PREDICTION OF COMPACTION CHARACTERISTICS OF LATERITIC SOILS IN GHANA

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Ellen ADU PARKOH: PREDICTION OF COMPACTION CHARACTERISTICS OF LATERITIC SOILS IN GHANA

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To my family...

ABSTRACT

Soil is one of the most common construction materials. Naturally occurring soils need improvement in their engineering properties. The determination of these engineering properties becomes a vital process for the successful design of any geotechnical structure. Laboratory determination of compaction properties namely; maximum dry unit weight (γ_{dmax}) and optimum water content (w_{opt}) is laborious and time - consuming in view of large quantities of soils.

In this study, an attempt to develop predictive models between Atterberg limit, Gradational parameters and compaction test parameters is made. To achieve this purpose, 168 lateritic soils in Ghana were subjected to Atterberg limit, Gradation and compaction laboratory tests. 77 samples were tested using standard Proctor and 70 samples for modified Proctor compaction tests.

Stepwise multiple linear regression analyses were carried out on the experimental data and predictive models were developed in terms of liquid limit (w_L), plasticity index (I_p) and fines content percentage (FC). A new set of 21 samples, 11 for standard Proctor and 10 for modified Proctor were obtained and their compaction results were used to validate the proposed models.

The results showed that these proposed models had R^2 values greater than 70% and the variation of error between the experimental and the predicted values of compaction characteristics was less than ± 2. It has been shown that these models will be useful for a preliminary design of earthwork projects which involves lateritic soils in Ghana.

***Keywords**:* Lateritic soils; compaction; Ghana; standard Proctor; modified Proctor; stepwise regression; models

ÖZET

Zemin doğada en fazla bulunan yapı malzemesidir. Doğal oluşumlu zeminlerin mühendislik özelliklerinin artırılması gerekir. Mühendislik özelliklerinin belirlenmesi başarılı bir yapının tasarımı için önemli bir süreçtir. Kompaksiyon (sıkıştırma) en önemli zemin iyileştirme tekniklerinden birisidir. Maksimum kuru birim hacim ağırlığı ($\gamma dmax$) ve optimum su içeriği ($wopt$) gibi kompaksiyon özelliklerinin laboratuvarda belirlenmesi yorucu ve fazla vakit gerektirir.

Bu çalışmada, Atterberg (kıvam) limitleri, dane çapı dağılımı parametreleri ve kompaksiyon (sıkıştırma) parametreleri arasında öngörü modellerinin geliştirilmesi için bir girişim yapılmıştır. Bu amaç doğrultusunda Gana'da 168 lateritik zemin üzerinde Atterberg (kıvam) limitleri, dane çapı dağılımı parametreleri ve kompaksiyon (sıkıştırma) laboratuvar testlerine tabi tutulmuştur. 77 numune standart Proktor kullanılarak, 70 numune değiştirilmiş Proktor sıkıştırma testleri kullanılarak test edildi.

Deneysel veriler üzerinde aşamalı çoklu doğrusal regresyon analizleri yapılmış ve öngörü modelleri, likit limit (w_L), plastisite indeksi (I_p) ve ince tane içerik yüzdesi (FC) yönünden geliştirilmiştir. 11 standart Proktor, 10 değiştirilmiş Proktor için olacak şekilde 21 numuneli yeni bir dizi elde edilmiş ve bunların sıkıştırma sonuçları önerilen modelleri doğrulamak için kullanılmıştır.

Öngörü modelleri, standart ve değiştirilmiş Proktor sıkıştırma parametreleri için belirgin biçimde önerilmiştir. Sonuçlar, önerilen modellerin R^2 değerlerinin %70'ten fazla olduğunu ve kompaksiyon özelliklerinin deneysel ve öngörülen değerleri arasındaki hata varyasyonunun ±2den az olduğunu göstermiştir. Ayrıca, bu modellerin Gana'daki lateritik zeminleri içeren hafriyat projelerinin ön tasarımı için yararlı olacağı gösterilmiştir.

Anahtar Kelimeler: Lateritik zeminler; zemin kompaksiyonu (sıkıştırması); Gana; standart Proktor; değiştirilmiş Proktor; aşamalı regresyon; öngörü modelleri

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LIST OF SYMBOLS AND ABBREVIATIONS

CE	Compaction energy(kN-m/m^3)
CL	Lean clay
C_u	Uniformity coefficient
E	compaction energy (unknown) kJ/m^3
E_k	compaction energy (known) kJ/m^3
FC	Fines content
G	Gravel content
GC	Clayey gravel
G_s	Specific Gravity
I_p	Plasticity index
MP	modified Proctor compaction test
n	Sample size
p	Number of selected independent variables
R^2	Coefficient of determination
RP	Reduced Proctor compaction test
S	Sand content
SC	Clayey sand
SM	Silty sand
SP	standard Proctor compaction test
SEE	Standard Error of Estimate
SSE_p	the sum of squares of the residual error for the model with p parameters
SST	Total sum of squares
w_L	liquid limit
w_{opt}	Optimum water content
w_p	plastic limit
ρ_{dmax}	Maximum dry density
γ_{dmax}	Maximum dry unit weight

CHAPTER 1

INTRODUCTION

1.1. Background

Compaction of soil is a conventional soil modification method by the application of mechanical energy to improve the engineering properties of the soil. The soil is densified by the removal of pore spaces and the particles are rearranged. Since the soil particles are closely packed together during this process, the void ratio is reduced thus making it difficult for water or other fluid to flow through the soil.

Due to the automobile invention in the 20^{th} century, soil compaction investigations were initiated along the roads. Since then, many efficient and cost effective methods came up; different compaction methods were used for different type of soils. Proctor, a pioneer in soil compaction established this fact in 1933. It was also established that the moisture content affected the degree of compaction for any compaction method used.

The soil phase is comprised of the solid, the liquid, and the gaseous phase. The liquid and gaseous phases are known as the void ratio. The solid phase is made up of mineral particles of gravels, sands, silts, and clays. The particle size distribution method is used to determine the range of soil particles. The liquid phase consists primarily of water and the principal component of the gaseous phase is air.

Soil compaction just affects the air volume and has no effect on the water content or the volume of solids. The air ratio in the void ratio is to be removed completely during an efficient compaction process, however, in practice, this is not so. The diminution of the pore spaces leads to rearrangement of the soil particles making it denser.

The importance of this property is well appreciated in the construction of earth dams and other earth filling projects. It is a vital process and is employed during the construction projects such as; highway, railway subgrades, airfield pavements, landfill liners and in earth retaining structures like Tailings Storage Facility (TSF). The main goals of soil compaction are:

i. Reduction in permeability of the compacted soil,
ii. Increase in the shear strength of the soil and,
iii. To reduce the subsequent settlement of the soil mass under working loads.

In the laboratory, soil compaction is conducted using the Proctor compaction test device. In the field, the compaction of the soil is achieved by different equipment with different compaction energy. The characteristics of the compaction test are optimum water content (w_{opt}) and maximum dry density or unit weight. (ρ_{dmax} or γ_{dmax}). These parameters are used to determine the shear strength and bearing capacity of the subgrade, platforms, landfills etc.

1.2. Problem Statement

Considerable time, effort and cost is used during a compaction test in order to determine the optimal properties i.e. maximum dry unit weight and optimum water content hence, there is the need to develop predictive models using simple soil tests like Atterberg limit tests and Gradation tests especially, when these are known already from project reports, bibliographies, and from database of the engineering properties of quarried soil within the geographical area or soils of similar properties. The predicted maximum dry unit weight and optimum water content can be used for the preliminary design of the project.

1.3. Hypothesis

This dissertation will test whether it is possible to estimate the compaction characteristics of lateritic soils from Atterberg limit test and Gradation parameters.

1.4. Research Objectives

The main objective of this study is to determine the relationship between the compaction test characteristics both standard and modified Proctor compaction test and the other soil variables such as Atterberg limit test parameters and Gradation properties of lateritic soils in Ghana. Thus, the specific goals are:

i. To develop an appropriate empirical predictive model relating optimum water content to Atterberg limit test parameters and Gradation properties of lateritic soils in Ghana.

ii. To develop an appropriate empirical predictive model relating maximum dry unit weight to Atterberg limit test parameters and Gradation properties of lateritic soils in Ghana.
iii. To validate the empirical models and draw appropriate conclusions from them.

1.5. Organization of the Study

In order to successfully accomplish the above objectives, the following scope of activities was performed and a flow chart presenting the activities is shown in Figure 1.1.

The first Chapter highlights the introduction of the subject study. The second Chapter deals with the review of published literature (thesis, journals, and conference papers). A discussion of the methodology of the research area, test samples, and test procedures were conducted in Chapter 3. In Chapter 4, the regression analysis and the developed correlations for the variables were carried out. Comparison of the developed models with other existing models was also performed under this chapter.

Lastly, the conclusions and recommendations of the study are given in Chapter 5. Enclosed in the Appendix section are the details of the test methods and some laboratory test results. The structure of the thesis is presented in the flow chart shown below:

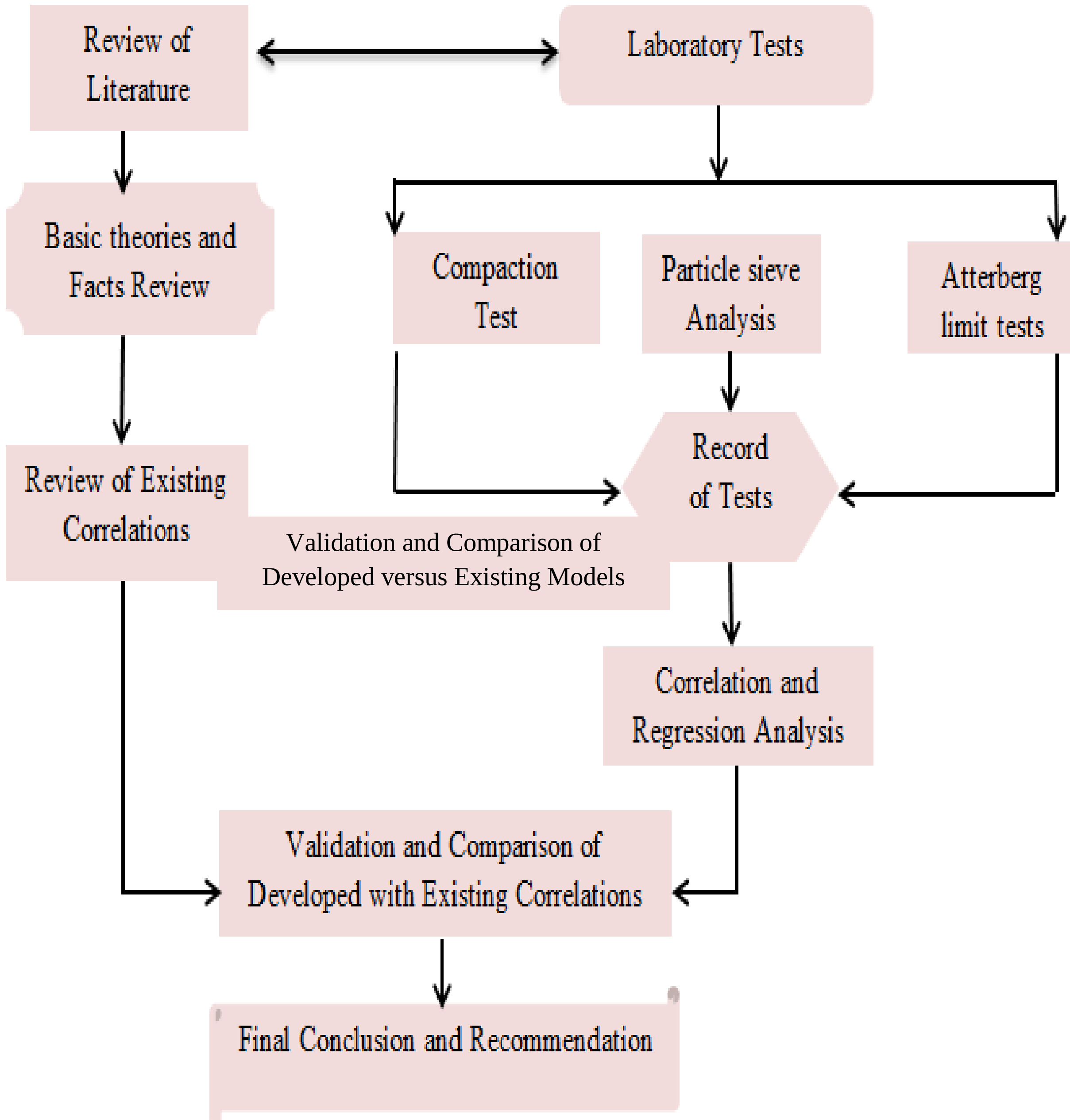


Figure 1.1: Flow chart of the study

CHAPTER 2

LITERATURE REVIEW

2.1. Background

Soil compaction is defined as a mechanical process of increasing the density of a soil by reducing the air volume from the pore spaces (Holtz et al., 2010). This leads to changes in the pore space size, particle distribution, and the soil strength. The main aim of the compaction process is to increase the strength and stiffness of the soils by reducing the compressibility and to decrease the permeability of the soil mass by its porosity (Rollings and Rollings, 1996). The type of soil and the grain sizes of the soil play a significant role in the compaction process as a reduction in the pore spaces within the soil increases the bulk density. Soils with higher percentages of clay and silt have a lower density than coarse-grained soils since they naturally have more pore spaces.

The compaction curve obtained in the laboratory tests or field compaction represents the typical moisture-density curve which explains the compaction characteristics theory (Hausmann, 1990).

Proctor (1933), pioneered the procedure of determining the maximum density of a soil as a function of the water content and compactive effort. Since then, many studies have been carried out on the basic phenomena. The concept of lubrication, pore water and air pressures, and the soil microstructures were studied under different theories. Each of these theories has its merits and demerits as soil mechanics was at the state of its development during that era and the nature of the soil and the compaction method employed in obtaining the experimental data played a significant role.

2.2. Soil compaction

Soil compaction is a common process in today's construction, it is employed in earthworks constructions, like roads and dams and the foundation of structures. The standard requirement for soil compaction in the field is more than 90% or 95% of the laboratory maximum dry unit weight. Effective methods have to be employed in order to measure soil compaction in the field as visual inspection cannot be used to determine whether the soil is compacted or not. The most common measure of compaction is bulk density (weight per unit volume).

*Compaction***:** The process of packing soil particles closely by the expulsion of the pore space, usually by mechanical means, increasing the density of the soil.

Optimum water content (w_{opt}*):* The water content of the soil at which a specified amount of compaction will generate maximum dry density.

Maximum dry density: The dry density obtained using a specified amount of compaction at the optimum water content

Dry density-water content relationship: The relationship between dry density and water content of a soil under a given compactive effort.

Percentage air voids (V_a*):* the volume of air voids in a soil expressed as a percentage of the total volume of the soil.

Air voids line: A line showing the dry density-water content relationship for a soil containing a constant percentage of air voids.

Saturation Line (Zero air void line): The line showing the dry density-water content relationship for a soil containing no air voids.

2.2.1. Compaction characteristics of soils

The water content placed and the compaction effort affects the density of the soil that is used as fill or backfill. Typical engineering properties of compacted soils are presented in Table 2.1.

Table 2.1: Typical engineering properties of compacted soils

(US. Army Corps of Engineers, 1986).

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, gm/cm^3 (lbs/ft^3)	Range of Optimum Moisture, Percent	Typical Value of Compression		Typical Strength Characteristics				Typical Coefficient of Permeability cm/min (ft/min)	Range of CBR Values	Range of Subgrade Modulus k gm/cm^3 (lbs/ft^3)
				At 134 kPa (1.4 tsf)	At 345 kPa (3.6 tsf)	Cohesion (as Compacted) kPa (lb/ft^2)	Cohesion (Saturated) kPa (lb/ft^2)	Effective Stress Envelope Deg	Tan			
GW	Well graded, clean gravel – sand mixtures	2.002 – 2.162 (125 – 135)	11 – 8	0.3	0.6	0	0	>38	>0.79	9×10^{2} (3×10^{2})	40 – 80	8,300 – 13,800 (300 – 500)
GF	Poorly graded, clean gravel – sand mixtures	1.842 – 2.002 (115 – 125)	14 – 11	0.4	0.9	0	0	>37	>0.74	3.0 (10^{1})	30 – 60	6,900 11,100 (250 – 400)
GM	Silty gravels, poorly graded gravel-sand-silt	1.922 – 2.162 (120 – 135)	12 – 8	0.5	1.1	--	--	>34	>0.67	3×10^{4} ($>10^{6}$)	20 – 60	2,800 – 11,100 (100 – 400)
GC	Clayey gravels, poorly graded gravel-sand-silt	1.842 – 2.082 (115 – 130)	14 – 9	0.7	1.6	--	--	>31	>0.60	3×10^{6} ($>10^{7}$)	20 – 40	2,800 – 8,300 (100 – 300)
SW	Well graded clean sands, gravelly sands	1.762 – 2.082 (110 – 130)	16 – 9	0.6	1.2	0	0	38	0.79	3×10^{2} ($>10^{3}$)	20 – 40	5,500 – 8,300 (200 – 300)

Table 2.1: Continued.

SP	Poorly graded clean sands, sand gravel mix	1.602 – 1.922 (100 – 120)	21 – 12	0.8	1.4	0	0	37	0.74	3 x 10^{-2} (>10^{-3})	10 – 40	5,500 – 8,300 (200 – 300)
SM	Silty soils, poorly graded sand-silt mix	1.762 – 2.002 (110 – 125)	16 – 11	0.8	1.6	50 (1050)	20 (420)	34	0.67	1.5 x 10^{-3} (>5x > 10^{-5})	10 – 40	5,500 – 8,300 (200 – 300)
SM – SC	Sand-silt clay mix with slightly plastic fines	1.762 – 2.082 (110-130)	15 – 11	0.8	1.4	50 (1050)	14 (300)	33	0.68	>6 x 10^{-5} (>2 x 10^{-6})	5-30	2,800 – 8,300 (100-300)
SC	Clay like sands, poorly graded sand/ clay mix.	-2.002 (105-125)	19-11	1.1	2.2	74 (1550)	11 (230)	31	0.60	>2 x 10^{-7} (>2 x 10^{-7})	5-20	2,800 – 8,300 (100-300)
CL	Inorganic clays of low to medium plasticity	1.922 (95-120)	24-12	1.3	2.5	86 (1800)	13 (270)	28	0.54	>3 x 10^{-8} (>10^{-7})	15 or less	1,400 – 5,500 (50-200)
OL	Organic silts and silt clays low plasticity	80-100	33-21	***	***	****	*****	***	***	***	5 or less	1,400 – 2,800 (50-100)

2.3. Compaction Theory

Field density tests usually give an indication of the performance of a standard laboratory compaction test on the material since it relates to the optimum water content and maximum dry density of the in-place material on the site. Field density testing is a must in earthworks fills and the laboratory compaction tests characteristics of the material is used as a reference. It is possible to test in the field since it does not keep pace with the rate of fill placement.

Nonetheless, before the commencement of any construction, standard compaction tests should be performed on the materials to be used for the construction during the design stage in order to be used as criteria during the construction phase. There is also a need to perform the tests on a newly borrowed material, and when a material similar to that being placed has not been tested previously. There should be a periodic laboratory compaction test on each fill material type so as to check the maximum dry density and optimum water content being used for correlation with field density test results.

Mitchell and Soga (2005) stated that the mechanical behaviour of a fine-grained soil is significantly influenced by the nature and magnitude of compaction. It is generally known that when a clayey soil is compacted to a given dry density (or relative compaction), it is stiffer if it is compacted wet of optimum.

Lambe and Whitman (1969), Hilf (1956), and Mitchell and Soga (2005) attributed this effect to soil fabric, as a result of different remolding water contents. However, these references imply that for sand, the drained shear strength and compressibility are independent of the remolding water content; i.e., these properties are uniquely determined, once the relative compaction, or void ratio, is specified.

The composition of soil is organic matter, minerals and pore space. The mineral fraction of the soils consists of gravel, sand, clay, and silt. There have been several studies on clay mineralogy as they play a significant role on the water holding content of the soil. There are pore spaces between gravel, sand, silt, and clay particles and these can be filled completely by air in the case of dry soil, water in a saturated soil or by both in a moist soil. As said previously, the compression of soil by reducing the pore spaces is compaction, and an important factor to the soil compaction potential is the amount of water in the soil. A dry soil is not easily compacted due to the friction between the soil particles hence the need of water as it serves as a lubricant between the particles.

However, a very wet or saturated soil does not compact well as a moderately moist soil. This is an assertion to the fact that as the soil water content increases, a point is reached when the pore space is filled completely with water, not air. Since water is incompressible, water between the soil particles carries some of the load thus resisting compaction.

Compaction can be applied to improve the properties of an existing soil or in the process of placing fill. There are three main objectives:

i. Reduction in permeability of the compacted soil,

ii. Increase in the shear strength of the soil and,
iii. To reduce the subsequent settlement of the soil mass under working loads

Mitchell and Soga (2005) also found that the samples compacted dry of optimum were to be stiffer than samples compacted wet-of-optimum at the same relative compaction. This difference in stress-strain behaviour is not generally expected for sand; fabric and/or over-consolidation may explain these results. Thus, for the case of shallow depth (such as backfill for a flexible conduit located within a few meters of the ground surface), it is important to consider the water content and the method of compaction, as the degree of compaction by itself will not necessarily achieve the desired modulus.

2.4. Factors affecting compaction

Researchers such as Turnbull and Foster (1956) cited in Guerrero (2001), D'Appolonia et al. (1969), Bowles (1979), and Holtz et al. (2010) have identified the soil type, molding water content, compaction effort, and method 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.

2.4.1. Effect of soil type

Soil parameters such as initial dry density, grain size distribution, particle shape, and molding water content are important material properties in controlling how well the soil can be compacted (Rollings and Rollings, 1996; Holtz et al. 2010). Different soils may show different compaction curves as is shown in Figure 2.1.

Coarse- graded soils like well-graded sand (*SW*) and well-graded gravel (*GW*) are easier and more efficient to compact using vibration since the particles are large and gravity forces are greater than surface forces. Furthermore, they may have two peaks in the compaction curve; this means that a completely dry soil can be compacted at the same density using two different optimum water contents (Rollings and Rollings, 1996). Also coarse-grained soils tend to have a steeper compaction curve, making them more sensitive to changes in molding water content (Figure 2.1).

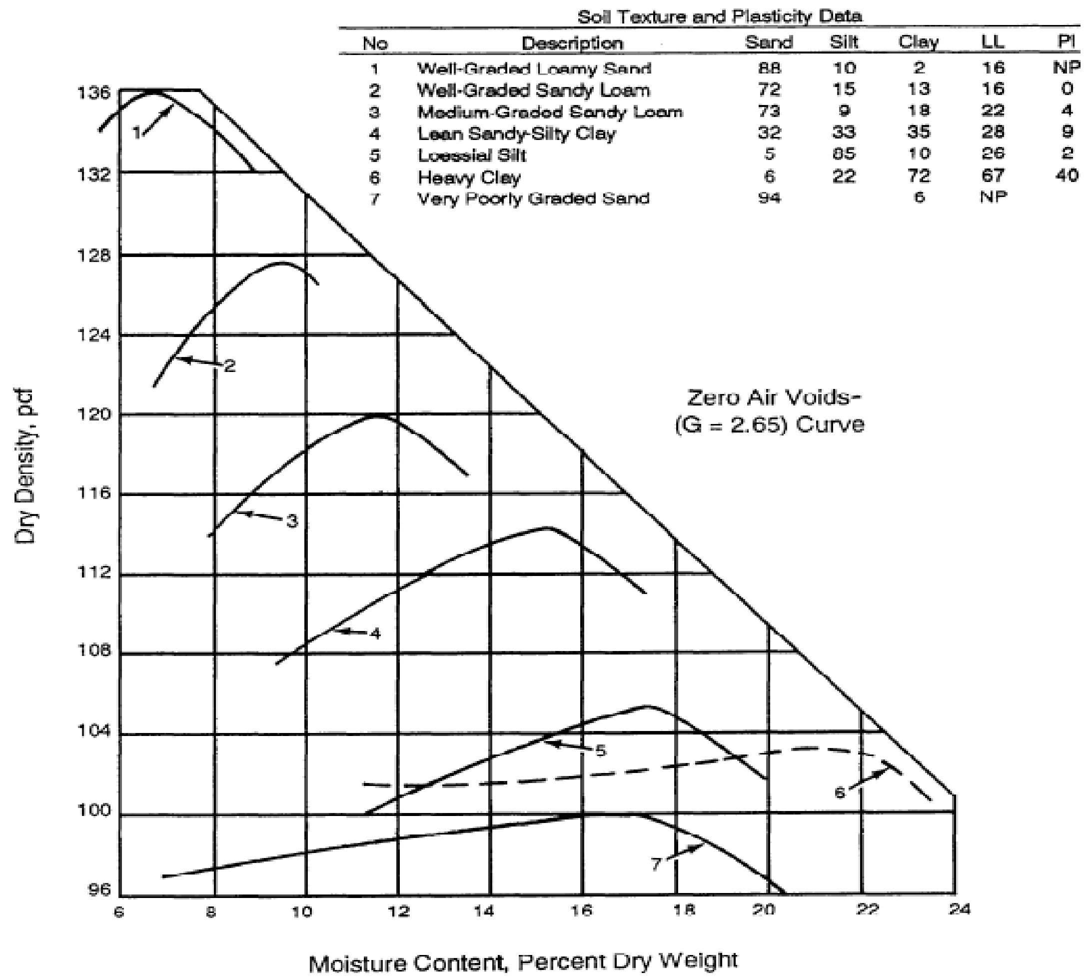


Figure 2.1: Compaction curves for different types of soils using the standard effort (Rollings and Rollings, 1996 after Johnson and Salberg, 1960)

The compaction method and the compactive effort have a higher influence in the final dry density of finely graded soils, than in coarse graded soils (Bowles, 1979). As is shown in Figure 2.1, the shape of the compaction curve when the soil has a larger content of silt or clay has a sharp peak. When the soil is more plastic the difference of compaction curves for standard effort and modified effort is larger (Rollings and Rollings, 1996).

2.4.2. Water content

The amount of water added to the soil during the compaction process may be controlled. The optimum water content determined by Proctor test is added to the soil in order to attain the standard specifications (90% or 95% of the maximum dry density measured by the ASTM

D698-12).

According to Mitchell and Soga (2005), it is recommendable to use different molding water contents than the optimum water content, since different water contents may give a range of soil properties. Compacting the soil at the dry or wet side of the optimum water content yields different soil fabric configurations which allow a range of suction and conduction phenomena such as hydraulic and thermal conductivity.

Daniel and Benson (1990) propose different ranges of water content and dry density for a compacted soil to be used as an impervious barrier or liner (low hydraulic conductivity) or zones where it may be used as embankment where low compressibility and high shear strength are needed. Table 2.2 and Figure 2.2 show different ranges of molding water content in terms of soil properties and applications.

Table 2.2: Acceptable range of water content (Daniel and Benson, 1990).

