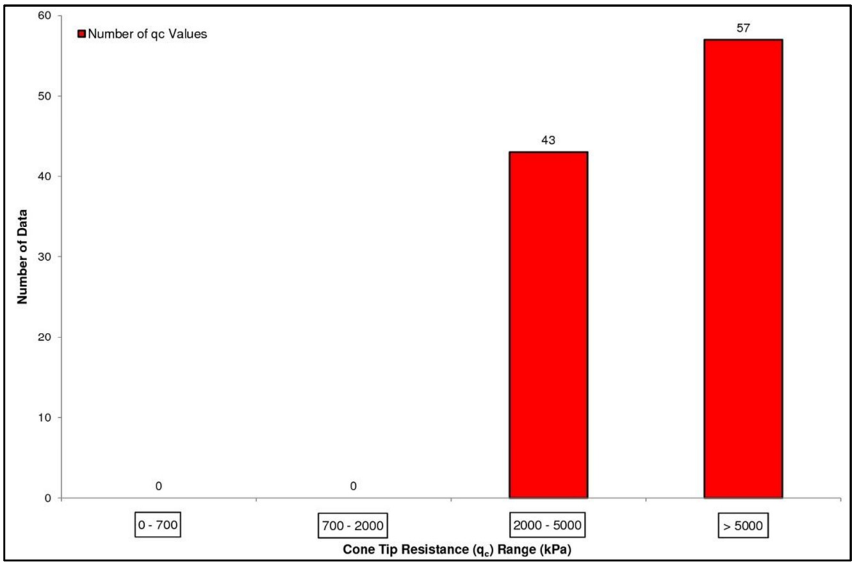

Figure 3.47 CPT cone tip resistance (q_c) distributions in Pliocene & Quaternary deposits

Flat dilatometer tests (DMT) performed during the site investigation studies were in limited numbers and only executed in station areas. In the scope of this study, only the index parameter called dilatometer modulus (E_d) was used in the analyses for which the distribution of the data is given in Figure 3.48.

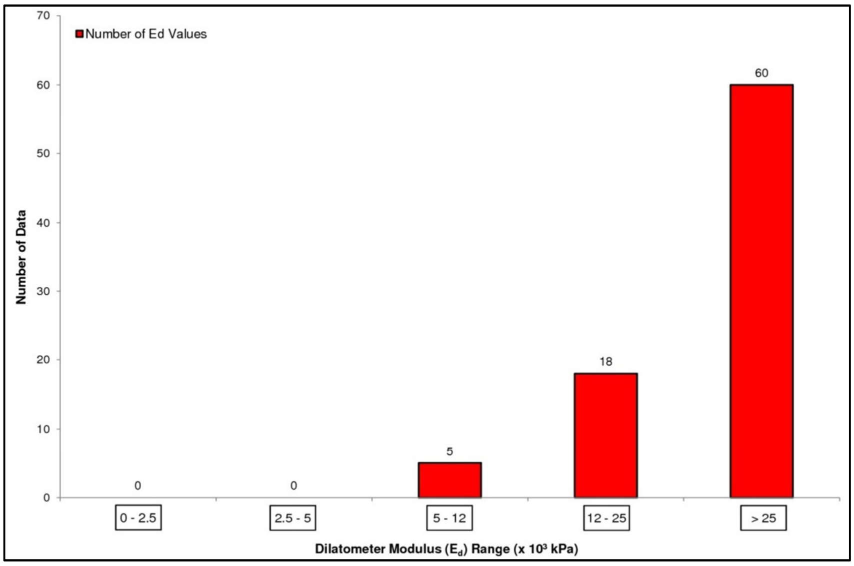

Figure 3.48 Dilatometer modulus (E_d) distributions in Pliocene & Quaternary deposits

Additionally, a small number of pressuremeter tests were also performed over the station areas mostly near to the dilatometer tests. Although the numbers of tests are very limited, sufficient number of pressuremeter modulus (E_p) data is found to be used especially in the correlations between the dilatometer modulus (E_d). The results of pressuremeter tests in terms of E_p are presented in Figure 3.49 with the ranges of data.

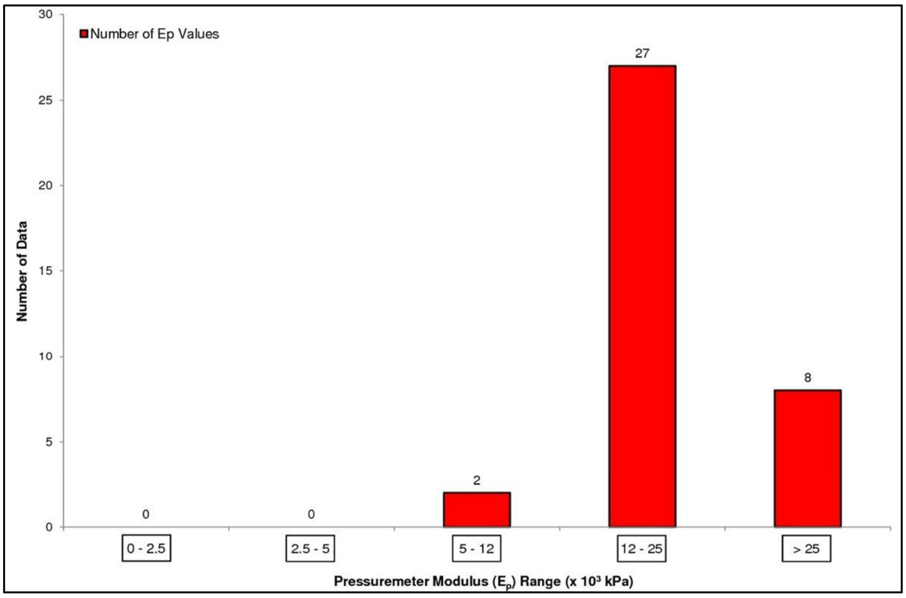

Figure 3.49 Pressuremeter modulus (E_p) distributions in Pliocene & Quaternary deposits

CHAPTER 4

DISCUSSION OF THE ESTIMATED PARAMETERS FROM IN SITU TESTS

In this chapter, detailed discussions of performed analyses are presented regarding the in situ and laboratory test data presented in Chapter 3. Prediction of three fundamental geotechnical parameters of cohesive soils namely as undrained shear strength (c_u), coefficient of volume compressibility (m_v) and deformation modulus (E_s) from in situ tests by using the laboratory test results is the main subject of this study. In addition, cross correlations between some of the in situ tests were also evaluated. Parameters and correlations are derived by empirical approaches and the results are compared with the similar studies presented in the Chapter 2.

4.1. Reduction of Data

Processing the raw data obtained from in situ tests is one of the challenging works in geotechnical design. Reliability of an analysis result is mostly defined by the accuracy of selected data rather than the method used for the analysis. Therefore, selection of the most representative parameters for a site is the key to a successful design.

In order to evaluate the correlation between in situ test results and geotechnical properties of cohesive soils more accurately, classification and data reduction methodologies for both in situ and laboratory tests are adopted for the compiled database.

4.1.1. Data Reduction for In Situ Tests

Due to the differences in scope, equipment and measurement techniques, elimination of inappropriate data from in situ test results yields to a more compatible relationship between the tests. Methods of data reduction for the analyzed in situ tests are explained below.

4.1.1.1. Standard Penetration Test (SPT)

Standard penetration test indicates the resistance of a soil against a dynamic penetration. The blow count required for the penetration of 30 cm in to the soil is defined as the SPT N value which is a unitless number. Although the SPT N numbers are required to be corrected due to the reasons mentioned in Chapter 2, uncorrected raw data from SPT was used in the scope of this study mainly to see the results when corrections are not applied. By this method, the capability of the correlations to predict the geoetechnical parameters from the uncorrected field SPT N were investigated when quick analyses are needed especially in the site.

Reductions for SPT N value from the database are performed on the basis of following considerations:

- Reasonability of SPT N numbers were evaluated according to the consistency of the tested soils. Low N values due to artificial fill or organic soil layers close to surface were determined and deducted from the database. Similarly, high SPT N values or refusals at deep levels were compared with borehole descriptions and laboratory tests and N values belong to weathered rock units are eliminated.

- General trend of the SPT N values in soil layers were considered in the analyses. High N numbers due to the presence of gravel or cobble particles in the soil layers were disregarded for a more refined analysis.

- Compatibility of the N numbers was checked with the ranges in commonly used literature studies for similar type of soils.

- Laboratory test results were compared with the N numbers to validate the reasonability of estimations.

4.1.1.2. Cone Penetration Test (CPT)

Eventhough, CPT provides a continuous soil profile and related parameters, reduction and idealization of the data is essential for the interpretation of the test. CPT logs usually present a scattered pattern for cone tip resistance and side friction data which is mainly due to different resistance characteristics of each sub layers of a soil body. Therefore, idealization of the raw data is commonly preferred in geotechnical practice.

Identification of main soil layers by CPT logs can be made according to three basic criteria for sandy and clayey soils as follows:

<u>Sandy Soils</u>:

- Cone tip resistance (q_c) values are generally large because of the particle sizes.

- Side friction (f_s) values are usually low due to absence of cohesion.

- Pore pressure measurements are usually low because of the quick dissipations of excess pore pressures in sands.

<u>Clayey Soils</u>:

- Cone tip resistance (q_c) values are generally low compared to sands.

- Side friction (f_s) values are usually high due to cohesion.

- Pore pressure measurements are usually high because of the slow dissipations of excess pore pressures.

Idealization and reduction of the CPT data is performed according to following issues:

- Soil layers were identified by using the basic guidelines mentioned above and the idealized CPT profile was compared with borehole logs.
- Scattered cone tip resistance (q_c) values were idealized by averaging considering the consistency of soils. (Figure 4.1)
- High q_c values of coarse grained soil layers (Sands, gravels, cobbles etc.) inside clays were mostly disregarded to obtain reasonable values for clays.
- Ranges of q_c values were determined compatible to similar literature studies for comparison purposes.

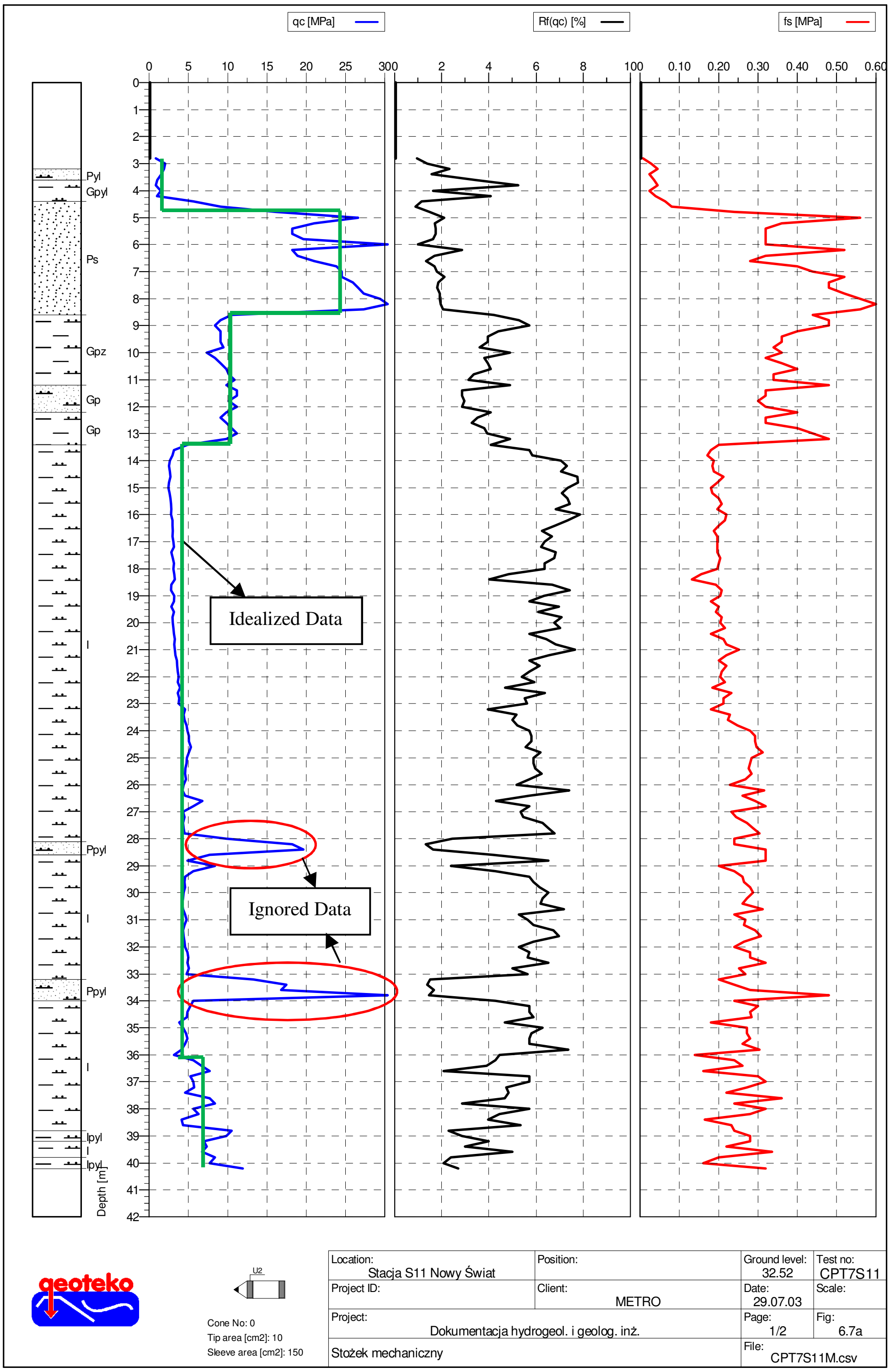

Figure 4.1 Typical idealization of cone tip resistance (q_c) in CPT results

4.1.1.3. Pressuremeter Test (PMT)

Raw pressure and volume recordings of pressuremeter tests are usually analyzed by software. By this method, necessary corrections due to calibrations are reflected on the results and accuracy of corrected pressuremeter curves can be enhanced. Even though, there are still some modifications and assessments that may be required on corrected curves regarding the following issues:

- Slope of the curve in the elastic range may vary slightly due to non linear behavior of the soil. Thus, a best fit line for the linear portion is drawn to estimate the pressuremeter modulus (E_p).

- In hard soils, complete failure of the soil may not be achieved because of the high resistance of soil. In that case, soil is assumed to be failed at the two times of the initial volume ($2V_0$). The pressure value corresponding to volume of $2V_0$ was accepted to be the limit pressure (p_l) for that test.

Aside from these considerations, shape of the pressuremeter curves was analyzed and compatibility of the net limit pressure (p_{ln}) and pressuremeter modulus (E_p) values was checked with the typical soil behavior and ranges in the literature.

4.1.1.4. Flat Dilatometer Test (DMT)

Interpretation of the DMT is mostly based on three main index parameters namely as the material index, I_d, horizontal stress index, K_d and dilatometer modulus, E_d which are derived from the raw data of the test. DMT logs usually present a continuous data for the tested soil profile with a similar scattered pattern as in CPT. Therefore, an idealization and reduction in the data was performed by taking the overall response of the soil into account. A typical DMT log showing the change of index parameters with respect to depth is given in Figure 4.2 with the idealization and reduction methods of data.

Among the index parameters of DMT, dilatometer modulus (E_d) is mainly focused in the content of this study for which the idealization and reduction of the DMT data was performed according to following issues:

- Soil types were identified by using the material index (I_d) values and the compatibility is checked with the nearby borehole logs and laboratory tests.

- Scattered dilatometer modulus (E_d) values were idealized by averaging according to the general trend of the soil layer. (Figure 4.2)

- Spiking values of E_d were mostly disregarded to obtain reasonable values.

- Obtained ranges of E_d values were compared with the similar literature studies to check the reasonability of the results.

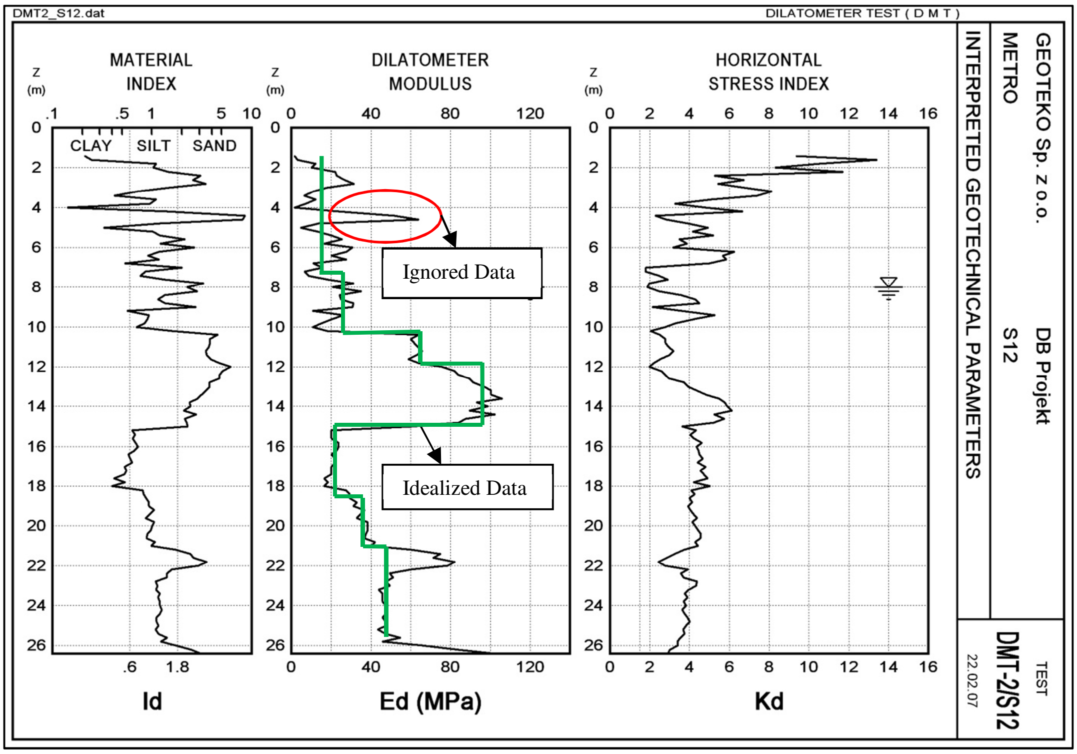

Figure 4.2 Typical idealization of dilatometer modulus (E_d) in DMT results

4.1.2. Evaluation and Data Reduction of Laboratory Tests

Laboratory test results were evaluated in a detailed manner for this study. The quality of the taken samples is the main concern regarding the evalaution of strength and compressibility characteristics. Considering the fact that the laboratory data of this study was compiled from the projects with a different aim than research purposes, it can be stated that the quality of taken samples are expected to be low. Beside the consistency and compatibility issues mentioned for in situ tests, some additional subjects were taken into account for laboratory tests.

Plasticity of fine grained soils is one of the most important properties which define the ability of a soil to show unrecoverable (plastic) deformation without cracking or crumbling. Generally clays with low plasticity index values can be loaded to significant levels. However, at the peak strength, crushing of the soil occurs in a brittle manner. Highly plastic clays on the other side can take lesser loads compared to clays of low plasticity but deform more before the failure describing a more ductile behavior.

Since plasticity index of soils is very effective on the behavior, some reductions were done from the laboratory test results. Clays of low plasticity (CL) having a plasticity index (PI) value less than 10 % are deducted in the analyses. These soils can be considered to show the behavior of non plastic sands rather than plastic clayey soils under loading conditions. In addition, clayey sands (SC) and silts of low plasticity (ML) in the database were also evaluated in a similar manner.

The main purpose of triaxial tests conducted in laboratory is determination of shear strength parameters of cohesive soils. It has the ability to simulate the real drainage and loading conditions in site. In an undrained unconsolidated (UU) traixial test in saturated soils consolidation and drainage of water is not allowed during the loading. Effective stress on the tested specimen is assumed to be constant as the loading proceeds due to increase in power water pressure. Therefore, for a series of tests with different all around pressure values, equal principle stress differences are observed at the failure. As a result, failure envelope becomes horizontal with shear strength parameters $\phi_u = 0°$ and $c = c_u$. However, due to disturbance effects on the sample and uncontrollable drainage of water from the samples yield Mohr circles with unequal radii and so ϕ_u values larger than 0. Among the triaxial test database of this study, almost all of the ϕ_u values are observed to be larger than 0. Therefore, the tests with ϕ_u value up to15° were considered to be reasonable for the analyses and any other results were disregarded. Undrained shear strength of samples were determined then by taking the average of c_u values obtained from each test with different all around pressures.

Consolidated undrained (CU) test results were also available in some of the projects and they are included in the correlations. Differently from UU tests, before the loading of the specimen, consolidation and dissipation of excess pore water is allowed. After the consolidation, sample is sheared while the drainage of remaining pore water is being prevented. The results of a series of tests is represented by a linear failure envelope with $c = c_{cu}$ and $\phi = \phi_{cu}$. Then, undrained shear strength (c_u) was estimated by the following equation.

$$c_u = c_{cu} + \sigma_{v0}' \tan \phi_{cu} \qquad \text{(Equation 4.1)}$$

Effective stress to be used in the equation is calculated by considering the depth of the sample to estimate the strength of soil under in situ stresses. Typical accepted Mohr – Coulomb failure definitions related to triaxial tests for this study is given in Figure 4.3.

