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Recalibration of Structural Layer Coefficient and Drainage Coefficient for Sub –Base and Base

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A thesis submitted to the school of graduate studies of Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science in civil engineering *(Road and Transport Engineering Stream)*

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DECLARATION

I declare and undersigned, this thesis is my original work performed under the supervision of Dr. Ephrem Taddesse following the rule and regulations of the institute. This thesis is not presented in any Institution or university for the award of degree or diploma. Also, all materials used as a source are duly acknowledged in the thesis work.

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ABSTRACT

The performance of flexible pavement structures is governed by the supporting capacity of the component layer determined by the structural layer coefficient and drainage coefficient. Also, the performance of the pavement will depend on the pavement material, the traffic level, the environmental condition, and the failure criterion. The value of structural coefficients and drainage coefficients used for pavement design are taken directly from AASHTO 1993. Using AASHTO value has a great influence on the structural thickness of the pavement conversely on the capacity of the pavement.

The focus of this study were concentrated on recalibration of the structural layer coefficient and drainage coefficient values for some base and sub-base pavement material in Addis Ababa by the AASHTO material characterization method. Sieve analysis, Atterberg limit, proctor test, specific gravity, and CBR were conducted for subgrade, sub-base, and base. However, a UU Triaxial test was conducted for subgrade and sub-base to determine the resilient modulus (Mr). The constitutive equation for resilient Modulus was also developed after material constants K_1, K_2, and K_3 were determined from multiple linear regressions using STATA software.

Structural layer coefficient (a) was determined from respective SN by rearranging and averaged for each segment. Drainage coefficient (m) was calculated from the ratio of site layer coefficient to AASHTO layer coefficient. New correlation equations were developed from average structural layer coefficients and resilient modulus (Mr). Then, the results were compared with each other as well as with that of given on AASHTO 1993, AACRA, and ERA **v**alue.

From the result obtained for Shiro Meda – Kidane Mihiret Hamere Nooh, the average thickness obtained for base course is 99.17cm which is greater than the value of 50cm obtained from engineering report which shows addition of 49.17cm. For sub - base course, the average thickness is 25.75cm which is less than the value of 50cm obtained from engineering report which shows a save of 24.25cm. However, this method should be calibrated by cyclic triaxial test to be used routinely.

Key word:- Recalibration, structural layer coefficient, drainage coefficient

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Symbols and Abbreviations

Symbols	Designations
AACRA	Addis Ababa City Road Authority
AADT	Average Annual Daily Traffic
AAR	Average Annual Rainfall
a_1, a_2, a_3	Structural layer coefficient or Strength coefficients.
AG-UU-MH	Ambesa Garage - Unity University - Mebrat Haile
AASHO	American Association of State Highway Officials
AASHTO	Association of State Highway and Transportation Officials
CBR	California Bearing Ratio
ERA	Ethiopian road Authority
FWD	Falling Weight Deflectometer
GE	Granular Equivalency
Sg	Specific Gravity
h_1, h_2, h_3	Thicknesses of pavement surface, base and sub-base layers (existing or required).
m_2, m_3	Drainage coefficients. Factors used to modify layer coefficients in flexible pavements
MDD	Maximum dry density
Mr	Resilient modulus
NCAT	National Center for Asphalt Technology
NMSA	National Meteorological Services Agency
OMC	Optimum moisture content
PSI	Present Serviceability Index
SB	Sub – base
SLC	Structural Layer Coefficient
SM - KMHN	Shiro Meda - Kidane Mihiret Hamere Nooh
SN	Structural number

CHAPTRE 1 INTRODUCTION

1.1 Background

The economic and social developments of nations depend on the quality of road infrastructure they have. Heavy loads and high speed are expected with the development of the national economy. Developing proper design, analysis, construction, and planning method are important to withstand the emerged heavy load and high speed. In this context, the current traffic data collection method and geotechnical investigation methods should be sufficient to get an actual condition that happens on road pavement. Since Loading and unloading traffic condition is acted upon the Road pavement, Static design methods such as California Bearing Ratio (CBR) will not be able to meet strength and deformation requirements of subgrade and pavement materials.

In the late 1950‟s and early 1960‟s, the American Association of State Highway Officials (AASHO) conducted a limited road test, the purpose of which was to determine a methodology for designing pavement structures. This organization, which later became the American Association of State Highway and Transportation Officials (AASHTO) used the AASHO Road Test results and The first interim design guide was published in 1961. Subsequent revisions occurred in 1972, 1981, 1986, and 1993 with the expanded and some edition. This design guide incorporates several design variables such as traffic loading, environmental effects, serviceability, pavement layer thickness, and pavement layer materials. Also, it incorporates a level of uncertainty in the process to ensure that the design will last. The guide uses the concept of layer coefficients for Asphalt pavements; these coefficients are based on research conducted on some pavement and material properties. The coefficients are indicators of the relative ability of a material to function as a structural component within the asphalt pavement.

The concept of structural number (SN) was first introduced as a result of the AASHO Road Test as a measure of overall pavement strength. It is essentially a measure of the total thickness of the road pavement weighted according to the strength of each layer. In the AASHTO method of flexible pavement design, the structural number is an index of pavement strength which reflects the degree to which the sub-grade must be protected from the effects of traffic based on layer thickness and material properties. SN is commonly used in pavement design practices and expresses the capacity of pavements to carry loads for a given combination of estimated traffic(W_{I8}), reliability(R), the overall standard deviation(So), terminal serviceability($\Delta PSI = (P_O - P_i)$ and environment.

The AASHTO Guide for the Design of Pavement Structures, based on the pavement performance- serviceability concept developed from the American Association of State Highway Officials (AASHO) Road Test, utilized the concept of "layer coefficients" to synthesize a pavement structure employing materials of varied supporting capacities.

A value for this coefficient is assigned to each layer material in the pavement structure to convert actual layer thickness into structural numbers (SN). This layer coefficient expresses the empirical relationship between SN and thickness; it is a measure of the relative ability of the material to function as a structural component of the asphalt pavement. The structural layer coefficients are obtained from experimentation and test result for a specific project.

The treatment for the expected level of drainage for a flexible pavement is made through the integration of modified layer coefficients into the structural number (SN) equation along with layer coefficient (ai) and thickness (Di); thus:

$$SN = a_1d_1 + a_2d_2m_3 + a_3d_3m_3\ldots\ldots\ldots\ldots\ldots\ldots \quad (1.1)$$

Where;-

a_1, a_2, a_3 =structural layer coefficients for surfacing, base and sub-base layers respectively

d_1, d_2, d_3 = thickness of surfacing, base and sub-base layers respectively.

The SN combines the impacts of the layer coefficients, layer thicknesses, and drainage coefficients on the pavement structure. The original layer coefficients were regression coefficients developed by relating layer thickness to a road performance determined on basis of the parameters of the AASHO Road Test. The development of the layer coefficients has been evolving; the most recent AASHTO Design Guide stipulates that layer coefficients can vary depending on several factors. These factors include material type, material properties, type of layer, traffic level, and failure criterion. The principal variables are material types and material properties. Materials types vary everywhere across the country and the material properties are dependent on construction practices and local environments. These conditions as well as traffic levels exhibit a wide range across the country. Therefore, the layer coefficients given in the AASHTO Design Guide are expected to be used universally, whereas different layer coefficients are expected to be developed for local conditions.

The methodologies used for recalibrations (developing) of the Layer coefficient are based on the following. These are: - Pavement Structural Response, Pavement Performance, Mechanistic - Empirical Design Approach, Material Properties Characterization, and Falling Weight

Deflectometer. The 1993 AASHTO Design Guide recommends Material Properties Characterization using the resilient modulus (Mr) as the standard material quality measure to account for material types and material properties since resilient modulus is not only a measure of stiffness but also can be an indicator of the strength of the material.

It is impossible to determine structural layer coefficient without the concept of drainage coefficient. The drainage coefficient is the modifier of the structural layer coefficient. The methodologies used to recalibrate drainage coefficient are the ratio of the layer coefficient of road test granular material adjusted for any specific site conditions (a site) to the layer coefficient of the road test material for road test conditions (a road test). Also, m is calculated from the ratio of respective average site layer coefficient to AASHTO layer coefficient. On the other hand, m is calculated based upon the percent passing the number 200 sieve of the material in question and the Average Annual Rainfall in inches for the project location. Then, m values obtained from the three methods are compared.

Therefore, this study is concerned with the determination of structural layer coefficient and drainage coefficient for the different types of new and processed sub-base and base material based on the AASHTO Material Properties Characterization method used to determine the resilient modulus (Mr) value obtained from the UU test method and drainage coefficients determination for the different types of new and processed sub-base and base material based on the ratio of the layer coefficient of road test granular material adjusted for any specific site conditions (**a** site) to the layer coefficient of the road test material for road test conditions (**a** road test).

1.2 problem statement

A pavement layer has to support the stress that comes to each layer. For this, more durable pavement materials were needed to support the ever-increasing traffic loads. Recently, the use of new and reprocessed (stabilized) sub-base and base course material has become more common. The capacity of each layer is determined by the layer coefficient of each layer.

The stress that comes to each layer is supported by the stiffness of the pavement material. This procedure does not tell us the thickness of each layer such as HMA, base, sub-base, and capping layer if it exists. To overcome this problem, the stress in each pavement layer is supported by the thickness of each layer's capacity. This thickness capacity is determined by the thickness and the structural coefficient of each layer. These structural coefficients are taken directly from

AASHTO 1993. Using AASHTO value has a great influence on the structural thickness of the pavement conversely on the capacity of the pavement. But, AASHTO 1993 Warns the design Engineer to develop structural layer coefficient project-specific and country-specific based on the country construction practice, traffic, environment, and material type. So, developing structural layer coefficients for different pavement materials is important according to local conditions.

1.3 Motivation of the thesis

The research was initiated by my own observation when I teach at Ambo University about the pavement design concept. The structural layer coefficient and drainage coefficient values are taken from the AASHTO guide. However, AASHTO warns the design Engineer to develop structural layer coefficient and drainage coefficients according to local conditions. So, it is my long-time question why we do not develop these values in the Ethiopian context. So I use this opportunity to do more in this area.

1.4 Significance of the study

The result of the thesis is used for mining easy method to determine the structural layer coefficient and drainage layer coefficient values for different categories of new and processed base and sub-base materials of some selected road projects in Addis Ababa. Using this trend, the structural layer coefficient and drainage coefficient of any project in and around Addis Ababa will be recommended. Finally, this paper gives some recommendations on the structural coefficient and drainage for Ethiopia in general. The obtained structural layer coefficient and drainage layer coefficient may be used for;-

- For pavement designers to determine the thickness of each layer of new pavement without intensive testing.
- Determining the overlay thickness needed for pavement rehabilitation.
- It will pave an easy way for pavement design and analysis.

1.5 Objective of the study

1.5.1 General Objective

The objective this thesis is to develop city wide structural layer coefficient and drainage coefficient for Addis Ababa. Then, using the trend of city wide, to recommend for the country Ethiopia in general.

1.5.2 Specific objective

- To find the structural layer coefficients and drainage coefficient for different categories of new and processed sub-base and base pavement materials for different projects under construction in Addis Ababa.
- To find the structural layer coefficient and drainage coefficient for sub-base and base pavement material by considering the average layer coefficient calculated at each segment of the road for different road projects under construction in Addis Ababa.
- To analyze the variation of structural layer coefficients and drainage coefficient values for different projects.
- To compare the new values of layer coefficient and drainage coefficients of sub-base and base pavement materials with that of given on AACRA, ERA, and AASHTO 1993.

1.6 Scope of the Study

This paper deal with the determination of the structural layer and drainage coefficient of new and processed pavement materials for some project in Addis Ababa by use of the AASHTO pavement material characterization method. The resilient modulus for pavement material in the selected projects is determined from the laboratory UU triaxial test. The average value of the structural layer and drainage coefficient will be determined at different selected segments for each project and analyzed with the value given on that the AASHTO 1993, ERA, and AACRA. New correlation equations were developed from average structural layer coefficients and resilient modulus (Mr) value obtained from data of 4(four) samples for Shiro Meda and 3(three) samples for Ambessa garage only for sub-base material independently. The result obtained will be expressed by the use of tables and graphical presentations. Using this trend, the structural layer coefficient and drainage coefficient of any project in/around Addis Ababa will be recommended. Finally, this paper can give some recommendations on the structural coefficient and Drainage coefficient for Ethiopia in general.

1.7 Limitation of the Thesis Work

- The study is limited to flexible road pavement under the jurisdiction of the AACRA.
  The results obtained are not calibrated by cyclic triaxial test method.

1.8 Organization of the Thesis

This thesis contains six chapters. Chapter one provides an introduction to the research project. It provides a general background and provides the scope, research questions, objectives and

limitation of the thesis work. Chapter two provides a literature review of the issues surrounding structural layer coefficient and drainage coefficient. Literatures were reviewed to obtain a theoretical basis for the Research, formulate the research questions and define the scope. To this effect, the AASHTO 1993 main guide and ERA 2013 were reviewed. Thereafter, relevant articles from journals and other publications were searched to conceptualize the structural layer coefficient and drainage coefficient. Apart from searching in libraries, internet sources were used to obtain recent articles and research papers in the area.

Chapter three deals with research methodology and laboratory test methods. In chapter Four, Test results are discussed. Chapter Five is the main part of this thesis work which is a recalibration of structural layer coefficient and drainage layer coefficient for new and processed sub-base and base pavement material. In chapter six, the Interpretation and Discussion of test results are presented. Conclusion and recommendations are presented in chapter seven. Finally, references and appendixes are presented.

CHAPTER 2 LITERATURE REVIEWS

2.1 Introductions

The economic and social development of nations all over the world relies to a large extent on the quality of their road transportation system. The economic role of the road infrastructure and the amount of money invested in its construction and maintenance indicate the importance of good roads. Poorly designed road pavements will suffer from premature failure, which will lead to high reconstruction costs and great economic losses. Over-designed pavements, on the other hand, will involve a waste of limited funds. Pavement design is a process intended to find the most economical combination of layer thicknesses and material types for the pavement, taking into account the properties of the subsoil, the traffic to be carried during the service life of the road and the climatic conditions [17].

Structural design is the process of developing the most economical combination of pavement layers with both material type and thickness to suit the soil foundation to the traffic load during the design period [16].

Structural design is based on the concept of limiting stresses and deformations to prevent excessive damage and deterioration of pavements. Overstressed pavements due to traffic loads and environmental effects will result in pavement distress. Road design approach may be empirical method or mechanistic method. An empirical design method is based solely on the results of experiments or experience. An empirical method is the use of charts and equations developed from experimental studies carried out with a set of traffic, environment, environment and Mechanistic method is, in which concepts of mechanics are used to predict responses and performance of the pavement. It is also data-intensive [42, 46]. For this thesis work, the empirical method is used since the mechanistic method is data intensive.

A pavement layer has to support the stress that comes to each layer. For this, more durable pavement materials were needed to support the ever-increasing traffic loads [2]. In general, AASHTO Soil classifications A -1 - a, A - 1 - b, A - 2 - 4, A - 2 - 5, and A - 3 can be used satisfactorily as subgrade or sub-base material if properly drained. Also, such soils must be properly compacted and covered with an adequate thickness of pavement (base and/or surface cover) for the surface load to be carried. The Materials classified as A - 2 - 6, A - 2 - 7, A - 4, A - 5, A - 6, A - 7 -5, and A - 7 - 6 will require a layer of sub-base material if used as subgrade [42].

Recently, the use of a new and reprocessed sub-base and base course material has become more common. New pavement materials are 100% natural gravels which include granular pavement materials without any admixtures of stabilizers and Reprocessed pavement materials are crushed pavement material. For this thesis work, however, both projects i.e. Shiro Meda - Kidane Mihiret Hamere Nooh and Ambessa garage - Unity University - Mebrat Haile, the pavement materials used for sub-base and base layers are new pavement materials obtained from nearby quarry site since no road construction material problem in Addis Ababa.

2.2 Definitions of Structural Layer Coefficient and Drainage Coefficient

The capacity of each layer is determined by the layer coefficient of each layer [2, 3]. The layer coefficient expresses the empirical relationship between SN and thickness and is a measure of the relative ability of the material to function as a structural component of the pavement [2, 7]. In other words, layer coefficient provides a "yardstick of equivalence" of layer strength and thickness and is indispensable in designing multilayer pavement systems [10].

Drainage coefficients are essentially modifiers of the layer coefficients and take into account the relative effects of pavement structure internal drainage on the performance of the pavement [11, 48].If the sub-base and base course layer coefficients are multiplied by some factor (called drainage coefficients). There is a corresponding increase or decrease in the thickness of the pavement layer while the structural number (SN) is maintained as a constant. It should be noted that by definition the m2 and m3 coefficients only address the effects of drainage in the unbound granular base and sub-base, respectively. They do not address the effects of moisture in the asphalt-bound layer(s), other stabilized layers, or the subgrade. The effects of moisture in the subgrade should be addressed during the calculation of the effective subgrade modulus in the design phase of a given project [50].

The approach used to evaluate the effect of drainage for flexible pavement design was to include adjustment factors (m values) to the structural number equation that modify the layer coefficients according to the anticipated moisture (drainage) conditions[50].

$SN = a_1D_1 + a_2m_2D_2 + a_3m_3D_3$ …….……. (2.1)

Where; SN= structural number

a_1, a_2, a_3 = structural layer coefficients,

D_1, D_2, D_3= layer thicknesses

m_2, m_3 = effect of water on the stiffness (strength) of each material.

2.3 Background of AASHTO Layer Coefficients

The solution to the design equations in the different versions of the AASHTO pavement design guide is in terms of a structural number (SN). A structural number is an abstract number expressing the structural strength of pavement required for a given combination of soil support value, total equivalent 18-kip (80 KN), single axle loads (ESALs), terminal serviceability index, and regional factor. The magnitude of the SN reflects the degree to which the sub-grade must be protected from the effects of traffic. The AASHTO Guide for the Design of Pavement Structures, based on the pavement performance- serviceability concept developed from the AASHO Road test, utilized the concept of "layer coefficients" (ai values) to synthesize a pavement structure employing materials of varied supporting capacities. A value for this coefficient is assigned to each layer material in the pavement structure to convert actual layer thickness into structural numbers (SN) [9].

2.4 Origins of Structural Layer Coefficient

It was from the fourth-power relationship that the concept of pavement capacity was derived. Knowing the expected traffic that will load a pavement and its associated damage, the pavement must have a certain capacity to withstand said traffic and resulting damage. The researchers at the AASHO Road Test developed an equation termed the "thickness index" (similar to the structural number), which can be mathematically expressed as [11]:-

$SN = a_1D_1 + a_2D_2 + a_3D_3$ ……..……. (2.2)

Where the terms are the layer coefficients, the D terms are the thicknesses of each layer, and the subscripts 1, 2, and 3 represent the HMA, base, and sub-base pavement layers, respectively.

The AASHO Interim Guide proposed numerical values for the structural layer coefficient of materials. Because of wide variations in environments, traffic, and construction practices, it is suggested that each design agency establish layer coefficients based on its own experience and applicable to its practice [10].

In 1990, WisDOT published a report on layer coefficients for flexible pavements. The purpose of the study entitled "Layer Coefficients for Flexible Pavements" was to evaluate new materials that are used in flexible pavements. The goals were to establish AASHTO layer coefficients for use in the design, establish a procedure that would allow for the determination of these coefficients, and provide input for the new 1986 AASHTO design procedure [9].

Research conducted in National Center for Asphalt Technology (NCAT) revealed that the layer coefficient followed by traffic level and resilient modulus are the most influential factors in the determination of the pavement thickness considering the AASHTO 1993 design equation. Although the original layer coefficients from the AASHTO road test are reliable, they apply only to the types of material, traffic, and the environmental conditions under which they have been generated [12].

2.5 Factors of Structural Layer Coefficient and Drainage Coefficient

According to the most recent AASHTO guide (1993), the a-value can vary considerably depending upon several factors. These involve: Layer thickness, Material type, Material properties, Layer location (base, sub-base), Traffic level, and Failure criterion. Because of the wide variations in environments, traffic, and construction practices, the proposed layer coefficient values in the AASHTO guide are not expected to apply to all cases. As a result, different layer coefficient values should be determined based on local conditions [9].

Five types of existing data were necessary to evaluate the drainage coefficient. These were 1) routine soil properties and location, 2) soil and material thermal/moisture-related properties, 3) climatological data, 4) pavement material structural properties, 5) unbound granular material drainability properties, and 6) typical pavement geometry information. These, in turn, affected base thickness calculation significantly [51].

2.6 Overview of the AASHTO Empirical Structural Layer Coefficient and Drainage Coefficient Determination Procedure

The AASHO Road Test was conducted from 1958 to 1960 near Ottowa, Illinois. The primary purpose of the road test was to determine the effect of various axle loadings on pavement behavior. Both flexible and rigid pavements were tested in the study, along with several short-span bridges. Six two-lane test loops were created for trafficking, including four large loops and two small loops. The AASHO Road Test was developed in the late 1950s to provide information that could be used to develop pavement design criteria and procedures. When completed, the road test directly leads to the 1961 AASHO Interim Guide for the Design of Rigid and Flexible Pavements. This guide was modified in the following decade and published in 1972 as the AASHTO Interim Guide for Design of Pavement Structures - 1972. After that, major revisions to the guide were completed in 1986 and 1993[9].

2.7 Limitation of the AASHTO Empirical Structural Layer Coefficient and Drainage Coefficient Determination Procedure

Only 2 million equivalent single axle loads (ESALs) were applied throughout the test. The test vehicles ranged in gross weight from 2,000 lb to 48,000 lb. The improved paving materials that are used today such as Superpave mixes, stone matrix asphalt, and open-graded friction courses were not available during the road test. Within the pavement cross-section, only one type of HMA, granular base material, and subgrade soil were used. The thickest HMA pavement was 6 inches. All results from the road test are a product of the climate of northern Illinois within two years [11].

The limitations inherent in the original AASHO road test equations are still applicable;-

- Specific set of pavement materials and one roadbed soil.
- Single environment.
- An accelerated procedure for accumulating traffic (a 2 - year testing period extrapolated to a 10 0r 20 - year design), and
- Accumulating traffic on each test section by operating vehicles with identical axle loads and axle configurations, as opposed to mixed traffic.

These basic limitations are reduced to some extent by experiences of various agencies.

2.8 Reviews on Structural Layer Coefficient and Drainage Coefficient Calibration Methods

Layer Coefficients were first developed after the AASHTO Road Test. The structural number (SN) was determined from the AASHTO equation, which was developed based on performance. The layer thicknesses were then used as independent variables to predict SN. The regression coefficients were used as the structural coefficients [54].

Since layer coefficient has a significant influence in determining the layer thickness and consequently on the construction expenses as well as the long term performance of the pavement, it is essential to determine the calibrated and reliable layer coefficient values for different regions and materials. For this purpose, many of the countries that implement AASHTO 1993 design procedure or use layer coefficient as part of their specific design methodology have tried to evaluate their own commonly used materials and assign new a-values to them. Old studies which are mainly experimental based have shown that a-values are correlated with gradation, thickness, abrasion of aggregate and more important the strength or stability of asphalt mixtures [12].

To account for material type and properties in the 1993 AASHTO guide, the elastic (resilient) modulus (M_r) is used as the standard material quality measure. The relationship between the layer coefficients and the resilient modulus has been investigated in several research efforts. The layer coefficient does not only reflect the stress distribution in the layer, but is also an index of the strength of the material. The position of the material in the structure and the mode of distress may, therefore, influence the relation between the layer coefficient and the resilient modulus [9].

The coefficients can also be computed with regression equations. For a base and sub-base courses, the coefficients are determined from the following equations [2, 9, 13];-

$a_2 = 0.249(\log Mr) - 0.977$…….. (2.3)

$a_3 = 0.227(\log Mr) - 0.839$…….. (2.4)

Where Mr is the resilient moduli for the base and sub-base courses can be determined either by testing or from an AASHTO equation as a function of the stress state and moisture conditions; - Mr. = $K_1\theta^{K2}$ where K_1 and K_2 = material constants, and θ = bulk stress

Many researchers have studied the resilient modulus i.e. nonlinear stress-strain characteristics of sub-grade soils. Pavement engineers have taken the stress-strain models developed by this research and applied them to asphalt layers. These models `proved to be powerful and more realistic representation of material performance than conventional tests, such as the California Bearing Ratio (CBR). The values estimated from field measurements are sensitive to the back calculation procedure and the assumptions made in the analysis. It is, therefore, more realistic to measure resilient modulus in the laboratory under different stress and temperature conditions [2]. Most of the methods used to evaluate either the layer coefficient or the layer thickness equivalency are based on the evaluation of limiting criteria at specific points in the pavement structure. To establish layer coefficients for the 1986 version of the AASHTO guide under various conditions and materials, three criteria surface deflection, tensile strain in the asphalt layer, and vertical compressive strain on the roadbed soil are used [9].

The layer coefficients were empirically developed during the AASHTO Road Test. These coefficients are reliable, but only directly applicable to the types of materials and environmental conditions represented in the Road Test. Procedures similar to those used at the original Road Test have since been used at satellite projects to determine structural coefficients for a specific area and local materials. Reliable coefficients are important since they can have a big influence

on the design thickness. High stability plant mixed asphalt concrete has the highest coefficient (0.44) and strengths of the other materials (or their influence on performance) are expressed as a fraction of 0.44 to give the structural coefficients [11, 54].

As originally used in the AASHO Road Test results, layer coefficients were regression coefficients which were the result of relating layer thicknesses to road performance under the conditions of the Road Test. The problem is to translate the Road Test findings to other geographic areas where the construction materials and climate are different. Drainage coefficients must be determined to use Eq. 1 above for design purposes [51].

The approach used to evaluate the effect of drainage for flexible pavement design was to include adjustment factors (m values) to the structural number equation that modify the layer coefficients according to the anticipated moisture (drainage) conditions. It was assumed initially that the *m* values might vary from 0.5 to 1.4. Note that an m value greater than 1.0 would result in a net decrease in required total SN or, if applied only to the surface layer, a decrease in surface thickness of about 1 to 2.5 in., depending on base thickness. Similarly, selection of an m value less than 1.0 would result in an increase in SN or in surface thickness [50].

2.9 Drainage Coefficient and Structural Layer Coefficient Recalibration Methods/ Approaches

2.9.1 Drainage Coefficient Recalibration Methods / Approaches

Recalibration of drainage coefficient is to calibrate (to make some change of) values again to represent accurate value of drainage coefficient of specific project or some area rather than using the values developed by AASHTO. The values developed by AASHTO are used over wider range to include the values of structural layer coefficient and drainage coefficient developed at Ottowa, Illinois over the world. Designers should consider carefully before adopting values developed by others.

The drainage coefficients are, in essence, modifiers of layer coefficients (see Equation 1) that adjust for conditions of drainage of a given project relative to the conditions of drainage at the road test. Each drainage coefficient developed in this study is a ratio of the layer coefficient of road test granular material adjusted for any specific site conditions (*a* site) to the layer coefficient of the road test material for road test conditions (*a* road test) [48].

$$m = \frac{\text{a site}}{\text{a road test}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.5)$$

On The other hand, drainage coefficient (mi) are calculated based upon the percent passing the number 200 sieve (p200) of the material in question and the Average Annual Rainfall in inches (AAR) for the project location[11]. Also Drainage coefficient can also be determined from AASHTO drainage coefficient table [51].

