PREDICTION OF GEOTECHNICAL PROPERTIES OF COHESIVE SOILS FROM IN-SITU TESTS: AN EVALUATION OF A LOCAL DATABASE

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ABSTRACT

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In any geotechnical design procedure, the fundemantal point to be initially clearified is the characterization of existing soil profile at a site. This requires a great deal of planning a suitable site investigation program including borings, sampling, laboratory and in situ testing etc. Laboratory and in-situ (field) tests are important tools leading to the estimation of soils properties in geotechnics. Beside laboratory tests, the measurement of engineering properties in situ is a continuously growing and developing trend, particularly in materials difficult to obtain perfect undisturbed samples.

For the purpose of this study, two large volumed geotechnical investigation reports are collected from a wide archive of 30 years experiences. Different soil types are encountered during the study like alluvial deposits of soft to stiff cohesive materials, hard clays in appearance of highly weathered rocks.

The in-situ tests mostly being focused and studied on are "Pressuremeter Test" and "Standard Penetration Test" on cohesive materails. Over 350 standard penetration test results are recorded together with the pressuremeter results of relevant soils. Besides, the corresponding laboratory test results of oedometer, triaxial loading and all index properties of soils are assembled.

The results of in-situ tests are evaluated together with the results of laboratory tests performed on the samples obtained from related sites. The correlations between in-situ & laboratory test results on shear strength, compressibility and deformation characteristics of soils are analysed and compared with the existing correlations in literature.

The correlations are generally obtained to be in agreement with the ones in common literature in cases where the soil conditions, particularly saturation, are same in both laboratory and in-situ tests.

Keywords : *Soil Properties, In-Situ Test, Laboratory Test, Pressuremeter Test, Standard Penetration Test, , Oedometer Test, Compressibility, Shear Strength, Deformation, Correlation*

ÖZ

KOHEZYONLU ZEMİNLERİN GEOTEKNİK ÖZELLİKLERİNİN ARAZİ DENEYLERİ İLE TAHMİN EDİLMESİ: YEREL BİR VERİ TABANININ DEĞERLENDİRİLMESİ

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Herhangi bir geoteknik tasarım prosedüründe ilk olarak netleştirilmesi gereken temel husus, çalışma sahasında mevcut zemin profilinin karakterize edilmesidir. Bu süreç; sondaj yapılması, numune alımı, laboratuvar ve arazi deneyleri vs. gibi işlemleri kapsayan uygun bir zemin etüdü programının dikkatlice planlanmasını gerektirir. Laboratuvar ve arazi deneyleri, geoteknik biliminde zemin özelliklerinin tahmin edilmesine olanak sağlayan önemli araçlardır. Laboratuvar testlerinin yanısıra; zeminlerin mühendislik özelliklerinin yerinde (arazide) deneylerle saptanması, özellikle kusursuz, örselenmemiş numune tesbitinin zor olduğu zeminler için sürekli büyüyen ve gelişen bir yaklaşımdır.

Bu çalışma kapsamında, yaklaşık 30 yıllık bir tecrübenin birikimi olan büyük bir arşivden seçilen çok geniş kapsamlı 2 adet geoteknik araştırma raporu taranmıştır. Çalışma süresince; kıvam durumu yumuşak – katı arasında değişkenlik gösteren kohezyonlu alüvyonal depozitler, aşırı ayrışmış kayaç formundaki sert killer gibi farklı zemin türleri değerlendirilmiştir.

Çalışma kapsamında dikkate alınan ve üzerinde değerlendirmeler yapılan arazi deneyleri, kohezyonlu zeminlerde gerçekleştirilen "Presiyometre Deneyi" ve "Standart Penetrasyon Deneyi" olmuştur. 350'nin üzerinde standart penetrasyon deney sonucu, sözkonusu zeminlerdeki presiyometre deney sonuçları ile birlikte kayıt altına alınmıştır. Bunun yanısıra, ilgili zeminlere karşılık gelen tüm ödometre, üç eksenli yükleme ve indeks özellikleri sonuçları derlenmiştir.

Arazi deney sonuçları, söz konusu sahalardan alınan numuneler üzerinde gerçekleştirilen laboratuvar deney sonuçları ile birlikte değerlendirilmiştir. Zeminlerin kayma dayanımları, sıkışabilirliği, ve deformasyon özelliklerini kapsayan arazi ve laboratuvar deney sonuçları arasındaki korelasyonlar analiz edilmiş ve literatürdeki mevcut korelasyonlarla karşılaştırılmıştır.

Çalışma kapsamındaki korelasyonların; başta suya doygunluk olmak üzere zemin özelliklerinin arazi ve laboratuvar koşullarında birbirine benzer olduğu durumlarda, literatürde önerilen yaygın korelasyonlarla genel olarak uyum gösterdiği gözlenmiştir.

Anahtar Kelimeler : *Zemin Özellikleri, Arazi Deneyi, Laboratuvar Deneyi, Presiyometre Deneyi, Standart Penetrasyon Deneyi, Ödometre Deneyi, Sıkışabilirlik, Kayma Dayanımı, Deformasyon, Korelasyon*

To My Wife

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CHAPTER 1

INTRODUCTION

Soil - one of the most complex engineering materials on earth – is the representation of underground world hidden down. The uncertainties kept in secret along the soils are usually what attracted the researchers through it for years.

The formation of soil is to be occured during a geological time history of millions of years in nature and still carrying on as an out of control process for humanbeing. So, soils originally do not exist as man-made formations in nature, which causes the designation of exact engineering properties of soils to be extremely difficult or nearly impossible.

In today's world, it is clearly understood and accepted that nothing is deterministic in engineering, especially in soil mechanics. Based on this phenomena, any geotechnical design could be named as successful as well as the characterization of soil profile and estimation of soil parameters used are accurate.

The uncertainties of soils in composition disscussed above require different methodologies in order to obtain the soil parameters in geotechnical engineering. For the purpose, sets of laboratory procedures on obtained soil samples and a variety of in-situ tests are improved and they are now to be very common in practical use. Since none of them is fully accurate on its own, they are both used together for the design.

The laboratory tests present direct results for soil parameters, whereas in-situ test are used with emprical correlations and calibrations in most cases to convert the results to appropriate engineering properties for design.

The in-situ tests, “Standard Penetration Test” and “Pressuremeter Test” are well-known and very common tools used in geotechnical design practice.

Standard Penetration Test (SPT) indicates the resistance of soil strata against the blows of a special test equipment trying to penetrate. The number of blow counts, N is like a useful number that is correlated to many properties of soils.

Pressuremeter Test reflects the resistance of soil towards a radial pressure and hence the deformation. The pressumeter modulus E_p, the limit pressure p_l and net limit pressure p_{ln} are the values obtained from the pressumeter curve plotted for each test.

According to the Stroud (1974), it is well established in the literature that, in saturated insensitive clays, N values are in some measure directly related to the undrained shear strength (Terzaghi & Peck, 1948; Schultze & Knausenberger, 1957; Sowers, 1954; De Mello, 1971). Beside, a correlation between N values and the coefficient of volume compressibility, m_v, is given again by Stroud (1974) depending on many oedometer test results. Gibson & Anderson (1961) and Menard (1967) propose that the undrained shear strength of clay materials could be predicted by net limit pressure value, p_{ln}, that is obtained from pressuremeter test.

The deformation modulus of soils also might be estimated from the pressuremeter tests by converting the pressuremeter modulus E_p somehow to a vertical stress-strain modulus of E (Centre d'Etudes Menard, 1975).

In addition to the correlations between laboratory & in-situ tests, the relation between SPT N values and E_P or N & p_l are also searched by some authors (Cassan, 1968-1969; Hobbs and Dixon, 1969; Waschkowski, 1976).

This study includes the correlations mentioned above relating to a large amount of experimental data of both laboratory & in-situ test results together.

CHAPTER 2

REVIEW ON TESTING FOR SOIL CHARACTERIZATION

2.1 Site Investigation

The term “site investigation” in geotechnics involves mainly the exploration of the general subsurface conditions. A site investigation for the purpose of assessment of the characteristics of soil is the first and most important part of the geotechnical design process.

Since it is the nature that has furnished the materials on which the humanbeing found his structures, it is for sure that no construction material could be more variable than the soil itself (Bowles, 1988). This enormous variety in both lateral and vertical directions requires a great deal of “site specific” investigations including subsurface explorations by the methods such as geological and geophysical surveys, in-situ testing, boring and sampling, visual inspection, local experience, laboratory testing of samples of the subsurface materials, and groundwater observations and measurements (Figure 2.1).

The proper investigation of the underground conditions at a site is also an obvious prerequisite to the feasibility and economical design of the substructure elements.

Figure 2.1 A Site Investigation Study on Ankara Rail Transit System / Stage 3 Construction Project (Batıkent)

2.1.1 General Objectives & Importance

The general objectives of a site investigation in geotechnics include the determination of:

- Lateral distribution and thickness of the soil and/or rock strata within the zone of influence of the proposed construction (Hunt, 1984)

- Detailed and representative soil profiles including description of subsoils, their degree of density if cohesionless or degree of stiffness if cohesive and groundwater conditions

- Engineering properties of soil and/or rock in situ such as permeability, compressibility and shear strength (Fang, 1991)

- Information so that the identification and solution of construction problems (adjacent structures, sheeting and dewatering or rock excavation)

- Hazardous conditions (unstable slopes, active or potentially active faults, regional seismicity, floodplains, ground subsidence, collapse and heave potential) (Hunt, 1984)

In addition to the objectives above, it may be necessary to perform an exploratory site investigation program on existing structures for two main reasons:

a) to investigate the ability of the foundation to carry additional loads to be imposed from improved superstructure

b) to search for the current status of the safety of the structure if the foundation behaviour is not what the designer may have expected

The importance of a site investigation process comes from the fact that the engineer, to prepare his design, must learn what underlying materials are present and what properties they possess (Peck, 1974).

The adequate knowledge of subsurface conditions obtained from proper site investigations at sites both informs the construction engineer about the materials/conditions he will encounter in the field and leads to a safe and economical design of a project. Elimination of the site exploration, which usually ranges from about 0.5 to 3 % of the total construction costs, may cause to a false safety and economy in case of a failure or when the performance of the structure on underlying strata is insufficient and redesign of foundations is required.

It is doubtful if any major structures are constructed without site exploration being undertaken, but for smaller structures there is a wide practice of little or no exploration, however, this practice is not recommended (Bowles, 1988).

2.1.2 Methodology & Planning

The most widely used method of subsurface investigation for most sites is boring holes into the ground from which samples may be collected for either visual inspection or laboratory testing. Beside, in-situ (field) tests are quite suitable for exploration process. The major steps in site investigations including basic methodologies are presented in Figure 2.2.

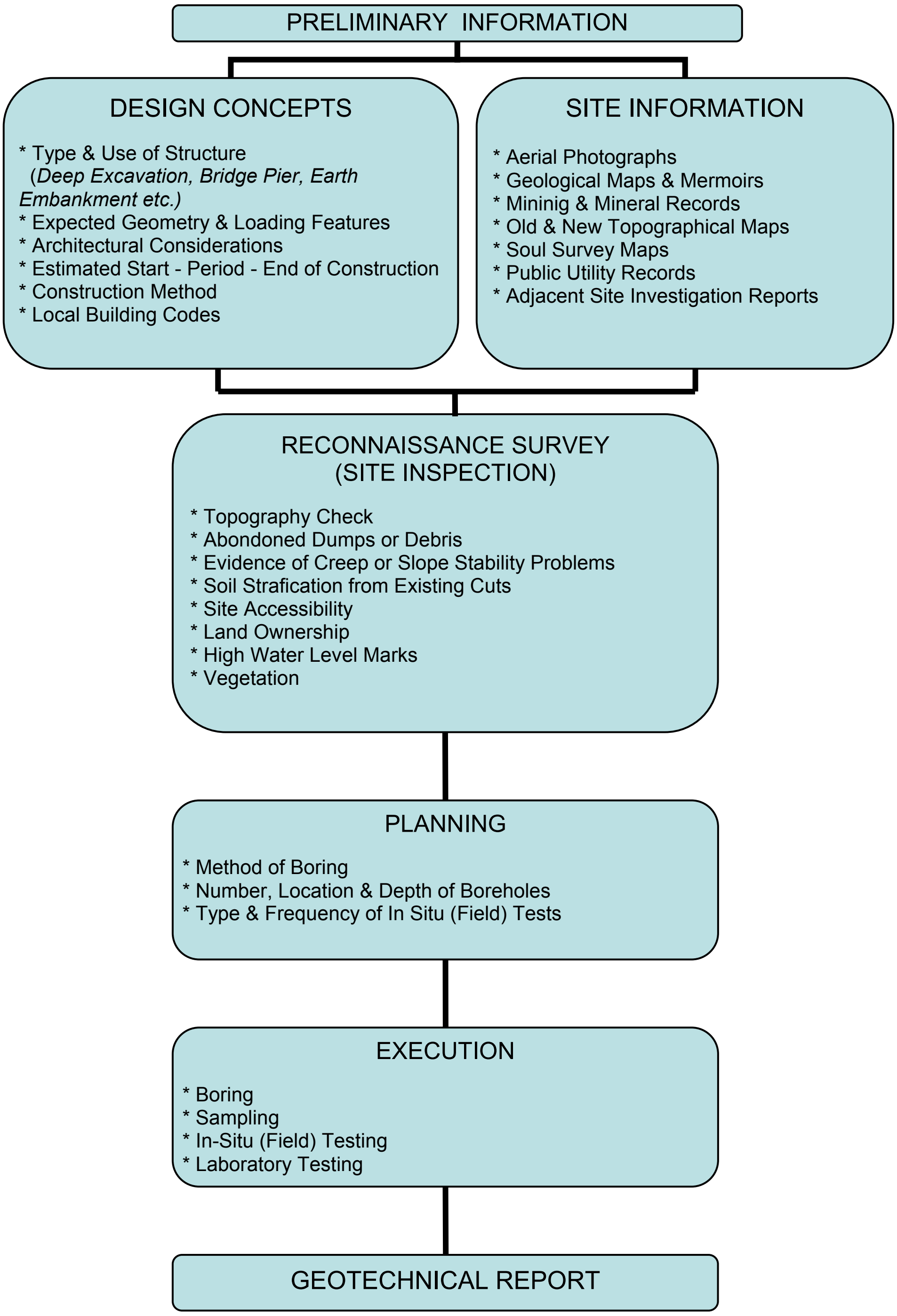


Figure 2.2 General Stages of a Site Investigation

For small projects, only one performance of the procedure may be sufficient to establish geotechnical criterias, whereas extent of site investigation may be required in details if risks are higher in case of a critical project or the soil is of poor quality and/or erratic soils are encountered.

2.2 Measurement of Properties

The site investigations truly aims to determine vertical and horizontal variations in type and properties of ground which include in situ stress conditions, compressibility & deformation characteristics, strength parameters and parameters defining time-dependent behaviour (Clarke, 1995). A design engineer can then make a prediction of the behaviour of the ground when it is subject to any change such as that caused by loading or unloading.

According to Hunt (1984), the properties of the geological materials are measured to provide the basis for:

a) Identification & Classification

b) Correlations between properties including measurements made during other investigations in similar materials

c) Engineering Analysis & Evaluations

2.2.1 Geotechnical Properties

The geotechnical properties of ground materials may be divided into some general groups:

1) Basic Properties

Basic properties include the fundamental characteristics of the materials and are used for identification and correlations. Some are used in engineering calculations. (Unit weight/density, specific gravity, moisture content, void ratio)

2) Index Properties

Index properties define certain physical characteristics used basically for classifications, but also for correlations with engineering properties. (Gradation, liquidity, plasticity, organic content)

3) Hydraulic Properties

Hydraulic properties, expressed in terms of permeability, are engineering properties. They involve the flow of fluids through geological media (permeability & seepage characteristics).

4) Mechanical Properties

Strength and deformation characteristics are mechanical properties. They are also engineering properties and are grouped as static or dynamic (Shear strength, deformation modulus, volume compressibility, in situ stress condition).

Measurements of the mechanical & hydraulic properties, which provide the basis for all engineering analysis, are often costly or difficult to obtain, especially with reliable accuracy. Correlations based on basic or index properties, with data obtained from other investigations in which extensive testing was employed or engineering properties were evaluated by back analysis of failures, provide data for preliminary engineering studies as well as a check on the reasonableness of data obtained during investigation (Hunt, 1984)

2.2.2 Methods of Measurement

"In-Situ" and "Laboratory Testing" techniques represent the two principal approaches for the measurement and determination of engineering properties during geotechnical investigations.

An in-situ (field) test simply means bringing the test equipment to the field and testing the soil in-place. In general, it includes borehole tests, full scale tests such as preloading trials, and non-destructive tests such as a surface geophysical testing. Borehole (in-situ) tests generally include those that;

* penetrate the ground (SPT, etc.)
* statically load the ground (Pressuremeters, etc.)
* dynamically load the ground (Crosshole geophysics, etc.)

Laboratory tests are performed on samples retrieved from the investigation sites. They usually include those that test elements (samples) of soil, such as a triaxial test, and those that test prototype models, such as a centrifuge tests.

Laboratory tests directly measure the engineering properties of soils whereas in-situ tests usually do not. However, use of emprical correlations and calibrations to convert in-situ test results to appropriate engineering properties for design purposes is a continuously growing methodology, since the determination of properties of soils as they exist in nature –free from the disturbances due to sampling and laboratory handling- is a useful and often necessary step toward proper design (Campbell, 1969)

Under ideal testing conditions, both approaches involve considerable compromise, so a careful combination of the two which will provide the most relevant information is preferred more oftenly.

A general list of in-situ & laboratory tests are presented in Figure 2.3.

2.3 In-Situ (Field) Tests

In-situ tests in daily words are to be defined as taking the laboratory to the site, rather than taking the soil to the laboratory as alternative approach in property measurement.

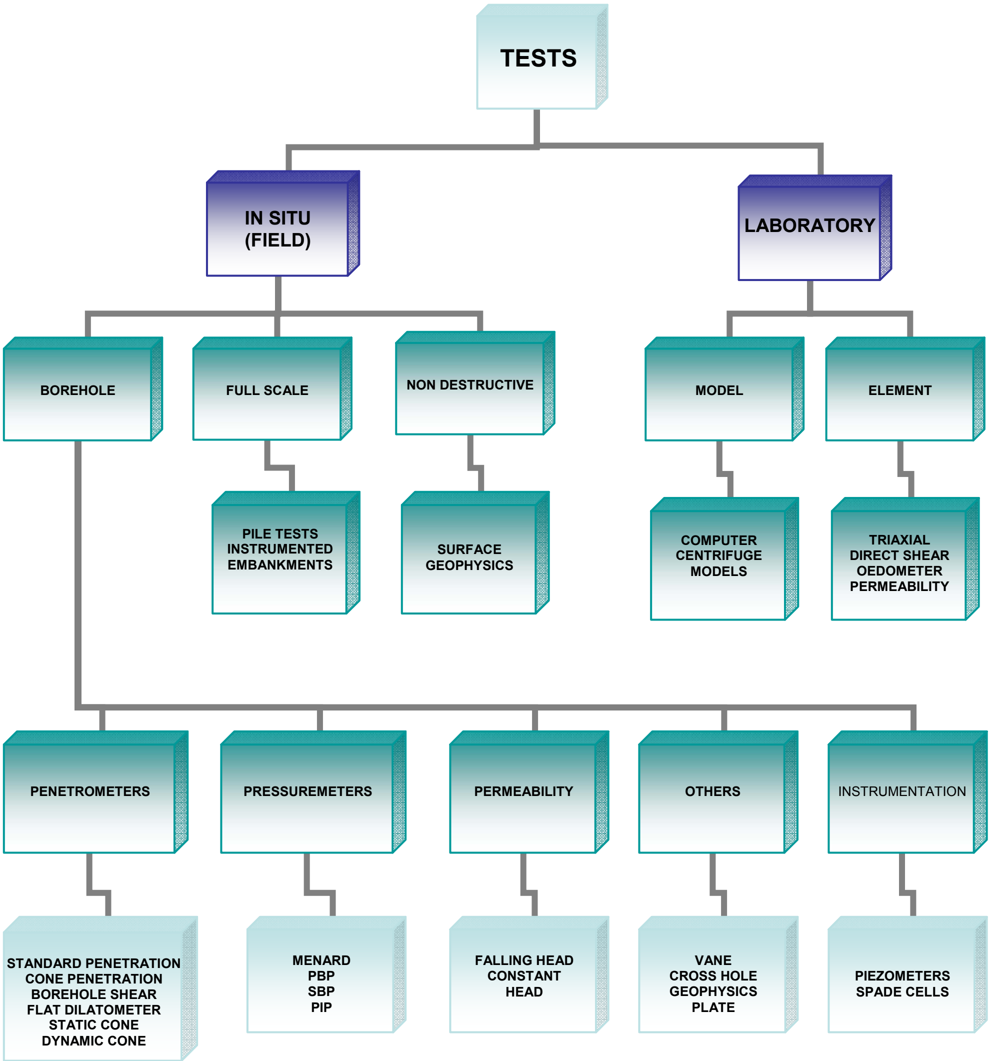


Figure 2.3 General Types of In-Situ & Laboratory Tests for Site Investigation

2.3.1 General

Soils are usually tested in-situ to obtain measures of engineering properties to supplement laboratory data, and in conditions where undisturbed sampling is difficult or not practical. Beside, field tests are to be desirable where it is considered that the mass characteristics of the ground would differ appreciably from the material characteristics determined by laboratory testing (BS 5930, 1999). These differences normally arise from several factors; the most important of two are how much representative the laboratory sample is and the amount of quality of sample. It is a fact that laboratory tests on “undisturbed” samples are no better than the quality of the sample.

Figure 2.4 A Penetration Type In-Situ Test Work on Field (CPT)

2.3.2 Advantages & Limitations

In-situ tests with their all advantages & shortcomings constitutes an important part of experimantal soil engineering.

In comparison with the laboratory studies, in-situ testing has some general advantages that of are:

- Larger volume of soil is tested which leads to the reflections of some macrofabric effects in soil (cementation, strong layering, fissures, etc.).
- A continuous record of soil profile is possible to obtain (Cone Penetration Test).
- In-situ tests are applicable to cases where undisturbed soil sampling is not efficient or possible (cohesionless granular soils, highly organic materials, intensly layered soils, highly fissured hard clays and cohesive soils with large granular particles such as glacial till, residual soils, etc.).
- They are performed under the natural environment and in-situ stresses with so little or no release of stress.
- The in-situ tests are usually less expensive, so a greater number of tests can be performed, thus characterizing the soil in more detail.
- Tests in field are less time consuming and results are available immediately.

Together with the advantages, in-situ tests are limited by some disadvantages and shortcomings:

- Stress and strains are not defined (except in pressuremeter test).

- The engineer has less control over drainage conditions, they are unknown and can not be controlled.

- The strain fields are not uniform, and strain rates are high as compared to real foundation loadings.

- Nature of soil may not be identified due to the lack of obtaining sample, which makes the soil classification more difficult (except in SPT).

- Interpretation techniques are mostly emprical.

Results obtained from in-situ tests vary in quality, quantity and applicability. Parameters are test dependent. A parameter obtained from one test may not have the same value as that obtained from another despite being given the same time.

In-situ tests are carried out as a part of a design exercise and the results can be used either indirectly or directly in design.

Data from an in-situ test can be converted to soil properties using correlations of either theoretical or emprical based. These properties are used in a design method, the choice of which depends on the parameter used, the method by which it was obtained and the soil model chosen (Clarke, 1995).

Jamiolkowski et al. (1985), in their report to the 11th International Conference on Soil Mechanics and Foundation Engineering (ICSMFE) outlined advantages and disadvantages of in-situ tests and suggested that the "self boring pressuremeter" has the most potential of all in-situ devices since soil properties are derived using primary correlations which are theoretical based to analyse a test with only a few assumptions.

Alternatively, the in-situ test data can be used directly in design methods developed specifically for that test (SPT, CPT, Pressuremeter Test, etc.). Baguelin (1989), in his report to the 12th ICSMFE, considered that the theoretical interpretation of in-situ tests to derive basic soil parameters could only be justified if the application of those parameters in theories of design could be calibrated and validated against an extensive database of the behaviour of full-size structures.He concluded that direct methods of design are more efficient in many cases.

2.3.3 Standard Penetration Test (SPT)

Since its introduction in the United States in 1902 as driving a 25 mm diameter open-end pipe into soil during a wash boring process, Standard Penetration Test (SPT) has become one of the most important and widely used test among the in-situ field works.

2.3.3.1 General

SPT (Standard Penetration Test) is an important in-situ test in site exploration and foundation design due to its results that give a qualitative guide to the in-situ engineering properties and design concept in addition to its capability of providing samples of soils for identification and classification studies to be performed in laboratory.

The test was originally developed in the late 1920s and has been used most extensively in North and South America, the United Kingdom, and Japan. Because of this long record of experience, the test is well established in engineering practice despite of many problems that affect its accuracy and reproducibility.

2.3.3.2 Test Procedure

The test procedure of Standard Penetration Test (SPT) is established by ASTM D1586 in 1958:

1. Drill a 2.5 to 8 in (60-200 mm) diameter exploratory boring to the depth of the first test (Figure 2.5).

2. Insert the SPT sampler (also known as a split-spoon sampler) into the boring. The shape and dimensions of this sampler are shown in Figure 2.6. It is connected via steel rods to a 140 Ib (63.5 kg) hammer, as shown in Figure 2.7.

Figure 2.5 Exploratory boring drilled for SPT performance on field during a site investigation
Black Sea Region / *TURKEY*

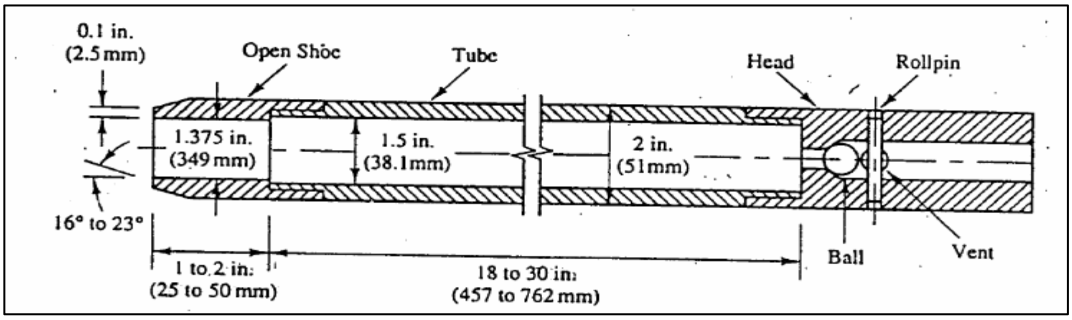


Figure 2.6 A Typical Split-Spoon Sampler

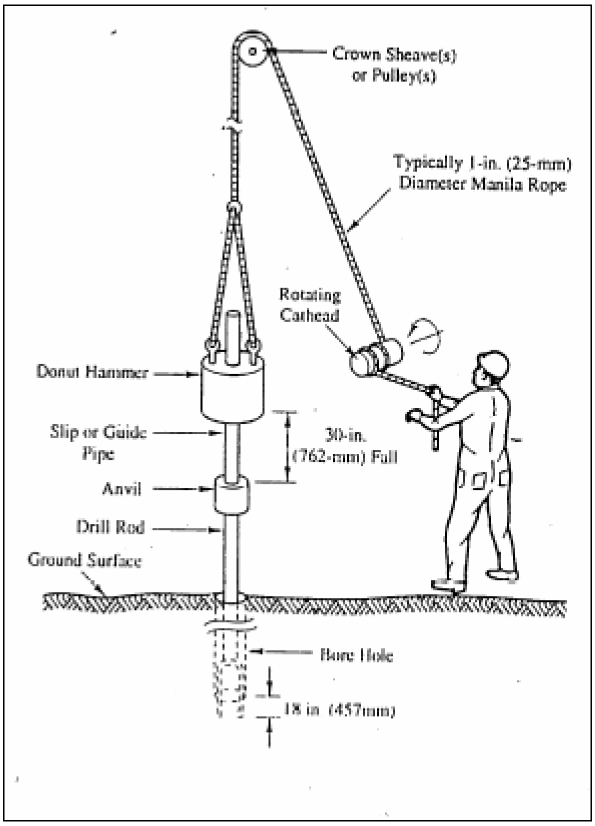


Figure 2.7 SPT Performance Illustration

3. Using either a rope and cathead arrangement or an automatic tripping mechanism, raise the hammer a distance of 30 in (760 mm) and allow it to fall. This energy drives the sampler into the bottom of the boring. Repeat this process until the sampler has penetrated a distance of 18 in (450 mm), recording the number of hammer blows required for each 6 in (150 mm) interval. Stop the test if more than 50 blows are required for any of the intervals, or if more than 100 total blows are required. Either of these events is known as refusal and is so noted on the boring log.

4. Compute the N-value by summing the blow counts for the last 12 in (300 mm) of penetration. The blow count for the first 6 in (150 mm) is retained for reference purposes, but not used to compute N because the bottom of the boring is likely to be disturbed by the drilling process and may be covered with loose soil that fell from the sides of the boring. Note that the N-value is the same regardless of whether the engineer is using English or SI units.

5. Remove the SPT sampler; remove and save the soil sample.

6. Drill the boring to the depth of the next test and repeat steps 2 through 6 as required.

2.3.3.3 Interpretation of Test Results

The interpretation of standard penetration test result, number N, is sensitive to the variations in test procedure and poor workmanship, although a standard is established in general. These variations are primarily resulted due to method of drilling, cleaning of the bottom of bore hole, type and location of hammer, number of turns around cathead, etc.

The raw SPT data could be improved partially by applying certain correction factors for the purpose of compensation of these variatons in testing procedures (Skempton, 1986):

$$N_{60} = \frac{E_m C_B C_S C_R N}{0.60} \quad (2.1)$$

where:

N_{60} = SPT N value corrected for field procedures
E_m = hammer efficiency
C_B = borehole diameter correction
C_s = sampler correction
C_R = rod length correction
N = measured SPT N value

The use of SPT N corrections in field procedures (emprical correlations or direct design methodologies) are usually limited to their clear definitions and indications in any approach of property estimation or design process.

Despite of some disadvantages and limitations, SPT has major advantages over other in-situ test methods. Most of the methods do not include sample recovery , so soil classification must be based on conventional sampling from nearby borings and on correlations between the test results and soil type. However, SPT obtains a sample of the soil being tested which permits direct soil classification. Another advantage is that it is very fast and inexpensive because it is performed in borings that would have been drilled in anyway. Finally, nearly all drill rigs used for soil exploration are equipped to perform this test, whereas other in-situ tests require specialized equipment that may not be readily available.

The standard penetration test and the number N should be used only very cautiously for quantitative analysis since they have substantial qualitative value. But, when the use of N number is considered with the sample of the soil obtained and related to a site and “site-specific” experience, prediction by the crude and decried SPT tests does not come out worse than predictions by other method of analysis (Fang, 1991).

2.3.3.4 Predictions Based on SPT correlations in Cohesive Soils

The long record of experience on Standard Penetration Test established many prediction methodologies for soils in general, but much more for granular soils, rather than cohesive materials. The interpretation and use of SPT N number for cohesive soils is usually a kind of cross-check for laboratory test results performed on samples. However, estimations from SPT results could be a leading method to the design, especially in case of lack of opportunity for laboratory study.

The emprical correlations of SPT results in cohesive materials in general cover consistency, unconfined compressive strength (q_u), undrained shear strength (c_u), coefficient of volume compressibility (m_v) and modulus of deformation (E_s).

The British Standard "Code of practice for site investigations" presents a scale for strength of clays estimated in the field (Table 2.1). The scale is described in terms of undrained shear strength.

The assessment of undrained shear strength is affected by a number of factors including fabric, sample disturbance, moisture content and stress changes. Where such assessment is critical, appropriate testing should be carried out. This may be particularly marked in very soft or very stiff clays, but may occur throughout the strength range. (BS 5930, 1999)

Table 2.1 The scale for strength of clays (BS 5930, 1999)

Consistency	Undrained Shear Strength, c_u (kN/m^2)
Very soft	less then 20
Soft	20 to 40
Medium	40 to 75
Stiff	75 to 150
Very stiff	150 to 300
Hard (or very weak mudstone)	Greater than 300

Clays with undrained shear strength greater than about 300 kN/m^2 can be described as hard clay. The field assessment of 300 kN/m^2 is not easy, being beyond the range of hand penetrometers and thumb nails, but such clays, in their saturated condition, break in a brittle manner.

Some of the common properties of clay soils including relationships between consistency, strength and SPT N values according to Hunt (1984) are given in Table 2.2.

Table 2.2 Common properties of clay soils (Hunt, 1984)

Consistency	N	Field Identification (Hand Test)	Unconfined Compressive Strength, q_u (kN/m^2)
Very soft	< 2	Extrudes between fingers	< 25
Soft	2-4	Molded by slight pressure	25 - 50
Medium	4-8	Molded by strong pressure	50 - 100
Stiff	8-15	Indented by thumb	100 - 200
Very stiff	15-30	Indented by thumbnail	200 - 400
Hard	>30	Difficult to indent	> 400

Unconfined compressive strength q_u is usually taken as equal to twice the cohesion c or the undrained shear strength c_u:

$$q_u = 2c_u \tag{2.2}$$

Similar to undrained shear strength, an accurate measurement of unconfined compressive strength in laboratory is highly susceptible to the fabric of material and sample quality. It could be indicated that for N>30 (hard clays), significant amount of disturbance in samples are possible arising from sampling process, laboratory handling and sample preparation in the lab. In addition, since hard clays have fissures, laboratory samples could not be representative for the mass behaviour of the material and may mislead the designer.

The consistency, and so SPT N number, is somehow an indication for cohesive materials in the manner of their consolidation histories, ages and cementations (Table 2.3). The increasing stiffness -together with standard penetration resistance- generally implies older, cemented clay materials as a geological formation, and also an increasing overconsolidation ratio whereas very soft to medium clays are usually younger deposits of normally consolidated materials.

Table 2.3 Consistency of Saturated Cohesive Soils*
Bowles (1996)

Consistency		N'_{70}	q_u (kPa)	Remarks
Very soft	NC, Young Clay	0-2	< 25	Squishes between fingers when squeezed
Soft		3-5	25-50	Very easily deformed by squeezing
Medium		6-9	50-100	-
Stiff	Aged / Cemented, Increasing OCR	10-16	100-200	Hard to deform by hand squeezing
Very stiff		17-30	200-400	Very hard to deform by hand squeezing
Hard		>30	> 400	Nearly impossible to deform by hand

* Blow counts and OCR division are for a guide - in clay where “exceptions to the rule” are very common.

In addition to the tabulated values in the form of ranges, the standard penetration tests performed on cohesive materials have been studied by several authors to be used for the prediction of unconfined compressive strength of that material in graphical forms (Figure 2.8).

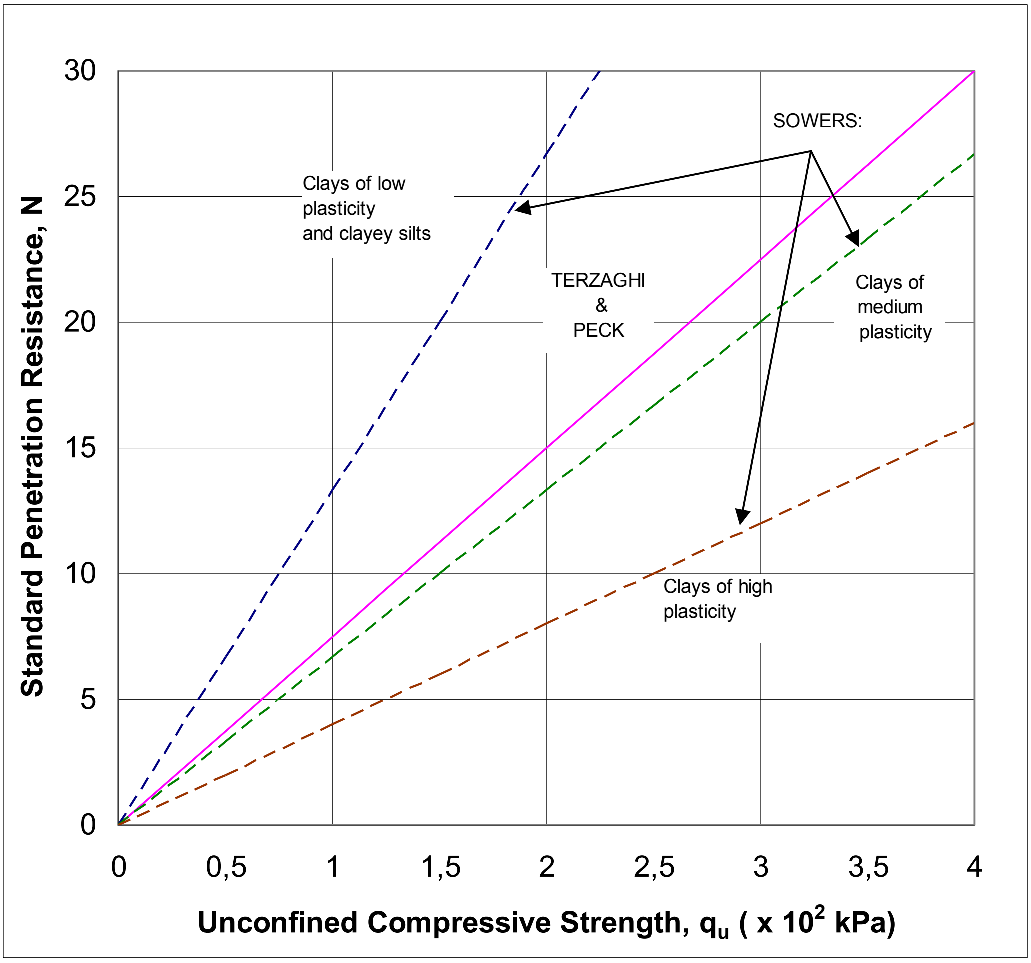


Figure 2.8 Correlations of SPT N values & q_u (NAVFAC, 1971)

Sowers (1954) correlates N and q_u values for highly plastic clays, clays of medium plasticity and clays of very low plasticity.

If the study of Terzaghi & Peck (1967) given in Figure 2.8 is investigated in a quantitative way (Table 2.4), the N & q_u values come out in an agreement with those of Hunt (1984).

Table 2.4 N & q_u correlation (Terzaghi & Peck, 1967)

N (blows/300 mm)	< 2	2-4	4-8	8-15	15-30	> 30
q_u (kPa)	< 25	25-50	50-100	100-200	200-400	>400

Peck & Reed (1954) have plotted hundreds of data on Chicago clays and suggested a conservative boundary of $q_u = N/6$, for use in estimating the allowable bearing pressure of footings, although their data approximates $q_u = N/4$.

De Mello et al. (1959) obtain a statistical relationship for an unsaturated silty clay:

$$q_u = 0.061N + 1.3 \quad (kg/cm^2) \qquad (2.3)$$

One of the most wide-spread correlations used in practice is proposed by, Stroud, in 1st Europian Conference on Penetration Testing, Stockholm (1974), who mainly demonstrates that the SPT can be a reliable and valuable means of estimating the properties of clays in situ with the considerable advantages of cheapness and simplicity.

He clearly states that for granular materials practising engineers in many countries have over the years built up their own body of experience using the S.P.T. which is perhaps the most widely used in-situ test in site investigation work, on the other hand in cohesive materials its application has not been widespread and has been treated with much greater caution.

In United Kingdom, engineers have preffered to obtain their clay characteristics by sampling in the field and testing in the laboratory. For many clays, this technique work well but for very stiff to hard fissured clays, weak rocks and for many glacial deposits the difficulties of obtaining undisturbed samples are great and the final laboratory results are often unrepresentative of the in situ characterisitcs or they exhibit so much scatter as to be practically meaningless (Stroud, 1974).