Compactive efforts	**Acceptable range of water content (%) for hydraulic conductivity**	**Acceptable range of water content (%) for Volumetric shrinkage**	**Acceptable range of water content (%) Unconfined compressive Strength**
Modified Compaction	16.5 to $>$26	$<$16 to 21.1	$<$16 to 23.3
Standard Compaction	25.1 to 31.9	$<$22 to 23.1	$<$22 to 29
Reduced Compaction	27.1 to 27.9	$<$23 to 23.8	$<$23 to 28.8

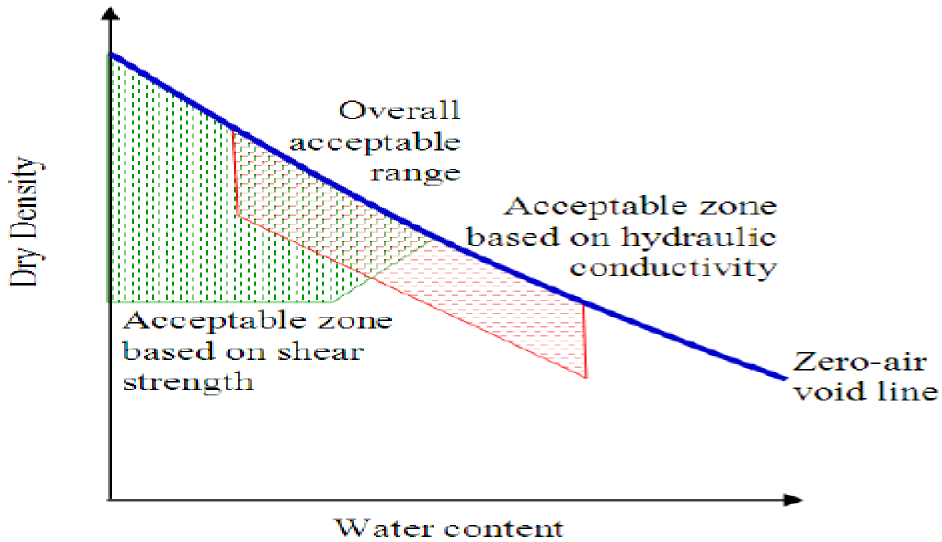


Figure 2.2: Scheme of ranges of soil properties and applications as a function of molding water content (Daniel and Benson, 1990)

The matric suction of a compacted soil changes the shape of the soil-water characteristic curve (*SWCC*) due to different pore structures or soil fabrics created during the compaction process (Tinjum et al., 1997). Figure 2.3 shows the differences in the *SWCC* for a clay soil *CL* and *CH* compacted at the dry side, wet side and optimum water content using different compactive effort. As matric suction and thus, the long-term water content of the fill is affected by the molding water content at compaction, other soil properties such as the small strain shear modulus and the thermal conductivity are affected in the long-term as well.

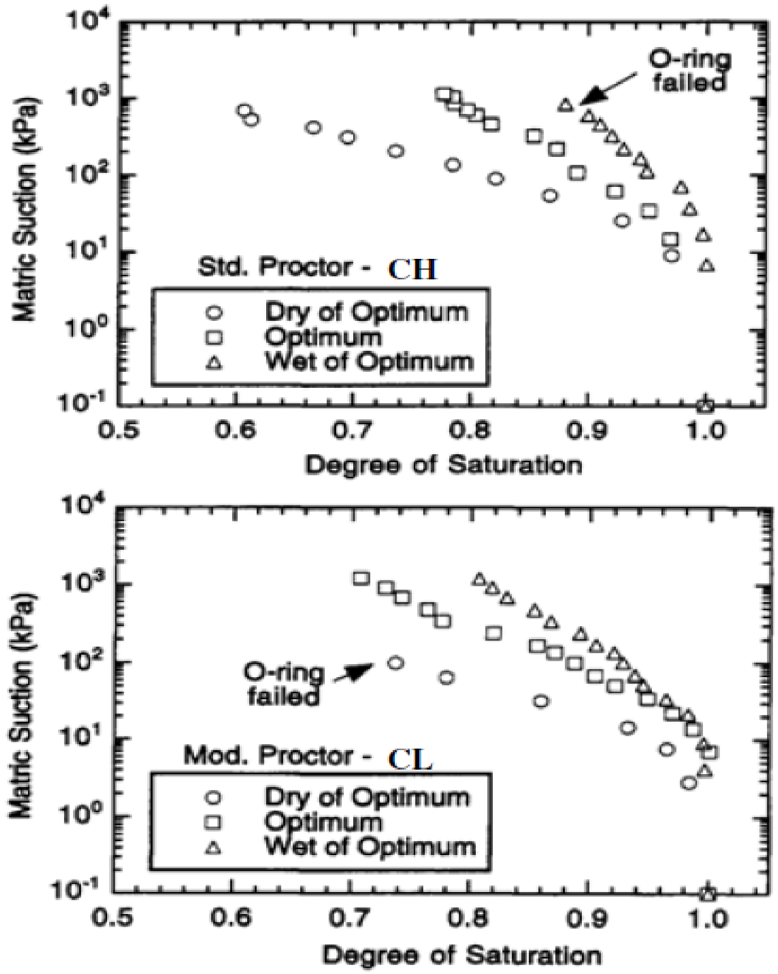


Figure 2.3: *SWCC* for a *CH* and *CL* soil compacted at dry of optimum, wet of optimum and optimum water content (Tinjum et al. 1997)

2.4.3. Compaction effort

As mentioned previously, compaction of soil is reducing the pore space in the soil. In controlling the final reduction of the void ratio during this mechanical process, the compactive effort is one of the most important variables to control this. Hence, there is a need to know how the compactive effort affects the soil in compaction process. The compactive effort is the amount of energy or work necessary to induce an increment in the density of the soil. D’Appolonia et al. (1969) cited in Guerrero (2001) stated that the compactive effort is controlled by a combination of the parameters such as weight and size of the compactor, the frequency of vibration, the forward speed, the number of roller passes, and the lift height.

The measurement of the compactive effort is specific energy value *(E);* applied energy per unit volume. The energy applied has a positive relation with the maximum dry unit weight and a negative relation with the optimum water content. Thus, an increase in the applied energy increases the maximum unit weight and decreases the optimum water content. This is represented in Figure 2.4.

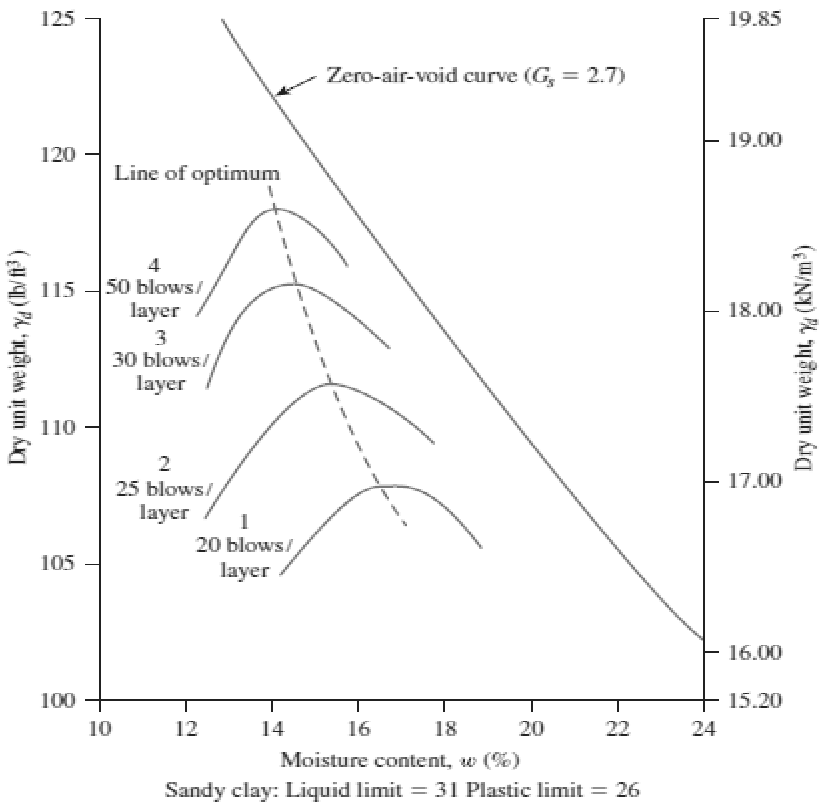


Figure 2.4: Effect of compaction energy on the compaction of sandy clay (Das, 2010)

It can be seen that when the energy is increased all the densities are higher between the moisture contents range. The process efficiency is better for lower water contents and becomes practically useless when the water content is too high. A common characteristic among the shown curves is that when the water content is very high, the compaction curves tend to come closer. Another detail is that after the maximum value in the compaction curves is reached, the curves tend to align parallel to the Zero Air Void curve (Das, 2010).

The compaction energy per unit volume used for the standard Proctor test can be given by;

$$\mathrm{E} = \frac{1}{\text{Volume of mold}}\left[(\text{Number of blows per layer}) \times (\text{Number of layers}) \times (\text{weight of hammer}) \times (\text{Height of drop of hammer})\right] \tag{2.1}$$

2.4.4. Compaction method

Different shear strength and volumetric stability of soils are produced when soils are compacted using different compaction methods and water content since different compaction methods yield different results (Seed and Chan, 1959 cited in Guerrero, 2001). This is shown in Figure 2.5.

The influence of the compaction method can be observed in Figure 2.6 as well, where the same soil was compacted using different methods of compaction; obtained by (1) laboratory static compaction, 13700 kPa; (2) modified effort; (3) standard effort; (4) laboratory static compaction 1370 kPa; (5) field compaction rubber – tire load after 6 coverages; (6) field compaction sheepfoot roller after 6 passes.

The differences observed are produced by factors acting at laboratory scale for the design, and at field scale during compaction (Holtz et al., 2010). As an example, one of these factors is the presence of oversize material in the field that is not considered in laboratory tests.

Furthermore, particles of soil may break down or degrade under the compaction hammer during the test, increasing the fine content in the specimen (Holtz et al., 2010).

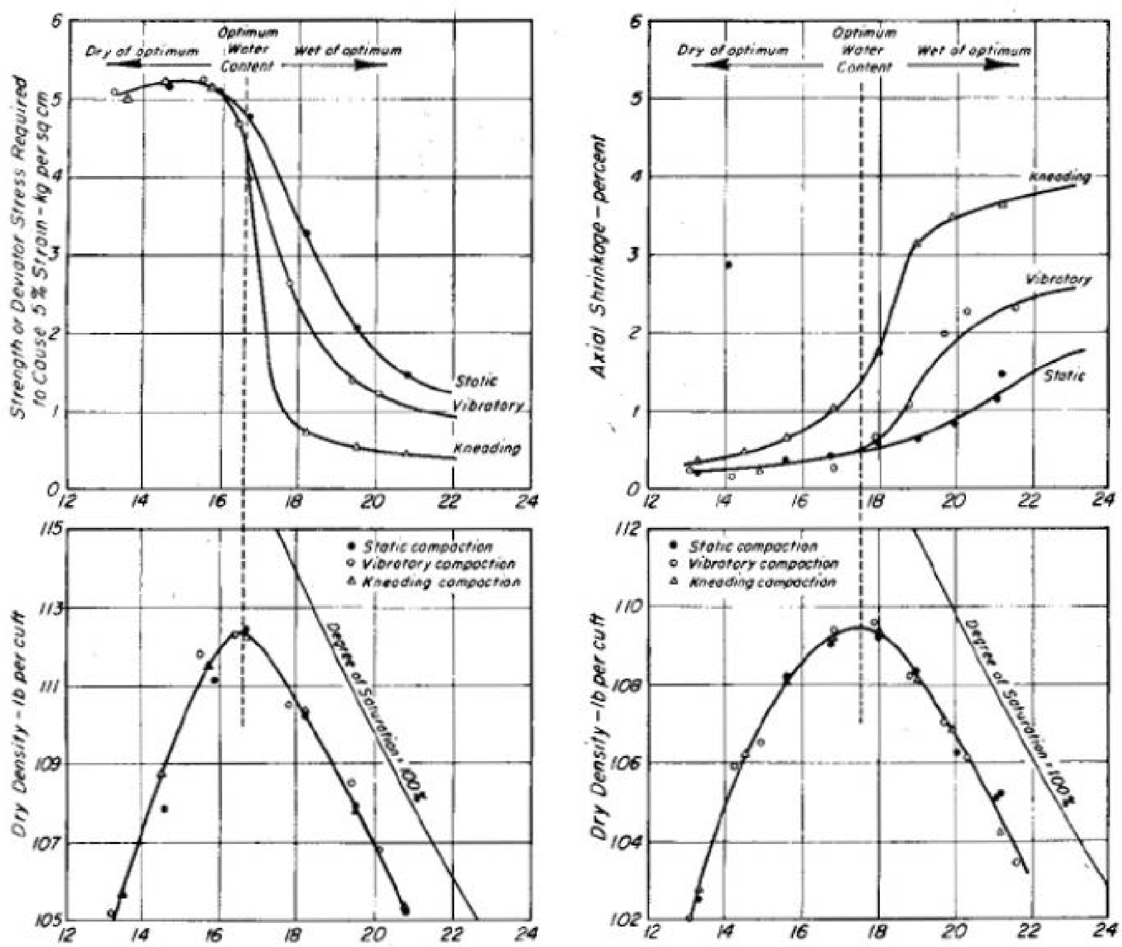


Figure 2.5: Strength and volumetric stability as a function of water content and compaction methods (Seed and Chan, 1959)

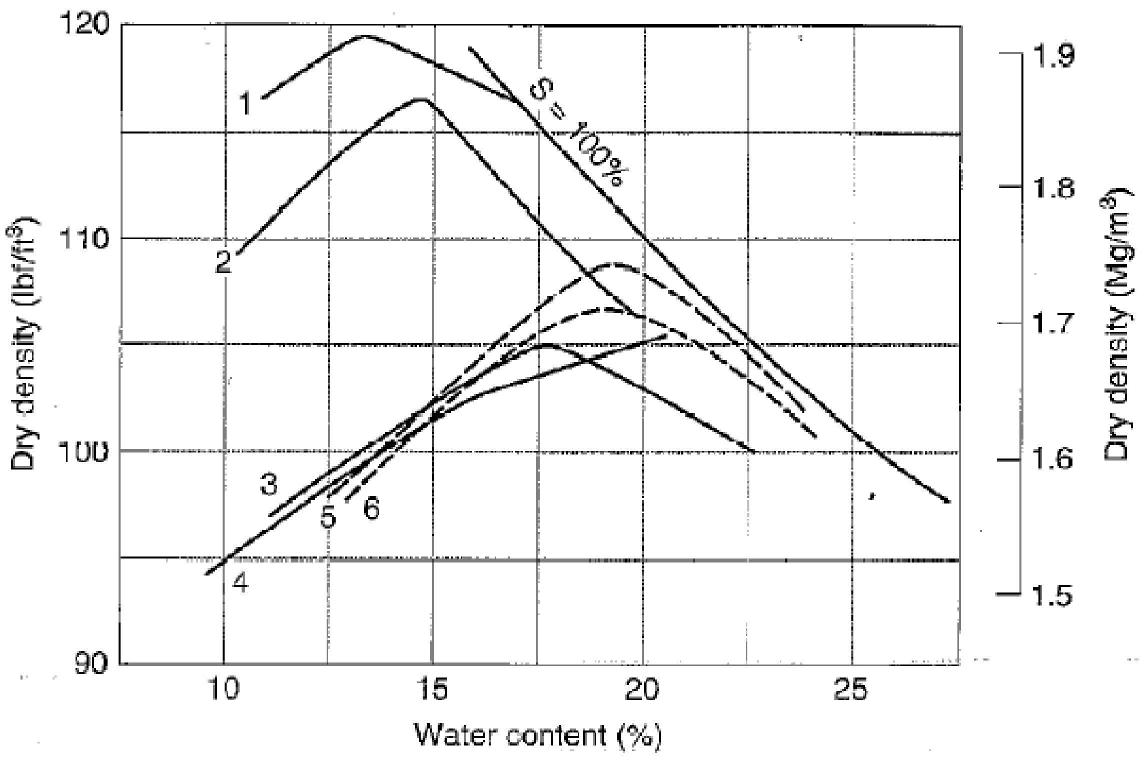


Figure 2.6: Compaction curves by different compaction methods (Holtz et al. 2010 adapted after Turnbull and Foster, 1956)

2.5. Dry-Density versus Water-Content Relationship

Figure 2.7 shows the typical compaction relationship found by Proctor (1933) for different compaction energies. This relationship shows how dry density initially increases when the water content increases until reaching the maximum dry density at the optimum water content. Afterwards, any further increase in the water content leads to a reduction in the dry density. As the energy of compaction increases, similar convex curves are obtained and the curves are shifted to the left and up. That is, increasing compaction energies yield higher dry densities and lower optimal water contents. The Figure also presents the Zero Air Void line and line of optimums. The Zero Air Void line associates the dry unit weight that corresponds to soil fully saturated with water. This line represents a boundary state that cannot be crossed by the compactive process. The line of optimums joins the points that correspond to the maximum dry density and optimum water content for different compaction efforts. The line of optimums corresponds to approximately 75 to 80% degree of saturation (Holtz and Kovacs, 1981).

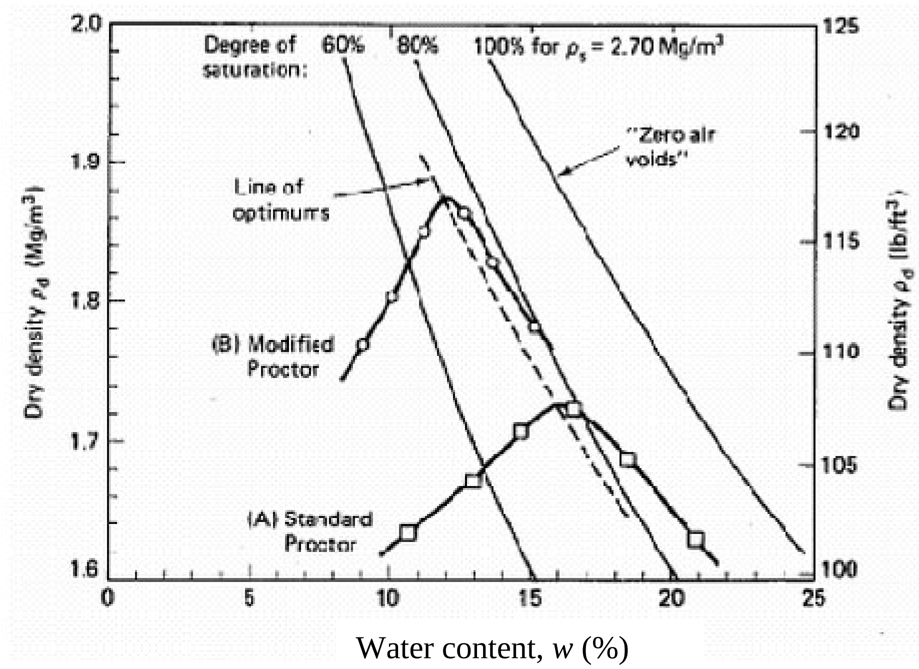


Figure 2.7: Compaction curves with the Zero Air Void line and line of optimus (Holtz and Kovacs, 1981)

2.5.1. The Compaction curve

The compaction curve is the representation of the dry densities versus the water contents obtained from a compaction test. The achieved dry density depends on the water content during the compaction process. When samples of the same material are compacted with the same energy, but with different water contents, they present different densification stages, as shown on Figure 2.8.

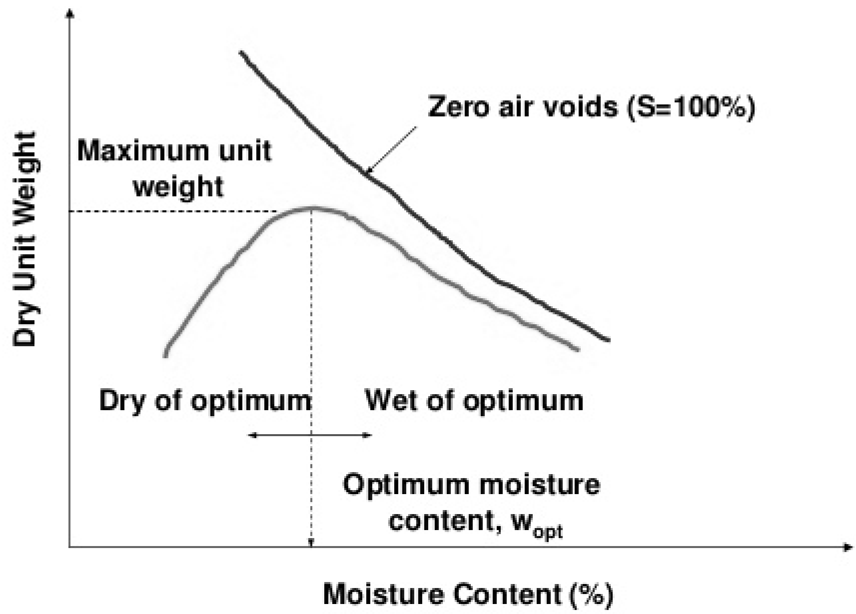


Figure 2.8: Typical compaction moisture/density curve

This densification stage is represented in the compaction curve, which has a particular shape. Many theories have tried to explain the shape of this curve. The principal theories are presented below:

- Proctor (1933) cited in Holtz and Kovacs (1981), believed that the humidity in soils relatively dry creates a capillarity effect that produces tension, stress, and grouping of the solid particles, that results in a high friction resistance that opposes the compaction stresses. For instance, it is very difficult to compact soils with low water content. He obtained a better rearrangement of the soil particles by compacting it with higher water content, because of the increment of lubrication from the water. By compacting the soil whilst the water content is increased, the lubrication effect will continue until a point where the water combined with the remaining air is enough to fill the voids. At this stage, the soil is at its maximum dry density (ρ_{dmax}) and optimum water content (w_{opt}). For any increment in the water content after the "optimum water content", the volume of voids tends to increase, and the soil will obtain a lower density and resistance.
- Hogentogler (1936) cited in Guerrero (2001) considered that the compaction curve shape reflects four stages of the soil humidity: hydration, lubrication, expansion, and

saturation. These stages are represented in Figure 2.9.

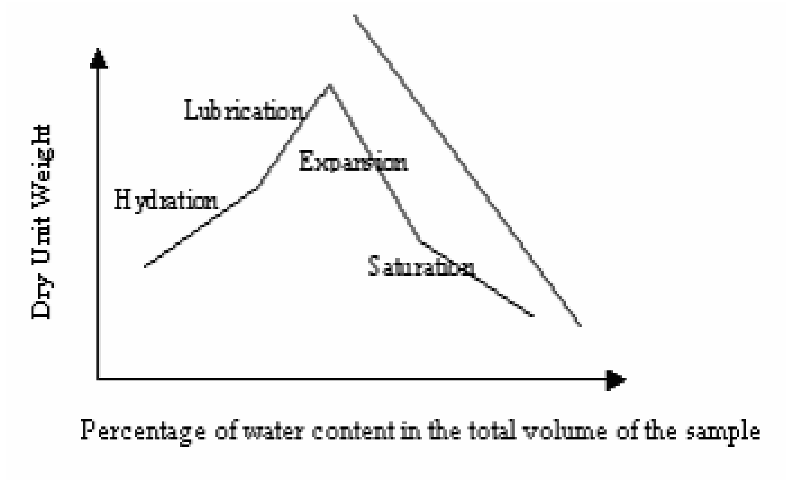


Figure 2.9: Compaction curve (Guerrero, 2001 after Hogentogler, 1936)

As shown in Figure 2.9, Hogentogler's moisture-density curve differs from the Proctor's curve in the abscissa axes. Hogentogler used for this axis the percentage of water content in the total volume of the sample. Hogentogler believed that by using that chart, the compaction curve becomes four straight lines that represent his humectation stages. "Hydration" is the stage where the water incorporation creates a surface coat in the solid particles providing viscosity. "Lubrication" is the stage where the coat is increased by the addition of water acting as a lubricant, and making possible the rearrangement of the soil particles without filling all the air voids. The maximum water content in this stage corresponds to the maximum dry density obtained from the compaction. Hogentogler (1936) cited in Guerrero (2001) believed that more water after the lubrication stage will create the "expansion" of the soil mass without affecting the volume of the air voids, so the additional water in this stage acts in the displacement of the soil particles. The addition of more water to the soil produces its "saturation", which is the stage where the air content is displaced.

Hilf (1956) cited in Guerrero (2001) gave the first modern type of compaction theory by using the concept of pore water pressures and pore air pressures. He suggested that the compaction curve is presented in terms of void ratio (volume of water to the volume of solids). A curve similar to the conventional compaction curve results, with the optimum moisture content corresponding to a minimum void ratio. In his chart the zero air voids curve is shown as a straight line and so are the saturation lines, all originating at zero void ratios and zero water contents. Points representing soil samples with the equal air void ratios (volume of air to the volume of solids) plot on lines parallel to the zero air voids or 100% saturation line.

- According to Hilf, dry soils are difficult to compact because of high friction due to capillary pressure. Air, however, is expelled quickly because of the larger air voids. By increasing the water content, the tension in the pore water decreases, reducing friction and allowing better densification until a maximum density is reached. Less-effective compaction beyond the optimum water content is attributed to the trapping of air and the increment of pore air pressures and the added water taking space instead of the denser solid particles.
- Olson (1963) cited in Guerrero (2001) confirmed that the air permeability of a soil is dramatically reduced at or very close to the optimum water content. At this point, high pore air pressures and pore water pressures minimize effective stress, allowing adjustments of the relative position of the soil particles to produce a maximum density. At water contents below optimum, Olson attributes resistance to repeated compaction forces to the high negative residual pore pressures, the relatively low shear-induced pore pressures, and the high residual lateral total stress. On the wet side of optimum, Olson explains the reduced densification effect by pointing out that the rammer or foot penetration during compaction is larger than in drier soil, which may cause temporary negative pore pressure known to be associated with large strains in overconsolidated soil; in addition, the soil resists compaction by increasing the bearing capacity due to the depth effect.
- Lambe and Whitman (1969) explained the compaction curve based on theories that used the soils surface chemical characteristics. In lower water contents, the particle flocculation is caused by the high electrolytic concentration. The flocculation causes lower compaction densities, but when the water content is increased the electrolytic concentration is reduced.
- Barden and Sides (1970), made experimental researches on the compaction of clays

that were partially saturated, reporting the obtained microscopic observations of the modifications in the clay structure. The conclusions they obtained can be summarized as follows:

1. The theories based on the effective tensions used to determine the curve shape are more reliable than the theories that used viscosity and lubrication.
2. It is logical to suppose that soils with low humidity content remain conglomerated due to the effective tension caused by the capillarity. The dryer these soils are, the bigger the tensions are. In the compaction process, the soil remains conglomerated. By increasing the water content, these tensions are reduced and compaction is more effective.
3. The blockage of the air in the soil mass provides a reasonable explanation of the effectiveness of use compaction energy.
4. If by increasing the water content, the blocked air is not expelled and the air pressure is increased, the soil will resist the compaction.

- Lee and Suedkamp (1972), studied compaction curves for 35 soil samples. They observed that four types compaction curves can be found. These curves are shown in Figure 2.10. Type A compaction curve is a single peak. This type of curve is generally found in soils that have a liquid limit between 30 and 70. Curve type B is a one-and-one-half-peak curve, and curve type C is a double-peak curve. Compaction curves of type B and C can be found in soils that have a liquid limit less than about 30. The compaction curve of type D does not have a definite peak. This is termed an “odd shape”. Soils with a liquid limit greater than 70 may exhibit compaction curves of type C or D, such soils are uncommon (Das, 2010).

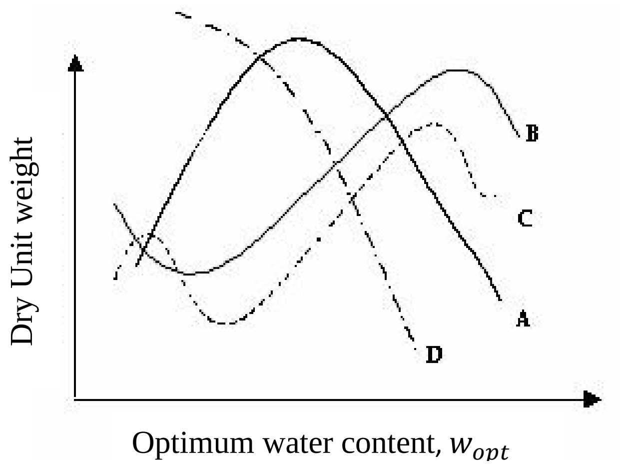


Figure 2.10: Types of compaction curves (Das, 2010)

2.6. Soil Classification

Soils exhibiting similar behaviour can be grouped together to form a particular group under different standardized classification systems. A classification scheme provides a method of identifying soils in a particular group that would likely exhibit similar characteristics. There are different classification devices such as USCS and AASHTO classification systems, which are used to specify a certain soil type that is best suitable for a specific application. These classification systems divide the soil into two groups: cohesive or fine-grained soils and cohesion-less or coarse-grained soils.

