ADDIS ABABA INSTITUTE OF TECHNOLOGY

(AAiT)

CIVIL AND ENVIRONMENTAL ENGINEERING

SCHOOL OF GRADUATE STUDIES

PERFORMANCE OF ROADS CONSTRUCTED ON BLACK COTTON SUBGRADE SOIL WITH RESPECT TO REPLACEMENT MATERIALS TYPE AND REPLACEMENT DEPTH

(A CASE STUDY ON MODJO - EDJERE ROAD)

A thesis submitted to

The school of Graduate Studies of Addis Ababa Institute of Technology in partial fulfillment of the Requirement for the Degree of Master of Science in Road and Transport Engineering

Advisor: Efrem Gebregziabher (Eng.)

By: Melik Yunus

March 2015

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LIST OF ABBREVIATIONS & DEFINITIONS

AASHTO	American association of state highway and transportation officials method
ASTM	American society for testing materials
CBR	California bearing ratio
CEC	Cation Exchange Capacity
E	Modulus of elasticity
F	percent passing 0.075mm (no. 200)
G	gravitational acceleration
GI	Group index
GW	well graded gravels
H	Active depth of swelling
Ip	plasticity index
LL	liquid limit
MDD	maximum dry density
Mr	Elastic or resilient modulus
OMC	optimum moisture content
PE	potential expansivity',
Pi	Swelling pressure
Ps	swelling pressure,

PVC Potential Volume Change test

R-value Resistance value

SC plastic fine sands

SP poorly graded sand

USCS unified soil classification system

Vdry Dry volume

Vwet Wet volume

WL Liquid limit

WN Natural Moisture Content

XRD x-ray diffraction

ρd dry density in kg/m3

ρ Bulk density of the soil,

ABSTRACT

The performance of roads depends mainly on the strength of the underlying subgrade soil. Black cotton soil gives rise to the failure of the road constructed on it. These soils have cyclic volume change due to fluctuation of moisture content. The degree of volume change or expansiveness depends on the mineralogical composition of the soil. Black cotton soil with montmorillonite is a dominating mineral and experiences high volume changes cause considerable failure to the structures constructed on it.

The current study was conducted on modjo –edjere road project where subgrade soil is black clay soil. The road failed with in liability period showing longitudinal cracks on carriage way and shoulder of the road. This thesis was conducted to evaluate the adequacy of the replacement depth and replacement materials type made for the road. Accordingly, both disturbed and undisturbed soil samples were brought from the site for laboratory tests both for subgrade soil and replacement materials.

Hence, swelling pressure and bulk density of the soil were determined from undisturbed subgrade soil samples. Cracked sections of the road stretch were selected and excavation was made to know the origin of the cracks. Hence, cracks were originated from subgrade soil and depths of cracks were measured for the selected sections of the road from natural ground level. Replacement depth was determined from measured swelling pressure and compared to the crack depth actually measured on the field. Accordingly, replacement depth of 60cm which was made for the road under study is insufficient to counter balance upward swelling pressure from native subgrade soil. Moreover, It was also seen that overburden pressure of the pavement structure is less than measured swelling pressures.

Key Words: Road performance, Black Cotton Soil, Swelling Pressure, Replacement Depth, Replacement Materials Types

1. Introduction

1.1. Back Ground

Black cotton soils are widely distributed in Ethiopia, especially in highlands Known as vertisols. They are fertile and used intensively for agriculture. It is estimated that 7.6 million hectares of vertisols area are located in the highlands with a height of greater than 1500m above sea level. The general slope range of the landscape in which vertices occur is 0 – 8 %. They are more frequent in 0 – 2% slope range and are usually found in landscapes of restricted drainage such as seasonally inundated depressional basins, alluvial and colluvial plains, undulated plateaus, valleys and undulating side slopes (16).

The current study set to focus on modjo – edjere road project which is located in the central part of Ethiopia, in oromia region. The project road starts off from Addis – Adama-Djibouti road at modjo and extends about 71 km east of Addis Ababa. The road starts at rotary junction where the road to Awasa joins the Adama - Djibouti road. Subgrade soil of the road under study is black cotton soil which can change in volume due to moisture fluctuation. This change in volume leads to premature failure of the road. Construction of Modjo – Edjere road project was made by replacing black cotton subgrade soil by materials that have better engineering properties and yet the road fails even before liability periods resulting in riding discomfort and cracks appeared along the edge of the shoulder and along the carriage way of the road. The behavior of expansive soil varies from place to place based on parent materials, climate and topography. Replacement materials type and replacement depth might also varies from place to place or even from chainage to chainage on the same stretch of the road.

1.2. Problem of black cotton soil

Light weight structures, including pavement laid on black cotton subgrade soil cannot withstand upward pressures which come from the underlying subgrade soil characterized by high swelling pressure and volume changes. Roads fail during construction or immediately after construction before their design life or even before its liability periods after construction in the case of Modjo - Edjere road due to volume change of the black clay subgrade soil. The depth of these black clay soils might be varies from place to place depending on environmental conditions and geographic location of the area. Road of the Modjo - Edjere, on which this thesis was conducted, was underlain by black cotton soil which results in riding discomfort of the road, cracks on the surface of the carriage way, on the shoulder, and heave of the carriage way. These problems lead to investigate the cause of the problem for premature failure of the road in terms of replacement material type and replacement depth. The depth of replacements varies from place to place depending on depth at which volume change of the soil might be minimized. Therefore, replacement materials type and depth of replacement should be considered to overcome failure of the road and hence the aim of this thesis is to evaluate the adequacy of depth of replacement made for the road under study and materials type used to replace black cotton soil.

1.3. Objectives of the Study

The study includes tests for subgrade soils, evaluation of replacement materials, and depth of replacement materials. Replacement depth and replacement material type plays a great role in performance of roads constructed on black cotton soil. Accordingly, the objectives of the study are given as follows:

1.3.1. General objective

The main objective of this thesis is to evaluate adequacy of the replacement depth and CBR value of replacement materials according to ERA specification for road constructed on black cotton subgrade soil for the case study of Modjo – Edjere road project.

1.3.2. Specific objectives

The specific objectives of the present research work include the following:

- ❖ characterize the subgrade soil of a trial section of the road under study
- ❖ evaluate swelling pressure and swelling potential of the subgrade soil
- ❖ evaluate replacement materials type and depth of replacement

1.4. Methodology of the Study

In order to achieve the above objectives, the following methods were used: A visual site visit was made to know the performance of the road, and road sections were identified depending on the degree of their damages and photographs were taken to show the existing pavement condition. Soil sampling was made for both replacement materials and subgrade soil taken from test pits both disturbed and undisturbed soil samples and transported to Core Consulting Engineers laboratory and Saba Engineering PLC materials laboratory and subjected to soil tests such as; Atterberg limits and grain size analysis, Modified Proctor Density (MDD), Optimum Moisture Content (OMC), California Bearing Ratio (3 point CBR), bulk density, free swell and swelling pressure. Moreover, Petrography analysis of the rock sample was made by preparing a thin section.

A literature review was undertaken to provide understanding regarding black clay soil and the problems associated with such subgrade soil. Literature sources include books, reports, journals and online materials available on the internet. Experience of other countries with similar physical characteristics to Ethiopia has been referred for the study for the replacement materials type and depth of replacement.

Depth of replacement was determined from predicted swelling pressure and from swelling pressure obtained in the laboratory. The existing replacement materials type and depth of replacement was compared to the result obtained from measured swelling pressure and predicted swelling pressure. This data together with data obtained from literatures have been compiled and based on the result obtained conclusions and recommendations have been made.

1.5. Outcome of the Study

The research was conducted to evaluate performance of road constructed on black cotton soil with respect to replacement materials type and depth of replacement. Adequate material type for replacement of black cotton soil and sufficient depth of replacement will minimize damages caused due to black cotton subgrade soil and its susceptibility to moisture which could result in distress in road and causes to premature failures. The research will provide indicative information on depth of replacement and the replacement materials type of Modjo – Edgere road project. It also gives opinion of replacement depth and type of replacement materials.

1.6. Scope

The scope of the research is to evaluate performance of the road constructed on black cotton soil with respect to replacement materials type and depth of replacement on a case study basis for Modjo – Edgere road project in which samples taken from km 9+020 to 28+400 based on the degree of damages. The road section from Modjo – Edgere was identified and classified into three sections based on its degree of damages as good, bad, and worse.

1.7. Limitations

Characteristics of black clay soil vary from one location to another and hence, the behaviour of soils in the laboratory is likely to fall short of a close relation to the behaviour in their natural original state. It is from this unquestionable fact that measurement errors due to insufficient control of testing procedures and equipments were inescapable.

Another limitation was X-ray diffraction machine. It is known that degree of expansiveness of black clay soil depends on its mineralogical composition. Depth of replacement for subgrade soil would also be influenced by mineral type that the soil is composed of. Accordingly, it is very important to undertake mineral composition tests using x-ray diffraction machine. XRD machine is found at Ethiopian Geological Survey Laboratory but it stopped working and not maintained until the completion of this thesis. Due to this reason, mineral composition of the subgrade soil under study was not conducted.

In Ethiopia, there is no previous study which was made on the replacement materials type and depth of replacement for road constructed on black cotton subgrade soil. Hence further study would require based on topography, environment, and clay mineral compositions of the soil. Therefore, it is strongly recommended that the results and the findings of the present study must be considered as a complete only for road under study. However, further studies and additional tests are required before implementing these results or findings to other road project, hence shall be considered indicative only.

1.8. Organization of the Thesis

Chapter 1 Introduces the general statement of the problem, objectives, methodologies, scope and limitations of this study is also included in this chapter.

Chapter 2 Focus on the performance of road constructed on black cotton soil, black cotton soil swelling mechanisms and its effect on road performance, and construction practice of black cotton soils.

Chapter 3 Briefly discusses about locations, topography and climatic condition of the study area

Chapter 4 Discusses about pavement condition survey of the study area.

Chapter 5 Presents laboratory tests for subgrade soil and pavement materials

Chapter 6 Analysis and interpretation of laboratory tests were discussed.

Chapter 7 Main conclusion drawn from the study and recommendations provided with suggestion for future study were presented

Chapter 8 References and appendices were presented

2. LITERATURE REVIEW

2.1. Pavement Performance

Performance is a general term for how pavements change their condition or serve their intended function with accumulating use. Performance is defined by the distress, loss of serviceability index and skid resistance, loss of overall condition, and by the damage that is done by the expected traffic. The deterioration accumulates with the passage of time and results in failure of the pavement structure. There are two types of pavement distress or failure [15].

The first is a structural failure, in which a collapse of the entire structure or a breakdown of one or more of the pavement components renders the pavement incapable of sustaining the loads imposed on its surface. The second type of failure is a functional failure; it occurs when the pavement, due to its roughness, is unable to carry out its intended function without causing discomfort to drivers or passengers or imposing high stresses on vehicles. The cause of these failure conditions may be due to inadequate maintenance, excessive loads, climatic and environmental conditions, poor drainage leading to poor subgrade conditions, and disintegration of the component materials.

