PREDICTION OF COMPACTION CHARACTERISTICS OF OVER-CONSOLIDATED SOILS

A THESIS SUBMITTED TO THE GRADUATE SCHOOL OF APPLIED SCIENCES OF NEAR EAST UNIVERSITY

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In Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

NICOSIA, 2016

Arhaiem H Abdrabba HUSSAIN: PREDICTION OF COMPACTION CHARACTERISTICS OF OVER-CONSOLIDATED SOILS

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ACKNOWLEDGEMENTS

First and foremost, my deepest gratitude goes to Allah Almighty, who has helped me and my family throughout these years. Then I would like to thank the staff of the Department of Civil engineering for their encouragement, guidance, and assistance throughout my years study of.

I would like to address my appreciable gratitude to my advisor Prof. Dr. Cavit Atalar for his precious assistance in all of this research work. I am thankful for his valuable encouragement, direction, advice, and helpful contribution during the preparation of this work. I am grateful to him for holding me to a higher research standard, and for requiring me to validate my research results.

I am also grateful to Prof. Dr. Ali Ünal Şorman Chair of the Civil Engineering Department, and Dean of Engineering Faculty in Near East University for providing me with useful feedback, and helping me to understand statistical analysis, thus enriching my ideas.

I am also grateful to Asst. Prof. Dr.Pınar Akpınar for supporting me all these years.

I would like to thank Dr.Anoosheh Iravanian for providing me valuable information for my research.

I would also like to thank Assoc.Prof.Dr.Huriye Bilsel, comittee chairman in Cyprus International University. I thank her greatly.

I am also indebted to the staff members of the Civil Engineering Department of NEU; Asst. Prof. Dr. Rıfat Reşatoğlu, Assoc. Prof. Dr. Gözen Elkıran, Prof. Dr. Ata Atun, Assoc. Prof. Dr. Kabir Sadeghi, Dr. Fatemeh Nouban, Ayten Altınkaya, and Mustafa Turk, for their various forms of support during my Graduate studies in the department. Thank you very much.

Most importantly, none of this would have been possible without Allah, then the love and patience of my family. My immediate family, to whom this dissertation is dedicated to, has been a constant source of love, concern, support and strength through all these years. To my mum , the spirit of my father, my brothers and my sisters, thanks for everything.

Finally, I am grateful to my wife for supporting me throughout my college years, which without her support I would not have been able to complete this work.

To my family....

ABSTRACT

The compaction of soil is regarded among the most significant engineering techniques which is generally opted in order to implement projects in the field of engineering such as roads, railways, airfields, earth dams, landfill, and foundations. The main goal of soil compaction is to advance the features in engineering aspects such as shear strength and density increases with compressibility and permeability decreases.

In this study, attempts to develop predictive models between Atterberg limit parameters gradational parameters, and compaction test parameters is used. For this purpose, 99 soil samples of North Nicosia were subjected to Atterberg limit, gradation and laboratory compaction tests. 52 samples tested using standard Proctor and 47 samples for checking the results of a new relationship. Part analysis, the software SPSS, Minitab 17, and MS Excel spreadsheet that was used in the scatter plot, correlation and regression analysis. Attempts were made to find the relationships of all parameters (OWC, MDD, LL, PL, and PI*)*.

The results of the analyses reveal that both OWC and MDD have strong correlation with the LL than the other Atterberg limits. The OWC is particularly found to be about 92% of the LL. Therefore, it can be suggested that during prediction of OWC and MDD from Atterberg limits, the LL should be used rather than other Atterberg limits. However, it shall be noted that MDD has a better correlation with OWC than the LL.

The outcome of this thesis may be applied in different civil engineering sectors, and it has been shown that these models will be useful for preliminary design of earthwork projects which involves North Nicosia soils in Cyprus such as construction of roads, earth dams, the earth fills, and other works that involve soil compaction.

Keywords: Nicosia soils; compaction; standard Proctor; stepwise regression; models

ÖZET

Zeminlerin sıkıştırılması yollar, demiryolları, havaalanları, baraj, depolama alanları ve temeller gibi mühendislik projelerinin uygulanması için tercih edilen en önemli mühendislik teknikleri arasında kabul edilir. Zeminlerin sıkıştırılmasının temel amacı kayma direnci ve yoğunluğun arttırılması ile sıkıştırılabilirlik ve geçirgenliğin azaltılmasıdır.

Bu çalışmada Atterberg limit parametreleri, dane çapı dağılımı parametreleri ve sıkıştırma parametreleri arasında öngörü modellerinin geliştirilmesi için bir girişim yapılmıştır. Bu amacı gerçekleştirmek için, Lefkoşa'nın kuzeyinden 99 zemin örneği Atterberg limitleri, dane çapı dağılımı ve sıkıştırma testlerine tabi tutulmuştur. 52 örnek standart Proktor kullanarak test edildi ve 47 örnek yeni bir ilişkinin sonuçlarını kontrol etmek için test edildi. Bölüm analizi, dağılım çizim, korelasyon ve regresyon analizinde kullanılan SPSS yazılımı, Minitab 17 ve MS Excel ile yapılmıştır. Tüm parametreler arasındaki ilişki bulunmaya çalışılmıştır (OMC, MDD, LL, PL ve PI).

Analizlerin sonuçları, hem OWC parametresinin hem de MDD parametresinin LL parametresi ile diğer Atterberg limitlerinden daha güçlü bir derecede anlamlı ilişkisi olduğunu göstermektedir. Optimum su içeriği, özellikle LL'nin yaklaşık % 92'si olarak bulunmuştur. Bu nedenle, Atterberg limitinden OMC ve MDD tahmini sırasında, LL'nin diğer Atterberg limitleri yerine kullanılması gerektiği ileri sürülebilir. Ancak MDD'nin LL'den çok OMC ile daha iyi bir anlamlı ilişkisi olduğu görülmektedir.

Bu tezin sonucu farklı inşaat mühendisliği sektörlerinde uygulanabilir ve bu modellerin, yol, baraj ve zemin dolgu yapımı gibi Kıbrıs'ta Kuzey Lefkoşa zeminlerinin kullanılabileceği hafriyat projelerinin ön tasarımı için yararlı olacağı gösterilmiştir.

Anahtar Kelimeler: Lefkoşa zeminleri; sıkıştırma; Standart Proktor; aşamalı regresyon; modeller

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LIST OF SYMBOLS AND ABBREVIATIONS

C:	Constant
CE:	Compaction energy(kN.m/m^3)
CH:	Highly plastic clays and sandy clays
CL:	Low plasticity clays, sandy or silty clays
Cu:	Uniformity coefficient
E:	Compaction energy (unknown) kJ/m^3
Ek:	Compaction energy (known) kJ/m^3
FC:	Fine content
Gs:	Specific gravity
LL:w_L:	Liquid limit
MDD:	Maximum dry density
MP:	Modified Proctor compaction
OH:	Organic silts and clays of high plasticity
OL:	Organic silts and clays of low plasticity
OWC:	Optimum water content
P:	Number of selected independent variables
PI:I_p**:**	Plasticty index
PL:w_p:	Plastic limit
R^2**:**	Coefficient of determination
SC:	Clayey sand
SEE:	Standard Error of Estimate
SM:	Silty sand
SP:	Standard Proctor compaction
SSE:	The sum of squares of the residual error
SSR:	total sum of squares of the response variable
SSY:	Total sum of squares
γ_{dmax}:	Maximum dry unit weight
γ_d:	Dry unit weight

CHAPTER 1

INTRODUCTION

1.1. Background

Soil is a substance which are not produced on purpose, so it is not formed of the necessary features in order to adapt to the system of the earth. This is why soil is modified in order to carry out constructions with the desired engineering properties. One of the techniques used to realize this without wasting a lot of money is called soil compaction. The reason why soil is compacted is to improve the shear strength and to decrease permeability and settlement. It is necessary to compact the soil in order to complete geotechnical constructions. For example, railway subgrades, airfield pavements and earth retaining structures require earth filling compaction. In general, pavement design requires the soil compacted at the laboratory and field California Bearing Ratio, CBR values (Horpibulsuk et al., 2013).

Soil compaction, which is generally used in engineering requiring projects such as railway sub-grades, earth dams, landfill, highways, airfield pavements and foundations, a significant engineering technique, aims to improve the soil engineering properties, including high density, low compressibility leading to a low settlement, a low permeability, a high shear strength and a high bearing capacity.

Soil compaction refers to the process of mechanically squeezing the particles of soil so that they have a close contact, leaving the air particles outside of the soil. In this process, the void number and size in a given soil mass will be reduced, and therefore, the density of the soil increases, and the engineering property changes significantly.

It has been stated to compact soil in the beginning of 1940s. Most of these modelling attempts included correlation equations for estimating the compaction characteristics (OWC and MDD) of soil in terms of soil index properties and grain size distribution (Davidson and Gardiner, 1949). Ramiah et al. (1970) correlated both OWC and MDD solely to LL. Jeng and Strohm (1976) correlated the standard energy Proctor (OWC and MDD) to index properties of 85 soil samples.

Blotz et al (1998) used Proctor compaction test data from 22 fine-grained soil samples to correlate OWC and MDD with LL and CE. Gurtug and Sridharan (2002 and 2004) correlated OWC and MDD of fine-grained soils compacted by various CE to PL.

Joslin (1959) conducted a research on compaction curves yielded 26 typical standard Proctor curves, called Ohio's curves in other terms, which are considered to be similar to the soil on earth. It is possible to calculate an approximate compaction curve of a specific soil by these curves by making use of a water content–bulk density data point identified by using a standard Proctor penetration needle. A model has been developed by Pandian et al. (1997) and Nagaraj et al. (2006) which allows finding out the density and water content relationship of fine-grained soils for both dry and wet sides of optimum according to LL as well as Gs. Different curves are found out by the study that were similar to the results in the study of Joslin (1959). Nevertheless, this method is only valid for standard Proctor energy compacted fine-grained soil.

Horpibulsuk et al. (2008 and 2009) promoted a phenomenological model which explains the case of the compaction curves of fine- and coarse-grained soils which change according to various CE. The model figures satisfactorily all the compaction curves. The Ohio's compaction curves for energies of 296.3, 1346.6 and 2693.3 (MP) kJ/m^3 which were changed have additionally been presented. They were found utile in the fast identification of the compaction curves trough the results of a single trial test. Pavement engineers also find it highly significant in terms of the compacted soil CBR values. Ohio's and the modified Ohio's curves would be very useful in terms of approximating the CBR values by using γ_d. However, in order to save energy and to maintain the economic balance, it is necessary to have the optimal roller pass number at the target γ_d and CBR values for the best field compaction.

Fine-grained soils exhibit considerable variance in the physical properties features with a water content variance. It would be ideal to use dry clay as a foundation for heavy loads on condition that dryness is preserved because it may transfer into swamp in wet condition (Chen, 2010). In a dry state, fine soils might shrink whereas they may expand when in a wet state. This would negatively affect the buildings as the foundation of the buildings.

Despite the stable water content, the starting condition and the changed state of the fine-grained soils would change in terms of their features. Silts differ from clays in many aspects expect their appearance, but they can be distinguished due to their reaction in water (Chen, 2010).

The interaction between coarse-grained soil particles is controlled by the forces that are applied at the particle-to-particle contacts. In contrast, clay particles are small enough that their behaviour is significantly affected by the molecular-level interactions that occur between individual particles. When examining the molecular structure of an individual clay particle, it can be observed that clay particles have a negatively charged surface. When in contact with water, positive cations (normally Na+ together with their molecules of hydration water) are attracted onto this surface (Mitchell, 1976). Adsorbed water particles have the clay surrounded hydrosphere have the soluble cations of various charges. These exchangeable cations establish the balance among the negative charges on clay because they establish a double layer in the surrounding. One effect of this diffuse double layer is that two clay particles will begin to repel each other when the double layer of each particle begins to overlap. In this way, the diffuse double layer controls both flocculation and dispersion. The smaller the clay particle size, the greater is the effect of the double layer.

1.1. Problem Definition

Compaction characteristics of soil are usually determined by conducting specified method of testing (eg. standard Proctor compaction test) in the laboratory, and the test results are utilized in the field to ensure the quality of construction for the desired purposes (Nerea, 2012). However, when the extent of construction is very large (such as construction of long roads and large embankment dams that require massive materials), number of compaction tests are to be performed. Obtaining this compaction achievement requires relatively elaborated laboratory procedures and is time consuming. Thus, it is very important to obtain the index property parameters that involve simpler, quicker, and cheaper method of testing and the compaction characteristics can be predicted satisfactorily from empirical correlations.

1.2. Significance of the Study

Correlations are essential to measure the characteristics of soils specially built for the task where the barrier by price, absence or limited time test hardware, and overall related are used as part of the preparation stage any initiative (Nigel, 2009). Several attempts have been made to obtain the OWC and MDD fine-grained soil compaction. The correlation equations for fine-grained soils relate OWC as well as MDD with index properties (Sridharan, 2004).

1.3. Aim of the Study

This study seeks to find out or determine the type of relationship between compression and Atterberg limits features over-consolidated soils in Northern Cyprus. To achieve this main goal, we need some specific aims to be a reference;

1. To build significant relationships between the characteristics of compaction and Atterberg limits of over-consolidated soils, and to develop appropriate empirical correlations among the corresponding soil parameters.

2. To examine the validity of the correlations, and to draw appropriate conclusions on the relationships of each empirical equations.

1.4. Overview of the Thesis

Chapter 1 gives details about the general introduction of soil, soil compaction, and the problem definition, the significance of the study, the aim of study, and most importantly the breakdown of this study.

Chapter 2 presents the related research work on soil compaction, properties of fine soils, and various methods of soil compaction, etc.

Chapter 3 presents the material and methodology of this study.

Chapter 4 is the section where the results and discussions were discussed in details.

Chapter 5 mentions the conclusion of the entire research as well as the thesis recommendations, suggestions, and future studies. Structure of the thesis is presented in the flow chart shown below.

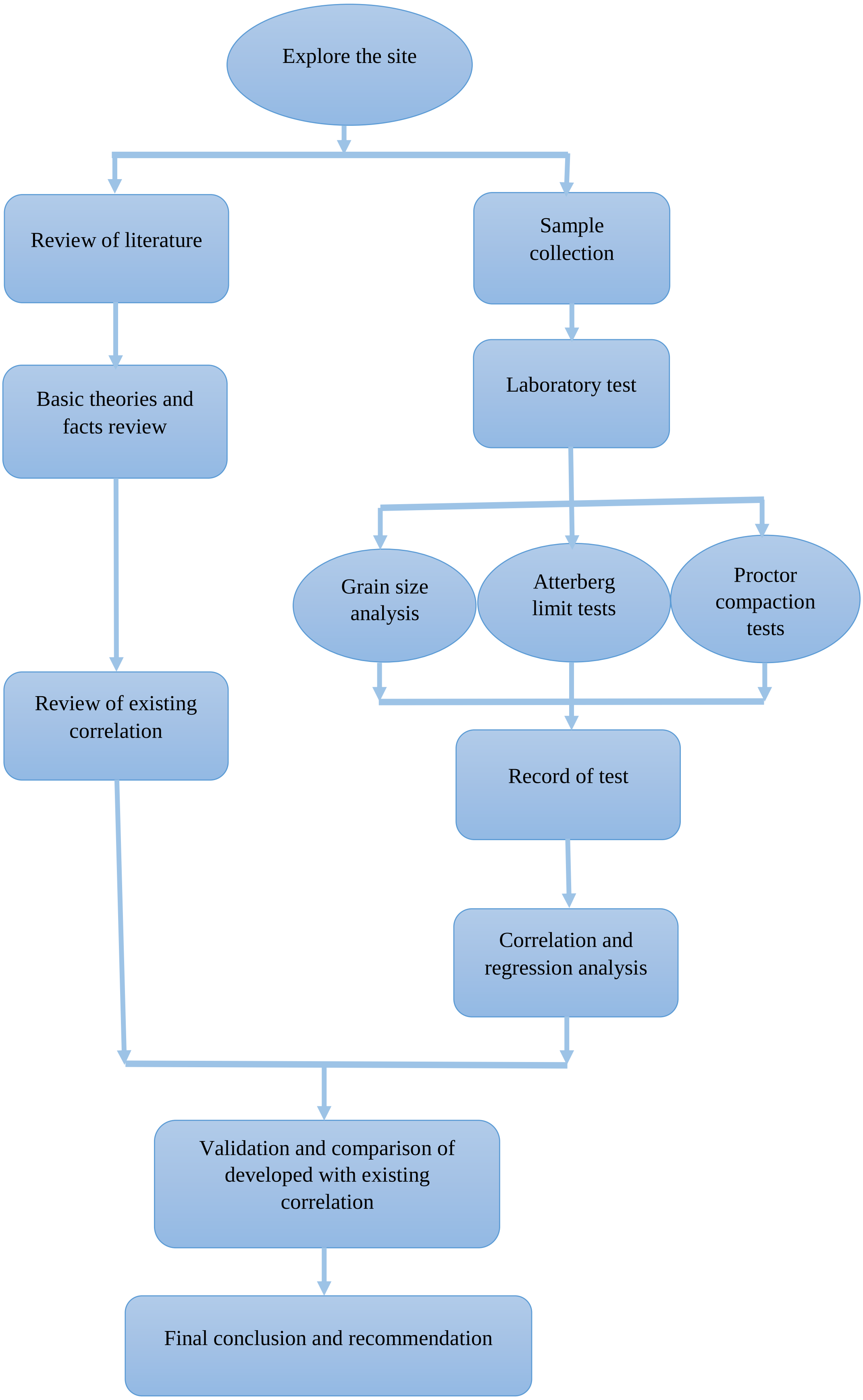


Figure 1.1: Flow chart of the study

CHAPTER 2

LITERATURE REVIEW

2.1. Properties of Soils

Fine-grained soil particles are generally characterized as finer than 0.075 mm (ASTM D422-63; Holtz and Kovacs, 1981). Fine-grained soils in the federal building where 50% or more of the particles (by dry mass) in a sample given in finer than 0.075 mm. Typically, the proportion of fine-grained soil contains a mixture of both silt and clay. The cutoff between the two comparative particle size ranges are commonly referred to as the clay fraction, it is often assumed to be either a 0.005 mm particle (ASTM D422-63) or a 0.002 mm particle (Taylor, 1948). The cutoff in particle size is somewhat arbitrary, as the behaviour of the clay particles are more appropriately associated with their plasticity (Holtz and Kovacs, 1981).

One of the earliest theories of the arrangement of soil particles in a compacted clay soil was presented by Lambe (1958). This theory, often referred to as the Gouy-Chapman theory, used to explain the different services of clay particles that are believed to exist in compacted clays. For clay soils compacted dry of work, the relatively small amount of waste water that is now a high concentration of electrolytes, which prevents the full development of the double layer of ions surrounding each particle clay. This double layer depression results in a small inter particle repulsion, which therefore leads to a tendency towards a flocculated soil structure, which has a low orientation of clay. Due to the compaction water content service approaches, the electrolyte concentration is reduced, which causes an expansion of the double layer that makes the malignant forces between particles and which also increases the degree of particle orientation. It should be noted that all the behavioural observations were based on samples of a compacted using a kneading-type compaction process at the Harvard low compaction facilities (Wilson, 1950).

Seed and Chan (1959) discussed different roles of compacted clay structure in terms of shrinkage, swelling, swollen enjoyable, stress-deformation characteristics, undrained strength, strain water-hole, and effective power characteristics. Increase water content from dry to wet work is believed to play an important role in producing an increased degree of particle orientation and clay particle dispersion, which then had a significant effect on the clay behaviour. More specifically, the samples compacted dry of work (which tended to have more flocculated) demonstrated less shrinkage, large hepatic presentations, large swell strain, and steeper stress-strain curves than the same sample of soil was compacted wet (which tended to be more scattered feature).