2.6.1. Grain size analysis (Gradation)

For coarse-grained materials, the grain size distribution is determined by passing soil sample either by wet or dry shaken through a series of sieves placed in order of decreasing standard opening sizes and a pan at the bottom of the stack. Then the percent passing on each sieve is used for further identification of the distribution and gradation of different grain sizes. Particle size analysis tests are carried out in accordance to ASTM D6913-04. Besides, the distribution of different soil particles in a given soil is determined by a sedimentation process using hydrometer test for soil passing 0.075mm sieve size. For a given cohesive soil having the same moisture content, as the percentage of finer material or clay content decreases, the shear strength of the soil possibly increases.

2.6.2. Atterberg Limits

Historically, some characteristic water contents have been defined for soils. In 1911, Atterberg proposed the limits of consistency for agricultural purposes to get a clear concept of the range of water contents of a soil in the plastic state (Casagrande, 1932). They are liquid limit (w_L), plastic limit (w_P), and shrinkage limit (SL). Atterberg limits for a soil are related to the amount of water attracted to the surface of the soil particles (Lambe and Whitman, 1969).

Therefore, the limits can be taken to represent the water holding capacity at different states of consistency. The consistency limits as proposed by Atterberg and standardized by Casagrande (1932, 1958) form the most important inferential limits with very wide universal acceptance. These limits are found with relatively simple tests, known as Index tests, and have provided a basis for explaining most engineering properties of soils met in engineering practice.

Based on the consistency limits, different indices have been defined, namely, plasticity index (I_p), liquidity index (*LI),* and consistency index *(CI)* (Figure 2.11). These indices are correlated with engineering properties. In other words, all these efforts are principally to classify the soils and understand their physical and engineering behaviour in terms of these limits and indices.

a. *Liquid limit*: The liquid limit (w_L) 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 liquid limit machine (Casagrande apparatus). The liquid limit of a soil highly depends upon the clay mineral present. The conventional liquid limit 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.

b. *Plastic limit:* The plastic limit (w_P) 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 conventional plastic limit test is carried out as per the procedure of AASHTO T 90 or ASTM D 4318-10. 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.

c. *Plasticity Index:* The plasticity index (I_p) is the difference between the liquid limit and the plastic limit of a soil using Equation 2.2,

$$I_p = w_L - w_P \quad (2.2)$$

The Plasticity index is important in classifying fine-grained soils. It is fundamental to the Casagrande Plasticity chart, which is currently the basis for the Unified Soil Classification System.

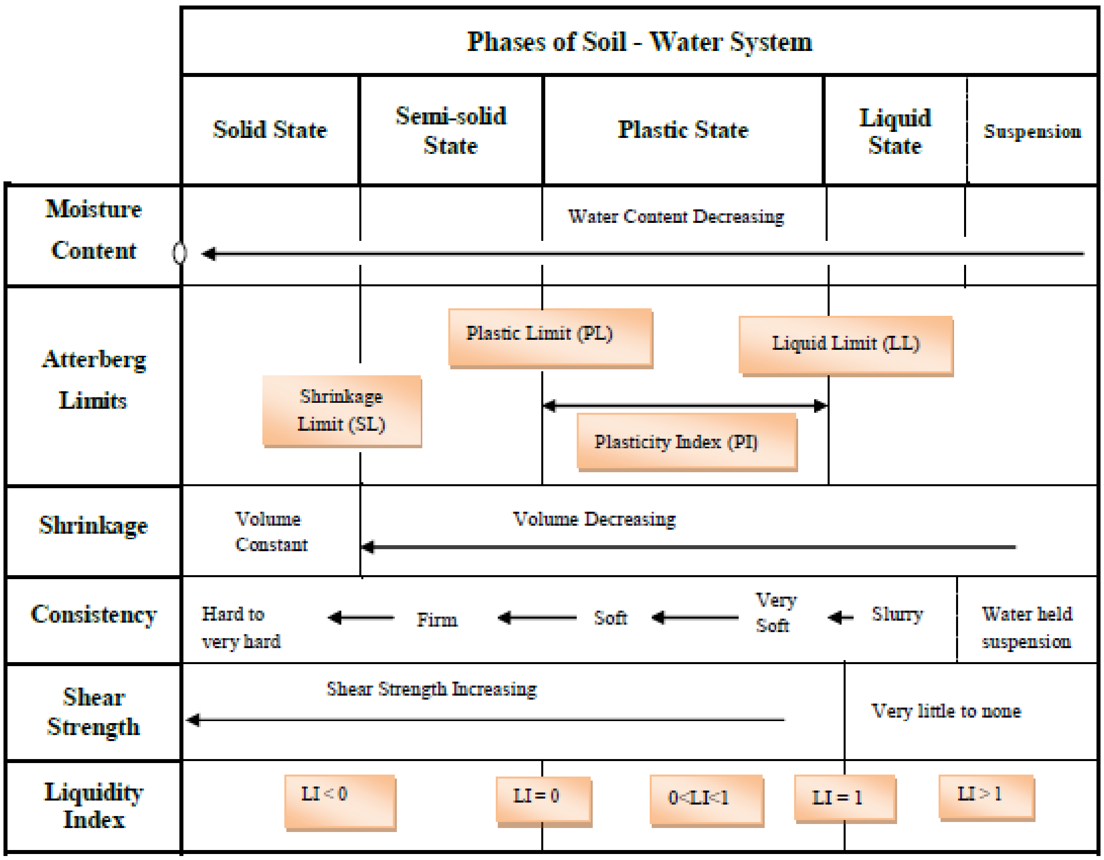


Figure 2.11: Changes of the volume of soil with moisture content with respect to Atterberg limits

2.7. Some Existing Correlations

Many researchers have made attempts to predict compaction test parameters from several factors such as soil classification data, index properties, and grain size distribution.

An early research done by Joslin (1958) was 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 optimum water content, w_{opt} and maximum dry density, ρ_{dmax} 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 standard Proctor 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.

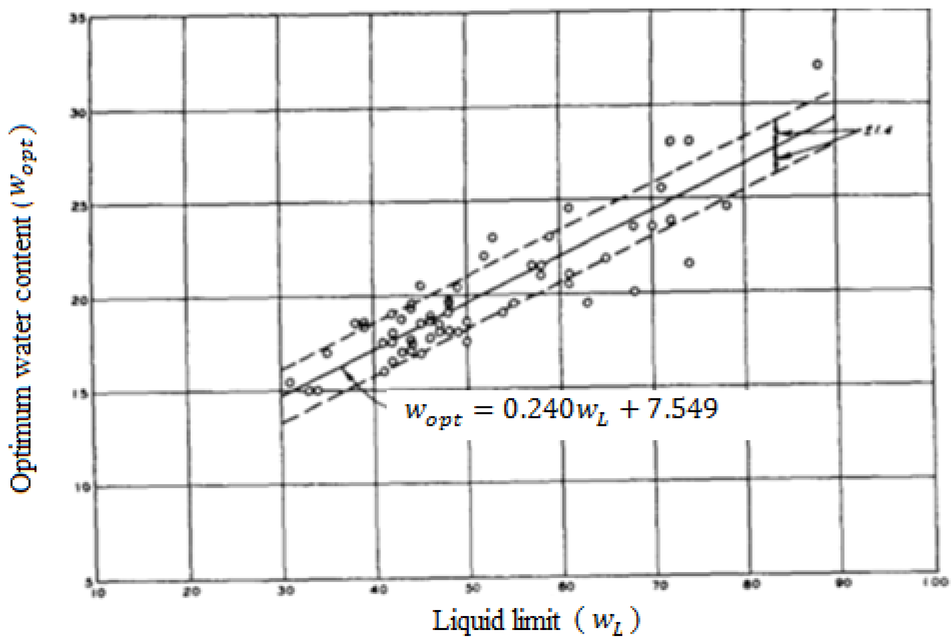
Torrey (1970), in his research, made an interesting discussion on correlating compaction parameters with Atterberg limits. He remarked in this research that in order to determine a mathematical relationship between independent variables, i.e. liquid limit, plastic limit, and dependent variables (optimum water content and maximum dry density) 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 2.12a, 2.12b, 2.13a, and 2.13b represent the results of the analysis done by Torrey (1970). It shows the linear relation between optimum water content and liquid limit (Figure 2.13a) and also Figure 2.13b shows the relation between maximum dry density and liquid limit. These models can estimate 77.6 and 76.3 percent of the variables. Similarly, Figure 2.14 (a) and (b) shows the linear relation between the compaction test parameters with plasticity index. He proposed the following Equations 2.3, 2.4, 2.5, and 2.6:

$$w_{opt} = 0.240w_L + 7.549 \tag{2.3}$$

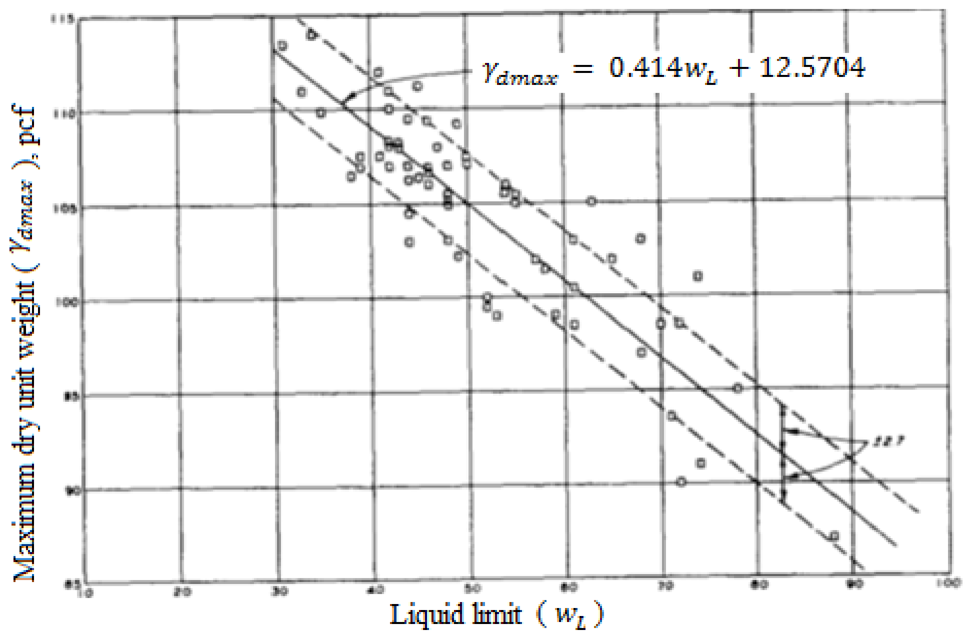
$$\gamma_{dmax} = 0.414w_L + 12.5704 \tag{2.4}$$

$$w_{opt} = 0.263I_p + 12.283 \tag{2.5}$$

$$\gamma_{dmax} = 0.449I_p + 11.7372 \tag{2.6}$$

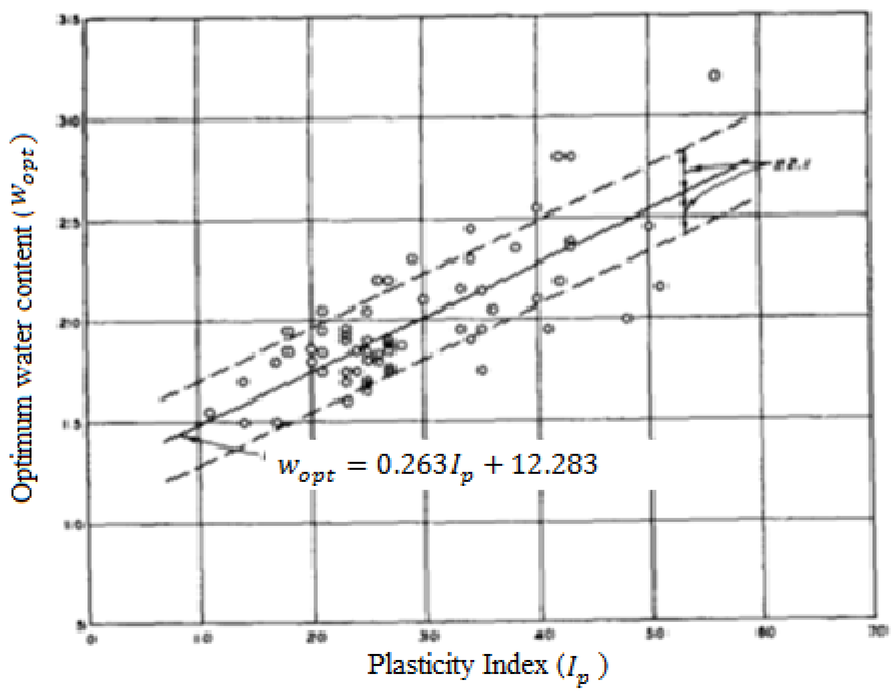


(a)

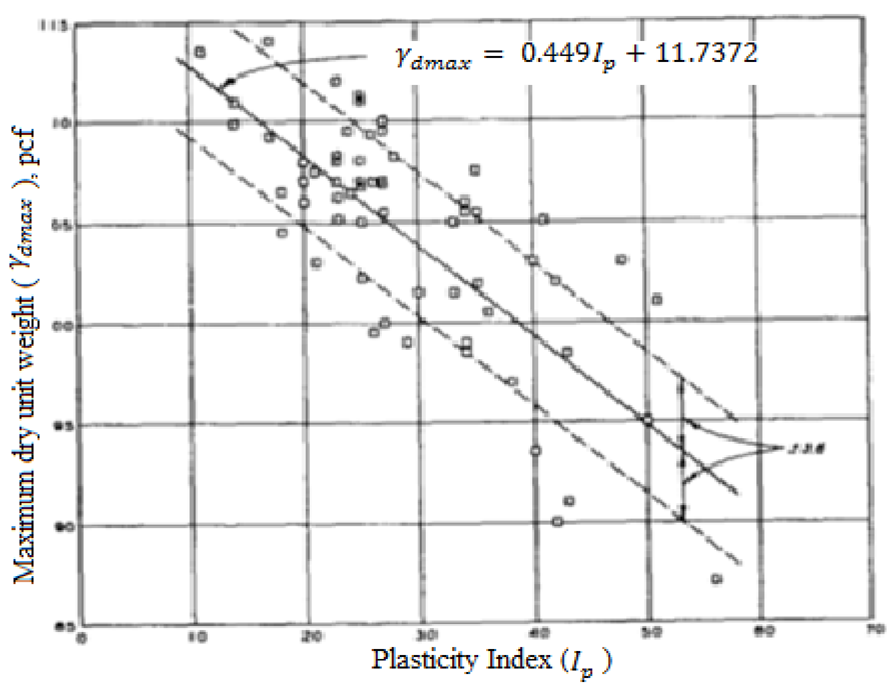


(b)

Figure 2.12: Plots of compaction characteristics versus liquid limit (Torrey, 1970)



(a)



(b)

Figure 2.13: Plots of compaction characteristics versus plasticity index (Torrey, 1970)

Jeng and Strohm (1976), correlated w_{opt} and ρ_{dmax} of testing soils to their Atterberg limits properties. Standard Proctor test was conducted on 85 soil samples with liquid limit ranging from 17 to 88 and plastic limit from 11 to 25. The statistical analysis approach was used in their study to correlate the compaction test parameters with Index properties.

In Ghana, the area of study, Hammond (1980) studied three groups of soils and proposed a linear regression model relating w_{opt} to either w_p, w_L, I_p or % fines. The proposed Equations are below:

For lateritic soils (predominantly clayey and sandy gravels), Equation 2.7 is used:

$$w_{opt} = 0.42w_p + 5 \tag{2.7}$$

For micaceous soils (clayey silty sands with Atterberg limits of the fines plotted below the A-line), Equations 2.8 and 2.9 can be used:

$$w_{opt} = 0.45w_p + 3.58 \tag{2.8}$$

$$w_{opt} = 0.5w_L - 6 \tag{2.9}$$

For black cotton clays (silty clays), Equation 2.10 can be used:

$$w_{opt} = 0.96w_p - 7.7 \tag{2.10}$$

Similarly, Korfiatis and Manikopoulos (1982) by 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 2.14 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 2.11:

$$Ds = \frac{1}{InD_1 - InD_2} = \frac{1}{2.303log\frac{D_1}{D_2}} \tag{2.11}$$

The definitions of D_1 and D_2 are shown in Figure 2.15. Once the magnitude of D_s is determined, the value of γ_{dmax} (based on the modified Proctor test) can be estimated as using Equations 2.12 and 2.13.

$$\gamma_{dmax} = \frac{G_s\gamma_w}{\left[\frac{100-FC}{100 \times a}\right] + \left[\frac{FC}{100 \times q)}\right]} \tag{2.12}$$

(for $0.5738 < D_s < 1.1346$)

$$\gamma_{dmax} = \frac{G_s\gamma_w}{\left[\frac{100-FC}{100 \times (c-ds)}\right] + \left[\frac{FC}{100 \times q)}\right]} \tag{2.13}$$

(for $0.2 < D_s < 0.5738$)

Based on statistical relationships,

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

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

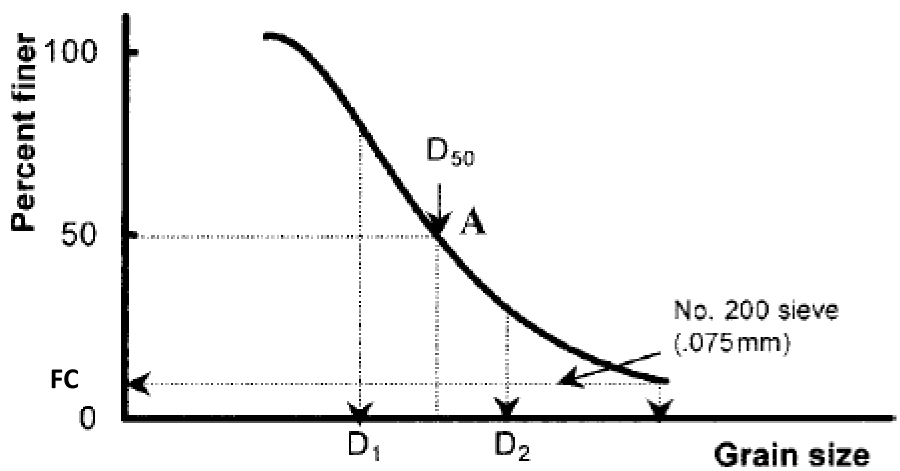


Figure 2.14: Definition of D_s in Equation 2.7 (Korfiatis and Manifopoulos, 1982)

Also, Wang and Huang (1984) developed correlation Equations for predicting w_{opt} and ρ_{dmax} for synthetic 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 w_{opt} and ρ_{dmax} to specific gravity, fineness modulus, plastic limit, uniformity coefficient, 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 standard Proctor test. He used soils from Iraq and USA to carry out his test in order to develop empirical Equations relating liquid limit (w_L) and plastic limit (w_p) to maximum dry density (ρ_d) and optimum water content (w_{opt}). 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 standard Proctor 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 following Equations 2.14 and 2.15 were derived from Iraqi soils;

$$\rho_{dmax} = 2.44 - 0.02w_p - 0.008w_L \tag{2.14}$$

$$w_{opt} = 0.24w_L + 0.63w_p - 3.13 \tag{2.15}$$

Similarly, for USA soils, the Equations 2.16 and 2.17 below were proposed;

$$\rho_{dmax} = 2.27 - 0.019w_p - 0.003w_L \tag{2.16}$$

$$w_{opt} = 0.14w_L + 0.54w_p \tag{2.17}$$

Blotz et al. (1998) correlated maximum dry unit weight and optimum water content of clayey soil at any compactive effort, *E*. Compactive efforts; including standard Proctor (ASTM D698-12), modified Proctor (ASTM D1557-12), "Reduced Proctor" and: Super-Modified Proctor" were used to compact the soils. One variation of the method uses the liquid limit (w_L) and one compaction curve, whereas the other uses only w_L. Linear relationships between γ_{dmax} and the logarithm of the compactive effort (*log E*), and between w_{opt} and *log E*, both of which a function of w_L, are used to extrapolate to different compactive energies. They used twenty two clayey soils to develop the empirical Equations and five different samples were used to validate the models. The variation in employing $w_L$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 employing only w_L for w_{opt} and γ_{dmax} are about ±2% and ±6% respectively. The empirical Equations 2.18 and 2.19 obtained were:

$$\gamma_{dmax,E} = \gamma_{dmax,k} + (2.27w_L - 0.94)log\left(\frac{E}{E_k}\right) \tag{2.18}$$

and

$$w_{opt,E} = w_{opt,k} + (12.39 - 12.21w_L)log\left(\frac{E}{E_k}\right) \tag{2.19}$$

where:

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

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

Figure 2.15 shows the relationships between γ_{dmax} , w_{opt} and w_L with Reduced Proctor (*RP*), standard Proctor (*SP*) and modified Proctor (*MP*) corresponding to Reduced, standard and modified Proctor efforts respectively. They also observed that when w_L becomes larger, w_{opt} increases and γ_{dmax} decreases. These curves can be used to directly estimate the optimum point for standard or modified Proctor effort if the w_L is known.

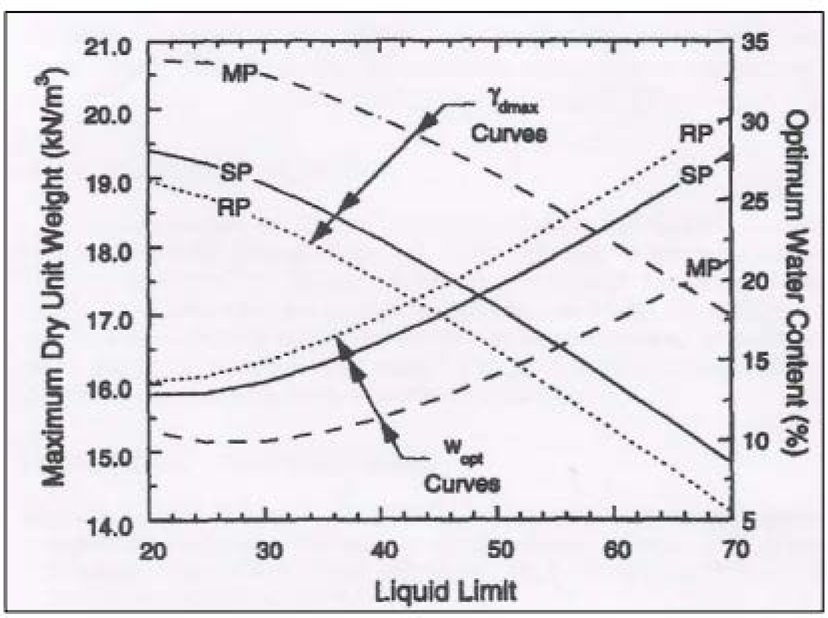


Figure 2.15: Maximum dry unit weight and optimum water content versus liquid limit for *RP*, *SP* and *MP* Compactive Efforts (Blotz et al., 1998)

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), liquid limit, plasticity index and specific gravity of soil solids). Of these samples, 45 were gravelly soils (*GP*, *GP-GM*, *GW*, *GW-GM, and GM*), 264 were sandy soils (*SP*, *SP-SM*, *SW-SM*, *SW*, *SC-SM*, *SC*, and *SM*) and two were clayey soils with low plasticity, *CL*. They used modified Proctor compaction test on the soils and developed the Equations 2.20 and 2.21 below:

$$\rho_{dmax}(\mathrm{kg/m^3}) = [4804574G_s - 195.55({w_L}^2) + 156971(R\#4)^{0.5}]^{0.5} \qquad (2.20)$$

$$In(W_o) = 1.195 \times 10^{-4}({w_L}^2) - 1.964G_s - 6.617 \times 10^{-3}(R\#4) + 7.651 \qquad (2.21)$$

Also, Gurtug and Sridharan (2004) studied the compaction behaviour and prediction of its characteristics of three cohesive soils taken from the Turkish Republic of Northern Cyprus and other two clayey minerals based on four compaction energy namely, standard Proctor, modified Proctor, Reduced standard Proctor and Reduced modified Proctor to develop relationship between maximum dry unit weight and optimum water content and plastic limit with particular reference to the compaction energy. They proposed the Equations 2.22 and 2.23 below:

$$w_{opt}(\%) = [1.95 - 0.38(\log CE)]w_P \qquad (2.22)$$

$$\gamma_{dmax}(\mathrm{kN/m^3}) = 22.68e^{-0.0183w_{opt}(\%)} \qquad (2.23)$$

where,

w_P= plastic limit, CE = compaction energy (kN-m/m^3)

Recently, Sridharan and Nagaraj (2005) conducted a study of five pairs of soils with nearly the same liquid limit but different plasticity index among the pair and made an attempt to predict optimum moisture content and maximum dry density from plastic limit of the soils. They developed with the following Equations 2.24 and 2.25:

$$w_{opt} = 0.92w_p \qquad (2.24)$$

$$\gamma_{dmax} = 0.23(93.3 - w_p) \qquad (2.25)$$

They concluded that w_{opt} is nearly equal to plastic limit.

Sivrikaya et al. (2008) correlated maximum dry unit weight and optimum water content of 60 fine-grained soils from Turkey and other data from the literature using standard Proctor and modified Proctor test with a plastic limit based on compaction energy. They developed the following Equations 2.26 and 2.27 which are similar to what Gurtug and Sridharan (2004) found in their study.

$$w_{opt} = Kw_p \tag{2.26}$$

and,

$$\gamma_{dmax}(\text{kN/m}^3) = L - Mw_{opt} \tag{2.27}$$

where;

$K = 1.99 - 0.165InE$

$L = 14.34 - 0.195InE$

$M = -0.19 + 0.073InE$

E in kJ/m^3

Thus, at any compactive effort, w_{opt} can be predicted from plastic limit (w_p) and the predicted optimum water content can be used to estimate maximum dry unit weight (γ_{dmax}).

Matteo et al. (2009) analyzed the results of 71 fine-grained soils and provided the following correlation Equations 2.28 and 2.29 for optimum water content (w_{opt}) and maximum dry unit weight (γ_{dmax}) for modified Proctor tests (E= 2700 kN-m/m^3)

$$w_{opt} = -0.86(w_L) + 3.04\left(\frac{w_L}{G_s}\right) + 2.2 \tag{2.28}$$

$$\gamma_{dmax}(\text{kN/m}^3) = 40.316\left(w_{opt}^{-0.295}\right)\left({I_p}^{0.032}\right) - 2.4 \tag{2.29}$$

where,

w_L = liquid limit. (%)

I_p = plasticity index (%)

G_s = Specific Gravity

Gurtug (2009) used three clayey soils from Turkish Republic of 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 2.30 and 2.31 below using 152 soil samples.

$$\rho_{dmax} = 15.665SG + 1.526w_L - 4.313FC + 2011.960 \quad (2.30)$$

$$w_{opt} = 0.129FC - 0.0196w_L - 1.4233SG + 11.399 \quad (2.31)$$

where,

w_L=liquid limit (%)

FC= Fines Content (%)

G_s = Specific Gravity

Mujtaba et al. (2013) conducted laboratory compaction tests on 110 sandy soil samples (*SM*, *SP-SM*, *SP*, *SW-SM*, and *SW*). Based on the tests results, the following correlation Equations 2.32 and 2.33 were proposed for γ_{dmax} and w_{opt}:

$$\gamma_{dmax}(\mathrm{kN/m^3}) = 4.49 \times \log(C_u) + 1.51 \times \log(E) + 10.2 \quad (2.32)$$

$$\log w_{opt}(\%) = 1.67 - 0.193 \times \log(C_u) - 0.153 \times \log(E) \quad (2.33)$$

where,

Cu= uniformity coefficient

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

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 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 Equation 2.34 using plastic limit (w_p) in order to determine the compaction characteristics namely, optimum water content $\left(w_{opt}\right)$ of these soils.

$$w_{opt} = 12.001e^{0.0181w_p} \qquad R^2 = 0.84 \quad (2.34)$$

CHAPTER 3

METHODS AND LABORATORY TEST RESULTS

3.1. Geoenvironmental Characteristics and Geology of the Study Area

Ghana is underlain partly by what is known as the Basement complex. It comprises a wide variety of Precambrian igneous and metamorphic rock which covers about 54% of the country's area; mainly the southern and western parts of the country (Figure 3.1). The primary components are gneiss, phyllites, schists, migmatites, granite-gneiss, and quartzites. The rest of the country is underlain by Paleozoic consolidated sedimentary rocks referred to as the Voltaian Formation consisting mainly of sandstones, shale, mudstone, sandy and pebbly beds, and limestones (Gyau-Boakye and Dapaah-Siakwan, 2000).

