Suitability Analysis of Excavated Roadway Materials In-terms of In-situ Moisture Prior To Direct Use as Subgrade Material at Edo-Sorefta-Werka Road Project

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A Thesis Submitted to the Department of Civil Engineering,

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Degree of Master's in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies

Adama Science and Technology University

May, 2024

Adama, Ethiopia

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Declaration

I hereby declare that this Master Thesis entitled "Suitability Analysis of Excavated Roadway Materials In-terms of In-situ Moisture Prior To Direct Use as Subgrade Material at Edo - Sorefta - Werka Road Project" is my original work. That is, it has not been submitted for the award of any academic degree, diploma or certificate in any other universities. All sources of materials that are used for this thesis have been duly acknowledged through citation.

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I/we, the advisor(s) of this thesis, hereby certify that I/we have read the revised version of the thesis entitled “Suitability Analysis of Excavated Roadway Materials In-terms of In-situ Moisture Prior To Direct Use as Subgrade Material at Edo -Sorefta - Werka Road Project” prepared under my/our guidance by Seid Nuru Alemu submitted in partial fulfillment of the requirements for the degree of Mater’s of Science in Geotechnical Engineering.

Therefore, I/we recommend the submission of revised version of the thesis to the department following the applicable procedures.

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Major Advisor	Signature	Date
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Co-advisor	Signature	Date

APPROVAL PAGE

I/we, the advisors of the thesis entitled "Suitability Analysis of Excavated Roadway Materials In-terms of In-situ Moisture Prior To Direct Use as Subgrade Material at Edo - Sorefta - Werka Road Project" and developed by Seid Nuru Alemu, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

Major Advisor	Signature	Date
Co-advisor	Signature	Date

APPROVAL OF BOARD OF REVIEWERS

We, the undersigned, members of the Board of Examiners of the thesis by Seid Nuru Alemu have read and evaluated the thesis entitled "Suitability Analysis of Excavated Roadway Materials In-terms of In-situ Moisture Prior To Direct Use as Subgrade Material at Edo - Sorefta - Werka Road Project" and examined the candidate during open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in Geotechnical Engineering.

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ACKNOWLEDGMENT

First and for most, I would like to show my deepest gratitude to my advisor Dr. Argaw Asha for the continuous support and guidance he has given me, for being there full of support at times when I need motivation and encouragement the most, and for giving me invaluable inspirations in class sessions to read different journals. Next, I would like to thank my whole family and children for being always there when I need them the most. Finally, I would like to thank Mr. Ashebir Tajebe for his role in assisting me in all aspects as brotherhood; and Gondwana Engineering Plc for its excellent welcome to conduct the laboratory tests of this study and invaluable support.

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List of Acronyms and Abbreviations

ASTM: American Society for Testing Materials

AASHTO: American Association of Highway and Transportation Officials

ASTU: Adama Science and Technology University

CBR: California Bearing Ratio

ERA: Ethiopian Roads Authority

I_C: Consistency Index

I_F: Flow Index

I_L: Liquidity Index

LL: Liquid Limit

OMC: Optimum Moisture Content

NMC: Natural Moisture Content

MDD: Maximum Dry Density

masl: Meter Above Sea Level

PL: Plastic Limit

PI: Plasticity Index

SL: Shrinkage Limit

UCS: Unconfined Compressive Strength

USCS: Unified Soil Classification System

Abstract

Physical properties of soil often impose limitations on engineering uses of soil. As physical properties of soil such as strength, compressibility, permeability, plasticity, density, grain-size distribution, cohesiveness and natural moisture content are of great significance are in dictating the usability of soil. Understanding these properties and how they change with environmental conditions and over time enables to design any civil engineering structures. The author believes that it is self-evident that the criteria for a suitable fill material is that it should form an embankment with stable side slopes, support construction traffic and provide a surface of adequate bearing capacity on which the structural layers can be placed and compacted. It is now generally accepted that in the case of clay soils all these factors can be directly related to the undrained shear strength of the compacted fill. Accordingly, this study covers the suitability analysis of roadway excavated materials in-terms of in-situ moisture content prior to direct use as embankment fill material and investigated their properties like soil classification, workability/plasticity, consistency, strength, and stability aspects of soil samples collected for an embankment fill of road construction project.. In addition, 24 samples (71%) are classified as sensitive soils, indicating potential stability issues. The remaining 10 samples (29%) are classified as plastic soils, while no samples are over-consolidated. In conclusion, while the majority of the soil samples meet the basic project specification criteria, a significant number (71%) are unsuitable for embankment fill due to high natural moisture content and sensitivity issues. Further evaluation and potential soil treatment/stabilization measures may be required to ensure the long-term stability and performance of the embankment.

Key words: *Roadway excavated material, Plasticity, In-situ Moisture Content, Liquid Limit, Liquidity/Consistency index*

CHAPTER ONE: INTRODUCTION

1.1. General

Soil is one of the most important civil engineering materials in the construction process, and the determination of geotechnical properties of soil is the most important first phase of work for every type of civil engineering structure. Geotechnical engineering properties of soil are determined by both field and laboratory test methods. There are several laboratory tests to be conducted for identifying index and engineering properties of soil that are very commonly performed to evaluate and compare their engineering functions versus problems. These properties are essential for the design of both foundation and earth structures like buildings, bridges, roads, dams, sewer lines etc. (Cheng Liu, 2009).

The engineering properties of soil are compaction, permeability, compressibility, and shear strength. Tests required for determination of engineering properties are generally elaborate and time consuming, and hence sometimes we only need rough assessment of the engineering properties are determined only if index properties determined first. So, index properties are indicative of engineering properties of soil. As soils are classified and identified based on index properties, simple tests are required for the determination of index properties and these tests are called classification tests. The index properties of soils can be determined in laboratory using tests like water content test, density test, compaction test, sieve and hydrometer analyses tests, specific gravity test and Atterberg limit tests; whereas, engineering properties of soil can be determined in laboratory using tests like consolidation test, triaxial test, direct shear test, unconfined compressive strength test and permeability test. For instance; index properties of fine-grained soil are like water content, specific gravity, voids ratio, porosity, plasticity, unit weight, density, and particle size distribution. These properties are very important for empirical and analytical evaluation of geotechnical problems caused by soils.

The development of transportation infrastructure plays a crucial role in the economic growth and social well-being of a nation. Roads are a vital component of any transportation network, connecting regions and facilitating the movement of goods and people. In the construction of roadways, the availability and suitability of construction materials are of utmost importance to ensure the durability, stability, and cost-effectiveness of the project. One approach to optimizing road construction is by utilizing locally available excavated roadway materials for various purposes, such as cut-to-fill and improved subgrade materials.

This study aims to investigate the potential suitability of excavated roadway materials in-terms of in-situ moisture prior to direct use as subgrade material for the construction of embankment work.

1.2. Background of the Study

The Edo-Sorefta-Werka road project is a significant infrastructure development initiative aimed at improving the connectivity and accessibility of Oromia and Sidama regions. The successful execution of this project requires a thorough understanding of the available construction materials within the project area. Excavated materials from road construction activities often possess varying engineering properties that can influence their suitability for specific applications, such as cut-to-fill and subgrade material. Traditionally, the selection of construction materials involves sourcing them from distant locations, leading to increased project costs and environmental impacts due to transportation. However, by evaluating the suitability of locally available excavated materials, the project can potentially benefit from reduced costs and environmental footprint while promoting sustainable practices.

This research will involve conducting comprehensive laboratory testing and analysis of the excavated roadway materials in terms of their geotechnical properties as well as in-situ moisture. The findings of this study will provide valuable insights into the engineering behavior and suitability of the excavated materials for direct use as cut-to-fill and improved subgrade materials at the captioned project.

By optimizing the utilization of excavated roadway materials, the project can potentially achieve economic and environmental benefits while ensuring the construction of a safe and sustainable road infrastructure. The outcomes of this research will contribute to the body of knowledge in the field of geotechnical engineering and provide practical recommendations for the selection and application of excavated materials in road construction projects, particularly in the context of the Edo-Sorefta-Werka Road Project. The use of locally available excavated materials in road construction projects can have both positive and negative impacts on the quality and durability of the infrastructure. For instance, the quality and durability of road construction projects are influenced by the properties of the materials used. Locally available excavated materials may vary in their engineering properties, such as gradation, compaction characteristics, shear strength, and durability. Proper testing and analysis are essential to determine the suitability of these materials for specific applications. If the excavated materials meet the required specifications and are suitable for the intended use as cut-to-fill or subgrade material, they can contribute to the overall quality and durability of the road construction. In addition, Local regulatory bodies and engineering standards provide guidelines and specifications for road construction materials. It is important to ensure that the locally available excavated materials meet these standards to ensure the desired quality and durability of the road infrastructure. Compliance with appropriate design and construction practices in utilizing excavated materials is crucial to maintaining the longevity and performance of the road.

The use of locally available excavated materials can impact the quality and durability of road construction projects. Proper material selection, testing, processing, compliance with engineering standards, and quality control measures are essential to ensure that the excavated materials meet the required specifications and contribute to the long-term performance and durability of the road infrastructure. Furthermore, utilizing locally available excavated materials in road construction projects offers cost savings, reduces environmental impact, promotes resource efficiency, improves project timelines, allows customization of material properties, and supports local economic development. These benefits make it an attractive and sustainable approach for road construction projects.

1.3. Project Location and Accessibility

The project road is located in West Arsi Zone, Oromia Region, connecting Dodola Woreda with Nansebo Woreda. The project start, Edo Town, is about 305 km from Addis Ababa and 55 km from the Zonal Capital, Shashemene, which can be accessed via Addis-Modjo-Shashemene-Dodola-Goba main trunk road.

The 20 km gravel road that stretches from Edo to Serofta is part of the Edo-Serofta-Kokosa rural road (65 km) that have been said to be studied recently. Riding is over thin natural scoria gravel cover. The roadway has approximate width of 6.5m except in Edo village where extended to 10m. The road traverses over largely flat land that is occasionally interfered by rolling terrain.

Table 1: Basic Project Features

Project Name	**Edo - Serofta – Werka Road Project**
Region	Oromia Regional State
Starting Point Description	Edo Town located in Oromia region, West Arsi Zone at approximately 70°0’42.9’’ Latitude and 39°02’34.4’’ Longitude
Ending Point Description	Werka Town located in Oromia region, West Arsi Zone at approximately 6°35’11’’ Latitude and 39°06’12’’ Longitude
Length	71 km
Road Functional Classification	Class III - Main Access
Climate Classification	‘Dega’ and ‘Weyina Dega’
Elevation	1800 to 3050 m asl
Existing Traffic Volume per day	> 100

The section from Serofta-Werka is on an existing gravel road, and has average 6m wide carriageway with an approximate length of 50 km. The existing gravel road has been constructed in the early 1970’s mainly for military purposes during the Ethio-Somalia war. As a result, it is known by its switching curves and steeper slopes. The topographic feature of this section of the project is generally mountainous with elevations in the order of 1,822 m-asl to over 3,050 m-asl.

The Woredas traversed by the proposed road project are Dodola and Nansebo Weredas with the total population in the order of 232,410 and 135,903 respectively. The total number of Kebeles within these Weredas are 29 and 20 for Dodola and Nansebo, respectively. Out of these totals, Seven Kebeles from Dodola Woreda and four Kebeles from Nansebo Weredas are directly traversed by the proposed road alignment.