2.2. Swelling and Shrinkage in Road Performance

The mechanism of swelling in expansive soil is complex and is influenced by a number of factors. Expansion is the result of changes in the soil, water system that disturbs the internal force equilibrium. There must be a potential gradient, which can cause water migration and a continuous passage through which water transfer can take place to cause volumetric expansion. Fractures and fissures, shrinkage cracks, capillary rise, vapor transfer, thermal gradients, etc. are some of the sources that cause moisture migration and swelling of expansive soils [10].

In General, the movement of expansive soil occurs in uneven pattern and the resulting expansion is a magnitude that cannot be predicted by the classical elastic plastic theory [31]. However, the swelling behavior can be basically related to the combined effect of interacting factors that can be grouped into:

(a) Local Geology
(b) Engineering properties
(c) Local Environment of deposition.

The Main geological factors include the rock type and age as related to the type and amount of clay minerals, type and amount of cementing materials and the soil particle arrangement. The engineering factors include the moisture content, Atterberg limits, and the dry density. The environmental factors include the confining pressure, type and degree of weathering as related to the amount of clay fraction, initial water content and water. Thickness and location of potentially expansive layers into a profile considerably influence potential movement. The movements are higher around the ground surface and decrease with depth. Less movement will occur if the expansive soil is overlain by non-expansive material or have got shallow depths.

2.3. Evaluation of the swelling potential of Expansive soil

Swelling pressure is defined as the pressure required for preventing volume expansion in soil in contact with water [38]. The swell potential of a soil is a measure of the ability and the degree to which a soil might swell if its environment were to be changed in some definite way. Moisture content alone is not a good indicator of shrink – swell potential. Instead, the moisture content relative to limiting moisture contents such as the plastic limit and shrinkage limit must be known. Water content changes below the shrinkage limit produce little or no change in volume. The availability of water to an expansive soil profile is influenced by many environmental factors and man-made factors. Generally the upper few meters of the soil profiles are subject to the widest ranges of potential moisture variation and also overburden stress is low and the soil is not restrained against movement at shallow depth. The swell potential depends on the following factors which influence the volume change:

- Mineral type and Amount
- Density
- Surcharge loads
- Soil structure, time and water content

The differential free swell may also be expressed by the term ’free swell index’. The ‘potential expansivity’ PE, or the “degree of expansion” and consequent damage to structures with light loading are qualitatively judged from the Atterberg limit and free swell tests.

2.3.1. Indirect Measurement of Potential Swell

a. Classification of potential swell based on plasticity index and liquid limit

The plasticity Index (IP) and Liquid limit (WI) are useful indices for determining the swelling characteristics of most clays, since the liquid limits and the swelling of clays both depends on the amount of water a clay tries to absorb [8,38]. A soil sample with liquid limit exceeding 70% and plasticity index greater than 35% is judged to have a very high potential swell.

Table 2.1 Potential swell based on plasticity (source: Chen 1975)

Classification of potential swell	Liquid limit (LL), %	Plasticity index (PI), %	Shrinkage limit (SL), %
Low	20-35	< 18	>15
Medium	35 – 50	15 – 28	10 – 15
High	50 – 70	25 – 41	7 – 12
Very high	>70	> 35	< 11

b. Linear shrinkage

The swelling potential of a soil can be estimated from linear shrinkage in combination with shrinkage limits [31]. They propose values given below to classify the given soil swell potential.

Table 2.2 Soils swell potential based on linear shrinkage (Source: Nelson and Miller)

Linear shrinkage	Shrinkage limit (%)	Probable swell (%)	Degree of expansion
<5	>12	<0.5	Noncritical
5-8	10-12	0.5-1.5	Marginal
>8	<10	>1.5	Critical

c. Prediction of swelling potential

According to seed at al cited in V.N.S. Murthy [2008], the swelling potential is given as a function of the plasticity index by the formula:

$$SP = 60k\,(PI)^{2.44} \qquad 2.1$$

where PI is the plasticity index in percent,

SP is swelling potential in percent and k is the constant, equal to $3.6*10^{-5}$

Table 2.3 Category for Swelling Potential Classification (Source: Murthy, 2008)

Swell pressure	Expansiveness
Sp <1.5	Low
1.5≤sp<5	Medium
5≤sp<15	High
Sp>25	Very high

2.3.1.1. Swelling pressure prediction model

Many relationships have been established from which swelling pressure can be estimated based on physical property of soils such as consistency limits, clay content, initial moisture content and initial dry density which do not require sophisticated laboratory procedure and apparatus. These empirical equations are easy to apply and give satisfactory results when applied to the particular soils for which they were developed [3]. Though evaluation of swelling pressure is not a part of this thesis, it was used models developed by different researchers to determine the replacement depth from predicted swelling pressure to compare with the actual swelling pressure obtained from the laboratory. Accordingly, empirical equation developed by Daniel Teklu and Abraham Mengistie on undisturbed soil samples of Addis Ababa and Assella town respectively were used.

a) Daniel Teklu in his MSC thesis entitled to examine swelling pressure of expansive soil in Addis Ababa developed a model for predicting swelling pressure in July 2003. He considered to use a combinations of parameters such as liquid limit, plasticity index, natural water content as follows:

$$\text{Log Ps} = -9.384 + 0.02748W + 0.006307PI + 0.008359\gamma d \qquad 2.2$$

where Ps = swelling pressure (kPa),

W = natural moisture content (%)

PI = plasticity index (%)

ϒd = initial dry density (gm/cc)

b) Abraham Mengistie in his MSC thesis entitled correlation between index properties, and swelling characteristics of Assella expansive soil in July 2014, developed a swelling pressure as follows:

$$\text{Log Ps} = 0.756\gamma d - 0.007SL - 0.003PI + 1.387 \qquad 2.3$$

where Ps = swelling pressure (Kpa)

PI = plasticity index (%)

ϒd = initial dry density (gm/cc)

SL = shrinkage limits (%)

c) Bantayehu Uba with thesis entitled unsaturated shear strength and swelling characteristics of expansive soils of Arba Minch in July 2011 was developed a model to estimate swelling pressure of expansive soil of Arba Minch given as follows:

$$\log \text{Ps} - -7.908 + 0.058wi - 0.019LL + 0.042PI + 6.394\gamma d \qquad 2.4$$

where wi = natural moisture content (%)

LL = liquid limit (%)

PI = plasticity index (%)

ϒd = dry density (g/cm3)

According to Komornik and David (cited in Bowles J.E. 1996) developed a correlation between index properties to estimate swelling pressure as follows:

$$\log Ps = 2.132 + 0.0208Wl + 0.000665\gamma d - 0.0269w \qquad 2.5$$

Where Ps is swelling pressure (kg/m^2),

Wl is the liquid limit (%),

ϒd is dry density (kg/m3),

w is initial moisture content (%).

2.3.2. Direct Measurement of Swelling Pressure

Loading expansive soil with pressure greater than the swelling pressure is a method by which swelling can be prevented. However, pavement loads are generally insufficient to prevent expansion and this method is usually applied in the case of large building or structures imposing high loads. Therefore, highways should be based partly on the requirement that the pavement weight should be enough to prevent expansion of the subgrade. Common Laboratory test methods to determine the swell of soil are free swell test and swell in oedometer test.

a. Free swell Test

The free swell consists of placing a known volume of dry soil passing the No. 40 sieve into a graduated cylinder filled with water and measuring the swelled volume after it has completely settled. The free swell of the soil can be determined as the ratio of volume change to the initial volume expressed as a percentage.

$$Free\ swell = \frac{V_{final} - V_{initial}}{V_{inital}} * 100 \qquad 2.6$$

Soils having free swell less than 100% may exhibit moderate expansion in the field when wetted under light loading. Soils with free swell values below 50% are not considered to exhibit appreciable volume change if there is no extreme climatic condition that may result in expansion characteristics of the soil. Values of 100% or more are associated with clay which could swell considerably [31].

b. Consolidometer Testing

The most reliable method for the estimation of swelling pressure would involve utilizing odometer test on undisturbed samples. Zero swell tests were used for the current study. This test is conducted by applying a small incremental load of 6.9KPa on a compacted specimen. Water is then added to the sample. As expansion starts, pressure is added in small increment to prevent swelling. This is continued until the specimen cease to swell. The total load required to prevent swelling divided by the area of the sample defines the swelling pressure **[9].**

2.4. Construction Practices on black cotton soil

2.4.1. General

The construction of roadways often requires traversing areas that contain materials that are unsuitable for the subgrade soils that lie beneath the pavement. These materials can be expansive, highly plastic, soft, wet, and/or weak. The exact nature of potential construction problems depends on whether or not the natural grade is to be excavated or if an embankment is to be constructed. The supporting soils may be susceptible to Excessive consolidation, shrinking and/or swelling with changes in moisture conditions or heave - induced volume changes due to the excavation of overlying soils, i.e. a cut section [34].

When black cotton subgrade soil encounters along the stretch of the road, undercut and replacing with better material is a viable alternative. At present, when poor subgrade soils are encountered, four approaches are taken, individually or in combination. These approaches are:

I). Remove and replace of black cotton soil
II). Apply chemical stabilization
III). Employ Reinforcement Geosynthetics
iv). Install subsurface drainage using vertical or horizontal drainage elements.

For this thesis emphasis will be given to replacement practices which were done for the selected case study project from Modjo – Edjere road section. Black cotton soils are those that exhibit particularly large volumetric changes, both shrinkage and swell, due to variations in their moisture content. They exhibit poor bearing capacity and also some stability problems. When the subgrade is a particular black cotton soil, it may be necessary to replace the expansive material with granular materials to a depth affected by seasonal moisture changes depending on the degree of expensiveness of the soil. Problems associated with construction over black cotton soils are usually the seasonal Changes in these soils rather than the low bearing strength, as these soils are often relatively strong at equilibrium moisture content [18]. Distress occur as seasonal wetting Causes soils at the edge of the pavement to wet and dry out at rates differing from those under the bituminous surfacing. This mechanism causes differential movements over the roadway cross section and associated crack development, beginning at the shoulder and proceeding towards the carriageway.

During construction, the roadbed of expansive soil should be kept moist and covering with earthworks prior to any drying. Attempts to process and compact the soil beyond the normal density requirement is not required [12]. Several empirical relationships have been developed to identify expansive soils, although a standard classification procedure does not exist. Generally, soils with a plasticity index (PI) of less than 15% and liquid limit below 55% do not exhibit expansive behavior [16]. Clays that have developed from volcanic ash may also have a fragile structure prone to collapse under embankment loads.

2.4.2. Removal and Replacement of the Black Cotton Soil

Removal of natural black cotton subgrade soil and replace with a non-expansive material is the most common method of eliminating swell problems. In some cases this approach may be economical if the expansive stratum is thin and replacement materials are available. The selection of the particular nonexpansive backfill materials is critical. Replacement soils are impervious or granular materials (oversized cobbles) as pervious soils may create conditions conducive to the collection of water. Removing subgrade and replacing with sand – gravel material could perform well if gravel is not highly compacted so that some compression of the gravel would occur, thereby relieving part of the expansive force [33]. Several problems may be encountered when using the removal and replacement method. If subgrade material lacks strength because of a high water table, the properties of the material used as backfill may also be adversely affected by the water conditions. Unless some type of separation membrane is used between the subgrade and granular layer, material from the soft subgrade may migrate into the granular material, significantly reducing the effectiveness of the granular layer over time. Since pavement performance depends on subgrade uniformity even if an ideal subgrade is difficult to achieve due to natural variability of the soil, the influence of water, temperature, construction activities, proper soil properties, proper grading practices, and quality control testing are needed to be conducted to achieve quality sub grade replacement materials.

