Sudan University of Science and Technology
College of Graduate Studies

PREDICTION OF COMPACTION PARAMETERS FROM FINE SOIL INDEX PROPERTIES

التنبؤ بمعاملات الدمك من مؤشر خصائص التربة الناعمة

Case Study: Dam Complex of Upper Atbara Project

A Thesis Submitted in Partial Fulfillment for the Degree of Master of Science in Construction Engineering at Sudan University of Science and Technology

By

Mohamed Ibrahim Ahmed Ibrahim

Supervisor: Dr. Abusamra Awad Attaelmanan

March 2018

الآية

بسم الله الرحمن الرحيم

قال الله تعالى

هُوَ الَّذِي خَلَقَكُم مِّن طِينٍ ثُمَّ قَضَى أَجَلًا وَأَجَلٌ مُّسَمًّى
عِندَهُ ثُمَّ أَنتُمْ تَمْتَرُونَ

صدق الله العظيم

سورة الأنعام – الآية 2

DEDICATION

This Research is lovingly dedicated to our respective parents who have been our constant source of inspiration. They have been giving us the drive and discipline to tackle any task with enthusiasm and determination. Without their love and support this research would not have been made possible.

ACKNOWLEMENT

Foremost, I would like to express my sincere gratitude to my advisor

Dr: Abusamra Awad Attaelmanan for the continuous support of my M.Sc. study and research, for his patience, motivation, enthusiasm, and immense knowledge. His guidance helped me in all the time of research and writing of this thesis. I could not have imagined having a better advisor and mentor for my M.Sc. study.

I would also like to Thank Sudan University of science and technology, Dams Implementation Unit, Dam Complex of Upper Atbara Project and Prof. Ahmed Mohamed Elsharief at Building and Road Research Institute.

Last but not the least; I would like to thank my family: my parents Ibrahim Ahmed and Omkalthoum Nasor, for giving birth to me at the first place and supporting me spiritually throughout my life, and my wife Tiseer Mohamed Adam.

Abstract

Soil compaction is a vital part of the construction process. It is used for support of structural entities such as building foundations, roadways, walkways, and earth retaining structures to name a few. For a given soil type certain properties may deem it more or less desirable to perform adequately for a particular circumstance.

The main objective of this thesis is to obtain correlations between compaction characteristics of fine grained soils and their Atterberg limits. For this purpose, 40 samples have been collected from a borrow area of Burdana Quarry, which is located at the right bank upstream of Setit River, then the soil samples were tested at the laboratory of Dam complex of upper Atbara project.

In the analysis section, the Microsoft Office Excel software was used for the regression analysis. Attempts were made to obtain the relationships between Atterberg limits (Liquid Limit, Plastic Limit, and Plasticity Index) with the compaction parameters (Optimum Moisture Content, and Maximum Dry Density).

The results have shown that the Optimum Moisture Content and Maximum Dry Density have an excellent relationship with the Liquid Limit, other than the Plastic Limit and Plasticity Index. It was noted that the Optimum Moisture Content has also an excellent correlation with Maximum Dry Density other than the remaining parameters. Therefore, for the prediction of Optimum Moisture Content, and Maximum Dry Density of fine grained soils from the Atterberg Limits' correlations, it is recommended to use the compaction parameters and Liquid Limits' correlations due to their reliability in comparison with the other correlations.

The outcome of this thesis could be useful and applicable in different civil Engineering sectors, especially for preliminary investigations and prefeasibility study of civil engineering works such as construction of roads, earth dams, and other works that involve soils.

المستخلص

يعتبر دمك التربة جزءا اساسيا فى عمليات التشيد. يتم استخدامه لدعم الاجزاء الهيكلية مثل الاساسات والطرق والممرات والحوائط الساندة. وبالنسبة لدمك نوع معين من انواع التربة يمكن تحقيق خصائص معينة مرغوب فيها لمقاومة الظروف البيئية المطلوبة.

يتمثل الهدف الرئيسي لهذا البحث في دراسة العلاقة بين خصائص الدمك مع حدود اتربيرج . وإيجاد علاقات تستخدم للتنبؤ بخصائص الدمك من حدود اتربيرج. لهذا الهدف، تم جمع 40 عينة اختبار من المقلع الذي يقع بالضفة اليمنى لنهر ستيت,يمثل المقلع الرئيسي للمواد الطينية و تم إختبار العينات بمختبر مجمع سدي اعالي عطبرة وستيت.

في الجزء الخاص بتحليل النتائج، تم استخدام برنامج مايكروسوفت أوفيس اكسل لايجاد قيمة الارتباط بين العلاقات. وقد بذلت المحاولات للحصول على علاقات بين خصائص الدمك (المحتوى الرطوبي الأمثل و الكثافة الجافة القصوى) مع حدود اتربيرج المختلفة (حد السيولة ، حد اللدونة و مؤشر اللدونة).

أظهرت النتائج علاقات ممتازة تربط المحتوى الرطوبي الأمثل و الكثافة الجافة القصوى مع حد السيولة أفضل من علاقة نفس الخصائص مع حد اللدونة و مؤشر اللدونة. وجد أن هنالك علاقة ممتازة تربط ما بين المحتوى الرطوبي الامثل مع الكثافة الجافة القصوى أفضل من علاقة هذه الخاصية مع بقية الخصائص. لذلك، يوصى باستخدام علاقات خصائص الدمك مع حد السيولة نسبة لدقتها عند مقارنتها مع بقية العلاقات التي تربط خصائص الدمك مع بقية الخصائص.

نتائج هذه الأطروحة قد تكون مفيدة و قابلة للتطبيق في مختلف مجالات الهندسة المدنية، وخاصة بالنسبة لفحوصات المواقع الأولية ودراسات الجدوى للأعمال الهندسة المدنية مثل بناء الطرق والسدود الترابية وغيرها من الأعمال الترابيةالاخرى.

Table of Contents

Contents

List of Tables

List of Figures

List of Symbols and Abbreviations

AASHTO	American Association of Highway and Transportation Officials
ASTM	American Society of Testing And Materials
BS	British Standard Methods of Test
CE	Compaction Energy
CH	Inorganic Clays of High Plasticity
CL	Inorganic Clays of Low To Medium Plasticity
CMS	Construction Method Statement
C	CONSTANT
COW	Cut –Off Wall
DCUAP	Dam Complex of Upper Atbara Project
FC	Fine Content (Percentage of soil passing no. 200 sieve)
I_p	Plastic index
Gs	Specific Gravity
LL	Liquid Limit
MDD	Maximum Dry Density
MDD_{Equ}	Maximum Dry Density Found by The Proposed Equation
MDD_{pro}	Maximum Dry Density Found by Proctor Test
MIP	Mixed in Place
ML	Inorganic Silts of Low Plasticity
MH	Inorganic Silts of High Plasticity
MS	Method Statement
N	Number of Sample
OH	Organic clays of medium of high plasticity
OL	Organic Silts of Low Plasticity
OMC	Optimum Moisture Content
OMC_{Equ}	Optimum Moisture Content Found by the Proposed Equation
OMC_{pro}	Optimum Moisture Content Found by Proctor Test
OWC	Optimum Water Content
PL	Plastic Limit
PI	Plastic Index
R^2	Correlation Coefficient
SE	Standard Error of The Estimate
SU	Consistency Versus Un Drained Shear Strength
SL	Shrinkage Limit
T S	Technical specifications
USCS	Unified Soil Classification System
γ_d	Dry Unit Weight
$\gamma_{d\,max}$	Maximum Dry Unit Weight
W_l	Liquid Limit
W_p	Plastic limit
W_{opt}	Optimum Moisture Content

CHAPTER ONE

INTRODUCTION

CHAPTER ONE

INTRODUCTION

1.1 General

Every structure made by man, needs to be resting on a safe and stable ground. The engineering properties of the soil underneath the structure must be recognized to achieve the requirements of the ground safety and stability. However, more time and money are relatively required to obtain these engineering properties of the soil. On the other hand, investigating the index properties of the soil is much easier than investigating the other engineering properties; most of the engineering properties depend on the index properties, which involve simpler and quicker methods of testing to obtain, then the other engineering properties can be predicted satisfactorily from empirical correlations, which save time, money and effort.

One of the most important engineering techniques is the soil compaction, which is commonly performed in the engineering projects such as highways, railway subgrades, airfield pavements, earth dams, landfills and foundations. The soil compaction targets mainly the improvement of the engineering properties of the soil, like increasing the density, reducing the settlement, reducing the permeability, increasing the sheer strength and increasing the bearing capacity.

1.2 The Problem Statement

Compaction characteristics of soil are usually determined by performing specified method of testing in the laboratory and the test results are used in the field to ensure the quality of construction for the desired purposes. However, in the execution of mega projects e.g. (long road, and embankment dams), number of compaction test are to be executed is time consuming.

Thus it is very important to find a relation between the Atterberg limits and compaction characteristics, so it will be quicker cheaper and simpler method of testing.

1.3 Research Objectives

1. The main objective of this work is to obtain applicable relationships between Atterberg limits and compaction parameters of some soil samples collected from Dam complex of upper Atbara project borrow areas.

2. To Determine the Maximum dry Density of Fine soils.

3. To Determine the Optimum Moisture Content of fine soils.

1.4 Methodology

The literature on clayey soils and its properties was reviewed. Sources include books, journals, scientific papers, standards and online material from the internet. It is from the literature review that conceptual and methodological background of the entire research was established, and the Microsoft Office Excel software is used for the regression analysis. The methodology is based on laboratory evaluation of properties of soil. They were obtained from borrow areas of Dam Complex of Upper Atbara Project.

1.5 Research Outline

A brief description of each chapter included in this research is presented. Chapter one includes the need for the present research and briefly describes the contents of each chapter. Chapter two presents a review of soil classification, type of soils, tests and previous studies. Chapter three outlines the experimental program for achieving the objectives. The soils used in the research, the tests which have been carried out and the tests results are given. Chapter four discusses test results obtained from the liquid limit, plastic limit and plasticity index with maximum dry density (MDD) and optimum moisture content (OMC). Chapter five summarizes conclusions of this research and offers recommendations for future related research.

CHAPTER TWO

LITERATURE REVIEW

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

This chapter outlines the literature review on Properties of soils and their Types, classification of soils, factors affecting of the compaction, compaction test, Atterberg limit tests and previous studies.

2.2 Properties of the Soils

Fine soils exhibit considerable changes in physical properties with change of water content. Dry clay may be suitable as a foundation for heavy loads as long as it remains dry, but may turn into swamp when wet. Many of the fine soils shrink on drying and expand on wetting, which may adversely affect structures founded on them. Even when moisture content does not change, the properties of fine grained soils may vary considerably between their natural condition in the ground and their state after being disturbed (Raymond, 1997).

Silts are different from clays in many important respects, but because of their similarity in appearance, they often have been mistaken to distinguish one from the other, but they are easily identified by their behavior in the presence of water (Raymond, 1997).

2.3 Type of Soils

2.3.1 Silts

Silts are the non-plastic fine grained soils. They are unstable in the presence of water and have a tendency to become quick when saturated. Silts are fairly impervious, difficult to compact and are highly susceptible to frost heaving. Thus, silts have relatively low plasticity compared with clays. In terms of the classification chart they plot below the `A' line. The dilatancy property of silts, together with the quick reaction to vibration, affords a means of identifying typical silt in the loose wet state. When dry, silt can be pulverized easily under finger pressure and will have a smooth feel between the fingers in contrast to the grittiness of fine sand. For similar conditions of previous loading, the higher liquid limit of silt the more compressible it is (Raymond, 1997).

2.3.2 Clays

Clays are the plastic fines. Thus, they plot above the `A' line on the plasticity chart. They have low resistance to deformation when wet but become hard cohesive masses when they dry. Clays are virtually impervious, difficult to compact when wet, and impossible to drain by ordinary means. Large expansion and contraction with changes in water content are characteristics of clays. The higher the liquid limit of a clay, the more compressible it will be, and hence, in the most cases the liquid limit is used to distinguish between clays of high compressibility (H) and those of low compressibility (L) (Raymond, 1997).

In general, the higher the liquid limit and thus the plasticity index the more cohesive is the clay. Field differentiation among clays is accomplished by the toughness test in which the moist soil is molded and rolled into threads until crumbling occurs and by the dry strength test which measures the resistance of the clay to breaking and pulverizing (Raymond, 1997).

2.3.3 Organic Matter

Organic matter in the form of partly decomposed vegetation is the primary constituent of peaty soils. Thus, organic silts of low plasticity and organic clays of medium to high plasticity are found. Organic soils are dark grey or black in color, and usually have a characteristic odor of decay. Organic clays feel spongy in the plastic range as compared to inorganic clays. Soils containing organic matter are significantly more compressible and less stable than inorganic soils and they are undesirable for engineering uses (Nerea, 2012).

2.4 Soil Water Content

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

2.5 Soil Compaction

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

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

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

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

The importance of compaction as a practical means of achieving the desired strength, compressibility and permeability characteristics of fine-grained soils has been appreciated since the time as early as earth structures were built (Pandian et al, 1997).

The theory of why compaction results in a denser material and why there is a limit to the water content has been studied since Proctor first introduced his findings (Robert et al 2000) Proctor recognized that water content affects the compaction process. He believed the reason why a moisture-density curve “breaks over” at OWC was related to capillarity and frictional forces. He also believed that the force of the compactive effort was applied to overcoming the inter-particle friction of the clay particles. As the water content increased from dry of optimum to wet of optimum he believed that the water acted as a lubricant between the soil particles. The next compaction theory can be illustrated as: Compaction along the moisture density curve from dry to wet has four-step process (Robert et al 2000). First, the soil particles become hydrated as water is absorbed. Second, the water begins to act as a lubricant helping to rearrange the soil particles into a denser and denser state until OWC is reached. Third, the addition of water causes the soil to swell because the soil now has excess water. Finally, the soil approaches saturation as more water is added (Sivrikay et al, 2008).

2.5.1 Purposes of Soil Compaction

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

- The increase in density by compaction usually increases shearing resistance. This effect is highly desirable that it may allow the use of thinner pavement structure over a compacted subgrade or the use of steeper side slopes for an embankment. For the same density, the highest strengths are frequently obtained by using greater compactive efforts. Large-scale experiments have indicated that the unconfined compressive strength of clayey sand could be doubled by compaction (Alemayehu et al, 2009).
- When soil particles are forced together by compaction, both the number of voids contained in the soil mass and the size of the individual void spaces are reduced. This change in voids has an obvious effect on the movement of water through the soil. One effect is to reduce the permeability, thus reducing the seepage of water in earth dams, road embankments and water loss in reservoirs through deep percolation (Arora, 2004).
- Swelling characteristics is an important soil property. For expansive clay soils, the greater the density the greater the potential volume change due to swelling unless the soil is restrained. An expansive clay soil should be compacted at moisture content at which swelling will not be excessive. Although the conditions corresponding to a minimum swell and minimum shrinkage may not be exactly the same, soils generally may be compacted so that these effects are minimized (Amer et al ,2006)
- The primary advantage resulting from the compaction of soils used in embankments is that it reduces settlement that might be caused by consolidation of the soil within the body of the embankment. This is true because compaction and consolidation both bring about closer arrangement of soil particles. Densification by compaction prevents later consolidation and settlement of a structure (Alemayehu et al, 2009).

2.5.2 Factors Affecting Compaction Characteristics

Many researchers have identified the soil type, molding water content, amount of CE, method of compaction and admixtures (Teragahik, 1943).as the main parameters controlling the compaction behavior 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.

2.5.3 Compaction Energy Amount

The compactive effort is the amount of energy applied on the soil. A soil of given water content, if the amount of *CE* increases, the soils particles will be packed so that the γ_d increases. For a given *CE*, there is only one water content which gives the $\gamma_{d\ max}$. If the *CE* is increased the $\gamma_{d\ max}$ also increases, but the w_{opt} decreases (Alemayehu et al, 2009).

2.5.4 Necessity of Compaction

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

1. It increases the erosion resistance which helps in maintaining the ground surface in serviceable condition.
2. Compaction improves the engineering properties like shear strength, density, permeability etc. of the fill.
3. It reduces the amount of water that can be held in the soil by decreasing the void ratio and thus helps in maintaining the required strength.
4. It reduces the chances of slope stability problems like landslides.

2.5.5 Types of Compaction

There are four types of compaction effort on soil or asphalt:

1. Vibration
2. Impact
3. Kneading
4. Pressure

These different types of effort are found in the two principle types of compaction force: static and vibratory.

Static force is simply the deadweight of the machine, applying downward force on the soil surface, compressing the soil particles.

The only way to change the effective compaction force is by adding or subtracting the weight of the machine. Static compaction is confined to upper soil layers and is limited to any appreciable depth. Kneading and pressure are two examples of static compaction. Vibratory force uses a mechanism, usually engine-driven, to create a downward force in addition to the machine's static weight.

The vibrating mechanism is usually a rotating eccentric weight or piston/spring combination (in rammers). The compactors deliver a rapid sequence of blows (impacts) to the surface, thereby affecting the top layers as well as deeper layers. Vibration moves through the material, setting particles in motion and moving them closer together for the highest density possible. Based on the materials being compacted, a certain amount of force must be used to overcome the cohesive nature of particular particles. (Soil compaction handbook)

2.5.6 Factors Affecting Compaction in the Field

Compaction of a particular soil is affected by following given factors –

(i) Compactive Effort

In modern construction projects, heavy compaction machinery is deployed to provide compaction energy. Types of machinery required are decided based on type of soil to be compacted. The method of compaction is primarily of four types such as kneading, static, dynamic or impact and vibratory compaction. Different type of action is effective in different type of soils such as for cohesive soils; sheepsfoot rollers or pneumatic rollers provide the kneading action. Silty soils can be effectively compacted by sheepsfoot roller/pneumatic roller or smooth wheel roller. For compacting sandy and gravelly soil, vibratory rollers are most effective. If granular soils have some fines, both smooth wheel and pneumatic rollers can be used.

