CHARACTERIZATION OF COHESIVE SUBGRADE SOIL FOR FLEXIBLE PAVEMENT DESIGN (IN CASE OF WEZEKA-KONSO ROAD)

MSc THESIS

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CHARACTERIZATION OF COHESIVE SUBGRADE SOIL FOR FLEXIBLE PAVEMENT DESIGN IN CASE OF WEZEKA – KONSO ROAD

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ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled "**characterization of cohesive subgrade soil for flexible pavement design In Case of Wezeka-Konso Road**" submitted in partial fulfillment of the requirements of the degree of Master's with specialization in **Road and Transport Engineering** the graduate program of the school of civil engineering , and has been carried out by **Mulugeta Regassa** Id. No. **PGRo/029/09**, under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the school.

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EXAMINERS' APPROVAL SHEET

We, the undersigned, members of the board of examiners of the final open defense by **Mulugeta Regassa** have read and evaluated his thesis entitled" **Characterization of Subgrade Soil for Flexible Pavement Design: in case of Wezeka to Konso road**", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of masters in Civil Engineering specialization in Road and Transportation Engineering.

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DECLARATION

I, the undersigned declare that the study intitled "**Characterization of cohesive subgrade soil for flexible pavement design in case of Wezeka-Konso road**" is my original work. The work has not been presented for the award of any degree. Materials used for this study are duly acknowledged and referenced in a proper format.

Mulugeta Regassa ____________________

Name of student signature

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LIST OF ABBREVIATIONS

AASHTO	American Association of State highway and Transportation Officials
CBR	California Bearing Ratio
ERA	Ethiopian Road Authority
LL	Liquid Limit
MDD	Maximum Dry Density
ME	mechanistic - empirical
MR	Resilient Modulus
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
w	Moisture Content
P_{200}	Percent of Soil Finer than sieve number 200
qu	unconfined compressive strength

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ABSTRACT

Road transport has a vital contribution for a nation development. Road has to be designed in a way that it can function in all season of the year, this could be achieved if the road is paved and the structural elements of the pavement is well designed. In general pavements are classified as rigid and flexible pavements. In rigid pavement loads are carried by the concrete itself thus the design requirement is the flexural strength of the concrete whereas in flexible pavement, the intention of the design is to reduce the deformation of the subgrade under a certain loading condition. Thus, flexible Pavement design needs proper material characterization so that failures in structural as well as functional could be mitigated. The aim of this study is characterization of cohesive subgrade soil for flexible pavement design in case of Wezeka – Konso road, Ethiopia. Identification of the Physical properties and classification of the soil was done according to AASHTO criteria. Resilient modulus was predicted from unconfined compressive strength. Additionally, a method has been developed for prediction of resilient modulus from unconfined compressive strength and index properties of soil. For both prediction models Microsoft excel was used to conduct the statistical analysis. The soils used for this study were low to medium plastic fine-grained soils. According to AASHTO soil classification system the soils were group in A-4, A-5, A-6 and A-7-5 categories. Statistical analysis shows that the relation between resilient modulus and unconfined strength was linear correlation. For this correlation the coefficient of determination value of $R^2 = 0.83$ *was obtained. For the second model which is prediction of resilient modulus from unconfined compressive strength and index properties the correlation was multilinear with coefficient of determination* $R^2=0.87$*. Due to the inclusion of index properties in the second model a better correlation was observed.*

Key words*: - Subgrade soil, Resilient Modulus, Unconfined compressive strength, index properties*

1. INTRODUCTION

1.1 Background

Road transport has a vital contribution for a nation development. Road has to be designed in a way that it can function in all season of the year, this could be achieved if the road is paved and the structural elements of the pavements are well designed. In general pavements are classified as rigid and flexible. In rigid pavement loads are carried by the concrete itself thus the design requirement is the flexural strength of the concrete. whereas in design of flexible pavement, the main aim of the design is to reduce the deformation of the subgrade under a certain loading condition. Design of flexible pavements for airport and highway requires the characterization of the foundation soil as well as the paving material and their behavior under all climatic conditions (Yoder and Witczak, 1975).

According to Yoder and Witczak (1975) the performance of pavements is highly dependent on the strength of subgrade soil. In the early, all pavements were constructed with the same thickness. The thickness provision was based on experience observed on other roads. The strength of the subgrade soil was not considered for the thickness requirement (Huang, 2004; Yoder and Witczak, 1975). However, the thickness provided should account the strength of the subgrade soil since the performance of the pavement depends on it. Information on the strength of the soil is obtained from the result of characterization of the soil in laboratory and field. Road agencies use different pavement material characterization technique depending on the design method implemented in the pavement design.

In most part of the world, pavements are designed from empirically developed equations or charts. For this design methods subgrade characterization is mostly carried out in terms California bearing ratio (CBR). Characterization of the subgrade soil by CBR its own has limitations. One of the limitations is CBR is not a direct strength measuring test (Roger, 2003). In addition, CBR is related to shear strength of the soil however, most roads failure occurs to deformation of the subgrade soil not by shear failure (Hossain and Kim, 2014). Thus, it is important to characterize subgrade soil in terms of a soil parameter that accounts the stress-strain properties and deformation of the soil under a certain stress level. This advantage could be obtained through the characterization of the soil by resilient modulus. Resilient modulus test is the recommended test to characterize subgrade soil for pavement design in 1993 AASHTO pavement design guide as well as newly proposed mechanistic – empirical pavement design guide (MEPDG). The direct laboratory determination of resilient modulus is carried out as per AASHTO T-307 test procedure. However, resilient modulus test requires significant resources including high level of technical capability to conduct. To minimize this drawback, modulus of resilient has been correlated with many soil tests. Among these correlations, correlation with CBR is widely in use. Even though resilient modulus is predicted using CBR, the relation between the two is not good enough. This is because of the fact that CBR is a measure of shear strength whereas resilient modulus is a measure of stiffness. Thus, it is more appropriate to relate modulus of resilient with those tests which show the stress state and deformation properties of the soil (Hossain and Kim, 2014).

In Ethiopia, the design of flexible pavement is carried by using charts which relate the traffic class with that of subgrade class (ERA, 2013). The characterization of the subgrade soil is in terms of CBR. According to Rogers (2003) CBR test is not a direct test for determination of strength of pavement material. Therefore, t is more appropriate to characterize the subgrade soil by soil properties that show more clearly the stress level and deformation under a certain loading condition. In this study the cohesive subgrade soil along Wezeka to Konso is characterized through resilient modulus which is the elastic property of subgrade soil. The study shows possibilities on its part on the characterization of subgrade soil for mechanistic-empirical design in Ethiopia. Mechanistic empirical method is a pavement design method which correlate the properties of the pavement material, the traffic load and climatic condition with that of pavement performance (Huang, 2004). The characterization of the subgrade material in terms of resilient modulus is one of the inputs for mechanistic – empirical method. The study considered characterization of the cohesive subgrade soil by determining modulus of resilient from unconfined compressive test and developed correlation between modulus of resilient and soil index property tests. The developed model is multilinear regression model.

1.2 Statement of the problem

Pavement failures mostly occur due to improper consideration of materials and due to implementation of inadequate design method. Until the current time in Ethiopia pavements are designed using charts provided by ERA. In ERA pavement design manual, granular material and subgrade soil characterization is in terms of CBR. CBR is not measure of deformation and it does not show any relation with pavement failures like permanent deformation and fatigue cracking (Drumm et al, 1990). To understand failures related to subgrade soil deformation it is necessary to characterize the materials in terms of its elastic property i.e., by resilient modulus. Resilient modulus is an input for mechanistic-empirical design method. Mechanistic empirical design method correlates the properties of the pavement material, the traffic load and climatic condition with that of pavement performance to determine the thickness of each pavement layer (Huang, 2004). Determination of resilient modulus in laboratory is done by repeated load triaxial test and in field it could be obtained applying back calculation technique from deflection measurements. However, this method requires well trained personnel, cost and time. In addition, it is difficult to interpret the result and the equipment are rarely available in Ethiopia. To overcome this problem and to start applying mechanistic-empirical method in Ethiopia it is necessary to determine resilient modulus from other simple tests. Thus, this study determined resilient modulus from simple tests by determining MR value from UC-test and tried to develop a regression (both simple and multi linear) model which relates MR with index properties.

1.3 Objective

1.3.1 General objective

The general objective of this study is to characterize the cohesive subgrade soil for flexible pavement along the road from Wezeka to konso, Ethiopia.

1.3.2 Specific objectives

The specific objectives of this study are: -

- To investigate the physical properties of the soil for classification the of soil as per AASHTO soil classification system
- To develop linear regression model between the resilient modulus and unconfined compressive strength
- To develop multilinear regression model between resilient modulus and unconfined compressive strength and index properties of soil

1.4 Research question

- What does the physical properties of the soil indicate and in which group the soil exist according to AASHTO soil classification system?
- How resilient modulus could be modeled from unconfined compressive strength?
- How resilient modulus could be correlated with both unconfined compressive strength and the index properties of soil?

1.5 Scope and limitation

Scope

This study considered only soil along the road from Wezeka to Konso, taking cohesive soil samples from different section along the road. The characterization of the subgrade soil is limited on laboratory tests conducted on disturbed samples.

Limitation

Due to lack of readily available in-situ MR measuring equipment the study does not considered the field measurements of subgrade soil elastic property.

1.6 Significance of the study

It may help graduate students who want to do their thesis on similar topic. It is also valuable for those researchers working on characterization of pavement materials. Additionally, for designers and consultants may give a guide on developing similar models on material characterization. Even the study is significant for Ethiopian Road Authority in showing determination of MR from simple soil tests for subgrade material characterization in terms of MR.

2. LITERATURE REVIEW

2.1 General

Subgrade layer, which represents the pavement foundation, plays a key role in the pavement structure. The overall performance of the pavement is controlled by the stiffness and saturation of the subgrade soil. Subgrade soil provides support to the pavement. Subgrade soils may deform under heavy loads consequently it contributes to distress in the overlaying pavement structure. Thus, excessive plastic deformation on the top of subgrade layer may deteriorate the pavement in the form of rutting or cracking. Especially weak subgrade soils largely contribute a significant portion of the total pavement rutting. Fine grained soil has the ability to retain water within the pores between them due to their low permeability.

According to Khasawneh (2017) Pore water has three major effect on fine grained soils: - high water contents in cohesive soils cause a lubrication effect at the soil particle contact points, thereby increasing the plastic deformation in wet soils under continuous deviatoric loading, strengths of wet (saturated) soils is lower and hence continuous loading induces more damage to subgrade soils, and positive pore pressures in near saturated conditions increase with the number of load applications and hence result in softening and the likelihood of damage occurring from heavy traffic loads increases drastically

Resilient modulus of subgrade soils in a pavement structure is one of the most important inputs in AASHTO and MEPDG method for the design of new and the rehabilitated flexible pavements (Kim, 2004). Designers and engineers associated with pavement design and road maintenance need to understand the cause of fatigue failure of a flexible pavement subjected to repeated wheel loads and the elements influencing MR of subgrade soils. Most pavement engineering or engineering mechanics textbooks will have a similar definition for resilient modulus. Generally, it is described as the ratio of applied deviator stress to recoverable or resilient strain. Researchers such as Drumm et al. (1990), Zumrawi and Awad (2017) and Soliman and Shalby (2010) have studied the characteristics of various subgrade soils and attempted to relate it to engineering properties and stress state. In order to reduce the complexities related to direct laboratory determination of resilient modulus they have developed different mathematical models based on other properties of soils.

2.2 Resilient modulus

Resilient modulus is a measure of the elastic behavior of subgrade soil under traffic loading. Resilient modulus is a stiffness measurement, it does not measure the strength of a material. The design thickness of a pavement structure and its predicted performance are highly dependent on subgrade modulus. Characterization of subgrade resilient modulus involves conducting advanced repeated loading triaxial testing that requires special equipment and technical experience that are not available in many soil laboratories. Resilient modulus is the primary design input for subgrade soil in AASHTO design method or mechanistic-empirical method of design. Many studies show that MR can be obtained from index properties of soil and unconfined compressive test (Soliman and Shalby, 2010; Kim, 2004; Zumrawi and Awad, 2017). AASHTO recommends the determination of modulus of resilient from CBR to characterize subgrade soils. But CBR is a measure of shear strength, which is not necessarily expected correlate with a measure of stiffness or elastic modulus. CBR based relation fails to consider stress dependence of modulus of resilience. there is a limited correlation between CBR and resilient modulus for soils, thus CBR method has limit utility (Drumm et al, 1990 quoting Robnett and Thompson 1976).