2.4.2.1. Types of Replacement Materials

The replacement materials type should be nonexpansive soil. All granular soils ranging from GW to SC in the united soil classification system may fulfill the nonexpansive soil requirement. However, for clean, granular soils such as GW and SP surface water can travel freely through the soil and cause wetting of the lower swelling soils. At the other extreme, SC materials with a high percentage of plastic clay will sometimes be exhibits swelling potential [10]. The table below shows suitable fills materials by correlating liquid limit and % pass no. 200 sieve.

Table: 2.4 Correlation of liquid limit and % pass no. 200 sieve (Source: Chen, 1975)

Liquid limit (%)	Percent Minus No 200 sieve
Greater than 50	15 – 30
30 – 50	10 – 40
Less than 30	5 – 50

It might be difficult to locate materials which fulfill the above requirement. If the above requirement fails to meet selected fill will be satisfactory, provided that the material is inexpensive and it is preferable to conduct a swell test beside plasticity tests.

One key factor that influences the replacement material type is the hauling distance. Local material usage is appreciated to minimize cost of the project. To minimize the cost of the project with imported granular fill, mixing of locally available expansive soil with granular materials has got acceptance theoretically, and it should be tried on actual construction on trial section. If the subgrade or open excavation becomes wetted excessively before placement of the fill materials, the trapped water will cause heaving. In this case, the detrimental amount of heave will occur regardless of the thickness of the selected fill materials, but if pre-wetting is followed by covering with gap graded materials (over- sized materials), will stabilize the heaving or the heaving can be controlled with weight of the oversized cobbles, boulders and the void between them [29].

2.4.3. Depth of Replacement

The method used to reduce or eliminate ground movements due to expansive soil is to replace or partially replace them with non expansive soils to provide a stable foundation. The expansive strata may be removed wholly or partially. Depth of expansive soil may extend to a depth too great to economically allow complete removal and replacement. Then the depth of excavation to be removed and fill will be necessary to prevent excessive heave. Most of the heave occurs near ground surface where there is the largest difference between the swelling pressure and overburden pressure. The depth at which the corrected swelling pressure equals the total overburden pressures is known as the active depth of swelling with the assumption that the total final pore-water pressure throughout the profile go to zero and the soil is assumed to be homogeneous [4]. Active depth of swelling can be estimated from the following equation.

$$H = \frac{p_i}{\rho g} \qquad 2.7$$

Where: H= active depth of swelling

ρ = bulk density of the soil, which is assumed to remain constant with depth.

g = gravitational acceleration, Pi = Swelling pressure

Replacement depth of expansive soil has got not defined depth, but depends on the local bearing capacity of expansive soil and the quantity of the uplift pressure. This can be obtained by conducting swell tests during dry and wet season. The depth at which both wet and dry seasons swell test with corresponding depth will be plotted and the depth at which swell becomes constant is the active depth that enhances roughness and failure to the road. To define this depth, tests should be conducted for each soil type available locally considering environmental condition.

The swelling potential of the soil beneath the fill is very important as the density and moisture condition change at various locations. Depending on the degree of swelling, the surcharge load could be placed to counteract swelling pressure [10].

Table: 2.5 Typical over excavation depths recommended by Capa (Source: Colorado asphalt pavement association (Capa))

Subgrade plasticity index	Depth of over-excavation below normal subgrade elevation
10 – 20	0.7 meters
20-30	1.0 meters
30-40	1.3 meters
40-50	1.7 meters
More than 50	2.0 meters

The depth to which nonexpansive backfill should be placed will be governed by the weight necessary to restrain the expected uplift pressure and the ability of the backfill to mitigate differential displacements. ERA site investigation manual recommend 0.6m depth of replacement. This depth may not be sufficient for areas where the ground water table is shallow and black cotton soil with montmorillonite is a dominating mineral.

The experience of road from Megenegna to Hayat, which was underlain by black cotton soil with low CBR and high swell indices shows that expansive soil was removed to a depth of 1m and filled the section with improved subgrade materials of minimum CBR of 8% and swell index less than 2% or rock fill materials. The road serves high traffic flow and the road is performing well [30]. In order to overcome failures due to the black cotton soil, controlling of moisture content in the underlying black clay layer is a mitigating measure. If the clay exhibits a moderate to low expansion potential, the reduced volume of expansive clay in the upper top moisture variation layer may be sufficient to prevent large movements at the surface. If there is a high potential for volume change in the underlying soil, the reduced volume of expansive material may not adequately prevent surface heave or shrinkage, if water can infiltrate into the clay layer due to surface runoff, waterline leakage, or other excessive movement will most likely results.

2.5. Pavement Design Methods and Road Performance

Pavement design is used to provide a structural and economical combination of materials such that it serves the intended traffic volume in a given climate over the existing soil conditions for a specified time interval. The structural design of a pavement system must be done with a clear understanding of the factors that affect the life and serviceability of the pavement. According to pavement design criteria manuals of city of Colorado, there are multiple factors in the design and construction of a pavement system. Factors that have a significant effect on the pavement life and serviceability includes: subgrade, traffic loads, pavement material, future maintenance and special considerations such as swelling or collapse prone soils, slope instabilities and frost susceptible soils [36]. Accordingly, the specification produced should incorporate climate, environmental condition, topography, geologic formation of the soil [32]. These parameters will help to define the native subgrade soil property with its mineral constituents and makes to decide depth of replacement. Moreover, it helps to decide replacement materials type to be used for the undercut depth.

Oversea Road Note 31 design procedures are the most widely used methods in tropical and sub-tropical developing countries like Ethiopia. The guide gives a procedure to estimate the moisture content at which the bearing capacity of the subgrade is determined using the CBR test, to be carried out on the soil in the wet condition likely to occur, and sub grouped into six categories for design purposes [1]. Most of the failures noted on the pavements resulted from improper design or construction method adopted for the particular soil condition. This shows that the applicability of any design or construction method adopted for pavements on expansive sub grade is a function of the environmental conditions that prevailed on the site.

3. The Study Area

3.1. Introduction

The Ethiopian roads authority as part of road sector development program plans to upgrade and construct the Modjo- edjere- Arerti and Sambo- Gobensa and Metahbila-Metehara road project. Accordingly, the preparation of detailed engineering design and tender document preparation and also the construction supervision work were signed a contract between the Ethiopian road authority the client and the consulting firms associate engineering consultant (AEC) in joint venture with CORE Consulting engineers. The consultancy service agreement was signed between ERA and AEC on July 11, 2005 and the commencement date was taking place on July 26, 2005.

3.2. Project Location

The Modjo – Edjere road begins in a Modjo town in the Eastern Shewa Zone of Oromia. The road branches off from Addis – Nazareth main highway at Modjo town and extends to North East direction for about 30 km until it reaches the town of Edjere. .

Table 3.1 the Geographical position of the road stretch from Modjo to Edjere (Source: final engineering report of the Modjo-Edjere- Arerti road)

Towns/Village	Latitude (N)	Longitude (E)
Modjo	8° 35' N	39° 07' E
Deke Bora	8° 46' N	39° 19' E
GPS 06	8° 44' N	39° 13' E
Edjere	8° 46' N	39° 15' E

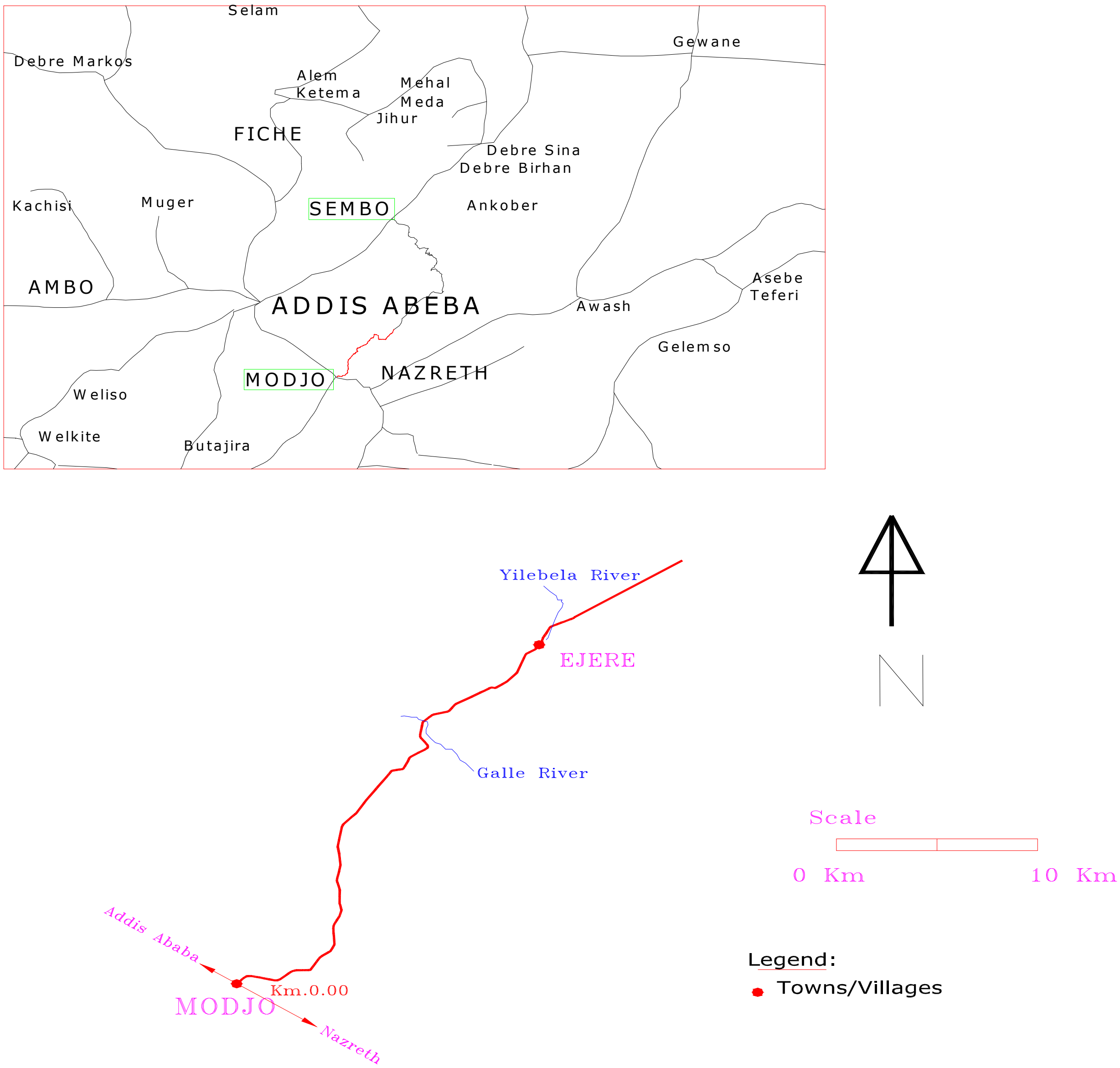

Figure 3.1 Project Road Alignment of the Study Area

3.3. Topography

One of the main determinants of the geometric design parameters of a rural road is the terrain upon which the route traverses. The terrain is classified by the general slope of the country across the road alignment. The portion of the road from Modjo (0+000) to Edjere (28+700) on which this paper focuses lies dominantly on the flat terrain and with very few rolling terrain.