(ii) Moisture Content

Proper control of moisture content in soil is necessary for achieving desired density. Maximum density with minimum compacting effort can be achieved by compaction of soil near its OMC (Optimum Moisture Content). If natural moisture content of the soil is less than OMC, calculated amount of water should be added to soil with sprinkler attached to water tanker and mixed with soil by motor grader for uniform moisture content. When soil is too wet, it is required to be dried by aeration to reach up to OMC.

(iii) Soil Type

Type of soil has a great influence on its compaction characteristics. Normally, heavy clays, clays and silt offer higher resistance to compaction whereas sandy soils and coarse grained or gravelly soils are amenable for easy compaction. The coarse-grained soils yield higher densities in comparison to clays. A well-graded soil can be compacted to higher density.

(iv) Layer Thickness

The more the thickness of layer of earth subjected to field compaction, the less the energy input per unit weight of soil and hence, less is the compaction under each pass of the roller. Suitable thickness of soil of each layer is necessary to achieve uniform thickness. Layer thickness depends upon type of soil involved and type of roller, its weight and contact pressure of its drums. Normally, 200-300 mm layer thickness is optimum in the field for achieving homogeneous compaction.

(v) Contact Pressure

Contact pressure depends on the weight of the roller wheel and the contact area. In case of pneumatic roller, the tyre inflation pressure also determines the contact pressure in addition to wheel load. A higher contact pressure increases the dry density and lowers the optimum moisture content.

(vi) Number Of Roller Passes

Density of the soil increases with the number of passes of rollers but after optimum number of passes, further increase in density is insignificant for additional number of cases. For determination of optimum number of passes for given type of roller and optimum thickness of layer at a predetermined moisture content.

(vii) Speed Of Rolling

Speed of rolling has a very important bearing on the roller output.

The greater the speed of rolling, the more the length of embankment that can be compacted in one day. Speed was found to be a significant factor for vibratory rollers because its number of vibrations per minute is not related to its forward speed. Therefore, the slower the speed of travel, the more vibrations at a given point and lesser number of pass required to attain a given density. (manak nagar, lucknow-11)

2.5.7 Field Tests

Several different methods are used to determine the in-situ density of a soil;

1. Rubber balloon method

2. Sand-replacement (sand cone) method,

3. Core cutter method

4. Nuclear moisture-density meter method.

1. Balloon Density Apparatus

• The balloon density apparatus determines the in-place density of soil using a volume displacement method, similar to the sand cone density method.

• The apparatus consists of a graduated cylinder, an aluminum guard, reversibl aspirator type pump density plate and 10 pump, rubber balloons. [See Figure 2.1]

Figure 2.1: Balloon Density Apparatus

2. Sand Cone Test (ASTM D1556-90 &D4643)

A small hole (6" x 6" deep) is dug in the compacted material to be tested. The soil is removed and weighed, then dried and weighed again to determine its moisture content. A soil's moisture is figured as a percentage. The specific volume of the hole is determined by filling it with calibrated dry sand from a jar and cone device. The dry weight of the soil removed is divided by the volume of sand needed to fill the hole. This gives us the density of the compacted soil in lbs per cubic foot. This density is compared to the maximum

Proctor density obtained earlier, which gives us the relative density of the soil that was just compacted. [See Figure 2.2]

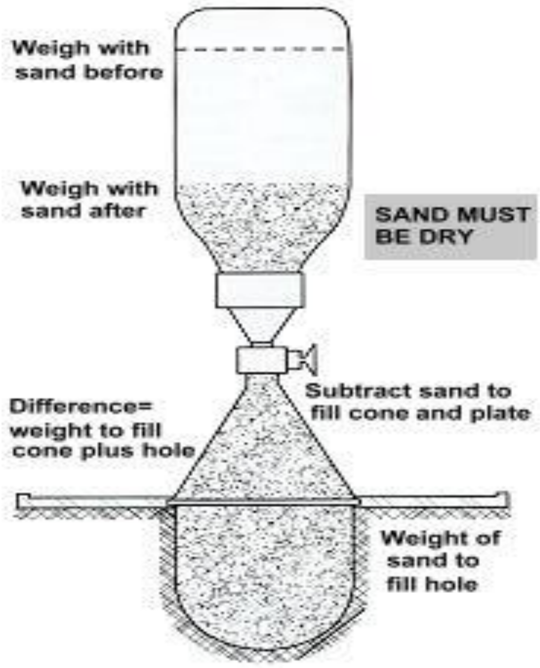


Figure 2.2: Sand Cone test

3. CORE CUTTER METHOD (ASTM D 2937)

Core cutters are used for testing the compaction of cohesive/clay soils placed as fill.

The cylindrical cores of standard volume, 13cms long and 10cms diameter., they have a sharpened edge at one end to improve penetration of the soil surface.

These cores are driven fully into the surface to be tested, they are removed from the ground without disturbing the core contents. In the laboratory they are cut flush top and bottom and weighed.

Bulk density can be quickly calculated, and by determining the moisture content of the soil the dry density of the fill can be calculated and hence the voids percentage. [See Figure 2.3]

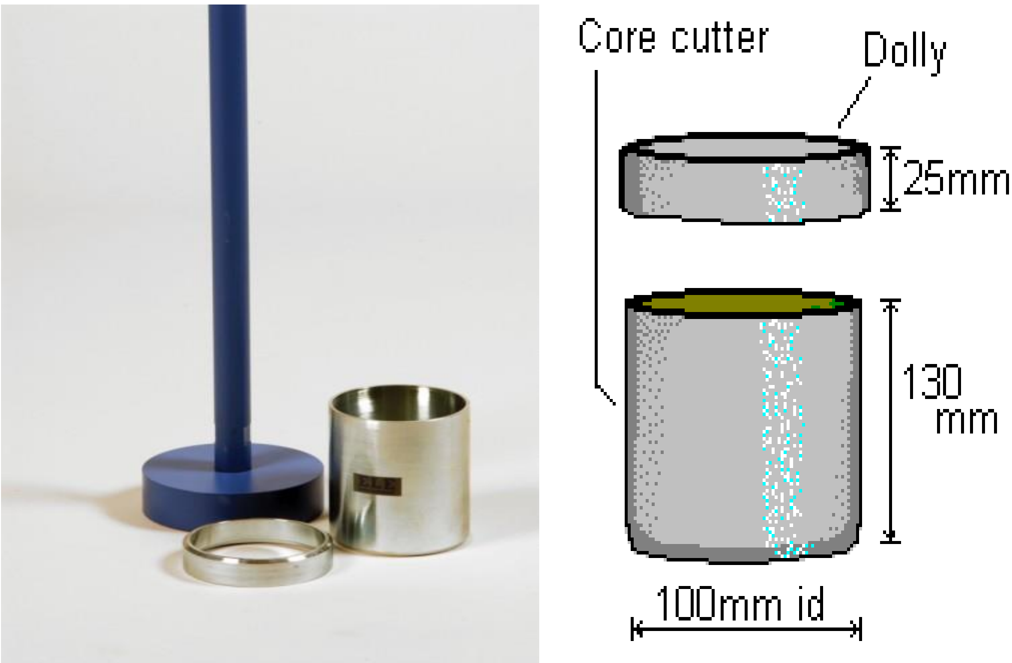


Figure 2.3: Core cutter test

4. Nuclear Density (ASTM D2292-91)

Nuclear Density meters are a quick and fairly accurate way of determining density and moisture content.

The meter uses a radioactive isotope source (Cesium 137) at the soil surface (backscatter) or from a probe placed into the soil (direct transmission).

The isotope source gives off photons (usually Gamma rays) which radiate back to the mater's detectors on the bottom of the unit.

Dense soil absorbs more radiation than loose soil and the readings reflect overall density.

Water content (ASTM D3017) can also be read, all within a few minutes. A relative Proctor density with the compaction results from the test. [See Figure 2.4]

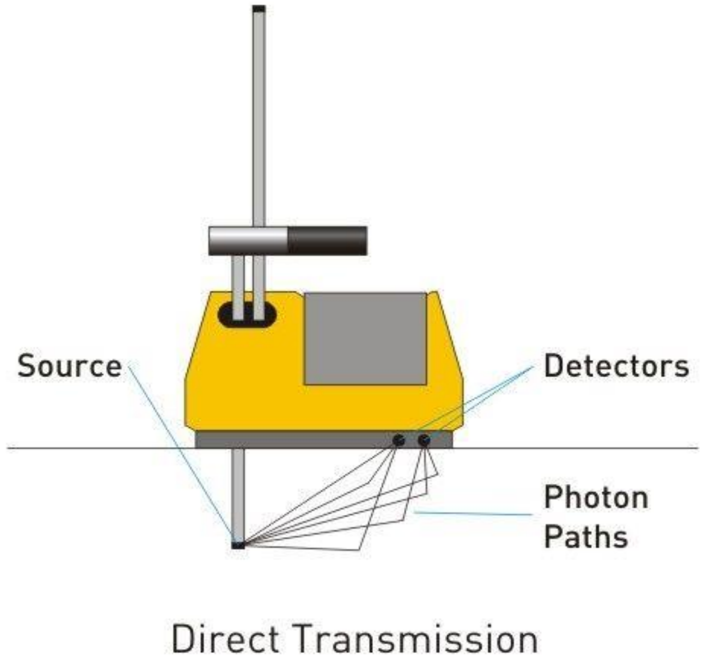


Figure 2.4 :Nuclear Gauge test

2.5.8 Types of Compacting Equipment

A large variety of mechanical equipments is available for compaction of soil but soil type and moisture condition will often dictate the type of equipments and method of use. (manak nagar, lucknow-11)

Some important compacting equipment are given below: -

1. Light compacting equipments (Rammers/Plate compactors)

2. Smooth wheel rollers

3. Sheepsfoot rollers

4. Pneumatic tyred rollers

5. Vibratory rollers

6. Grid rollers

2.6 Laboratory Compaction Test

To attain the required MDD in the field, first appropriate tests are determined in the laboratory and this laboratory results must be confirmed in the field. The following tests are normally carried out in a laboratory as shown Figure 2.5.

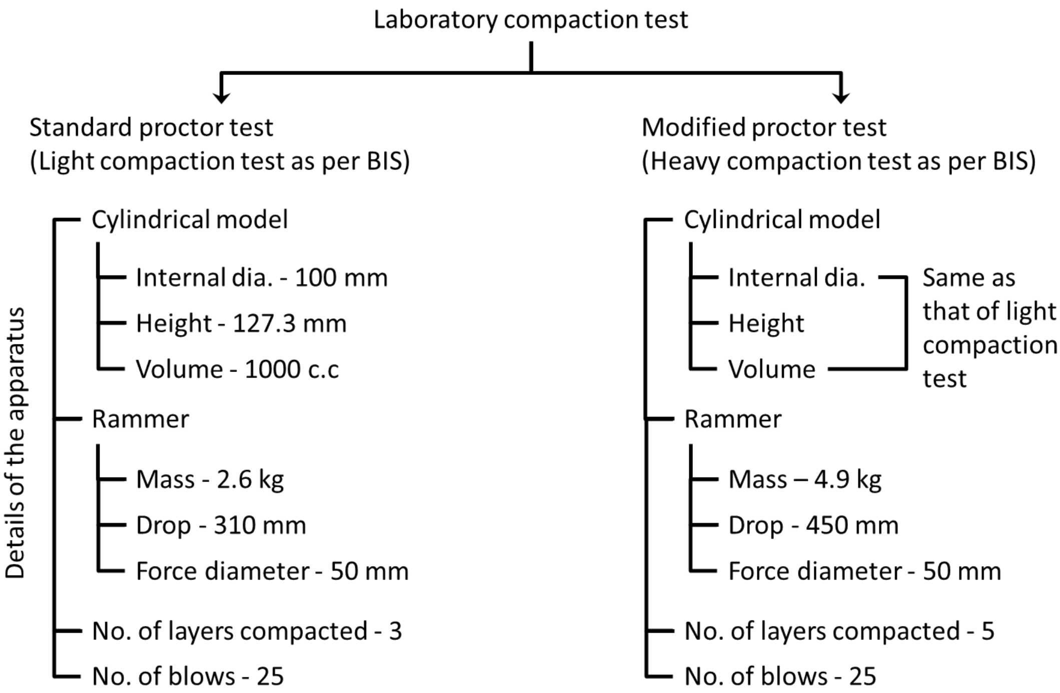


Figure 2.5: Schematic diagram showing different laboratory compaction test (Khan, 2014)

2.6.1 Standard Proctor Compaction Test

Proctor developed this test in connection with the construction of earth fill dams in California in 1933. It gives the standard specifications for conducting the test. A soil at a selected water content is placed in three layers into a mold of 101.6mm diameter, with each layer compacted by 25 blows of a 2.5 kg hammer dropped from a height of 305 mm, subjecting the soil to a total CE of about 600 kN/m2, so that the resulting γd at OWC is determined. The apparatus consists of a cylindrical metal mould of internal diameter 100 mm, 127.3 mm height and 1000 cm3 volume. The rammer used for this test is 2.6 kg mass, 310 mm free drop and a face diameter of 50 mm. The mould is fitted with detachable base plate and a 60 mm high collar (Murthy, 2007).

2.6.2 Modified Proctor Compaction Test

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

2.7 Laboratory Atterberg Limit Tests

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

2.7.1 Liquid Limit Tests (Cone Penetration Method)

The liquid limit is the empirically established moisture content at which a soil passes from the liquid state to the plastic state. The liquid limit provides a means of identifying and classifying fine grained cohesive soils especially when also the plastic limit is known. Variations in the moisture content in a soil may have significant effect on its shear strength, especially on fine-grained soils. The cone penetrometer method is the preferred method to the Casagrande test as it is essentially a static test depending on soil shear strength. This method covers the determination of the liquid limit of a sample in its natural state, or a sample from which material retained on a 425 mm test sieve has been removed. It is based on the measurement of penetration into the soil of a standardized cone (Zelalem, 2010).

2.7.2 Plastic Limit and Plasticity Index

The Plastic Limit is the empirically established moisture content at which a soil becomes too dry to be plastic. It is used together with the Liquid Limit to determine the Plasticity Index which when plotted against the Liquid Limit on the plasticity chart provides a means of classifying cohesive soils. The Plasticity Index is the difference between the Liquid Limit and the Plastic Limit. The Plasticity Index is the range of moisture content in which a soil is plastic; the finer the soil, the greater the Plasticity Index. This method covers the determination of the liquid limit of a sample in its natural state, or a sample from which material retained on a 425 μm test sieve has been removed (Zelalem, 2010).

2.8 Soil Classification

The purpose of soil classification is to provide the geotechnical engineer with a way to predict the behavior of the soil for engineering projects. There are many different soil classification systems in use, and only three of the most commonly used systems will be discussed in this section (Robert, 2004).

2.8.1 Unified Soil Classification System

As indicated in Table 2.1, this classification system separates soils into two main groups: coarse-grained soils (more than 50% by weight of soil particles retained on No. 200 sieve) and fine-grained soils (50% or more by weight of soil particles pass the No. 200 sieve). The coarse-grained soils are divided into gravels and sands. Both gravels and sands are further subdivided into four secondary groups as indicated in Table 2.1.

The four secondary classifications are based on whether the soil is well graded, poorly graded, contains silt-sized particles, or contains clay-sized particles. These data are obtained from a particle size distribution, also known as a ‘‘grain size curve,’’ which is obtained from laboratory testing (sieve and hydrometer tests). Figure 2.6 presents examples of grain size curves. The Atterberg limits are used to classify fine-grained soil, and they are defined As follows:

1. Liquid Limit (LL) is water content corresponding to the behavior change between the liquid and plastic state of a silt or clay. The liquid limit is determined in the laboratory by using a liquid limit device; the liquid limit is defined as the water content at which a part of soil, cut by a groove of standard dimensions, will flow together for a distance of 12.7 mm (0.5 in) under the impact of 25 blows in a standard liquid limit device.
2. Plastic Limit (PL) is defined the water content corresponding to the behavior change between the plastic and semisolid state of a silt or clay. The plastic limit is also determined in the laboratory and is defined as the water content at which a silt or clay will just begin to crumble when rolled into a tread approximately 3.2 mm (0.125 in) in diameter.
3. The plasticity index (PI) is defined as the liquid limit minus the plastic limit (PI = LL - PL). With both the liquid limit and plasticity index of the fine-grain soil known, the plasticity chart (Figure 2.7) is then used to classify the soil. There are three basic dividing lines on the plasticity chart, the LL = 50 line, the A- line, and the U -line. The LL =50 line separates soils into high and low plasticity, the A-line separates clays from silts and the U-line represents the upper-limit line.

As indicated in Table 2.1, symbols (known as ‘‘group symbols’’) are used to identify different soil types. The group symbols consist of two capital letters. The first letter indicates the following: G for gravel, S for sand, M for silt, C for clay, and O for organic (Robert, 2004).

The second letter indicates the following: W for well graded, which indicates that a coarse-grained soil has particles of all sizes; P for poorly graded, which indicates that a coarse-grained soil has particles of the same size, or the soil is skip-graded or gap-graded; M for a coarse-grained soil that has silt-sized particles; C for a coarse-grained soil that has clay-sized particles; L for a fine-grained soil of low plasticity; and H for a fine-grained soil of high plasticity. An exception is peat, where the group symbol is PT. Also note in Table 2.1 that certain soils require the use of dual symbols.