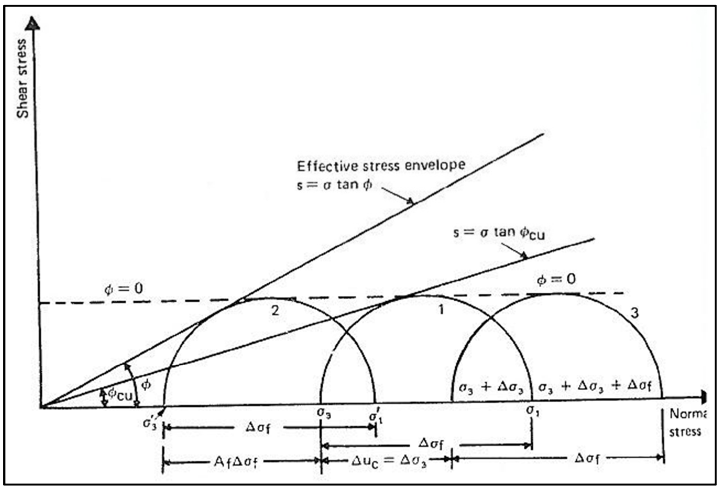

Figure 4.3 Typical Mohr – Couloumb envelopes for triaxial test types (Das, 1985)

The results of oedometer tests were mainly interpreted to estimate the coefficient of volume compressibility (m_v) values in the content of this study. Compressibility of soils depends on the effective stresses acting upon them and usually decreases with increasing effective stresses. The m_v defines the change in volume or void ratio with respect to increase in effective pressure. Thus, m_v value represents the slope of the curve for a given stress range in void ratio (e) or volume (V) vs. log σ_v' graph. Effective stresses at the depth of samples were estimated and the slope of the curve between the related ranges is calculated from the oedometer test graph.

4.1.3. Analysis Methodology

The main methodology accepted for this study is evaluation of all in situ and laboratory tests for a sub layer in the soil body. For example, Measured SPT N number in the field corresponding to an interval of 15.0 – 15.45 meters depth, pressuremeter test results at 14.50 meters depth and laboratory tests from an undisturbed sample at 14.50 – 15.00 meter depth are all taken into consideration for being in the same soil. Since CPT and DMT were performed near to the borehole locations, the results were interpreted in accordance to the boreholes. For instance, laboratory test results of a sample at 14.50 – 15.00 meters depth are analyzed with the data of nearest CPT at the same depth. If the correlated data was determined to be in different soil types at the same depth, then the nearest data representing the related soil conditions were taken into account.

4.2. Assessment of Parameters from SPT Results

Standard penetration test (SPT) results in this study were mainly used to estimate the parameters of undrained shear strength (c_u), coefficient of volume compressibility (m_v) and deformation modulus (E_s) in cohesive soils.

4.2.1. Estimation of Undrained Shear Strength (c_u)

Regarding the undrained shear strength (c_u), two commonly used laboratory tests, unconfined compression (UC) and triaxial tests (UU and CU) were considered to establish a reference strength in the predictions. UC and triaxial tests were analyzed separately according to the base of existing correlations.

4.2.1.1. Correlations Based on Unconfined Compression Tests

Most of the empirical correlations in the literature were based on the unconfined compression test (UC) results for being quick and simple. UC test can be defined as a different kind of triaxial test for which all around pressure value is zero. By theory, undrained shear strength (c_u) is defined as half of the unconfined compression strength.

Estimations from SPT N numbers recorded in field (N_f) and c_u from UC tests were compared with two commonly used correlations proposed by Terzaghi & Peck (1967) (Equation 2.2) and Sowers (1979) (Equation 2.3, 2.4 and 2.5). A total number of 103 data from Eurostar CCPP, Bursa Ring Road Project and İzmir Port Rehabilitation Project were evaluated and the distributions of the data along with the correlations proposed by Terzaghi & Peck as given Figure 4.4.

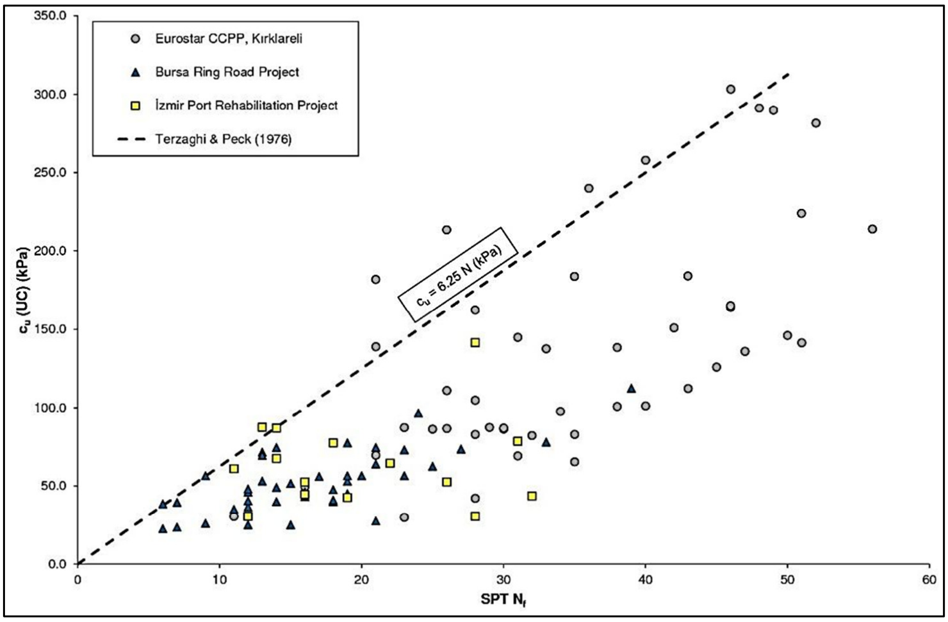

Figure 4.4 SPT N_f vs. c_u compared with Terzaghi & Peck (1967)

From the comparison of the results, it can be clearly identified that the correlation proposed by Terzaghi & Peck seems to form an upper boundary to the obtained c_u values. On the other hand, it is observed that unconfined compression tests underestimate the in situ shear strength by a factor of 2 or even more due to sampling disturbances (Lambe & Whitman, 1979). Since the tested samples of these projects are mostly consists of CL and SC type of soils, disturbance effects during sampling and handling might have affected the results more dramatically. If a best line is fitted for the data, the ratio of the constants before SPT N_f value of best fit and Terzaghi & Peck's proposal can be estimated approximately as 1.67 which may validate the under estimation of c_u values from laboratory tests (Figure 4.5)

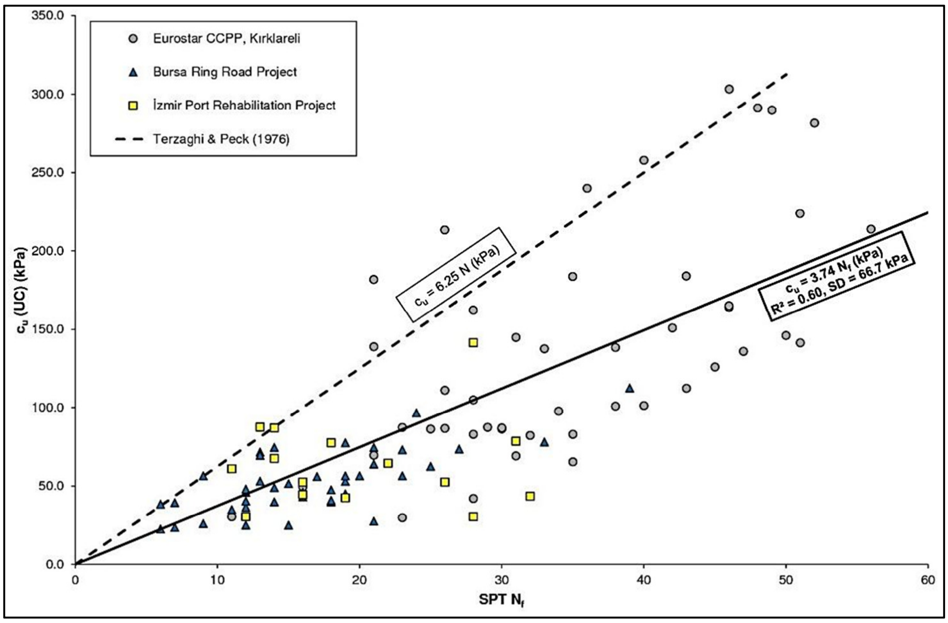

Figure 4.5 Best fit analysis for SPT N_f vs. c_u compared with Terzaghi & Peck (1967)

The scattering of the data is considerable which causes the coefficient of correlation (R^2) to be low (R^2 = 0.6). In addition, scattering seems to be less for the Bursa and İzmir data than the Kırklareli data especially for SPT N_f values larger than 30. This can be related to either disturbance effects on the UC test results of CL type clays or the inefficiency of SPT in hard clays which can be observed for Kırklareli data.

Similarly, SPT N_f and c_u values obtained from UC tests are compared with the equations suggested by Sowers (1979). Differently from Terzaghi & Peck, Sowers classified the soils in to three categories according to their plasticity. Eventhough, Sowers (1979) did not defined the plasticity ranges for the equations, for the sake of comparison, soils are classified according to their plasticity indexes by using British soil classification system (BSCS) for this analysis. Comparison of the values in the database in terms of project and plasticity of soils with the correlations proposed by Sowers are presented in Figure 4.6 and Figure 4.7.

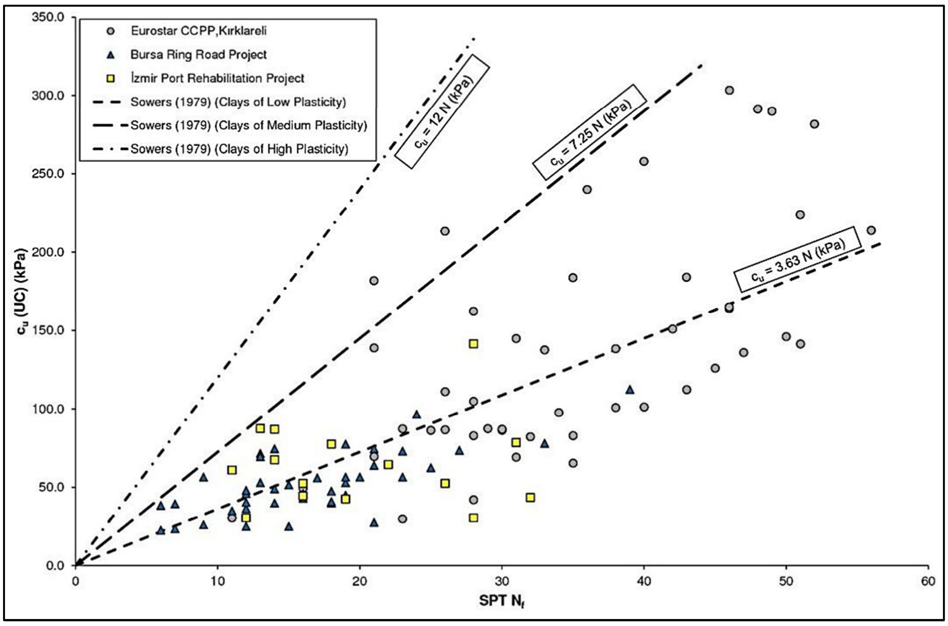

Figure 4.6 SPT N_f vs. c_u for clays compared with Sowers (1979)

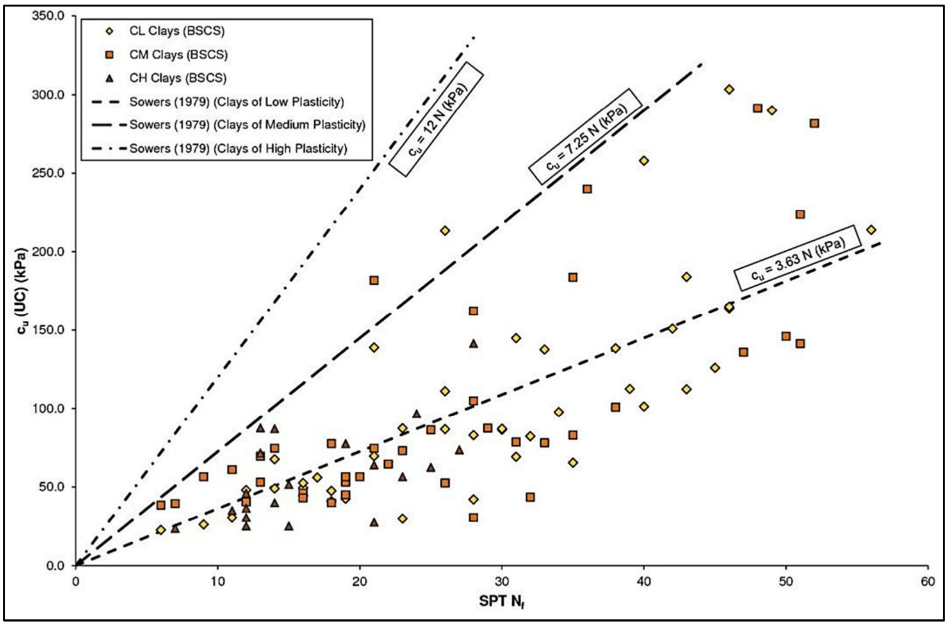

Figure 4.7 SPT N_f vs. c_u according to the classification of BSCS for clays

Results indicate that the data is in good conformity with the equation proposed for clays of low plasticity (Equation 2.3). This can be considered as a reasonable result because the database is mostly consisted of CL type soils. A best line fitted for the soils with low plasticity in the data also gives a similar relationship between SPT N_f and c_u with the Equation 2.3 as given in Figure 4.8.

On the other hand, CM and CH type clays show inconsistencies with the equations presented by Sowers. A best fit line drawn for these soils as given in Figure 4.9 indicates a lower trend than the relationships suggested by Sowers. This can be related to previously mentioned disturbance effects on the soil samples although it is unclear whether the correlations include compensations for disturbances.

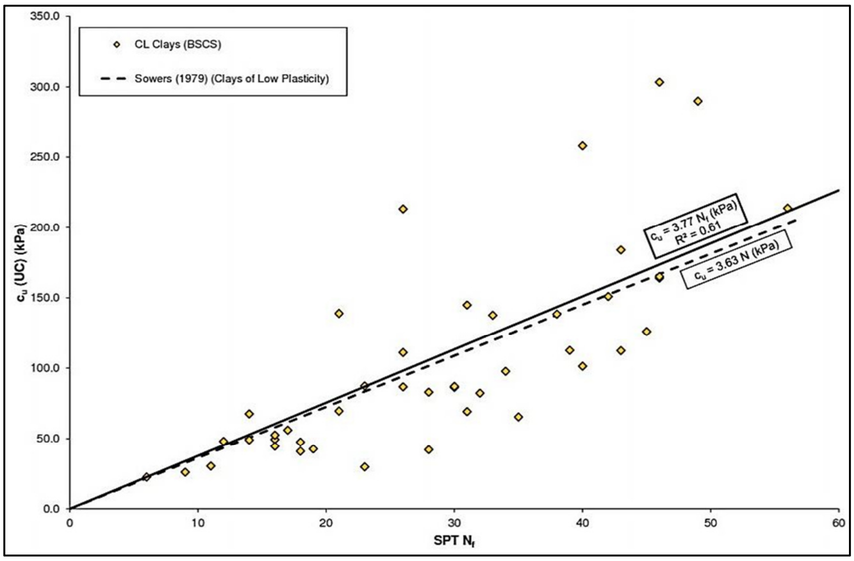

Figure 4.8 SPT N_f vs. c_u for CL clays according to BSCS

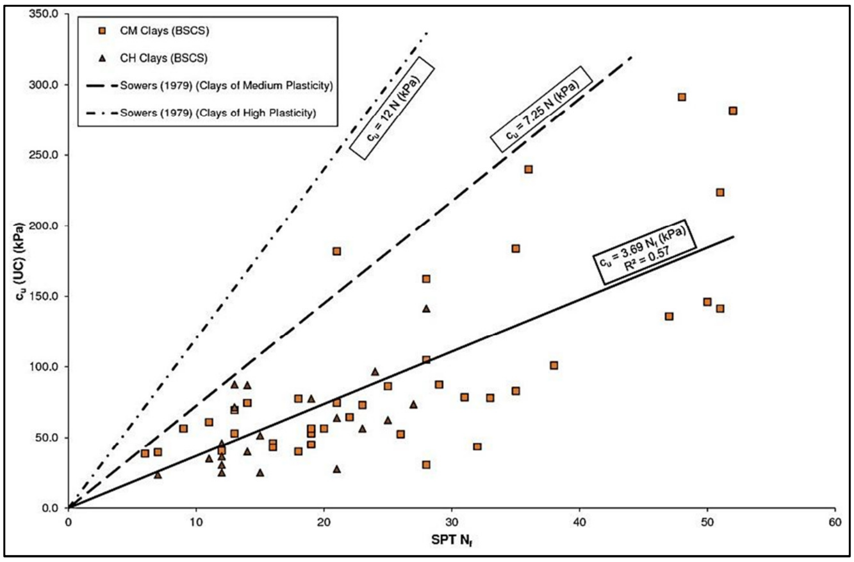

Figure 4.9 SPT N_f vs. c_u for CM and CH clays according to BSCS

Finally, the correlation proposed by Sivrikaya & Toğrol (2007) for SPT N_f and c_u from UC test results (Equation 2.6) was compared with the results of this study as presented in Figure 4.10.

Similar to the other studies, a general compatibility can be observed with the equation proposed by Sivrikaya & Toğrol (2007) and the best fit line of the analyzed soils in Figure 4.10. The difference between the coefficients may mostly related to variations in the soil characteristics of the compared databases and testing equipments for SPT.

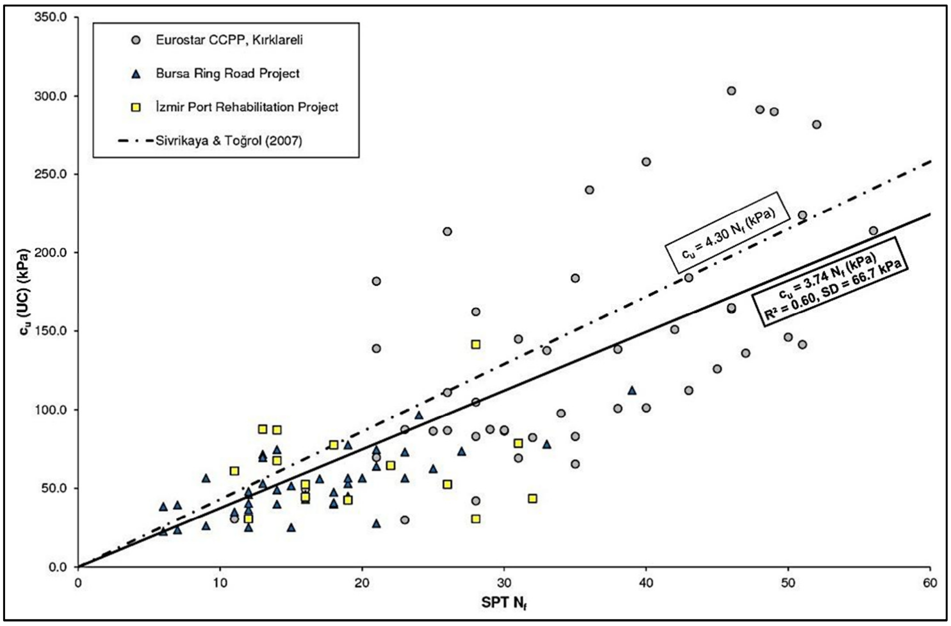

Figure 4.10 Best fit analysis for SPT N_f vs. c_u compared with Sivrikaya & Toğrol (2007) from UC test results

4.2.1.2. Correlations Based on Triaxial Tests

Triaxial tests are considered to be the best method to estimate the in situ undrained shear strength of cohesive soils mainly because of the ability to simulate the real stress and drainage conditions. Triaxial test results in the database were also correlated with SPT N values as done for the unconfined compression tests. In this content, some of the literature studies based on the triaxial tests were evaluated and among them Stroud's (1974) commonly used correlation and the relationship suggested by Sivrikaya & Toğrol (2007) were mainly focused in the analyses.

In the scope of the analyses, a total number of 100 triaxial test results from Eurostar CCPP, Bursa Ring Road Project, Warsaw Subway II Line Project and İzmir Port Rehabilitation Project were compiled and studied. Undrained shear strength values calculated by the Stroud's method were indicated as c_u (Stroud) whereas the corresponding laboratory test results are indicated as c_u (Lab) on the graphs.

Two different approaches are selected for the comparison of the results with the Stroud's work. In the first approach, coefficient of f_1 is determined by using the plasticity index (PI) values of soils according to the graph presented in Figure 2.5. Then, SPT N numbers are multiplied with f_1 coefficients and calculated values of c_u were plotted against the related laboratory triaxial test results for comparison purposes. (Figure 4.11)

As it can be seen from the Figure 4.11, calculated undrained shear strengths are compatible with the laboratory test results up to c_u values of 150 kPa, but for the values larger than 150 kPa significant amount of scattering can be observed especially for Kırklareli and Warsaw data.

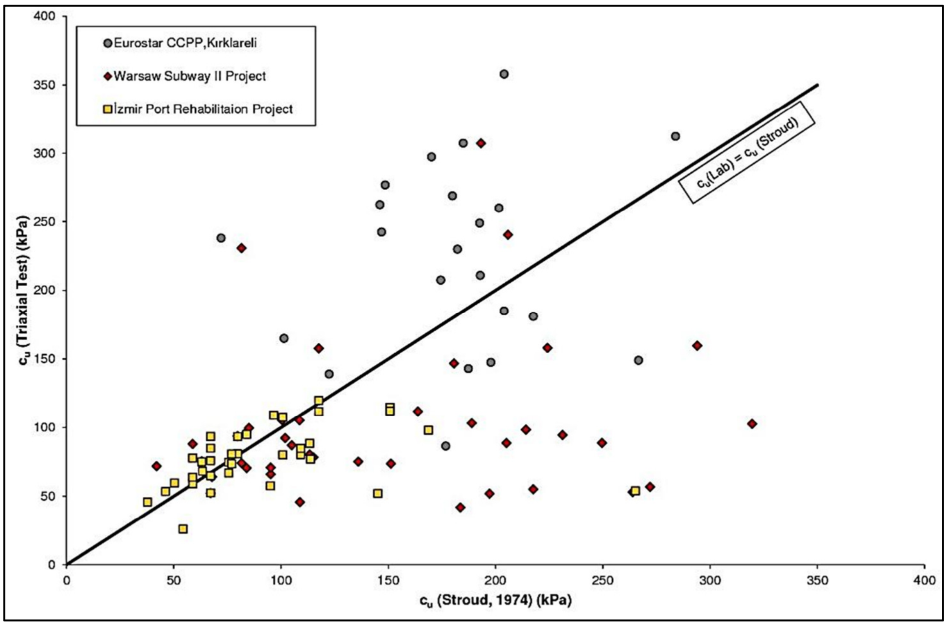

Figure 4.11 c_u (Stroud) vs. c_u (Lab) for analyzed soils

In order to comprehend the range of scatter in the plot, upper and lower limits of the data were determined as given in Figure 4.12. At this stage, some of the data is reduced from the correlation, based on the criteria presented in Section 4.1 to get reasonable limits.