Table 3.2: Terrain classification of Modjo- Edjere road project

(Source: Feasibility study of the project)

Item No.	Station		Terrain Type/Town
	From	To	
1	0+000	1+300	Modjo town
2	1+300	2+600	Flat
3	2+600	3+800	Rolling
4	3+800	4+800	Flat
5	4+800	5+800	Rolling
6	5+800	7+900	Flat
7	7+900	8+500	Rolling
8	8+500	17+300	Flat
9	17+300	18+600	Dekabora town
10	18+600	20+000	Flat
11	20+000	23+300	Rolling
12	23+300	25+000	Flat
13	25+000	26+500	Rolling
14	26+500	28+700	Flat

The starting point of the project is in Modjo town. The altitude of the Modjo town where the project starts is 1790m. Table below summarizes the elevation of the project under study.

Table 3.3 Elevation of the towns along the route

Town/village	Station (km)	Elevation
Modjo	0	1790
Dakabora	19	2151
Edgere	30	2271

3.4. Climate of Project Area

3.4.1. Temperature

Modjo – Edjere road traverses through warm temperate climatic zones. The observed maximum and minimum monthly temperature data obtained from the meteorological department at Modjo is given below in Table 3.4

Table 3.4 Maximum and minimum mean temperature at Modjo

Temperature	Month											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Maximum Mean	28.6	29.6	30.2	30.5	31.2	28.7	26.3	25.8	27.6	28.4	28.4	27.8
Minimum Mean	9.50	10.5	12.9	13.5	13.6	13.4	13.1	13.2	12.6	11.1	10.1	8.3

The highest air temperature observed is in May and the maximum temperature varies in the range of 25 to 31 ^{o}c. The mean maximum temperature varies from 25.8 to 31.2^{0}c.

3.4.2. Rainfall

The mean annual rainfall ranges between 500mm and 2000mm. The mean annual rainfall at Modjo is 875mm while at Metehara it is 512mm according to data obtained from the Ethiopian Meteorological Service Agency.

3.5. Soil of the study area

From the visual inspection of the site during the site visit, the study area was covered with black clay soil. According to the feasibility study of the road under study, the predominant soil type observed for almost the whole alignment is dark clay soil, which is usually considered as unsuitable roadbed material.

4. Pavement Condition Survey

4.1. General

Condition survey was conducted to evaluate the functional aspect of the road. Functional evaluations identify the capability of the pavement structure to provide a comfortable and safe service. Visual inspection requires the rating of the degree and extent of the various distresses. Typical pavement conditions evaluated visually include surface conditions (roughness), potholes, deformations (ruts), cracks, edge-breaks, raveling, bleeding (flushing), and patching. The roughness of a road pavement is the major parameter used to determine of the functional conditions. Before the performance evaluation starts the whole road stretch of the Modjo - edjere was surveyed to know the types and extent of failure occurred that influence the performance of the road. During initial survey, the failed sections of the road were inspected visually for the next selection of the sampling program. At the time of the site visit, it was found necessary to start with a visual inspection of the whole stretch of the road under study, which is Modjo – Edjere road project. During the first site visit, the road sections were covered to identify the failed section. Accordingly, failed sections were identified visually and categorized according to their degree of failures as slightly damaged, moderately damaged, and extremely damaged. Hence, once, failed sections were identified visually, an approximate measurement of the width of cracks and length of cracks were recorded and tabulated in Table 4.1.

The site visit (data collection) was made three times to notice if any additional damages were occurring and also to take samples. Accordingly, the first site visit was made at the beginning of February, 2014 to see overall conditions of the road. The second site visit was made in March , 2014 and samples were taken from replacement materials from the shoulder of the road at different stations considering the severity of the road failure. It is observed that the cracks noticed were increasing in width from 10mm to 27mm and also in length in some areas. There was also new crack that was noticed on the road surface and on the side or edge of the road even though, the cracks were varying from hair cracks to easily visible cracks. The failures vary from the rough surface of the road to surface cracks on the pavement. Moreover, there were edge cracks on the side of the pavement which varies from small cracks to very sever cracks which varies from 10mm to 27mm in width and from 6m to 18m in length.

Table 4.1 Summary of pavement condition for Modjo –Edgere DSD road

Station	Crack Length (m)	Defects observed	Crack width (mm)	Status
9 + 567 – 9 + 650 LHS	12	Surface crack	15	The cracks observed increasing in length and some depression of the surface was seen
10+346 – 10 + 600 LHS	8	Surface cracks	10	The cracks observed increasing in length along the center of the road
12 + 182 LHS	6	Surface cracks	12	New crack observed on the surface of the road
24 + 590 – 24 + 610 RHS	20	Edge cracks	21	Crack was seen on the shoulder of the road filled with an embankment
28 + 600 RHS and LHS	17	Both surface and edge cracks were observed	27	The crack observed on the shoulder was considerably wide and deep.

To know the extent of the crack and its origin, excavation of the cracked section of the road were carried out and it was noticed that the crack extends below the replacement materials, even beyond the rock fill. It was seen on some section of the road that, borrow materials were placed directly after treating the top 200 mm thickness of the sub-grade by ripping, removing all loose and oversize material from the surface and compacting to the minimum density requirement. This leads to the rock fill exposed directly to runoff and rainfall, Moreover, there is moisture variation from the underlying subgrade. It was seen around station 24+560 that borrow materials was exposed and seen under the embankment above natural ground elevation. Rock fill were simply placed on cleared surface of the top soil without excavating the natural soil to 60cm replacement depth. Figure below shows the appearance of fill (borrow) materials at the site.

Figure 4.1 Exposed fill materials where the rock floats on the fine materials below embankment around station 24+560

During the sampling of subgrade soil, it was observed that subgrade soil gets wet about 1m below the surface where samples were taken. Though; the degree of wetting was variable, the degree of wetting was sever at station 24 + 575 ,24 + 787 , 28 + 149, 28 + 400 during sampling.

It was seen that drainage structures provided for the road under study are far away from each other and also there are some places that require drainage structures for the relief of the moisture. Lack of the drainage structures produces slide of the road side soil, which leads to erosion as indicated on the figure 4.2 and 4.3 below. The degree of damage occurring from moisture due to lack of drainage structure should be within the acceptable limit so that drainage structures should be designed and placed within acceptable distances where moisture is suspected.

Figure.4.2 Lack of drainage facility **Figure 4.3 Improper drainage facility**

4.2. Soil Sampling

It was not possible to take samples for subgrade soil and replacement materials under the roadway by cutting the asphalt. The contractor was maintaining the road for its liability period and he was not willing to give permission to cut roadway and take samples under the roadway. Hence, sub grade samples were taken from different places along the roadway nearby sides to the roadway to investigate the engineering properties of the soils for all three sections that were identified during site visits with 10 test pits. In addition, replacement material samples were taken from the shoulder of the roadway to check if the materials engineering properties met specification and submitted to the core consulting laboratory for the tests to be conducted.

4.2.1. Subgrade Soil Sampling

The purpose of sub grade soil sampling is to carry assessment on the nature of subgrade soil characteristics along the study area and to identify problems caused to the road constructed on black cotton soil along the study area and also to identify what causes the problems to the road. Sampled sections and depth of sampling is summarized as given in Table 4.2 below.

Table: 4.2 Sampled Stations and Depth of Sampling of Subgrade soil

S/ No	Station (KM)	Depth (m)	Color of sampled soil
1	9+020	0.95	Dark clay
2	9+375	1.00	Dark clay
3	9+656	1.10	Dark clay
4	10+080	1.30	dark clay
5	10+500	1.15	dark clay
6	24+575	1.10	dark clay
7	24+787	1.20	dark clay
8	28+149	1.00	dark clay
9	28+400	1.20	Dark clay
10	28+602	1.15	dark clay

5. Laboratory Tests

5.1. Introduction

Laboratory tests are carried out in accordance with AASHTO standard testing methods. In order to obtain the intended purpose of the research the following laboratory tests are carried out:

Sieve analysis (Gradation), atterberg limits (liquid limit, plastic limit and plasticity index), optimum moisture content and maximum dry density (modified proctor tests), natural moisture content, 3 Point CBR and swell after 4 days soak, free swell, linear shrinkage limit tests were conducted. Additional samples were collected and submitted to SABA engineering material testing laboratory to conduct swelling pressure tests of native subgrade.

5.2. Laboratory Investigation

5.2.1. Replacement Materials

Different laboratory tests were conducted both for subgrade soils and replacement materials. Test for the replacement materials for road under study were conducted by taking the samples from the edge of the road shoulder. Tests conducted for these materials include: gradation test, proctor density test and 3- point CBR tests. Replacement materials taken from the embankment were contaminated by subbase materials and laboratory test result might be exaggerated. An effort was made to take samples with less contamination even if, it is not possible to get uncontaminated replacement materials from the side of the embankment. Comparisons were made between laboratory test results of replacement materials taken from the side of the embankment and borrow materials test result obtained from the contractor. Summary of the test results is given in the Tables 5.1 and 5.2.

Table 5.1 replacement material test results

S/ No	Station (KM)	Material Description	AASHTO T 27 % Passing Sieve (mm) 2	0.425	0.075	AASHTO T 89 & 90 Atterberg Limit LL	PL	PI	Soil Classification	Proctor Density AASHTO T 180 MDD (g/cc)	OMC %	3 - Point CBR AASHTO T 193 No of Blows	Dry Density (g/cc)	CBR %	Swell %	AASHTO T 92 Shrinkage Limit (%)
												10	1.16	9	1.83	
												30	1.42	23	1.55	
1	9+567	white silty clay	92	89	75	34	22	12	A-7-6-(12)	1.54	17	65	1.54	32	1.38	13
												10	1.54	13	1.59	
												30	1.83	22	1.37	
2	24+575	light brown silty clay	99	90	60	34	22	12	A-7-5(12)	1.97	18	65	1.97	34	1.21	13
												10	1.22	10	1.88	
												30	1.47	19	1.61	
3	24+757	white brown silty clay	91	64	49	36	22	14	A-7-5(4)	1.58	18	65	1.58	31	1.35	13
4	28+400	brown silty clay with gravel	45	23	19	51	39	12	A-2-7-(0)	1.54	12	10	1.43	8	1.43	13

Table 5.2 Summary of laboratory test results for borrow materials obtained from the contractor