Table 2.1: Unified Soil Classification System. (Robert, 2004)

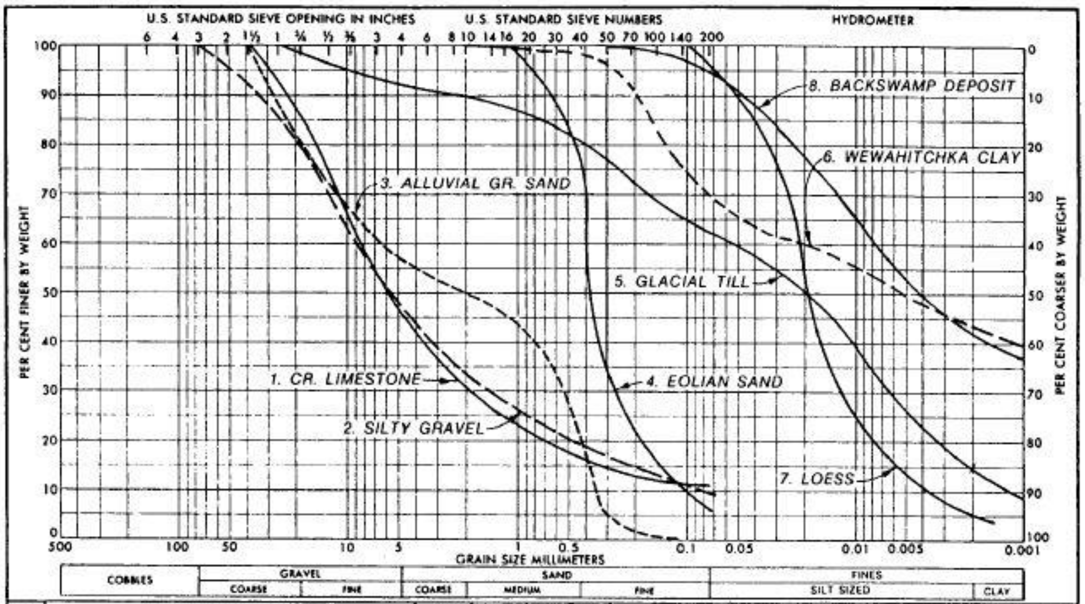
Major Divisions	Subdivisions	USCS Symbol	Typical Names	Laboratory Classification Criteria	
Coarse-grained soils (More than 50% retained on No. 200 sieve)	Gravels (More than 50% of coarse fraction retained on sieve No. 4)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$ [c]
		GP	Poorly graded gravels or gravelly sands, little or no fines	Less than 5% fines[a]	Does not meet C_u and/or C_c criteria listed above [c]
		GM	Silty gravels, gravel-sand-silt mixtures	More than 12% fines[a]	Minus No. 40 soil plots below the A-line
		GC	Clayey gravels, gravel-sand clay mixtures	More than 12% fines[a]	Minus No. 40 soil plot on or above the A-line
	Sands (50% or more of coarse fraction passes sieve No. 4)	SW	Well-graded sands or gravelly sands, little or no fines	Less than 5% fines[a]	$C_u \geq 6$ and $1 \leq C_c \leq 3$ [c]
		SP	Poorly graded sands or gravelly sands, little or no fines	Less than 5% fines[a]	Does not meet C_u and/or C_c criteria listed above [c]
		SM	Silty sands, sand-silt mixtures	More than 12% fines[a]	Minus No. 40 soil plots below the A-line
		SC	Clayey sands, sand-clay mixtures	More than 12% fines[a]	Minus No. 40 soil plot on or above the A-line

Fine-grained soils (50% or more Passes the No. 200 sieve)	Silts and clays (liquid limit less than 50)	ML	Inorganic silts, rock flour, silts of low plasticity	Inorganic soil	$PI > 4$ or plots below A-line
		CL	Inorganic clays of low plasticity, gravelly clays, sandy clays, etc.	Inorganic soil	$PI \leq 7$ and plots on or above A-line[b]
		OL	Organic silts and organic clays of low plasticity	Organic soil	$\frac{LL_{(oven\ driend)}}{LL_{(not\ dried)}} > 0.75$
	Silts and clays (liquid limit 50 or more)	MH	Inorganic silts, micaceous silts, silts of high plasticity	Inorganic soil	Plots below A-line
		CH	Inorganic highly plastic clays, fat clays, silty clays, etc.	Inorganic soil	Plots on or above A-line
		OH	Organic silts and organic clays of high plasticity	Organic soil	$\frac{LL_{(oven\ driend)}}{LL_{(not\ dried)}} > 0.75$
Peat	Highly Organic	PT	Peat and other highly organic soils	Primarily organic matter, dark in color, and organic odor	

a. ‘‘Fines’’ are those soil particles that pass the No. 200 sieve. For gravels with between 5% to 12% fines, use of dual symbols required (i.e., GW-GM, GW-GC, GP-GM, or GP-GC). For sands with between 5% to 12% fines, use of dual symbols required (i.e., SW-SM, SW-SC, SP-SM, or SP-SC).

b. If $4 \leq PI \leq 7$ and plots above A-line, then dual symbol (i.e., CL-ML) is required.

c. $C_u = D_{60}/D_{10}$ and $C_c = (D_{30})^2 / (D_{10}.D_{60})$ where D_{60} = soil particle diameter corresponding to 60% finer by weight (from grain size curve).



NO.	SOIL DESCRIPTION	W_l	W_p	PI	NO.	SOIL DESCRIPTION	W_l	W_p	PI
1	CRUSHED LIMESTONE, TENNESSEE	18	11	7	5	GLACIAL TILL, ILLINOIS	25	15	10
2	SILTY GRAVEL FROM WEATHERED GABBRO, OMAN	23	20	3	6	WEWAHITCHKA SANDY CLAY, FLORIDA	65	25	41
3	ALLUVIAL GRAVELLY SAND, MISSISSIPPI	-	-	-	7	LOESS, MISSISSIPPI	29	24	5
4	EOLIAN SAND, OMAN	-	-	-	8	BACKSWAMP DEPOSIT, MISSISSIPPI RIVER	59	18	41

Figure 2.6: Examples of grain size curves and Atterberg limit test data for different soils

Note that w_l = liquid limit and w_p = plastic limit.

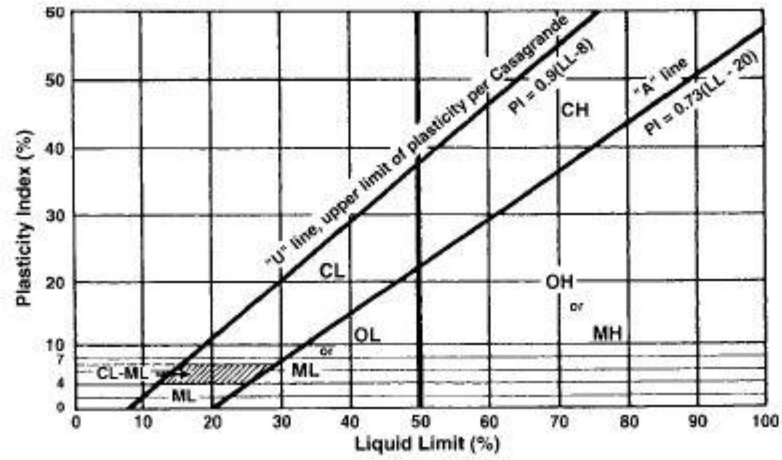


Figure 2.7: Plasticity chart. (Robert, 2004)

2.8.2 AASHTO Soil Classification System

This classification system was developed by the American Association of State Highway and Transportation Officials as shown in Table 2.2.

AASHTO Soil Classification system Notes are summarized as follows:

1. Classification Procedure: First decide which of the three main categories (granular materials, silt-clay materials, or highly organic) the soil belongs.

Then proceed from the top to the bottom of the chart and the first group that meets the particle size and Atterberg limits criteria is the correct classification.

2. Group Index = (F - 35) [0.2 + 0.005(LL - 40)] + 0.01(F - 15) (PI -10), where F =percent passing No. 200 sieve, LL = liquid limit, and PI =plasticity index. Report group index to nearest whole number, for negative group index, report as zero when working with A-2-6 and A-2-7 subgroups, and use only the PI portion of the group index equation.
3. Atterberg limits are performed on soil passing the No. 40 sieve. LL = liquid limit, PL = plastic limit, and PI = plasticity index.
4. AASHTO definitions of particle sizes are as follows: (a) boulders: above 75 mm, (b) gravel: 75 mm to No. 10 sieve, (c) coarse sand: No. 10 to No. 40 sieve, (d) fine sand: No 40 to No, 200 sieves, and (e) silt-clay size particles: material passing No. 200 sieve.
5. An example of an AASHTO classification for a clay is A-7-6 (30), or Group A-7, subgroup 6, group index 30.eighth group (A-8) reserved for highly organic soils. Soil types A-1, A-2, and A-7 have subgroups as indicated in Table 2.2. Those soils having plastic fines can be further categorized by using the group index (defined in Table 2.2). Groups A-1-a, A-1-b, A-3, A-2-4, and A-2-5 should be considered to have a group index equal to zero. According to AASHTO, the road supporting characteristics of a subgrade may be assumed as an inverse ratio to its group index. Thus, a road subgrade having a group index of 0 indicates a ‘‘good’’ subgrade material that will often provide good drainage and adequate bearing when thoroughly compacted. A road subgrade material that has a group index of 20 or greater indicates a ‘‘very poor’’ subgrade material that will often be impervious and have a low bearing capacity (Robert, 2004).

Table 2.2: AASHTO Soil Classification System. (Robert, 2004)

Major Divisions	Group	AASHTO Symbol	Typical Names	Sieve Analysis (Percent Passing)	Atterberg Limits
Granular materials (35% or less passing No. 200 sieve)	Group A-1	A-1-a	Stone or gravel fragments	Percent Passing: No. 10 ≤ 50% No. 40 ≤ 30% No. 200 ≤ 15%	PI≤ 6
		A-1-b	Gravel and sand mixtures	No. 40 ≤ 50% No. 200 ≤ 25%	PI≤ 6
	Group A-3	A-3	Fine sand that is non-plastic	No. 40 > 50% No. 200 ≤ 10%	PI = 0 (non-plastic)
	Group A-2	A-2-4	Silty gravel and sand	Percent Passing: No. 200 ≤ 35%	LL ≤ 40 PI ≤ 10
		A-2-5	Silty gravel and sand	Percent Passing: No. 200 ≤ 35%	LL > 40 PI ≤ 10
		A-2-6	Clayey gravel and sand	Percent Passing: No. 200 ≤ 35%	LL ≤ 40 PI > 10
		A-2-7	Clayey gravel and sand	Percent Passing: No. 200 ≤ 35%	LL > 40 PI > 10
Silt-clay materials (more than 35% passing No. 200 sieve)	Group A-4	A-4	Silty soils	Percent Passing: No. 200 > 35%	LL ≤ 40 PI ≤ 10
	Group A-5	A-5	Silty soils	Percent Passing: No. 200 > 35%	LL > 40 PI ≤ 10
	Group A-6	A-6	Clayey soils	Percent Passing: No. 200 > 35%	LL ≤ 40 PI > 10
	Group A-7	A-7-5	Clayey soils	Percent Passing: No. 200 > 35%	LL > 40 PI ≤ LL – 30 PI > 10
		A-7-6	Clayey soils	Percent Passing: No. 200 > 35%	LL > 40 PI > LL – 30 PI > 10
Highly organic	Group A-8	A-8	Peat and other highly organic soils	Primarily organic matter, dark in color, and organic odor	

2.9 The Previous Studies

Many soil mechanics experts have made serial attempts to predict compression tests, exception of a few elements, for example, soil grouping information, recording properties, grain size and conveyance.

These attempts produced relation between compaction characteristics and soil properties, also provided equation and graphs, in this research we will take some of them.

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

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

(Torrey, 1970) in his research, made an interesting discussion on correlating compaction parameters with Atterberg limits. He remarked in this research that in order to determine mathematical relationship between independent variables, i.e. LL, PL, and dependent variables (OWC and MDD) using the method of statistics, it is necessary to assume a frequency distribution between the variables. An assumption was made that there is normal or Gaussian distribution between the variables. A normal distribution has a very specific mathematical definition and although, the assumption of normal distribution is reasonable, it must be pointed out there is no assurance this is valid. Additionally, it was assumed that the relationship between the variables of interest is linear.

Figure 2.8 and Figure 2.9 show the results of analysis carried out by Torrey (1970). It shows the linear relation between w_{opt} and w_l and also aims the relation between $\gamma_{d\,max}$ and w_l. These models can estimate 77.6 and 76.3 percent of the variables. Also,

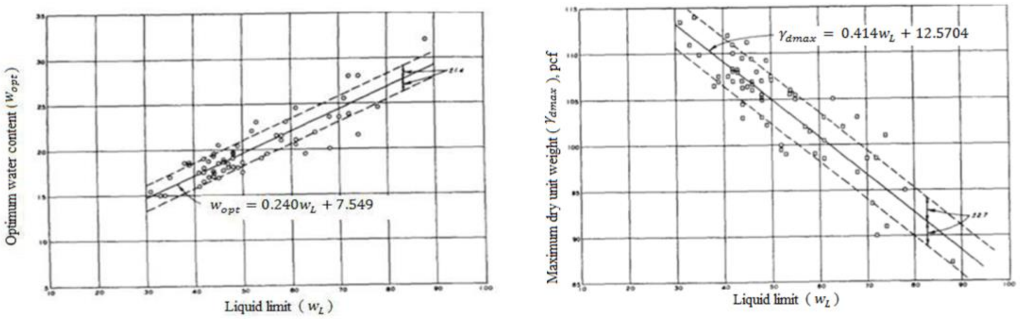
Figure 2.8 shows the linear relation between the compaction test parameters with I_p. He proposed the following equations:

$$\boldsymbol{w_{opt} = 0.24\, w_l + 7.549} \qquad (2.1)$$

$$\boldsymbol{\gamma_{d\,max} = 0.41\, w_l + 12.5704} \qquad (2.2)$$

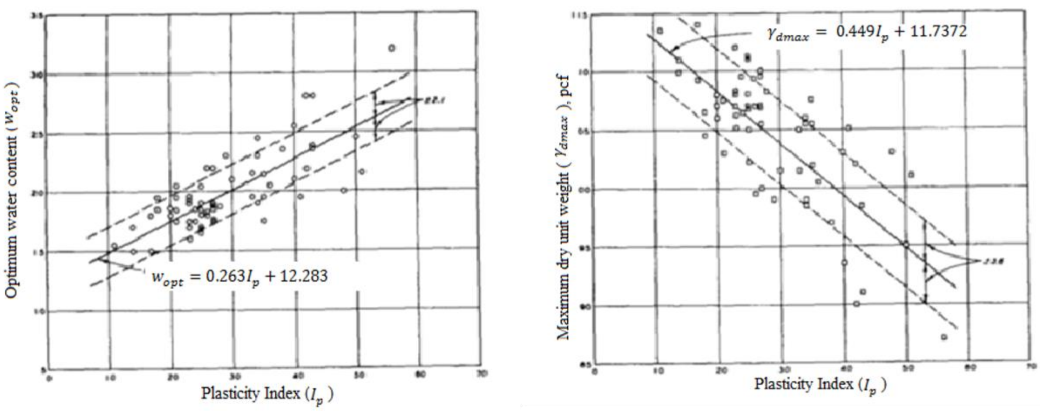
$$w_{opt} = 0.449\, I_p + 12.283 \qquad (2.3)$$

$$\gamma_{d\,max} = 0.449\, I_p + 11.7372 \qquad (2.4)$$



(a) Plots of w_{opt} versus w_L (b) Plots of $\gamma_{d\,max}$ versus w_L

Figure 2.8: Plots of compaction characteristics versus w_L.



(a) Plots of w_{opt} versus I_p (b) Plots of $\gamma_{d\,max}$ versus I_p

Figure 2.9: Plots of compaction characteristics versus I_p.

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

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

The accompanying equations were from Iraqi soils:

$$\gamma_{d\,max} = 2.44 - 0.02\, w_p + 0.008\, w_l \qquad (2.5)$$

$$w_{opt} = 0.24\, w_l + 0.63\, w_p - 3.13 \qquad (2.6)$$

Likewise, for USA soils, the equations underneath were proposed:

$$\gamma_{d\,max} = 2.27 - 0.019\, w_p + 0.003\, w_l \qquad (2.7)$$

$$w_{opt} = 0.14\, w_l + 0.54\, w_p \qquad (2.8)$$

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

$$\gamma_{d\,max} = \left[4804574\, Gs - 195.55\, {w_l}^2 + 15697\, (R \neq 4)^{0.5} - 9527830\right]^{0.5} \qquad (2.9)$$

$$w_{opt} = 1.195 \times 10^{-4}\, {w_l}^2 - 1.964\, Gs - 6.61 \times 10^{-3}(R \neq 4) + 7.651 \qquad (2.10)$$

where; $\gamma_{d\,max}$ *in kg/m*^{3}

(Gurtug and Sridharan, 2004) also studied the compaction behavior and prediction characteristics of three cohesive soils taken from the Northern Cyprus and other two clayey minerals based on four compaction energy namely, standard Proctor compaction, modified Proctor compaction, Reduced standard Proctor and Reduced modified Proctor to develop relationship between $\gamma_{d\,max}$ and w_{opt} and I_p with particular reference to the *CE*. They proposed the equations below:

$$w_{opt} = (1.95 - 0.38\, log\, CE) w_p \qquad (2.11)$$

$$\gamma_{d\,max} = 22.68\, e^{-0.0183\, w_p} \qquad (2.12)$$

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

$$w_{opt} = 0.92\, w_p \qquad (2.13)$$

$$\gamma_{d\,max} = 0.23(93.3 - w_p) \qquad (2.14)$$

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

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

$$\boldsymbol{w_{opt} = K \times w_p} \qquad (2.15)$$

$$\boldsymbol{\gamma_{d\,max} = L - M \times w_{opt}} \qquad (2.16)$$

where;

$K = 1.99 - 0.165 \ln CE$

$L = 14.34 - 0.195 \ln CE$

$M = -0.19 + 0.073 \ln CE$

CE in kJ/m^3, $\gamma_{d\,max}$ *in kN/m*^{3}

Therefore, at any compactive effort, w_{opt} can be anticipated from w_p and the anticipated w_{opt} can be utilized to gauge $\gamma_{d\,max}$.