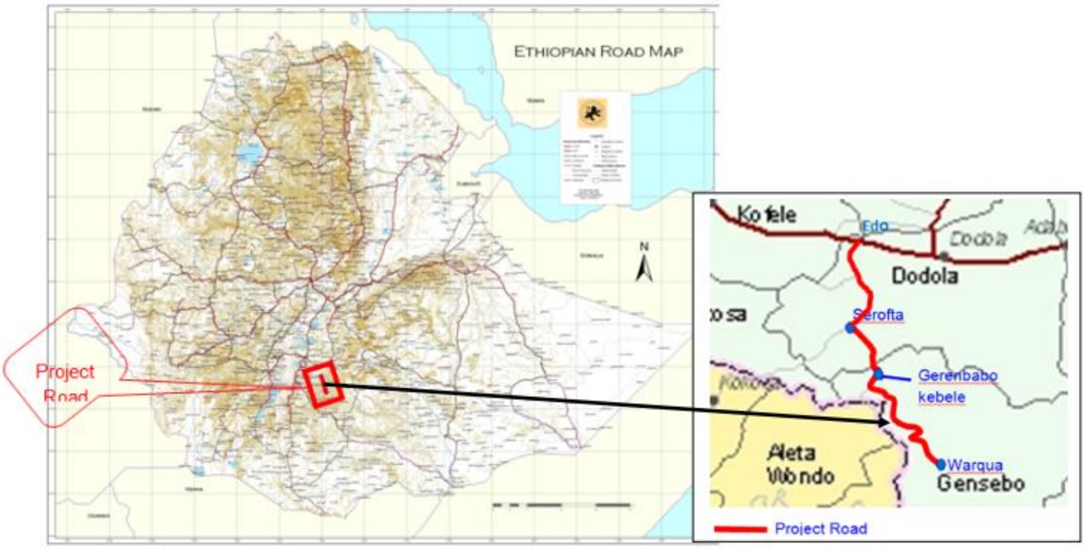

Figure 1: Project Location Map

1.4. Climate, Geology and Topography of the Area of the Study

According to the materials and site investigation report for the design purpose of the subject project by Value Engineering PLC (2016), the climate, geology and topography of the study area is as illustrated hereunder.

1.4.1. Climate of the Study Area

Pavement performance is highly influenced by environmental factors, particularly by temperature and moisture (related with rainfall). This information is used basically to account for the detrimental effects of climate on the ability of the structural cross-section to support traffic loading, and take the appropriate climatic factors when the selecting material type (e.g. in selecting asphalt grade). Climatic information has immense use in design as well as construction stage of a project.

In design stage, it is basically used to account for the detrimental effects of climate on the ability of the structural cross-section to support traffic loading, and take the appropriate climatic factors when designing pavement structure and selecting material type, i.e., temperature for surface treatment, and moisture for likely moisture content of subgrade and unbound materials. In construction stage, it is used to plan construction activities.

Climate is influenced by latitude, altitude, land and water surfaces, mountain barriers, local topography and atmospheric features such as prevailing winds, air masses and pressure centers. Although Ethiopia is located in the tropics, temperatures vary greatly with altitude and large climate variation, from hot arid to cool temperate, exists in the country. The climate in the country is broadly divided into five zones as extracted in Table-1 below.

Table 2: Ethiopia's Climatic Zones (ERA Design Manual for Low Volume Roads, Part-D, 2013)

Climate Zone	Altitude (m)	Temperature (°C)	Rainfall (mm)	Remark
Bereha	< 500	> 28	< 400	CZ_1
Kola	500-1500	20-28	600-1000	CZ_2
Weina Dega	1500-2500	16-20	About 1200	CZ_3
Dega	2500-3200	10-16	1000-2000	CZ_4
Wurch	> 3200	< 10	< 800	CZ_5

From the past ten years rainfall and temperature data that has been collected at 'Adaba' and 'Werka' stations were obtained from Ethiopian Meteorological Services Agency for the analysis. Based on this information (Temperature and rainfall) and the physiographic (particularly altitude), generally the project is found in Weina Dega (CZ_3) for the first 50 km, and in Dega (CZ_4) for last 20 km climatic locations.

1.4.2. Temperature of the Study Area

The meteorological data collected at Adaba shows that the project road beginning experiences temperature of highest mean maximum of average 24°C throughout the year while a range from 25-30 °C is evident in the project end area (Werka station). The stations again show that mean minimum monthly temperature of 7.5 °C and 15 °C are evident in the project beginning and end corridor, respectively. It seems that working temperature that ranges from 10-30 °C is prevalent in the project.

Table 3: Mean Monthly Minimum Temperatures in °C (National Meteorological Service Agency, NMSA)

	Months											
Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Adaba	6.9	6.8	7.1	7.8	7.8	7.5	8.0	7.9	7.5	7.4	6.7	7.1
Werka	14.7	15.1	15.4	16.0	15.1	15.1	14.8	15.6	15.2	14.6	13.6	13.9

Table 4: Mean Monthly Maximum Temperatures in °C (National Meteorological Service Agency, NMSA)

	Months											
Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Adaba	24.0	23.8	24.2	24.2	24.0	23.8	23.1	23.5	23.3	23.1	23.3	23.5
Werka	27.9	28.5	29.6	27.6	26.2	25.1	25.3	25.3	25.9	25.9	26.9	27.3

1.4.3. Rainfall of the Study Area

As per the last 10 years data, the mean annual rainfall varies from 70-250 mm in the project beginning area and increases 120-380 mm around the project end. It receives relatively light rainfall in the month of December, January and February, otherwise wet. The annual rainfall is by far greater than 500 mm. Generally, the project area is wet for the majority period in the year, particularly in the last 20 km, and construction schedule need to take this into consideration.

Table 5: Mean Monthly Rainfall in cm (National Meteorological Service Agency, NMSA)

	Months											
Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Adaba	7	10	15	16	16	24	25	25	19	13	11	8
Werka	12	13	17	29	38	30	26	27	22	26	16	14

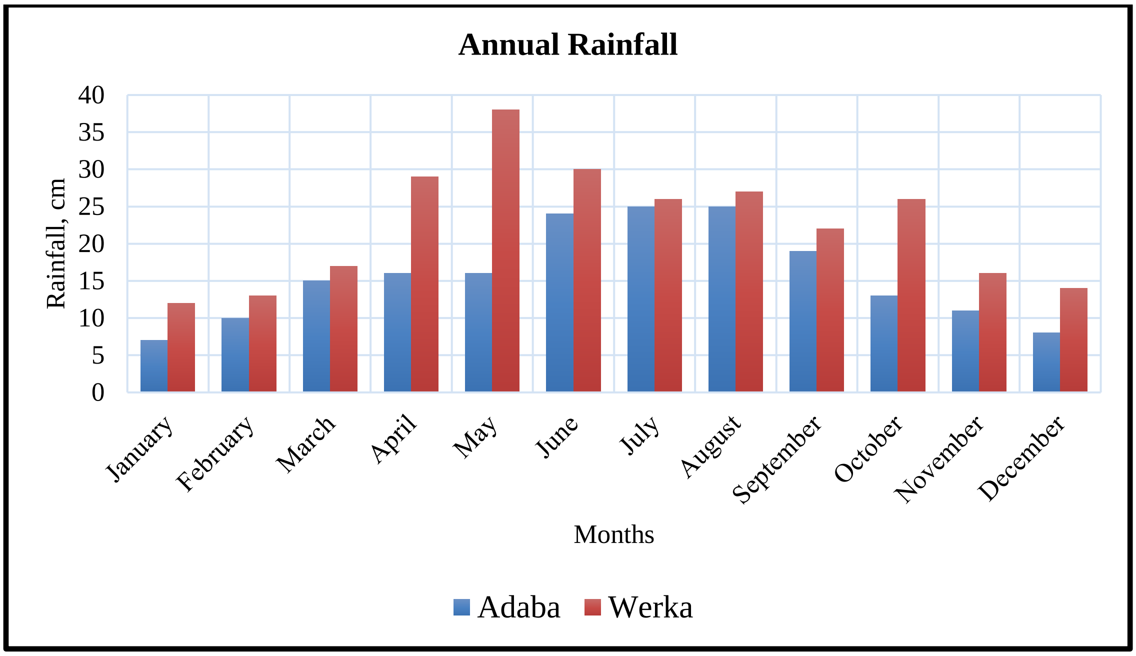

Figure 2: Bar Chart for the Mean Monthly Rainfall of the Study Area

1.4.4. Geology of the Study Area

General geologic features pertaining to the project road were evaluated aided by the "Geological Map of Dodola" with a scale of 1:250000, and the accompanying explanatory note "Geology of the Dodola Area" compiled by G. Hambisa et al (1997), Geological Survey of Ethiopia. According to the map, three units of the Bale massif volcanic succession are prevalent: alkaline trachyte flows with some plugs, trachytic tuffs with minor basalt and alkali trachyte flows, sandstone and limestone, and aphyric to porphyritic basalts. A geologic map of the project area is shown on figure-3 below.

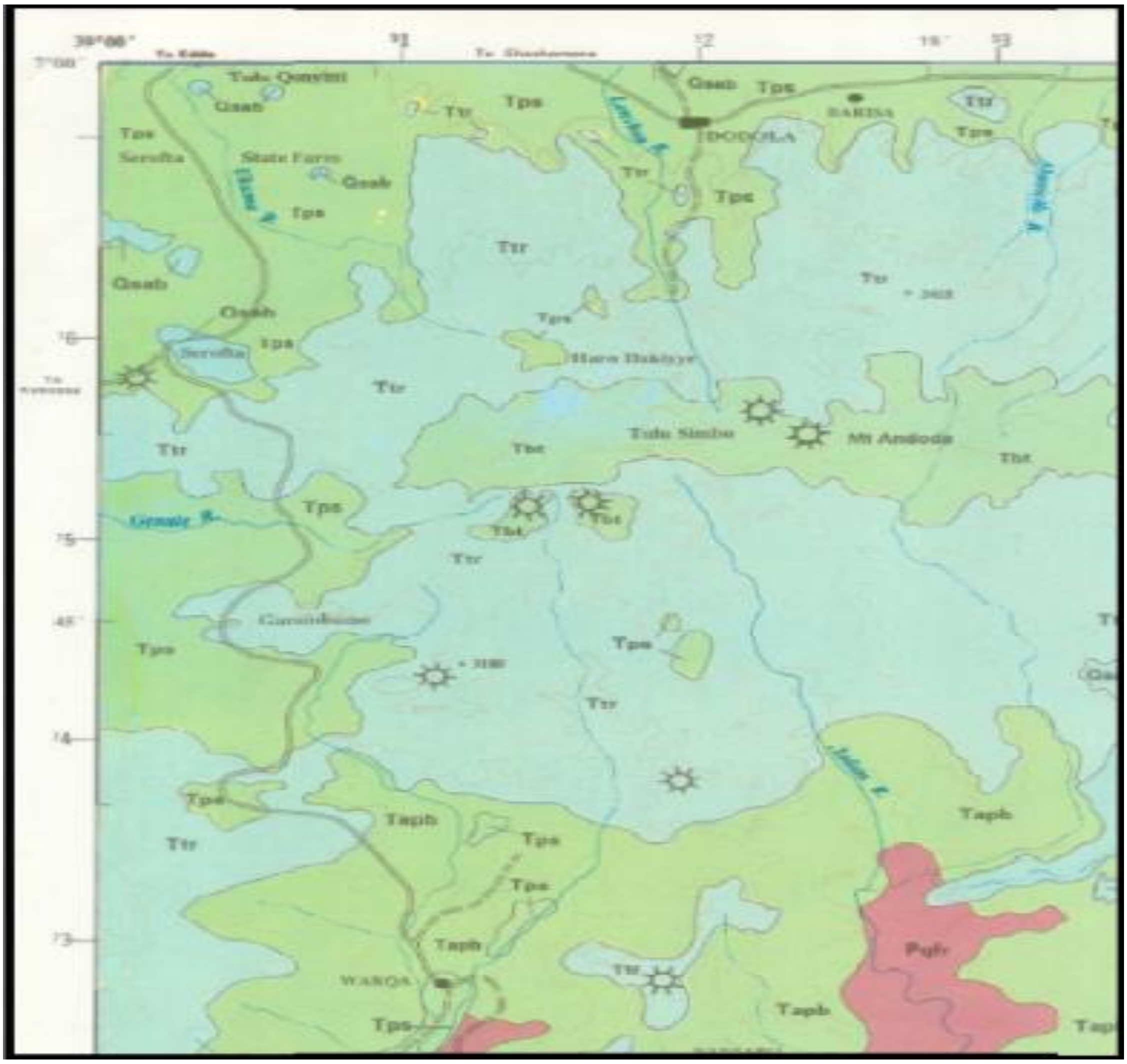

Figure 3: Geological Map of The Study Area

1.4.5. Topography of the Study Area

According to ERA Route Selection Manual (2013), the terrain can be classified as flat, rolling, mountainous and escarpment on the basis of the ground slope as defined below.