Mitchell (1993) stated that the extended shear strains that are induced by the compaction rammer compaction impact (e.g., Proctor compaction) is a real effect on the fabric are formed in the resulting compacted fine-grained soil. The compaction and water content are two major factors that affect the formation of the resulting compacted soil structure. If compaction too, tamper, or piston does not produce appreciable shear deformation at soil, which usually occurs when the soil is compacted dry of work, there may be a general alignment of the particles or particle groups in the horizontal plane. If the soil is compacted wet work, too, tamper, or piston tends to penetrate the soil surface and produce large shear strains, which leads to a greater alignment of the particles with the failure surface. A folded or convoluted structure may result with repeated blows to the top of the soil layer.

2.1.1. Silts

Silt, that is a granular material sized like a material between sand and clay, which is minerally similar to quartz and feldspar. Silt appears in terms of a soil which is a suspension of water in rivers, lakes, etc. At the same time, this suspended load is generally not stick and it feels like plastic. It also has a moderate specific area. In a dry state, it is floury and in a wet state, it is slippery. Silt can be seen by using a hand lens.

Silt can be produced through different processes physically by using sand-sized quartz crystals of primary rocks. This is carried out by exploiting deficiencies in their lattice (Moss and Green, 1975). The rocks and regolith are chemically weathered, followed by other types similar to frost shattering and haloclasty (Nahon and Trompette, 1982). It is in semi-arid environments that substantial quantities of silt are produced (Wright et al.,1998). Silt is also called stone dust and rock flour due to the glacial action production (Haberlah, 2007). In terms of minerals, quartz and feldspar are the main components of silt. Siltstone is the composition of sedimentary rock.

2.1.2. Clays

Clay is a fine-grained natural rock or soil material that combines one or more clay minerals with traces of metal oxides and organic matter. Clays are plastic due to their water content and become hard, brittle and non–plastic upon drying or firing (Guggenheim and Martin, 1995). Geologic clay deposits are mostly composed of phyllosilicate minerals containing variable amounts of water trapped in the mineral structure (Scarre, 2005). Depending on the soil's content in which it is found, clay can appear in various colours from white to dull gray or brown to deep orange-red.

Clays are distinguished from other fine-grained soils by differences in size and mineralogy. Silts, which are fine-grained soils that do not include clay minerals, tend to have larger particle sizes than clays. There is, however, some overlap in particle size and other physical properties, and many naturally occurring deposits include both silts and clay. The distinction between silt and clay varies by discipline. Geologists and soil scientists usually consider the separation to occur at a particle size of 2 μm (clays being finer than silts), sedimentologists often use 4–5 μm, and colloid chemists use 1 μm (Garcia-Sanchez et al., 2002).

2.1.3. Organic Matter

Soil organic matter is the fraction of the soil that consists of plant or animal tissue in various stages of breakdown (decomposition). Most of our productive agricultural soils have between 3 and 6% organic matter. Soil organic matter contributes to soil productivity in many different ways.

Organic matter in the form of partly decomposed vegetation is the primary constituent of peaty soils. Thus, we have organic silts of low plasticity and organic clays of medium to high plasticity. Organic soils are dark grey or black in color, and usually have a characteristic odor of decay. Organic clays feel spongy in the plastic range as compared to inorganic clays. Soils containing organic matter are significantly more compressible and less stable than inorganic soils and they are undesirable for engineering uses (Raymond, 1997).

2.2. Soil Compaction

In geotechnical engineering, soil compaction is the process in which a stress applied to a soil causes densification as air is displaced from the pores between the soil grains. When stress is applied that causes densification due to water (or other liquid) being displaced from between the soil grains, then consolidation, not compaction, has occurred. Normally, compaction is the result of heavy machinery compressing the soil.

Soil compaction is a vital part of the construction process. It is used for support of structural entities such as building foundations, roadways, walkways, and earth retaining structures to name a few. For a given soil type certain properties may deem it more or less desirable to perform adequately for a particular circumstance. In general, the preselected soil should have adequate strength, be relatively incompressible so that future settlement is not significant, be stable against volume change as water content or other factors vary, be durable and safe against deterioration, and possess proper permeability (McCarthy, 2007).

Determination of adequate compaction is done by determining the in-situ density of the soil and comparing it to the MDD determined by a laboratory test. The most commonly used laboratory test is called the Proctor compaction test and there are two different methods in obtaining the MDD. They are the standard Proctor compaction tests (SP) and modified Proctor compaction tests (MP); the MP is more commonly used. For small dams, the SP may still be the reference (Murthy, 2007).

There are four major groups of soil modification techniques used in construction today: mechanical, hydraulic, chemical, and confinement (Robert et al., 2000). The most common technique is mechanical modification of the soil by increasing its density with mechanical force applied using compaction equipment.

The importance of compaction as a practical means of achieving the desired strength, compressibility and permeability characteristics of fine-grained soils has been appreciated since the time as early as earth structures were built (Pandian et al., 1997).
The theory of why compaction results in a denser material and why there is a limit to the water content has been studied since Proctor first introduced his findings (Robert et al., 2000). Proctor recognized that water content affects the compaction process. He believed

the reason why a moisture-density curve “breaks over” at OWC was related to capillarity and frictional forces. He also believed that the force of the compactive effort was applied to overcoming the inter-particle friction of the clay particles. As the water content increased from dry of optimum to wet of optimum he believed that the water acted as a lubricant between the soil particles. The next compaction theory can be illustrated as: Compaction along the moisture density curve from dry to wet has four-step process (Robert et al., 2000). First, the soil particles become hydrated as water is absorbed. Second, the water begins to act as a lubricant helping to rearrange the soil particles into a denser and denser state until OWC is reached. Third, the addition of water causes the soil to swell because the soil now has excess water. Finally, the soil approaches saturation as more water is added.

Some of the studies attempted to correlate OWC and MDD to LL alone (Sivrikaya et al., 2008), and others correlated OWC and MDD to LL and PL.

2.2.1. Purpose of Soil Compaction

Compaction increases the strength characteristics of soils, which in turn increases the bearing capacity of foundations, decreases the amount of excessive settlement of structures, increases the stability of slopes of embankments. Generally, compaction is used as practical means of achieving the following characteristics of soils (Arora, 2004).

- ***Increase of shear strength*****:** The increase in density by compaction usually increases shearing resistance. This effect is highly desirable that it may allow the use of thinner pavement structure over a compacted subgrade or the use of steeper side slopes for an embankment. For the same density, the highest strengths are frequently obtained by using greater compactive efforts. Large-scale experiments have indicated that the unconfined compressive strength of clayey sand could be doubled by compaction (Alemayehu et al., 2009).
- ***Seepage and permeability reduction:*** When soil particles are forced together by compaction, both the number of voids contained in the soil mass and the size of the individual void spaces are reduced. This change in voids has an obvious effect on the movement of water through the soil. One effect is to reduce the permeability, thus reducing the seepage of water in earth dams, road embankments and water loss in reservoirs through deep percolation (Arora, 2004).

- ***Shrinkage characteristics and swelling optimization:*** Swelling characteristics is an important soil property. For expansive clay soils, the greater the density the greater the potential volume change due to swelling unless the soil is restrained. An expansive clay soil should be compacted at moisture content at which swelling will not be excessive. Although the conditions corresponding to a minimum swell and minimum shrinkage may not be exactly the same, soils generally may be compacted so that these effects are minimized (Amer et al., 2006).
- ***Compressibility and excessive settlement reduction:*** The primary advantage resulting from the compaction of soils used in embankments is that it reduces settlement that might be caused by consolidation of the soil within the body of the embankment. This is true because compaction and consolidation both bring about closer arrangement of soil particles. Densification by compaction prevents later consolidation and settlement of a structure (Alemayehu et al., 2009).

2.3. Factors Affecting Compaction Characteristics

Many researchers have identified the soil type, molding water content, amount of CE, method of compaction, and admixtures (Terzaghi, 1943) as the main parameters controlling the compaction behaviour of soils. A description of the influence of these factors on the process of compaction and on the final performance of the compacted fill is done in this section.

2.3.1. Type of Soil

The nature of a soil itself has a great effect on its response to a given compactive effort. Compaction characteristics of soils are divided in to three groups, Compaction of cohesionless soils, compaction of sandy or silty soils with moderate cohesion, and compaction of clay (Terzaghi, 1943). In general, coarse-grained soils can be compacted to higher γ_d than fine-grained soils. The amount of fines and the voids of the coarse-grained soils are about the same highest γ_d can be achieved (Arora, 2004). The well graded sand attains higher γ_d than poorly graded sand. Cohesive soils with high plasticity have, generally, low γ_d and high OWC.

2.3.2. Soil Water Content

The water content of a soil affects its γ_d. A soil with very low water content is difficult to compress into close state of particles. This results in higher void ratio (e) and hence lower γ_d for the same CE. On the other hand when the water content increases excessively, the soil grain tends to move apart and the total e continues to increase where as the γ_d falls. However, if the water content of the soil is of some intermediate specific value, the water acts as lubricant causing the soil to soften and become more workable. In this case the soil grains are close packed thus lowering the void content and increasing the γ_d (Terzaghi, 1943).

2.3.3. Compaction Energy Amount

The compactive effort is the amount of energy applied on the soil. A soil of given water content, if the amount of CE increases, the soils particles will be packed so that the γ_d increases. For a given CE, there is only one water content which gives the MDD. If the CE is increased the MDD also increases, but the OWC decreases (Alemayehu et al., 2009).

2.4 Theory of Compaction

Compaction is the process by which soil particles are packed more closely together by dynamic loading such as rolling, tamping or vibration it is achieved through the reduction of air voids with little or no change in water content of soil. In other words, compaction is the use of equipment to compress soil into smaller volume thereby increasing its γ_d and improving its engineering properties, (Khan, 2014). Compaction is achieved by reduction in the volume of air, as solid and water are virtually incompressible as shown in the figure 2.1.

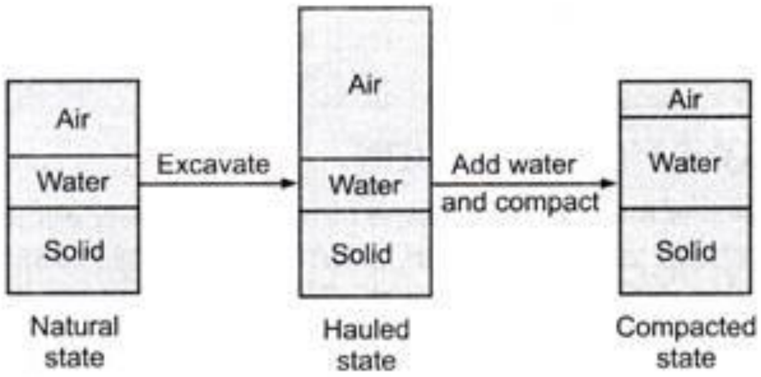


Figure 2.1: Schematic diagram showing three phase changes in the soil when it move from location to compacted fill

Compaction of soil is measured in terms of the dry unit weight achieved. γ_d is weight of soil solid per unit of total volume of the soil mass. Proctor showed that compaction depends upon water content, effect of soil type, and compaction effort. Proctor suggested laboratory method of study compaction in which soil sample is compacted in to a cylindrical mould of 1000 cm^3 by using standard compaction effort. Soil in the mould is weighted and its water content is measured (Khan, 2014).

The γ_d is computed by utilizing the accompanying expression in Equation 2.1:

$$\gamma_d = \frac{\gamma}{1+m} \tag{2.1}$$

Where m is the water content

γ is acquired by taking ratio of mass of moist soil to the volume of soil.

γ_d is expressed in gm/cm^3 or kg/m^3 or ton/m^3.

2.4.1. Necessity of Compaction

Soil compaction is one of the most important parts of earth work for soil engineering and it is required for these following reasons:

- It increases the erosion resistance which helps in maintaining the ground surface in serviceable condition

- Compaction improves the engineering properties like shear strength, density, permeability etc. of the fill.
- It reduces the amount of water that can be held in the soil by decreasing the void ratio and thus helps in maintaining the required strength.
- It reduces the chances of slope stability problems like landslides.

2.5. Laboratory Compaction Test

To attain the required MDD in the field, first appropriate tests are determined in the laboratory and this laboratory results must be confirmed in the field. The following tests are normally carried out in a laboratory (ASTM, 1998).

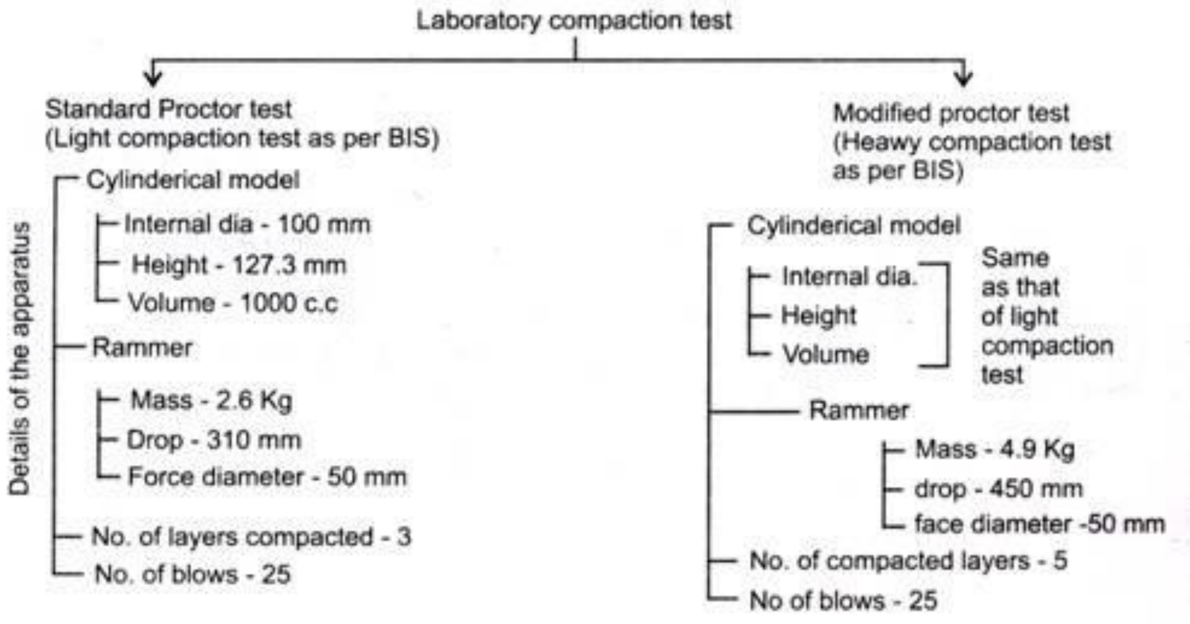


Figure 2.2: Schematic diagram showing different laboratory compaction test (Khan, 2014)

2.5.1. Standard Proctor Compaction Test (ASTM D-698)

Proctor developed this test in connection with the construction of earth fill dams in California in 1933. It gives the standard specifications for conducting the test. A soil at a selected water content is placed in three layers into a mold of 101.6mm diameter, with each layer compacted by 25 blows of a 2.5 kg hammer dropped from a height of 305 mm, subjecting the soil to a total CE of about 600 kN/m^2, so that the resulting γ_d at OWC is determined. The apparatus consists of a cylindrical metal mould of internal diameter 100 mm, 127.3 mm height and

1000 cm^3 volume. The rammer used for this test is 2.6 kg mass, 310 mm free drop and a face diameter of 50 mm. The mould is fitted with detachable base plate and a 60 mm high collar (Murthy, 2007).

2.5.2. Modified Proctor Compaction Test (ASTM D-1557)

This test method covers laboratory compaction procedures used to determine the relationship between water content and γ_d of soils, compacted in 5-layers by 101.6mm diameter mold with a 4.5kg hammer dropped from a height of 457mm producing a CE of 2,700 kN/m^2 (Murthy, 2007).

2.6. Atterberg Limit Tests

The Swedish soil scientist Albert Atterberg originally defined six ‘Limits of consistency’ to classify fine-grained soils, but in current engineering practice only three of the limits, i.e. liquid (LL), plastic (PL) and shrinkage (SL) limits are used (Dessalegn, 2003). In fact, he was able to define several limits of consistency and he has developed simple laboratory tests to define these limits.They are:

2.6.1. Liquid Limit Test

The liquid limit of a soil is the water content, expressed in percent, at which the soil changes from a liquid state to a plastic state and principally it is defined as the water content at which the soil pat cut using a standard groove closes for about a distance of 13cm (1/2 in.) at 25 blows of the LL machine (Casagrande apparatus).However, subsequent studies have indicated that the LL for all fine-grained soils corresponds to shearing resistance of about 1.7-2.0 kPa (Nagaraj, 2000). The LL of a soil highly depends upon the clay mineral present. The conventional LL test is carried out in accordance with test procedures of AASHTO T 89 or ASTM D 4318-10. A soil containing high water content is in the liquid state and it offers no shearing resistance. Currently two methods are popular in practice for the determination of the LL of fine-grained soils, they are: the percussion cup method and the cone penetration method.

2.6.2. Plastic Limit test

Plastic limit is the water content, expressed in percentage, under which the soil stops behaving as a plastic material and it begins to crumble when rolled into a thread of soil of 3.0mm diameter. The soil in the plastic state can be remolded into different shapes. When the water content has reduced, the plasticity of the soil decreases changing into semisolid state and it cracks when remolded. The range of water content from the LL to PL is known as the plasticity of the soil. Plasticity is represented by plasticity index PI which is numerically equal to the difference between the LL and the PL water contents of the soil. PI is used in the classification of fine-grained soils, through the plasticity chart. The plasticity chart is widely used to differentiate between clays and silts and further, to subgroup them according to the degree of their compressibility.

The PI is used in a number of correlations with many engineering properties such as the compression index, the coefficient of consolidation, swelling potential, the friction, the coefficient of earth pressure at rest, and the undrained shear strength etc. (Nagaraj, 2000).

2.7. Some Existing Correlations

Many scientists have made an effort to anticipate compression tests exception of a few elements, for example, soil grouping information, recording properties, grain size and conveyance.

Joslin (1958) carried out by testing a large number of soil samples. He revealed 26 different compaction curves known as Ohio compaction curves. Using these curves, the OWC, and MDD, of a soil under study can be determined by plotting the compaction curve of the soil on the Ohio curves with the help of one moisture – density point obtained from conducting a single SP test.

Ring et al (1962) also conducted a study to predict compaction test parameters from index properties, the average particle diameter, and percentage of fine and fineness modulus of soils.

