“Stabilizing expansive subgrade soil on a selected road section in eastern part of Ethiopia in Somalia region Jigjiga-Tuli”

A Thesis Submitted to the School of Earth Science Presented in partial fulfillment of the requirements for the degree of Master of Science (Engineering Geology).

BY

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CERTIFICATE OF APPROVAL

This is to certify that the thesis prepared by Sisay Negash, entitled "Stabilizing expansive subgrade soil on a selected road section in eastern part of Ethiopia in Somalia region Jigjiga-Tuli" and submitted in partial fulfillment of the requirements for the degree of masters of Arts in Engineering Geology complies with the regulations of the university and meets the prevalent requirements with recognize to originality and best.

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DECLARATION

I, Sisay Negash, the undersigned, declare that this thesis entitled; “**Stabilizing expansive subgrade soil on a selected road section in eastern part of Ethiopia in Somalia region Jigjiga-Tuli**” is my original work. I conducted the research work on my own, with the practice and guide of the research advisor. This research study has not been submitted for any diploma or diploma software in this or any other institution, and that all sources of materials used for the thesis have been appropriately acknowledged. It is supplied here in partial success of the requirements for the award in Masters of Engineering Geology.

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List of Acronyms

AASHTO	American Association of state Highway and transportation officials
ASTM	American standard of testing materials
ERA	Ethiopian Roads Authority
CH	Inorganic clay with high plasticity
e	Void ratio
LL	Liquid limit
M.a.s.l	Mean Annual Sea Level
MDD	Maximum Dry Density
MH	Inorganic silt with high plasticity
NMC	Natural moisture content
PI	Plasticity index
PL	Plastic limit
OMC	Optimum Moisture Content
TP	Test pit
USCS	Unified soil classification system

ABSTRACT

The long-term performance of any construction project depends on the soundness of the underlying soils. Unstable soils can create significant problems for pavements or structures. In order to avoid most of the problems that has been appearing on the upper layer of any pavement it is better to treat the subgrade soil. To achieve the research objectives, subgrade soils samples were collected along the corridor of the road section and evaluated for their physical and geotechnical properties. The laboratory results of previous investigations on the area show that approximately 71 percent of the existing soil has a plastic index (PI) ranging from 27 to 35 percent and a liquid limit (LL) ranging from 55 to 70 percent with an average of 62.5 percent and <5% California bearing ratio (CBR) and approximately 89 percent of the swell values are greater than 2%. This reveals that the sub-grade soil along the route project is primarily characterized by A-7-5 and A-7-6 according to the American Association of State Highway and Transportation Officials (AASHTO) and fat clay (CH) and elastic silt (MH) of UCS soil classification which is considered as a problematic soil for subgrade materials. The swelling pressure of natural subgrade soil was determined using the laboratory to evaluate the fill height of the embankment on the geotechnical properties of soil. Because the swelling pressure of the sub-grade material has a medium swell potential and low potential heave, so soil replacement of only shallow soil not exceeding 30 cm is sufficient (only for the selected stretches) to equalize the swelling pressure of the existing soil with lower swelling potential, higher workability, and stabilized soils were feasible to be used as subgrade materials.

CHAPTER ONE

1. INTRODUCTION

1.1 Background of the Study

Expansive or swelling soil is highly plastic soil that typically contains montmorillonite and other active clay minerals. Such soils are universally recognized as problematic soils, which has made experts concerned about the design, protection, and operation of highway and structural systems. Expansive soils can be found in arid/semi-arid areas where even moderate expansive soils can cause serious damages to the structure, or in humid environments where expansive soils with high plasticity index (Ip) can lead the structure to be damaged. Different factors are responsible for the behaviors of expansive soil, among which the principal ones are the fluctuation of moisture and the amount and type of clay minerals in the soil. It is worth mentioning that when the water content changes in expansive soil, the volume would be changed as well. These volume changes can lead to either swelling or shrinkage and that is why expansive soils are also known as swell/shrink soils (Ardani, 1992; Day, 2000; Ito and Azam, 2010; Jones & Jefferson, 2012; Liu et al., 2015, as cited by Faezehossadat, K and Jeff. B.,2016). Predicting volume change and swelling potential of expansive soils is of great significance for plentiful engineering and industrial applications. Even know a time having many researchers have provided different kinds of stabilization techniques expansive soils cause a serious of Damage for each year, for example, has been reported to cost more than all other natural hazards combined, including earthquakes, fires, floods, and tornadoes (Chen, 1988). Prediction methods for ground movements and swelling pressures are mostly empirical or semi-empirical (Hunt, 1986; Nelson & Miller, 1992), where most predictive methodologies are derived from experimental tests that are relatively easy and inexpensive.

To evaluate the soil shrinkage and swelling amount, the water content in the near-surface zone can be measured. Normally, most of the significant action happens in a depth not more than 3 meters, although if the tree roots exist in the area, this amount can be extended (Driscoll, 1983; Biddle.1998).

Approximately all types of transportation facilities have been affected by expansive soil behavior and, as a result, many have failed or are no longer serviceable. It is very critical to control the damage caused by these expansive soils through the application of robust soil stabilization methods. (Petry.M and J. Clyde.2002)

Roads and runways have suffered from destructive differential movements caused by such kinds of soils. The nature of these soils has led to many slope failures and, retaining walls and bridge abutments have experienced extreme distortion and have been overturned by swelling pressures associated with these soils. Track systems have been moved out of alignment, both vertically and horizontally, by the effects of expansive soils, and port facilities have been affected by both the influence and the amount of volume change. Even pipelines have had their share of damage, as exhibited in changes in alignment and crushing. (Petry.M and J. Clyde.2002).

<<The two major factors in treating of expansive soils are: (1) Identifying, and (2) estimating the anticipated potential volume change of the subgrade soils (Ardani, 1992). >>

1.2 Problem statement

Construction of civil structure requires information all about the geology and engineering property of soils and rock the presences of expansive soil causes swelling and shrinking these soils are susceptible to volume change in response to fluctuations in groundwater table and moisture content following seasonal climate variations.

This can cause severe damage to infrastructures unless proper measures are taken in the design and construction phases. Therefore, identifying the problem related to subgrade soil is very important to improve their strength and increases the engineering life of roads.

Ethiopian roads authority (ERA) is responsible for the construction, administration as well as maintenance of roads network. Currently there is huge construction of roads in the country and the road coverage is increasing from time to time, but it is common to see a lot of defects in both the newly constructed roads as well as existing roads.

Sub-grade investigation and improvement were not practically fulfilled, where less and less routine subsurface investigation is conducted. In the study area, distress of existing ground, some distortion, potholes, swelling upward displacement, uneven road surface, and cracking occurred on existing gravel roads and footpaths Although geotechnical investigation of sub-grade soil was carried out from the period December 2019 by China Civil Engineering Construction Corporation (CCECC). This study aims to investigate the swelling pressure to check the replacement thickens of expansive soil which was recommended by Ethiopian Road Authority (ERA).

1.3 Objective

1.3.1 General objectives

The main objective of the research is to assess the engineering property of Jigjiga-Tuli subgrade soil along the proposed road corridor and suggest possible mitigation measures along the study area.

1.3.2 Specific objectives

- Characterizing the sub-grade expansive soil.
- To identify the problematic sections of the proposed road corridor.
- Formulating a proper and robust stabilizing methodology for the suitability of expansive subgrade soil.

1.4 Significance of the study

The outputs and findings of the research will be a bench for both governmental and non-governmental organizations (organizations that participated in the road construction sectors, design, and supervision companies) and also serve as a resource for further research in areas of subgrade soil stabilization. It will be feedback for ERA to evaluate its stabilization practice.

1.5 Scope and organization of the thesis

The research focuses on subgrade soil stabilization practice of ERA. In terms of area its scope is limited in eastern part of Ethiopia in Somalia region Jigjiga-Tuli, by a proposed road and assessing the swelling pressure of the expansive soil of the area and evaluating its possible potential heave only on the stabilization practice of Ethiopian Roads Authority.

1.6 Limitations

- The number of samples collected is limited to only 10 representative samples to characterize 30km of the road corridor.
- The depth of investigation in this research is limited to the maximum depth not greater than two meters since it is difficult to excavate and sampling manually beyond this depth.

1.7 Outline of the thesis

This thesis consists of seven chapters and an Appendix. In the **first chapter** of the research a general introduction comprises the background of the study, problem statement, objectives of the study, significance of the research, scope of the study, limitation of the research, and outline of the thesis is briefly noted.

Chapter 2 Discuss the theoretical framework for the study. These frameworks were subsequently considered engineering characteristics of expansive soils and possible undertaken measures.

Chapter 3 Provides a good understanding of the study area including the location, climate, geology, accessibility, with very descriptive maps and figures.

Chapter 4 Presents the research method and materials in detail.

Chapter 5. Laboratory test results of all data were presented in detail. The primary experimental observations are summarized.

Chapter 6 Focuses on the analysis of results and discussions.

Chapter 7 Summarizes the conclusions obtained in the finding and recommendations.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Introduction

Expansive soil is a type of soil that changes in volume with changes in moisture conditions. Many soils that exhibit swelling and shrinking behavior contain expansive clay minerals, such as smectite, that absorb water; the more clay soil contains the higher its swell potential and abundant water it can absorb. As a result, these materials swell, and thus increase in volume, when they get wet and shrink when they dry. The more water they absorb the more their volume changes, for the most expansive clays expansions of 10% are not uncommon (Chen 1988; Nelson and Miller, 1992).

The extent of shrink and/or swell is determined by the water content in the near-surface zone; significant activity usually occurs to about 3m depth, unless this zone is extended by the presence of tree roots (Driscoll, 1983; Biddle 1998). Fine-grained clay-rich soils can absorb large amounts of water after a rainfall, becoming sticky and heavy. On the other hand, they can also become very hard when dry, resulting in shrinking and cracking of the ground. This hardening and softening are known as 'shrink-swell behavior.

Swelling and shrinkage are not fully reversible processes (Holtz & Kovacs, 1981).

The development of shrinkage causes cracks, which on re-wetting, do not close perfectly and hence causing the soil to bulk out slightly, and also allow the entrance of water for the swelling process. When it is exposed to a long period of geological time scales shrinkage cracks may become in-filled with sediment, thus affecting the homogeneity of the soil. When the material is filled with cracks the soil is unable to move back, thus resulting in enhanced swelling pressure.

The main characteristics of Expansive clay among others are (Gidigasu and Gawu,2013):

1. Black or dark grey to brown color
2. High content of expansive clay mineral montmorillonite
3. Poses the tendency to shrink and swell with change in moisture condition
4. Exhibit's heave and crack as geo-environmental phenomena.

2.2 The genesis of Expansive Soil

Igneous, sedimentary and metamorphic rocks oversee the existence of expansive clay. Expansive Soil is formed mainly by the chemical weathering of mafic (basic) igneous rocks such as basalt, norite, andesites, diabases, dolerites, gabbros, and volcanic rocks and their metamorphic derivatives (e.g., gneisses) which is made up of calcium-rich feldspars and dark minerals which are high in the weathering order, in poorly drained areas with well-defined wet and dry seasons. All constituents whether to form amorphous hydrous oxides and under suitable conditions clay minerals develop. The absence of quartz leads to the formation of fine-grained, mostly clay size, plastic soils which are highly impermeable and easily become waterlogged. In addition, abundant magnesium and calcium present in the rock add to the possibility of the formation of black cotton soil with its attendant swelling problem (Ola, 1983). The Expansive soils have also formed into sedimentary materials such as shales, limestones, slates, etc. (Ahmad,1983., as cited by Gidigasu and Gawu,2013).) found that although the parent materials are diverse, one striking feature which is common to all is the fact that the parent materials are rich in feldspar and ferromagnesian minerals which yield clay residue on weathering. He also noted that where the parent rock is not mafic (basic), alkali earth elements can be added through seepage or by flooding waters.

2.3 Characterization and Classification of Expansive Soils

2.3.1 Characterization of Expansive Soils

About 40 to 60% of expansive soils have grain sizes less than 0.001mm. Expansive Soils generally have higher liquid limit and plasticity index and extremely low CBR values. At their liquid limit, the volume change is of the order of 200 to 300% and results in swelling pressure as high as 8kg/cm2 to 10kg/cm2. Soaked laboratory CBR values of expansive soils are generally found to be in the range of 2 to 4%. expansive soils absorb water heavily, swell, become soft and lose strength. These soils are easily compressible when wet and possess a tendency to heave during the wet condition and shrink in volume and develop cracks during dry seasons of a year and they show extreme hardness and cracks when they are in dry condition. The seasonal change in volume of expansive soils is manifested by both horizontal and vertical movements, the horizontal movement leads to fissure opening during dry seasons and closing during wet seasons whereas the vertical movement leads to cyclic changes in levels. The magnitude of these movements decreases with depth where there are no seasonal moisture changes (Seehra,2008 and Reo,2007., as cited by Tagel. M,2016).

1. Expansive Soil characteristics ERA, (2002)

- Spacing and width of wide or deep shrinkage cracks.
- High dry strength and low wet strength (indicates high plasticity).
- Stickiness and low trafficability when wet.
- Scraped or cut surfaces have a glazed or shiny appearance, like soap.

2.3.2 Identification and Classification of Expansive soils

Various criteria adopted to recognize the presence of expanding lattice-type clay minerals in natural soil can be broadly classified into two categories namely, mineralogical identification and inferential testing methods (Sridharan and Prakash, 2016). Excessive movements can lead to both damage and a negative impact on the structural performance in terms of cost and time Therefore, identifying the expansive soils can help researchers reduce the imposed damages resulting from the expansive soil to the structure. (Reddy et al., 2009,as cited by Faezehossadat, K and Jeff. B.2016). Figure 1 presents the methods used in each group. It is worth mentioning that mineral identification methods are not only time-consuming but also requires special expertise and equipment. Therefore, most experts prefer simple identification tests based on the physical properties of the soil.

a. Visual inspection (field identification)

- They commonly have a color of black and gray.
- A shiny surface over a bulging surface.
- The wet samples of the soil are sticky and very strong when they get dry.
- The presence of elongated cracking (distress) on the existing structures.

On the area of seasonal moisture variation

- Open or closed fissures.
- Shattering or micro shattering are common.

Table 1 Features of Expansive Soils Descriptions ERA, (2002)

Soil Description	**Typical Features of Expansive Soils**
Soil Type	More clayey soils are likely to be expansive
Consistency when slightly moist to dry	Stiff to very stiff
Consistency when wet	Soft to firm and sticky
Structure	Typical cracked surface, slick-sided fissures
Color	Only a reliable indicator when combined

b. Laboratory identification

Generally, there are three different methods of identifying expansive soil in the laboratory.

2.3.2.1 Indirect methods

In this method Soil clay Mineralogical composition with index properties of fine-grained soils is examined and hence, to estimate their swell potential (Sridharan and Prakash, 2016). Measurement of the plasticity and the shrinkage characteristics of the soil is conducted for identification of soils and provides a wide acceptable means of rating by using liquid limit, plastic limit, shrinkage limit, free swell tests, colloid content test, etc.

I.Particle size associated properties:

Soil swelling is controlled by the relative particle size distribution of fine-grained soils. Many researchers have proposed criteria based on percentage clay size fraction (i.e., <0.002 mm size) or colloid content (i.e., the content of particles of size less than 0.001 mm) to predict the swell potential of fine-grained soils, presented in Table 2.

Table 2 Grain size and swell potential relation

Degree Of Expansivity/Swell Potential	Percent Clay Size Fraction (Chen,1965)	Colloid Content (Holtz and Gibbs 1956)
Low	<30	<15
Medium	30–60	13–23
High	60–95	20–31
Very High	>95	>28

II.Liquid limit related properties:

The relationship between moisture content and consistency defined by the Atterberg limits recognizes the degree of soil swell potential based on the liquid limit of fine-grained soils (Table 3).

Table 3 Liquid limit and Swell potential relation

Swell potential	Liquid limit (%)		
	Chen (1965)	Snethan et al. (1977)	IS: 1498 (1970)
Low	<30	<50	20–35
Medium/marginal	30–40	50–60	35–50
High	40–60	>60	50–70
Very high	>60	–	70–90

III. Plasticity Index related properties:

The most widely used parameter for determining the shrinkage and swelling potential of soil is the Plasticity Index (IP). (BRGM,2009). The higher the plasticity index, the more plastic the soil is and higher will be the soil swell potential. schemes available to recognize the soil expansivity based on the plasticity index presented in Table 4.

Table 4 Expansive soil classification based on plasticity index (BRGM,2009)

Classification of potential swell	Plasticity index
Low	<12
Medium	12-25
High	25-40
Very high	>40

IV. A 'Modified Plasticity Index' (IP')

A 'Modified Plasticity Index' (IP') is proposed in the Building Research Establishment (BRE, 1993).

Table 5 Classification for shrink-swell clay soils (BRE, 1993)

Ip' (%)	Volume Change Potential
> 60	Very high
40 - 60	High
20 - 40	Medium
< 20	Low

Where: $Ip' = Ip \times (\%<425\mu m) / 100\%$

V. Shrinkage and Degree of expansion

The swell potential is presumed to be related to the shrinkage limits as well as linear shrinkage. Altmeyer (1955) suggested the values presented in Table (6) as a guide to the determination of potential expansiveness based on shrinkage limits and linear shrinkages.

Table 6 Relation between swelling potential, shrinkage limits, and linear shrinkages (Altmeyer, 1955)

Shrinkage limit (%)	Linear shrinkage (%)	Degree of expansion
<10	>8	Critical
10-12	5-8	Marginal
>12	0-5	Non-critical

VI. Free swell and Degree of expansion

This test was first proposed by Holtz and Gibbs (1956). This test consists of pouring slowly 10 cm3 of oven-dried soil passing 425 μm sieve into a 100 cm3 measuring jar filled with distilled water and noting the equilibrium volume of the sediment formed. The free swell value is then calculated as the increase in the volume of the soil expressed as a percentage of the initial volume. The percent of free swell is expressed as:

Free swell percent = $\Delta V V * 100\%$

where $\Delta V = V_f - V$ = change in initial volume (V) of a specimen and

V = initial volume (10 mm3) of the specimen

Vf = final volume of the specimen

Soils with free swell less than 50% are not likely to show an expansive property, while soils with the free swell in excess of 50 percent could present swell problems. Value of 100% or more is associated with clay which could swell considerably, especially under light loadings.

Table 7 swell potential classification (Holtz and Gibbs,1956)

Swell potential	FS value (%)
Low	<50
Medium	50 – 100
High	100 – 200
Very high	>200

VII. Activity

Skempton (1953) defined activity (A) as the ratio of plasticity index (PI) to < 0.002 mm clay fraction (CF) and showed that activity could be related to the mineralogy and geotechnical history of the clay. He classified clays into three groups; "in-active," "normal" and "active" clays.

$$\textbf{Activity} = \frac{PI}{<0.002\%\ clay}$$

Table 8 table for evaluation of potential expansiveness (Skempton,1953)

Activity	Nature of soil	Swell potential
<0.75	Inactive Clay	Low
0.75 - 1.25	Normal Clay	High
>1.25	Active Clay	Very high

VIII.Expansive Soils- Classification ERA, (2002)

According to ERA, (2002), Extended investigations are advisable if:

• The results of the field reconnaissance indicate expansive soils, and

• PIw >20%

Where PIw = Plasticity Index tested on fraction <425μm, weighted for the sample’s actual content of particles <425μm as follows:

PIw = PI x (% passing 425μm)/100

To estimate expansiveness ($Є_{ex}$)

$Є_{ex} = 2.4\ w_p - 3.9\ w_s + 32.5$

Were

w_p = PI x (% passing 425mm)/100

w_s = Shrinkage Limit x (% passing 425μm)/100

Table 9 Classification of Expansive soils. ERA, (2002)

Expansiveness Єex	Classification
<20	Low
20-50	Medium
>50	High

2.3.2.2 Direct measurement

The most reliable method of determining the swelling potential (% swell) and swelling pressure of expansive clay is by direct measurement using the Oedometer testing technique.