(Matteo et al., 2009) analyzed the after effects of 71 fine-grained soils and gave the following correlation equations 2.17 and 2.18 for OMC and $\gamma_{d\,max}$ for modified proctor tests (*CE= 2700 kN.m/m*^{3})

$$\boldsymbol{w_{opt} = -0.86\, w_l + 3.04\, {}^{w_p}/_{Gs} + 2.2} \qquad (2.17)$$

$$\boldsymbol{\gamma_{d\,max} = 40.316\, (w_{opt})^{-0.295} (I_p)^{0.032} - 2.4} \qquad (2.18)$$

where; $\gamma_{d\,max}$ *in kN/m*^{3}

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

(Ugbe, 2012) studied the lateritic soils in Western Niger Delta, Nigeria and he developed the equations 2.19 and 2.20 underneath utilizing 152 soil samples.

$$\boldsymbol{\gamma_{d\,max} = 15.665\, Gs + 1.52\, w_l - 4.313\, FC + 2011.960} \qquad (2.19)$$

$$\boldsymbol{w_{opt} = 0.129\, FC + 0.019\, w_l - 1.4233\, Gs + 11.399} \qquad (2.20)$$

where; fine content (FC) and liquid limit (w_l*) in %.*

(Mujtaba et al., 2013) did laboratory Proctor compaction tests on 110 sandy soil tests (SM, SP, SP-SM, SW, SW-SM). In view of the tests outcomes, the following correlation equations were proposed for w_{opt} and $\gamma_{d\,max}$:

$$\log w_{opt} = 1.67 - 0.193 \log C_u - 0.153 \log CE \qquad (2.21)$$

$$\gamma_{d\,max} = 4.49 \log C_u + 1.51 \log CE + 10.2 \qquad (2.22)$$

where; CE in kN.m/m³, $\gamma_{d\ max}$ in kN/m³, w_{opt} in %.

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

(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 2.23 utilizing w_p in order to determine the compaction characteristics namely, w_{opt} of these soils.

$$w_{opt} = 12.001\, e^{0.0181\, w_p}, \qquad R^2 = 0.84 \qquad (2.23)$$

Most recently (Hussain, 2016) studied the prediction of compaction characteristics of over-consolidated soils, M.Sc. of near East University, the following correlation equations were proposed for OWC and MDD.

$$w_{opt} = 9.71 + 0.270\, I_p \qquad R^2 = 0.88 \qquad (2.24)$$

$$w_{opt} = 6.86 + 0.206\, w_l \qquad R^2 = 0.926 \qquad (2.25)$$

$$w_{opt} = 4.00 + 0.609\, w_p \qquad R^2 = 0.752 \qquad (2.26)$$

$$\gamma_{d\,max} = 22.9 - 0.128\, w_l - 0.028\, I_p \qquad R^2 = 0.707 \qquad (2.27)$$

$$\gamma_{d\,max} = 22.5 - 0.0926\, w_l \qquad R^2 = 0.702 \qquad (2.28)$$

$$\gamma_{d\,max} = 25.7 - 0.453\, w_{opt} \qquad R^2 = 0.774 \qquad (2.29)$$

CHAPTER THREE

EXPERIMENTAL METHODOLOGY

CHAPTER THREE

EXPERIMENTAL METHODOLOGY

3.1 Introduction

This chapter includes the area of study, soil sampling, sample collection, testing Methods and preliminary investigation results. This is followed by results obtained from the tests.

3.2 Brief Introduction for the Project

Dam Complex of Upper Atbara Project is located at the boundary between Kassala State and Gedaref State, 20 km away from the U/S of the confluence between the Atbara River and Setit River. This Project, 460 km away from the capital city Khartoum and 659km away from Port Sudan, is situated 30 km away from Showak which is located at the U/S of Gedaref.

For flood control and protection of the embankment dams from overtopping, large reinforced concrete spillway structures for each river are constructed with a maximum discharge of 5300 m3/s and 9800 m3/s respectively. Through this impoundment a maximum gross head of 38.85 m is created. The stored water will provide drinking water to about 3 million inhabitants of the region and water to the by 5000 km2 extended New Halfa irrigation scheme via the existing Kashm El Girba Reservoir. The head will be utilized for hydropower generation by 4 Units, each 80 MW installed capacity as shown in **Figure 3.1** The construction costs for the civil works amount to about 1100 million US$ (MS, MIP-COW).

The Rumela Dam on Atbara will have a height of 55 meters and the Burdana Dam on Setit will have a height of 50 meters. The two dams will be connected and have a total length of 13 kilometres. The twin dam complex will thus have a joined reservoir with a storage capacity of about 2.7 billion cubic meter of water. The maximum filling level will be 523,3 meters above sea level (MS ,MIP-COW).

Based on hydraulic design calculations and considerations, the spillway structure of the Rumela Dam consists of 4 bottom outlets and 1 surface sluice as shown in **Figure 3.3.**

Based on hydraulic design calculations and considerations, the spillway structure of the Burdana Dam consists of 6 bottom outlets and 2 surface sluices as shown in **Figure3.4.**

The most critical condition might be the condition with completed spillway if the diversion of a serious flood event only through the 6 bottom outlets of the spillway structure takes place (MS, MIP-COType equation here.W).

Figure 3.1: Unit two

Figure 3.2: The Pwer House

Figure 3.3 Rumela Spillway

Figure 3.4 Burdna spillway

3.3 Visual Identification of Soils in the Field

Field identification of soils was carried out according to ASTM D-2488 “Standard Practice for Description and Identification of Soils”. The field description and classification of soil were based on the size and distribution of coarse-grained particles and on the behavior of fine-grained particles. The first step used in describing soil under the visual-manual method was to determine whether the soil is fine-grained or coarse-grained by visually observing the soil sample to be taken.

3.4 Sample Collection

The samples used were obtained primary data for this work are taken from Borrow Area BU3-QF, which is located at the right bank upstream of Setit River, with Elevation of 520.00 m above Alexandria Mean Sea Level. This borrow area served as the major source of core materials for Burdana dyke and dam construction. The quantity of (BU3-QF) about 9,745,50.00 m^3 (CMS-No. 49, Rev. D).

3.5 Work Content and Testing Methods

According to the obtained data from Dam complex of upper Atbara project, preliminary investigation and test results, clay material balance, borrow area planning and test fill have been conducted. Each sample were collected and brought to the geotechnical engineering laboratory of Dam Complex of Upper Atbara Project Samples were collected from each pit at a depth ranging from 1.00 m to 7.00 m and transported to the laboratory.

Once in the laboratory the soil was allowed to be air dried and each soil sample was mechanically pulverized over 4.75 mm sieve before testing (CMS-No. 49, Rev. D).

3.6 The Soil Tests

The details of tests carried out are listed as follows:

3.6.1 Grain Size Analysis

The amount of soil materials finer than 0.075mm was determined using T.S, section 02222, 1.3.3, Method for Amount of Material in the Soil Finer than the No. 200 Sieve.

3.6.2 Atterberg Limits

The Liquid Limit, Plastic Limit, and Plasticity Index were determined according to technical specification, DCUAP Contract Test Method (T.S, section 02222, 1.3.3) and BS 1377 part 2 (Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils). The standard three -point method for determining the liquid limit was used for all tests.

3.6.3 Specific Gravity

The specific gravity of each type of soil was determined according to BS1377 part 2 (Standard Test Method for Specific Gravity of Soils). The precision and bias of each pair of tests were investigated and all are within the BS accepted range.

3.6.4 Moisture-Density Relationship

Each sample extracted from the different sites was sieved over a 4.75mm sieve for testing and compacted in a 105mm- diameter mold as described in Procedure of the BS 1377 part 4. Each sample was immediately tested for moisture content according to ASTM D-2216, the moisture content obtained from this procedure was used for generation of a compaction curve according to BS1377 part 4. Finally the Maximum Dry Density (**γ**dmax) and the corresponding Optimum Moisture Content (OMC) were computed using spread sheet and chart plots.

The density of the clay material (Zone 1) of the earth core rock fill dam after placing and compaction shall be not less than 99% of standard proctor density according to ASTM D 698 as an average and not less than 97% as a minimum of individual test results.

The moisture content of the material after compaction in the dam embankment shall be within 2% above and 1% below the Optimum Moisture Content as determined by ASTM D 698 in order to permit the specified density to be achieved using the approved compaction equipment, except where otherwise specified hereunder, material with Moisture Content outside these limits shall not be incorporated into the dam embankment. (T.S DCUAP, section 02222, 1.3.3).

3.6.5 Organic Matter

The clay material (Zone 1) of the earth core rock fill dam shall consist of a material with an organic matter content of less than 3%, according to the ASTM D-2974.

3.6.6 Permeability

The clay material (Zone 1) of the earth core rock fill dam shall consist of a material with a permeability coefficient of less than 10^{-7} m/s after compaction. The permeability coefficient shall be determined on samples from the compacted embankment and extracted according to the drive-cylinder-method. . (T.S DCUAP, section 02222, 1.3.3).

3.7 Preliminary Investigation Result (BU3-QF)

Considering that sampling works were conducted under the supervision of the Engineer on Site, the test pits are analyzed with every 1.5m as a range. The amount of samples was 35 groups. Samples within gradation envelope are 9 groups and samples beyond gradation envelope are 26 groups from BU3-QF Borrow Area (MS).

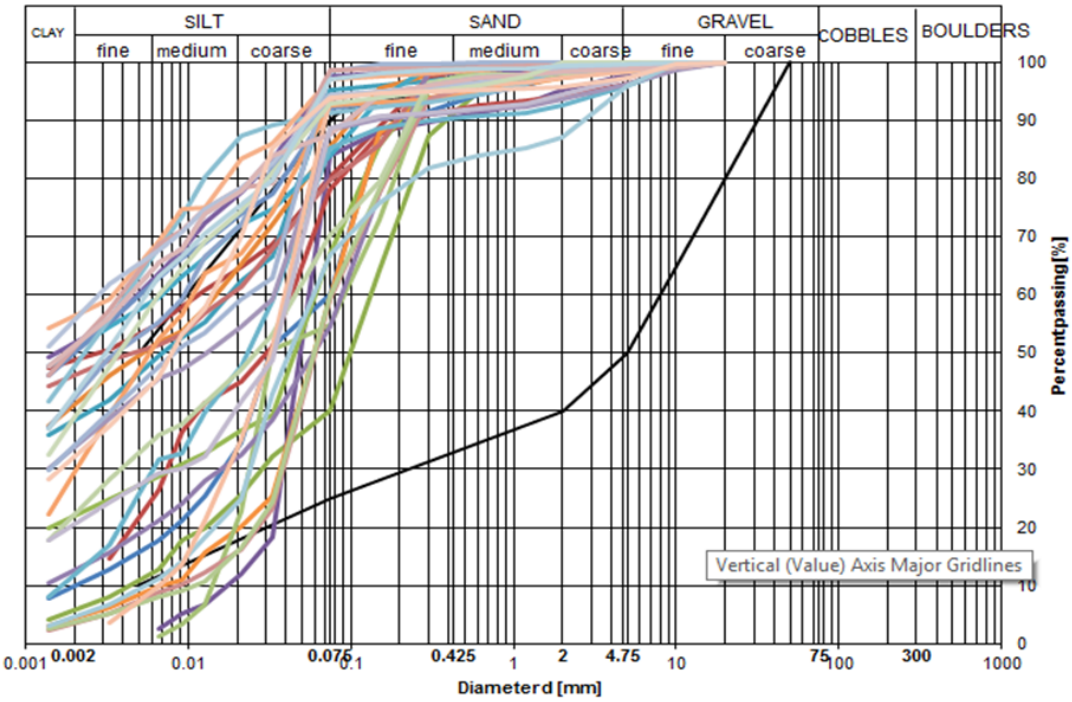


Figure 3.5: Sieve Analysis Test

Sieve analysis test was carried out for samples (BU3-QF) as shown in Figure 3.5. The organic content of all samples exceeds 1% has maximum, minimum and average values of 6.4, 1.3 and 3 respectively as presented in Figure 3.6. The maximum, minimum and average values of specific gravity were 2.84, 2.51 and 2.65 respectively as illustrated in Figure 3.7.

Atterberg limits tests were carried out for 35 groups for BU3-QF Borrow Area. 27 groups of samples (i.e. 77% of the samples) with LL between 30% and 70%, and 15 groups of samples (i.e. 43% of all samples) with PI between 12%, and 40% as illustrated in Figure 3.8.

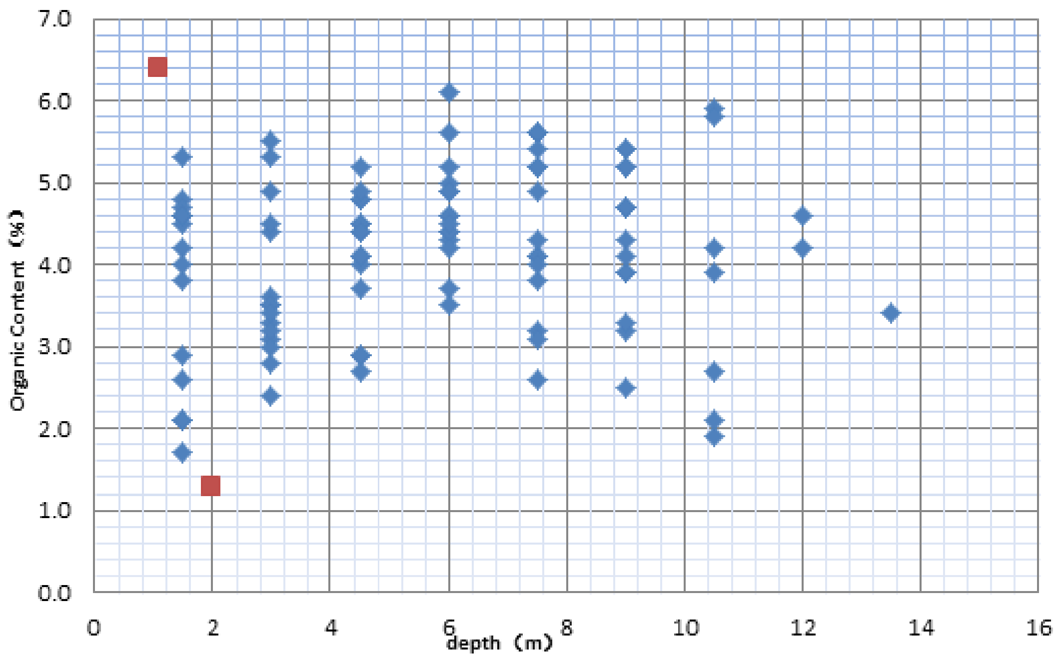


Figure 3.6: Organic matter chart

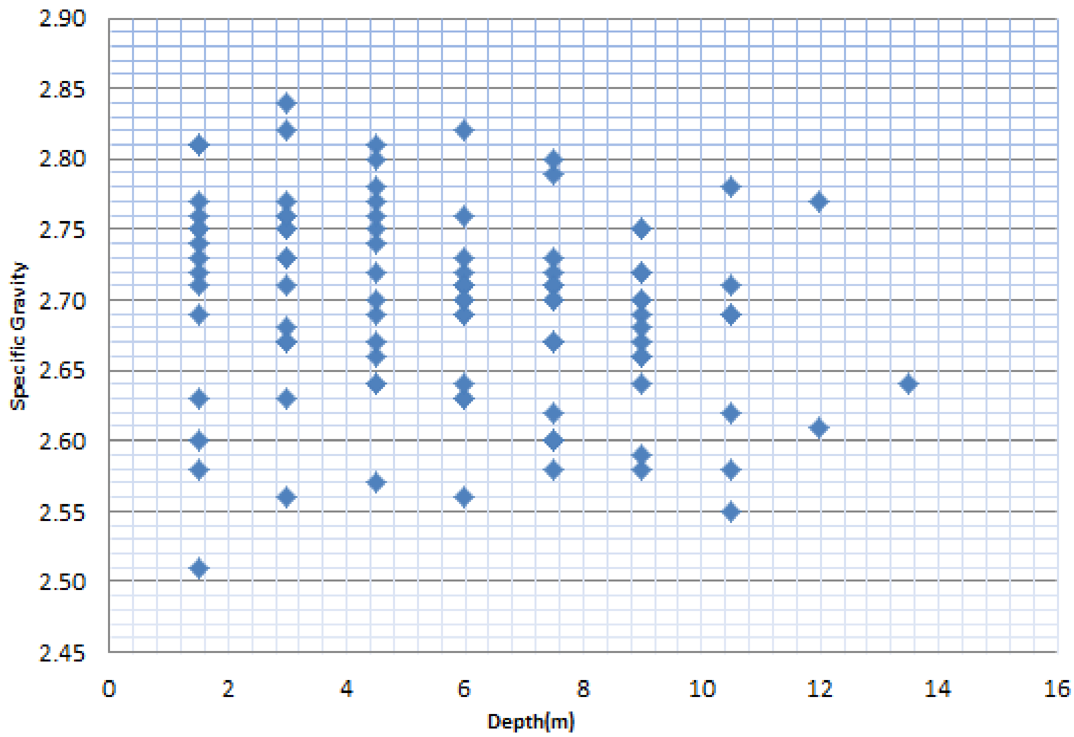


Figure 3.7: Specific gravity chart

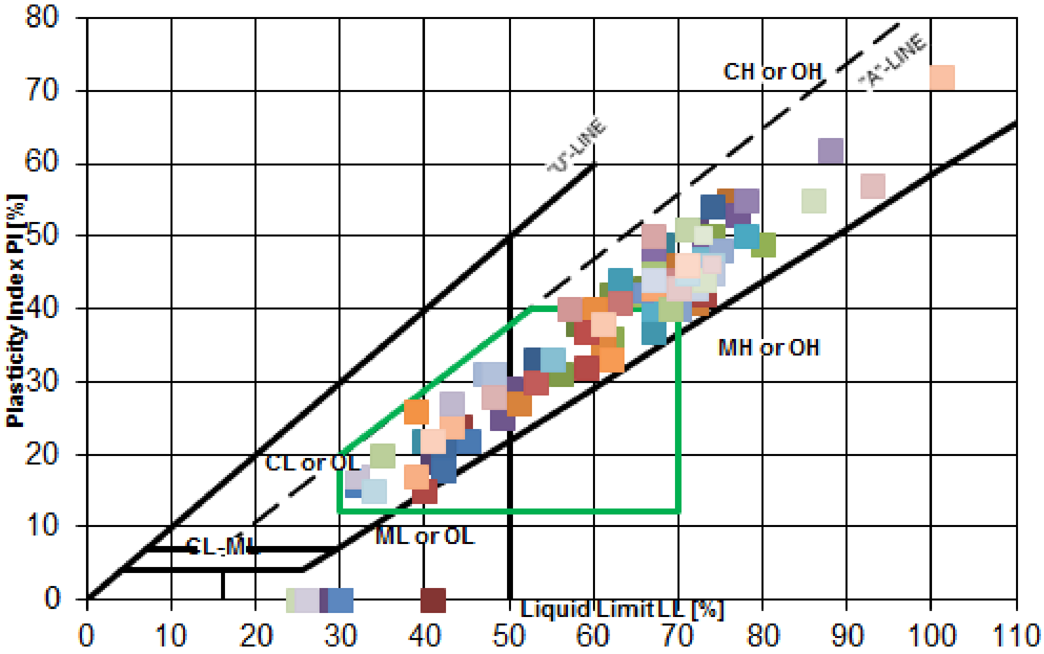


Figure 3.8: Plot of primary data on the plasticity chart

Note: CL = inorganic Clay of low plasticity, ML = inorganic Silt of low plasticity, OL = Organic clay or silt of low plasticity, CH = inorganic clay of high plasticity, MH = inorganic silt of high plasticity, OH = organic clay or silt of high plasticity.