- **Flat**: The natural ground slopes perpendicular to the ground contours are generally below 3%.
- **Rolling**: The natural ground slopes perpendicular to the ground contours are generally between 3 and 25%.
- **Mountainous**: The natural ground slopes perpendicular to the ground contours are generally above 25%.

- **Escarpment**: Escarpments require special geometric standards because of the engineering risks involved. Typical gradients are greater than those for mountainous terrain.

Based on the above criteria the project road is classified as illustrated in the table-6 below.

Table 6: Terrain Classification of the Project Route

Chainage		Dominant Terrain Type	Length (Km)	Percent (%)
From	To			
0+000	19+000	Flat	19.00	27.4%
19+000	25+000	Rolling	5.00	7.0%
25+000	31+000	Mountainous	6.00	8.4%
31+000	37+000	Rolling	6.00	8.4%
37+000	40+000	Mountainous	3.00	4.2%
40+000	47+000	Rolling	7.00	9.8%
47+000	68+000	Mountainous	21.00	29.4%
68+000	End	Rolling	3.88	5.4%

1.4.6. Land Use and Land Cover

The project area is endowed with extensive natural forests which are the part of the Harena National forest priority area that normally stretches into West Arsi, Bale and Guji Zones of Oromia Region as well as Sidama Region. The Harena-Kokosa National Forest priority area is also comprising more than half (107,500ha) of the total land mass of the Bale Mountain National Park (215,000ha). The part of Dodola woreda and almost all land masses of Nansebo woreda are also falls within the Harena Natural Forest. Harena-Kokosa Forest is also located adjacent to Mena-Angetu, Anferera-Wader and Bale forest priority areas. Moreover, Dodola woreda is also shares part of the Adaba-Dodola Natural Forest priority area. As a result, the forest cover of the Dodola and Nansebo Woredas is said to be in the order of 44% (73,495) and 75% (135,315) in the same order. The land of the study area is predominantly occupied by natural forests (44-72%), cultivated areas (13-26%), grazing grounds (5-12%) and settlements and infrastructures (5.6%). The cultivated lands are dominated by coffee and ‘enset’ plantations in Nansebo woreda and wheat and barley production in Dodola woreda.

1.5. Problem Statement

One of the main concerns that initiated the author for this study is the excessive in-situ moisture content present in the roadway excavated materials from km 20+900 to km 55+500 for direct use as cut-to-fill and improved subgrade at Edo-Serofta-Werka Road Project. However, the Contractor had to discard excavated roadway materials that were already classified as suitable in terms of contract specification geotechnical parameters but having high in-situ moisture content. This is due to the fact that made it impractical to dry them and directly use in cut-to-fill and improved subgrade of embankment construction.

The suitability of the excavated material was be evaluated based on the requirements specified in the ERA Standard Specifications and Special Provisions for the Project Works Contract. According to the Works Contract specification sub-clause 2408 (d), "Suitable materials that are wet (significantly above OMC) at their in-situ state should be dried out by the Contractor until they reach an adequate dryness for compaction. However, the Contractor should plan the construction program to use high in-situ moisture materials after drying during the dry season whenever practical. The method of drying is at the Contractor's discretion, and the cost of drying is included in the tender rates unless otherwise specified in the Special Specification."

During the progress of the roadway excavations, it has been observed that the in-situ moisture content of the excavated materials often significantly exceeded their OMC (±2% of OMC). This high in-situ moisture content has caused difficulties during the construction activities. The excavated materials were unstable while loading onto trucks, resulting in an unstable final subgrade level that hindered the passage of trucks and compaction equipment. Moreover, due to the prevailing wet weather conditions, it takes a considerable amount of time to dry the excavated materials to make them suitable for both embankments fill and subgrade construction. However, drying the materials is a time-consuming process, and the project corridor experiences high annual rainfall of over 1500mm distributed throughout the year. The construction season in this area typically lasts for only six months, from October to March, with a mean annual temperature below 20°C.

Despite being aware of the situation and the adverse effect of excess in-situ moisture content on the progress of the work, the Contractor decided, at its own risk, to consider the excavated roadway materials with high in-situ moisture as unsuitable and discarded them as cut-to-spoil. This decision was communicated to the Engineer through letters. To further complicate matters, the bill of quantities (BoQ) in the Works Contract didn't separate the quantities of imported borrow materials for filling and replacement of unsuitable subgrade soil. This lack of clarity led to a major setback during the design review, requiring a section-by-section review of the whole project length to quantify earthwork and other construction items. As a result of replacing unsuitable soil with suitable borrow materials, particularly those with LL> 60% and excessive in-situ moisture content, the volume of borrow materials has significantly increased. The total quantity of suitable borrow material specified by the contract can only be compared to the borrow material used for subgrade replacement. Most of the roadway excavations are suitable as sources of embankment materials.

The lack of specific provisions in the Contract Specification regarding the suitability of excavated materials with excess in-situ moisture in areas with high rainfall posed a practical challenge. For instance, it is unreasonable to expect the Contractor to dry excavated soil that is around its LL under any practical condition.

The author was unable to find any officially published articles or technical specification manuals specifically addressing the suitability analysis of predefined suitable roadway excavated materials with regards to their high in-situ moisture content, prior to direct use as construction materials. Consequently, the author conducted this investigation into this matter. In this study, the author examined the predefined suitable roadway excavated materials and assessed their potential for the direct use as construction materials. One crucial aspect focused on was the high in-situ moisture content (significantly beyond the OMC) found within these materials at the excavation site. However, no existing sources or publications directly addressed this particular issue. To bridge this knowledge gap, the author conducted an independent analysis to determine the suitability of the materials for construction purposes despite their high in-situ moisture content. This analysis involved assessing the materials' physical properties, such as their strength, durability, and compaction characteristics, and considering the potential effects of moisture on these properties with respect to the optimum moisture content. The findings and analysis would also contribute valuable insights to the field of geotechnical engineering, especially in addressing challenges associated with high in-situ moisture content in predefined suitable roadway excavated materials.

1.6. The Objectives of the Study

1.6.1. General Objectives

The main objective of this study is carrying out suitability analysis of excavated roadway materials prior to direct use as cut-to-fill and improved subgrade material at Edo-Serofta-Werka Road Project regarding road construction.

1.6.2. The Specific Objectives

The specific objectives of this study aimed to address the captioned challenges are:

- Assess the geotechnical properties of the excavated roadway materials for the suitability analysis at different chainage locations of the road project.
- Evaluate the suitability of the excavated roadway materials for direct use based on their in-situ moisture content.
- Analyze the potential impact of varying in-situ moisture content on the performance and durability of the road construction.
- Compare the properties of the excavated materials with established standards or specifications to determine their compliance.
- Provide recommendations for appropriate moisture control measures for the excavated roadway materials to ensure optimal performance in road construction.

1.7. The Scope of the Study

- The experimental research type was used for this study. It used quantitative approach for data analysis with inductive method of reasoning for discussion of results.
- The specific project site geographical location where the study was carried out is at the Edo-Serofta-Werka road project, which constitutes the primary area of investigation, and the findings may not be directly applicable to other road projects or locations.
- This study was limited to laboratory testing and analysis of the collected samples. No field tests were conducted as part of this research. Laboratory-based investigations involved conducting various tests to determine the geotechnical properties and characteristics of the materials, such as grain size analysis, compaction test, CBR test, plasticity tests, specific gravity test and natural moisture contest test.
- The study focused on roadway excavated material samples within the specified section of the road, ranging from km 21+900 to 55+500. These samples were collected within the specified section to ensure representative sampling, and the findings may not be representative of the entire road project or other projects.
- The primary purpose of this study was to contribute academic knowledge in the field of Geotechnical Engineering and fulfill the partial requirements for a Master of Science degree. Hence, the practical implications and direct applications of the study's findings were limited but it will be used as a springboard for future studies.

Based on the availability of apparatuses, ASTM and AASHTO standard manuals were interchangeably used for the laboratory procedures to conduct the designated tests and all the laboratory tests within the time table shown in section-3.

1.8. The Significance & Outcomes of the Study

At the end of this study, the possible outcomes that the author expected to deliver are shortlisted hereunder.

- To resolve the outstanding issues for the utilization of the roadway excavated material for the sole purpose of cut-to-fill and subgrade replacement that have been in pending. This was when the soils are suitable in terms of requirements set in the contract except due to excess natural moisture contents beyond acceptable construction practice.

- The excessive in-situ moisture content in the roadway excavated material has posed significant challenges during construction activities such as instability during loading onto trucks and hindrance to the passage of trucks and compaction equipment. By analyzing the material's suitability and proposing recommendations, this study aims to address these challenges and improve the construction process.
- Drying materials for use in embankments and improved subgrade construction is time-consuming and costly. By determining the suitability of the excavated material, this study can potentially reduce the need for drying and associated costs, leading to cost and time efficiency in the construction process.
- The lack of clarity in the bill of quantities regarding borrow materials for filling and replacement of unsuitable subgrade soil has led to increased volumes of borrow materials. Analyzing the excavated material's suitability will help optimize resource utilization by identifying which materials can be used as embankment materials and which are suitable for subgrade replacement, thus avoiding unnecessary use of borrow materials.
- To provide sensible recommendations regarding the suitability of excavated materials with excess in-situ moisture content in areas with high rainfall. This will address the practical challenge faced and provide guidance on how to handle excavated soil around its LL under practical conditions, considering the high annual precipitation and limited construction season.
- It will provide valuable insights into the material's characteristics and help determine if it meets the necessary criteria for direct use as cut-to-fill and improved subgrade material, ensuring the quality and performance of the road project.
- The study will contribute to the existing body of knowledge in Geotechnical Engineering and serve as a foundation for future studies and practical applications in similar contexts.

CHAPTER TWO: LITERATURE REVIEW

In this chapter, the author presents the reviewed relevant journals, studies, reference books and standard manuals from different authors and institutions on the topic related to the study.

2.1 General

In soil, there are three states of constituents: solid, liquid, and gas. The interaction and behavior of these three phases, along with their inherent variability, lead to non-uniformity in soil composition, characteristics, and properties. Due to the complex interactions and reactions, it becomes challenging to precisely predict how soil will respond to external factors (R.N. Yong, 1975).

2.2 Physical Properties of Soil

Weight and volume are among the most common physical properties of soil. Soil density is an important characteristic, and the degree of saturation is another obvious physical property. Saturation influences water movement, swelling upon wetting, and cracking upon drying. The presence of water affects the diffusion of air in the soil. Soil is composed of particles of different sizes, arranged in various ways, with voids or pore spaces between them. It can be described as a dynamic equilibrium system consisting of the three phases, and the equilibrium between these phases is continuously changing. During rainfall, the liquid phase increases while some gases are displaced. Particle movement occurs as a result of external stresses induced by engineering construction. On a physical level, soil is comprised of minerals of different sizes, with certain organic molecules strongly bonded to the minerals and some organic matter physically mixed in. Clay and silt, the smallest particles, are not uniformly distributed throughout the soil (R.N. Yong, 1975).