2.9.2 Structural Layer Coefficient Recalibration Methods / Approaches

Recalibration of structural layer coefficient is to calibrate (to make some change of) values again to represent accurate value of structural layer coefficient of specific project or some area rather than using the values developed by AASHTO. The values developed by AASHTO are used over wider range to include the values of structural layer coefficient and drainage coefficient developed at Ottowa, Illinois over the world. Designers should consider carefully before adopting values developed by others.

Layer coefficient Recalibration is conducted based on different methodologies which will be discussed briefly in this section.

- **Pavement Structural Response**

Some mechanistic-based studies have tried to use the concept of equivalent deflection by assigning a reference mixture with a defined thickness and compare other mixtures to that by determining the required thickness to result in the identical deflection to that of the reference mixture under the same loading magnitude. Similarly, the identical maximum vertical stress on top of subgrade soil for different types of hot mix asphalt mixtures has widely been used to recalculate the a-values. Maximum tensile strain at the bottom of the asphalt layer has also been used to determine the layer coefficient of recycled mixes. The thickness of the recycled layer to give the equivalent number of load repetitions to failure (N_f) of the standard reference hot mix asphalt on the same subgrade soil is used to determine the layer coefficient since the SN is equal for both cases [15].

- **Pavement Performance**

AASHTO pavement performance analysis has also been used as another practical method for layer coefficient calibration. This method monitors the serviceability indicators (rut depth, cracks, patching, IRI and etc.) and calculates the PSI. The rate of change in serviceability for a given pavement structure with known thicknesses for different layers is then converted to SN value. From this SN the structural layer coefficient for respective layer is calculated [15].

- **Mechanistic-Empirical Design Approach**

A more sophisticated way to calibrate the a-value is to use the mechanistic-empirical method (MEPDG). This method is highly data-intensive and is recommended to be used by the agencies that are in the process of transforming from the empirical to the mechanistic-empirical design approach and have enough databases available for the calibration. Once the database is available, the calibration can be simply done by designing the required thickness through MEPDG approach and then calibrate the a-value in the AASHTO design method to obtain the same thickness for the structure [15].

- **Falling weight Deflectometer**

According to the AASHTO 1993 design guide, a more reliable way to determine the layer coefficients is to back-calculate the moduli from Falling Weight Deflectometer (FWD) test on the road in lieu of lab testing since there might be a variation between the lab-made samples and the mixture placed in the field and also FWD is considered as a way to simulate the dynamic loading of a moving wheel in a wide range of loading level which is a more realistic way of loading in the field [3, 12]. However, this method is new technology and costly.

- **Material Properties Characterization**

Among all the factors that influence the layer coefficient, the material type and properties have the highest impact. The AASHTO 1993 design guide proposes the resilient modulus (Mr) of the material since it is not only a measure of stiffness but also can be an indicator of the strength of the materials [12].

The Resilient Modulus (MR) is a measure of material stiffness. A material's resilient modulus is actually an estimate of its modulus of elasticity (E). The modulus of elasticity is stress divided by strain for a slowly applied load. Resilient modulus is stress divided by strain for rapidly applied loads – like those experienced by pavements [42].

There are many types of tests designed to determine the mechanical properties of soil, such as stiffness tests, shearing tests, and penetration tests. Shear strength is one of the most important soil mechanical properties which quantify the stress by which the soil fails. Shear strength is the maximum force a material can withstand before it ruptures. Soil shear strength represents the resistance that can be sustained by the soil due to traffic load. The stiffness test such as the cyclic triaxial test best simulates the traffic load and pavement soil interaction. However, the cyclic triaxial test is not affordable in many poor countries for testing pavement material for design and research activities. This is because; the cyclic resilient test is a difficult and complex test that

requires a high cost of the testing system and skilled technicians. Thus, the development of alternative simple and reliable resilient testing techniques is important to implement Mr Test routinely in pavement design and analysis. The alternative methods should be simple enough for the practitioner to carry out on a routine basis and the testing method should represent the stress and strain states in the pavement material under vehicle loading [32, 33, and 35].

In the past, the resilient modulus is correlated to UCS, CBR, or other pavement material properties which can be obtained in most geotechnical laboratories of road construction contractors, consultants, and Universities such as Atterberg limit (LL and PL), specific gravity, optimum moisture content, and dry density. Some of the early attempts were based on a simple empirical relationship between the cyclic California bearing ratio (CCBR) and Mr Value [36], CU and Mr [37], UCS and Mr [38]. In any case, a well-planned experiment design is essential in order to obtain reliable correlations [2].

There is a general consensus that the stiffness's determined by dynamic, cyclic, or static tests are equivalent if the effects of strain amplitude, loading frequency, confining pressure, and loading cycles are properly taken into account in the comparison [32, 33].

The predicted Mr values from the static Triaxial test method match well with those determined by standard cyclic Mr showing the capability of the static triaxial test method be used for determining Mr for both sub-grade and sub-base. Modulus values obtained by the standard Mr Test method and static Mr Test method overlap nicely [32, 33].

All of these methods have been shown to be reliable in a recalibration of a-values. AASHTO 1993 design guide recommends the use of Resilient Modulus (Mr) of the mixtures for a-value recalibration purposes.

Layer coefficient values for different layers made of different materials can be determined using empirical equations derived from field experiments. The following equations are some of the commonly used relationships (Mustaque et al. 1997.) [2, 12].

- Bituminous-treated Base: $a_2 = 0.3\log (E/3000Mpa) + 0.15$ $0.1 < a_2 < 0.3$
- Granular Sub-base: $a_3 = 0.23\log (E/160Mpa) + 0.15$ $0.06 < a_3 < 0.2$

In the AASHTO design procedure a number of relationships have been derived using layered elastic theories to evaluate a combination of pavement cross-sections and material properties. In addition to the following relationships, charts for estimation were developed for other materials.

- Granular Base: $a_2 = 0.249 \log E_2 – 0.977$

- granular Sub-base: $a_3 = 0.227 \log E_3 - 0.839$

Where ai values are the layer coefficients and the Ei values are the resilient modulus values. The relationship between layer coefficients and resilient moduli is generally based on layered elastic theory. Layer coefficients are affected by more factors than just material stiffness and strength; this is why these empirical relationships are derived [12].

2.10 Triaxial Compression Tests

As explained above, resilient modulus can be determined from the static triaxial compression test. There are three types of triaxial compression tests based on the consolidation and drainage conditions [22, 36].

- Unconsolidated undrained (UU) confined triaxial compression tests; this method is to quickly load the sample without pre-consolidation and without the ability to drain water during the load.
- A consolidated drained (CD) Triaxial compression test is carried out with preliminary consolidation and with the possibility of draining. Consolidated, Drained (CD) used to estimate the effective shear strength of cohesive soil. This test is performed by allowing drainage (i.e., no excess pore pressure) and consolidation while the shearing stress is applied. Since cohesive soil generally drains very slowly, the shear stress must be applied at a very low rate. Consequently, the test takes considerable time to conduct, is costly and is therefore rarely performed. In lieu of this test, the triaxial CU test with pore water pressure measurements is typically performed to estimate effective (i.e., drained or long term strength) shear strength.
- Consolidated undrained (CU) Triaxial compression tests; the method is to load the sample after initial consolidation without the possibility of water draining. Unlike the UU test, the CU test is performed by consolidating the sample prior to applying the shear stress. The CU test can be used to represent conditions in the field where load has been applied and foundation soils have had time to consolidate but excess pore water pressure is still present.

The most reliable soil strength parameters can be obtained by using a triaxial compression apparatus. With these tests, geotechnical parameters such as the angle of friction, cohesion, Young's modulus can be determined. Studies are carried out on samples with a diameter (D) around two times lower than its high - H *(*H = 2D*).* All test methods can be used to determine

the resilient modulus, although different test conditions should be taken into account in the calculation. The resilient modulus is determined in a triaxial compression apparatus under cyclic loading conditions which most accurately reflects the behavior of the soil subgrade subjected to cyclical stress [36].

2.10.1 Unconsolidated Undrained (UU) Triaxial Compression Tests

For this thesis work, however, static Triaxial compression test of unconsolidated undrained (UU) triaxial test is used since it is obtained in several universities, road constructing contractors and consultants' geotechnical laboratory which is help full to use UU test to carry out on the routine basis for Mr Test. The advantages of UU confined Triaxial tests include uniform stress distribution, total control over drainage conditions, accurate for research purposes, [22], it is affordable in most geotechnical laboratories, can measure the confining pressure that occurred, it is used for either undisturbed or remolded samples and it can be effectively used to study soil mechanical behavior for pavement design and research work. On the other hand, UU test is used for cohesive soil of low permeability. However, it is used for soil of all type which has good permeability when compared to CU and CD triaxial test. Therefore, this type of test method relatively describes the traffic load effect on pavement structures.

The principal stresses (σ_1, σ_2, σ_3) and axial strain (ε) can be obtained from the test result. From these results, deviatoric stress is calculated for the construction of stress-strain diagram. Stress - strain relationships were presented by plotting accumulated axial strains with deviator stresses. Deviator stresses were calculated using raw data from quick shear tests, and axial strains were corrected by counting the permanent strains from the Mr tests on the sample. Strains were calculated by dividing the displacements by the sample height that was calculated after Mr Test. Then, the cross area of the test sample was corrected by accounting for strain effects on the sample [23].

2.10.2 Unconsolidated Undrained (UU) Triaxial Compression Test Procedure

The UU triaxial test is performed by applying lateral confining stress (chamber pressure) and then applying an axial load without allowing the sample to drain. Normally three samples are sheared, and the chamber pressure is increased for each sample. The chamber pressures must be specified by the engineer and communicated to the laboratory, and these pressures should "bracket" the soil pressure expected in the field [23, 43]. For this thesis work 100KN, 200KN and 300KN are used for Ethiopia since it is near-tropical.

Before starting a test, it is essential to de-air the whole system i.e. the cylindrical chamber and the pressure lines, as air could cause changes in the pressure and volume as it compresses. After the soil specimen was prepared according to the desired moisture content and bulk density as it was covered in the rubber membrane and mounted on the test assembly. The cylindrical chamber was mounted and water was filled in it to provide uniform confinement. The soil specimen was subjected to a confining fluid (water) pressure in a triaxial chamber. As the soil specimen is surrounded by water, it was subjected to the same pressure as that of the water around it. The soil specimen was loaded by the top and bottom plates. The normal load and the displacement were recorded in the prepared paper format since the triaxial testing machine is manual. The specimen loading was stopped when the reading of the dial gauge is constant. This may take 20 to 30 minutes. The failure was observed by the bulging of the soil specimen.

2.10.3 Determination of Soil Modulus of Elasticity from UU Triaxial Test Result

The static (monotonic) Young's modulus was determined from the stress - strain graph after making any necessary zero strain adjustments. The secant modulus was determined to half of the maximum deviator stress [50].

Shear strength and modulus of elasticity of soil can be measured from deviatoric stress versus axial strain result obtained from UU triaxial test. Initially, the deviatoric stress increased rapidly with the increase in axial strain. It formed a smooth curve as the axial strain increased. Typically the deviatoric stress at the peak of the stress-strain curve is taken as the shear strength. However, a well-defined peak was not observed. Failure can be taken as the stress value which corresponds to the maximum deviatoric stress attained or the deviatoric stress at 15% axial strain. The axial strain at 50% of the deviatoric stress was found and this point is highlighted on the curve. A line was drawn through the origin and this point. The slope of this line is found by regression and this slope equivalent to Elastic modulus (E) as shown in figure 5.1 below [22]. The secant Mr value obtained by this method is used for calculation of structural number (SN) from excel solver as shown in part 4.3

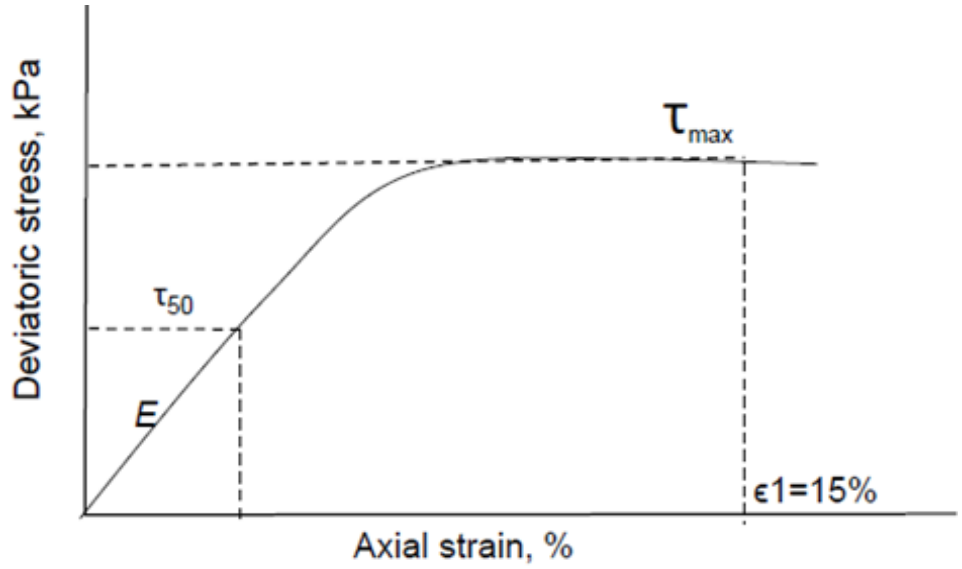


Figure 2.1 Criteria for calculating Young's modulus, E.

2.11 Conclusion from Literature Review

Among the above methods of determining the structural layer coefficients, the AASHTO empirical method of Material Properties Characterization method is used since, among all the factors, material type and material properties have the highest impact and account for the layer coefficient. Also, AASHTO 1993 design guide proposes the resilient modulus (Mr) of the material since it is not only a measure of stiffness but also can be an indicator of the strength of the material.

Among the above method of determining m, the drainage coefficient from the AASHTO drainage coefficient table is used to be used as an initial value. Then, drainage coefficient from SN determined from resilient modulus value is determined for sub–base and base material. From these results, drainage coefficient from ratios of the layer coefficient of Road Test granular material adjusted for specific site conditions (a site) to the layer coefficient of the Road Test material for Road Test conditions (a road test). Finally, all values from the three methods are compared.

CHAPTER 3 RESEARCH METHODOLOGY AND SAMPLING METHODS

3.1 Background of the study area

3.1.1 Descriptions of the study area

For this paper work, two under-construction projects namely Shiro Meda – Kidane Mihiret Hamere Nooh (SM-KMHN) and Ambesa Garage - Unity University - Mebrat Haile (AG - UU - MH) are selected randomly in Addis Ababa.

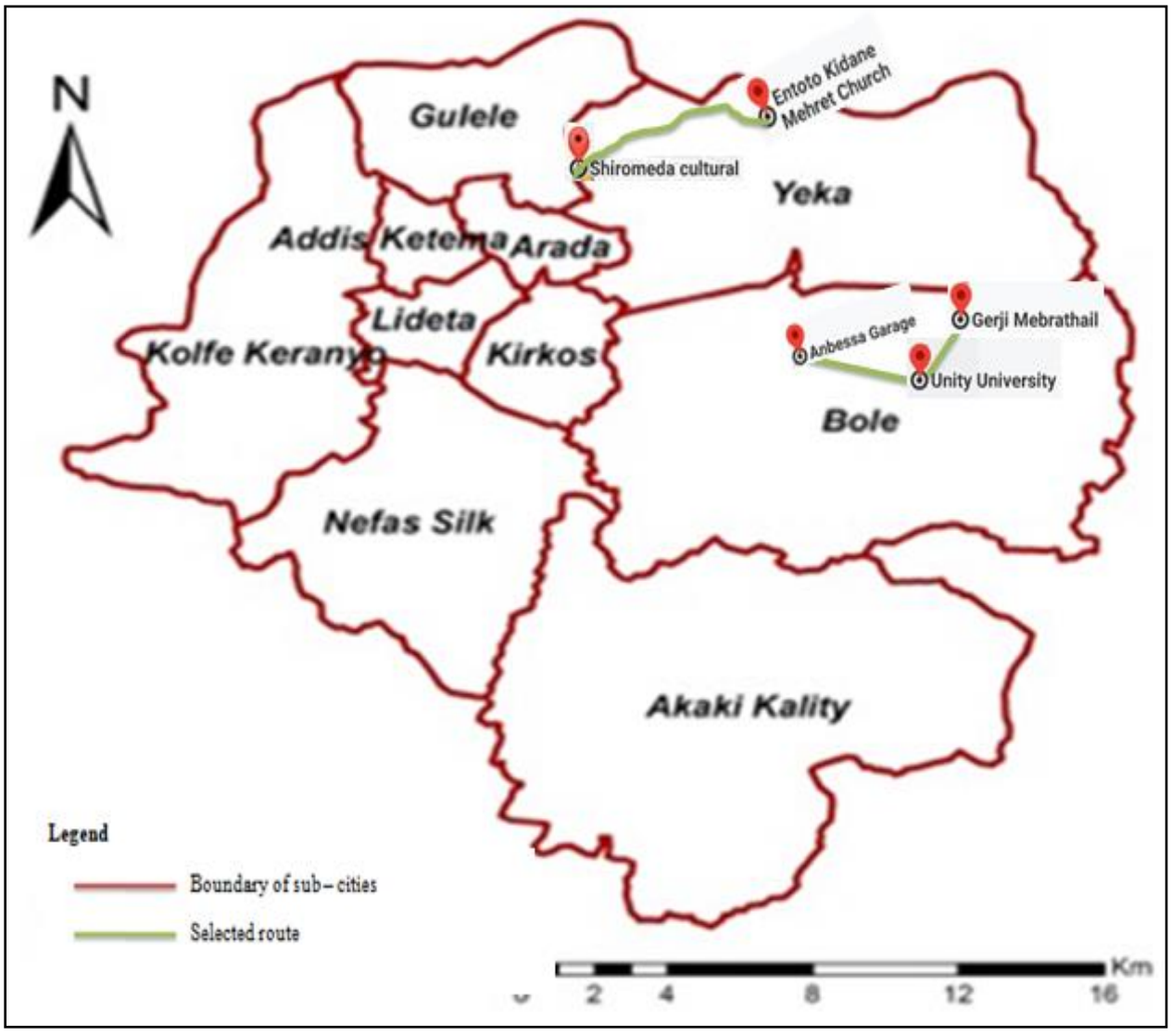


Figure 2.1 Specific Locations of study

3.1.2 Rain fall of the study area

In Addis Ababa, the rainfall intensity variation is attributed to differences in Topography. The high elevated areas such as the Entoto receive relatively greater precipitation than lowland areas around Bole and Akaki. According to National Meteorological Services Agency (NMSA)

located Addis Ababa Observatory (Teklehaymanote) and Entoto Stations data from the mean monthly rainfall the months from May to September receives the highest rainfall (Table 3.1). The annual rainfall for the year is about 1275mm. According to table 3.2, the years 2016, 2013and 2010 received the highest rainfall as compared to the rest of the years [41].

Table 3.1 Mean Monthly Rainfall (Source: National Meteorological Services Agency)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Seb	Oct	Nov	Dec
Mean Monthly Rainfall	10	22	47	71	104	151	307	302	201	37	20	3

Table 3.2 Mean annual rainfall (Source: National Meteorological Services agency)

year	2016	2015	2014	2013	2012	2011	2010	2009	2008	2007	2006
Mean annual Rainfall.mm	141.6	92	94.3	125.7	104.6	81.9	117.9	106.3	103.5	102.1	112.4

3.1.3 Temperature of the Study Area

The mean monthly maximum and minimum temperature records of the National Meteorological Services Agency (NMSA) for the last ten years between 2006 and 2016 were utilized to calculate the annual average temperature. Furthermore, Entoto station which is located at a higher elevation records the smallest amount of average annual temperature [41].

Table 3.3 Maximum and Minimum Annual Temperature (2006-2016) Source: NMSA

Average annual Temperature (°C)	Year										
	2016	2015	2014	2013	2012	2011	2010	2009	2008	2007	2006
Minimum	11.4	10.2	10.5	10.1	9.8	10.0	9.7	9.5	10.4	10.2	9.2
Maximum	22.4	21.4	21.4	20.7	21.2	21.1	20.0	21.8	17.0	20.7	22.2

3.1.4 Drainage of the study area

The streams of Addis Ababa drain towards the south from Entoto ridge, South-east from Mount Wechecha and mount Furi and towards southwest from Mount Yerer and other elevated areas of the streams of Addis Ababa drain towards south from Entoto ridge, southeast from Mount

Wechecha, and Mount Furi and towards southwest from Mount Yerer and other elevated areas of the eastern outskirts of the city. The potential streams in the city are Little Akaki, Bantyiketu, Kurtume, Kebena, Ginifile, and Big Akaki. Streams are dense with deep valleys on top of mountains such as Entoto ridge forming radial and dendritic drainage patterns. There is a general decrease in elevation from north to the south direction [41].

3.2 Methodology of the thesis

3.2.1 Data Collection Methods

This study is focused on recalibration of the structural layer coefficient and drainage coefficient values for a new and processed base and sub-base pavement material in Addis Ababa by the AASHTO material characterization method. The methods followed to achieve the specified objectives include:

- **Collecting primary data**

Primary data's are those data's directly collected from site or from test data. The primarily datas obtained from the laboratory test for this thesis work are Atterberg limit, proctor test, specific gravity, CBR , and UU triaxial test result. For this two under-construction projects namely Shiro Meda – Kidane Mihiret Hamere Nooh (SM-KMHN) and Ambesa Garage - Unity University - Mebrat Haile (AG - UU - MH) are selected randomly assuming soil variation is present in Addis Ababa to projects. After this, Sample collections were made purposely considering some soil variation visually from left and right carriageway for sub - grade (14 samples), sub - base (14 samples) and base (4 samples) from both projects to conduct laboratory testing. Accordingly, 14 samples of sub - grade, 14 samples of sub - base and 4 samples of base from both projects are collected.

- **Collecting secondary data**

Secondary data are those data collected from previous records that were collected or conducted at the concerned site by different stakeholders. The secondary data's used for this thesis work are AADT, thickness of each layer of pavement and geometric standard of the two road projects. AADT is obtained from AACRA, 2004 manual as function of Road functional classification. For this thesis work the road functional are Collector Street (CS) and Sub-arterial Street (SAS). Thickness of each layer of pavement and geometric standard of the two road projects obtained from the Engineering report done by the Best consulting Engineers plc.

- Analysis (determination) of the data to meet the specific objectives

The data which have collected from the laboratory test are properly compiled and made ready in Micro Soft excel for easy use of computer processing. The Software that fit for determination of regression coefficient (K1.K2 and K3) is STATA from LL, PL, OMC, γd and Sg. This software has been selected among the available tools for it determines many parameters at one time. Also, The Matrix is used for determining regression coefficients (K1, K2, and K3) from Mr values obtained from graph of UU triaxial test. Also, logarithmic regression is used to determine the correlation of structural layer coefficient (a) and Resilient Modulus (Mr).

3.2.2 Statistical Methods used

This research use a method of Evaluation research methodology in which it tries to find a solution or recommendation for the current problem in structural layer coefficient and drainage coefficient determination. This type of research is applied research to solve practical problems and apply to specific real situation to check the system is working.

This research tries to use analytical research approach which means it uses facts or information already available in the concerned bodies related to structural layer coefficient.

Also, quantitative research method is used as it is based on measurement and testing. It also involves collecting and analyzing numerical data.

3.3 Sampling Method

Two under construction project in Addis Ababa are selected namely Shiro Meda - K/Mihiret H/Nooh and Ambessa Garage - Unity University - Mebrat Haile. Under construction projects are selected because it is simple to get a sample of pavement material layer by layer during construction at each stage. Two projects are selected to see the structural layer coefficient values obtained is compared for two projects to each other and also these values are compared with the values of AASHTO, ERA, and AACRA.A sufficient amount of subgrade, sub - base, and base pavement material sample is collected from different stations selected purposively on the left and right carriageway for each project. Based on this 4 (four) location (stations) are selected for Shiro Meda - K/Mihiret H/Nooh and 3(three) location (stations) for Ambessa Garage - Unity University - Mebrat Haile project.

The sample for sub-grade, sub-base, and base is taken from the top of sub-grade, sub-base, and base layer respectively at different stations from each lane for both projects. Sub-grade material sample is collected at end of sub-grade excavation. For sub-base and base pavement material after the sample is dumped on site, scarified, and ready for compaction.

Figure 3.3 Sampling stages for sub-grade, sub –base and base pavement material

Table 3.4 Sampling plan for test

Projects	pavement Material Type	Locations (station)	Number of samples collected from Right carriageway	Number of samples collected from Left carriageway	Total
Shiro Meda – Kidane Mihiret Hamere Nooh	Sub -grade	0 + 050, 0 + 560, 0 +960 and 1+380	4	4	8
	Sub -base	0+050, 0+560, 0 +960 and 1+380	4	4	8
	Base	0+980	1	1	2
Ambessa Garage - Unity University - Mebrat Haile	Sub -grade	0+120,0+380 and 0+740	3	3	6
	Sub -base	0+120,0+380 and 0+740	3	3	6
	Base	0+460	1	1	2

CHAPTER 4 LABORATORY TEST METHODS AND TEST RESULT

4.1 Laboratory Test Methods

To conduct the recalibration of structural layer coefficient and drainage coefficient for sub-base and base different test of soil and aggregate is done. These tests are particle size analysis, liquid limit (LL), plastic limit (PL), proctor test (MDD and OMC), specific gravity (Sg), California bearing ratio (CBR), and triaxial test of unconsolidated undrained (UU). Liquid limit, plastic limit, proctor test (MDD and OMC), and specific gravity are conduct to determine material constants (K_1, K_2, and K_3) and to determine different parameters used for the next tests such as California bearing ratio and triaxial test of unconsolidated undrained. Also, tests are done to check the property of the material taken as a sample from the selected project are within the range of the property of the material within the manuals such as AACRA, ERA, and AASHTO.

First, the following laboratory tests are performed in this thesis work;-

- Soil and aggregate pavement materials Gradation test using AASHTO T 27
- Soil and aggregate pavement materials liquid limit using AASHTO T 89
- Soil and aggregate pavement materials plastic limit using AASHTO T 90
- Soil and aggregate materials Proctor test (for MDD and OMC result) AASHTO T 99
- Soil and aggregate pavement materials specific gravity using AASHTO T 100
- Soil and aggregate pavement materials one point CBR tests were performed in accordance with AASHTO T 193.

Secondly, an unconsolidated undrained (UU) triaxial compression test is done. The triaxial unconsolidated undrained (UU) test is done to determine axial stress (σ_1) and confining stress (σ_3). Also from unconsolidated undrained (UU) test graph of deviator versus axial stress draw to determine resilient modulus (Mr), Confining stress (σ_3) is taken to be 100KN, 200KN, and 300KN for countries near tropical.

The sample for unconsolidated undrained (UU) is prepared at optimum moisture content and dry density obtained from proctor test results at confining pressures of 100, 200, and 300 Kpa. Three replicates samples were prepared at optimum moisture content for each confining pressure for each sample. Thus a total of 84 UU triaxial tests were conducted for this thesis work as show by the test plan table below.