Torrey (1970), in his research, made an interesting discussion on correlating compaction parameters with Atterberg limits. He remarked in this research that in order to determine a mathematical relationship between independent variables, i.e. LL, PL, and dependent variables (OWC and MDD) using the method of statistics, it is necessary to assume a frequency distribution between the variables. An assumption was made that there is normal or Gaussian distribution between the variables. A normal distribution has a very specific mathematical definition, and although, the assumption of normal distribution is reasonable, it must be pointed out there is no assurance this is valid. Additionally, it was assumed that the relationship between the variables of interest is linear. Figure 2.3a, 2.3b, 2.4a, 2.4b and to the results of the analysis carried out by Torrey (1970). It shows the linear relation between w_{opt} and w_L (Figure 2.3a) and also aims 2.3b the relation between γ_{dmax} and w_L. These models can estimate 77.6 and 76.3 percent of the variables. Also, Figure 2.4a and 2.4b shows the linear relation between the compaction test parameters with I_p. He proposed the following equation 2.2, 2.3, 2.4, and 2.5:

$$w_{opt} = 0.24\, w_L + 7.549 \tag{2.2}$$

$$\gamma_{dmax} = 0.41 w_L + 12.5704 \tag{2.3}$$

$$w_{opt} = 0.263\, I_P + 12.283 \tag{2.4}$$

$$\gamma_{dmax} = 0.449\, I_P + 11.7372 \tag{2.5}$$

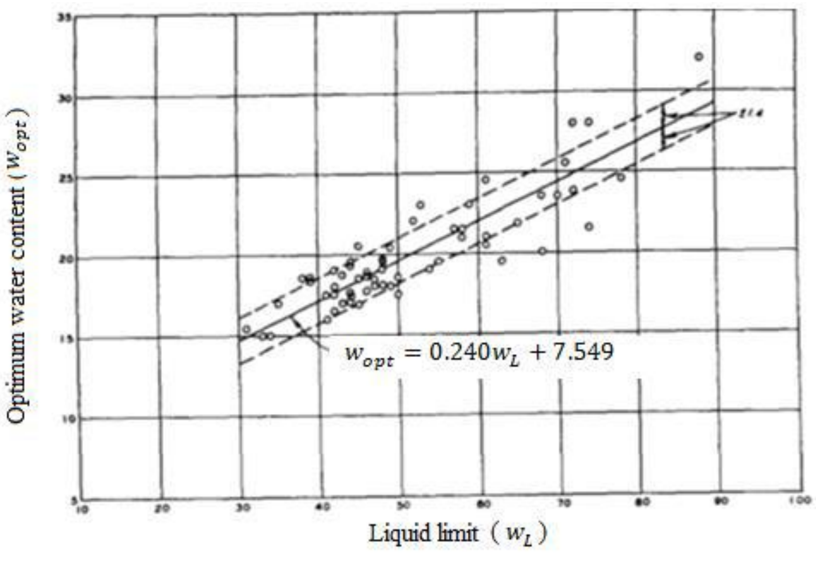


Figure 2.3 a: Plots of w_{opt} versus w_L

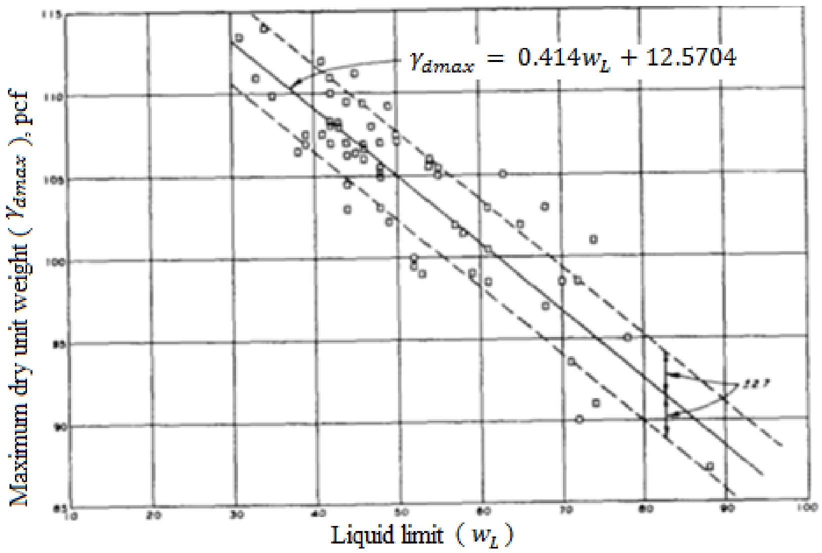


Figure 2.3 b: Plots of γ_{dmax} versus w_L

Figure 2.3 : Plots of compaction characteristics versus w_L (Torrey, 1970)

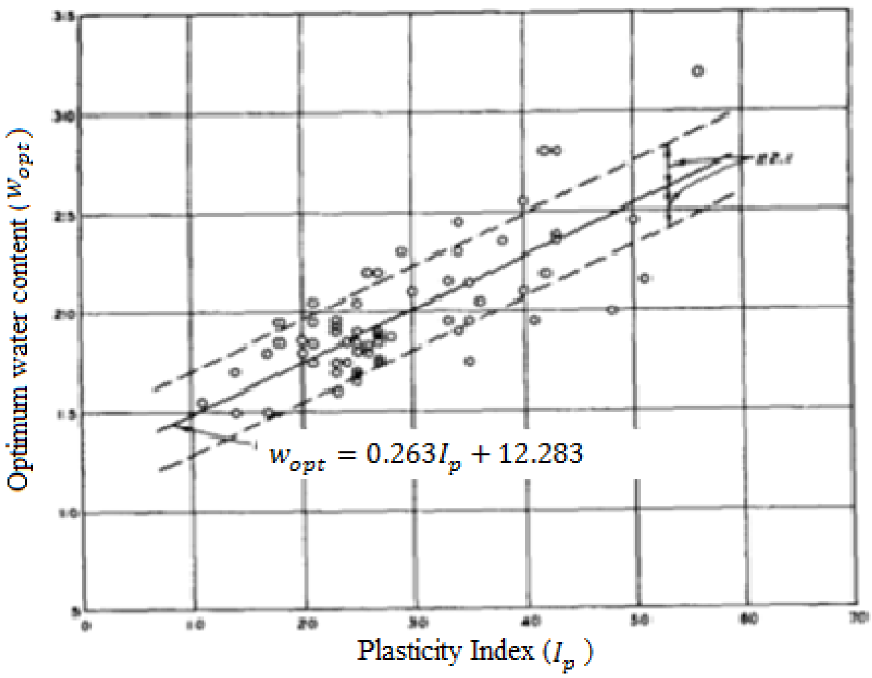


Figure 2.4 a: Plots of w_{opt} versus I_P

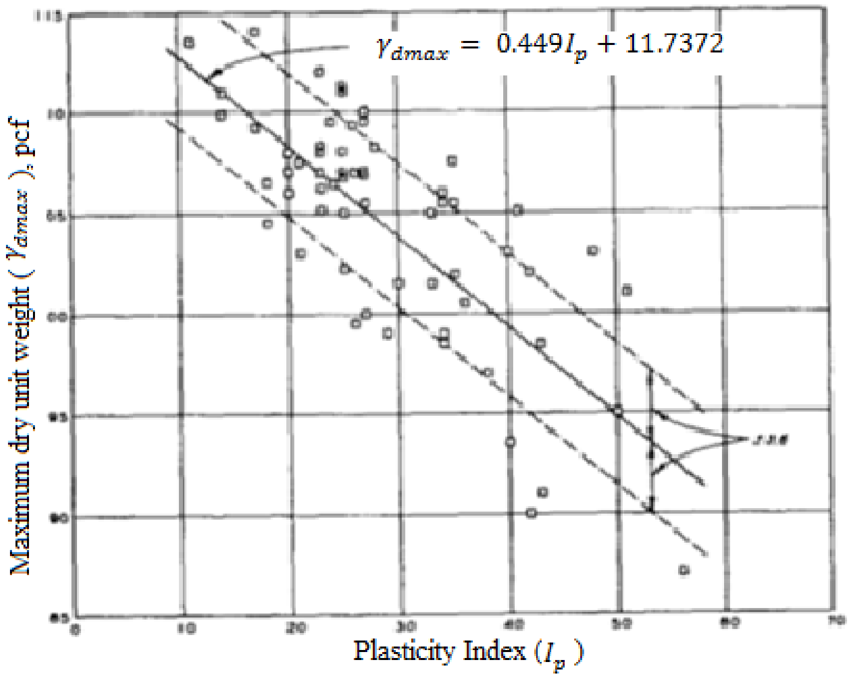


Figure 2.4 b: Plots of γ_{dmax} versus I_P

Figure 2.4 : Plots of compaction characteristics versus I_P (Torrey, 1970)

Jeng and Strohm (1976), correlated of testing soils to their Atterberg limits properties. The SP test was conducted on 85 soils with LL ranging between 17 to 88 and PL between 11 to 25. The statistical analysis approach was used in their study to correlate the compaction test parameters with Index properties.

Korfiatis and Manikopoulos (1982) using granular soils developed a parametric relationship for estimating the maximum modified Proctor dry density from parameters related to the grain size distribution curve of the tested soils such as percent fines and the mean grain size. Figure 2.5 summarizes the results of their study. The Figure is a typical grain size distribution curve of a soil in which FC is equal to the percent of fines (that is, the percent passing through the no. 200 US Sieve); and D_{50} is the mean grain size, which corresponds to 50% finer. The slope of the grain-size distribution in a lognormal plot at point A can be given by Equation 2.6:

$$D_s = \frac{1}{\text{In}D_1 - \text{In}D_2} = \frac{1}{2.303\log\frac{D_1}{D_2}} \quad (2.6)$$

The meaning of D_1 and D_2 appear in Figure 2.5. Once the magnitude is determined, the value (based on the modified Proctor test) can be estimated as using Equations 2.7 and 2.8.

$$\gamma_{dmax} = \frac{G_s\gamma_w}{\left[\frac{100\text{-}FC}{100 \text{ X } a}\right] + \left[\frac{FC}{100 \text{ X } q}\right]} \tag{2.7}$$

(for $0.5738 < D_s < 1.1346$)

$$\gamma_{dmax} = \frac{G_s\gamma_w}{\left[\frac{100\text{-}FC}{100 \text{ X } (c\text{-}ds)}\right] + \left[\frac{FC}{100 \text{ X } q}\right]} \tag{2.8}$$

(for $0.2 < D_s < 0.5738$)

Based on statistical relationships,

$a \cong 0.6682 \pm 0.0101$ $d \cong 0.3282 \pm 0.0267$

$c \cong 0.8565 \pm 0.238$ $q \cong 0.7035 \pm 0.0477$

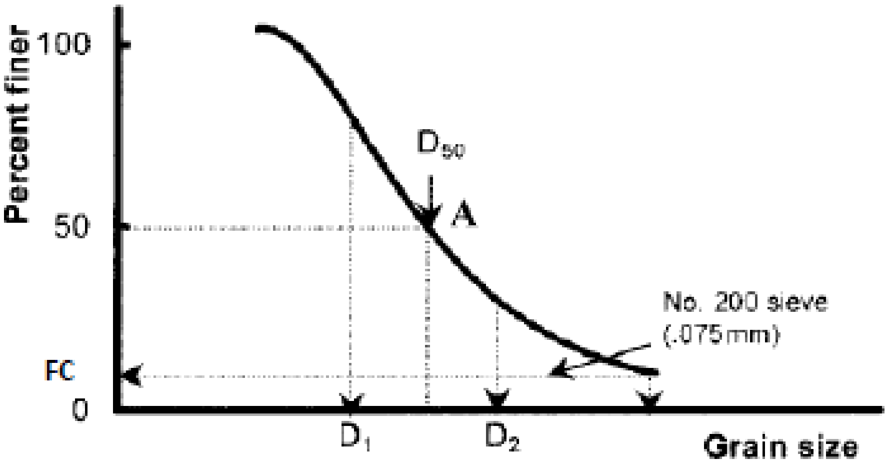


Figure 2.5: Definition of D_s in Equation 2.6 (Korfiatis and Manifopoulos, 1982)

Likewise, Wang and Huang (1984) created correlation equations for predicting OMC and MDD for manufactured soils made up of mixtures of bentonite, silt, sand and fine gravel. The backward elimination procedure (a statistical analysis approach) was used to develop models correlating OMC and MDD to Gs, PL, Cu, fineness modulus, bentonite content, and particle diameters corresponding to 10% and 50% passing (D_{10} and D_{50}).

Al-Khafaji (1993) examined the relation between the index properties and soil compaction by SP test. He used soils from Iraq and USA to carry out his test in order to develop empirical equations relating LL and PL to MDD and OWC. The equations and charts developed were done by the means of curve fitting techniques. From these, it is possible to estimate the compaction test characteristics of a SP test from index properties. The precision of these charts is considered in relation to the basic data. He also did the comparison for the compaction parameters of the Iraqi and USA soils.

The accompanying Equations 2.9 and 2.10 were from Iraqi soils;

$$MDD=2.44-0.02PL+0.008LL \tag{2.9}$$

$$OWC=0.24LL+0.63PL-3.13 \tag{2.10}$$

Likewise, for USA soils, the Equations 2.11 and 2.12 underneath were proposed;

$$MDD=2.27-0.019PL+0.003LL \tag{2.11}$$

$$OWC=0.14LL+0.54PL \tag{2.12}$$

Blotz et al. (1998) correlated γ_{dmax} and w_{opt} of clayey soil at any compactive effort, E. Compactive efforts; including standard Proctor compaction (ASTM D698-12), modified Proctor compaction (ASTM D1557-12), "Reduced Proctor" and: Super-Modified Proctor" were used to compact the soils. One variation of the method uses the w_L and one compaction curve, whereas the other uses only w_L. Linear relationship between γ_{dmax} and the logarithm of the compactive effort (log E), and between w_{opt} and log E, both of which an element of, is utilized to extrapolate to various compactive energies. They utilized 22 clayey soils to build up the observational equations and five distinctive examples were utilized to accept the models. The variation in employing and one compaction curve is slightly more accurate with percentage of errors of about ±1% for w_{opt} and ±2% for γ_{dmax}. Typical errors in variation utilizing w_L for w_{opt} and γ_{dmax} are about ±2% and ±6% respectively. The exact Equations 2.13 and 2.14 acquired were:

$$\gamma_{dmax,E} = \gamma_{dmax,k} + (2.27w_L - 0.94)\log(\frac{E}{E_k}) \quad (2.13)$$

$$w_{opt,E} = w_{opt,k} + (12.39\text{-}12.21w_L)\log(\frac{E}{E_k}) \quad (2.14)$$

Where;

E= compactive effort (unknown) kJ/m^3

Ek= compactive effort (known) kJ/m^3

Figure 2.6 demonstrates the connections amongst w_L, w_{opt} and γ_{dmax} and with modified Proctor test (MP) reduced Proctor test (RP), standard Proctor test (SP) corresponding to modified, standard, and reduced Proctor endeavors individually. They additionally watched that when gets to be bigger, w_{opt} increments and γ_{dmax} decreases. These curves can be utilized to straightforwardly estimate the optimum point for standard or modified Proctor effort if the w_L is known.

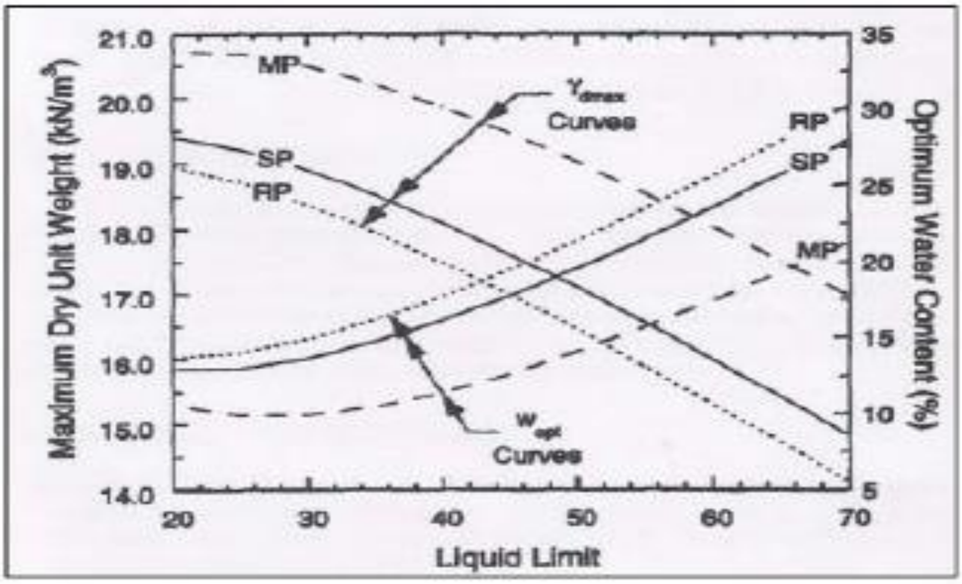


Figure 2.6: γ_{dmax} and OWC versus LL for MP, SP and RP Compactive Efforts (Blotz et al., 1998)

Omar et al. (2003) conducted studies on 311 soils in the United Arab Emirates in order to predict compaction test parameters of the granular soils from various variables (percent retained on US sieve # 200 (P#200), LL, PI and Gs of soil solids). Of these samples, 45 were gravelly soils (GP-GM, GP, GW-GM, GM and GW), 264 were sandy soils (SP-SM, SP, SW-SM, SM SW, SC-SM, and SC) and two were clayey soils with low plasticity, CL.

They used MP test on the soils and developed the Equation 2.15 and 2.16 beneath:

$$MDD = [4804574G_s - 195.55(LL^2) + 156971(R\#4)^{0.5} - 9527830]^{0.5} \quad (2.15)$$

$$In(OWC) = 1.195 * 10^{-4}(LL^2) - 1.964G_s - 6.617 * 10^{-3}(R\#4) + 7.651 \quad (2.16)$$

Where;

MDD in (kg/m^3)

Gurtug and Sridharan (2004) also studied the compaction behaviour and prediction characteristics of three cohesive soils taken from the Northern Cyprus and other two clayey minerals based on four compaction energy namely, standard Proctor compaction, modified Proctor compaction, Reduced standard Proctor and Reduced modified Proctor to develop relationship between γ_{dmax} and OWC and PL with particular reference to the CE. They proposed the Equations 2.17 and 2.18 below:

$$OWC = [1.95 - 0.38(\log CE)]PL \quad (2.17)$$

$$\gamma_{dmax} = 22.68e^{-0.0183PL} \quad (2.18)$$

Where;

CE = compaction energy (kN.m/m^3)

Sridharan and Nagaraj (2005) conducted a study of five pairs of soils with nearly the same LL but different PI among the pair and made an attempt to predict OWC and MDD from PL of the soils. They developed the following Equations 2.19 and 2.20:

$$OWC = 0.92PL \quad (2.19)$$

$$MDD = 0.23(93.3 - PL) \quad (2.20)$$

They presumed that *OWC* is almost equivalent as far as possible.

Sivrikaya et al. (2008) correlated MDD and OWC of 60 fine-grained soils from Turkey and other data from the literature using SP and MP test with a PL based on CE. They developed the following Equations 2.21 and 2.22, which are similar to what Gurtug and Sridharan (2004) found in their study.

$$OWC=K*PL \tag{2.21}$$

$$MDD = L - M*OWC \tag{2.22}$$

Where;

K = 1.99 – 0.165InE

L = 14.34 – 0.195 InE

M = -0.19 + 0.073 InE

E in kJ/m^3

MDD in kN/m^3

Therefore, at any compactive effort, OMC can be anticipated from PL and the anticipated OMC can be utilized to gauge γ_{dmax}.

Matteo et al. (2009) analyzed the after effects of 71 fine-grained soils and gave the following correlation Equations 2.23 and 2.24 for OMC and γ_{dmax} for MP tests (E= 2700 kN-m/m^3)

$$OMC= -0.86(LL)+3.04\left(\frac{PL}{G_s}\right)+2.2 \tag{2.23}$$

$$\gamma_{dmax} =40.316(OMC^{-0.295})(PI^{0.032})-2.4 \tag{2.24}$$

Where,

γ_{dmax} in kN/m^3

Gurtug (2009) used three clayey soils from Northern Cyprus and montmorillonitic clay to develop a one point method of obtaining compaction curves from a family of compaction curves. This is a simplified method in which the compaction characteristics of clayey soils can be obtained.

Ugbe (2012) studied the lateritic soils in Western Niger Delta, Nigeria and he developed the Equations 2.25 and 2.26 underneath utilizing 152 soil samples.

$$MDD = 15.665 * G_s + 1.52 * LL - 4.313 * FC + 2011.960 \qquad (2.25)$$

$$OWC = 0.129 * FC + 0.019 * LL - 1.4233 * Gs + 11.399 \qquad (2.26)$$

Where;

G_s = Specific Gravity

FC = Fine Content (%)

LL= Liquid limit (%)

Mujtaba et al. (2013) did laboratory Proctor compaction tests on 110 sandy soil tests (SM, SP, SP-SM, SW, SW-SM). In view of the tests outcomes, the following correlation Equations 2.27 and 2.28 were proposed for OWC and γ_{dmax}.

$$\log(OWC) = 1.67 - 0.193 \log(C_u) - 0.153 \log(E) \qquad (2.27)$$

$$\gamma_{dmax} = 4.49 * \log(C_u) + 1.51 * \log(E) + 10.2 \qquad (2.28)$$

Where;

E=compaction energy (kN.m/m^3)

γ_{dmax} in $\left(kN/m^3\right)$

OWC in %

Sivrikaya et al. (2013) used Genetic Expression Programming (GEP) and Multi Linear Regression (MLR) on eighty-six coarse-grained soils with fines content in Turkey to develop the predictive equation for the determination of the compaction test characteristics. He conducted standard and modified Proctor compaction tests on these soils.