2.3 Importance of Physical Properties

Physical properties of soil often impose limitations on engineering uses of soil. As physical properties are more difficult to alter compared to chemical properties, they largely dictate the usability of soil. Physical properties of soil such as strength, compressibility, permeability, plasticity, density, grain-size distribution, cohesiveness and natural moisture content are of great significance. Understanding these properties and how they change with environmental conditions and over time enables to design any civil engineering structures (R.N. Yong, 1975).

The process of exploring to characterize or define small scale properties of substrata at construction sites is unique to geotechnical engineering. In other engineering disciplines, material properties are specified during design, construction or manufacture, and then controlled to meet the specification. Unfortunately, subsurface properties cannot be specified; they must be deduced through exploration (C.H. Dowding, 1979).

The utilization of clay as a natural material for earthwork is a possibility worth considering. In various construction applications, compacted clay serves as a popular choice for filling embankments, creating cores in dams and dikes, and establishing liners on slopes. The engineering characteristics of clay are influenced by several factors, including water content, density, and general engineering properties. While it is widely acknowledged that the behavior of clay is significantly impacted by its water content and bulk density, the primary factor contributing to variations in its engineering properties such as shear strength, compressibility, and permeability is the alteration of water content. Clay is a readily available raw material in many regions, either sourced directly from the borrow sites or obtained from roadway excavations during construction activities. In certain regions, clay soil contains a significant amount of organic matter. This type of clay is characterized by its exceptionally high-water content, low strength, and substantial compressibility. Traditionally, this clay has not been utilized in engineering projects and has been considered useless or waste material, often disposed of in designated areas. However, with ongoing projects, the volume of this material, generated as spoil from construction sites, is increasing. considering environmental concerns and limited disposal space, managing this clay poses a challenge. It is crucial to find an efficient solution to this problem, and one potential approach is to explore the reuse of this unsuitable clay, which could help address at least a portion of the issue (C. Limsiri, 2008).

In general, the engineering properties of fine-grained soils depend significantly on their in-situ moisture content. For the same moisture content, the consistency of two soils could be markedly different. For example, at the same moisture content, a sample of silt could be very soft, whilst clay would be somewhat stiff and unpliable due to its plasticity nature and structure. This fact can, therefore, be conveniently used to differentiate between different types of soil, and the geotechnical property is known as Plasticity property of the soil (B. Bodo, 2013).

2.4 Plasticity Behavior of Soils in Relation to Their Natural/In-situ Moisture

The behavior of soil, particularly clay soils, undergoes changes with variations in water content. At high water contents, soils behave like suspensions with liquid-like flow properties. As the water content gradually decreases, the flow properties of clay soils transition to non-Newtonian flow, resembling paste-like materials. With further drying, the clay becomes moldable, exhibiting plasticity. At this water content, the soil is considered plastic. As the water content decreases even more, the plasticity diminishes, making the soil harder to work with and gradually assuming solid-like properties at low water contents. Building on clay-rich sediments can be challenging and includes several geotechnical issues. They have a complex interaction between clay minerals and water, which in turn provides low bearing and strength capacity. As the physicochemical interaction between clay particles, the water that fills the voids between the particles and the ions in the water affect the mechanical behavior of clays, high clay mineral content in the soil contributes to clay minerals being more "active" as a result, such deposits are highly compressible, very plastic, have a low permeability and are subjected to shrinkage and swell. Water is strongly connected to clay particle surfaces, which results in plasticity (Mitchell and Soga, 2005).

Plasticity refers to the ability of a material to continuously change shape under applied stress and retain the new shape when the stress is removed. This sets it apart from elastic materials, which regain their original shape after stress removal, and liquids, which do not maintain their own shape. Only very fine particles of soils with the appropriate mineralogy, such as clays and to some extent silts, exhibit plastic behavior under low stresses. The deformation behavior of soil at a specific water content is referred to as its consistency. Consistency represents the soil's resistance to flow and provides information about its deformation behavior. Different soils exhibit varying consistencies at different water contents, and specifying this condition offers insights into the type of soil material. The range of water content over which a soil displays plastic behavior under low stresses is defined as the range between the plastic limit and liquid limit. The plastic limit is the water content below which the soil is considered non-plastic and crumbles under pressure when worked. The liquid limit marks the transition from plastic to flow behavior. These measurements of plasticity are based on the work of Atterberg, hence the term "Atterberg limits" (Budhu M., 2000).

Measurements of plasticity are typically conducted on remolded soil. Plasticity, along with particle-size distribution, is widely used in the evaluation and classification of soil for engineering purposes. The liquid (LL) and plastic (PL) limits depend on the type and quantity of cohesive particles present in the soil. The difference between these two values is referred to as the plasticity index (PI). The compressibility of remolded soil increases with increasing water content at the plastic limit, while the strength of the soil decreases under the same conditions. The shrinkage limit (SL) indicates the moisture content at which further decreases in moisture content do not cause additional shrinkage. However, this limit is rarely used in routine engineering work (Rollings et al, 1996).

In 1911, the Swedish scientist Albert Atterberg described two qualitative (LL and PL) limits based on the varying water content of clay. These limits are used for identification, characterization and classification of clay-rich sediments. Plasticity index (PI) is the difference between liquid and plastic limits and is the range of water content at which clay-rich sediments display plastic properties, while the liquidity index (I_L) describes the relative consistency of the clay-rich sediments. After Terzaghi, K. (1925) discovered the potential of the Atterberg limits in soil mechanics, many further studies were performed to establish relationships between various mechanical properties of clayey sediments, such as compression modulus and shear strength, and liquidity and plasticity indices. While there have been many studies carried out worldwide to establish the correlation between liquidity index and undrained shear strength, such investigations are not common in Sweden.

Larsson (2008) and Vardanega and Haigh (2014) also write that the Atterberg limits give an introductory assessment of mechanical properties of soils. Interestingly, the plasticity index also gives a preliminary insight into the potential geotechnical issues since the clay content is quantitatively estimated by the plasticity index (Duncan et al., 2014). Therefore, the higher the index, the more clay in the soil.

Geotechnical investigations are very important since they provide significant information about various physical and mechanical soil properties which affect the design of upcoming constructions. This study investigates the relationship between the liquidity index and undrained shear strength. Undrained shear strength of clay-rich sediments is not a characteristic property of the material, but is dependent on the in-situ moisture content. Thus, it is suggested to correlate to a state-dependent index parameter, the liquidity index (Knappett et al, 2012).

According to Das, B.M. (2018); 'Principles of Geotechnical Engineering' (9th Edition), when clay minerals are present in fine-grained soil, the soil can be remolded in the presence of some moisture without crumbling. This cohesive nature is caused by the adsorbed water surrounding the clay particles. The Swedish scientist, A. Atterberg, developed a method to describe the consistency of fine-grained soils with varying moisture contents. At a very low moisture content, soil behaves more like a solid. When the moisture content is very high, the soil and water may flow like a liquid. Hence, on an arbitrary basis, depending on the moisture content, the behavior of soil can be divided into four basic states solid, semi-solid, plastic, and liquid (as shown in Figure-4 below). The moisture content at which the transition from solid to semi-solid state takes place is defined as the shrinkage limit. The moisture content at the point of transition from semisolid to plastic state is the plastic limit, and from plastic to liquid state is the liquid limit. These parameters are also known as Atterberg limits.

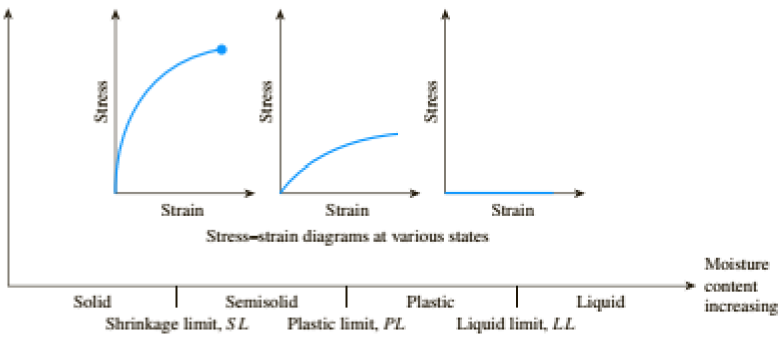

Figure 4: Atterberg limits (Casagrande, 1932)

2.4.1 Atterberg Limits

The accepted empirical method for the determination of consistency was devised by A. Atterberg (1911), based on the fact that, if sufficient water is added to a soil, its state changes from solid to liquid. Considering it the other way around, if the soil in liquid state is dried, it solidifies. The transformation during the drying process occurs in three stages as defined by the three Atterberg Limits: Liquid Limit (LL), Plastic Limit (PL) and Shrinkage Limit (SL).

i) Liquid Limit

When soil paste, wet enough to flow under its own weight, is dried, it changes into a plastic mass. The moisture content at which the change occurs is the Liquid Limit (LL). The volume of soil has decreased and it is fully saturated at this stage. In general, the value of LL increases as the size of soil particles gets smaller. If the natural moisture content of a soil is near to its LL, then it is susceptible to large deformation and shear failure when loaded. Usually, the Liquid Limit of a soil is defined as that moisture content at which the soil passes from a plastic to a liquid state (B. Bodo, 2013).

The percussion method was developed by Casagrande (1932) and used throughout the world (Figure 5). This device consists of a brass cup and a hard rubber base. The brass cup can be dropped onto the base by a cam operated by a crank. To perform the liquid limit test, one must place a soil paste in the cup. A groove is then cut at the center of the soil pat with the standard grooving tool. The moisture content of the soil and the corresponding number of blows are plotted on semilogarithmic graph paper (Figure 6). The logarithmic scaled relationship between moisture content and number of blows is approximated as a straight line. This line is referred to as the flow curve. The moisture content corresponding to 25 number of blows determined from the flow curve, gives the liquid limit of the soil. The slope of the flow line is defined as the flow index.

Figure 5: Percussion Method Apparatus for LL Determination (Casagrande, 1932)

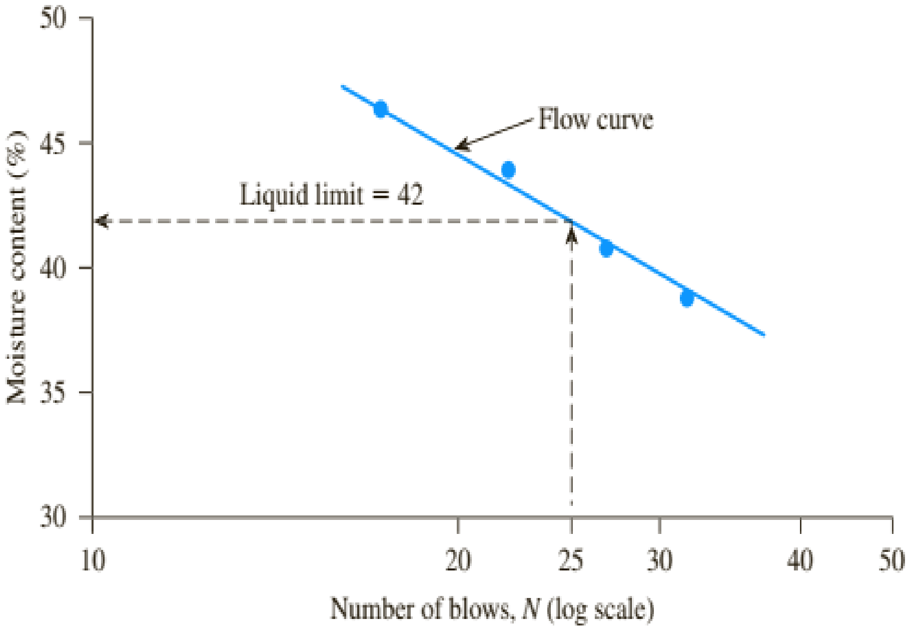

Figure 6: Flow Curve for Liquid Limit Determination of a Clay Soil (Casagrande, 1932)

ii) Plastic Limit

As the soil gets drier, it reaches a water content below which the material becomes friable when rolled into threads of 3.2 mm in diameter. This particular value of is the Plastic Limit (PL) of the soil. Usually, the Plastic Limit is defined as the lowest moisture content at which the soil is plastic. At PL the soil crumbles and falls apart easily, under pressure. If the natural moisture content of a soil is near its PL then, from an engineering point of view, it is easy to excavate and compact to its smallest volume (B. Bodo, 2013).