$\%\ swell = \Delta H / H * 100$

According to Alemayehu and Mesfin, (1999), the Oedometer testing techniques have become popular and are extensively used. The different types of techniques under these methods are the Constant Volume Method, Swell-Consolidation Method, Different Pressure Method, and Double-Oedometer Method. By Abdelkader j and Abdelmalek. B (2001). Table 10.

Table 10 Swelling pressure and Swelling potential after Ghen (1988)

%<75um	W_L (%)	Swelling pressure (kpa)	Swelling potential
>90	>60	250-1000	Very high
60-95	40-60	150-250	High
30-60	30-40	50-150	Moderate
<30	<30	<50	Low

2.1.1 Mineralogical Identification

In this method, the mineralogical composition of clay particles will be identified, such as characteristic crystal dimensions, characteristic reaction to heat treatment, size, and shape of clay particles, and charge deficiency and surface activity of clay particles. These properties are a fundamental factor controlling expansive soil behavior. The various techniques under these methods are:

-X-ray diffraction

-Differential thermal analysis

-Dye absorption

-Electron microscope

-Cation exchange capacity, etc.

But these methods are not suitable for routine tests because of the following reason; they are time-consuming, require expensive test equipment and, the results are interpreted by specially trained technicians (Sridharan and Prakash, 2016). Figure 1.

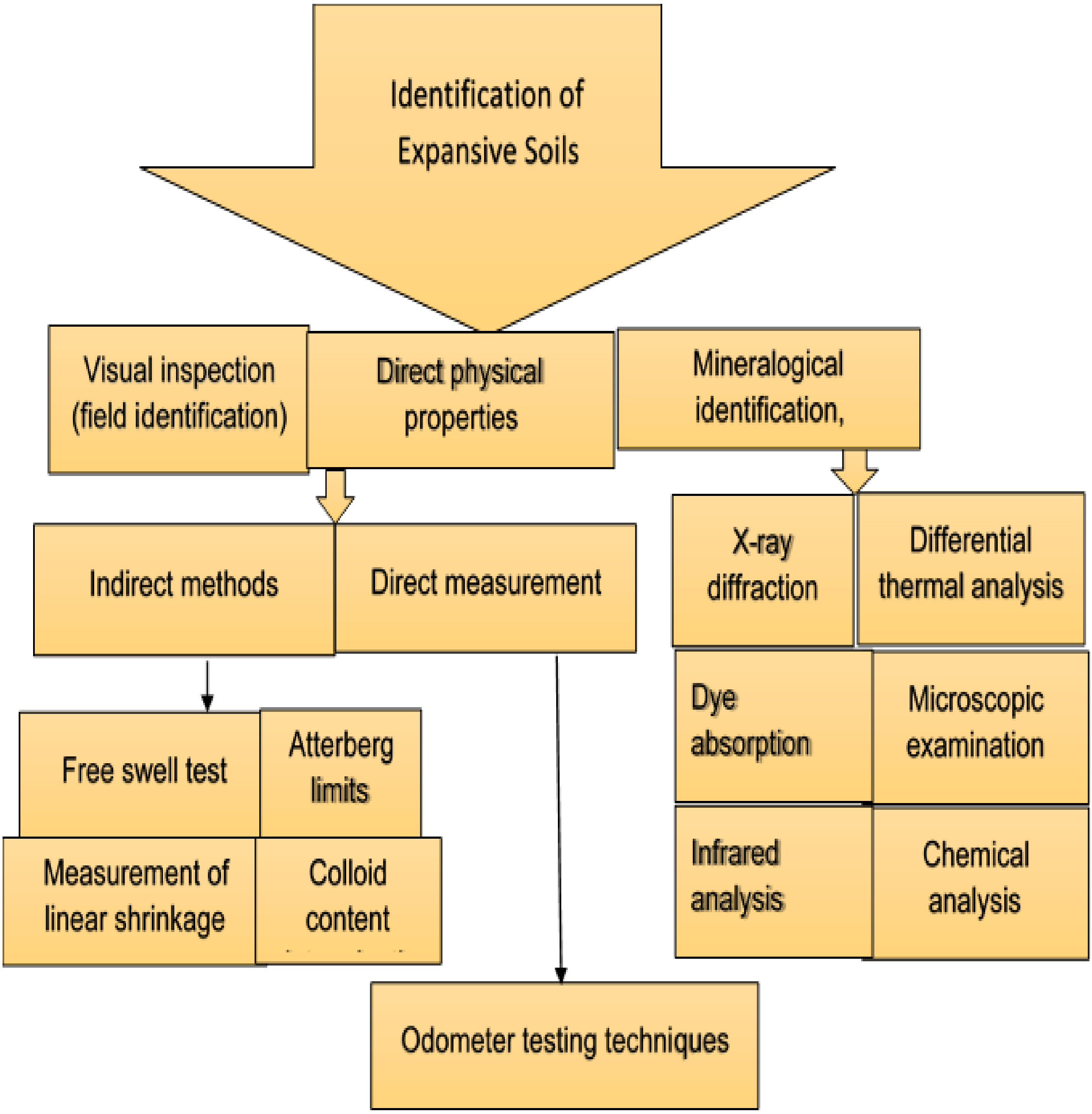


Figure 1 types of expansive clay identification approaches

2.4 Distribution of Expansive soils in Ethiopia

Local climatic conditions, sedimentation, and geological history are the dominant aspects of the Distribution of expansive soil. Arid climatic conditions and severe weathering environments prevailing in the northeastern part of Africa endorse the widespread occurrence of expansive soils. In Ethiopia, virtually 40% of the surface area is covered by such soils. It is also observed in an area such as central Ethiopia, following the major trunk roads like Addis Ababa - Ambo, Addis Ababa - Weliso, Addis Ababa - Debere Berehan, Addis Ababa - Gohatsion, Addis Ababa - Mojo. Also, they cover the area like Mekelle, Bahirdar, Gambela, Arba Minch, and the most Southern, South-west and south-east part of the capital Addis Ababa area in which the most major recent construction is being carried out (Affework Sisay, 2004; Tewodros Alene, 2010).

2.5 Problems Associated with Expansive Soils

According to ERA, 2002, most of the problems associated with expansive soils arise mostly from the nature of the soil itself and the drainage facilities provided. As a result of their low CBR and strength, expansive soils fail to support the loads transmitted from the pavement structure and cause excessive deformation beyond permissible limits. The common problems associated with expansive soils are described below.

❖ Volume change

Expansive soils tend to heave during the wet condition, shrink and develop cracks during dry seasons which makes expansive soils a problem to road pavements. The cracks developed during dry seasons allow water to penetrate deep into the soil during rainy seasons, hence causing considerable heave and expansion. This results in the deformation of road surface constructed on expansive soils as the expansion and the subsequent heave are never uniform. Figure 2. Furthermore, the shrink-swell behavior of expansive soils may lead to lateral displacements/ creep of the pavement layer on expansive soils, if the side slopes are not gentle enough. During seasonal change (rainy and dry seasons), the road edges get wet at a faster rate than the surfacing of the road. This results in differential movements over the road cross-section and associated crack development, first occurring in the shoulder areas and developing to carriageways.

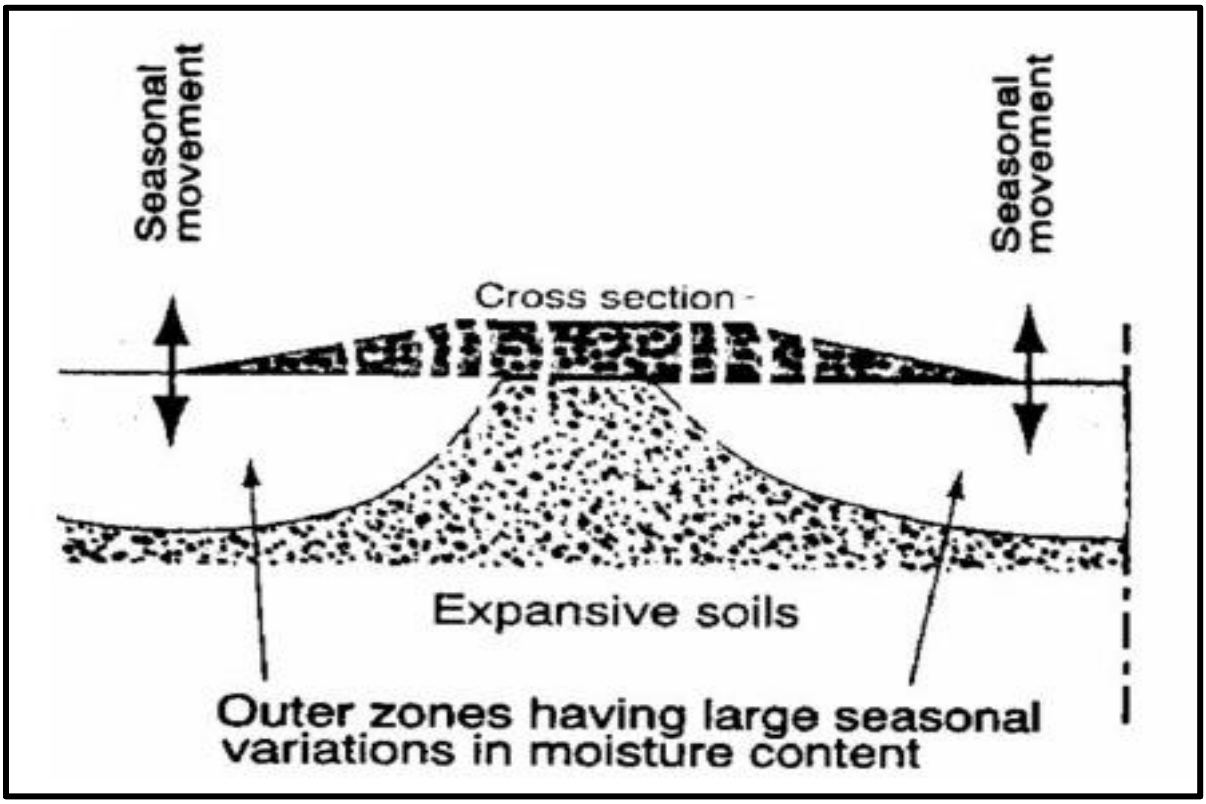


Figure 2 Moisture content in Expansive Soils (ERA Site Investigation Manual-2002)

❖ Bearing capacity

When the moisture content increases, expansive soils swell and become loose and hence their bearing capacity reduces dramatically. If the soil becomes fully saturated, the CBR value reduces to a value under 2% which makes such soils unsuitable to be used as road subgrade material.

2.6 Factors Influencing Swelling and Shrinking of expansive Soil

The factors influencing the shrink-swell potential of soil can be considered in three different groups, Nelson and millor, (1992).

❖ Soil characteristic that influences the basic nature of the internal force field. These include: -

Clay mineralogy, Plasticity, Dry density, Soil suction, Soil water chemistry, Soil structures, and fabrics.

❖ The environment factor influences the changes that may occur in the internal force system. These include;

Initial moisture condition, Moisture variation, Climate (Ground water, Drainage, and manmade water source, Vegetation, Permeability, Temperature)

- State of stress, which include

Stress history, Surcharge load

2.6.1 Effect of Clay mineralogy

The expansiveness of soils is due to the presence of clay minerals. Clay particles have sizes of 0.002mm or less. However, according to Chain. (1988), the grain size alone does not determine clay minerals and the most important property of fine-grained soils is their mineralogical composition. Clay minerals are crystalline hydrous alumino-silicates derived from parent rock by weathering. Figure 3. The basic building blocks of clay minerals are the silica tetrahedron and the alumina octahedron and combine into tetrahedral and octahedral sheets to form the various types of clays. Kaolinite, Illite, and Montmorillonite are the common groups of clay minerals most important in engineering studies.

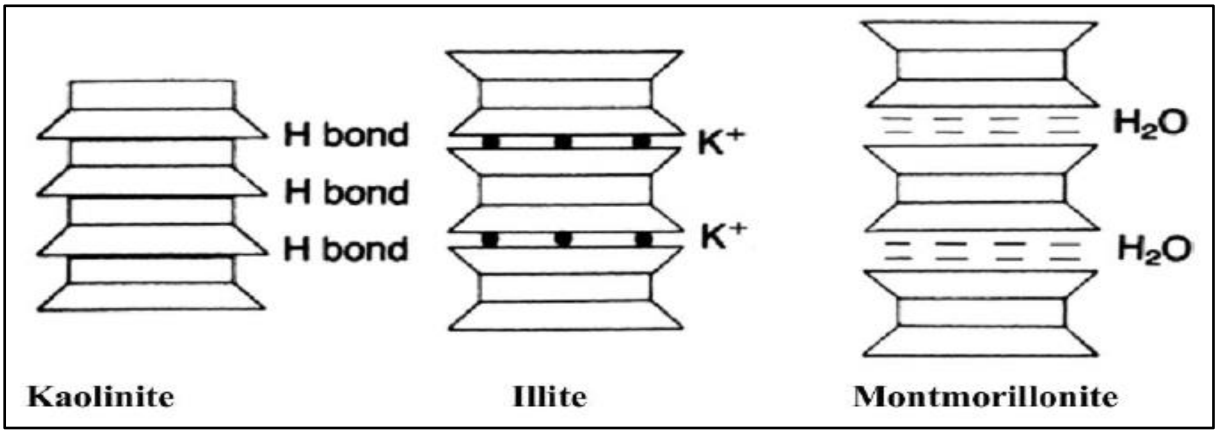


Figure 3 Schematic representation of clay minerals (Craig, 1997)

Kaolinite is a typical two-layered mineral having a tetrahedral and an octahedral sheet joined to form a 1 to 1 layer structure held by a relatively strong hydrogen bond. Kaolinite does not absorb water and hence does not expand when it comes in contact with water. The montmorillonite groups of clay minerals have a 2 to 1 layer structure formed by an octahedron sandwich between two tetra hydrons (Nelson, 2010). These clay groups have a significant amount of magnesium and iron sandwiched into octahedral layers. The most important aspect of the montmorillonite clay mineralogy group is the ability for water molecules to be absorbed between the layers, causing the volume of the minerals to increase when they come in contact with water. The Illite clay minerals have a structure similar to that of kaolinite, but are typically deficient in alkalis, with less aluminum substitution for silicon, magnesium, and calcium can also sometimes substitute for potassium and illites are the non-expanding type of clay minerals (Walsh et al., 2009).

2.6.2 Effect of Seasonal water content variations

The mechanism of swelling on expansive soils is complex and is influenced by many factors, and water is the most important factor. In this context, soils volume changes (shrink/swell), as well as shear strength changes, are a result of changes in the soil–water system. Especially a few meters below the ground surface. the water contents are influenced by climatic environmental factors.

Change in water content is responsible for the problem of Expansive soil in the upper few meters, with deep-seated heave being rare (Nelson and Miller 1992). The water content in these upper layers is significantly influenced by climatic and environmental factors and is generally termed the zone of seasonal fluctuations or active zone as shown in Figure (4).

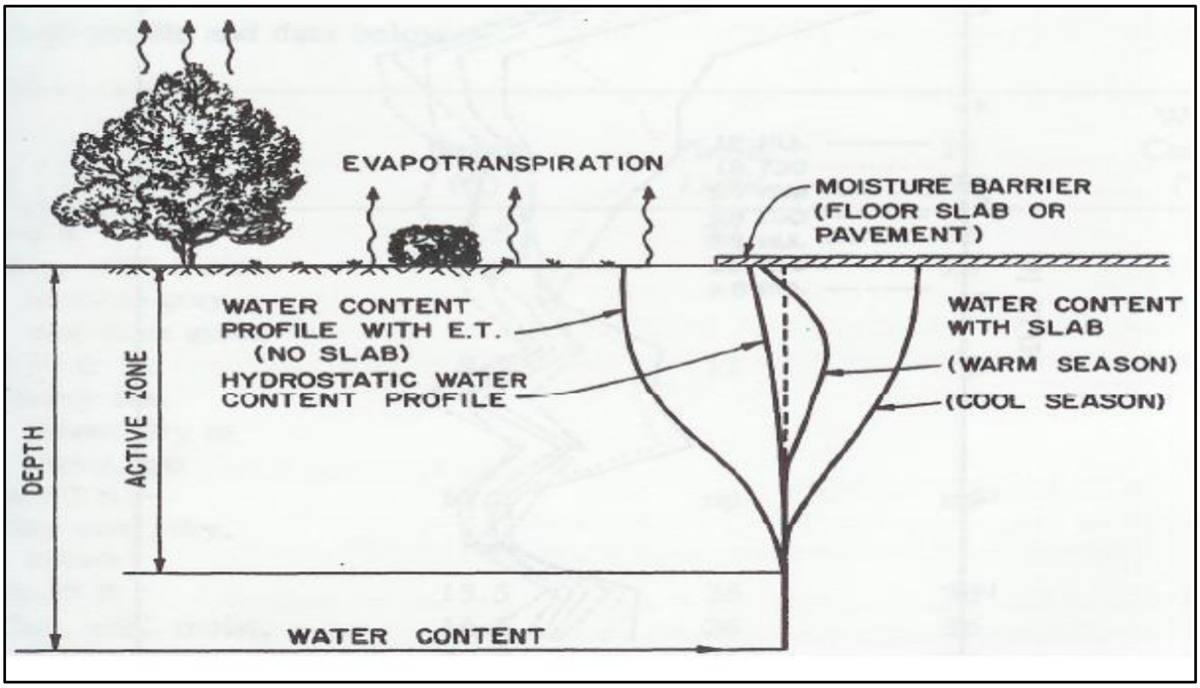


Figure 4 Water content profiles in the active zone (Nelson and Miller, 1992)

In the active zone, negative pore water pressures exist, however, if excess water is added to the surface or if evapotranspiration is eliminated then water contents increase, and heave will occur. Migration of water through the zone is also influenced by temperature as presented in Figure 4, with further details provided by Nelson et al., (2001). Thus, it is important to determine the depth of the active zone during a site investigation. The problem is it requires a time-consuming observation period for a soil investigation program. To obtain representative data, a measurement program should be conducted in 2 ~ 3 wet – dry seasons. In other words, it takes 2 ~ 3 years.

According to ERA, GDM (2013), Seasonal moisture variations in the central part of Ethiopia and in some northern and western highlands have been reported to occur up to a depth of between 3 and 5m. In lowland and semi-arid areas with reduced sources of water, variation is limited to a depth of 2m.

The term 'Active Zone' can have different meanings. Nelson et al. (2001) provides four definitions for clarity:

❖ **Active Zone:** The zone of soil that contributes to soil expansion at any particular time

❖ **Zone of Seasonal moisture fluctuation**: The zone in which water content changes due to climatic changes at the ground surface.

❖ **Depth of wetting**: The depth to which water contents have increased due to the presence of water from external sources

❖ **Depth of potential heave:** the depth at which the overburden vertical stress equals or exceeds the swelling pressure of the soil. This is the maximum depth of the active zone.

The depth of wetting is particularly important as it is used to estimate heave by integrating the strain produced over the zone in which water contents change (Walsh et al., 2009).

Details of how this can be achieved and the relative merits of regional and site-specific approaches are considered in detail for a post-development profile by the relationship between climatic condition and depth of active zone is presented in table 11.

Table 11 Relationship between climate and active zone (as cited by Ijara,2020).

Climate description	Active zone depth (m)
Alpine/west coastal	1.5
Wet temperate	1.8
Temperate	2.3
Dry temperate	3.0
Semi-arid	4.0

2.6.3 Potential heave

Heave is the upward movement of an underlying supporting soil stratum usually due to the addition of water to an unsaturated expansive soil in the active zone. When moisture is added to soil with clay content, expansion occurs within the structure of the soil, and the corresponding area of the subgrade and superstructure is moved upward. Heave normally only occurs within clayey soils that have a high suction potential and an available moisture source. (Kelm, P.E., Nicole. W,2007). Potential Vertical Rise (PVR) is an index used to quantify the swelling potential of a particular soil stratum. and has been successfully used to identify the swelling potential of the soil layer underlying pavements. Table 12. It represents the simplest and most applicable method to predict the volume change in expansive soil deposits.