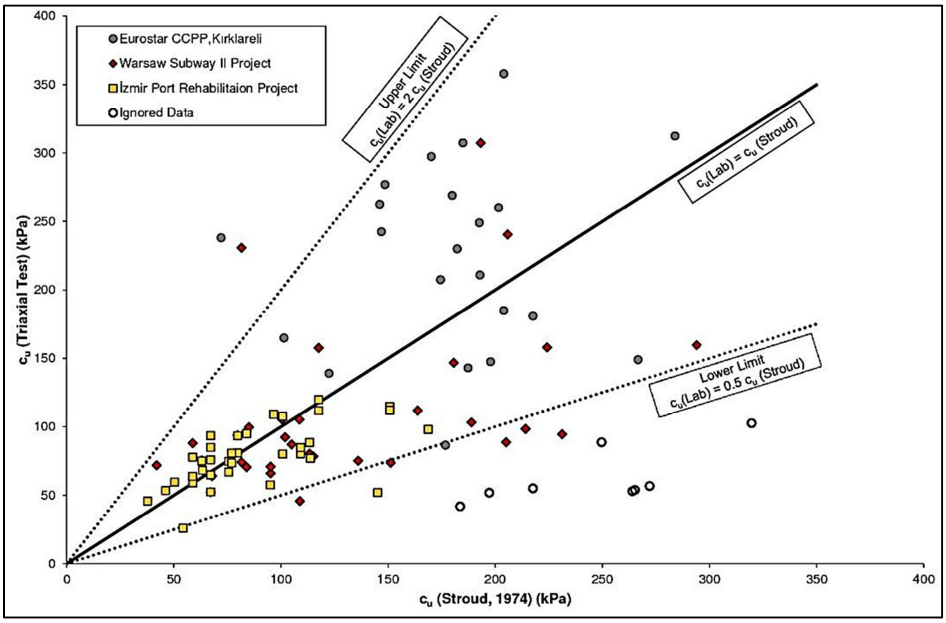

Figure 4.12 Limit ranges of c_u (Stroud) vs. c_u (Lab) for analyzed soils

From the Figure 4.12 it can be stated that the equation proposed by Stroud (1974) can estimate the undrained shear strength somehow up to a certain level but it also may result both over estimations up to 100% and under estimations up to 50% of the c_u values for very stiff or hard clays. The reason of the scatter may be related to the following issues:

- Disturbance effects on the samples as in unconfined compression tests.
- Laboratory tests having $\phi_u > 0$ values.
- Presence of low plasticity clays (CL) in the Kırklareli data that yields to low c_u values in calculations.
- High SPT N_f values of Warsaw data causing overestimation of c_u values compared to in situ strengths.
- Heterogeneity of the sampled soil.

The other method used for comparison of the results with Stroud's correlation was the back calculation of the coefficient f_1 from laboratory test results and SPT N_f values. After then, compatibility of the derived f_1 coefficients (f_1*) were checked by plotting against the corresponding plasticity index (PI) values as given in Figure 2.5. The graph obtained from this method and the study of Stroud (1974) is given in Figure 4.13.

Newly derived f_1* coefficients have a tendency to decrease with the increase in PI as Stroud (1974) suggested but a considerable scattering is also observed in the results.

As done previously, upper and lower limits for the scattering is determined and presented in Figure 4.14 along with the lower limit of Stroud's study. The lower limit established for the obtained values are found to be close to the Stroud's lower limit whereas the upper limit for the data can be considered as very wide.

Additionally, the results of the analyses are presented according to SPT N_f blowcount ranges in the database in order to evaluate the distribution of the f_1* values with respect to the consistency of soils. Resulting plots for the SPT N_f ranges of $0 < N_f < 15$, $15 < N_f < 30$ and $N_f > 30$ are given in Figures 4.15, 4.16 and 4.17 respectively.

From the Figures 4.15 4.16 and 4.17, it can be stated that the stiff to very stiff clays having SPT N_f blowcount between 15 and 30 in the database showed a more compatible distribution to the graph proposed by Stroud (1974). This result can be considered as reasonable because the database of the Stroud's study was mainly consisted of stiff, very stiff and hard clays.On the other hand, hard clays with SPT N_f larger than 30 shows a rather more scattered distribution than the stiff to very stiff clays which is possibly due to the higher disturbance effects and inconsistent SPT N_f values as mentioned in the previous sections.

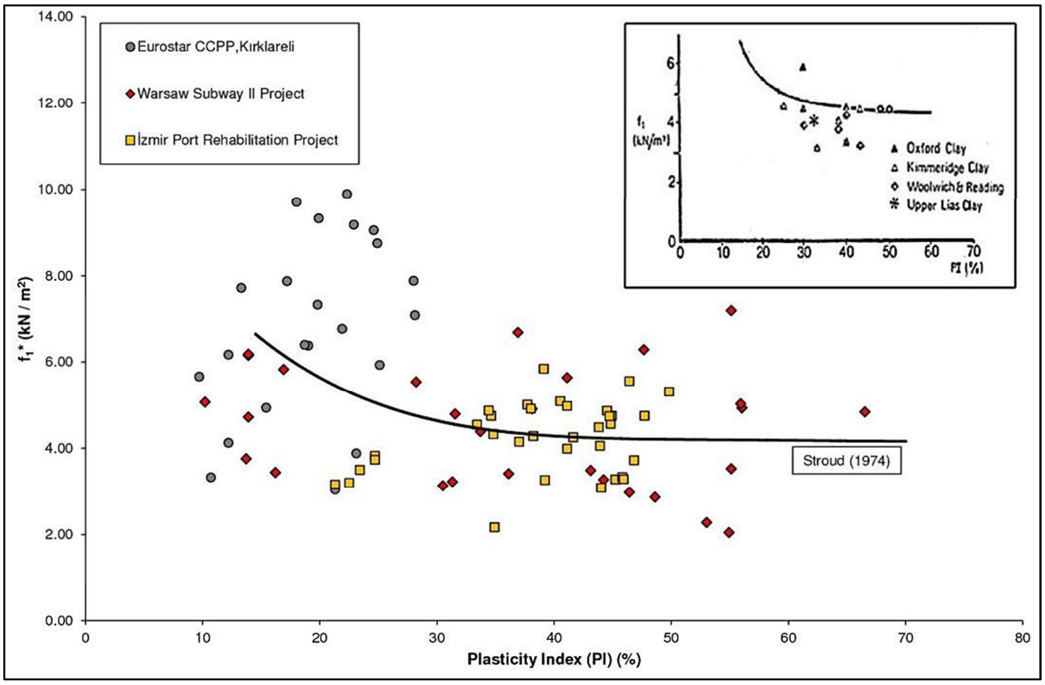

Figure 4.13 Comparison of derived f_1* and f_1 (Stroud, 1974) for analyzed soils

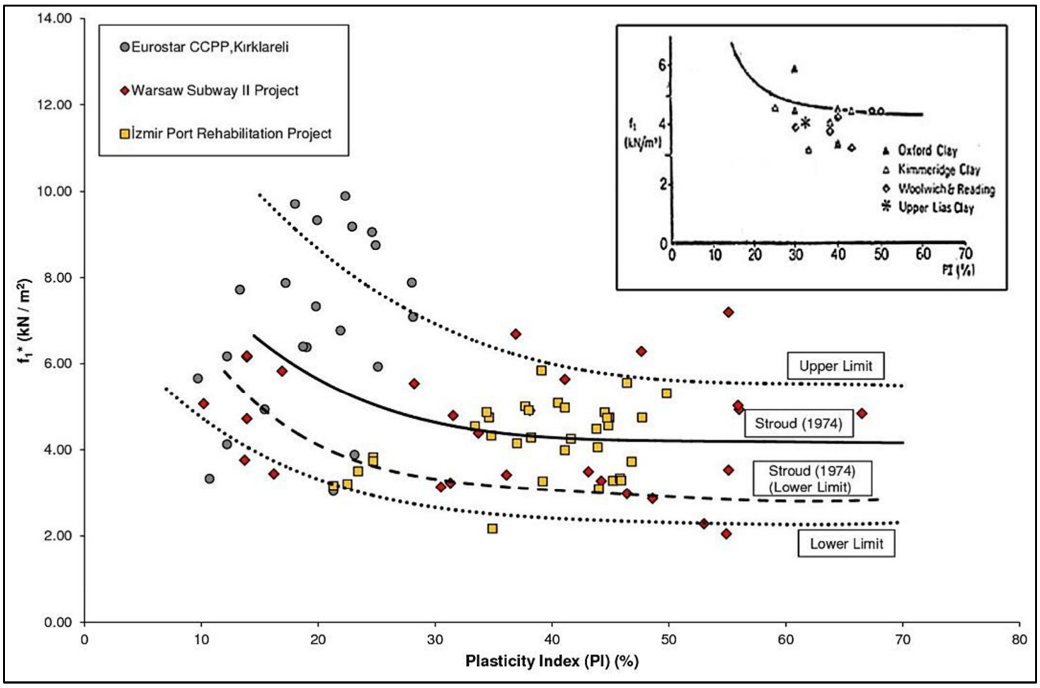

Figure 4.14 Limit ranges of derived f_1^* and f_1 (Stroud, 1974) for analyzed soils

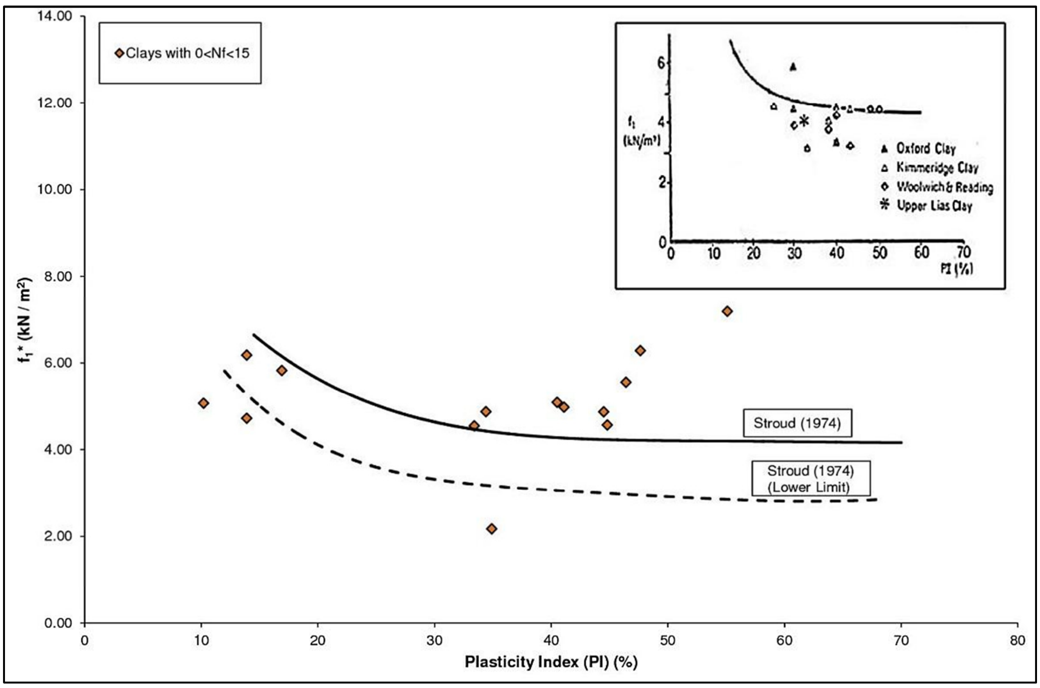

Figure 4.15 Distribution of f_1^* values for SPT N_f values less than 15 for analyzed soils

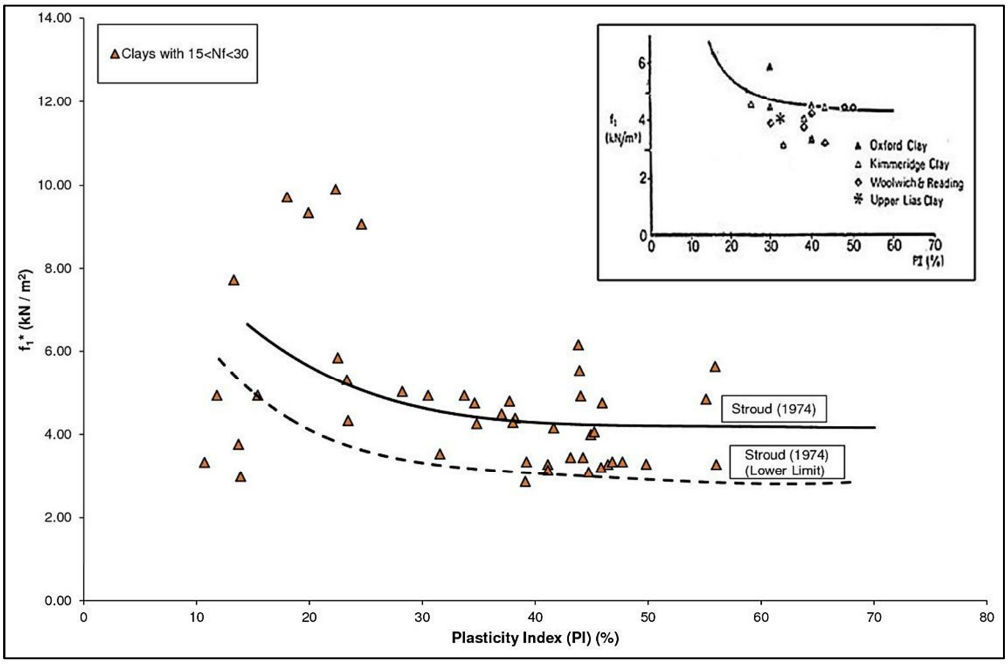

Figure 4.16 Distribution of f_1* values for SPT N_f values between15 and 30 for analyzed soils

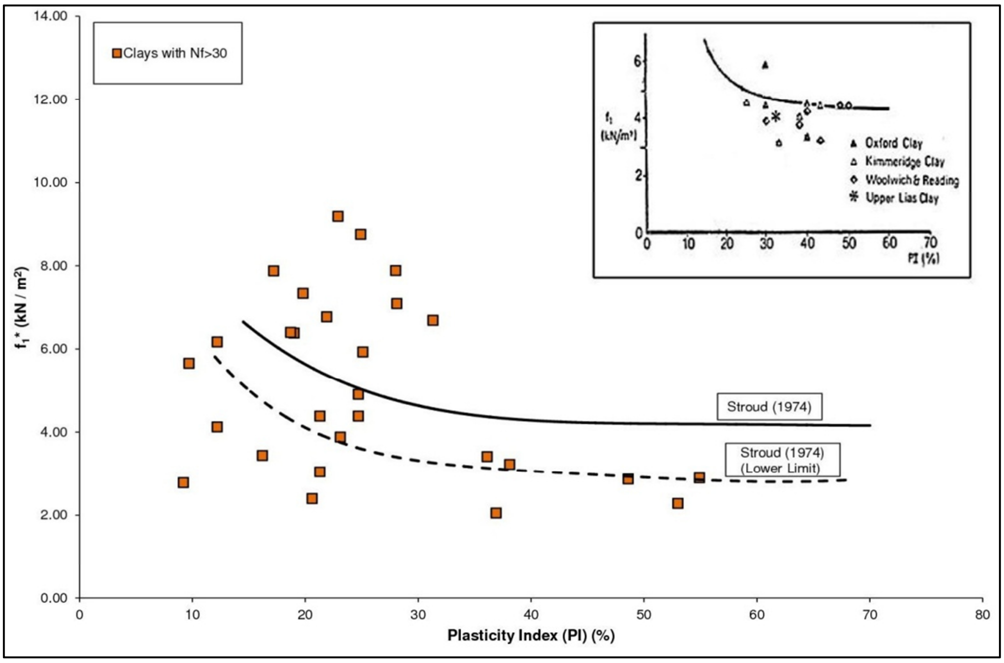

Figure 4.17 Distribution of f_1* values for SPT N_f values between15 and 30 for analyzed soils

The relationship between SPT N_f and c_u from triaxial test results were also compared with the study of Sivrikaya & Toğrol (2007) that is given in Equation 2.7. As from the results presented in Figure 4.18, the relationship between SPT N_f and c_u were found to be very compatible with the study of Sivrikaya & Toğrol (2007) especially for the stiff – very stiff clays in the analyzed soil database whose SPT N_f values changing between 10 and 30.

On the other hand, hard clays with SPT N_f larger than 30, showed a considerable scatter, particularly for Kırklareli data, which yielded the overall coefficient of correlation to be low (R^2 = 0.40). The significant variation in the c_u values of hard clays in the database can be considered as an expected result because of the pronounced effects of sample disturbances and heterogeneity of the tested samples which is mostly valid for the Kırklareli clays due to presence of clayey sand (SC) units in the hard clay layers.

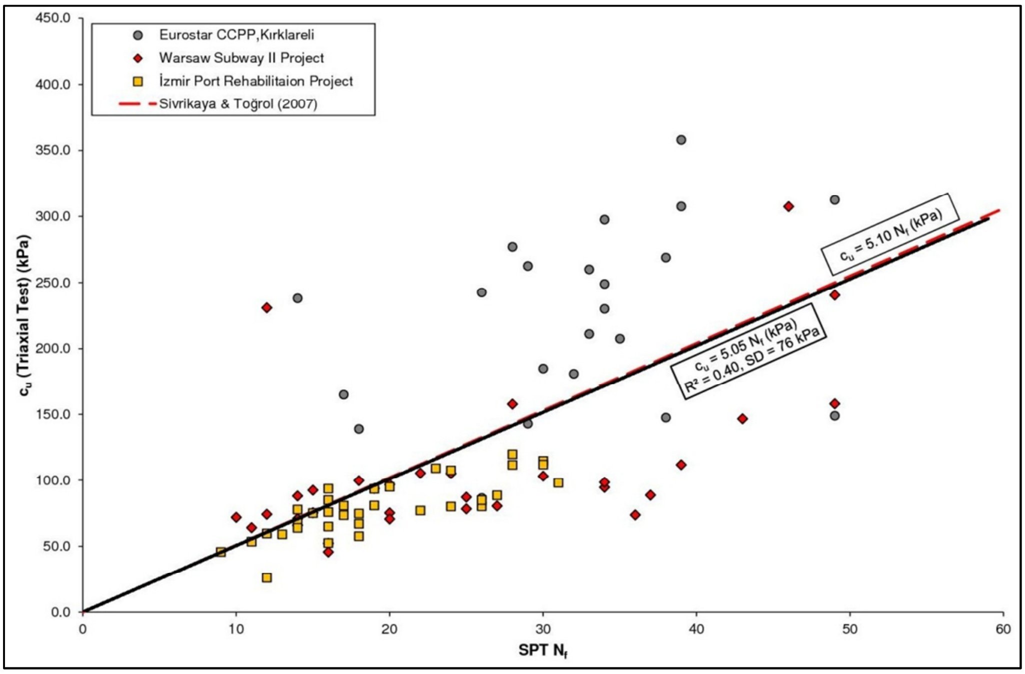

Figure 4.18 Best fit analysis for SPT N_f vs. c_u compared with Sivrikaya & Toğrol (2007) from triaxial test results

4.2.2. Estimation of Coefficient of Volume Compressibility (m_v)

Prediction of coefficient of volume compressibility (m_v) directly from SPT N values were performed in accordance with the commonly used equation proposed by Stroud (1974) (Equation 2.15). A total number of 157 data from Eurostar CCPP, Bursa RRP, Warsaw Subway II Line Project and İzmir Port RP were considered in the analyses. The methods used for determination of undrained shear strength from triaxial tests were also accepted for the estimation of m_v.

By using plasticity index (PI) of soils, firstly the coefficient of f_2 is calculated. After then, f_2 values are multiplied with the corresponding SPT N_f values to estimate the m_v values as stated in the Equation 2.15. On the other hand, m_v values of undisturbed soils were obtained from oedometer tests for the suitable effective stress ranges. The plot of the estimated m_v values from Stroud's correlation and oedometer tests are plotted on the same graph as given in Figure 4.19. A similar notation with the undrained shear strength analyses was used in the plots for which m_v values calculated by Stroud's method is denoted as m_v (Stroud) and the values obtained from laboratory tests as m_v (Lab).

At the first impression from Figure 4.19, m_v coefficients determined from oedometer tests seems to highly overestimate the values calculated from Stroud's correlation. For a detailed assessment, upper and lower limits for the data were determined as shown in Figure 4.20. The upper limit of the data indicates an over estimation up to 250 %, which is a serious amount, whereas an under estimation is observable only up to 20% with a small effect on the general trend.