3.8 Soil Test Results of Borrow Area BU3-QF

All laboratory tests were performed at dam complex of upper Atbara project in two groups. The first group results (20 samples) from Borrow Area, BU3-QF for all pits was presented in Table 3.1. The second group result (20 samples) for four pits in BU3-QF Borrow Area was presented **Table 3.2.**

Table 3.1: Laboratory test results first group

Test No.	Borehole No.	Depth (m)	Gs g/cm^3	Atterberg limits test			Compaction Test	
				LL%	PL%	PI%	OMC %	MDD g/cm^3
1	1-3-1	1.5 -5.5	2.76	37	20	17	22.5	1.59
2	1-3-2	1.5-5.5	2.78	35	21	14	22.6	1.59
3	2-1-1	0-0.8	2.77	50	20	30	24.5	1.54
4	2-1-2	0-0.8	2.79	55	22	33	25.0	1.53
5	4-5-6	1.5-6.5	2.80	46	22	24	21	1.62
6	7-1-1	0-1.4	2.76	68	26	42	29.0	1.44
7	8-1-1	0-1.2	2.81	59	19	40	23.5	1.52
8	10-1-1	0-4	2.72	43	19	24	22.9	1.58
9	10-1-2	0-4	2.74	44	21	23	23.1	1.56
10	11-1-1	0-1	2.79	32	16	16	23.5	1.60
11	11-1-2	1-2	2.77	30	16	14	24.0	1.60
12	13-2-1	0-1	2.77	35	18	17	24	1.61
13	14-6-1	0-1	2.79	37	14	23	23.5	1.58
14	14-6-2	1-2	2.77	39	15	24	23.7	1.59
15	15-6-1	3-4	2.78	63	21	42	26.0	1.51
16	16-2-1	0-1.2	2.75	66	26	40	30.0	1.40
17	17-1-1	0-2	2.75	68	25	43	26.3	1.46
18	17-1-2	2-4	2.76	69	25	44	25.8	1.52
19	18-1-1	0-1.7	2.75	67	26	41	28.0	1.48
20	18-1-2	0-1.7	2.75	62	23	39	27.0	1.48

Note: LL = Liquid limit, PL = Plastic Limit, PI = plasticity Index, MDD = Maximum Dry Density, OMC = Optimum Moisture Content, Gs = Specific Gravity.

Table 3.2: Laboratory test results second group

Test No.	Borehole No.	Depth (m)	Gs g/cm^3	Atterberg limits test			Compaction Test	
				LL%	PL%	PI%	OMC%	MDD g/cm^3
1	10-1-1	0-4	2.72	43	19	24	22.9	1.58
2	10-1-2	0-4	2.74	44	21	23	23.1	1.56
3	10-1-3	0-4	2.75	42	20	22	24.8	1.59
4	10-1-4	0-4	2.72	40	19	21	24.6	1.59
5	10-1-5	0-4	2.74	39	18	21	24.5	1.61
6	10-1-6	0-4	2.75	37	18	19	23.9	1.58
7	10-1-7	0-4	2.79	36	18	18	22.4	1.61
8	10-1-8	0-4	2.78	39	18	21	22.7	1.60
9	1-3-1	1.5-5.5	2.76	37	20	17	22.5	1.59
10	1-3-2	1.5-5.5	2.78	35	21	14	22.6	1.59
11	2-1-1	0-0.8	2.77	50	20	30	24.5	1.54
12	2-1-2	0-0.8	2.79	55	22	33	25.0	1.53
13	2-1-3	0-0.8	2.77	58	22	36	25.7	1.53
14	2-7-1	0-1	2.79	53	21	32	25.5	1.54
15	2-7-2	0-1	2.78	54	19	35	24.0	1.54
16	2-7-3	0-1	2.78	57	22	35	25.5	1.49
17	18-1-1	0-1.7	2.75	67	26	41	28.0	1.48
18	18-1-2	0-1.7	2.75	62	23	39	27.0	1.48
19	18-2-1	0-1.4	2.73	70	24	46	26.0	1.50
20	18-2-2	0-1.4	2.73	63	21	42	27.5	1.48

Note: LL = Liquid limit, PL = Plastic Limit, PI = plasticity Index, MDD = Maximum Dry Density, OMC = Optimum Moisture Content, Gs = Specific Gravity.

3.9 Data Analysis Methods

The relationship of two or more variables can be expressed in mathematical form by determining an equation connecting the two variables. In this work primary data (40samples) from two Groups, Results from Borrow Area BU3-QF were collected as tabulated in the previous section. In this Chapter the analysis have been done to develop possible relationships among the parameters.

There are many methods that can be used to check the validity of the relationships between two or more variables. However, in this study the Microsoft Excel are used to determine the scatter plot, correlation and regression. Before the application of the analysis methods some important terms are discussed below.

3.9.1 Level of Significance

The probability of making an error to reject a hypothesis while it happens to be true is called the level of significance (Zelalem,2010) In practice it is customary to use 5% level of significance. This means 95% is confident that could be made the right decision and wrong probability is 5%.

3.9.2 One Tailed and Two Tailed Tests

When a hypothesis is tested assuming that one process is better or worse than the other, then it is called one tailed or one sided test. However, if the hypothesis is tested assuming that the extreme values of the statistics score on both sides of the mean in both tails of the distribution, the tests are called two tailed or two sided tests (Nerea, 2012).

3.9.3 Standard Error

Standard error is the average measure of error of each sample points about the best-fit line. Out of all curves, the best-fit curve has the smallest standard error (Nerea, 2012).

3.9.4 Correlation Coefficient (R)

Correlation coefficient (sometimes called the regression coefficient) is the act of the linear correlation between two variable x and y, between +1 and -1 for sale inclusive. R = 1 indicates a perfect linear correlation and linear regression perfect, R = 0 is no correlation, and R = -1 total negative correlation. **Table 3.3** states the accuracy of the correlation coefficient is measured by the determination, R^2 (Husain, 2016).

Table 3.3: 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

CHAPTER FOUR

ANALYSIS AND DISCUSSION OF RESULTS

CHAPTER FOUR

ANALYSIS AND DISCUSSION OF RESULTS

4.1 Introduction

This chapter describes the analysis and discussion of results obtained from Atterberg limits and compaction parameters of soils.

4.2 Scatter Plot and Best-Fit Curve

In conducting the statistical analysis, Microsoft Office Excel software was used to determine the scatter plot, correlation and regression. Excel spreadsheet found to be the most powerful and handy tool for analyzing scatter plot and determining the correlation between two or more diverse.

The relationship between the dependent and independent variables are examined separately for the first group data as presented in Figures below.