Table 12 Potential Heave and expansiveness, Kazi M (2017)

Classification	Potential Heave (cm)	expansion potential
problematic	>2.54 cm	High expansion potential
non-heave zones	0-2.54 cm	Low expansion potential

I.Calculation of Potential Heave from Swell Pressure: M. Aniculaesi1, I Lungu (2019).

The anticipated heave in terms of swell pressure is:

$$S_{max} = \sum_{j=1}^{n} \frac{C_{sj}}{1+e_{oj}} . \log_{10} \frac{\sigma_{sj}}{\sigma'_{fj}} . H_j$$

where

C_{sj} = swell index of stratum j,

σ_{sj} = swell pressure of stratum j , tsf (1 tsf = 95.76 kN/m^2),

σ'_{fj} = final or equilibrium average effective vertical pressure of stratum j , $\sigma_{fj} - u_{wfj}$, tsf (1 tsf = 95.76 kN/m^2),

σ_{fj} = final average total vertical pressure of stratum j , tsf (1 tsf = 95.76 kN/m^2), and

u_{wfj} = average equilibrium pore water pressure in stratum j , tsf (1 tsf = 95.76 kN/m^2).

The number of strata (n) required in the calculation is that observed within the depth of the active zone for heave.

Swell index

Vanapalli and Lu (2012) suggested that the swelling index, Cs increases with increasing plasticity, Ip, and proposed a relationship between Cs and Ip (Equation). using the published experimental results from the literature. However, the Equation is limited for Ip values lower than 65.

$$C_s = 0.019 e^{0.0343 I_p}$$

2.6.4 Effect of Topography and Drainage

Drainage features of an area are dependent on the Topography nature on which in turn have a major effect on soil mineralogy. Its control over soil properties is predominantly robust in a tropical environment reflecting the importance of the lateral movement of water and soil materials (Taylor, 1990).

Katti et al., (2002) reported that the Expansive soil deposits are formed under conditions where the slope of the terrain is less than 3%. The most frequent physiographic position of expansive soils is flat, alluvial plains.

2.7 Effect of expansive clay on road construction

Distresses in the pavement over expansive clay subgrade are observed due to different causes which can be grouped into three categories. The first is due to overloading which includes excessive gross loads, high repletion of loads, and high tire pressure. Second climatic environmental conditions may cause surface irregularities and structural weaknesses on the pavement. For example, the volume change of soil due to wetting and drying resulting from improper drainage and lack of design consideration may be the prime cause of pavement distress. The third cause may be the disintegration of the paving materials due to the method of construction and quality of construction material. The use of contaminated aggregate and inadequate construction supervisors are also factors that may aggravate pavement distress. Lack of maintenance will further aggravate pavement distress.

The research intention is more on Distress as a result of expansive subgrade soil.

✔ Climate /Durability Associated Distress: - Bleeding, Block cracking, Joint reflection cracking, Line cracking /longitudinal/ transversal/, Patching of climate/durability swell caused distress, Weathering and raveling, Shoving; Figure 5 (a).

➢ Drainage/ moisture Associated Distress: - Bumps & sags, Lane/ shoulder drop off, Depression, Swell). Except for safety considerations associated with the surface coefficient of friction, any kind of distress can be interrelated individually or collectively to rupture, distortion, and collapse modes of failure. Figure 6 (a) The distress of the road in Ethiopia consisted of longitudinal cracking of the shoulders and the asphalt in the outer portion of the roadway blamed on the foundation soils. The problem urges the need for wider application of cost-effective and environmentally friendly technologies of improving soil properties to be customized and adapted to the current road construction trend in the country. Paved roads in tropical and subtropical climates often deteriorate in different ways to those in temperate regions, because of the harsh climatic conditions, lack of proper design and quality control, high loads, and inadequate assessment for identifying causes of distresses before carrying out maintenance and rehabilitation

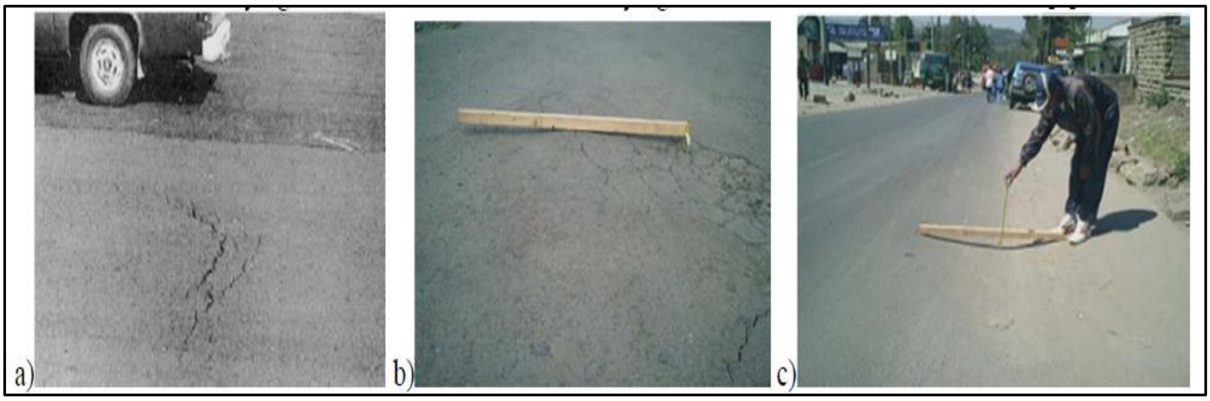


Figure 5 (a) Slippage cracks, (b) Swelling-upward displacement of a pavement, (c) Sag in asphalt pavement

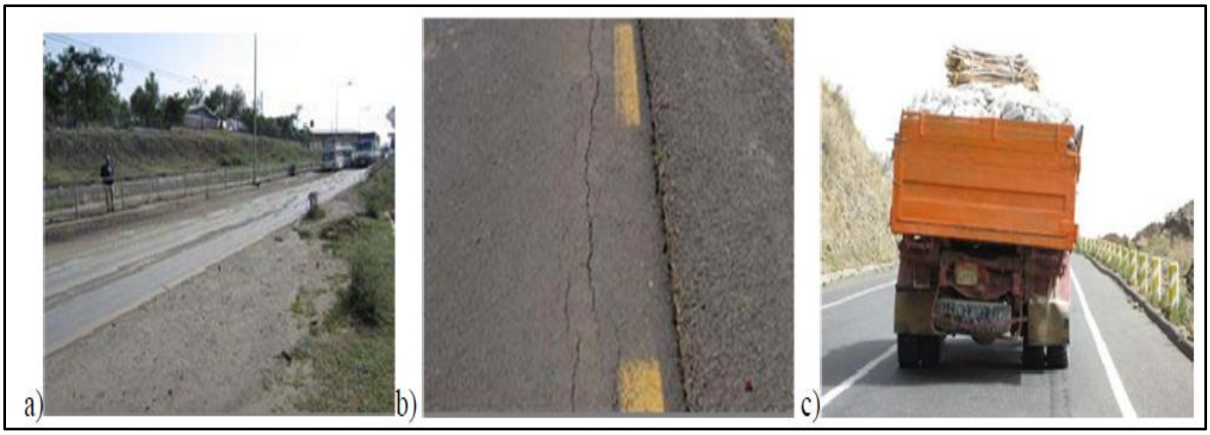


Figure 6 (a) urban trunk road under the influence of expansive soil, Rutting occurred on the surface. (b) Rural trunk road-Vertical rutting of the pavement edge. (c) Heavy vehicles hauling lime-stone have been passing in Abay. Traveling speed is lower than walking

2.8 Soil Stabilization approaches applied on expansive soils

2.8.1 Mechanical Stabilization

These techniques for soil stabilization have been applied for many years. The most widely known practices are soil compaction, dynamic, and Vibro-compaction; soil replacement and blending; and soil reinforcements and installation of barriers. The effect of replacement of expansive bentonite with blended sandy-recycled EP (expanded polystyrene) was investigated by Abdelrahman et al. (2013). The soil replacement reduced the volumetric expansion reduction of the expansive soil and increased the optimum moisture content with increasing EP beads content Soil replacement is among the most applied mechanical soil stabilization techniques.

❖ Pre-wetting

Pre-wetting of expansive soil has also been experienced over time. The main idea is to provide moisture to the expansive soil to allow the heaving to occur before the construction. However, the moisture content must be kept at a higher value to maintain the soil volumetric variation at a fixed state (Chen, 2012). Different types of expansive soil stabilization approaches are discussed and summarized in table 13.

❖ Removal & replacement

When moderately expansive soils of low thickness are present near the ground surface, they can be removed and replaced by less expansive soils and then compacted properly. A thorough investigation should be carried out to determine the thickness and extension of expansive strata before replacement.

This method is recommended for cases where the thickness of expansive soil is small, less than 2.5m. The depth at which the soil is to be replaced depends on the depth of the active zone. However, it has a disadvantage in causing failure during construction due to water ingress unless the fill is impervious, which is expansive and the thickness required may also be impractical (Nelson and Miller,1992).

According to ERA, (2014), Remove problem soils be found (and especially expansive soils with PI >30%, and swell > 2.0%), and replaced by a non-expansive material with CBR of 5-7% as presented in figure 7.

The following minimum treatment shall apply

❖ Removal of problem soil;

- Where the finished road level is designed to be less than 2m above ground level, remove the problem soil to a minimum depth of 600mm over the full width of the road.
- Where the finished road level is designed to be greater than 2m above ground level, remove the problem soil to a depth of 600mm below the ground level under the unsurfaced area of the road structure; or
- Where the expansive soil does not exceed 1m in depth, remove it to its full depth.

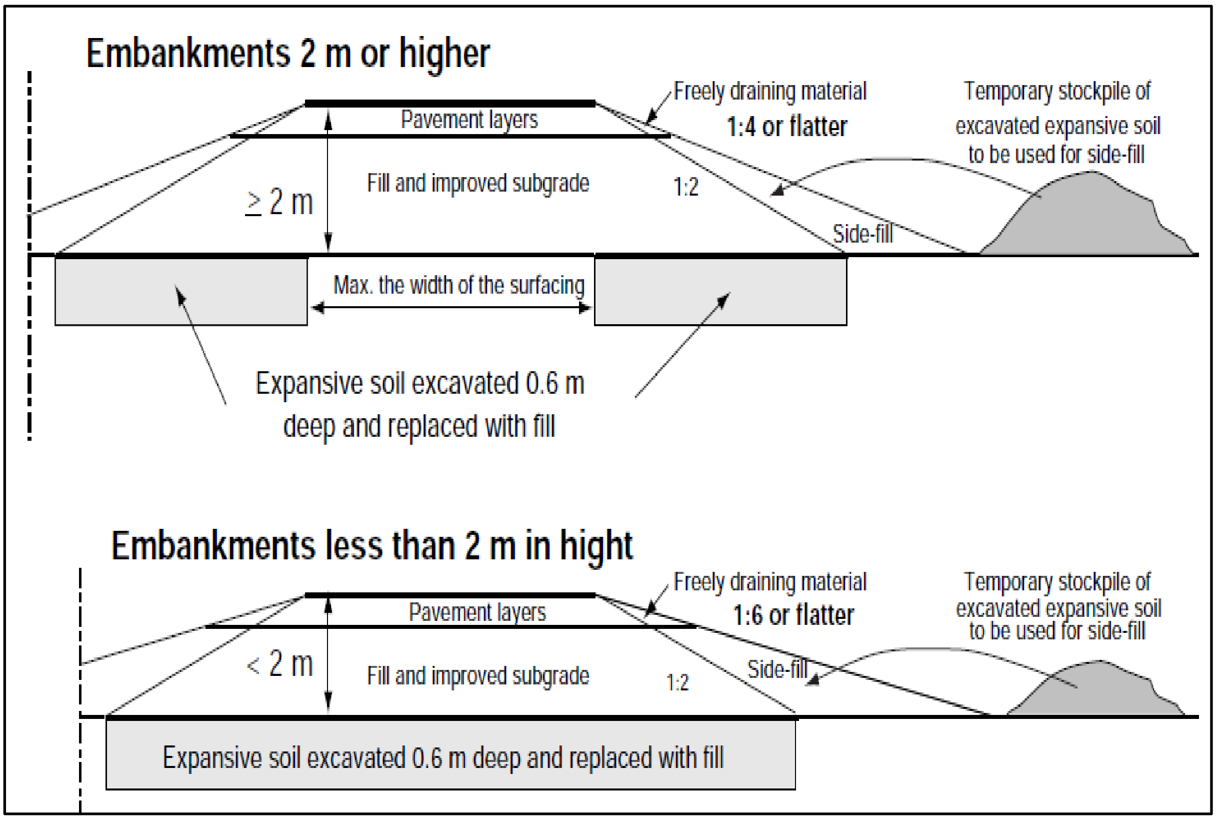


Figure 7 Cross-section and construction on expansive soil with soil replacement depth (Tanzania PDM URTMW, 1999)

2.8.2 Chemical and Additive Materials Stabilization

To improve their stability the commonly used method of soil treatment is chemical stabilization. This can be achieved by increasing the particle size of soil through a decline in the plasticity index, decreasing the shrinking-swelling potential and cementation. In chemical stabilization, soil stabilization is achieved by introducing a specific amount of a chemical compound to the expansive soil. Over the years lime, fly ash, cement, and some other chemical compounds have been successfully employed for soil stabilization (Neeraja D. and Rao Narsimha A.V., 2010). Sometimes, a more stable soil is needed to support the structural works and or as a subgrade for road works (pavements). Specifically for the pavements, the additives that are commonly used as ingredients of pavements are cemented base additives. (Petry and Little,2002) have listed some cemented stabilizing additives and divided them into traditional, by-product, and non-traditional stabilizers. The traditional stabilizers include Portland cement, fly ash, and hydrated lime while by-product stabilizers incorporate the dust from cement, lime kiln, and slag. Other additives such as potassium compounds, polymers, ammonium chloride, sulphonated oils, and enzymes are categorized as nontraditional stabilizers.

Texas Department of Transportation (2005) recommended several factors that should have to be considered before a suitable stabilizer is employed namely;

- soil classification which includes gradation and plasticity,
- soil mineralogy and content (sulphates and organics),
- Objectives of treatment,
- mechanisms of additives,
- Material properties (modulus, strength),
- Environmental conditions (water table, drainage),
- Design life, and desired engineering, and economic constraints (cost-effectiveness).

Table 13 Soil Stabilization approaches applied to expansive soils (Nelson and Miller, 1992)

Improvement approach	Outline of approach	Advantage	Disadvantage
Removal & replacement	Expansive soil removed and replaced by non-expansive fill to a depth necessary to prevent Excessive heave. Depth governed by weight is needed to prevent uplift and mitigate Differential movement. Chen (1988) suggests a minimum of 1 to 1.3m.	Non-expansive fill can achieve increased earing capacities; Simple and easy to undertake; Often quicker than Alternatives.	It is difficult to apply this approach of borrowing a source having low quantity quality and distance to the work area of replacing the material. Preferable to use impervious fill to prevent water ingress, which can be expensive. The thickness required may be impractical. Failure can occur during construction due to water Ingress.

Remolding & compaction	Less expansion was observed for soil compacted at low densities above OWCa than those at high densities and below OWC Standard compaction Methods and control can be used to achieve target densities.	Uses clay on site eliminating the cost of imported fill; Can achieve a relatively impermeable fill minimizing water ingress; swell potential reduces without introducing excess Water.	Low-density compaction maybe detrimental to bearing capacity; It may not be effective for the soil of high swell potential; Requires close and careful Quality control.
Pre-wetting or ponding	Water content increased to promote heave before Construction. Dykes or berms used to impound water in Flooded area. Alternatively trenches may be used and vertical drains can be used to also, speed infiltration of water Into soil.	Has been used successfully when soils have sufficiently high permeabilities to allow relatively quick water ingress, e.g., with fissure Clays.	It May require several years to achieve adequate wetting; Loss of strength and failure can occur; Ingress limited to a depth less than the active zone; Water redistribution can occur causing heave after Construction.
Chemical Stabilization	Lime (3 to 8% by weight) Common with cement (2 to 6% by weight) sometimes used, and salts, fly ash and organic compounds are less commonly used. Generally, lime mixed into the surface (~300mm), sealed, cured And then compacted. Lime may also be injected in slurry	All fine-grained soils can be treated by chemical Stabilization; Is effective in reducing plasticity and swell the potential of an expansive	Soil chemistry maybe detrimental to chemical treatment; Health and safety needs careful consideration as chemical stabilizers carry potential risks; Environmental risk may also occur – e.g., quick lime

	form. Lime is generally best when dealing with highly plastic clays.	Soil.	is particularly reactive; Curing inhibited in colder Temperatures.

OWC – optimum water content as determined by standard proctor test, BS 1377: 1990.

2.9 Previous study

Detailed geotechnical investigation of sub-grade soil was carried out from the period December 2019 by China Civil Engineering Construction Corporation (CCECC). including all the required field activity the essential laboratory tests are conducted on the representative soil samples including, Atterberg Limits tests carried out following (AASHTO T 89, T 90). The moisture content in the laboratory is determined following (AASHTO T 265) Standard, Compaction Characteristics (Modified proctor) carried out by (AASHTO T180) the California Bearing Ratio (CBR) carried out in accordance (AASHTO T193) Linear Shrinkage limit ASTM D4943-89 and AASHTO soil classification is done. Presented in Chapter 5 in detail.

Structural and magmatic evolution of the area was reported by Mohr, 1971 (reprint) and Pilger and Rosler (1975 & 1976) and also compiled in the geology of Ethiopia (Mengesha et al. 1996). Tamirat (1988), Gebreyohannes (1990 and 1989), and Shigut (1989) have classified the sedimentary lithologies of southeastern Ethiopia.

CHAPTER THREE

3. THE STUDY AREA

3.1 Introduction

Jijiga is the capital city of the Somali Region of Ethiopia. It became the capital of the Somali Regional State in 1995 after it was moved from Godey. Located in the Fafan Zone with 60 km (37 mil) west of the border with Somaliland, Jijiga has an estimated total population of 763,509. Settlement and lifestyle are more or less similar to the Somalis. Chat is a major plantation for cash and subsistence.

3.2 Location and Accessibility of the study area

The study area is located in the Eastern part of the country within the Somalia Region in Tuli and Dembel Woredas of Fafen and Shinile Zone. The road starts at the outskirts of Jigjiga town and heads in the northwest direction towards Fedaad, Tuli, Sedir Kebele, Lowwanja, Samekab, and Harmukale Junction (4.2km away from Harmukale Kebele). The end of the study is located about 30km away from Jijiga to the Northeast direction towards Dire Dawa-Dewele-Djibouti road. The exposed section of the study area covers about 30km in length starting from Jijiga to Tuli which is as presented in figure 8.