The plastic limit test is simple and is performed by repeated rolling of an ellipsoidal-sized soil mass by hand on a ground glass plate (Figure 7).

iii) Plasticity Index

The plasticity index (PI) is the difference between the liquid limit and the plastic limit of a soil, or

PI = LL – PL---Equation 2.1

The plasticity index is important in classifying fine-grained soils. It is fundamental to the Casagrande plasticity chart, which is currently the basis for the Unified Soil Classification System (Casagrande, 1932).

iv) Shrinkage Limit

Soil shrinks as moisture is gradually lost from it. With continuing loss of moisture, a stage of equilibrium is reached at which more loss of moisture will result in no further volume change (Figure 8). The moisture content at which the volume of the soil mass ceases to change is defined as the shrinkage limit. Shrinkage limit tests are performed in the laboratory with a porcelain dish (B. Bodo, 2013).

Shrinkage limit can be determined by:

$SL = \omega_i - \Delta\omega$--- Equation 2.2

Where;

- ω_i = initial moisture content when the soil is placed in the shrinkage limit dish
- $\Delta\omega$ = change in moisture content (i.e, between the initial moisture content and the moisture content at the shrinkage limit)

However,

$$\omega_i = \frac{(M1-M2)}{M2} * 100 \text{--- Equation2.2}$$

Where;

- M_1 = mass of the wet soil pat in the dish at the beginning of the test (g)
- M_2 = mass of the dry soil pat (g)

Shrinkage Limit test is difficult to carry out by taking selected representative samples to undertake volumetric shrinkage tests as the test results vary according to the test method and initial moisture content. This is more pronounced when the soils are remolded or disturbed to undertake the test. Hence, Casagrande (1932) suggested that the initial moisture content for shrinkage limit tests should be slightly above the plastic limit, but difficult to prepare specimen to such low moisture content without entrapping air bubbles. It has been found that for soils prepared in this way and that plots near the A-line of a plasticity chart the shrinkage limit is about 20. If the soil plots an amount ΔP vertically above or below the A-line, then the shrinkage limit will be less than or greater than 20 by ΔP.

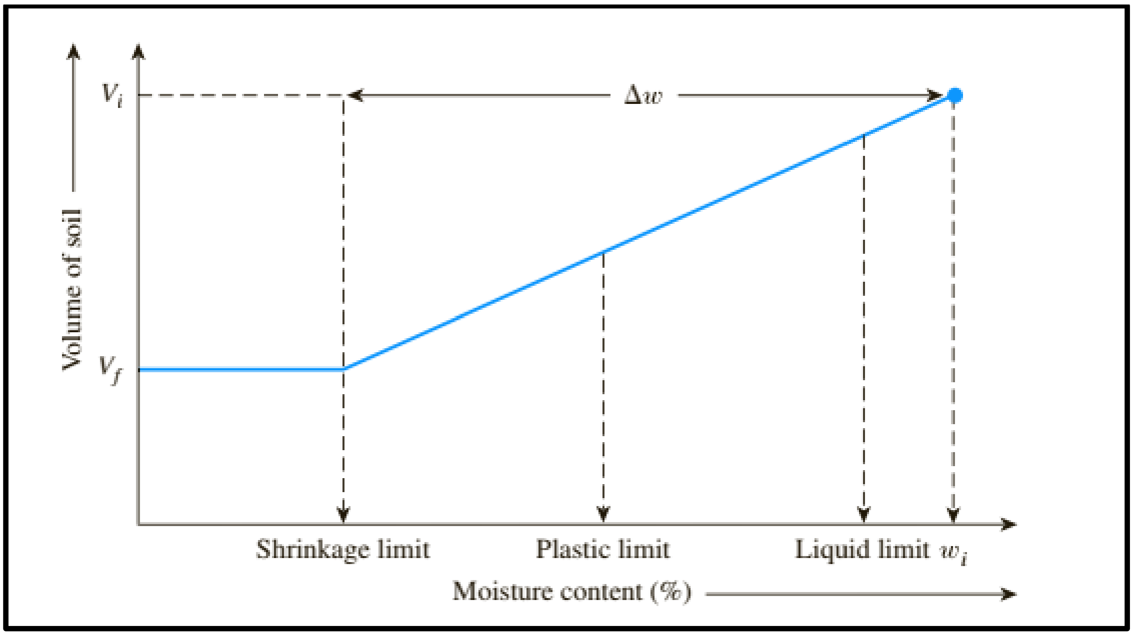

Figure 7: Definition of Shrinkage Limit (Das, B.M., 2018)

v) Liquidity Index and Consistency Index

According to Das, B.M. (2018), ‘Principles of Geotechnical Engineering’ (9th Edition); the relative consistency of a cohesive soil in the natural state can be defined by a ratio called the liquidity index (I_L). Liquidity index of fine-grained refers to how much the in-situ moisture content of soil closer to LL of soil at its natural state, which is given by:

$$I_L = \frac{\omega - PL}{LL - PL} \quad \text{……………………………………………………} \textit{ Equation } 2.1$$

Where;

ω = in-situ moisture content of soil.

From equation (3), we can demonstrate:

i) The in-situ moisture content for a sensitive clay may be greater than the liquid limit; i.e.: $I_L > 1$; These soils, when remolded, can be transformed into a viscous form to flow like a liquid.
ii) Soil deposits that are heavily over-consolidated may have a natural moisture content less than the plastic limit; i.e.: $I_L < 0$.

Another index that is commonly used for engineering purposes is the consistency index (I_C). Consistency index of fine-grained refers to how much the in-situ moisture content of soil closer to PL of soil at its natural state, which may be defined as:

$$I_C = \frac{LL - \omega}{LL - PL} \quad \text{------------------------------------} \textit{ Equation } 2.2$$

Where;

ω = in-situ moisture content of soil.

From equation (4), we can demonstrate:

i) If ω is equal to the liquid limit, the consistency index is zero.
ii) Again, if ω = PL, then $I_C = 1$.

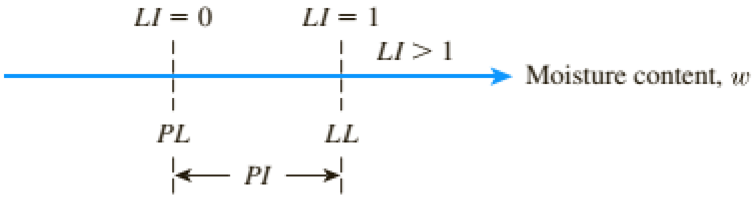

Figure 8: Liquidity Index (Das, B.M., 2018)

vi) Plasticity Chart

Casagrande (1932) studied the relationship of the plasticity index to the liquid limit of a wide variety of natural soils and proposed a plasticity chart as shown in figure-10 below. The important feature of this chart is the empirical A-line equation. An A-line separates the inorganic clays from the inorganic silts. Inorganic clay values lie above the A-line, and values for inorganic silts lie below the A-line. Organic silts plot in the same region (below the A-line and with LL ranging from 30 to 50) as the inorganic silts of medium compressibility. Organic clays plot in the same region as inorganic silts of high compressibility (below the A-line and LL greater than 50). The information provided in the plasticity chart is of great value and is the basis for the classification of ne-grained soils in the Unified Soil Classification System. Note that a line called the U-line lies above the A-line. The U-line is approximately the upper limit of the relationship of the plasticity index to the liquid limit for any currently known soil.

The equations for the A-line and U-line can be given as:

A-line: PI = 0.73(LL - 20) --------------------------------------- *Equation 1*.3

U-line: PI = 0.9(LL - 8) -- *Equation* 1.4

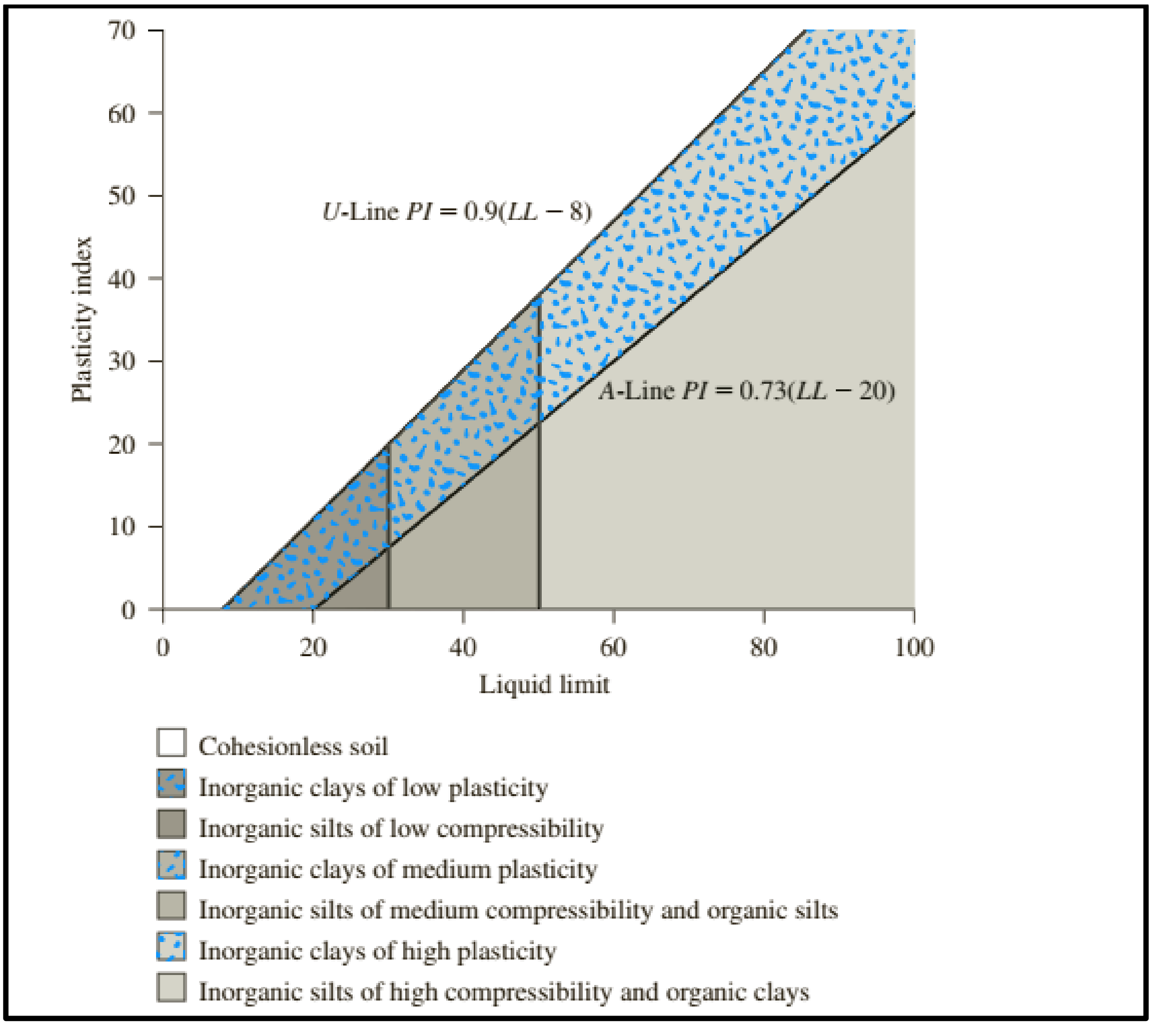

Figure 9: Plasticity Chart (Casagrande 1932)

Casagrande (1932) has suggested another use for the A-line and the U-line. Once the shrinkage limit of a soil can be approximately determined if its plasticity index and liquid limit are known (adopted from Holtz and Kovacs, 1981). This can be done in the following manner with reference to figure 11 below.

i) Plot the plasticity index against the liquid limit of a given soil such as point A in Figure 10.

ii) Project the A-line and the U-line downward to meet at point B. Point B will have the coordinates of LL = 43.5 and PI = 46.4.

iii) Join points B and A with a straight line. This will intersect the liquid limit axis at point C. The abscissa of point C is the estimated shrinkage limit.