Table 4.1 Plan for triaxial UU test

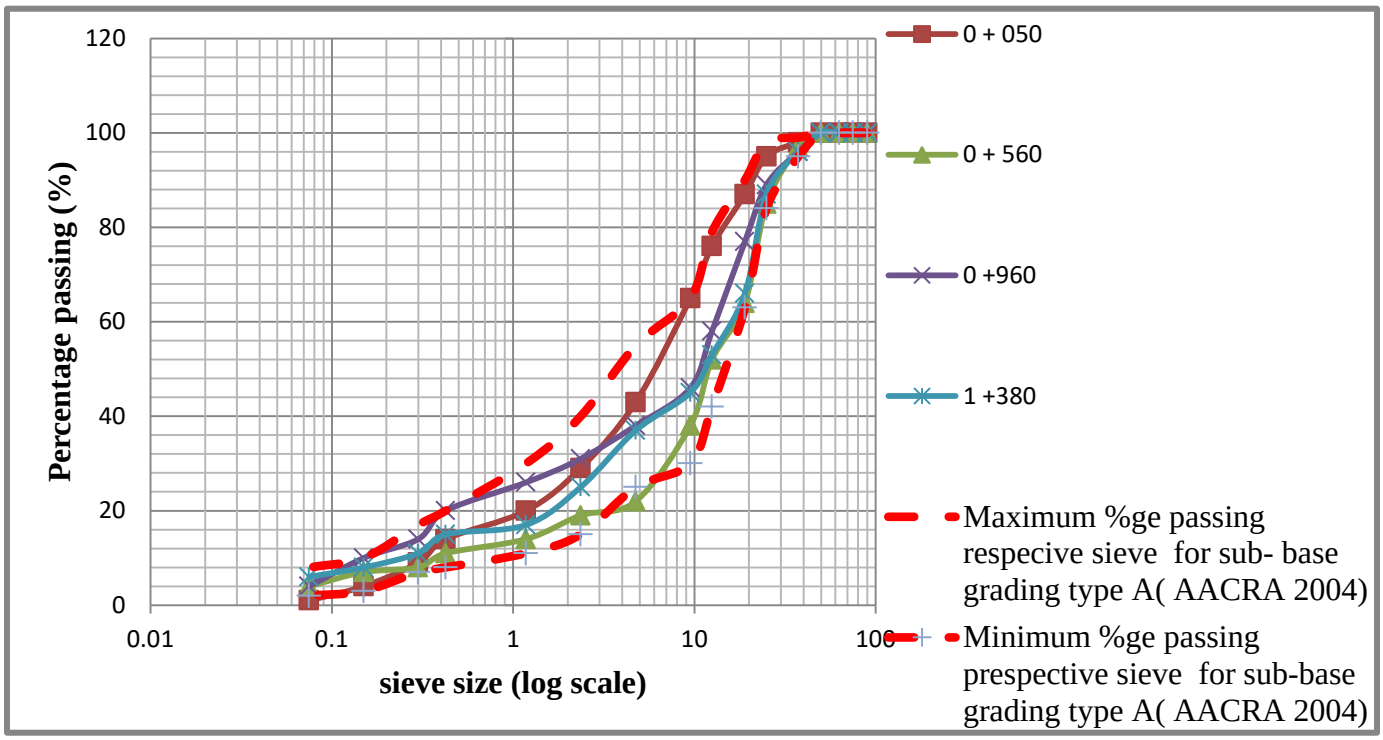
Location (project)		Left		Right	
		Sub –grade	Sub – base	Sub - grade	Sub -base
Shiro Meda – Kidane Mihiret Hamere Nooh	Number of sample	4	4	4	4
	Number of OMC for each sample	1	1	1	1
	Number of confinement for each OMC	3	3	3	3
	Number of UU test conducted **(Sub –total)**	4*1*3 =**12**	4*1*3 =**12**	4*1*3 =**12**	4*1*3 =**12**
Ambessa Garage - Unity University - Mebrat Haile	Number of sample	3	3	3	3
	Number of OMC for each sample	1	1	1	1
	Number of confinement for each OMC	3	3	3	3
	Number of UU test conducted **(Sub –total)**	3*1*3 = **9**	3*1*3 = **9**	3*1*3 = **9**	3*1*3 = **9**
Total		21	21	21	21

Unconsolidated undrained (UU) triaxial compression test is performed according to the test procedure AASHTO T-234. It is performed for sub-grade and sub-base only since the layer coefficient for sub-base and base layer is based on top of bottom layer resilient modulus.

To avoid failure due to buckling the height (H) to diameter (D) ratio is 2. Therefore, the dimension of the cylindrical specimen is 38mm diameter and 76mm height. The specimen is divided into three for the three confinement levels i.e. 100, 200, and 300Kpa levels. First, one-third of the sample is again divided into three and compacted into the mold by tamping in three levels. The specimen was extruded from the mold with the specimen extruding device. The dimensions were also measured again to make sure that the specimen was accurate in size and shape. This process was repeated for another two remaining samples to complete the specimen. This method provided a nearly consistent specimen in terms of bulk density. Take some soil samples to determine the moisture content of the specimen; Soil was oven-dried at 110ºC for 24 hours. Some test preparation methods and equipments used are shown in appendix J.

4.2 Laboratory Test Result

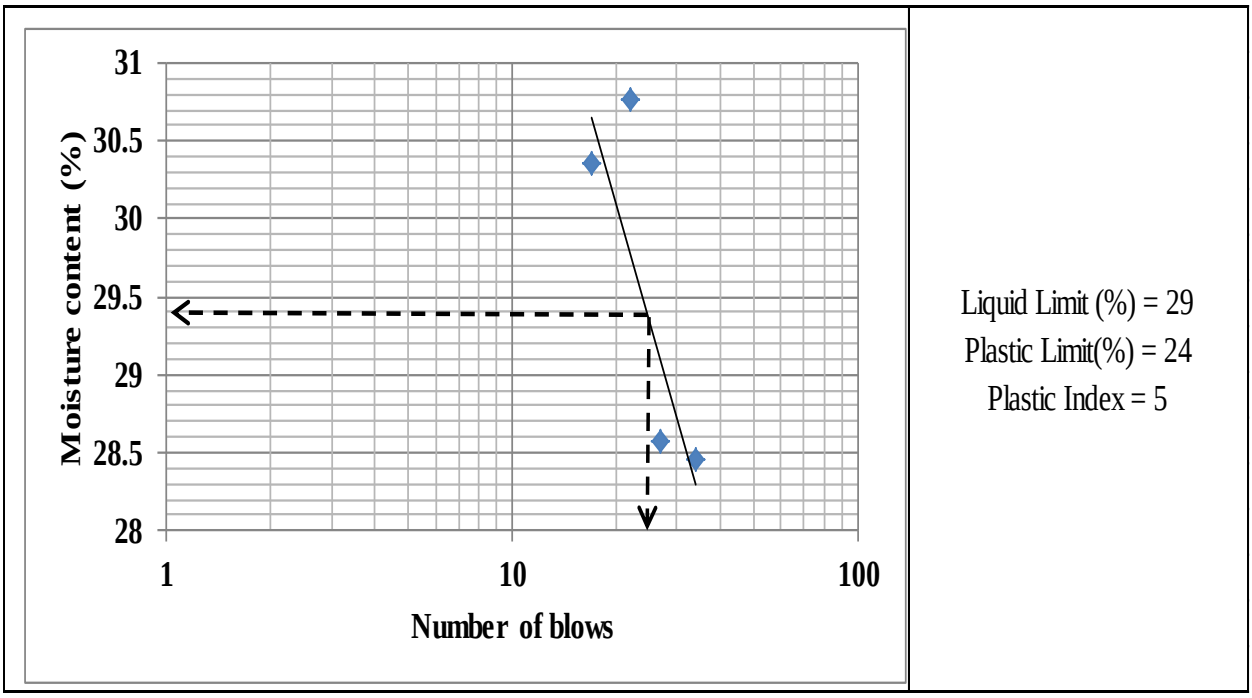
Some typical test results are presented in this section and some will be presented in the appendix part at the end of this material.



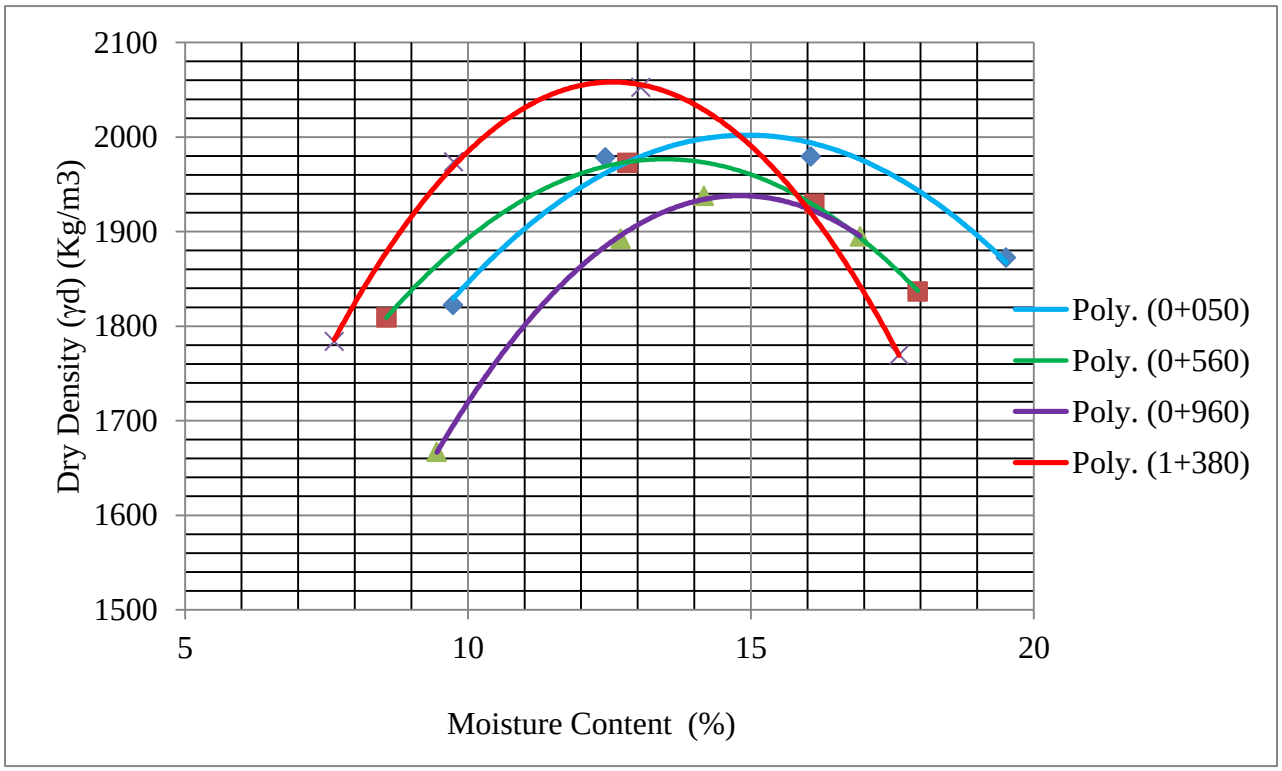
Graph 4.1 Summary of sieve analysis for sub-base pavement materials for Shiro Meda – Kidane Mihiret Hamere Nooh site for right carriageway.

Table 4.2 Liquid Limit and Plastic limit Result for Sub - base Pavement Material of Kidane Mihiret - Hamere Nooh project of 0 + 050 for left carriageway.

	Liquid Limit				Plastic Limit	
Container number	99	77	66	88	9	60
Number of blows	17	22	27	34		
Mass of Empty container (A)	15.5	15.5	15.4	18.6	15.6	15.9
Mass of container + Wet soil (B)	56.3	54.6	57.7	52.9	25.5	24.6
Mass of container + Dry soil(C)	46.8	45.4	48.3	45.3	23.7	22.8
Mass of wet soil (D = B – A)	40.8	39.1	42.3	34.3	9.9	8.7
Mass of dry soil (E = C – A)	31.3	29.9	32.9	26.7	8.1	6.9
Moisture content([D –E]/E *100)	30.35%	30.77%	28.57%	28.46%	22.22%	26.09%
	Average PL(%) = 24.16					



Graph 4.2Number of blows vs Moisture content for plastic limit determination for sub - base Pavement Material of Kidane Mihiret - Hamere Nooh project of 0 + 050 for left carriageway



Graph 4. 1 Moisture Content - Dry Density Relationships for sub - base Pavement Material of Shiro Meda - Kidane Mihiret Hamere Nooh project for Right carriageway of all stations.

Table 4. 3 Coarse Aggregate SG Observation Sheet for Station 0 + 050 for SM - KMHN

SN	Description	Test Number	
		I	II
1	Weight Of Sample	1000	1000
2	Weight Of Vessel + Sample + Water (A)	1690.3	1658.0
3	Weight Of Vessel + Water (B)	1074.5	1074.5
4	Weight Of Saturated & Surface Dry Sample (C)	977.5	961.1
5	Weight Of Oven Dry Sample (D)	952.9	936.40
6	SG=D/C-(A -B)	2.6	2.50

Specific Gravity (Sg) = (2.6+2.5)/2=**2.55**

Table 4. 4 Fine Aggregate Observation Sheet for Station 0 + 050 for SM - KMHN

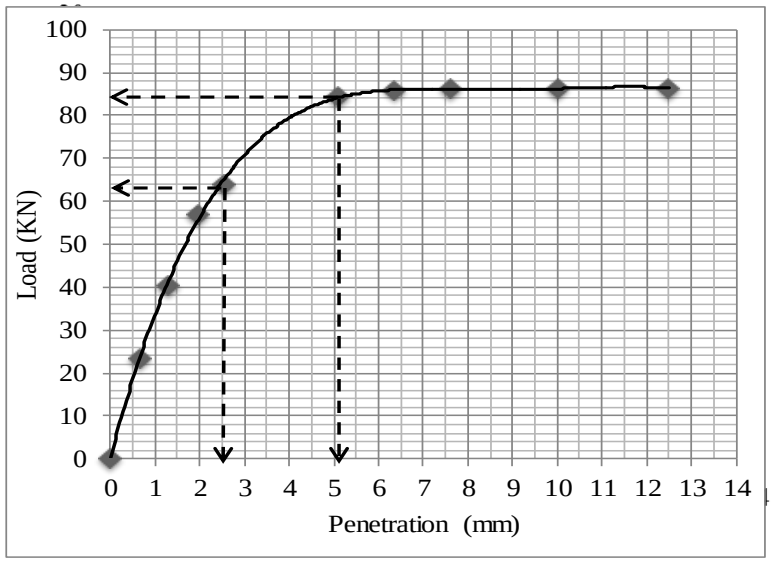
DESCRIBTION		Test Number	
		I	II
1	Weight Of Sample (Wo)	30	30
2	Weight Of Pycnometer + Water (WA)	169.24	169.24
3	Weight Of Pycnometer +Water + Sample (WB)	187.66	187.52
4	$SG = \frac{Wo}{Wo + (Wa - WB)}$	**2.59**	**2.56**

Specific Gravity (Sg) = (2.59+2.56)/2=**2.58**

The specific gravity of the material is the weighted average of the fine aggregate and coarse aggregate result i.e. SG = [(2.58*30) + (2.55*1000)]/1030= **2.55**

Penetration data

Test NO:-0+980		Ring constant (f)= 0.02415KN/DIV	
Penetration	Load dial reading (Div *5)	Total load *f	CBR
0	0	0	
0.64	977	23.5946	
1.27	1669	40.3064	
1.97	2348	56.7042	
2.54	2650	63.9975	91.4%
5.08	3498	84.4767	80.5%
6.35	3553	85.8049	
7.62	3565	86.0948	
10	3569	86.1914	
12.5	3573	86.288	



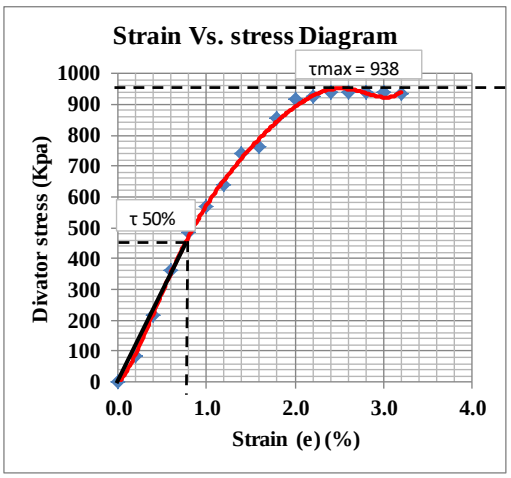
Graph 4. 2 Base pavement material California bearing ratio (CBR) result for Kidane Mihiret - Hamere Nooh for Station 0 + 980 of Right carriageway

$$CRB = \frac{\text{Load at 2.54mm}}{\text{standard Load at 2.54mm}} * 100\% = \frac{64.0KN}{70KN} * 100\% = 91.4\%$$

$$CRB = \frac{\text{Load at 5.08mm}}{\text{standard Load at 5.08mm}} * 100\% = \frac{84.5KN}{105KN} * 100\% = 80.5\%$$

Since CBR At 2.54mm > CBR at 5.08mm, CBR of the sample is equal to **91.4%**

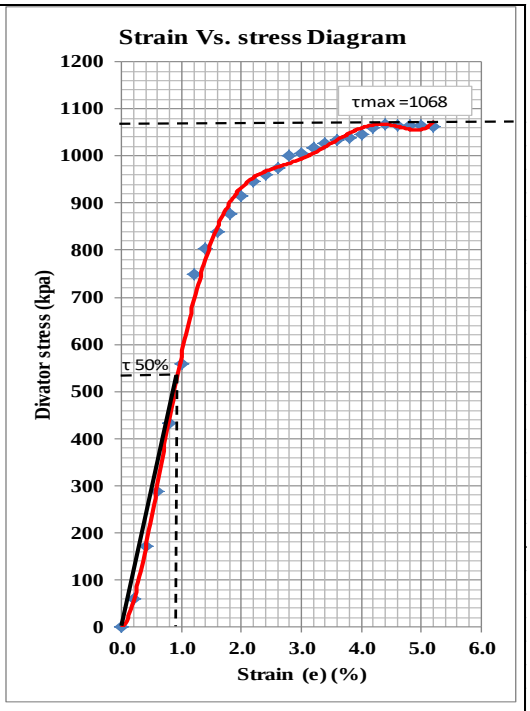
strain Dial reading mm	change in h at failure mm	strain (e) =dh/H0	original area	Corected Area Ao/(1-e)	Devator dial reading	devator stress kn	devator kn/m2
0	0	0.0000	0.001075	0.00108	0	0.0000	0
20	0.2	0.0027	0.001075	0.00108	45	0.0923	86
40	0.4	0.0054	0.001075	0.00108	114	0.2337	216
60	0.6	0.0081	0.001075	0.00108	192	0.3936	363
80	0.8	0.0108	0.001075	0.00109	258	0.5289	487
100	1.0	0.0135	0.001075	0.00109	302	0.6191	568
120	1.2	0.0162	0.001075	0.00109	342	0.7011	641
140	1.4	0.0189	0.001075	0.00110	395	0.8098	739
160	1.6	0.0216	0.001075	0.00110	408	0.8364	761
180	1.8	0.0243	0.001075	0.00110	460	0.9430	856
200	2.0	0.0270	0.001075	0.00111	494	1.0127	916
220	2.2	0.0297	0.001075	0.00111	501	1.0271	927
240	2.4	0.0324	0.001075	0.00111	508	1.0414	937
260	2.6	0.0351	0.001075	0.00111	510	1.0455	938
280	2.8	0.0378	0.001075	0.00112	511	1.0476	937
300	3.0	0.0405	0.001075	0.00112	512	1.0496	937
320	3.2	0.0432	0.001075	0.00112	512	1.0496	934



Origin coordinate		Coordinte at point	
(0,0)		(0.8, 469)	
X1	X2	Y1	Y2
0	0.8	0	469
slope = E = Mr			586.25

Graph 4. 3 The result of UU test for Resilient Modulus (Mr) calculation for sub-base of Sh/Meda - K/Mihiret H/Nooh project for 0 + 050 station of left carriageway for σ_3 = 100kpa

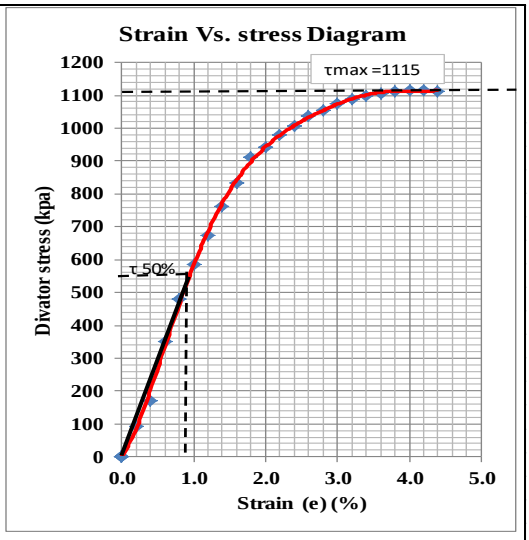
strain Dial reading mm	change in h at failure mm	strain (e) =dh/H0	original area	Corected Area Ao/(1-e)	Devator dial reading	devator stress kn	devator kn/m2
0	0	0.0000	0.001075	0.00108	0	0.0000	0
20	0.2	0.0027	0.001075	0.00108	32	0.0656	61
40	0.4	0.0054	0.001075	0.00108	90	0.1845	171
60	0.6	0.0081	0.001075	0.00108	152	0.3116	287
80	0.8	0.0108	0.001075	0.00109	229	0.4695	432
100	1.0	0.0135	0.001075	0.00109	297	0.6089	559
120	1.2	0.0162	0.001075	0.00109	399	0.8180	748
140	1.4	0.0189	0.001075	0.00110	430	0.8815	804
160	1.6	0.0216	0.001075	0.00110	450	0.9225	839
180	1.8	0.0243	0.001075	0.00110	472	0.9676	878
200	2.0	0.0270	0.001075	0.00111	493	1.0107	915
220	2.2	0.0297	0.001075	0.00111	512	1.0496	947
240	2.4	0.0324	0.001075	0.00111	520	1.0660	959
260	2.6	0.0351	0.001075	0.00111	530	1.0865	975
280	2.8	0.0378	0.001075	0.00112	545	1.1173	1000
300	3.0	0.0405	0.001075	0.00112	550	1.1275	1006
320	3.2	0.0432	0.001075	0.00112	558	1.1439	1018
340	3.4	0.0459	0.001075	0.00113	564	1.1562	1026
360	3.6	0.0486	0.001075	0.00113	570	1.1685	1034
380	3.8	0.0514	0.001075	0.00113	574	1.1767	1038
400	4.0	0.0541	0.001075	0.00114	580	1.1890	1046
420	4.2	0.0568	0.001075	0.00114	589	1.2075	1059
440	4.4	0.0595	0.001075	0.00114	595	1.2198	1067
460	4.6	0.0622	0.001075	0.00115	596	1.2218	1066
480	4.8	0.0649	0.001075	0.00115	597	1.2239	1064
500	5.0	0.0676	0.001075	0.00115	599	1.2280	1065
520	5.2	0.0703	0.001075	0.00116	600	1.2300	1064
540	5.4	0.0730	0.001075	0.00116	601	1.2321	1062
560	5.6	0.0757	0.001075	0.00116	601	1.2321	1059
580	5.8	0.0784	0.001075	0.00117	602	1.2341	1058



Origin coordinate		Coordinte at point	
(0,0)		(0.9, 534)	
X1	X2	Y1	Y2
0	0.9	0	534
slope = E = Mr			593.333

Graph 4. 4 The result of UU test for Resilient Modulus (Mr) calculation for sub - base of Sh/Meda - K/Mihiret H/Nooh project for 0 + 050 station of left carriageway for σ_3 = 200kpa

strain Dial reading mm	change in h at failure mm	strain (e) =dh/H0	original area	Corected Area Ao/(1-e)	Devator dial reading	devator stress kn	devator kn/m2
0	0	0.0000	0.001075	0.00108	0	0.0000	0
20	0.2	0.0027	0.001075	0.00108	49	0.1005	93
40	0.4	0.0054	0.001075	0.00108	90	0.1845	171
60	0.6	0.0081	0.001075	0.00108	185	0.3793	350
80	0.8	0.0108	0.001075	0.00109	254	0.5207	479
100	1.0	0.0135	0.001075	0.00109	312	0.6396	587
120	1.2	0.0162	0.001075	0.00109	360	0.7380	675
140	1.4	0.0189	0.001075	0.00110	408	0.8364	763
160	1.6	0.0216	0.001075	0.00110	446	0.9143	832
180	1.8	0.0243	0.001075	0.00110	490	1.0045	912
200	2.0	0.0270	0.001075	0.00111	507	1.0394	941
220	2.2	0.0297	0.001075	0.00111	529	1.0845	979
240	2.4	0.0324	0.001075	0.00111	546	1.1193	1007
260	2.6	0.0351	0.001075	0.00111	564	1.1562	1038
280	2.8	0.0378	0.001075	0.00112	575	1.1788	1055
300	3.0	0.0405	0.001075	0.00112	587	1.2034	1074
320	3.2	0.0432	0.001075	0.00112	596	1.2218	1087
340	3.4	0.0459	0.001075	0.00113	604	1.2382	1099
360	3.6	0.0486	0.001075	0.00113	610	1.2505	1106
380	3.8	0.0514	0.001075	0.00113	615	1.2608	1112
400	4.0	0.0541	0.001075	0.00114	618	1.2669	1115
420	4.2	0.0568	0.001075	0.00114	620	1.2710	1115
440	4.4	0.0595	0.001075	0.00114	621	1.2731	1114



Origin coordinate		Coordinte at point	
(0,0)		(0.9, 557.5)	
X1	X2	Y1	Y2
0	0.9	0	557.5
slope = E = Mr			619.444

Graph 4. 5 The result of UU test for Resilient Modulus (Mr) calculation for sub - base of Sh/Meda - K/Mihiret H/Nooh project for 0 + 050 station of left carriageway for σ_3 = 300kpa