2.3 Factors Affecting Resilient Modulus

The resilient response of a soil has been studied and documented by several researchers over the past years. Drumm, et al 1990; Kim, 2004 studied and evaluated the characteristics of resilient modulus of subgrade soils in association with the stress and engineering properties, and developed procedures for estimating resilient modulus. Different studies by different authors shows that modulus of resilient of soils depends on deviator stress, confining stress, water content, degree of saturation, index properties (Soliman and Shalby, 2010; Kim, 2004; Zumrawi and Awad, 2017). Among the above-mentioned factors deviatoric stress, water content, dry density and index properties are the major ones for fine grained soils whereas for coarse grained soils both confining pressure and deviatoric stress affect the resilient modulus.

2.3.1 Moisture content

Most of the time at the construction stage pavements are compacted at 95% percent of laboratory maximum dry density. After the end construction stage due to environmental and traffic factors the moisture content as well the dry density of the compacted pavement layer changes. Both in laboratory and in situ conditions the resilient response of cohesive soils highly affected by the moisture content of the soil. Zumrawi and Awad (2017); Richard et al. (2014) found that decrease in resilient modulus both below and above the optimum moisture content. As the moisture content increases and saturation is approached, positive pore pressure may develop under rapid applied loads which may cause a reduction in effective stress and permanent deformation resistance of soil (Zumrawi and Awad, 2017).

Soliman and Shalaby (2010) after studying six samples taken from different parts of Manitoba, Canada observed that the moisture sensitivity of resilient modulus. The degree of sensitivity of resilient modulus to moisture content variation is not the same for different soils. For high plastic clay and sandy clay soils the sensitivity is higher whereas sandy silt/silty sand soils showed less sensitivity to moisture content. In general, in all soil types it is observed that a decrease in resilient modulus due to increase in moisture content. Soliman and Shalaby (2010) used straight line to show pattern of decrease in modulus of resilient value due to increase in moisture content, but the actual decline is resilient modulus in relation the moisture content was not straight line. Similarly, Drumm et al. (1997) showed the effect of moisture content on the resilient modulus by conducting tests on 11 soil samples taken from Tennessee. The result showed that as the degree of saturation increases the resilient modulus decreases tremendously. However, the magnitude of decrease in resilient modulus was found to depend on the soil type. Moisture content of subgrade is the major factor for prediction of the resilient modulus of cohesive soils (Zainorabidin and Agustina, 2018).

2.3.2 Deviatoric stress

Resilient modulus of fine-grained soils decreases as the deviator stress increases (Drumm, et al., 1990; Kim, 2004). Resilient modulus for cohesive soils steeply decreases with the amplitude of cyclic load up to a deviator stress, called the break point (Kim, 2004 quoting Thomson and Robnett, 1976). Rahim (2005) after studying both fine and course (sand) grained soil concluded that the deviatoric stress significantly affects the resilient modulus of fine-grained soil but for that of coarse-grained soils both the deviatoric and confining pressure are influencing factors on the resilient modulus result. Khasawneh (2017) after conducting resilient modulus test on two fine grained soils namely A-4 and A-6 soils stated that as the moisture content and deviatoric stress increases the value of resilient modulus decreases. The deviatoric stress model for fine-grained cohesive soil does not picture the influence of confining pressure on resilient modulus accurately. Still, this aspect is considered insignificant since the cohesive soils derive their overall strength mainly from cohesion rather than from frictional characteristics. This kind of modeling is perhaps adequate for cohesive soils found at shallower depths. But if the traffic load is higher and the soil sample under consideration is at greater depth it is necessary to consider confining pressure in the deviatoric stress model.

2.3.3 Confining stress

Kim (2004) showed that the effect of confining stress on resilient modulus of cohesive soils, it gradually decreases with an increase in the moisture content. However, other researcher has suggested that the confining stress around cohesive soils has no significant effect on the resilient modulus (Kim, 2004 quoting Fredlund, et al., 1977). Ooi et. al (2004) stated that resilient modulus of resilient value of granular materials is affected by confining and deviatoric stresses, or the stress state, but the effect of confining stress on resilient modulus of cohesive soils is negligible. Rahim (2005) also stated that the influence of confining pressure on resilient modulus of fine-grained soil is insignificant. But for coarse grained soils the resilient modulus increased with an increase in confining pressure. Khasawneh (2017) after conducting resilient modulus test on A-4 and A-6 at OMC and wet of OMC, concluded that the effect of confining pressure on the resilient modulus of fine-grained soil is insignificant. That is the increase in resilient modulus of the soil due to the presence of confining pressure very much less.

2.3.4 Index properties

The plasticity index and percent of fine particles (P_{200}) affects resilient modulus. Low clay content and high silt content results in lower resilient modulus values (Zumrawi and Awad, 2017). Rahim (2005) studied the attribute of index properties of soil on resilient modulus of subgrade soil. The researcher concluded that for fine grained soils resilient modulus is a function of liquid limit, water content, dry density and percent passing the No. 200 sieve. Soliman and Shalby (2010) stated the dependence of resilient modulus of subgrade soils on grain size distribution. Murthy (2003) classified fine grained soils based their plasticity index, stating that for a certain range of plasticity index value the soil could be non-plastic, low-plastic, medium plastic or highly plastic soil (see table 2.1).

Table 2.1 Classification of soils based on plasticity index (Murthy, 2003)

Plasticity index	Degree of Plasticity
0	Non-plastic
<7	Low plastic
7-17	Medium plastic
>17	Highly plastic

2.4 Existing models

The resilient modulus is highly dependent upon the magnitude of the applied cyclic deviator stress. The requirement specialized equipment makes determination of resilient modulus complex and difficult. Therefore, approximate methods are often used to obtain an estimated value

AASHTO method relates resilient modulus with California bearing ratio in the form the equation 2.1.

$$MR = 1500CBR \qquad \text{Equation 2.1}$$

Where MR = resilient modulus (psi)

CBR = California bearing ratio (%)

Drumm et al (1990) proposed resilient modulus model from unconfined compression test. The model was developed by conducting unconfined compressive test on 11 soil samples taken from Tennessee. The researchers assumed that the initial modulus obtained from the unconfined compression test, may be a better measure of MR than quantities related to shear strength. The model assumes a hyperbolic relationship between stress and strain developed in unconfined compression test.

$$MR = \frac{a' + b'\sigma d}{\sigma d} \; for \; _{\sigma d} > 0 \qquad \text{Equation 2.2}$$

Where MR = resilient modulus in ksi

$a' = 318.2 + 0.337(qu) + 0.73(\%\ clay) + 2.26\ (PI) - 0.915\ (\gamma) - 2.19(S) - 0.304\ (\%finer\ \#200)$ Equation 2.3

with a coefficient of determination, $R^2 = 0.81$

$b' = 2.1 + 0.00039(1/a) + 0.104(qu) + 0.09(LL) - 0.1(\%finer\ \#200)$ Equation 2.4

with a coefficient of determination, $R^2 = 0.73$,

σd = deviatoric stress in psi

% clay = percent soil particles finer than 0.002mm (%)

PI = plasticity index,

γ = dry unit weight in pcf

S = degree of saturation (%)

% finer # 200 = percent soil particles finer than 0.075mm (%)

a = material parameter for hyperbolic representation of stress -strain response, and

LL = liquid limit

The material parameters a can be obtained from the unconfined compression test as the intercept of the linear relation by plotting the stress – strain data in the transformed coordinates ε/σ and ε as shown in figure 2.1. The model requires unconfined compression test, liquid and plastic limit tests, sieve analysis, and the measurement of moisture content and dry density for the specimen.

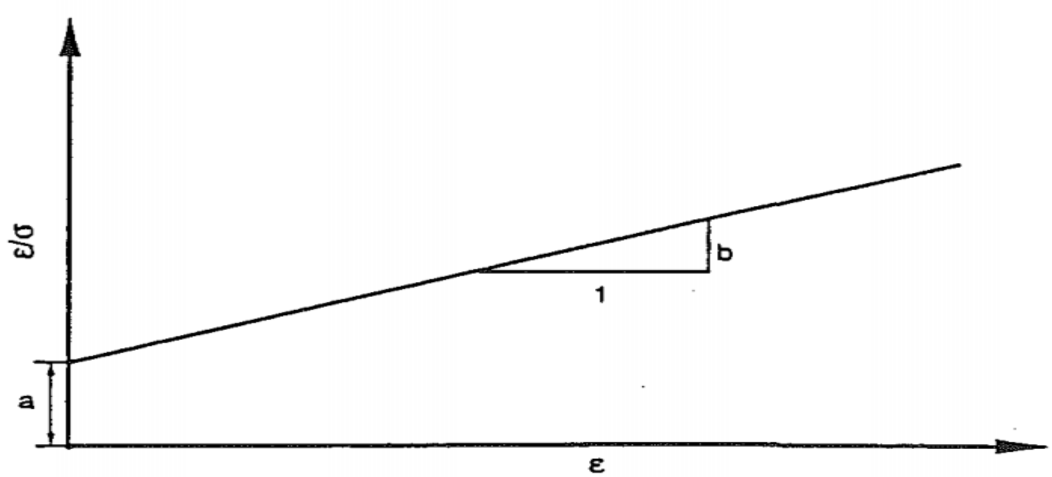


Figure 2.1 transformed coordinates for determination of hyperbolic parameters a and b (Drumm, 1990)

Lee et al. (1997) developed a simple relationship between unconfined compressive strength and the resilient modulus for cohesive soils. Three clayey soils from Indiana were investigate.

According to AASHTO soil classification system the soils were A-4, A-6 and A-7-6. Two representative stress states were selected deviatoric stress of 6psi and a confining pressure of 3psi for comparison purpose. The correlation below was developed between MR and $S_{u1\%}$ independent of actual moisture content and /or compaction density: -

$$MR = 695.4(S_{u1\%}) - 5.93(S_{u1\%})^2 \qquad \text{Equation 2.5}$$

with a coefficient of determination $R^2 = 0.97$

Hossain and Kim (2014) developed model of resilient modulus for subgrade soils found in Virginia. The model is based on stresses at 1.0% strain from UC test. the researchers developed the model 29 soil samples taken from Virginia and soils used for under their study were A-4 to A-7. The researchers conducted the resilient modulus test at cyclic deviatoric stress of 6psi and confining pressure 2psi and the correlation with that of the stress at 1% strain shows value of $R^2 = 0.97$.

$$MR\ (Psi) = 657S_{u1\%} - 6.75(S_{u1\%})^2 \qquad \text{Equation 2.6}$$

On this review part of the paper it discussed that the most influencing factors on the determination of resilient modulus. In general, among these factors deviatoric stress, water content and index properties are the major factors that affect resilient modulus of fine-grained soils whereas for coarse-grained soils both deviatoric stress and confining pressures are the main influencing factors. In this paper, the researcher developed similar model of resilient modulus for subgrade soil with that has been discussed in the previous section. Since all the above models where developed outside of Ethiopia, they do not satisfy the local condition because weathering process of the soil as well as the environmental condition is different. thus, the currently developed model simulates the local condition. the study also fills the gaps in characterizing of subgrade soils using CBR by showing the possible ways of using stress dependent parameters.

3 MATERIALS AND METHODS

3.1 Introduction

This chapter shows both materials used and methods followed in this study to reach at each specific objective. Particularly, the chapter discusses about description of study area, research design, sample size and selection, sampling and sampling techniques and procedures, source of data, data collection, data quality control, procedure of data collection, data analysis and ethical consideration.

3.2 Description of the study area

The location of the study area is part of Arbaminch-konso trunk road. This road is part of Addis Ababa –Hossana-arbamich-konso- yabello road which provides alternative road for Addis Ababa – Moyale road and it covers a total distance of 70KM. The specific location of the study area begins at wezeka which is located at 540km from the capital Addis Ababa and it ends at konso town which is located at 570km from the capital Addis Ababa.