NO.	STATION	DATE OF SAMPLING	SOIL DESCRIPTION	AASHTO SOIL CLASSIFI CATION	SIEVE ANALYSIS (% PASSING) Sieve Opening (mm) 2	0.425	0.075	Atterberg limits LL	PL	PI	MDD	OMC	CBR at 95% MDD	Swell (%)	Remark
1	3+100 LHS 1.5KM "A"	25/11/08	Brown Silty Clay	A-4(5)	96.6	85	59.2	38	28	10	1.6	22	7.2	3.5	Borrow
7	3+900 LHS 400M	31/07/08	Dark brown silty clay	A-2-7(0)	52.9	43.7	28.8	42	32	10	1.6	19.4	10	0.5	Borrow
8	4+850 LHS1KM	21/08/08	Light brown silty clay with gravel	A-2-7(0)	60.2	40.6	29.5	40	31	9	1.54	20	7	6	Borrow
15	8+600 LHS1KM" B"	1/10/2008	White Silty Clay With Gravel	A-4(2)	89.4	75	59.4	38	34	4	1.57	23	4.3	1.45	Borrow
16	8+600 LHS1KM" C"	1/10/2009	Light brown silty clay	A-5(3)	96	75.6	46.4	39	26	13	1.66	21.6	4.2	0.21	Borrow
17	9+350 RHS	2/12/2008	Yellowish silty clay	A-7-6(0)	78.9	65.2	38.3	51	24	27	1.66	16.6	3.9	4.8	Borrow
21	11+500 LHS 3.5 KM	5/1/2009	White brown silty clay mat'l	A-7-5 (0)	99	90	60	43	30	13	1.508	19.8	4.2	7.59	Borrow

23	18+240RHS 700M "A"	30/03/09	Dark reddish silty clay with gravel	A-2-6(0)	61	49	32	38	21	17	1.821	14.7	7	2.21	Borrow
24	18+240RHS 700M	3/4/2009	Reddish silty clay with gravel	A-2-7(0)	51	37	30	42	28	14	1.769	15.6	5.5	3.46	Borrow
25	18+240RHS 700M "C"	6/4/2009	Light Reddish silty clay with gravel	A-2-4(0)	51	37	30	32	22	10	1.883	11.5	27	0.78	Borrow
26	18+240RHS 700M "D"	6/5/2009	Light Reddish silty clay with gravel	A-2-6(4)	75.7	65	46.7	31	13	18	1.792	15.8	5.2	1.6	Borrow
27	18+240RHS 700M	6/6/2009	Reddish silty clay	A-7-6(4)	57.4	50.4	39.2	41	20	21	1.814	11.4	6.6	1.88	Borrow
35	21+470 LHS 200M	14/03/09	Weathered rock gravel material	A-2-7(0)	36.3	32.8	28.6	40	20	20	1.468	21.8	4	5.66	Borrow
38	23+000 RHS 700M	1/4/2009	Brown silty clay with gravel	A-2-6(0)	37.1	20.7	8.2	40	23	17	1.767	14	5	0.26	Borrow
43	25+470 LHS50M	7/3/2009	Grayish silty clay with gravel	A-2-7(0)	38.1	21.8	17.5	46	34	12	1.832	11.4	14	3.48	Borrow
47	25+470 RHS "B"	7/3/2009	Weathered rock gravel material	A-2-7(0)	57.6	39.7	25.4	43	29	14	1.562	19.5	4	0.94	Borrow

CBR for replacement materials were conducted at 3 point system for the current road under study and the value considered for design is a CBR value of 95% of MDD. Accordingly, the determination of CBR at 95% maximum dry density is given as follows.

Table 5.3 replacement materials CBR values at 95% MDD (km 9+ 567)

Number of blows	Dry density (gm/cc)	MDD(gm/cc)	CBR (%)	CBR swell %	CBR swell at MDD
10	1.16	1.54	9	1.83	1.38
30	1.42		25	1.55	
65	1.54		32	1.38	

95% of MDD = 0.95*1.54 = 1.463g/cc.

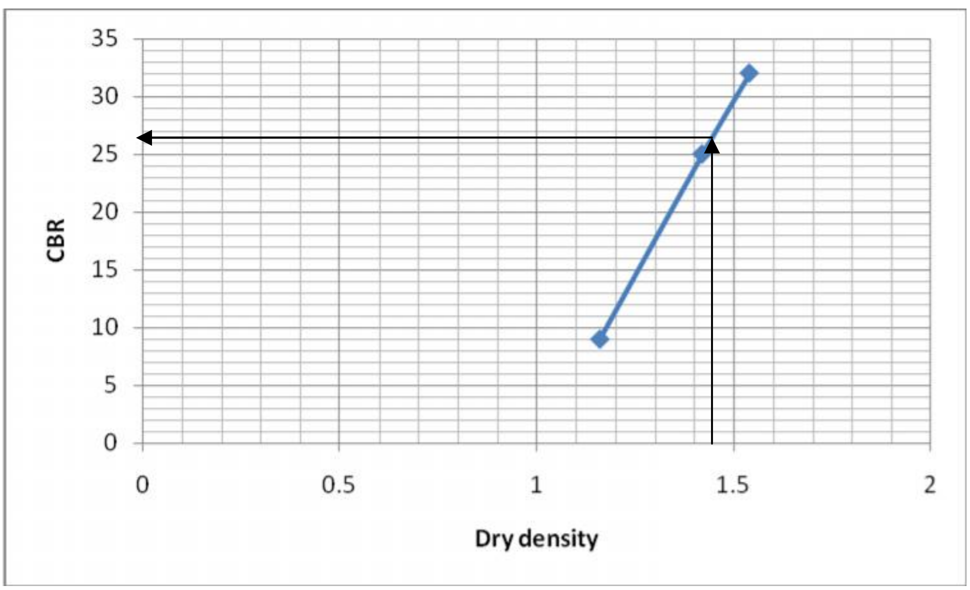

Figure 5.1 replacement materials dry density vs. CBR at 9+567

MDD*95% of replacement materials is 1.463g/cc and hence, CBR at 95% of MDD at station 9+567 from the above graph is 25.8 %.

Table 5.4 replacement materials CBR values at 95% MDD (km 24+575)

Number of blows	Dry density (gm/cc)	MDD(gm/cc)	CBR (%)	CBR swell%	CBR swell at MDD
10	1.54	1.97	13	1.59	1.21
30	1.83		22	1.37	
65	1.97		36	1.21	

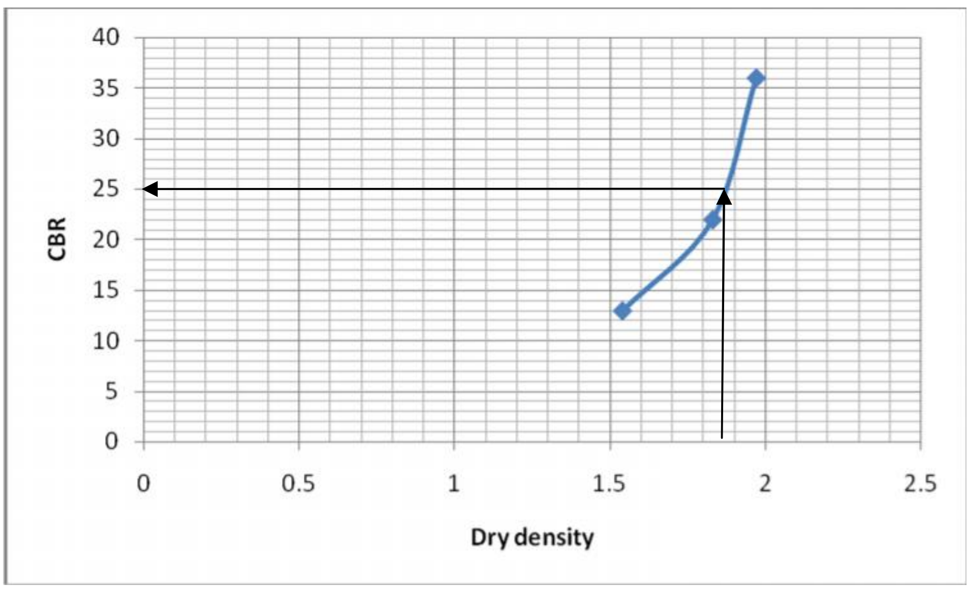

Figure 5.2 replacement material dry density vs CBR at station 24+575

MDD*0.95 = 1.872 for replacement materials at station 24+575, hence CBR at 95% of MDD from the above graph is 25%.

Table 5.5 replacement materials CBR values at 95% MDD (km 28+400)

<table>
<tr><th>Number of blows</th><th>Dry density (gm/cc)</th><th>MDD (gm/cc)</th><th>CBR (%)</th><th>CBR swell%</th><th>CBR swell at MDD</th></tr>
<tr><td>10</td><td>1.43</td><td rowspan="3">1.75</td><td>13</td><td>1.59</td><td rowspan="3">1.21</td></tr>
<tr><td>30</td><td>1.64</td><td>22</td><td>1.37</td></tr>
<tr><td>65</td><td>1.75</td><td>36</td><td>1.21</td></tr>
</table>

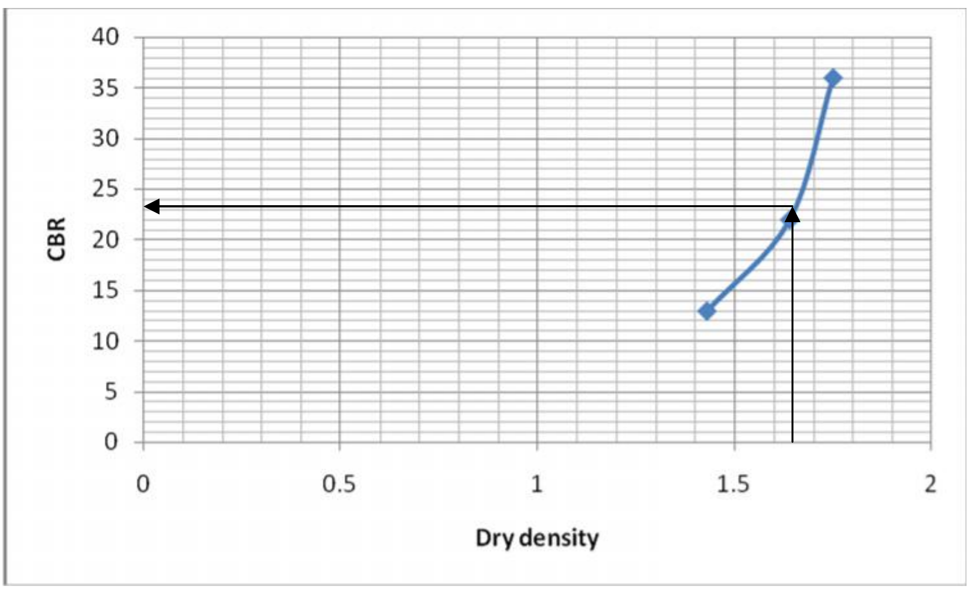

Figure 5.3 replacement material dry density vs CBR at station 28+400

MDD*0.95 = 1.66g/cc for capping materials at station 28+400, hence CBR at 95% of MDD from the above graph is 23%.

5.2.2. Petrography Investigation for rock used with capping materials

Rock samples that were used for capping layer (soil mixed with some rock) on the weak subgrade were collected and investigation was done for Petrographic analysis at the Ethiopian Geological survey central laboratory. The test results for both water absorption and Petro graphic analysis are presented in the appendix 5. The Petrographic analysis results were summarized in the following Table.