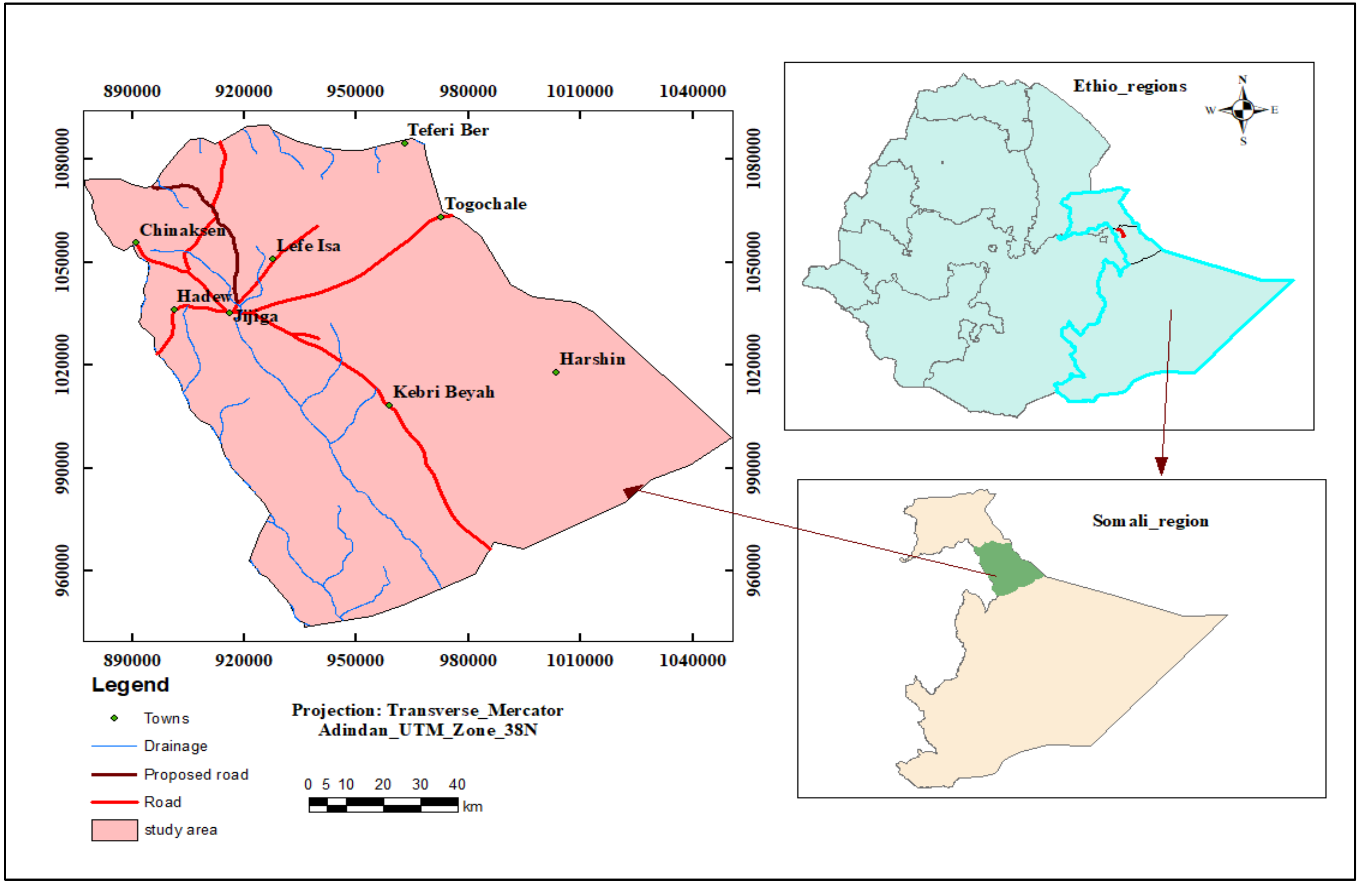


Figure 8 Location map of the study area

The study area can be accessed along the main asphalt road linking Addis Ababa both in Jigjiga and Diredawa cities through two main road namely: Addis Ababa-Harer-Jigjiga road and Addis Ababa-Diredawa-Dewel road and about 3km drive through asphalt road 2km gravel road from Jigjiga town.figure 9.

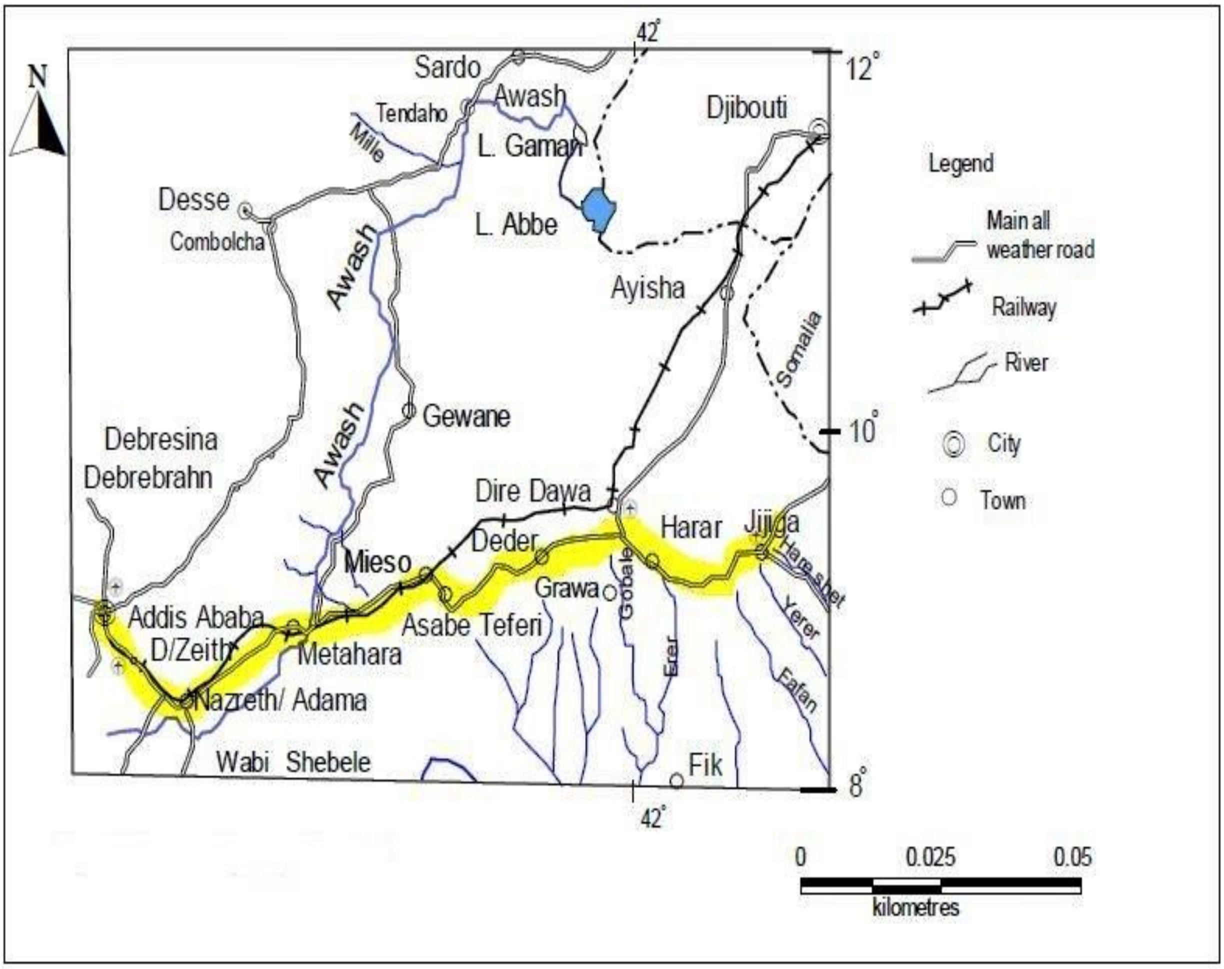


Figure 9 Accessibility map of the area

3.3 Physiography, Topography, Climate, and Geology

3.3.1 General

To investigate the study and research area well, it's imperative to understand its geologic and physiographic nature. The engineering characteristics of subgrade materials, the volume of earthworks, and the pattern of drainage are generally connected to the Geologic and Physiographic situations of the study area. Given this, the local climate condition of the area is among the essential factors that would be considered in the research.

3.3.2 Physiography

Physiographically, the area belongs to the southeastern highlands and associated lowlands of Ethiopia. In the area and its Surroundings is possible to subdivide the region into three physiographic regions. Figure 10.

Plateau areas; this extends to high rising mountain ranges and rugged hills (Karamara Mountains).

The rift escarpment; The rift escarpment is characterized by steep slopes and badland topography. Cliffs and rugged terrain are common. This is terminated at the foot slope and sharply passes to the rift plain. V-shaped streams are common with deep gorges.

The rift plains; The low lands in the rift zone form flat undulating lands ornamented by volcanic cones. Uniformly flat surfaces occur in the northwestern part where alluvial fan is developed due to subsidence at the axial ranges of the rift floor.

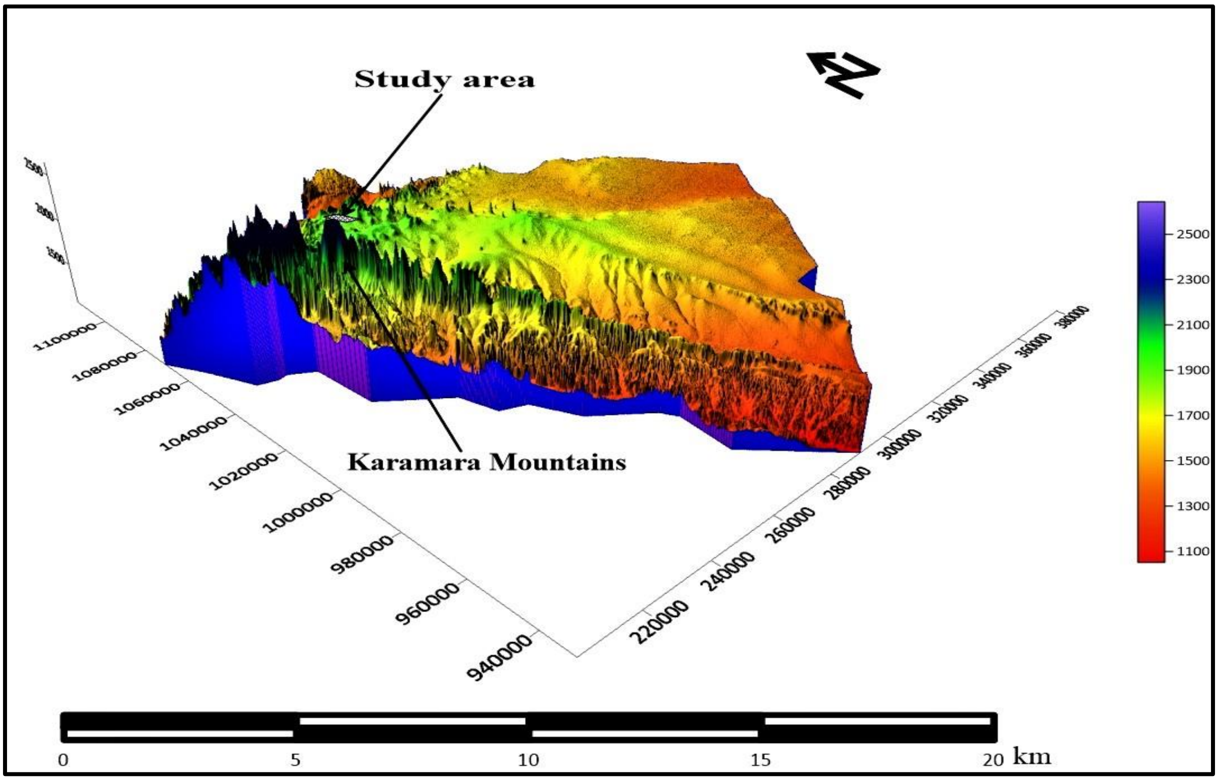


Figure 10. 3D view of the study area.

3.3.3 Topography

The topography of the study area consists of almost uniform features of land forms, the area is characterized by flat topographical features. The start of the road lies at 1656m around Jijiga and the end of the study area Tuli have1930m above sea level with an average slope of 1.7% which is less than 2% Figure 11. The ground profile of the study area for the section is considered here is from km 0+000 to km 30+000.

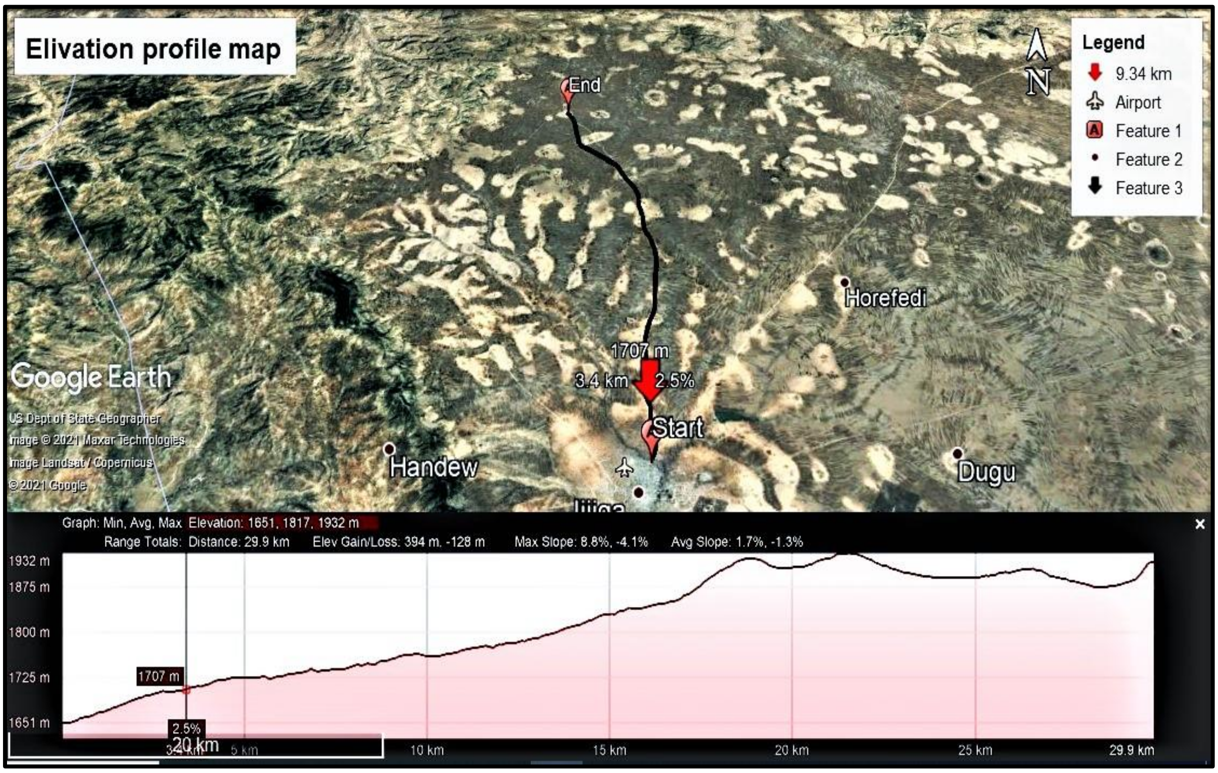


Figure 11 Ground profile of the study area

3.3.4 Climate and drainage

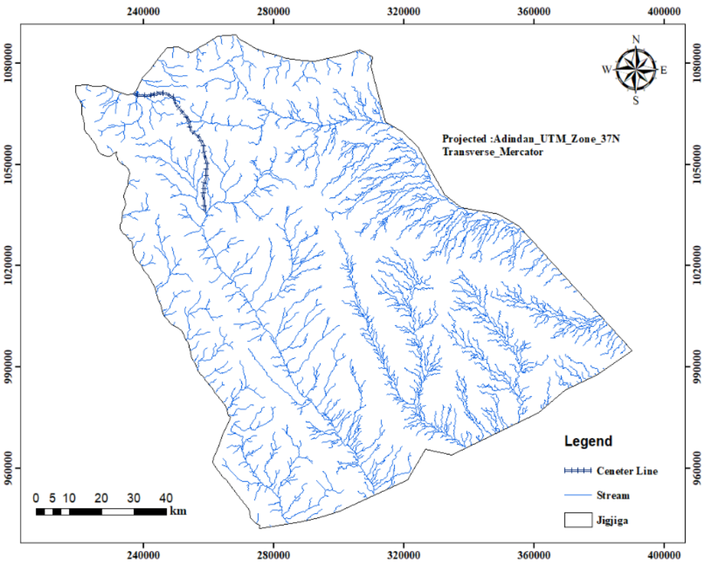


Figure 12 drainage map of the study area

Climatic conditions in Ethiopia are largely governed by altitudinal variations that are controlling rainfall distributions to some degree and temperature variation to a very large extent. Based on Mean Seasonal Precipitation and Mean Seasonal Temperature variations, three operational seasonal periods are commonly known in Ethiopia. These are named "Bega", "Belg" and "kiremt" and are occurring in October - January, February - May, and June - September, respectively. The long rainy season in the summer from May to September known locally as Kiremt, is primarily controlled by the seasonal migration of the inter-tropical convergence zone (ITCZ), which lies to the north of Ethiopia at that time.

According to the National Atlas of Ethiopia (1981), the climatic condition of the country is classified into the following climatic zones:

"Kur"; 3300m a.s.l. and above (annual mean temp. of <10^{o}C.)

"Dega"; 2300 – 3300-meter a.s.l. (annual mean temp. of 10 to 15^{o}C.)

"Weina Dega"; 1500 – 2300-meter a.s.l. (annual mean temp. of 15 to 20^{o}C.)

"Kola", 500 – 1500m a.s.l. (annual mean temp of about 30^{o}C)

"Berha", below 500-m. a.s.l. (with annual mean temperature of 30-40^{o}C.)

From the above classification, the study area falls within "Weina Dega" as the elevation of the route corridor as well as the vicinity of the study area is varying from 1656m around Jijiga and 1930m around the project end Tuli.

❖ Temperature

As per meteorological data of Ethiopia, The mean maximum monthly temperature ranges between 25 ^{0}C and 30 ^{0}C in the study area. In the Jigjiga area, the mean maximum seasonal temperature occurs in February-May (Belg) is in the order of 25 ^{0}C-35 ^{0}C. Table 14.

Mean monthly maximum temperature ranges from 25 to 28^{0}C and mean minimum monthly temperature ranges from 7.1 ^{0}C-15 ^{0}C.

Table 14 Mean Seasonal Maximum Temperature

Physiographic Region		October - January (Bega)	February - May (Belg)	June - September (Kiremt)
Jigjiga area	Maximum	20^{0}C - 25^{0}C	25^{0}C - 35^{0}C	25^{0}C - 30^{0}C
	Minimum	5^{0}C - 12^{0}C	10^{0}C - 15^{0}C	15^{0}C - 20^{0}C

Rainfall

The mean monthly rainfall of the project corridor is presented in figure 12. The mean annual rainfall reaches 600mm. Accordingly, the project area climate mainly shares a semi-Kola category with low intensive rainfall pattern during flood events.

As per the description of the Ethiopian National Meteorological Agency, the rainfall regimes of the country are classified as Mono-modal, Bi-modal Type I, Bi-modal Type II, and Diffused pattern. The rainfall regime of the project area is Bi-modal which is characterized by two peak rainfalls in March to April and a second peak around August/September July to August as presented in figure 13 and 14.