Most recently, Jyothirmayi et al. (2015) used nine types of fine-grained soils like black cotton soil, red clay, china clay, marine clay, silty clay etc. which were taken from different parts of Telengana and Andhra Pradeshin, India to propose a correlation 2.29 utilizing PL in order to determine the compaction characteristics namely, OWC of these soils.

$$OWC= 12.001e^{0.0181PL} \qquad R^2=0.84 \tag{2.29}$$

CHAPTER 3

MATERIALS AND METHODS

3.1. Area of Study and Soil Sampling

The samples used to collect basic data for this work are taken from Northern Cyprus Near East University. The northern part of Nicosia district covers 7.0km east-west and 2.0-4.0km directions from north to south. For political and geographical location of the town spread to the north and west-northwest. Northern Nicosia is almost flat lying around 100-150m above sea level. The northern part of the study area reaches up to about 180m above of the sea level. Ninety nine (99) samples collected from different sampling places of the sampling location. The samples were taken from a depth of 1.00m to 3.00m below ground surface.

Figure 3.1: Near East university (Google earth images of Cyprus, 35 13 38.59" N 33 19 15.86" E, 519 ft)

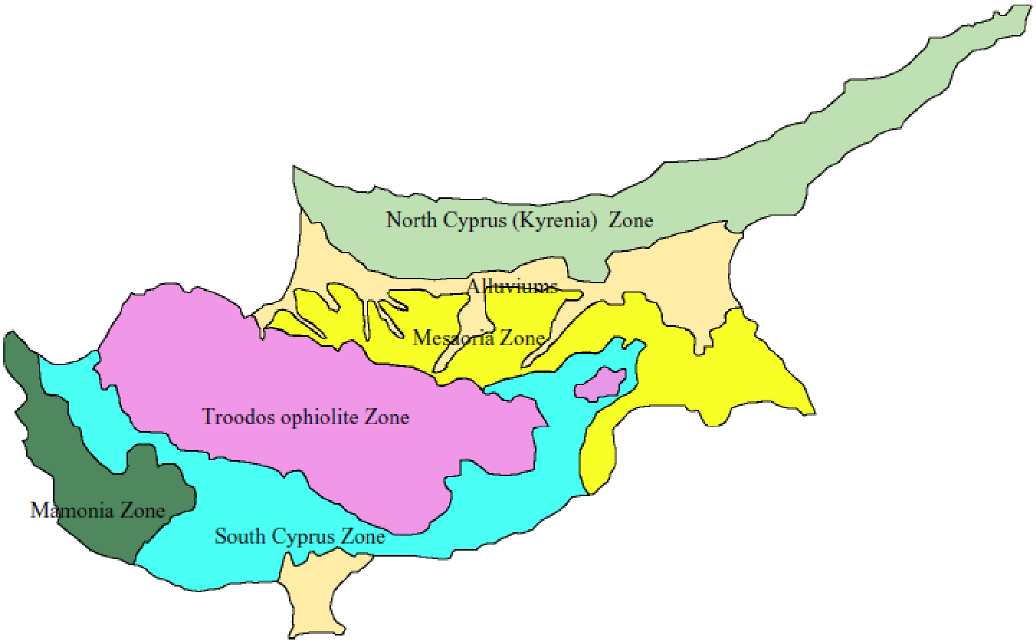


Figure 3.2: Geological Map of Cyprus (Atalar and Kilic, 2006)

3.2. Visual Soils Identification in the Field

Field tests done by ASTM D-2488 "Standard Practice for Identification and Description of soils". The image field and description of the soil depending on the size and distribution of coarse-grained particles and maintain fine-grained particles. The first step has been used as part of the same soil under the visual-manual approach is to figure out if the soil is fine grained or coarse-grained soil testing by visually observing the soil sample to be taken.

3.3. Sampling Methods and Sample Preservation

Clear and precise information are required to portray the soil profile and test areas. Test pits were unearthed utilizing hand devices with plan area of 1.5m by 1.5m and delegate disturbed examples were taken. The study samples had been handled and preserved to prevent contamination with other materials and to guarantee that the in situ soil conditions are saved. Efforts are made to take the test should be illustrative of the soil at the depth where the specimen was taken. The saving of transport and the examples were made by D-4220-95 (Standard Practice for preserving and board test).

3.4. Grain Size Distribution

In this specific study, two types of testing were used: sieve analysis and hydrometer as follows:

- ***Sieve Analysis Test***

Mechanical sieve analyzes were made on each sample ASTM D6913-04 as grain size distribution determination. sieve analysis was conducted using U.S. sieve size # 40, # 60, # 100, and # 200 A sample of soil is dried in the oven at a temperature of 105ºC–110ºC overnight. The sample was allowed to cool and ideal weight is taken. The sample is placed in the nested sieves are arranged in order to reduce the sieve with a hole on top followed by the others. Subsequently, the mass retained on each sieve.

- ***Hydrometer Test***

The test was done according to standard ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils;

- Soil samples were from pan bottom of fine sieve set, set in a beaker, and 125ml of dispersing agent (sodium Hexametaphosphate (40 g / L)) was added and the mixture solution was stirred until it is completely wet, and finally the soil is allowed to soak for about 10 minutes.

The clay soil was transferred into a mixer by adding more distilled water, if necessary, at least until mixing cup half full. Then mix the solution for a period of two minutes.

- Just the clay was transferred into the soil sedimentation cylinder empty. Add distilled water up to the mark.
- The open end of the cylinder is coated with a stopper and was secured with the palm of my hand. Then I turned the cylinder upside down and back upright for a period of one minute about 30 times.
- Cylinder down laid down and recorded the time. The lid is removed from the cylinder. Upon elapsed time of one minute and forty seconds, very slowly and carefully insert the hydrometer for the first reading.
- The reading is taken by observing the top of the meniscus formed by the suspension and stem hydrometer. The hydrometer will be moving slowly and put back into the

cylinder control. Very light spin in a control cylinder to remove any particles that may be stuck.

- The hydrometer readings is taken after elapsed time of 2 to 5, 8, 15, 30, 60 minutes and 24 hours.

The range of grain size distribution is presented at Figure 3.3.

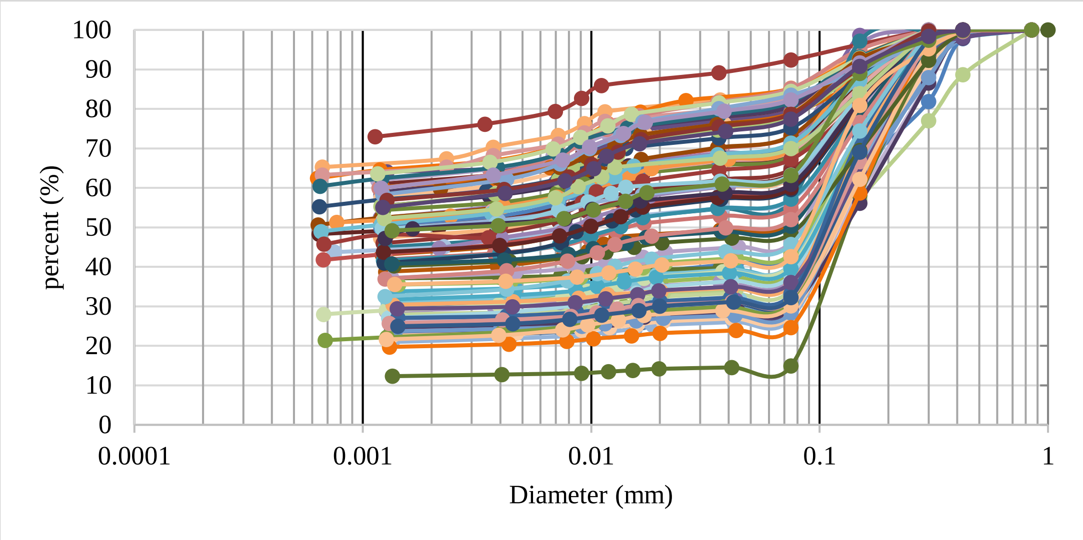


Figure 3.3: Grain size curves for all samples

3.5. Atterberg Limits

The Atterberg limits (LL and PL) are determined at each of the ninety nine samples using distilled water as the wetting agent. The experiment was performed using ASTM D4318-98 (Standard Test Method for LL, PL of soils).

Approximately 200 grams of soil needs to pass No.40 (0.425mm) sieve to complete Atterberg limits test. Water is added to the soil samples and was covered and left for 16 hours. About 20 grams reserved for determining PL and the remainder was used for determining the LL.

3.6. Compaction

Standard Proctor tests conducted on soil samples manually. It was performed on 52 samples of soil. The testing procedure ASTM D698-98 is summarized as follows:

Soil water content chosen was placed in three layers into a mold of dimensions given, with each layer compacted by 25 blows of 24.4kN rammer dropped from a distance of 305mm, subjecting the soil to a total of about 600 kN of compactive effort/m^2. The resulting MDD was determined. The procedure is repeated for a sufficient number of water content to establish a relationship between the dry density and water content.

The compression curves of soil samples for testing SP are shown in Figure 3.4.

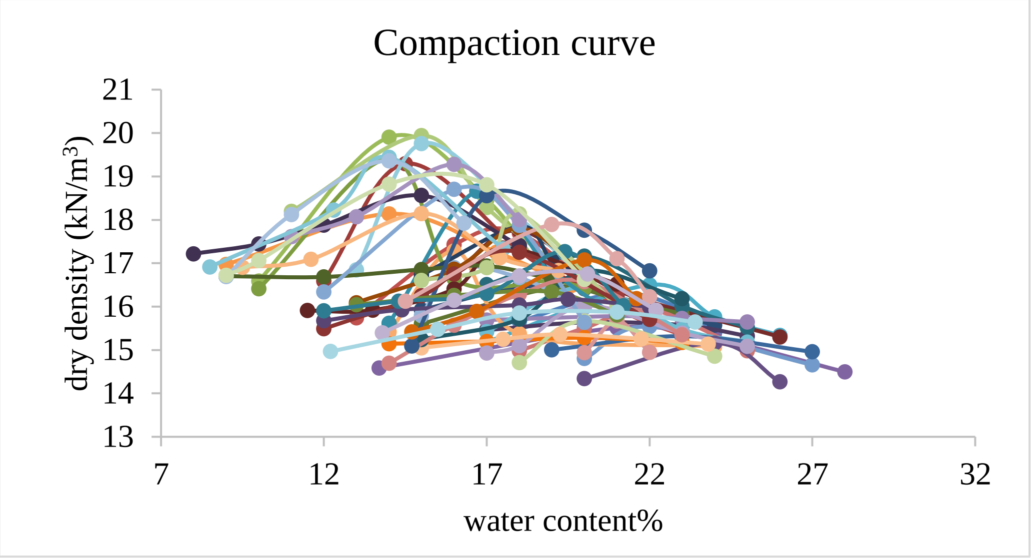


Figure 3.4: SP curves for the soil samples

Consequently, a compilation of the laboratory test results for the soil samples for the SP tests results is shown in Table 3.2. Soils samples taken for the regression analysis for SP is 45. With respect to validation of the regression models, 7 soil samples not seen by the model were used to verify the model See Table 3.4.

Table 3.1: Laboratory test results for regression analysis of Atterberg test

Test no	sand%	silt%	clay%	Atterberg limits test			water content%	soil type
				LL%	PL%	PI%		
1	57.15	12.85	30	45.8	22.6	23.2	7.45	SC
4	31.3	10.7	58	71	14.3	56.7	8.65	CH
5	59.72	11.28	29	40	21.3	18.7	5.1	SC
6	29.14	18.86	52	65.9	23.8	42.1	8.93	CH
7	71.25	8.75	20	35.5	18.6	16.9	5.22	SC
8	20.16	17.84	62	71.4	32.5	38.9	13.81	OH
9	59.84	14.16	26	48	21.9	26.1	4.78	SC
10	19.17	20.83	60	77.6	33.3	44.3	12.27	OH
11	70.55	7.45	22	38.2	19.6	18.6	6.93	SC
12	39.9	16.1	44	58.9	23.8	35.1	11.02	CH
13	70.03	9.97	20	33.4	17.3	16.1	4.63	SC
14	31.62	15.38	53	69	28.3	40.7	15.18	OH
15	70.35	9.65	20	36.1	18.9	17.2	3.5	SC
16	28.7	17.3	54	72	33.3	38.7	14.27	OH
17	68.62	9.38	22	39.5	19.8	19.7	8.87	SC
21	67.26	10.74	22	41.9	22.8	19.1	1.8	SC
22	20.37	15.63	64	71	18.3	52.7	11.73	CH
23	68.09	5.91	26	40	25.5	14.5	3.31	SM
24	14.92	20.08	65	78.9	30	48.9	16.77	OH
25	73.11	8.89	18	36	20.8	15.2	13.9	SC
26	17.32	17.68	65	81.8	30.3	51.5	4.57	OH
27	65.66	6.34	28	43.4	22.3	21.1	11.33	SC
28	17.68	18.32	64	79.5	26.5	53	15	OH
31	69.81	6.19	24	39.6	22	17.6	9.22	SC
32	30.84	15.16	54	64.2	20	44.2	13.57	CL
33	70.09	4.91	25	36.4	18.9	17.5	9.04	SM
34	29.56	12.44	58	66.7	26.6	40.1	11.21	OL
35	69.1	6.9	24	41.6	23.9	17.7	4.51	SM
36	26.83	19.17	54	74.5	28.57	45.93	10.82	OH
37	69.14	8.86	22	34.6	15.2	19.4	7.98	SC
38	14.79	23.21	62	81	37.5	43.5	14.47	OH
39	64.84	6.16	29	38	19.7	18.3	6.75	SC
40	63.34	8.66	28	46	23.8	22.2	7.54	SM
43	68.41	8.59	23	37.6	17.3	20.3	1.38	SC

Table 3.1: Continued

Test no	sand%	silt%	clay%	Atterberg limits test			water content%	soil type
				LL%	PL%	PI%		
44	35.29	16.71	48	56	28.3	27.7	3.89	OH
46	71.08	9.92	19	36.7	20	16.7	28.37	SC
54	72.26	7.74	20	34.7	18	16.7	1.17	SC
57	56.52	8.48	35	45.9	20	25.9	5.46	SC
58	67.27	12.73	20	37.7	21	16.7	1.45	SC
59	60.43	8.57	31	42	16.7	25.3	4.43	SC
60	30.31	16.69	53	59.9	33.3	26.6	6.95	OH
63	69.3	4.7	26	37	20.2	16.8	1.39	SC
64	70.53	5.47	24	36.5	15.5	21	0.97	SC
75	68.82	10.18	21	39.5	20.3	19.2	4.15	SC
84	69.83	9.17	21	39.2	19.6	19.6	7.08	SC
91	16.5	23.5	60	61.2	30.4	30.8	8.55	OH
96	57.4	10.6	32	44.8	21.7	23.1	6.08	SC

Table 3.2: Laboratory test results for regression analysis of SP test

Test no	sand%	silt%	clay%	LL%	PL%	PI%	OWC%	MDD (kN/m^3)	soil type
2	27.03	16.97	56	68.3	26.7	41.6	21.5	16.35	CH
3	61.34	8.66	30	44.3	23.6	20.7	17	17.92	SC
18	14.71	17.29	68	82.5	31.2	51.3	22.8	15.59	CH
19	71.28	10.72	18	34.6	15.79	18.81	14	19.98	SC
20	7.61	14.39	78	87.5	32.3	55.2	24	15.17	CH
29	67.37	6.63	26	40	23.5	16.5	14.8	19.3	SC
30	21.05	18.95	60	75.5	31	44.5	23	15.31	CH
41	59.3	9.7	31	45	23.81	21.19	17.2	18.145	SC
42	37.6	12.4	50	58.9	23.6	35.3	19.5	16.58	CH
45	31.95	12.05	56	63.8	28.3	35.5	21	15.68	CH
47	43.29	8.71	48	54.5	24.44	30.06	19.4	17.27	CH
48	21.24	16.76	62	70.4	25	45.4	20.5	16.02	CH
49	43.15	11.85	45	53.2	22.92	30.28	16.8	18.78	CH
50	30.76	15.24	54	66.5	26.25	40.25	20	15.36	CH
51	59.6	10.4	30	39.2	18.8	20.4	15	19.76	SC
52	68.09	8.91	23	36.7	17.4	19.3	14.3	19.41	SC
53	62.04	10.96	27	44.5	20.6	23.9	16	17.29	SC
55	29.81	18.19	52	67.9	25	42.9	21.2	15.78	CH
56	39.19	16.81	44	58.7	24.3	34.4	20	16.46	CH
61	24.79	17.21	58	68.4	28.7	39.7	22	15.54	CH

Table 3.2: Continued

Test no	sand%	silt%	clay%	LL%	PL%	PI%	OWC%	MDD (kN/m^3)	soil type
62	36.46	12.54	51	60.3	23.8	36.5	20	15.67	CH
65	18.24	16.76	65	69.5	27.8	41.7	22	15.93	CH
66	48.93	12.07	39	54.6	24.3	30.3	17.5	18.67	CH
67	71.62	8.38	20	33.8	15.39	18.41	14	19.95	SC
68	45.21	12.79	42	58.5	23.8	34.7	18.2	17.84	CH
69	22.15	19.85	58	67.5	28.6	38.9	20.5	15.28	CH
70	36.8	18.2	45	59.1	24.29	34.81	19.2	18.9	CH
71	28.69	19.31	52	65.4	23.8	41.6	19.5	17.78	CH
72	64.45	5.55	30	44.3	23.8	20.5	17	16.93	SC
73	66.81	6.19	27	43.7	23.8	19.9	15	18.59	SC
74	32.81	19.19	48	60.8	26.7	34.1	20.5	16.82	CH
76	63.98	7.02	29	43.8	21.8	22	15	18.15	SC
77	42.85	15.15	42	56.8	23.6	33.2	19.2	16.82	CH
78	65.89	4.31	29.8	47.3	22	25.3	16	19.27	CL
79	37.4	11.6	51	54	27.8	26.2	16.8	17.04	CH
80	67.98	7.02	25	40.1	20.3	19.8	14	19.35	SC
81	15.51	18.49	66	80.9	31.3	49.6	22.5	16.48	CH
82	53.63	9.37	37	39.7	18.6	21.1	14.5	18.15	SC
83	36.73	13.27	50	60.7	24.8	35.9	20.3	17.07	CH
85	40.38	17.62	42	57.8	23.2	34.6	19	16.41	CH
86	40	15	45	56.3	22.7	33.6	19	17.18	CH
87	51.54	7.46	41	39	16.7	22.3	14	19.44	SC
88	39.11	9.89	51	58.5	24.29	34.21	19.5	16.17	CH
89	49.67	9.33	41	55	19.6	35.4	18	17.35	CH
90	21.21	18.79	60	65	21.6	43.4	20.5	15.87	CH

Table.3.3: Data samples for validation for SP test

Comparison no	Test no	sand%	silt%	clay%	LL%	PL%	PI%	OWC%	MDD (kN/m^3)	Soil Type
1	92	48.02	14.98	37	48.8	18.5	30.3	17.5	17.26	CL
2	93	30	16	54	60	22.6	37.4	19	16.42	CH
3	94	17.73	20.27	62	76.8	25.7	51.1	22.5	15.76	CH
4	95	54.32	12.68	33	48.5	19.5	29	18	17.76	SC
5	97	21.35	19.65	59	63	21.7	41.3	19	16.36	CH
6	98	21.35	28.65	50	49.2	18	31.2	18	17.82	CL
7	99	22.55	19.45	58	65.7	23.9	41.8	21	15.63	CH

From the table above, the following general observations:

i) All in Fine-grained soils (clay, silt (sand> 12% fines)) limit Atterberg tests are carried out on the soil and can be used accordingly in the statistical analysis.

ii) Based on the classification system (ASTM D-2487-11) in Table 3.4 and 3.5.