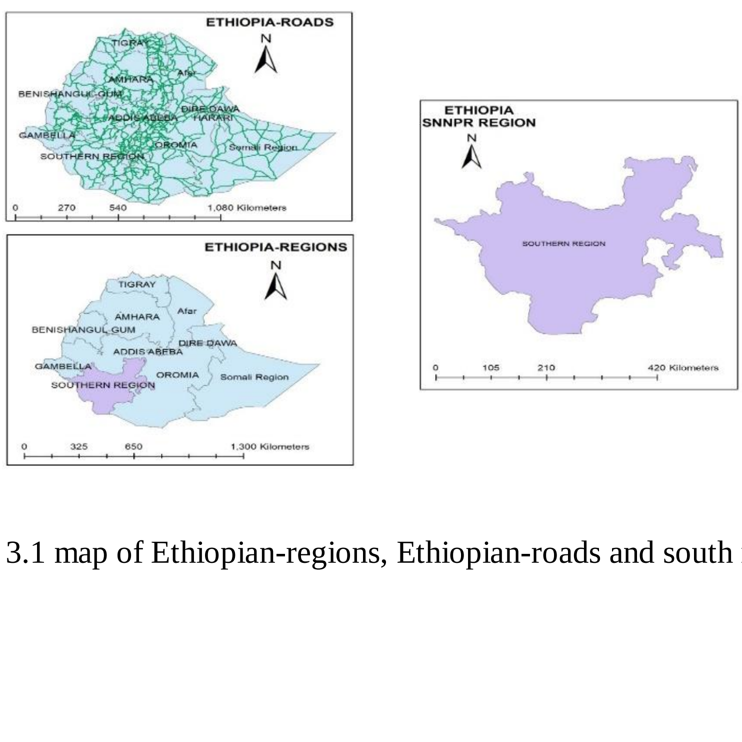


Figure 3.1 map of Ethiopian-regions, Ethiopian-roads and south region (GIS)

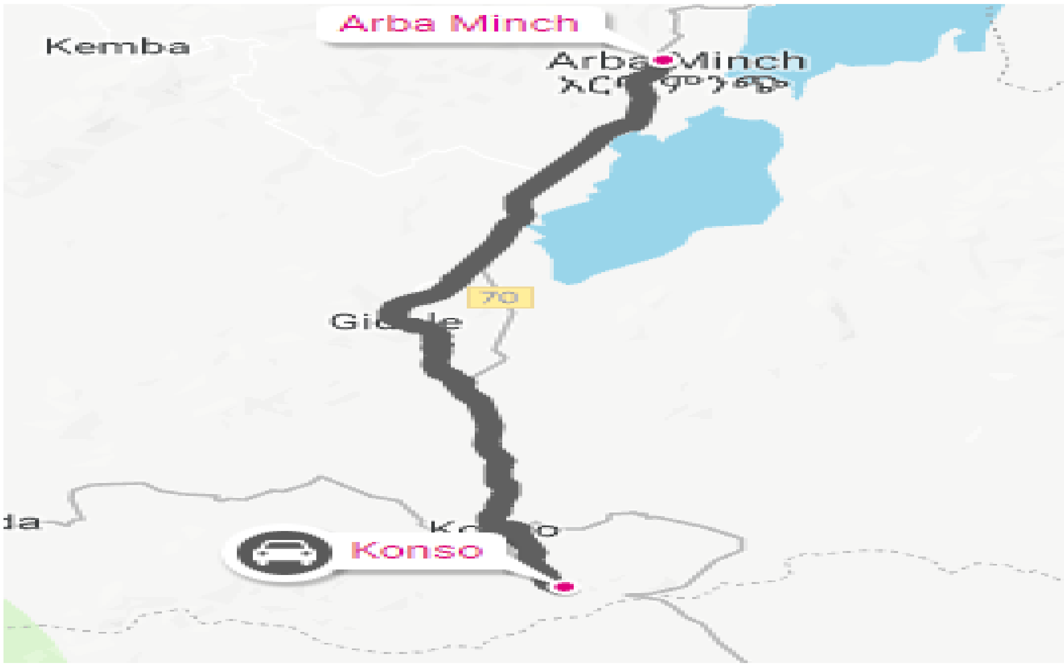


Figure 3.2 map of the study area (google map, 2018)

3.3 Study type and Research design

The study type is both experimental (soil laboratory test results) and analytical (analyzing the data obtained from laboratory tests numerically). Experiments are conducted in laboratory on disturbed samples collected from the field. Moreover, the experiments conducted are those tests relating the independent and dependent variables. Analysis was carried on the relation between the dependent and independent variables. Factors affecting resilient modulus and its complexity in determine from the repeated load triaxial test is mentioned in the literature part of this paper. Additionally, the current subgrade soil characterization technique which is the CBR method has its own draw back. The research design is drawn in relation to the procedures to follow in such a way that appropriate data can be gathered for analyzing and evaluation of the research problem and its forwarded questions. The flow chart for the research design of this study is well described in figure 3.3.

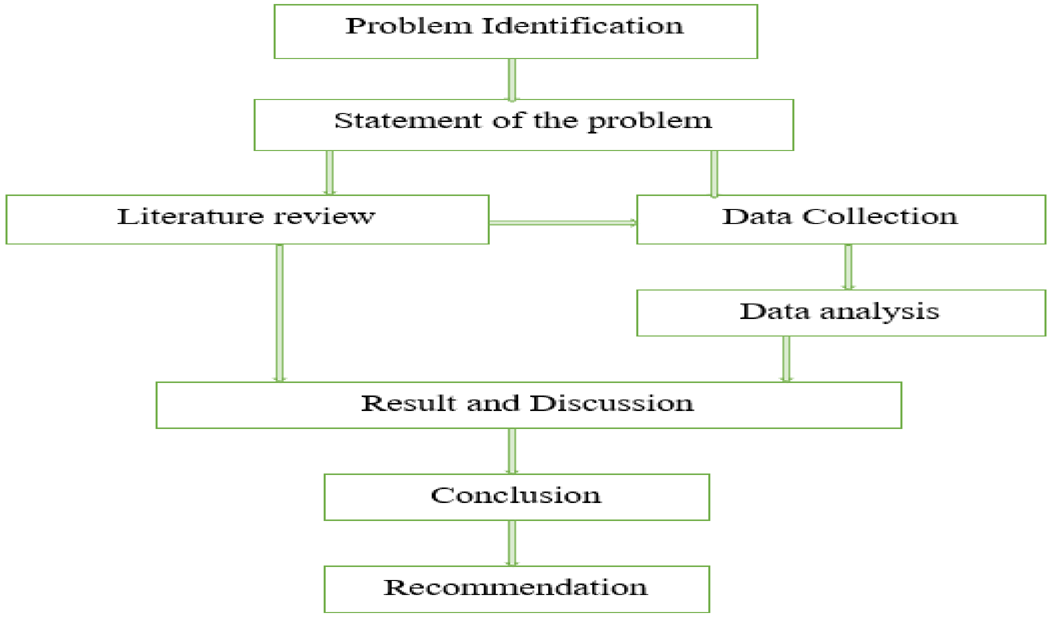


Figure 3.3 research design flow chart

3.4 Sampling and sampling technique

Since the study was basically carried out on cohesive soils, to collect those cohesive soil samples the study followed a non-probabilistic purposive sampling technique. The samples used in this study for conducting laboratory tests are basically disturbed samples. The samples were collected from 14 different location along Wezeka to Konso road. The tests were conducted as the procedures stated in AASHTO soil testing standards. In the literature review part of this study, it is discussed about those factors influencing the resilient response of cohesive subgrade soils. Based on this the independent variables (soil physical properties as well as stress states) that affect the resilient response of the subgrade soil were identified.

- Independent Variables includes liquid limit, plastic limit, OMC, MDD and P_{200}
- Dependent variable is modulus of resilient

3.5 Instruments and Software

Basically, the triaxial and unconfined compressive strength test equipment was used to characterize the resilient response of the subgrade soil under this study. In addition to the above equipment, other soil testing equipment used for preliminary characterization and sampling equipment are used. To collect soil sample from the field and testing in laboratory the following instruments are used; shovel, plastic bags, manuals for soil testing and necessary laboratory equipment that are required for determination of index properties of soils are used. This laboratory equipment includes liquid limit testing equipment, sieving equipment, compaction equipment. For data analysis and publication both Microsoft word and excel are used for this study.

Repeated load triaxial testing equipment

Resilient modulus is a major material property used to characterize unbound/subgrade pavement materials. Resilient modulus is a measure of stiffness of materials. It provides a way of analyzing stiffness of materials under different conditions, such as moisture content, density and stress level. The equipment used for resilient modulus testing is triaxial machine. The equipment contains: - triaxial pressure chamber, loading device (either a top-loading, closed loop, electrohydraulic, or electropneumatic testing machine with a function generator), axial deformation measuring equipment, specimen preparation equipment.

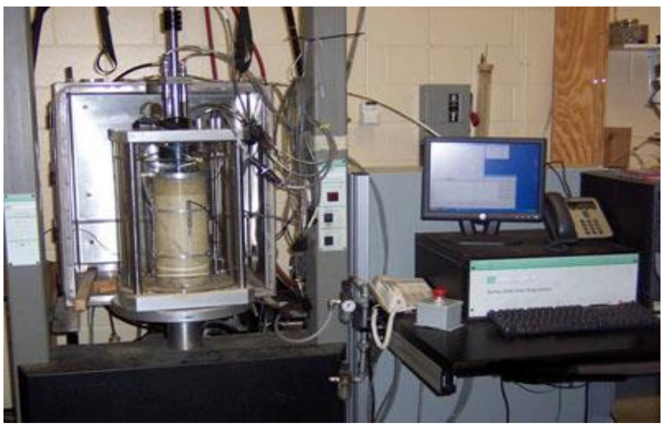


Figure 3.4 Repeated load triaxial equipment

Unconfined compressive strength testing equipment

The unconfined compressive test is special type of triaxial test in which the confining pressure in all direction is zero. The test is always conducted on cohesive soil. The unconfined compressive strength test is the quickest test to determine the shear strength of cohesive soils as well as the modulus of elasticity of the soil from the stress-strain relationship (Murthy, 2003). The slope of the strain-stress shows the modulus of elasticity of the material under static loading. Modulus of elasticity shows the recoverable strain after removal the applied of static load. The modulus of elasticity is the capacity of soils not to deform excessively during loading. It shows stiffness of the material not the strength of the material. figure 3.5 shows the required test equipment for determination of unconfined compressive strength of soils.

Figure 3.5 Unconfined compressive testing equipment (Reddy, 2002)

Compaction equipment

Compaction is a densification of soil mass by application of dynamic load. Compaction characteristics of soil influences the strength of soils. The maximum density and optimum density of soil is obtained in laboratory by conducting standard proctor test or modified proctor compaction test. The usual practice in the field is compacting the soil in the field to get 90-95% dry density of laboratory depending on the specification described. In this study modified compaction equipment is used to compact the soil. Apparatus required for this test are cylindrical metal mold with 15.3cm internal diameter and length 11.6cm with its accessories, rammer of weight 4.5Kg and balance.

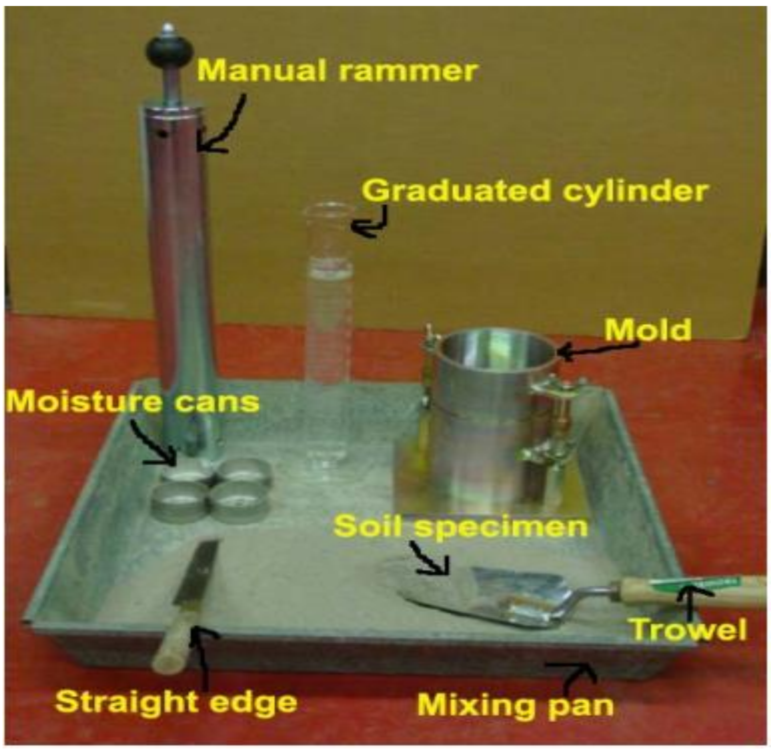


Figure 3.6 Compaction equipment (Reddy, 2002)

Atterberg limits and gradation test equipment

Atterberg limits are moisture contents which show the change in the soil state with increase in moisture content. The Atterberg limits of soils are liquid limit and plastic limits, the difference of these two limits is known as plasticity index. Plasticity index shows the plasticity characteristics of fine-grained soils. Liquid limit of a soil under this study is determined by Casagrande apparatus and the plastic limit is determined by using frictionless glass plate.