Table 5.6 Mineral Composition of Rock Fill Sample

Name of the mineral	Composition (%)	Texture
Sanidine	42	Euhedral, microlitic
Microlite Andesine	30	Microlitic
Hornblende	15	Needle, elongated
Rock fragment	5	
Quartz	4	Fine anhedral
Volcanic glass	2	
Opaque (Fe-oxide)	2	Euhedral - Anhedral
Rock type	Lithic hornblende trachyandesite	

5.3. Laboratory Investigation for subgrade soil

Soils are used as construction materials and also as a foundation for engineering structures. However, the wide range of properties under different conditions affects their performance and use. For this reason, soils have to be properly sampled and subjected to various tests so as to understand their properties towards engineering applications.

For the present study, the sub-grade soil was properly sampled for laboratory testing from each test pit and these soil samples were subjected to various tests to determine their physical properties. The types of tests carried out in the laboratory include: soil classification tests such as; grain size distribution and atterberg limits, moisture-density relationship, California Bearing Ratio (CBR), CBR swell tests and swelling pressure of subgrade soil. The summary of the laboratory test results for various soil properties is tabulated in Appendix 1-4 together with individual test results.

5.3.1. Grain Size Distribution

Sieve analysis was carried out to determine the grain size distribution of sub-grade soil and used in the classification of the soil. Accordingly, the wet sieve analysis was employed to determine the grain size distribution of sub-grade soil samples in accordance with AASHTO T-88Test Method for Particle-size Analysis of Soils. The grain size distribution of the soil samples is presented in Table 5.7 below.

Table: 5.7 Gradation test result of subgrade materials

Sample no.	Station (sample designation)	Depth of sample (Cm)	Visual Description	% passing			
				4.75	2.36	0.425	0.075
1	9+020	80	Black clay	100	95	93	92
2	9+375	100	Black clay	100	98	96	95
3	9+656	110	Black clay	100	99	97	96
4	10+080	100	Black clay	100	98	97	95
5	10+500	90	Black clay	100	96	96	95
6	24+575	110	Black clay	100	97	96	95
7	24+787	120	Dark Gray	100	98	97	95
8	28+149	110	Black clay	100	97	95	93
9	28+400	120	Black clay	100	98	96	95
10	28+602	115	Dark Gray	100	97	95	94

5.3.2. Atterberg Limits

Expansive clays exhibit higher shrinkage and swelling upon a change in moisture content [8]. These clays are checked for their liquid limits and plasticity Index in accordance with AASHTO T 89 and 90 to determine the nature and response of sub-grade soils upon change to moisture content. Only the materials passing through sieve size 0.425mm (No 40) are considered for Atterberg limit tests. The liquid limit and plastic limit tests collectively are called the tests for Atterberg limits. The test results were presented in appendix 1.

5.3.3. California Bearing Ratio (CBR) Test

The CBR is the most widely used methods for designing pavement structures. The method is primarily intended, but not limited to evaluate the strength of cohesive materials. The test procedure is based on, American society for testing and materials, AASHTO T 193. The CBR value for a soil depends upon its density, moulding moisture content and moisture content after soaking. For this research, that is, from km 8+500 to 28+700 lengths, three points CBR tests were carried out with 4 days soaking which helped to anticipate the sub-grade soils at the worst moisture conditions.

The subgrade soil of the current study area is black cotton soil with high volume change due to moisture fluctuation. This change in volume results in shrinking and swelling of the soil. The CBR value of these swelling soils was determined by 4 days soaking, 3-point CBR loaded with a 9kg surcharge load. CBR can be performed either for one point system or three points system for road materials. Three points CBR was conducted for subgrade soil in the laboratory and design CBR value considered at 93% of MDD as summarized below.

TABLE 5.8 SUBGRADE CBR VALUES AT 93% MDD (KM 9 + 375)

Number of blows	Dry density (gm/cc)	MDD(gm/cc)	CBR (%)	CBR swell %	CBR Swell	Penetration	
						2.54mm	5.08mm
10	1.26	1.66	0.6	4.22	3.22	13.33	20.00
30	1.52		1.1	3.62		13.33	20.00
65	1.66		1.5	3.22		13.33	20.00

CBR at 93% of MDD = 0.93*1.66 = 1.544g/cc.

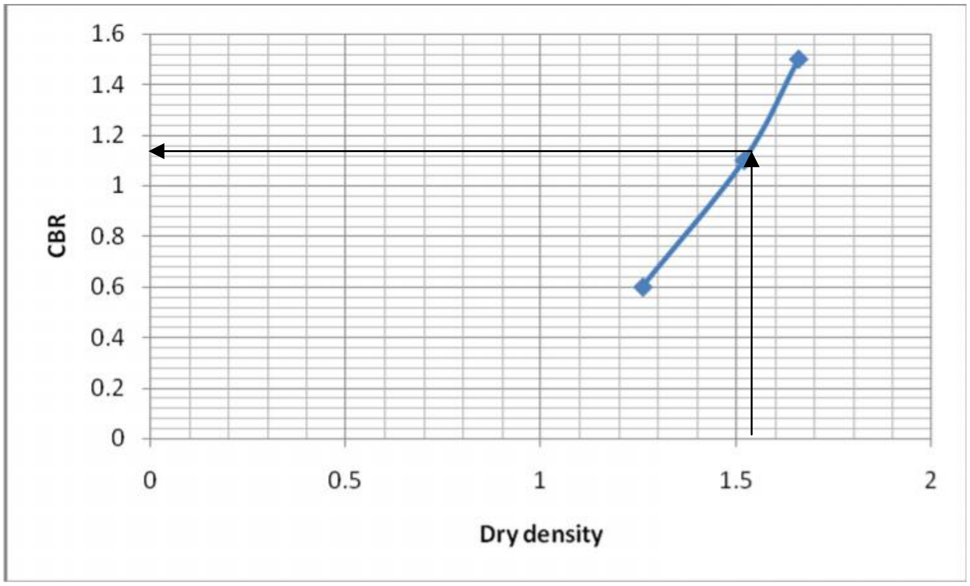

Figure 5.4 CBR VALUE AT 93% MDD

The CBR value at 93% of the MDD is 1.18% and hence the design CBR value is 1.18%.

Table 5.9 Subgrade CBR values at 93% MDD (km 9 + 656)

Number of blows	Dry density (gm/cc)	MDD(gm/cc)	CBR (%)	CBR swell %	CBR Swell	penetration	
						2.54mm	5.08mm
10	1.14		0.6	4.47		13.33	20.00
30	1.4	1.53	1.1	3.76		13.33	20.00
65	1.53		1.5	3.37	3.37	13.33	20.00

CBR at 93% of MDD = 0.93*1.53 = 1.423g/cc.

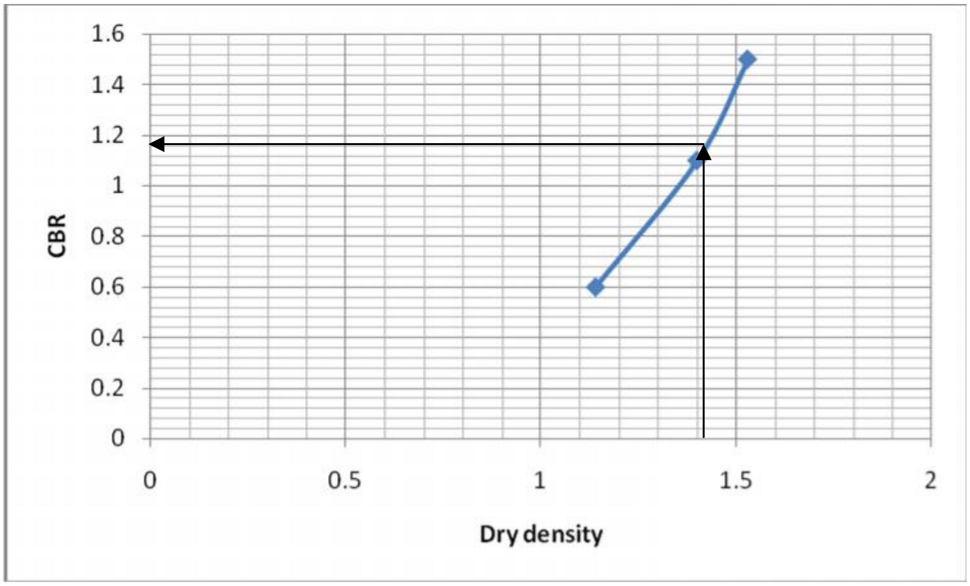

Figure 5.5 CBR Value at 93% MDD at station 9+656

The CBR value of 93% of the MDD is 1.19% from figure 5.5 above for station 9+656

Table 5.10 Subgrade CBR values at 93% MDD (km 24 + 575)

Number of blows	Dry density (gm/cc)	MDD(gm/cc)	CBR (%)	CBR swell %	CBR Swell	penetration	
						2.54mm	5.08mm
10	1.22	1.61	0.4	4.41	3.3	13.33	20.00
30	1.49		1.1	3.72		13.33	20.00
65	1.61		1.7	3.30		13.33	20.00

The CBR value of 93% of the MDD is 1.16% from figure 5.6 below for station 24+575.

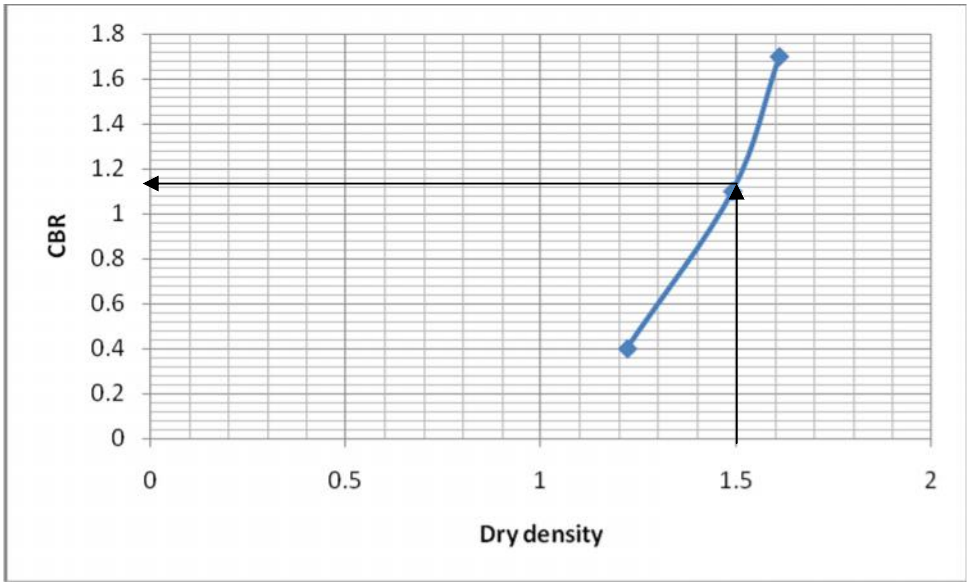

Figure 5.6 CBR VS Dry density at station 24+575

The above selected three sections of the road for determination of the CBR (design CBR) for subgrade at 93% of the maximum dry density are for the sake of illustration. When CBR tests are conducted for cohesive soils by three point methods at 4 days soaking conditions, the minimum density obtained at 65 blows of CBR value shall be the maximum dry density [13].