Table 3.4: Unified soil classification system (more than 50% passing no 200 sieve)

Fine-grained (Over 50% by weight passing the 0.075 mm No. 200) sieve)	Low compressibility (LL less than 50)	ML	Silts,very fine sands, silty or clayey fi ne sands, micaceous silts
		CL	Low plasticity clays, sandy or silty clays
		OL	Organic silts and clays of low plasticity
	High compressibility (LL more than 50)	MH	Micaceous silts, diatomaceous silts, volcanic ash
		CH	Highly plastic clays and sandy clays
		OH	Organic silts and clays of high plasticity

Table 3.5: Unified soil classification system(more than 50% retained on no 200 sieve)

Sand ≥gravel	>12% fines	Fines = ML or MH	SM	<15% gravel	silty sand
		Fines = CL or CH	SC	<15% gravel	clayey sand
		Fines = CL - ML	SC - SM	<15% gravel	silty clayey sand

CHAPTER 4

DATA ANALYSIS

The relationship of two or more variables are expressed in the form of mathematical equations with two variables by deciding. Before the relationship between two or more variables, useful data must be collected. In this work a total of 52 soil samples were used to develop and validate a model that can predict the standard Proctor compaction characteristics of Nicosia soils from gradation and Atterberg limit test parameters, and The statistical descriptive of the dependent and independent variables for the samples used for the regression analysis excluding the 7 data for validation . Also, there are 47 samples of Atterberg limit test to apply the results of extracted relationships. Proctor compaction as tabulated in the previous section.

4.1. Data Analysis Methods

Many methods can be used to check the validity of the relationship between two or more variables. However, in this study two methods commonly used are: scatter plot and linear regression analysis .The variables are separated into independent and dependent variables. The compression test parameters are dependent variables and parameters when gradation is Atterberg limit (forecasting) independent variables. Before analytical method some important terms are discussed below.

- ***T-test value***: The probability of making a mistake to reject a hypothesis when it happens to be true at the level of significance. In practice it is usual to use the 5% level of significance. This means that we are 95% confident that we can make the right decision and we cannot go wrong with a probability of 5%, and you can get the *t-value* by dividing the standard error of the coefficient through its independent variables.
- ***P-value:*** It is the most important term in the opinion of the statistical significance of the independent variables. It also represents a significant predictive power of the model. *P-value* is simply the ratio of the model mean square error to the mean square.
- ***Standard error:*** The average error of each measurement sample points on the line of best fit. Out of all the curves, the best-fit curve through the standard error smaller, and it is important because it is used to calculate other measures, such as confidence intervals

and margin of error. The efficiency of regression line can also be evaluated through the estimation of standard error given as

$$SEE = \sqrt{\frac{SSY}{(n-p)}} \quad \text{or} \quad SEE^2 = \frac{SSY}{(n-p)} \tag{4.1}$$

- ***Correlation coefficient (R):*** correlation coefficient (sometimes called the regression coefficient) is the act of the linear correlation between two variable x and y, between +1 and -1 for sale inclusive. R = 1 indicates a perfect linear correlation and linear regression perfect, R = 0 is no correlation, and R = -1 total negative correlation.

  The coefficient of determination (R^2) is determined by:-

$$R^2 = \frac{SSR}{SSY} \quad \text{or} \quad R^2 = \frac{SSR}{SSR+SSE} \tag{4.2}$$

Where

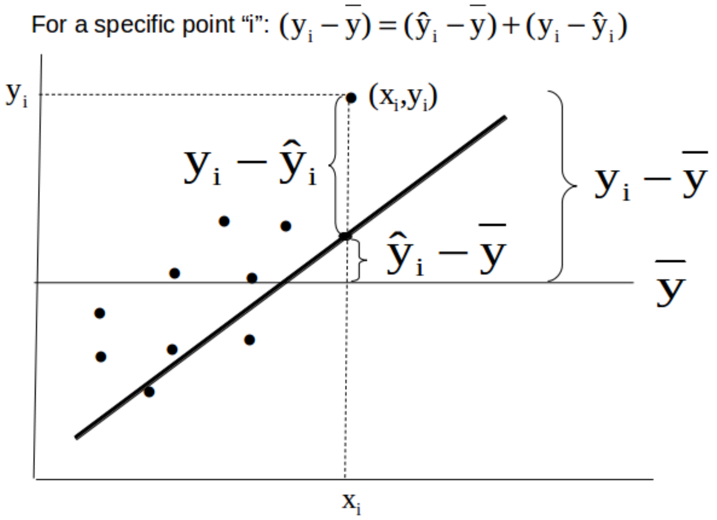


Figure 4.1: An illustration of the components contributing to the difference between the average y-value $\bar{y}$ and a particular point (x_i; y_i)

$$SSR = \sum(\hat{y}i-\bar{y})^2 \tag{4.3}$$

$$SSE = \sum(yi-\hat{y}i)^2 \tag{4.4}$$

$$SSY = \sum(yi-\bar{y})^2 \tag{4.5}$$

SSY = SSR + SSE

P = Number of selected Independent variables

4.1.1. Scatter Plot and Best-Fit Curve

All the analytical process, in this study, the amount of OWC and MDD considered as the dependent variable as the Atterberg limits where values are grain size (forecasting) independent variables.

In conducting the statistical analysis, statistical software program Minitab and Excel spreadsheets are used to determine the scatter plot, correlation and regression. Excel spreadsheet found to be the most powerful and handy tool for analyzing scatter plot and determine the correlation between two or more diverse.

However, when necessary to determine the relationship between more than two variables (the dependent variable requires two or more independent variables) and regression analysis is applied to the SPSS software to be the most powerful instrument and descriptive.

The relationship between the dependent and independent variables are examined separately for the data as presented in Figures 4.2 to 4.14.

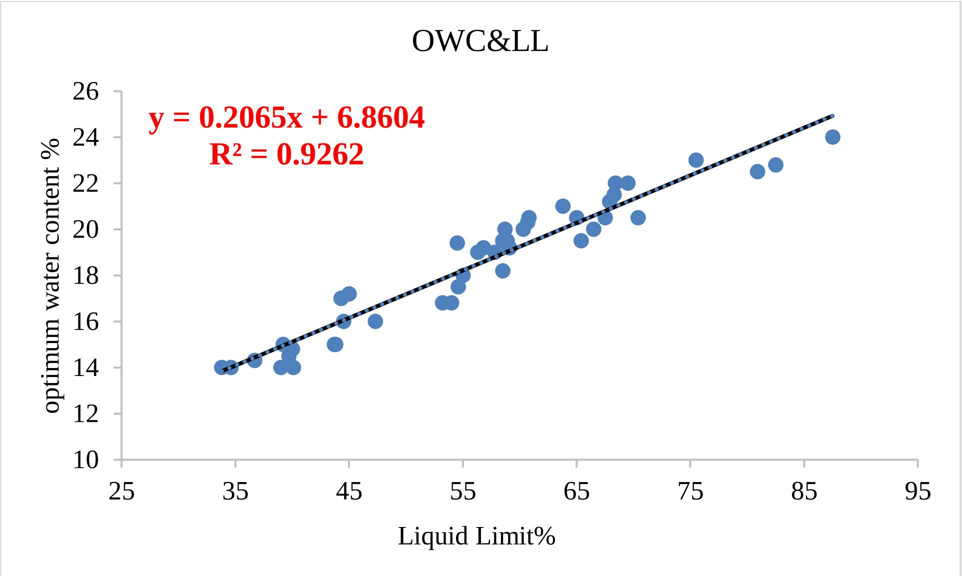


Figure 4.2: Scatterplot of LL and OWC

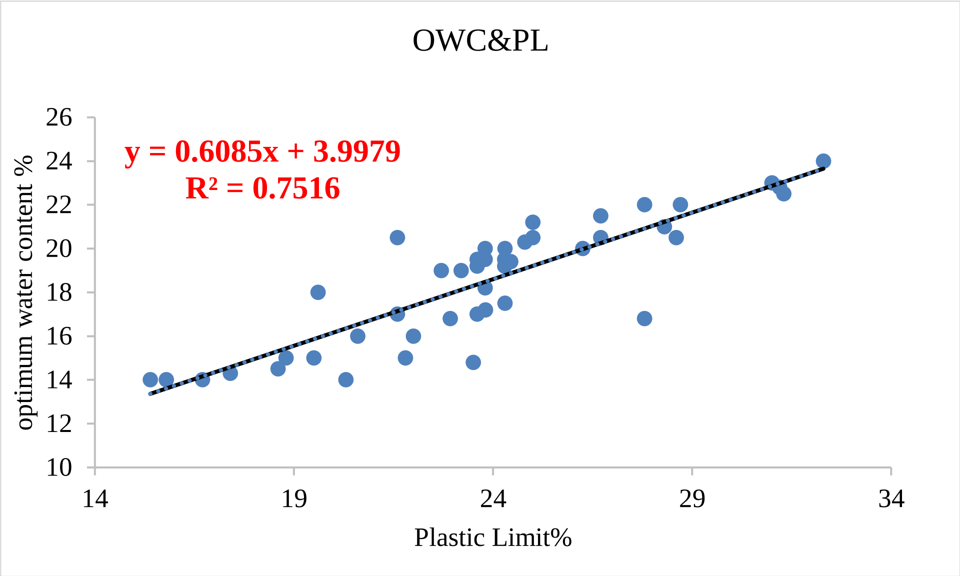


Figure 4.3: Scatterplot of PL and OWC

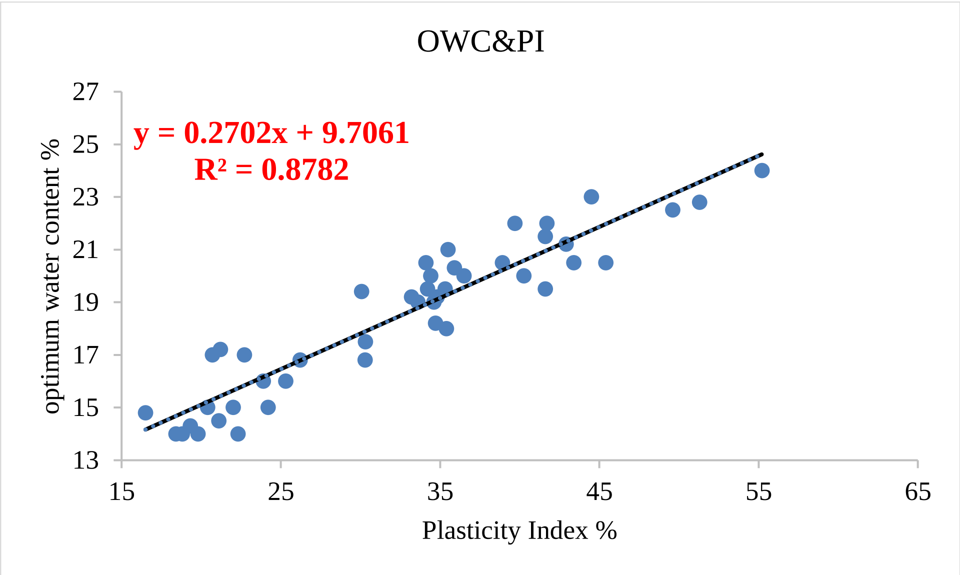


Figure 4.4: Scatterplot of PI and OWC

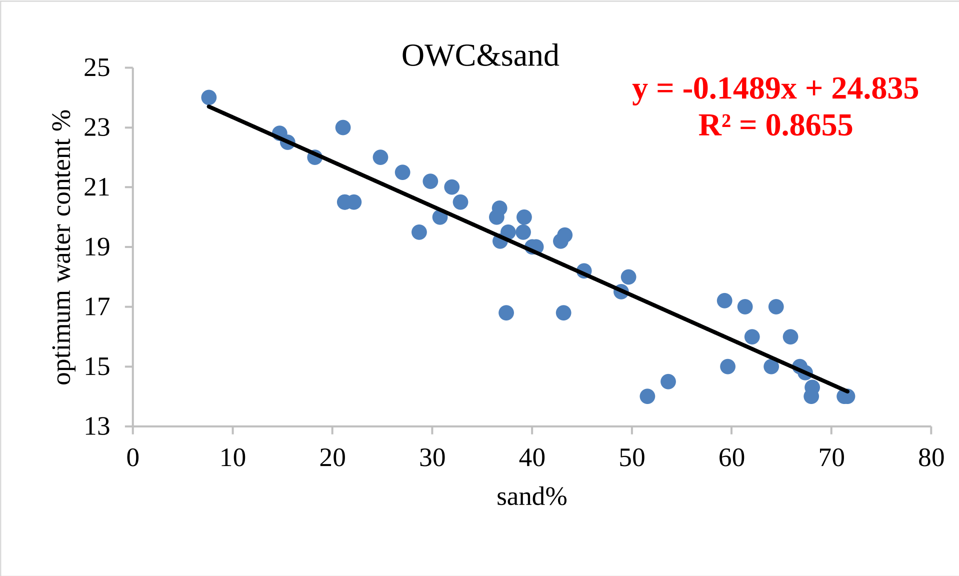


Figure 4.5: Scatterplot of sand and OWC

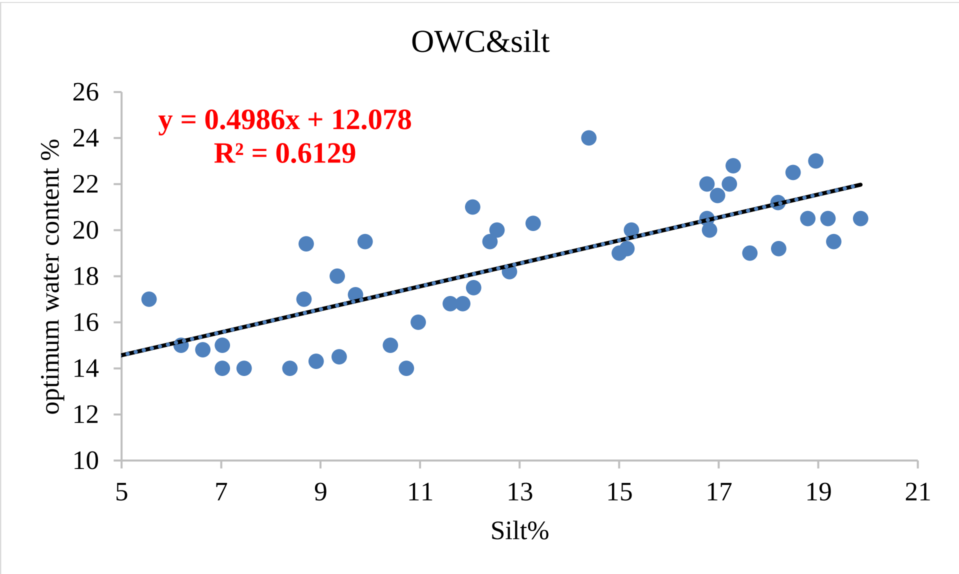


Figure 4.6: Scatterplot of silt and OWC

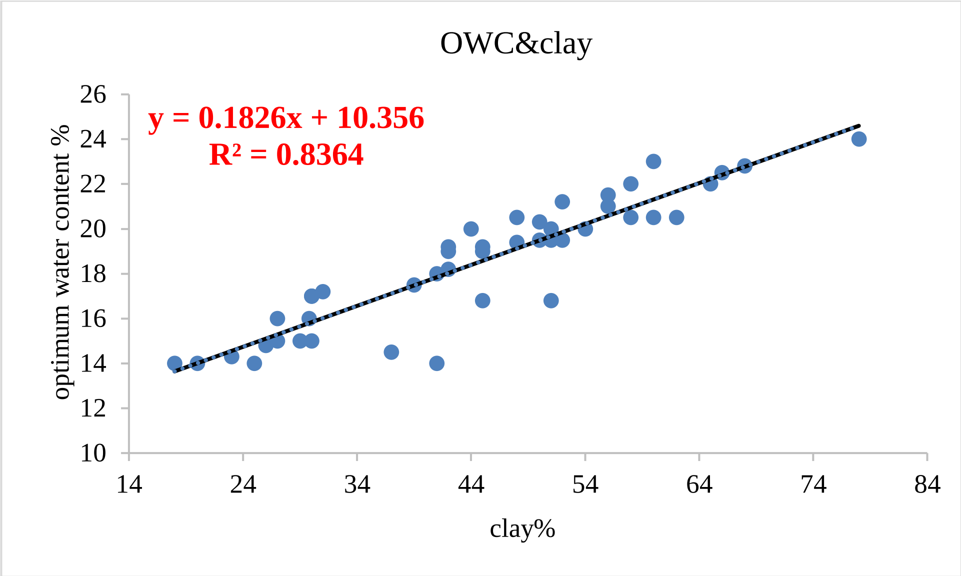


Figure 4.7: Scatterplot of clay and OWC

As shown in Figures 4.2 to 4.7, the dependent variable and independent variables in OWC Atterberg limit parameters, and grain size. Note that to predict the dependent variable in the regression analysis of more than one independent variable can take part at a time. However, at the scatter plot of Figures from 4.2 to 4.7, only one independent variable is called to predict a dependent variable.

The OWC has a strong correlation with LL than PL, PI, and fine-grain size. On the other hand, as shown in Figure 4.3, the relationship between the OWC and PL is the weakest of all the Atterberg limits.

In general, it can be concluded that the assessment of soil water content of overconsolidated soils, a compression standard Proctor, can be predicted from LL without significant error.