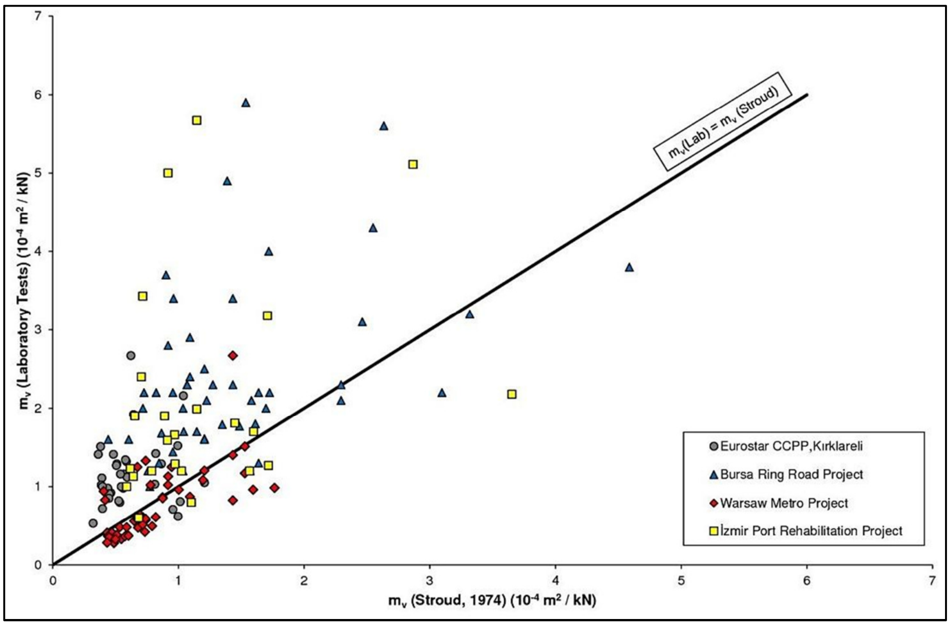

Figure 4.19 m_v (Stroud) vs. m_v (Lab) for analyzed soils

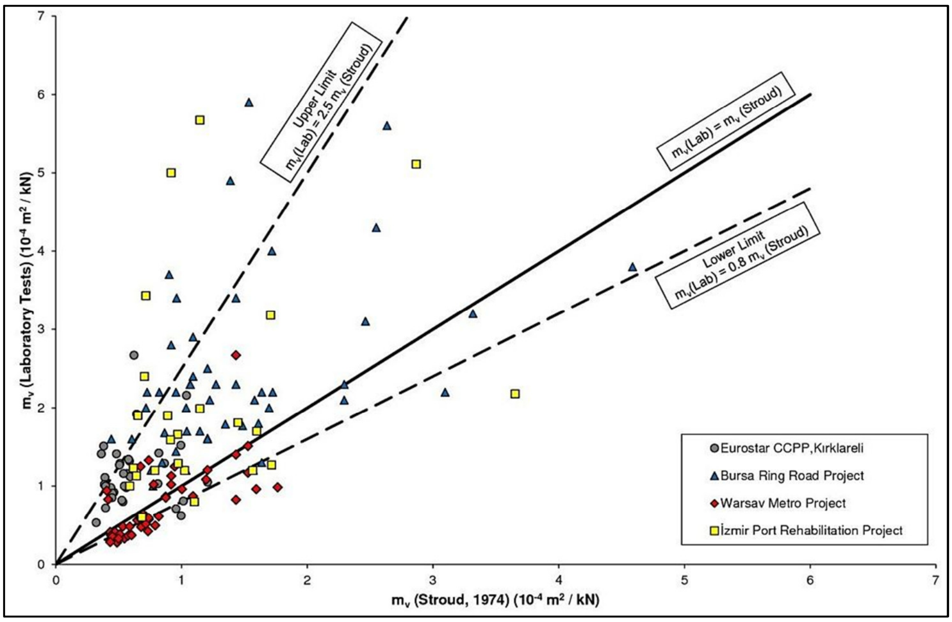

Figure 4.20 Limit ranges of m_v (Stroud) vs. m_v (Lab) for analyzed soils

Significant amount of over estimations would likely to be due to following issues in general:

- Disturbances on the tested soil samples have been known to affect the outcome of the laboratory test results.

- Release of in situ stresses during sampling may have affected the compressibility characteristics since sampling of soils were performed up to 40 meters depth in all of the projects. For instance, a sample taken from deeper parts of the soil mass would show more relaxations than a sample obtained a location closer to the surface. As a result, recovered deformations due to relaxations might have been measured instead of the deformations under the desired ranges of stresses.

- Differences in saturation conditions between samples and in situ state can be considered an important factor that could drastically affect the compressibility properties. Although all of the laboratory test samples were saturated by flooding and back pressure methods, the soil might be in partially saturated state at the site. Effects of saturation can especially be observed for the Kırklareli data where the ground water level is approximately 20.0 meters below the surface.

- Inefficiencies of oedometer test to represent the real compressibility conditions can be considered as one the reasons for deviation and scattering. One of the major factors affecting the oedometer test results is frictional forces on the sides of the sample. As loading of the sample proceeds, shear forces developed on the sides disturbs the one dimensional strain state of the sample which may result in unequal stress distributions between top and bottom of the specimen. This could have caused the high m_v values in the database.

As for the second method of analysis, f_2 coefficient is back calculated from laboratory m_v values and related SPT N_f numbers. Similarly done for the estimation of c_u in previous section, newly derived f_2 coefficients (f_2*) are plotted against the plasticity index (PI) as given in Figure 4.21 and the results were compared with the graph (Figure 2.6) suggested by Stroud (1974).

With respect to the high m_v trend of laboratory tests, most of the derived f_2* values fall below the Stroud's f_2 curve and considerable amount of scatter can be observed. In order to get a rough estimate for the scattering, upper and lower limits are defined for the plot which is shown in Figure 4.22. Limits on the studied data indicate a very wide range especially for the case $f_2^* < f_2$ (Stroud).

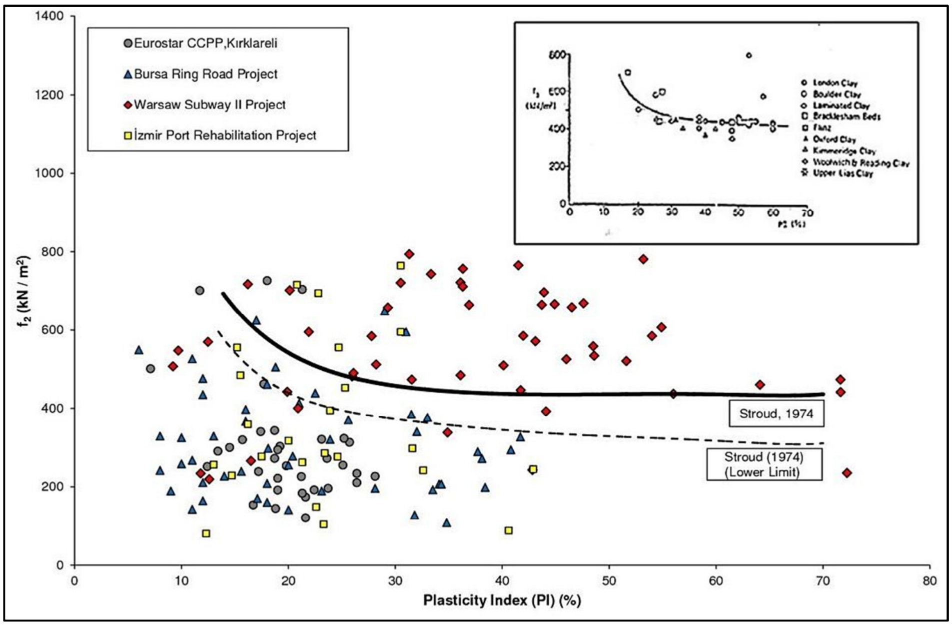

Figure 4.21 Comparison of derived f_2* and f_2 (Stroud) for analyzed soils

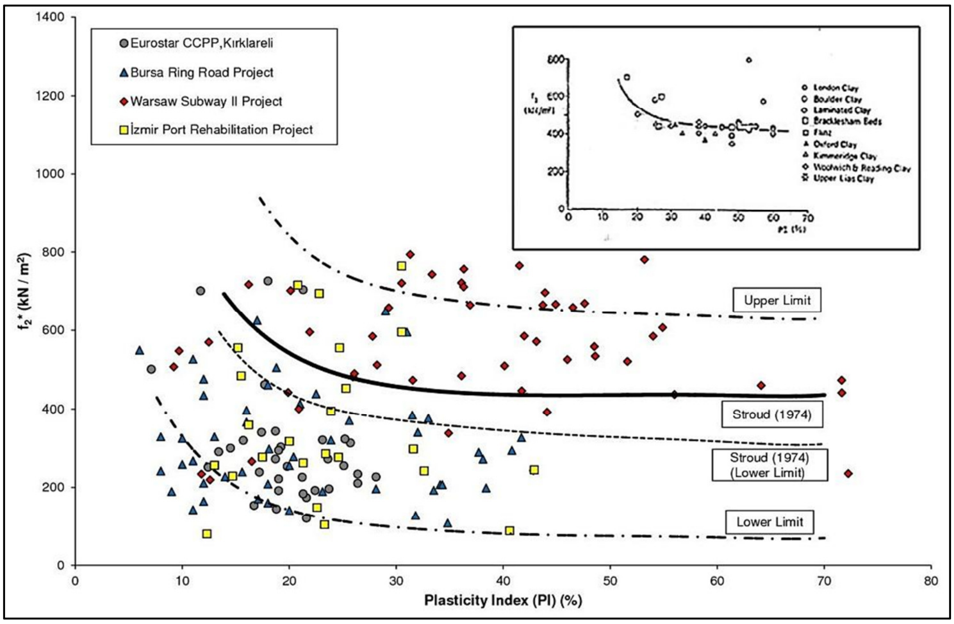

Figure 4.22 Limit ranges of derived f_2* and f_2 (Stroud) for analyzed soils

4.2.3. Estimation of Deformation Modulus (E_s)

Deformation modulus of clays was estimated by using the coefficient of volume compressibility (m_v) values from oedometer tests. However, it is well known that during the oedometer test, sample is confined from sides and only vertical deformation of the sample is allowed. Thus, m_v of the oedometer tests defines the compressibility of soil only in vertical direction. Even though oedometer represents the vertical loading conditions for a soil mass at the site, deformability of soil on the other directions are also needs to be reflected for a more accurate analysis. In the scope of this study, vertical deformability characteristics (m_v, M) were converted to three dimensional parameters by using the theory of elasticity as given in Equations 2.13 and 2.14 with a reasonable assumption of $\nu = 0.3$.

Correlations of SPT N_f and deformation modulus (E_s) were analyzed by using the same database discussed for estimation of volume compressibility coefficient. The plot of SPT N_f and calculated E_s values are given in Figure 4.23 with upper, lower limits and the best fitting line.

The upper and lower limits indicate a very wide range for the E_s values varying between 150 N_f and 500 N_f. Variations in the results were observed to be larger for very stiff to hard clays. Best fit line indicates an approximate average value of 295 N_f with a low coefficient of correlation ($R^2 = 0.46$). The scatter and wide range of E_s values was expected, especially for stiff and hard clays, because of sample disturbances, heterogeneity of soils, assumption of ν and various factors affecting SPT N results. Furthermore, best line fitted for the data indicates a weak correlation meaning that the real parameter might be quite different from the estimation.

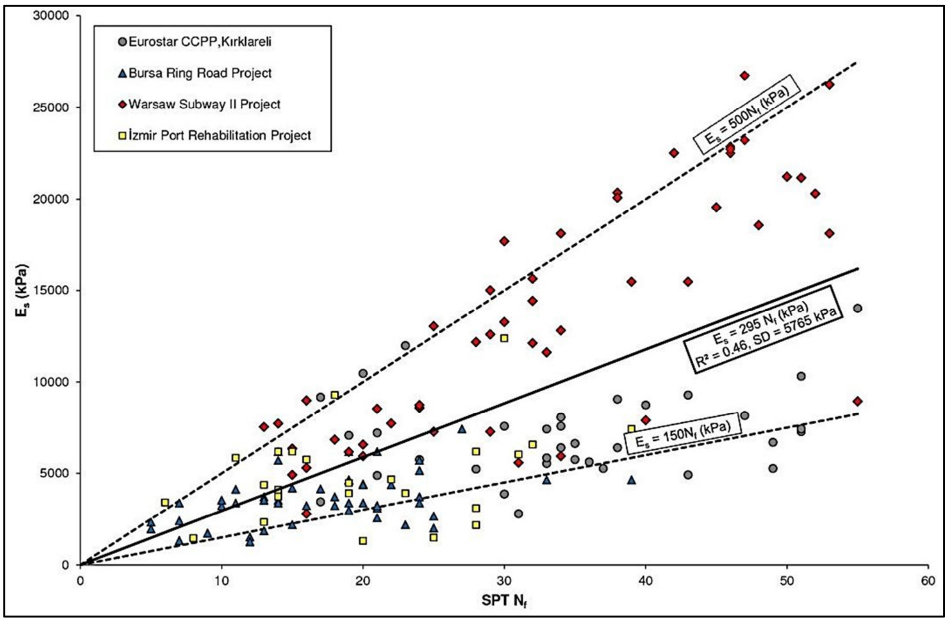

Figure 4.23 Limits and average of SPT N_f vs. derived E_s for analyzed soils

4.3. Assessment of Parameters from CPT Results

Cone penetration test (CPT) test has become widely used for estimations of geotechnical parameters beside the direct geotechnical design applications. Total cone tip resistance (q_c) data of CPT was mainly used for the prediction of undrained shear strength (c_u) and deformation characteristics by the means of coefficient of volume compressibility (m_v) was analyzed in the scope of this study.

4.3.1. Estimation of Undrained Shear Strength (c_u)

Estimations of undrained shear strength from CPT test results were mainly based on the Equation 2.16 as commonly referred in the literature. For this purpose, a total number of 75 CPT data with related laboratory tests were gathered from Warsaw Subway II Line project and Braila WWTPP in Romania. According to the studies in the literature, c_u is related with q_c by a factor of N_k. In order to determine the N_k factors, net cone resistance ($q_c - \sigma_{v0}$) values were plotted against the c_u values of laboratory test results by using the Equation 2.16 (Figure 4.24).

The scattering of the data is considerable when the plot in the Figure 4.24 is inspected but an increasing trend of undrained shear strength can be observed as the net cone resistance increase which is an expected result. To evaluate the ranges of N_k values of the database, upper and lower limits are determined along with the average values in Figure 4.24.

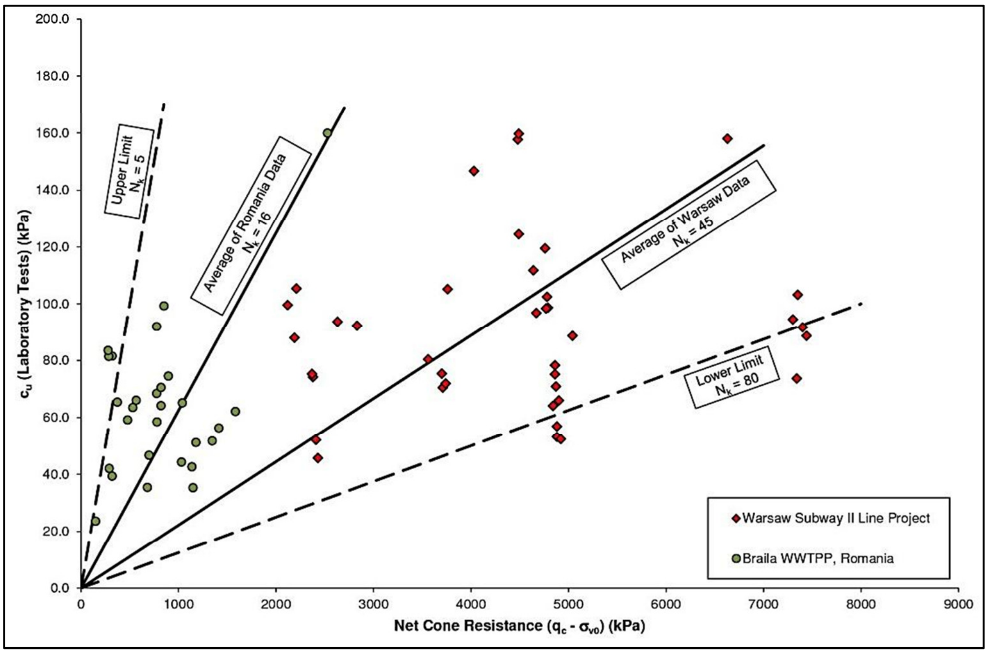

Figure 4.24 Limits and averages of q_c vs. c_u for analyzed soils

First impression of the given plot in Figure 4.24 is that the Romanian data shows less scatter with an average N_k value of 16. The reason for this can be explained by the consistency between net cone tip resistance and undrained shear strength. On the other hand, for higher values of net cone resistance, as in the data of Warsaw, scatter reaches a significant level with an average N_k value of 45. This can be considered as a result of low c_u values from laboratory tests that prevent the data to be consistent with q_c values. Either way, regarding the general limits given in Figure 4.24, the range of N_k values in the database are found to be very wide.

The results of the study between cone tip resistance (q_c) and undrained shear strength (c_u) was also compared with some of the suggested N_k values in the literature (Figure 2.9). Resulting plot is given in Figure 4.25. The N_k values recommended in the literature also show great variations so the studied database seems to be in the limits of the literature values. The reason for such differences in N_k values can be related to factors such as inefficient idealization of q_c data, absence of pore water corrections, inconsistent reference strengths from different laboratory tests, disturbances on the laboratory samples, differences in the results of cone types etc.

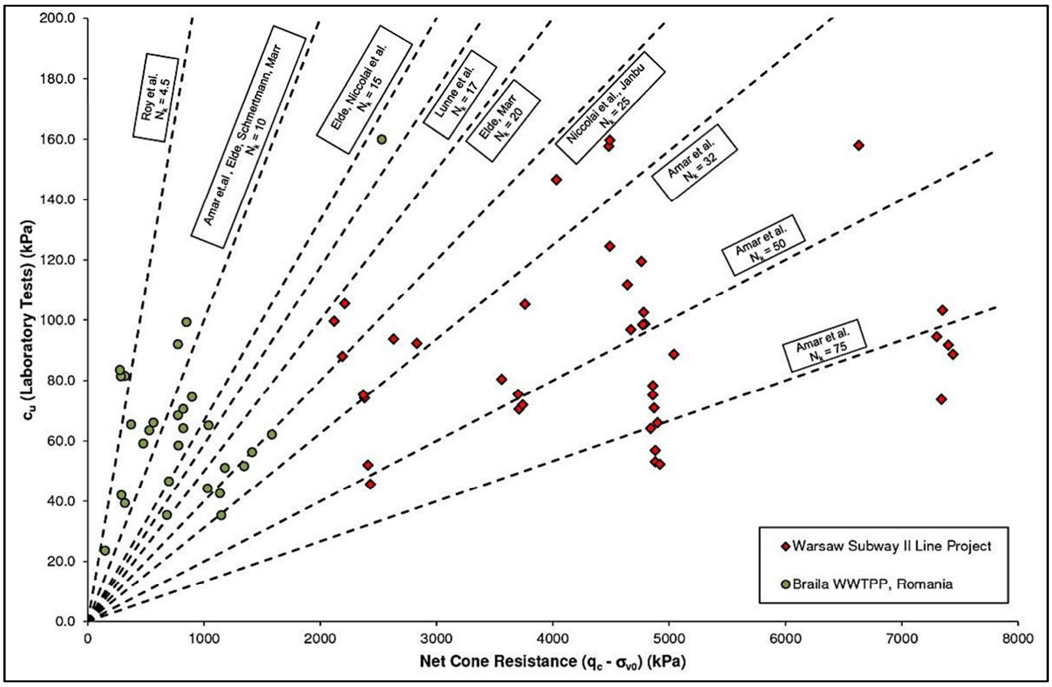

Figure 4.25 Comparison of the q_c vs. c_u with the literature studies for analyzed soils

4.3.2. Estimation of Coefficient of Volume Compressibility (m_v)

Predictions about compressibility characteristics from CPT are also preffered as the prediction of undrained shear strength in the geotechnical practice. According to the extensive studies about the subject, coefficient of volume compressibility (m_v) or one dimensional deformation modulus (M) is found to be related with the cone tip resistance (q_c) by a factor of α as given in Equation 2.17.

The analyses in this study are also performed in a similar manner and α value in the equation was tried to be estimated by using the results of oedometer tests and CPT q_c values. A total number of 108 data from Warsaw Subway II Line project and Braila WWTPP in Romania is analyzed for this purpose. By using q_c and the corresponding m_v values in the database, α coefficients were calculated according to the Equation 2.17. In order to evaluate the differences in the relationship, soil types (CL and CH) in the database were analyzed separately (Figure 4.26 and 4.27).

Regarding the plots given in Figure 4.26 and 4.27, variation in the ranges of α value with respect to q_c can be determined for CL and CH soils seperately. Graphical inspections indicate that the α coefficient varies between 3 – 7.5 for q_c < 2000 kPa, 2.5 – 7 for 2000 < q_c < 5000 kPa and 3.5 – 6 for q_c > 5000 kPa values for CH type of clays. (Figure 4.26)

The contraction in both upper and lower limits of the α coefficient up to q_c = 5000 kPa for CH clays could mostly be related to decrease in compressibility with the increase in stiffness of soils which is an expected result. On the other hand, eventhough the upper limit of the α coefficient decreases for q_c > 5000 kPa values, lower range of the values increase with respect to 2000 < q_c < 5000 kPa range. Unexpected increase in the lower limit of α values for q_c > 5000 kPa range might be related to the accuracy of the oedometer test results for stiff clays due to disturbances. Furthermore, release of the in situ confining stresses for the samples taken from deeper levels might have caused the oedometer test results to overestimate the compressibility.

On the other hand, despite the low amount of data regarding the CL type of clays, variation of α coefficient were found to be vary between 2 – 5 regardless of the cone tip resistance, q_c (Figure 4.27).