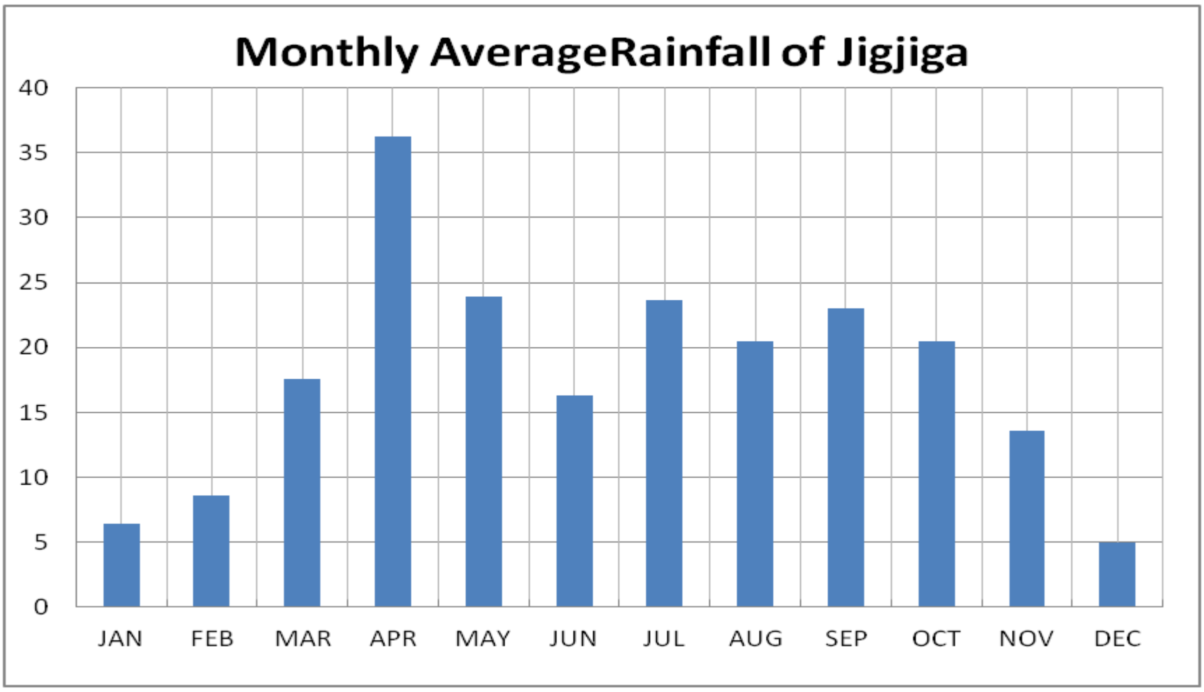


Figure 13 Monthly Rainfall pattern,2019

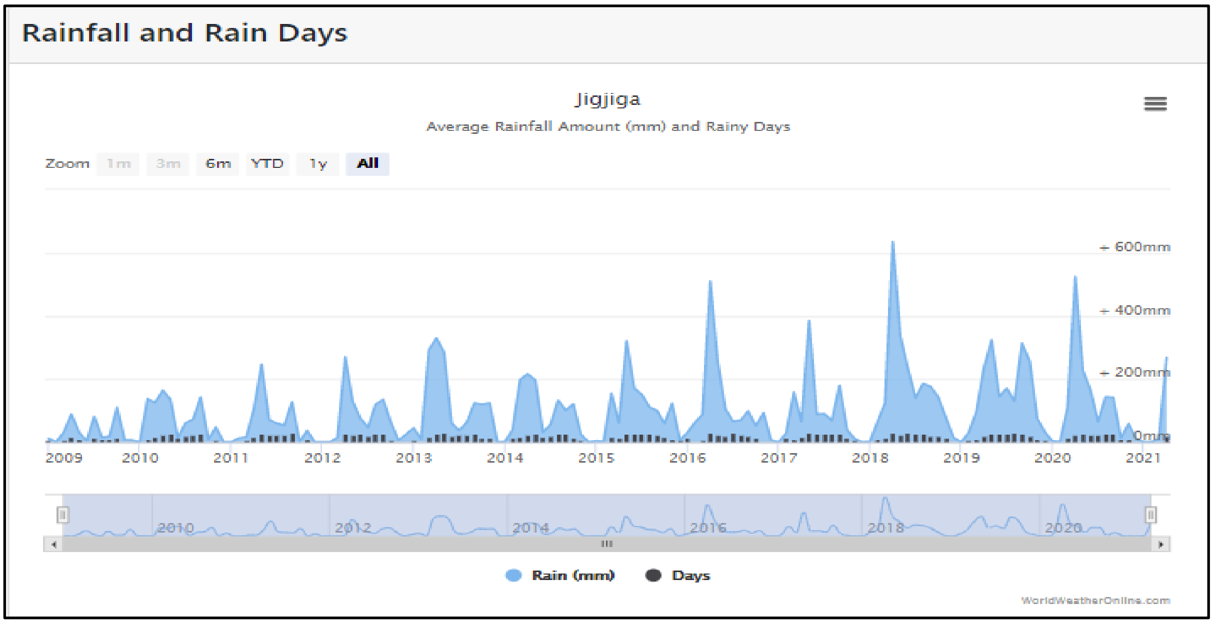


Figure 14. 13 Years of max, min, and average rainfall data (New_LocClim).

3.4 Geological Setting of The Project Area

3.4.1 Regional Geological Formations

Referring to the general 1:2,000,000 scale geological map of Ethiopia (second edition, 1996), and project report (EIGS,2010). supplemented by a specific project area i.e., 1:250,000 scale geological map of HARAR sheet, the general geology of the area has been described in detail below.

The geomorphic setting of the region along the Jigjiga-Tuli-Lowwanja-Dul’ad-Samekab-Harmukale Junction Road project is mainly controlled by internal (the tectonic) and external forces activity that took place in the past geologic time one event after the other.

The tectonic activities are associated with the formation of Mesozoic Sedimentary Rock, the Quaternary superficial and the volcanic activity in the region was the main cause of the formation of flat terrain, mountains, and separate hills in the area surrounding the route corridor. Furthermore, the external activities are associated with the formation of a valley system and develop to dissect the region into the formation of a valley through which major drainage patterns flow.

The general geology of the south-eastern low land of the country where the road project crosses is dominated by major sedimentary complexes with superficial deposits having various formational histories.

In addition to the general geology of the country, the specific geological setting of the project has been referred to from the available geological map of the HARAR sheet. Figure 13.

As presented in figure 15, the geological setting of the project area can be represented as follows;

- **Quaternary alluvium (Qel)**

This alluvium occurs both on the plateau and in the rift zones along major river valleys. It shows varied thicknesses from place to place; which range from a meter to 10 m. or up to 20 m. in some river beds. Composition varies from silty mud to sandy-conglomerates. These loose materials often show bedding and cross-bedding which are not continuous. In the rift zones, it occupies a low plain in the interfluves and is considered to be Pleistocene in age. Along with the present-day valley floors, it is washed away by channel flows of recent valley floors. Quaternary alluvium1 covers a total area of 857 sq. km (5.15%). Quaternary alluvium (Qal1) forms relatively wide valley plains and at places even altered to new soil generation at their top part. Alluvial plain soil is mixed with scattered gravels and pebbles, moved from the surrounding uplands. Such deposits are assumed to be older than Holocene and are assumed to be Pleistocene pluvial deposits.

- **Lower limestone (Mllst)**

Jurassic limestones are widely exposed in many areas extending from the western to the eastern part of the map. The limestones are down-fault and form part of the Afar Rift in the northern part of the area. The lower limestone covers an area of 2704 sq. km (16.2%). These lower limestones on the plateau occupy many areas from western, southern & eastern & extend to Karamara ridge, and also form the plateau of Jijiga, Chinaksen and Sheder areas. In many of its exposures, the limestone forms mountain chains and isolated hills. This unit is variably thick, and horizontally bedded, but in the rift areas, tilted beds are common due to listric movement during the fault. The bottom section of the limestone succession is marked by sandy limestone grading upward to calcareous sandstone and then to marl limestone interbedded with calcilutite.

- **Granodioritic gneiss (Pgdgn)**

The granodioritic gneiss is exposed in south-central and southwestern parts of the map area. It is exposed in Abdi Buchi, Awubare, and Bishan Babile localities. In this unit dominant lithology is granodioritic gneiss, but also occur minor granitic gneiss, amphibolite, and gabbroic bodies. The granodioritic gneiss covers an area of 398 sq. km (2.4%).

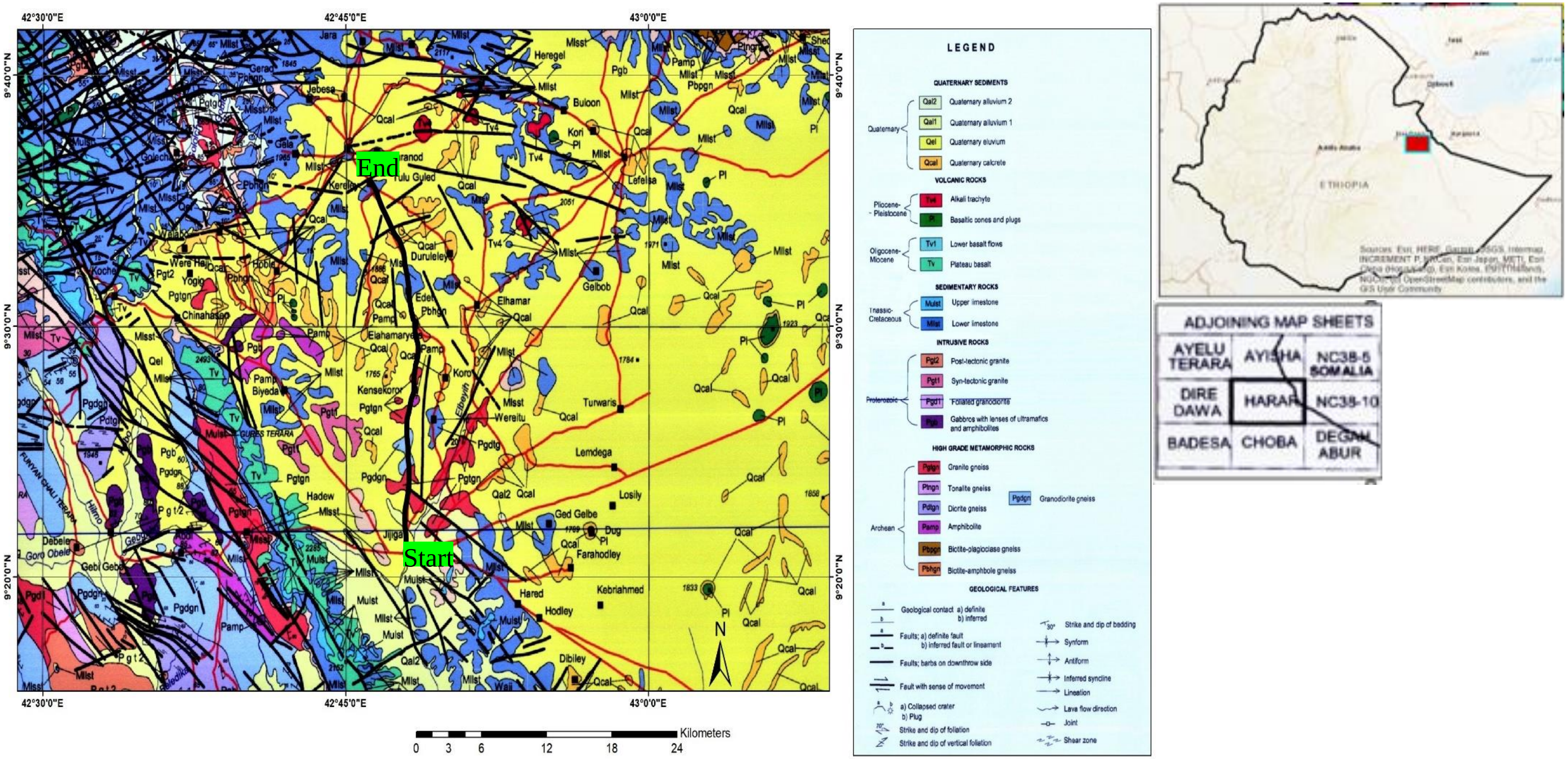
Granodioritic gneiss is dark grey, medium-grained, and shows pronounced compositional banding of dark (mafic) and lighter minerals. The mafic minerals are biotite and amphibole, while the lighter ones are quartz and feldspars. They define composite foliation up to 5mm in width. Feldspar megacrysts occur along foliation planes.

- **Granite gneiss (pgtgn)**

This unit is exposed mainly in the northeastern part of the Harar map sheet, northeast of Lafa Isa and central parts of Jijiga, west of Deweronache, and on the Sewu mountains. The granite gneiss covers a total area of 368 sq. km (2.2%).

The granitic gneiss form rock blocks of big boulders, with the rugged terrain of erosion-resistant hills. Compositional banding is characteristic of this rock at places massive, and in medium-coarse grain. As it is generally stated in the above section, the region/general geology of the project area is part of the south-eastern of the rift valley and the flank of the rift system. The regional geological constituents are those Sedimentary and volcanic complexes.

Geological Map of Study Area Modified from 1:250,000 Harar Map



Project Road Jijiga to Tulli

Figure 15 Geological map of the study area.

3.4.2 Local Geology of the research area

The geology of the project road was assessed during the field trip and data collected during the soil and material investigations were also assessed. The geological map of the HARAR area, with a scale of 1:250,000, was also referred to. The Engineering geological units that are crossed by the alignment are also studied regarding their suitability for sub-grade material furthermore their engineering physical property is also observed visually.

According to the geological map and field observation, the project corridor is dominantly covered by a sedimentary rock that is overlined by thick expansive clay deposits. These are described in the following sections.

- **Expansive Soil: -** soils in the study area have, dark grey color over a flat topography, and the thickness of expansive soil in the area ranges from 1.5m to 3m and Thick expansive soil formations were found around km1+100-km3+000, km7+000-km10+000, km11+500-km15+502, km 25+000-km28+000, and km28+500-km29+500.figure 16.
- **Limestone**: - this geologic material is dominantly observed interchangeably with expansive clay as a surface outcrop. the geologic material is moderately affected by weathering. The study area is flat in nature and types of structures are not observed in the area.

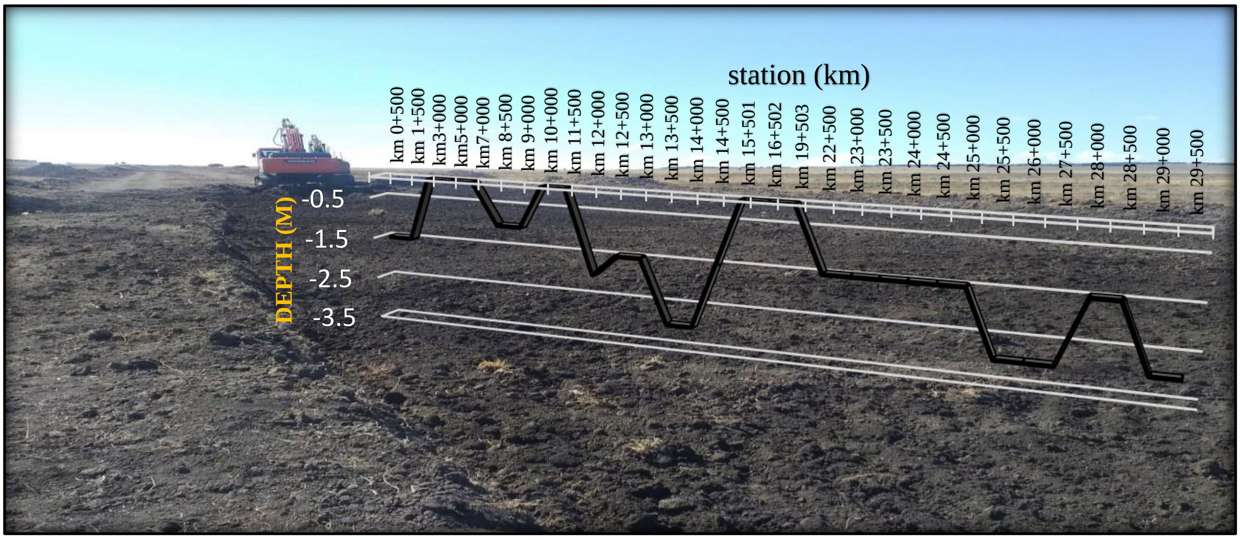


Figure 16 Thickness of expansive clay of the study area

3.5 Geological structure of the area

3.5.1 Folds

Folds in the map area are formed by compressional deformation with penetrative planar structure found in high-grade gneisses. Most of the time closure of the fold/ plunge is lacking or obliterated by complex tectonic episodes or later shearing. Mostly the gneissic layering is folded or intrafolially folded.

Meso-scale migmatite folds and shortening due to compression or ptygmatic folding of pegmatites or dikes of orthogneisses are common within high-grade rocks. Inferred folds are observed in some areas, for instance, west of Wubale in biotite-plagioclase gneiss was assumed closure/ plunge strikes 70-75º ENE and dip of 70º SW. Granitic gneisses along with amphibolites are also intrafolially folded. adopted from project report (EIGS,2010).

3.5.2 Mesozoic Structures

The Mesozoic structures are mainly primary depositional structures. The Mesozoic Sea was formed as a result of the down warping of the Horn of Africa. The lower sandstone, the lower limestones, upper sandstones and the limestone-sandstone interbeds are generally horizontal to sub-horizontal. Bed thickness varies from fine lamination to thickly bedded. Large cross-bedding is restricted to the bottom part of the lower sandstone bed. Many of the beds of the lower sandstone show medium cross-bedding or cross-lamination. Such structures are also common within the upper sandstone.

The limestones are mainly horizontally bedded and vary from thin lamination to thickly bedded units. The faults in the Mesozoic are restricted to those producing ocean floor spreading and no recognized faults are affecting the sediments of this time. Cretaceous uplift caused the marine transgression from the Horn of Africa. adopted from project report (EIGS,2010).

3.5.3 Cenozoic structures

Older Cenozoic (Oligocene) structures recorded in the area are the faults producing lava flows on the ridges extending from Karamara to Goro areas. This fault belt is known to be the Marda Fault Belt (Gouin & Mohr 1964 (in Merla et al 1979); Kazmin 1972, Black et al., 1974; Merla et al., 1979; Senbeto et al., 1981; Mengesha et al. 1996). The Marda Fault Belt has an NNW-SSE trend and passes southwest of Jijiga.

On the plateau area, east of Jijiga minor volcanic cones and plugs are related to deep-seated faults in line with the Red Sea trend and were formed during Plio-Pleistocene. adopted from project report (EIGS,2010).

CHAPTER FOUR

4. RESEARCH METHODOLOGY

4.1 Introduction

This chapter deals with the methods and approaches adopted for the accomplishment of the general and specific objectives of the study where the research approach, the research design, sample and sampling techniques, source of data collection, data collection instruments, laboratory investigations, analytical calculations, and the method of data analysis are discussed. The research method that was used for this study is the qualitative method combined with the quantitative method. The subject of the study focusing on the characteristics and swelling pressure of expansive soil.

As such, the sample collection will then be based on the method of the research being done and will also be utilizing a combination of both amounts of swell pressure of laboratory results (active zone taken from the literature) as with the calculated surcharge.

4.2 Data Source and Data Type

To accomplish and fulfill this research both primary and secondary data are collected.

Primary data; this data was collected from the actual study area by direct participation of the researcher, thus representative data were collected. this will indicate the contribution and scientific view of the researcher with the following approaches:

- Site reconnaissance visit, Observations of local landforms, land cover, lithological attributes (color, grain size, and composition).
- Visualization of natural features and phenomena prevalent to each of the distress sites.

Subsurface exploration through test pit and representative samples are collected.

4.2.1 Test pit

To obtain primary data five (5) test pit locations were selected based on the findings of a geotechnical report of the study area. figure 17.

The pits were dug from the route alignment of the newly proposed road corridor by excavator and manpower; logging and pictures were taken, and presented in appendix-I, Gps coordinate of test points presented in table 15.