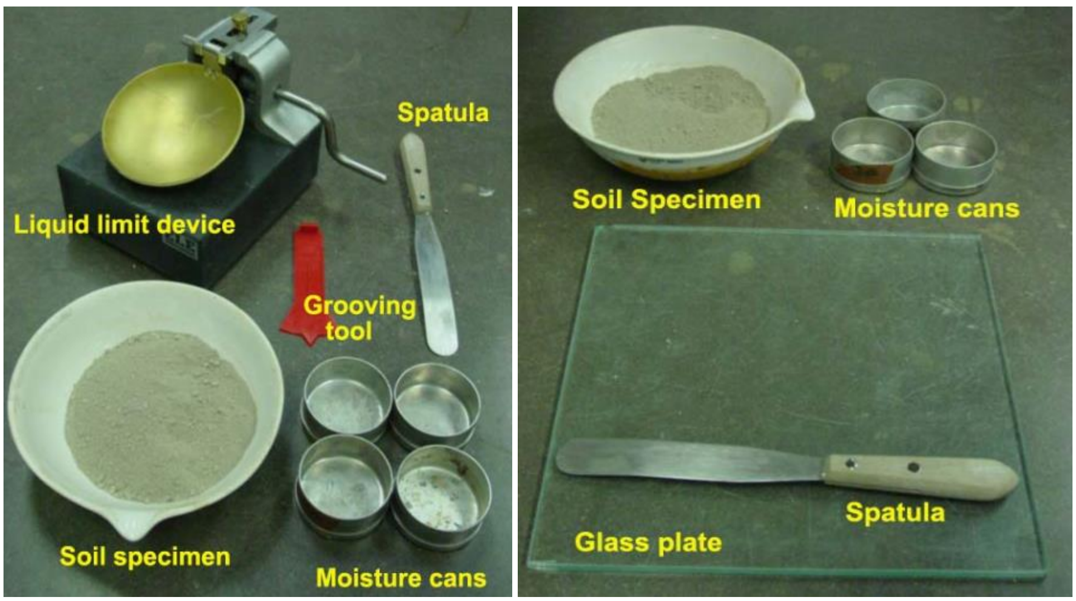


Figure 3.7 Liquid limit and plastic limit apparatus (Reddy, 2002)

The grain size distribution of the soil is determined using both sieve of different size openings and hydrometer the apparatus requirements are shown in figure 3.8.

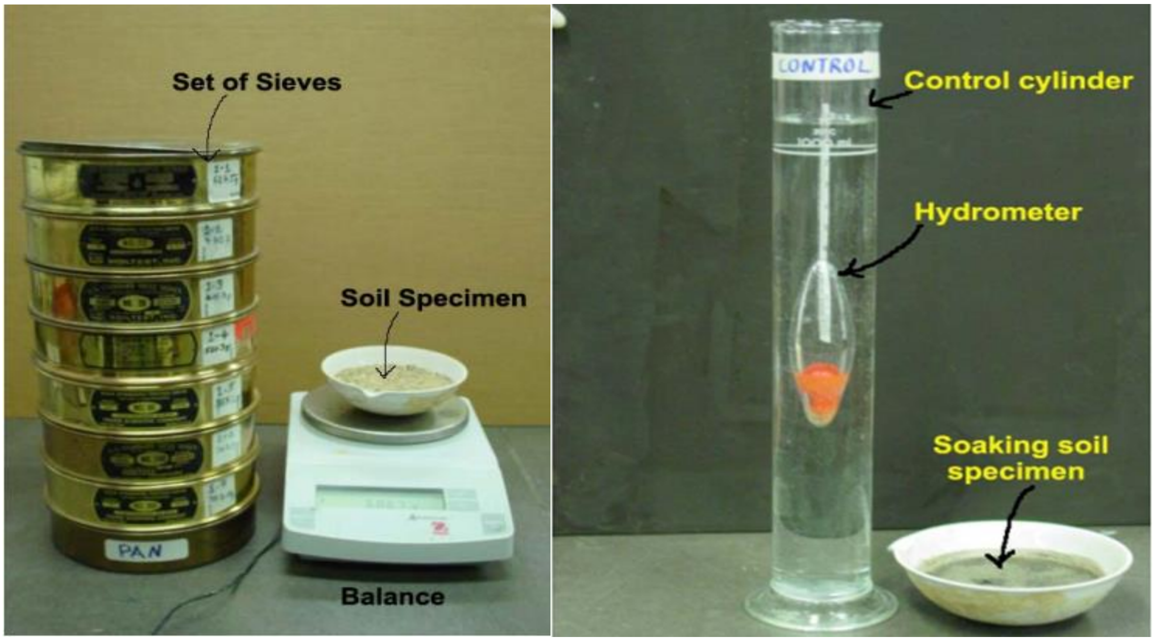


Figure 3.8 Sieving and hydrometer apparatus (Reddy, 2002)

3.6 Method of data collection

The source of data which is used in this study were both primary and secondary data. The primary data is the laboratory test result of soil sample which is taken from the study area. The secondary data includes literatures written on the similar topics with this study i.e. journals, books, dissertations and conference proceedings.

Disturbed soil samples were collected from the study area from 14 locations. Purposely the selected samples were cohesive soils. The samples were collected from Arbaminch konso road, wezeka to Konso road section, Ethiopia. The samples were collected at depth ranging from about 0.5m-1m below existing ground level, the depth declared to be the subgrade level in each section (ERA, 2013). Optimum moisture content, maximum dry density, particle size distribution, liquid limit, plastic limit, shrinkage limit, plasticity index and unconfined compressive strength tests are conducted in laboratory per AASHTO soil testing manual.

Laboratory experimentation

- **Resilient modulus test**

Resilient modulus is elastic behaviour of unbound/subgrade materials. Mathematically it is expressed as the ratio of applied deviatoric stress to recoverable strain. The determination of MR is usually conducted using repeated load triaxial testing equipment. MR shows the recoverable deformation of materials under a certain dynamic loading. In this study the test is conducted as the procedures indicated in AASHTO T-307 testing procedures. The samples were prepared at predetermined optimum moisture level and at maximum dry density per AASHTO T180-95. The specimens were prepared using mold diameter of 71mm and 147mm length. The specimen is compacted in three layer each layer receiving 25 blows.

- **Unconfined Compressive Strength test**

The unconfined compressive strength test is an unconsolidated and undrained load test performed for determining the unconfined compressive strength of cohesive soil samples where the lateral confining pressure is equal to zero during the test. UCS is a compressive stress at which an unconfined cylindrical soil sample fails in load test. UCS is taken as the maximum load attained per unit area during loading. For UCS test specimen's height to diameter ratio shall be between 2 and 2.5, the shear strength was calculated to be half of the compressive stress at failure. At first the optimum moisture content and maximum dry density was determined as per AASHTO T180-95 (moisture-density relation testing for soils). Using the maximum dry density and optimum moisture data specimen is prepared for test. Mold size of diameter 38mm and height 76mm is used. For each sample three specimens are prepared with the same sample condition and tested. The test is conducted as a procedure stated in the AASHTO T208-92 (unconfined compressive strength of soils testing).

- **Determination index properties**

Index properties of soils include Atterberg's limits (AASHTO T-89 and T-90) and grain size distribution (AASHTO T- 27). Most studies show that plasticity index and percent of clay within soil affects the resilient modulus of soils. With this regard laboratory tests like particle size distribution, Atterberg's limits, dry density and moisture content test was conducted on disturbed samples taken from the study area.

3.7 Method of data analysis

- **Determination of the physical properties of the soil and classification the soil as per AASHTO system**

After conducting laboratory tests on different soil properties like atterberg limits, dry density and moisture content, grain size distribution then excel sheet is used to analyze the data, using this result comparison has been carried out with different literatures. To classify the soil AASHTO classification is used. Initially the system was proposed in 1928 by the U.S. Bureau of Public Roads for use by highway engineers. Currently, in many parts of the world the method is in use. The system classifies inorganic soils into seven groups, A-1 to A-7 with 12 subgroups in all. For the classification the system uses three soil properties namely particle-size distribution, liquid limit and plasticity index. Additional parameter known as group index is introduced to further differentiate soils containing appreciable fine-grained materials. The group index is a measure of suitability of soil as a subgrade material. The higher the value of GI the poorer the material is as subgrade soil. Use GI equal to zero if the calculated value is less than zero but there is no upper limit value for it (Murthy, 2003).

$$\text{Group Index (GI)} = 0.2a + 0.005ac + 0.01bd \qquad \text{Equation 3.1}$$

a = that portion of percentage of soil particles passing No. 200 sieve greater than 35 = (F-35)

b = that portion of percentage of soil particles passing No.200 sieve, greater than 15 = (F-15).

c = that portion of the liquid limit greater than 40 = (LL – 40)

d = that portion of the plasticity index greater than 10 = (PI – 10)

F = percent passing No. 200 sieve. If (F < 35), use (F-35) = 0

Classification procedure: - With the required data in mind, go from left to right in the chart. The correct group will be obtained by a process of elimination. The first group from the left consistent with the test data is the correct classification. The A-7 is subdivided into A-7-5 or A-7-6 depending on the plasticity index PI.

For A-7-5, PI ≤ LL – 30

For A-7-6, PI > LL-30

General classification	Granular Materials (35 percent or less of total sample passing No. 200)							Silt-clay Materials (More than 35 percent of total sample passing No. 200)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-*a*	*A*-1-*b*		*A*-2-4	*A*-2-5	*A*-2-6	*A*-2-7				*A*-7-5 *A*-7-6
Sieve analysis percent passing											
No. 10	50 max										
No. 40	30 max	50 max	51 min								
No. 200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 max	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	Stone fragments—gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

Figure 3.9 AASHTO soil classification system (Murthy, 2003)

- **Modulus of Resilience from Unconfined Compressive Strength**

As stated in the literature review part of this research paper, different scholars showed method of determination of resilient modulus of cohesive soils from unconfined compressive strength test (Kim, 2004; Drumm et al., 1990; Soliman and Shalaby, 2010). Additionally, these researchers verified that the MR obtained from unconfined compressive test is highly correlated to MR obtained from repeated load triaxial test. The slope of stress-strain curve that is found from the unconfined compressive strength test is known as elastic modulus. This elastic modulus is determined under static loading condition. it does not show the response of the material under dynamic loading condition. whereas the modulus of resilient is a measure of stiffness of the soil under confinement and repeated loading. MR is usually the elastic modulus used in mechanistic analysis of pavement structures. It is obtained from repeated load cyclic triaxial tests and widely used to simulate the behaviour of subgrade soils under traffic loading. Due to its complexity of testing resilient modulus directly, researchers are providing alternative method of determination. researchers like Drumm et al. (1990) stated that the initial modulus obtained from the graph of stress-strain of unconfined compressive strength test highly represents the resilient modulus. But Hossain and Kim (2014) after comparing resilient modulus of cohesive soils with that of estimated resilient modulus from the initial tangent of unconfined compressive strength test concluded that the correlation between the measured and the estimated values have week correlation. The other researcher Lee et al. (1997) stated that resilient modulus of cohesive soils is highly correlated to the stresses at 1% strain ($S_{u1\%}$) obtained from the unconfined compressive strength than the initial modulus concept stated by Drumm et al 1990. This idea is supported by Hossain and Kim (2014). Hossain and Kim developed similar model with that of Lee et al. (1997) and got higher correlation between measured resilient modulus and the estimated resilient modulus from ($S_{u1\%}$). Based on the research of the above-mentioned scholars, in this study it is aimed to estimate the resilient modulus for each sample using unconfined compressive strength. The unconfined compressive test and resilient modulus test was conducted on replicate samples. After that regression model is developed between measured resilient modulus and unconfined compressive strength. Then after the prediction trend line is drawn to study the variation between the measured and predicted value of resilient modulus. Additionally, model validation has been done using four samples collected from the same study area.

- **Analysis of modulus of resilient from unconfined compressive strength and index properties**

Index properties that influence the resilient modulus of the subgrade soils include physical condition of the soil (moisture content and unit weight), loading condition or stress state, soil type and its structure and index properties (Zumrawi and Awad, 2017). Studies show that MR can be obtained from index properties of soil and unconfined compressive test (Soliman and Ahmed, 2013; Kim, 2004 and Zumrawi & Awad, 2017). Using the above resilient modulus, unconfined compressive strength test result and the index properties of the soils, multi linear regression analysis (MLRA) model was developed. The developed model considered MR value obtained from repeated load triaxial test as the dependent variable and the rest of soil properties as independent variables. Since MR vary with stress level, the stress state of MR selected for analysis was a deviatoric stress of 41.4KPa and confining stress of 13.8KPa. most of the studies conducted on prediction of resilient modulus of cohesive soils, stated that inclusion of the index properties provides better correlation (Soliman and Shalby 2010; Rahim, 2005). Based on the nature of the variables the model proposed is multi linear regression model. Model verification is done by drawing a trend line between measure and predicted resilient modulus. Model validation is done using our samples collected from the same study area. MLRA is carried out using data analysis tool bar of Microsoft Excel in order to derive the relationship statistically as follows:

$$MR = f\,(qu, PL, MDD, w, P_{200})$$

The objective function for this research study is formulated as follows:

$$Y = b_0 + b_1x_1 + b_2x_2 + b3x_3 + b_4x_4 + \ldots\ldots\ldots b_nx_n$$

Where,

Y = modulus of resilience

b1, b2, b3, b4…... and b_n are constants

X_1, X_2, X_3, X_4 ....and X_n are a soil property (unconfined compressive strength, liquid limit, plasticity index, optimum moisture content and fine fractions) and unconfined compressive stress that are considered for the analysis.