Table 4. 5 Summaries of all laboratory results sub - grade and sub - base

Location (Station) of sampling			LL	PL	PI	OMC	γd	SG	CBR (%)	ERA CBR specification	AASHTO soil classification	Resilient Modulus (Mr.) for confinement (σ3)		
												σ3 = 100 Kpa	σ3 =200 Kpa	σ3 = 300 Kpa
Sh/Meda – Kidane Mihiret Hamere Noh (sub - grade)	Left	0 +050	57	33	24	23.5	1.520	2.64	17.10	≥ 2 %	A -2 -7	366.00	347.08	461.82
		0 + 560	62	31	31	24.5	1.530	2.62	12.86		A -2 -7	362.78	418.00	498.89
		0 + 960	63	34	29	26.30	1.532	2.61	21.43		A -2 -7	320.00	323.50	345.50
		1 + 380	56	32	24	22.7	1.540	2.58	12.86		A -2 -7	244.54	213.44	255.00
	Right	0 +050	59	31	28	21.0	1.493	2.63	12.50		A -2 -7	227.92	250.77	297.86
		0 + 560	59	31	28	20.0	1.513	2.58	9.10		A -2 -7	386.67	281.00	284.58
		0 + 960	59	32	27	20.0	1.533	2.59	10.9		A -2 -7	346.67	389.00	306.79
		1 + 380	58	33	25	24.0	1.514	2.61	12.80		A -2 -7	328.57	303.64	303.85
A/garage-Unity Un.– M/Haile(Sub - grade)–	Left	0 + 120	61.0	32	29	23.0	1.560	2.56	11.73		A -2 -7	290.00	326.67	333.50
		0 + 380	57.0	29	28	24.0	1.525	2.61	18.87		A -2 -7	305.50	317.27	346.82
		0 + 740	56.0	28	28	25.5	1.460	2.61	14.00		A -2 -7	305.92	313.33	356.67
	Right	0 + 120	62	32	30	17.5	1.548	2.56	13.56		A -2 -7	293.33	355.56	364.44
		0 + 380	64	30	34	18.6	1.506	2.61	14.71		A -2 -7	312.50	315.42	347.50
		0 + 740	60	27	33	19.2	1.516	2.58	15.29		A -2 -7	332.50	336.5	343.64
Sh/Meda – K/Mihiret H/Noh (Sub - base)	Left	0 +050	29	23	6	13.4	2.040	2.64	49.14	≥ 30 %	A - 2 -5	586.25	593.33	619.44
		0 + 560	29	20	9	11.4	1.997	2.50	35.71		A -2 -5	306.50	397.86	417.27
		0 + 960	30	19	11	13.8	1.868	2.50	38.71		A - 2 -5	219.12	215.48	345.77
		1 + 380	33	23	10	8.6	2.080	2.55	43.19		A - 2 -5	303.24	247.92	332.78
	Right	0 +050	29	23	6	15.0	2.000	2.55	37.88		A - 2 -5	224.64	373.33	400.28
		0 + 560	30	21	9	13.4	1.980	2.65	48.71		A - 2 -5	435.00	579.29	586.15
		0 + 960	31	24	7	14.5	1.940	2.60	42.44		A - 2 -5	262.73	575.83	657.50
		1 + 380	31	25	6	12.5	2.220	2.60	40.1		A - 2 -5	320.77	375.00	373.21
A/garage-Unity Un.– M/Haile(Sub - base)	Left	0 + 120	28	20	8	9.7	1.925	2.55	44.02		A - 2 -5	220.42	273.44	277.50
		0 + 380	31	19	12	10	1.952	2.54	50.30		A - 2 -5	396.36	432.31	468.33
		0 + 740	30	22	8	10.5	2.050	2.54	40.10		A - 2 -5	532.50	598.33	621.43
	Right	0 + 120	35	28	7	11.2	1.900	2.55	51.20		A - 2 -5	305.91	353.33	367.65
		0 + 380	30	21	9	10.8	1.990	2.53	49.40		A - 2 -5	431.25	653.75	635.42
		0 + 740	34	22	12	11.0	2.000	2.57	57.00		A - 2 -5	349.44	442.92	451.00

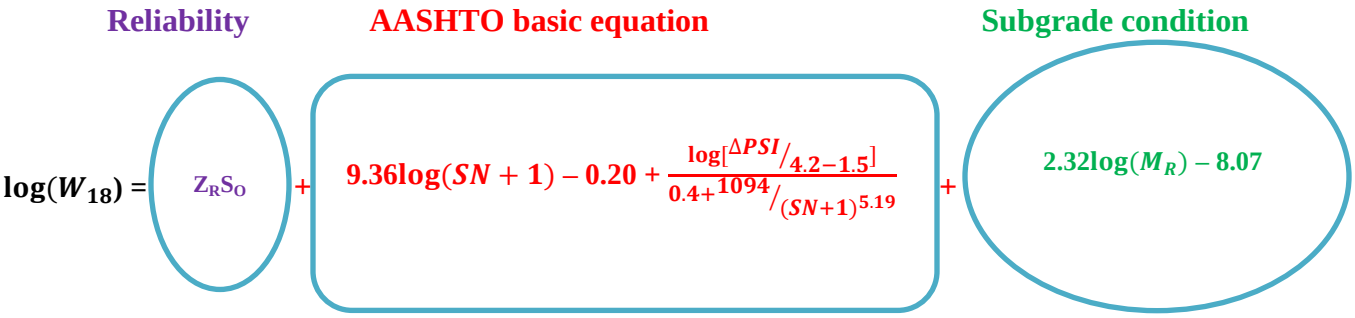
Table 4. 6 Summaries of all laboratory results for base

Location (Station) of sampling				LL	PL	PI	OMC	γd	SG	CBR (%)	ERA CBR specification	AASHTO soil classification
Base	Sh/Meda – K/Mihiret H/Noh	Left	0+980	23	19	4	9.8	2.100	2.75	82.00	≥ 80 %	A - 1- a
		right	0+980	23	18	5	12.5	2.240	2.60	82.00		A - 1- a
	A/garage – Unity Univ. - M/Haile	left	0+460	21	16	5	11.0	2.072	2.65	84.70		A - 1- a
		right	0+460	22	17	4	11.2	2.078	2.64	87.70		A - 1- a

CHAPTER 5 RECALIBRATION OF STRUCTURAL LAYER COEFFICIENT AND DRAINAGE LAYER COEFFICIENT

5.1 Data Needed for Structural layers coefficient determination

Input Data needed for recalibration of structure layer coefficient and drainage layer coefficients in mechanistic-empirical performance-based equation are traffic data (AADT), reliability (R), standard deviation (So), pavement layer thickness for each layer (hi), and overall geotechnical investigation as shown in the following AASHTO 1993 the basic equation for pavement design.



Basic structural layer coefficient Recalibration process follows the following Procedure:

1. Determination of Reliability (Z_R) and Standard Deviation (So)
2. Select the performance level (ΔPSI)
3. Determination of the traffic (ESAL)
4. Calculate the effective subgrade modulus (Mref)
5. Solve for the required SN needed to protect the subgrade (Excel solver is used)

In the following section, these parameters are determined from different sources such AACRA manual, ERA manual, AASHTO 1993, Engineering report of the projects and form different test result.

5.1.1 Determination of Reliability (ZR) and standard deviation (S_O)

Reliability of a pavement design performance process is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design speed [2]. The level of reliability should increase with traffic, traffic sensitivity, and public expectation. The reliability factor decreases the predicted number of ESALs that a particular design can accommodate. Reliability considering 95% confidence interval is 1.65[35].

The estimated overall standard deviations (So) for the case where the variance of projected future traffic is considered (along with the other variance associated with the revised pavement

performance model) are 0.39 for rigid pavements and 0.49 for flexible pavements [2].since the two projects are road with the flexible pavement, 0.49 value is used in this material.

5.1.2 Determination of the Performance Level (Δpsi)

Pavement performance considers structural performance, functional performance, and safety. Structural performance relates pavement physical condition i.e. occurrence of cracking, faulting, raveling or other condition which adversely affects the load-carrying capability of pavement structures or would require maintenance. The function performance of a pavement concerns how well the pavement serves the user. In this context riding comfort or riding quality is the dominant characteristic. In order to quantify riding comfort, the serviceability performance concept is used as the measure of performance [2].

The serviceability of pavement is expressed in terms of the present serviceability index (PSI) obtained from measurements of roughness and distress, e.g., cracking, patching, and rut depth at a particular time during the service life of the pavement [2].

The scale for PSI ranges 0 - 5 with values of 5 represent the highest index and 0 represents the lowest index. The initial serviceability index (Pi) is estimated immediately after construction. Pi 4.2 and 4.5 values are used for flexible and rigid pavement respectively. The terminal serviceability index (Pt) is the lowest acceptable level before resurfacing or reconstruction. An index of 2.5 or 3.0 is used for major and lower classification highways [2]. For this paperwork, Pi of 4.2 and Pt of 2.5 are used since the road projects considered are urban flexible pavement of higher class of CS - 2.

Therefore, $\Delta PSI = p_i - p_t = 4.2 - 2.5 =$ **1.7**

5.1.3 Determination of design $ESAL_S$

The determination of design equivalent standard axle load (ESALs) requires information of Annual average daily traffic (AADT), Growth Factor (G_F), and design period. These concepts are discussed below.

The number of traffic using the road facility over the design period is determined from the traffic count conducted. In the absence of traffic data count, typical values of the traffic and axle load study of AACRA manual, table 6.2 are used for each road classification in Addis Ababa. Road classification of Addis Ababa is based on the infrastructure master plane of the city [1].

The annual average daily traffic (AADT) for Addis Ababa city roads is determined from the functional classification of roads such as freeway, arterial, sub-arterial, Collector Street, and Local Street. The functional classification of the road is in-turn determined from the city's

infrastructure master plan. The functional classification of Shiro Meda - Kidane Mihiret and Ambesa Garage - Unity University - Mebrat Haile Projects is stated on the Engineering report as shown in table 5.1 and table 5.2 respectively. So, the annual average daily traffic (AADT) of the two Projects is shown in table 5.

Table 5. 1 Road class for Shiro Meda –Kidane Mihiret Hamere Nooh project

From station	To station	Road class	Road width
0 + 000	0 + 750	*CS - 2	20
0 + 750	2 + 081.7	*SAS - 2	20

*CS- Collector Street. *SAS- Sub-arterial Street

Table 5. 2 Road class Ambessa garage – unity university – Mebrat Haile

From station	To station	Road class	Road width
0+ 000	1 + 682.0	*CS - 2	20

*CS- Collector Street. *SAS- Sub-arterial Street

Table 5. 3 Typical value for Addis Ababa traffic by percentages and values (AACRA, 2004)

Road classification	AADT (two way)	Typical values				
		Car	light	medium	Heavy	articulated
Freeway	-	-	-	-	-	-
Arterial	9,410 (10,000)	7,506 (80%)	1,600 (17%)	180 (2%)	14 (0%)	110 (1%)
Sub – arterial	8,695 (9,000)	7,730 (89 %)	816 (9%)	86 (1%)	13 (0%)	50 (1%)
Collector	3,320 (4,000)	3,035 (91%)	252 (8%)	25 (1%)	2 (0%)	6 (0%)
Local	2,385 (1,500)	1,348 (97%)	35 (3%)	2 (0%)	0 (0%)	0 (0%)

From table 5.3 and Road class (table 5.1 and table 5.2) above:-

- Collector Street (CS - 2) is a road with two ways AADT of 3320 - 4000.
- Sub-arterial Street (SAS - 2) is a road with two ways AADT of 8695 - 9000

The design periods that will be used for flexible pavement design vary from 8 years to 20 years based on the type of construction proposed. The Pavement Design Engineer can adjust the design period within guidelines based on project - specific conditions and constraints [53]. For most

road projects an economic analysis period of 20 years from the date of opening is appropriate. Whatever time period is chosen for the appraisal of a project, the road will always have some residual value at the end of this period. Since the projects under consideration are in Addis Ababa with Urban collector street (CS - 2) and Sub - Arterial Street (SAS - 2) the design period is 20 years for new granular and semi - rigid pavements [2].

Growth Factor determined as follows:

$G_F = \frac{(1+g)^n - 1}{g}$ Where; - n is the design period = 20 years and g is traffic growth rate = 10 % for Addis Ababa current traffic condition according to AACRA officials.

Then, $G_F = \frac{(1+g)^n - 1}{g} = \frac{(1+0.1)^{20} - 1}{0.1} = \underline{\mathbf{57.27}}$

The traffic information required by the AASHTO design equation includes axle loads, axle configuration, and the number of repetitions (AASHTO 1993). The damaging effect of the passage of an axel of any traffic load can be represented by the number of 18 kip equivalent single axle loads of ESALs. Thus mixed traffic stream of different axle loads and axle configurations into a design traffic number is to convert each expected axle load into an equivalent number of 18 kip single axle loads and to sum these over the design period.

Table 5. 4 Typical ESA factor Values for Addis Ababa traffic (AACRA, 2004)

Vehicle class	AACRA			ERA		
	Typical	Lower	upper	typical	upper	lower
Car	0.03	0.00	0.10	0.00	_	_
Light	0.73	0.39	1.07	0.70	0.30	1.10
Medium	1.31	0.73	1.89	1.70	0.80	2.60
Heavy	1.61	1.05	2.18	1.80	0.80	3.00
Articulated	3.15	2.15	4.14	2.20	1.00	3.00

Because asphalt materials, cement materials, and sub-grade each have different performance relationships it is necessary to determine separately for each material the number of standard axles which will cause the same level of accumulated damage as the actual traffic load spectrum. As shown in table 5.5. The design loading is then calculated as the design number of standard axles for: (AACRA 2004).

Table 5. 5 Adjusted AADTo and ESAL determination for different pavement material.

Material type	Asphalt	Subgrade	Cemented material
Adjusted AADTo	Nsa = 1.1N	Nss = 1.1 N	Nsc = 1.0 N
ESALs	Nsa x 365 x G_F	Nss x 365 X G_F	Nsc x 365 x G_F

Table 5. 6 ESAL determination for Collector Street (CS - 2)

Vehicle type	Current traffic two-way	Current traffic one-way (AADTo)	AADTosa =1.1*AADTo	Growth factor (G_F)	Design traffic ($AADT_f$) = AADTo sa x 365 x G_F	ESALs factor	Design ESALs
Car	3,035	1517	1669	57.27	34,888,024.95	0.03	1,046,640.749
Light vehicle	252	126	139	57.27	2,905,593.450	0.73	2,121,083.213
Medium truck	25	13	15	57.27	313553.250	1.31	410,754.758
Heavy truck	2	1	1	57.27	20,903.550	1.61	33,654.712
Articulated vehicle	6	3	3	57.27	62,710.650	3.15	197,538.548
						W18 = Total Design ESALs	**3,809,671.980**

Table 5. 7 ESAL determination for Sub-Arterial Street (SAS - 2)

Vehicle type	Current traffic two way (AADTo) (AADTo)	Current traffic one –way (AADTo)	AADTosa =1.1*AADTo	Growth factor (GF)	Design traffic ($AADT_f$)= AADTosa x 365 x GF	ESALs	Design ESALs
Car	7,730	3865	4252	57.27	88,871,442.825	0.03	2,666,143.285
Light vehicle	816	408	449	57.27	9,381,513.240	0.73	6,848,504.665
Medium truck	86	43	47	57.27	988,737.915	1.31	1,295,246.667
Heavy truck	13	7	8	57.27	160,957.335	1.61	259,141.309
Articulated vehicle	50	25	28	57.27	574,847.625	3.15	1,810,770.019
						W18 = Total Design ESALs	**12,879,805.945**

5.1.4 Determination of resilient modulus

❖ **Determination of Resilient Modulus (Mr) from UU Triaxial Test Result**

As explained above literature, resilient modulus or Young's modulus is determined from the graph of the stress - strain graph. The secant modulus was determined to 50% of the maximum deviator stress as shown in graphs 4.5, 4.6, and 4.7 as well as appendix F at the end of this thesis.

Table 5. 8 Resilient modulus result Shiro Meda - Kidane Mihiret Hamere Nooh and Ambessa Garage - Unity - Mebrat Haile

Location (Station) of sampling and ESAL		Confinement	Sub - grade (Left and Right Carriageway)		Sub – base (Left and Right Carriageway)	
			Resilient Modulus (Mr) for confinement (σ3)		Resilient Modulus (Mr) for confinement (σ3)	
		Kpa	Kpa	Psi	Kpa	Psi
Shiro Meda – Kidane Mihiret Hamere Noh	0 +050 for 3,809,671.980 ESAL	100	366	53.07	586.25	85.01
		100	227.9	33.05	224.64	32.57
		200	347.1	50.33	593.33	86.03
		200	250.8	36.36	373.33	54.13
		300	319.5	46.30	619.44	89.82
		300	297.9	43.19	400.28	58.04
	0 + 560 for 3,809,671.980 ESAL	100	362.8	52.60	306.50	44.44
		100	311.9	45.20	435.00	63.08
		200	418.0	60.61	397.86	57.69
		200	281.0	40.75	579.29	84.00
		300	340.9	49.40	417.27	60.50
		300	284.6	41.26	586.15	84.99
	0 + 960 for 12,879,805.945 ESAL	100	320.0	46.40	219.12	31.77
		100	346.7	50.27	262.73	38.10
		200	323.5	46.91	215.48	31.24
		200	389.0	56.41	575.83	83.50
		300	345.5	50.10	345.77	50.14
		300	306.8	44.48	657.50	95.34
	1 + 380 for 12,879,805.945 ESAL	100	244.5	35.46	303.24	43.97
		100	328.6	47.64	320.77	46.51
		200	213.4	30.95	247.92	35.95
		200	303.6	44.03	375.00	54.38
		300	255.0	36.98	332.78	48.25
		300	303.9	44.06	373.51	54.16

Location (Station) of sampling and ESAL		Confinement	Sub – grade (Left and Right Carriageway)		Sub - base (Left and Right Carriageway)	
			Resilient Modulus (Mr) for confinement (σ3)		Resilient Modulus (Mr) for confinement (σ3)	
		Kpa	Kpa	Psi	Kpa	Psi
Ambessa Garage -Unity– M/Haile	0 + 120 for 3,809,671.980 ESAL	100	202.45	29.34	220.42	31.96
		100	293.33	42.53	305.91	44.36
		200	269.10	39.00	273.44	39.65
		200	298.77	43.30	353.33	51.23
		300	258.06	37.40	277.5	40.24
		300	364.44	52.84	367.65	53.31
	0 + 380 for 3,809,671.980 ESAL	100	305.50	44.30	396.36	57.47
		100	312.50	45.31	431.25	62.53
		200	317.27	46.00	432.31	62.68
		200	315.42	45.74	653.75	94.79
		300	346.82	50.29	468.33	67.91
		300	347.50	50.39	635.42	92.14
	0 + 740 for 3,809,671.980 ESAL	100	305.92	44.36	532.50	77.21
		100	332.50	48.21	349.44	50.67
		200	313.33	45.43	598.33	86.76
		200	336.50	48.79	442.92	64.22
		300	356.67	51.72	621.43	90.11
		300	343.64	49.83	451.00	65.40

❖ Constitutive Model for Resilient Modulus

A material is considered nonlinear if the value of elastic modulus depends on the state of the stress in the material. While many materials start to exhibit this behavior at very high stress states, the only materials that are considered to be nonlinear are unbound base/sub-base and sub-grade materials [24].

There is not a singular Mr value for a particular soil but rather the modulus is a function of the stress state. The resilient modulus test is inherently complicated; time-consuming, expensive, and/or not available at all. For these reasons, most commercial and design laboratories will not conduct these tests but instead, rely on an empirical relationship. Therefore, it has been recommended that an alternative test be developed to approximate resilient modulus. The alternative test method was designed to be an easier method obtaining an approximate Mr value [40].

The variations in the modulus values of sub-base soil with the number of loading cycles and loading frequency were almost negligible in a practical sense. The resilient modulus was affected by the confinement and the bulk stress. Thus, the bulk stress model was properly applicable for the tested sub-base soils also known as the K - θ model [9, 25, 32, 33]. For sub-grade soils, Mr decreases as the magnitude of the deviatory stress increases, almost independently of confining pressure, and the deviatory stress model known as K - σ_d model is also used in the design [25, 33]. Thus the following constitutive model can be used to determine the resilient modulus. Resilient modulus (Mr) obtained from the constitutive model is either from experience or from other material properties. The potential benefit of estimating Mr from soil physical properties is that the seasonal variations in the Mr can be determined from seasonal changes in the material's properties; however, the effect of stress sensitivity is not captured.

For non-cohesive granular pavement materials such as sub - base and base

$Mr = K_1\theta^{K2}$…….(4.1)

$\theta = \sigma_1 + \sigma_2 + \sigma_3$ and k_1, K_2 are material constants obtained from regression.

For cohesive pavement materials clay subgrade

$Mr = K_1\sigma d^{K2}$ ……..(4.2)

$\sigma_d = \sigma_1 - \sigma_3$ and k_1, K_2 are material constants obtained from regression.

Both equations 1 and 2 have a drawback to account for the effect of confining stress and deviator stress in the pavement structure. Thus, the Uzan model of which combined bulk stress model and

deviator stress model is for every soil type and thus used in this paper work. This equation is called Universal model.

$Mr = K_1\theta^{K2}\ \sigma d^{K3}$……(4.3)

Witczak and Uzan (1988) suggested replacing the deviator stress σ_d with the octahedral shear stress (τ_{oct}) as shown in the following equation. The bulk stress (θ) and the octahedral shear stress were normalized using atmospheric pressure (Pa).

$Mr = k_1\ Pa\ (\theta\ /Pa)^{K2}\ (\sigma_d\ /Pa)^{K3}$ ……(4.4)

Witczak and Uzan in 1992 introduced octahedral shear stress in place of deviator stress in Equation, which provided a better explanation for the stress state of the material, in which the normal and shear stress change during loading. The proposed model is known as the k_1- k_3 model or universal model. The universality of this model stems from its ability to conceptually represent all types of soils from pure cohesive soils to non-cohesive soils. The Witchzak and Uzan model is therefore considered more practical and realistic in the material characterizations [9, 25, and 27].

$Mr = k_1\ Pa\ (\theta\ /\ Pa)^{K2}\ (\tau oct\ /\ pa)^{K3}$…… (4.5)

NCHRP model:-

$Mr = k1\left(\frac{\sigma sum}{pa}\right)^{K2}\left(\frac{\tau oct}{pa}+1\right)^{K3}$. ……..(4.6)

Where, $\tau_{oct} = 1/3\ ((\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^3 + (\sigma_3 - \sigma_1)^2)^{½}$

Considering asymmetry, $\sigma_2 = \sigma_3$ and $\sigma_d = \sigma_1 - \sigma_3$ which result $\tau_{oct} = \frac{\sqrt{2}}{3}\sigma_d$

AASHTO pavement design guide recommends highway agencies to use representative confining and deviator stress in sub - grade and sub - base layers under traffic loading conditions. When simplifying Mr Test procedures it is necessary to investigate the range of confining and deviator stresses resulting from the traffic loading [34]. For tropical countries like Ethiopia, the confining stress is taken to be 100kpa, 200kpa, and 300kpa.

The coefficients k_1, K_2, and K_3 are constants, dependent on material type and physical properties, and are obtained from a regression analysis. Since coefficient k_1 is proportional to Young's modulus, it should always be positive as Mr can never be negative. The coefficient K_2 should be positive, because increasing the volumetric stress produces stiffening or hardening of the material, yielding higher modulus. The coefficient K_3 should be negative because an increase in the shear stress softens the material, thereby yielding a lower modulus. If nonlinear property

coefficients K_2 and K_3 are set to zero, then the model can be simplified as linear elastic. If K_3 is zero, the behavior could be non-linear hardening and if K_2 is zero, the behavior is non-linear softening [9, 25].

In Regression analyses, the more the variables the higher the coefficient of determination (R^2), however, "the more the variables" also mean less practical and less statistically sound expressions. Therefore, the most important and practical factors were included in the final regression equations. Also, in the multiple regression analyses, the risk of collinearity would be eliminated [39].

There are one variable and two coefficient in first three Models and two variables and three coefficients in three models. All models are not linear equations and have to be transferred into a linear equation in order to apply a linear regression analysis. The Mr model above can be linearized as following [55].

Log (Mr) = log (K_1pa) +K_2log (x_1) + K_3log (x_2)

X_1 stands for θ for 4.1 and 4.3 models, σ_d for 4.2 model, θ/pa for 4.4 and 4.5 models. σ_{sum}/pa for 4.6 respectively.

X2 stands for σ_d for 4.3 model, σ_d/pa for model 4.4, τoct/pa for 4.5 for model and (τoct/pa +1) for 4.6 model respectively.

Let, Y = log (Mr), a_0 = log (K_1pa), a_1 =K_2, x_1 = log(X_1), x_2 = log (X2)

Which form a simple linear equation, $Y = a_0 + a_1x_1 + a_2x_2$

Assume we have n set of data, then (Y_1, x_{11}, x_{21})

(Y_2, x_{12}, x_{22})

(……..)

(Yn, x_{1n}, x_{2n})

Where, y_i =log (Mr_i), x_{1i} = log (x_{1i}), x_{21} = log (x_{2i}) for i= 1 to n, coefficient a_o, a_1 and a_2 are solved from the following equation.

$$C'C\begin{pmatrix} ao \\ a1 \\ a2 \end{pmatrix} = C'\begin{pmatrix} y1 \\ y2 \\ : \\ : \\ yn \end{pmatrix} \qquad \text{Where, } C = \begin{pmatrix} 1 & X11 & X21 & \dots & Xm1 \\ 1 & X12 & X22 & \dots & Xm2 \\ \dots & \dots & \dots & \dots & \dots \\ 1 & X1n & X2n & \dots & Xmn \end{pmatrix} \text{ and } C' = \text{Transpose of } C$$

The task turns to solving a 3 by 3 linear equation for a_o, a_1 and a_2 and

$K_1 = {e^{ao}}/{pa}$, $K_2 = a_1$, $K_3 = a_2$

Table 5. 9 Summary of axial stress (σ_1), confining stress (σ_3), total stress (θ), Diviatoric stress (σ_d) and resilient Modulus (Mr) obtained from the graph of deviator stress versus axial strain from UU test result for Sh/Meda – K/Mihiret H/Nooh for sub - grade and sub-base material

Location (Station) of sampling			Confinment (Kpa)	σ1	stress (θ) =σ1 +2σ3	Divator stress(σd)	$\tau oct = \frac{\sqrt{2}}{3}\sigma d$	Resilient Modulus (Mr)		ln mr	ln(θ/pa)	ln(τoct/pa +1)
								Kpa	Psi			
Shiro Meda – Kidane Mihiret Hamere Noh	Left carriageway	0 +050	100	830	1030	730	343.10	366.00	53.07	5.90	2.33	1.49
		sub -	200	931	1331	831	390.57	347.08	50.33	5.85	2.59	1.59
		grade	300	1112	1712	1012	475.64	461.82	66.96	6.14	2.84	1.75
		0 + 560	100	753	953	653	306.91	362.78	52.60	5.89	2.25	1.40
		sub -	200	935	1335	835	392.45	418.00	60.61	6.04	2.59	1.59
		grade	300	997	1597	897	421.59	498.89	72.34	6.21	2.77	1.65
		0 + 960	100	676	876	576	270.72	320.00	46.40	5.77	2.17	1.31
		sub -	200	747	1147	647	304.09	323.50	46.91	5.78	2.44	1.40
		grade	300	788	1388	688	323.36	345.50	50.10	5.84	2.63	1.44
		1 + 380	100	638	838	538	252.86	244.54	35.46	5.50	2.13	1.26
		sub -	200	783	1183	683	321.01	213.44	30.95	5.36	2.47	1.44
		grade	300	813	1413	713	335.11	255.00	36.98	5.54	2.65	1.47
	Right carriageway	0 +050	100	647	847	547	257.09	227.92	33.05	5.43	2.14	1.27
		sub -	200	751	1151	651	305.97	250.77	36.36	5.52	2.44	1.40
		grade	300	933	1533	833	391.51	297.86	43.19	5.70	2.73	1.59
		0 + 560	100	564	764	464	218.08	386.67	56.07	5.96	2.03	1.16
		sub -	200	662	1062	562	264.14	281.00	40.75	5.64	2.36	1.29
		grade	300	782	1382	682	320.54	284.58	41.26	5.65	2.63	1.44
		0 + 960	100	722	922	622	292.34	346.67	50.27	5.85	2.22	1.37
		sub -	200	875	1275	775	364.25	389.00	56.41	5.96	2.55	1.54
		grade	300	956	1556	856	402.32	306.79	44.48	5.73	2.74	1.61
		1 + 380	100	560	760	460	216.20	328.57	47.64	5.79	2.03	1.15
		sub -	200	767	1167	667	313.49	303.64	44.03	5.72	2.46	1.42
		grade	300	890	1490	790	371.30	303.85	44.06	5.72	2.70	1.55
Shiro Meda – Kidane Mihiret Hamere Noh	Left carriageway	0 +050	100	1034	1234	934	438.98	586.25	85.01	6.37	2.51	1.68
		sub -	200	1258	1658	1058	497.26	593.33	86.03	6.39	2.81	1.79
		base	300	1414	2014	1114	523.58	619.44	89.82	6.43	3.00	1.83
		0 + 560	100	713	913	613	288.11	306.50	44.44	5.73	2.21	1.36
		sub -	200	907	1307	707	332.29	397.86	57.69	5.99	2.57	1.46
		base	300	1217	1817	917	430.99	417.27	60.50	6.03	2.90	1.67
		0 + 960	100	845	1045	745	350.15	219.12	31.77	5.39	2.35	1.50
		sub -	200	1104	1504	904	424.88	215.48	31.24	5.37	2.71	1.66
		base	300	1197	1797	897	421.59	345.77	50.14	5.85	2.89	1.65
		1 + 380	100	1128	1328	1028	483.16	303.24	43.97	5.71	2.59	1.76
		sub -	200	1388	1788	1188	558.36	247.92	35.95	5.51	2.88	1.88
		base	300	1488	2088	1188	558.36	332.78	48.25	5.81	3.04	1.88
Shiro Meda – Kidane Mihiret Hamere Noh	Right carriageway	0 +050	100	727	927	627	294.69	224.64	32.57	5.41	2.23	1.37
		sub -	200	973	1373	773	363.31	373.33	54.13	5.92	2.62	1.53
		base	300	1725	2325	1425	669.75	400.28	58.04	5.99	3.15	2.04
		0 + 560	100	966	1166	866	407.02	435.00	63.08	6.08	2.46	1.62
		sub -	200	1819	2219	1619	760.93	579.29	84.00	6.36	3.10	2.15
		base	300	1423	2023	1123	527.81	586.15	84.99	6.37	3.01	1.84
		0 + 960	100	678	878	578	271.66	262.73	38.10	5.57	2.17	1.31
		sub -	200	1573	1973	1373	645.31	575.83	83.50	6.36	2.98	2.01
		base	300	1510	2110	1210	568.70	657.50	95.34	6.49	3.05	1.90
		1 + 380	100	933	1133	833	391.51	320.77	46.51	5.77	2.43	1.59
		sub -	200	1175	1575	975	458.25	375.00	54.38	5.93	2.76	1.72
		base	300	1344	1944	1044	490.68	373.51	54.16	5.92	2.97	1.78
	pa =100kpa											

Table 5. 10 Summary of axial stress (σ_1), confining stress (σ_3), total stress (θ), Diviatoric stress (σ_d) and resilient Modulus (Mr) obtained from graph of deviator stress versus axial strain from UU test result for A/Garage - Unity University – M/Haile for sub - grade and sub - base material.