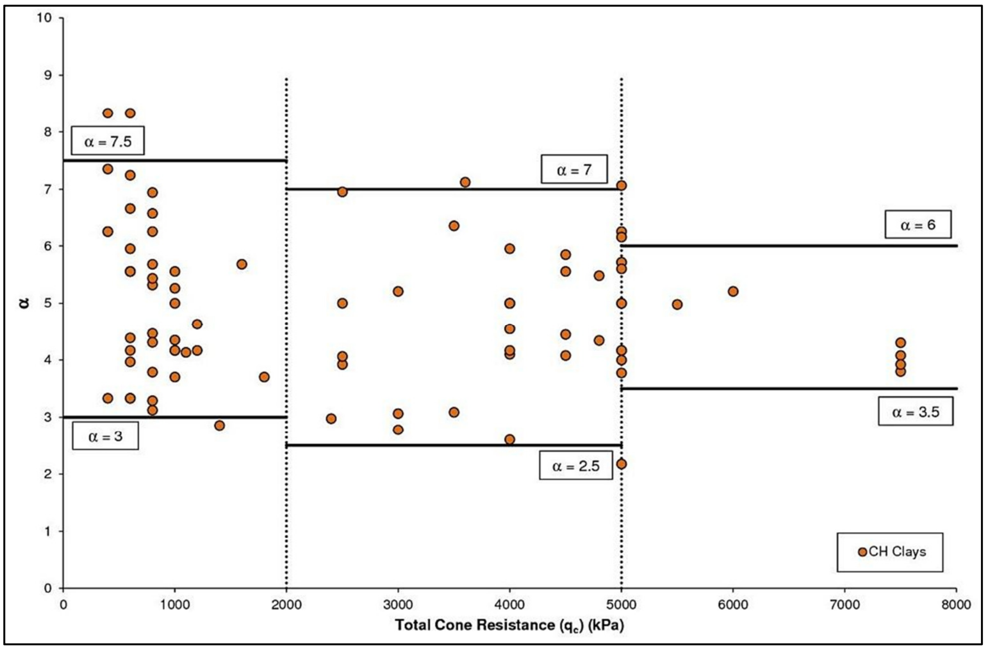

Figure 4.26 Ranges of derived α values for CH clays

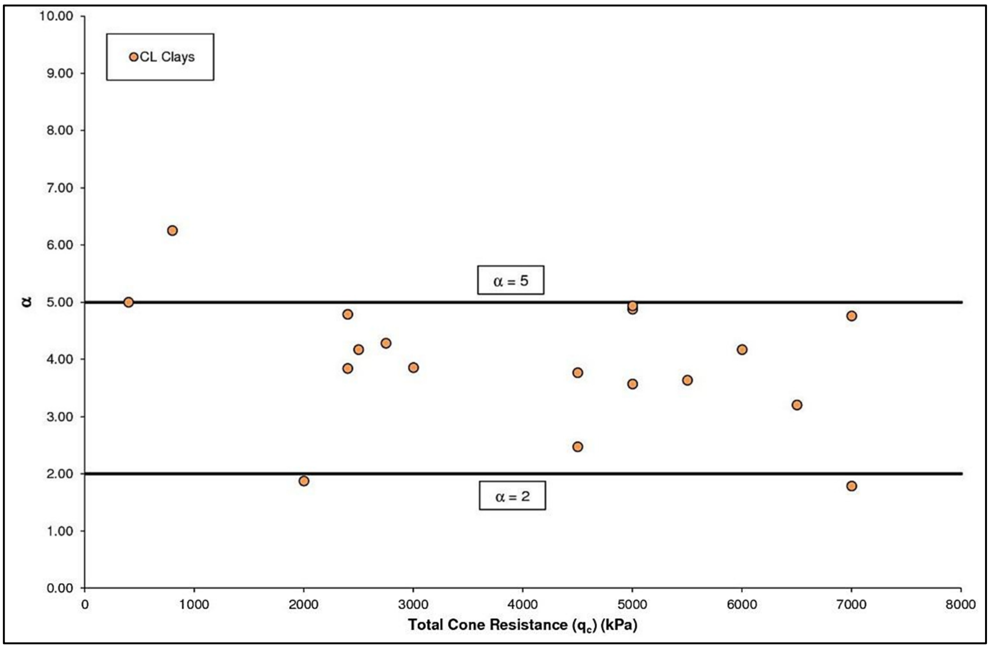

Figure 4.27 Ranges of derived α values for CL clays

The results were also compared with the ranges suggested in the literature. Sanglerat (1972) proposed a detailed range of α coefficient for various soil types (Table 2.4). Ranges of α coefficient for CL and CH type of soils were plotted on the estimated values for comparison purposes (Figure 4.28 and 4.29). Since there are significant numbers of $q_c > 2000$ kPa values in the database, α coefficient was extended according to q_c values, instead of limiting at the 2000 kPa as in Sanglerat's study (1972).

A general compatibility for the suggested limits of Sanglerat (1972) and the calculated α coefficients can be seen from the Figure 4.28 regarding the CH type of clays. Although, Sanglerat did not extend the limits for CH type of soils with $q_c > 2000$ kPa, most of the data stays in the range of $\alpha = 2 – 6$.

On the other hand, Figure 4.29 representing the CL soils are determined to be completely out of the ranges recommended by Sanglerat (1972). In addition to low amount of data for CL clays, there are some possible factors that could have caused the incompatibility such as overestimation of m_v values due to in situ stress release effects and disturbances on CL clay samples as previously mentioned for CH clays.

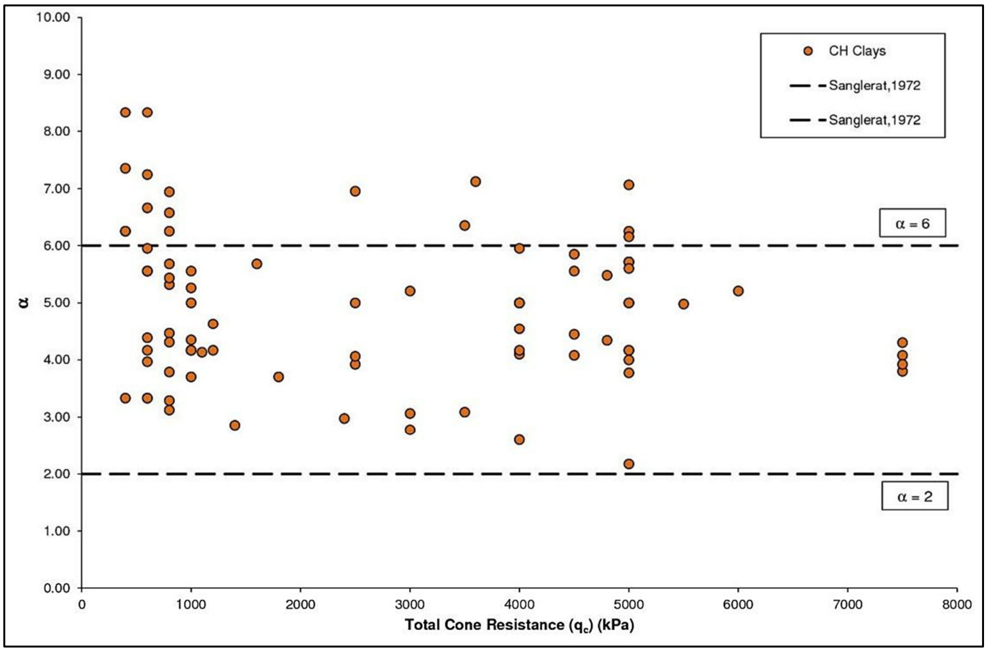

Figure 4.28 Comparison of derived α values with the ranges suggested by Sanglerat (1972) for CH clays

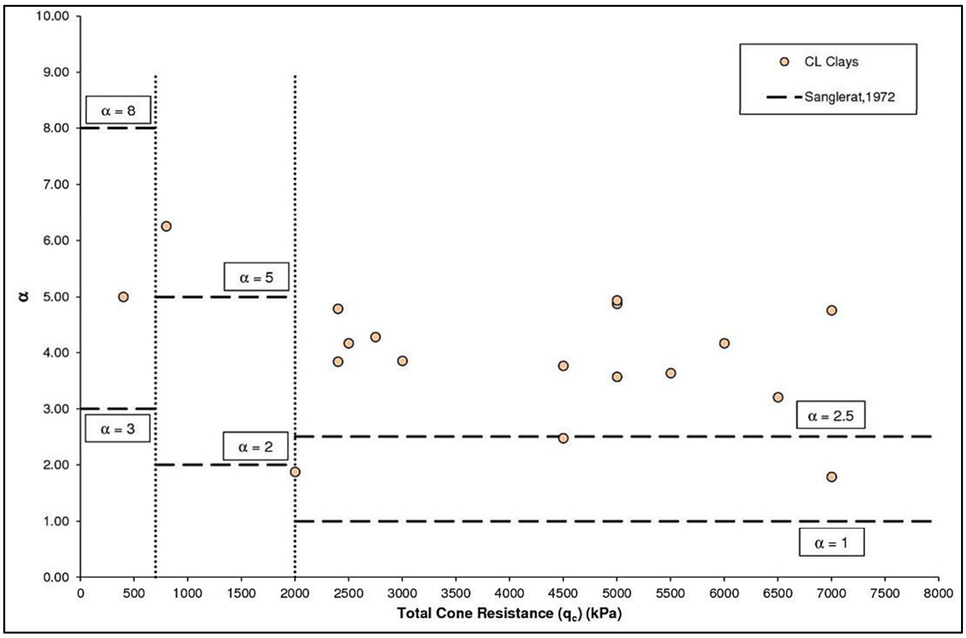

Figure 4.29 Comparison of derived α values with the ranges suggested by Sanglerat (1972) for CL clays

In a similar study performed by Kaplan et al. (2004), ranges of α coefficient was determined from Iskenderun and another Romanian site. According to Kaplan's study, α coefficient varies between 4 – 12 with a tendency to decrease as the tip resistance increases up to a q_c value of 750 kPa. It is also stated that the α coefficients stayed in a range of 2.7 – 4.7 independent form the q_c value. Since the limits did not differentiated according to soil types, the entire database was plotted at the same graph. In addition, as previously done, limits of α coefficient were extended according to q_c values in the database. Comparison of the derived α coefficients with the study by Kaplan et al. (2004) is presented in Figure 4.30. Ignoring a few points, most of the data can be considered to be in agreement with the ranges proposed by Kaplan et al. (2004).

Mayne (1990) recommended a more general approach by using cone data of 12 different sites (Figure 2.12). Since their study correlates the cone tip resistance with one dimensional modulus (constrained modulus), M, m_v values obtained from laboratory test results are converted to M by taking the reciprocal for an appropriate comparison. Best fit line of the data and the equation proposed by Mayne (1990) is given in Figure 4.31.

Although, Romanian data with low q_c values seems to be compatible with the study of Mayne (1990), almost all of the Warsaw data points fell below the compared correlation line by an approximate mean ratio of 1.75 as presented in Figure 4.31. A number of factors can be discussed about the deviations, which are most likely the characteristics of tested soils and differences in test conditions related to both CPT and laboratory. However, it can be concluded that a similar relationship to Mayne's exists between the net cone resistance and one dimensional modulus (M) of studied soil conditions.

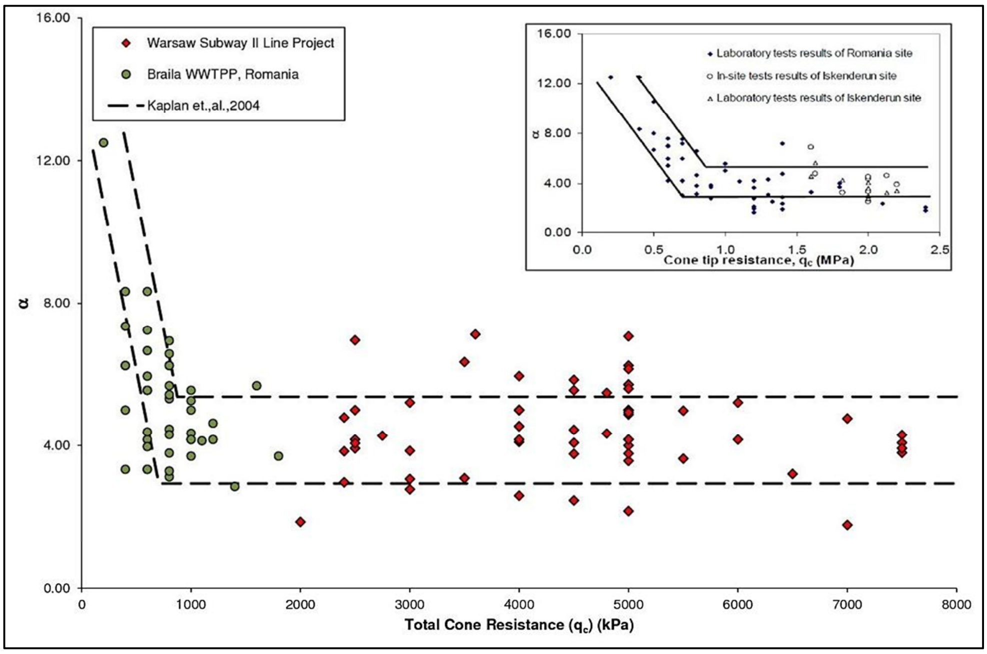

Figure 4.30 Comparison of derived α values with the ranges suggested by Kaplan et al. (2004)

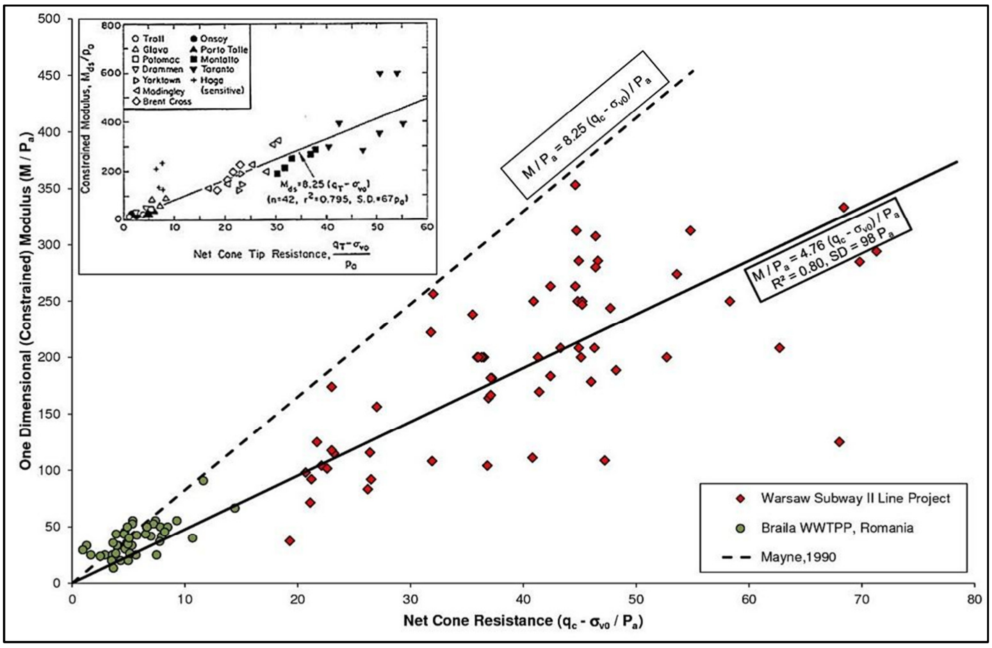

Figure 4.31 Comparison of M values with Mayne (1990)

4.3.3. Estimation of Deformation Modulus (E_s)

The number of correlations between cone tip resisitance (q_c) and deformation modulus (E_s) is small compared to correlations with one dimensional modulus (M) mainly due to nonhomogenity of soil, uncertainties regarding the Poission's ratio and anisotropical behavior of soil. Yet the results of oedometer tests are converted to deformation modulus (E_s) by using the Equation 2.13 with assuming a Poisson's ratio of $\nu = 0.3$ and analyzed with related cone tip resistance values (q_c).

In order to comprehend the magnitude of the relationship, a best fit for the data was established and predictions were compared with the recommended correlation proposed by Bowles (1997) for clays in Equation 2.20 (Figure 4.32).

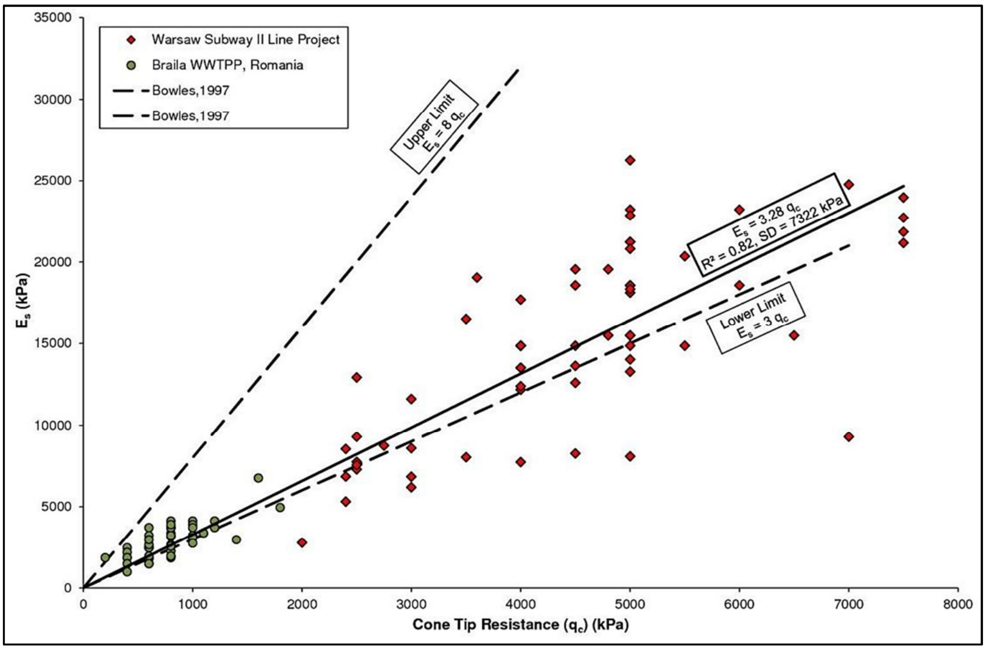

Figure 4.32 Comparison of q_c vs. E_s with Bowles (1997) for analyzed soils

A clear trend can be seen at the first impression for which the deformation modulus (E_s) increases with increasing cone tip resistance (q_c). in Figure 4.32. A good correlation was obtained by the best fit line of the analyzed soils which is observed to be very close to the lower limit proposed by Bowles (1997). However, the points below the best fit line also indicates that, even the lower limit value suggested by Bowles (1997) may overestimate the deformation modulus (E_s) by a certain amount. In any condition, it can be concluded that, cone tip resistance may be utilized for a prediction of deformation modulus if the overestimation possibility is taken into account.

4.4. Assessment of Parameters from PMT Results

Pressuremeter tests (PMT) have been become one of the commonly used tests for the estimation of geotechnical parameters. The results of PMT in the studied database were mostly analyzed to estimate the undrained shear strength (c_u) and deformation modulus (E_s) of cohesive soils.

4.4.1. Estimation of Undrained Shear Strength (c_u)

Predictions of undrained shear strength (c_u) from net limit pressure (p_{ln}) of PMT generally expressed as the ratio of p_{ln} / c_u which is referred as β in the literature. Therefore, analyses in this study were also performed with a similar methodology. PMT database is limited with a total number of 55 data obtained from Eurostar CCPP and Bursa RRP, along with UC and triaxial tests.

In order to estimate the β coefficient, net limit pressure (p_{ln}) and corresponding undrained shear strength (c_u) values obtained from laboratory testing are plotted with the upper and lower limits and best fitting average line in Figure 4.33.

From the plot of p_{ln} vs. c_u, a typical proportional relationship can be easily observed with a variation of β = 4 for the upper limit, β = 15 for the lower limit. The best linear fit for the data indicates the β = 11. This can be considered as a validation of a linear correlation with the β coefficient. In the following stages of this section, comparison of β values with other correlations in the literature (Section 2.4.5.2.1) is done.

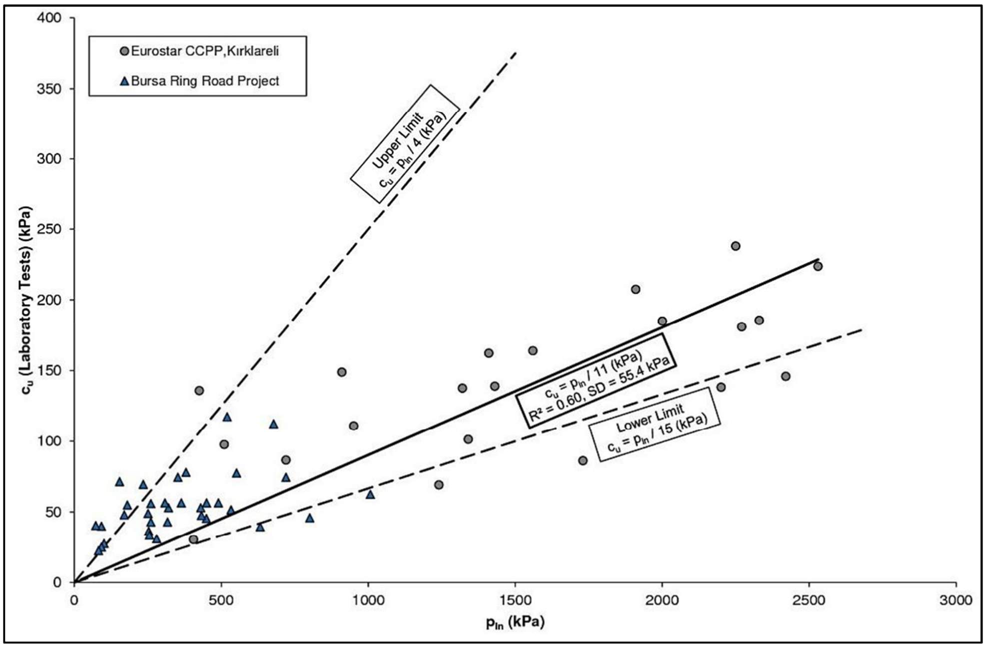

Figure 4.33 Limits and average of p_{ln} vs. c_u for analyzed soils

Comparison of the results regarding the β coefficient is firstly done with the graphical representation of compiled studies of Higgins (1969), Cassan (1972) and Komornik et al. (1970) by Baguelin et al (1978) (Figure 2.17). Best fitting β coefficient (β = 11) to the data of this study and β coefficients of presented studies were plotted together in Figure 4.34.