The values of the above constants are solved using MLRA in the data analysis toolbar a built-in add-in for Microsoft Excel. So, by inserting the values of constants from laboratory results like: liquid limit, plasticity index, optimum moisture content and fine fractions, the values of constants are obtained.

3.8 Data Quality Assurance and management

The quality of the data is managed and assured by following standard procedures. Each test is conducted three times to check the accuracy and precision as well as to minimize personal error while testing. Tests are conducted on calibrated and verified laboratory equipment to avoid instrumental error. To manage the data guide line is prepared and careful data recording steps are followed.

4 RESULTS AND DISCUSSIONS

4.1 Introduction

The main objective of this study is to predict resilient modulus from unconfined compressive strength test. This chapter in general discusses about the results obtained from laboratory tests conducted on soils. Specifically, it deals about the physical properties of the soil such as the grain size distribution, specific gravity, Atterberg limits, optimum moisture content and maximum dry density and classification of the soil based on AASHTO soil classification system. Moreover, the chapter discusses on estimated resilient modulus from the unconfined compressive strength test. it also deals about multilinear model developed for resilient modulus of cohesive soil from index properties and unconfined compressive strength. At the end it presents the comparison made between the estimated resilient modulus estimated from the unconfined compressive test only and estimated resilient modulus from unconfined compressive test and index properties. This comparison shows the model strength due to the inclusion of the index properties in the model developed.

4.2 Physical properties of soil

The physical state of the soil affects the performance the subgrade. These physical properties include specific gravity, grain size distribution, atterberg limits, optimum moisture content and maximum dry density. Identification the physical properties of the soil helps for preliminary characterization of the subgrade soil.

4.2.1 Natural moisture content

The natural moisture content gives some idea about the property of the soil in the field. The moisture content of the soil sample is determined after keeping the sample in oven dry for 24 hours. The average natural moisture of the studied sample by the sampling is presented in the table 4.1 below. The test is conducted three times to minimize errors in estimating the moisture content.

Table 4.1 average moisture content of the samples for different pits

Test pit	1	2	3	4	5	6	7	8	9	10	11	12	13	14
mc (%)	13	21.1	9.4	19.7	17.9	16.9	17.9	15	14.3	15.6	6.9	15.5	8.8	14

4.2.2 Specific gravity

The specific gravity of fine-grained solids (Gs) is an important parameter in determination of soil physical properties such as void ratio, degree of saturation, in hydrometer analysis in consolidation and compaction analysis. It shows how a soil grain denser than water. Usually specific gravity is determined in laboratory using pycnometer/density bottle method. Compared to coarse grained soils determination of specific gravity of fine-grained soils is difficult due to entrapped of air in the soil sample. Specific gravity value for inorganic fine-grained soils like silt and clays ranges from 2.6-2.8 (Prakash et.al, 2012). In this study the specific gravity of the soil is determined using 200ml volume pycnometer. For different samples under this study an average value ranging from 2.59-2.72 is obtained as shown in the table 4.2. Only value of sample of test pit 13 is out of the usual value. Depending on the result specific gravity test, it concluded that the collected soil samples in this study are inorganic in nature.

Table 4.2 Specific gravity values

Test pit	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Specific gravity	2.68	2.72	2.70	2.64	2.60	2.66	2.61	2.67	2.65	2.70	2.61	2.71	2.59	2.62

4.2.3 Grain size distribution

The particle size distribution of a soil sample is an important factor in evaluation and classification of a soil material as highway subgrade. This is because most roads failures that result from geotechnical factors are due to grading characteristic of the subgrade material itself. For example, the shrinking of clay as a result of withdrawals of water and expansion of clay when there is ingress of water which in return causes damage on the pavement structure. In general, keeping other factors being constant, soils having a considerable fine fraction (clay and silt size particles) is likely to have poor geotechnical properties as highway subgrade or subbase than a soil largely made up of coarse particles (particles greater than 0.075mm). The laboratory test results show most of the samples used in this study are soils belonging in the fine-grained soils containing silt and clay in large contents. The values are summarized in table 4.3. all the soil samples are fine grained soils having particle size less than 0.075 micro meter. Compared to percent of clay size particles percent of silt size particles are larger in percent, but for the test pit 9 the percent of clay soil is greater than that of silt size soils.

Table 4.3 Summary of wet sieve analysis

Pit no.	% of gravel	% of sand	% passing sieve no. 200 (silt and clay)	% silt	% clay
1	0	42.2	57.7	14	43.7
2	0	35	65	35	30
3	0	35.3	64.7	46.7	18
4	4.4	13.8	81.8	53.8	28
5	12	29.4	58.6	29.1	29.5
6	10.9	16.6	72.5	58.5	14
7	4.8	28.8	66.4	34.4	32
8	7.4	20.9	71.7	47.7	24
9	11.8	39.3	48.9	18.9	30
10	6.7	22.3	71	45	26
11	3.7	39.3	57	29	28
12	3.4	50.9	45.7	21.7	24
13	2.9	37.8	59.3	35.3	24
14	5.7	19.3	75	39.1	35.9

4.2.4 Atterberg limits

Usually atterberg limits are moisture contents showing the boundaries where the soil acts like plastic or fluid material. The determination of these limits (plastic limit and liquid limit) is known as atterberg limit tests. These tests are used to know about the consistency of clay, and used for characterization of fine-grained soils together with other soil properties or individually.

The concept of atterberg limits helps to correlate these features with engineering properties such as compressibility, permeability, swell, shrinkage and shear strength (Abbas, 2018). For the determination of these limits soil sample passing sieve size 0.425 micro meter is used as stated in AASHTO manual. Casagrande apparatus is used for determination of the liquid limit. according to this test procedure the liquid limit of a soil is minimum water content at which a pat of soil specimen cut by a groove of standard dimension will flow together for a distance of 13 mm under an impact of 25 number of drops in the device. Since it is difficult to get the above-mentioned value by single trial a line is drawn to facilitate simple determination of liquid limit. For the determination of plastic limit, a frictionless glass plate is used. To accurately determine the plastic limit, the test is repeated three times and an average value is taken as shown in table 4.4. the range of liquid limit, plastic limit and plasticity index of soil sample were 24 - 53, 16 - 35 and 6 - 17 respectively. Instead of using single value of liquid limit or plastic limit to characterize soils the common practice is to use plasticity index which is the difference of liquid limit and plastic limit. plasticity index of shows the degree of plasticity of a soil. Murthy (2003) stated the plasticity characteristics of soils depending on plasticity. The ranges of plasticity index values used by Murthy is listed in table (see table 2.1). Based on table 2.1 most of the soil samples used in this study are medium plastic soils the remaining are low plastic soils. The soil samples obtained from test pit 1,2,3,4,5,6,7,9,10, 11, 12 and 13 are medium plastic soil whereas the soils obtained from test pit 8 and 14 are low plastic soils.

Table 4.4 Atterberg limit values for the collected samples

Pit no.	Liquid limit (LL)	Plastic limit (PL)	Plasticity index PI = LL-PL
1	29	21	8
2	37.5	30	7.5
3	46	34	12
4	53	34	17
5	44.8	35	9.8
6	24	17	7
7	28	18	10
8	28	22	6
9	36	25	11
10	29	22	7
11	27	20	7
12	27	18	9
13	27	16	11
14	35	29	6

4.2.5 Optimum moisture content and maximum dry density

The physical and mechanical behaviour of soils could be improved through compaction. The water content of a soil generally influences the properties of soils. At low moisture content the soil is stiff and difficult to compress. As the moisture content increases, the water acts as a lubricant making the soil workable. Due to this effect the soil gets denser when compacted under a certain level of compaction energy (Murthy, 2003). When a fine-grained soil is densified under a constant compactive effort but with varying moisture contents, a typical dry density versus water content relationship develops. among these relations the optimum and maximum dry density is selected for design of any soil related civil engineering projects (Li H., 2001). In this study laboratory determination of the OMC and MDD was carried by using modified proctor test. The values obtained are summarized in the table 4.5 and the group of the soil as per AASHTO is summarized in table 4.6. For the A-4 group the optimum moisture content ranged from 19.1 to 25.2% and this group has maximum dry density value that range from 1.44 to 1.71. For that of A-6 group soil the optimum moisture range is from 19.63 to 27.69 whereas the dry density for this group ranges from 1.37 to 1.67. for the A-7-5 soils the optimum moisture content and the maximum dry density ranges from 21.3 to 22.54 and 1.47 to 1.72

respectively. The remaining A-5 soil has moisture content of 30.56 and maximum dry density of 1.47. the lowest density is observed A-6 (sample taken from test pit 9). all the graphs drawn for determination of the OMC and MDD are shown in appendix I.

Table 4.5 OMC and MDD values of the soil sample

Test pit	Optimum moisture content OMC in %	Maximum dry density in g/cm^3
1	19.1	1.58
2	25.2	1.57
3	21.3	1.72
4	22.54	1.47
5	30.56	1.47
6	23.4	1.64
7	22.89	1.59
8	19.72	1.5
9	27.69	1.37
10	21.55	1.67
11	22.1	1.71
12	23.21	1.67
13	19.63	1.56
14	19.32	1.44

4.3 Classification of the soil as per AASHTO soil classification system

The classification of the soil is done using figure 3.9 as a guide line and the parameters indicated in table 4.3 and 4.4 as an input for the classification. As summarized in the table 4.6, eight of the soil samples used for this study are grouped under A-4 category indicating that these soils are silty soils. three of the soil samples are grouped under group A-6 and the other two are grouped under A-7-5. These categories show the soil samples are clayey soils as per AASHTO classification system. The remaining A-5 soil is also a silty soil.

From the group index soils having lower values i.e., near to zero are good as a subgrade soil whereas soils having higher value of GI are poorer as a subgrade material. From the table it could be observed that soils belonging the same group may have different GI. This is because the GI value is influenced by the liquid limit, limit and fraction of fines present within a soil sample. Soil samples taken from pit 1,2,6,7,8,11,12,14 are in group A-4 but this does not mean that they are equally suitable as subgrade soil. The difference in GI value indicates that the difference in the liquid limit and plasticity index as well as percent of fine fraction between the soil samples. Comparing this A-4 soils soil samples obtained from sampling pit 12 is higher in strength than the others due to the fact that it has the lowest value of GI. On the other hand, soil sample from sampling pit 2 has lowest strength compared to the other A-4 soils because it has highest GI value. Thus, based GI the soils could be put in order of decreasing strength as test pit 12,11,1, (2&7),8, (6&14) for the soils belonging to A-4 soil group. For the soils belonging to A-6 the decreasing order of strength is pit 9, 13 and 10. Comparing to the to A-7-5 soils the soil from test pit 3 has higher strength than pit 4. Since soils having GI value near to zero has higher strength, soil sample from pit 5 could be considered as an intermediate strength.

Table 4.6 Soil classification based on AASHTO system

Pit no.	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Soil group	A-4	A-4	A-7-5	A-7-5	A-5	A-4	A-4	A-4	A-6	A-6	A-4	A-4	A-6	A-4
Group index	5	6	9	17	5	8	6	7	3	7	4	2	5	8

4.4 Unconfined compressive strength test

A sample 38mm in diameter and 76mm in length was compacted at optimum moisture content and maximum dry density using a rammer. The specimen is compacted in predetermined density and moisture content as per AASHTO T180-95 test procedure. The compaction is carried out in five layers of equal height. The specimens were prepared in accordance with AASHTO T 208-92. The data collection from UCS test is not limited to only ultimate compressive strength. A complete diagram showing the stress-strain response of the sample were prepared. For the 14 sampling points the strain stress is drawn as shown in figure 4.1 through figure 4.7. The grain size for the preparation of each specimen is that passed sieve size

2.36mm. the ultimate unconfined compressive strength for the samples used were 154.1, 131.1, 114.5, 85.9, 113.1, 98.8, 99.9, 105.8, 88.7, 84.2, 76.2, 94.3, 88 and 108.4KPa respectively for pit 1-14. Among the samples taken for this study the maximum observed value of qu is qu = 154.1KPa. The minimum qu = 84.2KPa.