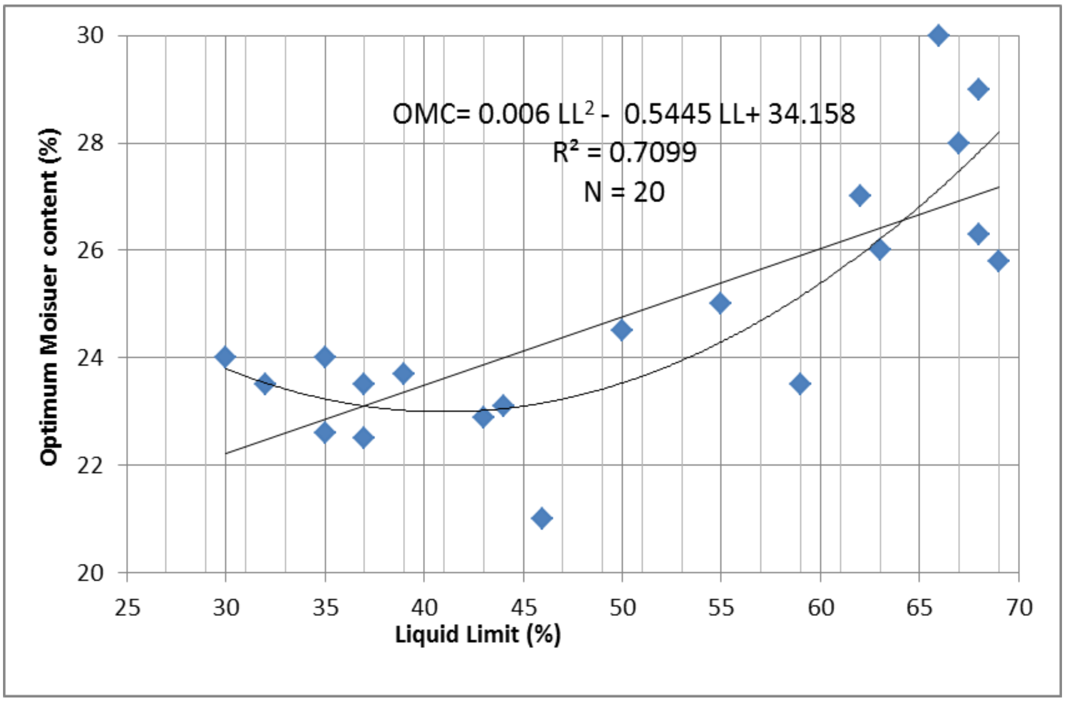


Figure 4.1: Scatter plot and best-fit line for liquid limit and OMC

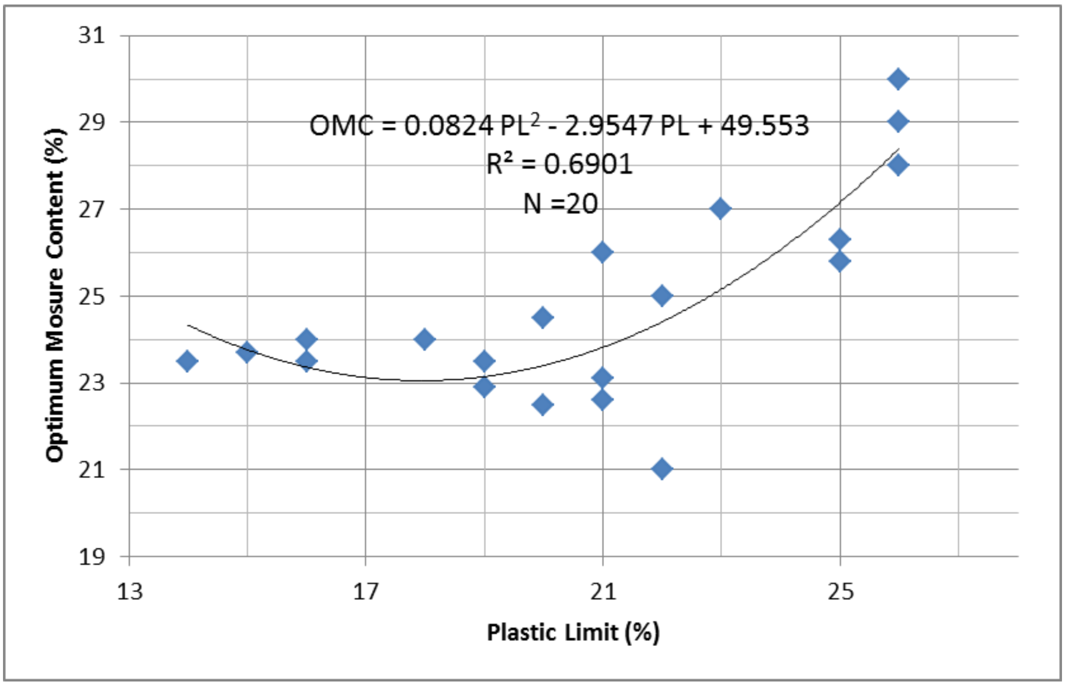


Figure 4.2: Scatter plot and best-fit line for Plastic limit and OMC

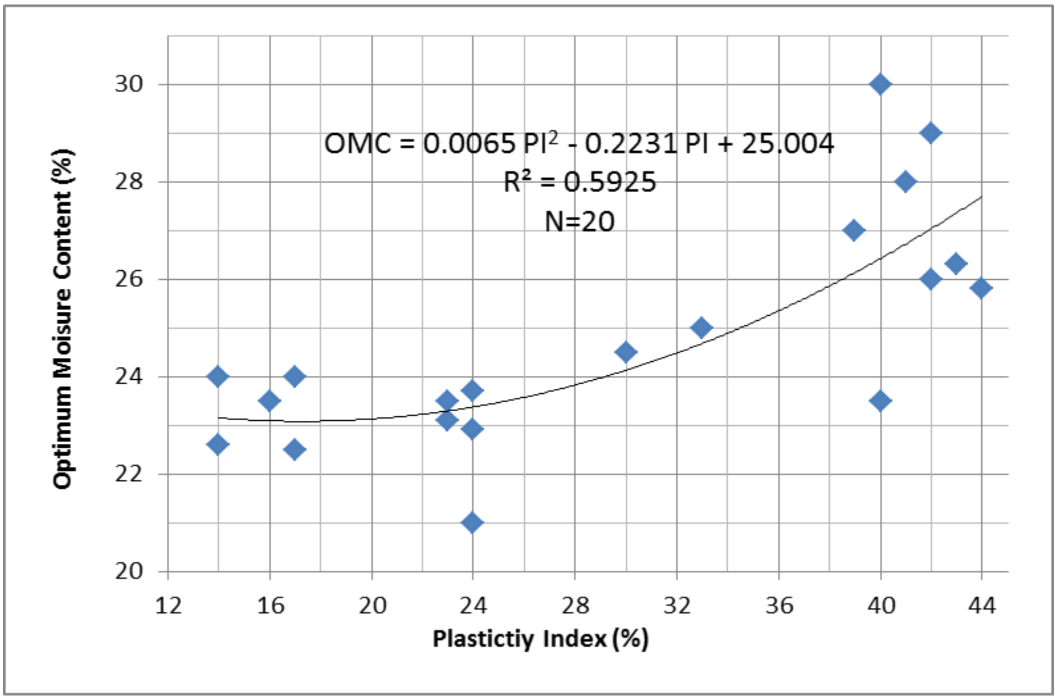


Figure 4.3: Scatter plot and best-fit line for Plasticity Index and OMC

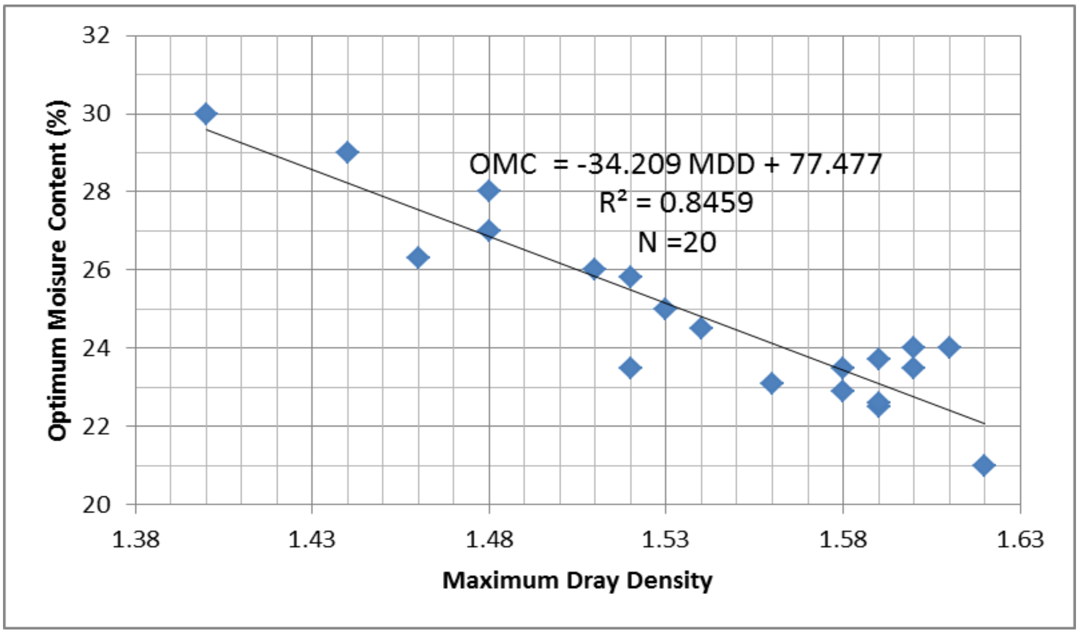


Figure 4.4scatter plot and best -fit line for MDD and OMC

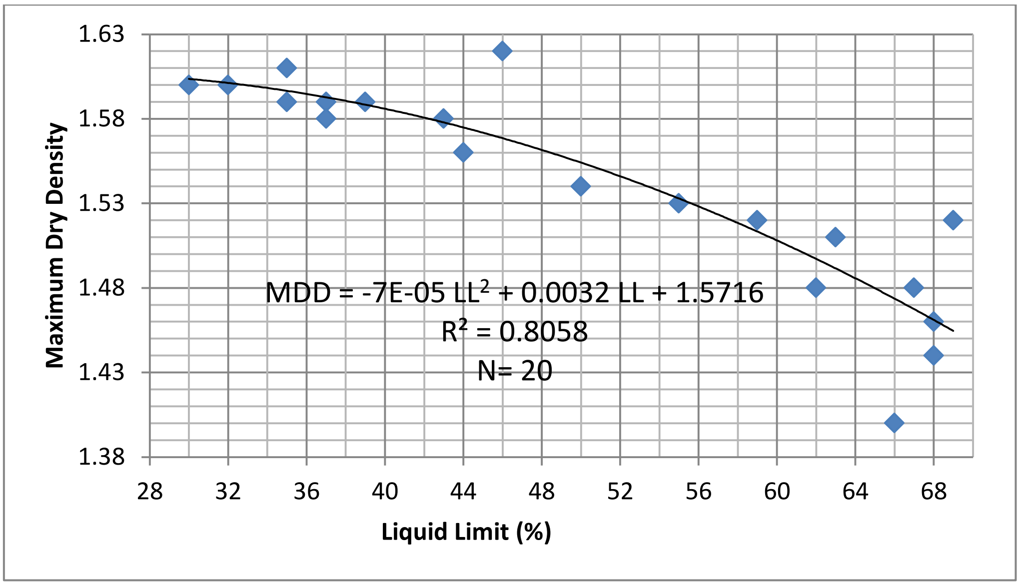


Figure 4.5scatter plot and best-fit curve for liquit limit and MDD

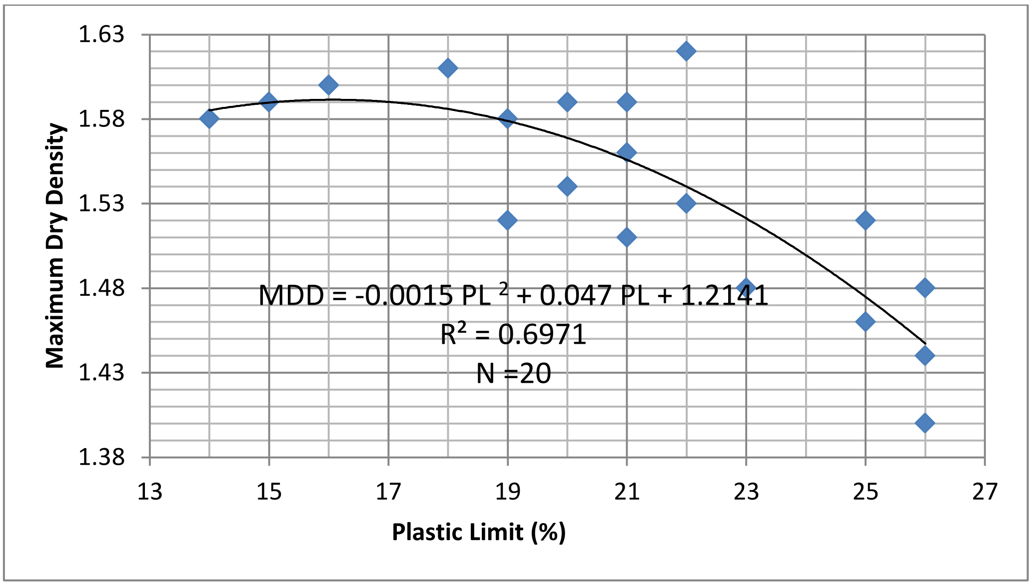


Figure 4.6 scatter plot and best -fit curve for plastic limit and MDD

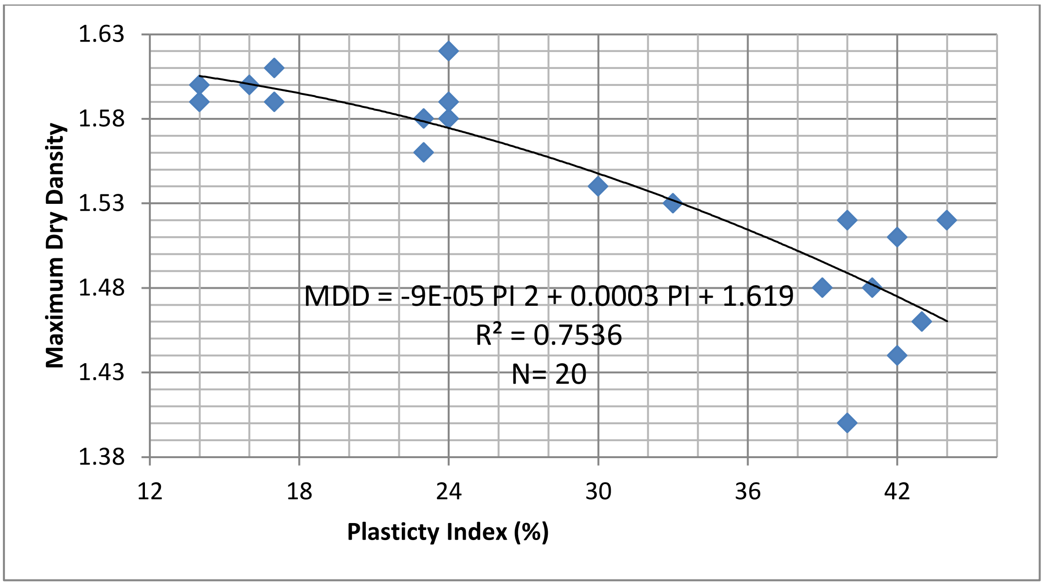


Figure 4.7scatter plot and best -fit curve for plasticty index and MDD

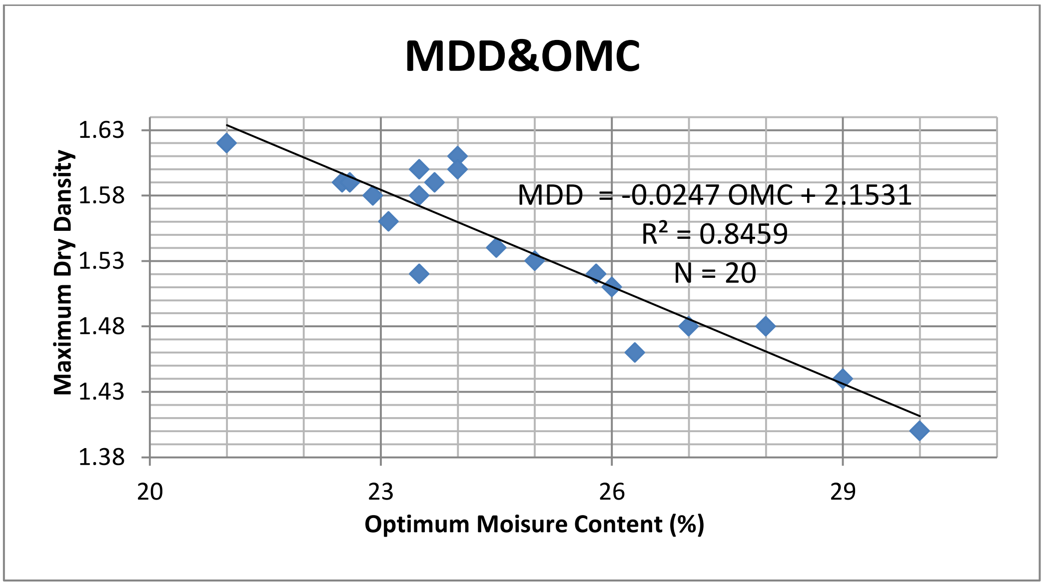


Figure 4.8 scatter plot and best -fit line for OMCand MDD

It was found that the OMC has a strong correlation with LL than PL, PI. On the other hand, as shown in **Figure 4.1**, the relationship between the OMC and PI is the weakest of all the Atterberg limits, as shown **in Figure 4.3**. It was observed that the OMC has the best relationship with MDD than all other parameters, as shown in **Figure 4.4.**

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

It was noticed that LL has a good relationship with MDD as shown **in Figures 4.5.** Both OMC and MDD can be predicted from LL only with acceptable accuracy. As shown in **Figure 4.6**, the relationship between the MDD and PL is the weakest of all the Atterberg limits. In addition, MDD has the best relationship with OMC than all other parameters. Thus, it can also be predicted MDD from OMC more accurately than LL versus the OMC.

The relationship between the dependent and independent variables are examined separately for the second Group data as presented in Figures below.