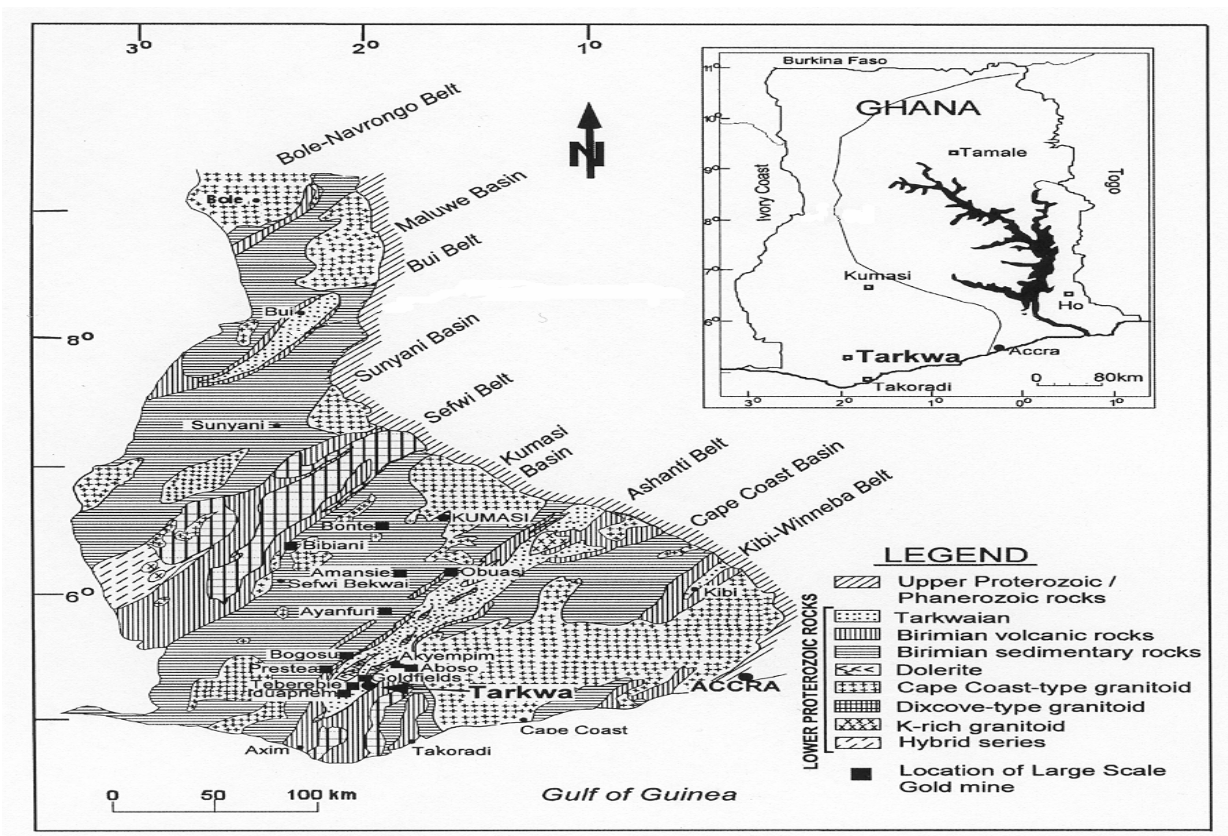


Figure 3.1: Simplified geological map of southwest Ghana (modified from Kuma, 2004)

The soil under study is laterite and it occurs in different parts of Africa. It is also called residual soils. It occurs in tropical and sub-tropical countries under certain climatic conditions. They are formed when the mean annual rainfall is about 1200mm with a daily temperature in excess of 25℃. They are used in the construction of roads, earth dams, etc. Though its occurrence can be found in different parts of Africa, its mineralogical composition is different. There have been many studies on lateritic soils and one of the most significant features is its red colour. There are many factors that affect the engineering properties and field performances. The two most important factors are;

i. Soil forming factors (e.g. parent rock, climatic and vegetation conditions, topography, and drainage conditions).
ii. The degree of weathering (degree of laterization) and the texture of soils, genetic soil type, the predominant clay mineral types, and depth of the sample.

A very distinctive feature of lateritic soils is the high proportion of sesquioxides of iron and/or aluminum. Physically similar laterite may have different chemical composition and chemically similar laterite may display different physical properties (Maignien, 1966).

The mineralogical characterization is considered to be the most important feature when describing the physical properties of lateritic soils.

The major constituents are oxides and hydroxides of aluminum and iron, with clay minerals and to a lesser extent, manganese, titanium, and silica. The minor constituents are residual remnants or classic minerals.

Kaolinite is the most common clay mineral in lateritic soils, halloysite may also be seen. The most common minerals encountered are quartz, feldspar, and hornblende.

When the desired engineering properties for specific projects are not met, they are usually stabilized with cement, lime, etc.

3.2. Site Plan of the Study Area

The construction area is within the Tarkwaian zone. The area was demarcated into several sections and designated for easy reference. Figure 3.2 shows the site plan of the Tailings Storage Facility, TSF dam. Also, it shows the major designated areas of about 17 in number. These areas were divided into smaller areas according to the cardinal coordinates.

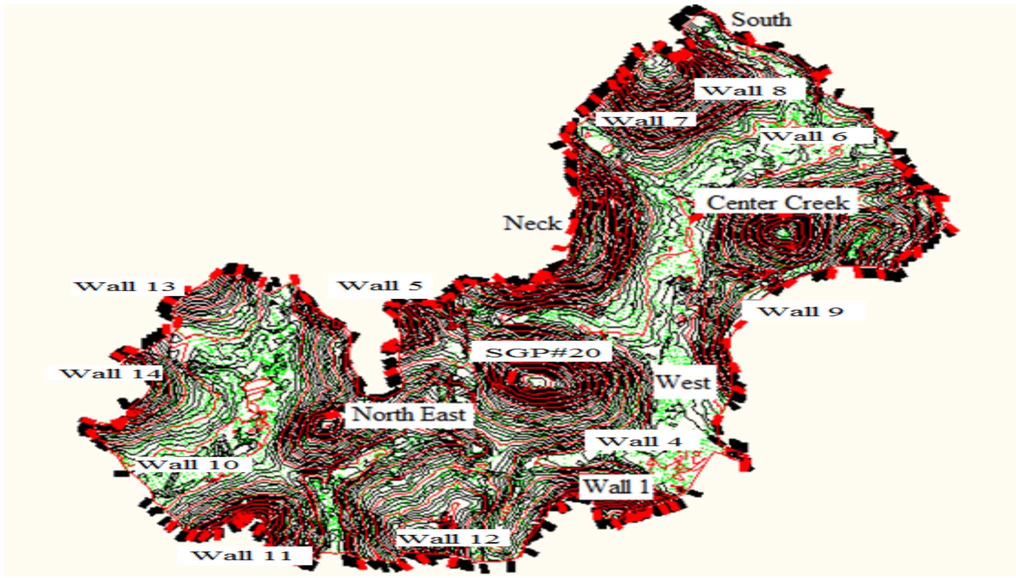


Figure 3.2: Site layout of the TSF dam, Tarkwa (ABP Gh Ltd., 2015)

3.3. Laboratory Tests

Fresh soil samples were obtained from depths of about 300mm to 2metres during the construction of Tailings Storage Facility, TSF dam for a gold mine in Tarkwa, Ghana. In total, 168 fresh samples were collected and they were subjected to particle size analysis test, Atterberg limit tests, and compaction tests. All the tests were performed by ABP Gh Ltd, a construction and building company in charge of the construction of the dam. The tests were performed in accordance to American Society for Testing and Materials (ASTM) standard specifications to determine the physical and compaction properties of the soils. The dam consists of about 14 embankments and these embankments are constructed in lifts, with each lift of about 300mm thick.

3.3.1. Gradation Analysis Tests

Mechanical sieve analyses were performed on each soil sample according to ASTM D6913-04 to determine the grain size distribution. Sieve analysis was conducted using U.S. Sieve sizes; 3/8”, #4, #10, #40, #60, #100, and #200. A sample of the soil was dried in the oven at a temperature of 105^{o}C - 110^{o}C for overnight. The whole specimen sample was allowed to cool and the weight was taken. The weighed sample was put in the nested sieves which are arranged in a decreasing order with the sieve with the largest aperture on top followed by the others. Subsequently, the mass retained on each sieve was taken. The percentage passing is then calculated from the mass retained. Figure 3.3 and 3.4 shows the range of grain size distribution curves for all samples used for standard and modified Proctor compaction tests respectively.

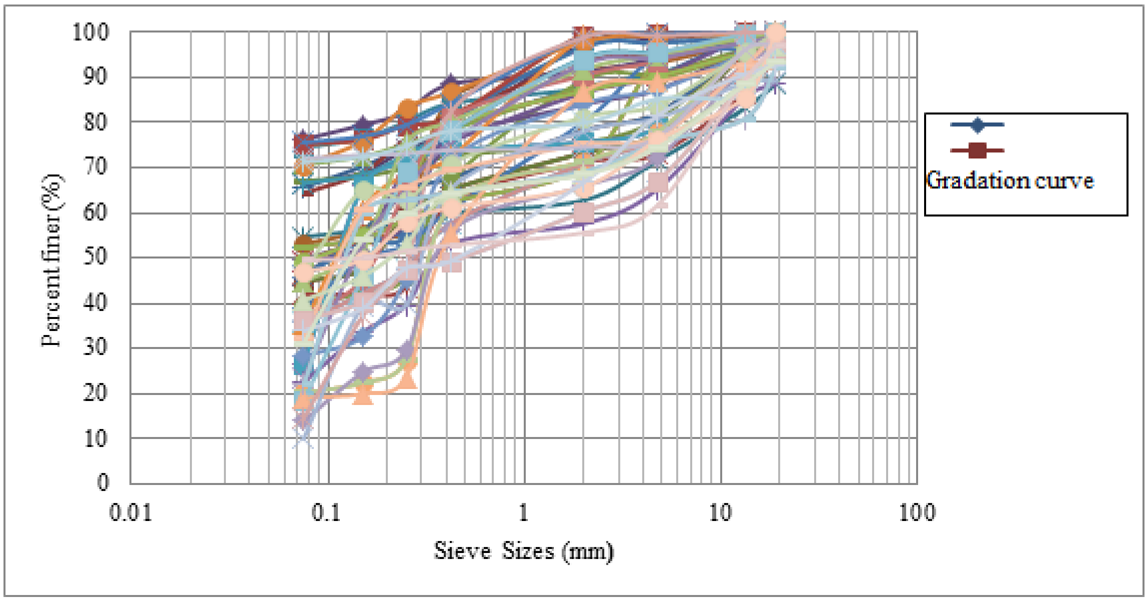


Figure 3.3: Grain size distribution curves for 88 lateritic soils used for standard Proctor tests

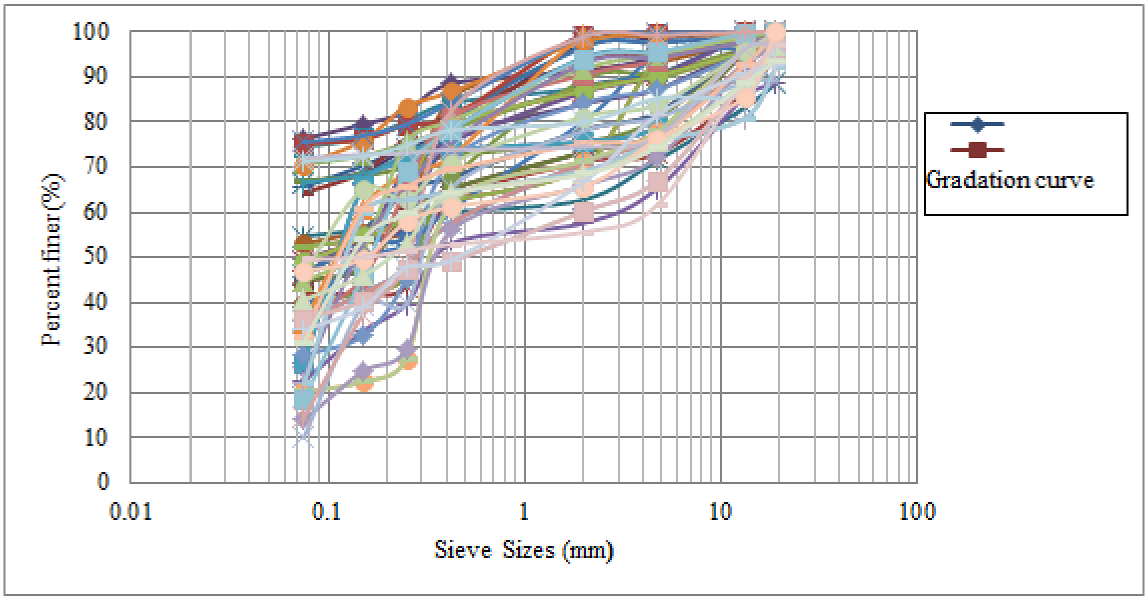


Figure 3.4: Grain size distribution curves for 80 lateritic soils used for modified Proctor tests

3.3.2. Atterberg limit tests

The Atterberg limits (plastic and liquid limit) were determined on all the 168 samples using distilled water as the wetting agent. The liquid limit test was done on the soil fraction passing through the U.S. No. 40 (0.425mm) sieve in accordance with ASTM D4318-10. This method involves finding the moisture content at which the groove cut in the wet sample with a standard grooving tool closes (Appendix A)

In accordance with ASTM D4318-10 procedure, the plastic limits were determined on the soil fraction passing the U.S. No. 40 sieve. This method involves finding the moisture content at which the wet soil just begins to crumble or break apart when rolled by hand, into threads of diameter, 3mm or one-eighth of an inch (Appendix A).The results are shown in Table 3.2, 3.3, 3.4 and 3.5.

Furthermore, the classification of the soils was done in accordance with the Unified Soil Classification System (ASTM D2487-11).

3.3.3. Proctor compaction tests

Two types of Proctor compaction test; standard and modified Proctor tests were conducted manually on the soil samples. Standard Proctor test was performed on 88 soil samples and modified Proctor was performed on 80 samples. This was used to determine the maximum dry unit weight and optimum moisture content of the soil. Compaction of the soil was done using the mechanical energy obtained from an impacting hammer. The mechanical energy is a function of hammer weight, height of the hammer drop, the number of soil layers, and number of blows per layer. The parameters of the standard and modified Proctor tests in accordance to ASTM D 698-12 and ASTM D 1557-12 respectively are shown in Table. 3.1.

Table 3.1: Standard and modified Proctor test parameters.

	Standard Proctor	**Modified Proctor**
Mold Volume(cm^3)	944	944
Hammer Weight (kN)	2.495	4.539
Hammer Drop(mm)	304.9	457
No of Soil layers	3	5
No. of Hammer blows per layer	25	25
Compaction Energy(kJ/m^3)	592.7	2693.0

The test procedures for the standard and modified Proctor compaction test can be seen in Appendix A. The compaction curves of the soil samples for standard and modified Proctor tests can be seen in Figure 3.5 and 3.6 respectively.

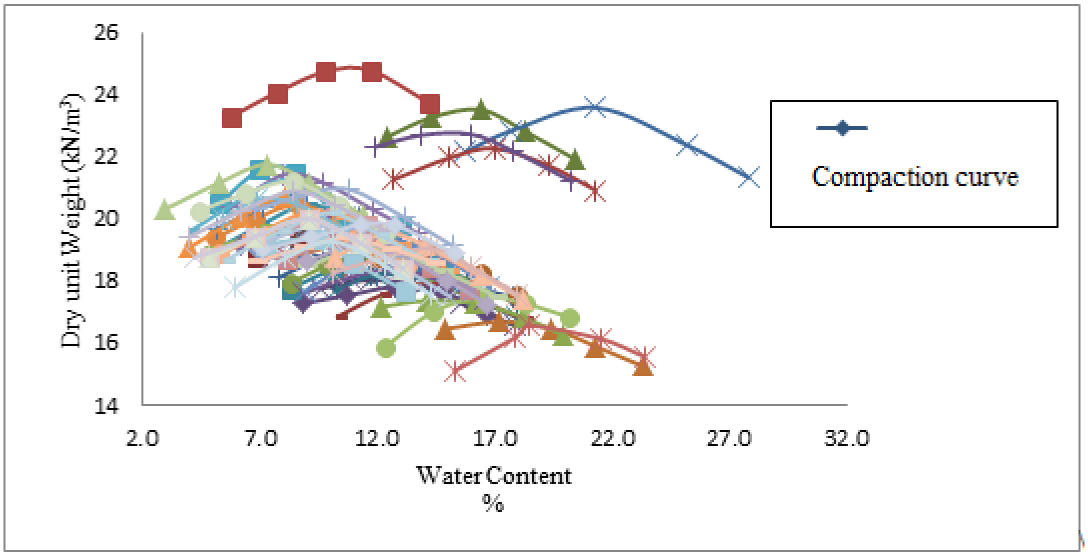


Figure 3.5: Standard Proctor compaction curves for the soil samples

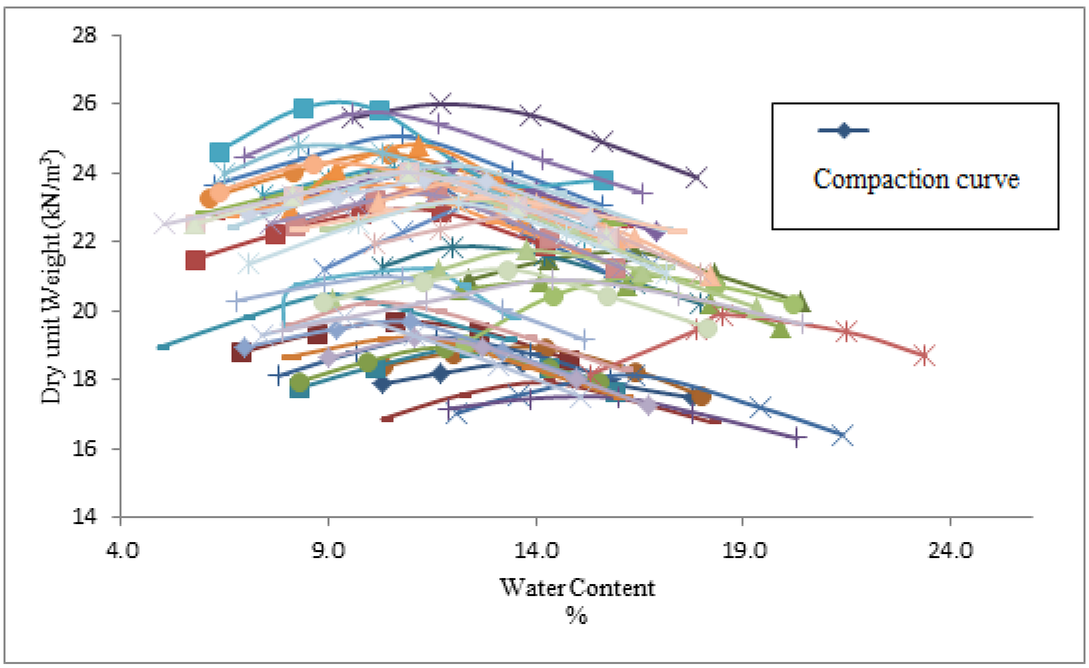


Figure 3.6: Modified Proctor compaction curves for the soil samples

Consequently, a compilation of the laboratory test results for the soil samples for the standard and modified Proctor tests results is shown in Table 3.2 and Table 3.3 respectively. Soils samples taken for the regression analysis for standard Proctor is 77 and that of modified Proctor is 70. With respect to validation of the regression models, 21 soil samples not seen by the model were used to verify the model i.e. 11 samples for standard and 10 samples for modified proctor compaction test (See Table 3.4 and Table 3.5).

Table 3.2: Laboratory test results for regression analysis of standard Proctor compaction test.

Sample No	Section	*Gravel* G (%)	*Sand* S (%)	FC (%)	w_L	w_p	I_p	γ_{dmax} kN/m^3	w_{opt} %	USCS
47	West of Wall 12	30.8	30.4	38.8	26.9	10.8	16.5	18.80	17.3	CL
48	GTSF – South	18.4	45.6	36.0	28.8	11.4	17.4	19.8	17.0	SC
49	Wall1 Stockpile	15.2	31.4	53.4	31.6	12.4	19.3	20.3	14.9	CL
50	GTSF North-East	43.3	12.8	43.9	33.2	13.3	19.9	19.7	15.6	CL
51A	West of Wall 12	32.6	24.8	42.6	34.8	14.0	20.9	20.2	14.4	CL
52	Wall 1 (2nd & 3rd Layer)	14.7	32.5	52.8	37.0	15.6	21.4	20.2	14.0	CL
53	West (Center Creek)	24.0	24.4	51.6	38.5	16.3	22.2	19.3	14.5	CL
54	Wall 1 Layer 7	24.0	22.1	53.9	42.2	17.5	24.7	18.7	17.5	CL
55	Wall 7 Approach	21.5	17.2	61.3	45.0	19.8	25.1	18.45	17.8	CL
56	Wall 1 Layer 8	23.9	16.3	59.8	45.9	21.4	24.6	19.41	15.6	CL
57	GTSF - West	13.3	49.8	36.9	34.9	11.4	23.5	20.45	14.1	SC
58	Neck - North	10.2	56.2	33.6	39.7	12.4	27.4	20.36	14.4	SC
59	Neck - Stockpile	23.0	45.3	31.7	37.6	13.3	24.3	20.45	13.9	SC
60	GTSF - South-West of Neck	40.0	20.0	40.0	39.8	15.3	24.5	20.80	13.5	CL
61	East of Neck	44.2	19.0	36.8	42.0	14.5	27.5	20.71	12.4	GC
62	GTSF - North/East	17.7	21.8	60.5	42.3	14.5	27.8	21.23	12.1	CL
63	North of Wall 7	24.4	18.8	56.8	42.8	15.2	27.6	20.10	13.0	CL
64	GTSF - South of Neck	34.0	26.5	39.5	45.8	18.1	27.7	20.36	13.5	CL
65	Wall 7 Base	23.9	22.6	53.5	50.0	17.1	32.9	19.6	15.2	CH
66	Neck - North	23.7	19.6	56.7	51.4	19.3	32.1	19.4	17.8	CH
67	GTSF - Wall 6	10.3	57.3	32.4	27.2	10.8	16.5	19.6	17.5	SC
68	Wall 6 Stockpile	14.1	54.7	31.2	33.6	12.8	20.7	19.8	16.9	SC
69	North-East	19.2	38.6	42.2	42.1	17.8	24.3	20.3	13.3	CL
70	SGP #20	16.9	61.8	21.3	27.9	9.5	18.4	20.0	12.5	SC
71	GTSF - Neck	52.0	19.7	28.3	34.4	13.2	21.2	19.7	14.4	GC
72	East of Neck	17.2	22.1	60.7	35.2	14.0	21.3	20.5	12.5	CL
73	GTSF - South-West of Neck	25.7	21.0	53.3	38.1	14.5	23.5	21.1	11.8	CL
74	GTSF - South	35.3	25.4	39.3	39.8	15.6	24.3	20.9	11.1	CL
75	GTSF - North-West of Neck	48.2	33.7	18.1	25.1	12.4	12.7	20.4	12.8	CL

Table 3.2: Continued

76	Neck - Stockpile	40.7	39.7	19.6	38.1	15.3	22.7	20.4	13.0	CL
77	GTSF - South-West of Wall 8	38.4	38.8	22.8	49.6	16.9	32.6	20.3	12.7	SC
78	GTSF North-East	14.3	56.0	29.8	28.7	23.7	5.0	16.6	22.4	SC
79	GTSF - South-West of Neck	15.8	48.5	35.8	27.5	24.6	2.9	16.9	20.1	SC
80	GTSF - North-West of Neck	30.6	50.5	18.9	28.3	25.0	3.3	17.3	21.3	SC
81	GTSF - South-West	17.9	53.4	28.7	19.6	17.7	1.9	17.3	21.7	SC
82	GTSF - West of Neck	18.5	65.3	16.2	24.3	23.4	0.9	17.8	19.5	SC
83	GTSF - South-West of Neck	32.8	56.0	11.2	30.9	30.3	0.7	17.4	21.2	SM
84	GTSF - North-East	26.7	58.5	14.9	32.9	25.8	7.1	17.1	20.9	SC
85	GTSF - Wall 7	28.8	56.0	15.3	29.4	26.3	3.1	17.3	20.0	SC
86	Wall 1 Approach	31.0	61.0	8.1	31.1	26.3	4.8	16.8	19.8	SM
87	GTSF - West	29.9	58.5	11.7	32.9	31.5	1.4	17.1	20.0	SM
88	Centre Creek	11.5	51.0	37.5	31.0	25.3	5.7	17.1	20.7	SC
89	Wall 4 Approach	13.9	68.5	17.6	24.7	18.8	6.0	17.3	20.4	SC
90	Wall 7	46.8	27.3	25.9	38.2	15.4	23.0	21.2	11.8	GC
91	Wall 6 Stockpile	48.8	22.0	29.2	40.7	17.1	23.6	21.2	11.1	GC
92	Wall 7 Base	40.5	26.8	32.6	42.4	17.9	24.4	20.3	13.0	GC
93	Wall1 Stockpile	17.8	24.3	57.9	46.5	19.3	27.2	19.6	15.9	CL
94	Wall 4 Approach	23.8	18.9	57.2	49.5	21.8	27.6	19.4	15.1	CH
95	GTSF - West of Neck	44.3	17.9	37.7	50.4	23.5	27.0	20.4	12.3	GC
96	West of Wall 8	19.0	53.8	27.2	38.4	12.5	25.9	21.5	11.4	SC
97	GTSF - South-West	46.9	13.3	39.8	43.7	13.6	30.1	21.4	12.5	GC
98	Wall 1	20.7	37.8	41.4	41.3	14.7	26.7	21.5	11.3	CL
99	South of Neck	9.9	65.0	25.1	43.8	16.9	26.9	21.8	10.5	SC
100	Wall 1	20.8	45.6	33.6	46.2	16.0	30.2	21.7	10.4	SC
101	GTSF – Neck	19.1	50.6	30.3	46.5	16.0	30.6	22.3	12.7	SC
102	East of Neck	48.1	20.7	31.2	47.1	16.7	30.4	21.1	13.6	GC
103	South of Wall 6	24.2	29.2	46.6	50.3	19.9	30.5	21.4	12.7	CH
104	North	27.1	50.2	22.7	38.2	15.4	23.0	21.2	13.8	SC
105	Wall 9 Approach	17.6	22.0	60.4	40.7	17.1	23.6	21.2	13.6	CL
106	Wall 7 Approach	17.5	26.8	55.6	42.4	17.9	24.4	20.3	15.3	CL
107	Centre Creek	16.6	24.3	59.1	46.5	19.3	27.2	19.6	15.9	CL
108	GTSF - South/West	16.7	18.9	64.4	49.5	21.8	27.6	19.4	15.1	CH
109	East	36.7	17.9	45.3	50.4	23.5	27.0	20.4	12.3	CH
110	Centre Creek	27.1	11.0	61.9	38.4	12.5	25.9	21.5	11.6	CL
111	North of Wall 7	29.4	13.3	57.2	43.7	13.6	30.1	21.4	11.3	CL

Table 3.2: Continued

112	Wall 7 Approach	15.4	37.8	46.8	41.3	14.7	26.7	21.5	12.4	CL
113	Wall 9 Approach	20.7	22.0	57.3	43.8	16.9	26.9	21.8	9.7	CL
114	Wall 1Approach	40.3	20.9	38.8	46.2	16.0	30.2	21.7	10.5	GC
115	GTSF – Neck	39.5	24.0	36.6	46.5	16.0	30.6	22.3	12.7	GC
116	GTSF North-East	36.4	20.7	42.9	47.1	16.7	30.4	24.3	8.8	CL
117	North of Wall 7	25.1	56.0	18.9	31.2	29.1	2.1	16.3	21.4	SC
118	Center Creek South	20.8	48.5	30.7	42.1	22.2	19.9	20.2	12.2	SC
119	GTSF - Center Creek	18.8	50.5	30.7	42.1	22.2	19.9	20.2	11.5	SC
120	GTSF - North/East	24.3	53.4	22.3	32.5	27.2	5.3	17.4	19.3	SC
121	West (Center Creek)	8.5	65.3	26.2	37.0	24.8	12.1	18.7	16.5	SC
122	Wall 5 Approach	17.8	56.0	26.2	37.0	24.8	12.1	18.7	16.5	SC
123	GTSF – Neck	16.1	58.5	25.4	36.1	25.3	10.7	18.5	15.6	SC

Table 3.3: Laboratory test results for regression analysis of modified Proctor compaction test.