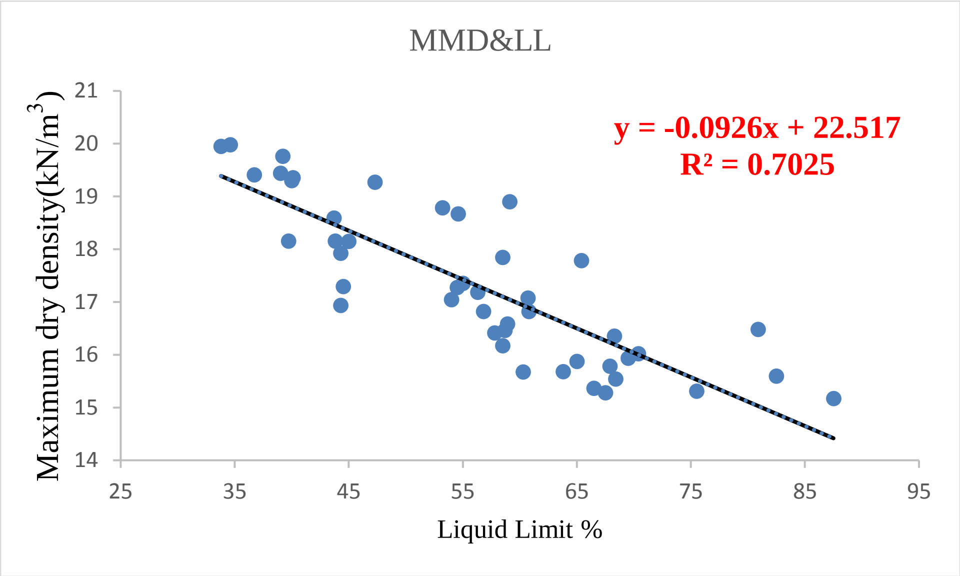


Figure 4.8: Scatterplot of LL and MDD

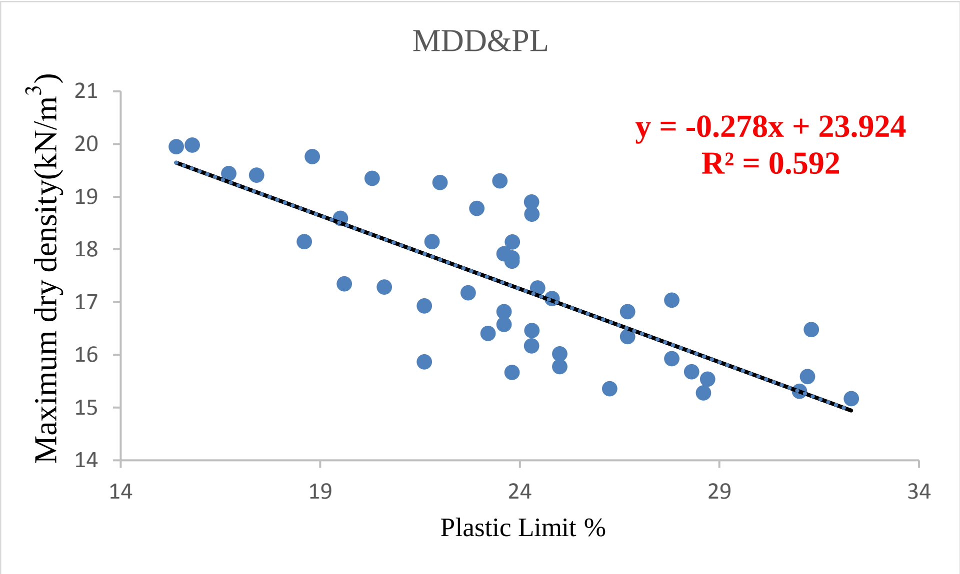


Figure 4.9: Scatterplot of PL and MDD

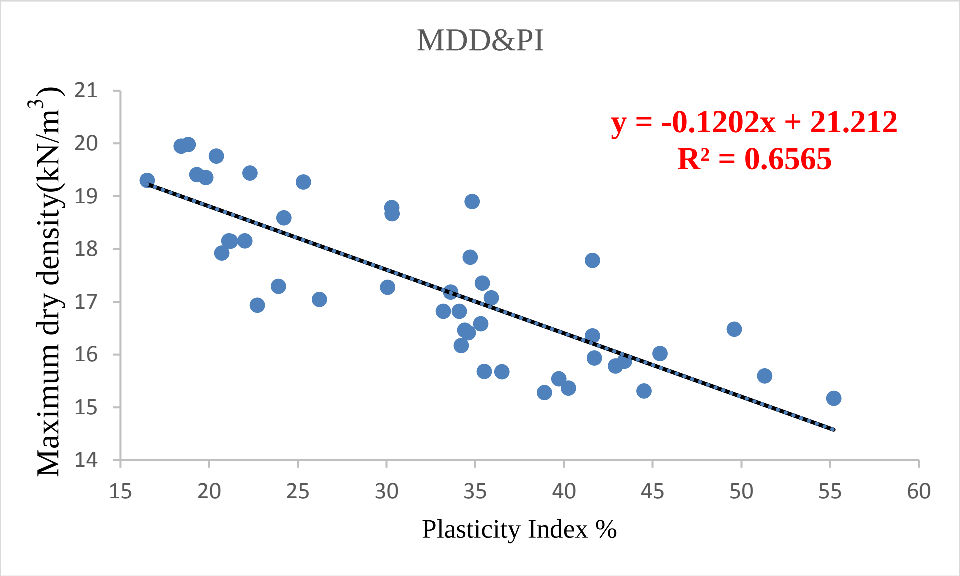


Figure 4.10: Scatterplot of PI and MDD

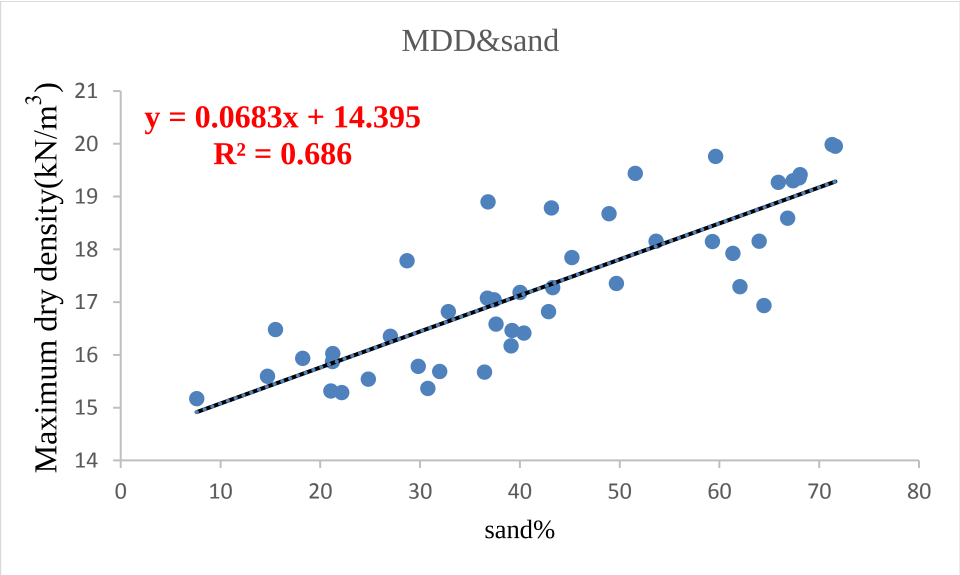


Figure 4.11: Scatterplot of sand and MDD

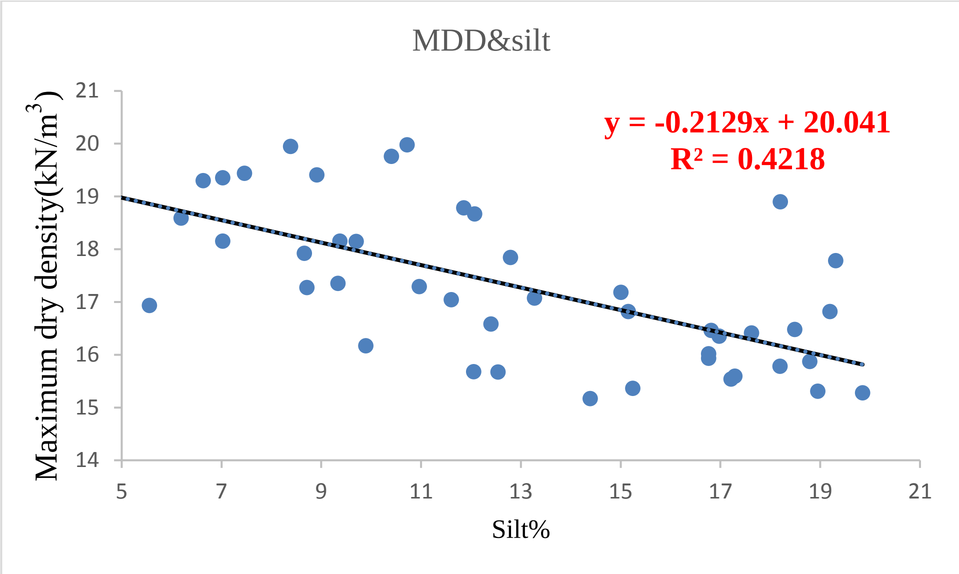


Figure 4.12: Scatterplot of silt and MDD

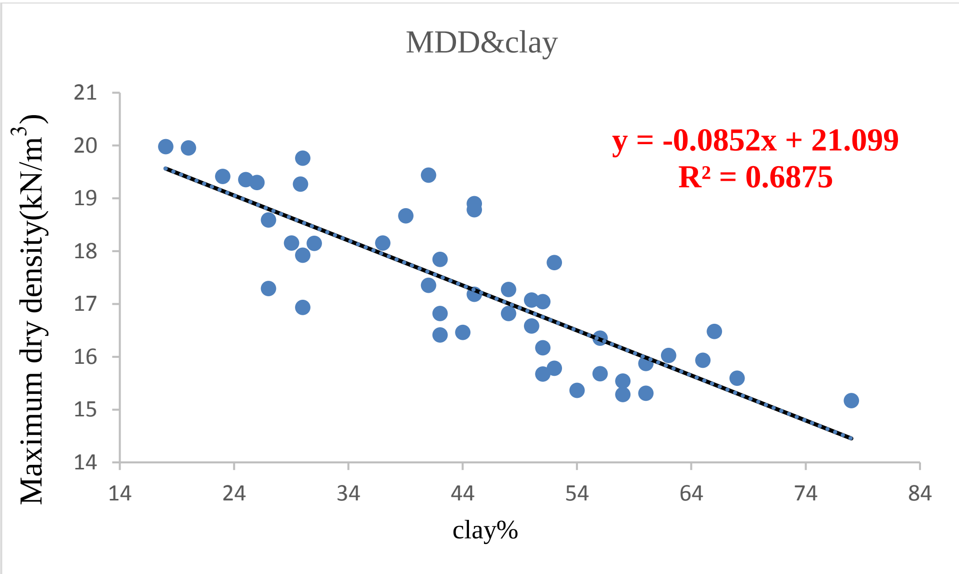


Figure 4.13: Scatterplot of clay and MDD

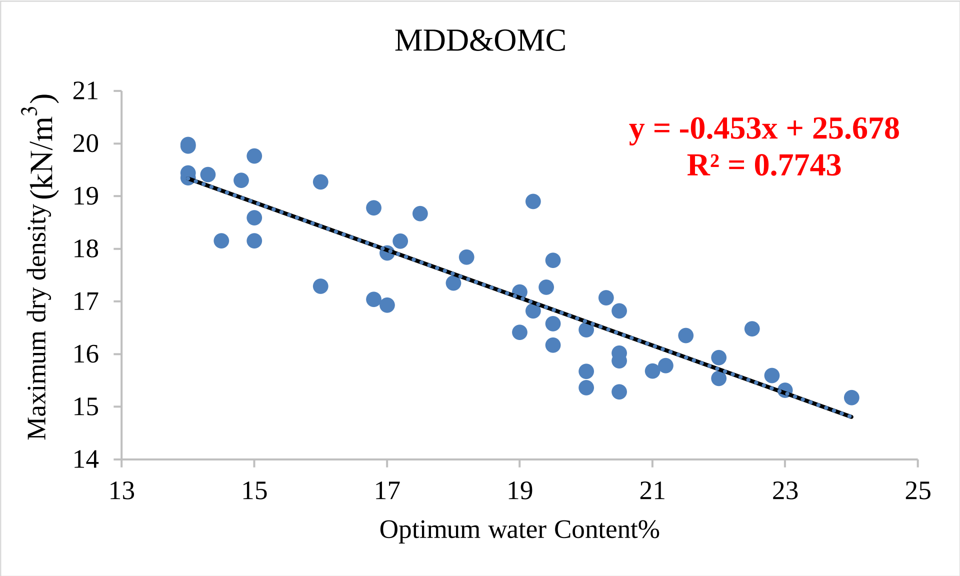


Figure 4.14: Scatterplot of OWC and MDD

Figures 4.8 to 4.14 shows the plot also and the corresponding dispersion curve of best fit for predicting the MDD. In the calculation the dependent variable is MDD, and the independent variables are included Atterberg limit, grain size, and OWC.

It can also be found in Figure 4.7 that LL has a good relationship with MDD. Both OWC and MDD can be predicted from LL only with acceptable accuracy. In addition, MDD has the best relationship with OWC than all other parameters. Thus, it can also be predicted MDD from OMC more accurately than LL, if the case, the value of the OWC.

The diagram shows the relationship between liquid limit, optimum water content, and maximum dry density.

The relationship between the OWC, MDD, and LL can be used to reduce using the compression equipment in the future for the same area of soils as shown in Figure 4.15. These curves can be used to directly estimate the OWC and MDD for SP if the LL is known.

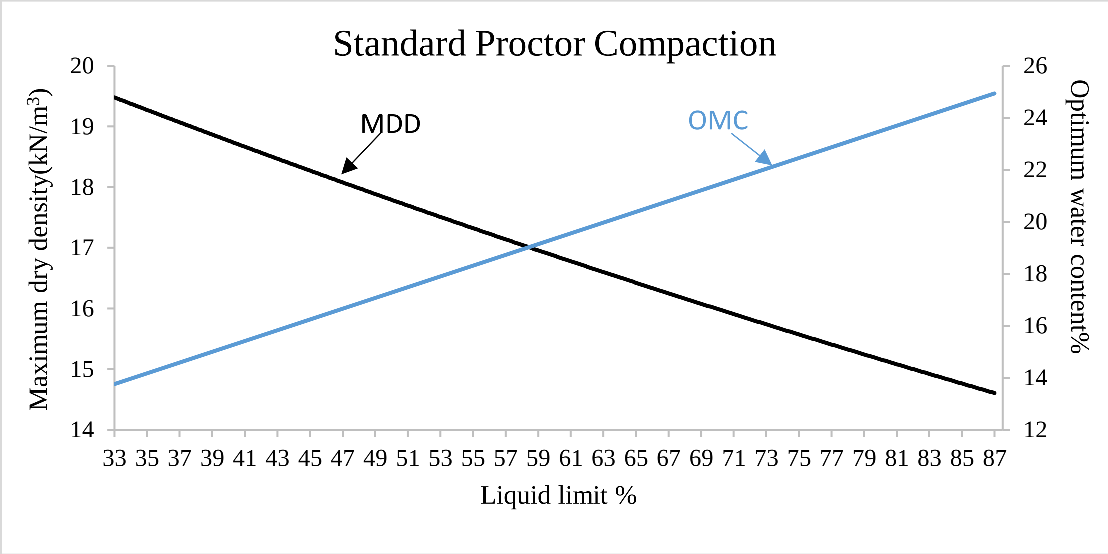


Figure 4.15: MDD and OWC versus LL for SP

4.1.2. Correlation matrix

The correlation coefficient, R, which is the relative predictive power of the model, is given for each analysis. It is between -1 and +1 descriptive measure. Table 4.1 states the accuracy of the correlation coefficient is measured by the determination, R^2. minus sign indicates inverse proportion between two variables while a plus sign represents direct proportion. A correlation matrix analysis shows the strength of the linear relationship between two variables randomly.

It is indicative tool to determine the independent variables that are highly correlated with the dependent variables. Moreover, it shows the linear interaction between two independent variable. Perhaps the high correlations between two independent variables shows in over-fit the model.

Table 4.1: A measure of correlation accuracy by R^2

R^2 **values**	**Accuracy**
<0.25	Not good
0.25-0.55	Relatively good
0.56-0.75	Good
>0.75	Very good

The correlation matrix for the representation of the linear interactions between the soil gradation and Atterberg limits and the SP test parameters are shown in Table 4.2.

Table 4.2: Correlation matrix results for SP data analysis

	sand	**silt**	**clay**	**LL**	**PL**	**PI**	**OWC**	**MDD**
sand	1	-0.835	-0.985	-0.957	-0.820	-0.950	-0.930	0.828
silt	-0.835	1	0.728	0.777	0.616	0.791	0.783	-0.649
clay	-0.985	0.728	1	0.950	0.830	0.936	0.915	-0.829
LL	-0.957	0.777	0.950	1	0.883	0.981	0.962	-0.838
PL	-0.820	0.616	0.830	0.883	1	0.775	0.867	-0.769
PI	-0.950	0.791	00.936	0.981	0.775	1	0.937	-0.810
OWC	-0.930	0.783	0.915	0.962	0.867	0.937	1	-0.880
MDD	0.828	-0.649	-0.829	-0.838	-0.769	-0.810	-0.880	1

4.1.3. Regression analysis

Regression analysis is a statistical technique for modeling and investigating the relationship between two or more variables. Called variable which value is predicted dependent variable or response. A variable used to predict the value of the dependent variable will be called independent variables or predict. Called regression model containing more than one predictive variable in multiple regression models. Otherwise, regression model consisting of one independent variable as a simple regression model. A number of techniques are used to demonstrate the adequacy of the multiple regression model; some of these are standard errors and regression coefficient R^2 value. A standard error statistic gives some idea about the precision of the estimates.

In the same way, all the regression analysis has been done and the outputs of the table tabulated as follows: -

1) Prediction of optimum water content from liquid limit, plastic limit, plasticity index, sand, and fine content `

Variables : LL, PL, PI,sand, FC, and OWC

- Independent variables : LL, PL, PL, sand, FC, and C
- Dependent variable: OWC
- R^2 = 0.932
- SEE = 0.776
- Proposed equation:

$$OWC = 6.82 - 0.0046\ Sand + 0.0650\ Silt + 0.162\ LL + 0.0802\ PL \qquad (4.6)$$

Table 4.3: Coefficients of predicting OWC from LL, PL, PI, sand, and FC

Variable	Coefficients	SEE	t Stat	P-value
Intercept	6.819	3.051	2.24	0.031
sand	-0.00459	0.02652	-0.17	0.864
silt	0.06498	0.04895	1.33	0.192
LL	0.16194	0.03791	4.27	0
PL	0.08025	0.06352	1.26	0.214

2) Prediction of optimum water content from liquid limit, plastic limit, and plasticity index

- Variables : LL, PL, PI, and OWC
- Independent variables : LL, PL, PI, and C
- Dependent variable: OWC
- R^2 = 0.927
- SEE = 0.786
- Proposed equation:

$$OWC = 6.38 + 0.192\ LL + 0.0555\ PL \qquad (4.7)$$

Table 4.4: Coefficients of predicting OWC from LL, PL, and PI

Variable	Coefficients	SEE	t Stat	P-value
Intercept	6.3826	0.7419	8.6	0
LL	0.19152	0.01896	10.1	0
PL	0.05546	0.06202	0.89	0.376

3) Prediction of optimum water content from liquid limit, and plasticity index

- Variables : LL, PI, and OWC
- Independent variables : LL, PI, and C
- Dependent variable: OWC
- $R^2 = 0.88$
- SEE = 1
- Proposed equation:

$$OWC = 9.71 + 0.270\ PI \tag{4.8}$$

Table 4.5: Coefficients of predicting OWC from LL and PI

Variable	Coefficients	SEE	t Stat	P-value
Intercept	9.706099	0.520233	18.6572	0
PI	0.270179	0.015342	17.61033	0

4) Prediction of optimum water content from liquid limit

- Variables : LL and OWC
- Independent variables : LL and C
- Dependent variable: OWC
- $R^2 = 0.926$
- SEE = 0.78

- Proposed equation:

$$OWC = 6.86 + 0.206LL \tag{4.9}$$

Table 4.6: Coefficients of predicting OWC from LL

Variable	Coefficients	SEE	t Stat	P-value
Intercept	6.86	0.513533	13.35928	0
LL	0.206483	0.008887	23.23375	0

5) Prediction of optimum water content from plastic limit

- Variables : PL and OWC
- Independent variables : PL and C
- Dependent variable: OWC
- $R^2 = 0.752$
- SEE = 1.43
- Proposed equation:

$$OWC = 4.00 + 0.609\,PL \tag{4.10}$$

Table 4.7: Coefficients of predicting OWC from PL

Variable	Coefficients	SEE	t Stat	P-value
Intercept	3.998	1.288	3.11	0.003
PL	0.60854	0.05335	11.41	0

6) Prediction of maximum dry density from liquid limit, plastic limit, plasticity index, sand, Fine content, and optimum water content

- Variables : LL, PL, PI, FC, OWC, and MDD
- Independent variables : LL, PL, PI, FC, OWC, and C
- Dependent variable: MDD
- $R^2 = 0.79$
- SEE = 0.71

- Proposed equation:

$$MDD = 22.4 + 0.0341\,sand + 0.0600\,silt + 0.0482\,LL - 0.0152\,PL - 0.526\,OWC \qquad (4.11)$$

Table 4.8: Coefficients of predicting MDD from LL, PL, PI, OWC, sand, and FC

Variable	Coefficients	SEE	t Stat	P-value
Intercept	22.44517	2.967611	7.563382	0
sand	0.034148	0.024336	1.403229	0.16846
silt	0.060039	0.045873	1.30882	0.198256
LL	0.04818	0.04196	1.15	0.258
PL	-0.01524	0.05941	-0.26	0.799
OWC%	-0.52565	0.145023	-3.62457	0.000826

7) Prediction of maximum dry density from liquid limit, plastic limit, plasticity index, and optimum water content

- Variables : LL, PL, PI, OWC, and MDD
- Independent variables : LL, PL, PI, OWC, and C
- Dependent Variable : MDD
- R^2= 0.776
- SEE = 0.716
- Proposed equation:

$$MDD = 26.2 + 0.0172\,LL - 0.0204\,PL - 0.505\,OWC \qquad (4.12)$$

Table 4.9: Coefficients of predicting MDD from LL, PL, PI, and OWC

Variable	Coefficients	SEE	t Stat	P-value
Intercept	26.15536	1.12976	23.15124	0
LL	0.01717	0.032168	0.533737	0.596405
PL	-0.02044	0.05736	-0.35631	0.723439
OWC%	-0.50473	0.141379	-3.57005	0.001

8) Prediction of maximum dry density from liquid limit, plasticity index, and optimum water content