For the materials mentioned above, the case in-situ testing is very convincing. Generally the strength of such deposits is high and on the borderline of the economic and physical capabilities of many static penetrometers.

The Standard Penetration Test on the other hand is capable of providing results in these materials and is moreover cheap and simple to operate.

Stroud (1974) has studied a total of 1200 SPT from a number of 42 sites around U.K. together with undrained triaxial tests and oedometer tests, extending the correlations to a wide variety of insensitive clays and weak rocks. He also indicated the correlations by a study of plate loading tests made at a further 13 sites and previously studied ones by Marsland (1971).

The standard penetration resistance (N) is to be correlated to the undrained shear strength of cohesive soils by means of the common description of Stroud (1974):

$$c_u = f_1 \times N \tag{2.4}$$

where the factor f_1 is a variable depending on the plasticity. The change of f_1 with plasticity index is given in Figure 2.9.

The compressibility of clay is also evaluated from the relationship between SPT N-values, plasticity index (PI) and volume compressibility coefficient proposed by Stroud (1974):

$$m_v = \frac{1}{f_2 N} \tag{2.5}$$

The factor f_2 is also a variable depending on the plasticity. The change of f_2 with plasticity index is given in Figure 2.10.

The undrained triaxial tests included in Stroud (1974) are quick tests and m_v values from oedemeter tests are obtained taking the effective stress at sample depth in to account.

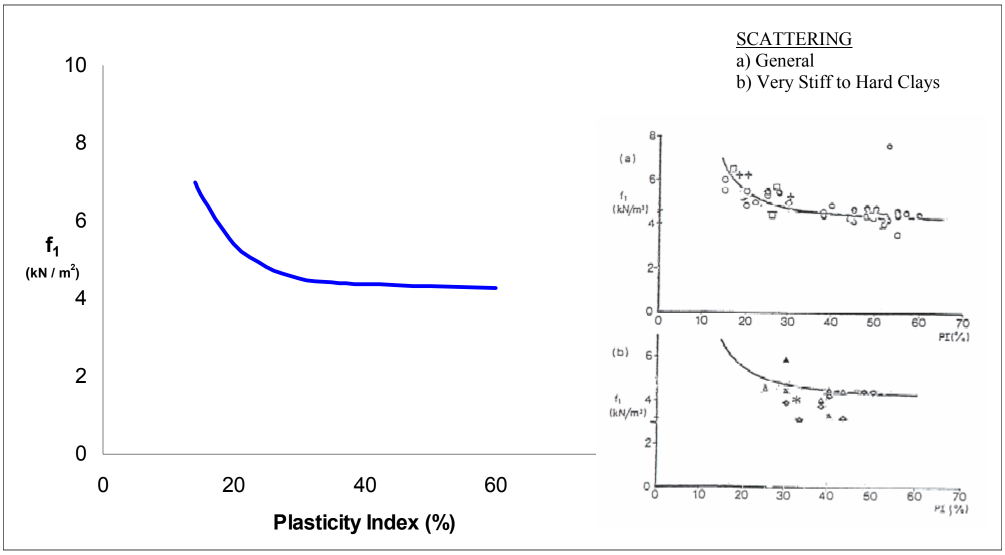


Figure 2.9 Change of the factor f_1 with plasticity index PI (Stroud, 1974) [together with scattering]

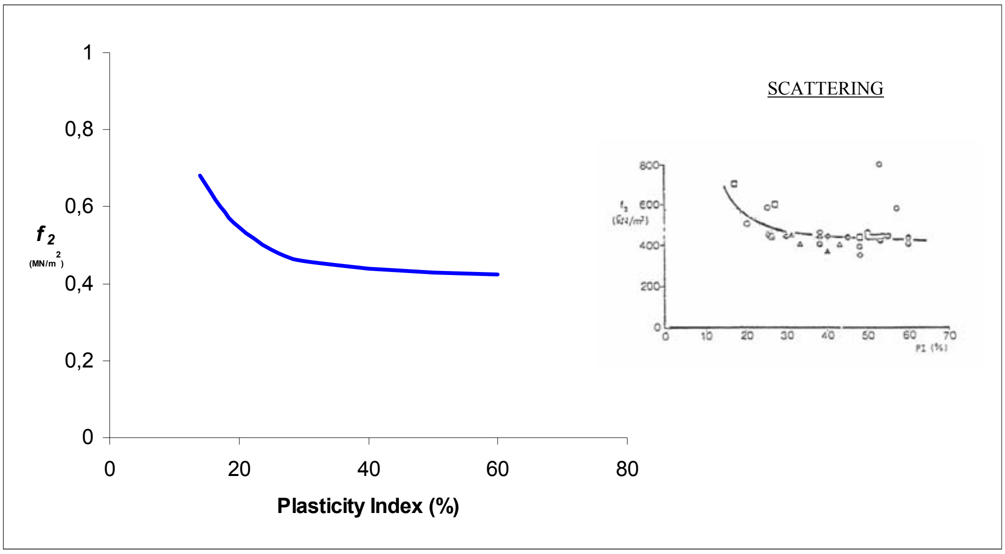


Figure 2.10 Change of the factor f_2 with plasticity index PI (Stroud, 1974) [together with scattering]

The correlations for the estimation of deformation (stress-strain) modulus (E_s) of soils based on SPT results are generally proposed for granular type materials, as it was for the whole SPT based predictions. However, there are some estimations using N and c_u of cohesive materials.

Yoshida & Yoshinaka (1972) proposed a crude correlation for the deformation modulus of cohesive soils presented in the Equation 2.6 with an average error close to ± 20 percent (Bowles, 1977).

$$E_s = 6\ N \quad (\mathrm{x}10^2\ \mathrm{kPa}) \qquad (2.6)$$

For clayey sand or sand-clay mixtures, the equation proposed by Bowles (1988) may be used in a condition that N value should be estimated as N_{55} :

$$E_s = 320\ [N+15] \quad (\mathrm{kPa}) \qquad (2.7)$$

The approximate predictions of deformation modulus using the undrained shear strength are presented in Tables 2.5 & 2.6.

Table 2.5 Typical E_s & c_u correlations (Bowles, 1988)

CLAY	PI > 30 or organic	E_s = 100 to 500c_u
	PI < 30 or stiff	E_s = 500 to 1500c_u

Table 2.6 Typical E_s & c_u correlations (Hunt, 1986)

CLAY	E_s
Soft Sensitive	$500c_u$
Firm to Stiff	$1000c_u$
Very Stiff	$1500c_u$

[After CGS (1978) and Lambe & Whitmann (1969)]

2.3.4 Pressuremeter Test (PMT)

An emprical approach has been developed and practised in France over the past years with considerable success: results from pressuremeter tests are emprically related to geotechnical parameters or, more directly, to performance of foundations (Mair & Wood, 1987)

The test, pressuremeter, is one of the most efficient and successfull in-situ tests with a great capability of that it can be used in all ground conditions.

2.3.4.1 General

The term pressuremeter was first used by Menard to describe the testing equipment he developed in 1955. Baguelin et al. (1978) referred to the pressuremeter probe as a device that applies hydraulic pressure through a flexible membrane to the borehole walls. Mair and Wood (1987) further restricted the definition of a pressuremeter to a cylindrical device and this definition is recognised internationally by the ISSMFE (Amar et al, 1991)

The definition given by Clarke (1995) is that a pressuremeter is a cylindrical probe that has an expandable flexible membrane designed to apply a uniform pressure to the walls of a borehole.

The pressuremeter, in general, can refer to the probe, drill rods and testing equipment.

Figure 2.11 A Pressuremeter Test Experiment on Field

The basic idea behind the pressuremeter test is the expansion of a cylindrical cavity formed in the ground in order to measure a relationship between pressure and deformation for the soil. In practice this is done by drilling a hole down to the level at which the test is to be made; a length of this hole forms the cylindrical cavity. The pressuremeter probe is inserted and then inflated to expand the cavity, while a record is kept of the resulting volume change (Figure 2.11). The probe is designed so that the length of the cavity does not change; the increase in volume is due only to radial expansion of the hole (Baguelin et al., 1978).

2.3.4.2 Test Equipment & Procedure

The pressuremeter consists of three parts as illustrated in Figure 2.12. These are the probe, the control unit and the tubing.

a) Probe (Menard Type)

The pressuremeter probe in sum consists of the tree cells, a steel core which ties them all together and keeps them in line, and a system of attachment to the tubing which comes from the control unit (also providing the lowering and raising of probe).

The two cells at the top and bottom part of the probe are named “guard cells” and then one between them is “measuring cell” which forms cavity. Water is used to pressurize the cavity (measuring cell) and to measure the resulting volume change.

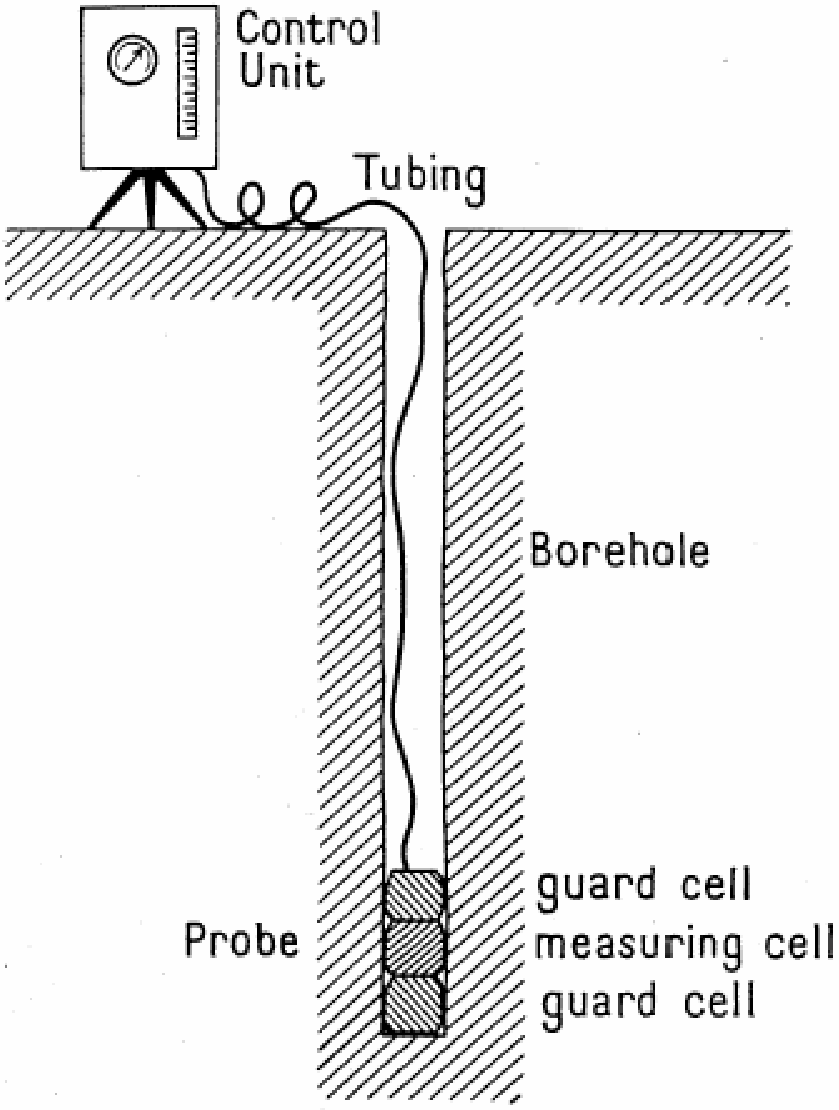


Figure 2.12 Basic Principles of the Pressuremeter
(Baguelin, 1978)

Measuring cell is a flexible, impervious rubber bladder and so the guard cells are. To make sure that the cavity expands as it should, the measuring cell is flanked top and bottom by guard cells which are inflated, usually by gas, to the same pressure as the measuring cell. The inflated guard cells effectively seal off the borehole and prevent the measuring cell membrane from expanding into the void of the hole (Baguelin et al., 1978).

b) Control Unit

The control unit is located at a suitable place on the ground surface close to the borehole. Its function is to control and monitor the expansion of the probe. It does this by applying a given pressure on command to the probe and then measuring the volume change of the measuring cell. The pressure source is a bottle of compressed gas and the flow of water to the measuring cell is monitored using a graduated cylinder which is called the volumeter (Baguelin et al., 1978).

c) Tubing

Tubing is required between the control unit and the probe to allow water and gas to be sent from one to another.

A major difference between categories of pressuremeter tests lies in the method of installation of the pressuremeter device in the ground. The following categories of test can be distinguished in terms of the installation method:

1) Menard-Type Pressuremeter (MPM): The device is lowered into a preformed hole of which is usually slightly oversized. (Different probe types are E, GC, GB)

2) Self-Boring Pressuremeter (SBP): The device bores its own way into ground.

3) Push-In Pressuremeter (PIP): The device is pushed into the ground below the base of a borehole, has been principally developed for offshore use.

The MPM category tests have been much more widely experienced since it is the origin of all devices which is named after the principal developer of the pressuremeter test, Louis Menard.

<u>Test Procedure</u>

The test is carried out by applying pressure in increasing steps of equal magnitude and duration. The pressure increments should be selected so that the limit pressure is attained in ten steps. Each step has a duration of 1 min., and the volume changes are read at 15, 30, and 60 s after the application of the pressure increment to obtain the creep deformations in the soil. The test is considered completed when the volume injected in the probe corresponds to 100% increase of the initial volume of the borehole. Special testing techniques apply for tests in rock.

2.3.4.3 Interpretation of Test Results

The result of any pressuremeter test is the raw data giving a pressure-volume curve, the pressure being the total pressure in the cavity. Before this can be interpreted, it has to be reduced to a corrected curve.

Corrections must be made for:

(a) The resistance of the probe itself to expansion. The probe normally consists of both a rubber membrane and a metallic protective cover, and a test is normally carried out at ground surface to determine the relation between volumetric expansion and required pressure for the unconfined probe. This is deducted from test results.

(b) The expansion of the nylon tubes connecting the probe with the pressure-volumeter. The effect can be assessed by a surface test in which the probe is confined, and all volume change takes place in the leads.

(c) Hydrostatic effects, due to the fact that the measuring cell and its leads are filled with water, while the guard cell and leads contain air. At depths in excess of 30 m it is necessary to use

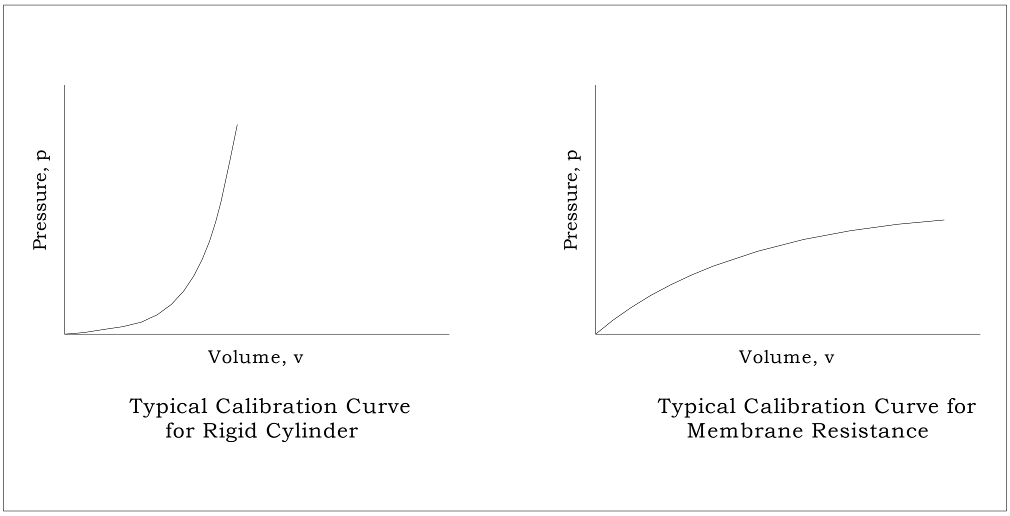


Figure 2.13 Typical Pressuremeter Calibration Curves

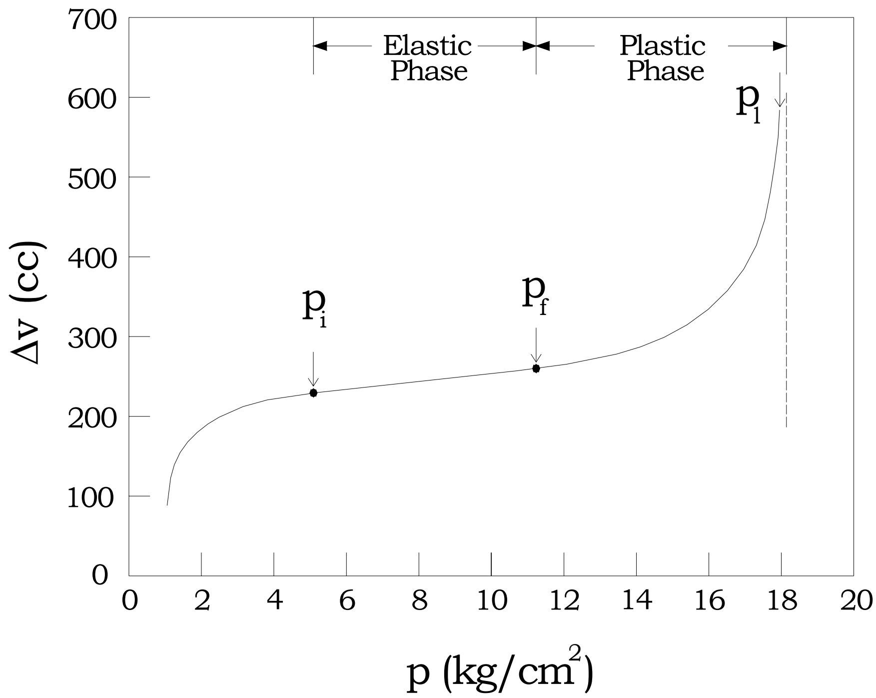


Figure 2.14 Corrected Pressuremeter Curve

two pressure sources in order to give equal guard and measuring cell pressures (Gibson and Anderson, 1961)

All the calibrations that of are vital parts of the pressuremeter test procedure are applied using the calibration curves (Figure 2.13) that should be obtained for each individual test equipment.

From the shape of the corrected pressuremeter curve, three characteristic pressures can be defined (Figure 2.14)

In the first stage of the test, the volume increases rapidly with pressure as the probe is inflated against the undisturbed soil. The pressure p_i at the end of this stage represents the start of elastic phase and gives a general idea for the estimation of the in-situ total horizontal stress in the ground although it is not possible to obtain the exact value.

Following the pressure p_i, the volume increases slowly and linearly with increasing pressure, which indicates the elastic behaviour of the soil. The slope of this part of pressure-volume curve is related to the elastic modulus of the soil. This linear stage ends at the yield pressure (creep pressure) p_f.

Beyond the yield pressure, the volume increase rapidly implying the development of soil failure around the probe. With increasing pressure, the pressure-volume curve tends to an asymptotic limit corresponding to the limit pressure p_l.

The volume measurements corresponding to p_i and p_f values are named as v_i and v_f, respectively.

The net limit pressure, p_{ln}, which is of great interest in foundation design is defined as:

$$p_{ln} = p_l - p_o \qquad (2.8)$$

The term, p_o is defined as the horizontal total pressure at rest condition in the ground at testing level. In terms of total stress, it is calculated from the equation:

$$p_o = [z\,\gamma - u]\,K_o + u \qquad (2.9)$$

where

z = depth from the ground surface to the center of the probe
γ = unit weight of the soil
u = pore water pressure at the level of the probe
K_o = coefficient of earth pressure at rest

While theoretically p_i should be equal to the value p_o, in practice it has been found that p_o is very difficult to determine accurately since in most tests there are only a few points on the test curve in this early phase of the test. In addition, it is this early phase that is most influenced by disturbance to the walls of the borehole (Baguelin et. al, 1978).

The approach given by Baguelin et al. (1978) propose the determination of p_o from the Equation 2.9 taking in to account the type and condition of the soil to estimate K_o (from literature) and u (from groundwater level). However, an exception to the rule is described relating to the overconsolidation of soils, since great inaccuracies are encountered to estimate K_o when the materials are to be overconsolidated (K_o between $0.5 \approx 3.0$).

The Menard's modulus of pressuremeter, E_M is determined from the slope of the linear section of the pressure-volume curve :

$$E_M = 2.66\left(Vc + \frac{v_f + v_i}{2}\right)\frac{p_f - p_i}{v_f - v_i} \qquad (2.10)$$

where $V_c = 535$ cm^3 as the volume of measuring cell.

While obtaining the modulus E_M, the shear modulus, $G = V^*[\Delta p/\Delta V]$ (Lame, 1852) of a cylindrical cavity is converted to something roughly equivalent to a Young's modulus E where $G = E/[2(1+\upsilon)]$ assuming the soil is elastic with a Poisson's ratio $\nu = 0.33$ which is chosen by Centre d'Etudes Menard (1967) as a constant value for all soils to compute E_M.

An important point concerning the interpretation of pressuremeter test results and hence their use in design is that the results of a pressuremeter test are functions of the installation procedure and test procedure as well as the methods of analysis and interpretation used. It is important to ensure that the installation procedure produces repetable minimum disturbance to the ground (Clarke, 1995)

2.3.4.4 Application of Pressuremeter Testing to Design

Mair and Wood (1987) states that the principal attraction of the pressuremeter test in geotechnical engineering practice is that the boundary conditions are controlled and well defined, as are the stress and strain conditions in the surrounding soil mass.

As a method of foundation engineering, the pressuremeter method has a number of advantages. It is based on an in-situ test thus meeting an important requirement of modern soil mechanics. Unlike other in-situ tests such as penetration or vane tests, the pressuremeter measures deformation properties of the soil in addition to a rupture or limit resistance. The engineer can now benefit from the considerable advantage of having deformation available as a matter of routine. A further advantage is that the pressuremeter results are based on a test which involves a fairly large volume of soil, something that can not be said of cone penetration tests (Baguelin et al., 1978).

Another statement is given by Clarke (1995) that the pressuremeter test most closely models an ideal condition in which the ground is positively loaded from the in-situ stress conditions.

If some other major advantages of the pressuremeter test such as its capability to work in most types of soil, to model the way in which an actual foundations behave and to be used directly for foundation performance prediction are also considered, its use in design could be accepted as an important in-situ tool.

The pressuremeter test results in geotechnical engineering practice are utilized in different ways that of are soil identification, parameter estimation and direct design.

Table 2.7 Approximate common values for the pressuremeter parameters (Briaud, 1992)

CLAY					
Consistency	Soft	Medium	Stiff	Very Stiff	Hard
p_{ln} (kPa)	0 - 200	200 - 400	400 - 800	800 - 1600	> 1600
E_M (kPa)	0 - 2500	2500 - 5000	5000 - 12000	12000 - 25000	> 25000

Table 2.8 Values of Menard pressuremeter modulus E_M and limit pressure p_l from Menard pressuremeter testing (Hunt, 1986)

SOIL TYPE	E_M (x 10^2 kPa)	p_l (x 10^2 kPa)
Peat and very soft clay	2 – 15	0.2 – 1.5
Soft clay	5 – 30	0.5 – 3.0
Firm clay	30 – 80	3.0 – 8.0
Stiff clay	80 – 400	6 – 25

From SOLS SOILS (1975)

The type of soil tested in fact can not be identified clearly by the pressuremeter test results alone because the pressuremeter only imposes a series of total stress changes in the ground in a relatively short time. If, however, the soil can be classified by some other method as clay, sand, or whatever, from augering cuttings or direct sampling, then the condition of the soil can be determined from the pressuremeter results (Table 2.7, 2.8, 2.9).

Table 2.9 Field guide to soil condition based on p_{ln} & c_u (Baguelin et. al., 1978)

p_{ln} (kPa)	Description	Field Test	Undrained Shear Strength c_u (kPa)
0 – 75	Very Soft	Penetrated by fist, squeezes easily between fingers	< 20
75 – 150	Soft	Penetrated readily by finger, easily moulded	20 – 40
150 – 350	Firm	Penetrated with difficulty, moulded by strong finger pressure	40 – 75
350 – 800	Stiff	Indented by strong finger pressure	75 – 150
800 – 1600	Very Stiff	Indented only slightly by strong finger pressure	> 150
> 1600	Hard	Can not be indented by finger pressure, penetrated by finger-nail or pencil point	

An alternative approach is that relationships between the pressuremeter characteristics p_f, p_l and E_M can also be used to assess the condition of the soil, but it must be born in mind that each of these characteristics is dependent on the quality of the borehole. The ratio of E_M/p_l has been found to be quite useful and typical values are presented in Table 2.10.

Table 2.10 Soil type from MPM tests (Clarke, 1995)

GROUND TYPE	E_M/p_l
Peat	8 – 10
Soft to firm clay	8 – 10
Stiff to very stiff clay	10 – 20
Loess	12 – 15

The ratio p_l/p_f is also useful, but it is more vague. In clays it will be in the neighbourhood of 1.6 to 1.8 (Baguelin et al., 1978).

The pressuremeter test results are utilized oftenly to predict the strength and deformation properties of clay type soils for use in traditional design calculations of bearing capacity and settlement.

The undrained shear strength (c_u) of a clay could be calculated from the limit pressure, p_l, obtained from pressuremeter test. Based on ideal elastic-plastic assumptions, three theoretical solutions have been proposed (Baguelin et.al., 1978):

Bishop, Hill & Mott (1945)

$$p_{\ln} = c_u \left[1 + \ln \frac{E}{2c_u(1+\nu)} \right] \qquad (2.11)$$

Hill (1950)

$$p_{ln} = c_u \left[1 + \ln \frac{E}{c_u(5-4\nu)} \right] \qquad (2.12)$$

Salençon (1966)

$$p_{ln} = c_u \left[1 + \ln \frac{E}{4c_u(1-\nu^2)} \right] \qquad (2.13)$$

During the pressuremeter test, the conditions for drainage is to be taken as undrained since no change in water content is assumed when the test duration and soil structure are considered. Therefore, a Poisson's ratio of $\nu = 0.5$ is to be assumed, which reduces all three equations of 2.11, 2.12, 2.13 to:

$$p_{ln} = c_u \left[1 + \ln \frac{E}{3c_u} \right] \qquad (2.14)$$

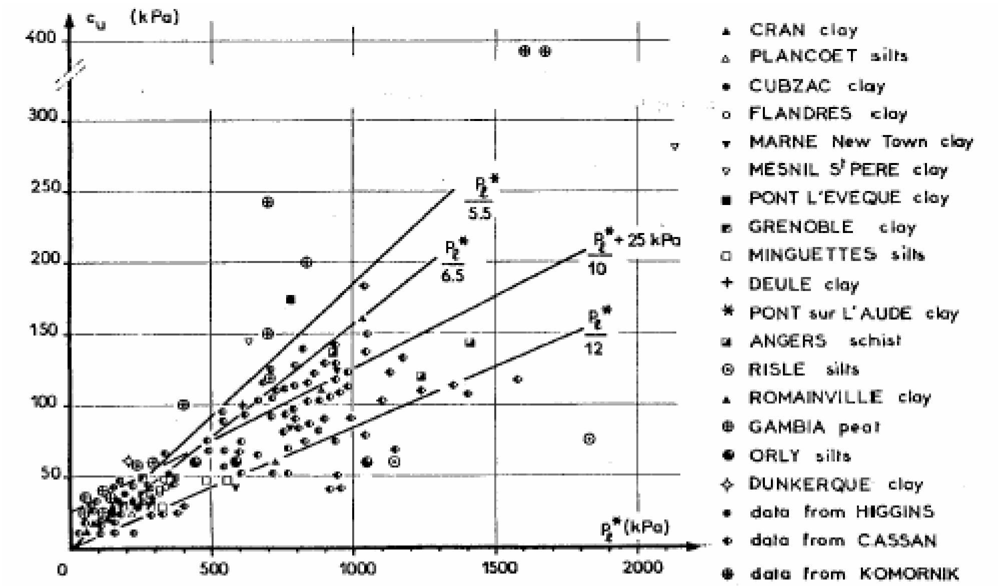
Equation 2.12 could be also written in the form

$$c_u = \frac{p_{ln}}{\beta} \qquad (2.15)$$

where $$\beta = 1 + \ln \frac{E}{3c_u} \qquad (2.16)$$

The values for $\frac{E}{c_u}$ are variable. Typical range of 200 to 2000 might be taken [D'appolina, Poulos and Ladd (1971)] and the β values then would come out as β =5.2 and β = 7.5 (Baguelin et al., 1978)

A number of published [Higgins (1969), Cassan (1972), Komornik et al. (1970)] and unpublished (on many types of cohesive soils from different cities) sources are assembled and inserted on a plot by Baguelin et al.(1978) and much of the points -ignoring some- are obtained to fall within a band given by β = 6.5 and β = 12 in the stiff to very stiff strength range with an average β of 9 (Figure 2.15).



p_l^* : p_{ln}

Figure 2.15 Scattering for experimental observations of c_u vs p_{ln} [Higgins (1969), Cassan (1972), Komornik et al. (1970) & others]

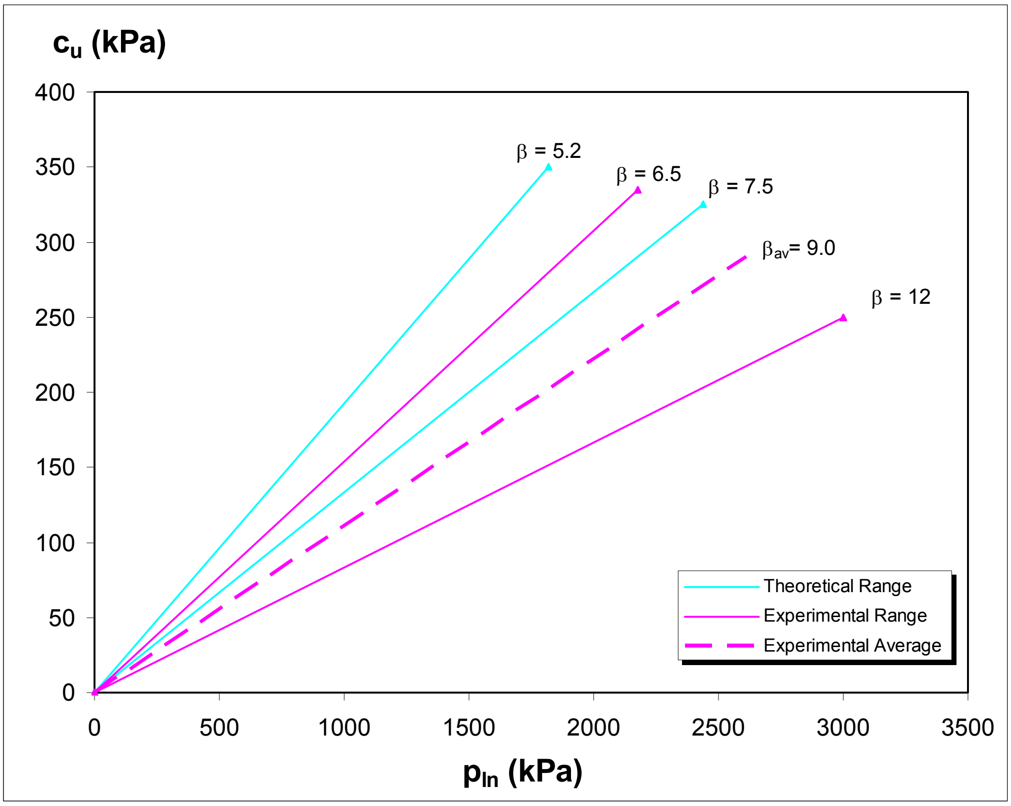


Figure 2.16 Comparison of β from theory and experiments

The theoretical values of β = 5.2 to 7.5 could be assumed as reasonably low at first sight when compared to the experimental results of β = 6.5 to 12 for clay of medium and higher strength (Figure 2.16). The probable cause could be that the values of c_u, which were measured in the laboratory by triaxial tests are too low. An example for this cause was faced by Meigh and Greenland (1965) in form of a plot in Figure 2.17 where β comes from the theoretical range.

Meigh & Greenland (1965) concluded that there was reasonable aggreement for shear strengths below about 150 to 200 kPa but above this the laboratory test values were significantly less than the pressuremeter values, demonstrating the effects of sample disturbance.

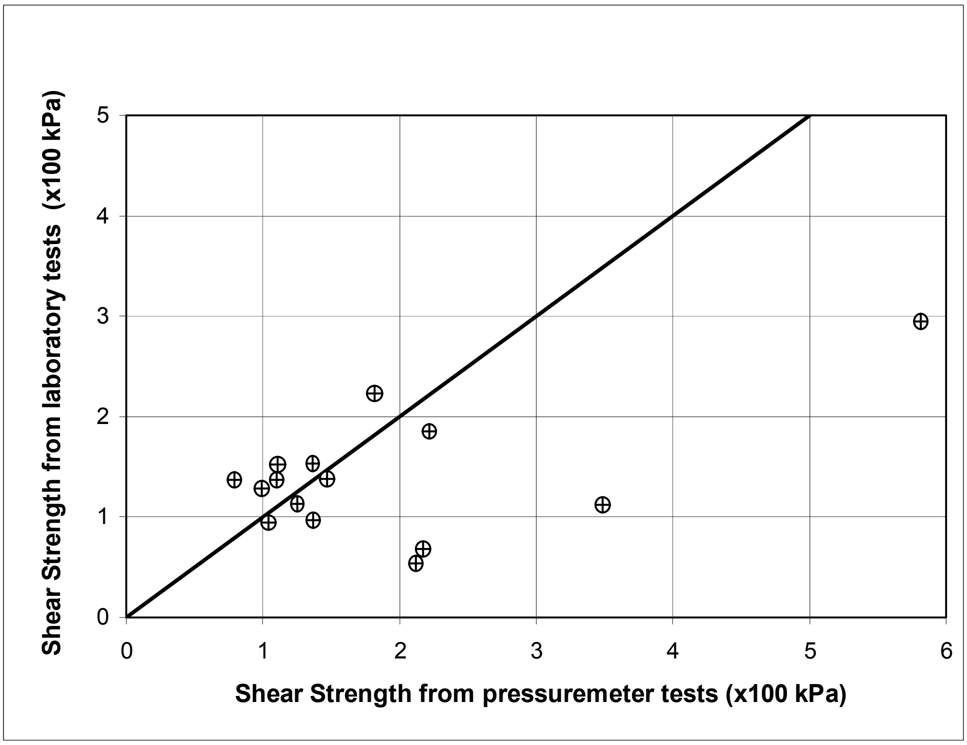


Figure 2.17 Pressuremeter c_u vs. Laboratory c_u
(Meigh & Greenland, 1965)

Another possibility is that the calculations of p_{ln} are in error (Baguelin et al., 1978). There might be many probable causes of that:

The value, p_o can neither be measured or calculated accurately, which directly effects the value p_{ln}. The long pressuremeter probe may bridge over thin or localized weak zones in the soil and tests a material that is stronger due to nonhomogenity in soil. The sensitivity also may be a factor in such a way that immediately adjacent to the probe the clay could be at disturbed or residual strength, while at some distance away it could be at its undisturbed, peak strength.

Relating to the low values of laboratory undrained shear strength, c_u, main causes are disturbance during sampling and handling which usually reduces the shear strength of test specimens. Recent work with a different type of pressuremeter (self-boring) by Amar et al., (1975) has shown that traditional ways of measuring c_u such as field vane or in the laboratory can seriously underestimate the true in-situ strength of clay soil.

Despite of having no precise values for p_{ln}, c_u and β, a number of recommendations are available by some authors in order to estimate c_u from pressuremeter tests:

Cassan (1972)	$\beta = 5.5$	(for low values of p_l)
	$\beta = 8$	(for middle values of p_l)
	$\beta = 15$	(for high values of p_l)
Amar & Jezequel (1972)	$\beta = 5.5$	(for $p_l < 300$ kPa)

Amar & Jezequel (1972)

$$c_u = \frac{p_{ln}}{10} + 25 \quad \text{(kPa)} \qquad (2.17)$$

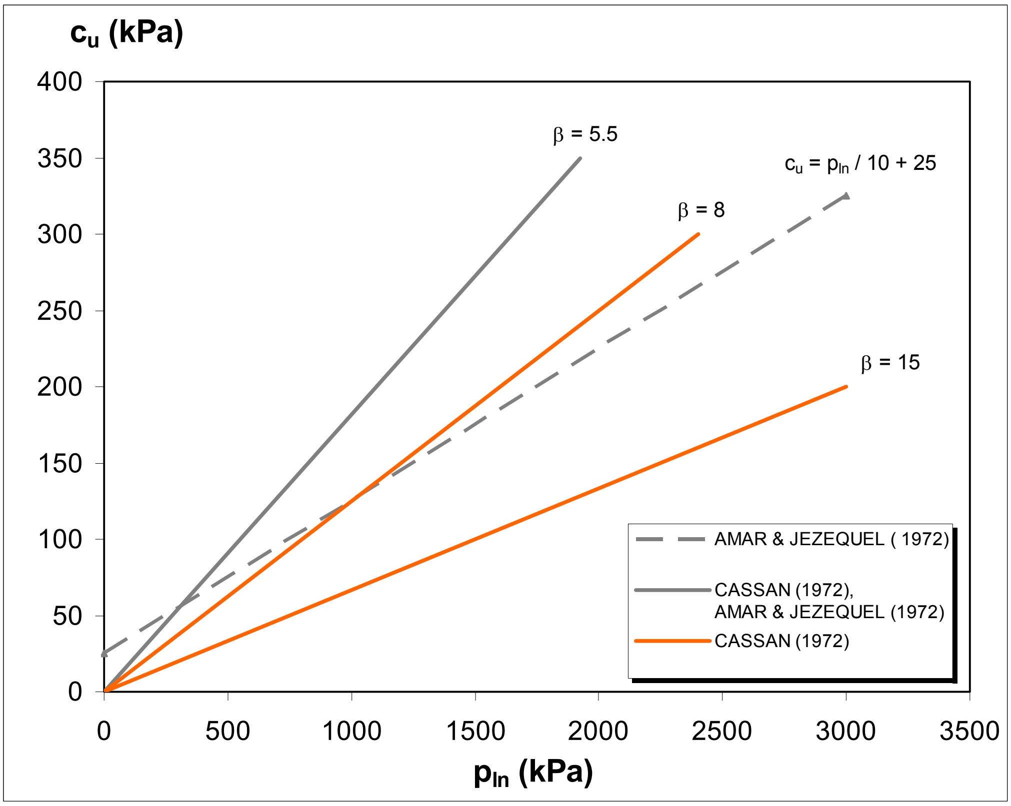


Figure 2.18 The variation of β values [Cassan (1972), Amar & Jezequel (1972)]

The value β=5.5 proposed by Cassan (1972) and Amar & Jezequel (1972) are usually for soft to firm clays, whereas the values β=8 (firm to stiff), β=15 (stiff to very stiff) by Cassan (1972) and Equation 2.17 by Amar & Jezequel (1972) are suggested for the strength range over firm (Figure 2.18).

Lukas & Leclerc De Bussy (1976) report on comparisons between p_{ln} and unconfined compressive strengths ($2c_u$) for hard clays in Chicago. They conclude that a value β = 5.1 best fits their results, whereas they have a spread in β values from about 3.8 to 6.4.

Clarke (1995) mentions practical values of β = 3.3 for soft clays and β = 12 for stiff clays. Some additional suggestions for the value β are as below:

Marsland & Randolph (1977) β = 6.8 (stiff clays)

Martin & Drahos (1986) β = 10 (stiff clays)

Mair and Wood (1987) have also indicated a value of β = 6.2 with a range of ± 10 %, and hence a β value from 5.5 to 6.8.