Sample No	Section	*Gravel* *G* (%)	*Sand* *S* (%)	*FC* (%)	w_L	w_p	I_p	γ_{dmax} kN/m^3	w_{opt} %	USCS
51	South West	21.3	30.4	48.3	33.5	13.4	20.5	21.60	11.2	CL
76	GTSF - North/West of Neck	9.6	45.6	44.8	35.9	14.2	21.7	22.80	11	SC
85	GTSF - North of Neck	2.1	31.4	66.5	39.4	15.4	24	23.30	9.6	CH
92	North West	32.5	12.8	54.7	41.4	16.6	24.8	22.60	10.1	CL
93	South of Wall 6	22.1	24.8	53.1	43.3	17.4	26	23.20	9.3	CL
95	Wall 1	1.8	32.5	65.7	46.1	19.4	26.7	23.20	9.05	CL
95	West of Wall 8	11.3	24.4	64.3	48	20.3	27.6	22.20	9.4	CL
96	Centre Creek	10.8	22.1	67.1	52.6	21.8	30.8	21.50	11.3	CH
97	Wall 1 Approach	6.5	17.2	76.3	56	24.7	31.3	21.20	11.5	CH
99	Wall10	9.2	16.3	74.5	57.1	26.6	30.6	22.30	10.1	CH
100	Wall 9 Approach	4.2	49.8	46	43.5	14.2	29.3	23.50	9.1	SC
102	East of Wall 10	1.9	56.2	41.9	49.5	15.4	34.1	23.40	9.3	SC
103	GTSF - South-West of Neck	15.2	45.3	39.5	46.8	16.6	30.2	23.50	9	SC
105	GTSF - West	30.2	20	49.8	49.6	19.1	30.5	23.90	8.7	CL
109	Centre Creek	35.2	19	45.8	52.3	18.1	34.2	23.80	8	CH
111	West Creek	2.9	21.8	75.3	52.7	18.1	34.6	24.40	7.8	CH
112	Wall 11 Approach	10.5	18.8	70.7	53.3	18.9	34.4	23.10	8.4	CH
113	Wall 11	24.3	26.5	49.2	57	22.5	34.5	23.40	8.7	CH
114	Wall 10	10.8	22.6	66.6	62.3	21.3	41	22.50	9.8	CH
115	Wall 4 Approach	9.8	19.6	70.6	64	24	40	22.30	11.5	CH

Table 3.3: Comtinued

116	Wall 1 Approach	2.3	57.3	40.4	33.9	13.4	20.5	22.50	11.3	SC
117	GTSF - South-West of Neck	6.5	54.7	38.8	41.8	16	25.8	22.70	10.9	SC
118	GTSF - West	8.9	38.6	52.5	52.4	22.2	30.2	23.30	8.6	CH
119	Centre Creek	11.7	61.8	26.5	34.7	11.8	22.9	23.00	8.1	SC
120	West Creek	45.0	19.7	35.3	42.8	16.4	26.4	22.60	9.3	GC
122	East of Wall 14	2.3	22.1	75.6	43.8	17.4	26.5	23.60	8.1	CL
123	Center Creek	12.6	21.0	66.4	47.4	18.1	29.3	24.20	7.6	CL
125	Wall 13, Wall 1 Approach	25.6	25.4	49.0	49.6	19.4	30.2	24.00	7.2	CL
126	Wall 14 Approach	43.7	33.7	22.6	31.3	15.5	15.8	23.40	8.3	GC
127	Wall 11	35.9	39.7	24.4	47.4	19.1	28.3	23.40	8.4	SC
128	North East Neck	32.8	38.8	28.4	61.8	21.1	40.6	23.30	8.2	SC
129	Wall 1 Approach	7	55.95	37.05	35.75	29.56	6.19	19.08	14.5	SC
131	Wall10	7	48.45	44.55	34.2	30.65	3.55	19.37	13	SC
132	Wall 9 Approach	26	50.5	23.5	35.25	31.18	4.07	19.89	13.8	SC
134	East of Wall 10	10.8	53.4	35.8	24.4	22.04	2.36	19.88	14	SC
136	GTSF - North of Neck	14.5	65.3	20.2	30.2	29.12	1.08	20.45	12.6	SC
137	North West	30.1	55.95	13.95	38.5	37.69	0.81	19.94	13.7	SC
138	South of Wall 6	23	58.45	18.55	41.03	32.18	8.85	19.70	13.5	SC
140	Wall 1	25	55.95	19.05	36.6	32.7	3.9	19.84	12.9	SC
143	GTSF - South-West of Neck	29	60.95	10.05	38.7	32.77	5.93	19.34	12.8	SM
144	GTSF – West	27	58.45	14.55	41	39.24	1.76	19.69	12.9	SC
145	Centre Creek	2.3	50.95	46.75	38.6	31.56	7.04	19.69	13.4	SC
146	West Creek	9.6	68.45	21.95	30.8	23.37	7.43	19.90	13.2	SC
148	Wall 11 Approach	40.5	27.28	32.22	47.63	19.14	28.6	24.36	7.6	GC
149	GTSF - North of Neck	41.6	22	36.4	50.71	21.34	29.37	24.36	7.2	GC
150	North West	32.5	26.84	40.66	52.8	22.33	30.36	23.31	8.4	CH
152	Wall 4	3.6	24.31	72.1	57.86	23.98	33.88	22.58	10.29	CH
153	Wall 11	9.8	18.9	71.3	61.6	27.2	34.43	22.26	9.8	CH
154	Wall 10	35.09	17.93	47.0	62.81	29.26	33.66	23.42	7.98	CL
155	Wall 4 Approach	12.3	53.8	33.9	47.85	15.62	32.23	24.68	7.4	SC
156	Wall 1 Approach	37.18	13.31	49.5	54.45	16.94	37.51	24.57	8.1	CL
158	Wall10	10.56	37.84	51.6	51.48	18.26	33.22	24.68	7.3	CL
159	Wall 9 Approach	3.8	65	31.2	54.56	21.01	33.55	25.10	6.8	SC
160	East of Wall 10	12.5	45.6	41.9	57.5	19.9	37.62	24.99	6.7	SC
161	North of Wall 7	11.7	50.6	37.7	57.97	19.91	38.06	25.62	8.19	SC
162	North of Wall 6	40.5	20.68	38.8	58.63	20.79	37.84	24.26	8.82	GC
163	GTSF - North of Neck	12.8	29.15	58.05	62.7	24.75	37.95	24.57	8.2	CH
164	North West	21.5	50.2	28.3	47.63	19.14	28.6	24.36	8.9	SC
165	South of Wall 6	2.8	22	75.2	50.71	21.34	29.37	24.36	8.82	CH

Table 3.3: Continued

166	Wall 1	3.9	26.84	69.26	52.8	22.33	30.36	23.31	9.87	CH
167	Wall 11	2.1	24.31	73.59	57.86	23.98	33.88	22.58	10.29	CH
168	Wall 10	0.9	18.92	80.18	61.6	27.17	34.43	22.26	9.765	CH
169	Wall 4 Approach	25.6	17.93	56.47	62.81	29.26	33.66	23.42	7.98	CH
170	Wall 1 Approach	11.9	11.0	77.1	47.9	15.6	32.23	24.68	7.5	CL
171	GTSF – West	15.4	13.3	71.3	54.5	16.9	37.51	24.57	7.3	CH
173	Centre Creek	3.9	37.8	58.3	51.5	18.3	33.22	24.68	8.0	CH
174	West Creek	6.7	22.0	71.3	54.6	21.0	33.55	25.10	6.3	CH
175	Wall 11 Approach	30.8	20.9	48.3	57.5	19.9	37.62	24.99	6.8	CL
176	East of Wall 10	30.5	23.98	45.5	57.97	19.91	38.06	25.62	8.19	CL
177	West (Subgrade Pad)	25.9	20.7	53.4	58.6	20.8	37.84	24.26	8.82	CL

Table.3.4: Data samples for validation for standard Proctor compaction test.

Sample No	Section	*Gravel* G (%)	*Sand* S (%)	FC (%)	w_L	w_p	I_p	γ_{dmax} kN/m^3	w_{opt} %	USCS
124	Wall 7	22.0	58.2	19.8	21.8	17.5	4.3	17.9	17.75	SC
125	Center Creek South	27.8	21.6	50.6	33.6	19.4	14.2	17.8	17.54	CL
126	GTSF - North/East	4.4	12.2	83.4	56.9	32.6	10.9	16.1	24.50	CL
127	Wall 5 Approach	10.9	76.3	12.8	25.1	16.0	9.1	19.8	17.5	SC
128	GTSF - South-West of Neck	19.0	24.2	56.8	33.3	18.8	14.5	17.9	18.2	CL
129	South of Wall 6	0.7	72.5	26.8	29.2	16.9	12.3	18.7	17.6	SC
130	Wall1 Stockpile	16.1	35.1	48.8	29.0	16.4	12.6	17.7	17.4	CL
131	Wall 1	36.6	31.7	31.7	48.6	25.9	22.7	20.1	16.5	GC
132	Wall 5 Approach	30.2	23.2	46.6	45.9	25.0	20.9	19.6	16.2	CL
133	SGP #20	32.9	28.1	39.0	33.8	17.9	15.9	19.4	15.8	CL
134	East of Neck	34.8	18.9	46.3	27.7	16.2	11.5	18.8	17.3	CL

Table 3.5: Data samples for validation for modified Proctor compaction test.

Sample No	Section	*Gravel* *G* (%)	*Sand* *S* (%)	*FC* (%)	w_L	w_p	I_p	γ_{dmax} kN/m^3	w_{opt} %	USCS
205	Wall 1 Approach	14.1	59.4	26.5	38	26	12	20.6	11.7	SC
206	Centre Creek	1.4	66.1	32.5	31	15	16	23.1	10.9	SC
207	GTSF – West	44.2	34.6	21.2	47	27	20	21.8	11.8	GC
208	North of Wall 6	9.1	54.1	36.8	34	14	20	22.3	10.3	SC
209	GTSF - North of Neck	11.2	42.6	46.2	19	16	3	21.1	12.3	CL
210	Wall 11 Approach	9.4	28.5	62.1	40	11	29	24.5	8.7	CL
211	West of Wall 8	0.6	86.5	12.9	43	25	18	22.6	10.4	SC
212	Centre Creek	4.3	78.9	16.8	47	27	20	21.9	10.8	SC
213	GTSF - South-West of Neck	13.5	57.9	28.6	44	22	22	23.1	10.1	SC
214	East of Wall 14	15.6	56.3	28.1	48	23	25	22.9	10	SC

From these tables above, the following general observations were made:

a) Most of the soils had fines content exceeding 12% hence Atterberg limit tests can be conducted on the soils and be used accordingly in the statistical analysis.

b) Based on the Unified soil classification system (ASTM D-2487-11), about 75% of the soils are *CL* and *CH*; *SC*, *GC* and *SM* making up the remaining 25%.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1. Statistical Analysis Procedure Used for Model Development

A brief review of statistical models, regression analyses, assumptions, limitations and practical considerations is given in this section. In this phase, the sampled data are analyzed fully to determine the range of the variables, mean, standard deviation and other statistical descriptive. Consequently, once the statistical descriptive has been established, statistical methods were used to identify and develop reliable correlation Equations. The variables were separated into dependent and independent variables. The compaction test parameters are the dependent variables whilst the Gradation and Atterberg limit parameters are the independent (predictor) variables.

In statistics, regression analysis examines the relation between a dependent variable (response variable) and a specified independent variable (predictor). The mathematical model of their relationship is known as the regression Equation. Statistica 13 statistical software was employed in the regression analysis.

4.1.1. Statistical terms and Definitions

Statistical terms used in the regression analysis are defined as follows:

Residual variance and R-square: An accurate prediction is achieved if the variability of the residual values around the line of best fit is smaller. This also shows the relationship between the X and Y variables. For example, if the ratio of the residual variability of the Y variable to the original variance, a value of 1.0 is obtained and 0.0 is achieved if there is a perfect relationship. In most cases, the ratio would fall somewhere between these extremes, that is, between 0.0 and 1.0. 1.0 minus this ratio is referred to as *R-square* or the *coefficient of determination*.

$$R^2 = 1 - \frac{SSE}{SST} \tag{4.1}$$

where; $SSE = \sum(y_i - \bar{y}_i)^2$ is the residual sum of square and SST is the total sum of squares, $\sum y_i^2$.

The *R*-square value is an indicator of how well the model fits the data (e.g., an *R*-square value close to 1.0 indicates that almost all the variables variability have been accounted in the model). However, *R*-square increases with increase in a number of predictors in the model, even when the role of the individual predictor is not significant.

Standard Estimate Error (SEE): The efficiency of regression line can also be evaluated through the estimation of standard error given as

$$SEE^2 = \frac{SSE}{(n-p)} \qquad OR \qquad SEE = \sqrt{\frac{SSE}{(n-p)}} \tag{4.2}$$

where;

SEE^2 is the unbiased estimator of variance and the smaller the variance, the better the model.

Degrees of Freedom (df): Total number of degrees of freedom is one less than the number of observations. Each sum of square is associated with the degrees of freedom.

*The sum of Squares***:** Sum of squares represents the total amount of variability in the data set that can be estimated by calculating the sum of the squared differences between each observation and the overall mean.

Mean Square: It is the sum of squares divided by the corresponding degrees of freedom.

P-value: It is the most important term in the estimation of the statistical significance of independent variables. It also represents whether the model has the significant predictive capability. *P-value* is simply the ratio of the model mean square to the error mean square.

t-test value: It is the coefficient divided by its standard error of independent variables .

The following steps were implemented for the procedure for model selection:

i. Scatterplots to inspect the possible relationship among the various variables.
ii. Correlation matrix of the dependent and independent variables.
iii. Analysis of Variance (*ANOVA*) table for identifying significant terms.
iv. Residual plots.
v. Engineering judgment.

Subsequently, the best model was selected if it possesses these statistical features;

a) Pass the *F* and t-tests with a pre-selected significance value (usually 0.05).
b) Possess a high value of R^2.
c) Have a low value of *SEE*.

4.2. Regression Analysis of Standard Proctor Compaction Test Parameters

As mentioned previously, a total of 88 soil samples seen in Table 3.2 and 3.4 were used to develop and validate a model that can predict the compaction characteristics of lateritic soils from gradation and Atterberg limit test parameters. The statistical descriptive of the dependent and independent variables for the samples used for the regression analysis excluding the 11 data for validation are shown in Table 4.1.

From the Table 4.1, the maximum dry unit weight of the samples ranges from 16.3kN/m^3 to 24.3kN/m^3; the optimum water content is between 8.8% and 22.4%. The highest liquid limit is 51.4 with the lowest being 19.6.

Table 4.1: Descriptive statistics of data for standard Proctor analysis.

	Gravel (G) %	Sand (S) %	FC %	w_L %	w_p %	I_p %	γ_{dmax} kN/m^3	w_{opt} %
N	77	77	77	77	77	77	77	77
Range	43.5	57.5	56.3	31.8	22.0	32.3	7.9	13.6
Minimum	8.5	11.0	8.1	19.6	9.5	.7	16.3	8.8
Maximum	52.0	68.5	64.4	51.4	31.5	32.9	24.3	22.4
Mean	25.9	36.4	37.7	38.7	18.1	20.6	19.9	14.9
Std. Deviation	11.1	16.8	14.85	7.54	5.08	9.37	1.62	3.37
Variance	123.6	282.3	220.5	56.8	25.8	87.8	2.6	11.3
Skewness	0.66	0.28	.081	-.33	.68	-.93	-.37	.63
Kurtosis	-0.56	-1.43	-1.00	-0.66	-0.30	-0.40	-0.13	-0.59

4.2.1. Scatter plots

A scatter plot matrix is shown in Figure 4.1; it indicates the relationship between the independent and dependent variables used for the analysis. Though it is a statistical fact that high correlations between the independent variables improve the regression coefficient R^2 of a model, it is sometimes unrealistic due to the interactions between the independent variables.

The statistical strength of the model does not change even though the R^2 increases, this is due to colinearity.

Consequently, scatter plots becomes a significant method to estimate the linearities and relationship between the quantitative variables in a data set.

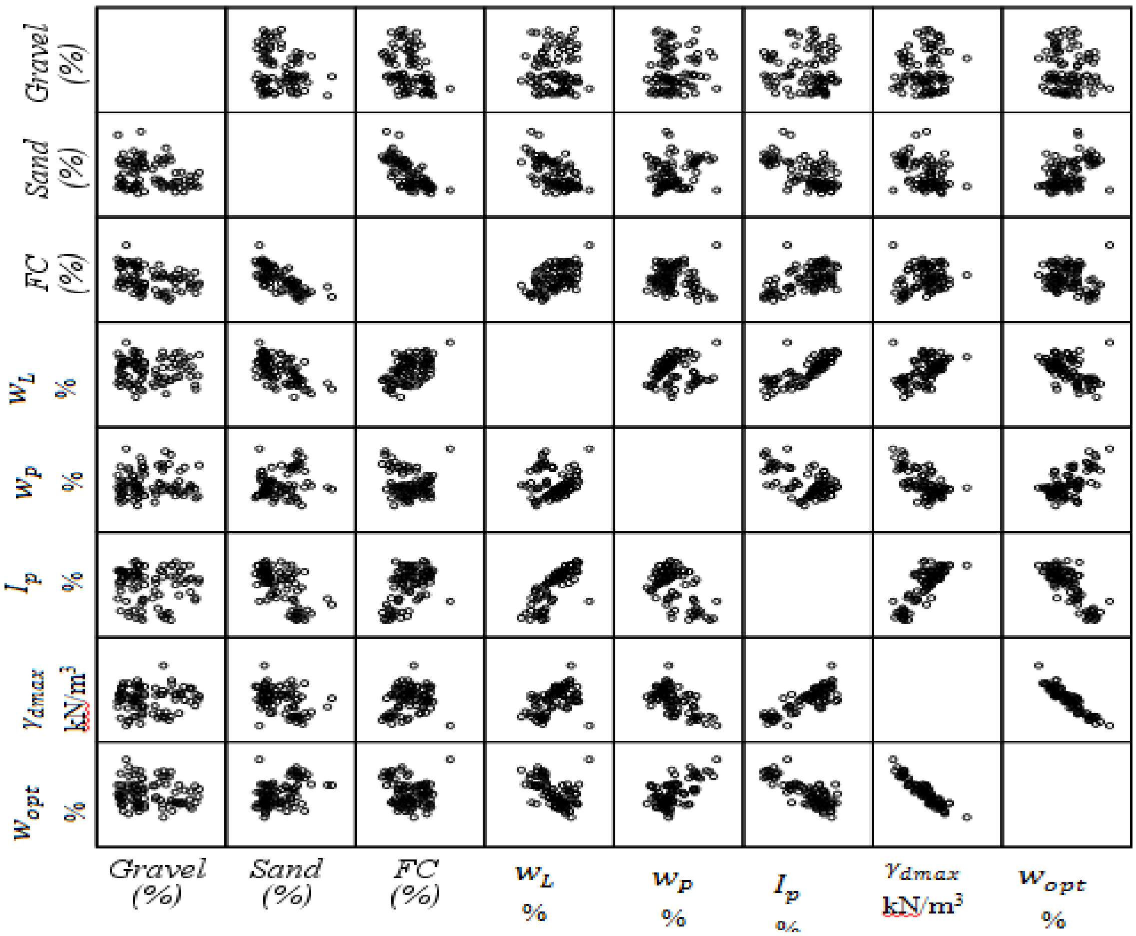


Figure 4.1: Scatterplot matrix for the demonstration of the interaction between independent and dependent variables of standard Proctor compaction analysis

4.2.2. Correlation matrix

The correlation coefficient, *R*, which is the relative predictive power of a model, is given for each analysis. It is a descriptive measure between -1 and +1. Table 4.2 states the accuracy of the correlation measured by the coefficient of determination, R^2. Minus sign indicates inverse proportion between two variables whilst plus sign represents a direct proportion. A correlation matrix analysis indicates the strength of the linear relationship between two random variables.

It is an indicative tool to determine the independent variables that are highly correlated with the dependent variables. Furthermore, it shows the linear interactions between two independent variables. High correlations between two independent variables may indicate over-fit in the model.

Table 4.2: 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 standard Proctor compaction test parameters are shown in Table 4.3.

Table 4.3: Correlation matrix results for standard Proctor compaction data analysis.

	Gravel (G) (%)	Sand (S) (%)	FC (%)	w_L (%)	w_P (%)	I_p (%)	γ_{dmax} kN/m³	w_{opt} (%)
Gravel, G (%)	1	-0.202	-0.476	0.021	0.038	-0.004	0.108	-0.160
Sand, S (%)	-0.202	1	-0.765	-0.611	0.359	-0.686	-0.514	0.521
FC (%)	-0.476	-0.765	1	0.534	-0.347	0.618	0.391	-0.362
w_L (%)	0.021	-0.611	0.534	1	-0.068	0.841	0.597	-0.609
w_P (%)	0.038	0.359	-0.347	-0.068	1	-0.598	-0.665	0.588
I_p (%)	-0.004	-0.686	0.618	0.841	-0.598	1	0.840	-0.809
γ_{dmax} (kN/m³)	0.108	-0.514	0.391	0.597	-0.665	0.840	1	-0.937
w_{opt} (%)	-0.160	0.521	-0.362	-0.609	0.588	-0.809	-0.937	1

4.3. Multiple Regression of Maximum Dry Unit Weight and Optimum Water Content of Standard Proctor Compaction

Pearson's product moment correlations were performed to determine variable colinearity and to aid in the selection of the independent variables i.e. Atterberg limits and gradation parameters as said earlier. A stepwise multiple regression method was conducted to evaluate the best model that contain statistically significant, intuitively meaningful predictive variables for predicting maximum dry unit weight and optimum water content of both standard and modified Proctor compaction tests. It also maximizes the accuracy of a model with an optionally reduced number of predictor variables.

In the stepwise approach,

- The first step identifies the "best" one-variable model. Subsequent steps introduce two, three etc. variables to the model.
- The addition of a subsequent predictor variable depends on the *F* or *t-test* value and also an increase in R^2 and decrease in *SEE* values.
- The addition of variables to the model stops when the "minimum F-to-enter" exceeds a specified probability level (i.e. 0.15).

For maximum dry unit weight, γ_{dmax}

At step 1 of the analysis, plasticity index, I_p entered into the regression Equation and was significantly related to maximum dry unit weight, γ_{dmax}, F=164.4, p< 0.05(pre-selected significant value). The R^2 was 0.682, indicating approximately 68.2% of the variance of maximum dry unit weight could be accounted for by plasticity index. Also the *SEE* of the model was 0.251. The regression Equation is represented in Equation (4.1).

In step 2, liquid limit, w_L, entered into the Equation and the R^2 increased to 0.732. The statistical significance values can be seen in the regression output below. Equation (4.2) represents the regression model at this step.

The predictor variable, fines content, *FC*, was entered at the third step and the R^2 value was 0.76 implying that 76% of maximum dry unit weight could be accounted for by the plasticity index, liquid limit and fines content. The standard error of estimate is 0.813 which is less than that of step two (0.851). It is noticed that all the predictor variables had *P value* less than 0.05 hence the regression model shown in Equation (4.3) is very significant. Also the *ANOVA* analysis shows the overall goodness of fit of the model.

The last step, step 4, plastic limit (w_p) entered the model and there was not any significant change in the R^2 value even though the number of predictor variables increased from three to four. The *P values* of I_p, w_L and w_p were 0.899, 0.955 and 0.988 respectively, these were all greater than the pre-significant value of 0.05 thus they are not significant. This explains that even though w_L and w_p is not strongly correlated (Table 4.3), the combination of the two in a model makes the model statistically insignificant. The regression Equation is Equation 4.4. The stepwise regression output of the maximum dry unit weight of standard proctor is shown below;

Stepwise Regression output of maximum dry unit weight, γ_{dmax}

<u>Step 1</u>

$$\gamma_{dmax} = 16.862 + 0.143I_p \quad (4.1)$$

R^2	*Adj.* R^2	*SEE*	*F*	*P(sig.)*
0.687	0.682	0.89	164.43	0.000

Coefficients

	B	*Std. Err of* β	*t*	*P-value*
Constant	16.862	0.253	66.45	0.000
I_p	0.143	0.01	12.82	0.000

<u>Step 2</u>

$$\gamma_{dmax} = 18.97 + 0.2I_p - 0.085w_L \quad (4.2)$$

R^2	*Adj.* R^2	*SEE*	*F*	*P(sig.)*
0.732	0.725	0.851	101.2	0.000

Coefficients

	B	*Std.Err of* β	*t*	*P-value*
Constant	18.97	0.639	29.68	0.000
I_p	0.20	0.019	10.4	0.000
w_L	-0.085	0.024	-3.55	0.000

<u>Step 3</u>

$$\gamma_{dmax} = 19.20 + 0.22I_p - 0.08w_L - 0.023FC \quad (4.3)$$

R^2	*Adj.* R^2	*SEE*	*F*	*P(sig.)*

0.76	0.75	0.813	76.78	0.000

ANOVA (Overall Goodness of Fit)

Effect	*Sum of Squares*	*Df*	*Mean Squares*	*F*	*P-value*
Regression	152.19	3	50.7	76.78	0.000
Residual	48.23	73	0.66		
Total	200.42	76			

Coefficients

	β	*Std.Err of* β	*t*	*P-value*
Constant	19.20	0.62	31.09	0.000
I_p	0.22	0.02	11.195	0.000
w_L	-0.08	0.023	-3.585	0.000
FC	-0.023	0.008	-2.86	0.005

<u>Step 4</u>

$$\gamma_{dmax} = 19.3 - 0.022\text{FC} + 0.25I_p - 0.11w_L + 0.02w_P \quad (4.4)$$

R^2	*Adj.* R^2	*SEE*	*F*	*P(sig.)*
0.759	0.746	0.88	56.79	0.000

Coefficients

	β	*Std.Err of* β	*t*	*P-value*
Constant	19.295	0.64	30.38	0.000
FC	-0.022	0.008	-2.84	0.006
I_p	0.25	1.974	0.13	0.899
w_L	-0.11	1.971	-0.06	0.955
w_P	0.02	1.976	0.01	0.988

Subsequently, based on the criteria for selecting the best model, the model for predicting maximum dry unit weight of a standard Proctor compaction test using Atterberg and gradation parameters of lateritic soils in Ghana is;

$$\gamma_{dmax} = 19.20 + 0.22I_p - 0.08w_L - 0.023FC \tag{4.3}$$

Using this Equation, when all the predictor variables are set to zero, γ_{dmax} is 19.2 kN/m^3, this falls within the mean value of 19.9 kN/m^3 with a standard deviation of ± 1.62 shown in Table 4.1. This also attests to the model's predictive power.