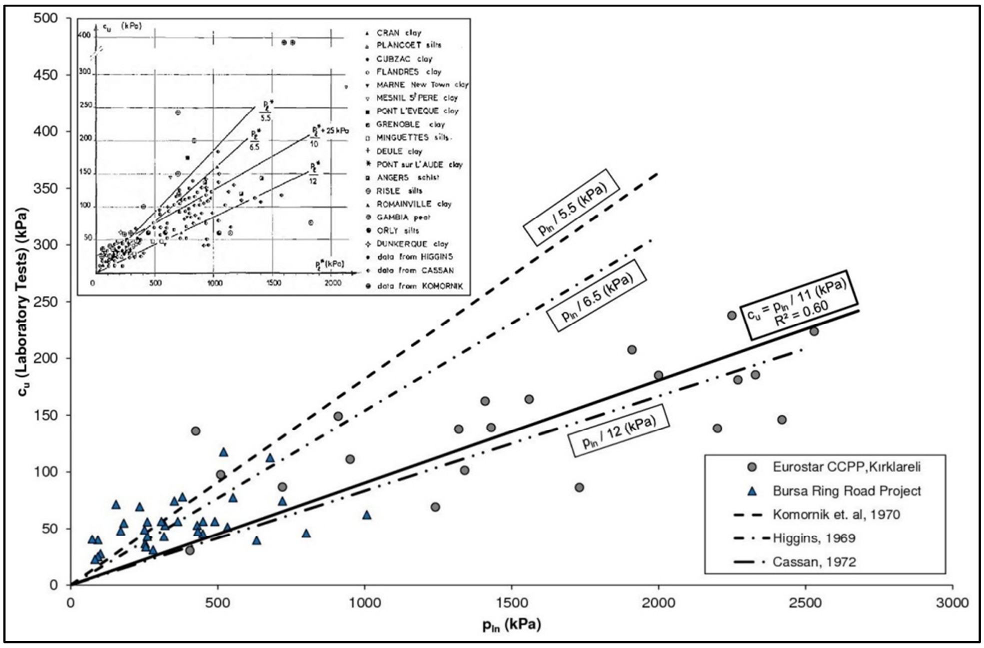

Figure 4.34 Comparison of p_{ln} vs. c_u with Higgins (1969), Cassan (1972) and Komornik et al. (1970) for analyzed soils

Considering the scatter and ranges of β values, it can be stated that the limits belong to Kırklareli and Bursa database are compatible with the values given in Baguelin's chart. The average β value determined from the analyses can be clearly seen to be located near the lower limit of the studies given by Baguelin. p_{ln} data of Bursa RRP, which is generally below 500 kPa, conforms with the β = 5.5 and β = 6.5 lines for soft to medium clays but as the p_{ln} gets larger in stiff to hard clays, as in the Kırklareli data, β coefficient also seems to have a tendency to increase.

Results of p_{ln} and c_u values from Bursa and Kırklareli database were also correlated in a similar way with the equation suggested by Amar & Jezequel (1972) as given below.

$$c_u \text{ (kPa)} = (p_{ln} / 10) + 25 \qquad \text{(Equation 4.2)}$$

For this case, best fit of the data was modified according to the form of Equation 4.2 for comparison purposes as presented in Figure 4.35.

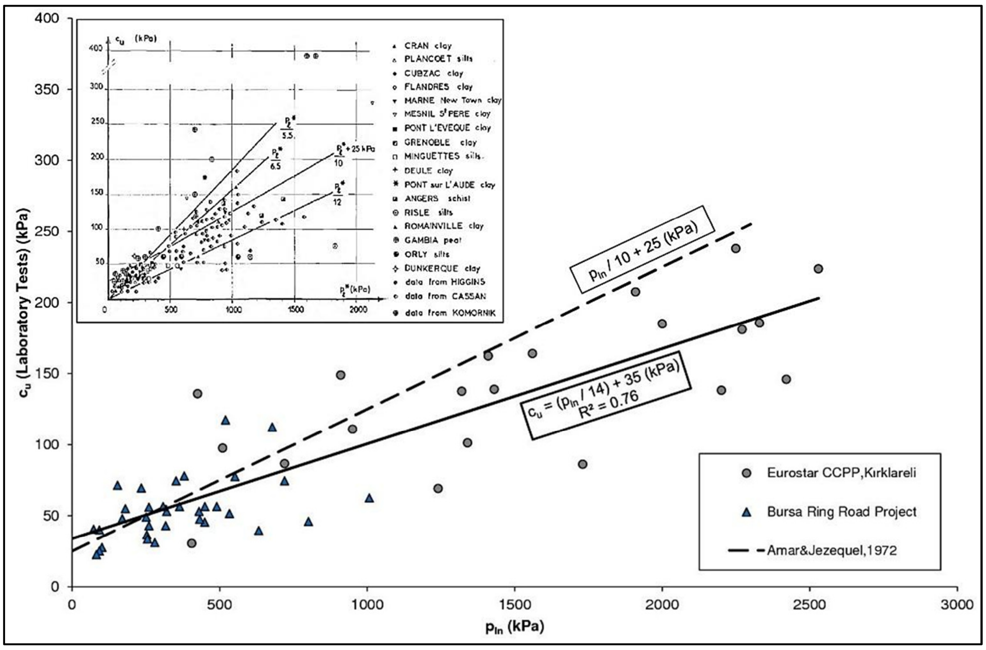

Figure 4.35 Comparison of p_{ln} vs. c_u with Amar & Jezequel (1972) for analyzed soils

The best fit line differs from the compared line by a certain amount but the square of correlation coefficient (R^2 = 0.76) for the fitted line indicates that the relationship can be defined better with an equation similar to the suggested by Amar & Jezequel (1972).

Another comparison of the results was done with the recommended values of β coefficient by Cassan (1972) which are summarized in Table 2.7. Cassan classified the soils based on their consistencies and proposed different β coefficients accordingly. Coefficient of β is suggested to be taken as 5.5 for soft to firm clays whereas β = 8 for firm to stiff clays and β = 15 for stiff to very stiff clays were recommended by Cassan. Comparison of the results with the suggested values by Cassan is presented in Figure 4.36.

A good agreement of the data with the proposed values of Cassan can be seen from the Figure 4.29 for p_{ln} > 500 kPa cases of Bursa and Kırklareli projects. Eventhough some of the Bursa data with p_{ln} < 500 kPa show deviations from the β = 5.5 line, they are generally located close to this line.

The other recommendations for β coefficient was made by Lukas & LeClerc de Bussy (1976) as β = 5.1, Marsland & Randolph (1977) as β = 6.8 and Martin & Drahos (1986) as β = 10 were also evaluated for the database. A general compatibility in the β values between Martin & Drahos (1986) can be observed with the stiff to hard clays of Kırklareli whereas the studies of Lucas & LeClerc de Bussy (1976) and Marsland & Randolph (1977) seems to predict the c_u better up to $p_{ln} \approx 750$ kPa of the evaluated data (Figure 4.37).

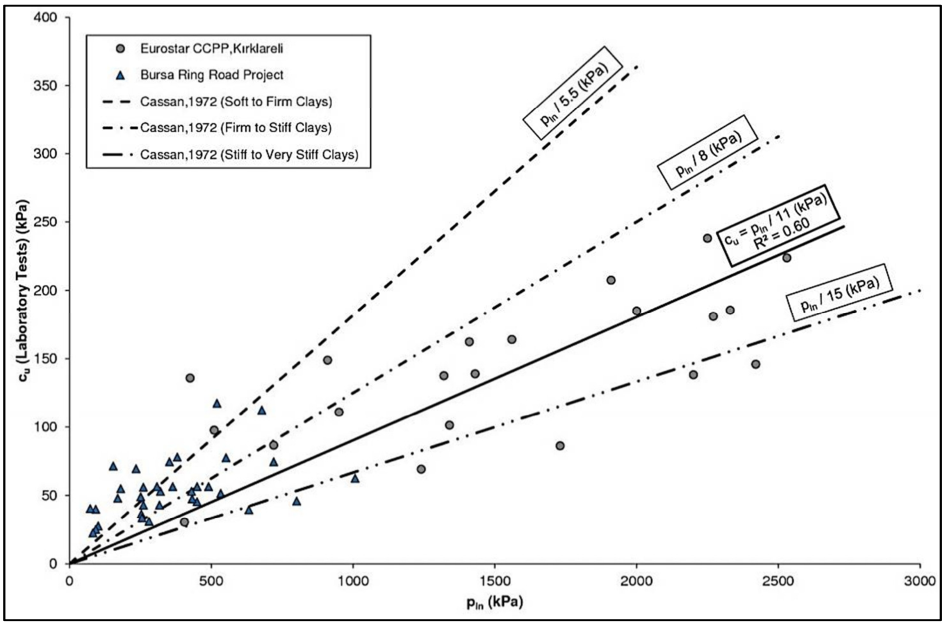

Figure 4.36 Comparison of p_{ln} vs. c_u with Cassan (1972) for analyzed soils

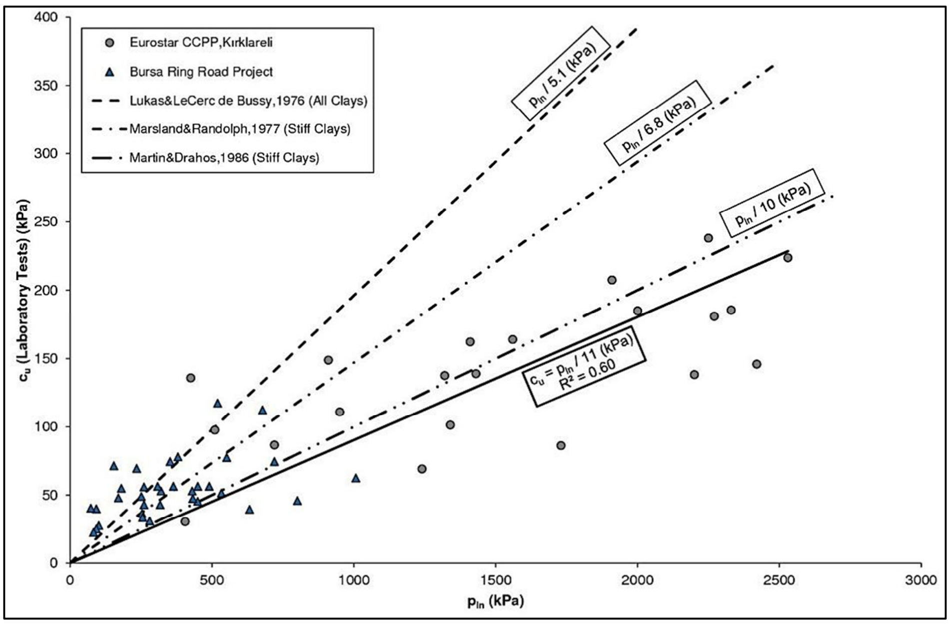

Figure 4.37 Comparison of p_{ln} vs. c_u with Lukas & LeClerc de Bussy (1976), Marsland & Randolph (1977) and Martin & Drahos (1986) for analyzed soils

A rather different and direct method to estimate the c_u from p_{ln} values was proposed by Baguelin et al. (1978) (Figure 2.18). Evaluation and comparison of the database with the equation recommended by Baguelin was performed and the results are plotted on the same graph as given in Figure 4.38. Similar to the nonlinear equation given by Baguelin, a nonlinear power function was fitted for the data as following:

$$c_u \text{ (kPa)} = 2.41\, p_{ln}^{0.55} \text{ (with } R^2 = 0.74) \qquad \text{(Equation 4.3)}$$

The numbers of determined equation are different from the ones in Baguelin's, possibly due to relatively lower c_u values corresponding to p_{ln} > 2000 kPa range. The scattering of data for p_{ln} > 1000 kPa values is also observable in the study of Baguelin et al. (1978) which could be related to typical factors such as quality of laboratory tests, borehole disturbances, nonhomogenous structure of the tested soil etc. However, general trends of the curves show similarities and it can be interpreted that a nonlinear correlation between p_{ln} and c_u is also possible.

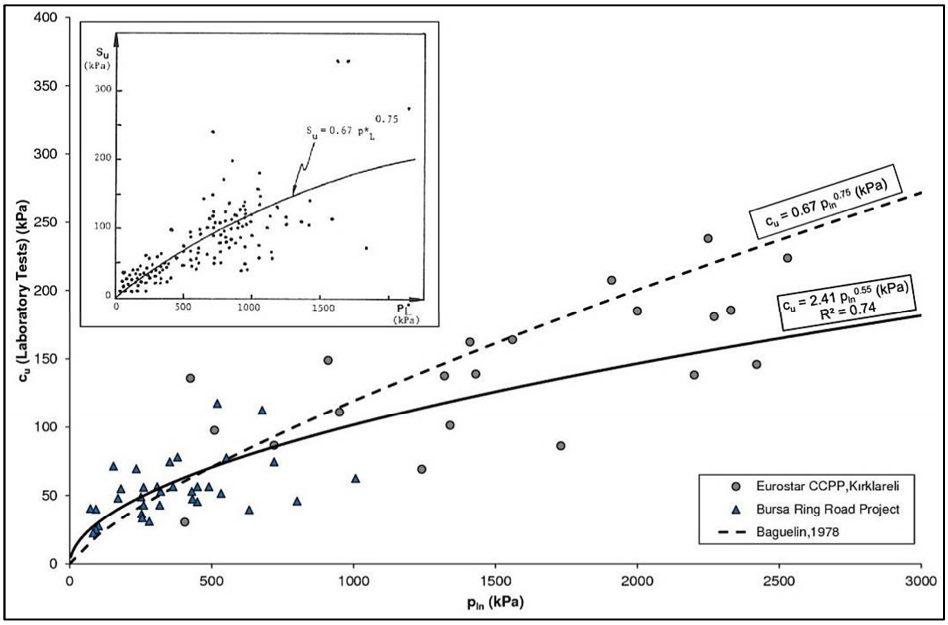

Figure 4.38 Comparison of p_{ln} vs. c_u with Baguelin et. al (1978) for analyzed soils

Besides the estimations of c_u from p_{ln} values of pressuremeter, Briaud (1992) presented a rough prediction of c_u from pressuremeter modulus (E_p) by using an extensive database of pressuremeter tests. Pressuremeter modulus (E_p) values in the database and corresponding undrained shear strength (c_u) values were plotted with the suggested correlation of Briaud on the Figure 4.39. Despite the large number of data in Briaud's study, it is mentioned that the scattering of the data makes difficult to establish a reasonable correlation between E_p and c_u which is also valid for the evaluated database of this study. Therefore, the estimation shall only be used to get an idea about the magnitudes (Briaud, 1992).

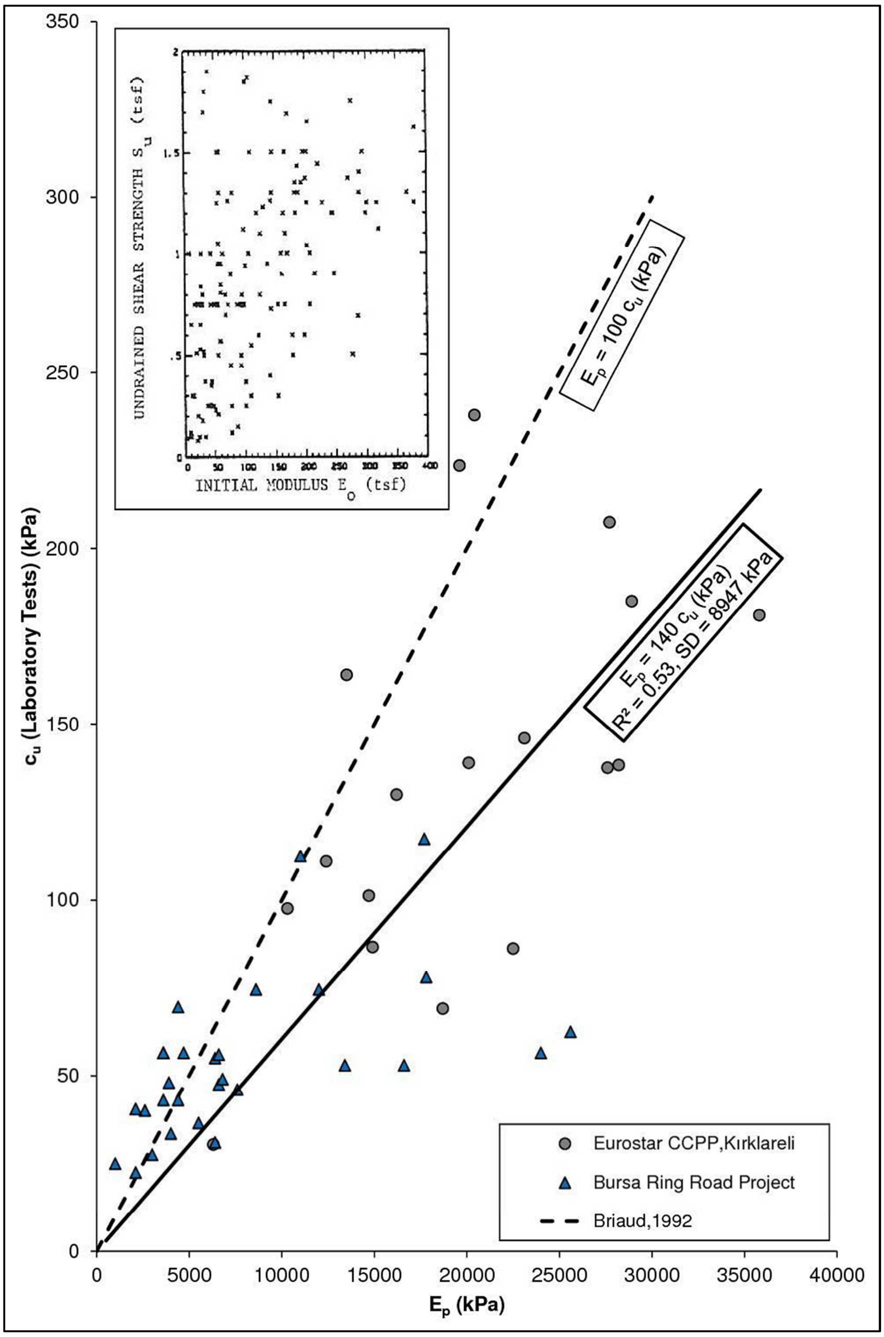

Figure 4.39 Comparison of E_p vs. c_u with Briaud (1992) for analyzed soils

4.4.2. Estimation of Deformation Modulus (E_s)

Pressuremeter modulus (E_p) is a commonly preferred parameter regarding the predictions of deformation modulus (E_s) in situ conditions. Assessments are mainly performed according to the method suggested by Menard (1975). Deformation modulus (E_s) values of the soils in the database were estimated by dividing the related E_p values by an α coefficient which was determined according to E / p_{ln} ratios as given in Table 2.8.

Since E_s can be related to one dimensional modulus (M) of soils (Equation 2.16), calculated E_s parameters were plotted against the M values obtained from oedometer tests for comparison purposes in Figure 4.40.

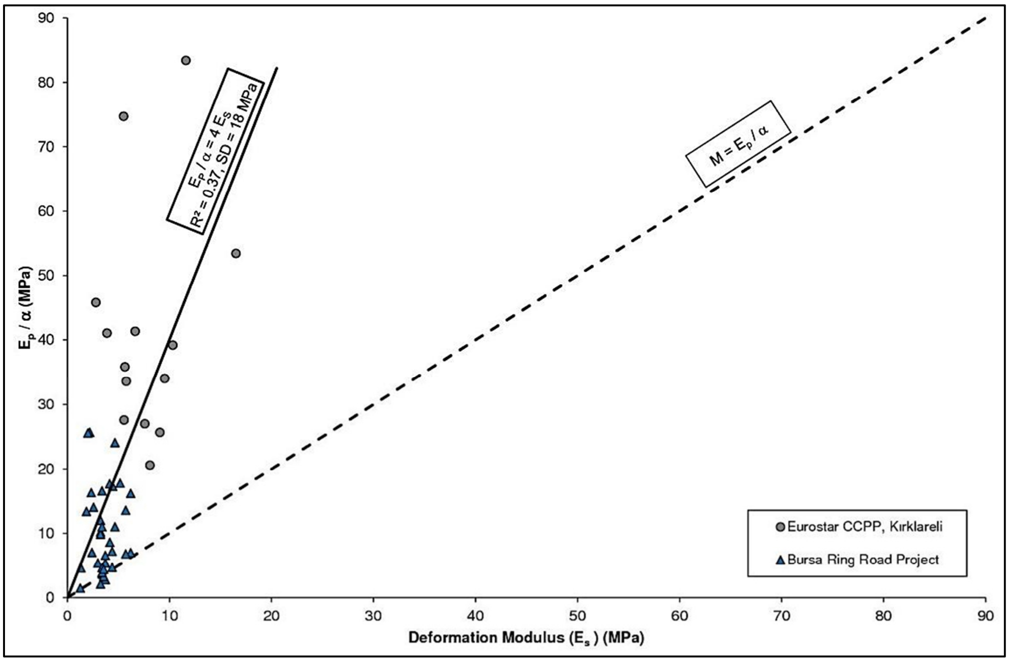

Figure 4.40 M ($1/m_v$) vs. E_s (E_p / α) for analyzed soils

One dimensional deformation modulus (M) values were observed to be much lower than the E_p / α values. This result is completely the opposite to the expected value if the Equation 2.13 was used to determine the E_s from M. Aside from a few data of Bursa, a consistent trend can be observed with a ratio of 2.98.

The unexpected result of the attempted correlation yields to the questionability of oedometer test results. Results of the laboratory oedometer tests supposed to give larger modulus values compared to the estimations from pressuremeter tests due to the restriction of the sample to deform in the lateral direction. As discussed in the previous chapters, adverse effects of disturbance on the oedometer samples, differences in saturation state and loading conditions on the soil can be stated as the main reasons of the result.

A similar study performed by Yaman (2007) was also reviewed about the related subject. Yaman (2007) founded a similar result regarding the database of his study. The outcome of the performed analysis and the study of Yaman (2007) can be compared from the Figure 4.41.

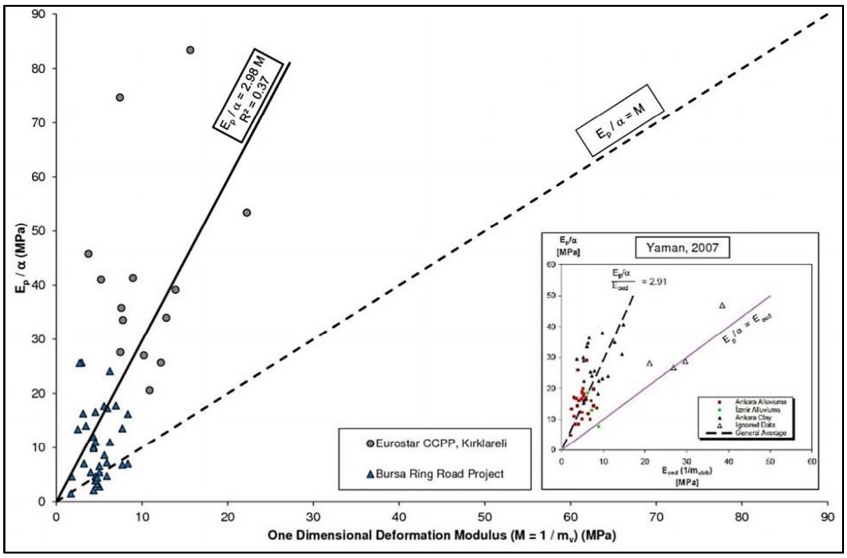

Figure 4.41 Comparison of M vs. E_p / α with Yaman (2007) for analyzed soils

Additionally, the correlation of E_p / α and E_s for which the M is converted by the elastic theory and a Poissons's ratio of ν = 0.3 was also investigated as presented in Figure 4.42. A significant deviation from the equality line was also observed by a factor of 4 in the correlation.