- Variables : LL, PI, OWC, and MDD
- Independent variables : LL, PI, OWC, and C
- Dependent Variable: MDD
- R^2 = 0.776
- SEE = 0.716
- Proposed equation:

$$MDD = 26.2 - 0.0033\ LL + 0.0204\ PI - 0.505\ OWC \tag{4.13}$$

Table 4.10: Coefficients of predicting MDD from LL, PI, and OWC

Variable	Coefficients	SEE	t Stat	P-value
Intercept	26.15536	1.12976	23.15124	0
LL	-0.00327	0.05484	-0.06	0.953
PI	-0.02044	0.05736	0.36	0.723439
OMC%	-0.50473	0.141379	-3.57005	0.001

9) Prediction of maximum dry density from liquid limit, and optimum water content

- Variables : LL, OWC, and MDD
- Independent variables : LL, OWC, and C
- Dependent variable: MDD
- R^2 = 0.78
- SEE = 0.708
- Proposed equation:

$$MDD = 26.0 + 0.0131\ LL - 0.512\ OWC \tag{4.14}$$

Table 4.11: Coefficients of predicting MDD from LL, and OWC

Variable	Coefficients	SEE	t Stat	P-value
Intercept	26.02651	1.05914	24.57326	0
LL	0.013077	0.029734	0.439798	0.662338
OMC%	-0.51161	0.138588	-3.69161	0.001

10) Prediction of maximum dry density from liquid limit, and plastic limit

- Variables : MDD, LL, and PL
- Independent variables : LL, PL, and C
- Dependent variable: MDD
- $R^2 = 0.707$
- SEE = 0.81
- Proposed equation:

$$MDD = 22.9 - 0.0795\ LL - 0.0484\ PL \quad (4.15)$$

Table 4.12: Coefficients of predicting MDD from LL, and PL

Variable	Coefficients	SEE	t Stat	P-value
Intercept	22.93386	0.76897	29.82412	0
LL	-0.0795	0.019647	-4.04618	0
PL	-0.04843	0.064278	-0.75343	0.455394

11) Prediction of maximum dry density from liquid limit, and plasticity index

- Variables : LL, PI, and MDD
- Independent variables : LL, PI, and C
- Dependent variable: MDD
- $R^2 = 0.707$
- SEE = 0.81

- Proposed equation:

$$MDD = 22.9 - 0.128\ LL + 0.0484\ PI \tag{4.16}$$

Table 4.13: Coefficients of predicting MDD from LL and PI

Variable	Coefficients	SEE	t Stat	P-value
Intercept	22.93386	0.76897	29.82412	0
LL	-0.12792	0.047834	-2.67433	0.010623
PI	0.048429	0.064278	0.753431	0.455394

12) Prediction of maximum dry density from liquid limit

- Variables : MDD and LL
- Independent variables : LL and C
- Dependent variable: MDD
- R^2 = 0.702
- SEE = 0.81
- Proposed equation:

$$MDD = 22.5 - 0.0926\ LL \tag{4.17}$$

Table 4.14 : Coefficients of predicting MDD from LL

Variable	Coefficients	SEE	t Stat	P-value
Intercept	22.51661	0.530813	42.41907	0
LL	-0.09256	0.009186	-10.0762	0

13) Prediction of maximum dry density from optimum water content

- Variables : OWC and MDD
- Independent variables : OWC and C
- Dependent variable: MDD

- R^2 = 0.774
- SEE = 0.702
- Proposed equation:

$$MDD = 25.7 - 0.453\ OWC \quad (4.18)$$

Table 4.15: Coefficients of predicting MDD from OWC

Variable	Coefficients	SEE	t Stat	P-value
Intercept	25.67839	0.697088	36.83666	0
OWC%	-0.45295	0.03729	-12.147	0

Table 4.16: Summary of linear equations, R^2, and SE in predicting OWC

no	Coefficients of Predictors						Equation	R^2	SEE
	sand	silt	LL	PL	PI	C			
1	-0.01	0.07	0.16	0.08		6.82	OWC = 6.82 - 0.01 sand + 0.0650 silt + 0.162 LL + 0.0802 PL	0.93	0.78
2			0.19	0.06		6.38	OWC = 6.38 + 0.192 LL + 0.0555 PL	0.93	0.79
3					0.27	9.71	OWC = 9.71 + 0.270 PI	0.88	1
4			0.21			6.86	OWC = 6.86 + 0.206 LL	0.93	0.78
5				0.61		4	OWC = 4.00 + 0.609 PL	0.75	1.43

Table 4.17: Summary of linear equations, R^2, and SE in predicting MDD

no	Coefficients of Predictors							Equation	R^2	SEE
	sand	silt	LL	PL	PI	OWC	C			
6	0.03	0.06	0.05	-0.02		-0.53	22.4	MDD = 22.4 + 0.0341 sand + 0.0600 silt + 0.0482 LL - 0.0152 PL - 0.526 OWC	0.79	0.71
7			0.02	-0.2		-0.51	26.2	MDD = 26.2 + 0.0172 LL - 0.0204 PL - 0.505 OWC	0.78	0.72
8			0		0.02	-0.51	26.2	MDD = 26.2 - 0.0033 LL + 0.0204 PI - 0.505 OWC	0.78	0.72
9			0.01			-0.51	26	MDD = 26.0 + 0.0131 LL - 0.512 OWC	0.78	0.71
10			-0.08	-0.05			22.9	MDD = 22.9 - 0.0795 LL - 0.0484 PL	0.71	0.81
11			-0.13		0.05		22.9	MDD = 22.9 - 0.128 LL + 0.0484 PI	0.71	0.81
12			-0.09				22.5	MDD = 22.5 - 0.0926 LL	0.70	0.81
13						-0.45	25.7	MDD = 25.7 - 0.453 OWC	0.77	0.7

4.2 Validation of the developed models

4.2.1. Using the equation

The models developed validated using a different set of data not seen by the model. The data in Table 3.3 will be used for the empirical models standard Proctor.

Compression for SP; Table 4.18 shows their results and MDD and OWC predicted. An absolute error between the highest MDD measurement unit forecast at 0.71 shown that this model is very accurate, in the same way for OWC higher absolute error is 1.10 which is still very low.

In addition, graphical representations of a validated model for MDD and OWC are shown in Figure 4.16 and Figure 4.17 respectively. The R^2 value is also displayed and put a very high amount of testimony to the strength and statistical models for OWC and MDD.

Table 4.18: Validation of SP parameters models(using equation)

Optimum Water Content (%) using LL			**Maximum Dry Density (kN/m^3) using OWC**		
Measured	**Predicted**	**Abs.Error**	**Measured**	**Predicted**	**Abs.Error**
17.5	16.96	0.54	17.26	17.75	0.49
19	19.28	0.28	16.42	17.07	0.65
22.5	22.76	0.26	15.76	15.49	0.27
18	16.90	1.10	17.76	17.52	0.24
19	19.90	0.90	16.36	17.07	0.71
18	17.04	0.96	17.82	17.52	0.30
21	20.46	0.54	15.63	16.17	0.54

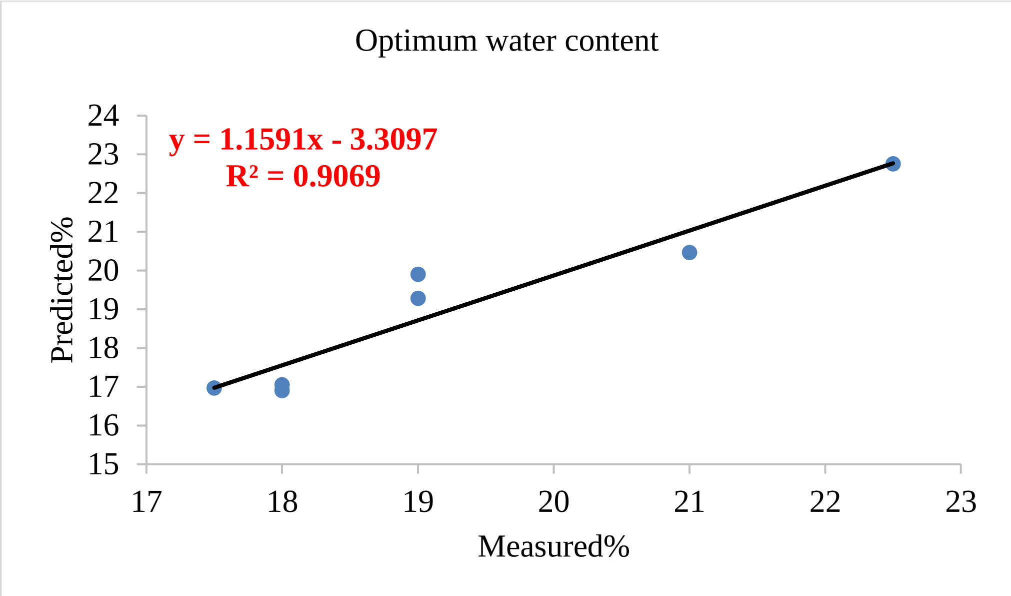


Figure 4.16: Plot of predicted and measured OWC for SP model validation(using equation)

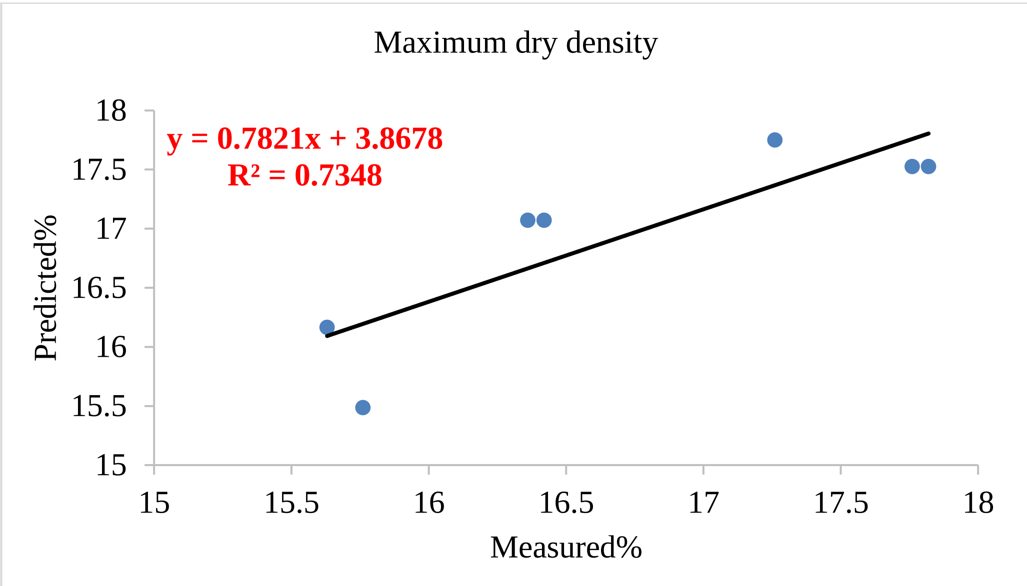


Figure 4.17: Plot of predicted and measured MDD for SP model validation(using equation)

4.2.2 Using the graph

The Table 4.19 shows the results of the measured and predicted MDD and OWC. Higher absolute error between them and predicted MDD 0.77 shows that this model is very accurate, OWC potential error in case 1 which is also very low.

Additionally, graphical representations of the validated model for MDD and OWC are shown in Figure 4.18 and Figure 4.19 respectively. The R^2 value is also displayed and put a very high amount of testimony to the strength and statistical models for OWC and MDD.

Table 4.19: Validation of SP parameters models (using the graph)

Optimum Water Content (%) using graph			**Maximum Dry Density (kN/m^3) using graph**		
Measured	**Predicted**	**Abs.Error**	**Measured**	**Predicted**	**Abs.Error**
17.5	17	0.5	17.26	17.8	0.54
19	19.2	0.2	16.42	16.9	0.48
22.5	22.5	0	15.76	15.5	0.26
18	17	1	17.76	18	0.24
19	19.8	0.8	16.36	16.7	0.34
18	17.2	0.8	17.82	18	0.18
21	20.5	0.5	15.63	16.4	0.77

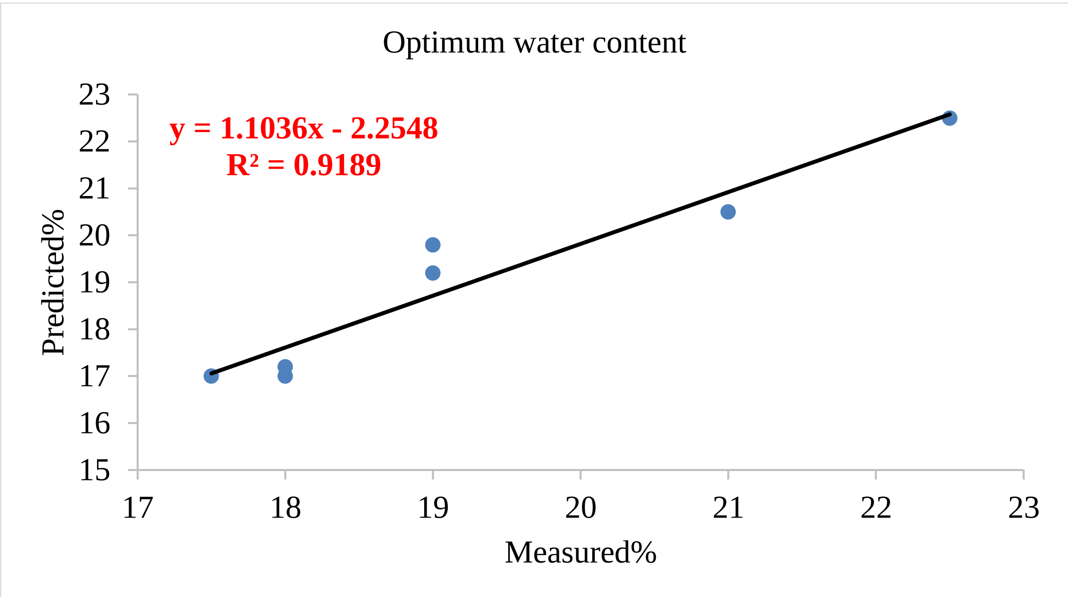


Figure 4.18: Plot of predicted and measured OMC for SP model validation (using the graph)

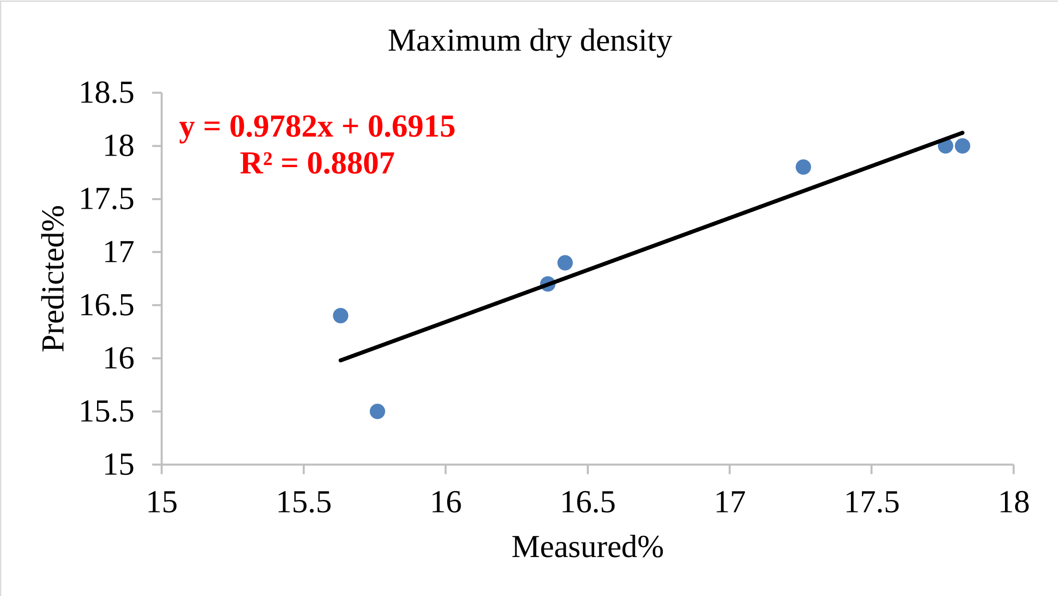


Figure 4.19: Plot of predicted and measured MDD for SP model validation(using the graph)

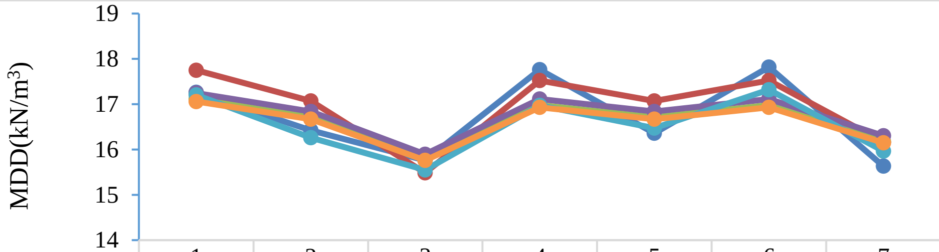
From the results, we found no significant difference between the use of leverage and the use of. Along these lines using the graph is less demanding and requires nothing but the need to detail when perusing information.

4.3. Comparison of Developed Models with Some Existing Models.

Some of the existing models are used to predict soil compaction test parameters used to validate models developed and compared with the models proposed in this study.

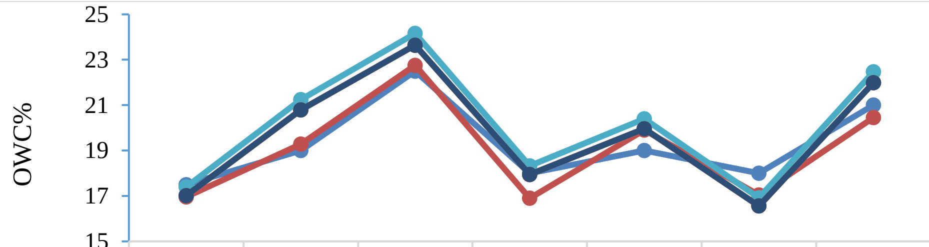
As seen in Figure 4.20 and Figure 4.22 using a series of equations and graphs all models can be used to predict the maximum dry weight of standard Proctor test. It was found that the predicted using these models is close to the measurement. However, these models should be used with caution in predicting the characteristics of Nicosia soils in Cyprus.

A similar observation can be seen in Figure 4.21 and Figure 4.23 using a series of equations and graphs while no modification of the conservation measure, the dose should be taken in the application of these models during the pre-feasibility studies of a project using Nicosia soils in North Cyprus.