Also, the residual plots of the model are shown in Figure 4.2. It consists of four graphs; (a) Normal Probability Plot, (b) Residual values versus fitted value, (c) Histogram of the Residual, and (d) Observation order of the Residual values. In the illustration of the residual versus the fitted values, the residual values should be close to zero for an accurate model. Besides, if a particular trend (linear, parabolic and hyperbolic etc.) is observed in the plot, the mathematical model must be changed to fit that trend. In the residual versus fitted values scatter plot, there is no particular pattern hence the linear model is acceptable.

In the Normal Probability and Histogram plot, it is noticed that the distribution of the residuals is very close to normal with one variable being far from the line of best fit. In the observation order, a demonstration of an outlier encountered during the model development is noticed.

Finally, a plot of the predicted and the measured γ_{dmax} is represented in Figure 4.3. The prediction and confidence intervals are also shown in this graph. It is noticed that all the samples fall within the prediction interval with the exception of one which represents the outlier which may be due to experimental error. This shows that the model can be used confidently to predict the maximum dry unit weight of standard Proctor compaction test using fines content, liquid limit, and plasticity index of lateritic soils in Ghana.

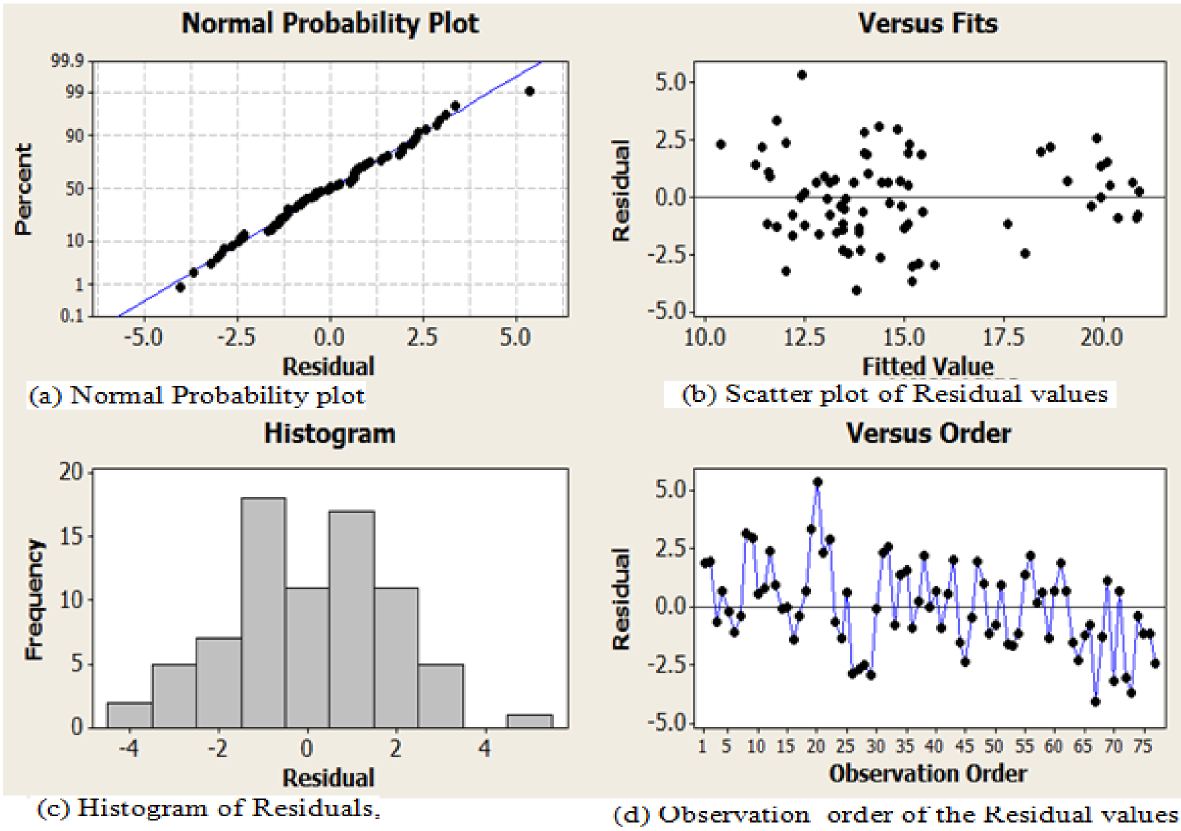


Figure 4.2: Residual plots for the multiple regression model correlating γ_{dmax} with Gradation and Atterberg limit parameters for a standard Proctor

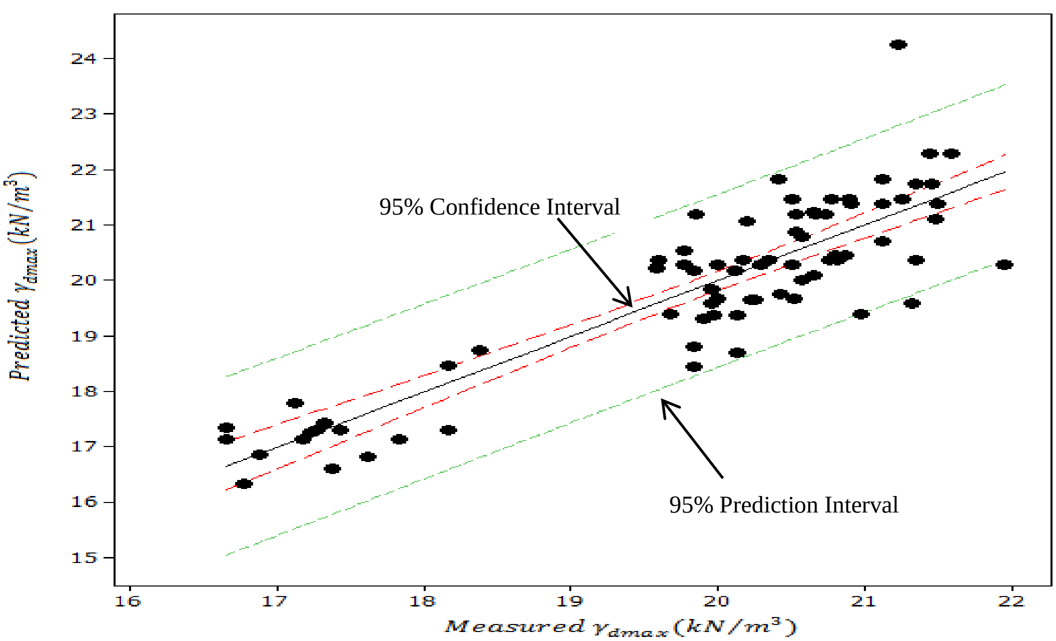


Figure 4.3: Plot of predicted and measured γ_{dmax} using Equation 4.3

For optimum water content, w_{opt}

At Step 1, plasticity index entered the model since it has a strong relation with optimum water content than the rest of the predictor variables. The R^2 was 0.66, indicating that approximately 66% of the variance of optimum water content could be accounted for by plasticity index. Also the *SEE* of the model was 1.98. The regression Equation is represented in Equation (4.5).

At the second step, liquid limit (w_L) was added to the model and the R^2 increased to 0.686. In step 3, fines content (*FC*) was included and the R^2 increased to 0.71 and the *SEE* reduced from 1.89 in step 2 to 1.85. At these stages, all the variables were statistically significant since the *P-values* were less than 0.05.

However, with the addition of a fourth variable i.e. plastic limit (w_p) there was no change in the R^2 value of the model and also the *P-value* of w_L, w_p and I_p were greater than 0.05 indicating that they are not statistically significant. The stepwise regression output is shown below.

<u>Step 1</u>

$$w_{opt} = 20.91 - 0.289I_p \tag{4.5}$$

R^2	*Adj. R^2*	*SEE*	*F*	*P(sig.)*
0.655	0.65	1.98	142.52	0.000

Coefficients

	β	*Std. Err of* β	*t*	*P-value*
Constant	20.91	0.548	38.1	0.000
I_p	-0.289	0.024	-11.94	0.000

<u>Step 2</u>

$$w_{opt} = 20.02 - 0.34I_p + 0.05w_L \tag{4.6}$$

R^2	*Adj. R^2*	*SEE*	*F*	*P(sig.)*
0.686	0.678	1.89	81.06	0.000

Coefficients

	B	*Std.Err of* β	*t*	*P-value*
Constant	20.02	0.619	32.3	0.000
I_p	-0.34	0.03	-11.44	0.000
w_L	0.05	0.02	2.73	0.008

<u>Step 3</u>

$$w_{opt} = 17.15 - 0.42I_p + 0.049\text{FC} + 0.117w_L \qquad (4.7)$$

R^2	*Adj. R^2*	*SEE*	*F*	*P(sig.)*
0.706	0.694	1.849	58.67	0.000

ANOVA (Overall Goodness of Fit)

Effect	*Sum of Squares*	*df*	*Mean Squares*	*F*	*P-value*
Regression	601.778	3	200.59	58.67	0.000
Residual	249.602	73	3.42		
Total	851.38				

Coefficients

	β	*Std.Err of* β	*t*	*P-value*
Constant	17.153	1.412	12.15	0.000
I_p	-0.416	0.045	-9.24	0.000
FC	0.049	0.018	2.69	0.008
w_L	0.117	0.052	2.24	0.03

<u>Step 4</u>

$$w_{opt} = 17.17 + 0.049\text{FC} + 0.56w_L - 0.445w_P - 0.86I_p \qquad (4.8)$$

R^2	*Adj. R^2*	*SEE*	*F*	*P(sig.)*
0.707	0.69	1.86	43.4	0.000

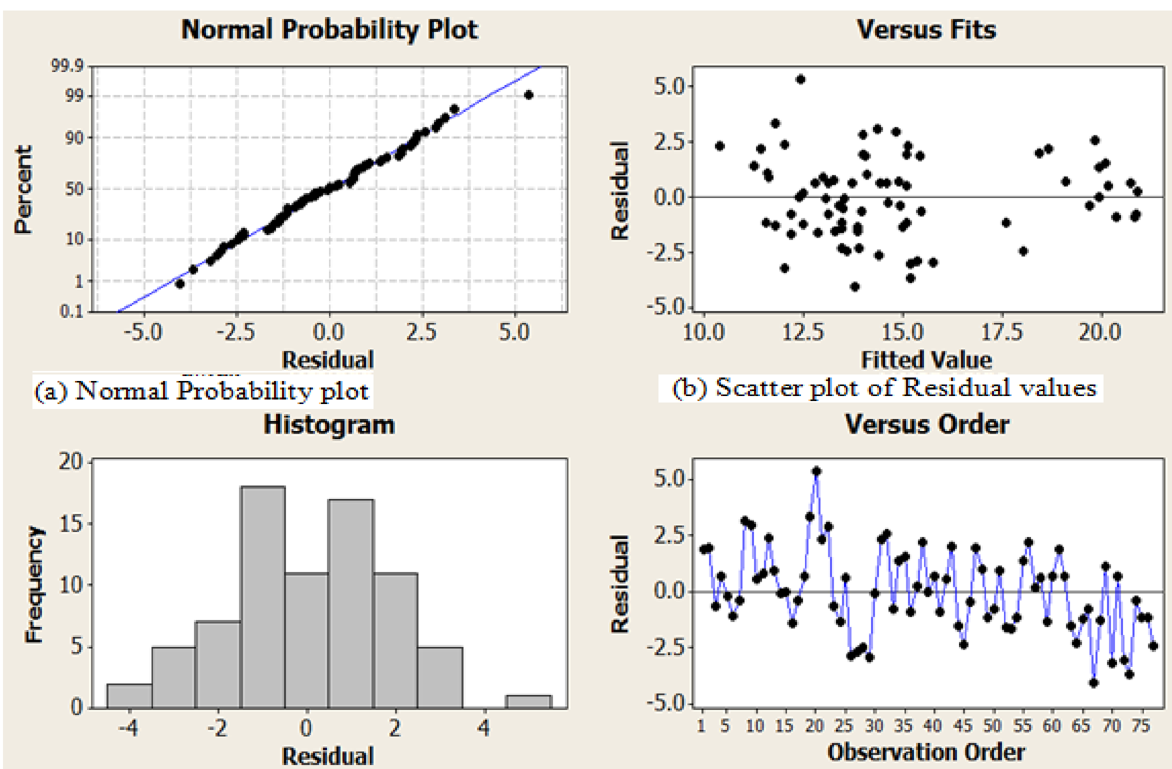
Coefficients

	β	*Std.Err of* β	*T*	*P-value*
Constant	17.17	1.45	11.88	0.000
FC	0.049	0.018	2.67	0.009
w_L	0.56	4.485	0.125	0.9
w_P	-0.445	4.494	-0.099	0.92
I_p	-0.861	4.49	-0.192	0.848

The best model for predicting the optimum water content of a standard Proctor compaction test is Equation (4.7):

$$w_{opt} = 17.15 - 0.42I_p + 0.049\text{FC} + 0.117w_L \tag{4.7}$$

The residual plots of this model are shown in Figure 4.4. The characteristics of the residual plots are similar to that in Figure 4.2 for maximum dry unit weight.



(a) Normal Probability plot

(b) Scatter plot of Residual values

(c) Histogram of Residuals,

(d) Observation order of the Residual values

Figure 4.4: Residual plots for the multiple regression model correlating w_{opt} with Gradation and Atterberg limit parameters for a standard Proctor

Final plot of the measured and predicted values using Equation (4.7) is shown in Figure 4.5.

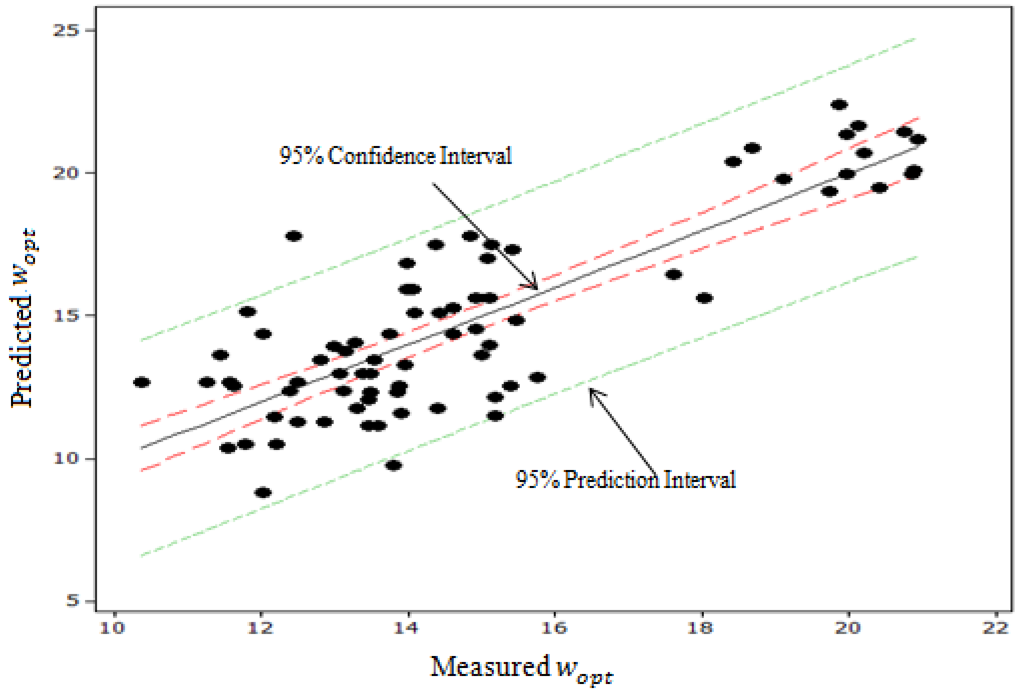


Figure 4.5: Plot of predicted and measured w_{opt} using Equation 4.7

It is noticed that majority of the data fall within the prediction interval for both the maximum dry unit weight and optimum water content using the proposed empirical Equations i.e. Equations 4.3 and 4.7 respectively. Subsequently, these models can be used to predict the standard Proctor compaction test parameters of lateritic soils in Ghana with much confidence during the preliminary stages of an earthwork construction projects.

4.4. Multiple Regression of Maximum Dry Unit Weight and Optimum Water Content of Modified Proctor Compaction

A similar approach used for the standard Proctor was used in this section. As already mentioned in Chapter 3, a total of 80 soil samples in Table 3.3 and 3.5 were used to develop and validate a model to predict the compaction characteristics from gradation and Atterberg limit test variables. The statistical analysis results for modified Proctor were similar to that found for standard Proctor. This is due to the fact that the testing procedure for the two is the same with only the compactive effort and number of layers changing.

Furthermore, although the two compaction tests were not conducted on the same soil samples, the soil samples differ slightly in their composition since the sampling was done within the same geographical location.

The statistical descriptive of the dependent and independent variables for the samples excluding the 10 samples for validation are shown in Table 4.4.

From the table, the maximum dry unit weight of the samples ranges from 19.1kN/m^3 to 25.6kN/m^3; the optimum moisture content is between 6.3% and 14.5%. The highest liquid limit is 64 with the lowest being 24.4. The correlation matrix between the dependent and independent variables are shown in Table 4.5.

Table 4.4: Descriptive statistics of data for modified Proctor analysis.

	Gravel G %	**Sand S %**	**FC %**	w_L **%**	w_P **%**	I_p **%**	γ_{dmax} **kN/m^3**	w_{opt} **%**
N	70	70	70	70	70	70	70	70
Range	44.1	57.5	70.1	39.6	27.4	40.2	6.5	8.2
Minimum	.9	11.0	10.1	24.4	11.8	.8	19.1	6.3
Maximum	45.0	68.5	80.2	64.0	39.2	41.0	25.6	14.5
Mean	17.1	34.5	48.39	48.44	21.67	26.78	22.91	9.58
Std. Deviation	12.74	16.37	18.70	9.70	5.90	11.41	1.74	2.11
Variance	162.19	267.89	349.75	94.02	34.76	130.24	3.02	4.47
Skewness	0.57	0.49	-0.09	-0.38	0.94	-1.16	-0.76	0.76
Kurtosis	-0.93	-1.20	-0.97	-0.72	0.56	0.16	-0.35	-0.45

Table 4.5: Correlation matrix results for modified Proctor compaction data analysis

	Gravel (G) (%)	**Sand (S) (%)**	**FC (%)**	w_L **(%)**	w_P **(%)**	I_p **(%)**	γ_{dmax} **kN/m^3**	w_{opt} **(%)**
Gravel, G (%)	1	-0.19	-0.51	0.01	0.08	-0.03	0.08	-0.15
Sand, S (%)	-0.19	1	-0.74	-0.62	0.24	-0.65	-0.46	0.5
FC (%)	-0.51	-0.74	1	0.53	-0.26	0.59	0.35	-0.34
w_L (%)	0.01	-0.62	0.53	1	-0.01	0.86	0.58	-0.59
w_P (%)	0.08	0.24	-0.26	-0.01	1	-0.53	-0.66	0.60
I_p (%)	-0.03	-0.65	0.59	0.86	-0.53	1	0.84	-0.81
γ_{dmax} (kN/m^3)	0.08	-0.46	0.35	0.58	-0.66	0.84	1	-0.94
w_{opt} (%)	-0.15	0.5	-0.40	-0.59	0.6	-0.81	-0.94	1

The scatterplot is of the same trend of the Standard Proctor in Figure 4.1.

For maximum dry unit weight, γ_{dmax}

In estimating the maximum dry unit weight of modified proctor compaction from Atterberg limit and gradation parameters, the regression output of the stepwise analysis is shown below;

<u>Step 1</u>

$$\gamma_{dmax} = 19.5 + 0.12I_p \tag{4.9}$$

R^2	*Adj. R^2*	*SEE*	*F*	*P(sig.)*
0.696	0.69	0.964	156.0	0.000

Coefficients

	β	*Std. Err of* β	*t*	*P-value*
Constant	19.5	0.29	65.92	0.000
I_p	0.12	0.01	12.49	0.000

<u>Step 2</u>

$$\gamma_{dmax} = 22.04 + 0.19I_p - 0.088w_L \tag{4.10}$$

R^2	*Adj. R^2*	*SEE*	*F*	*P(sig.)*
0.761	0.753	0.863	106.43	0.000

Coefficients

	β	*Std. Err of* β	*t*	*P-value*
Constant	22.04	0.656	33.62	0.000
I_p	0.19	0.018	10.85	0.000
w_L	-0.088	0.021	-4.24	0.000

<u>Step 3</u>

$$\gamma_{dmax} = 22.34 + 0.2I_p - 0.08w_L - 0.02\mathrm{FC} \tag{4.11}$$

R^2	*Adj. R^2*	*SEE*	*F*	*P(sig.)*
0.784	0.774	0.826	79.7	0.000

ANOVA (Overall Goodness of Fit)

Effect	*Sum of Squares*	*df*	*Mean Squares*	*F*	*P-value*
Regression	163.25	3	54.42	79.72	0.000
Residual	45.05	66	0.683		
Total	208.30				

Coefficients

	β	*Std. Err of* β	*t*	*P-value*
Constant	22.34	0.638	35.04	0.000
I_p	0.2	0.018	11.6	0.000
w_L	-0.08	0.02	-4.24	0.000
FC	-0.02	0.007	-2.66	0.009

<u>Step 4</u>

$$\gamma_{dmax} = 22.34 - 0.017\text{FC} - 0.06w_L - 0.024w_P + 0.181I_p \quad (4.12)$$

R^2	*Adj.* R^2	*SEE*	*F*	*P(sig.)*
0.784	0.77	0.83	58.8	0.000

Coefficients

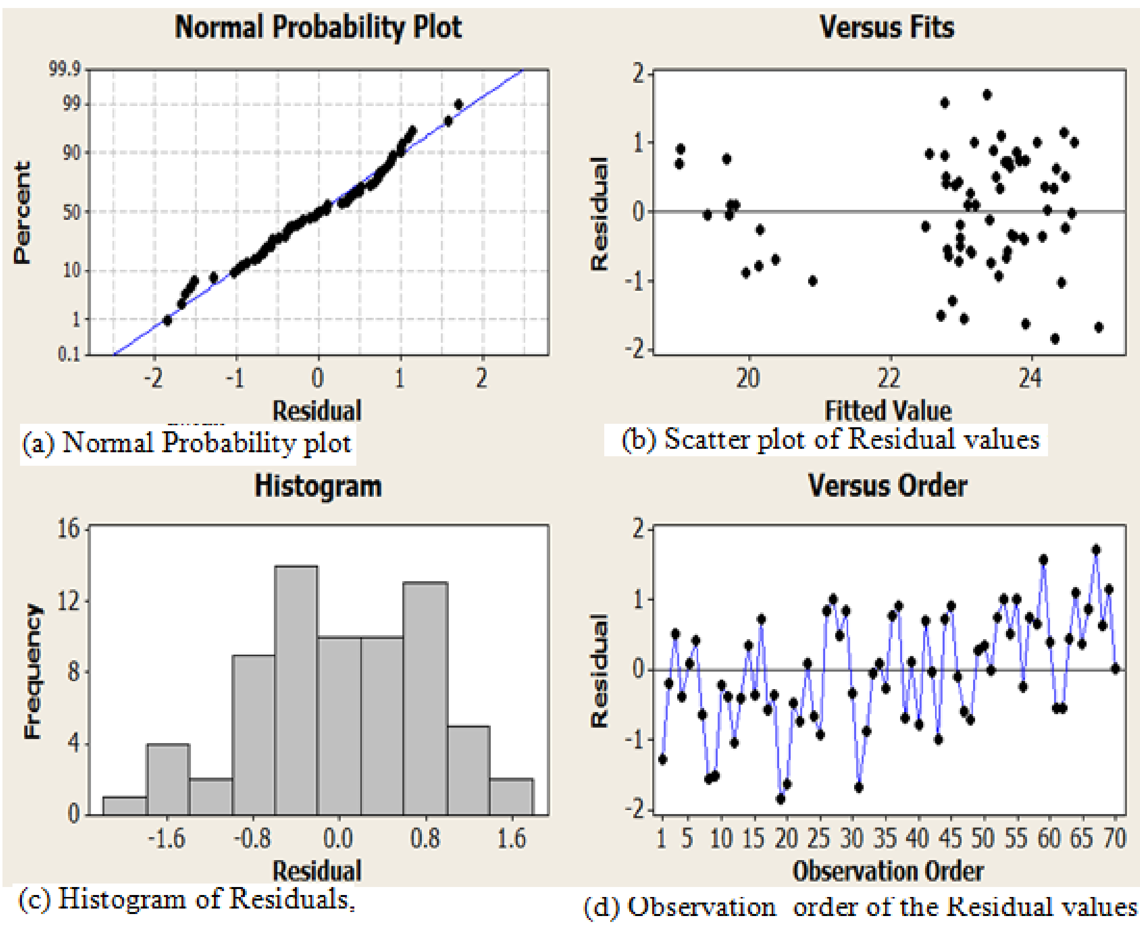
	β	*Std. Err of* β	*t*	*P-value*
Constant	22.34	0.65	34.23	0.000
FC	-0.017	0.006	-2.63	0.011
w_L	-0.06	1.61	-0.04	0.97
w_P	-0.024	1.614	-0.015	0.988
I_p	0.181	1.612	0.112	0.91

The best empirical model for estimating the maximum dry unit weight of a modified Proctor compaction test from Atterberg and gradation parameters of lateritic soils in Ghana is:

$$\gamma_{dmax} = 22.34 + 0.2I_p - 0.08w_L - 0.02\text{FC} \quad (4.11)$$

It has the highest R^2 value of 0.784. This indicates that 78.4% of maximum dry unit weight can be confidently estimated using this Equation above. The *ANOVA* analysis shows that the model is significant. Likewise, the same can be said about the *P-value* of the predictors.

Furthermore, residual plots of this model were done to see the statistical strength of the model and this can be seen in Figure 4.6. In the residual plots, it was observed that the linear model is acceptable since there is no peculiar trend in the scatterplot of the fitted value versus residual. Also, the model is very significant since the residuals are very close to the line of best fit. The final plot of the measured versus the predicted γ_{dmax} is shown in Figure 4.7. It is noticed that approximately, all the samples fall within the prediction band indicating the statistical strength of the model.



(a) Normal Probability plot

(b) Scatter plot of Residual values

(c) Histogram of Residuals.