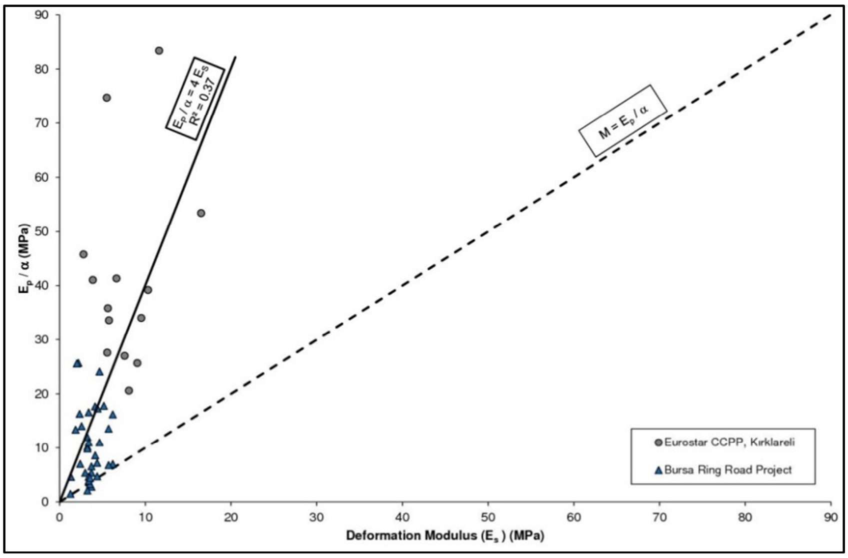

Figure 4.42 Comparison of derived E_s vs. E_p / α for analyzed soils

4.5. Cross Correlations between In Situ Tests

The use of in situ test results for estimation of the strength and deformation parameters is an essential procedure for a geotechnical design. However, possible cross correlations between in situ tests are also valuable for a designer to check the magnitude of the selected design parameters in the absence of one for a site investigation. In order to study the possible correlations between in situ test results, the entire database was evaluated, including the test results for both clay and sand type of soils, with any available in situ data.

4.5.1. Correlations between SPT and PMT

Despite their differences in measuring the in situ strength or deformation characteristics for a given soil, there are many attempts in the literature to correlate the SPT N and pressuremeter parameters, p_{ln} or E_p.

The variation of p_{ln} values with respect to SPT N_f number of clays for the studied soils are plotted on the Figure 4.43 with limit ranges and best fitting line.

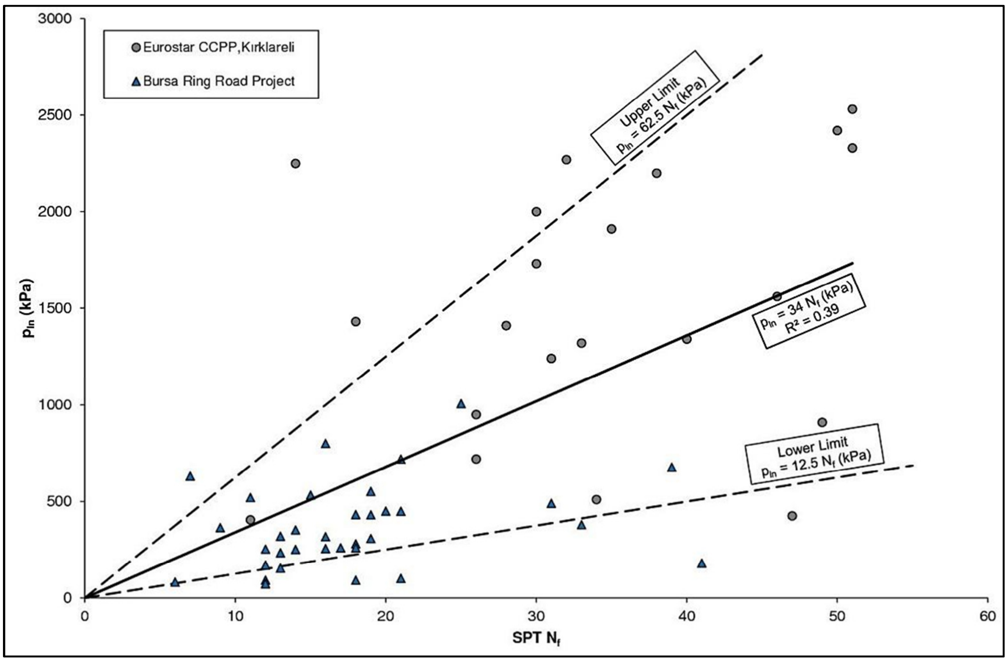

Figure 4.43 SPT N_f vs. p_{ln} for clays of the analyzed soils

As given by the best fit line of the data in Figure 4.43, correlation between SPT N_f and p_{ln} is found to be as;

$$p_{ln} \text{ (kPa)} = 34 N_f \; (R^2 = 0.39) \qquad \text{(Equation 4.4)}$$

An increasing trend for p_{ln} values with the increase of SPT N_f is observable from Figure 4.43. However, the square of the correlation coefficient (R^2) of the best fit line equation indicates a very weak correlation with a wide range of deviations.

Main reasons for the a scatter of the data and the weak correlation could be stated as ineffectiveness of SPT in clayey soils, disturbances in the borehole and anisotropic behavior of soil in the tested direction. In addition, the uncertainty regarding the failure of soil by SPT shall be considered whereas the net limit pressure (p_{ln}) defines an approximate failure of the soil. Since the SPT N is generally controlled by the consistency of cohesive soils, N numbers may both over and underestimate the strength of the soils compared to limit pressure values.

The relationship between SPT N_f and net limit pressure (p_{ln}) was also analyzed for sandy soils with a limited amount of data in Bursa and Kırklareli projects. Evaluated values of N_f and p_{ln} are plotted in a compatible format with the correlation suggested by Briaud (1992) for comparison purposes (Figure 4.44). A similar trend and correlation compared to Briaud's study was obtained but derivation of a reliable correlation is not seem to be possible due to the scatter in the data.

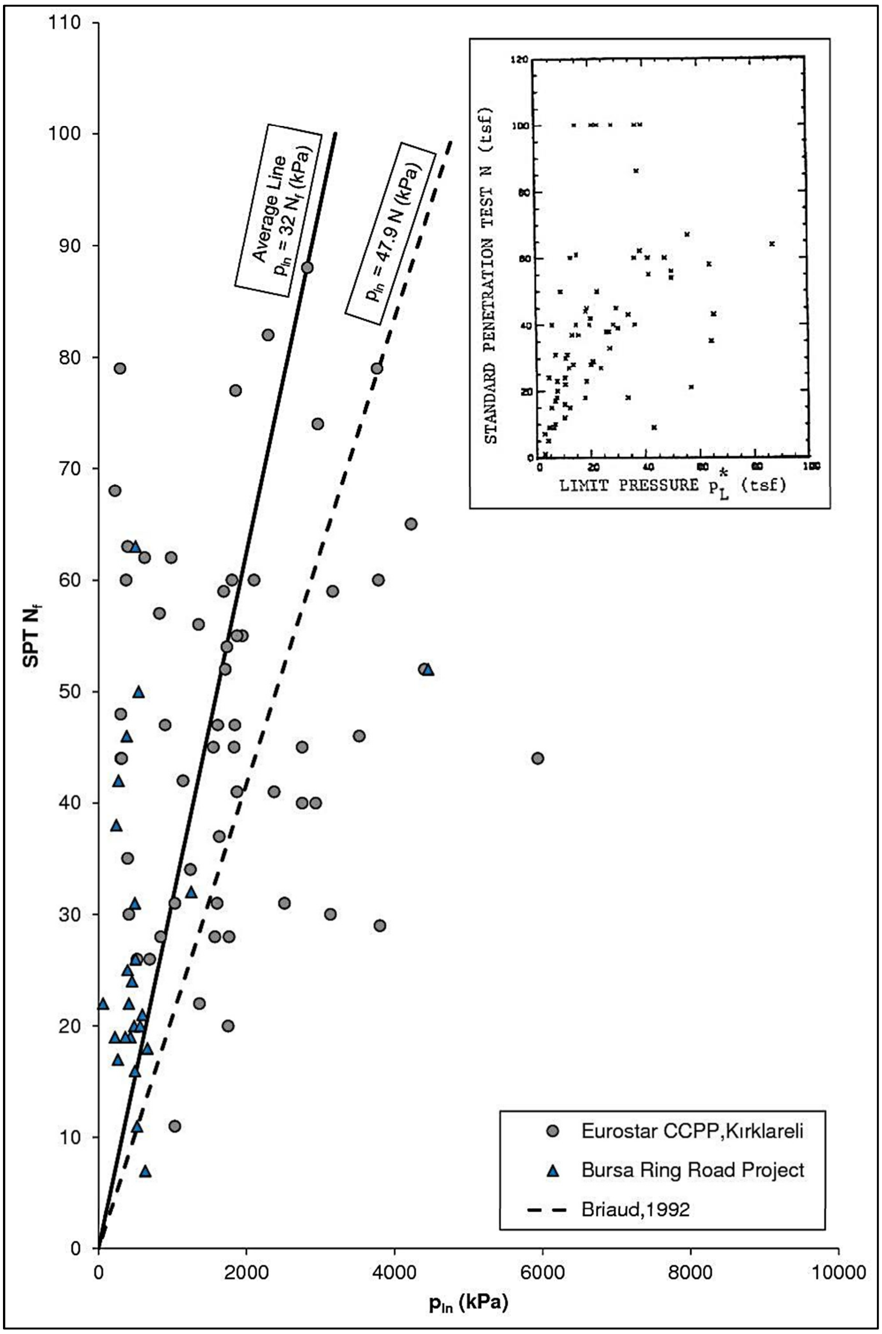

Figure 4.44 Comparison of SPT N_f vs. p_{ln} with Briaud (1992) for sands

In addition to net limit pressure (p_{ln}), the relationship between SPT N_f and pressuremeter modulus (E_p) was also investigated for the clay and sand type of soils from the same database. Evaluations of the SPT N_f and E_p correlation was compared with the similar studies in the literature to get an idea about the level of the variations. The comparisons about clay and sands for the SPT and PMT relationship, the studies by Ohya et al. (1982) and Briaud (1992) were taken as a guide.

Ohya et al. (1982) presented the possible relationships between SPT N_f and E_p on a log – log scaled chart for the alluvial and diluvial sands and clays in Japan. For the sake of a better interpretation, SPT N_f and corresponding E_p values of the analyzed soils of clays and sands were also plotted on a log – log scaled chart as illustrated in Figure 4.45 and Figure 4.46 respectively. Compared to the study of Ohya et al. (1982) range of SPT N values are in limited range.

Best fit line for the analyzed clay data indicates a similar but very weak correlation. Regarding the plot in Figure 4.45, it can be seen that the smaller SPT N values of Bursa data shows more scatter. Possible explanation for this could be the low measurement accuracy of SPT in soft clays. Since the SPT is a dynamic test, the large scatter could be anticipated especially when it is compared with a static test like PMT. Furthermore, the method of analysis could have played an important role about the results for which the SPT and PMT data of small sub layers are correlated rather than the values of overall soil mass.

As for the results of sands in Figure 4.46, insufficiency in the number of data lefts the predictions as incomplete hence, a proper correlation could not be established.

Beside the correlations proposed by Ohya et al. (1982), the data of sandy soils were also plotted in an appropriate format to compare with the study of Briaud (1992) (Figure 4.47). A certain amount of scattering can be seen from both Briaud's data and the values in the database. An average fit value for the analyzed data is provided with the approximate correlation suggested by Briaud (1992) in Figure 4.47.

Considering the results of the mentioned studies, a consistent correlation does not seem possible for both clay and sands due to large scattering, so the stated correlations should be preffered only for rough preliminary estimations or magnitude checks.

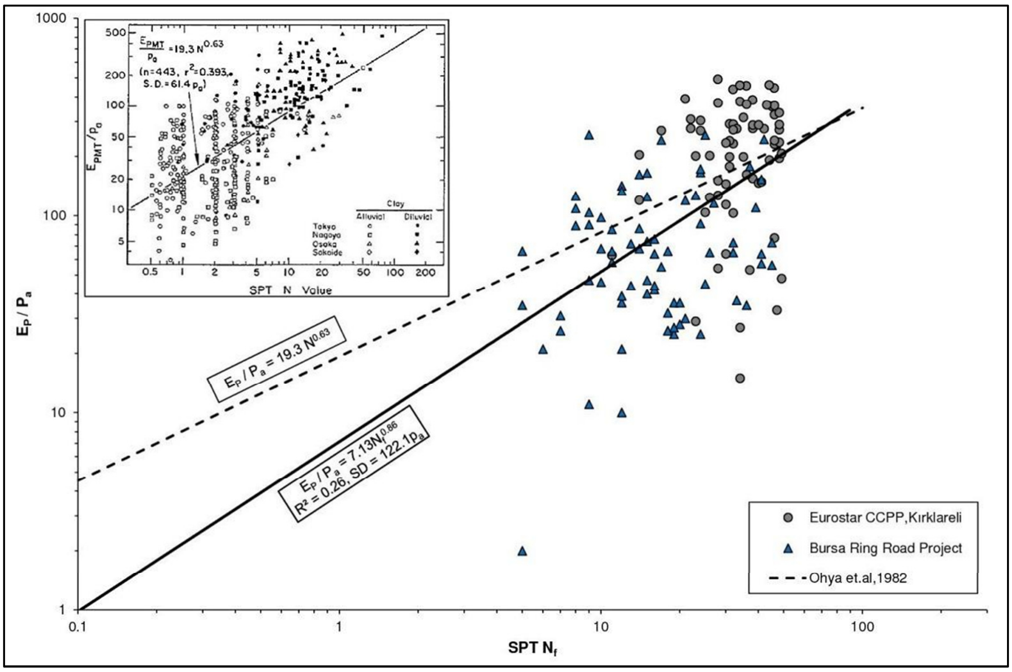

Figure 4.45 Comparison of SPT N_f vs. E_p /p_a with Ohya et al. (1982) for clays

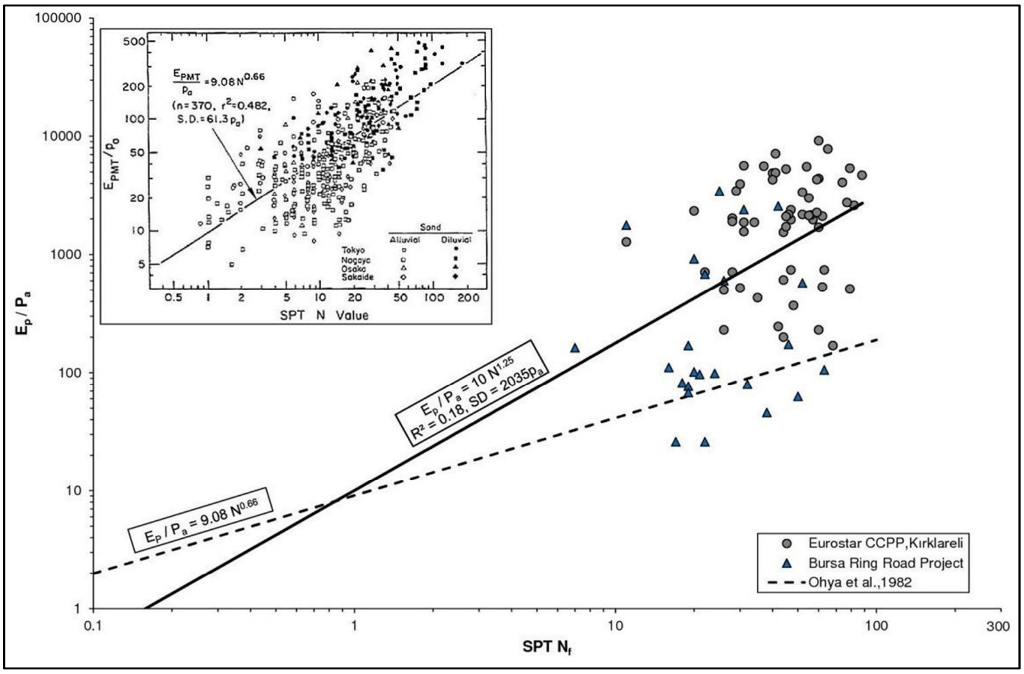

Figure 4.46 Comparison of SPT N_f vs. E_p /p_a with Ohya et al. (1982) for sands

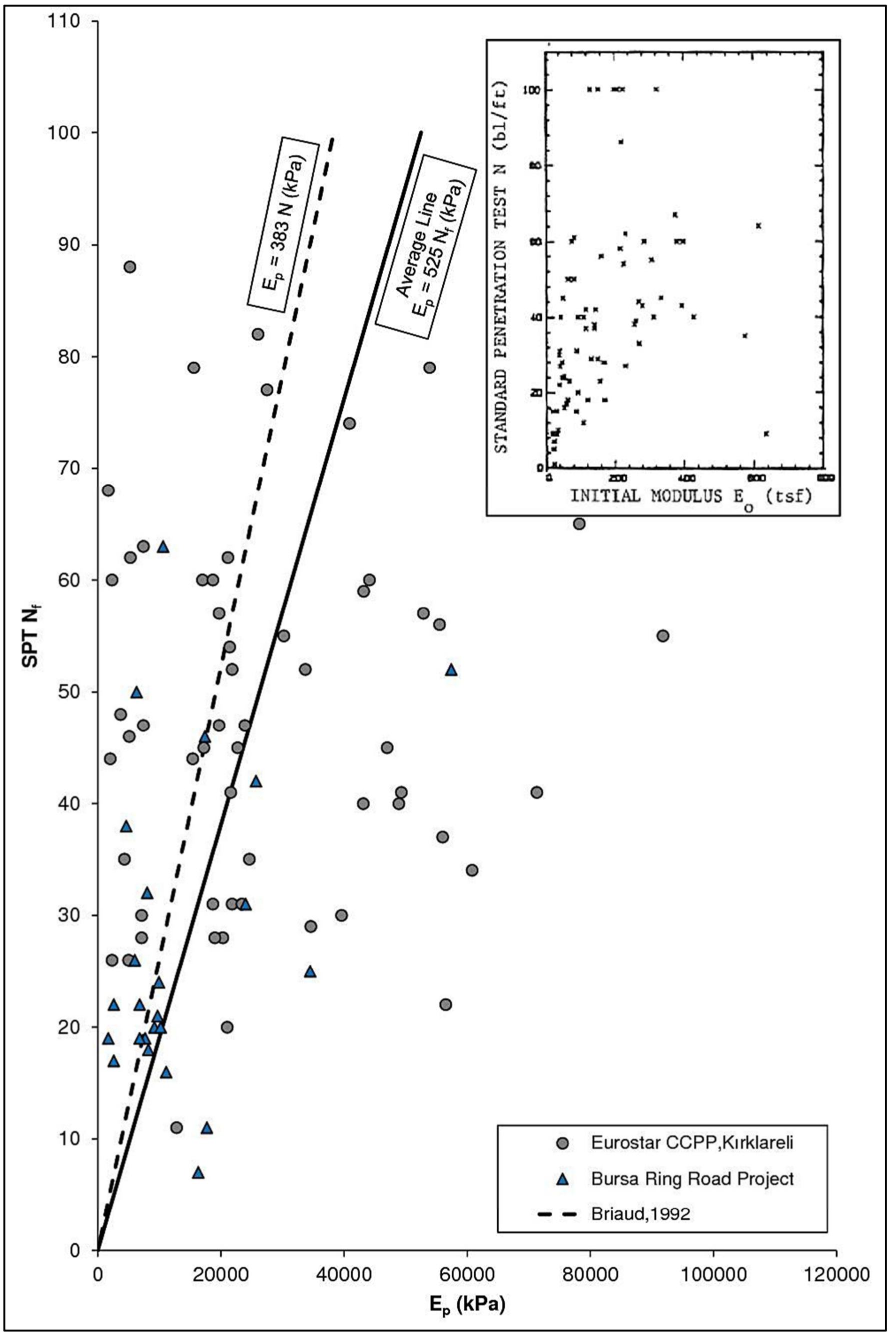

Figure 4.47 Comparison of SPT N_f vs. E_p with Briaud (1992) for sands

4.5.2. Correlations between SPT and DMT

Possible correlations between SPT N and dilatometer modulus (E_d) were investigated from a limited database of DMT test results in Warsaw subway project. Correlation studies are mostly performed for silty sand (SM) and clayey sand (SC) type of soils mainly because the availability of DMT results for these types of soils.

SPT N_f values and the corresponding E_d values from DMT are plotted in Figure 4.48. From the general layout of the data, a widely scattered but an increasing trend of E_d can be observed with the increase in SPT N_f. Despite the dispersion, a nonlinear best line was fitted to the data.