Location (Station) of sampling			Confinment (Kpa)	σ1	stress (θ) =σ1 +2σ3	Divator stress(σd)	$\tau oct = \frac{\sqrt{2}}{3}\sigma d$	Resilient Modulus (Mr)		ln mr	ln(θ/pa)	ln(τoct/pa +1)
								Kpa	Psi			
Ambessa Garage -Unity– M/Haile	Left carriageway	0 + 120 sub - grade	100	622	822	522	245.34	290.00	42.05	5.67	2.11	1.24
			200	686	1086	586	275.42	326.67	47.37	5.79	2.39	1.32
			300	765	1365	665	312.55	333.50	48.36	5.81	2.61	1.42
		0 + 380 sub - grade	100	711	911	611	287.17	305.50	44.30	5.72	2.21	1.35
			200	798	1198	698	328.06	317.27	46.00	5.76	2.48	1.45
			300	862	1462	762	358.14	346.82	50.29	5.85	2.68	1.52
		0 + 740 sub - grade	100	771	971	671	315.37	305.92	44.36	5.72	2.27	1.42
			200	851	1251	751	352.97	313.33	45.43	5.75	2.53	1.51
			300	952	1552	852	400.44	356.67	51.72	5.88	2.74	1.61
	Right carriageway	0 + 120 sub - grade	100	628	828	528	248.16	293.33	42.53	5.68	2.11	1.25
			200	740	1140	640	300.80	355.56	51.56	5.87	2.43	1.39
			300	756	1356	656	308.32	364.44	52.84	5.90	2.61	1.41
		0 + 380 sub - grade	100	725	925	625	293.75	312.50	45.31	5.74	2.22	1.37
			200	857	1257	757	355.79	315.42	45.74	5.75	2.53	1.52
			300	934	1534	834	391.98	347.50	50.39	5.85	2.73	1.59
		0 + 740 sub - grade	100	632	832	532	250.04	332.50	48.21	5.81	2.12	1.25
			200	771	1171	671	315.37	336.50	48.79	5.82	2.46	1.42
			300	855	1455	755	354.85	343.64	49.83	5.84	2.68	1.51
Ambessa Garage -Unity– M/Haile	Left carriageway	0 + 120 sub - base	100	629	829	529	248.63	220.42	31.96	5.40	2.12	1.25
			200	1073	1473	873	410.31	273.44	39.65	5.61	2.69	1.63
			300	1186	1786	886	416.42	277.50	40.24	5.63	2.88	1.64
		0 + 380 sub - base	100	971	1171	871	409.37	396.36	57.47	5.98	2.46	1.63
			200	1319	1719	1119	525.93	432.31	62.68	6.07	2.84	1.83
			300	1977	2577	1677	788.19	468.33	67.91	6.15	3.25	2.18
		0 + 740 sub - base	100	1038	1238	938	440.86	532.50	77.21	6.28	2.52	1.69
			200	1183	1583	983	462.01	598.33	86.76	6.39	2.76	1.73
			300	1204	1804	904	424.88	621.43	90.11	6.43	2.89	1.66
Ambessa Garage -Unity– M/Haile	Right carriageway	0 + 120 sub - base	100	773	973	673	316.31	305.91	44.36	5.72	2.28	1.43
			200	892	1292	692	325.24	353.33	51.23	5.87	2.56	1.45
			300	1165	1765	865	406.55	367.65	53.31	5.91	2.87	1.62
		0 + 380 sub - base	100	790	990	690	324.30	431.25	62.53	6.07	2.29	1.45
			200	1766	2166	1566	736.02	653.75	94.79	6.48	3.08	2.12
			300	1820	2420	1520	714.40	635.42	92.14	6.45	3.19	2.10
		0 + 740 sub - base	100	729	929	629	295.63	349.44	50.67	5.86	2.23	1.38
			200	1261	1661	1061	498.67	442.92	64.22	6.09	2.81	1.79
			300	1652	2252	1352	635.44	451.00	65.40	6.11	3.11	2.00
		pa =100kpa										

From the tables 5.09 and 5.10 above the values used for linearized Mr Equations are summarized below

Table 5. 11 Confining stress (σ_3), Diviatoric stress (σ_d) and Resilient modulus (Mr) for sub - grade

Location (station)			$\sigma_{3(psi)} = X_1$	σ_d(psi) = X_2		Mr (psi) = Y	
				Left	Right	Left	Right
Shiro Meda – Kidane Mihiret Hamere Nooh (Sub - Grade)	0 + 050	100	14.50	105.85	79.31	53.07	33.05
		200	29.00	120.50	94.39	50.33	36.36
		300	43.50	148.04	120.78	66.96	43.19
	0 + 560	100	14.50	94.68	67.28	52.60	56.07
		200	29.00	121.07	81.49	60.61	40.75
		300	43.50	130.06	98.89	72.34	41.26
	0 + 960	100	14.50	83.52	90.19	46.40	50.27
		200	29.00	93.81	112.37	46.91	56.41
		300	43.50	99.76	124.12	50.10	44.48
	1 + 380	100	14.50	78.01	66.70	35.46	47.64
		200	29.00	99.03	96.71	30.95	44.04
		300	43.50	103.38	114.55	36.98	44.06
Ambessa Garage – Unity University – Mebrat Haile (Sub –grade)	0 + 120	100	14.50	75.69	76.56	42.05	42.59
		200	29.00	84.97	92.80	47.37	51.58
		300	43.50	94.97	95.12	48.38	52.84
	0 + 380	100	14.50	88.59	90.62	44.30	45.31
		200	29.00	101.21	109.76	46.00	45.74
		300	43.50	110.49	120.93	50.29	50.39
	0 + 740	100	14.50	97.29	77.14	44.36	48.21
		200	29.00	108.89	97.29	45.43	48.79
		300	43.50	123.54	109.47	51.72	49.83

Table 5. 12 Matrix solutions for K_1, K_2 and K_3 from resilient Modulus (Mr) values obtained from UU test result for Sh/Meda – K/Mihiret H/Nooh sub grade for left carriageway.

For left carriageway

	y = lnMr
Y =	5.90
	5.85
	6.14
	5.89
	6.04
	6.21
	5.77
	5.78
	5.84
	5.50
	5.36
	5.54

	Y	X1 = ln(θ/pa)	X2 = ln(τoct/pa)
C left =	1	2.33	1.49
	1	2.59	1.59
	1	2.84	1.75
	1	2.25	1.40
	1	2.59	1.59
	1	2.77	1.65
	1	2.17	1.31
	1	2.44	1.40
	1	2.63	1.44
	1	2.13	1.26
	1	2.47	1.44
	1	2.65	1.47

Table 5. 13 transpose of C for left carriageway for S/Meda – K/Mihiret H/Nooh for sub grade material

C' Left =	1	1	1	1	1	1	1	1	1	1	1	1
	2.33	2.59	2.84	2.25	2.59	2.77	2.17	2.44	2.63	2.13	2.47	2.65
	1.49	1.59	1.75	1.40	1.59	1.65	1.31	1.40	1.44	1.26	1.44	1.47

Determination of liner coefficient for left side carriageway sub –grade

$$C' * C = \begin{pmatrix} 12 & 29.86 & 17.80 \\ 29.86 & 74.90 & 44.60 \\ 17.80 & 44.60 & 25.42 \end{pmatrix} \qquad C' * Y = \begin{pmatrix} 69.83 \\ 174.08 \\ 103.84 \end{pmatrix}$$

$$\begin{pmatrix} 12.00 & 29.86 & 17.80 \\ 29.86 & 74.90 & 44.60 \\ 17.80 & 44.60 & 25.42 \end{pmatrix} * \begin{pmatrix} ao \\ a1 \\ a2 \end{pmatrix} = \begin{pmatrix} 69.83 \\ 174.08 \\ 103.84 \end{pmatrix} \qquad \text{Then, } \begin{pmatrix} ao \\ a1 \\ a2 \end{pmatrix} = \begin{pmatrix} 4.365 \\ 0.538 \\ 0.077 \end{pmatrix}$$

For $Mr = k1\left(\frac{\sigma sum}{pa}\right)^{K2}\left(\frac{\tau oct}{pa}+1\right)^{K3}$ from the above values $K1 = \frac{e^{ao}}{pa}$, then K1 = 79,

$K_2 = a_1 = 0.538$, $K_3 = a_2 = 0.077$

Thus, for sub – grade (Left), $Mr = 79\left(\frac{\sigma sum}{pa}\right)^{0.538}\left(\frac{\tau oct}{pa}+1\right)^{0.077}$

Table 5. 14 Matrix solutions for K_1, K_2 and K_3 from resilient Modulus (Mr) values obtained from UU test result for S/Meda – K/Mihiret H/Nooh sub grade for Right carriageway.

For Right side carriageway

	y = lnMr
Y =	5.43
	5.52
	5.70
	5.96
	5.64
	5.65
	5.85
	5.96
	5.73
	5.79
	5.72
	5.72

	Y	X1 = ln(θ/pa)	X2 = ln(τoct/pa)
C left =	1	2.14	1.27
	1	2.44	1.40
	1	2.73	1.59
	1	2.03	1.16
	1	2.36	1.29
	1	2.63	1.44
	1	2.22	1.37
	1	2.55	1.54
	1	2.74	1.61
	1	2.03	1.15
	1	2.46	1.42
	1	2.70	1.55

Table 5. 15 Transpose of C for Right carriageway for Shiro Meda - Kidane Mihiret Hamere Nooh for sub grade material

C' Right =	1	1	1	1	1	1	1	1	1	1	1	1
	2.14	2.44	2.73	2.03	2.36	2.63	2.22	2.55	2.74	2.03	2.46	2.70
	1.27	1.40	1.59	1.16	1.29	1.44	1.37	1.54	1.61	1.15	1.42	1.55

Determination of linear coefficient for right side carriageway sub –grade

$C' * C = \begin{pmatrix} 12.00 & 29.03 & 16.79 \\ 29.03 & 70.99 & 41.05 \\ 16.79 & 41.05 & 22.15 \end{pmatrix}$ $C' * Y = \begin{pmatrix} 68.67 \\ 166.07 \\ 96.08 \end{pmatrix}$

$\begin{pmatrix} 12.00 & 29.03 & 16.79 \\ 29.03 & 70.99 & 41.05 \\ 16.79 & 41.05 & 22.15 \end{pmatrix} * \begin{pmatrix} ao \\ a1 \\ a2 \end{pmatrix} = \begin{pmatrix} 68.67 \\ 166.07 \\ 96.08 \end{pmatrix}$ Then, $\begin{pmatrix} ao \\ a1 \\ a2 \end{pmatrix} = \begin{pmatrix} 5.8993 \\ -0.0646 \\ -0.0147 \end{pmatrix}$

For Mr = $k1(\frac{\sigma sum}{pa})^{K2}(\frac{\tau oct}{pa}+1)^{K3}$ from the above values K1 = $\frac{e^{ao}}{pa}$, then K1 = 365,

$K_2 = a_1 = -0.0646$, $K_3 = a_2 = -0.0147$

Thus, for sub - grade (Right), Mr = $k1(\frac{\sigma sum}{pa})^{K2}(\frac{\tau oct}{pa}+1)^{K3} = 5.8993(\frac{\sigma sum}{pa})^{-0.0646}(\frac{\tau oct}{pa}+1)^{-0.0147}$

In the same way, resilient modulus regression constants for the remaining location (station) material can be calculated and some of the calculations are shown in the appendix part.

- **Multiple Regressions STATA software for Determination of regression constants(K1, K2 and K3)**

Multiple linear regressions coefficients obtained by STATA software from density, moisture content, degree of compaction, degree of moisture, Atterberg limits, specific gravity, and other soil properties is more recommended than regressions coefficients from CBR and UCS because it exhibits greater R^2[27]. Thus, for this paper work Dry density (γ_d), optimum moisture content (OMC), Specific Gravity (Sg), plastic limit (Pl), and Liquid Limit (LL) is taking into account collinearity.

$K_1 = ao + a_1 (LL) + a_2 (PL) + a_3 (OMC) + a_4 (\gamma_d) + a_5 (Sg)$ ……..(4.7)

$K_2 = bo + b_1 (LL) + b_2 (PL) + b_3 (OMC) + b_4 (\gamma_d) + b_5 (Sg)$ ……(4.7)

$K_3 = co + c_1 (LL) + c_2 (PL) + c_3 (OMC) + c_4 (\gamma d) + c_5 (Sg)$ ……..(4.7)

Take left and right carriageway data together for sub-grade and sub - base as shown below for conducting multiple regressions using STATA software for both projects independently. To do this, regression constants need to be determined from the matrix solution. Then, the coefficient for LL, PL, OMC, and γ_d is determined from multiple regressions using STATA software.

Table 5. 16 Multiple regressions using STATA software for K_1, determination from important soil properties such as LL, PL, OMC, γ_d and Sg obtained from laboratory test result for S/Meda - K/Mihiret H/Nooh sub - grade for left and right carriageway

```
. regress k1 ll pl omc mdd sg
```

Source	SS	df	MS		
Model	132532.82	5	26506.564	Number of obs	= 8
Residual	31059.18	2	15529.59	F(5, 2)	= 1.71
				Prob > F	= 0.4093
				R-squared	= 0.8101
				Adj R-squared	= 0.3355
Total	163592	7	23370.2857	Root MSE	= 124.62

k1	Coef.	Std. Err.	t	P>|t|	[95% Conf.	Interval]
ll	13.60645	23.58516	0.58	0.622	-87.87232	115.0852
pl	48.03535	59.16684	0.81	0.502	-206.539	302.6097
omc	-43.08639	39.34064	-1.10	0.388	-212.3555	126.1827
mdd	-7086.144	4540.071	-1.56	0.259	-26620.49	12448.21
sg	-2643.737	3115.229	-0.85	0.485	-16047.48	10760.01
_cons	16532.37	13575.04	1.22	0.347	-41876.3	74941.04

Table 5. 17 Multiple regressions using STATA software for K_2 determination from important soil properties such as LL, PL, OMC, γ_d and Sg obtained from laboratory test result for Shiro Meda - Kidane Mihiret Hamere Nooh sub - grade for left and right carriageway

```
. regress k2 ll pl omc mdd sg
```

Source	SS	df	MS	Number of obs =	8
				F(5, 2) =	1.71
Model	.360056418	5	.072011284	Prob > F =	0.4093
Residual	.08437953	2	.042189765	R-squared =	0.8101
				Adj R-squared =	0.3355
Total	.444435948	7	.06349085	Root MSE =	.2054

k2	Coef.	Std. Err.	t	P>|t|	[95% Conf.	Interval]
ll	-.0224269	.0388743	-0.58	0.622	-.1896894	.1448357
pl	-.0791744	.0975218	-0.81	0.502	-.498777	.3404283
omc	.0710172	.0648433	1.10	0.388	-.2079809	.3500153
mdd	11.67975	7.48318	1.56	0.259	-20.51778	43.87727
sg	4.357544	5.134681	0.85	0.485	-17.73521	26.45029
_cons	-26.58329	22.37508	-1.19	0.357	-122.8555	69.6889

Table 5. 18 Multiple regressions using STATA software for K_3 determination from important soil properties such as LL, PL, OMC, γ_d and Sg obtained from laboratory test result for Shiro Meda - Kidane Mihiret Hamere Nooh sub - grade for left and right carriageway

```
. regress k3 ll pl omc mdd sg
```

Source	SS	df	MS	Number of obs =	8
				F(5, 2) =	1.71
Model	.013624797	5	.002724959	Prob > F =	0.4093
Residual	.003192983	2	.001596491	R-squared =	0.8101
				Adj R-squared =	0.3355
Total	.01681778	7	.00240254	Root MSE =	.03996

k3	Coef.	Std. Err.	t	P>|t|	[95% Conf.	Interval]
ll	-.0043626	.0075621	-0.58	0.622	-.0368997	.0281744
pl	-.0154015	.0189706	-0.81	0.502	-.0970256	.0662225
omc	.0138148	.0126138	1.10	0.388	-.0404579	.0680874
mdd	2.272026	1.45568	1.56	0.259	-3.99126	8.535312
sg	.8476597	.9988338	0.85	0.485	-3.449975	5.145295
_cons	-5.198433	4.352555	-1.19	0.355	-23.92597	13.5291

- **Sub - base multiple regression constants value for Sh/Meda – K/Mihiret H/Nooh.**

$K_1 = 16532.37 + 13.61(LL) + 48.04(PL) – 43.09(OMC) – 7086.14(mdd) – 2643.74(Sg)$

$K_2 = -26.58 - 0.02(LL) - 0.08(PL) + 0.07 (OMC) + 11.68(mdd) + 4.36 (Sg)$

$K_3 = -5.20 – 0.01(LL) – 0.02(PL) - 0.01(OMC) + 2.27(mdd) + 0.85(Sg)$

For the remaining pavement material regression constants are calculated in the same way.

Generally, the value of Mr obtained by this method is used for the development of structural layer coefficient for sub - base and base course. However, the constitutive model is not used since regression constants K_1, K_2 and K_3 need to be calibrated first from the cyclic triaxial test. UU triaxial test for elastic modulus determination is not conducted for base pavement material since the layer coefficient is based on the elastic modulus of top of sub - base pavement material and the layer coefficient of sub - base material is based on the elastic modulus of top of sub - grade material. This will be discussed in detail in chapter 5.3.

5.2 Determination of AASHTO Drainage coefficient

To develop drainage coefficient by AASHTO guide, climate condition and quality of drainage are the two most important information. Since the length of climate seasonal condition for Ethiopia is almost 6 months wet and 6 months dry, it falls zone B (I) AASHTO climate condition zone as shown in table 5.19 below. Frozen and thawing climatic condition does not occur in Ethiopia particularly in Addis Ababa.

Table 5. 19 climate condition seasonal lengths and climate condition (AASHTO zone)

Climate condition (AASHTO Zone)	Season (months)			
	Road bed frozen	Road bed thawing	Road bed wet	Road bed Dry
A (IV)	0.0	0.0	2.0	10.0
B (I)	0.0	0.0	5.5	6.5
C (VI)	3.0	1.5	1.0	6.5
D (V)	0.5	0.5	1.5	9.5
E (III)	3.0	1.5	2.0	5.5
F (II)	0.5	0.5	5.0	6.0

Table 5. 20 recommended Drainage coefficient for flexible pavements with untreated base and sub base material.

Pavement Quality of Drainage	Climate Condition					
	A	B	C	D	E	F
Excellent	1.25 - 1.20	1.25 - 1.20	1.25 - 1.20	1.25 - 1.20	1.20 - 1.15	1.20 - 1.15
Good	1.25 - 1.20	1.20 - 1.15	1.20 - 1.15	1.20 - 1.15	1.20 - 1.15	1.20 - 1.15
fair	1.20 - 1.15	1.15 - 1.05	1.05 - 0.85	1.05 - 0.85	1.05 - 0.85	1.15 - 1.05
Poor	1.15 - 1.05	1.15 - 1.05	1.05 - 0.85	1.05 - 0.85	1.05 - 0.85	0.85 - 0.70
Very poor	1.05 -0.85	1.05 - 0.85	0.85 - 0.70	0.70 - 0.60	0.85 - 0.70	0.70 - 0.60

For both projects, the base material is AASHTO A -1- a crushed aggregate of stone which is a relatively good permeable pavement material with a drainage coefficient ranging 1.20 -1.15, and the sub - base material is AASHTO A -2- 5 which is fair permeability granular soil with drainage coefficient raging 1.15 - 1.05 respectively as shown in the table 5.20. The sub - grade is AASHTO A - 2- 7 which is poor permeability of gravel clay soil which needs improvement by good sub - base and base material above. Taking the average, the base drainage coefficient is 1.18 and for the sub - base is 1.10.

As explained in literature review, the drainage coefficients (mi) are also calculated based upon the percent passing the number 200 sieve (p200) of the material in question and the Average Annual Rainfall in inches for the project location as expressed by the following equations [11].however, this method is used for agricultural soil.

$Si = 1.2 - 0.6(AAR)/100$ $Dq = 1.2 = 0.6\ (p200)/100$

then $mi = Si * Dq$

Where Si is the level of saturation and Dq is used to describe the drainage quality. According to National Meteorological Services Agency (NMSA), Average Annual Rainfall (AAR) for Addis Ababa is 141.6mm for the year 2016 according to Medhanit Akalu (2017).

i.e. AAR for AA is 141.6mm = 5.57inch

Thus, the level of saturation; $Si = 1.2 - 0.6(AAR)/100 = 1.2 - 0.6(5.57)/100 = \underline{1.17}$

Dq is used to describe the drainage quality: $Dq = 1.2 - 0.6\ (p200)/100$

Table 5. 21 Percent passing #200 sieves for Base and Sub - base at each

Material type and carriageway	Station	Percent passing # 200 Sieve	drainage quality Dq = 1.2 - 0.6 (P200)/100
Sub base left carriageway for Shiro Meda - Kidane Mihiret Hamere Nooh	0 + 050	4	1.176
	0 + 560	3	1.182
	0 + 960	4	1.176
	1 + 380	1	1.194
Sub base Right carriageway Shiro Meda - Kidane Mihiret Hamere Nooh	0 + 050	1	1.194
	0 + 560	4	1.176
	0 + 960	4	1.176
	1 + 380	6	1.164
		Average	**1.18**
Sub base left carriageway Ambessa Garage - unity - M/Haile	0 + 120	2	1.188
	0 + 380	5	1.17
	0 + 740	2	1.188
Sub base right carriageway Ambessa Garage - unity - M/Haile	0 + 120	3	1.182
	0 + 380	4	1.176
	0 + 740	5	1.17
		Average	1.18

Table 5. 22 Percent passing #200 sieves for Base and Sub - base at each

Material type and carriageway	Station	Percent passing # 200 sieve	drainage quality Dq = 1.2 – 0.6(P200)/100
Base Shiro Meda - Kidane Mihiret Hamere Nooh	0 + 980 left carriageway	4	1.176
	0 + 980 Right carriageway	4	1.176
		Average	1.176
Base Ambessa Garage - unity Univ.- M/Haile	0 + 460 left carriageway	5	1.17
	0 + 460 Right carriageway	7	1.158
		Average	1.164

Thus the drainage coefficient is given by mi = Si * Dq

For sub base drainage coefficient considering both Shiro Meda and Ambessa Garage projects

Drainage coefficient (m_3) = Si *Dq = 1.17 * 1.18 = 1.38

For base drainage coefficient considering both Shiro Meda and Ambessa Garage projects

Drainage coefficient (m_2) = Si * Dq = 1.17 * 1.17 =1.37

5.3 Determination of structural Number and structural layer coefficient

Structural Number is determined by rearranging the basic AASHTO equation. For this paper, however, an excel solver is used by inserting resilient modulus (Mr), Reliability (Z_R), Standard Deviation (So), equivalent standard axle load (ESALs or W_{18}), and performance level (ΔPSI) to basic AASHTO equation as shown in table 5.23. For AASHTO structural layers calculation, Use resilient modulus (E_2 or Mr) of the base layer to obtain the structural number of asphalt layer (SN_1) required for protecting layer2, Use resilient modulus (E_3 as Mr) of sub-base layer to obtain the structural number of the base layer (SN_2) required for protecting layer3 and Use reference resilient modules (Mref) of sub - grade to obtain the structural number of sub - base layer (SN_3) required for protecting the sub - grade as shown figure 5.1 below. Since this thesis work intends to determine the structural layer coefficient of sub-base and base, the resilient modulus of sub-base and sub - grade is used respectively for each project at the randomly selected station for the left and right sides. Since the UU test is used for this thesis work, reference resilient modules (Mref) is equal to resilient modulus (Mr) obtained from stress - strain graph determined for each project at the purposely selected station for left and right side in chapter 3[44].

- $a1 \geq \frac{SN1}{D1}$
- $a2 \geq \frac{SN2 - a1D1}{D2m2}$
- $a3 \geq \frac{SN2 - a1D1 - a2D2}{D3m3}$

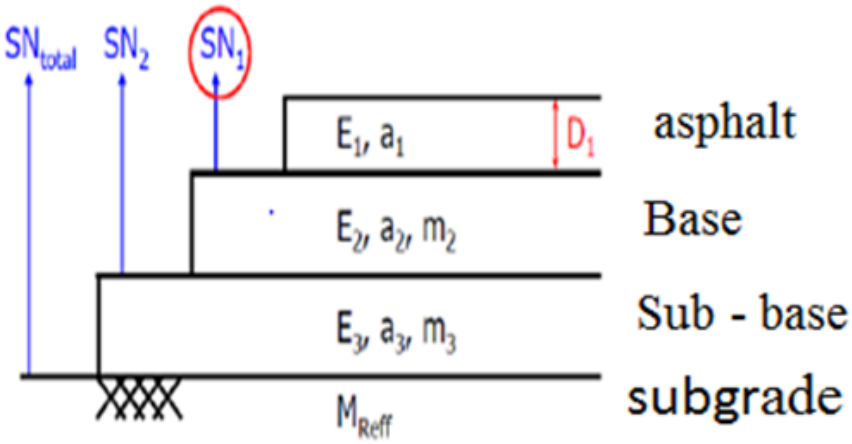


Figure 5. 1 structural number, layer coefficient, drainage coefficient, young's modulus (E or Mr) and layer coefficient calculation with respective pavement layer.