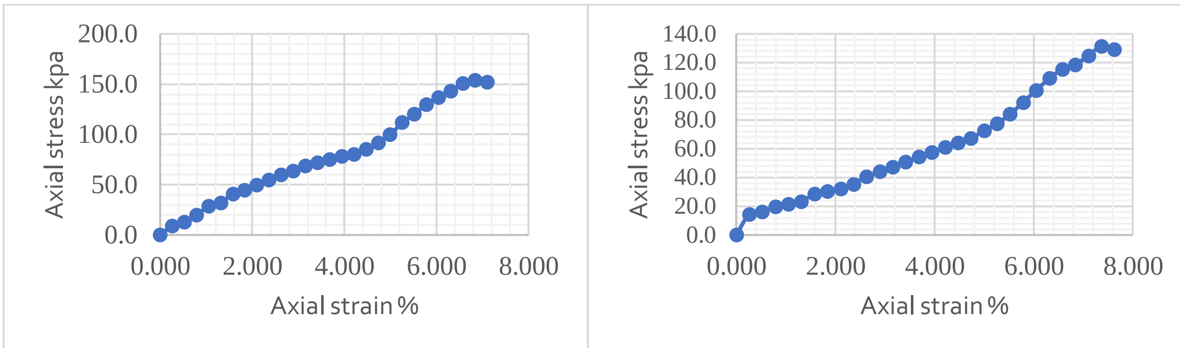


Figure 4.1 stress-strain graph of pit 1 & 2

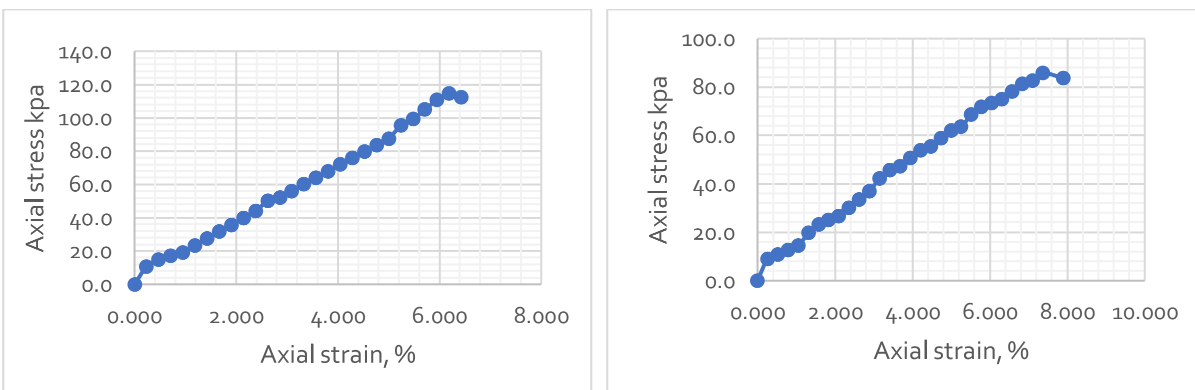


Figure 4.2 strain - stress curve of pit 3 & 4

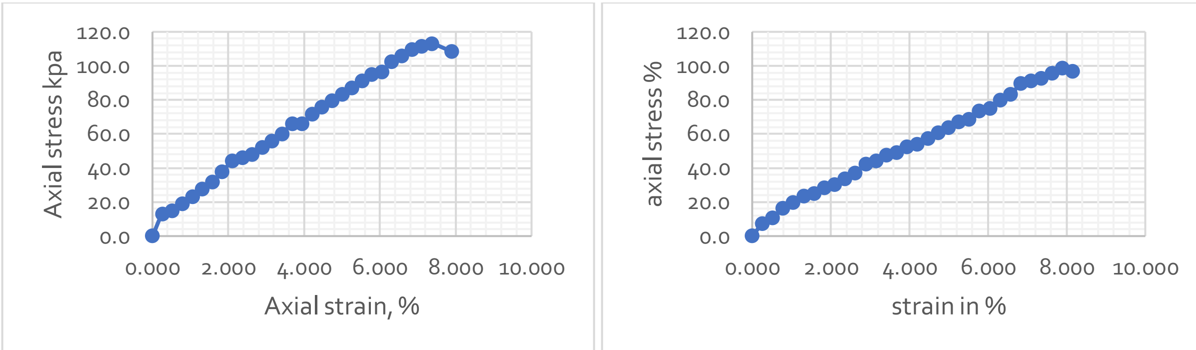


Figure 4.3 strain-stress curve of pit 5 & 6

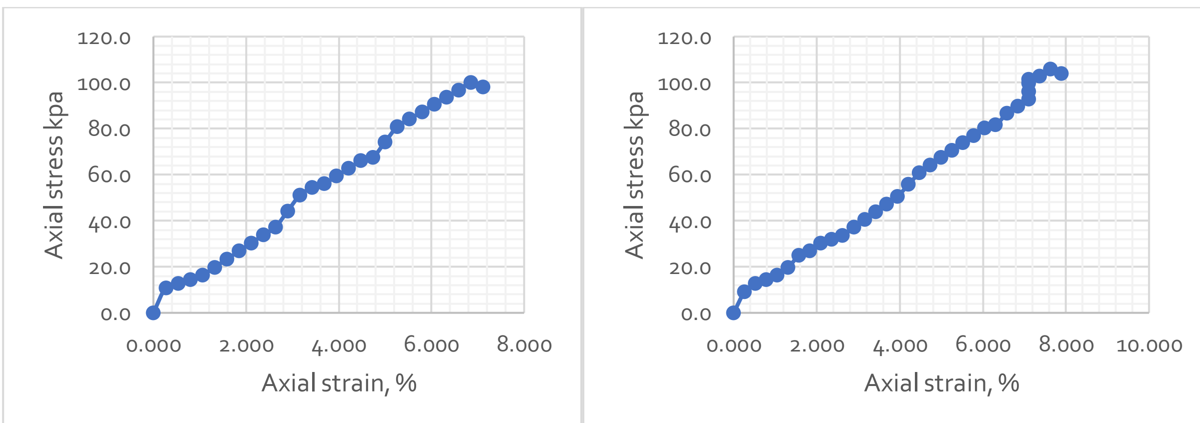


Figure 4.4 strain-stress curve of pit 7 & 8

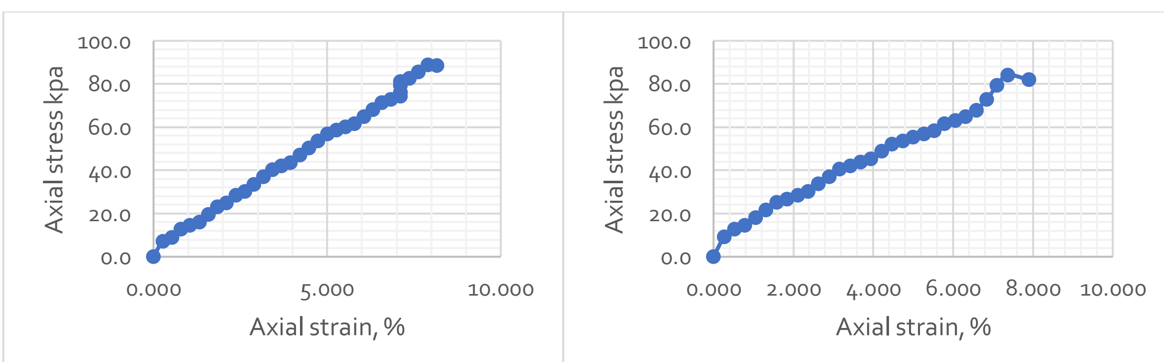


Figure 4.5 strain - stress curve of pit 9 & 10

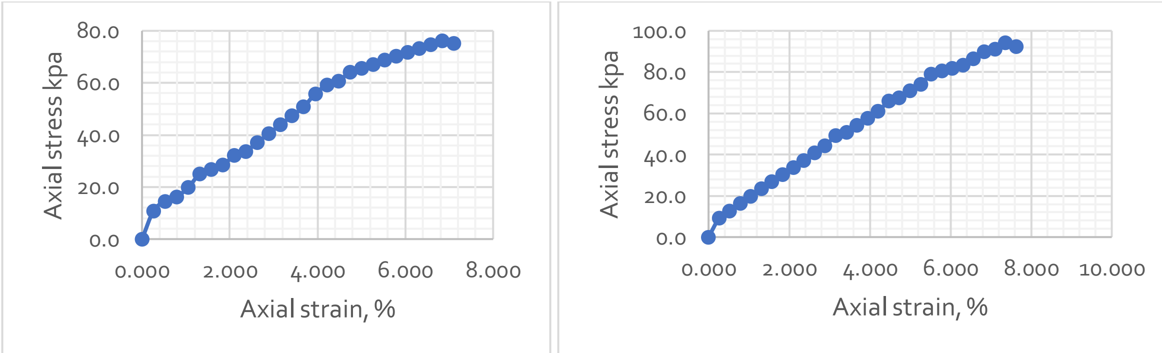


Figure 4.6 strain - stress curve of pit 11 & 12

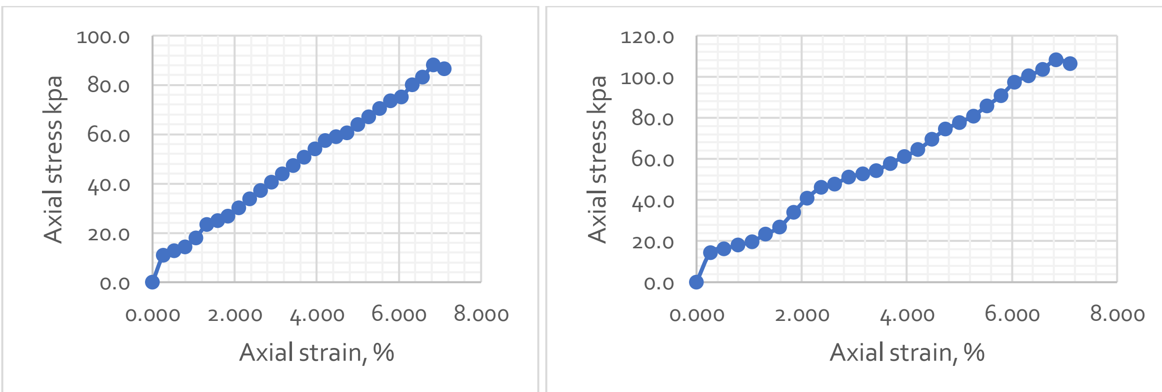


Figure 4.7 strain - stress curve of pit 13 & 14

4.5 Resilient modulus from unconfined compressive strength test

To develop the model between MR and unconfined compressive strength, MR was determined in laboratory with a replicate sample. The values at a confining stress of 13.8KPa and deviatoric stress of 41.4 KPa was used for this analysis. Using resilient modulus obtained from this stress levels and ultimate unconfined compressive strength from the unconfined compressive strength test regression model was developed. Measured resilient modulus at confining stress of 13.8KPa and deviatoric stress of 41.4KPa is listed in table (see table 4.7). For the prediction of the model the unconfined compressive strength measured from the unconfined compressive test is used. These values are 154.1, 131.1, 114.5, 85.9, 113.1, 98.8, 99.9, 105.8, 88.7, 84.2, 76.2, 94.3, 88 and 108.4KPa for pit one to fourteen respectively. To develop the regression model 71.43% of the sample used and the remaining 28.57% used for validation purpose. The result of linear regression analysis shows the following correlation between calculated resilient modulus and unconfined compressive strength value. The relation between the dependent variable (MR) and the independent variable (qu) is positive. The variables are directly related.

Table 4.7 Measured MR from repeated load triaxial test

Pit no.	MR (MPa)
1	17.4
2	14.4
3	14
4	8.3
5	13.4
6	10.9
7	10.1
8	10
9	8.5
10	10.9
11	12.1
12	11.2
13	10.6
14	12.2

$$MR = 0.122qu - 1.335 \quad \text{Equation 4.1}$$

with $R^2 = 0.85$ adjusted $R^2 = 0.83$ n= 10

Where MR = resilient modulus (MPa)

qu = unconfined compressive strength obtained from UCS test (KPa)

n = number of observations

R^2 = coefficient of determination

The output of the statistical analysis indicates that the model developed between calculated MR and unconfined compressive strength is significant and the relation between the two variables is good. The resulting statistical model was used to back-predict the MR of each soil sample in the study. The verification of the model is shown in the figure 4.8, the measured and predicted MR values are closer to the trend line indicating that the goodness of the predicted model. The variation of the predicted model and measured model may be attributed to the measurement made to get MR. The other reason for the variation is the sample preparation during the unconfined compressive strength test. the proposed model gives satisfactory prediction of resilient modulus and may be used as for flexible pavement design.