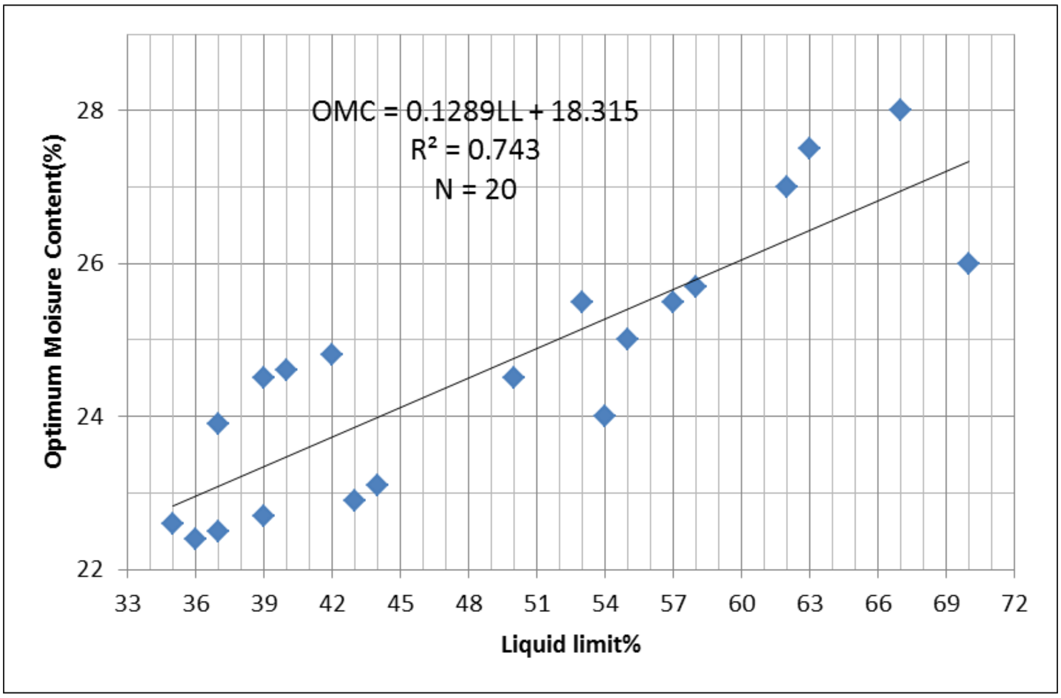


Figure 4.9 scatter plot and best -fit line for liquid limit and OMC

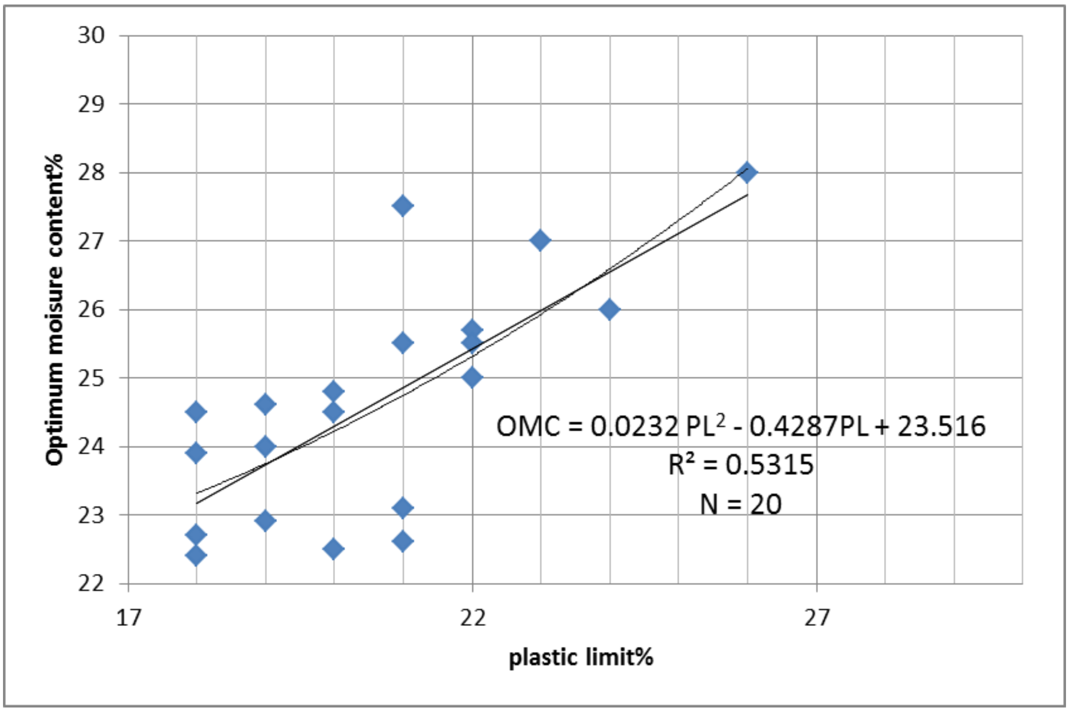


Figure 4.10 scatter plot and best- fit line for plastic limit and OMC

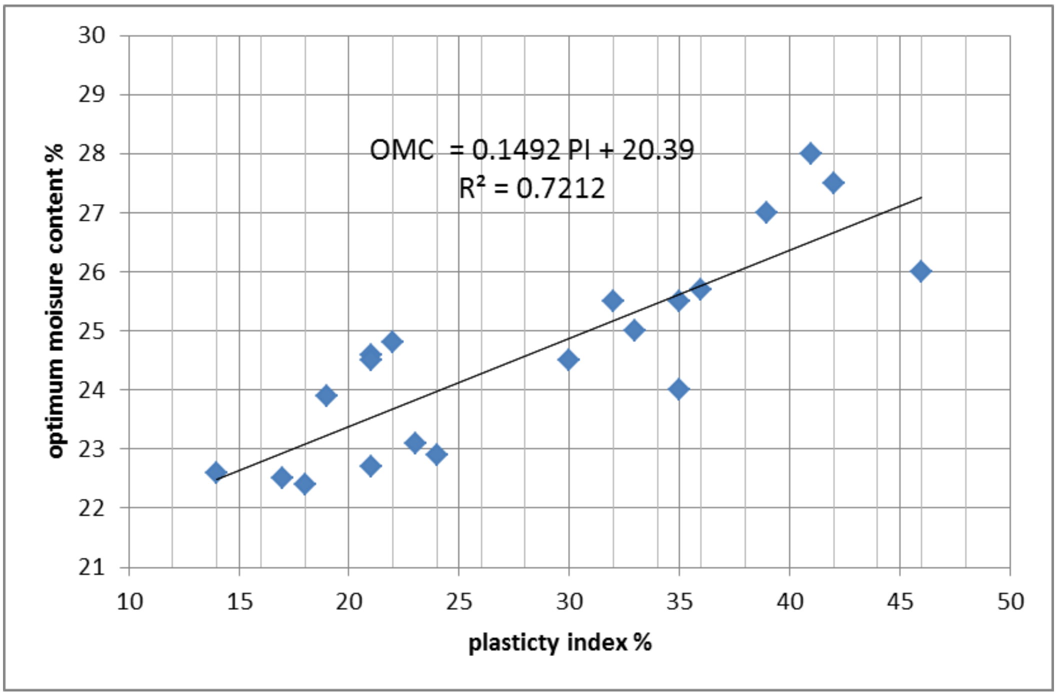


Figure 4.11 scatter plot and best -fit line for plasticty index and OMC

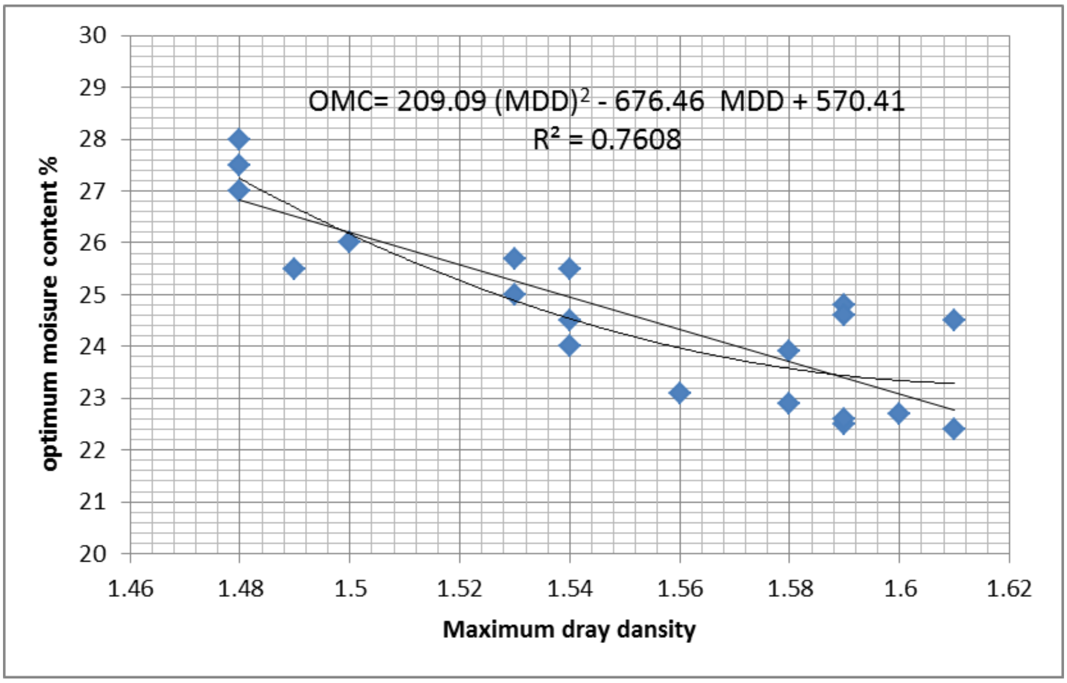


Figure 4.12 scatter plot and best -fit line for MDDand OMC

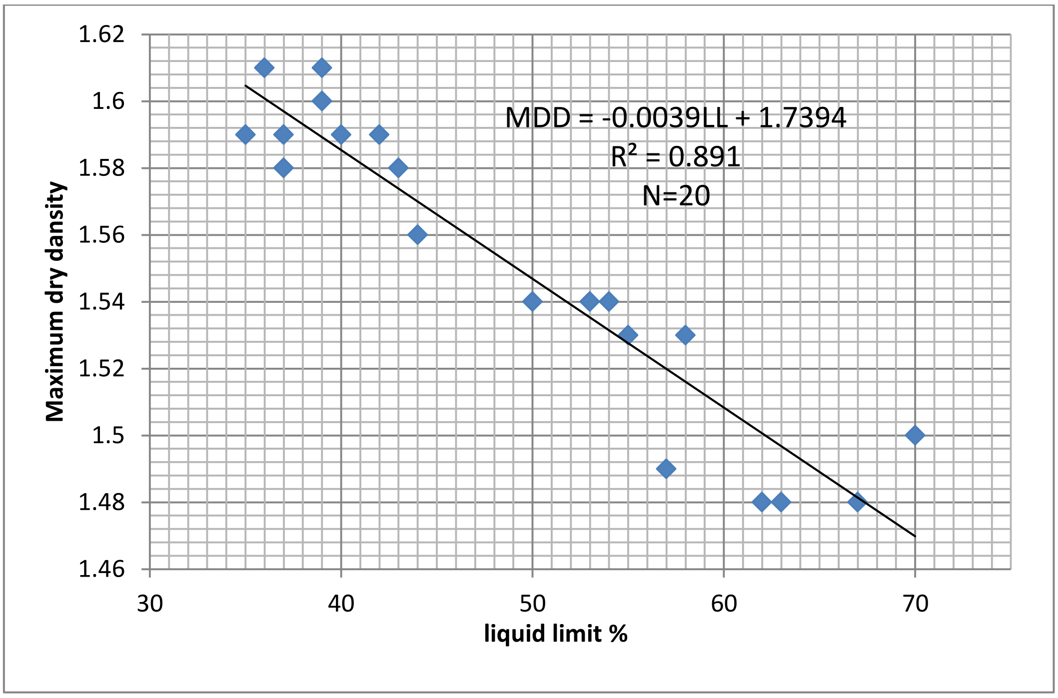


Figure 4.13 scatter plot and best -fit line for liquid limit and MDD

.

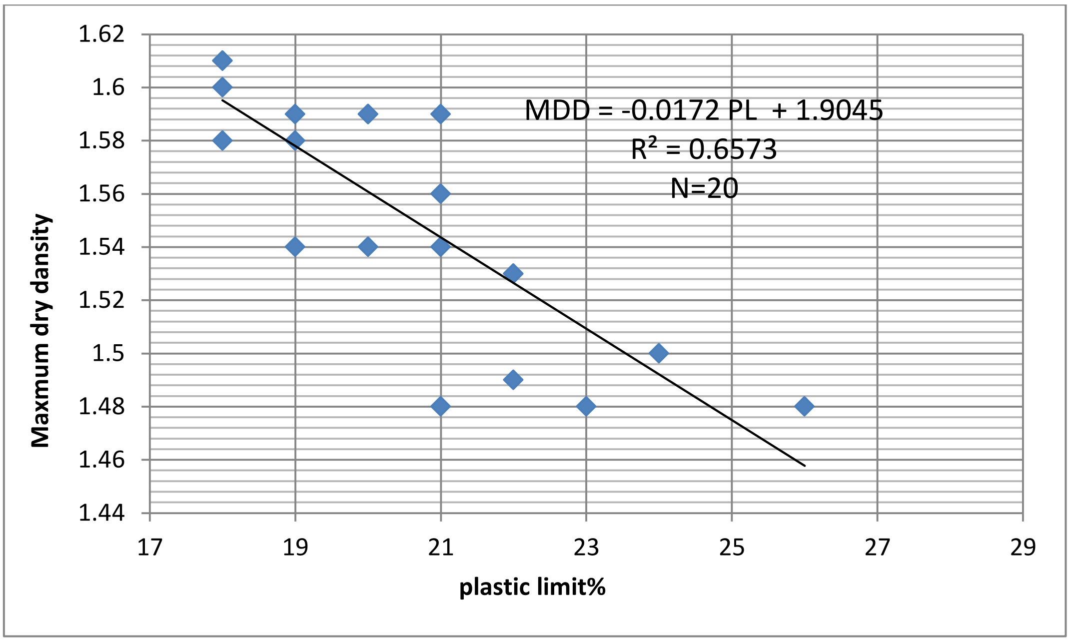


Figure 4.14 scatter plot and best - fit line for plastic limit and MDD

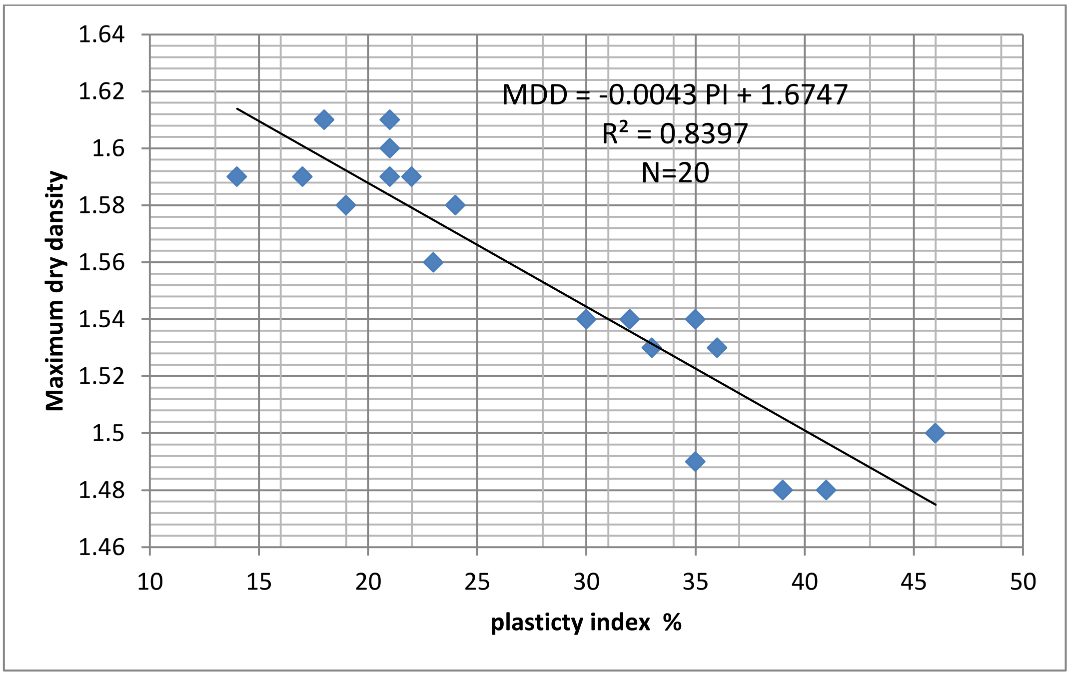


Figure 4.15 scatter plot and best -fit line for plasticity index and MDD

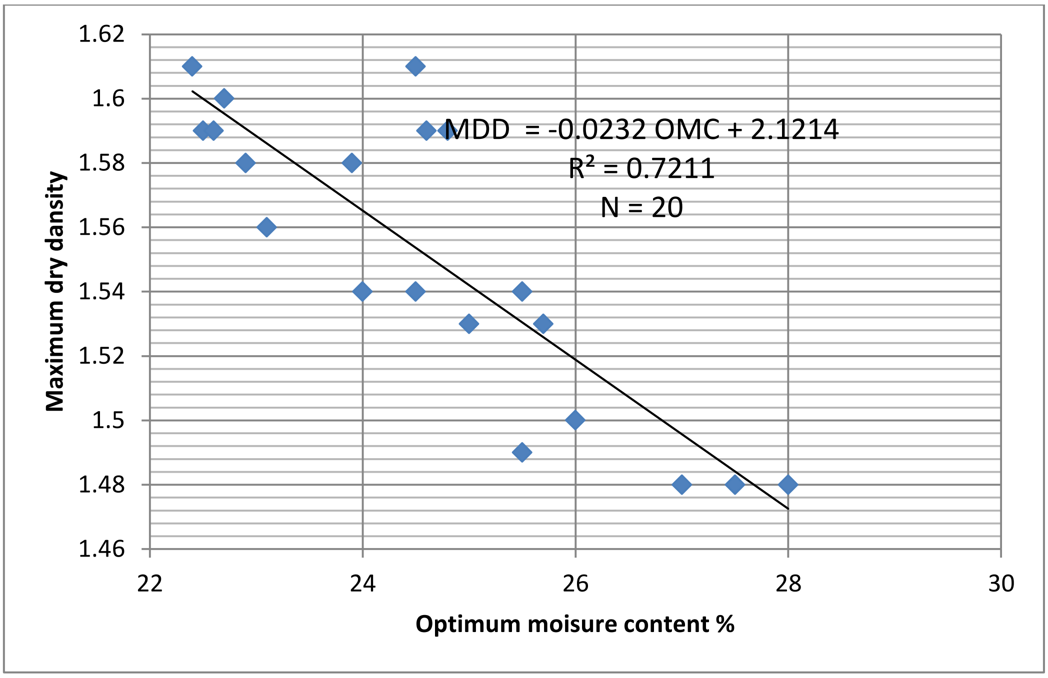


Figure 4.16 scatter plot and best - fit line for OMC and MDD

Similarly, the scatter plots of OMC and Atterberg parameters for second group were examined as presented in **Figures 4.9 – 4.12.** The OMC has a strong correlation with LL than PL, PI. On the other hand, as shown in **Figure 4.9,** while the relationship between the OMC and PL is the weakest of all the Atterberg limits as shown in **Figures 4.10**. It was noticed that OMC has the best relationship with MDD than all other parameters.

It was observed that the LL has a good relationship with the MDD. Both OMC and MDD can be predicted from LL only with acceptable accuracy. As shown in **Figure 4.13**, the relationship between the MDD and PL is the weakest of all the Atterberg limits as shown in **Figure 4.14**. MDD has the best relationship with OMC than all other parameters.

Analysis Results between first group and second group is same Results, The OMC and MDD has a strong correlation with LL than PL, PI.

A comparison between optimum moisture content and maximum dry density Obtained by proctor test and by the equation proposed was presented in the **Table 4.1.** It was concluded that characteristics of soils found by proctor test and proposed equations have good relationship as shown in below.

Table 4.1 a comparison between OMCand MDD obtained by proctor test and equation

No.	OMC (%)		MDD g/cm^3	
	By Proctor	By proposed Equation	By Proctor	By proposed Equation
1	22.5	23.05	1.59	1.60
2	24.5	23.43	1.54	1.55
3	29.0	27.1	1.44	1.47
4	25.0	24.45	1.40	1.41
5	22.9	23.8	1.60	1.58
6	24.5	23.32	1.58	1.57
7	27.0	26.21	1.61	1.59
8	25.0	24.88	1.48	1.49

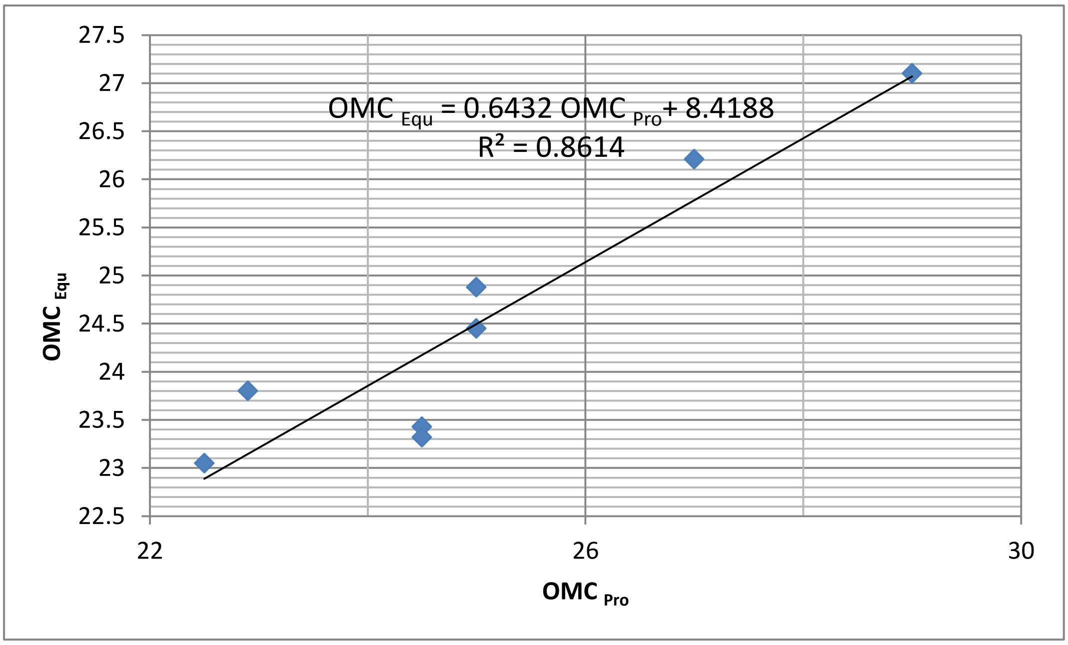


Figure 4.17scatter plot and best -fit line for OMCpro and OMC Equ

.

Figure: 4.18: Scatter Plot and Best-fit Line for MDD_{pro} and MDD

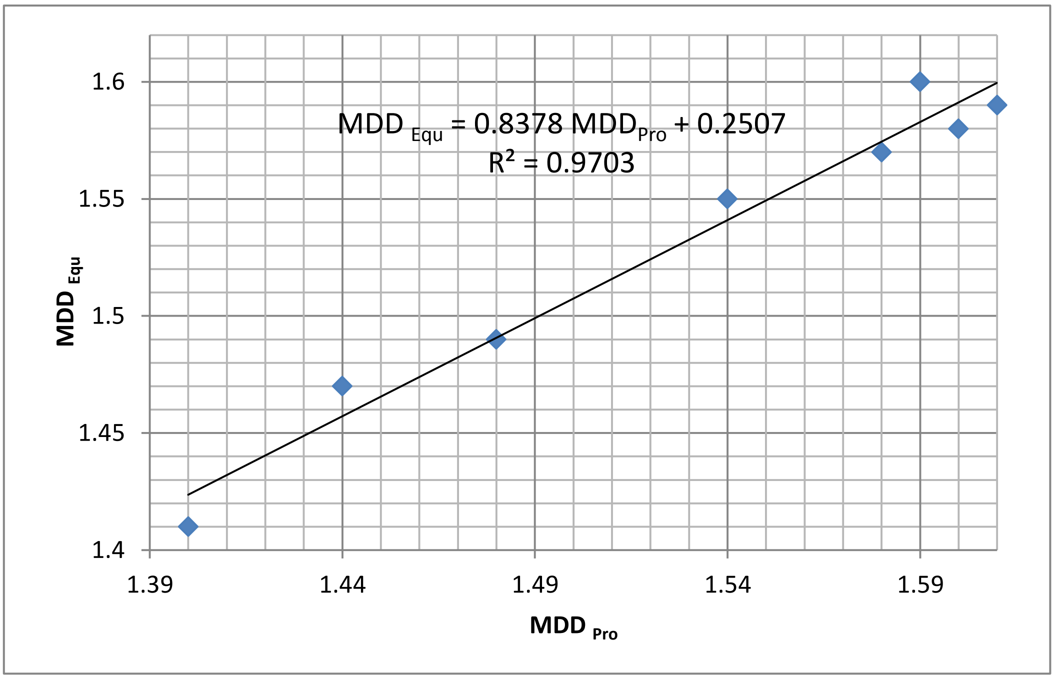


Figure 4.18 scatter plot and best -fit line for MDDpro and MDDequ

The comparison between optimum moisture content and maximum dry density Obtained by proctor test and optimum moisture content and maximum dry density Obtained by the equation proposed a very good relation between them.

4.3 Regression analysis of Two Groups

Regression analysis is a statistical technique for modeling and investigating the relationship between two or more variables. A variable whose value is predicted is called dependent variable or response. A variable used to predict the value of dependent variable is termed independent or predictor variable. A regression model that contains more than one predictor variable is called multiple regression models. Alternatively, Regression model containing one independent variable is termed as simple regression model.

A number of techniques can be used to indicate the adequacy of a multiple regression model; some of these are standard error and the coefficient of regression (R^2) values. The standard error of a statistic gives some idea about the precision of an estimate.