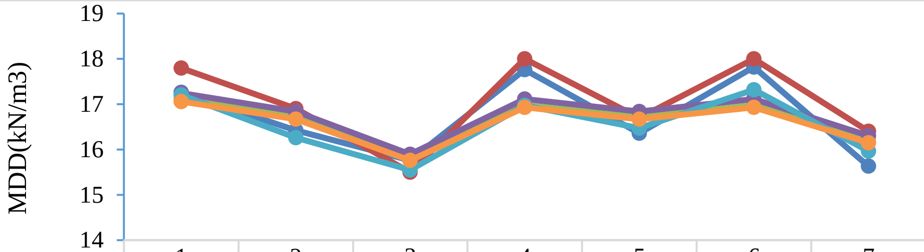
	1	2	3	4	5	6	7
Measured	17.26	16.42	15.76	17.76	16.36	17.82	15.63
Predicted	17.7505	17.071	15.4855	17.524	17.071	17.524	16.165
Sivrikaya & Soycan 2009	17.115	16.71	15.765	16.98	16.71	16.98	16.17
Sivrikaya 2008	17.245	16.84	15.895	17.11	16.84	17.11	16.3
Sridharan & Nagaraj 2005	17.205	16.262	15.549	16.975	16.469	17.32	15.963
Gurtug & Sridharan 2004	17.06	16.67	15.76	16.93	16.67	16.93	16.15

Figure 4.20: Comparison of some existing models with developed model for MDD for SP (using equation)



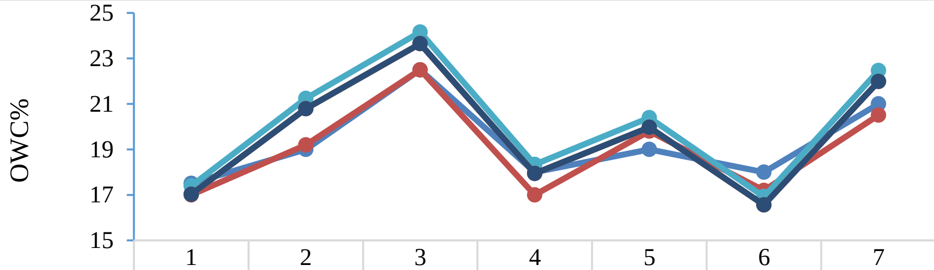
	1	2	3	4	5	6	7
Measured	17.5	19	22.5	18	19	18	21
Predicted	16.9616	19.28	22.7576	16.8995	19.901	17.0444	20.4599
Sivrikaya & Soycan 2009	17.02	20.792	23.644	17.94	19.964	16.56	21.988
Sivrikaya 2008	17.39	21.244	24.158	18.33	20.398	16.92	22.466
Sridharan & Nagaraj 2005	17.02	20.792	23.644	17.94	19.964	16.56	21.988
Gurtug & Sridharan 2004	17.02	20.792	23.644	17.94	19.964	16.56	21.988

Figure 4.21: Comparison of some existing models with developed model for OWC for SP (using equation)



	1	2	3	4	5	6	7
Measured	17.26	16.42	15.76	17.76	16.36	17.82	15.63
Predicted	17.8	16.9	15.5	18	16.7	18	16.4
Sivrikaya & Soycan 2009	17.115	16.71	15.765	16.98	16.71	16.98	16.17
Sivrikaya 2008	17.245	16.84	15.895	17.11	16.84	17.11	16.3
Sridharan & Nagaraj 2005	17.205	16.262	15.549	16.975	16.469	17.32	15.963
Gurtug & Sridharan 2004	17.06	16.67	15.76	16.93	16.67	16.93	16.15

Figure 4.22: Comparison of some existing models with developed model for MDD for SP (using the graph)



	1	2	3	4	5	6	7
Measured	17.5	19	22.5	18	19	18	21
Predicted	17	19.2	22.5	17	19.8	17.2	20.5
Sivrikaya & Soycan 2009	17.02	20.792	23.644	17.94	19.964	16.56	21.988
Sivrikaya 2008	17.39	21.244	24.158	18.33	20.398	16.92	22.466
Sridharan & Nagaraj 2005	17.02	20.792	23.644	17.94	19.964	16.56	21.988
Gurtug & Sridharan 2004	17.02	20.792	23.644	17.94	19.964	16.56	21.988

Figure 4.23: Comparison of some existing models with developed model for OWC for SP (using the graph)

4.4. Utilizing the equations as a part of the same area soil

In this section the values of optimum water content and maximum dry density were predicted for the soil samples and they were not tested by Proctor compaction parameter test. Table 3.1 shows the 47 samples that were predicted. To apply the correlation between Proctor compaction parameter (MDD and OMC) and Atterberg limit parameter, the grain soil can be use to determine the possibility of using these correlation in the future in the same sample area.

Table 4.20: Data samples for prediction for SP test

TEST NO	sand	silt	clay	Atterberg Limits Test			water content%	predict	
				LL	PL	PI		OWC	MDD
1	57.15	12.85	30	45.8	22.6	23.2	7.45	16.34	18.33
4	31.3	10.7	58	71	14.3	56.7	8.65	21.56	15.98
5	59.72	11.28	29	40	21.3	18.7	5.1	15.14	18.87
6	29.14	18.86	52	65.9	23.8	42.1	8.93	20.50	16.45
7	71.25	8.75	20	35.5	18.6	16.9	5.22	14.21	19.28
8	20.16	17.84	62	71.4	32.5	38.9	13.81	21.64	15.94
9	59.84	14.16	26	48	21.9	26.1	4.78	16.80	18.12
10	19.17	20.83	60	77.6	33.3	44.3	12.27	22.92	15.36
11	70.55	7.45	22	38.2	19.6	18.6	6.93	14.77	19.03
12	39.9	16.1	44	58.9	23.8	35.1	11.02	19.05	17.11
13	70.03	9.97	20	33.4	17.3	16.1	4.63	13.77	19.48
14	31.62	15.38	53	69	28.3	40.7	15.18	21.14	16.16
15	70.35	9.65	20	36.1	18.9	17.2	3.5	14.33	19.23
16	28.7	17.3	54	72	33.3	38.7	14.27	21.76	15.88
17	68.62	9.38	22	39.5	19.8	19.7	8.87	15.04	18.91
21	67.26	10.74	22	41.9	22.8	19.1	1.8	15.53	18.69
22	20.37	15.63	64	71	18.3	52.7	11.73	21.56	15.98
23	68.09	5.91	26	40	25.5	14.5	3.31	15.14	18.87
24	14.92	20.08	65	78.9	30	48.9	16.77	23.19	15.24
25	73.11	8.89	18	36	20.8	15.2	13.9	14.31	19.24
26	17.32	17.68	65	81.8	30.3	51.5	4.57	23.79	14.97
27	65.66	6.34	28	43.4	22.3	21.1	11.33	15.84	18.55
28	17.68	18.32	64	79.5	26.5	53	15	23.32	15.19
31	69.81	6.19	24	39.6	22	17.6	9.22	15.06	18.90

Table.4.19: Continued

TEST NO	sand	silt	clay	Atterberg Limits Test			water content%	predict	
				LL	PL	PI		OWC	MDD
32	30.84	15.16	54	64.2	20	44.2	13.57	20.15	16.61
33	70.09	4.91	25	36.4	18.9	17.5	9.04	14.40	19.20
34	29.56	12.44	58	66.7	26.6	40.1	11.21	20.67	16.38
35	69.1	6.9	24	41.6	23.9	17.7	4.51	15.47	18.72
36	26.83	19.17	54	74.5	28.57	45.93	10.82	22.28	15.65
37	69.14	8.86	22	34.6	15.2	19.4	7.98	14.02	19.37
38	14.79	23.21	62	81	37.5	43.5	14.47	23.63	15.05
39	64.84	6.16	29	38	19.7	18.3	6.75	14.73	19.05
40	63.34	8.66	28	46	23.8	22.2	7.54	16.38	18.31
43	68.41	8.59	23	37.6	17.3	20.3	1.38	14.64	19.09
44	35.29	16.71	48	56	28.3	27.7	3.89	18.45	17.38
46	71.08	9.92	19	36.7	20	16.7	28.37	14.46	19.17
54	72.26	7.74	20	34.7	18	16.7	1.17	14.04	19.36
57	56.52	8.48	35	45.9	20	25.9	5.46	16.36	18.32
58	67.27	12.73	20	37.7	21	16.7	1.45	14.66	19.08
59	60.43	8.57	31	42	16.7	25.3	4.43	15.55	18.68
60	30.31	16.69	53	59.9	33.3	26.6	6.95	19.26	17.01
63	69.3	4.7	26	37	20.2	16.8	1.39	14.52	19.15
64	70.53	5.47	24	36.5	15.5	21	0.97	14.42	19.19
75	68.82	10.18	21	39.5	20.3	19.2	4.15	15.04	18.91
84	69.83	9.17	21	39.2	19.6	19.6	7.08	14.97	18.94
91	16.5	23.5	60	61.2	30.4	30.8	8.55	19.53	16.89
96	57.4	10.6	32	44.8	21.7	23.1	6.08	16.13	18.42

Contrasting the outcomes and some Proctor test information results, was done in the research facility the outcomes are indistinguishable to a substantial degree, proposing that these relations can be utilized with a small percent of mistake which don't influence the work process.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

In order to assure the compression assessment quality level, field-based experiments were conducted, the compression test parameters, namely; OMC and MDD in the laboratory are dependable parameters. On the basis of the study findings, the objectives in this dissertation have been achieved. 99 Nicosia soils in Cyprus have been used to develop and validate empirical equation to estimate the parameters compression standard Proctor to Atterberg and gradation.

According to the finding analyses obtained from laboratory test data collection, the conclusions below were made and the following recommendations were given.

5.1 Conclusions

The primary aim of this research has been gathering useful relationships between features and cost parameters Atterberg fine-grained soils compression. Both MDD and OWC of over-consolidaed soils relate too well with LL than PL and PI of soils. The OWC has the best correlation with LL, and a MDD has satisfactory correlation. Thus, the two MDD and OMC of over-consolidaed soils predictable tests LL, especially for prefeasibility study of the project. LL of the sample used for regression analyzes ranged from 33.8% to 87.5% for SP. The PL range from 15.4% to 32.3% for SP. Empirical models offered in more than R^2 value of 0.7 and a standard error estimate, less than 1 indicates the high computing power of the models. Stepwise multiple linear regression analyses are used for the development of a model to reduce over-fit the model. There have been predicting for data untested by Proctor compaction tests, and the results were satisfactory. In conclusion, although the feasibility of any project conditions earthworks, which was completed by the use of Nicosia soils, the proposed equation can be used to estimate the characteristics compression test. It should be noted that these models will serve as a testing area, where the tests should be done accordingly, should only be used in the preliminary design phase where there is limited time, financial constraints and tests on large scale.

5.2 Recommendations

Cyprus soils are divided mainly as Troodos Ophiolite Zone or Troodos massive, Northern Cyprus (Kyrenia) Zone, Mamonia Zone or Mamonia Complex, South Cyprus Zone, Mesaoria Zone, and Alluviums. The soils of Nicosia are mainly Kythrea Group, Nicosia Formation, and Alluviums. The results of this study are limited only to Kythrea Group inNorthern Cyprus (Kyrenia) Zone in Cyprus. The tests were used to develop predictive models between Atterberg limit parameters gradational parameters, and compaction test parameters for standard energy. Further studies in other soil zones should be carried out in order to propose empirical equations to estimate Proctor compaction test parameters in the future. This work can be extended to correlate with different parameter tests such as modified Proctor compaction test parameter, etc.

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APPENDIX

LABORATORY TEST SHEETS

1. PARTICLE SIZE DISTRIBUTION (GRADATION ANALYSIS)

ASTM D 422

NEAR EAST UNIVERSITY LABROTORY

Test method :- Sieve analysis test

Date testes :- 23/12/2015

Tested by :- Arhaiem Hussain at Near East University

Project name :- Thesis

Sample number :- 96

sample 96		sieve anlysis no	mass of soil retained on eachbsievev(g)	passing (%)
sand	coarse	0.85	0	100
	mid	0.425	0	100
		0.3	3.32	95.26
	fine	0.15	10.18	80.71
	very fine	0.075	26.69	42.59
Clay and silt		pan	29.81	0
			70	

NEAR EAST UNIVERSITY LABROTORY

Test :- Hydrometer Analysis

Test date :- 13/4/2016

Tested by :- Arhaiem Hussain at Near East University

Hydrometer Number :- 151 H

Specific Gravity of solids :- 2.65

Dispersing Agent :- Sodium Hexametaphosphate

Wieght of Soil Sample :- 50 gm

Level Water :- 1.002

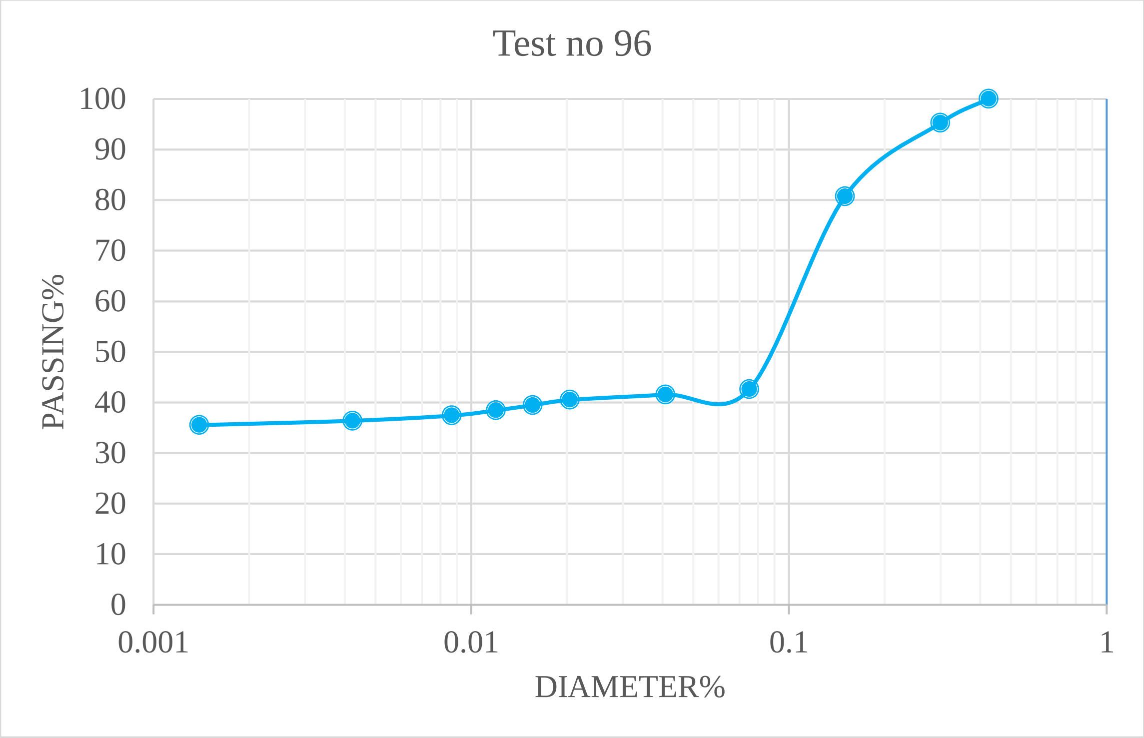
Zero correction :- 1001

Project name :-   Thesis

Sample number :- 96

Test no 96 13-04-16

Date	Time	Elapsed time (min)	Temp C	Actual Hydro.Rdg .Ra	Hyd.corr.f or meniscus	L	K	L/T	$\sqrt{L/T}$	D (mm)	CT	a	Corr.Hydr.Rdg.Rc	Finer P(%)	Adjusted finer PA (%)	
13-04-16	9:40	0	21	1.031	1.032	7.8	0.01348				0.2	1	0	0	0	42.5857143
		1	21	1.026	1.027	9.2	0.01348	9.2	3.03315018	0.040886864	0.2	1	1.225	2.45	1.043455	41.5422593
		5	21	1.017	1.018	11.5	0.01348	2.3	1.51657509	0.020443432	0.2	1	1.216	2.432	1.0357888	40.5064705
		9	21	1.015	1.016	12.1	0.01348	1.34444444	1.15950181	0.015630084	0.2	1	1.214	2.428	1.0340852	39.4723853
		16	21	1.013	1.014	12.6	0.01348	0.7875	0.88741197	0.011962313	0.2	1	1.212	2.424	1.0323816	38.4400037
		31	21	1.012	1.013	12.9	0.01348	0.41612903	0.64508064	0.008695687	0.2	1	1.211	2.422	1.0315298	37.4084739
		139	21	1.009	1.01	13.7	0.01348	0.09856115	0.3139445	0.004231972	0.2	1	1.208	2.416	1.0289744	36.3794995
14-04-16	9:37	1437	20	1.006	1.005	15	0.01365	0.01043841	0.10216855	0.001394601	0	1	1.005	2.01	0.856059	35.5234405



Test no 96
100
90
80
70
60
50
40
30
20
10
0
PASSING%
0.001
0.01
0.1
1
DIAMETER%

2. STANDARD PROCTOR COMPACTION TEST- ASTM D 698

NEAR EAST UNIVERSITY LABROTORY

Test method :- Standard Proctor compaction

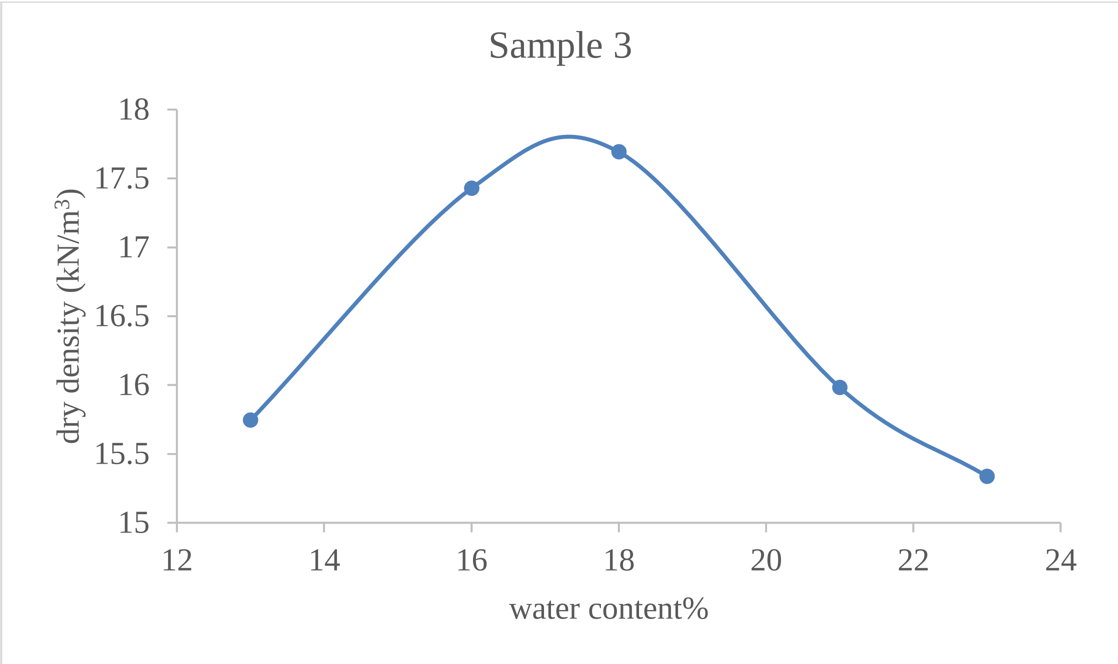
Date testes :- 27/1/2016

Tested by :- Arhaiem Hussain at Near East University

Project name :- Thesis

Sample number :- 3

Test no 3	1	2	3	4	5
Volume of mold (cm^3)	944	944	944	944	944
Assumed water content	13%	16%	18%	21%	23%
weight of the mold (gr)	4080	4080	4080	4080	4080
weight of mold and moist soil(gr)	5803.2	6050.6	6115.4	5948.2	5902.6
weight of moist soil (gr)	1723.2	1970.6	2035.4	1868.2	1822.6
moisture density (gm/cm^3)	1.83	2.09	2.16	1.98	1.93
moisture can number	S20	2S4	S13	S18	S19
weight of moisture can (gr)	40.4	38.1	38.6	38.7	33.5
weight of can +moist soil (gr)	230.8	187.8	246.5	288.2	307.9
weight of can +dry soil (gr)	207.8	165.5	212.5	244.1	255.7
water content %	13.74	17.50	19.55	21.47	23.49
dry density (gm/cm^3)	1.61	1.78	1.80	1.63	1.56
dry density (kN/m^3)	15.79	17.46	17.66	15.99	15.30



optimum water content	17%
maximum dry density	17.92 kN/m^3

3-ATTERBERG LIMIT TEST-ASTM D 4318

NEAR EAST UNIVERSITY LABROTORY

Test method :- Atterberg limit test

Date testes :- 15/12/2015

Tested by :- Arhaiem Hussain at Near East University

Project name :- Thesis

Sample number :- 20

sample 20	LL			PL	
number of blows	28	25	18	0	0
container number	S15	S19	S18	S5	S7
wight of container(gm)	40.9	33.3	39.1	26.1	22.4
wight of container +wet soil(gm)	46.4	39.5	46.7	28.4	24.6
wight of container +dry soil(gm)	43.9	36.6	43	27.8	24.1
wight of water	2.5	2.9	3.7	0.6	0.5
wight of dry soil(gm)	3	3.3	3.9	1.7	1.7
water content %	83.33	87.88	94.87	35.29	29.41
				32.35	

Liquid limit	Plastic limit	Plasticity index
87.5	32.35	55.15