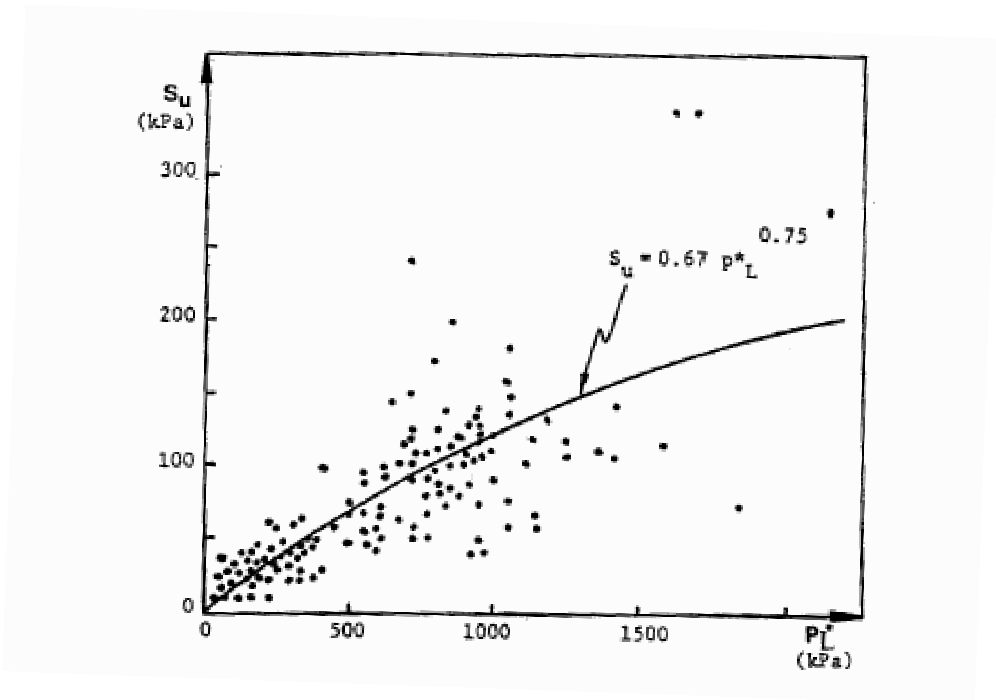


Figure 2.19 Correlation between c_u (s_u) and p_{ln} (p_L*)
(after Baguelin et al., 1978)

Baguelin et al. (1978) present an extensive comparison of undrained shear strength c_u and p_{ln}. The plot in Figure 2.19 shows that the ratio p_{ln} / c_u varies from about a 5.5 for clay with c_u values less than 50 kPa to 10 for clays with c_u values of about 150 kPa. This suggests a non-linear relationship between c_u and p_{ln} (Briaud, 1985):

$$c_u = 0.67\ p_{ln}^{0.75} \qquad (2.18)$$

One of the greatest potential for pressuremeter testing lies in the measurement of modulus. In many cases of geotechnical design, the soil or rock is assumed to behave elastically prior to failure, and calculations of ground and foundation deformation under working conditions can be performed with a knowledge of the elastic deformation modulus. Appropriate design values can be assessed from pressuremeter moduli (Mair and Wood, 1987).

Considering a corrected pressuremeter curve, between v_i and v_f the soil is said to behave as a more or less elastic material since the curve is approximately a straight line in this region. Then, from the slope of the linear part of pressure-volume expansion curve therefore a shear modulus, G is to be obtained based on the equation $G = V^*[\Delta p/\Delta V]$ by Lame (1852) for the radial expansion of a cylindrical cavity in an infinite elastic medium. However, G is often converted to an equivalent Young's modulus, E (for an isotropic material), using the expression of $G = E/[2(1+\upsilon)]$ where υ is Poisson's ratio (Mair and Wood, 1987). An assumption by Centre d'Etudes Menard (1967) of υ=0.33 for all soils leads to the Menard Pressuremeter Modulus, E_M, which results in $E_M = 2.66$ G.

The value of υ is often possible to be assumed reasonably in order to compute values of E, or instead E_p referring to a pressuremeter modulus since the modulus is obtained from the pressuremeter test. A saturated clay, for example, which does not change in water content during the test would have a Poisson's ratio of υ=0.5. Partially saturated soil and soil which drains slightly or completely during the test would have υ values as low as 0.1, but typically between 0.1 and 0.3. And for most foundation engineering problems it is really G and not E_p that matters. For these reasons, the proposal of Centre d'Etudes Menard (1967) has become as υ=0.33 value which is to be used to compute E_p and beside to convert E_p back to G whenever the shear modulus is required for all soils (Baguelin et al., 1978)

When using Young's Modulus in design, a distinction has to be made between undrained Young's modulus, E_u, and drained Young's modulus, E', depending on whether undrained or drained conditions apply to design in question (Burland et al., 1977).

The pressuremeter test measures G. The Young's modulus, E, deduced from a pressuremeter test, is dependent on the drainage conditions to which that value of E would be applied in design. It is not dependent on the drainage conditions operating during the pressuremeter test (Mair and Wood, 1987).

Menard and other investigators have long argued that the pressuremeter modulus may not be compared directly with a compression modulus such as Young's modulus due to some reasons (Baguelin et al., 1978):

a) The stress path followed in the soil around the pressuremeter probe is different from that in a compression test or under a plate or footing (Figure 2.20), and it is well known that the stress path has an important influence on the behaviour of soil.

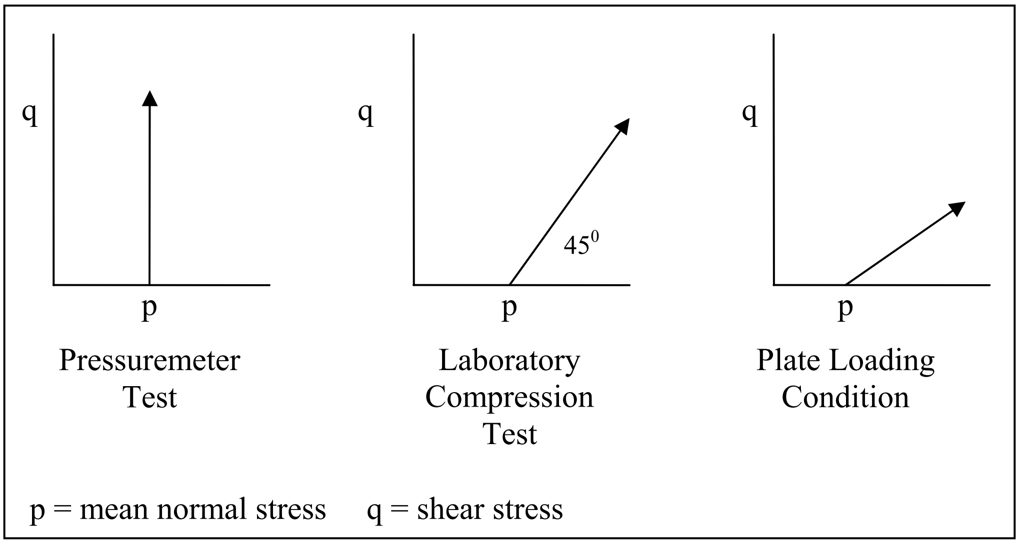


Figure 2.20 Comparison of stress paths

b) Simple elastic theory indicates that the increase in compressive stress in a radial direction equals the increase in tensile stress (actually the reduction in compressive stress) in a tangential direction during the elastic phase of a pressuremeter test. But, Menard argues that the compression modulus E^+, is different from the tension modulus E^-, and the pressuremeter modulus probably lies somewhere between two.

c) Menard states that the pressuremeter modulus is not a measure of what he calls the modulus of 'micro deformation' which is defined as the modulus of soil skeleton when it is subjected to very small strains. By very small strains, Menard means that the diameter of the probe could not increase by more than a few microns which is a precision required but beyond the capability of nearly all pressuremeters.

Combining the effects of the influence of the stress path, compression modulus E^+ not being equal to tension modulus E^- and the problem of microdeformation (small strain), the Centre d'Etudes Menard (1975) proposes a table of α values by which E_M should be divided to evaluate the Young's modulus E for a soil (Table 2.11).

$$E = \frac{E_M}{\alpha} \qquad (2.19)$$

Table 2.11 The rheological factor α for various soils (cited from Baguelin et al., 1978)

Soil Type	Peat		Clay		Silt		Sand		Sand & Gravel	
	E_M/p_{ln}	α	E_M/p_{ln}	α	E_M/p_{ln}	α	E_M/p_{ln}	α	E_M/p_{ln}	α
Over Consolidated			> 16	1	> 14	2/3	> 12	1/2	> 10	1/3
Normally Consolidated		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	Extremly fractured				Other				Slightly fractured or extremely weathered	
	α = 1/3				α = 1/2				α = 2/3	

In Equation 2.19, the value, E or E_s instead for Young's modulus could be defined as a soil deformation modulus in vertical direction (or under vertical loads) whereas the Menard Pressuremeter Modulus, E_M or Ep instead is a moduli measurement in lateral direction.

A compression modulus which is used in settlement calculations from the pressuremeter test is named as K_M which is equal to Young' modulus E and similarly defined as

$$K_M = E = \frac{E_M}{\alpha} \qquad (2.20)$$

by Menard and Rousseau (1962).

2.3.5 Correlations between SPT – PMT Results

Although it is an indirect method of predicting soil parameters, the results of in-situ tests are to be able to assembled and correlated with each other. This would give a brief idea and a rank for any in-situ design parameter such as N, p_l or E_M.

The comparison of Standard Penetration Test (SPT) and Pressuremeter Test (PMT) results has been generally performed on sand type of materials. Figure 2.21 shows the results obtained by some authors (Cassan, 1968-1969; Hobbs and Dixon, 1969; Waschkowski, 1976) from sand – silty sand and marl type soils.

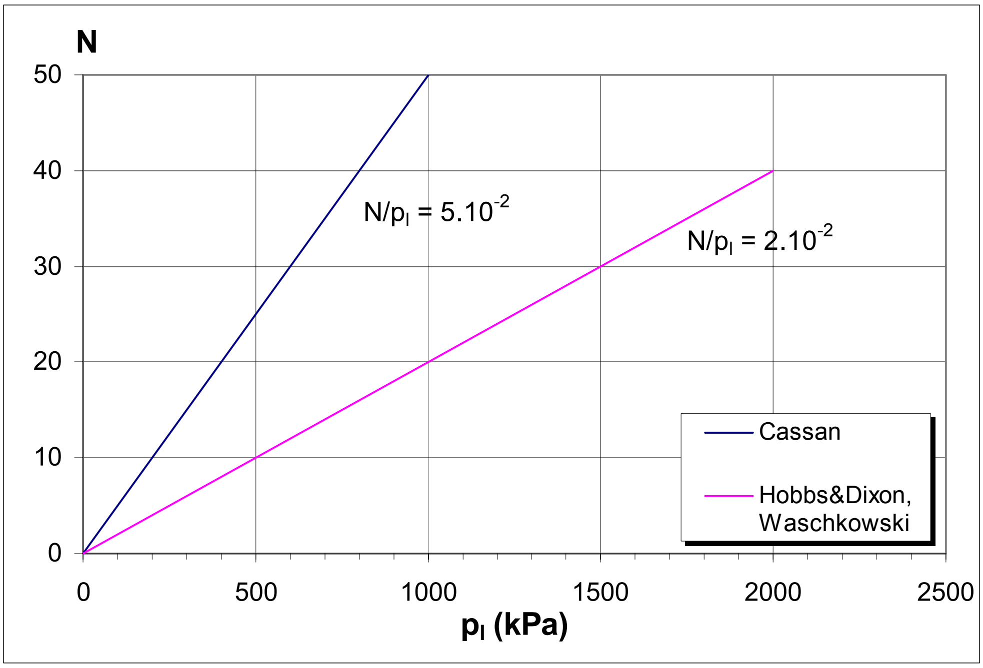


Figure 2.21 Correlation of SPT N vs. p_l (for sands)

The authors reported the observation of a large scatter for the comparison between SPT N and limit pressure p_l of pressuremeter. The ratio, N/p_l lies between $2x10^{-2}$ and $5x10^{-2}$. As a provisional recommendation, Baguelin et al. (1978) proposes that the value $N/p_l = 2x10^{-2}$ could be adopted for sands, and only for sands.

Waschkowski (1974) indicates that no relationship is proposed for clays, in view of the very large scatter obtained in the N measurements.

Pilot (1982) has listed a number of correlations between SPT N and PMT results. These are given in Table 2.12:

Table 2.12 Some correlations for SPT-PMT results (Pilot, 1982, cited in TDV)

Correlation	Soil Type		
	Clays	Silts	Sands
$\dfrac{N}{P_l - P_o}$	20 - 40	30	30 - 50
$\dfrac{N}{E_M}$	0.8 - 1.1	3	2 - 6

NOTE : Dimensions are in Mpa

CHAPTER 3

EXPERIMENTAL STUDIES

3.1 General

The experimental part of this study is covered by two main geotechnical investivatigon projects executed at different sites in Turkey:

* Ankara Rail Transit System / Stage 3 Works (Batıkent–Sincan)
* İzmir Bostanlı Shoreline / Atakent Project

Soil investigation reports that belong to above projects have been assembled and searched in detail with all appendices in order to form a database.

The number of investigation reports studied is 11 with a total number of 195 borings drilled in different type of soils in the projects. Eventually, a final database with a number of 351 data has been refined including both laboratory and in-situ tests together performed on cohesive materials (Appendix A).

Throughout the experimental study, especially two important in-situ tests, Standard Penetration Test (SPT) and Pressuremeter Test (PMT) have been investigated over the reports. As laboratory works, (UU) triaxial loading tests for undrained shear strength (c_u), and oedometer tests to obtain the coefficient of volume compressibility (m_v) were fixed together where in-situ tests exist for the same soils. Laboratory evaluations also involve huge number of sieve analysis and Atterberg limits tests that are used to identify and classify the soils. Then, the results of in-situ tests, SPT and PMT have been correlated to laboratory test results of c_u & m_v, furthermore to deformation modulus E using existing approaches and correlations in literature together with site-specific observations and evaluations.

The distributions of in-situ test results in terms of number of observations are given in Figures 3.1, 3.2 & 3.3:

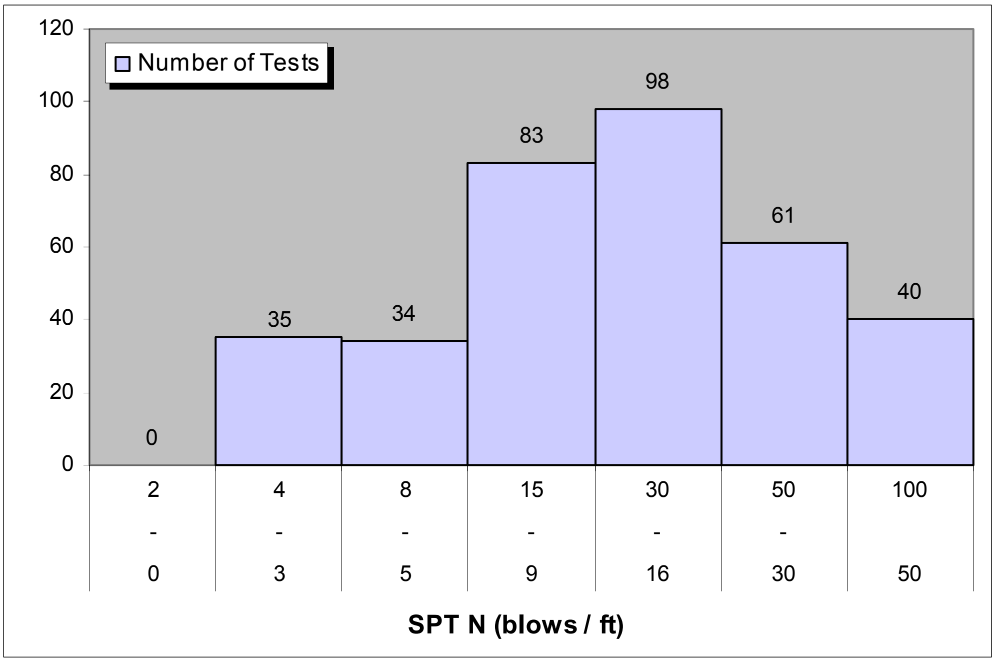


Figure 3.1 SPT N number distribution in database

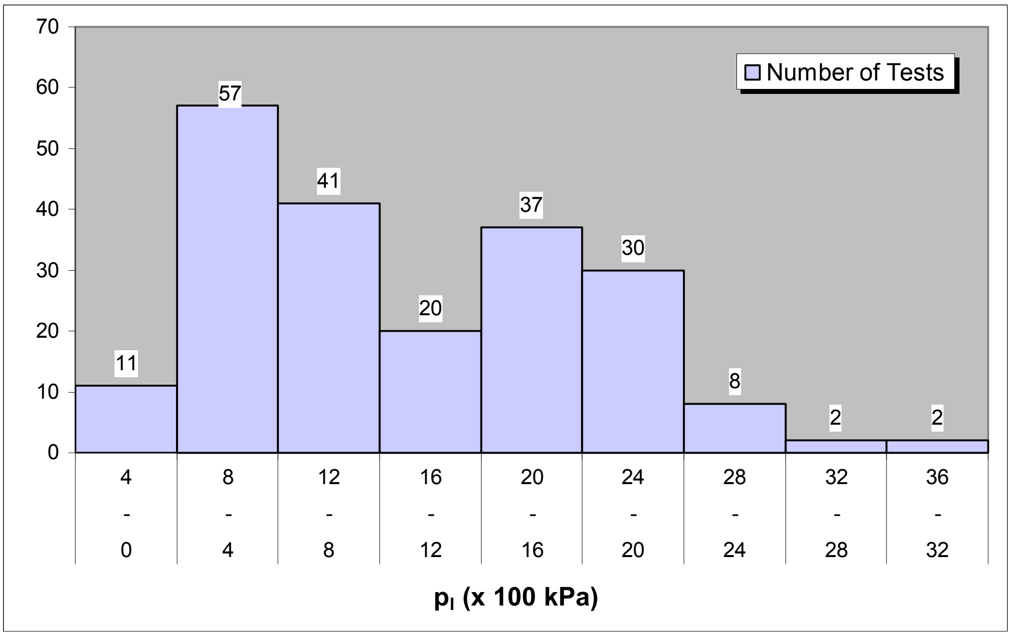


Figure 3.2 Limit Pressure, p_l distribution in database

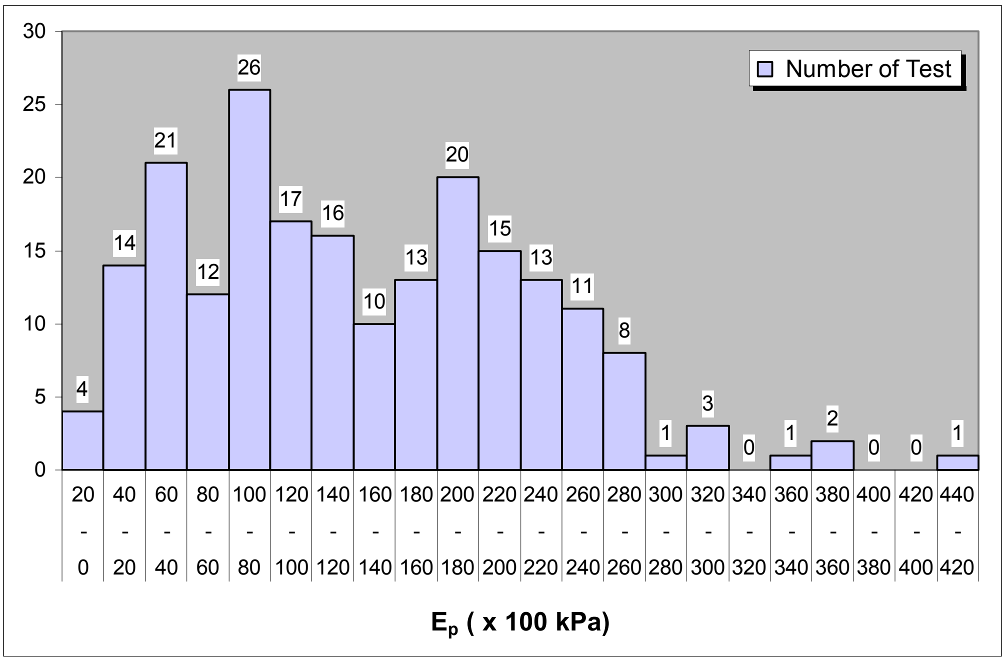


Figure 3.3 Pressuremeter Modulus, E_p distribution in database

The database study has been mostly focused on clay soils of CH and CL type according to Unified Soil Classification System (USCS). In addition, some clay-sand mixtures of SC class soils with cohesion were encountered and they were also included in database since there were both laboratory and in-situ tests performed on them.

The range of laboratory and in-situ test results for general database is tabulated in Table 3.1 in order to give a rough idea. Details for each soil type encountered during the study are discussed in proceeding sections.

Table 3.1 General ranges of properties in database

Soil Class (USCS)	CH – CL, rarely SC
Plasticity Index, PI (%)	12 - 91
Undrained Shear Strength, c_u (kPa)	12 - 384
Volume Compressibility, m_v (m^2/kN)	0.26 x10^{-4} - 5.09 x10^{-4}
SPT N (blow counts / ft)	3 - 95
Limit Pressure, p_l (x100 kPa)	1.94 - 33.50
Pressuremeter Modulus, E_p (x100 kPa)	13 - 430

The general ranges of properties are quite wide including from soft to hard clay type of materials in consistency.

3.2 Ankara Rail Transit System / Stage 3 Works

The project, Ankara Rail Transit System / Stage 3 Works has been executed as an extension of the existing subway line of Ankara, starting from the current final station in Batıkent and following the route towards west to north-west side of the city, ending in Sincan region (Figure 3.4).

Figure 3.4 Ankara Rail Transit System / Stage 3 Works Construction Part Passing Away from Batıkent

The total length of the route is about 15.5 km including different types of construction methodologies such as viaducts, stations, cuts, cut & cover tunnels, level crossing fills and cuts, tunnels to be drilled etc. More than 150 borings have been performed covering all the route, as both preliminary and final site investigation works with drilling rigs of 1 truck-mounted Mobile Drill B-53, 2 truck-mounted Acker Teredo Mark II and 1 skid-mounted Craelius XH-90.

Undisturbed samples were obtained whenever possible by using thin-walled steel (shelby) tubes and the ones with insufficient recovery due to the soil conditions were bagged as disturbed samples. Also, samples taken from SPT tests are used as disturbed samples in order to be tested in laboratory. In addition to SPT tests, pressuremeter tests were performed at every 3.0 m depths with Menard GA type equipment.

The geological formations observed along the construction line could be usually described in two main groups where a few more types also exist which are out of the scope of this study:

* Alluviums (Ankara Alluviums)
* Gölbaşı Formation (Ankara Clay)

3.2.1 Ankara Alluviums

The alluvium materials that are to be named as “Ankara Alluviums” during this study are made up of generally fine-grained (clayey, silty) units. They are usually observed in the form of brown to grayish-brown colored silty clay containing thin lenses of partially sand and so rarely gravel.

Ankara Alluviums are of generally Ouaternary aged in geological time and at some local zones early Pliocene aged followed by alteration. These ones -deposited just after the Pliocene- are usually stiffer in consistency, relatively preloaded & altered when compared to general alluvial deposits of Ankara.

Ankara Alluviums were deposited as a result of generally short distance transportation of the deeper units (such as gölbaşı formation, hançili formation, etc.) which takes place nearby vicinity in the environments of lake or stream. Beside, some deep deposits of thicker than 25 m are encountered that are made up of silt and clay deposits at the environment of lake-marsh (steady water).

The topography where alluvial deposits were obtained are in general smooth and alluviums are located in a floor of a valley that is formed by deeper and stiffer formations. So, materials being dragged along the sides of valleys also constitute the alluviums. The alluviums get younger and daily in age term as topography rises through the side slopes.

Groundwater was encountered along the alluviual deposits, especially where they are located in a valley floor.

The distributions of in-situ test results (SPT and PMT) in terms of number of observations are given in Figures 3.5, 3.6 & 3.7:

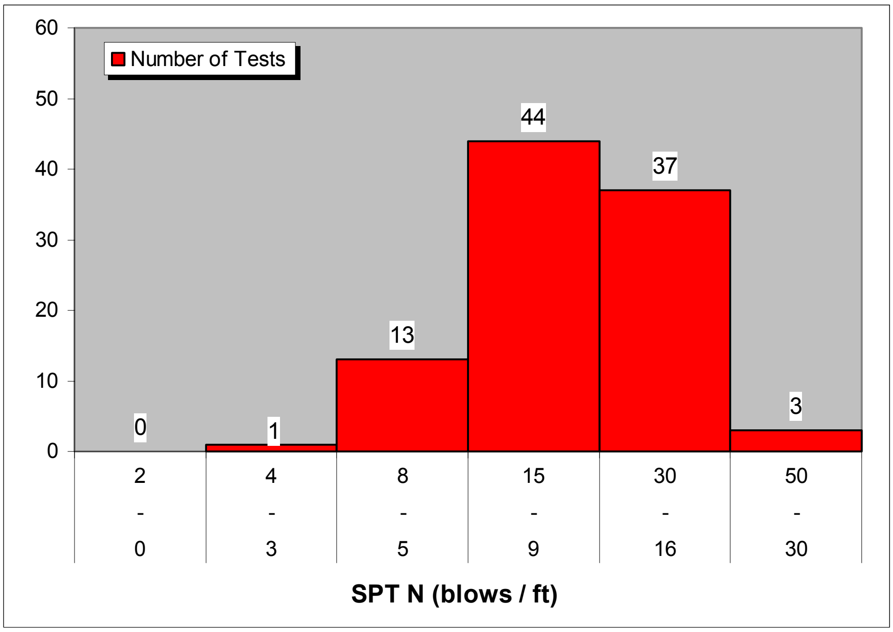


Figure 3.5 SPT N number distribution in Ankara Alluviums

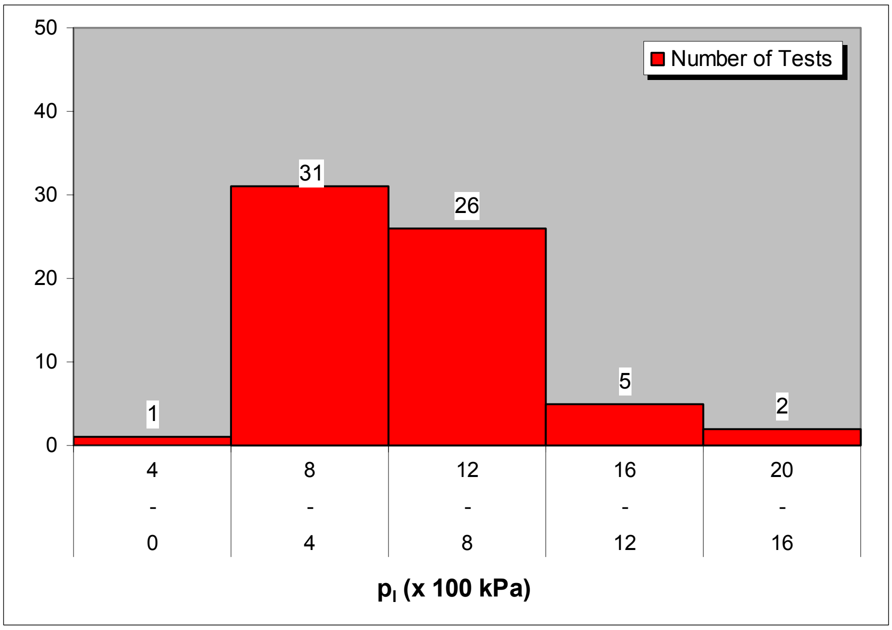


Figure 3.6 Limit Pressure, p_l distribution in Ankara Alluviums

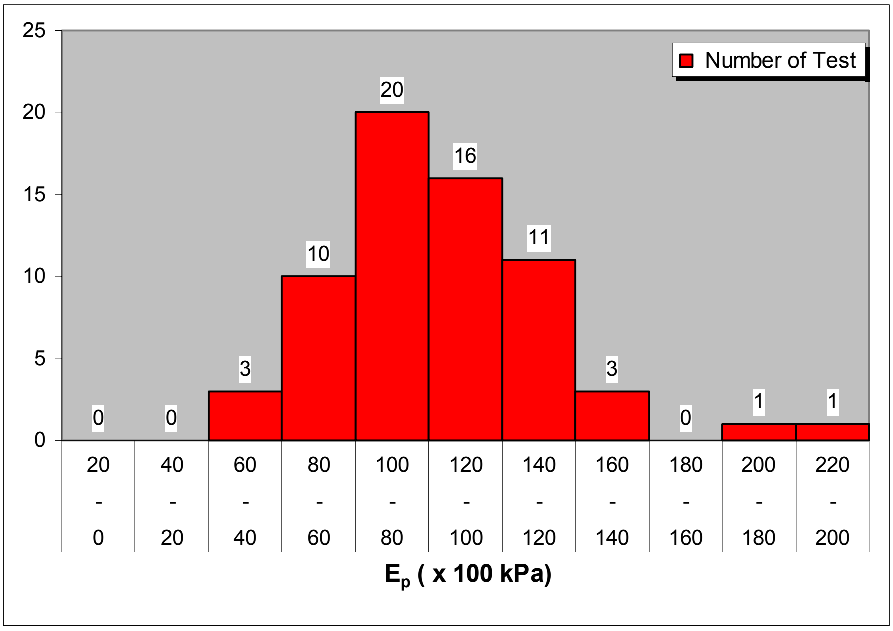


Figure 3.7 Pressuremeter Modulus, E_p distribution in Ankara Alluviums

Table 3.2 General ranges of properties for Ankara Alluviums

Soil Class (USCS)	CH – CL
Plasticity Index, PI (%)	19.6 – 87.2
Undrained Shear Strength, c_u (kPa)	21.6 – 170.0
Volume Compressibility, m_v (m^2/kN)	0.99 x10^{-4} - 5.09 x10^{-4}
SPT N (blow counts / ft)	4 - 35
Limit Pressure, p_l (x100 kPa)	3.30 - 19.20
Pressuremeter Modulus, E_p (x100 kPa)	49 - 212

According to the USCS, the materials refined from Ankara Alluviums for the purpose of database preparation are mostly of CH and CL type soils (Table 3.2). A few spontaneous SC samples resulting from the thin lenses of granular materials are ignorable for the study.

Plasticity of Ankara Alluviums is highly variable. Data obtained has PI values of commonly between 20–70 (%). The ones with low plasticty (PI<35) usually belong to CL class silty-sandy clays with relatively lower fine content, or high silt amount as fines. Some samples of PI>70% clays with CH class are also encountered together with potential of swelling.

Relating to SPT N number obtained in Ankara Alluviums, general consistency seems to be from medium stiff to very stiff (N=5-30) where few exceptions are available in lower and upper ranges.

General accumulations for pressuremeter results, p_l and E_p, are 4-16 (x100 kPa) and 40 -160 (x100 kPa), respectively.

3.2.2 Ankara Clay

The formation, which covers approximately 9 km of the project area at surface level and overlained by alluviums for most of the rest part comprises of Pliocene-aged units which are generally named as Ankara Clay.

Ankara Clay is made up of fine particles having the color of light brown to brown and rarely green to gray, with calcereous concretions and/or marl interlayers, partly lenses of gravelly sand and clayey sand (Figure 3.8, 3.9).

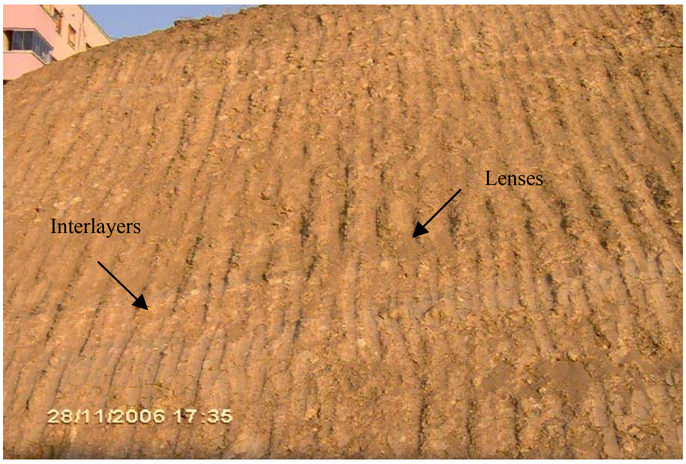


Figure 3.8 Ankara Clay in a cut slope (Batıkent)

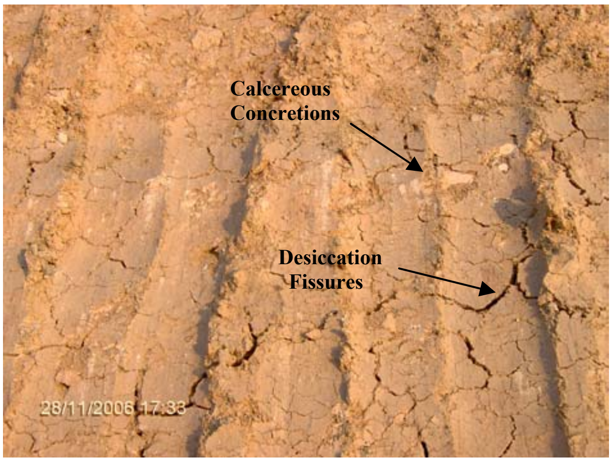


Figure 3.9 Calcereous concretions and desiccation fissures in Ankara Clay

It is also observed that the formation is fissured (Figure 3.9) and contains groundwater on the levels of sand lenses. Clays of the formation are occasionally in unconsolidated rock appearance in deeper parts.

Sands, on the other hand, are in thin layers form, uncemented and very dense. The layers of sand, clayey sand and marl have no extensions in lateral or horizontal whereas transitions within the unit are to be obtained.

Ankara Clay (Gölbaşı Formation) has been deposited in a river or lake basin (Akyürek et. al, 1996).

The distributions of in-situ test results (SPT and PMT) in terms of number of observations are given in Figures 3.10, 3.11 & 3.12:

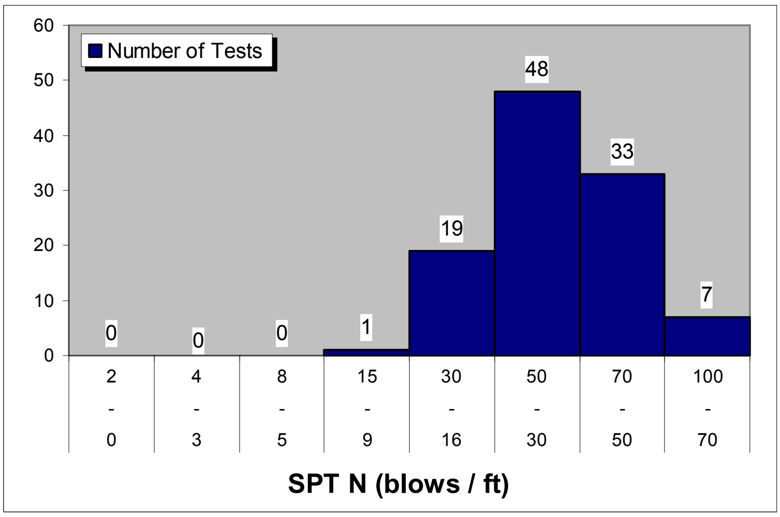


Figure 3.10 SPT N number distribution in Ankara Clay

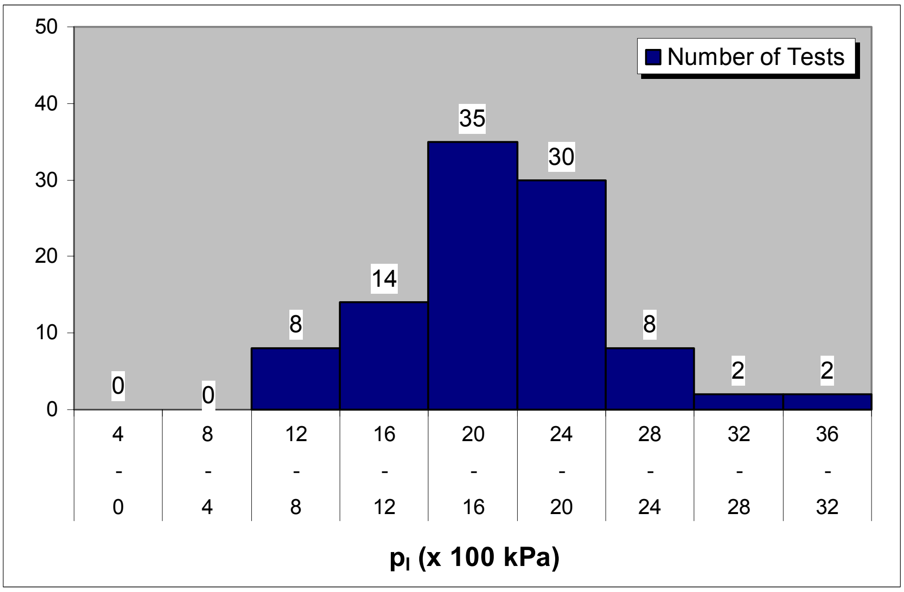


Figure 3.11 Limit Pressure, p_l distribution in Ankara Clay

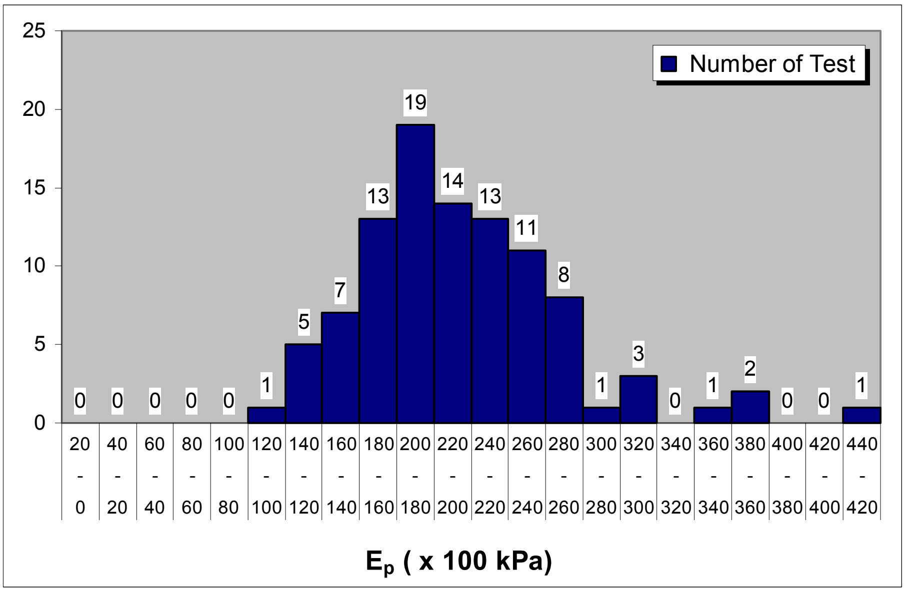


Figure 3.12 Pressuremeter Modulus, E_p distribution in Ankara Clay

Table 3.3 General ranges of properties for Ankara Clay

Soil Class (USCS)	CH
Plasticity Index, PI (%)	28.7 – 91.3
Undrained Shear Strength, c_u (kPa)	59.2 – 383.9
Volume Compressibility, m_v (m^2/kN)	0.26 $\times 10^{-4}$ – 3.311 $\times 10^{-4}$
SPT N (blow counts / ft)	14 - 95
Limit Pressure, p_l (x100 kPa)	8.80 - 33.50
Pressuremeter Modulus, E_p (x100 kPa)	111 - 430

According to the USCS, the clay soil that of named as Ankara Clay is completely CH type soil in class (Table 3.3).