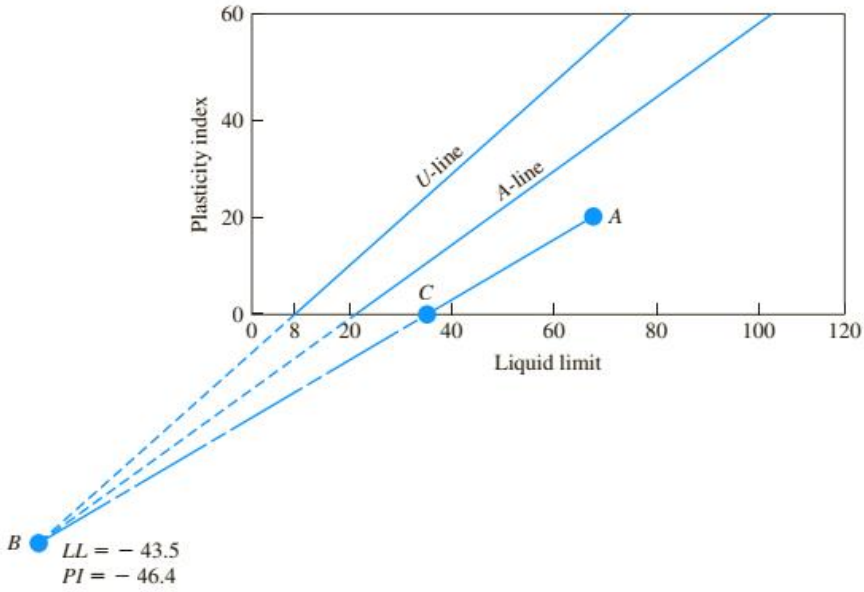

Figure 10: Estimation of Shrinkage from Plasticity Chart (Adapted from Holtz and Kovacs, 1981)

2.5 Factors Affecting Plasticity Behavior and Strength of Soil

2.5.1 Moisture Content

Aday, S. (1993) studied the impact of soil moisture content on soil shear strength in different soil textures. The study was conducted on four soil textures (loamy sand, sandy silt, silty loam, and silty clay) from five locations in southern Iraq. The soil samples were sieved and dried, and their moisture content was measured using the oven-drying method. Clod and bulk shear strength were measured using a shear box and linear shrinkage apparatus, while soil consistency limits were determined using various methods. The findings revealed a decrease in clod shear strength with increasing soil moisture content, particularly in the hard soil state. In contrast, bulk shear strength was lower in the hard state and increased with moisture content. The friable state was found to be the best for tillage operations, with the lowest soil shear strength. The intersection point between bulk and clod shear strength curves was identified as the area with the weakest soil. Based on these results, Aday, S. (1993) recommends conducting tillage operations in the friable state, with a moisture content of 14 to 18% for high clay fraction soils and 14 to 20% for heavy soils to reduce power requirements and potential damage to the soil structure. Further research is needed to determine the most effective methods for determining soil consistency limits, as they may vary based on soil texture and other factors.

Dahunsi, B.I. (2012) assessed the effects of natural moisture content on selected engineering properties of soils on the Irrua-Auchi road in Nigeria. Soil samples were collected at one-kilometer intervals, and both condition survey and laboratory tests were conducted using standard procedures. The findings revealed a significant relationship between natural moisture content and density index, while the relationships between optimum moisture content and density index, and plasticity index and density index were not significant. This suggests that natural moisture content can be used as a quick assessment tool for evaluating pavement materials before use in road construction. The study also highlighted common problems in road construction in developing countries, such as faulty design, poor construction techniques, and corruption. Based on these findings, the authors recommend using natural moisture content as a means of evaluating road failure and improving the quality of road construction.

P. Pezowicz and K. Choma-Moryl (2015) investigated the impact of moisture content on the mechanical properties of cohesive soils from the southern part of the Wielkopolskie Voivodeship in Poland. Two series of soil samples with different consistency levels were analyzed using soil paste. Physical properties, such as moisture content, bulk density, plasticity limit, and granulometric content, were determined via laboratory tests on samples collected from three locations in the region. The results showed that increasing moisture content led to a decrease in the angle of shearing resistance and cohesion, as well as an increase in compressibility of the soil. This was observed in both series of tests, regardless of the soil consistency. The clay fraction was also found to have an impact on these properties. Based on these findings, the authors recommend considering moisture content and clay fraction when using cohesive soils for construction purposes, as the transition from stiff to soft consistency can significantly diminish their mechanical properties. They also suggest further research on the mechanical properties of cohesive soils, particularly in relation to their application as waste landfill liners or in hydro-engineering structures.

Dasho, D.K. and Verma, R.K. (2019) conducted a geotechnical investigation and determine the effect of moisture content on subgrade CBR values for the Arbaminch-Chencha existing road in Ethiopia. Seventeen representative soil samples were collected and laboratory investigations were conducted, including in-situ moisture content, specific gravity, grain size analysis, and Atterberg's limit tests. The soils were then classified and their laboratory moisture-density relationship and CBR values were determined. The study found that the subgrade soils were classified as coarse and fine-grained, with a dominance of fine-grained soils. The laboratory maximum dry density ranged from 1.36 gm/cc to 2.11 gm/cc, and the OMC ranged from 10% to 30%. The CBR values ranged from 3% to 49%, with a specification of 95% MDD. The authors also observed a reduction in CBR values on both the dry and wet side of OMC and MDD, as well as a decrease in CBR values with longer saturation periods. Based on these findings, they recommended conducting geotechnical investigations before road construction, closely monitoring moisture content during construction, and classifying subgrade soils to determine their geotechnical properties for pavement design and performance.

Alsudays, O.M. (2022) studied to assess the effect of moisture content on the compressive strength of cement-stabilized local rammed earth construction material. Materials from the southeast of the Qassim region in Saudi Arabia, including sand, gravel, clay, and Type-I Portland cement, were used to create samples with varying levels of moisture content (4%, 6%, 10%, and 14%). The study found that moisture content significantly affects the compressive strength of stabilized rammed earth materials, with samples containing 10% moisture content having the highest compressive strength of 4 MPa and those with 4% moisture content having the lowest of 1.97 MPa. Based on these findings, the study recommends using a moisture content of 10% for optimal compressive strength and suggests further research on incorporating natural and synthetic fibers to improve the mechanical properties of rammed earth materials. This can help reduce the negative environmental impacts of conventional construction materials and provide a sustainable and economical alternative for building construction.

Shaochun Ma (2023) conducted to investigate the impact of soil moisture content on the properties of foundation soils supporting cultural relics. The objective of the study was to provide insights for the formulation and implementation of repair schemes for cultural relics in order to protect them. The study employed unconfined compression, triaxial, and consolidation tests on soil samples with varying moisture contents. The results revealed that soil moisture content exerts a significant influence on the strength and deformation properties of foundation soils. Moreover, specific regions were identified where mechanical parameters were particularly sensitive to moisture content, thereby affecting the stability and safety of overlying structures. Consequently, the study recommends that special attention be given to the moisture content of foundation soils in cultural relic protection projects. These findings are crucial for understanding and addressing the challenges associated with preserving cultural relics in areas impacted by the Yellow River.

Jianhua Wu (2023) examined the influence of moisture content and dry density on the compressibility of disturbed loess in Yan'an City, China. The study aimed to enhance understanding of the engineering properties of disturbed loess in the Chinese Loess Plateau and address potential environmental and geologic issues. The researchers collected 135 groups of disturbed loess samples from Yan'an City and conducted laboratory tests to analyze their physical properties. High-pressure consolidation experiments were performed to investigate the compression properties of loess with varying moisture contents and dry densities. The experimental results indicated that the optimal compaction performance of disturbed loess was achieved at a 16% water content rate. The compressive deformation coefficient generally decreased with increasing dry density and water content. However, in cases of low soil moisture, a small amount of water and salt concentrated at the contact position of the powder, resulting in increased intensity and compression-deformation coefficient with dryness density within the small load range. Based on these findings, the researchers recommend considering the moisture content and dry density of disturbed loess in engineering construction within the Chinese Loess Plateau. They suggest maintaining a 16% water content rate for optimal compaction performance and considering the effects of low soil moisture on the compressibility and strength of the loess. Further research is needed to gain a comprehensive understanding of the intricate interactions between water and soil, as well as their implications for the compressibility of loess under different conditions.

2.5.2 Strength Properties

Skok, E.L. et al (2003) studied to gain a comprehensive understanding of the techniques and materials used for subgrade enhancement in Minnesota and develop best practice recommendations for cities and counties in the state. The research team utilized various methods to gather information, including questionnaires to 40 counties and 17 cities, agency visits, and creating a database of existing subgrade enhancement installations. They also reviewed literature and consulted with experts in the field. The findings revealed a range of materials and techniques used, such as drainage, compaction, moisture content adjustment, modification, stabilization, reinforcement, separation, and substitution. The use of geosynthetics and alternative materials, like wood chips and shredded tires, is also increasing. Based on their findings, the research team developed best practice summaries and recommendations for each method, considering different soil types and moisture conditions. They emphasized the importance of proper documentation and monitoring for cost-effectiveness and following pollution control guidelines when using alternative materials.

Teklay, A. et al (2015) studied to investigate the applicability of conventional laboratory testing, classification, and evaluation systems on the geotechnical properties of tropically weathered residual laterite soils in Ethiopia. The study was conducted on laterite soils sampled from a highway project in western Ethiopia, utilizing conventional laboratory testing procedures and classification systems, as well as a combination of classification based on structure, mineralogical composition, and geo-morphological impact on soil formation. The findings revealed that residual laterite soils laterite soils in Ethiopia are sensitive to handling and disturbance, and revisions to sample preparation and testing procedures are necessary. The plasticity chart should be used as a guidance tool only, and the United States Classification System (USCS) chart was found to be inappropriate for laterite soils. The authors recommend further research on developing more precise definitions, evaluation, and standardization of appropriate characterizations for residual laterite soils laterite soils, as well as using a combination of classification methods for better evaluation of their engineering properties.

Kollaros, G. (2017) studied to assess the suitability of soils for pavement subgrade in Xanthi, Greece, using gyratory compaction and bearing capacity testing. Soil samples from four different locations were collected and tested for grain size distribution, Atterberg limits, and relationship between density and moisture. Cylindrical specimens were formed using both dynamic hammer compaction and gyratory compaction, and CBR tests were also conducted. The results showed that all soil samples were classified as A-2-4 soils, with OMC values ranging from 10.3% to 16.0% and MDD values ranging from 2040 kg/m^3 to 2280 kg/m^3. The compaction curves obtained from the standard Proctor test and gyratory compactor were generally similar, with the gyratory specimens yielding higher densities at higher moisture contents. CBR tests showed a decrease in bearing capacity with an increase in moisture content, with greater differences in samples prepared with dynamic compaction. Based on these findings, the use of the gyratory compactor was recommended as a feasible means of laboratory compaction for subgrade soils, and further research was suggested to consider the number of gyrations and confinement pressure as important parameters for compaction.