(d) Observation order of the Residual values

Figure 4.6: Residual plots for the multiple regression model correlating γ_{dmax} with Gradation and Atterberg limit parameters for a modified Proctor

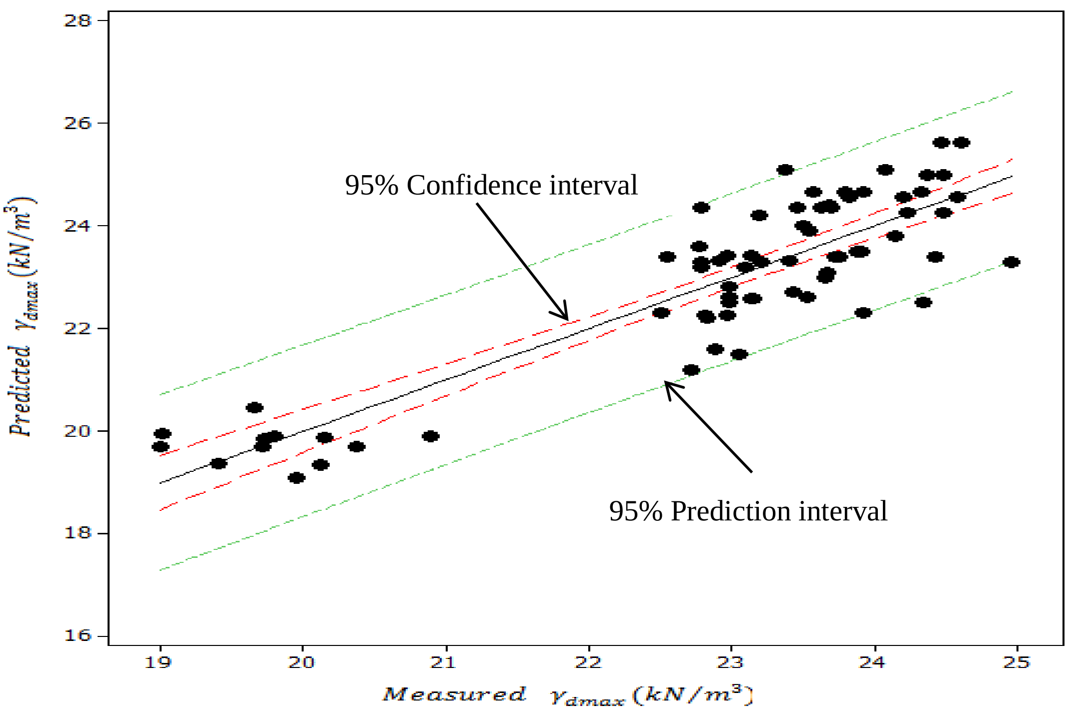


Figure 4.7: Plot of predicted and measured γ_{dmax} using Equation 4.11

For optimum water content, w_{opt}

In the same way, for optimum water content, the Equations below were proposed after the stepwise regression. The best empirical model is:

$$w_{opt} = 10.75 - 0.23I_p + 0.021FC + 0.08w_L \tag{4.15}$$

The R^2 was 0.713, indicating approximately 71.3% of the variance of optimum water content could be accounted for by using this model. The regression output of the stepwise analysis is shown below.

<u>Step 1</u>

$$w_{opt} = 13.58 - 0.15I_p \tag{4.13}$$

R^2	*Adj. R^2*	*SEE*	*F*	*P(sig.)*
0.649	0.644	1.261	126.01	0.001

Coefficients

	β	*Std. Err of β*	*T*	*P-value*
Constant	13.58	0.387	35.09	0.000
I_p	-0.15	0.013	-11.23	0.000

Step 2

$$w_{opt} = 11.1 - 0.212I_p + 0.086w_L \quad (4.14)$$

R^2	*Adj.* R^2	*SEE*	*F*	*P(sig.)*
0.691	0.681	1.19	74.83	0.000

Coefficients

	β	*Std. Err of* β	*T*	*P-value*
Constant	11.096	0.907	12.23	0.000
I_p	-0.212	0.024	-8.69	0.000
w_L	0.086	0.029	2.99	0.004

Step 3

$$w_{opt} = 10.75 - 0.23I_p + 0.021\text{FC} + 0.08w_L \quad (4.15)$$

R^2	*Adj.* R^2	*SEE*	*F*	*P(sig.)*
0.713	0.7	1.16	54.68	0.000

ANOVA (Overall Goodness of Fit)

Effect	*Sum of Squares*	*df*	*Mean Squares*	*F*	*P-value*
Regression	220.033	3	73.34	54.68	0.000
Residual	88.53	66	1.34		
Total	308.56				

Coefficients

	β	*Std. Err of* β	*T*	*P-value*
Constant	10.746	0.89	12.02	0.000
I_p	-0.23	0.024	-9.224	0.000
w_L	0.08	0.028	2.92	0.005
FC	0.021	0.009	2.27	0.003

Step 4

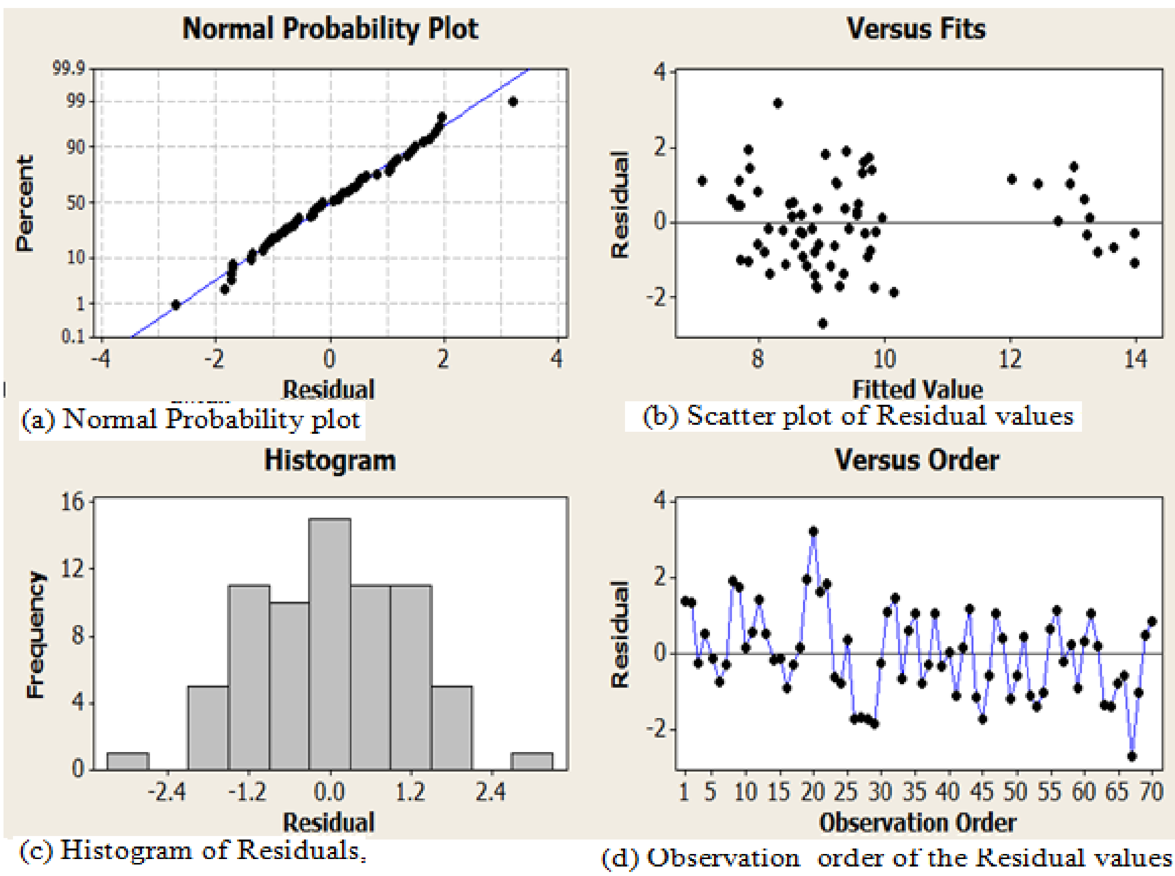
$$w_{opt} = 10.758 + 0.02\text{FC} + 0.26w_L - 0.183w_P - 0.412I_p \quad (4.16)$$

R^2	*Adj.* R^2	*SEE*	*F*	*P(sig.)*
0.713	0.695	1.17	40.40	0.000

Coefficients

	β	*Std. Err of* β	*T*	*P-value*
Constant	10.76	0.915	11.76	0.000
FC	0.02	0.009	2.25	0.028
w_L	0.26	2.258	0.12	0.91
w_P	-0.183	2.262	-0.08	0.936
I_p	-0.412	2.260	-0.182	0.856

The residual plots of the model and the plot of the measured versus the predicted have shown in Figure 4.8 and 4.9 respectively.



(a) Normal Probability plot

(b) Scatter plot of Residual values

(c) Histogram of Residuals,

(d) Observation order of the Residual values

Figure 4.8: Residual plots for the multiple regression model correlating w_{opt} with Gradation and Atterberg limit parameters for a modified Proctor

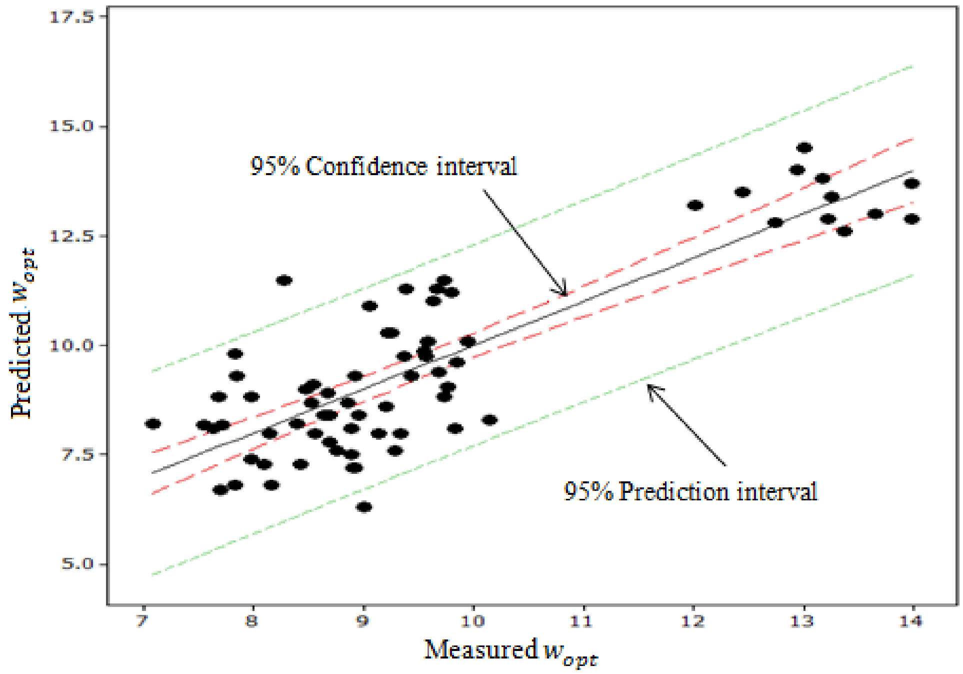


Figure 4.9: Plot of predicted and measured w_{opt} using Equation 4.15

4.5. Validation of the developed models

The developed models were validated using a different set of data not seen by the model. The data in Table 3.4 and 3.5 were used for the standard and modified Proctor empirical models respectively.

For standard Proctor compaction parameters;

Table 4.6 shows the results of the measured and the predicted maximum dry unit weight and optimum water content. The highest absolute error between measured and predicted maximum dry unit weight is 1.19 showing that this model is very accurate, likewise for optimum water content, the maximum absolute error is 1.65 which is also very small.

Additionally, graphical representations of the validated model for maximum dry unit weight and optimum water content are shown in Figure 4.10 and 4.11 respectively. The R^2 values are also shown and they show very high values which attest to the statistical strength of the models for γ_{dmax} and w_{opt}.

Table 4.6: Validation of standard Proctor compaction parameters models.

Maximum dry unit weight γ_{dmax} $(\mathrm{kN/m^3})$			**Optimum water content** w_{opt} **(%)**		
Measured	Predicted	Abs. Error	Measured	Predicted	Abs. Error
17.94	17.9	0.08	17.75	18.9	1.11
17.78	18.2	0.41	17.54	17.6	0.06
16.10	14.9	1.19	24.5	23.3	1.18
19.80	18.7	1.08	17.5	16.9	0.61
17.90	18.1	0.23	18.2	17.7	0.46
18.65	18.7	0.06	17.6	16.7	0.89
17.70	18.3	0.58	17.4	17.6	0.24
20.10	19.1	0.98	16.5	14.9	1.65
19.60	18.6	0.96	16.2	16.0	0.17
19.40	18.8	0.62	15.8	16.3	0.54
18.80	18.2	0.58	17.3	0.53	0.53

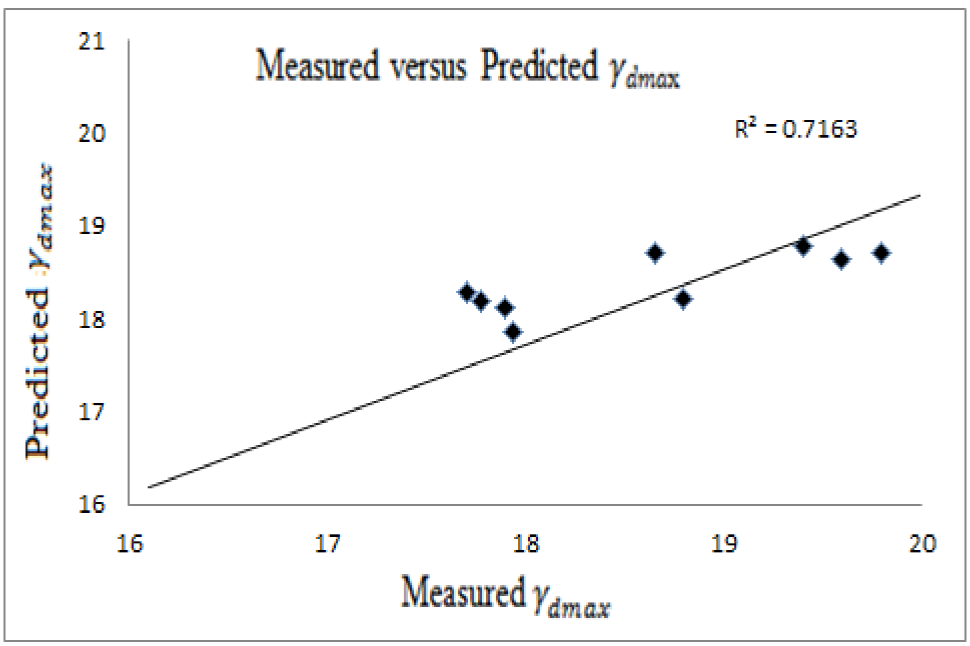


Figure 4.10: Plot of predicted and measured γ_{dmax} for standard Proctor model validation

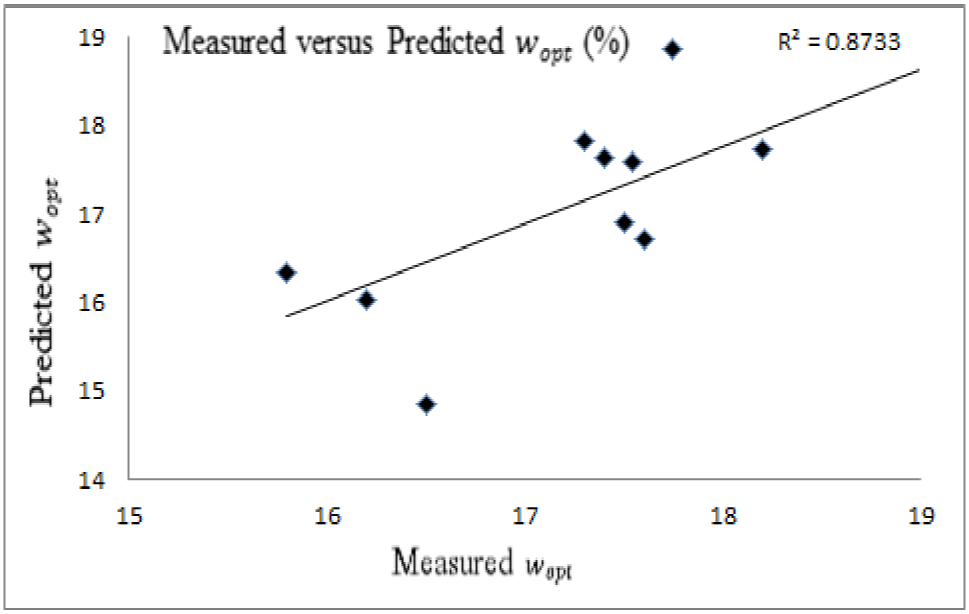


Figure 4.11: Plot of predicted and measured w_{opt} for standard Proctor model validation

Correspondingly, the output of the validation of the model results for modified Proctor compaction test parameters is shown in Table 4.7, Figure 4.12 and Figure 4.13. the high R^2 values show that these parameters can be accurately predicted from Gradation and Atterberg limit test parameters using these models.

Table 4.7: Validation of modified Proctor compaction parameters models.

Maximum Dry Unit Weight γ_{dmax} (kN/m^3)			**Optimum water Content w_{opt} (%)**		
Measured	Predicted	Abs. Error	Measured	Predicted	Abs. Error
20.6	21.2	0.57	11.7	11.6	0.11
23.1	22.4	0.69	10.9	10.2	0.67
21.8	22.2	0.36	11.8	10.4	1.44
22.3	22.9	0.58	10.3	9.6	0.66
21.1	20.5	0.6	12.3	12.6	0.25
24.5	23.7	0.8	8.7	8.6	0.12
22.6	22.2	0.36	10.4	10.3	0.08
22.9	22.2	0.34	10.8	10.3	0.54
23.1	22.6	0.45	10.1	9.8	0.29
22.9	22.9	0.04	10	9.4	0.57

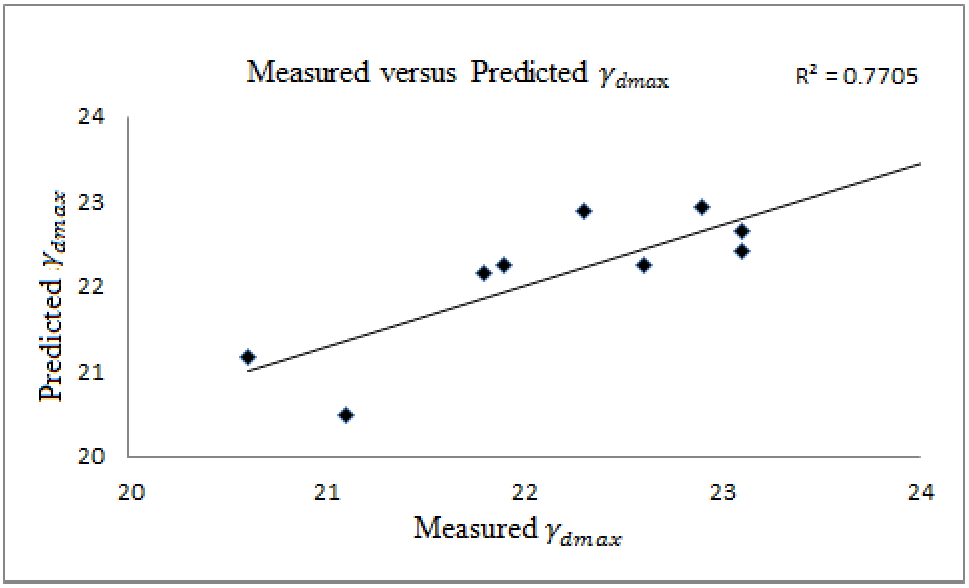


Figure 4.12: Plot of predicted and measured γ_{dmax} for modified Proctor model validation

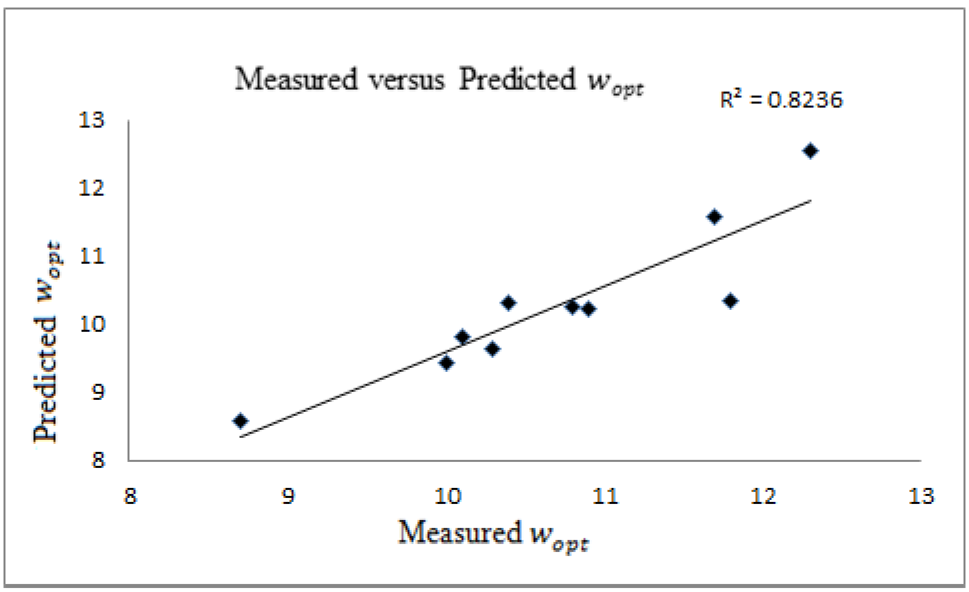


Figure 4.13: Plot of predicted and measured w_{opt} for modified Proctor model validation

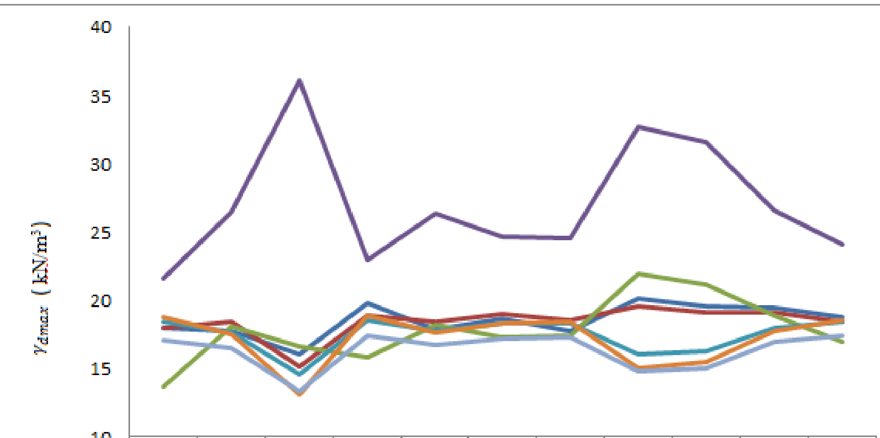
4.5. Comparison of Developed Models with Some Existing Models.

Some of the existing models were used to predict the compaction test parameters of lateritic soils that were used to validate the developed models and compared with the proposed models in this study.

For standard Proctor compaction parameters;

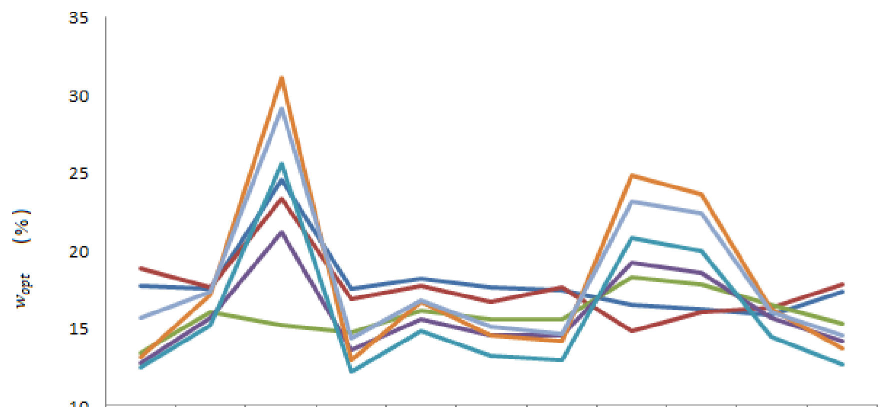
As observed in Figure 4.14, all the models could be used to predict the maximum dry unit weight of a standard Proctor test with the exception of Torrey using w_L Equation. It was noticed that the predicted γ_{dmax} using these models is close to the measured γ_{dmax}. However, these models should be used with caution when predicting the standard proctor compaction characteristics of lateritic soils in Ghana.

A similar observation is seen in Figure 4.15 though there was no extreme variation from the measured w_{opt}, extreme care should be taken in the application of these models during the pre-feasibility studies of a project using lateritic soils in Ghana.