5.3.4. CBR Swell values

The swelling potential tests conducted during CBR soaking for 4 days can also be used to estimate the expansive nature of the sub-grade soils. The swelling potential is defined as the percentage swell of a laterally confined sample which has been compacted to MDD at OMC and allowed to swell under a surcharge load of 4.5kg conducted on CBR [2]. For the road under study, the native subgrade minimum CBR swell value recorded under a surcharge load of 9kg was 3.22%.

5.3.5. Specific Gravity (ASTM D854-98)

The specific gravity of a substance is the ratio of the unit weight of that substance to the unit weight of water at 25 degree centigrade. The specific gravity of a soil depends on the mineralogy of the soil grains. Most soils are a blend of several basic minerals. The subgrade soil under study is expansive black cotton soil composed of different minerals. The average specific gravity of the soil under study was 2.69.

5.3.6. Density - Moisture Relationship

The most common measure of compaction of soil is its density. Soils density and optimum moisture content should be determined according to AASHTO T180. Optimal engineering properties such as shear strength for a soil type occur near its maximum dry density (MDD) and optimum moisture content (OMC). At this state, soils void ratio, potential to shrink and swell is minimized. Field density is then correlated to moisture density relationship in the laboratory for quality controlling purposes in the field [13].

The sub-grade soil samples were subjected to the determination of maximum dry density (MDD) and optimum moisture content (OMC) in the laboratory. The laboratory test result reveals that the range of maximum dry density of the native Subgrade lies in the range of 1.52g/cc to 1.66g/cc and optimum moisture content (OMC) lies in between 21% to 25%. Test results are attached in Appendix 1.

5.3.7. Natural Moisture Content of the soil under study

A soil under study for this thesis is characterized by high swelling behavior as it can be seen from the test results. The natural moisture content obtained from an undisturbed sample range in between 30 % to 40%.

The plasticity index of the soil on which the road under study was constructed varies from 38% to 50%. The natural moisture content of the native subgrade and the plasticity limit of the soil are almost within similar ranges. The soil remains plastic since moisture content lies in the range of liquid limit and plastic limits. Laboratory test results for plasticity index and natural moistures were attached in appendix 1 and 3 respectively.

5.3.8. Dry Density (Υ_d)

Dry density is an important factor to determine the amount of volume change. Swelling pressure increases with the increase in dry density for constant moisture content. It is because higher density results in closer particles spacing which result in high amounts of swelling clays are packed with in small volume.

Dry density can be obtained from the relations:

$$\gamma_{bulk} = \frac{\text{(weight of wet soil)}}{\text{volume of the ring}} \qquad\qquad \gamma_d = \frac{\gamma_{bulk}}{1+w}$$

Where, Υ_{bulk} is bulk density, w is natural moisture content of the soil. The value of dry densities obtained from undisturbed soil samples was attacked in appendix 3.

5.3.9. Linear Shrinkage

Shrinkage limit is the water content at which the volume of a soil begins to change as a result of a change in water content. The test was conducted according to AASHTO T-92. Linear shrinkage limit of the subgrade soil under study lies in between 6% to 17%. The test result is found in appendix 2.

5.4. Swelling Pressure of the soil under study

5.4.1. Laboratory Measurement of swelling pressure

The direct tests which provide information on the amount of swelling that are to be anticipated are free swell and swelling pressure tests [4]. Hence, disturbed soil sample and undisturbed soil samples were collected for the free swell and swelling pressure tests respectively. The swelling pressure tests were conducted at the SABA Engineering Material Laboratory and free swell tests was carried out in CORE Consulting Engineer's Materials Laboratory. The test result of the free swell test was attached in Appendix 2 and swelling pressure test result was attached in Appendix 3.

A). Zero Swell Tests: were used to evaluate the swelling pressure of the soil under study. To conduct the test, the specimen in the constant volume method is allowed to absorb water without any increase in volume by increasing the applied weight as the test proceeds until the sample reaches equilibrium for 24 hours. The tests were conducted using oedometer, where the load is added to keep the volume of the sample constant while the sample absorbs water.

B). Free Swell Test

The free swell test consists of placing a known volume of dry soil in water and noticing the swelled volume after the materials settled for 24 hours without any surcharge, to the bottom of the graduated cylinder. The difference between the final and initial volume, expressed as a percentage of the initial volume, is the free swell value.

5.4.2. Estimation of swelling pressure from indirect measurement

Three locations in the study area, according to degree of damages of the road under study, have been selected and hence the estimated swelling pressure from the selected section of the road from equation discussed in chapter two are given as follows:

Table 5.11 prediction of swelling pressure from index property (Equation 2.2)

Locations (stations)	NMC (%)	Liquid limit (%)	Plasticity index (%)	Dry density (g/cc)	Shrinkage limit (%)	Estimated Swelling pressure (kPa)
ST1 (9+656)	40.2	94	50	1.14	14	36.75
ST2 (24+ 575)	35.4	83	40	1.23	13	132.63
ST3(24+757)	36.7	86	46	1.27	10	339.29
ST3 (28+400)	30.4	95	49	1.37	8	1630.29

Table 5.12 Prediction of swelling pressures from index property (Equation2.3)

Locations (stations)	NMC (%)	Liquid limit (%)	Plasticity index (%)	Dry density (g/cc)	Shrinkage limit (%)	Estimated Swelling pressure (kPa)
ST1 (9+656)	40.2	94	50	1.14	14	100.19
ST2 (24+ 575)	35.4	83	40	1.23	13	127.61
ST3(24+757)	36.7	86	46	1.27	10	137.76
ST3 (28+400)	30.4	95	49	1.37	8	165.85

Table 5.13 Prediction of swelling pressure from index properties (Equation 2.4)

Locations (stations)	NMC (%)	Liquid limit (%)	Plasticity index (%)	Dry density (g/cc)	Shrinkage limit (%)	Estimated Swelling pressure (kPa)
ST1 (9+656)	40.2	94	50	1.14	14	106.36
ST2 (24+ 575)	35.4	83	40	1.23	13	129.66
ST3(24+757)	36.7	86	46	1.27	10	435.49
ST3 (28+400)	30.4	95	49	1.37	8	737.87

Table 5.14 Comparison of previously developed equations with measured values of swelling pressure

Station (km)	**Measured swelling pressure (kPa)**	**(Table 5.11) (kPa)**	**(Table 5.12) (kPa)**	**(Table 5.13) (kPa)**
9+656	20.4	36.75	100.19	106.36
24+575	17.2	132.63	127.61	129.66
24+757	85.2	339.29	137.76	435.49
28+400	38.2	1630.29	165.85	737.87

None of the previously developed prediction models to estimate swelling pressure can precisely predict the swelling pressure of the study area. The intention of the study is actually not to evaluate precision of previously developed swelling pressure prediction model, but to evaluate sufficiency or adequacy of replacement depth of 60cm recommended by ERA for road constructed on black cotton subgrade soil. depth determined from actually measured swelling pressure and swelling pressure determined from prediction models previously developed by different researchers in Ethiopia were used for comparisons. Even though prediction models previously developed were not for the current study area and also the values obtained was not precisely closer to the actual value of the swelling pressure, Abraham Mengistie’s prediction model was used since it is a lower value obtained from the rest of the models used. Moreover, depth of undercut recommended by Capa was also used for comparisons. Hence, replacement depth obtained from measured swelling pressures, predicted swelling pressure, and from literatures is summarized as follows:

Table 5.15 Replacement depth determination from measured swelling pressure using equation 2.7

Station	Bulk density (gm/cc)	Unit weight (kN/m3)	Measured swelling pressure (kPa)	Replacement depth (m)
9+656	1.6	16	20.4	1.28
24+575	1.66	16.6	17.2	1.04
24+757	1.74	17.4	85.2	4.90
28+400	1.79	17.9	38.2	2.13

Table 5.16 Replacement depth determination from predicted swelling pressure (from Table 5.12)

Station	Bulk density (gm/cc)	Unit weight (kN/m3)	Estimated swelling pressure (kPa)	Replacement depth (m)
9+656	1.6	16	100.19	6.26
24+575	1.66	16.6	127.61	7.68
24+757	1.74	17.4	137.76	7.9
28+400	1.79	17.9	165.85	9.265

Table 5.17 over excavation of study area according to CAPA

Station	Subgrade plasticity index	Depth of over excavation (m)
9+656	50	1.7
24+575	40	1.3
24+757	46	1.7
28+400	49	1.7

6. DATA ANALYSIS AND INTERPRETATION

6.1 Discussions on test results of subgrade soil

6.1.1. Gradation

The particle size analysis and plasticity characteristics of the soils are required to classify soil. According to AASHTO soil classification Soils with 35% minimum percent pass sieve no. 200 are classified as silty-clay materials. The minimum percent pass sieve no. 200 for the soil under study is 92% and the soil is categorized as poor clay subgrade soil.

6.1.2. Atterberg Limits

Atterberg limits (liquid limit, plastic limit) were determined according to AASHTO T 89 and 90 standard test method. The results of the Atterberg limits were shown in appendix 1. Measured liquid limit was found in the range of 83% to 94% and plasticity index in the range of 38% to 50%. Based on the Atterberg test results soil classification is made based on AASHTO soil classification system as shown in figure below and classified as A-7-5. This soil groups have high liquid limit and are highly plastic as well as causes considerable volume change upon moisture fluctuation.

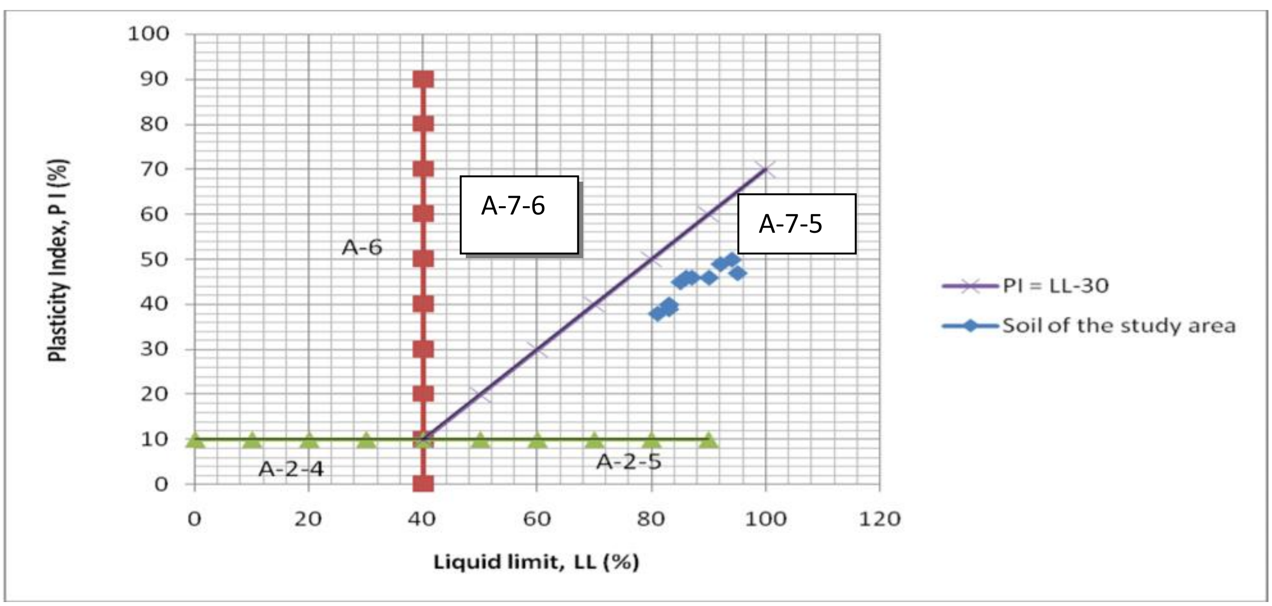

Figure 6.1.Plasticity chart for AASHTO classification methods

6.1.3. California Bearing Ratio (CBR)

Subgrade strength for pavement structural design is evaluated in terms of California Bearing Ratio (CBR). Strength indicators other than CBR may be used provided they are adequately correlated on CBR values. Design CBR of the subgrade under study is below 2% and subgrade class according to ERA pavement design manual is S2. According to ERA design manual subgrade class with CBR less than 2 cannot support structures constructed on it unless special treatment is made. The test was conducted for subgrade soil CBR at optimum moisture content and laboratory test result shows that the CBR value lie in between 1% - 2%. This shows that native subgrade soil is very poor soil to support structures constructed on it unless special treatment like removal and replacement of soil is made.