Table 15 Coordinate of test point location

Test points	Station	X Coordinate	Y Coordinate	Elevation
Tp-1	Km5+000	258185.01	1040824.01	1720
Tp-2	Km8+000	258403.88	1043796.30	1740
Tp-3	Km13+000	259226.80	1048672.88	1782
Tp-4	Km24+000	256178.25	1058774.41	1884
Tp-5	Km28+000	254004.12	1061784.82	1872

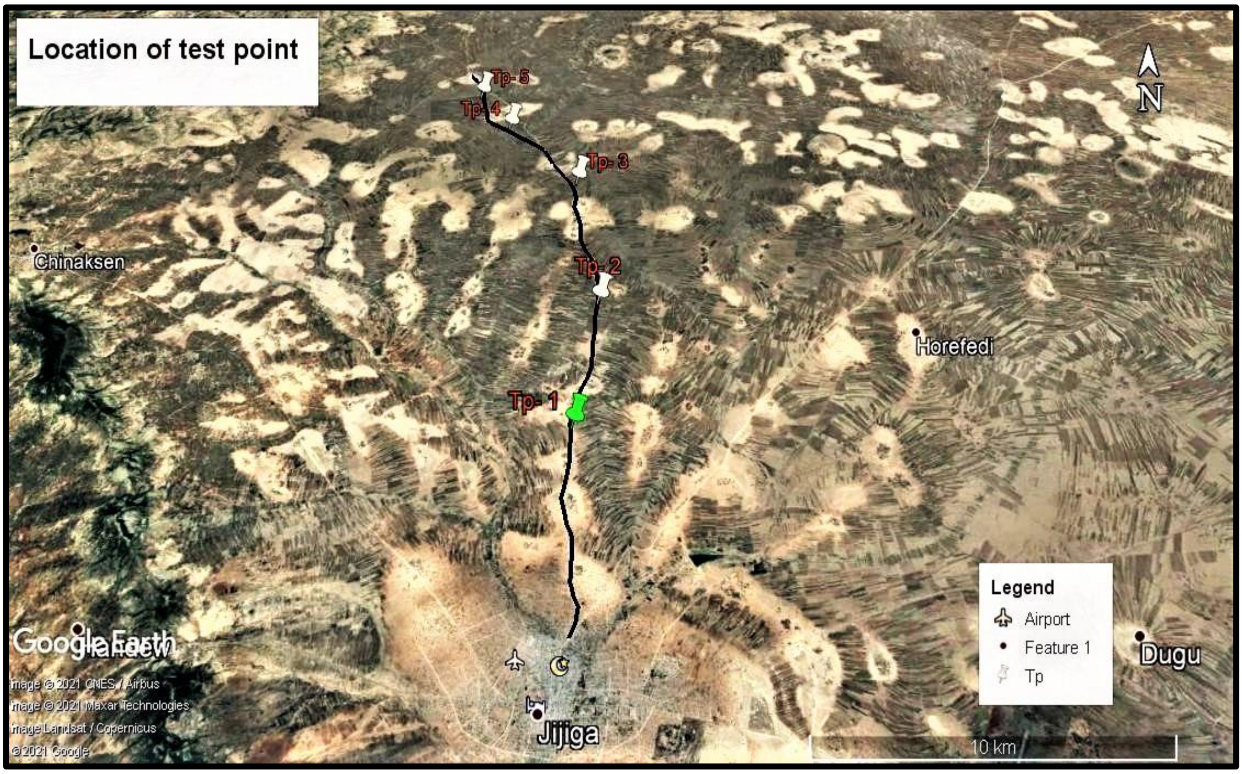


Figure 17 Test pit locations

4.2.2 Sample type

both disturbed and undisturbed samples were collected for further laboratory investigation.

4.2.3 Sample size

The choice of sample size is made after considering practical issues, and the availability of resources. The larger the sampling size of the research, the more accurate the data generated. To examine the swelling pressure of the study area, two representative undisturbed soil samples were collected from each pit and total of 10 samples were collected and 30 disturbed samples are collected from secondary source for characterizing the index property of expansive soil.

4.2.4 Sampling Material and Method

The development of collecting representative samples was by a standard Shelvey tube for the undisturbed soil samples and suitable sample bags for disturbed soil samples are used and proper method of undertaking the sample adopted, all collected samples carefully wrap with plastic sample-bag and plastered.

4.2.5 Sampling Techniques

A systematic selection of sampling locations by reviewing the profile section of the previous project purpose study. Following the route corridor, the research area is first divided into five mutually inclusive groups which infer thick expansive clay deposits having significant lateral extensions that are relevant, appropriate, and meaningful in the context of the study.

Secondary data; the secondary data was collected from different organizations (geological information including a map from Ethiopian geological survey, climatic data from Ethiopian meteorological agency and New_LocClim, detailed geotechnical and pavement report from China Communications Constructions Company.

4.2.6 Test Pitting and Sub- Grade Soil Sampling

Test pits were excavated along the left and right side of the centerline of the proposed road alignment at intervals of 500m (unless there is a soil variation within a short interval) and representative samples were collected for various tests and transported to ICT ENGINEERING PRIVATE LIMITED COMPANY in Addis Ababa. A total of 30 test pits have been excavated to a minimum depth of 150cm unless the material is firm. The pitting operation is conducted at the right and left edge alternatively along the existing gravel road and at the center of virgin ground. Representative samples were collected from the full depth of each of the test pits in which the vertical soil profile was seen to exhibit a reasonable degree of uniformity.

CHAPTER FIVE

5. SUB GRADE SOIL CHARACTERIZATION

5.1 Previous Sub-grade soil investigation

5.1.1 General

field investigation of sub-grade soil was carried out by China Civil Engineering Construction Corporation (CCECC) from the period December 2019. A total of 30 sub-grade samples were collected and tested. The fieldwork was carried out as part of the detailed field investigation and included the following activities:

• Visual identification of alignment soil.

• Test pitting and sampling of sub-grade soil.

• Locating and sampling potential construction material sources for embankment, pavement layers, and concrete works.

5.1.2 Visual Condition Survey

The activities made during the investigation included:

- Identifying the types of surface distresses and measuring their extent.
- Visual Sub-grade Soil Extension Survey along with the Road Project

5.1.3 Laboratory Investigation of Sub-Grade soil

A total of 30 sub-grade samples were collected and tested. The type of Tests conducted is presented in table 16.

Table 16 Type of Test conducted

Test Description	**Standard**
Grain size analysis	AASHTO T89-wet
Atterberg limit	AASHTO T89/ T90
Moisture Content	AASHTO T265
Compaction Characteristics (Modified proctor)	AASHTO T180
3- point CBR (at 100% of standard energy, 4 days soaking with 4.5kg surcharge for standard case, at 95% when modified energy is used	AASHTO T193
Linear Shrinkage limit	ASTM D4943-89)
AASHTO classification	

5.1.4 Sub-Grade Soil classification

The laboratory test result of Subgrade soils is classified under both AASHTO soil classification and USCS Soil classification system. AASHTO soil classification

the sub-grade soil along the study area is mainly classified under A-7-5 and A-7-6, almost all of the sub-grade soil samples were found to have Clayey Soil. presented in figure 18.

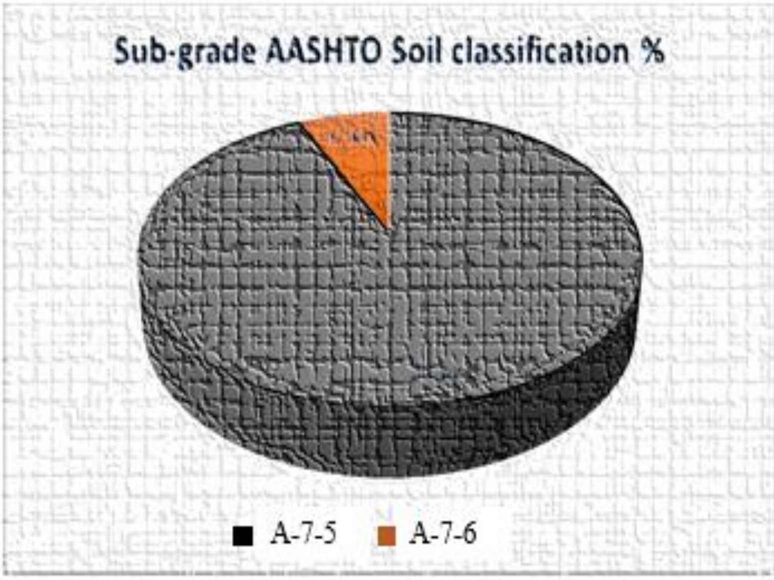


Figure 18 Percentage of different soils types as per AASHTO soil classification

I. USCS soil classification

The test result indicates the liquid limits of all samples are greater than 50 and their plasticity index ranges from 27 to 35. As presented in figure 19. Therefore, as per the unified soil classification system, the soils of the study area are grouped as MH or OH (inorganic/organic silts of high plasticity respectively) and CH or OH (inorganic/organic clay of high plasticity respectively).

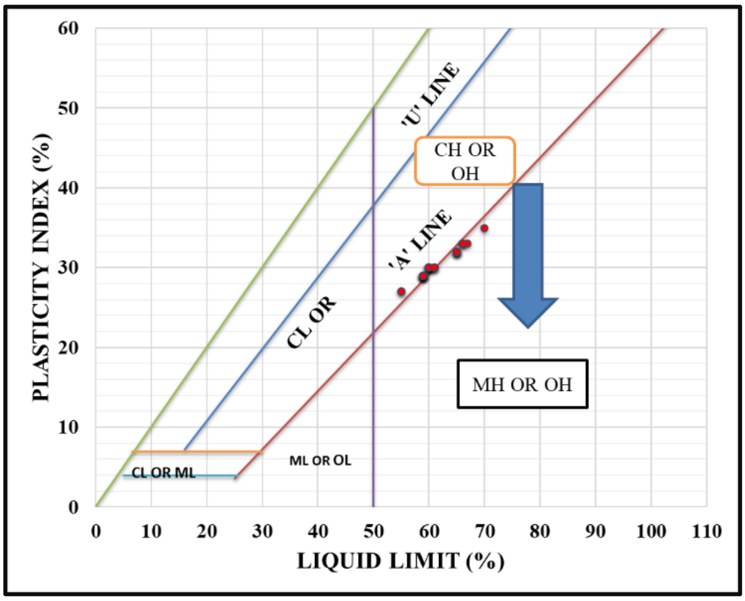


Figure 19 Plasticity charet of subgrade material

5.1.5 Soil Consistency Limit

I.Subgrade Soils Plasticity Index (PI)

Atterberg limit tests were conducted on all samples collected from the test pits to determine their plasticity behavior. The PI values generally range from 27 to 35% figure 20.

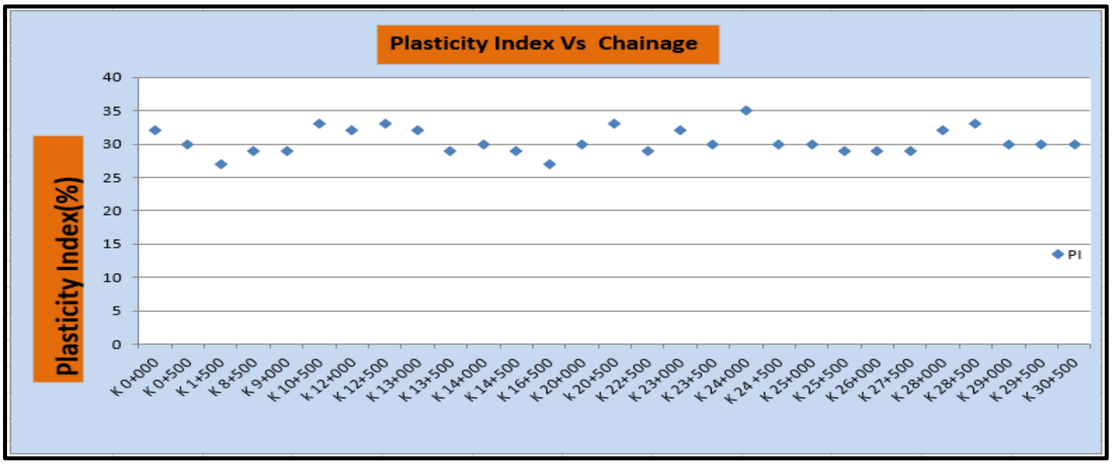


Figure 20 Plasticity index chart

The Modified IP' considers the calculated 'Modified Plasticity Index' (IP') of the study area and also the whole sample are evaluated based on the given formula.figure 21.

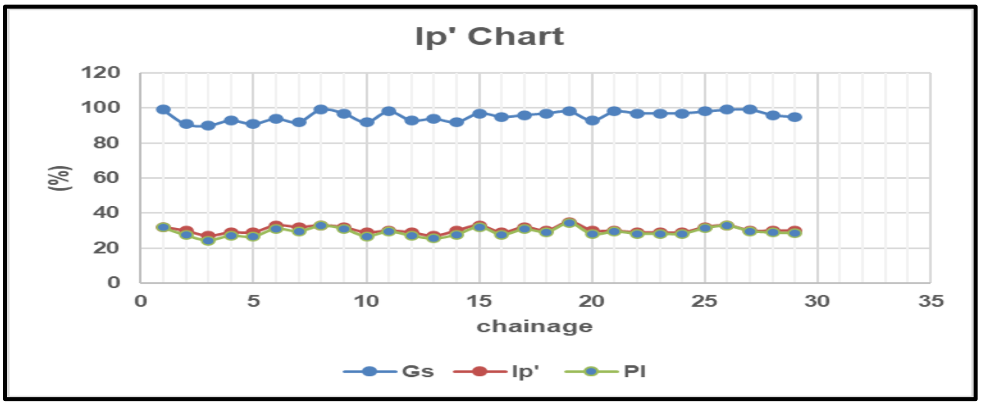


Figure 21 Calculated 'Modified Plasticity Index' (IP') of the study area

II.Subgrade Soils Liquid limit (LL)

The liquid limit of the subgrade samples of the study area is in the range of 55% to 70% with an average of 62.5%. Consistent with the PI distribution figure 22.

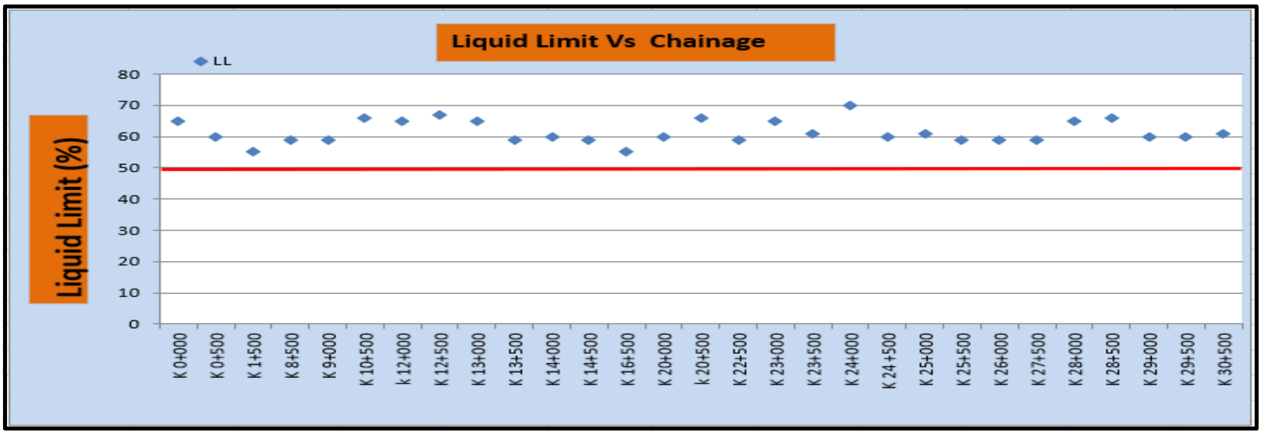


Figure 22 Sub-grade Liquid Limit Variation along the study area

III.Free swell

The free swell test results distributed along the road alignment of the study area are presented in figure 23, where the value generally ranges from 52 to 65.

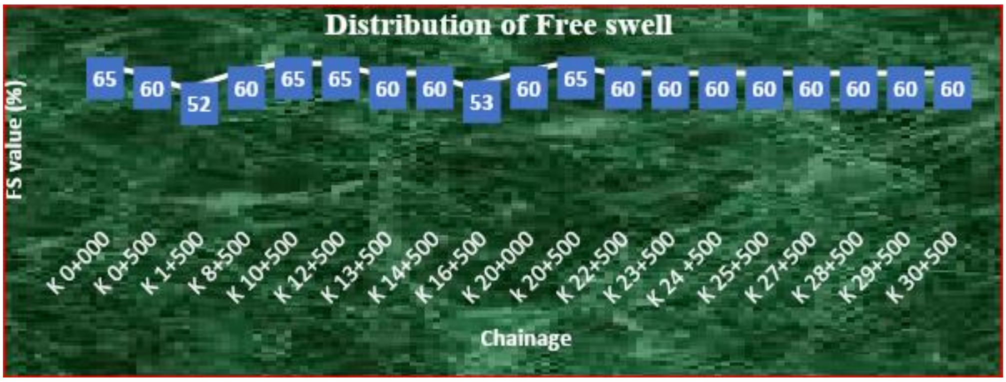


Figure 23 Sub-grade free swell distribution along the study area

IV.CBR and CBR Swell of Sub-grade Soil

In addition to the above parameters, the in-situ sub-grade soil has been subjected to compaction and the corresponding CBR tests to evaluate the bearing resistance of the same and corresponding potential swell of materials. The CBR values at 95% of MDD have been presented in figure 24.

CBR values are in the range of 2.04 % to 7.61 % and swelling potential of 1.6% to 3.8%. The project area has only 16.6% of CBR value (>5) figure 22. The project subgrade in the section considered almost 75% have a Swell value less than (<3%) and about 89% of the swell values are greater than 2%. Figure 24.

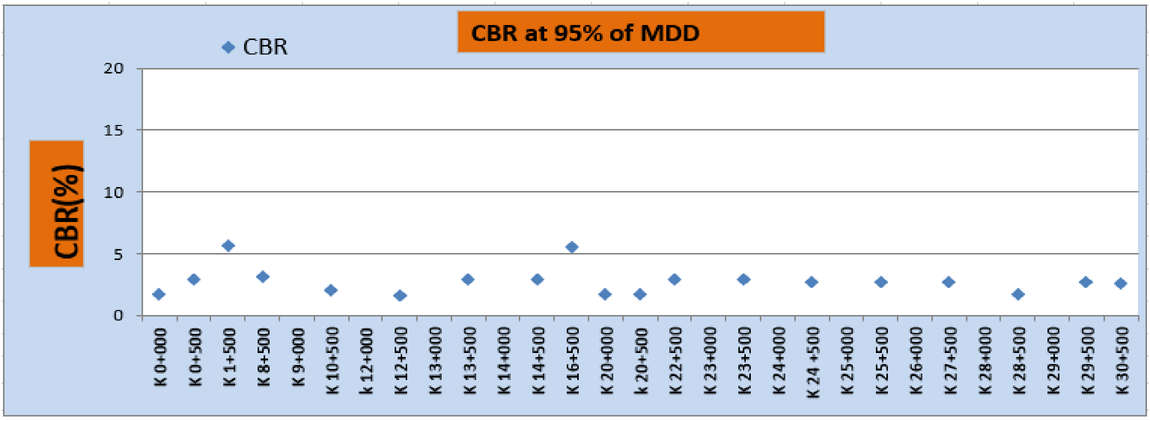


Figure 24 CBR chart of the study area

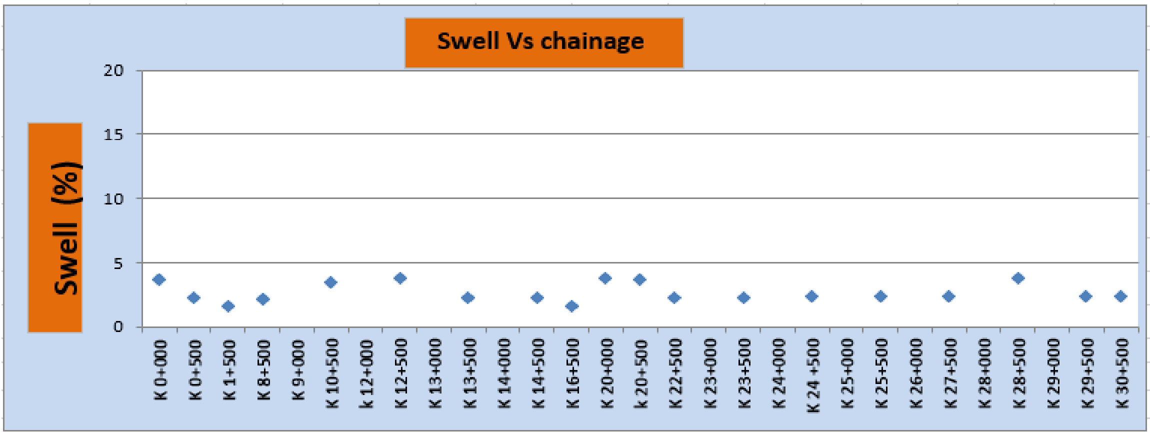


Figure 25 Swell chart of the study area

5.1.6 Identified expansive sections (ERA,2002).

Extended investigations are required when routine tests show signs of potential expansiveness. These include linear shrinkage tests of selected samples, calculation of expansiveness from the given formula, and classification into low, medium, and high. Table 17.