Andreasson, D. (2018) made the assessment of undrained shear strength from Atterberg limits and derived indices. The assessment of shear strength from Atterberg limits is closely related to the development of empirical calculations of shear strength from both CPT and VST. Studies by Skempton and Northey (1952) and by Schofiled and Wrother (1968) led Wroth and Wood (1978) to propose a new equation to estimate shear strength from liquidity index for clayey sediments (Vardanega and Haigh, 2014).

$C_u = 170e^{-4.6\ I_L}$ (kPa).. *Equation*.2.5

This equation indicates that the undrained shear strength should be 170 kPa at the plastic limit and 1.7 kPa at the liquid limit (Wroth and Wood, 1978). Later, an alternative correlation was proposed by Leroueil et al. (1983) which is applicable for liquidity indices between 0.5 and 2.5.

$C_u = 1/(I_L-0.21)^2$ (kPa), $0.5 < I_L < 2.5$……………………………………….*Equation* 2.6

Where, C_u is the undrained shear strength, e is a mathematical constant and I_L is the liquidity index.

Locat and Damers (1988) proposed an equation suited for sensitive clays with high liquidity indices:

$C_u = (1.167/I_L)^{2.44}$ (kPa), $1.5 < I_L < 6.0$…………………………….. *Equation* 2.7

Muhmed, A. et al (2023) studied was to evaluate the changes in microstructure and strength of highly reactive clays treated with lime over a period of curing time and varying moisture content. Three series of testing were conducted, including Unconfined Compressive Strength tests and Scanning Electron Microscopy, on specimens with different bentonite content and varying moisture content and curing temperatures. The results showed that the strength of treated bentonite increased with higher moisture content, curing time, and temperature, but substituting bentonite with sand led to a reduction in strength. California Bearing Ratio and Resilient Modulus values were also significantly enhanced with lime treatment. Microstructural analysis provided visual evidence of improved strength due to the pozzolanic reaction being affected by moisture content. Based on these findings, the study recommends compacting lime treated expansive clays with moderately higher moisture content for a significant enhancement in strength over the curing period.

2.6 Effect of Drying Methods in Plasticity Nature and Classification of Soil

Alias, R. (2018) studied to determine the effect of pre-drying methods on the Atterberg limits of granite residual soils in terms of plastic limit and liquid limit. The study collected granite residual soil samples from an area near the civil engineering laboratory building at Universiti Teknologi MARA Pahang and used three different pre-drying methods: 3 days air-drying at 37°C, 24 hours oven-drying at 50°C, and 24 hours oven-drying at 110°C. The plastic limit test and cone penetration test were conducted on the samples. The results showed that the plastic limit and liquid limit values decreased for the samples that were dried using the 24 hours oven-drying at 110°C method. Therefore, the study concluded that the air-drying method should be used instead of the oven-drying method in preparing soil samples for Atterberg limit tests, as it can lead to more accurate and reliable results. Furthermore, the study recommends conducting further research to compare the effects of different pre-drying methods on other properties of residual soils.

Al-Adhadh, A.R. et al (2019) investigated the impact of different soil drying methods on Atterberg limits and soil classification. The authors' objective was to determine the correlation between the drying method of soil and the values of liquid and plastic limits, and how it affects the soil classification. To achieve this, the authors collected soil samples from different cities in Iraq and used two methods of drying - oven drying and air weather drying. The soil was then tested for liquid and plastic limits according to ASTM standards. The results showed that the drying method had a significant effect on the consistency limits and subsequently on the soil classification. Based on their findings, the authors recommend using the multipoint liquid limit method and the hand roll method for plastic limit tests, as these were found to be more accurate. Overall, the article emphasizes the importance of considering the method of soil drying when conducting Atterberg limit tests, and how it can impact the classification of soil and subsequent geotechnical design. The authors suggest conducting further research in this area to gain more insights into the correlation between drying methods and consistency limits. Al-Adhadh, A.R. et al (2020) once again examined the impact of different methods of soil drying on the Atterberg limits and soil classification. The study aimed to investigate the effect of drying methods on the liquid and plastic limits of soil and determine the significance of this effect on soil classification. Soil specimens were collected from various cities in Iraq and tested using the one-point and multi-point liquid limit tests, as well as the hand roll method for plastic limit determination. The results showed that the method of soil drying has a significant effect on the Atterberg limits and soil classification, with higher liquid and plastic limits observed for soil dried in an oven compared to air-dried soil. This difference in limits can affect the soil classification, highlighting the importance of considering the drying method when conducting consistency tests for accurate geotechnical design. The researchers suggest further research in this area to gain more insights into the correlation between drying methods and soil consistency.

2.7 Embankment Construction Materials

According to ERA Geotechnical Design Manual (2013), Earth-fill embankments are typically built by compacting earthen materials such as natural soils. Hence the compaction properties of the soil materials (optimum water content and maximum dry density) are very important to the long-term performance of the embankments. Compressibility and shear strength can also be useful measures to determine the stability of earth embankments. In addition, drainage is an important issue to prevent the loss of shear strength due to saturation. The materials used in embankments typically consist of soil, aggregates, or rock materials. Embankments can be built on natural firm ground or by replacing the upper soft soil with a compacted fill of stronger soil that meets specific requirements. The behavior of embankments is influenced by the position of the groundwater table throughout their lifespan. The choice of soil types and compaction requirements for embankments is also influenced by the groundwater level. There are several types of embankment construction methods used in civil engineering and infrastructure projects. Among these, earth embankments are constructed by compacting layers of soil or fill material, such as clay, sand, or rock, to form the desired shape and slope. This is the most common type of embankment construction and is used in various applications, including road and railway construction, dams, and levees. The choice of embankment construction method is influenced by several factors.

According to ERA Site Investigation Manual (2013) also the selection of materials for embankment construction is primarily based on construction methods, standards and the availability of materials at a reasonable cost. The limited knowledge of material properties also influences material usage. Engineers consider several factors when determining the appropriate fill material for a particular project. The selection process involves evaluating project-specific requirements, site conditions, material properties, and considering various engineering considerations.

Here are some key factors that determine the selection of a particular embankment construction method:

- Soil Type and Properties – The characteristics of the soil or fill material at the construction site play a crucial role in determining the appropriate construction method. Factors such as soil composition, density, strength, in-situ moisture content and permeability influence the choice of construction method. For example, cohesive soils may require different construction techniques compared to granular soils.
- Site Conditions – The specific conditions at the construction site, including topography, water table, presence of groundwater, and environmental factors, are important considerations. Steep slopes, high water tables, or areas prone to erosion may require specialized construction methods or additional reinforcement.
- Project Requirements – The requirements of the project itself, such as the intended use of the embankment and the desired design life, play a significant role in selecting the construction method. Different projects have different load-bearing capacity requirements, durability expectations, and design standards, which influence the choice of construction method.
- Availability of Construction Materials – The availability of suitable construction materials, such as soil, rock, or fill material, can impact the choice of construction method. If the project site has limited access to specific materials, alternative methods may need to be considered.
- Time and Cost Considerations – Construction time and budget constraints are crucial factors in determining the choice of construction method. Some methods may require more time, specialized equipment, or skilled labor, which can affect the overall project schedule and cost.
- Environmental Impact – Environmental considerations, including the impact on natural habitats, ecosystems, and water bodies, are increasingly important in modern construction practices. The choice of embankment construction method

should consider minimizing environmental disturbance, erosion control measures, and compliance with relevant regulations.

- Engineering Expertise and Experience – The availability of engineering expertise and experience in a particular construction method also influences the choice. If the project team has extensive experience and knowledge of a specific method, it may be preferred due to familiarity, efficiency, and proven success.

Kollaros, G (2017) studied to assess the suitability of soils for pavement subgrade using gyratory compaction and bearing capacity testing. Four soil samples from different locations were collected and tested for Atterberg limits, moisture-density relationship, and California Bearing Ratio (CBR), with the soils classified as A-2-4 according to the AASHTO soil classification system. The results showed that the OMC values ranged from 10.3% to 16.0% and MDD values ranged from 2040 kg/m^3 to 2280 kg/m^3. Gyratory compaction was performed using a 500 kPa vertical stress at 30 rpm and a fixed gyration angle of 1.15°, and CBR tests were conducted using both standard Proctor and gyratory compaction methods. The researchers recommend using gyratory compaction as a feasible means of laboratory compaction for soils in highway projects and suggest considering the number of gyrations and confinement pressure as important parameters. Further research can be conducted to optimize the compaction parameters and compare the results with those obtained from field compaction.

Andreasson, D. (2018) focuses on the assessment of using the liquidity index for the approximation of undrained shear strength of clay tills in Scania. The objectives of the study were to investigate the potential relationship between the liquidity index and undrained shear strength, and to determine the consistency of clay-rich sediments in the province of Scania. The materials and methods used in the study included field and laboratory methods, such as disturbed sampling, cone penetration tests, and vane shear tests. The results showed a wide range of values for both the liquidity index and undrained shear strength. The study also found a partial relationship between the two parameters, with a stronger correlation at the Simrishamn site. Based on these findings, the author recommends further research to better understand the relationship between the liquidity index and undrained shear strength in clay-rich sediments.

Ugwuanyi, H.K. and Onyelowe, K.C. (2019) studied to investigate the suitability of Olokoro and Amaoba area lateritic soils as pavement construction materials. This was achieved by conducting preliminary tests (natural moisture content, specific gravity, particle size analysis, and Atterberg limits) and strength tests (compaction and California Bearing Ratio, CBR) on soil samples collected from the borrow sites. The results showed that both soils have similar characteristics, with a liquid limit of 44% and 40%, plastic limit of 30% and 18%, and plasticity index of 14% and 22%, respectively. However, modifications would be needed to improve the strength of these soils for use as sub base and base materials in pavement construction. Therefore, the study recommends further research and testing to determine the most effective methods for stabilizing these soils for use in pavement construction. It also suggests that both Olokoro and Amaoba lateritic soils can be used as fill materials in road construction.

Sorsa, A. et al (2020) investigates the characterization of subgrade soils in Jimma Town, Ethiopia for the purpose of designing roadways. The study aimed to determine the engineering properties of subgrade soils in the area by conducting various tests on both disturbed and undisturbed soil samples. The results showed that the predominant subgrade soil type in Jimma Town is soft clay, with low load-bearing capacity, high plasticity, low strength, and high compressibility. This makes the soil unsuitable for use as a highway subgrade without the implementation of soil improvement techniques. Based on these findings, the authors recommend implementing proper soil improvement techniques in the design and construction of roadways in Jimma Town and conducting further research on the properties of soft soils in the area. The study highlights the importance of adequate geotechnical investigations and soil characterization in preventing over-design, unexpected excavations, cost overruns, and construction delays. Overall, this study provides valuable insights into the engineering properties of subgrade soils in Jimma Town, Ethiopia and emphasizes the importance of proper soil characterization in the design and construction of civil engineering structures.

Nwaiwu, C.M. et al (2020) assessed the suitability of fifteen samples of lateritic soils for highway construction in Anambra State, Nigeria. The study conducted laboratory tests on the soil samples to obtain their index properties, compaction characteristics, and California Bearing Ratio (CBR). The findings showed that the soil samples belonged to the group of ferruginous tropical soils derived from acid igneous and metamorphic rocks and were classified as silty sand (SM) or silty sand/clayey sand (SM-SC) based on the Unified Soil Classification System (USCS) and as silty soils (A-4) or silty/clayey gravel and sand (A-2-4) based on the American Association of State Highway and Transportation Officials (AASHTO) classification. The study recommends further investigations to determine the suitability of the lateritic soils for different engineering purposes and proper selection and compaction methods to minimize the potential for road failures.