Tables 4.2 – 4.3 show the summary of output equations, R^2 and standard error (SE) for OMC and MDD of the two groups

Table 4.2: Summary of equations and their corresponding R^2 in predicting OMC and MDD first group

No	Coefficients of Predictors									Output Equation	R^2	SE
	LL	PL	PI	LL^2	PL^2	PI^2	OMC	MDD	C			
1	0.5445	--	--	0.0066	--	---	--	--	34.158	OMC = $0.0066LL^2 - 0.5445LL + 34.158$	0.7099	3.1228
2	--	-2.954	--	---	0.0824	--	---	----	49.553	OMC = $0.0824PL^2 - 2.954PL + 49.553$	0.6901	0.8362
3	---	---	-0.2231	--	--	0.0065	--	---	25.004	OMC=$0.0065\ PI^2 - 0.2231PI + 25.004$	0.5925	2.4660
4	--	--	--	--	--	--	--	-34.209	77.477	OMC= $-34.209MDD + 77.477$	0.8459	0.0140
5	0.0032	--	--	$-7 * \in^{-0.5}$	--	--	---	---	1.5716	MDD = $-7 \in^{-0.5} LL^2 + 0.0032LL + 1.5716$	0.8058	3.1228
6	--	0.047	--	--	-0.0015	--	--	--	1.2141	MDD= $-0.0015PL^2 + 0.047PL + 1.2141$	0.6971	0.8362
7	--	--	0.0003	--	--	$-9 * \in^{-05}$	--	--	1.619	MDD = $-9\epsilon^{-05}PI^2 + 0.0003PI + 1.619$	0.753	2.4660
8	--	--	--	--	--	--	-0.0247	--	2.15	MDD =- $0.0247\ OMC + 2.15$	0.8459	0.5211

Note: LL = Liquid limit, PL = Plastic Limit, PI = Plasticity index, OMC = Optimum Moisture Content, MDD = Maximum dry Unit Weight.

R^2 = Coefficient of regression SE = Standard error of estimate, C= Constant

Table 4.3 summary of equation and their corresponding R2in predicting OMC MDD second group

NO	Coefficients of Predictors									Output Equation	R^2	SE
	LL	PL	PI	MDD	PL^2	PI^2	OMC	MDD^2	C			
1	0.1289	---	--	--	--	--	--	--	18.315	OMC = $0.1289LL + 18.315$	0.743	2.511
2	--	0.4287	--	--	-0.0232	--	--	--	23.516	OMC =$0.0232PL^2 - 0.4287PL + 23.516$	0.5315	0.4834
3	--	--	0.1492	--	--	--	--	--	20.39	OMC = $0.1492PI + 20.39$	0.7212	2.1367
4	--	--	--	-676.46	--	--	--	676.46	570.41	OMC=$209.09MDD^2 - 676.46MDD + 570.41$	0.7608	0.01024
5	-0.0039	--	--	--	--	--	--	--	1.7394	MDD = $-0.0039LL + 1.7394$	0.891	2.511
6	--	-0.0172	--	-	--	--	--	--	1.9045	MDD = $-0.0172PL + 1.9045$	0.6573	0.4834
7	--	--	-0.0043	--	--	--	--	--	1.6747	MDD =- $0.0043PI + 1.6747$	0.8397	2.1367
8	--	--	--	--	--	--	-0.0232	--	2.214	MDD = $-0.0232OMC + 2.214$	0.7211	0.3754

Note: LL = Liquid limit, PL = Plastic Limit, PI = Plasticity index, OMC = Optimum Moisture Content, MDD = Maximum dry Unit Weight.

R2 = Coefficient of regression SE = Standard error of estimate, C= Constant

4.4 Discussions of Results

Tables 4.2- 4.3 show the summary of regression analysis results in predicting the optimum moisture content and maximum dry density first and second groups data from the corresponding Atterberg limits respectively. An attempt is made to obtain which one of the predictors can be strongly related with dependent variables. This has been done by predict the OMC and MDD from one or more independent variables.

The discussion of regression analysis was summarized as follows:

a) In the first group, it has been found that the optimum moisture content (OMC) has a strong correlation with liquid limit and weakest correlation with plastic limit. In addition, the maximum dry density (MDD) has also has strong correlation with the liquid limit (LL) than the other Atterberg limits. Thus, both OMC and MDD have good relationship with liquid limit (LL) than the plastic limit (PL) and plasticity index (PI). Therefore, one can conclude that both OMC and MDD can be predicted from the correlation equations without significant errors. However, it should be noted that the optimum moisture content (OMC) has strongest correlation with maximum dry density (MDD) than all other parameters.

b) In the second group, it has been found that the OMC has a strong correlation with liquid limit (LL) and weakest correlation with plastic limit (PL). In addition, the maximum dry density (MDD) has also has strong correlation with the liquid limit (LL) than the other Atterberg limits. Thus, both OMC and MDD have good relationship with liquid limit (LL) that the plastic limit (PL) and plasticity index (PI).

c) Therefore, one can conclude that both OMC and MDD can be predicted from the correlation equations without significant errors. However, it should be noted that the optimum moisture content (OMC) has strongest correlation with maximum dry density (MDD) than all other parameters.

d) Therefore, it is recommended that both OMC and MDD should be predicted from one independent variable (LL) without significant reduction in the correlation coefficient, instead of using two or more independent variables, since the value of regression coefficient is almost the same in both cases.

e) When soil is compacted at low water content, the soil is stiff and has more void space resulted in lower dry unit weight. If the water content is increased excessively, the space that might have been occupied by solid particles is occupied by water and also resulted in lower dry unit weight.

f) If the soil is compacted at OMC, the soil particles get lubricated and move easily in to close state position and the corresponding dry unit weight is higher. This specific water content (OMC) of fine soil is very close to LL. In addition, as the fine content of soil increases, both OMC and LL are increased but MDD is reduced. This condition might be the possible reason that the OMC and MDD have good correlation with liquid limit.

g) The comparison between optimum moisture content and maximum dry density obtained by proctor test and optimum moisture content and maximum dry density obtained by the equation proposed a very good relation between them.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Introduction

This concluding chapter summarizes the contribution of this study, the work done, conclusions obtained from the results analysis and recommendations for the future researches

5.2 Conclusions

The following conclusions can be drawn based on the laboratory test results:

1. The maximum dry density (MDD) and optimum moisture content (OMC) of fine soils have good relation with liquid limit (LL) than the plastic limit (PL) and plasticity index (PI).

2. The maximum dry density (MDD) has a stronger correlation with liquid limit (LL) and best relationship with optimum moisture content (OMC). Therefore, both OMC and MDD of soils can be predicted from liquid limit (LL) especially for prefeasibility study of projects.

3. The main objective of this thesis was to obtain valid relationships between Atterberg limits (LL, PL&PI) and compaction characteristics. However, it should be noted that the optimum moisture content (OMC) has a stronger correlation with maximum dry density (MDD) than all other parameters.

4. The comparison between optimum moisture content and maximum dry density Obtain by proctor test and optimum moisture content and maximum dry density Obtain by the equation proposed a very good relation between them, R2 = 0.9703 and R2 =0.8614.

5. It was concluded that previous studies of predicting compaction characteristics of over-consolidated soils (Husain, 2016) and the case study of Upper Atbara have found that (LL) a good relationship with (MDD) and (OMC). However (Nerea, 2012) has found that the plastic limit (PL) has a stronger relationship with (MDD) and (OMC) more than the Liquid limit (LL) and Plasticity Index (PI), which is differ to this study.

6. The relationships between compaction characteristics and Atterberg limits differ according to the type of soil.

5.3 Recommendations

The following recommendations are suggested for further studies:

1. This work can further be extended to relate the soil properties with other tests such as modified Proctor test, California Bearing Ratio (CBR) and Permeability test.
2. In the execution of mega projects e.g. (long road, and large embankment dams), number of compaction test are to be executed is time consuming.

Thus it is very important to find a relation between the Atterberg limits and compaction characteristics, so it will be quicker cheaper and simpler method of testing; such parameters could be used for prediction of compaction characteristics.

REFERENCES

1. Nerea, (2012), "Prediction of Compaction Characteristics from Atterberg Limits for Fine Grained Soils", M.Sc in civil engineering, Addis Ababa Univer.

2. Raymond, G. P. (1997). "Geotechnical Engineering”, Unified Soil Classification.

3. Terzaghi, K. (1943), "Theoretical Soil Mechanics”, John Wiley & Sons INK, London, UK.

4. McCarthy, David F. (2007), Essentials of, “soil mechanics and foundations”. USA NJ: Pearson Prentice Hall.

5. Murthy, V.N.S. (2007).” Geotechnical Engineering, Principles and Practice of Soil Mechanics and Foundation Engineering. New York, INC: Marcel Dekker.

6. Robert, L. P., Derek, H. F., & Stephen, A. C. (2000). “Compaction and settlement of existing embankments. Lawrence, Kansas: University of Kansas.

7. Pandian, N.S., Nagaraj, T.S., & Manoj, M. (1997).” Re-examination of compaction characteristics of fine-grained soils. Geotechnique, 47 (2), 363–366.

8. Sivrikaya, O. Togrol, E., & Kayadelen, C. (2008). “Estimating compaction behavior of fine-grained soils based on compaction energy. Canadian Geotechnical Journal, 45(6), 877-887.

9. Arora, K.R. (2004).” Soil mechanics and foundation engineering. New Delhi: NAI SARAK, 6th ed.

10. Alemayehu, T., & Mesfin, L. (2009).” Soil Mechanics. Addis Ababa: AAU press, 2nd ed.

11.Amer Ali, Al-Rawas, and Mattheus, F.A.G.(2006).” Expansive Soils. Taylor & Francis, London, UK .

12. Khan, Z. (2014),” Compaction of soil process, necessity and theory of compaction. Retrieved April 15, 2016 from http://www.yourarticlelibrary.com

13. Dessalegn Sissay (2003). “Computation of Soil Compressibility using Tangent Modulus approach

14 Robert W.Day (2004), chief engineering, American.” (Soil mechanics and foundations)

15. Husain, (2016),” prediction of compaction characteristics of over-consolidated soils, Near East University.

16. Method statement (MS) for mixed in place (MIP) cut –off wall (CWO) construction (DCUAP).

17. Construction Method statement (CMS-No. 49, Rev. D) of Test Fill for Materials from BU3_QF (DCUAP).

18. Technical specifications part 1 (2010), civil and hydro mechanical works, volume 20f 7(DCUAP).

19. Elalem Worku (2010). “Prediction of California Bearing Ratio (CBR) Value from Index Properties of Soil. MSC Thesis, Addis Ababa University, Addis Ababa.

20. BS-1377, Part 2 British Standard Methods of test for, Soils, for civil engineering purposes. Classification tests.

21. BS-1377, Part 4 British Standard Methods of test for, Soils, for civil engineering purposes. Compaction-related tests.

22. ASTM D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)

23. ASTM D2216-98 standard test method for, laboratory determination of water (moisture) content of soil and rock by mass1.

24. ASTM D2974 Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils1.

25. ASTM D698 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft3(600 kN-m/m3))

26. Joslin, J.C. (1959). Ohio’s typical water density curves. American Society for testing and materials, Special Technical Publication, STP 239, 111-118.

27. Ring, G., Sallberg, J., & Collins, W. (1962). Correlation of compaction and classification of test data. Highway Research Bulletin,325, 55-75. Washington, D.C:Highway Research Board, NRC.

2.8 Torrey, V. H. (1970). Analysis of field compaction control data, delaware river, kansas, miscellaneous paper S- 70-13, report 1, vicksburg, MS : US army corps of engineers waterways experiment station.

29. Jeng, Y. S., & Strohm, W. E. (1976). Prediction of shear strength and compaction characteristics of compacted fine-grained cohesive soils. Final Report, U.S. Army Corps of Engineers Waterways Experiment Station, Soils and Pavement Laboratory, Vicksburg.

30. Al-Khafaji, A. (1993). Estimation of soil compaction parameters by means of Atterberg limits. Quaterly Journal of Engineering Geology and Hydrogeology,26, 359-368.

31. Omar, M., Shanableh, A., Basma, A., & Barakat, S. (2003). Compaction characteristics of granular soils in United Arab Emirates. Journal of Geotechnical and Geological Engineering, 21, 283-295.

32. Gurtug, Y., and Sridharan, A. (2004). Compaction behaviour and prediction of its characteristics of fine-grained soils with particular reference to compaction energy. Journal of Soil and Foundations, 44(5), 27-36.

33. Sridharan, A., & Nagaraj, H. B. (2005). Plastic limit and compaction characteristics of fine-grained soils. Journal of Ground Improvement, 9(1), 17-22.

34. Sivrikaya, O. Togrol, E., & Kayadelen, C. (2008). Estimating compaction behavior of fine-grained soils based on compaction energy. Canadian Geotechnical Journal, 45(6), 877-887.

35. Gurtug, Y. (2009). Simplified method of prediction of compaction characteristics of soils. In Proceeedings of the 2nd International Conference on New Developments in Soil Mechanics and Geotechnical Engineering (pp. 568-572). Turkish Republic of North Cyprus : Near East University.

36. Ugbe, F. C. (2012). Predicting compaction characteristics of lateritic soils of Western Niger Delta, Nigeria. Research Journal of Environment and Earth Sciences, 4(5), 553-559.

37. Mujtaba, H., Farooq, K., Sivakugan, N., and Das, B. M. (2013). Correlation between Gradation Parameters and Compaction Characteristics of Sandy Soils. International Journal of Geotechnical Engineering, 7(4), 395-401.

38. Sivrikaya, O., Kayadelen, C., & Cecen, E. (2013). Prediction of the compaction parameters for coarse-grained soils with fines content by MLR and GEP. ACTA Geotechnica Slovenica Jurnal, 2, 29-41.

39. Jyothirmayi, K. H., Gnanananda, T., & Suresh, K. (2015). Prediction of compaction characteristics of soil using plastic limit. International Journal of Research in Engineering and Technology, 04 (06), 253-256.

40. soil compaction handbook, rev. a (0212_bd), copyright 2011, multiquip inc.

41. manak nagar, lucknow-11 (2005) study report on compaction equipments and construction machinery, report no. ge- r-76, geotechnical engineering directorate research designs & standards organization.

42. ASTM D2937: Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method.

42. ASTM D4643: Determination of Water (Moisture) Content of Soil by Microwave Oven Heating.

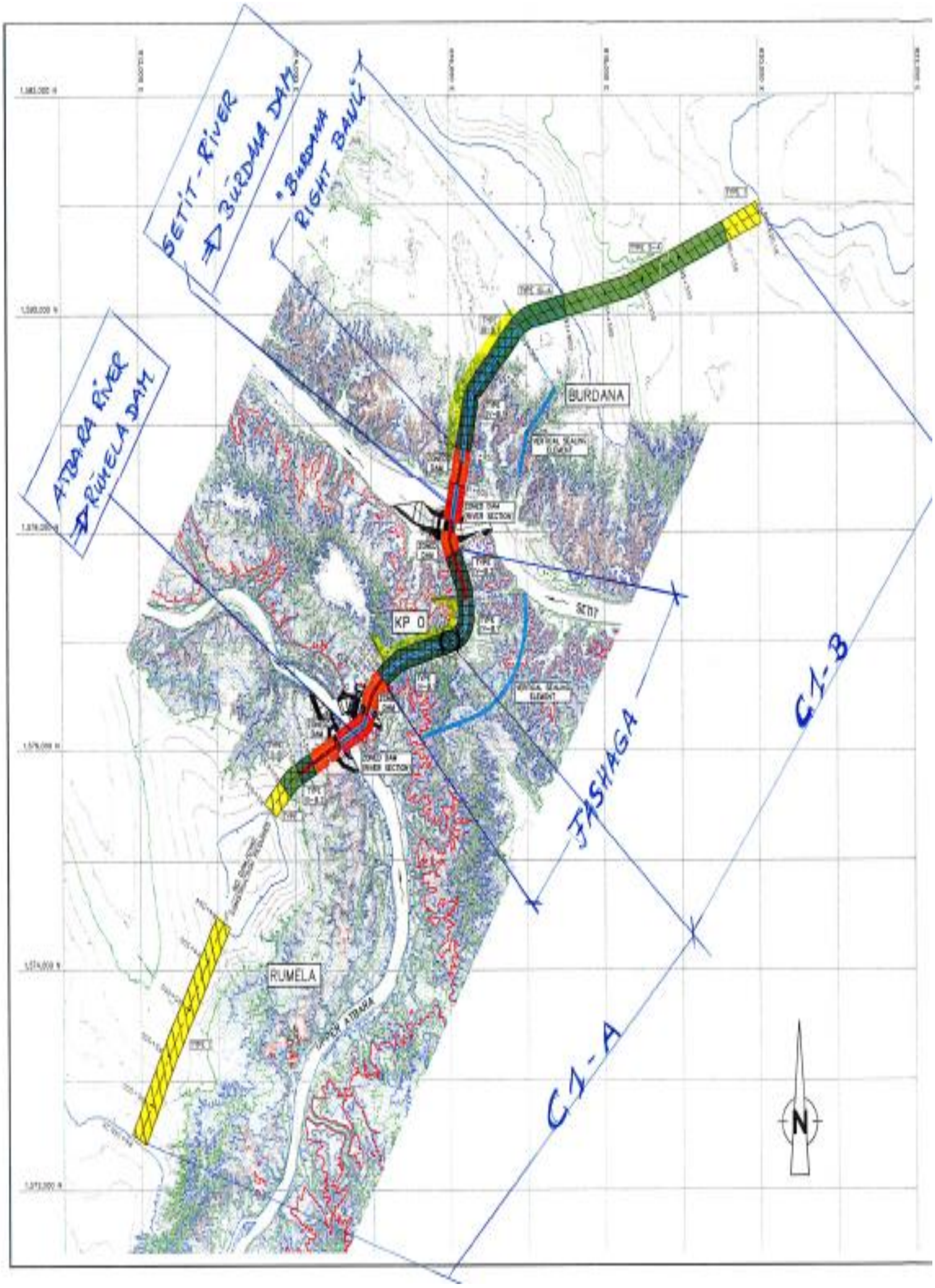
43. ASTM D2292-91: Standard Practice for obtaining Hydrostatic or Pressure Design Basis for" Fi berg liss'; (G lass-Fi ber-Rei nforced Thermosetti n g-Resi n) Pipe and Fittingsl .

APPENDECIES

APPENDEX A

Appendixes A: Sudan map

APPENDEX B



Appendixes B: General and specific DCUAP Dam conditions.

APPENDEX C

Appendixes C: Geo-investigation of BU3-QF Borrow Area