Ankara Clay could be described as a cohesive material with high plasticity. Most of the atterberg limit tests indicate plasticity index of PI = 40 – 80 (%) which is a high range. The plasticity mostly occurs from the mineral content and grain size distribution where nearly all the soil particles are finer than #200 sieve in ASTM standards. Ankara clay has a swelling potential that is to be observed in oedometer tests.

Standard penetration tests performed on Ankara Clay have given a general range of N = 20-70 where values of N = 50+ is often encountered in deeper parts that are indications of a bedrock formed by claystone or mudstone. Consistency for Ankara Clay is then defined as to be very stiff & hard.

Pressuremeter tests in Ankara Clay resulted in weighted ranges of parameters, limit pressure p_l = 10-30 (x100 kPa) and pressuremeter modulus E_p = 150 -300 (x100 kPa).

3.3 İzmir Bostanlı Shoreline / Atakent Project

Atakent Project mainly aimed the improvement and recreation of the field that was in a characteristics of an old river bed and delta along Bostanlı Shoreline nearby İzmir Bay (Figure 3.13).

Figure 3.13 A view from Delta of Gediz River, İzmir

The project consisted of constructions of 2844 residences. Topographical elevations of project area with respect to sea level were changing about (+) 0.60 – 1.00 m which was nearly sea level.

The construction region in general was the old delta of Gediz River and it was completely formed by an alluvial young fill of Quaternary aged that was composed of various types of materials changing from silty clays with organic content to sand and gravel that of all were transported by the river Gediz and deposited. Gediz river had followed the natural slope orientation while streaming in geological time. Displacements of river bed, seperation of main stream line at the exit point to a very calm sea of Izmir Bay and some geological movements (especially lowering in elevation) had resulted into the formation of a very wide and deep alluvial fill deposit around the region.

The number of borings drilled in the project is 37 with varying depths of 35.0 to 50.0 m. Drilling rigs of 3 skid-mounted Craelius XH-90 were used for site investigation purposes.

Casings of HW and NW types were used all along the borings since deep and soft deposits threatened the stability of boroholes. Disturbed and undisturbed samples were obtained as much as possible.

SPT tests with a standard split-spoon sampler and pressuremeter tests with Menard GC type probe had been performed.

For the purpose of this study, the geological formations observed along the construction site could be described in a single title that of called as:

* Gediz Alluviums (İzmir Alluviums)

İzmir alluviums involve also granular materials of sand and gravels but they are not included in evaluations and the title is used only for cohesive soils during the study.

3.3.1 İzmir Alluviums

The cohesive materials of İzmir Alluviums are made up of generally clayey materials with considerable amount of silt and sand content (Figure 3.14). An average of 22.0 m thickness from the ground surface is constituted by gray to dark gray colored silty clay – clayey silt with little organic content and very thin sand lenses (intensly layering). These deposits are soft in consistency. Between 22.0 – 27.0 m depths, they become medium stiff with increasing sand content.

The cohesive materials under 27.0 m depth are usually formed by light brown to brown colored stiff to very stiff silty clay. In deeper parts, silty clay turns out to be sandy or gravelly clay that causes an increase in stiffnes through hard class.

Figure 3.14 Gray colored silty clay deposits of İzmir Alluviums extracted during a bored-pile excavation

Groundwater levels encountered along the alluviual deposits are often in first a few meters from the surface and sometimes closer than a meter.

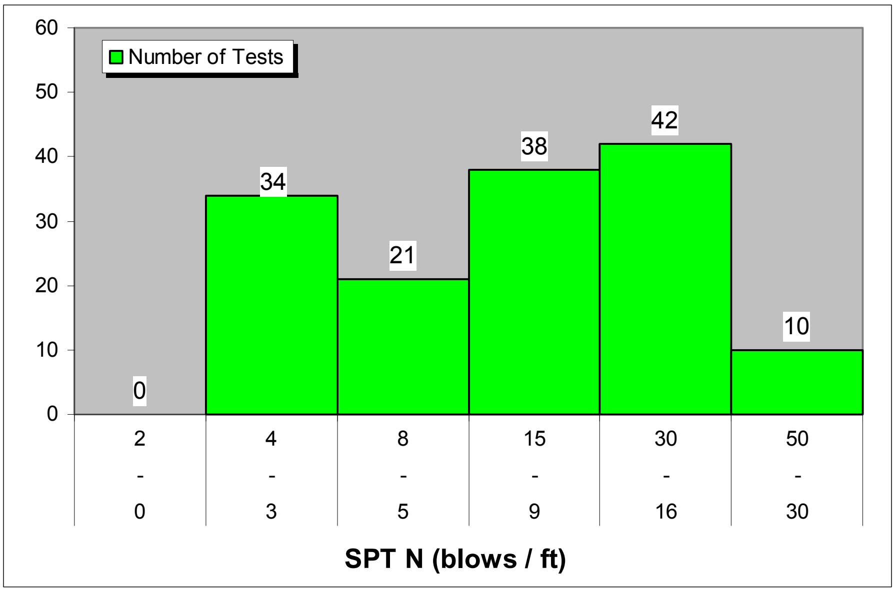


Figure 3.15 SPT N number distribution in İzmir Alluviums

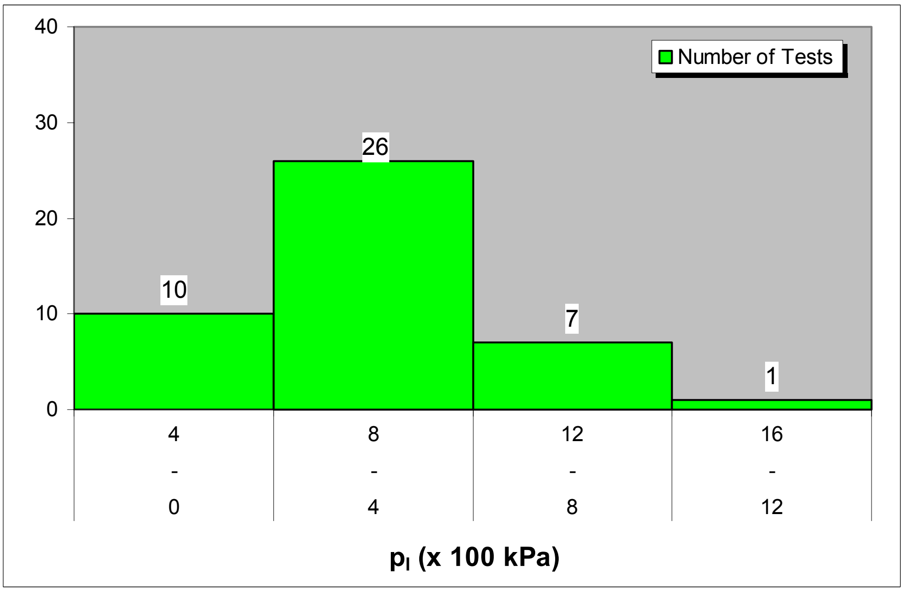


Figure 3.16 Limit Pressure, p_l distribution in İzmir Alluviums

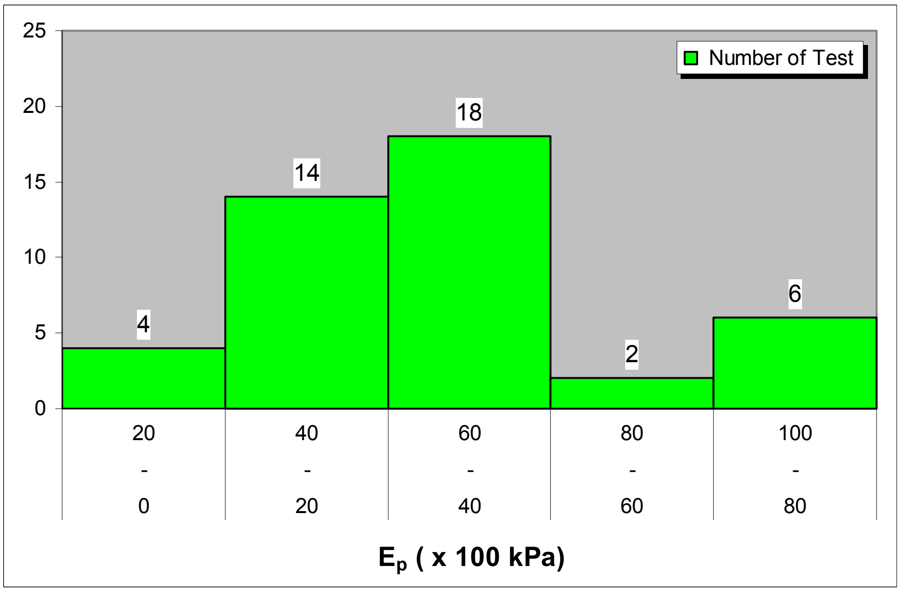


Figure 3.17 Pressuremeter Modulus, E_p distribution in İzmir Alluviums

The distributions of in-situ test results (SPT and PMT) in terms of number of observations are given in Figures 3.15, 3.16 & 3.17 above.

According to the USCS, the alluvial deposits of İzmir have a variety in classification since an active formation process of deposits is followed due to the river and sea environment. The deposits of the upper 27.0 m, gray to dark gray colored silty clay/clayey silt are in CH class with plastic silt and clay fines content, beside CL class samples are obtained with silty and sandy clays of low to medium plasticity. Furthermore, some materials of ML class are encountered, that confirms the content of silts and very fine clayey sands or clayey silts with slight plasticity (Table 3.4).

The cohesive materials that of are light brown to brown colored clays below 27.0 m are mostly of CL and SC class. CL materials refer to silty-sandy and gravelly clays of low to medium plasticity, whereas SC soils indicates the increase in sand content inside the clay medium with sand-clay mixtures or clayey sands with a slight plasticity. Very rarely some CH class of samples are obtained.

Table 3.4 General ranges of properties for İzmir Alluviums

Soil Class (USCS)	CH - CL – SC (ML)
Plasticity Index, PI (%)	12.1 – 63.2
Undrained Shear Strength, c_u (kPa)	11.8 – 190.5
Volume Compressibility, m_v (m^2/kN)	0.48 $x10^{-4}$ – 2.13 $x10^{-4}$
SPT N (blow counts / ft)	3 - 44
Limit Pressure, p_l (x100 kPa)	1.94 - 13.00
Pressuremeter Modulus, E_p (x100 kPa)	13 - 94

The plasticity of İzmir alluviums could be defined as dependent to composition characteristics. In the range of PI > 35, nearly all the samples are of CH class, rest is the materials with low to medium plasticity as CL class for PI < 35 and additionally SC classes usually for PI < 25.

SPT N numbers obtained in 22.0 m deep soft deposit is between N = 3-5, and between 22.0 – 27.0 m medium stiff consistency is common with SPT N = 5-8 for the same gray clayey deposit. For deeper deposits of brown colored clay, a wide range of

SPT N = 9 – 44 has been obtained with increasing stiffness in deeper parts, depending on sand and gravel sized particle content.

General values of pressuremeter results, p_l and E_p, are between 3-10 (x100 kPa) and 20-90 (x100 kPa), respectively.

CHAPTER 4

DISCUSSION ON PARAMETERS PREDICTED FROM IN-SITU TESTS FOR THE EXPERIENCED SOILS

This section deals with the reduction of data obtained from the soil investigations in terms of both laboratory & in-situ test results; methodology of data analysis, and the predictions of properties of soils obtained in projects described in Chapter 3. Parameter estimations have been based on emprical correlations and existing approaches in literature together with site-specific observations and evaluations. Two important in-situ tests, Standard Penetration Test (SPT) and Pressuremeter Test (PMT) results have been studied to predict the undrained shear strength (c_u), volume compressibility coefficient (m_v) and deformation modulus (E) of soils those obtained from the laboratory tests.

4.1 Data Reduction

Data reduction is certainly the fundemantal process in geotechnical engineering analysis and evaluation. As a rough definition, it is the “art” of refining the results from any engineering investigation so that the data could be transformed to a reasonable, reliable and consistent form in order to be used for engineering analysis and design purposes. Methods of emprical, semi-emprical, analytical or numerical analysis are never much more reliable than the reliability of data adopted for them. Then, an engineer should always give great effort to reduce his data to get a more reliable set of that.

In the scope of this study, the results of in-situ and laboratory tests that were performed on soils experienced in previously mentioned projects has been reduced and currently derived. Then, the database has been prepared from the new refinement and derivation. Database is used in parameter predictions from in-situ tests of SPT and PMT.

4.1.1 Data Reduction for In-Situ Tests

Data reduction for in-situ tests should be discussed seperately for SPT and PMT because they have many different perspectives such as purposes, procedures, measuring principles and interpretations:

Standard Penetration Test, SPT, data directly consists of an index number N, that is the number of blow counts required for last 30 cm penetration of totally 45 cm. An important reduction applied in this study is the clear identification of N number, means examining whether

- the number N is reasonable with no gross error by taking in to account the general conditions of soil within which test is performed.

- the number N is consistent with other results obtained from different depths in the same soil strata or not.

- the number N is also consistent with the accepted range of N in common literature for similar soils

- the laboratory tests roughly confirm the number N with an expected amount of consistency

In case where these conditions were destroyed seriously, the number N has been ignored and not included in evaluations.

Pressuremeter Test, PMT data obtained from the field is already a raw data set and the results of tests are presented in a corrected form -as corrected pressure-volume curves- in studied projects. Calibrations are specific to the equipment used in each project study. However, some modifications have been required on corrected curves. For example, the curves were smoothened much more, especially around the points of p_i-v_i and p_f-v_f in order to obtain them more accurately. An accurate prediction of p_i value, for example, provides a more reliable in-situ total horizontal stress, p_o, which is too difficult to calculate by ordinary methods. Furthermore, derivation of these points also modified the E_p values. Estimation of p_l value on curves, usually when the test could not be fully completed due to the high resistance of hard soils which require great limits of relevant pressuremeter probes and equipments, have been examined carefully and necessary modifications were applied in order to obtain a smooth curve that is close to an ideal shape. A check is performed on the calculation methodology of E_p, and it is determined that the presented E_p values directly correspond to E_M, with the assumption of υ=0.33 as Menard (1967) proposed.

The consistency and being reasonable conditions for the test results have also been examined for PMT, as it is explained above for SPT.

4.1.2 Data Reduction for Laboratory Tests

Laboratory tests have all been reduced according to the consistency and reasonability statements given in Section 4.1.1. UU triaxial loading tests and oedometer tests were studied additionally for some other reduction processes:

An unconsolidated-undrained triaxial loading test in saturated cohesive soils normally models the $\phi_u = 0°$ condition, which means that all the deviatoric stress (σ_d) is compensated by an increase in pore pressure u and no change in the diameter of Mohr's circles. Hovewer, in some test data of this study, examples of $\phi_u > 0°$ cases are observed(ϕ_u = 0 - 8°). The c_u values obtained from these tests have come out so low that they were not reasonable and no consistency was to be provided as discussed in Section 4.1.1. Then, undrained shear strength of these samples were obtained by considering that each Mohr's circle in fact represents a slightly different c_u value -where normally they were all to have an equal diameter (deviatoric stress, σ_d)- and taking the average of these different values of σ_d

might be an error minimizing methodology. The half of average σ_d was then taken as c_u for $\phi_u > 0°$ tests.

The coefficient of volume compressibility of soils in laboratory results presentations have been given for different pressure ranges, since m_v is a measure of relative change in void ratio Δe and effective stress $\Delta\sigma'$. The m_v value given for a certain range (for example $\sigma_o' = 50$ to $\sigma_1' = 100$ kPa) in fact represents the slope of the line from e_o-σ_o' to e_1 vs. σ_1' on the curve of e-σ'. Then, that m_v value has been referred to the slope of tangent line to the curve at the mid-point (average) of a range (say 75 kPa for 50 to 100 kPa). Now, each m_v value has been taken as representative of its mid-point of the range that it belongs. Rest is only interpolation of varying slopes on curve according to what the effective stress at sample depths were to be calculated.

4.2 Method of Data Analysis

One of the most critical point for a geotechnical analysis and evaluation should probably be the characterization of soils by appointing the results of in-situ and laboratory tests to soil samples or layers so that they are defined and qualified by those properties after the reduction of test data. When a borehole log is presented to a designer, simply attaching the results at relevant sample or test depths on log might be misleading. Instead, the consistency and reasonability should certainly be investigated over the soil profile, so that the characterization of soil strata could be reliable.

The database formed by the obtained and reduced in-situ & laboratory test results in this study has been examined also for general consistency and reasonability all over the soil profiles. For the purpose, two different soil characterization methodologies were studied:

1) Point data analysis
2) Layer idealization

For both methods, the initial database which consists of 351 data line has been used, whereas for the latter one this number was decreased to about 217 lines. The principals of methodologies could be summarized as below:

1) Point data analysis

In this methodology, the reduced in-situ and laboratory test results have been used as a point rather than characterizing the soil as an idealized layer. The obtained results are related to the other test results which were closest in soil profile, means for example, the results of a pressuremeter test performed at a depth of 2.70 m, a UU triaxial loading test executed on an undisturbed sample obtained from 3.00 m depth and a SPT test corresponding to 3.50 – 3.95 m depth interval have all been recorded together in the same data line and related to each other since they are assumed to be belonged to same soil.

2) Layer idealization

The method of layer idealization in this study mainly followed the principals of a geotechnical design process. The soil profiles were all arranged in a form that a profile has been divided into soil layers of having the most similar engineering characteristics using all the field examinations, local geological definitions, laboratory and in-situ test results etc. Data reduction process is doubled here because of that the consistency and reasonability have been checked one more time for the whole of a borehole log.

Both methodologies have been applied together for all correlation studies included in this study and it has been clearly observed that point data analysis always results in a larger scattering for any type of emprical correlation, when compared to layer idealization methodology.

An example for a comparison is given in Figures 4.1 & 4.2. The common description of Stroud (1974) relating the standard penetration number N vs. undrained shear strength c_u with a factor f_1 that depends on plasticity was discussed in Section 2.3.3.4 given by the Equation 2.4:

$$c_u = f_1 \times N$$

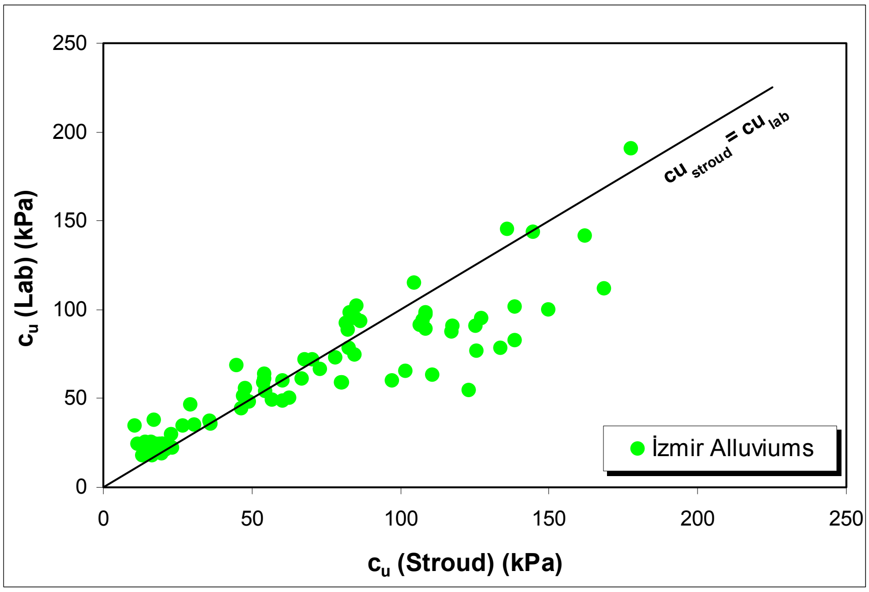


Figure 4.1 Comparison of $c_{u(Lab)}$ & $c_{u(Stroud)}$ by layer idealization methodology

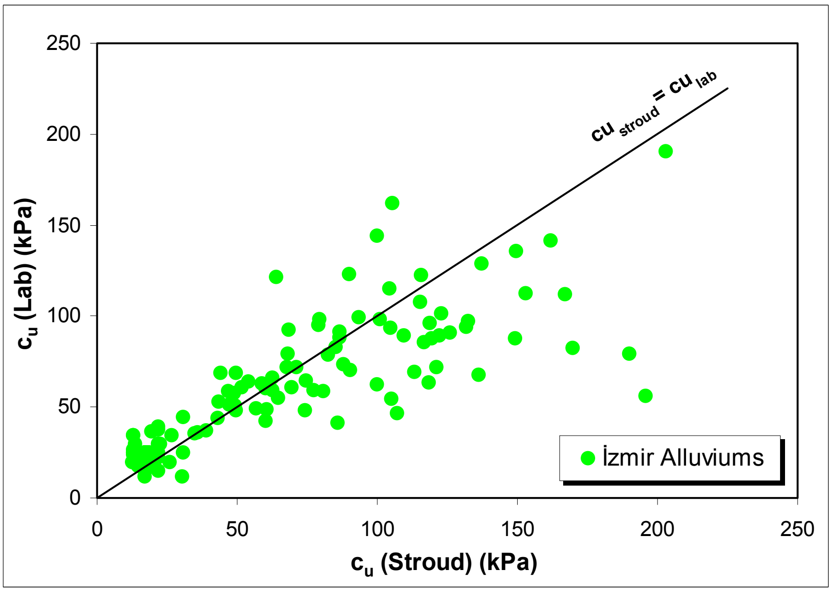


Figure 4.2 Comparison of $c_{u(Lab)}$ & $c_{u(Stroud)}$ by point data analysis

The estimations of c_u from number N as proposed by Stroud (1974) have been compared to the c_u values obtained from laboratory tests for İzmir Alluviums and the difference of scatterings for two methods in dimension were easily observed.

The reason for large scattering in point data analysis would probably result from the principles of methodology, which consists of attachment of engineering properties and test results with closest distance to any of samples or depths. Especially in transition zones of soil layers, within a close vicinity, the results from tests of in-situ and/or laboratory are discussable whether they belong to the upper zone or lower one. Point analysis methodology may not be able to differentiate this whereas in layer idealization, each test result is attached to the relevant layer with an extensive search. Beside, despite of a data reduction process, any kind of possible deviation in test results could be minimized in layer idealization method since an average characterizing is the matter, whereas a deviation in point data could not be refined once, and to be used directly.

The method of data analysis used during this study is “layer idealization” in order to get more accurate and reliable results.

4.3 Evaluation of SPT Results for Parameter Estimation

The use of Standard Penetration Test results to estimate undrained shear strength (c_u) and coefficient of volume compressibility (m_v) characteristics of cohesive materials is proceeded mainly on Equations 2.4 and 2.5 proposed by Stroud (1974). These wide-spread and commonly used predicitions of Stroud (1974) obtaining c_u and m_v from SPT N number are discussed seperately for three main soil types that were defined in Chapter 3; Ankara Alluviums, İzmir Alluviums and Ankara Clay.

The factors f_1 and f_2 are designated from the Figures 2.9 and 2.10, respectively, by means of plasticity index values of experienced samples obtained in laboratory tests. Then, using SPT N numbers, c_u values (from Equation 2.4) and m_v values (from Equation 2.5) are calculated as proposed by Stroud. These values are plotted against the laboratory obtained results of c_u and m_v for the same soils in order to make a comparison.

The values of undrained strength c_u and volume compressibility m_v obtained from Stroud (1974) is named as $c_{u(Stroud)}$ and $m_{v(Stroud)}$ whereas results from laboratory are mentioned as $c_{u(Lab)}$ and $m_{v(Lab)}$, respectively in the discussions.

4.3.1 Ankara Alluviums

The comparison of $c_{u(Stroud)}$ and $c_{u(Lab)}$ values in Ankara alluviums is plotted in Figure 4.3. The plot shows that despite of an amount of scattering, the general tendency of c_u values are along the line of equality, $c_{u(Stroud)} = c_{u(Lab)}$ which means that Stroud (1974) could somehow predict c_u values of cohesive materials. Numerical identification of scattering is defined in Figure 4.4.

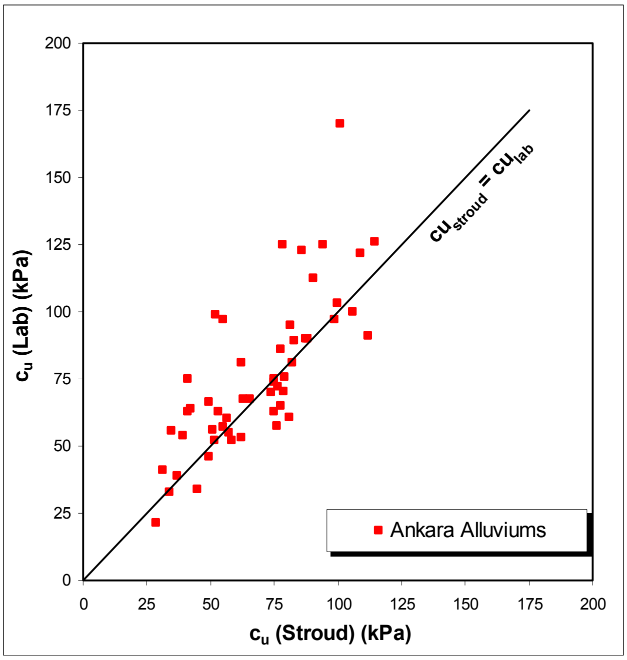


Figure 4.3 $c_{u(Stroud)}$ vs $c_{u(Lab)}$ in Ankara Alluviums

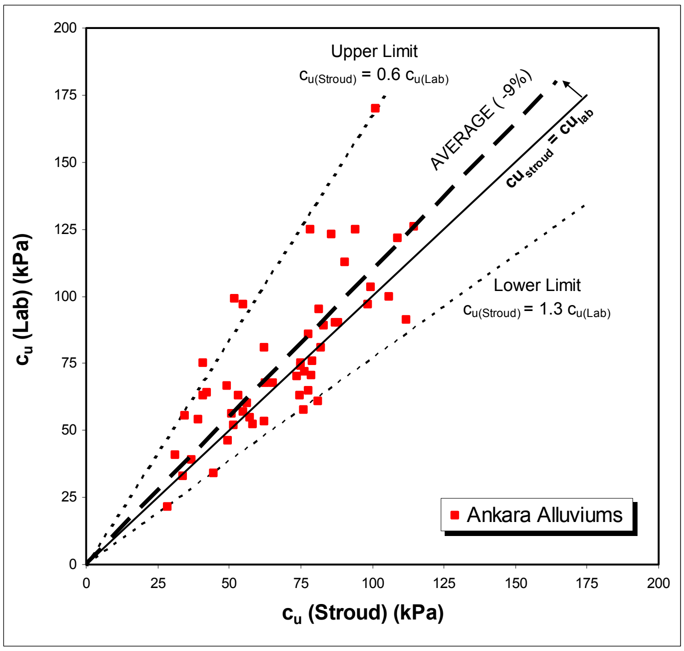


Figure 4.4 Accuracy evaluation of $c_{u(Stroud)}$ in Ankara Alluviums

A discussion point here is the upper and lower proximity observed for the plotted data. For the upper range of c_u values it could be stated that Stroud (1974) may underestimate measured $c_{u(Lab)}$ values by a max. amount of -40%, or another possibility is that laboratory evaluation and designation of c_u values with data reduction in cases where $\phi_u > 0°$ might be leading to overestimated $c_{u(Lab)}$ values of that are up to by +67% over the predictions of $c_{u(Stroud)}$ values. The lower range is more compact with an overestimation by Stroud (1974) at about 30% over $c_{u(Lab)}$ values, which may arise from possible uncomfortable conditions within sampling and laboratory testing.

If a best line is fitted throughout the whole plotted data, an average line of -9 % is obtained, which is an indication that in generally speaking for Ankara Alluviums, Stroud (1974) may be a useful tool to predict the undrained shear strength of cohesive soils with an underestimation of just by -9%, which could be commended as acceptable in a way.

Another study on c_u has been performed by a back calculation of f_1 values. The relationship between c_u and N number is examined in such a way that the $c_{u(Lab)}$ values are inserted into the Equation 2.4 together with SPT N numbers, than new f_1 factors (f_1') are derived. They are ordered according to PI values and compared with those of Stroud (Figure 4.5).

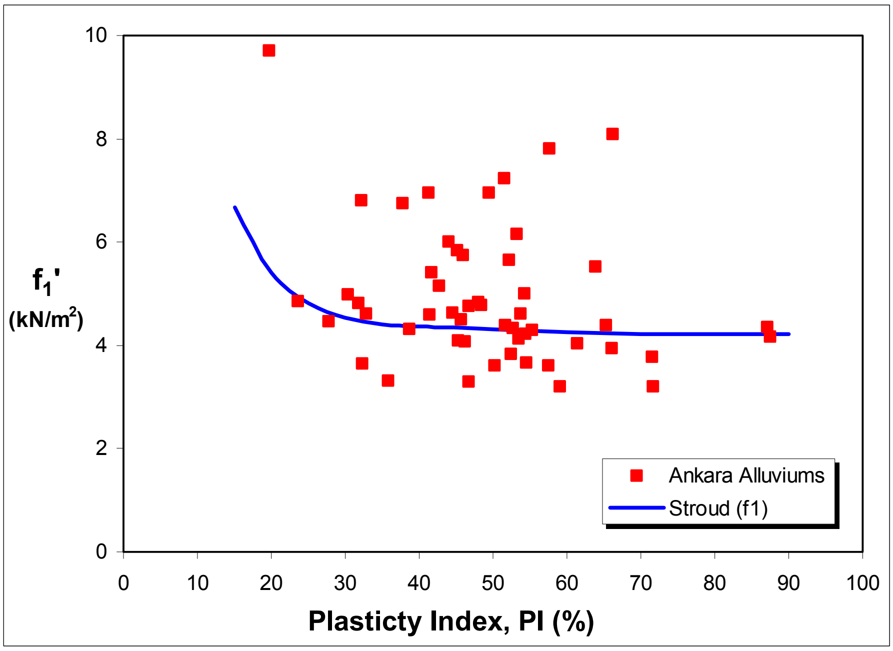


Figure 4.5 Comparison of derived factor f_1’ and $f_{1(Stroud)}$ in Ankara Alluviums

The currrently derived f_1’ factors are completly in agreement with the discussions above about c_u parameter, where a wide upper range is available together with a relatively small scatter for the lower values of $f_1' < f_{1(Stroud)}$.

The other comparison investigated is on $m_{v(Stroud)}$ and $m_{v(Lab)}$ values in Ankara alluviums. The values obtained are plotted in Figure 4.6. The plot clearly indicates that laboratory values of m_v are approximately all above the line of equality, that is the m_v values obtained from oedometer tests are much more higher than that of Stroud predicted. Accuracy of comparison is also examined in Figure 4.7.

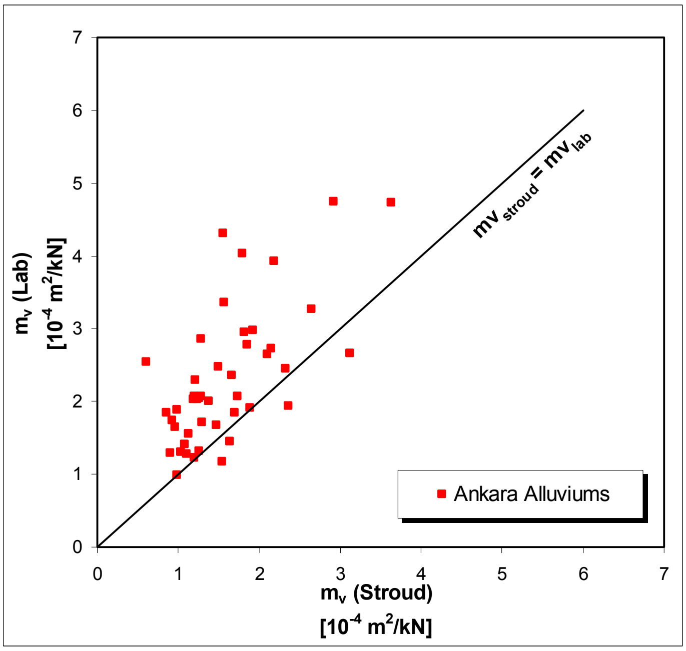


Figure 4.6 $m_{v(Stroud)}$ & $m_{v(Lab)}$ in Ankara Alluviums

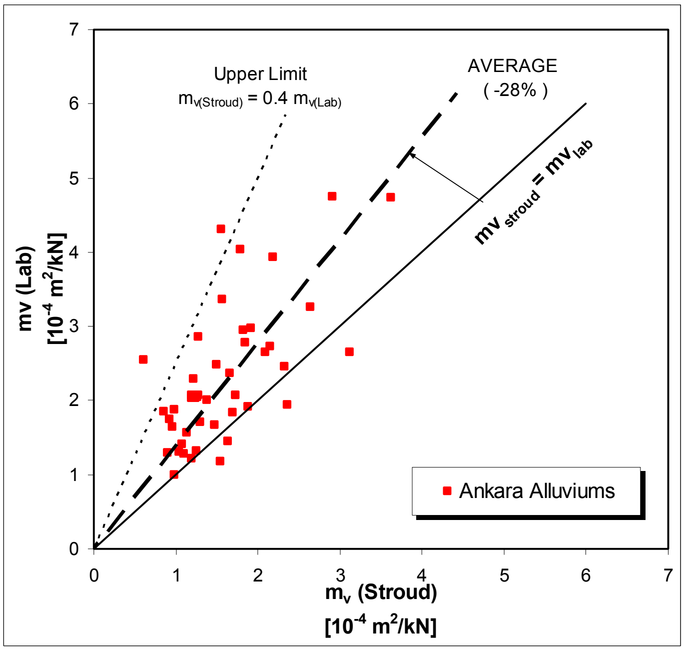


Figure 4.7 Accuracy evaluation of $m_{v(Stroud)}$ in Ankara Alluviums

The upper range of $m_{v(Stroud)}$ & $m_{v(Lab)}$ comparison in Ankara alluviums may rise up to an underestimation by Stroud on measured $m_{v(Lab)}$ values by a maximum amount of -60% , with an average of -28% in general. It is the lowest range of laboratory test results where they are nearly equal to those of estimated by Stroud. From a different point of view, laboratory (oedometer) tests are concluded to highly overestimate the coefficient of volume compressibility, at about 39% over $m_{v(Stroud)}$ values in average beside by a great upper limit of over 150% range when they are compared to the estimations of Stroud (1974).

The appearent laboratory overestimation may probably arised from combination of some major reasons:

The disturbance of specimens arising from sampling, handling, sample preparation, release of in-situ stress, etc. might explain the case. In addition, another possibility is the small size of laboratory specimens which may not comprise the macrofeatures of the soil mass (cementation, strong layering, etc.)

Moreover, differences in saturation conditions of in-situ soil layer and laboratory sample could also give rise to the difference between laboratory and predicted values of m_v. In an oedometer test, samples are formed in a fully saturated condition during the test, whereas the in-situ soil may not be fully saturated in case where groundwater does not exist or is not extensive which would lead to an in-situ predicted m_v value of unsaturated (partially saturated) and dessicated state of soil layers.

Similar to the factor f_1', f_2' is also derived by a back calculation from Equation 2.5 for the purpose of comparison with those of Stroud (Figure 4.8):

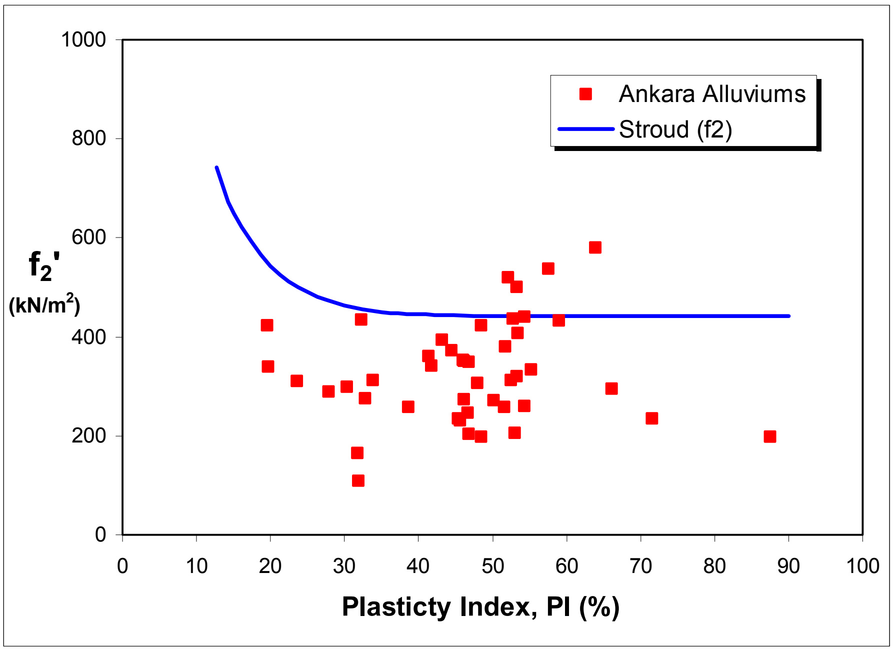


Figure 4.8 Comparison of derived factor f_2' and $f_{2(Stroud)}$ in Ankara Alluviums

Depending on the distribution of m_v values in Figure 4.6, high $m_{v(Lab)}$ values numerically result in low values of f_2' with a scatter located below the curve of Stroud (f2) where generally $f_2' < f_{2(Stroud)}$.

4.3.2 İzmir Alluviums

The undrained shear strength comparison of $c_{u(Stroud)}$ and $c_{u(Lab)}$ for İzmir alluviums is plotted in Figure 4.9.

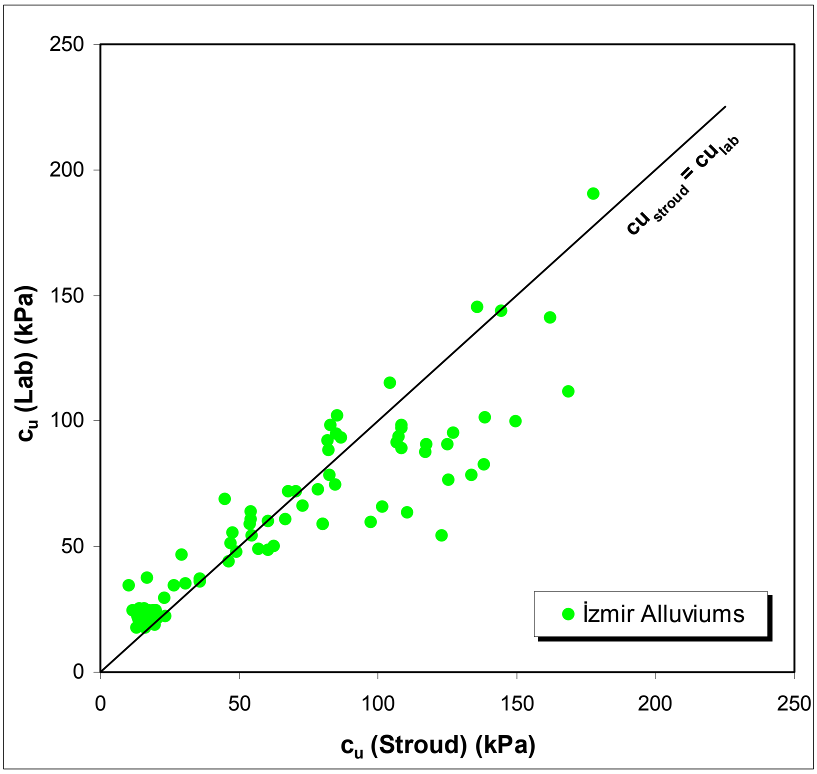


Figure 4.9 $c_{u(Stroud)}$ vs $c_{u(Lab)}$ in İzmir Alluviums

The plotted data shows that the reliability of Stroud's estimation in İzmir alluviums is partially variable depending on the range of c_u. For example, in the range where c_u<40 kPa, the accumulated data are generally over the line of equality, referring that $c_{u(Lab)}>c_{u(Stroud)}$. For the range $40<c_u<90$, laboratory and predicted values are in good agreement, whereas for c_u >90 kPa, laboratory tests underestimate the shear strength of soils.