As figure above the structural layer coefficient for sub - base is obtained by rearranging sub - base structural number (SN2) equation and for base is obtained by rearranging base structural number (SN3) as shown below[2, 44,47].

- $a_2 \geq \frac{SN2 - a2D2}{D2m2}$ ……for Base.
- $a_3 \geq \frac{SN2 - a1D1 - a2D2}{D3m3}$ ……for Sub – base.

Table 5. 23 AASHTO Flexible Pavement Design for Sub – Base for Shiro Meda – Kidane Mihiret Hamere Nooh Project Left Carriageway

SN Determination (solver) Design Inputs

W18	3,809,671.980		ESALs Applications Over Design Period	Typ. Range 0.1 to 80 million
R =	95	%	Reliability	Typ. Range 80 to 95%
So =	0.49		Standard Deviation	Typ. Range 0.3 to 0.5
Mr =	85	psi	Subgrade Resilient Modulus	Typ. Range 3000 to 9000 psi
Pi =	4.2		Initial Serviceability	Typ. Range 4.4 to 4.8
Pt =	2.5		Terminal Serviceability	Typ. Range 2.0 to 3.0
DESIGN SN =16.70				

The remaining calculation of the structural number for each material type at each station for the two projects is shown in appendix G at the end of this material. The summary of the structural number from the excel solver, structural Layer coefficient for each material type at each station for the two projects from the calculation, and appendix I is shown in table 5.24, 5.25, 5.26 and 5.27.

Table 5. 24 Summary of the structural layer coefficient (a_2) for Base for Shiro Meda - Kidane Mihiret Hamere Nooh from calculation in appendix G and data of appendix I

Location (Station) of sampling		Confinement (Kpa)	Resilient Modulus (Mr) for (σ_3)		h_1	h_2	a1	$SN_1 = H_1 * a_1$	SN_2 = Excel solver)	$*a_2$	a_2 average.
			Kpa	Psi							
Shiro Meda – Kidane Mihiret Hamere Noh	0 +050 sub - base (left and right) (3,908,671.980 ESAL)	100	586.25	85.01	15	50	0.44	6.6	16.75	0.17	
		100	224.64	32.57	15	50	0.44	6.6	21.45	0.25	0.21
		200	593.33	86.03	15	50	0.44	6.6	16.65	0.17	
		200	373.33	54.13	15	50	0.44	6.6	18.80	0.21	0.19
		300	619.44	89.82	15	50	0.44	6.6	16.46	0.17	
		300	400.28	58.04	15	50	0.44	6.6	18.46	0.20	0.18
	0 + 560 sub - base (left and right) (3,809,671.980 ESAL)	100	306.50	44.44	15	50	0.44	6.6	19.79	0.22	
		100	435.00	63.08	15	50	0.44	6.6	18.06	0.19	0.21
		200	397.86	57.69	15	50	0.44	6.6	18.49	0.20	
		200	579.29	84	15	50	0.44	6.6	16.75	0.17	0.19
		300	417.27	60.5	15	50	0.44	6.6	18.26	0.20	
		300	586.15	84.99	15	50	0.44	6.6	16.70	0.17	0.18
	0 + 960 sub - base (left and right) (12,879,805.945 ESAL)	100	219.12	31.77	15	50	0.44	6.6	24.73	0.31	
		100	262.73	38.1	15	50	0.44	6.6	23.60	0.29	0.30
		200	215.48	31.24	15	50	0.44	6.6	24.84	0.31	
		200	575.83	83.5	15	50	0.44	6.6	19.25	0.21	0.26
		300	345.77	50.14	15	50	0.44	6.6	21.98	0.26	
		300	657.50	95.34	15	50	0.44	6.6	18.60	0.20	0.23
	1 + 380 sub - base (left and right) (12,879,805.945 ESAL)	100	303.24	43.97	15	50	0.44	6.6	22.74	0.27	
		100	320.77	46.51	15	50	0.44	6.6	22.41	0.27	0.27
		200	247.92	35.95	15	50	0.44	6.6	23.96	0.29	
		200	375.00	54.38	15	50	0.44	6.6	21.52	0.25	0.27
		300	332.78	48.25	15	50	0.44	6.6	22.20	0.26	
		300	373.51	54.16	15	50	0.44	6.6	21.55	0.25	0.26

where;- $m_2 = 1.18$ * $a_2 = (SN_2 - SN_1) / H2 * m_2$ Range of $a_2 = [0.18 - 0.30]$

Table 5. 25 Summary of the structural layer coefficient (a_3) for Sub - Base for Shiro Meda - Kidane Mihiret Hamere Nooh from calculation in appendix G and data of appendix I.

Location (Station) of sampling		Confinement	Resilient Modulus (Mr) for confinement (σ_3)		h_1	h_2	h_3	a_1	$SN1 = H_1*a1$	SN_2 (Excel solver)	SN_3 = (Excel solver)	*a2	**a3	Average.(a3)
		Kpa	Kpa	Psi										
Shiro Meda – Kidane Mihiret Hamere Noh	0 +050 sub - grade (left and right) (3,809,671.980 ESAL)	100	366.00	53.07	15	50	50	0.44	6.6	16.70	19.83	0.17	0.057	0.03
		100	227.92	33.05	15	50	50	0.44	6.6	21.45	21.54	0.25	0.002	
		200	347.08	50.33	15	50	50	0.44	6.6	16.65	19.16	0.17	0.046	0.04
		200	250.77	36.36	15	50	50	0.44	6.6	18.80	20.85	0.21	0.037	
		300	319.47	46.30	15	50	50	0.44	6.6	16.46	19.63	0.17	0.058	0.04
		300	297.86	43.19	15	50	50	0.44	6.6	18.46	19.94	0.20	0.027	
	0 + 560 sub - grade (left and right) (3,809,671.980 ESAL)	100	362.78	52.6	15	50	50	0.44	6.6	19.79	20.35	0.22	0.010	0.02
		100	311.88	45.20	15	50	50	0.44	6.6	18.06	19.71	0.19	0.010	
		200	418.00	60.61	15	50	50	0.44	6.6	18.49	19.15	0.20	0.012	0.04
		200	281.00	40.75	15	50	50	0.44	6.6	16.75	20.24	0.17	0.063	
		300	340.86	49.40	15	50	50	0.44	6.6	18.26	19.32	0.20	0.019	0.04
		300	284.58	41.26	15	50	50	0.44	6.6	16.70	20.18	0.17	0.063	
	0 + 960 sub - grade (left and right) (12,879,805.945 ESAL)	100	320.00	46.4	15	50	50	0.44	6.6	24.73	24.93	0.31	0.004	0.01
		100	346.67	50.27	15	50	50	0.44	6.6	23.60	24.23	0.29	0.011	
		200	323.50	46.91	15	50	50	0.44	6.6	24.84	24.94	0.31	0.002	0.03
		200	389.00	56.41	15	50	50	0.44	6.6	19.25	21.99	0.21	0.050	
		300	345.50	50.1	15	50	50	0.44	6.6	21.98	23.74	0.26	0.032	0.05
		300	306.79	44.48	15	50	50	0.44	6.6	18.60	22.67	0.20	0.074	
	1 + 380 sub - grade (left and right) (12,879,805.945 ESAL)	100	244.54	35.46	15	50	50	0.44	6.6	22.74	24.04	0.27	0.024	0.02
		100	328.57	47.64	15	50	50	0.44	6.6	22.41	23.27	0.27	0.016	
		200	213.44	30.95	15	50	50	0.44	6.6	23.96	24.9	0.29	0.017	0.02
		200	303.64	44.03	15	50	50	0.44	6.6	21.52	22.73	0.25	0.022	
		300	255.00	36.98	15	50	50	0.44	6.6	22.20	23.78	0.26	0.029	0.03
		300	303.85	44.06	15	50	50	0.44	6.6	21.55	22.73	0.25	0.021	

Where, $*a_2 = (SN_2 - SN_1)/H_2*m_2$ $**a_3 = (SN_3 - a_2h_2m_2 - SN_1)/H_3*m_3$ Range of a_3 = [0.01 - 0.05]

Table 5. 26 summary of the structural layer coefficient (a_2) for Base for Ambessa Garage - Unity University - Mebrat Haile from calculation in appendix G and data of appendix I.

Location (Station) of sampling		Confinement (Kpa)	Resilient Modulus (Mr) for confinement (σ_3)		h_1	h_2	a_1	$SN_1 = H_1 * a_1$	SN_2 (Excel solver)	$*a_2$	a_2 average.
			Kpa	Psi							
Ambessa Garage -Unity– M/Haile	0 + 120 sub - base (left and right) (3,809,671.980 ESAL)	100	220.42	31.96	15	50	0.44	6.6	21.56	0.25	
		100	305.91	44.36	15	50	0.44	6.6	19.8	0.22	0.24
		200	273.44	39.65	15	50	0.44	6.6	20.39	0.23	
		200	353.33	51.23	15	50	0.44	6.6	19.07	0.21	0.22
		300	277.5	40.24	15	50	0.44	6.6	20.31	0.23	
		300	367.65	53.31	15	50	0.44	6.6	18.87	0.21	0.22
	0 + 380 sub - base (left and right) (3,809,671.980 ESAL)	100	396.36	57.47	15	50	0.44	6.6	18.51	0.20	
		100	431.25	62.53	15	50	0.44	6.6	18.09	0.19	0.20
		200	432.31	62.68	15	50	0.44	6.6	18.09	0.19	
		200	653.75	94.79	15	50	0.44	6.6	16.23	0.16	0.18
		300	468.33	67.91	15	50	0.44	6.6	17.71	0.19	
		300	635.42	92.14	15	50	0.44	6.6	16.35	0.17	0.18
	0 + 740 sub - base (left and right) (3,809,671.980 ESAL)	100	532.5	77.21	15	50	0.44	6.6	17.13	0.18	
		100	349.44	50.67	15	50	0.44	6.6	19.12	0.21	0.20
		200	598.33	86.76	15	50	0.44	6.6	16.61	0.17	
		200	442.92	64.22	15	50	0.44	6.6	17.98	0.19	0.18
		300	621.43	90.11	15	50	0.44	6.6	16.45	0.17	
		300	451	65.4	15	50	0.44	6.6	17.89	0.19	0.18

where;-m_2 =1.18

$*a_2 = (SN_2 - SN_1) / H_2 * m_2$

Range of a_2 = [0.18 - 0.24]

Table 5. 27 summary of the structural layer coefficient (a_3) for Sub - Base for Ambessa Garage - Unity University - Mebrat Haile from calculation in appendix G and data of appendix I.

Location (Station) of sampling		Confinement(Kpa)	Resilient Modulus (Mr)		h1	h2	h3	a1	SN1 =H1*a1	SN2 (Excel solver)	SN3 (Excel solver)	a_2	a_3	a_3 Average.
			Kpa	Psi										
Ambessa Garage -Unity– M/Haile	0 + 120 sub - grade (left and right(3,809,671.980 ESAL)	100	202.45	29.34	15	50	50	0.44	6.6	21.56	22.11	0.25	0.010	0.01
		100	293.33	42.53	15	50	50	0.44	6.6	19.80	20.02	0.22	0.004	
		200	269.10	39.00	15	50	50	0.44	6.6	20.39	20.52	0.23	0.002	0.01
		200	298.77	43.30	15	50	50	0.44	6.6	19.07	19.94	0.21	0.016	
		300	258.06	37.40	15	50	50	0.44	6.6	20.31	20.76	0.23	0.008	0.01
		300	364.44	52.84	15	50	50	0.44	6.6	18.87	18.98	0.21	0.002	
	0 + 380 sub - grade (left and right) (3,809,671.980 ESAL)	100	305.50	44.30	15	50	50	0.44	6.6	18.51	19.81	0.20	0.024	0.03
		100	312.50	45.31	15	50	50	0.44	6.6	18.09	19.69	0.19	0.029	
		200	317.27	46.00	15	50	50	0.44	6.6	18.09	19.61	0.19	0.028	0.04
		200	315.42	45.74	15	50	50	0.44	6.6	16.23	19.64	0.16	0.062	
		300	346.82	50.29	15	50	50	0.44	6.6	17.71	19.16	0.19	0.026	0.04
		300	347.50	50.39	15	50	50	0.44	6.6	16.35	19.15	0.17	0.051	
	0 + 740 sub – grade (left and right) (3,809,671.980 ESAL)	100	305.92	44.36	15	50	50	0.44	6.6	17.13	19.8	0.18	0.049	0.03
		100	332.50	48.21	15	50	50	0.44	6.6	19.12	19.37	0.21	0.005	
		200	313.33	45.43	15	50	50	0.44	6.6	16.61	19.68	0.17	0.056	0.04
		200	336.50	48.79	15	50	50	0.44	6.6	17.98	19.31	0.19	0.024	
		300	356.67	51.72	15	50	50	0.44	6.6	16.45	19.02	0.17	0.047	0.04
		300	343.64	49.83	15	50	50	0.44	6.6	17.89	19.21	0.19	0.024	

Where; -

$a_2 = (SN_2 - SN_1) / H_2 * m_2$

$a_3 = (SN_3 - a_2h_2m_2 - SN_1)/H_3*m_3$

Range of a_3 = [0.01 - 0.04]

Note: - The remaining calculation of the structural layer coefficients for each material type at each station for the two projects is shown in appendix G at the end of this material.

Table 5. 28 Regression equation determinations of structural coefficient for sub - base for Shiro Meda - K/Mihiret H/Nooh from resilient modulus and average structural layer coefficient

Shiro Meda - Kidane Mihiret - Hamere Nooh	
Resilient Modulus (Mr) (psi) of sub -base	average a3
58.79	0.03
70.08	0.04
73.93	0.04
53.76	0.02
70.845	0.04
72.745	0.04
34.935	0.01
57.37	0.03
72.74	0.05
45.24	0.02
45.165	0.02
51.205	0.03

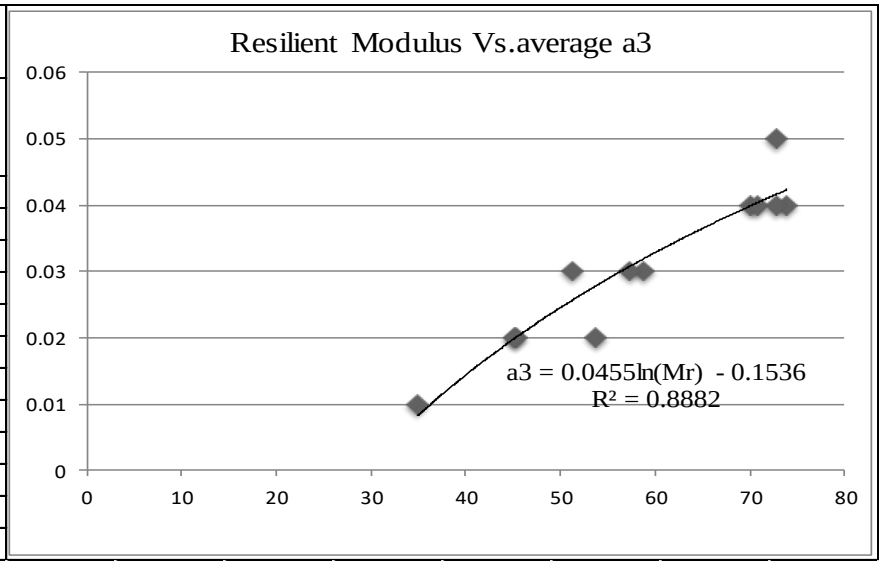
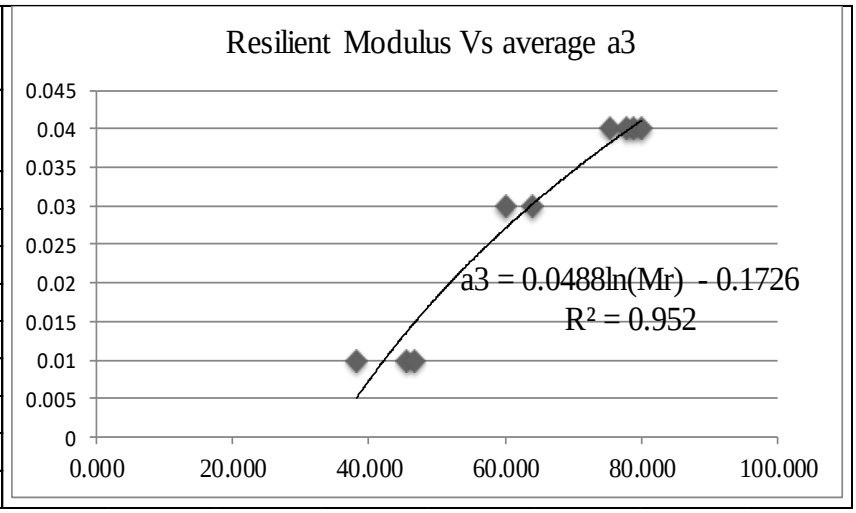


Table 5. 29 Regression equation determination of structural coefficient for sub - base for A/Garage - Unity Univ.- M/Haile from resilient modulus and average structural layer coefficient

Ambessa Garage - Unity University - Mebrat Haile	
Resilient Modulus (Mr) (psi) of sub -base	average a3
38.160	0.01
45.440	0.01
46.775	0.01
60.000	0.03
78.735	0.04
80.025	0.04
63.940	0.03
75.490	0.04
77.755	0.04



Finally, drainage coefficient (m) was developed as a ratio of the layer coefficient of road test site conditions under any given drainage and climate condition to layer coefficient of the AASHTO road test. The layer coefficients were calculated from resilient moduli determined for various conditions by use of elastic layered analysis [48].

For base material $m2 = \frac{\text{a2 Site}}{\text{a2 Road Test}} = \frac{0.24}{0.14} = 1.7$

For sub - base material $m3 = \frac{\text{a3 Site}}{\text{a3 Road Test}} = \frac{0.03}{0.11} = 0.27$

Both values are in the near range of AASHTO table values which is 0.4 -1.4 [2, 51].

CHAPTER 6 INTERPRETATION AND DISCUSSION OF TEST RESULTS

6.1 Interpretations of Test Results

As shown in the summary part of 4.7 the sieve analysis, PL, LL, PI, OMC, MDD, SG, CBR, and UU were conducted and the results were determined for sub-grade, sub-base, and base for each project.

Table 6.1 summary of interpretation of test results

project	Test type	Sub -grad	Sub -base	base
Shiro Meda – Kidane Mihiret Hamere Nooh	LL	56 -63	29 – 33	23
	PL	31 -34	19 -25	18 – 19
	PI	24 – 31	6 – 11	4 – 5
	Omc	20 – 26.30	8.6 – 15	9.8 left /12.5 right
	Mdd	1.49 – 1.54	1.868 – 2.220	2.1 left / 2.24 right
	CBR	9.10 – 21.43	35.71 – 49.14	82
	Sg	2.58 – 2.64	2.52 – 2.63	2.75left/ 2.67 right
Ambessa Garage – Unity university – Mebrate Haile	LL	56 – 64	28 – 35	21 - 22
	PL	27 – 32	19 – 28	16 - 17
	PI	28 – 34	7 – 12	4 - 5
	Omc	17.5 – 25.5	9.7 – 11.2	11 left / 11.2 right
	Mdd	1.46 – 1.56	1.925 – 2.050	2.072left /2.078 right
	CBR (%)	11.73 – 18.87	40.10 – 57.00	84.29
	Sg	2.56 – 2.61	2.53 – 2.57	2.63 left / 2.72 right
Shiro Meda area done by Ashenafi Mulgeta (2016)	LL	60 – 65	Not done	Not done
	PL	35 - 38		
	PI	25 - 27		
	Omc	31.13 – 33.33		
	Mdd	1.51 – 1.63		
	CBR (%)	12.46 – 14.27		
	Sg	2.60 – 2.64		
Addis Ababa Ethiopia and (AACRA/ERA)	LL	not limited	Not limited	26 max
	PL	Not limited	Not limited	20 max
	PI	< 30	6 - 12	6 max
	CBR (%)	20 - 40	30 min	80 min.

From the table 6.1 above, the test current research result is in conformance with the results obtained by Ashenafi Mulgeta and AACRA and/ or ERA.

The ranges of results of resilient modulus (Mr) obtained from the results of UU test is summarized in table 6.2 for Shiro Meda – Kidane Mihiret Hamere Nooh and Ambessa Garage – Unity University – Mebrate Haile projects.

Table 6.2 interpretation of resilient modulus (Mr) results

Projects	Confinement (σ_3) (KN)	Mr range for Sub – grade (KN)	Mr range for sub – base (KN)
Shiro Meda – Kidane Mihiret Hamere Nooh	σ_3 = 100	202.45 - 366.00	219.12 - 586.25
	σ_3 = 200	213.44 - 418.00	215.48 - 653.75
	σ_3 = 300	255.00 - 364.44	277.50 - 657.50
Ambessa – Garage – Unity university – Mebrate Haile	σ_3 = 100	290 – 332.50	220.42 – 532.50
	σ_3 = 200	313.33 – 355.56	273.44 – 653.75
	σ_3 = 300	333.5 – 364.44	277.50 – 635.52

Table 6.3 interpretation of Structural layer coefficient and Drainage coefficient results

Projects		Sub – base	base
Shiro Meda – Kidane Mihiret Hamere Nooh	Soil type	A – 2 -5	A – 1 -a
	Soil property	Fair permeability	Relatively good permeability
	Structural layer coefficient	a_3 = 0.01 – 0.05	a_2 = 0.18 – 0.30
	Drainage coefficient	m_3 = 1.15 – 1.05	m_2 = 1.20 – 1.15
Ambessa – Garage – Unity university – Mebrate Haile	Soil type	A – 2 -5	A – 1 -a
	Soil property	Fair permeability	Relatively good permeability
	Structural layer coefficient	a_3 = 0.01 – 0.04	a_2 = 0.18 – 0.30
	Drainage coefficient	m_3 = 1.15 – 1.05	m_2 = 1.20 – 1.15

On the other hand, Drainage coefficients obtained considering saturation and drainage quality for the base (m_2) is 1.37 and sub - base (m_3) is 1.38 for both projects.

Drainage coefficient (m) considering the ratio of the layer coefficient of road test site conditions under any given drainage and climate condition to layer coefficient of the AASHTO road test is for base material m_2 is 1.7 and for sub-base material m_3 is 0.27.

Generally, drainage coefficients obtained by all methods are in the range AASHTO drainage coefficient which is 0.4 - 1.4 except the base material drainage coefficient is 1.7 which has a little greater value. However, the drainage coefficient determined by this method is used for agricultural soil. Hence, in this thesis work the drainage coefficient obtained by the ratio of the layer coefficient of road test site conditions under any given drainage and climate condition to layer coefficient of the AASHTO road test is calculated by using the structural layer coefficient values obtained in the current thesis.

6.2 Discussion of Test Results for structural layer coefficient and Drainage coefficient

Generally, as shown in the summary result table 4.5, the sub-grade is AASHTO A-2- 7 which is poor permeability of gravel clay soil which needs improvement by good sub-base and base material above. For both projects, the base material is AASHTO A -1- a crushed aggregate of stone which is a relatively good permeable pavement material. The sub-base material is AASHTO A -2- 5 for both projects which are fair permeability granular soil. Considering both projects together LL is in the 56 - 64, PL is 27 - 34, PI is for 24 - 34, Sg is 2.58 - 2.64, CBR is 9.10 - 21.43 and Mr 202.45 - 366.00KN for σ_3 of 100KN, Mr is 213.44 - 418 KN for σ_3 of 200KN, Mr is 255.00 - 364.44 KN for σ_3 of 300KN for sub - grade. For sub - base LL is in the 28 - 35, PL is 19 - 28 and PI is for 6 - 12, Sg is 2.52 - 2.66, CBR is 35.71 - 57.00 and Mr is 219.12 - 586.25KN for σ_3 of 100KN, Mr is 215.48 - 653.75 KN for σ_3 of 200KN, Mr is 277.50 - 657.50 KN for σ_3 of 300KN. For base LL is in the 21 - 23, PL is 16 - 19 and PI is for 4 - 5, Sg is 2.63 - 2.75 and CBR is 82 - 87.70.

AS shown in table 5.24, 5.25, 5.26, and 5.27, the base layer coefficient (a_2) is in the range of 0.18 - 0.30, and the sub - base layer coefficient (a_3) is 0.01 - 0.05. Taking the average, the base structural layer (a_2) is 0.24 and the sub - base layer coefficient (a_3) is 0.03.

Generally, the layer coefficients values obtained in this research are in conformance to the AASHTO design requirement of 0.14, ERA Requirement of 0.135 and AACRA requirement of 0.15 for the layer coefficient of flexible pavement granular base and AASHTO and AACRA value of 0.11, ERA value of 0.105 for the layer coefficient of flexible pavement granular sub-base [1]. For base 0.20 and 0.26[1] which is obtained by the FWD test result, moreover, it is noteworthy, that irrespective of the correlation used.

For both projects, considering the AASHTO drainage coefficient table, the base material is AASHTO A-1-a crushed aggregate of stone which is a relatively good permeable pavement

material with a drainage coefficient ranging 1.20 - 1.15 average of 1.18, and the sub - base material is AASHTO A-2-5 which is fair permeability granular soil with drainage coefficient raging 1.15 - 1.05 average of 1.10 respectively as shown in the table 5.20. Also, the sub - grade is AASHTO A-2-7 which is poor permeability of gravel clay soil which needs improvement by good sub - base and base material above.

The following table compares the structural layer coefficient (ai) and Drainage coefficient (mi) test results of this thesis with the previous test results AASHTO, ERA, and AACRA.

Table 6.4 Comparison of test results for structural layer coefficient and drainage coefficient.