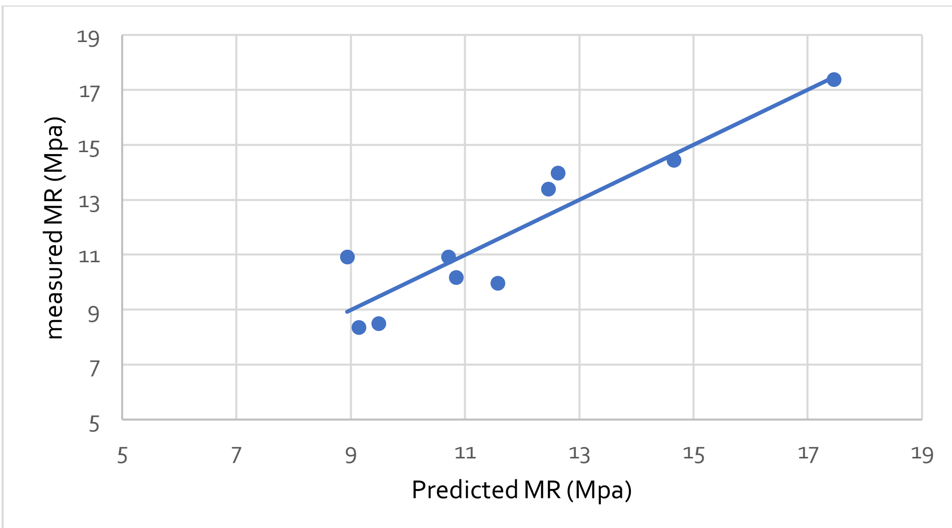


Figure 4.8 Measured MR vs. Predicted MR from qu

The validation of the model was done using the remaining 28.57% of the total data used. The minimum and the maximum error were 2.54% and 34.2% respectively. this variation may be attributed to error in sample preparation of the specimen for the UCS test. other reason this that MR test was not conducted on the sample thus the attribution of this factor may be significant. Thus, it is recommended to use the predicted MR value with safety factor. The result of the validation is summarized in table (see table 4.8).

Table 4.8 validation of the linear model between MR and qu

Pit no	Measured MR (MPa)	Predicted MR from the current model (MPa)	Error (MPa)	Error (%)
11	12.1	7.96	4.14	34.2
12	11.2	10.17	1.03	9.19
13	10.6	9.4	1.2	11.32
14	12.2	11.89	0.31	2.54

4.6 Resilient modulus from unconfined compressive strength test and index properties

For the multilinear analysis 71.43% of the sample observation is used to develop the model and the remaining 28.57% of the sample observation is used for validation of the model. The result of multilinear regression analysis shows correlation between measured resilient modulus as dependent variable and unconfined compressive strength, plasticity index, percent finer than sieve number 200, optimum moisture content and maximum dry density value as independent variables. Among the independent variables MDD is the most influencing factor of the predicted resilient modulus because it has highest coefficient. The correlation between MR and MDD is positive according to the model. The influence of OMC is also positive but as it can be seen from its coefficient the effect on MR is less compared to MDD. The relation between MR and qu is also positive meaning that they are directly related, as qu increases MR also increases. The coefficient PI is near to zero indicating that the effect on MR is less. But the sign confirms the direct relationship between MR and PI. This idea is agreeing with Kim (2004) which states that as the plasticity index increases MR also increases. the model indicates the indirect relationship between MR and P_{200}. This correlation satisfies the idea of both Kim (2004) and Drumm et. al. (1990) as the percent of fines passing sieve size 200 increases MR decreases. Figure 18 shows the measured and back calculated MR from the prediction model. It could be seen from the graph the predicted and measured MR values are close to one another. This indicates that the model best fits the situation. In table 4.9 measured MR value, independent variables used to develop the model and predicted MR are summarized.

Table 4.9 model variables, measured MR (MPa) and predicted MR (MPa)

Measured MR (MPa)	qu (KPa)	PI (%)	P_{200} (%)	OMC (%)	MDD (g/cm^3)	Predicted MR (MPa)
17.3628	154.1	8	57.7	19.1	1.58	17.8616
14.43174	131.1	7.5	65	25.2	1.57	15.5534
13.967	114.5	12	64.7	21.3	1.72	14.7764
8.345876	85.9	17	81.8	22.54	1.47	8.6376
13.37389	113.1	9.8	58.6	27	1.47	12.8894
10.89729	98.8	7	72.5	23.4	1.64	11.9138
10.16545	99.9	10	66.4	22.89	1.59	11.7715
9.949698	105.8	6	71.7	19.72	1.5	10.8286
8.47639	88.7	11	48.9	27.69	1.37	9.3681
10.89729	84.2	7	71	21.55	1.67	10.2729
12.1395	76.2	4	57	22.1	1.71	10.0932
11.19785	94.3	9	45.7	23.21	1.67	12.5397
10.55323	88	11	59.3	19.63	1.56	9.8661
12.22488	108.4	5	75	19.32	1.44	10.3364

$$MR = -17.85 + 0.12qu + 0.04PI - 0.03P_{200} + 0.13OMC + 10.22MDD \qquad \text{Equation 4.2}$$

With $R^2 = 0.94$ adjusted $R^2 = 0.87$ n = 10

Where MR = resilient modulus (MPa)

qu = unconfined compressive strength (KPa)

PI = plasticity index (%)

P_{200} = percent finer than sieve number 200 (%)

OMC = optimum moisture content (%)

MDD = maximum dry density (g/cm^3)

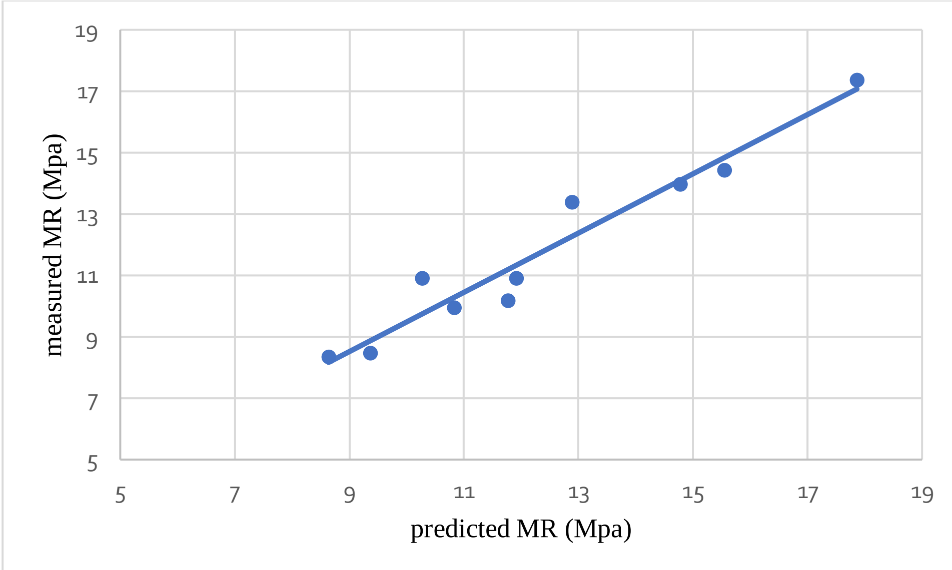


Figure 4.9 distribution of Measured MR and Predicted MR value

Using the remaining 28.57% the model validation is done as shown in table 4.10. Table 4.10 shows the measured MR and predicted MR from the current model. the minimum observed percent error is 6.89% and the maximum observed percent error is around 16.61%. for pit number 11 soil sample the predicted MR value is decreased by 16.61%. for the test pit number 12 sample the predicted MR is increased by 11.96%. for that of test pit number 13 the decrease in the predicted value of MR is 6.89%. the error for the test pit number 14 for the predicted MR value is 15.25% which shows the predicted MR is greater than measured by 15.25%. these all variation is attributed error caused during the sampling, specimen preparation and testing of the dependent and independent variables.

Table 4.10 Validation of predicted multilinear model

Pit no.	Measured MR (MPa)	Predicted MR from the current model (MPa)	Error (MPa)	Error (%)
11	12.1	10.09	2.01	16.61
12	11.2	12.54	-1.34	-11.96
13	10.6	9.87	0.73	6.89
14	12.2	10.34	1.86	15.25

5 CONCLUSIONS AND RECOMMENDATIONS

This chapter about the conclusion drawn based on the discussion carried out the result and discussion part of this paper and recommendation given for further study.

5.1 Conclusions

From the result and discussion part of this paper the following conclusions drawn.

The atterberg limit test conducted on the soil samples indicates that the soil samples were low to medium plastic soils. As per AASHTO soil classification the soil samples used in this study were A-4, A-5, A-6 and A-7-5 soils. A group index analysis of the soils shows that GI ranges from 2 to 17. The lowest GI shows good subgrade material and the higher being poor as subgrade material.

The resilient modulus of cohesive soils shows higher correlation with unconfined compressive strength (R^2 = 0.83). The regression analysis of resilient modulus of cohesive soil with unconfined compressive strength and index properties of the soils shows higher correlation value. The correlation value obtained from the analysis was R^2 = 0.87.

Comparing the two models developed due to the inclusion of the index properties in the second model the strongness of the model increased by 0.04.

5.2 Recommendations

- The developed prediction models are not for full characterization of fine-grained soils since the modulus of resilient test was conducted on replicate soil samples. Thus, it necessary to take into account and use the developed model with safety factor.
- The moisture sensitivity of resilient modulus is not included in this paper. To know the seasonal variation of resilient modulus and to include its effect in the predicted model it needs determination of resilient modulus in laboratory at varying moisture levels. The moisture variation should include dry of optimum and wet of optimum moisture levels.
- The model developed is based on a specimen size of 38mm diameter and 76mm length. It is recommended to developing the model with larger specimen sizes.
- It is well known that sample size affects any regression model. As the sample size increases the accuracy related to model increases. The sample data used for the model is minimum and region specific. In the future, it is necessary to study with larger sample sizes and soils collected from different areas should be included to get good result.

REFERENCES

Abbas H. (2018). Assessment of Accuracy in determining atterberg limits for four local soil laboratories. IOP conference series: materials science and engineering. 2nd international conference on engineering Science. 433, 0012030. doi:10.1088/1757-899x/433/1/012030

(AASHTO) American Association of State Highway and Transportation officials, (1993). Guide for Design of pavement structures Washington D.C.

Drumm C., Boateng-poku Y. and Pierce J. (1990), Estimation of Subgrade Resilient Modulus from Standard Tests, journal of Geotechnical Engineering vol. 116, no. 5, pp 774-789

Drumm C., Reeves S., Madgett R. & Trolinger D. (1997), Subgrade Resilient Modulus Correction for Saturation Effects, journal of geotechnical and geo-environmental engineering 123(7): 663 – 670

ERA, Pavement Design Manual (2013): Volume I Flexible Pavements and Gravel Roads. Ethiopian Roads Authority (ERA): Addis Ababa, Ethiopia.

Khasawneh A. (2017), investigation of factors affecting the behaviour of subgrade soils resilient modulus using robust statistical methods 1-14.

Kim M.S. (2004). Development of a Constitutive Model for Resilient Modulus of Cohesive Soils, PhD thesis, Ohio State University

Reddy K. (2002). Engineering properties of soils based on laboratory testing. University Illinois, Chicago

Lee W., Bohra N., Altschaeffl A., and White T. (1997). Resilient modulus of cohesive soils. Journal of geotechnical and geo-environmental engineering. 123(2), 131-135

Li H., (2001). The family of compaction curves for fine-grained soils and their engineering behaviors. MSc. Thesis. Alberta University, Edmonton, Canada. 10p.

Murthy S. (2003). Principles and practices of soil mechanics and foundation engineering 1st edition, Marcel Dekker Inc. New York. 56

Ooi P. Archilla A. & Sandefur K. (2004). Resilient modulus models for compacted cohesive soils. Journal of the transportation research board. No. 1874, 115 – 124

Prakash K., Sridharan A., Thejas H. and Swaroop H. (2012). A simplified approach of determining the specific gravity of soil solids. Journal of geotechnical and geological engineering, 30(4), 1063-1067

Rahim A. (2005). Subgrade soil index properties to estimate resilient modulus for pavement design. International journal of pavement engineering. 6(3), 163-169

Richard J., Nayyarzia S., Tommy N. and Daehyeon K. (2014), Evaluation of Resilient Modulus of Subgrade and Base materials in Indiana and its Implementation in MEPDG, the scientific world journal, vol. 2014

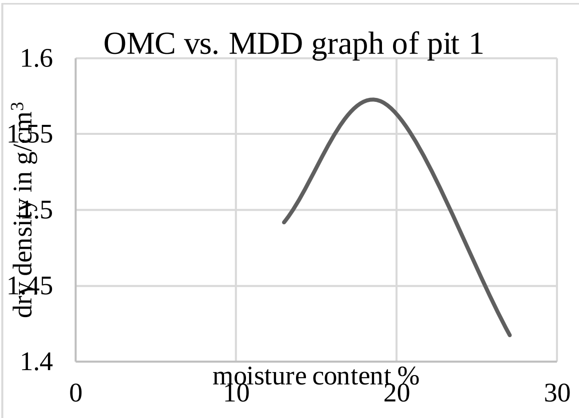
Soliman H. and Shalby A. (2010) Sensitivity of Subgrade Resilient Modulus to Moisture Variation, presented at the annual conference of the transportation association of Canada, Halifax, Nova Scotia.