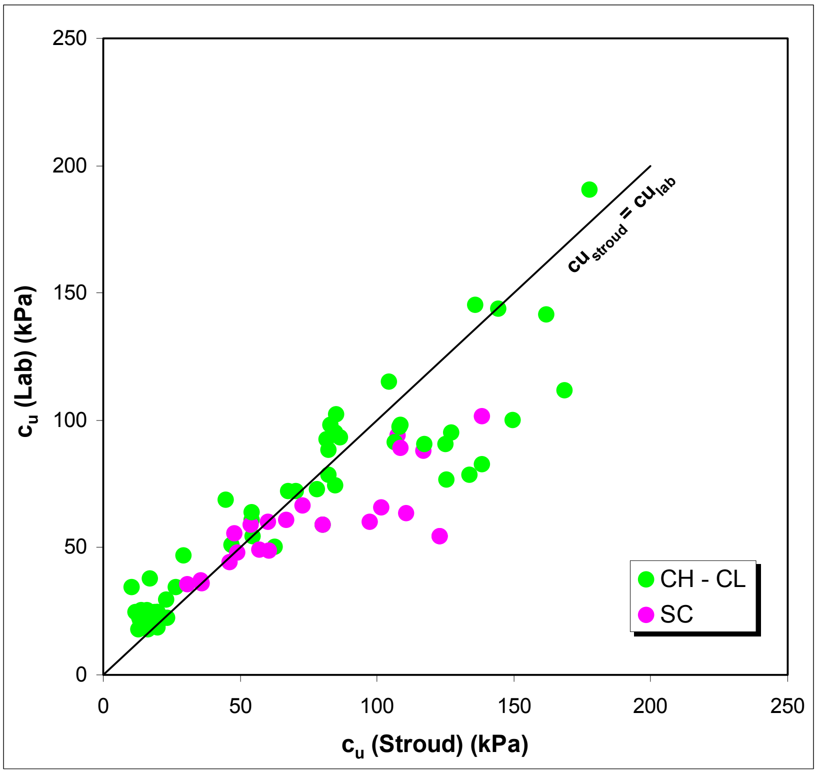


Figure 4.10 $c_{u(Stroud)}$ vs $c_{u(Lab)}$ in İzmir Alluviums for different soil classes

İzmir alluviums are composed of both silty-sandy-gravelly clays and sand-clay mixtures, furthermore clayey sands of low plasticity with cohesion as discussed in Chapter 3. For the purpose, Figure 4.9 could be modified as Figure 4.10 which presents the differentiation of SC class soil layers from others (CH-CL) along the comparison line of $c_{u(Stroud)}$ and $c_{u(Lab)}$. It is quite clear from the plot that clayey sands and sand-clay mixtures with cohesion are approximately in agreement with CH-CL soils following Stroud's prediction for the values of c_u<90 kPa. The range c_u<40 kPa is ignorable for SC soils because not enough data is available as SC class soils in that range probably because of combined sampling difficulties of low consistency (soft material) and lack of sufficient cohesion. Over 90 kPa, data from SC soils are scattered together with CH-CL type of data, but little bit more spreading and scattering at early stages of range for example between c_u = 90-100 kPa.

Due to the variability of data and prediction reliability, comparison of undrained shear strength c_u, proposed by Stroud (1974) and obtained from oedometer test results for İzmir alluviums is searched for different ranges those discussed above.

In the range c_u < 40 kPa, only a few SC soils are determined, nearly all of the soils are of CH-CL classes (Figure 4.11). The laboratory determined c_u values are likely to be higher than the ones that of Stroud's predictions and an average best line fitted over the whole data remarks that Stroud (1974) underestimates the measured $c_{u(Lab)}$ undrained shear strength by about -20% in average when low consistency of soft to medium stiff clays with c_u<40 kPa is the matter in İzmir Alluviums.

Figure 4.12 shows the comparison of c_u values together with SC class soil differentiation for the range of 40 < c_u < 90 (kPa).

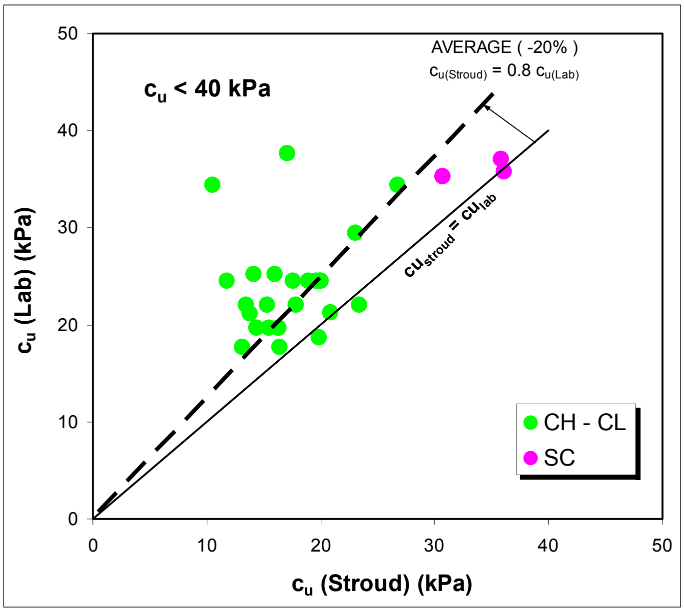


Figure 4.11 $c_{u(Stroud)}$ & $c_{u(Lab)}$ in İzmir Alluviums for c_u<40 kPa

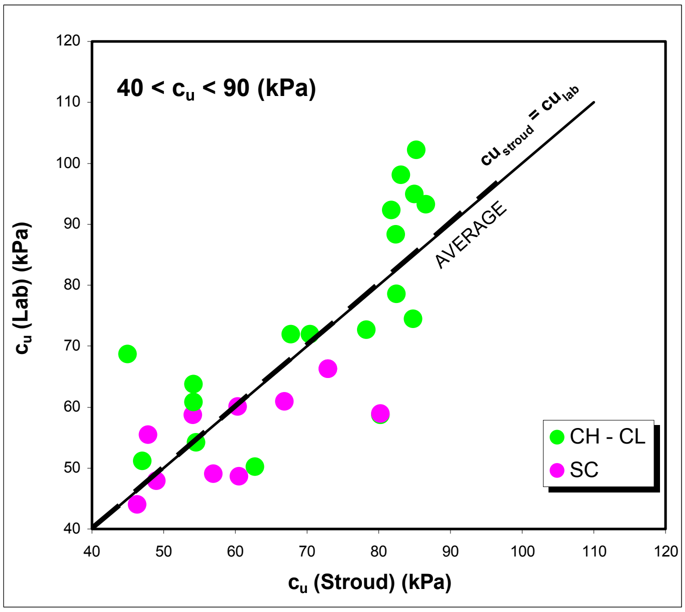


Figure 4.12 $c_{u(Stroud)}$ & $c_{u(Lab)}$ in İzmir Alluviums for 40<c_u<90 (kPa)

The range, $40<c_u<90$ (kPa) constitutes the part on comparison plot where most agreement is reached in average between $c_{u(Stroud)}$ and $c_{u(Lab)}$. CH-CL soils do follow the line of equality in a fitting way, means oedometer and predicted c_u values are generally close to each other with some scattering. Data from SC soils are also spreaded in relatively close vicinity of equality line, despite that general path they are located on seems somehow a bit under the line $c_{u(Stroud)}=c_{u(Lab)}$ as the c_u increase. This may be an indication of sample disturbance together with the effect of material composition with high sand content of SC soils. A best line for the whole of data is fitted and it could be seen on plot that as an average, best line exactly corresponds to line of equality indicating a high reliability of prediction in an average manner.

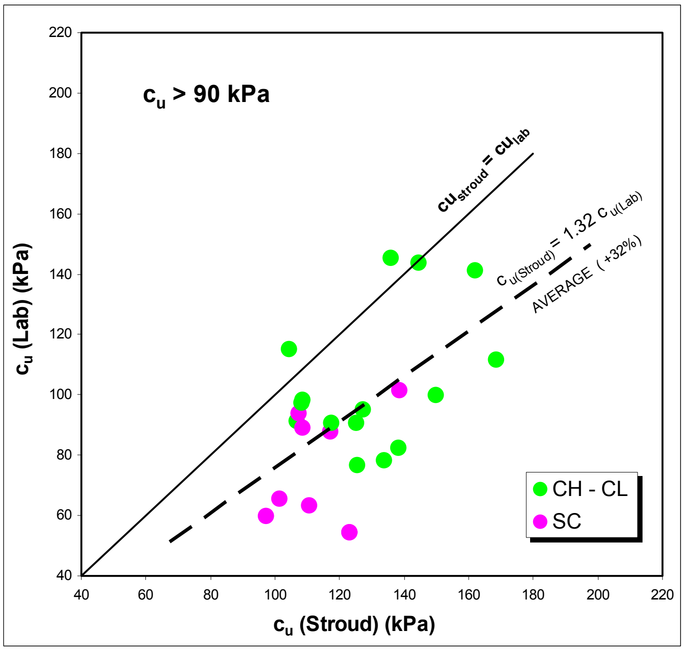


Figure 4.13 $c_{u(Stroud)}$ vs $c_{u(Lab)}$ in İzmir Alluviums for c_u > 90 kPa

The range which is formed by stiffer soils of c_u > 90 kPa is examined on Figure 4.13 together with a soil class differentiation. The plot of data obtained from relatively stiff soils clearly implies the importance of sample quality in strength measurement, especially when the depth of samples (all>30.0 m) obtained in stiff range of İzmir Alluviums is considered (overburden release). Nearly all data is under the equality line, showing that laboratory tests may underestimate the c_u value by a large amount of up to max. -40%, starting from SC class soils between 90-100 kPa and carrying on to higher ranges with other cohesive materials. An average best line for general data gives a laboratory underestimation of by -24% over Stroud's predictions.

Sample disturbance in stiff range would probably occur due to the release of overburden pressure, beside sampling-handling and sample preparation all of which lead to a laboratory specimen that is not representative for the underlying soil strata.

SC class soils in stiff range might give relatively high SPT N numbers due to grain size distribution. Relating to this possibility, predicted c_u values from Stroud (1974) for SC soils might seem to be high and not reasonable when compared to laboratory results.

Comparison of derived f_1' and f_1 proposed by Stroud is given in Figure 4.14. A large scatter is observed because of variations in reliability of c_u values predicted by Stroud (1974) between a PI range of mostly 20 to 50 %.

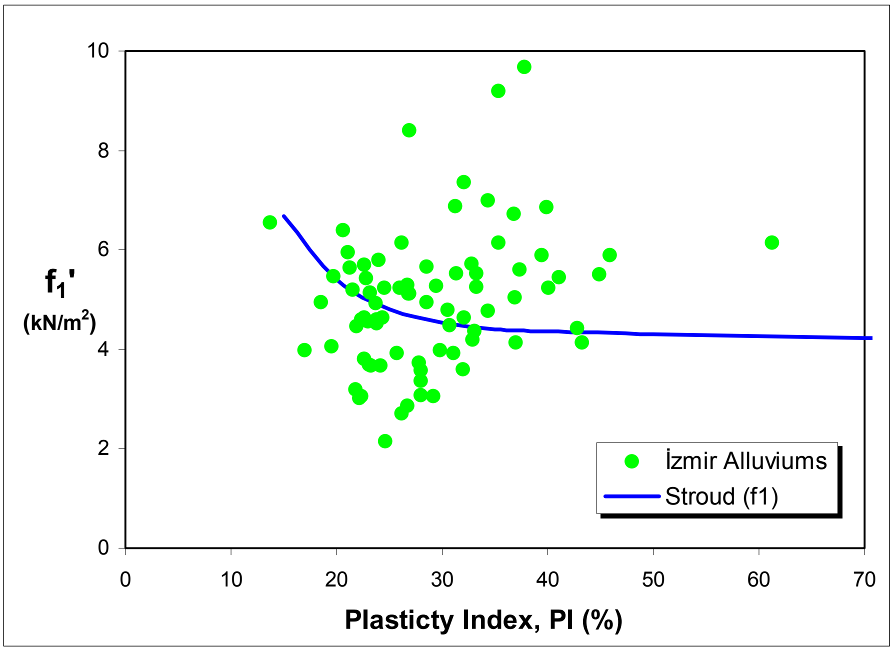


Figure 4.14 Comparison of derived factor f_1' and $f_{1(Stroud)}$ in İzmir Alluviums

The volume compressibility coefficients (m_v) that are predicted from Stroud (1974) and obtained from oedometer tests are compared in Figures 4.15 and 4.16 for İzmir Alluviums. Similar to Ankara Alluviums discussed in Section 4.3.1, laboratory measurements of m_v in alluvial deposits of İzmir site are generally higher than the values predicted by Stroud (1974) due to probable sample disturbances. It could be stated that oedeometer tests overestimate the coefficient m_v by an amount of up to +100% over the ones that of Stroud's with an average value of +13%. A relatively small proximity of maximum -26% underestimation is valid for laboratory test results over $m_{v(Stroud)}$ values with so less number of data.

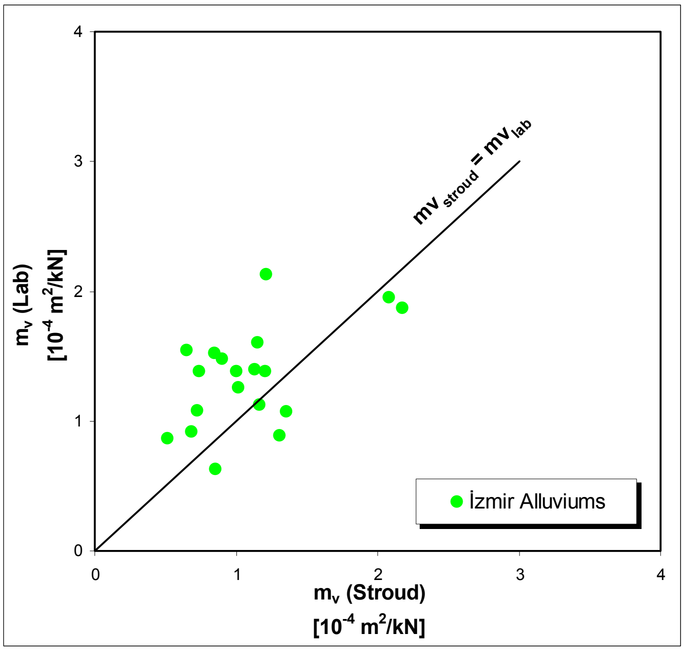


Figure 4.15 $m_{v(Stroud)}$ & $m_{v(Lab)}$ in İzmir Alluviums

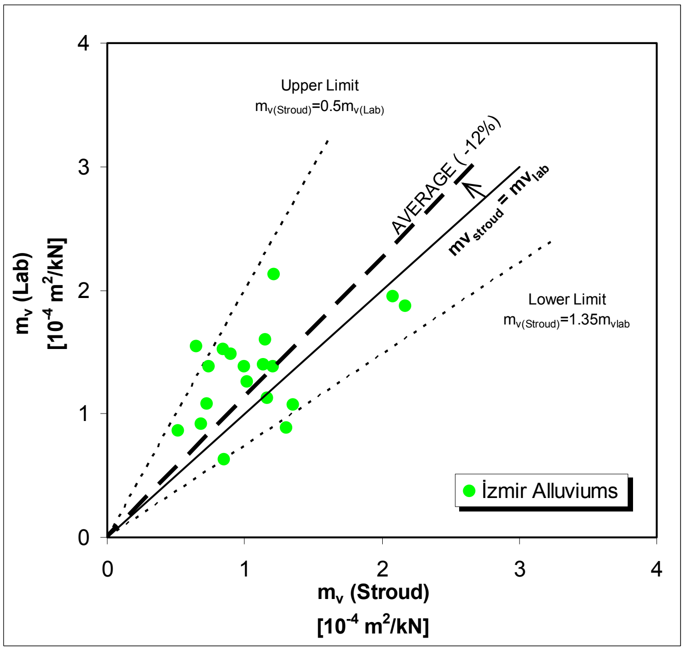


Figure 4.16 Accuracy evaluation of $m_{v(Stroud)}$ in İzmir Alluviums

The f_2' factors derived for İzmir Alluviums are given in Figure 4.17. Since the $m_{v(Lab)}$ values come out relatively high, a larger amount of f_2' data accumulates below the curve predicted by Stroud (1974).

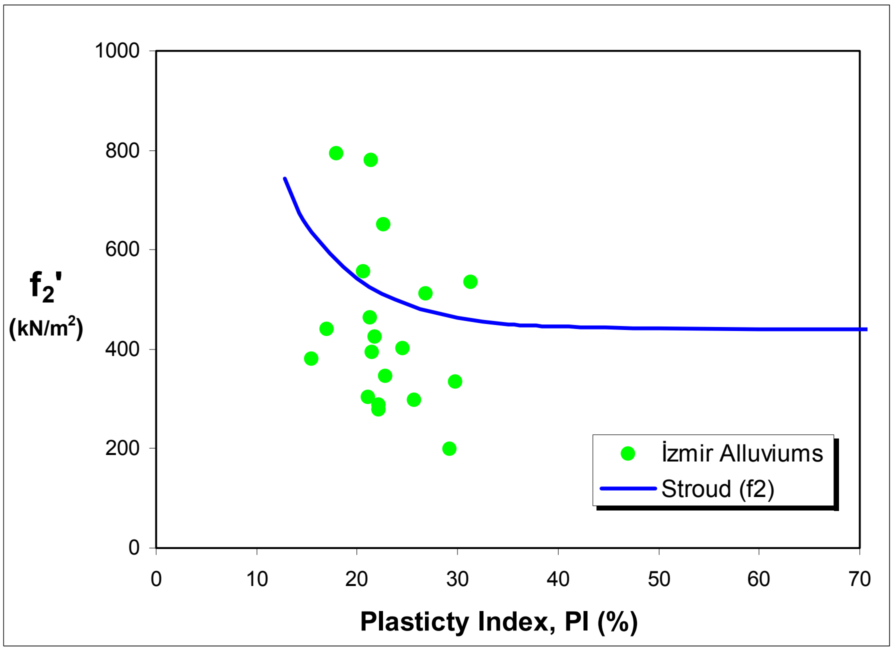


Figure 4.17 Comparison of derived factor f_2' and $f_{2(Stroud)}$ in İzmir Alluviums

4.3.3 Ankara Clay

The comparison of $c_{u(Stroud)}$ and $c_{u(Lab)}$ values in Ankara Clays, which is mostly stiff to very stiff and hard in consistency together with a fissured state is presented in Figure 4.18 & 4.19. Although there seems a roughly close alignment between the line of equality and the path on which data is distributed, very clear

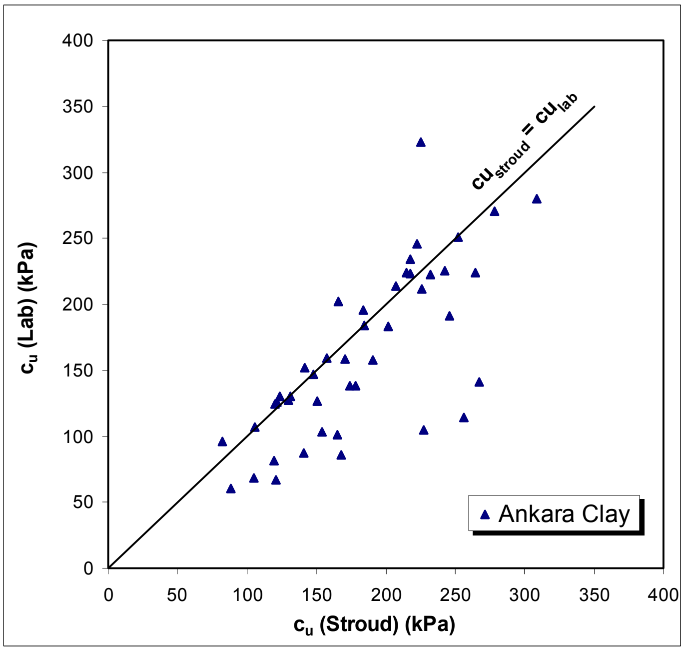


Figure 4.18 $c_{u(Stroud)}$ vs $c_{u(Lab)}$ in Ankara Clay

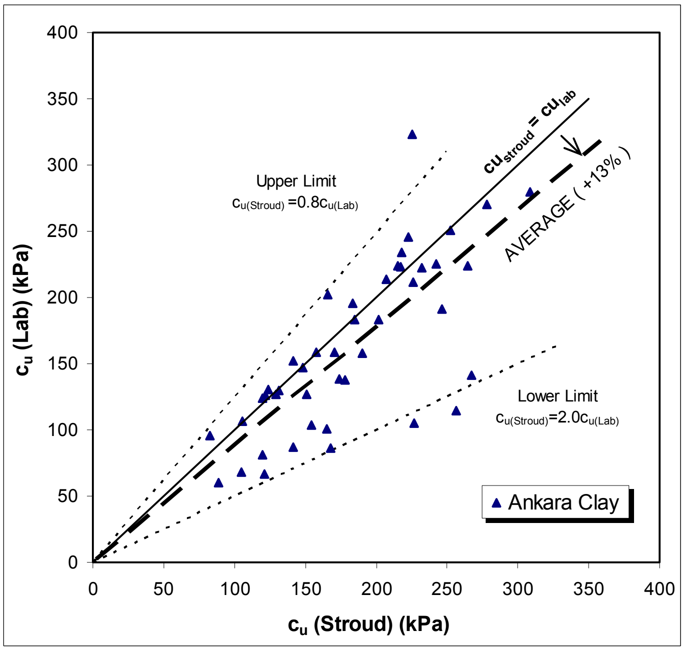


Figure 4.19 Accuracy evaluation of $c_{u(Stroud)}$ in Ankara Clay

distinctions could be made between $c_{u(Stroud)}$ and $c_{u(Lab)}$ values at the lower side of the equality line which shows that laboratory measurements of c_u could be significantly lower than the predictions by Stroud –up to almost half- within a large scatter, presenting an underestimation up to -50 %.

The reason for such a deviation is not only sample disturbance, but also fissured structure of Ankara Clay. During a loading application on a specimen in laboratory, samples would be broken along the weakest points of surfaces. Fissures taking place in Ankara Clay represent such weakining points and planes, where most of the time it may not be the case for the mass behaviour of soil. This is a limitation of laboratory testing methodology that of which is using small specimens having behaviours differing from the whole soil mass.

Furthermore, as the soil stiffness increases, the quality of obtained samples decrease due to the difficulties in operation where more disturbance might be imposed to the soil to take the sample out.

The average line for data set represents an underestimation of c_u about -11% by means of laboratory measurements . The average value could be commended as the emprical relation by Stroud and laboratory measurements are more or less to be on the same path in terms of c_u values, together with some clear discrepancies.

The laboratory derived f_1' value for Ankara Clay also implies that laboratory values of c_u are generally below the estimated values just Stroud (1974) has stated for stiff to hard clays (Figure 4.20).

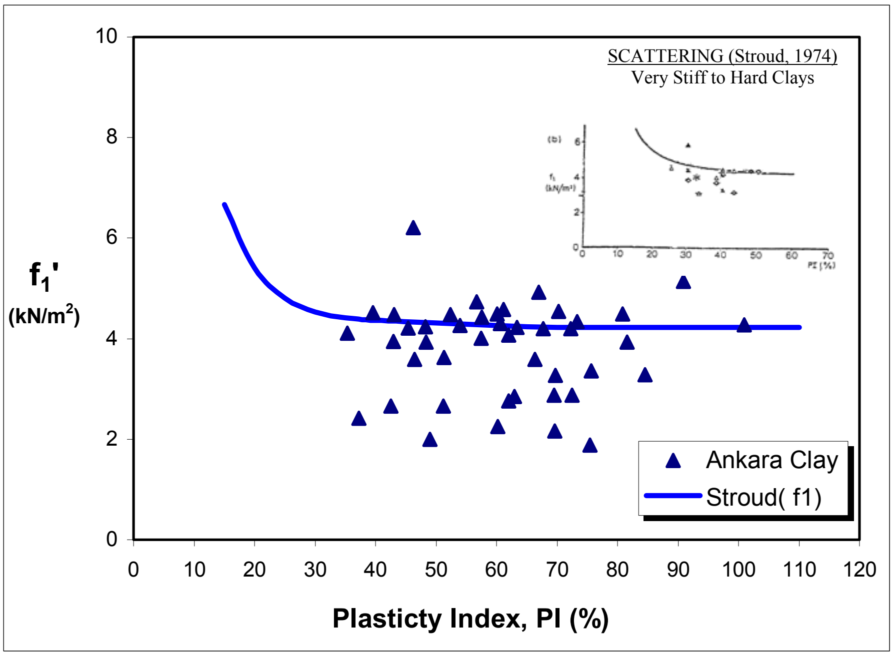


Figure 4.20 Comparison of derived factor f_1' and $f_{1(Stroud)}$ in Ankara Clay

The compressibility characteristics of Ankara Clay is examined by the comparison on $m_{v(Stroud)}$ and $m_{v(Lab)}$ values. The values obtained from laboratory and Stroud's predictions are plotted in Figure 4.20.

The plot is an indicator of typical characteristics of Ankara Clay, that of are stiff, hard to obtain good quality samples etc. Laboratory measurements of m_v may overestimate the predicted

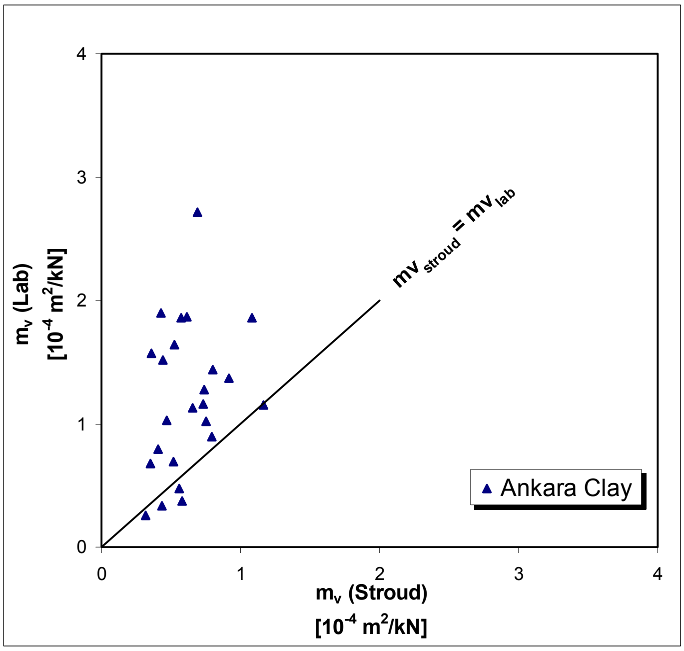


Figure 4.21 $m_{v(Stroud)}$ & $m_{v(Lab)}$ in Ankara Clay

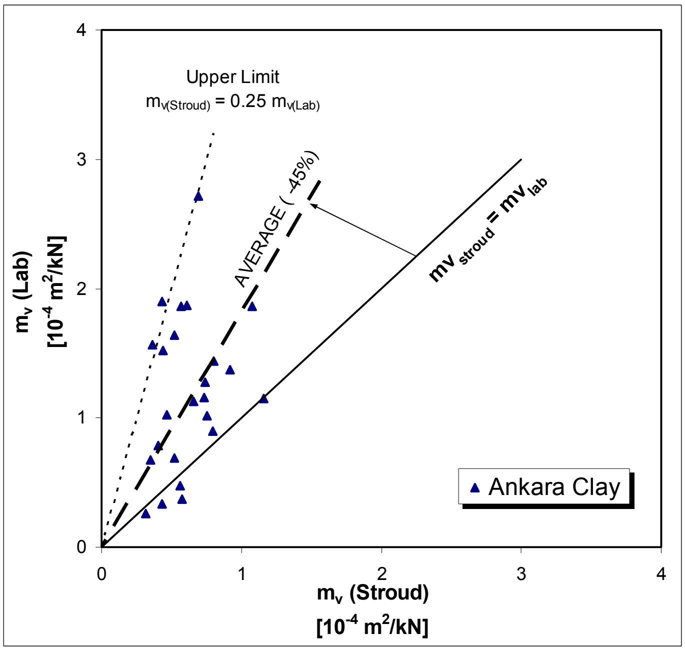


Figure 4.22 Accuracy evaluation of $m_{v(Stroud)}$ in Ankara Clay

values that of Stroud's up to by +300 %, which is seriously high. Even the average best line refers to an overestimation amount of +82% for laboratory results. For the case, the condition of saturation also would be an important effect because the in-situ state of Ankara Clay might not be fully saturated since generally no continuos groundwater table is encountered within Ankara Clay. There were only lenses of sands and gravels keeping some amount of water inside with no lateral or vertical extension. So, since samples are attained in saturated state during an oedemeter tests, laboratory test methodology may be misleading for the representation of in-situ conditions in Ankara Clay.

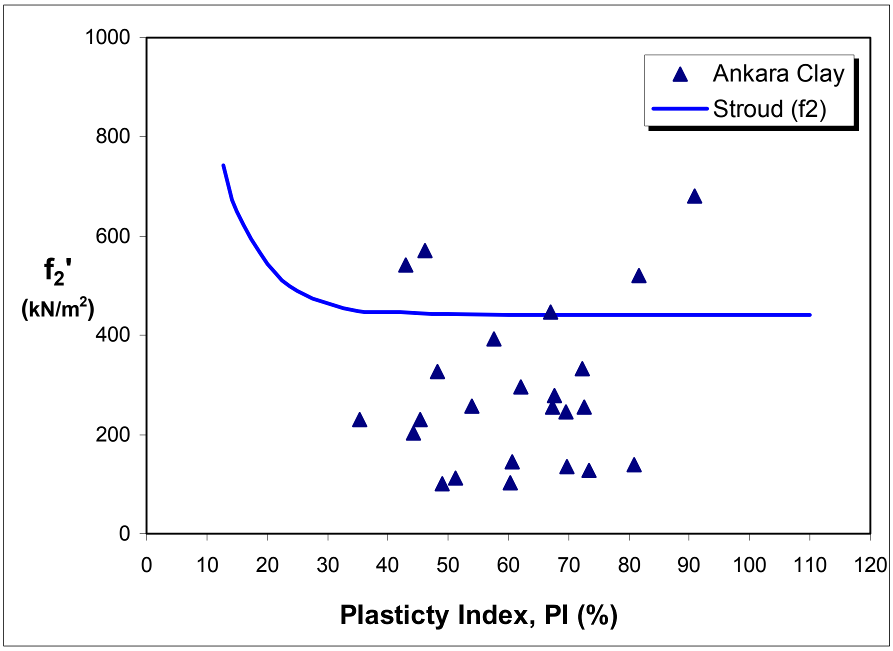


Figure 4.23 Comparison of derived factor f_2' and $f_{2(Stroud)}$ in Ankara Clay

The plot of factor f_2', that is derived from the emprical relation of Stroud by inserting laboratory measured m_v values in Equation 2.5 shows that f_2' values are too much lower than the $f_{2(Stroud)}$ values, resulting from the high $m_{v(Lab)}$ values (Figure 4.23).

4.3.4 General Evaluation

Following the discussions on each soil type experienced in this study, a general evaluation covering all soils together has been performed in order to investigate the reliability of the predictions by Stroud (1974) on c_u and m_v parameters. The laboratory and predicted values by Stroud (1974) are inserted on the same plots for all soils, for the purpose of general comparison.

The general view on plots indicate that the undrained shear strength values, $c_{u(Stroud)}$ predicted by Stroud (1974) and $c_{u(Lab)}$ measured in laboratory by triaxial tests, are likely to be agree for the common of data (Figure 4.24). Some amount of data just over the line of equality [$c_{u(Stroud)} = c_{u(Lab)}$] would probably result due to the evaluation of laboratory triaxial loading tests in data reduction process, which was described in Section 4.1.2. Much of the data -that of are deviating from the equality line- is clearly observed to take place in high c_u range, for mostly stiff to very stiff and hard cohesive materials. Sample disturbance and material characteristics of stiff to hard clays -those of fissures and discontinuties weakining the strength of a specimen- are probable major causes resulting the scatter Stroud (1974).

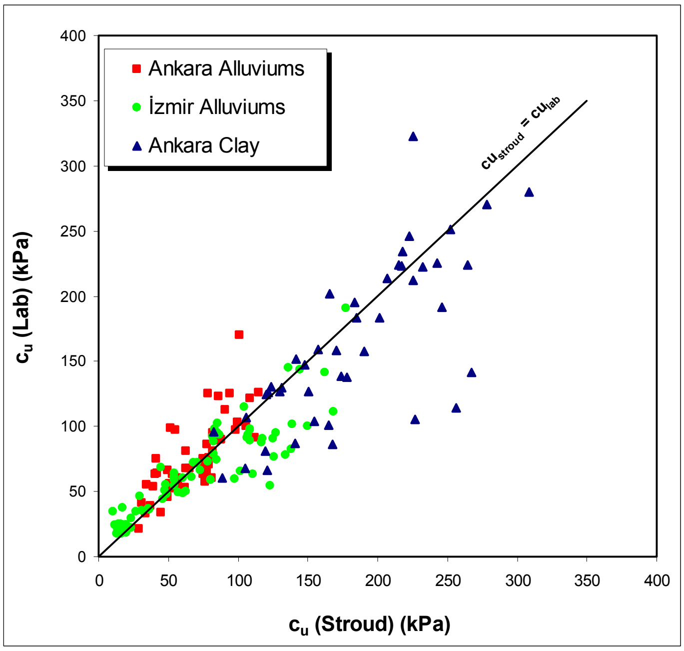


Figure 4.24 $c_{u(Stroud)}$ vs $c_{u(Lab)}$ in experienced soils

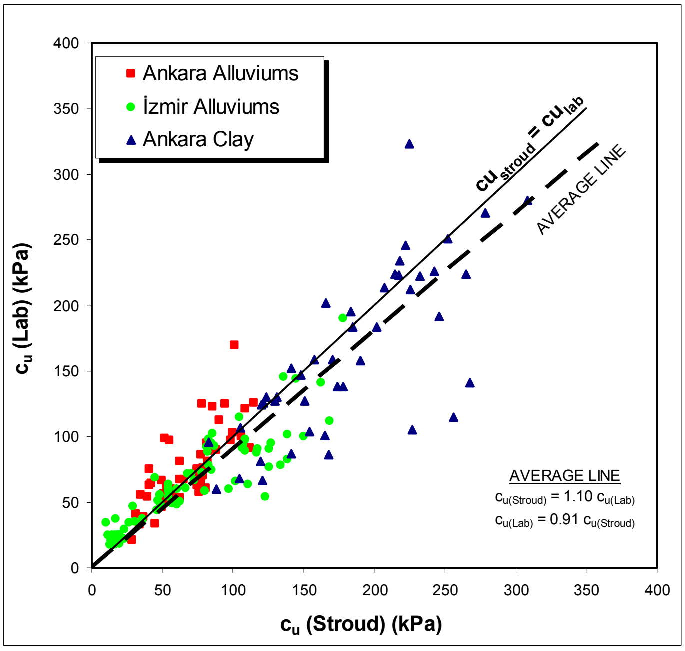


Figure 4.25 Accuracy evaluation of $c_{u(Stroud)}$ in experienced soils

Taking such factors into account, a statement on the reliability of the prediction proposed by Stroud (1974) could be explained that $c_{u(Stroud)}$ overestimates the laboratory measured undrained shear strength [$c_{u(Lab)}$] values by only +10% as seen from a general average line plotted through overall data (Figure 4.25). The value 10% would decrease much more if some of the deviating data of Ankara Clay resulting from sample disturbance & fissures were not included in evaluation.

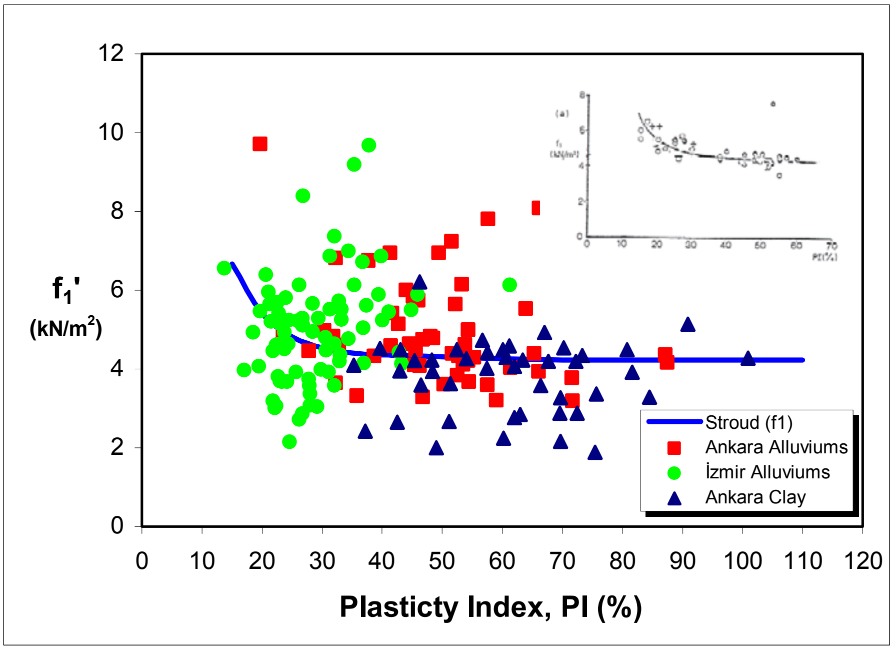


Figure 4.26 Comparison of derived factor f_1' and $f_{1(Stroud)}$ in experienced soils

The factor of f_1', designated from laboratory measured c_u values are compared with f_1 (Stroud) values for all experienced soils included in this study (Figure 4.26). The scatter of data is generally dense in the vicinity of the curve by Stroud (1974) whereas an amount of scattering could also be noticed due to the deviations in $c_{u(Stroud)}$ vs $c_{u(Lab)}$ comparison discussed above.

The coefficient of volume compressibility, m_v, for all soils are compared in terms of laboratory measured and predicted values (Stroud, 1974) in Figure 4.27. Approximately all of the data points are located above the line of equality, which is an indication of an important overestimation of m_v by laboratory tests. So few data is just below the equality line with no effect on general trend.