Researchers	year	Site (location)	SLC(ai)	Dc (mi)
AASHTO	1993	Ottowa, Illinois	*a_3 = 0.08	m_3 = 1.15 - 1.05
			a_2 = 0.14	m_2 = 1.20 - 1.15
AACRA	2004	Addis Ababa	a_3 = 0.11	m_3 = 1.15 - 1.05
			a_2 = 0.15	m_2 = 1.20 - 1.15
ERA	2013	All over Ethiopia	*a_3 = 0.08 - 0.105	m_3 = 1.15 - 1.05
			*a_2 =0.01- 0.145	m_2 = 1.20 - 1.15
Current research (sub - base)	2020	Shiro Meda (Gulele)	a_3 =0.01 - 0.05	m_3 = 0.25 - 1.00
Current research (sub - base)	2020	Ambessa garage (Bole)	a_3 = 0.01 - 0.04	m_3 = 0.13 - 0.88
Current research (base)	2020	Shiro Meda (Gulele)	a_2 = 0.18 - 0.30	m_2 =1.36 - 2.43
Current research (base)	2020	Ambessa garage (Bole)	a_2 = 0.18 - 0.24	m_2 = 1.29 - 2.00

ERA 2013 (Sub - Base material)

$a_3 = - 0.075 + 0.184 (\log_{10}^{CBR}) - 0.0444 (\log_{10}^{CBR})^2$ *CBR = 30% [0.105], CBR = 15% [0.08]

$a_2 = (29.14CBR - 0.1977CBR^2 + 0.00045CBR^3)*10^{-4}$ *CBR = [45 - 100] - [0.01 - 0.145]

- AASHTO structural layer coefficient, a_2, for a granular base material from its elastic (resilient) modulus,

  $a_3 = 0.249(\log_{10}^{Mr.Bs}) - 0.977$ ---------- [0.14] AASHTO

- AASHTO for sub – base material

  $a_3 = 0.227(\log_{10}^{Mr.SB}) - 0.839$ ---------- [0.08] AASHTO

- Shiro Meda – Kidane Mihiret Hamere Nooh (Sub –Base material)

  a3 = 0.0455(ln Mr_{SB}) – 0.1536 ---------- [0.01 – 0.05] R^2 = 0.8882 = 88.82%

- Ambessa Garage – Unity University – Mebrat Hail (Sub –Base material)

  a3 = 0.0488(ln Mr_{SB}) – 0.1726 ---------- [0.01 -0.04] R^2 = 0.9520 = 95.20%

6.3 Validation of the Result

The structural layer coefficient and drainage coefficient for sub-base and base could not be compared with the results obtained for Mr Results obtained from UU triaxial test results since none of them developed structural layer coefficient equation for sub - base and base from Mr Result obtained from UU test result. However, the results obtained are compared with structural layer coefficients and drainage for sub - base and base results obtained from AASHTO, ERA, and AACRA as discussed in the discussion part.

Hence to validate the proposed empirical equation one project is selected this is Ambessa Garage – Unity University – Mebrat Haile. For this project samples are collected at 3 (three) location. Like Shiro Meda – Kidane Mihiret Hamere Nooh project, all the required tests are performed for this project. Having all the test result of Ambessa Garage – Unity University – Mebrat Haile, the equation developed with the structural layer coefficient and drainage coefficient values for Shiro Meda – Kidane Mihiret Hamere Nooh project is compared as show in table below.

The validation of the Test result is carried out by Stata software **T – test** since it appropriate for two mean unpaired sample. The validation result is also shown in table 6.5.

- Label variable var1 "Shiro Meda Kidane Mihiret Hamere Nooh"
- Label variable var2 "Ambessa Garage Unity University Mebrat Haile"

Table 6.5 Validation of the developed empirical equation

```
. ttest var1 == var2, unpaired unequal

Two-sample t test with unequal variances
```

Variable	Obs	Mean	Std. Err.	Std. Dev.	[95% Conf.	Interval]
var1	12	.0308333	.0033616	.011645	.0234345	.0382322
var2	9	.0277778	.0046481	.0139443	.0170592	.0384963
combined	21	.0295238	.0027147	.0124403	.023861	.0351866
diff		.0030556	.0057363		-.0091383	.0152494

```
    diff = mean(var1) - mean(var2)                                t =   0.5327
Ho: diff = 0                     Satterthwaite's degrees of freedom =  15.4779

    Ha: diff < 0                 Ha: diff != 0                 Ha: diff > 0
 Pr(T < t) = 0.6991         Pr(|T| > |t|) = 0.6018          Pr(T > t) = 0.3009
```

The null hypothesis is H_O = there is no difference between the result of Shiro Meda - Kidane Mihiret Hamere Nooh and Ambessa Garage - Unity University - Mebrat Haile
Considering two tail test, since p = 0.6018 which is greater than the cutoff point p = 0.05 the null hypothesis is accepted. i.e. there is no significance deference between the structural layer coefficients of the two projects. Therefore the method used to develop structural layer coefficients is good to be used routinely.

6.4 Calculation of Pavement Thickness by Current Values

Taking the average structural layer coefficient and drainage coefficient for Addis Ababa from the current research result,

For sub – base average structural layer coefficient a_3 **is** a_3

$$a_3 = \frac{0.02+0.05}{2} = 0.03$$

For base, the average structural layer coefficient is a_2

$$a_2 = \frac{0.18+0.30}{2} = 0.24$$

For sub – base, the average drainage coefficient is $\mathbf{m_3}$ is

$$m_3 = \frac{0.25+0.88}{2} = 0.57$$

For base, the average drainage coefficient is $\mathbf{m_2}$

$$m_2 = \frac{1.43+2.86}{2} = 2.15$$

Then thickness calculation is conducted from the following equation

$SN_1 = a_1h_1$

$SN_2 = a_1h_1 + a_2h_2m_2$

$SN_3 = a_1h_1 + a_2h_2m_2 + a_3h_3m_3$

Calculation of pavement thickness h_3

$$h_2 = \frac{SN2 - a1h1}{a2m2}$$

$$h_3 = \frac{SN3 - a1h1 - a2h2m2}{a3m3}$$

From the above formula the calculation of the thickness sub – base and base is calculated and summarized below in table 6.3 and 6.4.for Shiro Meda - Kidane Mihiret Hamere Nooh project. For base course the average thickness obtained from the current structural layer and drainage coefficient is 99.17cm. Also, the average thickness obtained for sub - base course is 25.75cm. The negative value in table 6.4, shows no sub – base coarse is needed at all at some places for pavement design and construction.

Table 6.6 calculation of thickness for base coarse for Shiro Meda - Kidane Mihiret Hamere Nooh from current research result.

SN_2 = Excel (solver)	SN_1	a_2	m_2	$h2 = \frac{SN2 - a1h1}{a2m2}$
16.75	6.6	0.24	0.57	74.20
21.45	6.6	0.24	0.57	108.55
16.65	6.6	0.24	0.57	73.46
18.8	6.6	0.24	0.57	89.18
16.46	6.6	0.24	0.57	72.08
18.46	6.6	0.24	0.57	86.70
19.79	6.6	0.24	0.57	96.42
18.06	6.6	0.24	0.57	83.77
18.49	6.6	0.24	0.57	86.92
16.75	6.6	0.24	0.57	74.20
18.26	6.6	0.24	0.57	85.23
16.7	6.6	0.24	0.57	73.83
24.73	6.6	0.24	0.57	132.53
23.6	6.6	0.24	0.57	124.27
24.84	6.6	0.24	0.57	133.33
19.25	6.6	0.24	0.57	92.47
21.98	6.6	0.24	0.57	112.43
18.6	6.6	0.24	0.57	87.72
22.74	6.6	0.24	0.57	117.98
22.41	6.6	0.24	0.57	115.57
23.96	6.6	0.24	0.57	126.90
21.52	6.6	0.24	0.57	109.06
22.2	6.6	0.24	0.57	114.04
21.55	6.6	0.24	0.57	109.28
average				**99.17**

Table 6.7 calculation of thickness for sub - base coarse for Shiro Meda - Kidane Mihiret Hamere Nooh from current research result.

SN_3 = (Excel solver)	SN1= H_1*a1	SN_2 = (Excel solver)	h_2 (cm)	a_2	a_3	m_2	m3	$h3 = \frac{SN3 - a1h1 - a2h2m2}{a3m3}$
19.83	6.6	16.7	99.17	0.24	0.03	0.57	2.15	-5.22
21.54	6.6	21.45	99.17	0.24	0.03	0.57	2.15	21.30
19.16	6.6	16.65	99.17	0.24	0.03	0.57	2.15	-15.60
20.85	6.6	18.8	99.17	0.24	0.03	0.57	2.15	10.60
19.63	6.6	16.46	99.17	0.24	0.03	0.57	2.15	-8.32
19.94	6.6	18.46	99.17	0.24	0.03	0.57	2.15	-3.51
20.35	6.6	19.79	99.17	0.24	0.03	0.57	2.15	2.85
19.71	6.6	18.06	99.17	0.24	0.03	0.57	2.15	-7.08
19.15	6.6	18.49	99.17	0.24	0.03	0.57	2.15	-15.76
20.24	6.6	16.75	99.17	0.24	0.03	0.57	2.15	1.14
19.32	6.6	18.26	99.17	0.24	0.03	0.57	2.15	-13.12
20.18	6.6	16.7	99.17	0.24	0.03	0.57	2.15	0.21
24.93	6.6	24.73	99.17	0.24	0.03	0.57	2.15	73.85
24.23	6.6	23.6	99.17	0.24	0.03	0.57	2.15	63.00
24.94	6.6	24.84	99.17	0.24	0.03	0.57	2.15	74.01
21.99	6.6	19.25	99.17	0.24	0.03	0.57	2.15	28.27
23.74	6.6	21.98	99.17	0.24	0.03	0.57	2.15	55.40
22.67	6.6	18.6	99.17	0.24	0.03	0.57	2.15	38.81
24.04	6.6	22.74	99.17	0.24	0.03	0.57	2.15	60.05
23.27	6.6	22.41	99.17	0.24	0.03	0.57	2.15	48.12
24.9	6.6	23.96	99.17	0.24	0.03	0.57	2.15	73.39
22.73	6.6	21.52	99.17	0.24	0.03	0.57	2.15	39.74
23.78	6.6	22.2	99.17	0.24	0.03	0.57	2.15	56.02
22.73	6.6	21.55	99.17	0.24	0.03	0.57	2.15	39.74
average								**25.75**

CHAPTER 7 CONCLUSION AND RECOMMENDATION

7.1 Conclusion

Recalibration of structural layer coefficient and drainage coefficient for sub - base and base pavement material for some projects in Addis Ababa is the primary objective of this thesis by determining Mr from UU triaxial test. UU test is available in most geotechnical laboratories. Also it is cheap, fast, and easy to be used routinely for road research and design process.

Even though, UU test is used for cohesive soil of low permeability, it is used for soil of all type which has good permeability when compared to CU and CD triaxial test. In UU test, stress distribution is uniform, drainage conditions can be controlled totally and the confining pressure occurred can be measured. This type of test is used for either undisturbed or remolded samples and accurate for research purposes. Therefore, this type of test method relatively describes the traffic load effect on pavement structures.

Accordingly, the following conclusion is drawn;-

1. Based on the results obtained, the variation of structural layer coefficient and drainage coefficient for Addis Ababa is 0.01 - 0.05 for sub - base and 0.18 - 0.30 for base pavement material as shown in table 7.1. Also, the variation of drainage coefficient for Addis Ababa is 0.25 - 0.88 and 1.43 - 2.89 for base pavement material as shown in table 7.2.

Table 7.1 Conclusion table for structural layer coefficient

Pavement material type	Description	Shiro Meda - K/Mihiret H/Nooh (left and right carriageway)	Ambessa garage - unity university – M/Haile (left and right carriageway)
Sub – base	Structural layer coefficient	0.01 - 0.07	0.002 - 0.062
	average Structural layer coefficient	0.01 - 0.05	0.01 - 0.04
	variations of the structural layer coefficient for Addis Ababa	0.01 - 0.05	
	AASHTO ERA AACRA	0.11 0.105 0.11	
Base	Structural layer coefficient	0.17 - 0.31	0.16 - 0.25
	average Structural layer coefficient	0.18 - 0.24	0.18 - 0.24
	variations of the structural layer coefficient for Addis Ababa	0.18 - 0.30	
	AASHTO ERA AACRA	0.14 0.135 0.15	

Table 7.2 conclusion table for Drainage coefficient

Pavement material	Description	Shiro Meda - K/Mihiret H/Nooh (left and right carriageway)	Ambessa garage -Unity university-M/Haile for (left and right carriageway)
Sub – base	drainage coefficient	0.25 - 1.00	0.13 - 0.88
	average drainage coefficient	0.25 -0.875	0.25 - 0.625
	variations of the drainage coefficient for Addis Ababa	0.25 - 0.88	
	AASHTO/ ERA/ AACRA drainage coefficient value	1.15 - 1.05	
Base	drainage coefficient	1.36 - 2.43	1.29 - 2.0
	average drainage coefficient	1.43 - 2.86	1.43 - 1.89
	variations of drainage coefficient for Addis Ababa	1.43 - 2.86	
	AASHTO/ ERA/ AACRA drainage coefficient value	1.20 - 1.15	

2. As shown in table 7.3 the average thickness result obtained for base course for Shiro Meda - Kidane Mihiret Hamere Nooh is 99.17cm which is greater than the value of 50cm obtained from engineering report. Also, the average thickness obtained for sub - base course for Shiro Meda – Kidane Mihiret Hamere Nooh is 25.75cm which is less than the value of 50cm obtained from engineering report which shows a save in sub - base coarse. The negative value in table 6.3, shows no sub – base coarse is needed at all at some places.

Table 7.3 thickness saved from or added to the engineering report result

Pavement layer	Current research thickness result	Engineering report thickness value	Thickness added or saved
Sub – base	25.75cm	50cm	Saved 24.25cm
Base	99.17cm	50cm	Added 49.17cm

3. from the results shown in table 7.1 structural layer coefficient for Addis Ababa can be taken as between 0.01 - 0.05 for sub - base and 0.18 - 0.30 for base coarse. In the same way, the drainage coefficient for Addis Ababa can be taken as between 0.25 - 0.88 for sub - base and 1.43 - 2.86 for base coarse as shown in table 7.2.
4. since the values obtained by this method is near that of AACRA, ERA and AASHTO values, it can be taken as the structural layer coefficient and drainage coefficient values of Ethiopia.

7.2 Recommendation

Based on the following result of the this thesis work, the following recommendations were given below;

1. As shown in this thesis work, the UU test result is in agreement with the AASHTO test result with little conservatism. Since the UU test is obtained in most road constructing contractors, consultants and universities it can be used routinely to determine structural layer coefficients for the sub-base and base. However, the UU test for the determination of structural layer coefficients should be calibrated by the cyclic resilient modulus test (AASHTO T307) result.
2. If the UU test for the determination of SLC is calibrated, it is possible to determine SLC for a specific road project. After determining SLC for different projects in Addis Ababa as well as in Ethiopia, it is possible to make different Zones of similar or nearly similar narrow range SCL values for Addis Ababa as well as for Ethiopia in general. It is good to use a narrow range value for a specific zone rather than using the AASHTO value since these values can best simulate traffic and pavement conditions for that specific zone. This work is left for future work by me or other researchers.
3. Further studies for many soil types have to be carried out for other geographical areas of Addis Ababa which are not covered by this research to check the applicability of the equations for Addis Ababa.
4. It is also recommended to carry out such a study in other parts of Ethiopia to have more data.

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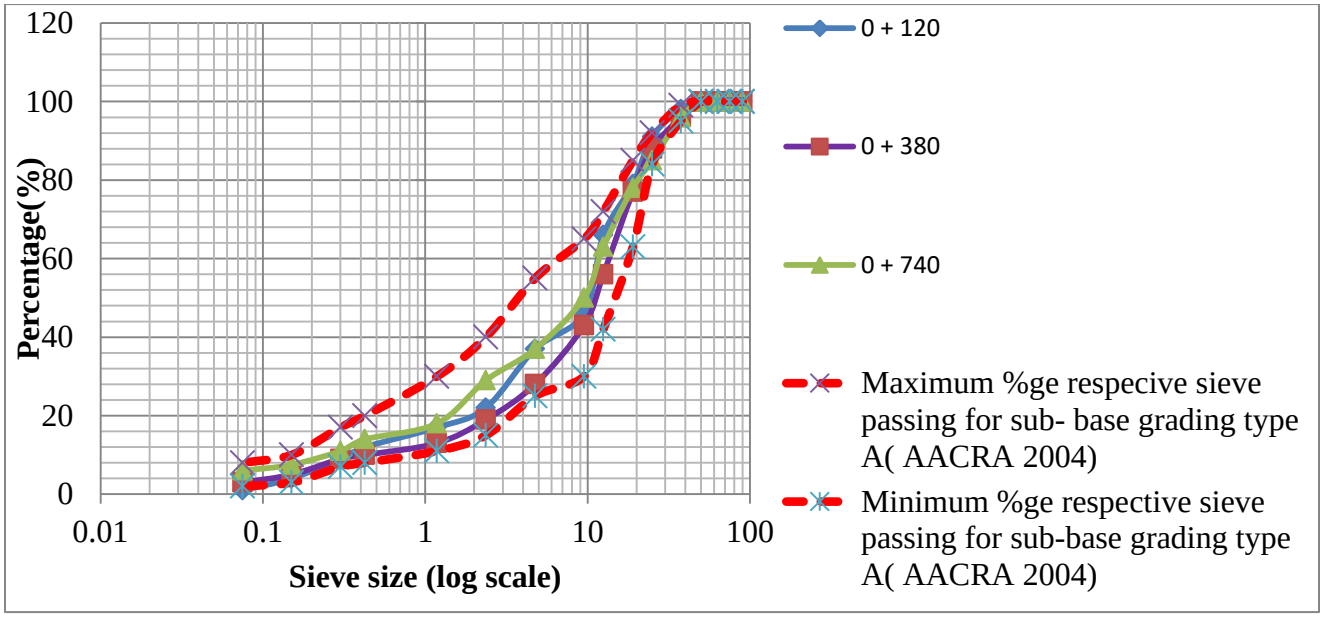
APPENDIX

Appendix A. Sieve analysis

Appendix A. 1 Summary of sieve analysis for sub-base pavement materials for Ambessa Garage – Unity University – Mebrat Haile site for right carriageway.

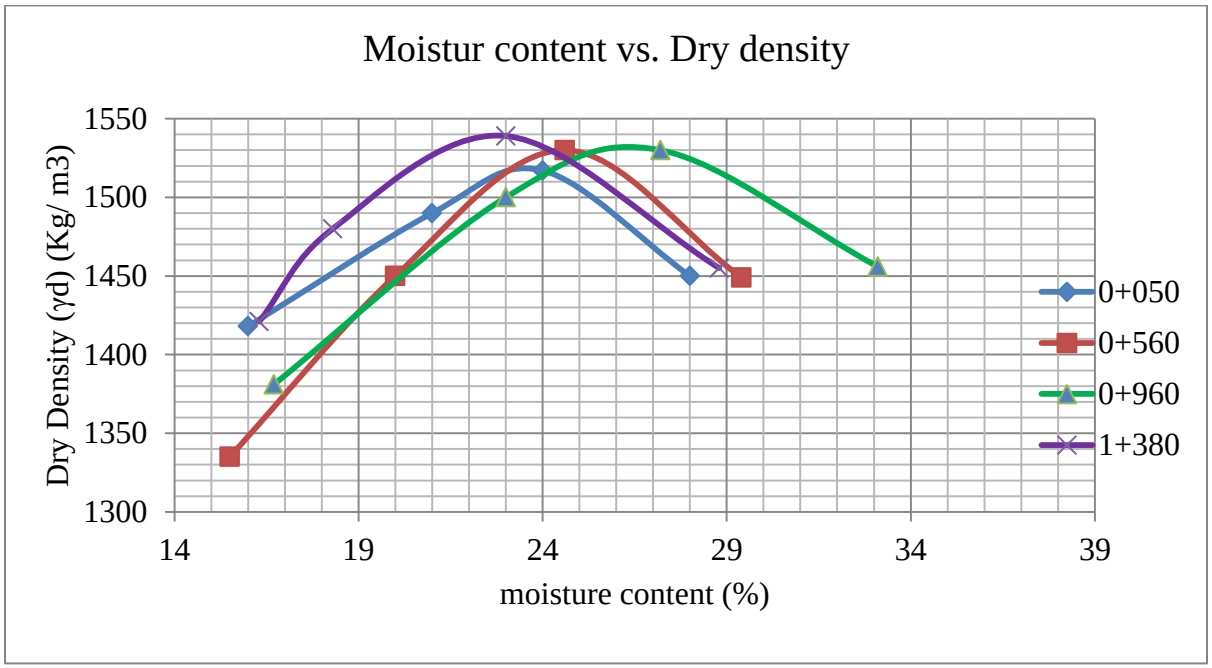
Sieve opening size	Total percent passing			Maximum percentage respective sieve passing for sub-base grading type A	minimum percentage respective sieve passing for sub-base grading type A
	0+120	0+380	0+740		
90	100	100	100	100	100
75	100	100	100	100	100
63	100	100	100	100	100
50	100	100	100	100	100
37.5	98	96	96	99	95
25	91	88	85	92	84
19	79	77	78	85	63
12.5	66	56	63	72	42
9.5	46	43	50	65	30
4.75	37	28	37	55	25
2.36	22	19	29	40	15
1.18	17	13	18	30	11
0.425	12	10	14	20	8
0.3	9	9	11	17	7
0.15	4	5	7.5	10	3
0.075	1	3	6	8	2



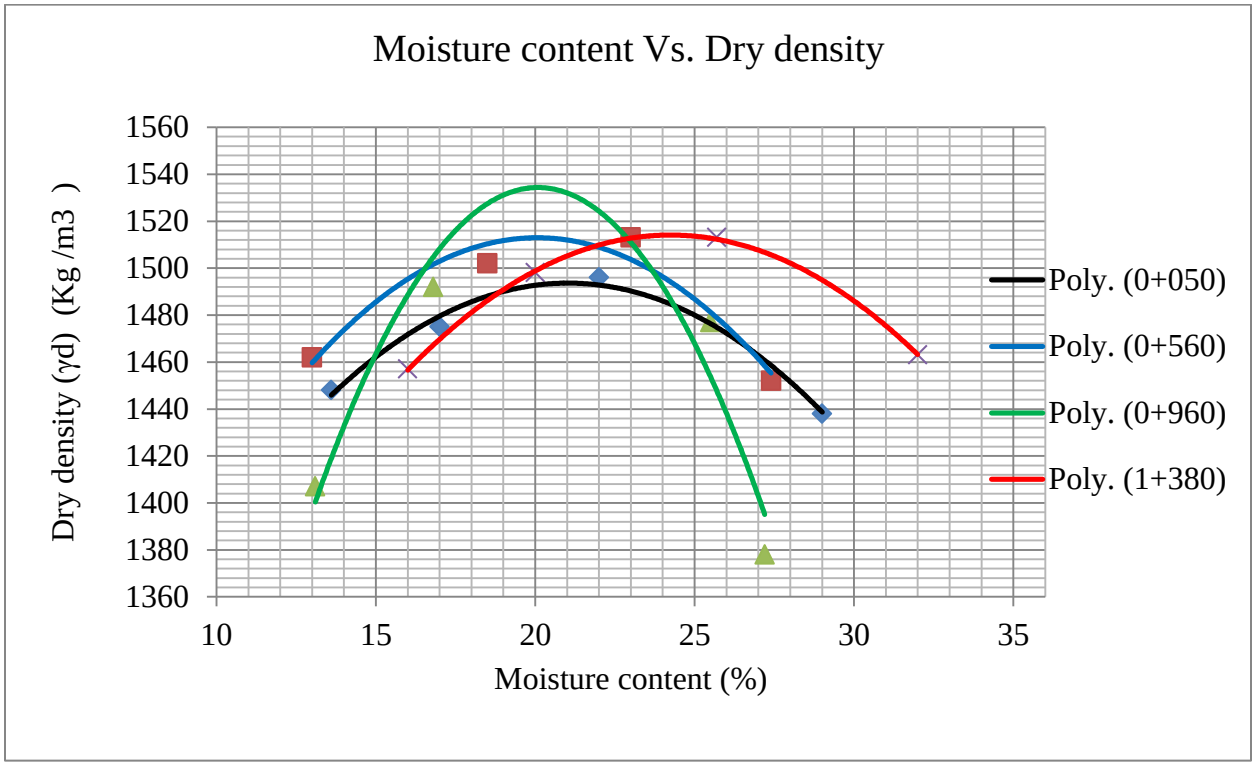
Appendix A. 6 Summary of sieve analysis for sub-base pavement materials for Ambessa Garage – Unity University – Mebrat Haile site for right carriageway.

Appendix B. Proctor Test Result

Appendix - B - 1. Proctor Test Result for Shiro Meda - Kidane Mihiret Hamere Nooh.



Appendix B -1 - i. Summary of Sub - grade soil Proctor Test Result for left carriageway.



Appendix – B -1 - ii. Summary of Sub - grade soil Proctor Test Result for right carriageway

Appendix C. Liquid limit and plastic limit test

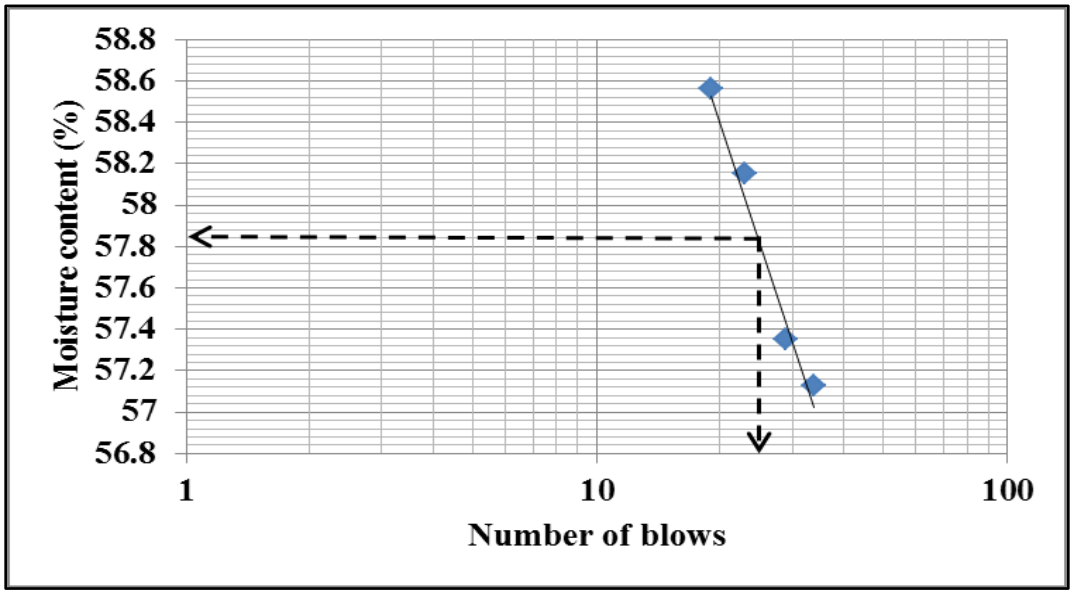
Appendix C.1. Sub - grade Pavement Material Liquid Limit and Plastic Limit Result for Shiro Meda - Kidane Mihiret Hamere Nooh project for 1 + 380 station (Right carriageway)

	Liquid Limit				**Plastic Limit**	
Container number	7	8	9	10	11	12
Number of blows	19	23	29	34		
Mass of Empty container (A)	15.0	16.2	15.5	16.3	15.5	15.0
Mass of container + Wet soil (B)	50.73	54.52	51.79	55.48	28.58	27.21
Mass of container + Dry soil (C)	37.53	40.43	38.56	41.23	25.40	24.18
Mass of wet soil (D = B - A)	35.73	38.32	36.29	39.18	13.08	12.21
Mass of dry soil (E= C - A)	22.53	24.23	23.06	24.93	9.90	9.18
Moisture content([D - E]/E *100)	58.56	58.15	57.35	57.13	32.12	32.94
	Average PL(%) = 32.53					

Liquid Limit (%) 58

Plastic Limit (%) 33

Plasticity Index (%) 25



LL = Moisture content at 25 blows = **57.82**

Graph C - 1 liquid limit **of** Sub - grade Result for Shiro Meda - Kidane Mihiret Hamere Nooh project for 1 + 380 station (Right carriageway)

Appendix D. Specific Gravity.