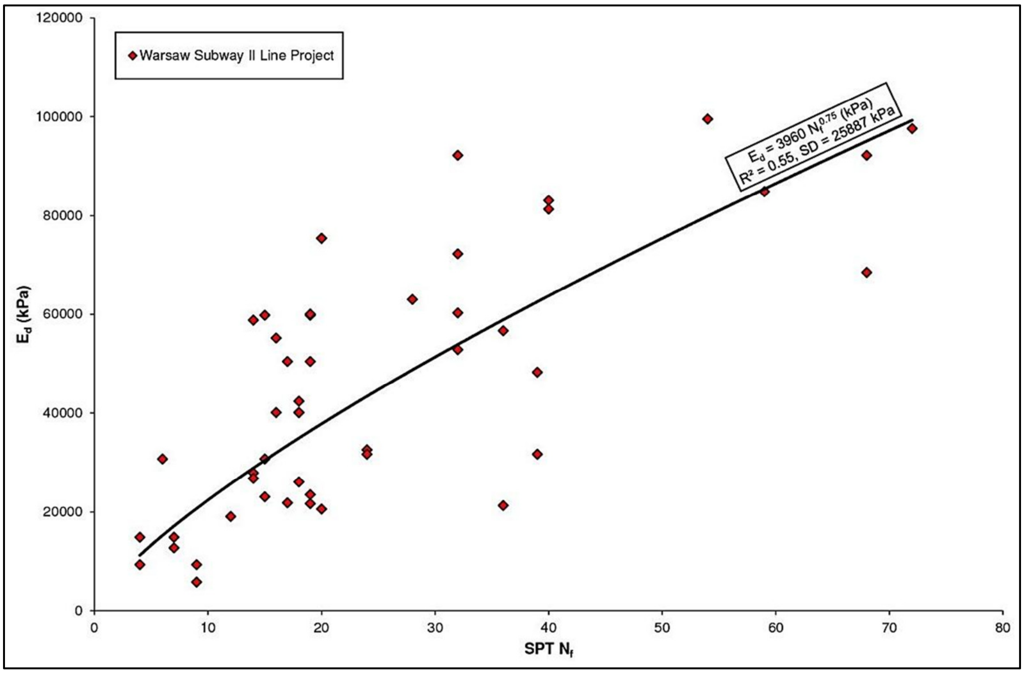

Figure 4.48 SPT N_f vs. E_d for sands

The obtained relationship was also compared with the study of Mayne & Frost (1989) for Piedmont residual sandy silts (Figure 4.49). A clear difference between the fitted line and the study of Mayne & Frost (1989) can be seen from the plot where almost all of the data is located above the curve of Mayne & Frost's study (1989). Variatons between the curves can be explained by the differences about types of the soils in the databases and equipment or procedure of the tests. Nevertheless, it can be stated that the relationship between SPT N_f and E_d can be expressed in terms of a similar nonlinear equation.

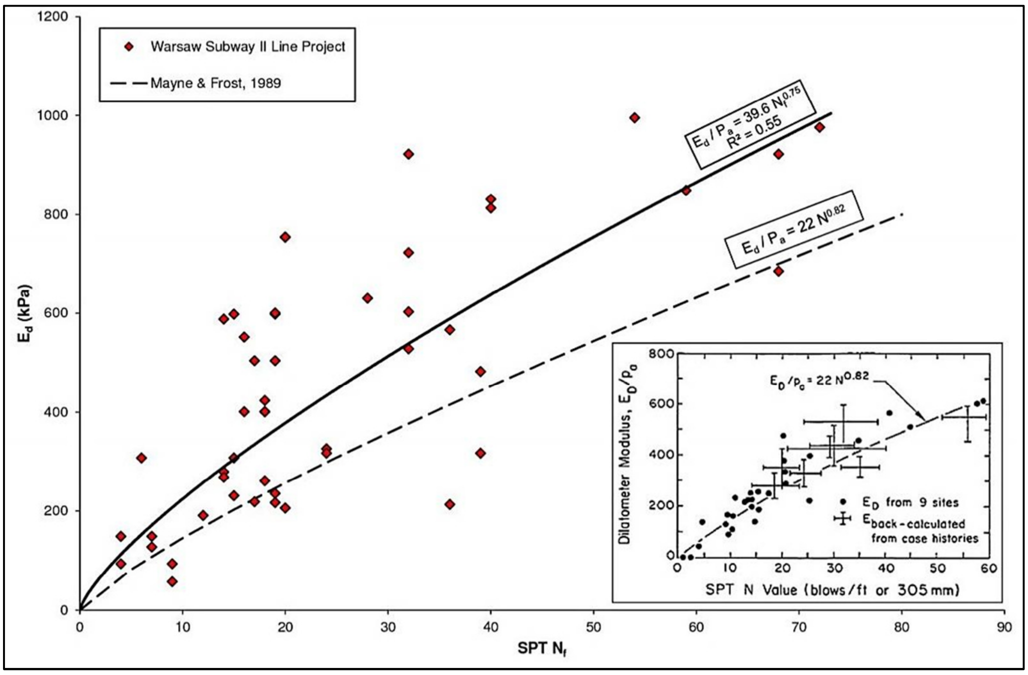

Figure 4.49 Comparison of SPT N_f vs. E_d with Mayne & Frost (1989) for sands

Additionally, the correlation between SPT N_f and E_d was also analyzed in accordance with the study of Tanaka & Tanaka (1998). The correlation derived by Tanaka & Tanaka (1998) for three sand sites in Japan was compared with the results of the analyzed database in Figure 4.50. For compatibility purposes, a linear best fit line is established for the data of Warsaw.

Tanaka & Tanaka (1998) founded a good correlation between N and E_d as a result of their study whereas the scatter in the Warsaw data causes a relatively weak correlation. Considering the square of correlation coefficient (R^2) of the trendline in Figure 4.50, a linear correlation between SPT N and E_d should be only preffered for rough estimations.

Even the correlations presented in Figures 4.49 and 4.50 indicate a relatively weak and highly variable relationship between SPT N and E_d, the correlations should be considered as site specific. Thus, the validity of a possbile correlation should always be checked before using in design as stated clearly by Marchetti et al. (1997).

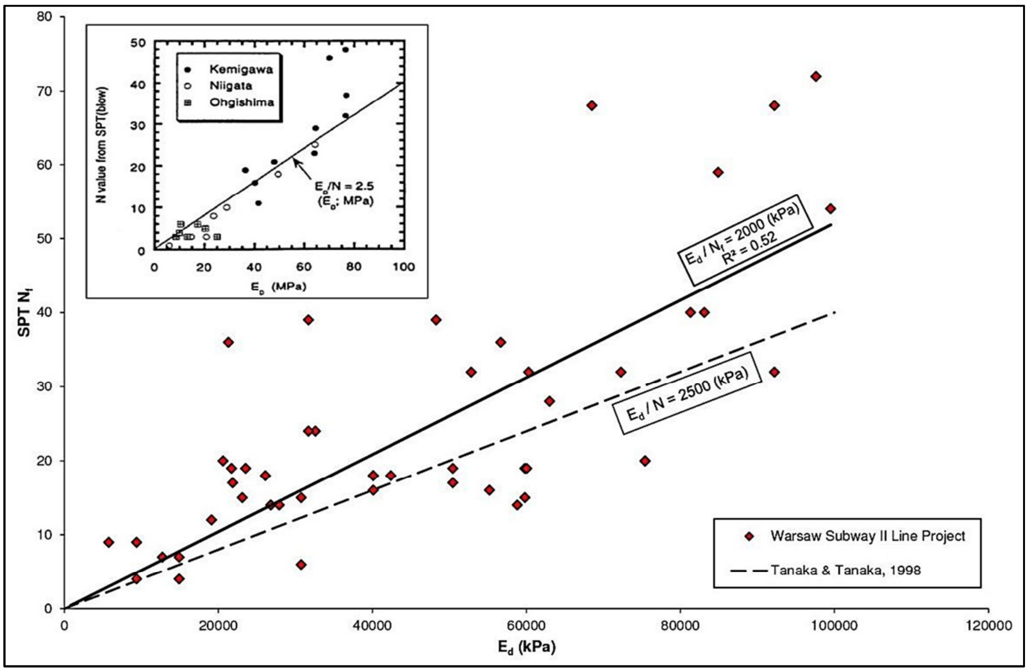

Figure 4.50 Comparison of SPT N_f vs. E_d with Tanaka & Tanaka (1998) for sands

4.5.3. Correlations between PMT and DMT

Another possible interpretation of DMT can be done with the PMT especially for pressuremeter modulus (E_p). Due to similarities in method, correlations between DMT and PMT seem reasonable, yet the number of literature studies is very limited with respect to other in situ tests. Since the DMT is a more recent in situ test than SPT or PMT, it is only used in a local scale. Furthermore, general database about the DMT has not been established well enough to be used in common design applications and it still is less preferred in site investigation studies.

In the scope of this study, correlation between pressuremeter modulus (E_p) and dilatometer modulus (E_d) is limited only for clay type of soils mainly due to the absence of comparable literature studies. For clayey soils of the Warsaw subway project, E_p and corresponding E_d values are given in Figure 4.51. A significant scatter and deviation from the line of equality can be seen in Figure 4.51 for the analyzed data. Therefore drawing a best fit line to the data would probably result in a very poor correlation.

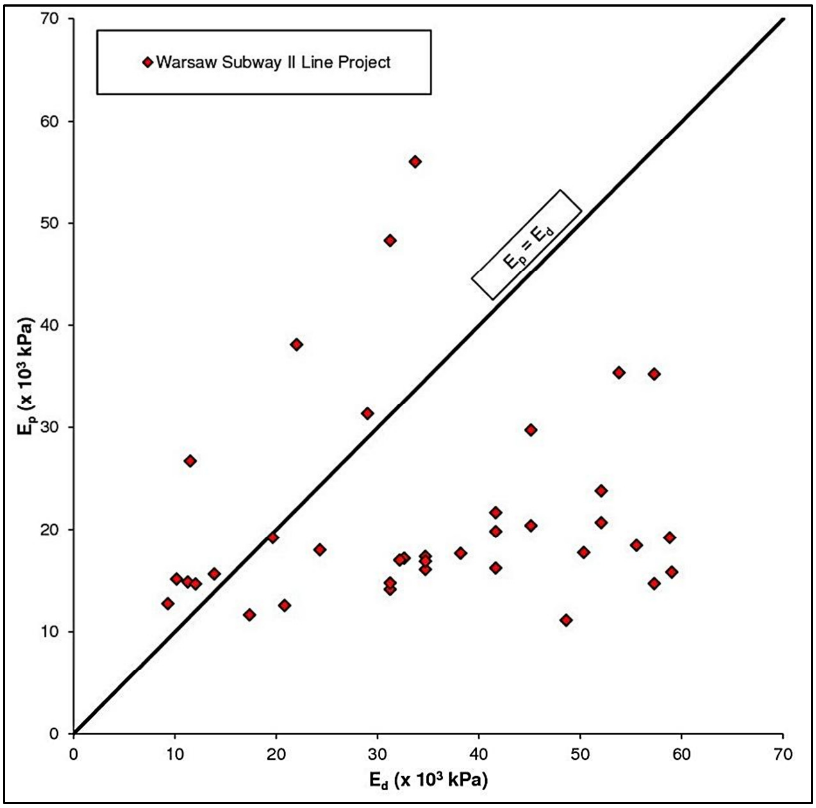

Figure 4.51 E_p vs. E_d for analyzed clays

In spite of the weak correlation obtained for E_p and E_d, comparison of the results with the literature studies was performed. The approximate correlation between E_p and E_d proposed by Kalteziotis et al. (1991) (Equation 2.38) is plotted on the Figure 4.52 with the studied data. Except some points, a reasonable compatibility was obtained for the relationship that the Kalteziotis et al. (1991) recommended.

Another comparison of the data was performed with the study of Lutenegger (1988) who used the data of Powell & Uglow's (1986) data for estimations (Figure 4.53). Even though Lutenegger (1988) did not propose an equation for the correlation, a similar distribution of the E_d with respect to E_p was found when the two studies are compared.

The main reason for the scatter and deviation in the data can be related to borehole disturbances prior to pressuremeter test as Lutenegger (1988) emphasized. On the other side, it is also mentioned by Lutenegger that a more reasonable correlation between E_d might exist not with the loading modulus but with the reload modulus of PMT due to induced displacements by DMT. Since the reload modulus of PMT is usually larger than the loading modulus, underestimation of E_p from E_d might be considered as an indication of correct trend.

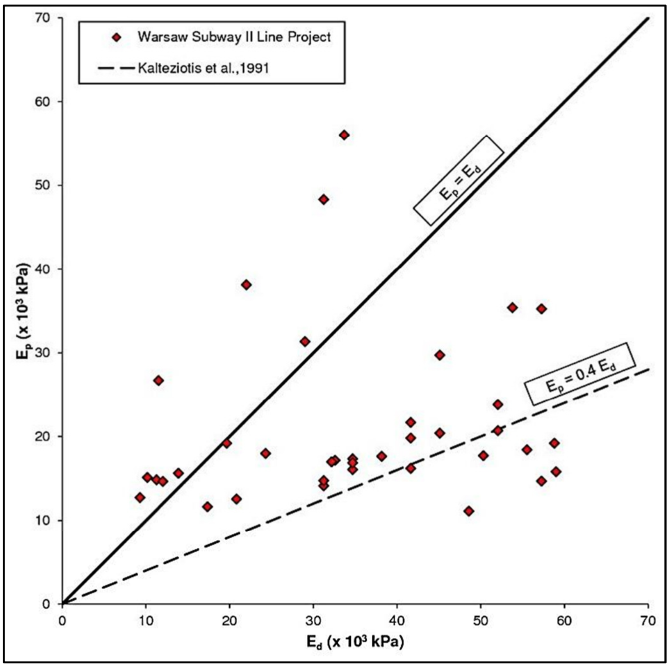

Figure 4.52 Comparison of E_p vs. E_d with Kalteziotis et al. (1991) for clays

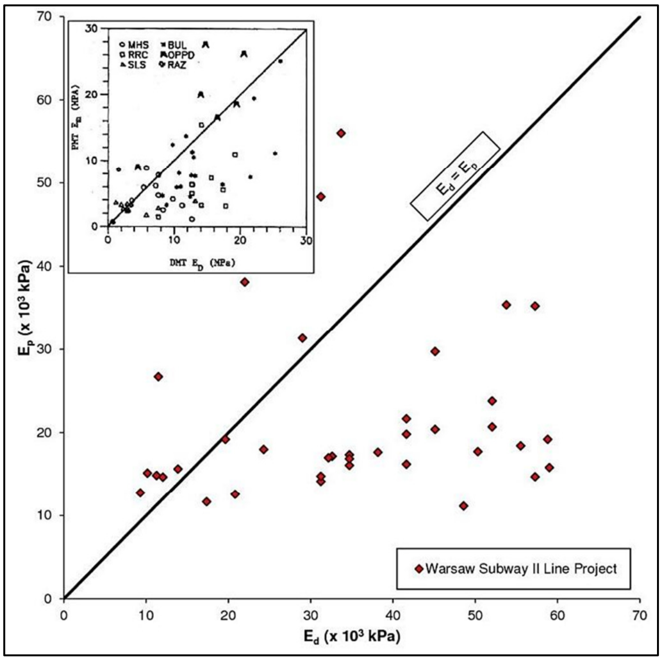

Figure 4.53 Comparison of E_p vs. E_d with Lutenegger (1988) for clays

CHAPTER 5

SUMMARY AND CONCLUSIONS

In the scope of this study, an extensive in situ test (SPT, CPT, PMT and DMT) data from five different site investigation studies was compiled along with laboratory test results in order to evaluate and compare the commonly preffered emprical correlations in the literature. Studies were mainly focused on the estimations of undrained shear strength (c_u), coefficient of volume compressiblity (m_v) and deformation modulus (E_s) of cohesive soils. In addition to parameter estimations, assessments about the reasonability of direct correlations between some of the in situ tests were also performed to establish a brief understanding.

5.1. Summary of Findings

As a result of the studies, interpretations have been done for each in situ test seperately about the predictions of geotechnical parameters as follows:

5.1.1. Standard Penetration Test (SPT)

- Estimations of undrained shear strength (c_u) from SPT N_f by using unconfined compression (UC) tests was found to be compatible with the study of Sowers (1979) and Sivrikaya & Toğrol (2007) whereas a general overestimation can be observed from the study of Terzaghi & Peck (1967) for the analyzed clays in the database.

- Estimations of undrained shear strength (c_u) from SPT N_f by using triaxial test results were found to be compatible with the study of Stroud (1974) and Sivrikaya & Toğrol (2007) for stiff to very stiff clays but, for the hard clays significant over or underestimation seems to be possible. In addition, back calculated f_1 coefficients seems to have a general compatibility with the study of Stroud (1974) but a considerable scattering was also observed in the results.

- Prediction of coefficient of volume compressibility (m_v) directly from SPT N values were also performed in accordance with the correlation proposed by Stroud (1974). However, m_v coefficients determined from oedometer tests seems to overestimate the values up to 250 % as compared to the calculated values by using f_2 coefficient as Stroud (1974) recommended. Additionally, back calculated f_2 coefficients was observed to fell mostly below the study of Stroud (1974) with a significant amount of scatter. This result might mostly be related to general issues about laboratory samples like disturbances, release of in situ stresses and differences in saturation conditions.

- Indirect estimations related to deformation modulus (E_s) of clays was also performed by converting the mv values to E_s using the theory of elasticity. Correlations with SPT N_f were resulted in a very wide range of E_s values varying between 150 N_f and 500 N_f as expected because of sample disturbances, assumption of ν and various factors affecting SPT N results. Furthermore, best line fitted for the data resulted in a weak correlation indicating that the real parameter might be quite different from the estimation.

5.1.2. Cone Penetration Test (CPT)

- Estimations of undrained shear strength from CPT test results were mainly based on the studies in the literature, for which cu is related with qc by a factor of N_k. The derived factors of N_k were observed to vary with a wide range yet it seems to be in the limits of the literature values. The reason for wide ranges for N_k values is mostly related to inefficient idealization of qc data, absence of pore water corrections, inconsistent reference strengths from different laboratory tests, disturbances on the laboratory samples, differences in the results of cone types.

- Predictions about compressibility characteristics from CPT were studied according to relation of one dimensional deformation modulus (M) with the cone tip resistance (q_c) by a factor of α. Different clay types (CL and CH) in the database were analyzed separately by using q_c and the corresponding m_v values in the database. The ranges of derived α values with respect to q_c were observed to vary between 3 – 7.5 for q_c < 2000 kPa, 2.5 – 7 for 2000 < q_c < 5000 kPa and 3.5 – 6 for q_c > 5000 kPa for CH type of clays whereas α values were observed to change between 2 and 5 independent from the q_c for CL type of clays. A general compatibility with the suggested limits of Sanglerat (1972) and Kaplan (2004) was also observed for CH clays. However, the data seems to be completely inconsistent according to the ranges of Sanglerat (1972) for CL clays possibly due to in situ stress release effects and disturbances on CL clay samples.

- Estimations of M and Es from q_c were also evaluated for the reviewed database referring the studies of Mayne (1990) and Bowles (1997). Eventhough the result differs from the study of Mayne (1990) a relatively strong linear relationship was obtained between the net cone resistance and M. On the other hand, the relationship between q_c and E_s was found to be compatible with the lower limit that Bowles (1997) recommended for which a certain amount of overestimation seems to be possible.

5.1.3. Pressuremeter Test (PMT)

- Predictions of undrained shear strength (c_u) from net limit pressure (p_{ln}) of PMT were expressed as the ratio of p_{ln} / c_u which is referred as β in the literature. Therefore, analyses in this study were also performed with a similar methodology. A typical proportional relationship was observed with a $\beta = 4$ as the upper limit and $\beta = 15$ as the lower limit for the analyzed soils. The best linear fit for the data resulted in an average value of β as 11. A general agreement was found when the obtained correlations were compared with an extensive number of similar studies in the literature. Besides the estimations from p_{ln}, pressuremeter modulus (E_p) values in the database were also correlated with c_u. In contrast to the good correlations with p_{ln}, a reasonable relationship has not been obtained for the analyzed soils as similarly observed by Briaud (1992) in his study.

- E_p / α values calculated for the estimation of deformation modulus (E_s) were found to be larger than constrained modulus (M) values by approximately three times which is completely the opposite to the expected value. Eventhough the oedometer tests supposed to give larger modulus values, adverse effects of disturbance on the oedometer samples, differences in saturation state and loading conditions on the soil can be considered as the main reasons for the conflict in the results.

- Possible cross correlations between SPT and PMT parameters were investigated although the amount of similar studies are scarce. According to the results of the analyses, correlations with the ratios of p_{ln} / N_f = 34 for clays, p_{ln} / N_f = 32 and E_p / N_f = 525 for sands were determined from the data. However, a reliable correlation seems to be impossible because of for both clay and sands due to large scattering as also present for the available studies in the literature.

5.1.4. Flat Dilatometer Test (DMT)

- Correlations between SPT N_f and dilatometer modulus (E_d) were studied from a limited database of DMT test results mostly for sandy soils. As a result of the analysis, both a nonlinear correlation of $E_d = 3960\ N_f^{0.75}$ (kPa) and a linear correlation of $E_d = 2500\ N_f$ (kPa) can be derived with low square of correlation coefficients (R^2) due to insufficient data. In spite of the differences in the the soil conditions, the results were found to be comparable with similar correlations proposed by Mayne & Frost (1989) and Tanaka & Tanaka (1998).

- Due to similarities in method, a possible direct relationship between DMT and PMT was investigated despite the limited number of literature studies. The plot regarding the E_p and E_d data showed a significant deviation from the line of equality. However, considering the studies of Kalteziotis et al. (1991) and Lutenegger (1988), a reasonable compatibility of the results can be stated for which the E_d overestimates the E_p. This result was mainly related with the misinterpretation of dilatometer modulus (E_d) with pressuremeter loading modulus (E_p) rather than pressuremeter reload modulus for which a more reasonable correlation would be obtained.

5.2. Conclusions and Recommendations for Future Study

In conclusion, it can be stated that the many commonly used correlations in the geotechnical practice to estimate the geotechnical parameters from in situ tests contain a certain amount of inaccuracy. The reasons for this result can easily be related to quality of the in situ and laboratory tests. Since the database of this study is mainly comprised of contracted construction projects, quality of the site explorations and testing of the soils are questionable parameters for this type of researches. In addition, there is also a more important reason that affects the obtained results which is the heterogenous nature of the soil. Therefore, applicability of these correlations should be evaluated in detail and the reasonability of the results should be checked with other available correlations. On the other hand, this study proves once more that the cross correlations between in – situ test parameters still involves a large amount of uncertainties as presented by many researchers and they should not be preffered unless there is not any other data available.

Aside from the mentioned issues above, the accuracy of the evalauted correlations can be increased by more carefully performed and well controlled in – situ testing, borehole sampling and laboratory testing. In this way, some of the uncertainties can be reduced and the reliability of the correlations would be enhanced.

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