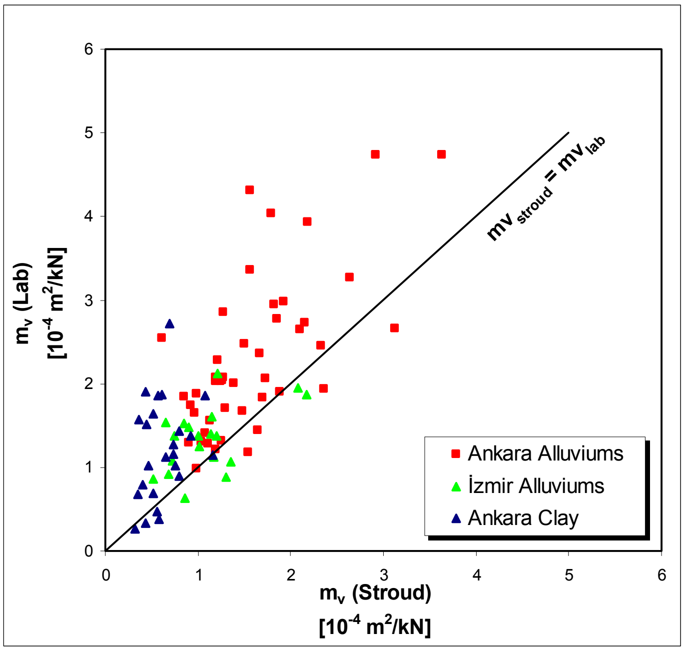


Figure 4.27 $m_{v(Stroud)}$ & $m_{v(Lab)}$ in experienced soils

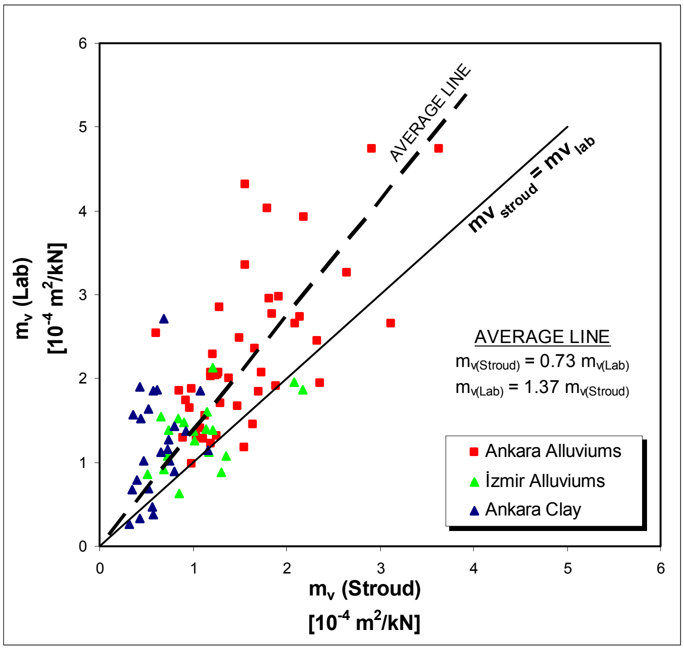


Figure 4.28 Accuracy evaluation of $m_{v(Stroud)}$ in experienced soils

An average best line fitted for data plots gives that laboratory values of m_v highly overestimates the values predicted by Stroud (1974), by an average of +37% and sometimes up to a serious amount of by +200% (Figure 4.28). The difference would likely to be due to the samples that is unable to comprise the macrofeatures of the soil mass; differences in the saturation of in-situ soil layer and laboratory sample and possible effect of disturbances arising from sampling.

Saturation, among the possible reasons of overestimated m_v values in laboratory, could somehow be explained on the plot in away that the only type of soil which is fully submerged due to the extensive groundwater level within the first meter of soil profile all around the project site is İzmir Alluviums. The results obtained from İzmir Alluviums seem least deviating ones among others. Furthermore, the type of soil with the lowest range of saturation would be Ankara Clay, due to the partial groundwater content of the in-situ soil that is kept only in thin sand-gravel lenses with no extension. Then, this may explain why data from Ankara Clay is deviating relatively high in general when compared to alluvial deposits.

The reason, sample disturbance is likely to be considered for all soils, whereas the increasing sample disturbance in stiff range as implied by Stroud(1974) for similar materials where intact clay becomes harder would be much more valid for Ankara Clay explaining high deviations within the data.

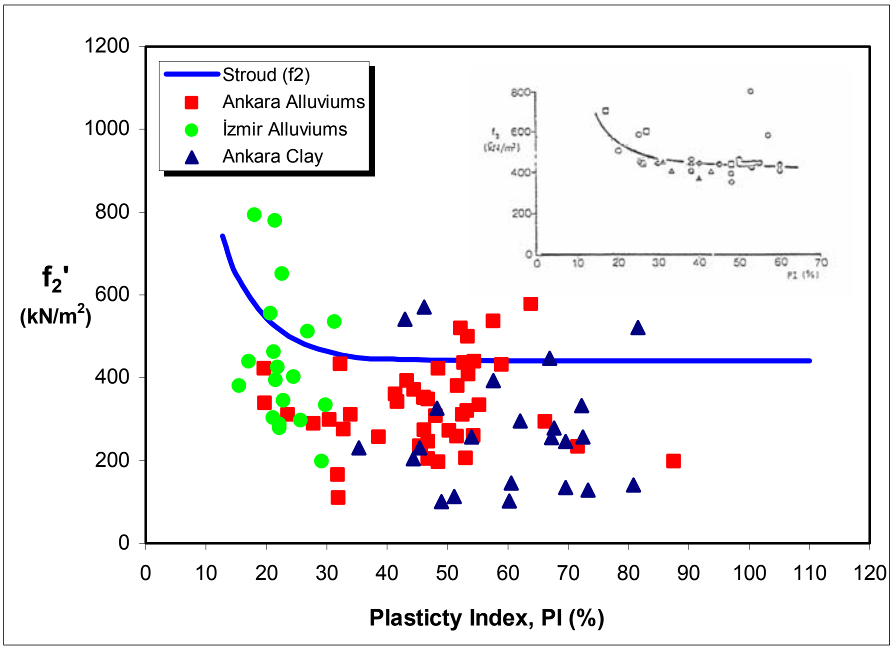


Figure 4.29 Comparison of derived factor f_2' and $f_{2(Stroud)}$ in experienced soils

The comparison between f_2' and the factor f_2 by Stroud (1974) is given in Figure 4.29. Relating to high m_v values measured by oedometer tests in laboratory, the data f_2 is mostly falling well below the line of Stroud (f2), where the lowest values are generally from Ankara Clay, as expected due to the discussions above.

4.3.5 General Evaluation Regarding Corrections in SPT

The discussion on the application of SPT results for parameter predictions as presented in previous sections could be also extended for the interpretation of number N, that is used to estimate c_u and m_v. The raw SPT result, N, could be partially improved by applying some correction factors for the compensation of possible variations in testing procedures as explained in Section 2.3.3.

The factors of SPT corrections are mainly defined for the concepts of borehole diameter, sampler, rod length and hammer efficiency. Considering the experimental studies included in this study and emprical methods of parameter predictions discussed, the corrections concerning borehole, sampler and rod length are either irrelevant or partially indefinite to apply. Moreover, they also have so little effect on the number N numerically, being minor factors in corrections.

Relating to the predictions of Stroud (1974), a correction for hammer efficiency might be discussed as the only reasonable one among others, although even it is not clearly defined and indicated. Since the description by Stroud (1974) generally consists of British practice in SPT, the hammer efficiency could be considered as 60 %, which is likely to be a common number in literature as N_{60}. Beside, the experimental part of this study regarding SPT works is possible to be assumed with an energy ratio of 45% representing Turkish practice. Then, the discussion performed in Section 4.3.4 could be also reviewed in a way for the number N that is corrected for energy ratio, where a factor of 0.45/0.60 is applied to the raw SPT N values measured in field. In case where N_{60} is established and used in predictions by Stroud (1974), the comparison of laboratory values and estimations by Stroud on undrained shear strength, c_u is presented in Figure 4.30. Similar to the view in Section 4.3.4, the data points are generally accumulated along the line of equality [$c_{u(Stroud)} = c_{u(Lab)}$], however, decrease in number N due to the energy correction resulted in underestimations by Stroud (1974) at about 17% in average over the laboratory measured values (Figure 4.31). It could also be stated that the evaluation of undrained shear strength in laboratory tests performed by data reduction process (regarding $\phi>0°$ cases) may also have little effect on the general view. Beside, there are still some amount of data for Ankara Clay, laying below the equality line, where laboratory measurements may be underestimating the c_u values compared to $c_{u(Stroud)}$. This is due to the factors discussed previously, such as sample disturbance, fissures and differences in saturation conditions in very stiff range of clays.

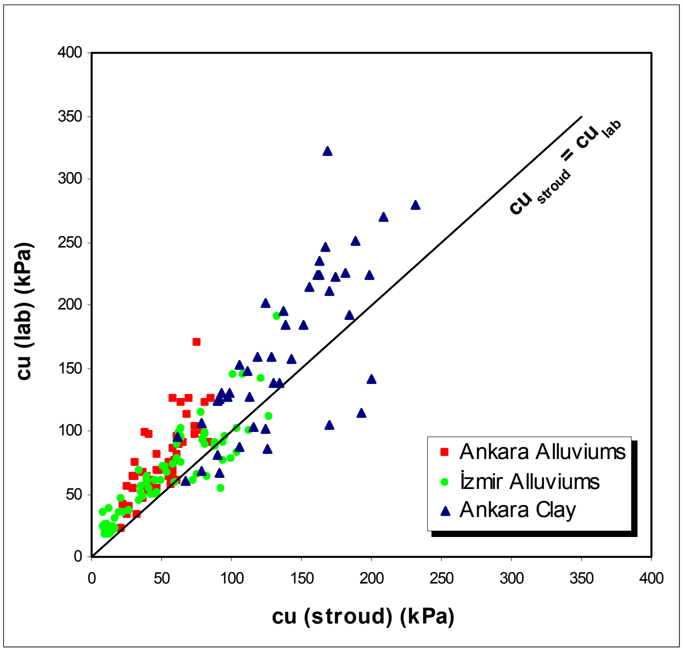


Figure 4.30 $c_{u(Stroud)}$ vs $c_{u(Lab)}$ in experienced soils (N_{60})

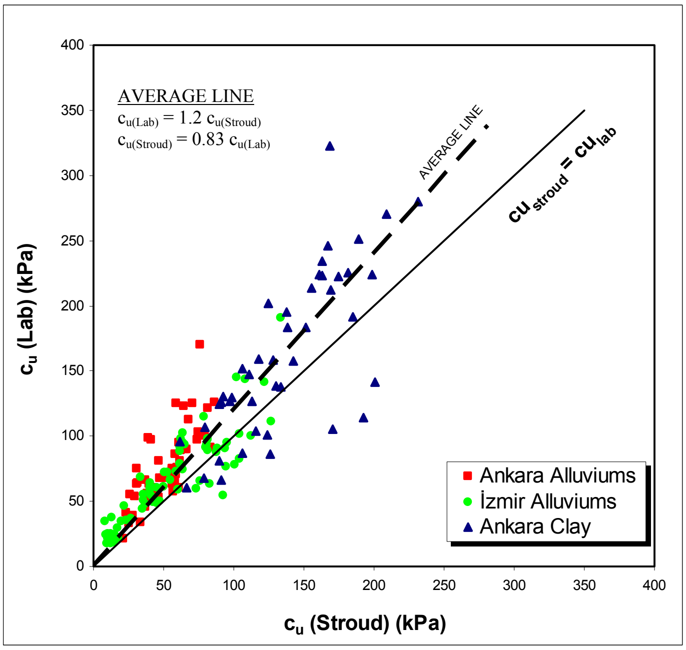


Figure 4.31 Accuracy evaluation of $c_{u(Stroud)}$ in experienced soils (N_{60})

The factor of f_1' obtained from laboratory measured c_u values by using N_{60} is plotted against PI values for the general of experienced soils in order to compare with $f_{1(Stroud)}$ values (Fig 4.32). The general of data points seem to take place over the predictions by Stroud, except for some in Ankara Clay. Then, the view in f_1 could be considered as a confirmation to the discussion above for c_u.

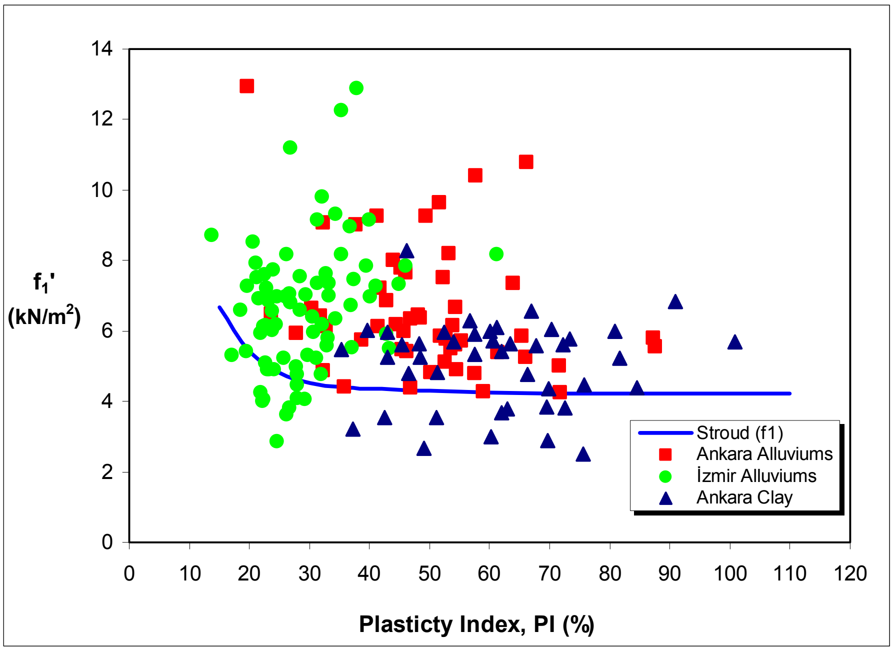


Figure 4.32 Comparison of derived factor f_1' and $f_{1(Stroud)}$ in experienced soils (N_{60})

The energy correction in SPT N number is also searched for the coefficient of volume compressibility predictions over the description by Stroud (1974). If the corrected N numbers, N_{60} has been applied for the estimation of m_v, the comparison of

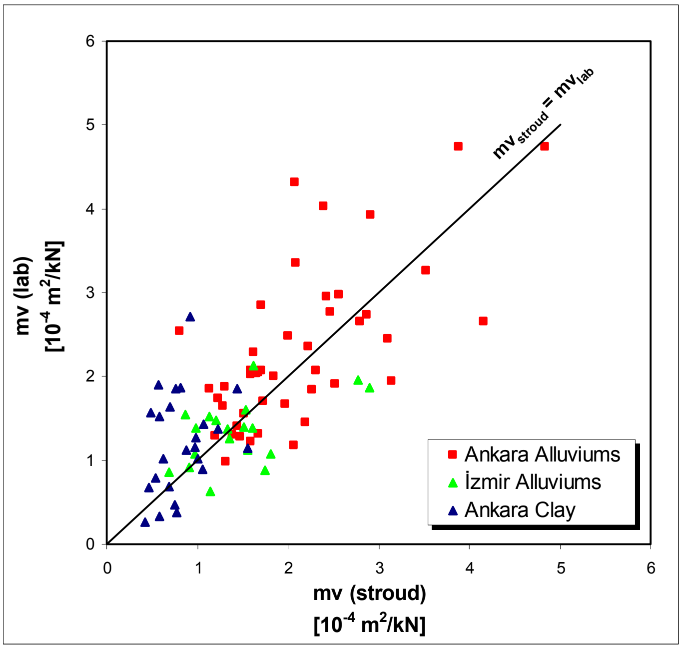


Figure 4.33 $m_{v(Stroud)}$ & $m_{v(Lab)}$ in experienced soils (N_{60})

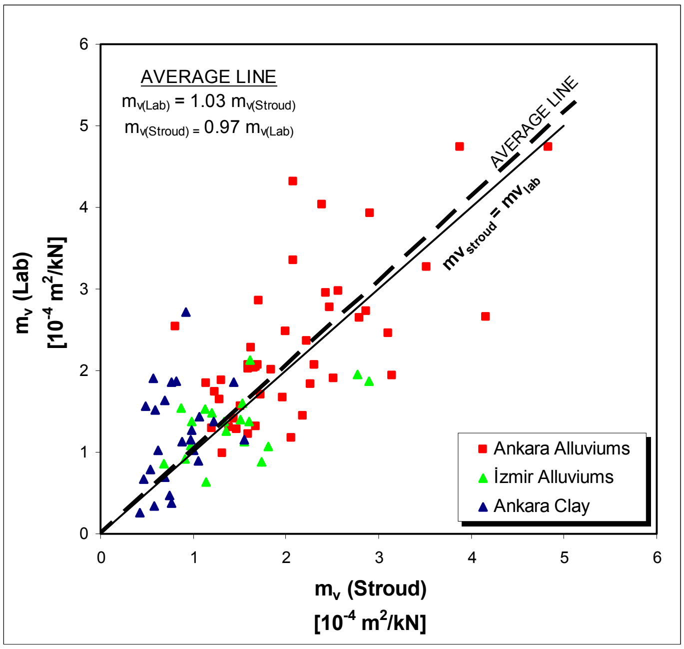


Figure 4.34 Accuracy evaluation of $m_{v(Stroud)}$ in experienced soils (N_{60})

laboratory measurements with estimated values by Stroud (1974) comes out as in Figure 4.33. Together with a scattering, $m_{v(Stroud)}$ values seem to be closer to the $m_{v(Lab)}$ values due to the decreasing N, when compared to the view in Section 4.3.4. The overestimation in m_v by laboratory measurements may be stated as just 3% in average for the case of SPT N corrected for hammer efficiency (Figure 4.34). Although $m_{v(Stroud)}$ and $m_{v(Lab)}$ values seem close in general, a clear deviation is still visible in the very stiff to hard clays of Ankara, indicating the overestimation of m_v by laboratory measurements, which is mainly due to the reasons of sample disturbance and saturation conditions as mentioned before.

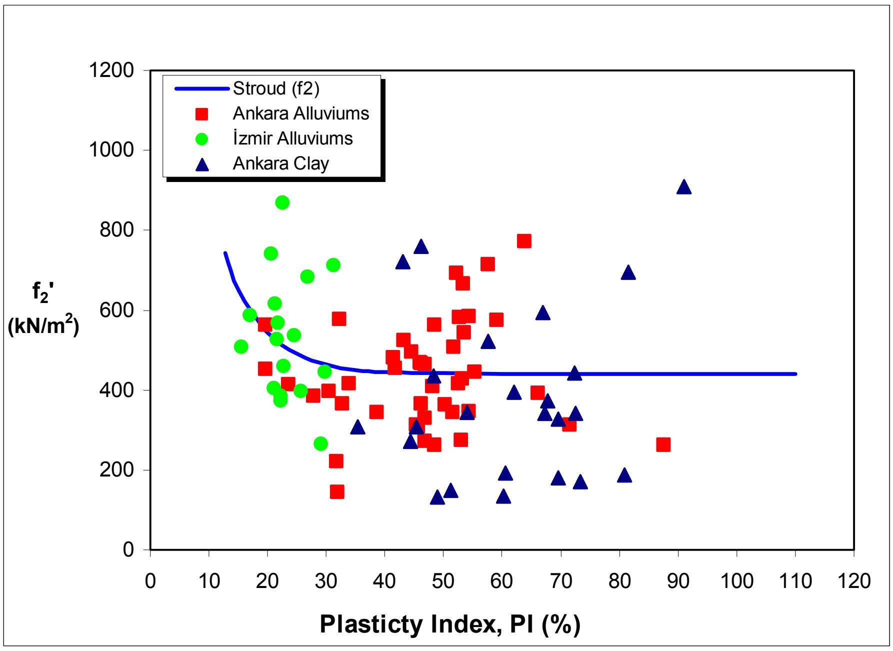


Figure 4.35 Comparison of derived factor f_2' and $f_{2(Stroud)}$ in experienced soils (N_{60})

The comparison of the factor f_2' with $f_{2(Stroud)}$ is plotted in Figure 4.35. The scattering for f_2' could be considered as laying both above and below the curve by Stroud (1974), together with the lowest values of f_2' designated in Ankara Clay that still indicates overestimation of m_v values in laboratory for very stiff to hard range of clays when compared to predictions by Stroud (1974).

4.4 Evaluation of PMT Results for Parameter Estimation

The pressuremeter test results, obtained from experienced soils included in this study, are mainly used for the prediction of undrained shear strength (c_u) and vertical stress-strain modulus (deformation modulus, E) of soils, relating to the laboratory test results.

Since the number of pressuremeter tests performed in soft soils of alluvial deposits with low limit pressure values is quite low and not reasonable for a seperate study when compared to general data distribution, no distinction is considered for soils from different sites and all the pressuremeter test results are demonstrated on a single figure for each parameter prediction.

The undrained shear strength of soils are investigated principally from the Equation 2.15, referring to the number β, where β is the ratio of net limit pressure to undrained shear strength, p_{ln}/c_u. For the purpose, the net limit pressures obtained from pressuremeter tests in this study are plotted against the $c_{u(Lab)}$ values obtained from triaxial loading tests (Figure 4.36)

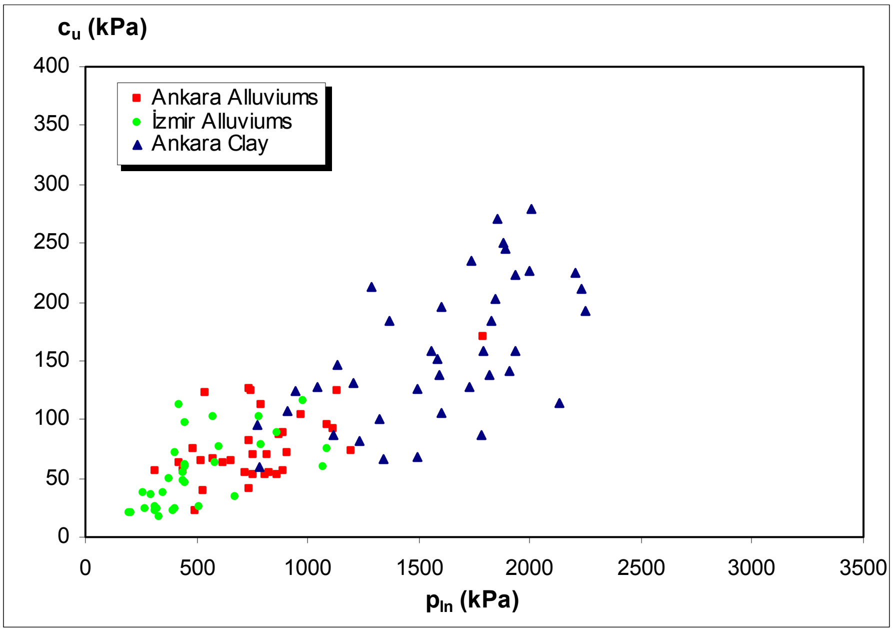


Figure 4.36 c_u vs p_{ln} for experienced soils

The plot of c_u vs p_{ln} for experienced soils seems to give a proportional relationship, a typical increase in c_u with increasing p_{ln}, which infact leads to provide a β value, named as β_{ave} for the purpose of this study. If a best line is fitted through the whole data, a general average value of β_{ave}=10.2 is provided within a range of 6.5 to 16 (Figure 4.37)

The general average value of β, β_{ave}, obtained in this study has been compared with various values proposed in the literature that were discussed in Section 2.3.4.4.

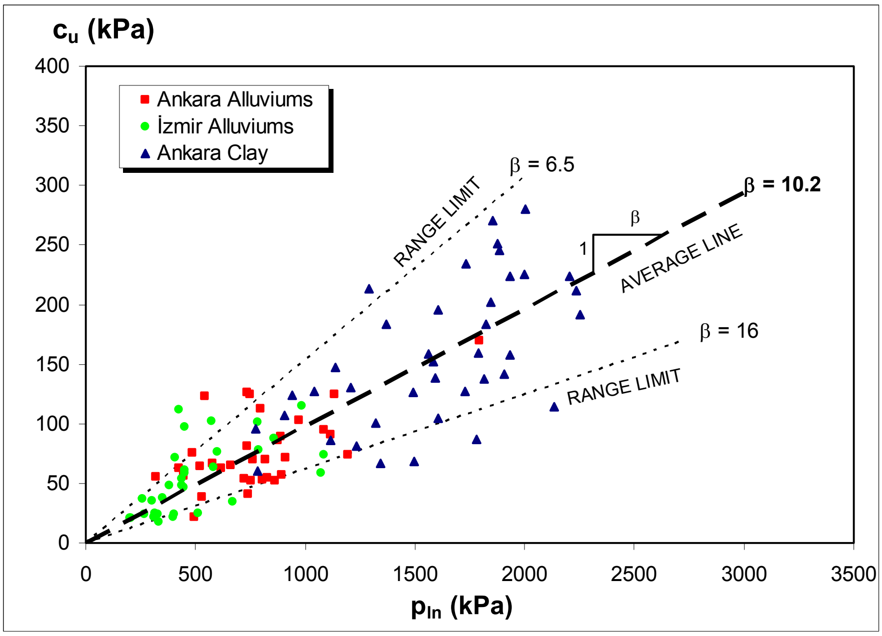


Figure 4.37 Evaluation of β_{ave} for experienced soils

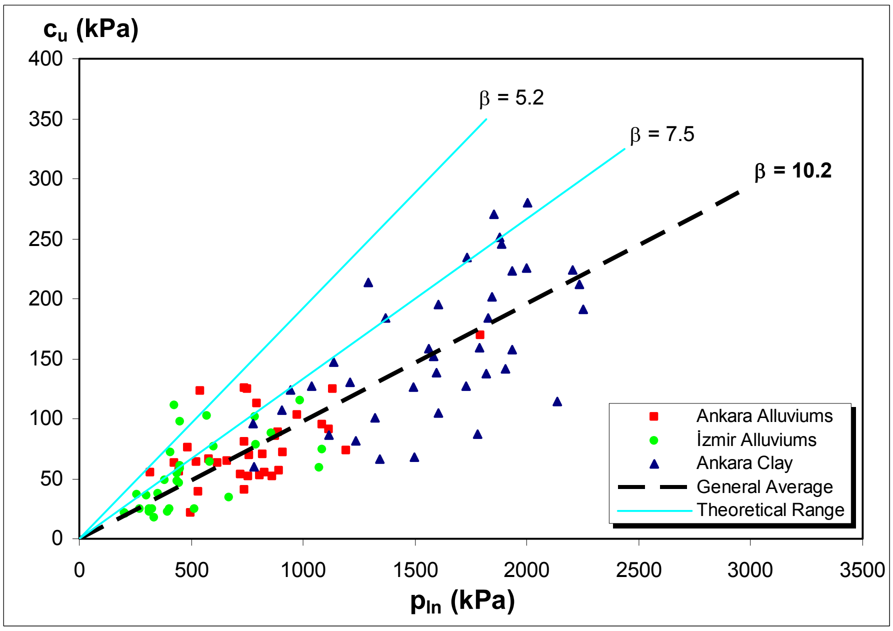


Figure 4.38 Comparison of β_{ave} with theoretical β values (Baguelin et al., 1978)

The first comparison is executed between theoretical values of β, which were determined from Equation 2.14 and β_{ave}, where theoretical values are ranging from β=5.2 to β=7.5 and β_{ave} is obtained as 10.2 for experienced soils (Figure 4.38)

Clearly, β_{ave} is higher than the theoretical values, but it could be because of some characteristic reasons: low value of c_u due to laboratory measurements, error in p_{ln} calculations, nonhomogenity, sensitivity and borehole disturbance. The effects of sample disturbance may come to the screen for the measurement of c_u in the laboratory, which were discussed together with other possible causes in Section 2.3.4.4.

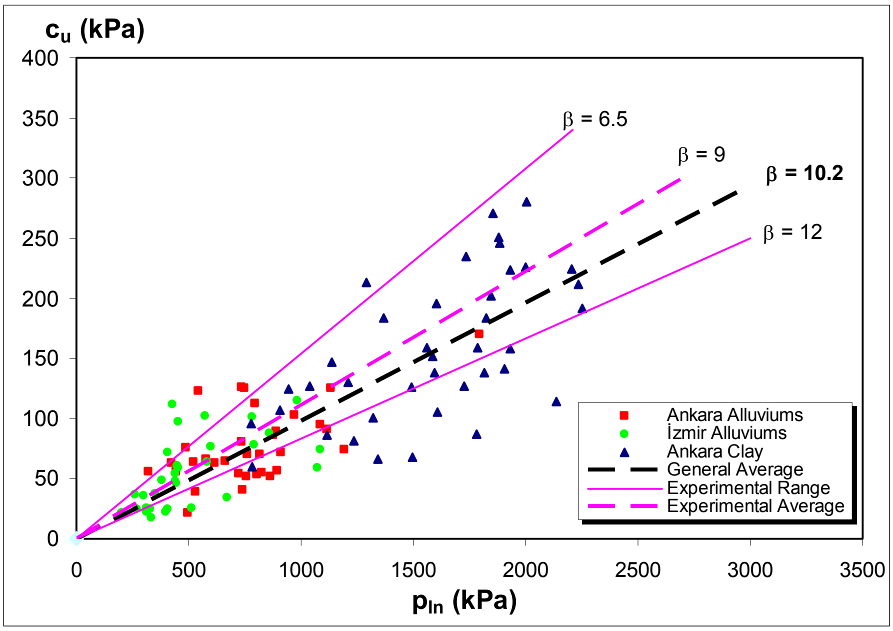


Figure 4.39 Comparison of β_{ave} with experimental β values [Higgins (1969), Cassan (1972), Komornik et al. (1970)]

Apart from the theoretical values, β_{ave} value is compared with an extensive experimental presentation given in Section 2.3.4.4, comprising the data from Higgins (1969), Cassan (1972), Komornik et al. (1970) and from some unpublished resources (cited from Baguelin et al., 1978,). The ranges and average of this experimental presentation is plotted on Figure 4.39 together with β_{ave} value from this study. It is observed for this study that if some of the results are ignored, the remaining data points would fall within the band formed by experimental ranges of β = 6.5 and β = 12 with an average of β=9 for stiff to very stiff clays where the ranges for β_{ave} = 10.2 in this study is obtained as 6.5 to 16. Average values of β for both studies are close and the lower range of β is exactly same, whereas the upper boundary of β is obtained higher in this study. The reason for that might be some data from Ankara Clay, resulting in very high β values, due to probable sample disturbance effecting the measured c_u values in a decreasing way.

The recommendations from Amar et al. (1972) and Cassan (1972) -which were discussed previously- are also evaluated as a comparison with the data from this study and presented in Figure 4.40. The value β=5.5 proposed by Cassan (1972) and Amar et al. (1972) for low values of p_l (<300 kPa) is not included in evaluations since not enough data is available for that range in this study. The relationship given by Amar et al. (1972) in Equation 2.17 is not relevant for the data set of this study, wheras the recommendation β = 8 (for medium values of p_l) and β = 16 (for high values of p_l) by Cassan (1972) is actually in a close agreement with the whole of data obtained in this study. When the range of data, β = 6.5 – 16 is considered for this study together with the included materials of alluviums with medium p_l and hard clays of Ankara with high p_l, this agreement could be extended till the boundary values of β successfully.

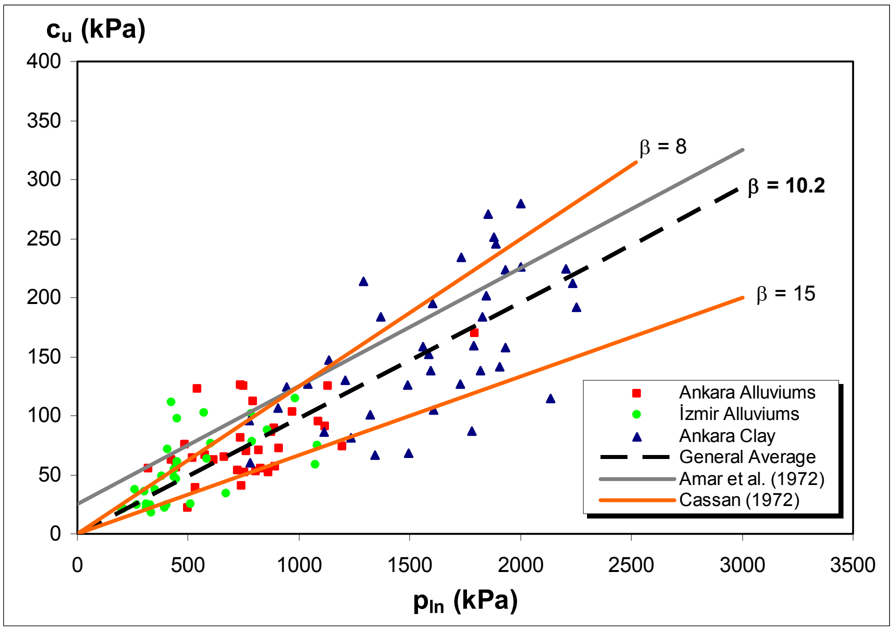


Figure 4.40 Comparison of β_{ave} with other recommended β values [Amar et al. (1972), Cassan (1972)]

The proposals by Clarke (1995) with a value of β = 12 and Martin & Drahos (1986) as β = 10 for stiff clays could also be concluded to be in a good agreement with β_{ave} = 10.2 of this study.

The direct correlation from Baguelin et al. (1978) as given in Equation 2.18 is compared with the whole data of this study in Figure 4.41:

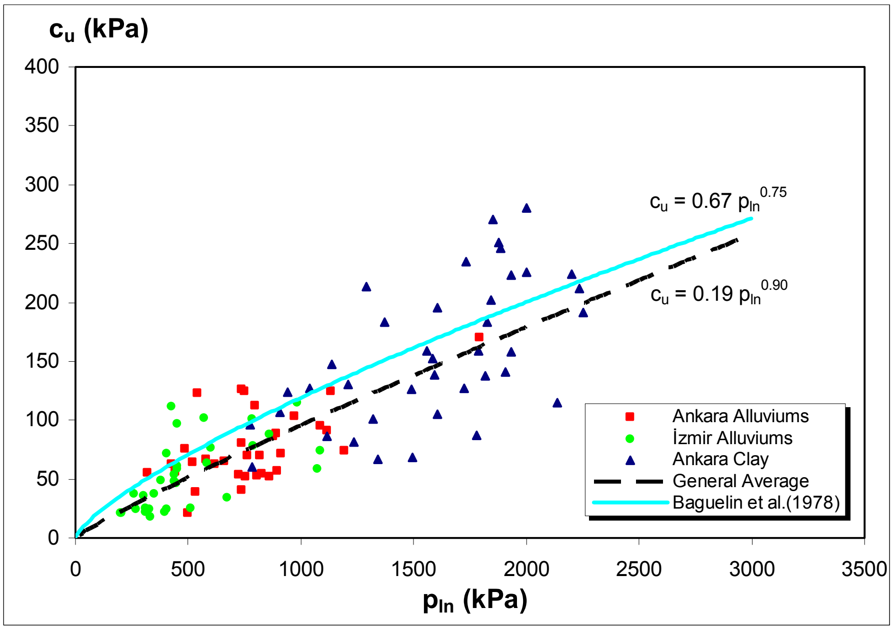


Figure 4.41 Comparison of β_{ave} with the direct correlation by Baguelin et al. (1978)

Following the plot of data points and the graph of equation by Baguelin et al. (1978), a power function is fitted to the whole data that is named as general average, similar to that of the proposed correlation.

$c_u = 0.67\ p_{ln}^{0.75}$ (Baguelin et al. (1978)

$c_u = 0.19\ p_{ln}^{0.90}$ (Experienced Soils)

Despite of numerical differences in equation, the plot of two equations seem to be on the same path and an agreement in a way. The steep slope of the plot by Baguelin et al. (1978) at low c_u values would result from the differences in databases. The database Baguelin et al. (1978) involves important amount of data at low c_u values (<50 kPa) with a recommendation of β= 5.5 and beside β=10 for c_u values of about 150 kPa. The latter one, β = 10 is infact the one exactly corresponding to the range of this study, with the value of β_{ave}=10.2.

In addition to undrained shear strength predictions, pressuremeter tests are utilized to estimate the deformation characteristics of experienced soils. The modulus of deformation for experienced soils is searched using the relationship stated by the Centre d'Etudes Menard (1975) in Equation 2.19. The pressuremeter modulus (E_p) values are determined from the tests performed on studied soils. The ratio E_p/α is referred as a vertical stress-strain modulus (Young's Modulus, E) in related equation. α values are obtained from Table 2.11 taking the E_p/p_{ln} ratio in to account and E_p/α ratio is plotted against $1/m_v$ values where $1/m_v$ is also a kind of representation for vertical deformation modulus, somehow indicating the deformation characteristics of soils. The related plot is given in Figure 4.42.

It is obviously implied on plot that the modulus of deformation values represented by oedometer tests as $1/m_v$ is much more lower than that of predicted by E_p/α. Ignoring a few data from Ankara Clay, a consistent average ratio of 2.91 has been calculated by means of an average best line fitted on.

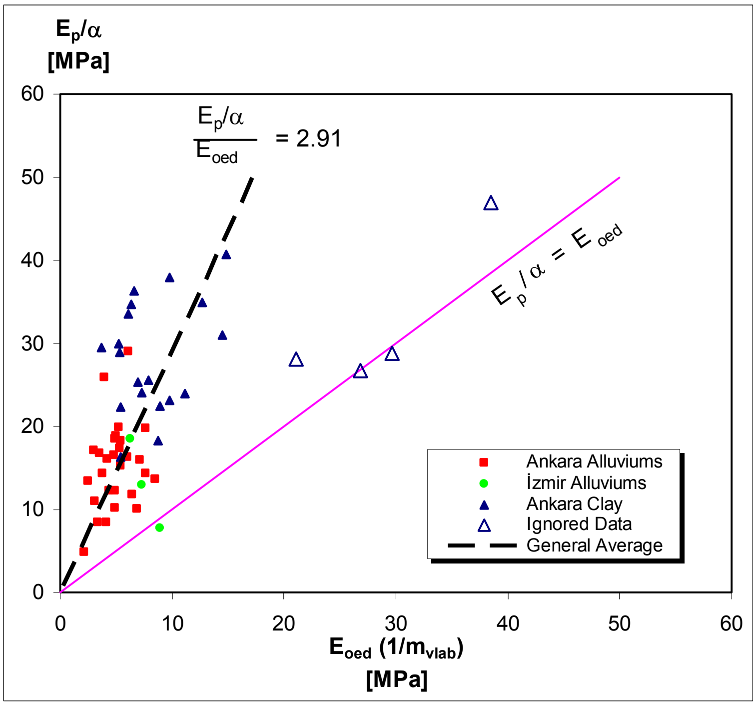


Figure 4.42 Comparison of E_p/α vs $1/m_v$

The relation coming out would probably focus the attention on the values of m_v, which was highly argumentative in Section 4.3 as a measurement by oedometer tests in laboratory. The laboratory measured m_v values might be supposed as considerably high in general, resulting low deformation modulus when compared to pressuremeter based moduli prediction. Beside the general effects of sample disturbance, saturation condition might partially lead to the situation in a way that pressuremeter tests evaluated for deformation modulus analysis are performed generally on partially saturated or even dry soils of in-situ (only a few data from İzmir Alluviums) whereas oedometer tests provide a saturation condition in laboratory.

4.5 SPT – PMT Relationship

The utilization of in-situ tests in a geotechnical design practice is a very common and accepted approach, where the predictions are generally between an in-situ test result and a measurable parameter or performance. However, any correlation between two in-situ tests could also be used as an additional alternative in order to check out the design values from an indirect way for a brief idea, especially where only one is possible for a site.

The in-situ tests, Standard Penetration Test and Pressuremeter Test covered in this study are examined in order to establish a correlation between the results of both. The previous studies were generally on granular type of materials.

The number N, from SPT is firstly plotted against the limit pressure values, p_l determined from pressuremeter tests (Figure 4.43). The geometry is so obvious that the resistance of a soil against penetration could be assumed as a measurement of its resistance against a pressure being applied on it through its failure. A numerical analysis is performed on the plot of data and presented in Figure 4.44. A best line fitted on data as an average is resulted in an equation that:

$$\frac{\boldsymbol{N}}{\boldsymbol{p_l}} = 2.25 \qquad (4.1)$$

[p_l : $\frac{1}{100}$ kPa]

where the square of correlation coefficient, R^2 = 0.843.