	1	2	3	4	5	6	7	8	9	10	11
γ_{dmax} (kN/m³)	17.9	17.8	16.1	19.8	17.9	18.7	17.7	20.1	19.6	19.4	18.8
Developed Model	17.9	18.5	15.1	18.9	18.4	19.0	18.5	19.6	19.1	19.1	18.4
Torrey (1970) Model using Ip	13.7	18.1	16.6	15.8	18.2	17.3	17.4	21.9	21.1	18.9	16.9
Torrey (1970) Model using wL	21.6	26.5	36.1	23.0	26.4	24.7	24.6	32.7	31.6	26.6	24.0
Al- Khafaji (1993) Model using USA soils	18.4	17.7	14.5	18.5	17.8	18.3	18.4	16.0	16.3	17.9	18.4
Al- Khafaji (1993) Model using Iraqi soils	18.8	17.5	13.1	18.8	17.6	18.3	18.4	15.0	15.4	17.8	18.6
Gurtug and Sridharan (2004) Model	17.0	16.5	13.3	17.5	16.7	17.2	17.3	14.8	15.1	16.9	17.4

Figure 4.14: Comparison of developed model with some existing models for γ_{dmax} for standard Proctor

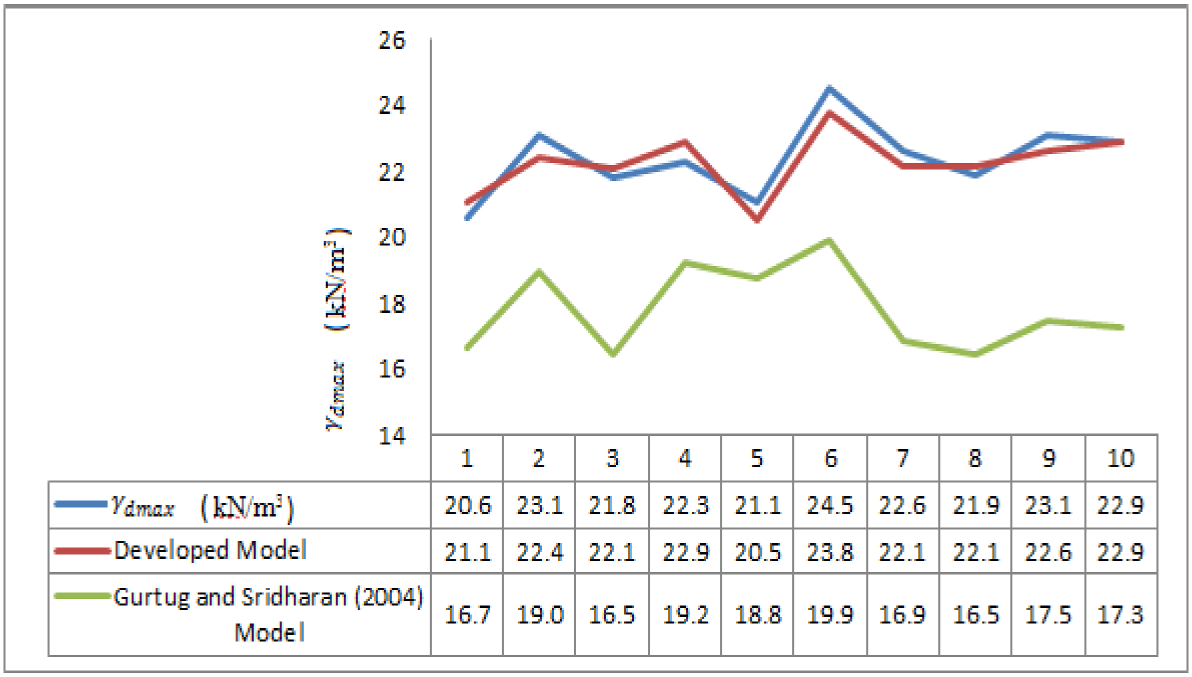


	1	2	3	4	5	6	7	8	9	10	11
w_{opt} (%)	17.75	17.54	24.50	17.5	18.2	17.6	17.4	16.5	16.2	15.8	17.3
Developed Model	18.9	17.6	23.3	16.9	17.7	16.7	17.6	14.9	16.0	16.3	17.8
Torrey (1970) Model using Ip	13.4	16.0	15.1	14.7	16.1	15.5	15.6	18.3	17.8	16.5	15.3
Torrey (1970) Model using wL	12.8	15.6	21.2	13.6	15.5	14.6	14.5	19.2	18.6	15.7	14.2
Al- Khafaji (1993) Model using USA soils	12.5	15.2	25.6	12.2	14.8	13.2	12.9	20.8	19.9	14.4	12.6
Al- Khafaji (1993) Model using Iraqi soils	13.1	17.2	31.1	13.0	16.7	14.5	14.2	24.9	23.6	16.3	13.7
Gurtug and Sridharan (2004) Model	15.7	17.3	29.2	14.3	16.8	15.1	14.7	23.2	22.4	16.0	14.5

Figure 4.15: Comparison of developed model with some existing models for w_{opt} for standard Proctor

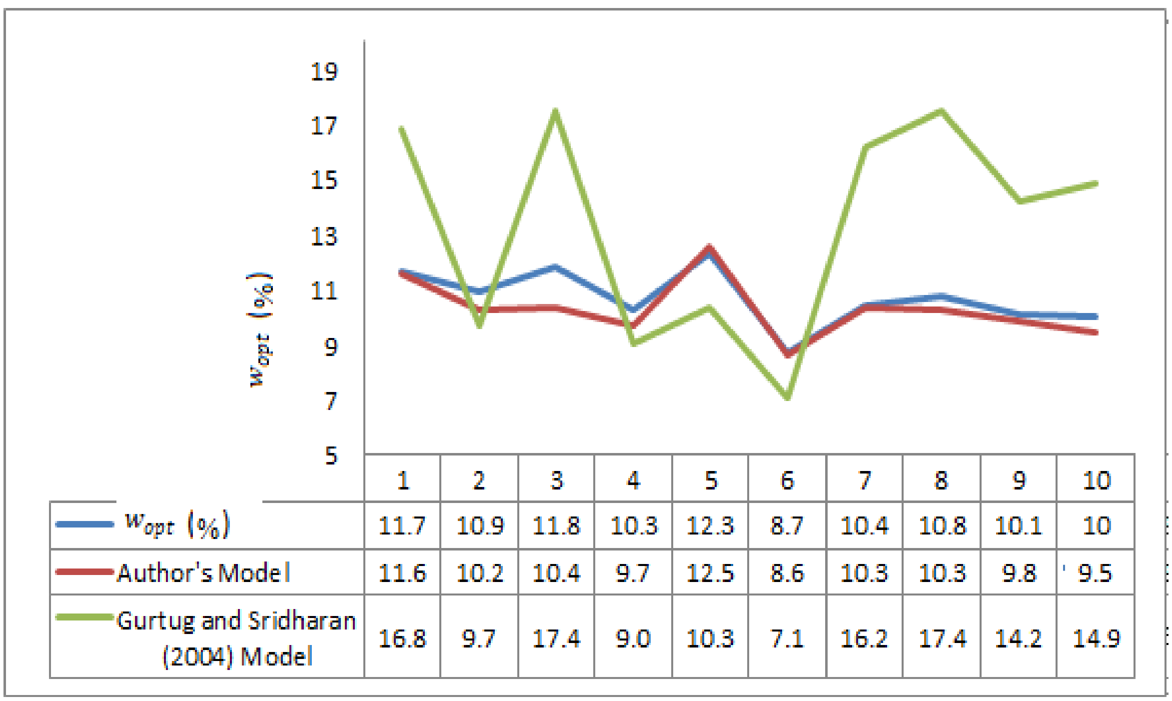
For modified Proctor compaction parameters;

As noticed in Figure 4.16 and 4.17, a wide variation between the experimental values and values estimated Gurtug and Sridharan's model was observed. This may be due to the fact that they used clayey soils in developing the model. This confirms that correlated models are used for particular soils or soils within the same geographical zone. Other models were not used for comparisons since most of them used just standard Proctor compaction tests parameters in developing their models. Also the availability of the parameters played a very important role in using the model.



	1	2	3	4	5	6	7	8	9	10
γ_{dmax} (kN/m³)	20.6	23.1	21.8	22.3	21.1	24.5	22.6	21.9	23.1	22.9
Developed Model	21.1	22.4	22.1	22.9	20.5	23.8	22.1	22.1	22.6	22.9
Gurtug and Sridharan (2004) Model	16.7	19.0	16.5	19.2	18.8	19.9	16.9	16.5	17.5	17.3

Figure 4.16: Comparison of developed model with some existing models for γ_{dmax} for modified Proctor



	1	2	3	4	5	6	7	8	9	10
w_{opt} (%)	11.7	10.9	11.8	10.3	12.3	8.7	10.4	10.8	10.1	10
Author's Model	11.6	10.2	10.4	9.7	12.5	8.6	10.3	10.3	9.8	9.5
Gurtug and Sridharan (2004) Model	16.8	9.7	17.4	9.0	10.3	7.1	16.2	17.4	14.2	14.9

Figure 4.17: Comparison of developed model with some existing models for w_{opt} for modified Proctor

CHAPTER 5

CONCLUSIONS AND RECOMMENDATION

5.1. Conclusions

In order to ensure the quality of compaction test carried out in the field, the compaction test parameters namely; maximum dry unit weight and optimum water content measured in the laboratory are dependable criteria. Based on the study's outcome, the objectives in this dissertation have been achieved. 88 lateritic soils in Ghana were used to develop and validate empirical Equations to estimate the standard Proctor compaction parameters from Atterberg and Gradation parameters. Similarly, 80 samples were used for modified Proctor compaction parameters.

Based on the analysis of laboratory data, the following conclusions were drawn;

1. The relationship between the Atterberg Limit parameters; liquid limit (w_L), plastic limit (w_P), plasticity Index (Ip), and the compaction test parameters are the same irrespective of the compaction type. A similar observation was seen with respect to Gradation parameters namely; Gravel percentage (G), Sand percentage (S), and Fine content (FC) percent.
2. It was observed that maximum dry unit weight (γ_{dmax}) and optimum water content, (w_{opt}) have better correlations with plasticity index than the liquid limit and plastic limit.
3. The liquid limit of the samples used for regression analyses ranges from 19.6% to 51.4% for standard Proctor and from 24.4% to 64% for modified Proctor. The plastic limit ranges from 9.5% to 31.5% for standard Proctor and from 11.8% to 39.2% for modified Proctor.
4. Stepwise multiple linear regression analyses were used for model development in order to minimize over-fit in the model.

5. The proposed empirical models all have R^2 values greater than 0.7 and the Standard Error of Estimate, *SEE* was less than 2 indicating the high statistical strength of the models.
6. Also, it was observed that the R^2 values for modified Proctor were higher than that of the standard Proctor.
7. Empirical correlation models were found separately for standard and modified Proctor compaction parameters. It must be stressed out that since different soil samples were used for these compaction test types, the developed Equations should be used in accordance with the specified type of compaction.
8. In conclusion, during the feasibility stages of any earthworks project that involves the use of lateritic soils, the proposed Equations could be used to estimate the compaction test characteristics. It should be noted that these models do not serve as a replacement of field test hence testing should be done accordingly, they should only be used in preliminary design phase where there are limited time, financial limitations and large-scale testing.

5.2. Recommendations

1. The study's result is limited to only lateritic soils in Ghana, thus, it is recommended that in future, a study should be done to estimate the compaction test parameters using lateritic soils from other tropical countries.
2. Moreover, this work can be further be extended to incorporate other soil parameters like specific gravity, uniformity coefficient, etc. to develop a model to predict the compaction test parameters of lateritic soils.
3. Also, since there are about predominantly 3 types of soils namely; laterites and lateritic soils, micaceous soils and black cotton clays in Ghana, these soils should be studied in order to propose empirical Equations to estimate the compaction test parameters in the future.

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APPENDICES

APPENDIX A
ASTM TESTING PROCEDURES

1. PARTICLE SIZE DISTRIBUTION (GRADATION ANALYSIS)

ASTM D6913-04

Introduction

A sieve analysis consists of passing a sample through a set of sieves and weighing the amount of material retained on each sieve. Sieves are constructed of wire screens with square openings of standard sizes. The sieve analysis is performed on material retained on an U. S. Standard No. 200 sieve. Table 1gives a list of the U. S. Standard sieve numbers with their corresponding size of openings.

Significance:

The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution, and it is required in classifying the soil.

Table 1. U. S. Sieve Numbers and Associated Opening Sizes

Sieve No.	Opening Size (mm)	**Sieve No.**	Opening Size (mm)
4	4.75	**35**	0.500
5	4.00	**40**	0.425
6	3.35	**45**	0.355
7	2.80	**50**	0.300
8	2.36	**60**	0.250
10	2.00	**70**	0.212
12	1.70	**80**	0.180
14	1.40	**100**	0.150
16	1.18	**120**	0.125
18	1.00	**140**	0.106
20	0.85	**200**	0.075
25	0.71	**270**	0.053

30	0.60	**400**	0.038

Apparatus

1. Sieves, a bottom pan and a cover
2. A balance sensitive to 0.1g
3. Mortar and rubber pestle
4. Oven
5. Paint brush for cleaning sieves

Preparation of sample

The material to be treated is first air-dried, after which the aggregates present in the sample are thoroughly broken up with the fingers or with the mortar and pestle. The specimen to be tested should be large enough to be representative of the soil in the field. It should also be small enough not to overload sieves. Large soil samples are divided by using a riffle to preserve their grain-size distribution. The size of a representative specimen depends on the maximum particle size.

Procedure

1. Collect a representative oven-dry soil sample. Samples having largest particles of the size of No. 4 sieve opening (4.75 mm) should be about 500 grams. For soils having largest particles of greater than 4.75 mm, larger weights are needed.

2. Break the soil sample into individual particles using a mortar and a rubber-tipped pestle. (Note: The idea is to break up the soil into individual particles, not to break the particles themselves.)

3. Determine the mass of the sample within 0.1g (**W**).

4. Prepare a stack of sieves. A sieve with larger openings is placed above a sieve with smaller openings. The sieve at the bottom should be a No. 200. A bottom pan should

be placed under the No. 200 sieve. The sieves that are generally used in a stack are Nos. 4, 10, 20, 40, 60, 140 and 200; however, more sieves can be placed in between.

5. Pour the soil prepared in Step 2 into the stack of sieves from the top.

6. Place the cover on the top of the stack of sieves.

7. Agitate the stack of sieves by hand for about 10 to 15 minutes.

8. Stop shaking the sieves and remove the stack of sieves.

9. Weigh the amount of soil retained on each sieve and the bottom pan.

Calculations

1. Determine the mass of soil retained on each sieve (i.e., M_1, M_2, $\cdot\cdot\cdot$ Mn) and in the pan (i.e., Mp)
2. Determine the total mass of the soil: $M_1 + M_2 + \cdot\cdot\cdot + Mi + \cdot\cdot\cdot + Mn + Mp = \sum M$
3. Determine the cumulative mass of soil retained above each sieve. For the *i*th sieve, it is $M_1 + M_2 + \cdot\cdot\cdot + Mi$
4. The mass of soil passing the ith sieve is $\sum M - (M_1 + M_2 + \cdot\cdot\cdot + M_i)$
5. The percent of soil passing the ith sieve (or percent finer) is

$$F = \frac{\Sigma M - (M_1 + M_2 + \cdots + M_i)}{\Sigma M} \times 100$$

2. ATTERBERG LIMIT TEST-ASTM D4318-10

LIQUID LIMIT TEST

Introduction

When a cohesive soil is mixed with an excessive amount of water, it will be in a liquid state and flow like a viscous liquid. When the viscous liquid dries gradually due to loss of moisture, it will pass into a plastic state. With further loss of moisture, the soil will pass into a plastic state. With even further reduction of moisture, the soil will pass into a semi-solid and then into a solid state.

The moisture content, *w*, (%) at which the cohesive soil will pass from a liquid state to a plastic state is called the liquid limit of the soil. Similarly, plastic limit and shrinkage limit can be explained. These limits are called Atterberg limits.

Atterberg Limits

<table>
<tr><td colspan="4">→ Moisture content increasing</td></tr>
<tr><td>Solid</td><td>Semisolid</td><td>Plastic</td><td>Liquid</td></tr>
<tr><td colspan="4">Shrinkage Limit (SL) Plastic Limit (PL) Liquid Limit (LL)</td></tr>
</table>

Equipment

1. Casagrande liquid limit device
2. Grooving tool
3. Moisture cans
4. Porcelain evaporating dish
5. Spatula
6. Oven
7. Balance sensitive up to 0.01g
8. Plastic squeeze bottle

9. Towels

Procedure

1. Determine the mass of moisture cans (W1).

2. Put 250g of air-dry soil, passed through No. 40 sieve into an evaporating dish. Add water and mix the soil to the form of a uniform paste.

3. Place some soil paste into the liquid limit device. Smooth the surface with a spatula such that maximum depth is 8 mm.

4. Using the grooving tool, cut a groove along the centerline of the soil pat.

5. Turn the crank at the rate of 2 revs. / second. Count the number of blows (N) for the groove in the soil to close through a distance of ½ in. If N = 25-35, collect a moisture sample from the cup to a moisture can and determine the mass (W2).

6. If N < 25, place the soil back to the evaporating dish and clean the device. Stir the soil (to dry it up) with spatula. Then redo steps 3, 4 and 5.

7. Remove the soil from the cup of LL device and clean it carefully.

8. Add more water to the soil paste in the evaporating dish and mix well. Repeat steps 3, 4 and 5 to get N = 20-25. Take a moisture sample from the cup. Clean the LL device.

9. Add more water to the soil paste in the evaporating dish and mix well. Repeat steps 3, 4 and 5 to get N = 15-20. Take a moisture sample from the cup. Clean the LL device.

10. Put three moisture cans in the oven to dry to constant mass (W3).

Calculation

1. Calculate mass of can, W1 (g)

2. Calculate mass of can + moist soil, W2 (g)

3. Calculate mass of can + dry soil, W3 (g)

4. Determine the moisture content for each of the three trials as

$$w\ (\%) = \frac{(W2 - W3)}{(W3 - W1)} \times 100\%$$

PLASTIC LIMIT TEST

Introduction

Plastic limit is defined as the moisture content, in percent, at which a cohesive soil will change from a plastic state to a semisolid state. In the lab, the plastic limit is defined as the moisture content (%) at which a thread of soil will just crumble when rolled to a diameter of 1/8 in. (3.18 mm).

Equipment

1. Moisture cans
2. Porcelain evaporating dish
3. Spatula
4. Ground glass plate
5. Balance sensitive up to 0.01 g
6. Plastic squeeze bottle
7. Oven

Procedure

1. Put 20g of air-dry soil, passed through No. 40 sieve into an evaporating dish.
2. Add water and mix the soil thoroughly.
3. Determine the mass of moisture cans (W1).

4. From the moist soil prepared in step 2, prepare several ellipsoidal-shaped soil masses by squeezing the soil with fingers.

5. Take one of the ellipsoidal-shaped soil masses and roll it on a glass plate using the palm of the hand. The rolling should be done at the rate of 80 strokes/min. Note that one complete backward and one complete forward motion of the palm constitutes a stroke.

6. When thread of soil reaches 1/8" in diameter, break it up in to several small pieces and squeeze it to form an ellipsoidal mass again.

7. Repeat steps 5 and 6 until the thread crumbles into several pieces when d = 1/8".

8. Collect the small crumbled pieces into the moisture can and put the cover on the can.

9. Take the other ellipsoidal soil masses formed in step 4 and repeat steps 5 through 8.

10. Determine the mass of moisture can plus wet soil (W2).

11. Place moisture can into the oven to dry to constant mass (W3).

<u>Calculations</u>

1. Calculate mass of can, W1 (g)

2. Calculate mass of can + moist soil, W2 (g)

3. Calculate mass of can + dry soil, W3 (g)

4. Calculate plastic limit

$$PL = \frac{(W2 - W3)}{W3 - W1} \times 100$$

5. Calculate plasticity index, PI = LL – PL.

3. STANDARD PROCTOR COMPACTION TEST- ASTM D698-12

Introduction

For construction of highways, airports, and other structures, it is often necessary to compact soil to improve its strength. Proctor (1933) developed a laboratory compaction test procedure to determine the maximum dry unit weight of compaction of soils, which can be used for specification of field compaction. This test is referred to as the Standard Proctor Compaction Test. It is based on compaction of soil fraction passing No. 4 U.S. sieve.

Equipment

1. Compaction mold
2. No. 4 U.S. sieve
3. Standard Proctor hammer (2.5kg)
4. Balance sensitive up to 0.01g
5. Balance sensitive up to 0.1g
6. Large flat pan
7. Steel straight edge
8. Moisture cans
9. Drying oven
10. Plastic squeeze bottle with water

Proctor Compaction Mold:

The Proctor compaction mold is 101.6mm in diameter. The inner volume is 944cm^3.

Procedure

1. Obtain a representative of air dry soil and break the soil lumps.
2. Sieve the soil on a No. 4 U.S. sieve. Collect all the minus 4 sieve materials.
3. Add water to the minus 4 sieve materials and mix thoroughly to bring the moisture content to about 8%.

4. Determine the weight of the Proctor Mold + base plate (not extension), **W1** (lb).

5. Attach the extension to the top of the mold.

6. Pour the moist soil in three equal layers. Compact each layer uniformly with the Standard Proctor hammer 25 times before each additional layer of loose soil is poured. At the end of the three-layer compaction, the soil should extend slightly above the top of the rim of the compaction mold.

7. Remove the extension carefully.

8. Trim excess soil with a straight edge.

9. Determine the weight of the Proctor Mold + base plate + compacted moist soil, **W2** (lb).

10. Remove the base plate from the mold. Extrude the compacted moist soil cylinder.

11. Take a moisture can and determine its mass, **W3** (g).

12. From the moist soil extruded in step 10, collect a moist sample in a moisture can (step 11) and determine the mass of moist soil + can, **W4** (g).

13. Place the moisture can with soil in the oven to dry to a constant weight.

14. Break the rest of the soil cylinder by hand and mix with leftover moist soil. Add more water and mix to raise moisture content by 2%.

15. Repeat steps 6-12. In this process, the weight of the mold + base plate + moist soil (**W2**) will first increase with the increase in moisture content and then decrease. Continue the test until at least two successive decreased readings are obtained.

16. The next day, determine the mass of the moisture cans + soil samples, **W5** (g) (from step 13).

<u>Calculation</u>

1. Determine weight of the mold **W1** (step 4).
2. Determine weight of the mold + compacted moist soil , **W2** (step 9).
3. Determine weight of the compacted moist soil = **W2-W1.**
4. Moist unit weight $\boldsymbol{\gamma}$ = weight of the compacted moist soil / volume of mold
5. Determine mass of moisture can, **W3** (step 11).
6. Determine mass of moisture can + moist soil, **W4** (step 12).

7. Determine mass of moisture can + dry soil, **W5** (step 16).
8. Compaction moisture content , w (%) = (**W4** - **W5**) x 100 / (**W5** - **W3**).
9. Dry unit weight $\boldsymbol{\gamma_d} = \boldsymbol{\gamma} / (1 + w (\%) / 100)$.

APPENDIX B

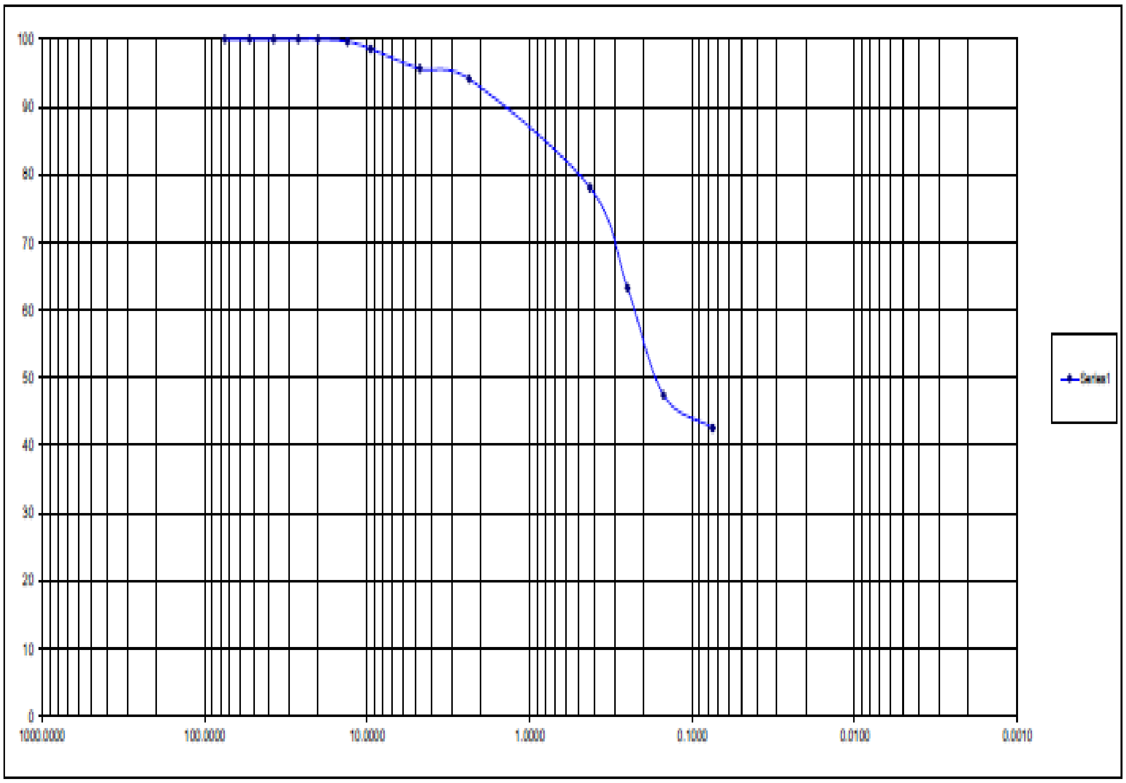
LABORATORY TEST SHEETS

Client	
Project	Tarkwa Iduapriem (GTSF)

ABP GHANA LTD

PARTICLE SIZE DISTRIBUTION

Sample ID	104	Sheet No.	Test #104
Sample Depth		Job No.	
Layer		Date	5th Sep, 2015
Sample Descript'n	Light-Brown, laterite	Locat'n	GTSF - South of Wall 6



SIEVE ANALYSIS

Sieve Size (mm)	75	63	53	37.5	26.5	20	13.2	9.5	6.7	4.75	3.35	2.36	1.18	0.6	0.425	0.3	0.25	0.15	0.075
% Passing	100	100	100	100	100	100	100	99		96		94			78		63	47	43

Tested By : VICTOR

Checked :

ATTERBERG LIMIT TESTS DATA SHEET

ABP GHANA LTD

Standard : ASTM D4318

Project : Tarkwa Iduaprim, Ghana

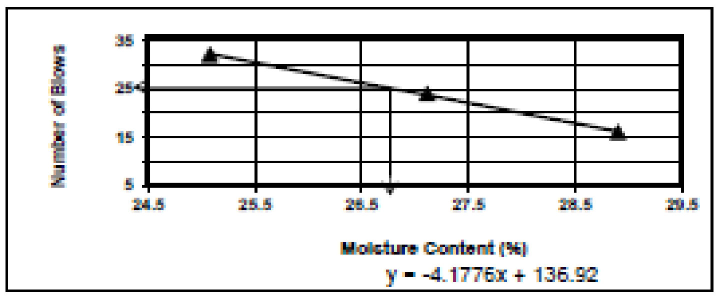
Project Name : GTSF

Client :

Sample Location: Greenfields Tailings Storage Facility		*Sample Number:*	Test #104
Test Location: GTSF - South of Wall 6		*Depth:*	
Description: Light-Brown, Laterite		*Layer*	

Liquid Limit - Casagrande Method:

Number of blows	32		24		16					
Container Number	B25	B54	B51	B22	B1	B4				
Mass of sample and container - wet (g)	38.4	38.2	40.1	39.3	43.9	42.9				
Mass of sample and container - dry (g)	34.3	34.1	35.4	34.7	38.1	37.2				
Mass of container (g)	17.9	17.8	18.0	17.8	17.8	17.7				
Mass of moisture (g)	4.1	4.1	4.7	4.6	5.8	5.7				
Mass of dry sample (g)	16.4	16.3	17.4	16.9	20.3	19.5				
Moisture content (%)	25.0	25.2	27.0	27.2	28.6	29.2				
Average moisture content (%)	25.1		27.1		28.9					

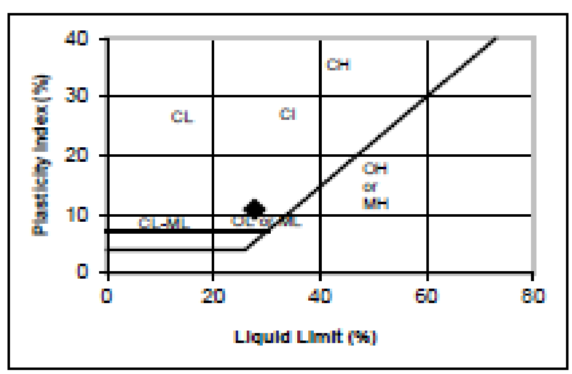


Plastic Limit

Container number	B58	B20								
Mass of sample and container - wet (g)	23.1	22.6								
Mass of sample and container - dry (g)	22.3	21.9								
Mass of container (g)	17.6	17.7								
Mass of moisture (g)	0.8	0.7								
Mass of dry sample (g)	4.7	4.2								
Moisture content (%)	17.0	16.7								
Average moisture content (%)	16.8									

Results:

Liquid Limit %	=	27.7
Plastic Limit %	=	16.8
Plasticity Index %	=	10.9
Classification	=	OL-ML



MODIFIED PROCTOR COMPACTION TEST- ASTM D1557

ABP GH. LTD

Client : AngloGold Ashanti - Tarkwa

Section: GTSF - South of Wall 6

Project: Iduapriem 'GTSF

Lab no: 104

Contract no:

Date: 5/9/2015

Request no: 104

MATERIAL DESCRIPTION: Light-Brown, Laterite

COMPACTION						
Drum No.		1	2	3	4	5
Mass of Sample		2000	2000	2000	2000	2000
Mould No.		2	2	2	2	2
Mass of Mould + Wet Mat	gm.	3986	4181	4243	4110	3944
Mass of Mould	gm.	1842.2	1842.2	1842.2	1842.2	1842.2
Mass of Wet Material (a-b)	gm.	2143.8	2338.8	2400.8	2267.8	2101.8
Volume of Mould	cm^3.	944	944	944	944	944
Wet Density (c/d*1000)	Kg/m^3	2271	2478	2543	2402	2226
Approx. D.D. (e/[h+100])	Kg/m^3	2122	2273	2291	2126	1936
Dry Density (e/[n+100])	Kg/m^3	2122	2273	2290	2129	1940
MOISTURE CONTENTS						
Moisture Added	%	7	9	11	13	15
Moisture Added	ml.					
Container No.		E21	E1	E27	E9	E2
Mass of Container	gm.	214.8	214.8	218.2	216.6	215.4
Mass of Cont. + Wet Mat.	gm.	971.6	972.6	973.8	980.0	942.4
Mass of Cont. + Dry Mat.	gm.	922.0	910.0	898.6	893.0	848.8
Mass of Moisture (j-k)	gm.	49.6	62.6	75.2	87.0	93.6
Mass of Dry Material (k-i)	gm.	707.2	695.2	680.4	676.4	633.4
Moisture Content (l*100/m)	%	7.0	9.0	11.1	12.9	14.8
Hygroscopic Moisture (n-h)	%	0.0	0.0	0.1	-0.1	-0.2