6.1.4. CBR swell values

The maximum CBR swell that has to be observed should not exceed 1.5 % as per the specification of the road under study. While, the value obtained from the laboratory test has minimum value of 3.22% which is twice higher than the specification CBR swell value under a surcharge load of 9kg. Hence, the soil has high swell value and cannot be used as subgrade soil or requires some kind of treatment.

6.1.5. Natural Moisture Content and Initial Dry Density

Natural moisture content of expansive soils controls the amount of swelling. Expansive soil with low moisture content can easily absorb moisture and can cause considerable swelling. Soil with high moisture content, swelling has already taken place and further expansion will be small. The natural moisture content of the soil under study ranges in between 30.4 to 40.2%. The swelling potential of subgrade soil at NMC of 30.4% at station 28+400 is 28.74 and at station 9+656 with NMC of 40.2% the swelling potential is 30.2%. The swelling potential of station 24+575 at NMC of 36.7% is 17.52%. The degree of expansiveness of the soil ranges from high to very high according to potential expansiveness of the soil based on swell potential given in Table 2.3.

Density is used as indicator of relative compactness of a soil. Density for expansive soil decreases with increasing water content. Increase in moisture content will leads to high volume change which enhances upward movement of the subsoil. The soil samples for subgrade for the current study were taken from the nearby side of the road, not from the beneath of the pavement structures. Accordingly, the initial dry density of soil under study lies in the range of 1.14gm/cc – 1.37gm/cc. according to Chen (1988) a soil having a density in excess of 1.76gm/cc would be suspected of a very high potential for swelling. However, the dry density obtained from the laboratory test is much lower than 1.76gm/cc, even if the swelling characteristics of the soil were found to be high to very high.

6.1.6. Free swell test

The free swell value of the soil of the study area varies from 140 % to 200%. The free swell value increases with plasticity index. Murthy, 2008 suggests that soils having a free swell value above 100 can cause damage where as free swell as low as 100 percent can cause considerable damage to light loaded structures and soils having a free swell value below 50 percent seldom exhibits appreciable volume change even under light loads. Hence the free swell value of the soil under study exceeds the mentioned value so that soil can cause considerable damage to the road constructed on it.

6.2. SWELLING PRESSURE TEST

6.2.1. Swelling pressure

The swelling pressures obtained from laboratory tests were varied from 17.2kPa to 85.2kPa. These swelling pressures are not expected to cause damage to the road overlaid on the soil. Though the soil is expected to swell more than these values, the season at which samples were taken will influence the swelling pressure value. The samples of the current study were taken during wet seasons and probably that may be the reason why measured swelling pressures of the native subgrade soil is very low, This is of that the soil has already absorbed sufficient moisture and subsequently swells to a certain level. Moreover, the soil samples were placed in odometer for 24 hours and test was conducted by applying a small incremental load to prevent swelling. However, one day only may not be sufficient to keep cohesive soil saturated and end up with volume change. The other factor that might could be accuracy of odometer equipment and testing methods. The cumulative effects of these factors influences the value of swelling pressure obtained in the laboratory.

6.3. Replacement depth and replacement materials type for study area

The replacement depth that was made during construction for Modjo – Edjere road was 60cm as obtained from as built drawing of the road. Replacement depth recommended in final engineering report of the road was 60cm too. Even though replacement of expansive soil was made for the road the road fails prematurely. The purpose of this thesis is to evaluate sufficiency of the replacement depth made and replacement materials type for the current study area road which was constructed on black cotton soil. Accordingly, depths of undercut of the road sections were determined from swelling pressures by correlating with unit weight of the soil of individual sections obtained from the same samples.

Replacement depth was determined from both measured swelling pressure and predicted swelling pressure of the selected section of the road. Hence, the replacement depth obtained from measured swelling pressure (Table 5.15) ranges in between 1.04m and 1.28m for intermediately damaged road section while 2.13m and 4.9m for severely damaged road sections.

Excavations of the cracked section were also made during site visits to know the depth of cracks and its origin. It was observed that the soil strata of the subgrade is black clay in colour for the first 80cm and it changes its color to gray soil beyond this depth even if the properties of the soil is similar from the laboratory test results conducted. The degree of expansion of the soil varies from high to very high according to swelling potential classification given in Table 2.3.

From field measurement made after excavation of the cracked sections (appendix 6, photo 1), it was seen that cracks were originated from subgrade soil and measured crack depths from original ground level for stations, 9+656, 24+575 and 28+400 were 85cm, 105cm and 95cm respectively. It is expected that these cracks were originated from subgrade soil due to swelling driven from an increase in water content of the underlying black cotton soil. Hence to control the swelling of the black cotton subgrade soil, soil shall be removed to the depth of moisture fluctuation and replaced by non-expansive fill. Replacement materials over expansive soil produces a surcharge loading that counterbalance uplift pressure comes from subgrade soil.

On the other hand, replacement materials used for the road under study were rock fill mixed with soil where fine fraction dominates and granular materials are floated on the finest materials as it can be seen from the photo. From the Thin Section report of the rock fills (appendix 5), Its Ground mass chiefly composed of very fine Microlite sanidine, Andesine and needle like elongated green hornblende showing parallel to sub parallel orientation or flow. Due to fine Microlite sanidine of the rock, there is a high porosity that tends the rock to absorb high water within the void. Literatures reviewed that the materials used as replacement for road constructed on expansive soil should be granular selected layers which are less sensitive to moisture change based on bearing capacity (CBR) strength. According to literatures, the minimum CBR value used is 15 percent at 93 percent of the modified AASHTO MMD at the highest anticipated moisture content in service.

For the current road under study, replacement materials with higher unit weight should be used to overcome damages caused due to upward pressure from underlying subgrade soil. Both depth of replacement and replacement materials used for the study area were not sufficient to counteract swelling pressure from subgrade.

7. GENERAL CONCLUSION AND RECOMMENDATION

7.1. Conclusion

- According to AASHTO classification system, the subgrade soil of the study area is classified as A-7-5. Furthermore, the CBR, plasticity index, and linear shrinkage test results of the soil shows that the subgrade soil has a very low bearing capacity and high swell potential which makes it unsuitable for subgrade soil without any suitable treatment measures.
- Tension cracks along carriage way and cracks on the shoulder of the road were seen during site visits. Cracks were originated from the native subgrade black cotton soil along the stretch of the road and it extends beyond 60cm replacement depth made for the road.
- Swelling pressure of the study area ranges from 17.2kPa – 85.2kPa. Due to inherent properties of black cotton soil upon wetting and drying, it is expected to record swelling pressure higher than measured swelling pressure. The reasons why swelling pressure of the soil is recorded low might be due to sampling season which was wet season that the soil has already absorb sufficient moisture and swell to a certain level. Though the value of measured swelling pressure is small, it is still higher than overburden pressure of the road and causes failure.
- Replacement depth of 60cm recommended by ERA for black cotton soil is not sufficient to counteract swelling pressure from subgrade soil. Failure of roads constructed on expansive soil is due to swelling driven from an increase in water content of the underlying subgrade soil. To control the swelling of the black cotton subgrade soil, soil shall be removed to the depth of moisture fluctuation and replaced by non-expansive fill.
- Replacement materials used over black cotton subgrade soil for the case study road were granular materials mixed with fine materials where fine materials dominates and granular materials floated on the fine materials. In addition to insufficiency of replacement depths made for the road, replacement materials made were also incapable to overcome compression from the subgrade soil.

7.2. Recommendation

Based on the above conclusions, the following recommendations are drawn.

- Degree of expansiveness of black cotton soil depends on mineralogical composition of the soil. There is also variation in their particle size distribution, clay and silt contents, liquid and plastic limits as well as swell potential. It is therefore necessary to develop design specifications for each category or groups or type of black cotton soil encountered in a particular area.
- To control swelling of black cotton subgrade soil, it is important to know depth of active zone and the potential uplift pressure of the soil. Depth of replacement can be determined from swelling pressure and also from active depth of moisture fluctuation. Hence, replacement depth shall be determined accordingly for road constructed on black cotton subgrade soil.
- Replacement materials should counteract swelling pressure from subgrade soil. Hence, replacement materials with higher unit weight should be used to counteract swelling pressure from underlying subgrade soil provided that adequate replacement depth was determined. According to some literatures like Tanzanian design manual, it is recommended to use replacement materials having minimum CBR value of ≥ 8%, preferably CBR value ≥ 15%.
- For the design of road pavement thickness, the strength of subgrade were taken in to account at its four days soaking moisture that assumed to simulate the actual wettest condition likely to occur after the road is opened to traffic. CBR tests do not show the actual swelling behavior of the soil and it is advisable to use swelling pressure tests of the subgrade soil in addition to CBR tests.

7.3. Suggestion for further study

- Further study is required to determine the moisture fluctuation zones, especially for arid climate and low land area to confirm the data obtained from this research for highland area hold true. This will help to set a standard for the country about the replacement depth.

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Appendix 1-4: Laboratory test results

Appendix 5: petrographic test of rock fill /Thin section/

Appendix 6: Photo reports

Photo 1 Cracks on the edge at station 24 + 590 – 24 + 610 RHS which extends beyond the rock fill

Photo 2 Edge cracks at station 28 + 600 RHS

Photo 3 Edge cracks at station 24 + 590 – 24 + 610 RHS

Photo 4 Surface cracks at station 10 + 346 – 10 + 600 LHS

Photo 5 Test pit taken for subgrade at station 10 + 080

Photo 6 Test pit taken at station 9 + 656

photo 7 Test pit taken for subgrade at 28 + 400

Photo 8 Test pit taken at station 24 + 757

photo 9 test pit taken at station 9 + 656

Photo 10 Sample taken for subgrade with its respective sampling station