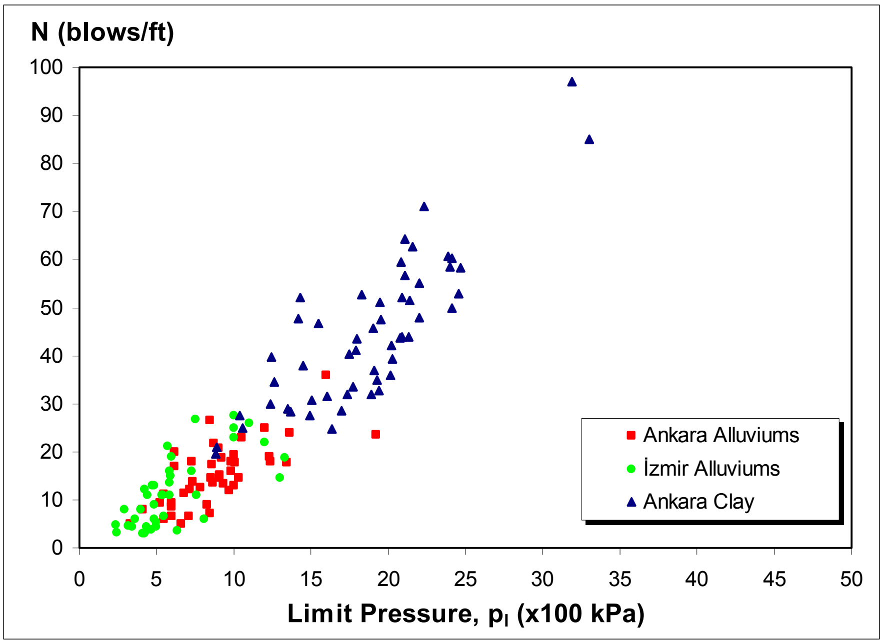


Figure 4.43 N vs p_l distribution for experienced soils

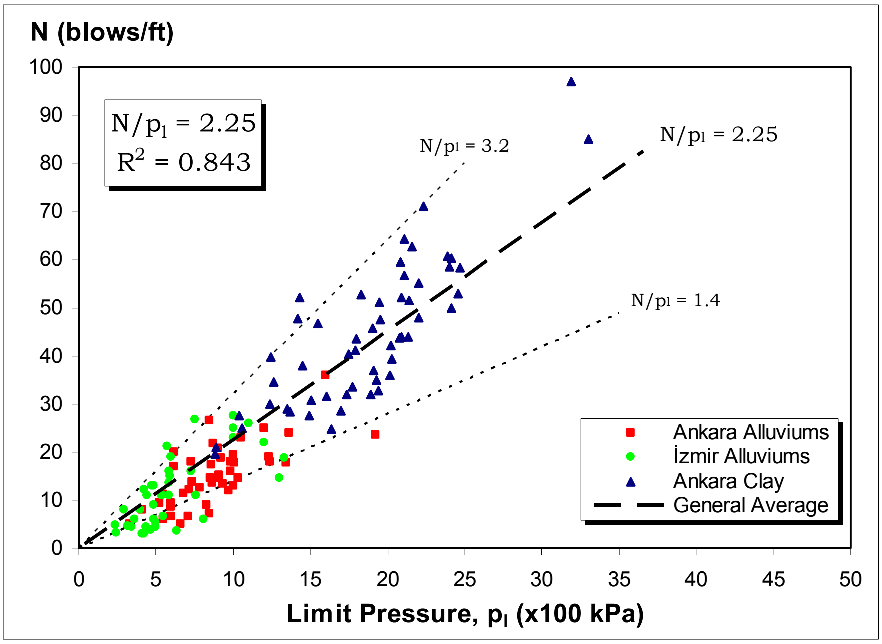


Figure 4.44 N vs p_l correlation for experienced soils

The ratio, N/p_l is generally changing between 1.4 and 3.2, where lower range is dominated generally by alluviual deposits. This indicates that together with a well established correlation by N/p_l = 2.25, stiffness of soils is a deterministic factor in fact to effect the ratio N/p_l within a range of 1.4 to 3.2, where the lower the stiffness, the lower the ratio and vice versa.

Beside limit pressure, the number N is also compared with the pressuremeter modulus, E_p. The data is plotted in Figure 4.45. A very clear tendency for a linear increase could be easily observed between two parameters, N and E_p. A fitted best line gives:

$$\frac{\boldsymbol{N}}{\boldsymbol{E_p}} = 0.20 \qquad (4.2)$$

$[E_p : \frac{1}{100}\ \mathrm{kPa}]$

where the square of correlation coefficient, R^2 = 0.883 (Figure 4.46).

The range of N/E_p is generally from 0.13 to 0.29 with a little amount of scattering in alluvial deposits. Pilot (1982) has proposed a general range for clay type of soils as presented in Section 2.3.5. He stated that N/Ep ratio is between 0.8 - 1.1 where E_p is in MPa. If the pressure unit used in this study (which were mostly the original measurement units of test equipment as kg/cm^2) is converted to MPa, the range obtained for soils experienced would turn out to be 1.3 – 2.9. These ranges are over the ones proposed by Pilot (1982), by an amount of more than 100%. The discreperancy is probably due to some

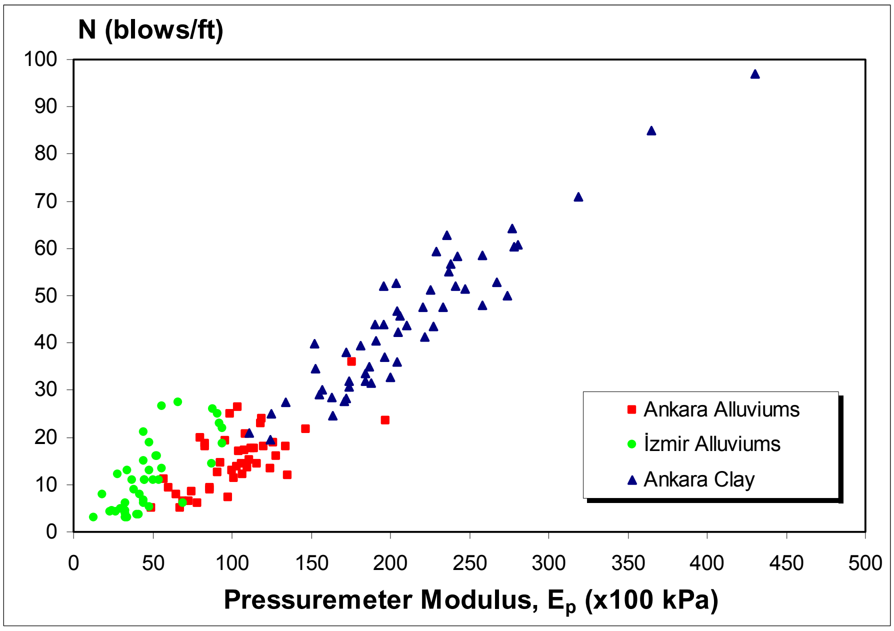


Figure 4.45 N vs E_p distribution for experienced soils

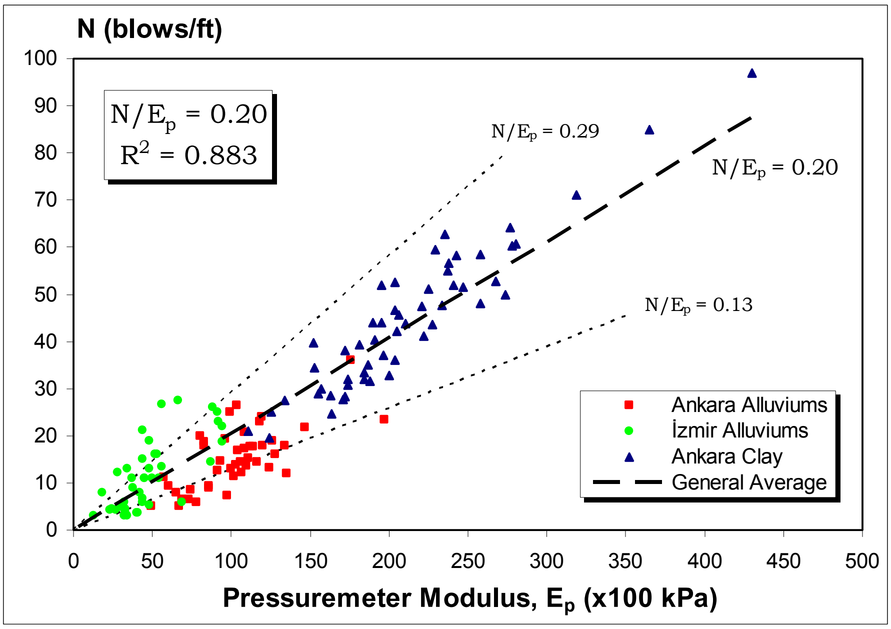


Figure 4.46 N vs E_p correlation for experienced soils

differences in parameter interpretations. For example, the parameter E_p obtained in this study has been interpreted depending on a Poisson ratio of υ=0.33 as a representative value for all soils chosen by Centre d'Etudes Menard (1967), whereas one may prefer to use a value of 0.1 to 0.5 depending on the soil conditions and evaluation. Another probable effect also might be arised from the differences in test techniques, especially pressuremeter test equipment, probe type etc.

The other parameter obtained from pressuremeter test, net limit pressure, p_{ln} is investigated in terms of change with SPT N numbers in Figures 4.47 and 4.48 respectively as distribution and correlation plots. The general trend of p_{ln} together with SPT N is similar to other correlations discussed above.

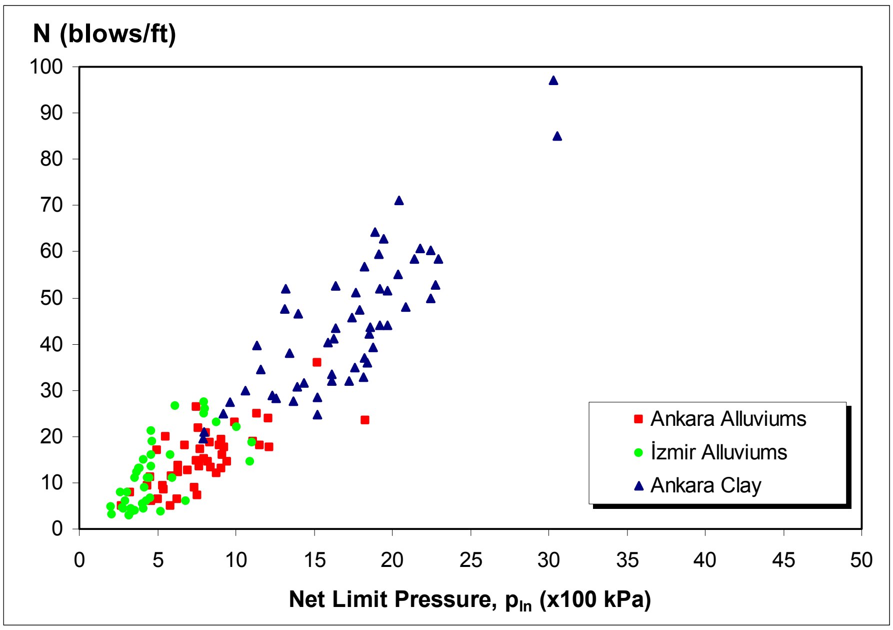


Figure 4.47 N vs p_{ln} distribution for experienced soils

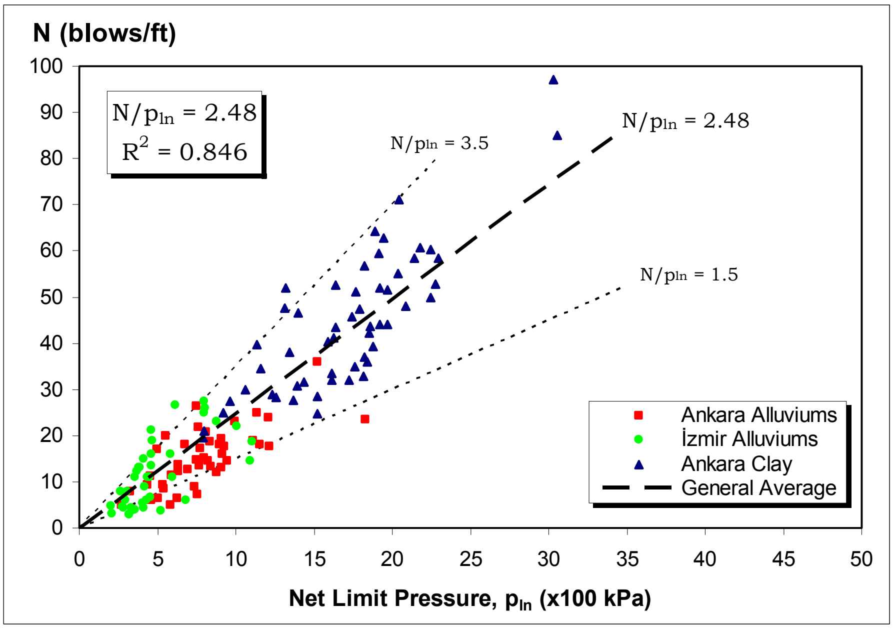


Figure 4.48 N vs p_{ln} correlation for experienced soils

The average of whole data is represented by a best line with an equation of:

$$\frac{N}{p_{ln}} = 2.48 \qquad (4.3)$$

[p_{ln} : $\frac{1}{100}$ kPa]

where the square of correlation coefficient, $R^2 = 0.846$.

The correlation and the value R^2 is somehow close to that of N&p_l correlation, since p_{ln} is a derivation of p_l in a way.

The range of N/p_{ln} could be stated as 1.5 – 3.5 where a little accumulation along the below line is likely to occur on alluvial soils, which indicates the same state that of obtained and explained in N/p_l study, that is the ratio is likely proportional to the stiffness within the given range.

Relating to N/p_{ln} recommendation from Pilot (1982), he has defined a range of 20 – 40 where E_p is in MPa. Converting the graphical units to MPa for comparison, the range comes out from this study would be 15 – 35 with an average of approximately 25. The range of ratio values then could be concluded to provide an agreement with those by Pilot (1982).

The agreement in N/p_{ln} ranges might also give an explanation to the commends on the inconsistency of N/E_p ranges from Pilot (1982) and this study. The parameter p_l, and hence p_{ln} with a slight numerical difference when compared to a large range, does not mainly depend on the interpretation technique of results, where E_p is highly sensitive on even a Poisson's ratio, υ choice. Then, agreement in p_{ln} may prove that disperancies in the ranges of N/E_p ratios would probably resulted from the evaluation of value E_p, rather than the differences in test equipment.

CHAPTER 5

CONCLUSIONS

The following statements can be presented as conclusions for the in-situ and laboratory test evaluations of cohesive soils in order to predict the related geotechnical properties.

- The undrained shear strength (c_u) values obtained from laboratory triaxial loading tests and predicted from the emprical correlation by Stroud (1974) are generally in good agreement. Stroud (1974) may overestimate the c_u value about only 10% over the laboratory measured values, which is practically acceptable.

- The prediction by Stroud (1974) to determine c_u value may sometimes results in relatively high overestimations, especially in case of very stiff to hard consistency in cohesive soils. This is in turn an underestimation by laboratory testing of samples which may rise up to 50% due to the reasons of sample disturbance (e.i. difficulties in sampling operation, release of in-situ stress, handling and preparation), differences in saturation conditions of in-situ soil and emprical correlation by Stroud (1974) which may effect the number N and soil characteristics (e.i. weakining effect of fissures). The case of laboratory underestimation is also confirmed by Stroud (1974) on the plot of factor f_1 derived for very stiff to hard intact clays.

- The coefficient of volume compressibility, m_v values obtained from laboratory tests are about 37% over the values estimated by Stroud (1974) in average, where the upper limit is nearly over 200%. An important reason for that is the saturation condition of the soil in-situ, in addition to sample disturbance. Soils experienced in this study are generally not in a fully saturated condition, except alluviums of İzmir. A dessication effect is possible for the relatively dry samples of unsaturated soils, which shows high resistance in SPT, together with high compressibility in a fully saturated condition imposed by odeometer tests.

- The interpretation of SPT result, number N for the prediction of parameters by Stroud (1974) is also possible to be partially modified for the energy ratio of general British practice and common literature as N_{60}, where it might be assumed as N_{45} for the present study. Regarding N_{60}, predictions by Stroud (1974) is likely to underestimate the c_u values at about 17% in average, whereas $c_{u(Lab)}$ values usually come out lower than predictions by Stroud (1974) in the very stiff to hard consistency range of clayey soils depending on undisturbed sample quality and fissures.

- The volume compressibility coefficent (m_v) obtained from oedometer tests shows somehow an agreement with the estimations by Stroud (1974) in average, with an overestimation of about 3% in case of the interpretation of N_{60}, that is corrected for energy ratio. But, in the range of very stiff to high clays, the amount of overestimation may rise up to 150% due to the disturbance of samples and differences in saturation conditions of soil in field and laboratory.

- The prediction of undrained shear strength, c_u from pressuremeter tests is one of the reliable methods in parameter estimations. The common result of data from this study, where $p_{ln}/c_u = \beta_{ave} = 10.2$ mostly agree with many of the experimental values proposed in literature.

- The deformation modulus obtained from pressuremeter test is approximately three times the one determined from oedometer test, as $1/m_v$. In order to evaluate the deformation characteristics, the values of m_v should certainly represent the entire soil mass in-situ, that is partially related to the saturation condition in addition to sample quality. The pressuremeter performances studied for deformation modulus evaluation are generally on semi-dry to dry soils, whereas m_v corresponds to saturated conditions of soils in the laboratory.

- The results of Standard Penetration Tests and Pressuremeter Tests on cohesive soils are possibly related to each other, although not much study is available on the subject. The ratio of $N/E_p = 0.20$ (ranges between 0.13-0.29) and $N/p_{ln} = 2.48$ (ranges between 1.5-3.5) represent clear relations and likely to agree with the few proposed correlations in the literature, where the ratio N/p_l = 2.25 (ranges between 1.4-3.2) is also a possible value that is observed in order to give a brief idea, although no proposed values are available in general.

- Correlations are valid if the soil conditions, particularly saturation, are same in both laboratory and in-situ tests.

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APPENDIX A

The general database consisting of laboratory and in-situ test results obtained from all experienced soils in this study are given in Appendix A (refer to Chapter 3).

DATA NO	PROJECT	FORMATION	BORE HOLE NO	SOIL DEFINITION	DEPTH	USCS	PI	Stroud f_1	Stroud f_2	SPT N	$c_{ustroud}$ (kPa)	c_{ulab} (kPa)	$m_{vstroud}$ $(1*10^{-4})$ m^2/kN	m_{vlab} $(1*10^{-4})$ m^2/kN	E_P (kg/cm^2)	p_l (kg/cm^2)	p_{ln} (kg/cm^2)
1	ANKARA RAILWAY	Ankara Alluviums	BMİ-4	Brown Clay	3.00	CH	33,9	4,44	0,45	16			1,382	2,008	128	9,85	9,16
2	ANKARA RAILWAY	Ankara Alluviums	BM-24	Sandy Clay	3.00	CH	42,2	4,35	0,45	26	113	103	0,864	1,879	118	10,50	9,91
3	ANKARA RAILWAY	Ankara Alluviums	BM-24	Brown Silty Clay	6.00	CL	32,3	4,47	0,46	30	134	91			99	12,00	11,38
4	ANKARA RAILWAY	Ankara Alluviums	BM-23	Sandy Clay	3.00	CH	41,3	4,35	0,45	44			0,510	2,547	212	15,10	14,25
5	ANKARA RAILWAY	Ankara Alluviums	BM-23	Sandy Clay	6.00	SC-CL				34					140	16,90	16,18
6	ANKARA RAILWAY	Ankara Alluviums	BM-21	Brown Clay	3.00	CH	56,0	4,28	0,44	15	64	150	1,510	1,140	77	6,60	6,02
7	ANKARA RAILWAY	Ankara Alluviums	BM-21	Brown Clay	6.00	CH	50,5	4,31	0,44	19	82	96	1,191	1,979	83	5,70	5,02
8	ANKARA RAILWAY	Ankara Alluviums	BM-21	Gray Silty Clay	9.00	CH	33,1	4,46	0,45	10	45	72	2,199	2,591	91	7,50	6,87
9	ANKARA RAILWAY	Ankara Alluviums	BM-21	Gray Silty Clay	13.50	CH	31,4	4,50	0,46	9	40	56	2,417	2,321	81	4,50	3,76
10	ANKARA RAILWAY	Ankara Alluviums	BM-21	Brown Sandy-Silty Clay	15.00	CL				21					149	6,40	5,30
11	ANKARA RAILWAY	Ankara Alluviums	BM-21	Brown Sandy-Silty Clay	16.50	CL	29,2	4,57	0,47	15			1,424	2,033	72	6,40	5,32
12	ANKARA RAILWAY	Ankara Alluviums	BM-21	Brown Sandy-Silty Clay	19.50	CL	26,5	4,73	0,48	14	66	76			91	5,60	4,22
13	ANKARA RAILWAY	Ankara Alluviums	BM-21	Brown Sandy-Silty Clay	22.50	SC	14,8	6,80	0,66	13			1,160	1,372			
14	ANKARA RAILWAY	Ankara Alluviums	BM-21	Brown Sandy-Silty Clay	24.00	CH	47,2	4,32	0,44	8	35	97			114	5,40	4,00
15	ANKARA RAILWAY	Ankara Alluviums	BM-18	Brown Clay	3.00	CH	61,6	4,25	0,44	23	98	61	0,986	1,220			
16	ANKARA RAILWAY	Ankara Alluviums	BM-16	Brown Clay	3.00	CH	54,8	4,29	0,44	16	69	65	1,416	2,044	83	7,30	6,74
17	ANKARA RAILWAY	Ankara Alluviums	BM-15	Brown Silty Clay	3.00	CH	66,3	4,23	0,44	9	38	25					
18	ANKARA RAILWAY	Ankara Alluviums	BM-15	Brown Silty Clay	6.00	CH	54,0	4,29	0,44	11	47	43	2,059	3,930			
19	ANKARA RAILWAY	Ankara Alluviums	BM-12	Brown Silty Clay	3.00	SC	20,4	5,36	0,54	12	64	97	1,547	2,950			
20	ANKARA RAILWAY	Ankara Alluviums	BM-11	Brown Silty Clay	3.00	CL	28,9	4,59	0,47	18	83	75	1,183	5,060			
21	ANKARA RAILWAY	Ankara Alluviums	BM-11	Brown Silty Clay	6.00	CH	32,8	4,46	0,46	11	49	60	1,995	3,570			
22	ANKARA RAILWAY	Ankara Alluviums	BM-9	Light-Brown Clay	3.00	CH	55,3	4,28	0,44	22	94	75	1,030	1,710			
23	ANKARA RAILWAY	Ankara Alluviums	BM-8	Gray-Light Brown Clay	3.00	CH	41,9	4,35	0,45	7	30	28	3,209	2,885	68	4,80	4,35
24	ANKARA RAILWAY	Ankara Alluviums	BM-8	Gray-Light Brown Clay	6.00	CH	36,8	4,39	0,45	7	31	50	3,189	3,649	81	7,10	6,47
25	ANKARA RAILWAY	Ankara Alluviums	BM-7	Brown Clay	3.00	CH	80,2	4,22	0,44	13	55	44					
26	ANKARA RAILWAY	Ankara Alluviums	BM-7	Brown Clay	6.00	CH	69,9	4,22	0,44	14	59	49					
27	ANKARA RAILWAY	Ankara Alluviums	S-125	Brown Clay	3.00	CH	54,5	4,29	0,44	13	56	53			106	9,10	8,21
28	ANKARA RAILWAY	Ankara Alluviums	S-124	Brown Clay	3.00	CH	44,0	4,34	0,44	10	43	54			86	8,30	7,36
29	ANKARA RAILWAY	Ankara Alluviums	S-118	Silty Clay	6.00	CH	41,3	4,35	0,45	18	78	125			120	12,40	11,54
30	ANKARA RAILWAY	Ankara Alluviums	S-111	Silty Clay	3.00	CH	46,2	4,33	0,44	17	74	72	1,326	2,071	112	10,10	9,27
31	ANKARA RAILWAY	Ankara Alluviums	S-110	Silty Clay	3.00	CH	48,8	4,32	0,44	8	35	63			60	5,20	4,33
32	ANKARA RAILWAY	Ankara Alluviums	S-109	Silty Clay	3.00	CH				6					67	6,60	5,85
33	ANKARA RAILWAY	Ankara Alluviums	S-108	Silty Clay	3.00	CH	61,4	4,25	0,44	14	60	70			108	8,60	7,75
34	ANKARA RAILWAY	Ankara Alluviums	S-102	Silty Clay	3.00	CL	25,6	4,78	0,49	10	48	57	2,054	2,870	57	5,50	4,53
35	ANKARA RAILWAY	Ankara Alluviums	S-102	Silty Clay	6.00	CH	35,2	4,41	0,45	11	48	55	2,025	3,090			

DATA NO	PROJECT	FORMATION	BORE HOLE NO	SOIL DEFINITION	DEPTH	USCS	PI	Stroud f_1	Stroud f_2	SPT N	$c_{ustroud}$ (kPa)	c_{ulab} (kPa)	$m_{vstroud}$ ($1*10^{-4}$) m^2 / kN	m_{vlab} ($1*10^{-4}$) m^2 / kN	E_P (kg/cm^2)	p_l (kg/cm^2)	p_{ln} (kg/cm^2)
36	ANKARA RAILWAY	Ankara Alluviums	S-101	Silty Clay	3.00	CL	20,5	5,35	0,54	5			3,720	5,089			
37	ANKARA RAILWAY	Ankara Alluviums	S-101	Silty Clay	6.00	CH	26,7	4,71	0,48	9	42	33	2,309	4,396			
38	ANKARA RAILWAY	Ankara Alluviums	S-100	Silty Clay	3.00	CL	19,6	5,51	0,55	4			4,533	4,737	49	3,30	2,72
39	ANKARA RAILWAY	Ankara Alluviums	S-99	Silty Clay	3.00	CL	51,7	4,30	0,44	25	108	90	0,905	1,284			
40	ANKARA RAILWAY	Ankara Alluviums	S-91	Brown Clay	3.00	CH	65,5	4,24	0,44	5					69	7,10	6,27
41	ANKARA RAILWAY	Ankara Alluviums	S-89	Brown Sandy Clay	3.00	SC	43,1	4,34	0,44	8			2,811	4,783	77	6,30	5,43
42	ANKARA RAILWAY	Ankara Alluviums	S-89	Brown Sandy Clay	6.00	SC	46,7	4,33	0,44	9			2,506	3,284	105	9,40	8,46
43	ANKARA RAILWAY	Ankara Alluviums	S-88	Brown Clay	3.00	CH	50,7	4,31	0,44	6	26	25			90	7,50	6,54
44	ANKARA RAILWAY	Ankara Alluviums	S-88	Brown Clay	6.00	CH	53,7	4,29	0,44	7	30	57	3,235	2,657	105	9,40	8,53
45	ANKARA RAILWAY	Ankara Alluviums	S-86	Silty Clay	3.00	CH	58,0	4,27	0,44	14	60	67	1,619	2,920			
46	ANKARA RAILWAY	Ankara Alluviums	S-86	Silty Clay	6.00	CH	31,0	4,51	0,46	12	54	53	1,808	1,217			
47	ANKARA RAILWAY	Ankara Alluviums	S-85	Silty Clay	3.00	CH	53,0	4,30	0,44	11			2,058	3,358	116	10,30	9,43
48	ANKARA RAILWAY	Ankara Alluviums	S-84	Dark Brown Clay	3.00	CH	65,3	4,24	0,44	13	55	57			100	10,00	9,11
49	ANKARA RAILWAY	Ankara Alluviums	S-83	Grayish-Brown Clay	3.00	CH	51,0	4,31	0,44	7	30	56			65	4,10	3,25
50	ANKARA RAILWAY	Ankara Alluviums	S-77	Silty Clay	3.00	CH	64,1	4,24	0,44	18	76	95	1,260	2,029	126	12,30	11,08
51	ANKARA RAILWAY	Ankara Alluviums	S-76	Brown Silty Clay	3.00	CH	61,0	4,26	0,44	23	98	74	0,986	2,450	112	15,90	14,48
52	ANKARA RAILWAY	Ankara Alluviums	S-76	Brown Silty Clay	6.00	CH	118,8	4,22	0,44	12			1,890	3,263	116	11,00	9,85
53	ANKARA RAILWAY	Ankara Alluviums	S-75	Brown Clay	3.00	CH	79,9	4,22	0,44	20	84	70			86	9,60	8,71
54	ANKARA RAILWAY	Ankara Alluviums	S-75	Brown Clay	6.00	CH	63,3	4,25	0,44	22	93	71	1,031	2,288	80	8,80	7,95
55	ANKARA RAILWAY	Ankara Alluviums	S-74	Silty Clay	3.00	CH	60,8	4,26	0,44	13	55	81	1,744	1,178	93	8,50	7,50
56	ANKARA RAILWAY	Ankara Alluviums	S-73	Brown Silty Clay	3.00	CH	53,8	4,29	0,44	18	77	89			96	10,00	9,06
57	ANKARA RAILWAY	Ankara Alluviums	S-72	Brown Silty Clay	3.00	CH	48,4	4,32	0,44	18	78	63					
58	ANKARA RAILWAY	Ankara Alluviums	S-70	Brown Silty Clay	3.00	CH	56,3			22					119	13,60	12,10
59	ANKARA RAILWAY	Ankara Alluviums	S-67	Brown Silty Clay	3.00	CH	87,2	4,22	0,44	21	89	90					
60	ANKARA RAILWAY	Ankara Alluviums	S-66	Brown Clay	1.00	CH	46,7			6					78	5,50	4,63
61	ANKARA RAILWAY	Ankara Alluviums	S-65	Brown Silty Clay	3.00	CH	54,4	4,29	0,44	11	47	52	2,059	2,011	131	8,50	7,40
62	ANKARA RAILWAY	Ankara Alluviums	S-65	Brown Silty Clay	6.00	CH	62,2	4,25	0,44	13	55	52	1,744	2,713	88	8,85	7,98
63	ANKARA RAILWAY	Ankara Alluviums	S-64	Brown Silty Clay	3.00	CH	51,7	4,30	0,44	20	86	86	1,132	1,318	134	9,85	8,93
64	ANKARA RAILWAY	Ankara Alluviums	S-60	Brown Silty Clay	6.00	SC	21,1	5,28	0,53	15			1,255	2,480			
65	ANKARA RAILWAY	Ankara Alluviums	S-60	Brown Silty Clay	9.00	CH	44,5	4,34	0,44	16	69	68					
66	ANKARA RAILWAY	Ankara Alluviums	S-39	Grayish-Brown Silty Clay	3.00	CH	66,2	4,24	0,44	12	51	99	1,890	2,776			
67	ANKARA RAILWAY	Ankara Alluviums	S-38	Grayish-Brown Silty Clay	3.00	SC	31,4			14					91	6,60	5,62
68	ANKARA RAILWAY	Ankara Alluviums	S-38	Grayish-Brown Silty Clay	6.00	CH	55,1	4,28	0,44	22			1,030	1,670	131	11,60	10,33
69	ANKARA RAILWAY	Ankara Alluviums	S-48	Grayish-Brown Silty Clay	3.00	CH	49,7			10					118	8,00	7,02
70	ANKARA RAILWAY	Ankara Alluviums	S-48	Grayish-Brown Silty Clay	6.00	CH	57,0	4,28	0,44	20			1,133	1,450	88	6,70	5,66

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71	ANKARA RAILWAY	Ankara Alluviums	S-47	Grayish-Brown Silty Clay	6.00	CH	62,1	4,25	0,44	13			1,744	2,652			
72	ANKARA RAILWAY	Ankara Alluviums	S-37	Grayish-Brown Silty Clay	3.00	CH	71,7	4,22	0,44	15	63	58					
73	ANKARA RAILWAY	Ankara Alluviums	S-36	Grayish-Brown Silty Clay	3.00	CH	41,9	4,35	0,45	9	39	44			93	6,10	5,10
74	ANKARA RAILWAY	Ankara Alluviums	S-36	Grayish-Brown Silty Clay	6.00	CH	48,5	4,32	0,44	11	47	89			110	7,40	6,67
75	ANKARA RAILWAY	Ankara Alluviums	S-46	Silty Clay	3.00	CH	35,8	4,40	0,45	5	22	22			73	6,00	5,06
76	ANKARA RAILWAY	Ankara Alluviums	S-46	Silty Clay	6.00	CH	71,5	4,22	0,44	15	63	55			130	7,50	6,42
77	ANKARA RAILWAY	Ankara Alluviums	S-46	Silty Clay	9.00	CH	35,5	4,41	0,45	15			1,486	1,840	118	11,20	10,43
78	ANKARA RAILWAY	Ankara Alluviums	S-45	Grayish-Brown Silty Clay	6.00	CH	73,3	4,22	0,44	16			1,417	0,990			
79	ANKARA RAILWAY	Ankara Alluviums	S-45	Grayish-Brown Silty Clay	9.00	CH	35,5	4,41	0,45	27	117	97					
80	ANKARA RAILWAY	Ankara Alluviums	S-34	Grayish-Brown Silty Clay	3.00	CH	52,5	4,30	0,44	12	52	52			129	9,20	8,22
81	ANKARA RAILWAY	Ankara Alluviums	S-34	Grayish-Brown Silty Clay	6.00	CH	53,0	4,30	0,44	13			1,742	1,910	141	10,20	9,35
82	ANKARA RAILWAY	Ankara Alluviums	S-44	Grayish-Brown Silty Clay	3.00	CH	39,2	4,37	0,45	14	61	63			98	7,70	6,79
83	ANKARA RAILWAY	Ankara Alluviums	S-44	Grayish-Brown Silty Clay	6.00	CH				15					115	6,60	5,79
84	ANKARA RAILWAY	Ankara Alluviums	S-43	Grayish-Brown Silty Clay	6.00	CH	52,9	4,30	0,44	11			2,058	2,732			
85	ANKARA RAILWAY	Ankara Alluviums	S-35	Grayish-Brown Silty Clay	6.00	CH	61,2	4,22	0,44	12	51	75	1,890	1,942			
86	ANKARA RAILWAY	Ankara Alluviums	S-33	Grayish-Brown Silty Clay	6.00	CH	42,2	4,35	0,45	25	109	81	0,899	2,072			
87	ANKARA RAILWAY	Ankara Alluviums	S-22	Grayish-Brown Clay	6.00	CH	51,6	4,30	0,44	24	103	170	0,943	1,647	197	19,20	18,28
88	ANKARA RAILWAY	Ankara Alluviums	S-21	Grayish-Brown Clay	6.00	CH	61,6	4,22	0,44	20	84	100	1,134	1,838			
89	ANKARA RAILWAY	Ankara Alluviums	S-21	Grayish-Brown Clay	9.00	CL	24,0	4,93	0,50	22			0,908	1,653			
90	ANKARA RAILWAY	Ankara Alluviums	S-20	Grayish-Brown Clay	3.00	CH	41,3	4,35	0,45	16	70	85					
91	ANKARA RAILWAY	Ankara Alluviums	S-20	Grayish-Brown Clay	6.00	CH	51,5	4,30	0,44	30	129	160	0,754	1,129	135	10,20	9,29
92	ANKARA RAILWAY	Ankara Alluviums	S-20	Grayish-Brown Clay	9.00	CL	32,3	4,47	0,46	15	67	93	1,458	1,689	82	7,80	6,90
93	ANKARA RAILWAY	Ankara Alluviums	S-18	Grayish-Brown Clay	6.00	CH				24					138	7,50	6,41
94	ANKARA RAILWAY	Ankara Alluviums	S-18	Grayish-Brown Clay	9.00	CL	37,2	4,39	0,45	20	88	125	1,117	1,305	155	9,90	8,86
95	ANKARA RAILWAY	Ankara Alluviums	S-17	Grayish-Brown Clay	6.00	CH	51,8	4,30	0,44	35	151	109	0,647	1,349			
96	ANKARA RAILWAY	Ankara Alluviums	S-17	Grayish-Brown Clay	9.00	CH	44,3	4,34	0,44	26	113	135	0,866	1,235			
97	ANKARA RAILWAY	Ankara Alluviums	S-16	Grayish-Brown Clay	6.00	CH	45,0	4,34	0,44	29			0,777	1,715	83	6,50	5,61
98	ANKARA RAILWAY	Ankara Alluviums	S-16	Grayish-Brown Clay	9.00	CH	48,6	4,32	0,44	27	117	126	0,837	1,988	124	10,40	9,40
99	BOSTANLI / ATAKENT	İzmir Alluviums	S-1	Dark Gray Clay	12.00	CH-ML	34,4	4,42	0,45	4	18	16					
100	BOSTANLI / ATAKENT	İzmir Alluviums	S-1	Dark Gray Clay	18.00	CH-ML	34,4	4,42	0,45	4	18	20			41	4,65	3,39
101	BOSTANLI / ATAKENT	İzmir Alluviums	S-1	Dark Gray Clay	21.00	CH	32,8	4,46	0,46	6	27	34			69	8,10	6,84
102	BOSTANLI / ATAKENT	İzmir Alluviums	S-1	Light-Brown Silty Clay	24.00	CL	26,0	4,75	0,48	22	105	115			94	12,00	10,04
103	BOSTANLI / ATAKENT	İzmir Alluviums	S-1	Light-Brown Clay	30.00	SC	19,5	5,54	0,55	14	78	59			87	13,00	10,94
104	BOSTANLI / ATAKENT	İzmir Alluviums	S-1	Light-Brown Silty Clay	36.00	SC	19,7	5,49	0,55	11	60	60					
105	BOSTANLI / ATAKENT	İzmir Alluviums	S-2	Dark Gray Clay	12.00	CH	39,0	4,37	0,45	5	22	15					