And the result also elistrated under figure 26.

Table 17 summarizes prone areas having a high degree of expansiveness

Test pit No.	Station (Km)	Depth (cm)	% pass0.425 mm	PI w (%)	W p (%)	Ws (%)	SL (%)	Swell (%)	CBR at 95% of MDD	Expansiveness (Ɛx)	Degree of Expansion
TP -2	km 0+500	0.00-1.00	91	27	27.3	11.8	13	2.3	2.9	51.9	High
TP -3	km 1+500	0.30-0.70	95	24	26.6	11.4	12	1.6	5.7	51.9	High

TP-4	km 8+500	0.00-1.00	93	27	27.0	12.1	13	2.2	3.1	50.1	High
TP-5	km 9+000	0.00-1.00	91	26	27.3	11.8	13			51.9	High
TP-8	km 12+000	0.00-1.90	97	29	31.0	14.6	15			50.3	High
TP-9	km 12+500	0.00-1.50	99	33	33.7	15.8	16	3.8	1.6	51.5	High
TP-11	km 13+000	0.00-1.20	97	31	32.0	14.6	15			52.6	High
TP-12	km 13+500	0.00-3.00	92	27	27.6	12.0	13	2.3	3.0	52.1	High
TP-13	km 14+000	0.00-3.00	99	29	30.2	13.9	14			50.9	High
TP-14	km 14+500	0.00-1.30	93	27	27.0	12.1	13	2.3	2.9	50.1	High
TP-16	km 16+500	1.60-3.00	94	25	26.3	11.3	12	1.6	5.5	51.7	High
TP-21	km 22+500	0.00-1.00	95	28	27.6	12.4	13	2.3	3.0	50.5	High
TP-23	km 23+000	0.00-0.90	96	31	30.7	14.4	15			50.1	High
TP-25	km 23+500	0.00-1.10	98	29	30.4	13.7	14	2.3	3.0	51.9	High
TP-27	km 24+000	0.00-1.50	99	35	34.7	15.8	16			53.9	High
TP-28	km 24+500	0.00-1.20	96	29	29.8	13.4	14	2.4	2.7	51.5	High
TP-29	km 25+000	0.00-1.30	98	29	30.4	13.7	14			51.9	High
TP-30	km 25+500	0.00-3.00	97	28	28.1	12.6	13	2.3	2.7	50.8	High
TP-31	km 26+000	0.00-3.00	97	28	28.1	12.6	13			50.8	High
TP-31	km 27+500	0.00-3.00	97	28	28.1	12.6	13	2.3	2.8	50.8	High

TP -32	km 28+000	0.00-1.20	98	31	31.4	14.7	15			50.4	High
TP -34	km 28+500	0.00-1.00	99	33	32.7	14.9	15	3.8	1.7	53.0	High
TP -36	km 29+000	0.50-3.00	99	30	29.7	12.9	13			53.6	High
TP -37	km 29+500	0.60-3.00	98	29	30.4	13.7	14	2.3	2.7	51.9	High

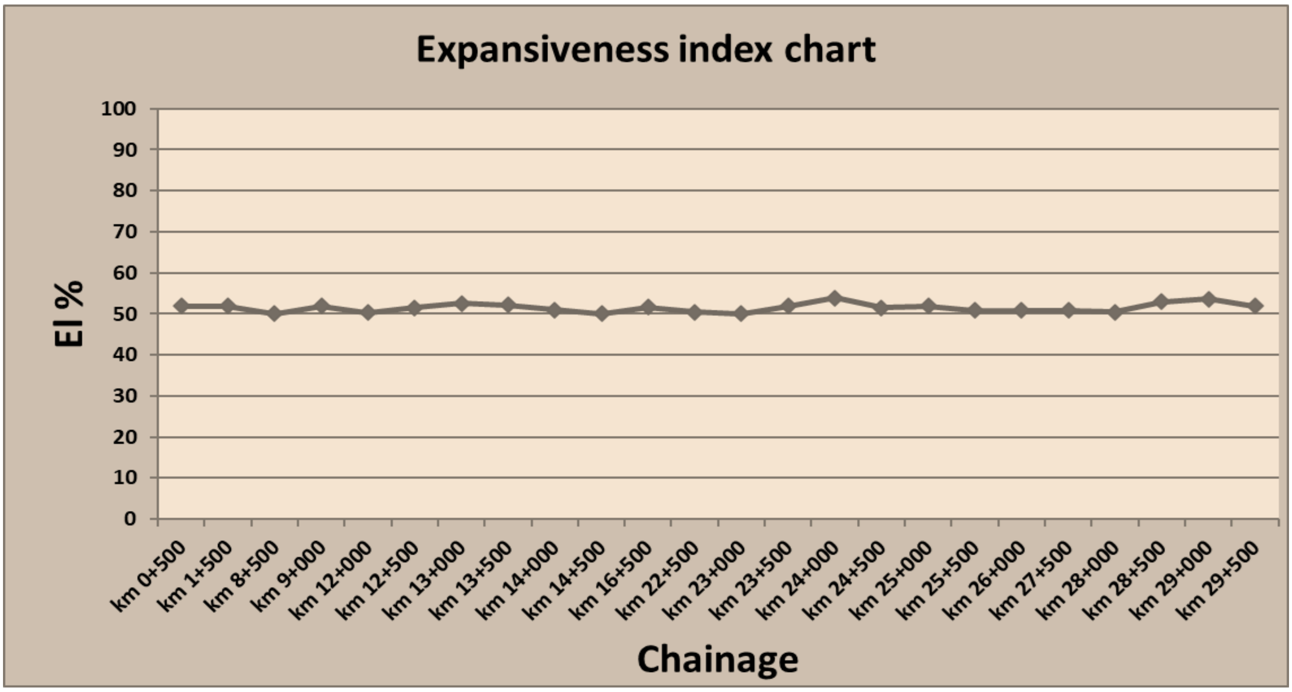


Figure 26 Expansiveness index chart of the study area

CHAPTER SIX

6 RESULTS AND DISCUSSION

6.1 Introduction

This chapter is all about the finding of the research. The results of the study are presented and discussed in detail. 1), characterization of expansive clay from the index property concerning its swell potential.2), the result of laboratory tests (Swelling pressure (Kpa) and intensity of its swell potential, Treatment Recommended on Expansive Soils and Surcharge load as per Final Recommended Pavement Structures is analyzed.

6.2 Index property of the soil

Grain size

Subgrade soils characterized both on (AASHTO) and (USCS) soil classification, the soil falls A-7-5, A-7-6, (susceptible to volumetric change, fair to poor subgrade engineering property) and fat clays (CH) (swells excessively), elastic silts (MH) (indicate expansive properties), mineralogically (illite and kaolinite) respectively.

Liquid Limit

The liquid limit of the subgrade samples is in the range of 55% to 70% with an average of 62.5%. Consistent with the PI distribution, it indicates **High swell potential**.

Plasticity index

The plasticity index of the study area ranges from 27%-35% with an average of 31% and it directs **high swell potential**.

linear shrinkage

The linear shrinkage of the study area ranges generally have a value greater than ten (10) and directing the degree of expansion is **critical**.

Free swell

The free swell of the study area is generally greater than 50% which indicates the area has **medium** swell potential.

Climate and drainage

the study area falls within “Weina Dega” The mean annual rainfall reaches 600mm. the project area climate have low intensive rainfall with unprotected drainage pattern.

6.3 Direct methods

6.3.1 Swelling pressure (Kpa)

The methods coming under this category measure the swell potential of soil directly. Oedometer Tests: According to Winterkorn and Fang (1986), the most useful and reliable assessment of the swell potential of soil could be obtained from the conventional oedometer swell tests. Soils with a liquid limit of less than 35 percent and plasticity index of less than 12 percent have a relatively low potential for swell and may not require swell testing, (EM 1110-1-1904, 1990).

One-dimensional swell tests performed on the collected 10 undisturbed soil samples. The result of test swelling pressure (oedometer) value of the study area ranges from 88Kpa -95Kpa on the top (0.6m) sample and 88.kpa – 95kpa bottom layer of the area with an average value of **91** and **91** respectively. Table 18.

Table 18 Laboratory test Result of collected soil samples

station (km)	swell pressure of clay(kpa)	
	Top (60cm)	Bottom (120cm)
km0+00-6+500	92	89
km6+500-12+500	89	88
km12+500-18+500	88	94
km18+500-24+00	88	89
km24+00-30+000	95	95

The value obtained from Oedometer Tests is within the range of (88-95) kpa, thus swelling pressure value indicates the soil exabit **Medium Swell potential**.

6.3.2 Depth of Active Zone

The fact that the groundwater table is deep in the area, the depth of the active zone is governed by climatic changes rather than the water table depth.

Concerning climatic conditions and due geologic aspects, the anticipated depth of the active zone ranges between 1.5 m and 3.0 m, below which moisture fluctuation was found to be small.

6.3.3 Calculation of Potential Heave from Swell Pressure:

Nb. In this case the strata are homogeneous n=1

Given

$^{\delta}$'fj = $^{\delta}$fj-µwf were µwf = 0 the groundwater table is dip

$^{\delta}$'fj = $^{\delta}$fj

=Yb * H Yb= Ydry*(1+w%)

Yb =bulk unit weight

H = thickness of the soil

Ydry = dry unit weight

W% = water content

Table 19 Average Potential heave (mm) of the study areas

Station (km)	Swell Index (csj)	$^{\delta}$'fj or $^{\delta}$fj (kpa)	e_o	Swell Pressure (kpa)	Smax (mm)
0+00-6+500	0.052	56.01	0.77	91	18.7
6+500-12+500	0.058	56.1	0.77	89	20.1
12+500-18+500	0.05	56.2	0.77	91	18.1
18+500-24+00	0.053	56	0.77	89	18
24+00-30+000	0.053	54	0.77	96	22

The calculated potential heave value ranges 18mm-22mm which is less than the maximum threshold value (24mm) a soil can exabit potential problem, thus the study area has low heave potential.

6.4 Treatment Recommended on Expansive Soils

6.4.1 Treatment of Road Sections with Expansive Soils

As discussed earlier, the sub-grade soil is found to be fair to the poor subgrade. All stretches of the road section are also covered by unsuitable soil of medium to high expansiveness.

As per the project aimed, expansive soils are recommended to be removed to a depth of 600mm and replaced with suitable non-swelling material having a minimum CBR value of 7%. Replacement over the full formation width of the pavement is required when the road height (embankment + Pavement) is less than 2m. Where the road fill height is more than 2m (embankment +pavement), replacement of the natural subgrade is required for the outer parts of the pavement at both sides of the shoulder

Weak soils with CBR values below 5 (mainly CBR of 2 and 3) have been observed in parts of the subgrade layer. Weak soils that were ever encountered below the sub-base bottom shall also be treated by a replacement method. The weak soils are also found to be expansive in the section considered. The top 600mm of weak soil encountered below the bottom of the sub-base shall be replaced with suitable, non-swelling material.

6.4.2 Final Recommended Pavement Structures

The recommended pavement structure type and thickness for the road section considered from k0+000 to k30+500 is presented in Table (20).

Table 20 Final Pavement Structures Provided

Pavement Composition						
From, km	To, km	Subgrade	Surfacing(mm)	Base Course GB1(mm)	Granular Sub Base(mm)	Capping Layer(mm)
0+000	30+500	S3	50 (AC)	200	200	200
0+000	30+500	S4/S5	50 (AC)	200	200	200

6.5 Surcharge load of the recommended embankment Height

Depending on the relative fill Height (including 600mm replacement) recommended under the design and pavement design report, the amount of fill Height is tabulated with every one-hundred-meter (100m) interval for our activity the average Height of every five-hundred-meter interval is taken to account, Height for Capping, Granular Sub Base, Base-Course, and Layer Surfacing is set under the pavement design report except layer surfacing (50mm) all have 200mm thickens for purpose of calculation as presented in Table 21. In infield conditions, it is difficult to obtain a material with its maximum dry density (MDD), it is expected to convert moist unit weight (Ymoist).

$$Y_{moist} = \frac{Gs*Y_{W}(1+\%w)}{1+e}$$

Were

$$Y_{dry} = \frac{G_{s}*Y_{W}}{1+e}$$

$$Y_{moist} = Y_{dry}*(1+\%w)$$

$$Y_{dry} = MDD$$

$$Y_{moist} = MDD*(1+\%w)$$

The total recommended surcharge is calculated by multiplying the bulk unit weight of construction material and thickness of each layer as shown hereunder.

Table 21 average surcharge load for the entire stretches

Surcharge load									
chainage (km)	**(Total Embankment) 0.6+fill**	**MDD (kg/m^3)**	**1+%w/100**	**Load (kg/m^2)**	**Average load (kpa)**	**capping(m)**	**MDD (kg/m^3)**	**1+%w/100**	**Average load (kpa)**
0+000	1.04	2090	1.08	2352		0.2	1824	1.13	
0+500	1.38	2090	1.08	3127		0.2	1824	1.13	
1+000	1.98	2090	1.08	4477		0.2	1824	1.13	
1+500	1.33	2090	1.08	3023		0.2	1824	1.13	
2+000	1.93	2090	1.08	4361		0.2	1824	1.13	
2+500	1.83	2090	1.08	4136		0.2	1824	1.13	
3+000	1.80	2090	1.08	4067		0.2	1824	1.13	
3+500	2.06	2090	1.08	4673		0.2	1824	1.13	
4+000	1.75	2090	1.08	3965		0.2	1824	1.13	
4+500	1.61	2090	1.08	3655		0.2	1824	1.13	
5+000	1.80	2090	1.08	4071		0.2	1824	1.13	
5+500	1.82	2090	1.08	4125		0.2	1824	1.13	
6+000	2.08	2090	1.08	4701		0.2	1824	1.13	
6+500	1.78	2090	1.08	4040	**39.1243**	0.2	1824	1.13	
7+000	1.71	2030	1.11	3855		0.2	1824	1.13	**4.12954**
7+500	1.69	2030	1.11	3807		0.2	1824	1.13	
8+000	2.11	2030	1.11	4769		0.2	1824	1.13	
8+500	2.17	2030	1.11	4885		0.2	1824	1.13	
9+000	2.15	2030	1.11	4838		0.2	1824	1.13	
9+500	1.43	2030	1.11	3216		0.2	1824	1.13	
10+000	1.69	2030	1.11	1563		0.2	1824	1.13	
10+500	2.17	2030	1.11	4903		0.2	1824	1.13	
11+000	1.60	2030	1.11	3619		0.2	1824	1.13	
11+500	1.63	2030	1.11	3676		0.2	1824	1.13	
12+000	1.29	2030	1.11	2909		0.2	1824	1.13	
12+500	1.60	2030	1.11	3651	**38.0752**	0.2	1824	1.13	
13+000	1.90	2091	1.08	4335		0.2	1824	1.13	
13+500	2.20	2091	1.08	5122		0.2	1824	1.13	
14+000	1.70	2091	1.08	3954		0.2	1824	1.13	
14+500	1.96	2091	1.08	4408		0.2	1824	1.13	
15+000	1.70	2091	1.08	4041		0.2	1824	1.13	
15+500	1.80	2091	1.08	4176		0.2	1824	1.13	
16+000	1.81	2091	1.08	4076		0.2	1824	1.13	

16+500	1.76	2091	1.08	3975	**39.7138**	0.2	1824	1.13	
17+000	1.75	2091	1.08	3946		0.2	1824	1.13	
17+500	1.80	2091	1.08	4062		0.2	1824	1.13	**4.12954**
18+000	1.39	2091	1.08	3126		0.2	1824	1.13	
18+500	1.08	2091	1.08	2436		0.2	1824	1.13	
19+000	2.09	1960	1.11	4533		0.2	1824	1.13	
19+500	1.30	1960	1.11	2953		0.2	1824	1.13	
20+000	2.08	1960	1.11	4518		0.2	1824	1.13	
20+500	2.27	1960	1.11	4929		0.2	1824	1.13	
21+000	2.10	1960	1.11	4633		0.2	1824	1.13	
21+500	1.60	1960	1.11	3536		0.2	1824	1.13	
22+000	2.00	1960	1.11	4404		0.2	1824	1.13	
22+500	1.70	1960	1.11	3749	**40.2112**	0.2	1824	1.13	
23+000	1.14	1960	1.11	2491		0.2	1824	1.13	
23+500	1.60	1960	1.11	3632		0.2	1824	1.13	
24+000	2.20	1960	1.11	4853		0.2	1824	1.13	
24+500	1.90	1980	1.11	4213		0.2	1824	1.13	
25+000	2.19	1980	1.11	4841		0.2	1824	1.13	
25+500	1.50	1980	1.11	3463		0.2	1824	1.13	
26+000	1.28	1980	1.11	2837		0.2	1824	1.13	
26+500	1.30	1980	1.11	3054		0.2	1824	1.13	
27+000	0.41	1980	1.11	920		0.2	1824	1.13	
27+500	1.80	1980	1.11	4037		0.2	1824	1.13	
28+000	1.70	1980	1.11	3847	**33.4817**	0.2	1824	1.13	
28+500	1.20	1980	1.11	2744		0.2	1824	1.13	
29+000	1.70	1980	1.11	3752		0.2	1824	1.13	
29+500	1.90	1980	1.11	4358		0.2	1824	1.13	
30+000	0.90	1980	1.11	2111		0.2	1824	1.13	

Surcharge load

chainage (km)	Sub-base (m)	MDD (kg/m3)	1+%w/100	Average load (kpa)	Base-course(m)	MDD (kg/m3)	1+%w/100	Average load (kpa)	Ac(m)	Y_{moist}	Average load (pa)
0+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
0+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
1+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
1+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
2+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
2+500	0.2	2000	1.11	**4.42**	0.2	2250	1.05	**4.71**	0.05	23	**1.2**
3+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
3+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
4+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
4+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
5+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
5+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
6+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
6+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
7+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
7+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
8+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
8+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
9+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
9+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
10+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
10+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
11+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
11+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
12+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
12+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
13+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
13+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
14+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
14+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	

15+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
15+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
16+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
16+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
17+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
17+500	0.2	2000	1.11	**4.42**	0.2	2250	1.05		0.05	23	
18+000	0.2	2000	1.11		0.2	2250	1.05	**4.71**	0.05	23	**1.2**
18+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
19+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
19+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
20+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
20+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
21+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
21+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
22+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
22+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
23+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
23+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
24+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
24+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
25+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
25+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
26+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
26+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
27+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
27+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
28+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
28+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
29+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
29+500	0.2	2000	1.11		0.2	2250	1.05		0.05	23	
30+000	0.2	2000	1.11		0.2	2250	1.05		0.05	23	

Table 22 summary of average surcharge load swell pressure of clay

station (km)	total surcharge load (kpa)	swell pressure of clay(kpa)	Pressure variance (kpa)
0+00-6+500	53	91	38
6+500-12+500	52	89	37
12+500-18+500	54	91	37
18+500-24+00	54	89	35
24+00-30+000	48	95	47

The average surcharge load analyzed based on the recommended fill height and the laboratory result complete section of the study area is **52kpa** and **91kpa** respectively with a variation of approximately 38 kpa which is less than 50kpa.

In respect of safety, the value indicates the study area is in the safest condition to be affected by seasonal moisture fluctuation of low swelling potential.

6.6 Loading expansive soil to 0.30 (m) replacement to balance swell pressure:

For the study area starting from km0+000-km30+000, only 0.30 (m) removal of expansive clay and replacement of suitable non-expansive material is sufficient to equate swelling pressure of existing clay. table (23).