Appendix D.1 Base Pavement Material Specific Gravity Result for Ambesa Garage - Unity University - Mebrat Haile Left Carriageway

Observation Sheet for Station 0 + 460 (base material)

i) Coarse Aggregate

	Description	Test Number(left)		Test Number(Right)	
		I	II	I	II
1	Weight Of Sample	1000	1000	1000	1000
2	Weight Of Vessel + Sample + Water (A)	1698.2	1702.8	1709.1	1702.8
3	Weight Of Vessel + Water (B)	1074.5	1074.5	1074.5	1074.5
4	Weight Of Saturated & Surface Dry Sample(C)	967.4	965.7	989.2	965.7
5	Weight Of Oven Dry Sample (D)	946.6	945.2	978.7	945.2
6	SG =D/C- (A - B)	2.6	2.7	2.8	2.7

Sg (Left) = (2.6+2.7)/2 = **2.65** Sg(Right) = (2.8+2.7)/2 = **2.80**

ii) Fine Aggregate

	DESCRIBTION	Test Number(Left)		Test Number(Right)	
		I	II	I	II
1	Weight Of Sample (Wo)	30	30	30	30
2	Weight Of Pycnometer + Water (WA)	169.24	169.24	169.24	169.24
3	Weight Of Pycnometer+Water+Sample(WB)	187.66	187.79	187.79	187.96
4	$SG = \frac{Wo}{Wo + (Wa - WB)}$	2.59	2.62	2.62	2.66

Sg (Left) = (2.59 + 2.62)/2 = **2.61** Sg (Right) = (2.62+2.66)/2 = **2.64**

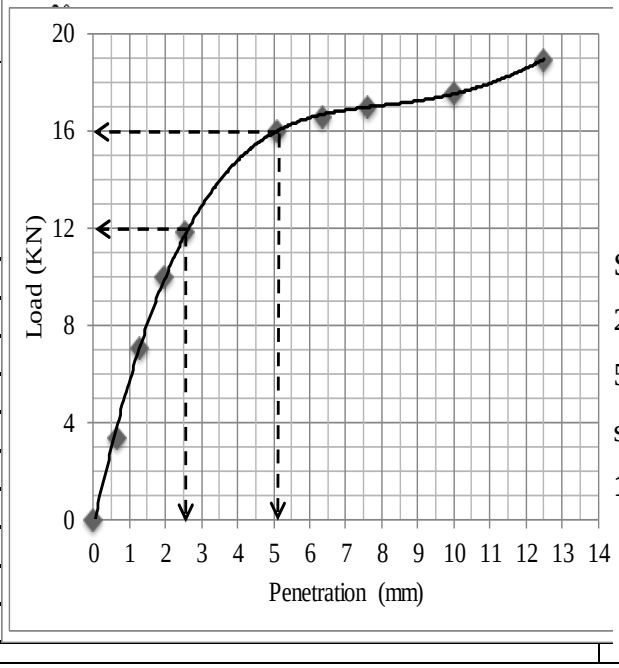
The specific gravity of the material is the weighted average of the fine aggregate and coarse aggregate result for left carriageway. i.e. SG = [(2.65*1000) + (2.61*30)]/2 = **2.65**

The specific gravity of the material is the weighted average of the fine aggregate and coarse aggregate result for right carriageway. i.e. SG = [(2.64*1000) + (2.80*30)]/2= **2.64**

Appendix E. California Bearing Ratio (CBR)

Appendix E. 1 Sub - grade pavement material California bearing ratio (CBR) result for Kidane Mihiret - Hamere Nooh for left carriageway and right carriageway

Test NO:-**0 + 050**		Ring constant (f)= 0.02415KN/DIV	
Penetration	Load dial reading (Div *5) (KN)	Total load *f	CBR
0	0	0	
0.64	220	3.313	
1.27	310	7.0371	
1.97	413	9.9735	
2.54	491	11.8513	17.14%
5.08	662	15.9800	15.24%
6.35	686	16.5584	
7.62	703	16.9845	
10	727	17.5571	
12.5	783	18.9165	

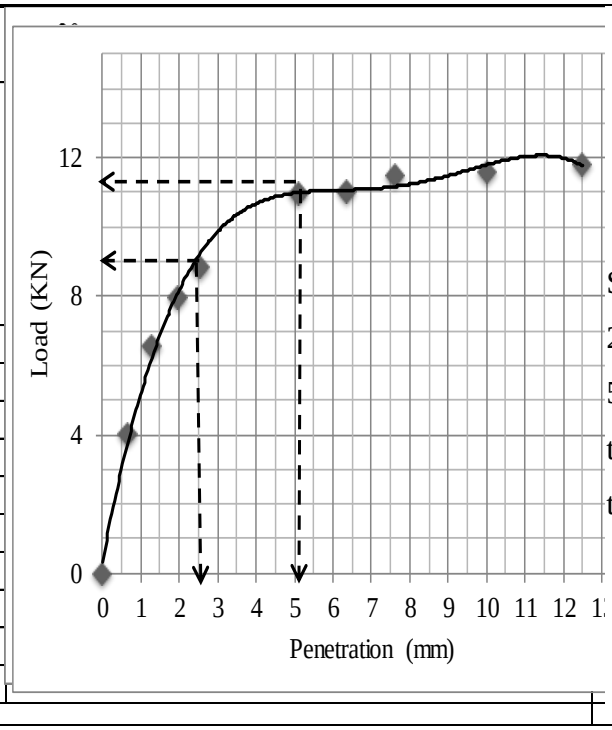


- CRB@2.54mm = 17.1%
- CRB@5.08mm = 15.2%

Since CBR At 2.54mm > CBR at 5.08mm, CBR of the sample is equal to **17.1%**

Graph E. 1.1.CBR Penetration data Observation Sheet for Station 0 + 050 (left carriageway)

Test NO:-0 +05**0**		Ring constant (f)= 0.02415KN/DIV	
Penetration	Load dial reading (Div *5) (KN)	Total load *f	CBR
0	0	0	
0.64	170	4.1055	
1.27	259	6.2548	
1.97	324	7.8246	
2.54	361	8.7182	12.5%
5.08	462	11.1573	10.60%
6.35	471	11.3747	
7.62	473	11.423	
10	481	11.6162	
12.5	487	11.7611	



- CRB@2.54mm = 12.50%
- CRB@5.08mm = 10.60%

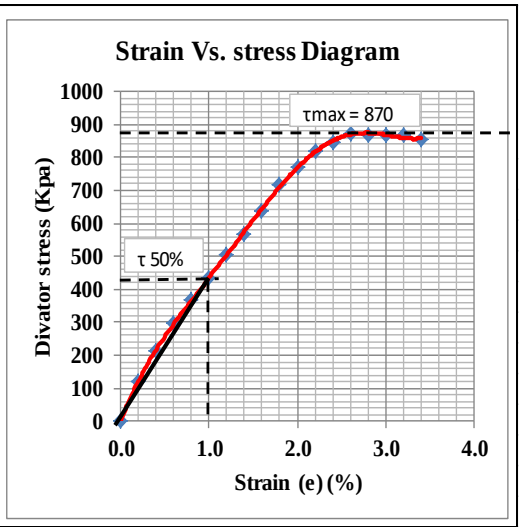
Since CBR At 2.54mm > CBR at 5.08mm, CBR of the sample is equal to **12.50%**

Graph E. 1.2 CBR Penetration data Observation Sheet for Station 0+050(Right carriageway)

Appendix F. Unconsolidated Undrained (UU) Test

Appendix F.1.i. Sub - base pavement material UU test result for Shiro Meda – Kidane Mihiret Hamere Nooh for Right carriageway

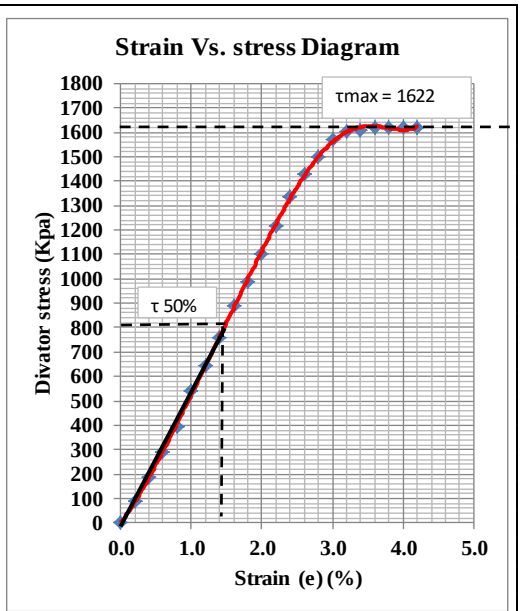
strain Dial reading mm	change in h at failure mm	strain (e) =dh/H0	original area	Corected Area Ao/(1-e)	Devator dial reading	devator stress kn	devator kn/m2
0	0	0.0000	0.001075	0.00108	0	0.0000	0
20	0.2	0.0027	0.001075	0.00108	63	0.1292	120
40	0.4	0.0054	0.001075	0.00108	112	0.2296	212
60	0.6	0.0081	0.001075	0.00108	158	0.3239	299
80	0.8	0.0108	0.001075	0.00109	194	0.3977	366
100	1.0	0.0135	0.001075	0.00109	232	0.4756	436
120	1.2	0.0162	0.001075	0.00109	270	0.5535	506
140	1.4	0.0189	0.001075	0.00110	303	0.6212	567
160	1.6	0.0216	0.001075	0.00110	342	0.7011	638
180	1.8	0.0243	0.001075	0.00110	385	0.7893	716
200	2.0	0.0270	0.001075	0.00111	416	0.8528	772
220	2.2	0.0297	0.001075	0.00111	443	0.9082	820
240	2.4	0.0324	0.001075	0.00111	459	0.9410	847
260	2.6	0.0351	0.001075	0.00111	473	0.9697	870
280	2.8	0.0378	0.001075	0.00112	474	0.9717	870
300	3.0	0.0405	0.001075	0.00112	474	0.9717	867
320	3.2	0.0432	0.001075	0.00112	475	0.9738	866



Origin coordinate		Coordinte at point	
(0,0)		(1.0, 435)	
X1	X2	Y1	Y2
0	1	0	435
slope = E = Mr			435

UU test result Observation Sheet for Station 0 + 560 for σ3 = 100kpa (Machine factor; - 2.05)

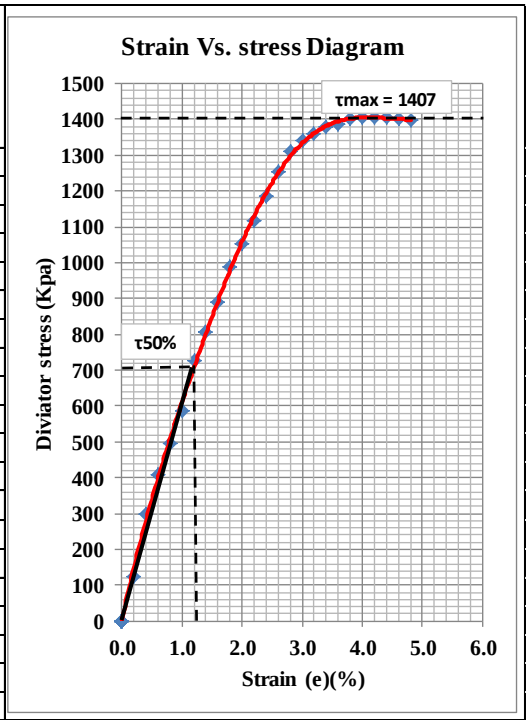
strain Dial reading mm	change in h at failure mm	strain (e) =dh/H0	original area	Corected Area Ao/(1-e)	Devator dial reading	devator stress kn	devator kn/m2
0	0	0.0000	0.001075	0.00108	0	0.0000	0
20	0.2	0.0027	0.001075	0.00108	47	0.0964	89
40	0.4	0.0054	0.001075	0.00108	98	0.2009	186
60	0.6	0.0081	0.001075	0.00108	152	0.3116	287
80	0.8	0.0108	0.001075	0.00109	209	0.4285	394
100	1.0	0.0135	0.001075	0.00109	288	0.5904	542
120	1.2	0.0162	0.001075	0.00109	343	0.7032	643
140	1.4	0.0189	0.001075	0.00110	407	0.8344	761
160	1.6	0.0216	0.001075	0.00110	477	0.9779	890
180	1.8	0.0243	0.001075	0.00110	532	1.0906	990
200	2.0	0.0270	0.001075	0.00111	595	1.2198	1104
220	2.2	0.0297	0.001075	0.00111	657	1.3469	1215
240	2.4	0.0324	0.001075	0.00111	726	1.4883	1339
260	2.6	0.0351	0.001075	0.00111	776	1.5908	1428
280	2.8	0.0378	0.001075	0.00112	818	1.6769	1501
300	3.0	0.0405	0.001075	0.00112	859	1.7610	1571
320	3.2	0.0432	0.001075	0.00112	878	1.7999	1602
340	3.4	0.0459	0.001075	0.00113	885	1.8143	1610
360	3.6	0.0486	0.001075	0.00113	893	1.8307	1620
380	3.8	0.0514	0.001075	0.00113	897	1.8389	1622
400	4.0	0.0541	0.001075	0.00114	899	1.8430	1621
420	4.2	0.0568	0.001075	0.00114	900	1.8450	1619



Origin coordinate		Coordinte at point	
(0,0)		(1.4, 811)	
X1	X2	Y1	Y2
0	1.4	0	811
slope = E = Mr			579.286

UU test result Observation Sheet for Station 0 + 560 for σ3 = 200kpa (Machine factor; - 2.05)

strain Dial reading mm	change in h at failure mm	strain (e) =dh/H0	original area	Corected Area Ao/(1-e)	Devator dial reading	devator stress kn	devator kn/m2
0	0	0.0000	0.001075	0.00108	0	0.0000	0
20	0.2	0.0027	0.001075	0.00108	65	0.1333	124
40	0.4	0.0054	0.001075	0.00108	157	0.3219	298
60	0.6	0.0081	0.001075	0.00108	216	0.4428	408
80	0.8	0.0108	0.001075	0.00109	263	0.5392	496
100	1.0	0.0135	0.001075	0.00109	311	0.6376	585
120	1.2	0.0162	0.001075	0.00109	387	0.7934	726
140	1.4	0.0189	0.001075	0.00110	431	0.8836	806
160	1.6	0.0216	0.001075	0.00110	478	0.9799	892
180	1.8	0.0243	0.001075	0.00110	531	1.0886	988
200	2.0	0.0270	0.001075	0.00111	567	1.1624	1052
220	2.2	0.0297	0.001075	0.00111	604	1.2382	1117
240	2.4	0.0324	0.001075	0.00111	643	1.3182	1186
260	2.6	0.0351	0.001075	0.00111	681	1.3961	1253
280	2.8	0.0378	0.001075	0.00112	714	1.4637	1310
300	3.0	0.0405	0.001075	0.00112	733	1.5027	1341
320	3.2	0.0432	0.001075	0.00112	745	1.5273	1359
340	3.4	0.0459	0.001075	0.00113	758	1.5539	1379
360	3.6	0.0486	0.001075	0.00113	765	1.5683	1388
380	3.8	0.0514	0.001075	0.00113	775	1.5888	1402
400	4.0	0.0541	0.001075	0.00114	780	1.5990	1407
420	4.2	0.0568	0.001075	0.00114	782	1.6031	1406
440	4.4	0.0595	0.001075	0.00114	783	1.6052	1404
460	4.6	0.0622	0.001075	0.00115	784	1.6072	1402
480	4.8	0.0649	0.001075	0.00115	784	1.6072	1398



Origin coordinate		Coordinte at point	
(0,0)		(1.2, 691)	
X1	X2	Y1	Y2
0	1.2	0	703.382
slope = E = Mr			586.152

UU test result Observation Sheet for Station 0 + 560 for σ3 = 300kpa (Machine factor; - 2.05)

Appendix G. Structural Number Calculation

Appendix G.1 SN result for unity to Ambesa garage - Unity University –M/Haile

Appendix G.1.1 sub - grade pavement material SN calculation using excel solver result for Ambessa garage - Unity University - M/Haile for Right carriageway

SN calculation result Observation Sheet for Station 0 + 380 for σ3 = 100kpa

AASHTO FLEXIBLE PAVEMENT DESIGN

SN Determination (solver) Design Inputs

W18 =	3,809,672.980		*ESALs Applications Over Design Period*	Typ. Range 0.1 to 80 million
R =	95	%	*Reliability*	Typ. Range 80 to 95%
So =	0.49		*Standard Deviation*	Typ. Range 0.3 to 0.5
MR =	45.31	psi	*Subgrade Resilient Modulus*	Typ. Range 3000 to 9000 psi
Pi =	4.2		*Initial Serviceability*	Typ. Range 4.4 to 4.8
Pt =	2.5		*Terminal Serviceability*	Typ. Range 2.0 to 3.0
	DESIGN SN =	19.21		

SN calculation result Observation Sheet for Station 0 + 380 for σ3 = 200kpa

AASHTO FLEXIBLE PAVEMENT DESIGN

SN Determination (solver) Design Inputs

W18 =	3,809,672.980		*ESALs Applications Over Design Period*	Typ. Range 0.1 to 80 million
R =	95	%	*Reliability*	Typ. Range 80 to 95%
So =	0.49		*Standard Deviation*	Typ. Range 0.3 to 0.5
MR =	46	psi	*Subgrade Resilient Modulus*	Typ. Range 3000 to 9000 psi
Pi =	4.2		*Initial Serviceability*	Typ. Range 4.4 to 4.8
Pt =	2.5		*Terminal Serviceability*	Typ. Range 2.0 to 3.0
	DESIGN SN =	19.64		

SN calculation result Observation Sheet for Station 0 + 380 for σ3 = 300kpa

AASHTO FLEXIBLE PAVEMENT DESIGN

SN Determination (solver) Design Inputs

W18 =	3,809,672.980		*ESALs Applications Over Design Period*	Typ. Range 0.1 to 80 million
R =	95	%	*Reliability*	Typ. Range 80 to 95%
So =	0.49		*Standard Deviation*	Typ. Range 0.3 to 0.5
MR =	50	psi	*Subgrade Resilient Modulus*	Typ. Range 3000 to 9000 psi
Pi =	4.2		*Initial Serviceability*	Typ. Range 4.4 to 4.8
Pt =	2.5		*Terminal Serviceability*	Typ. Range 2.0 to 3.0
	DESIGN SN =	19.15		

Appendix H. Regression Constants K1, K2, K3 Calculation

H.1 Matrix solution for K_1, K_2 and K_3 from resilient Modulus (Mr) values obtained from UU test result Shiro Meda - Kidane Mihiret Hamere Nooh sub - base for left and right carriageway.

For left carriageway

	y = lnMr
Y =	6.37
	6.39
	6.43
	5.73
	5.99
	6.03
	5.39
	5.37
	5.85
	5.71
	5.51
	5.81

	Y	X1 = ln(θ/pa)	X2 = ln(τoct/pa)
C left =	1	2.51	1.68
	1	2.81	1.79
	1	3.00	1.83
	1	2.21	1.36
	1	2.57	1.46
	1	2.90	1.67
	1	2.35	1.50
	1	2.71	1.66
	1	2.89	1.65
	1	2.59	1.76
	1	2.88	1.88
	1	3.04	1.88

Transpose of C for left carriageway for Shiro Meda - Kidane Mihiret Hamere Nooh for sub - base material

C' Left =	1	1	1	1	1	1	1	1	1	1	1	1
	2.51	2.81	3.00	2.21	2.57	2.90	2.35	2.71	2.89	2.59	2.88	3.04
	1.68	1.79	1.83	1.36	1.46	1.67	1.50	1.66	1.65	1.76	1.88	1.88

Determination of liner coefficient for left side carriageway sub –base

$$C' * C = \begin{pmatrix} 12 & 32.46 & 20.12 \\ 32.46 & 88.56 & 54.82 \\ 20.12 & 54.82 & 34.42 \end{pmatrix} \qquad C' * Y = \begin{pmatrix} 70.58 \\ 191.23 \\ 118.50 \end{pmatrix}$$

$$\begin{pmatrix} 12.00 & 32.46 & 20.12 \\ 32.46 & 88.56 & 54.82 \\ 20.12 & 54.82 & 34.42 \end{pmatrix} * \begin{pmatrix} ao \\ a1 \\ a2 \end{pmatrix} = \begin{pmatrix} 70.58 \\ 192.23 \\ 118.50 \end{pmatrix} \qquad \text{Then, } \begin{pmatrix} ao \\ a1 \\ a2 \end{pmatrix} = \begin{pmatrix} 4.418 \\ 0.333 \\ 0.167 \end{pmatrix}$$

For $Mr = k1\left(\frac{\sigma sum}{pa}\right)^{K2}\left(\frac{\tau oct}{pa}+1\right)^{K3}$ from the above values $K1 = \frac{e^{ao}}{pa}$, then K1 = 93.00

$K_2 = a_1 = 0.333$, $K_3 = a_2 = 0.167$

For the Left carriageway,

$$Mr = k1\left(\frac{\sigma sum}{pa}\right)^{K2}\left(\frac{\tau oct}{pa}+1\right)^{K3} = 93\left(\frac{\sigma sum}{pa}\right)^{0.333}\left(\frac{\tau oct}{pa}+1\right)^{0.167}$$

H.2 multiple regressions using STATA software for K_1, K_2 and K_3 for Shiro Meda – Kidane Mihiret Hamere Nooh Mihiret Hamere Nooh sub - base for left and right carriageway.

```
. regress k1 ll pl omc mdd sg
note: omc omitted because of collinearity

      Source |       SS       df       MS              Number of obs =       8
-------------+------------------------------           F(  4,     3) =    1.55
       Model |  3927.12199     4  981.780498            Prob > F      =  0.3752
    Residual |  1904.87801     3  634.959336            R-squared     =  0.6734
-------------+------------------------------           Adj R-squared =  0.2379
       Total |       5832     7  833.142857            Root MSE      =  25.198

------------------------------------------------------------------------------
          k1 |      Coef.   Std. Err.      t    P>|t|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
          ll |    12.15673   12.23135     0.99   0.394     -26.7689    51.08235
          pl |   -1.403077   10.09382    -0.14   0.898    -33.52612    30.71996
         omc |           0  (omitted)
         mdd |    16.18921   9.572332     1.69   0.189    -14.27422    46.65264
          sg |    154.3355   191.1532     0.81   0.479    -453.9992    762.6702
       _cons |   -789.2316   588.8797    -1.34   0.273     -2663.31    1084.846
------------------------------------------------------------------------------

. regress k2 ll pl omc mdd sg
note: omc omitted because of collinearity

      Source |       SS       df       MS              Number of obs =       8
-------------+------------------------------           F(  4,     3) =    1.55
       Model |  .309645012     4  .077411253            Prob > F      =  0.3752
    Residual |   .15019548     3   .05006516            R-squared     =  0.6734
-------------+------------------------------           Adj R-squared =  0.2379
       Total |  .459840492     7  .065691499            Root MSE      =  .22375

------------------------------------------------------------------------------
          k2 |      Coef.   Std. Err.      t    P>|t|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
          ll |    .1079472   .1086099     0.99   0.394    -.2376979    .4535923
          pl |   -.0124588   .0896294    -0.14   0.898    -.2976995    .2727819
         omc |           0  (omitted)
         mdd |    .1437542   .0849988     1.69   0.189    -.1267498    .4142582
          sg |    1.370442   1.697369     0.81   0.479    -4.031345    6.772229
       _cons |    -7.02139   5.229034    -1.34   0.272    -23.66251    9.619728
------------------------------------------------------------------------------

. regress k3 ll pl omc mdd sg
note: omc omitted because of collinearity

      Source |       SS       df       MS              Number of obs =       8
-------------+------------------------------           F(  4,     3) =    1.55
       Model |  .030788635     4  .007697159            Prob > F      =  0.3752
    Residual |  .014934243     3  .004978081            R-squared     =  0.6734
-------------+------------------------------           Adj R-squared =  0.2379
       Total |  .045722878     7   .00653184            Root MSE      =  .07056

------------------------------------------------------------------------------
          k3 |      Coef.   Std. Err.      t    P>|t|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
          ll |   -.0340388   .0342478    -0.99   0.394    -.1430306    .0749529
          pl |    .0039286   .0282627     0.14   0.898    -.0860159    .0938731
         omc |           0  (omitted)
         mdd |   -.0453298   .0268025    -1.69   0.189    -.1306274    .0399678
          sg |   -.4321393   .5352289    -0.81   0.479    -2.135476    1.271198
       _cons |    2.486049   1.648863     1.51   0.229     -2.76137    7.733467
------------------------------------------------------------------------------
```

Therefore,

K_1 = -789.23 + 12.16(LL) – 1.40(PL) + 16.19(mdd) + 154.34(Sg)

K_2 = -7.02 + 0.12(LL) - 0.01(PL) + 0.1(mdd) + 1.37(Sg)

K3 = 2.49 – 0.03(LL) + 0.01(PL) - 0.04(mdd) - 0.43(Sg)

Appendix I. Secondary Data

Secondary Data (from engineering report of BEST consulting Engineers and AACRA)

Table I.1 Road class for Shiro Meda –Kidane Mihiret Hamere Nooh project

From station	To station	Road class	Road width
0 + 000	0 + 750	*CS - 2	20
0 + 750	2 + 081.7	*SAS - 2	20

*CS- Collector Street. *SAS- Sub-arterial Street

Table I.2 Road class Ambessa garage – unity university – Mebrat Haile

From station	To station	Road class	Road width
0+ 000	1 + 682.0	*CS - 2	20

*CS- Collector Street. *SAS- Sub-arterial Street

Table I.3 Design standards for Shiro Meda - Kidane Mihiret Hamere Nooh project and Ambessa garage - unity university - Mebrat Haile.

Horizontal alignment		Vertical alignment		Cross section element	
Design Item	Design Value	Design Item	Design Value	Design Item	Design Value
Road type	Collector street	Road Type	Collector Street	Type of surfacing	Paved
Design speed	40 km/hr	Design speed	40km/hr	Carriageway cross fall	2.5%
Minimum horizontal radius	85m	Max. Grade	5% - flat 12% - hilly	Walkway cross fall	3.00%
Normal Crown	2% - 4%	Minimum grade	0.50%	Curb requirement	Yes
Max. Supper Elev.	4%	K value	3-Crest, 5 - Sag	Road side ditches	As per the project typical section
		Minimum length of curve	20m		

Table I.4 Thickness for Sh/Meda - K/Mihiret H/Nooh project and A/garage -Unity Un. -M/Hail

Layer	Thickness(cm)
Asphalt	15
Base	50
Sub - base	50

Table I.5 Terrain Type for Ambessa garage - unity university - Mebrat Haile

Station		Length	Terrain Type
From	To		
0 + 000	1 + 540	1540	flat

Table I.6 Terrain Type for Shiro Meda - Kidane Mihiret Hamere Nooh project

Station		Length	Terrain Type
From	To		
0 + 000	0 + 710	710	Rolling
0 + 710	0 + 840	130	Mountainous
0 + 840	1 + 440	600	Rolling
1 + 440	1 + 660	220	Mountainous
1 + 660	1 + 800	140	Escarpment
1 + 800	2 + 081.70	281.7	Mountainous