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106	BOSTANLI / ATAKENT	İzmir Alluviums	S-2	Dark Gray Clay	18.00	CL	22,6	5,10	0,52	6	31	12					
107	BOSTANLI / ATAKENT	İzmir Alluviums	S-2	Dark Gray Clay	21.00	CH	30,5	4,52	0,46	5	23	29					
108	BOSTANLI / ATAKENT	İzmir Alluviums	S-2	Gray Sandy Clay	24.00	CL	22,6	5,10	0,52	14	71	72					
109	BOSTANLI / ATAKENT	İzmir Alluviums	S-2	Light-Brown Silty Clay	27.00	CL	26,7	4,71	0,48	11	52	61					
110	BOSTANLI / ATAKENT	İzmir Alluviums	S-2	Brown Silty Clay	36.00	SC	15,3	6,59	0,64	20	132	94					
111	BOSTANLI / ATAKENT	İzmir Alluviums	S-3	Dark Gray Clay	15.00	CH	42,1	4,35	0,45	4	17	20					
112	BOSTANLI / ATAKENT	İzmir Alluviums	S-3	Dark Gray Clay	18.00	CH	48,6	4,32	0,44	6	26	20			34	2,82	2,51
113	BOSTANLI / ATAKENT	İzmir Alluviums	S-3	Dark Gray Clay	21.00	CH	37,7	4,38	0,45	7	31	25			25	1,94	1,56
114	BOSTANLI / ATAKENT	İzmir Alluviums	S-3	Light-Brown Gravelly Clay	24.00	GC	12,7			19					48	6,00	4,68
115	BOSTANLI / ATAKENT	İzmir Alluviums	S-3	Light-Brown Silty Clay	26.00	SC	19,0			9					38	4,85	4,17
116	BOSTANLI / ATAKENT	İzmir Alluviums	S-3	Light-Brown Sandy Clay	30.00	CL	29,2	4,57	0,47	29	133	97			56	7,50	6,12
117	BOSTANLI / ATAKENT	İzmir Alluviums	S-3	Light-Brown Sandy Clay	33.00	SC	24,3	4,89	0,50	40	196	56					
118	BOSTANLI / ATAKENT	İzmir Alluviums	S-3	Brown Silty Clay	36.00	SC	21,4	5,24	0,53	16			1,183	0,885			
119	BOSTANLI / ATAKENT	İzmir Alluviums	S-3	Brown Gravelly Clay	39.00	CL	28,5	4,61	0,47	44	203	190					
120	BOSTANLI / ATAKENT	İzmir Alluviums	S-4	Gray Clay	15.00	CL	27,0	4,70	0,48	4	19	25					
121	BOSTANLI / ATAKENT	İzmir Alluviums	S-4	Gray Clay	18.00	CH	40,5	4,36	0,45	3	13	20			16	2,60	1,84
122	BOSTANLI / ATAKENT	İzmir Alluviums	S-4	Brown Gravelly Clay	25.00	CL	28,0	4,64	0,47	36	167	112					
123	BOSTANLI / ATAKENT	İzmir Alluviums	S-4	Brown Silty Clay	32.00	CL	24,9	4,82	0,49	24	116	122					
124	BOSTANLI / ATAKENT	İzmir Alluviums	S-4	Brown Silty Clay	35.00	SC	24,8	4,83	0,49	13	63	59					
125	BOSTANLI / ATAKENT	İzmir Alluviums	S-4	Brown Silty Clay	36.00	SC	22,3	5,13	0,52	37	190	79					
126	BOSTANLI / ATAKENT	İzmir Alluviums	S-4	Brown Silty Clay	39.00	CL	27,3	4,68	0,48	32	150	136					
127	BOSTANLI / ATAKENT	İzmir Alluviums	S-4	Brown Silty Clay	42.00	SC	21,5	5,23	0,53	13	68	79					
128	BOSTANLI / ATAKENT	İzmir Alluviums	S-5	Dark Gray Clay	12.00	CH	56,2	4,28	0,44	3	13	20					
129	BOSTANLI / ATAKENT	İzmir Alluviums	S-5	Dark Gray Clay	15.00	CH	35,7	4,40	0,45	3	13	20					
130	BOSTANLI / ATAKENT	İzmir Alluviums	S-5	Dark Gray Clay	18.00	CH	36,1	4,40	0,45	5	22	22					
131	BOSTANLI / ATAKENT	İzmir Alluviums	S-5	Dark Gray Clay	21.00	CH	38,0			6					48	5,00	4,04
132	BOSTANLI / ATAKENT	İzmir Alluviums	S-5	Light-Brown Silty Clay	30.00	CL	26,3	4,74	0,48	19	90	123			44	5,75	4,61
133	BOSTANLI / ATAKENT	İzmir Alluviums	S-5	Light-Brown Silty Clay	33.00	SC	21,2	5,27	0,53	23	121	72					
134	BOSTANLI / ATAKENT	İzmir Alluviums	S-6	Gray Clay	12.00	CH	46,3	4,33	0,44	3	13	26					
135	BOSTANLI / ATAKENT	İzmir Alluviums	S-6	Gray Clay	15.00	CH	43,4	4,34	0,44	3	13	25					
136	BOSTANLI / ATAKENT	İzmir Alluviums	S-6	Gray Clay	18.00	CH	38,0	4,38	0,45	5	22	25			40	6,35	5,23
137	BOSTANLI / ATAKENT	İzmir Alluviums	S-6	Light-Brown Silty Clay	30.00	CL	26,4	4,73	0,48	18	85	83			53	7,25	5,83
138	BOSTANLI / ATAKENT	İzmir Alluviums	S-6	Light-Brown Silty Clay	40.00	CL	24,0	4,93	0,50	13	64	121	1,537	1,125			
139	BOSTANLI / ATAKENT	İzmir Alluviums	S-7	Gray Clay	12.00	CH	40,1	4,36	0,45	4	17	20					
140	BOSTANLI / ATAKENT	İzmir Alluviums	S-7	Gray Sandy Clay	19.00	CH	42,9	4,35	0,44	8	35	35					

DATA NO	PROJECT	FORMATION	BORE HOLE NO	SOIL DEFINITION	DEPTH	USCS	PI	Stroud f_1	Stroud f_2	SPT N	$c_{ustroud}$ (kPa)	c_{ulab} (kPa)	$m_{vstroud}$ ($1*10^{-4}$) m^2 / kN	m_{vlab} ($1*10^{-4}$) m^2 / kN	E_P (kg/cm^2)	p_l (kg/cm^2)	p_{ln} (kg/cm^2)
141	BOSTANLI / ATAKENT	İzmir Alluviums	S-7	Brown Gravelly Clay	27.00	SC	25,6	4,78	0,49	22	105	54					
142	BOSTANLI / ATAKENT	İzmir Alluviums	S-7	Brown Gravelly Clay	30.00	CL	26,8	4,71	0,48	10	47	51	2,081	1,953			
143	BOSTANLI / ATAKENT	İzmir Alluviums	S-8	Dark Gray Silty Clay	12.00	CH	43,4	4,34	0,44	4	17	25					
144	BOSTANLI / ATAKENT	İzmir Alluviums	S-8	Brown Gravelly Clay	22.00	SC	30,1	4,53	0,46	13	59	63					
145	BOSTANLI / ATAKENT	İzmir Alluviums	S-8	Brown Gravelly Clay	24.00	SC	25,0	4,81	0,49	13	63	66					
146	BOSTANLI / ATAKENT	İzmir Alluviums	S-8	Brown Gravelly Clay	26.00	SC	20,7	5,33	0,54	17	91	70	1,098	1,072			
147	BOSTANLI / ATAKENT	İzmir Alluviums	S-9	Dark Gray Clay	12.00	CL	19,0	5,66	0,56	3	17	12					
148	BOSTANLI / ATAKENT	İzmir Alluviums	S-9	Dark Gray Clay	15.00	CH	52,3	4,30	0,44	3	13	24					
149	BOSTANLI / ATAKENT	İzmir Alluviums	S-9	Light-Brown Silty Clay	21.00	SC	25,1	4,80	0,49	9	43	44					
150	BOSTANLI / ATAKENT	İzmir Alluviums	S-9	Light-Brown Gravelly Clay	33.00	SC	17,2	6,12	0,60	20			0,830	0,607			
151	BOSTANLI / ATAKENT	İzmir Alluviums	S-9	Light-Brown Gravelly Clay	39.00	SC	18,8	5,71	0,57	40			0,440	0,481			
152	BOSTANLI / ATAKENT	İzmir Alluviums	S-10	Gray Clay	12.00	CH	36,7	4,39	0,45	3	13	34					
153	BOSTANLI / ATAKENT	İzmir Alluviums	S-10	Gray Clay	18.00	CH	40,5	4,36	0,45	3	13	34					
154	BOSTANLI / ATAKENT	İzmir Alluviums	S-10	Brown Silty Clay	21.00	CH	34,4	4,42	0,45	10	44	69					
155	BOSTANLI / ATAKENT	İzmir Alluviums	S-10	Brown Silty Clay	29.00	SC	28,0	4,64	0,47	13	60	42					
156	BOSTANLI / ATAKENT	İzmir Alluviums	S-10	Light-Brown Silty Clay	32.00	CL	31,3	4,50	0,46	11	49	69	1,976	1,870			
157	BOSTANLI / ATAKENT	İzmir Alluviums	S-10	Brown Silty Clay	42.00	CL	27,6	4,66	0,48	17	79	98					
158	BOSTANLI / ATAKENT	İzmir Alluviums	S-13	Dark Gray Clay	15.00	CH	27,6	4,66	0,48	4	19	25					
159	BOSTANLI / ATAKENT	İzmir Alluviums	S-13	Dark Gray Clay	18.00	CH	36,0	4,40	0,45	4	18	20					
160	BOSTANLI / ATAKENT	İzmir Alluviums	S-13	Light-Brown Silty Clay	21.00	CH	33,0	4,46	0,46	21	94	99					
161	BOSTANLI / ATAKENT	İzmir Alluviums	S-13	Light-Brown Silty Clay	24.00	CL	23,6	4,98	0,50	15	75	64					
162	BOSTANLI / ATAKENT	İzmir Alluviums	S-13	Light-Brown Silty Clay	27.00	CL	23,6	4,98	0,50	13	65	55					
163	BOSTANLI / ATAKENT	İzmir Alluviums	S-13	Brown Sandy Clay	30.00	SC	22,2	5,15	0,52	23			0,837	1,525			
164	BOSTANLI / ATAKENT	İzmir Alluviums	S-14	Gray Clay	15.00	CH	45,6	4,33	0,44	4	17	25					
165	BOSTANLI / ATAKENT	İzmir Alluviums	S-14	Gray Clay	18.00	CH	37,3	4,39	0,45	3	13	25					
166	BOSTANLI / ATAKENT	İzmir Alluviums	S-14	Brown Silty Clay	22.00	CL	30,0	4,53	0,46	15	68	72			44	5,90	4,14
167	BOSTANLI / ATAKENT	İzmir Alluviums	S-14	Brown Silty Clay	29.00	CH	34,0	4,43	0,45	26	115	108					
168	BOSTANLI / ATAKENT	İzmir Alluviums	S-14	Brown Silty Clay	32.00	SC	25,6	4,78	0,49	18	86	41	1,141	1,601	94	10,30	8,06
169	BOSTANLI / ATAKENT	İzmir Alluviums	S-15	Gray Clay	14.00	CL	24,3	4,89	0,50	4	20	36					
170	BOSTANLI / ATAKENT	İzmir Alluviums	S-15	Gray Clay	15.00	CH	37,8	4,38	0,45	5	22	39					
171	BOSTANLI / ATAKENT	İzmir Alluviums	S-15	Gray Clay	18.00	CH	35,2	4,41	0,45	5	22	37					
172	BOSTANLI / ATAKENT	İzmir Alluviums	S-15	Gray Clay	21.00	CH	39,5			7					33	4,50	3,58
173	BOSTANLI / ATAKENT	İzmir Alluviums	S-15	Brown Silty Clay	24.00	SC	17,9			11					37	5,40	4,38
174	BOSTANLI / ATAKENT	İzmir Alluviums	S-15	Brown Gravelly Clay	29.00	SC	20,6	5,34	0,54	23	123	101			66	10,00	7,98
175	BOSTANLI / ATAKENT	İzmir Alluviums	S-15	Brown Silty Clay	32.00	CL	22,0	5,17	0,52	17	88	73					

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176	BOSTANLI / ATAKENT	İzmir Alluviums	S-15	Brown Silty Clay	35.00	CL	20,5	5,35	0,54	19			0,979	1,255			
177	BOSTANLI / ATAKENT	İzmir Alluviums	S-15	Brown Silty Clay	38.00	SC	27,7	4,66	0,48	23	107	46					
178	BOSTANLI / ATAKENT	İzmir Alluviums	S-16	Grayish-Brown Clay	21.00	CH	44,8	4,34	0,44	10	43	53					
179	BOSTANLI / ATAKENT	İzmir Alluviums	S-16	Grayish-Brown Clay	24.00	CL	23,8	4,95	0,50	15	74	48					
180	BOSTANLI / ATAKENT	İzmir Alluviums	S-16	Light-Brown Silty Clay	30.00	CL	28,9	4,59	0,47	23	106	162	0,926	1,162			
181	BOSTANLI / ATAKENT	İzmir Alluviums	S-16	Light-Brown Silty Clay	38.00	CL	22,7	5,09	0,51	27	137	129	0,720	1,002			
182	BOSTANLI / ATAKENT	İzmir Alluviums	S-17	Light-Brown Gravelly Clay	30.00	CL	31,2	4,50	0,46	28	126	91					
183	BOSTANLI / ATAKENT	İzmir Alluviums	S-17	Light-Brown Gravelly Clay	33.00	SC	16,5	6,29	0,62	18	113	69	0,900	1,379			
184	BOSTANLI / ATAKENT	İzmir Alluviums	S-17	Light-Brown Gravelly Clay	36.00	CL	15,0	6,67	0,65	15	100	62					
185	BOSTANLI / ATAKENT	İzmir Alluviums	S-18	Light-Brown Gravelly Clay	33.00	CL	25,8	4,77	0,49	22	105	93					
186	BOSTANLI / ATAKENT	İzmir Alluviums	S-18	Light-Brown Gravelly Clay	36.00	SC	22,1	5,16	0,52	15			1,280	1,399			
187	BOSTANLI / ATAKENT	İzmir Alluviums	S-22	Light-Brown Clay	40.00	CL	25,8	4,77	0,49	21	100	144					
188	BOSTANLI / ATAKENT	İzmir Alluviums	S-22	Light-Brown Clay	42.00	SC	19,1	5,64	0,56	32			0,556	0,916			
189	BOSTANLI / ATAKENT	İzmir Alluviums	S-23	Gray Clay	18.00	CL				4					25	2,90	2,38
190	BOSTANLI / ATAKENT	İzmir Alluviums	S-23	Gray Clay	21.00	CL				5					28	3,90	3,30
191	BOSTANLI / ATAKENT	İzmir Alluviums	S-23	Gray Clay	24.00	CH				6					44	4,85	4,27
192	BOSTANLI / ATAKENT	İzmir Alluviums	S-23	Brown Silty Clay	27.00	CL	24,4	4,88	0,50	10	49	57			50	5,85	4,47
193	BOSTANLI / ATAKENT	İzmir Alluviums	S-23	Brown Silty Clay	30.00	SC	24,0	4,93	0,50	10	49	51					
194	BOSTANLI / ATAKENT	İzmir Alluviums	S-23	Brown Silty Clay	33.00	SC	26,9	4,70	0,48	10	47	59					
195	BOSTANLI / ATAKENT	İzmir Alluviums	S-24	Dark Gray Silty Clay	12.00	CL	22,0	5,17	0,52	4	21	25					
196	BOSTANLI / ATAKENT	İzmir Alluviums	S-24	Gray Clay	18.00	ML				6					50	5,30	4,84
197	BOSTANLI / ATAKENT	İzmir Alluviums	S-24	Gray Clay	21.00	CH	34,4	4,42	0,45	7	31	44			44	5,50	4,54
198	BOSTANLI / ATAKENT	İzmir Alluviums	S-24	Light-Brown Silty Clay	24.00	SC	23,8	4,95	0,50	14	69	61			56	5,85	4,58
199	BOSTANLI / ATAKENT	İzmir Alluviums	S-24	Brown Gravelly Clay	27.00	CL	31,1	4,50	0,46	36	162	141					
200	BOSTANLI / ATAKENT	İzmir Alluviums	S-24	Light-Brown Silty Clay	33.00	CL	22,6	5,10	0,52	17	87	88	1,141	1,380			
201	BOSTANLI / ATAKENT	İzmir Alluviums	S-25	Gray Clay	12.00	CH	59,3	4,26	0,44	4	17	25					
202	BOSTANLI / ATAKENT	İzmir Alluviums	S-25	Gray Clay	15.00	CH	63,2	4,25	0,44	4	17	20					
203	BOSTANLI / ATAKENT	İzmir Alluviums	S-25	Gray Clay	18.00	CH	28,0	4,64	0,47	4	19	25			23	4,35	3,34
204	BOSTANLI / ATAKENT	İzmir Alluviums	S-25	Brown Silty Clay	24.00	SC	27,8	4,65	0,48	13	60	49			34	4,82	3,88
205	BOSTANLI / ATAKENT	İzmir Alluviums	S-25	Brown Silty Clay	30.00	SC	21,8	5,19	0,52	23	119	88			92	10,00	8,76
206	BOSTANLI / ATAKENT	İzmir Alluviums	S-26	Gray Clay	18.00	CH	38,4	4,38	0,45	5	22	29					
207	BOSTANLI / ATAKENT	İzmir Alluviums	S-26	Greenish Gray Clay	21.00	SC	19,5			13					48	4,75	3,83
208	BOSTANLI / ATAKENT	İzmir Alluviums	S-26	Light-Brown Silty Clay	23.00	CL	19,6			11					45	4,40	3,56
209	BOSTANLI / ATAKENT	İzmir Alluviums	S-26	Brown Gravelly Clay	27.00	CL				25					91	10,00	7,98
210	BOSTANLI / ATAKENT	İzmir Alluviums	S-26	Light-Brown Silty Clay	30.00	SC	22,9	5,06	0,51	16	81	59			52	5,85	4,61

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211	BOSTANLI / ATAKENT	İzmir Alluviums	S-26	Brown Gravelly Clay	33.00	SC	15,5	6,54	0,64	30			0,522	0,863			
212	BOSTANLI / ATAKENT	İzmir Alluviums	S-26	Brown Silty Clay	36.00	CL	21,2	5,27	0,53	15	79	95					
213	BOSTANLI / ATAKENT	İzmir Alluviums	S-27	Dark Gray Clay	12.00	CH	41,7	4,35	0,45	3	13	20					
214	BOSTANLI / ATAKENT	İzmir Alluviums	S-27	Gray Silty Clay	20.00	SC	21,9	5,18	0,52	11	57	49					
215	BOSTANLI / ATAKENT	İzmir Alluviums	S-27	Brown Silty Clay	21.00	CH	43,3	4,34	0,44	19	83	78					
216	BOSTANLI / ATAKENT	İzmir Alluviums	S-27	Brown Sandy Clay	24.00	SC	26,2	4,74	0,48	25	119	63					
217	BOSTANLI / ATAKENT	İzmir Alluviums	S-27	Brown Silty Clay	27.00	CL	23,0	5,05	0,51	20	101	98					
218	BOSTANLI / ATAKENT	İzmir Alluviums	S-27	Light-Brown Silty Clay	36.00	CL	21,1	5,28	0,53	13	69	92	1,448	2,127			
219	BOSTANLI / ATAKENT	İzmir Alluviums	S-28	Dark Gray Clay	12.00	CH	47,3	4,32	0,44	4	17	25					
220	BOSTANLI / ATAKENT	İzmir Alluviums	S-28	Dark Gray Clay	15.00	CL	24,0	4,93	0,50	3	15	18			13	2,44	2,10
221	BOSTANLI / ATAKENT	İzmir Alluviums	S-28	Dark Gray Clay	18.00	CH	46,1	4,33	0,44	9	39	37			18	2,90	2,64
222	BOSTANLI / ATAKENT	İzmir Alluviums	S-28	Light-Brown Silty Clay	24.00	SC	23,7			12					25	4,25	3,69
223	BOSTANLI / ATAKENT	İzmir Alluviums	S-28	Light-Brown Silty Clay	33.00	CL	22,6	5,10	0,52	17	87	91					
224	BOSTANLI / ATAKENT	İzmir Alluviums	S-29	Gray Clay	12.00	CH	45,4	4,33	0,44	4	17	25			20	2,85	2,49
225	BOSTANLI / ATAKENT	İzmir Alluviums	S-29	Grayish-Brown Silty Clay	21.00	SC	12,1			6					27	3,85	3,15
226	BOSTANLI / ATAKENT	İzmir Alluviums	S-29	Brown Sandy Clay	24.00	SC	30,7	4,51	0,46	8	36	36			42	3,95	3,07
227	BOSTANLI / ATAKENT	İzmir Alluviums	S-29	Brown Clay	30.00	SC	32,0			12					45	5,35	4,47
228	BOSTANLI / ATAKENT	İzmir Alluviums	S-29	Brown Clay	33.00	CL	30,2	4,53	0,46	11	50	48					
229	BOSTANLI / ATAKENT	İzmir Alluviums	S-29	Brown Clay	39.00	SC	25,7	4,77	0,49	23	110	89	0,894	1,480			
230	BOSTANLI / ATAKENT	İzmir Alluviums	S-29	Light-Brown Silty Clay	43.00	SC	27,4	4,68	0,48	25	117	85					
231	BOSTANLI / ATAKENT	İzmir Alluviums	S-29	Light-Brown Silty Clay	46.00	CL	26,0	4,75	0,48	25	119	96					
232	BOSTANLI / ATAKENT	İzmir Alluviums	S-31	Gray Clay	12.00	CH	30,2	4,53	0,46	3	14	29					
233	BOSTANLI / ATAKENT	İzmir Alluviums	S-31	Gray Clay	15.00	CH	29,5	4,56	0,47	3	14	26					
234	BOSTANLI / ATAKENT	İzmir Alluviums	S-31	Gray Clay	21.00	CL	21,0	5,29	0,53	3	16	21			34	4,25	3,19
235	BOSTANLI / ATAKENT	İzmir Alluviums	S-31	Dark Gray Clay	24.00	CL	24,0	4,93	0,50	11	54	64			54	7,60	5,94
236	BOSTANLI / ATAKENT	İzmir Alluviums	S-31	Brown Silty Clay	27.00	CL	16,5	6,29	0,62	27	170	82					
237	BOSTANLI / ATAKENT	İzmir Alluviums	S-31	Brown Silty Clay	35.00	CL	30,4	4,52	0,46	33	149	87					
238	BOSTANLI / ATAKENT	İzmir Alluviums	S-31	Brown Silty Clay	38.00	CL	28,0	4,64	0,47	33	153	112	0,639	1,543			
239	BOSTANLI / ATAKENT	İzmir Alluviums	S-32	Dark Gray Clay	14.00	CH	52,0	4,30	0,44	4	17	25					
240	BOSTANLI / ATAKENT	İzmir Alluviums	S-32	Dark Gray Clay	15.00	CL	22,1	5,16	0,52	4	21	20					
241	BOSTANLI / ATAKENT	İzmir Alluviums	S-32	Dark Gray Clay	21.00	CL	36,4	4,40	0,45	5	22	29			33	5,00	4,12
242	BOSTANLI / ATAKENT	İzmir Alluviums	S-32	Brown Gravelly Clay	30.00	CL	27,0	4,70	0,48	26	122	89	0,802	1,380	88	11,00	8,04
243	BOSTANLI / ATAKENT	İzmir Alluviums	S-32	Brown Gravelly Clay	33.00	SC	21,4	5,24	0,53	26	136	68					
244	ANKARA RAILWAY	Ankara Clay	BMD-1	Brown Clay	3.00	CH	35,8	4,40	0,45	45	198	270			193	11,50	10,50
245	ANKARA RAILWAY	Ankara Clay	BMD-1	Brown Clay	6.00	CH	41,4	4,35	0,45	55	239	157			274	16,90	15,80

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246	ANKARA RAILWAY	Ankara Clay	BMD-1	Brown Clay	12.00	CH	44,3	4,34	0,44	65	282	101	0,346	1,052	249	21,30	19,35
247	ANKARA RAILWAY	Ankara Clay	BMD-1	Brown Clay	15.00	CH	44,4	4,34	0,44	66	286	204	0,341	0,568	317	20,50	18,31
248	ANKARA RAILWAY	Ankara Clay	BMD-1	Brown Clay	18.00	CH	47,5	4,32	0,44	75	324	337	0,301	0,780	264	21,50	19,04
249	ANKARA RAILWAY	Ankara Clay	BM-25	Brown Silty Clay	2.00	CH	39,2			44					190	20,90	19,21
250	ANKARA RAILWAY	Ankara Clay	BM-24	Brown Silty Clay	9.00	CH	42,5	4,35	0,45	38	165	101			172	14,50	13,45
251	ANKARA RAILWAY	Ankara Clay	BMİ-3	Brown Clay	3.00	CH	39,6	4,36	0,45	22	96	124			134	10,40	9,61
252	ANKARA RAILWAY	Ankara Clay	BMİ-3	Brown Clay	6.00	CH	71,1	4,22	0,44	30	127	87	0,756	1,860	136	11,80	11,16
253	ANKARA RAILWAY	Ankara Clay	BMİ-3	Brown Clay	9.00	CH	68,2	4,23	0,44	51	216	85			168	13,10	11,56
254	ANKARA RAILWAY	Ankara Clay	BMİ-3	Brown Clay	12.00	CH	48,4	4,32	0,44	46	199	184			204	15,50	13,96
255	ANKARA RAILWAY	Ankara Clay	BMİ-2	Brown Clay	3.00	CH	49,9	4,31	0,44	47	203	321			195	13,40	12,22
256	ANKARA RAILWAY	Ankara Clay	BMİ-2	Brown Clay	6.00	CH	42,5	4,35	0,45	55	239	325	0,409	0,337	196	15,20	14,15
257	ANKARA RAILWAY	Ankara Clay	BM-14	Brown Clay	3.00	CH	35,3	4,41	0,45	54	238	226	0,413	0,790	235	22,00	20,35
258	ANKARA RAILWAY	Ankara Clay	BM-14	Brown Clay	5.00	CH				56					239	22,00	20,43
259	ANKARA RAILWAY	Ankara Clay	BM-14	Brown Clay	9.00	CH	67,3	4,23	0,44	27			0,840	1,422	158	14,10	13,32
260	ANKARA RAILWAY	Ankara Clay	BM-14	Brown Clay	12.00	CH	67,2	4,23	0,44	38			0,597	1,126	190	16,05	14,55
261	ANKARA RAILWAY	Ankara Clay	BM-10	Light-Brown Clay	3.00	CH	72,5	4,22	0,44	22	93	60	1,031	1,860	111	8,90	7,97
262	ANKARA RAILWAY	Ankara Clay	BM-10	Light-Brown Clay	6.00	CH	57,2			72					259	23,00	21,43
263	ANKARA RAILWAY	Ankara Clay	BM-10	Light-Brown Clay	9.00	CH	66,1	4,24	0,44	46	195	251			194	16,75	14,85
264	ANKARA RAILWAY	Ankara Clay	BM-10	Light-Brown Clay	12.00	CH	66,8			63					235	22,75	21,18
265	ANKARA RAILWAY	Ankara Clay	BM-8	Brown Clay	9.00	CH	65,9	4,24	0,44	23	97	60	0,986	0,923			
266	ANKARA RAILWAY	Ankara Clay	BM-8	Brown Clay	12.00	CH	48,2	4,32	0,44	37	160	235	0,610	1,329	154	13,60	12,47
267	ANKARA RAILWAY	Ankara Clay	BM-8	Brown Clay	15.00	CH				30					151	11,70	10,70
268	ANKARA RAILWAY	Ankara Clay	BM-6	Brown Clay	3.00	CH	48,3	4,32	0,44	32	138	127	0,706	1,020	157	12,40	10,60
269	ANKARA RAILWAY	Ankara Clay	BM-6	Brown Clay	6.00	CH	43,0	4,35	0,44	57	248	280	0,394	0,260	319	22,30	20,42
270	ANKARA RAILWAY	Ankara Clay	BM-6	Brown Clay	9.00	CH	83,2	4,22	0,44	41	173	145			190	15,20	13,55
271	ANKARA RAILWAY	Ankara Clay	BM-6	Brown Clay	12.00	CH	80,0	4,22	0,44	54	228	173	0,420	0,475	215	21,40	19,81
272	ANKARA RAILWAY	Ankara Clay	BM-6	Brown Clay	15.00	CH				34					168	15,90	14,34
273	ANKARA RAILWAY	Ankara Clay	BM-6	Brown Clay	18.00	CH	69,0	4,22	0,44	58	245	99			272	21,80	20,17
274	ANKARA RAILWAY	Ankara Clay	BM-6	Brown Clay	21.00	CH	82,0	4,22	0,44	72	304	130			289	25,90	23,37
275	ANKARA RAILWAY	Ankara Clay	BM-5	Brown Clay	3.00	CH	53,2	4,29	0,44	66	283	122	0,343	1,514	241	19,20	17,10
276	ANKARA RAILWAY	Ankara Clay	BM-5	Brown Clay	6.00	CH	67,3	4,23	0,44	63	267	161	0,360	1,623	230	23,90	21,75
277	ANKARA RAILWAY	Ankara Clay	BM-4	Brown Clay	3.00	CH	48,0	4,32	0,44	49	212	105			184	17,70	15,46
278	ANKARA RAILWAY	Ankara Clay	BM-4	Brown Clay	6.00	CH	44,6	4,34	0,44	62			0,363	1,902	227	19,15	17,51
279	ANKARA RAILWAY	Ankara Clay	BM-4	Brown Clay	9.00	CH	54,4			52					200	18,00	16,12
280	ANKARA RAILWAY	Ankara Clay	BM-2	Brown Clay	3.00	CH	69,1	4,22	0,44	27	114	59	0,840	1,912			

DATA NO	PROJECT	FORMATION	BORE HOLE NO	SOIL DEFINITION	DEPTH	USCS	PI	Stroud f_1	Stroud f_2	SPT N	$c_{ustroud}$ (kPa)	c_{ulab} (kPa)	$m_{vstroud}$ $(1*10^{-4})$ m^2/kN	m_{vlab} $(1*10^{-4})$ m^2/kN	E_P (kg/cm^2)	p_l (kg/cm^2)	p_{ln} (kg/cm^2)
281	ANKARA RAILWAY	Ankara Clay	BM-2	Brown Clay	6.00	CH	70,0	4,22	0,44	31	131	104	0,731	0,960	172	13,70	12,58
282	ANKARA RAILWAY	Ankara Clay	BM-2	Brown Clay	9.00	CH	66,2	4,24	0,44	47	199	85			191	18,65	17,09
283	ANKARA RAILWAY	Ankara Clay	BM-2	Brown Clay	12.00	CH				53					273	20,25	18,50
284	ANKARA RAILWAY	Ankara Clay	BM-2	Brown Clay	15.00	CH	56,1	4,28	0,44	54	231	384			212	19,45	17,46
285	ANKARA RAILWAY	Ankara Clay	S-125	Brown Clay	9.00	CH	79,8	4,22	0,44	32	135	195	0,709	1,640	215	16,00	14,40
286	ANKARA RAILWAY	Ankara Clay	S-125	Brown Clay	12.00	CH	81,8	4,22	0,44	49					240	19,90	18,30
287	ANKARA RAILWAY	Ankara Clay	S-124	Brown Clay	9.00	CH	75,5	4,22	0,44	40	169	138			253	18,80	17,20
288	ANKARA RAILWAY	Ankara Clay	S-124	Brown Clay	12.00	CH				34					191	17,00	15,30
289	ANKARA RAILWAY	Ankara Clay	S-123	Brown Clay	3.00	CH	59,1	4,26	0,44	31	132	145			199	18,20	16,83
290	ANKARA RAILWAY	Ankara Clay	S-123	Brown Clay	6.00	CH	68,7	4,23	0,44	30	127	198			158	16,00	14,30
291	ANKARA RAILWAY	Ankara Clay	S-123	Brown Clay	9.00	CH	82,9	4,22	0,44	40	169	113			196	19,00	17,30
292	ANKARA RAILWAY	Ankara Clay	S-123	Brown Clay	15.00	CH				35					192	18,80	17,20
293	ANKARA RAILWAY	Ankara Clay	S-123	Brown Clay	18.00	CH				50					243	18,80	17,20
294	ANKARA RAILWAY	Ankara Clay	S-123	Brown Clay	20.00	CH				55					227	20,90	19,30
295	ANKARA RAILWAY	Ankara Clay	S-122	Brown Clay	6.00	CH	58,9			30					174	18,90	17,21
296	ANKARA RAILWAY	Ankara Clay	S-122	Brown Clay	9.00	CH	67,3	4,23	0,44	45	190	169			246	20,90	19,20
297	ANKARA RAILWAY	Ankara Clay	S-122	Brown Clay	12.00	CH	46,1	4,33	0,44	46	199	323			225	19,60	18,00
298	ANKARA RAILWAY	Ankara Clay	S-122	Brown Clay	15.00	CH				55					252	22,10	20,50
299	ANKARA RAILWAY	Ankara Clay	S-121	Brown Clay	3.00	CH	41,5			64					237	23,80	22,00
300	ANKARA RAILWAY	Ankara Clay	S-121	Brown Clay	6.00	CH	68,5	4,23	0,44	43	182	153			202	19,60	17,90
301	ANKARA RAILWAY	Ankara Clay	S-121	Brown Clay	9.00	CH	71,0	4,22	0,44	44	186	123			208	20,85	19,16
302	ANKARA RAILWAY	Ankara Clay	S-121	Brown Clay	12.00	CH	84,5	4,22	0,44	58	245	191			248	25,10	23,50
303	ANKARA RAILWAY	Ankara Clay	S-121	Brown Clay	15.00	CH				55					237	24,30	22,45
304	ANKARA RAILWAY	Ankara Clay	S-120	Brown-Beige Silty Clay	3.00	CH	28,7			35					188	16,05	14,36
305	ANKARA RAILWAY	Ankara Clay	S-120	Brown-Beige Silty Clay	6.00	CH	67,0	4,23	0,44	14	59	96	1,620	1,149	124	8,80	7,91
306	ANKARA RAILWAY	Ankara Clay	S-120	Brown-Beige Silty Clay	9.00	CH				30					178	16,90	16,04
307	ANKARA RAILWAY	Ankara Clay	S-120	Brown-Beige Silty Clay	11.00	CH				34					191	17,80	16,20
308	ANKARA RAILWAY	Ankara Clay	S-120	Brown-Beige Silty Clay	14.00	CH				60					238	21,10	18,24
309	ANKARA RAILWAY	Ankara Clay	S-119	Brown-Beige Silty Clay	6.00	CH	57,5	4,27	0,44	36	154	87	0,630	3,311	192	17,90	17,08
310	ANKARA RAILWAY	Ankara Clay	S-119	Brown-Beige Silty Clay	9.00	CH	59,0	4,27	0,44	30			0,756	2,124			
311	ANKARA RAILWAY	Ankara Clay	S-119	Brown-Beige Silty Clay	12.00	CH	37,0			37					208	20,90	19,24
312	ANKARA RAILWAY	Ankara Clay	S-118	Brown-Beige Silty Clay	9.00	CH	60,6	4,26	0,44	33	141	159	0,687	1,867	173	17,10	16,05
313	ANKARA RAILWAY	Ankara Clay	S-118	Brown-Beige Silty Clay	12.00	CH				44					220	21,10	20,39
314	ANKARA RAILWAY	Ankara Clay	S-117	Brown Silty Clay	3.00	CH	53,3	4,29	0,44	56	240	184			216	22,50	19,84
315	ANKARA RAILWAY	Ankara Clay	S-117	Brown Silty Clay	6.00	CH	91,3	4,22	0,44	49			0,463	0,689	205	19,10	17,39

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316	ANKARA RAILWAY	Ankara Clay	S-116	Brown Silty Clay	3.00	CH	50,6	4,31	0,44	32	138	99			171	18,50	16,80
317	ANKARA RAILWAY	Ankara Clay	S-116	Brown Silty Clay	6.00	CH	52,1	4,30	0,44	38	163	155			202	20,10	18,40
318	ANKARA RAILWAY	Ankara Clay	S-116	Brown Silty Clay	9.00	CH	73,3	4,22	0,44	50	211	223	0,454	1,520	247	21,40	19,70
319	ANKARA RAILWAY	Ankara Clay	S-116	Brown Silty Clay	12.00	CH				82					430	31,90	30,30
320	ANKARA RAILWAY	Ankara Clay	S-116	Brown Silty Clay	15.00	CH				61					258	24,00	21,40
321	ANKARA RAILWAY	Ankara Clay	S-115	Brown Clay	3.00	CH				40					204	20,15	18,38
322	ANKARA RAILWAY	Ankara Clay	S-115	Brown Sandy Clay	6.00	CH	44,3	4,34	0,44	48			0,469	1,024	258	22,00	20,83
323	ANKARA RAILWAY	Ankara Clay	S-115	Brown Clay	9.00	CL				43					151	19,50	17,90
324	ANKARA RAILWAY	Ankara Clay	S-115	Brown Clay	12.00	CH	68,2	4,23	0,44	54	228	158			240	23,20	21,50
325	ANKARA RAILWAY	Ankara Clay	S-115	Brown Clay	18.00	CH				95					373	33,50	31,00
326	ANKARA RAILWAY	Ankara Clay	S-115	Brown Clay	20.00	CH				85					357	32,60	30,10
327	ANKARA RAILWAY	Ankara Clay	S-114	Brown Clay	3.00	CH	60,1	4,26	0,44	25	106	130			155	13,50	12,33
328	ANKARA RAILWAY	Ankara Clay	S-114	Brown Silty Clay	6.00	CH	100,9	4,22	0,44	21	89	107			125	10,55	9,23
329	ANKARA RAILWAY	Ankara Clay	S-114	Brown Silty Clay	9.00	CH	85,7	4,22	0,44	31	131	202	0,731	0,373	164	19,45	18,14
330	ANKARA RAILWAY	Ankara Clay	S-114	Brown Silty Clay	12.00	CH				38					199	21,15	19,46
331	ANKARA RAILWAY	Ankara Clay	S-114	Brown Silty Clay	15.00	CH				63					278	24,10	22,46
332	ANKARA RAILWAY	Ankara Clay	S-113	Brown Silty Clay	6.00	CH	54,0	4,29	0,44	40	172	130	0,566	1,156			
333	ANKARA RAILWAY	Ankara Clay	S-113	Brown Silty Clay	12.00	CH	57,3	4,27	0,44	54	231	224					
334	ANKARA RAILWAY	Ankara Clay	S-112	Brown Silty Clay	3.00	CH	68,2	4,23	0,44	40	169	68					
335	ANKARA RAILWAY	Ankara Clay	S-112	Brown Silty Clay	6.00	CH	57,8	4,27	0,44	36	154	139					
336	ANKARA RAILWAY	Ankara Clay	S-112	Brown Silty Clay	9.00	CH	72,8	4,22	0,44	56	236	215					
337	ANKARA RAILWAY	Ankara Clay	S-112	Brown Silty Clay	12.00	CH	51,3	4,30	0,44	61	263	230					
338	ANKARA RAILWAY	Ankara Clay	S-111	Brown-Beige Clay	9.00	CH	57,6	4,27	0,44	30	128	126	0,755	0,895	163	17,00	15,20
339	ANKARA RAILWAY	Ankara Clay	S-111	Brown-Beige Clay	12.00	CH				40					206	19,05	17,44
340	ANKARA RAILWAY	Ankara Clay	S-111	Brown-Beige Clay	15.00	CH				82					369	29,90	28,30
341	ANKARA RAILWAY	Ankara Clay	S-110	Brown Clay	6.00	CH	60,4			26					165	16,50	15,28
342	ANKARA RAILWAY	Ankara Clay	S-110	Brown Clay	9.00	CH	52,5	4,30	0,44	16			1,415	1,130	162	16,20	15,20
343	ANKARA RAILWAY	Ankara Clay	S-110	Brown Clay	12.00	CH	73,2	4,22	0,44	28	118	68	0,810	1,614	210	20,85	19,21
344	ANKARA RAILWAY	Ankara Clay	S-109	Brown Clay	6.00	CH	42,6	4,35	0,44	22	96	67			140	10,45	9,37
345	ANKARA RAILWAY	Ankara Clay	S-109	Brown Clay	9.00	CH				30					175	16,05	15,05
346	ANKARA RAILWAY	Ankara Clay	S-109	Brown Clay	12.00	CH				33					197	18,35	16,62
347	ANKARA RAILWAY	Ankara Clay	S-109	Brown Clay	15.00	CH	52,4	4,30	0,44	50	215	224			274	24,10	22,46
348	ANKARA RAILWAY	Ankara Clay	S-108	Brown Clay	6.00	CH	58,8	4,27	0,44	46	196	236			224	19,30	18,11
349	ANKARA RAILWAY	Ankara Clay	S-108	Brown Clay	9.00	CH	59,8	4,26	0,44	53	226	177			313	27,10	24,50
350	ANKARA RAILWAY	Ankara Clay	S-108	Brown Clay	12.00	CH	53,8	4,29	0,44	59	253	223			265	25,90	24,17
351	ANKARA RAILWAY	Ankara Clay	S-108	Brown Clay	15.00	CH				57					268	26,05	24,38