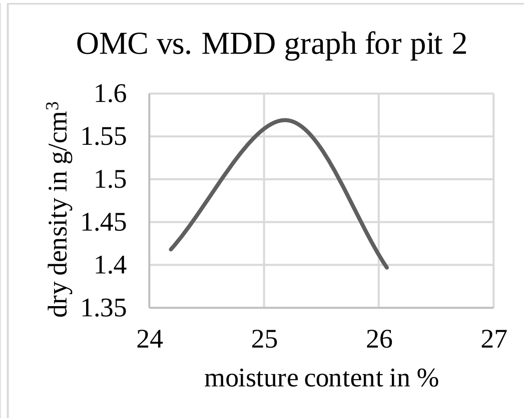
Zainorabidin A. and Agustina H. (2018). Effect of moisture content of cohesive subgrade soil, MATEC web of conferences 195(2), 03010.

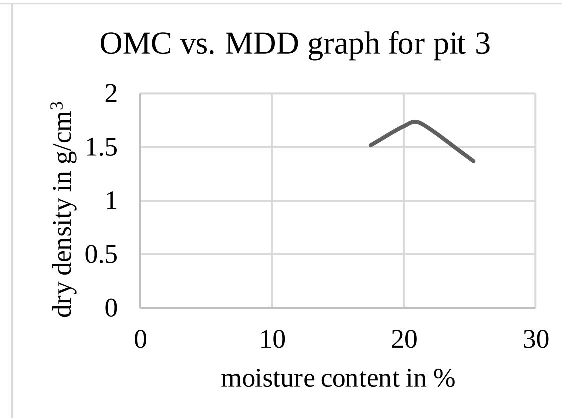
Zumrawi M. and Awad M. (2017), Estimation of Subgrade Resilient Modulus from Soil Index properties, International Journal of Geology & Environmental Engineering 11(9): 856 – 862

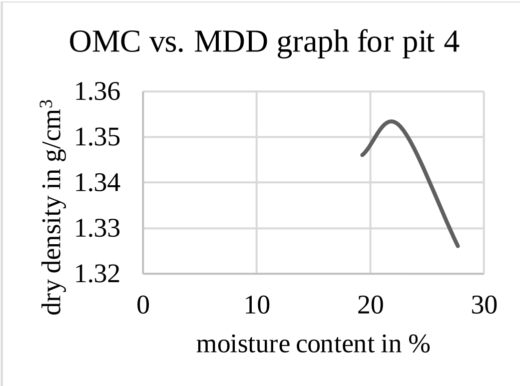
APPENDIX

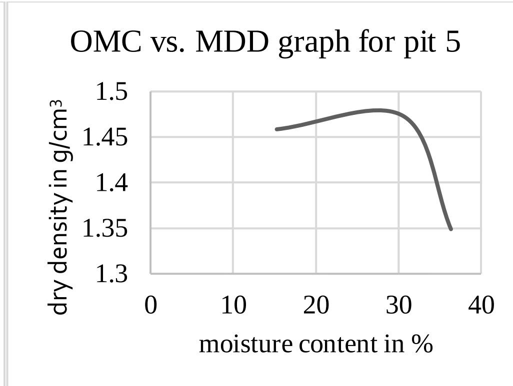
Appendix 1 OMC vs. MDD graphs for different sampling test pits

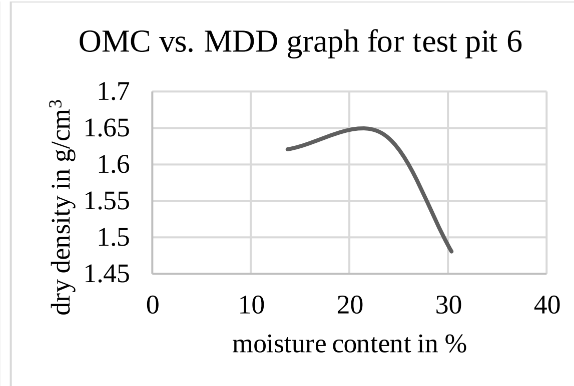


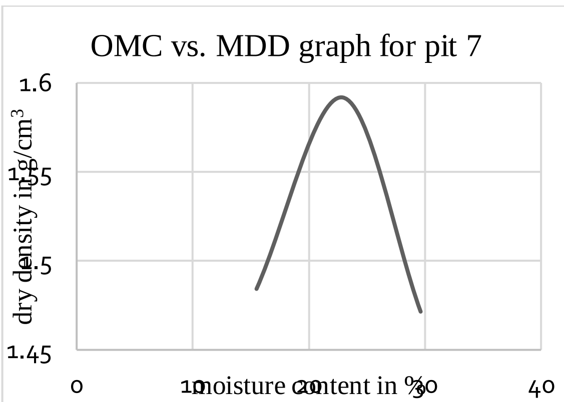


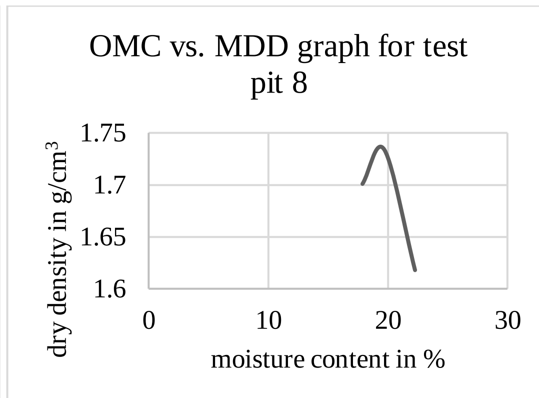


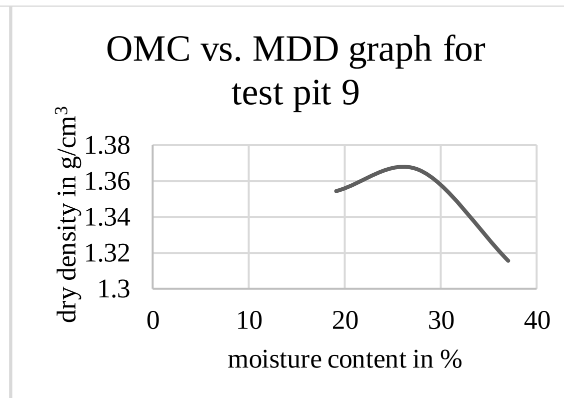

OMC vs. MDD graph for pit 4
dry density in g/cm3
1.36
1.35
1.34
1.33
1.32
0
10
20
30
moisture content in %

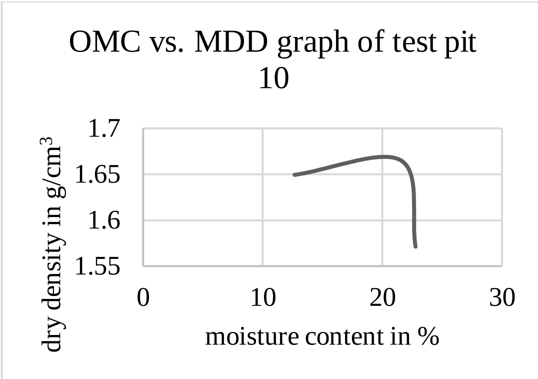

OMC vs. MDD graph for pit 5
dry density in g/cm3
1.5
1.45
1.4
1.35
1.3
0
10
20
30
40
moisture content in %

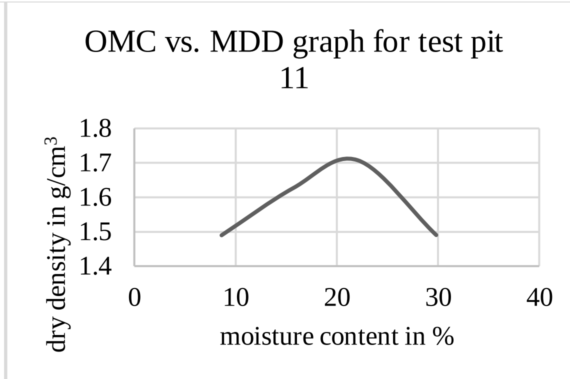

OMC vs. MDD graph for test pit 6
dry density in g/cm3
1.7
1.65
1.6
1.55
1.5
1.45
0
10
20
30
40
moisture content in %

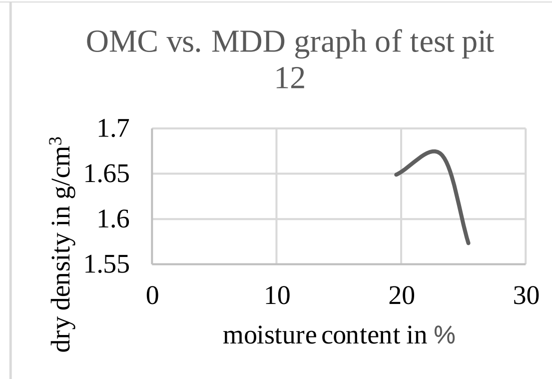

OMC vs. MDD graph for pit 7
dry density in g/cm3
1.6
1.55
1.5
1.45
0
10
20
30
40
moisture content in %

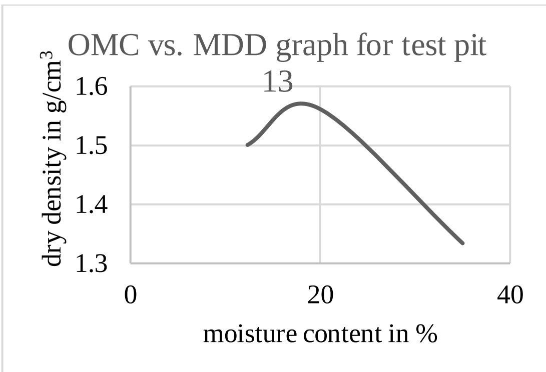

OMC vs. MDD graph for test pit 8
dry density in g/cm3
1.75
1.7
1.65
1.6
0
10
20
30
moisture content in %

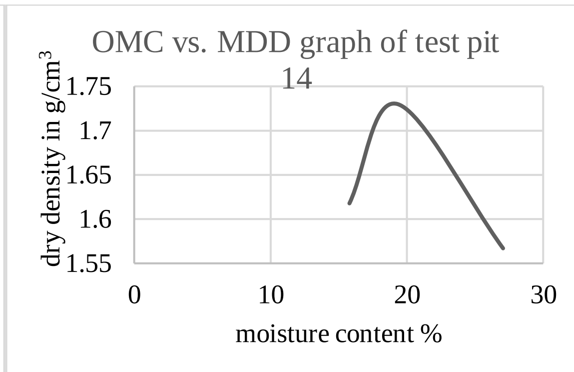

OMC vs. MDD graph for test pit 9
dry density in g/cm3
1.38
1.36
1.34
1.32
1.3
0
10
20
30
40
moisture content in %


OMC vs. MDD graph of test pit 10
dry density in g/cm3
1.7
1.65
1.6
1.55
0
10
20
30
moisture content in %


OMC vs. MDD graph for test pit 11
dry density in g/cm3
1.8
1.7
1.6
1.5
1.4
0
10
20
30
40
moisture content in %


OMC vs. MDD graph of test pit 12
dry density in g/cm3
1.7
1.65
1.6
1.55
0
10
20
30
moisture content in %


OMC vs. MDD graph for test pit 13
dry density in g/cm3
1.6
1.5
1.4
1.3
0
20
40
moisture content in %


OMC vs. MDD graph of test pit 14
dry density in g/cm3
1.75
1.7
1.65
1.6
1.55
0
10
20
30
moisture content %

Appendix II summary of statistical analysis of MR with qu

Regression Statistics	
Multiple R	0.922296
R Square	0.85063
Adj. R Square	0.831959
Standard Error	1.189219
Observations	10

ANOVA					
	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	64.43065	64.43065	45.55844	1E-04
Residual	8	11.31393	1.414242		
Total	9	75.74459			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	-1.33456	1.980025	-0.674	0.519299	-5.90	3.231	-5.9	3.23
qu	0.1219	0.01806	6.749699	0.000145	0.08	0.164	0.08	0.16

Appendix III summary of statistical analysis MR with qu and index properties

Regression Statistics	
Multiple R	0.97091
R Square	0.94266
Adj. R Square	0.87099
Standard Error	1.042
Observations	10

ANOVA					
	df	*SS*	*MS*	*F*	*Significance F*
Regression	5	71.4016	14.2803	13.1524	0.01357
Residual	4	4.34302	1.08575		
Total	9	75.7446			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*	*Upper 95.0%*
Intercept	-18	10.527	-1.6959	0.16515	-47.08	11.37507	-47.080439	11.3751	1.33724
PI	0	0.11932	0.33916	0.75155	-0.2908	0.371764	-0.29082471	0.37176	0.14383
OMC	0.1	0.17353	0.73939	0.50071	-0.3535	0.610096	-0.35348719	0.6101	0.24482
MDD	10	4.07081	2.50992	0.06606	-1.085	21.51976	-1.08497955	21.5198	0.28294
p200	-0	0.05408	-0.5619	0.60418	-0.1805	0.119761	-0.1805347	0.11976	18.4888
qu	0.1	0.02058	5.59088	0.00502	0.05791	0.172168	0.057910581	0.17217	0.04951