Table 23 Estimated sufficient pressure intensity to balance swell pressure with only 0.30 (m)replacement

station (km)	average Embankment load (kpa)	capping load (kpa)	Sub-base load (kpa)	Base-course load (kpa)	Ac load (kpa)	total surcharge load(kpa)	swell pressure of clay (kpa)	Pressure variance (kpa)
0+00-6+500	31.7	4.1	4.4	4.7	1.1	46	91	45
6+500-12+500	30.1					44	89	44
12+500-18+500	32.3					46	91	44
18+500-24+00	33.1					43	88	45
24+00-30+000	33.5					47	95	47

CHAPTER SEVEN

7 CONCLUSION AND RECOMMENDATION

7.1 Conclusion

Expansive soils can be found all over the world, but they are most common in arid/semi-arid areas, where their strong suction and potential for big water content changes on exposure/deficiency can cause significant volume changes in the soil. The results of the investigation revealed that Index Properties indicated a medium to high swell potential, whereas odometer-tested swelling properties of expansive soil in the research area showed a medium swell potential.

The fill weight proposed by the design with the replacement of expansive clay 0.60 (m) is significantly safer in terms of soil swelling pressure and balances the swell pressure, but it is not economical. The replacement height can be 0.3(m), and the quantity of removing expansive clay and replacing material will be reduced.

The expansive soil, as well as its causes and remedies, have been thoroughly examined in this research. The following conclusions have been drawn from other researchers based on their findings.

Removing the expansive soil and replacing it with non-expansive soil is a dependable solution, although it can be time-consuming and costly. This strategy can be useful in projects with a limited stretch of expansive soil, but it can generate complications as well as a lot of costs in larger projects. Furthermore, Granular soils should not be utilized alone for sub excavation and soil replacement because they lead to a collection of water at the surface of the underlying in situ components.

Buildings and structures have a higher loading pressure than pavements, thus applying an equivalent load to counteract the swelling pressure is more effective for engineering structures. As a result, this technology should primarily be utilized for designed constructions on expansive clay. Chemical Stabilization can be an effective method of sealing and reducing the swelling behavior only if it is combined with the soil appropriately to a suitable depth. Because this procedure is so complicated, further research is needed to determine the ideal depth of treatment for various expansive soils and an acceptable amount of lime. Chemical stabilizers pose health and safety issues; thus, they must be carefully considered. Quick lime, for example, is extremely reactive in the environment, posing a concern to the environment. The process of remodeling and compaction is the consequence of the mechanical energy that is mostly used, and it may not be effective if the soils are subjected to considerable moisture variations.

Low-density compaction may reduce bearing capacity; it may not be successful in high-swell potential soils, and it necessitates meticulous quality control.

Ponding has been identified as the most effective method for pre-wetting of soil. It works well in soils with sufficiently high permeabilities to allow relatively quick water ingress, such as fissure Clays. More research should be done to gain the satisfying depth to moisture and the length of time the material should be ponded. Dry seasons have been suggested as the best time for ponding because natural cracks are open due to desiccation. Preventing water access to the soil through the encapsulation method can be a satisfactory method if careful consideration is given to the material to be used as well as the situation of the expansive soil. It could take a long time to acquire appropriate wetness; It is possible to lose strength and fail. Ingress is restricted to a depth less than that of the active zone. After construction, water redistribution may occur, resulting in heave.

Finally, determining the height of replacement and loading the soil to sufficient pressure intensity to balance swell pressure is a practical solution, but it can only be employed in specific circumstances.

7.2 Recommendation

The client, policymakers, and other concerned parties shall be aware of the potential problems associated with engineering structures on this soil and evaluating of swelling potential and potensial heave before remedial measures are implemented to mitigate the risk and save money. Loading the soil to a sufficient stress depth to balance swell pressure: This technique is used in many fills where the fill weight balances the swell pressure.

- Removal and replacement of 0.30(m) are sufficient for the entire 30km stretch.
- Scarify, stabilize, and re-compact the upper portion of the expansive clay subgrade. The amount of heave of expansive soils can also be reduced if compacted to low densities at moisture contents wet of optimum. However, if the soils are compacted below optimum, they may exhibit excellent immediate stability, but they may fail to satisfy specified density requirements. Upon saturation, the strength of this material could also significantly reduce.
- Any drainable waterlogged existing ground and/or roadbed, such as saturated material overlying less pervious strata, shall first be drained by the installation of all permanent surface or subsoil drainage

List of References

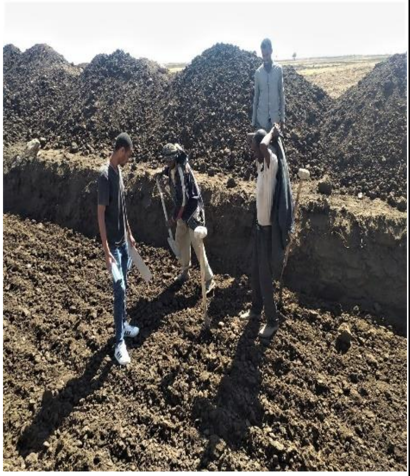
- A bdelkader D and Abdelmalek B. (2001). Identification and prediction of the swelling behavior of some soils from the Tlemcen region of Algeria.
- A.Ardani,.(1992). Expansive Soil Treatment Methods in Colorado.
- Abdelrahman, G.E., H.K. Mohamed and H.M. Ahmed, (2013). New replacement formations on expansive soils using recycled EPS beads. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, at Paris, France.
- Affework Sisay. (2004). Assessment of Damage of Buildings Constructed in Expansive Soil Areas of Addis Ababa, MSc Thesis, Addis Ababa University, Ethiopia, Pp 29-46.
- Alemayehu Tefera and Mesfin, (1999). L. Soil Mechanics. Addis Ababa: AAU Printing Press.
- Altmeyer, W. T. (1955). Discussion of engineering properties of expansive clays. In Proc. ASCE (Vol. 81, No. 658, pp. 17-19).
- BRE (1993) Low-rise buildings on shrinkable clay soils: BRE Digest, Vols. 240, 241.& 242.CRC, London.
- Chen, F. H. (1998). Foundation on Expansive Soil. Elsevier Science Publishers.
- Chen, F.H., (2012). Foundations on expansive soils. Vol. 12: Elsevier Science.
- China Civil Engineering Construction Corporation (CCECC) (2020). Jigjiga-Tuli-Lowwanja-Dul’ad-Samekab-Harmukale Junction Road Project Soils and Materials Investigation Report. Unpublished technical report, CCECC, Addis Ababa Ethiopia, 44 pp.
- crystalline swelling regime. A Thesis Presented to the Faculty of the Graduate School University of Missouri, pp152.
- Driscoll, R. (1983) The influence of vegetation on the swelling and shrinking of clay soils in Britain. Geotechnique, 33, 93-105.
- EIGS (Ethiopian Institute of Geological Survey) (2010). A Report on Basic Geoscience Mapping Core Process, Geology of the HARAR AREAS (NC 38/9). Unpublished Report, Addis Ababa, Ethiopia.
- Ethiopian Roads Authority (ERA) Site Investigation Manual-2002.
- Ethiopian Roads Authority (ERA), 2014. Standard Technical Specification and Method of Measurement. Addis Ababa Ethiopia. Pp. 831

- Faezehossadat.K and Jeff.B, (2016). Expansive Soil Causes and Treatments; i-manager's Journal on Civil Engineering, Vol. 6 l No. 3.
- Gidigasu S. S. R and Gawu S. K. Y, (2013). The Mode of Formation, Nature and Geotechnical characteristics of Black Cotton Soils - A Review. Research and Essays Vol1 (14): 377-390.
- Holtz, R.D. & Kovacs, W.D. (1981) An introduction to geotechnical engineering. Prentice.
- Holtz, W.G., & Gibbs, H.J. (1956) Engineering Properties of Expansive Clays. Transactions of the American Society of Civil Engineers, 121, 641-663.
- Ijara,Tesfaye. (2020). Shrink swell susceptibility zonation of Addis Ababa. Unpublished M.Sc. Thesis, Addis Ababa University, Addis Ababa, Ethiopia, 95 pp.
- Katti RK, Katti DR, Katti AR(2002). Behaviour of saturated expansive soils and control methods. A. A. Balkema Publishers. Pp. 1270
- Kazi Moinul. (2017). Subsurface characterization of flexible pavements constructed over Expansive Soil subgrade and selection of suitable rehabilitation alternatives. Unpublished M.Sc. Thesis, Boise State University,159pp.
- Kibaha, Tanzania. Ph.D. thesis, Royal Institute of Technology Stockholm, Department of Civil and Architectural engineering, Sweden 2008, Pp.1-13.
- M.Aniculaesi1, I Lungu (2019). Evaluation of the swelling pressure for expansive soils. Computational Civil Engineering Conference (CCEC).586.
- Neeraja, D. and A.V. Rao Narsimha, (2010). Use of certain admixtures in the onstruction of pavement on expansive clayey subgrade. International Journal of Engineering Science and Technology no. 2(11): 6108-6114.
- Nelson S.A, 2010, Weathering and clay minerals, lecture note, Tulane University, USA.
- Nelson, D. and Miller, J. (1992). Expansive Soils: Problems and Practice in Foundation and Pavement Engineering. John Wiley & Sons, Inc, New York, 573pp.
- Nelson, J.D., Overton, D.D. & Durkee, D.B. (2001). Depth of wetting and the active zone. Expansive clay soils and vegetative influence on shallow foundations, ASCE Geotechnical
- Ola SA.(1983). The geotechnical properties of the black cotton soil of Northeastern Nigeria. In Tropical soils of Nigeria in Engineering Practice, Ola S.A. (Ed). A. A. Balkema Publishers, Rotterdam. Pp.85-101.
- Petry.M and J. Clyde. (2002) Stabilization of Expansive Clay Soils.

- Ron Kelm, P.E., Nicole Wylie. (2007). Guidelines for Diagnosing Heave, Subsidence, and Settlement. Forensic Engineers Inc., Houston TX,22pp.
- Skempton, A. W, (1953). The colloidal "activity" of clays. Proceedings, 3rd International Conference on Soil Mechanics and Foundation Engineering, Zurich, vol.1, pp. 57-61.
    - special publications No.116., 95-109.
- Sridharan. A. and Prakash, K, (2016). Expansive Soil Characterization: An Appraisal, INAEL. Springer. 1:29–33.
- Tagel Mada. (2016). Application of Marble Dust to Improve the Engineering Properties of Expansive Soils to be used as Subgrade Material. M.Sc. Thesis, Addis Ababa University, Addis Ababa, Ethiopia, 89 pp.
- Taylor R. M.(1990) Tropical Residual soils The Quaternary Journal of Engineering Geology,London .
- Texas Department of Transportation (TxDOT), September 2005. Modification and Stabilization of Soils and Base for Use in Pavement Structures.
- The united republic of Tanzania ministry of works (1999) Pavement and material design manual Unpublished technical manual, united republic of Tanzania,185pp.
- Vanapalli, S. K., & Lu, L. (2012). A state-of-the-art review of 1-D heave prediction methods for expansive soils. International Journal of Geotechnical Engineering, 6, 15–41. http://doi.org/10.3328/IJGE.2012.06.01.15-41
- Walsh, K.D., Colby, C.A., Houston, W.N. & Houston, S.L. (2009). Method for Evaluation of Depth of Wetting in Residential Areas. Journal of Geotechnical and Geoenvironmental Engineering, 135 (2), 169-176.

Annexes

APPENDIX -I TEST PIT LOG

Test points	picture	Coordinates	Field Descriptions
Tp-1 K5+000		X - 258185.01 Y - 1040824.01 Z- 1720	Medium stiff to stiff, dark, high plastic silty CLAY.
Tp-2 K8+000		X-258403.88 Y- 1043796.30 Z- 1740	Medium stiff to stiff, dark grey, high plastic silty CLAY/clayey SILT.
Tp-3 K13+000		X- 259226.80 Y- 1048672.88 Z- 1782	Medium stiff to stiff, dark, high plastic silty CLAY.

Tp-4 K24+000		X- 256178.25 Y-1058774.41 Z- 1884	Medium stiff to stiff, dark grey, high plastic silty CLAY/clayey SILT.
Tp-5 K28+000		X- 254004.12 Y- 1061784.82 Z- 1872	stiff, dark grey, high plastic silty CLAY/clayey SILT.

APPENDIX-II LABORATORY TEST RESULTS

CLIENT	***CONSULTANT***	***CONTRACTOR***
ETHIOPIAN ROADS AUTHORITY (ERA)	**STADIA ENGINEERING WORKS CONSULTANT PLC**	**China Civil Engineering construction corporation (CCECC)**

Test for = Subgrade

NO	TYPE OF TEST CONDUCTED / SOURCE	MDD	OMC%	CBR at 95%of MDD	Grain size passing	Swell %	Liquid limit%	Plastic	plastic index	Free swell index%	Linear shrinkage	NMC
1	K 0+000	1.46	27.9	1.7	99	3.69	65	33	32	65	16	29.9
2	K 0+500	1.48	27.1	2.9	91	2.29	60	30	30	60	13	31.2
3	K 1+500	1.571	25.6	5.65	90	1.6	55	28	27	50	12	26.2
4	K 8+500	1.488	26.2	3.14	93	2.18	59	30	29	60	13	28.8
5	K 9+000				91		59	30	29		13	31.3
6	K 10+500	1.465	27.5	2.08	94	3.46	66	33	33	65	16	31.9
7	k 12+000				92		65	33	32		15	32.2
9	K 12+500	1.458	28	1.64	99	3.8	67	34	33	65	16	30.2
10	K 13+000				97		65	33	32		15	29.8
11	K 13+500	1.483	26.8	2.96	92	2.25	59	30	29	60	13	30.4

12	K 14+000				98		60	30	30		14	29.6
13	K 14+500	1.481	26.9	2.91	93	2.27	59	30	29	60	13	28.9
14	K 16+500	1.569	25.7	5.5	94	1.64	55	28	27	50	12	26.1
15	K 20+000	1.458	28	1.68	92	3.8	60	30	30	60	14	29.3
16	k 20+500	1.461	27.8	1.72	97	3.68	66	33	33	65	16	31.3
17	K 22+500	1.479	27.1	2.95	95	2.3	59	30	29	60	13	30
18	K 23+000				96		65	33	32		15	31.5
19	K 23+500	1.482	26.8	2.96	97	2.27	61	31	30	60	14	30.8
20	K 24+000				98		70	35	35		16	31.4
21	K 24 +500	1.47	27.4	2.7	93	2.34	60	30	30	60	14	31.5
22	K 25+000				98		61	31	30		14	32.4
23	K 25+500	1.471	27.3	2.7	97	2.33	59	30	29	60	13	23.9
24	K 26+000				97		59	30	29		13	26.3
25	K 27+500	1.47	27.3	2.75	97	2.34	59	30	29	60	13	23.8
26	K 28+000				98		65	33	32		15	31.2
27	K 28+500	1.459	28	1.7	99	3.79	66	33	33	60	15	30.6
28	K 29+000				99		60	30	30		14	
29	K 29+500	1.47	27.4	2.7	96	2.34	60	30	30	60	14	
30	K 30+500	1.468	27.5	2.6	95	2.36	61	31	30	60	14	

Represented section	***5+400 RHS***			*Source :*	***Borrow pit***
Sampled By:	***Jointly***				
Sampling date :	***7/08/2019***			*Lab #*	JTLDH-DBRP-St-02/2019
Visual Descri.	***silty clay with gravel material***			*Material for:*	Embankement fill
Blending Ratio					
TYPE OF TEST CONDUCTED		TEST RESULT	**SPECIFICATION LIMIT**	**REMARK**	
COMPACTION TEST	MDD (g/cm^3)	**2.090**	/		
	OMC (%)	**8.4**	/		
CBR TEST	**95% of MDD (g/cm^3)**	**1.99**		**-**	
	CBR at 95% of MDD	**24.0**	**>5%**	**-**	
SWELL (%)		**0.19**	**< 2%**	**-**	

Represented section	***9+700 RHS***	*Source :*	***Borrow pit***
Sampled By:	***Jointly***		
Sampling date :	***23/07/2019***	*Lab #*	JTLDH-DBRP-Sg-01/2019

Visual Descri.		***Whitish silty clay with gravel material***		*Material for:*	Embankement fill
Blending Ratio					
TYPE OF TEST CONDUCTED		TEST RESULT	**SPECIFICATION LIMIT**	**REMARK**	
COMPACTION TEST	MDD (g/cm^3)	**2.030**	/		
	OMC (%)	**11.1**	/		
CBR TEST	**95% of MDD (g/cm^3)**	**1.93**		**-**	
	CBR at 95% of MDD	**18.0**	**>5%**	**-**	
SWELL (%)		**0.08**	**< 2%**	**-**	

Represented section	***19+240 LHS***	*Source :*	***19+240 LHS***
Sampled By:	***Jointly***		
Sampling date :	***29/01/2021***	*Lab #*	JTLDH-DBRP-St-249/2021
Visual Descri.	***Whitish silty soil material***	*Material for:*	Embankement fill & Capping
Blending Ratio			
TYPE OF TEST CONDUCTED	TEST RESULT	**SPECIFICATION LIMIT**	**REMARK**

COMPACTION TEST	MDD (g/cm^3)	**1.959**	/		
	OMC (%)	**10.9**	/		
CBR TEST	**95% of MDD (g/cm^3)**	**1.824**		**-**	
	CBR at 95% of MDD	**50.0**	**>5%**	**-**	
SWELL (%)		**0.63**	**< 2%**	**-**	

Represented section	26+400 RHS @ 200m		*Source:*		***Borrow pit***					
Sampled By:	***Jointly***									
Sampling date :	***6/03/2021***		*Lab #*		JTLDH-DBRP-St-335/2021					
Visual Descri.	***Whitish silty gravel material***		*Material for:*		Embankment Fill & Capping					
Blending Ratio										
TYPE OF TEST CONDUCTED		TEST RESULT	**SPECIFICATION LIMIT**	**REMARK**						
COMPACTION TEST	MDD (g/cm^3)	**2.202**	/							
	OMC (%)	**6.3**	/							
CBR TEST	**95% of MDD (g/cm^3)**	**2.09**		**-**						

	CBR at 95% of MDD	**46.0**	> **5%**	-	
SWELL (%)		**0.44**	**<2%**	-	

Represent section ;		***29+500 LHS***	*Source :*	***29+500 LHS***
Sampled By:		***Jointly***		
Sampling date :		***9/12/2020***	*Lab #*	JTLDH-DBRP-St-162/2020
Visual Descri.		***Light silty gravel material***	*Material for:*	Embankment fill
Blending Ratio				
TYPE OF TEST CONDUCTED		TEST RESULT	**SPECIFICATION LIMIT**	**REMARK**
COMPACTION TEST	MDD (g/cm^3)	**1.98**	/	
	OMC (%)	**11.4**	/	
CBR TEST	**95% of MDD (g/cm^3)**	**1.88**		-
	CBR at 95% of MDD	**27.5**	> **5%**	-
SWELL (%)		**0.36**	< **2.0%**	-

Represented section	***11+500 LHS***	*Source :*	***Borrow pit***	

Sampled By:		***Jointly***			
Sampling date :		***4/03/2021***		*Lab #*	JTLDH-DBRP-St-331/2021
Visual Descri.		***Whitish gravel material***		*Material for:*	**Sub-base**
Blending Ratio					
TYPE OF TEST CONDUCTED		TEST RESULT	**SPECIFICATION LIMIT**	**REMARK**	
COMPACTION TEST	MDD (g/cm^3)	**2.000**	/		
	OMC (%)	**10.6**	/		
CBR TEST	**95% of MDD (g/cm^3)**	**1.90**		**-**	
	CBR at 95% of MDD	**66.0**	**>5%**	**-**	
SWELL (%)		**0.31**	**< 2%**	**-**	

Sampled By:		***Jointly***			
Sampling date :		***5/03/2021***		*Lab #*	JTLDH-DBRP-St-333/2021
Visual Descri.				*Material for:*	**Base-course**
TYPE OF TEST CONDUCTED		TEST RESULT	**SPECIFICATION LIMIT**	**REMARK**	
COMPACTION TEST	MDD (g/cm^3)	**2.25**	/		
	OMC (%)	**4.8**	/		

Cheers

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