Karakan, E. (2022) studied to determine a reliable method for obtaining the undrained shear strength of remolded clay mixtures using Atterberg limit test results in various states of consistency. The study conducted an experimental study using a wide range of clay mixtures with varying plasticity and geological origin and measured the undrained shear strength based on the cone penetration depth, water content, flow index, liquidity index, and log liquidity index. The findings showed that the highest undrained shear strength was observed in 100% Na-montmorillonite at 171.89 kPa and 56.60% water content, while the lowest undrained shear strength was observed in 100% Sepiolite at 9.28 kPa and 31.65% water content. The study also proposed equations for the liquid limit-flow index and plasticity index-flow index. Based on these findings, it is recommended to consider the interdependence between undrained shear strength, liquidity index, log liquidity index, and flow index when determining the engineering properties of soils. It is also suggested to use the Fall cone method as a more reliable and reproducible method for determining the liquid limit of fine-grained soils and to conduct further research to explore the correlation between the plasticity index and flow index in different types of clay mixtures.

In summary, the natural moisture contents (NMC) are compared with the OMC, LL and PL. The OMC is an ideal moisture content at which neither drying nor wetting is needed. It is also a perfect moisture content at which the maximum dry density (MDD) and then the highest CBR could be mobilized through compaction. Moisture content above optimum implies that the excavated roadway materials requires drying for their use both as subgrade and embankment fill. Here an important question is how high should the NMC be above the optimum and for how long should it be dried, days weak or months, so that a reasonable construction progress could be attained. Big and challenging Question. An attempt is made to visualize the impact of water content above OMC on compaction and hence on the CBR of materials. The OMC is the moisture content at which the Maximum Dry Density (MDD) is attained. moisture content to the left (<OMC) or to the right (>OMC) of the OMC both lead to lower dry density.

2.7.1 Classification and Application of Consistency Parameters for Suitability Analysis of Embankment Fill Materials

According to Nowak, P. (2015), the classification of the earthwork materials involved during excavation, transportation and deposition can vary, hence soil may be classified at any of the following stages:

- In situ – classification in undisturbed condition prior to excavation;
- On excavation – disturbed material after excavation; and
- On stockpile or deposition – classification following placing and prior to compaction.

The option for classifying soil should be selected which is most appropriate for the particular project logistics and materials to be worked with, and should maximize the potential to win suitable fill from the site. The consistency (Atterberg) limits are mainly used to determine the behavior of fine-grained soils at various moisture contents. When we compare the natural moisture content of the soil with the consistency limits, we may get an estimate of its strength. For the sake of this comparison, it is beneficial to use some dimensionless coefficients (A. Palali, 2006). Among these, either of the "liquidity index" in equation (3) and the "consistency index" in equation (4) are often used prior to direct use as fill material.

The purpose of assessing the consistency parameters is to get an idea of the suitability of a cohesive soil for engineering purposes. It must be remembered that apart from the natural moisture content and the shrinkage limit, tests are carried out on reconstructed materials. Because of the empirical nature of the experiments, the achievement of standardized results depends largely on the skill of the person carrying them out. Despite the inherent errors a reasonable, notional, estimate can be made of the consistency and the type of soil at hand (B. Bodo, 2013). The relationship between the consistency index and water content of a particular soil may be illustrated as in figure-12 below.

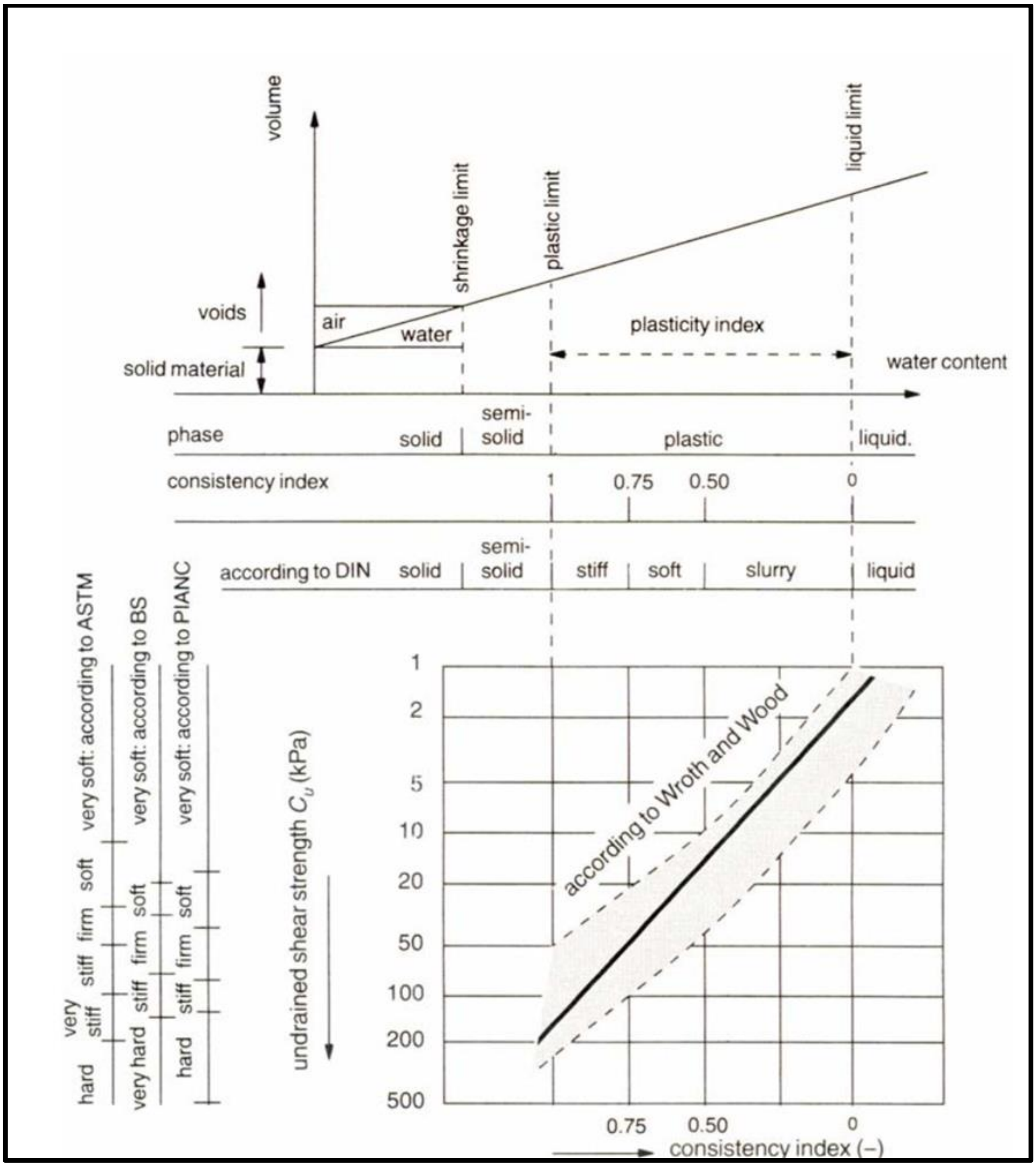

Figure 11: Relationship between undrained shear strength and the consistency index (CUR, 1996)

The in-situ moisture content for a sensitive clay may be greater than the liquid limit, NMC > LL. When these soils remolded, can be transformed into a viscous form to flow like a liquid. While, soil deposits that are heavily over-consolidated may have an in-situ moisture content less than the plastic limit, NMC < PL (K. Terzaghi, 1925).

Table 7: Consistency Parameters and States of Clay Materials (After Terzaghi, K. (1925), Venkatramaiah (2006), K. Özaydın et al (1997) and Bodo, B. (2013))

Consistency Stage	Consistency Index (I_C)	Liquidity Index (I_L)	Material State
Viscous Liquid	< 0	> 1	Liquid
Very Soft	0 – 0.25	0.76 – 1	Plastic
Soft	0.26 – 0.50	0.51 – 0.75	
Medium Stiff	0.51 – 0.75	0.26 – 0.50	
Stiff	0.76– 1	0.25 – 0	
Very Stiff to Hard	> 1	< 0	Solid

Table 8: Consistency of Clay In terms of Unconfined Compressive Strebgth - UCS (K. Terzaghi: Soil Mechanics in Engineering Practice, 3rd Edition)

Consistency Stage	Unconfined Compressive Strength, q_u (kN/m^2)
Viscous Liquid	< 0
Very Soft	0 – 25
Soft	26 – 50
Medium Stiff	51 – 100
Stiff	101 – 200
Very Stiff	201 – 400
Hard to Very Hard	> 400

The quality requirement stated on ERA Standard Specification (2013) for roadbed preparation, subgrade at fill and cut sections and fill materials and replacement excluding rock fill is given in the following table.

Table 9: Summary of Quality Requirements of Fill Materials (ERA Specification, 2013)

Item No.	Test Description	Project Specification for Embankment Fill Materials		Test Method
		Minimum Requirement	Maximum Requirement	
Quality Requirements for Fill Materials				
1	CBR (%)	3	-	AASHTO T-193
2	Swell (%)	-	2	AASHTO T-193
3	LL (%)	-	60	AASHTO T-90
4	PI (%)	-	30	AASHTO T-89
5	Field Compaction	95% of MDD	-	AASHTO T-191

2.7.2 Clay as Embankment Material

The quality of the soil being excavated can vary as a result of changes in ground condition, in soil type or in the weather due to the sensitivity for changes in water content. Control of the quality of the material is necessary to qualify the suitability of the material. Allowance has to be made, however, for the effects of weather conditions on the quality of the soil during the construction process. The suitability criterion is determined from considerations of trafficability by earthmoving plants, embankment stability, and embankment settlement. Clay is commonly used as embankment material in various civil engineering projects, including the construction of dams, levees, and earth-filled structures. Its properties make it suitable for creating stable embankments that can withstand the forces of water, weather, and the weight of the soil itself. The assessment of clayey fill materials is necessary for the construction of embankments. The process involves excavating, transporting, placing, and compacting the fill material. It is important to consider the workability and suitability of the fill material in order to estimate the cost and time required for construction. The efficiency and costs of embankment construction are influenced by the ability of construction equipment to operate effectively and the impact of plant operations on the quality of the fill material (Dennehy, 1978).

Clay is commonly used as a fill material in embankments, and its water content and density play a crucial role. To ensure that the embankment functions as intended, it is necessary to control the quality of the fill material. The specifications aim to address the stability and settlement requirements, impermeability for long-term stability and potential difficulties in operation. However, there are some considerations and challenges associated with using clay as embankment material for roads as embankment fill material. Here are some of the main challenges associated with using clay for road embankments:

- **Poor Load-Bearing Capacity**: Clay has relatively low strength compared to other fill materials such as granular soils. It may not have the required load-bearing capacity to support heavy traffic loads, leading to deformation, settlement, and rutting of the road surface. This can result in a rough ride for vehicles and increased maintenance costs.
- **High Plasticity and Shrink-Swell Behavior:** Clay exhibits high plasticity, meaning it is highly susceptible to changes in moisture content. During wet periods, clay can become soft and lose its strength, leading to instability and reduced load-bearing capacity. Conversely, during dry periods, clay can shrink and crack, resulting in surface irregularities and potential damage to the road structure.
- **Subsidence and Differential Settlement**: Clay embankments are prone to subsidence and differential settlement due to their compressible nature. Differential settlement can cause unevenness in the road surface, leading to a rough ride and safety hazards for vehicles. It can also result in the development of potholes and pavement distress.
- **Slope Stability**: Clay embankments may be susceptible to slope failures, especially if they have steep slopes or if there are weak layers or discontinuities within the clay. Slope instability can lead to slope failures, landslides, and road closures, posing a risk

to road users. Detailed geotechnical investigations and appropriate slope stability analyses are crucial to ensure safe embankment design.

- **Drainage Issues**: Clay has low permeability, which means it has limited ability to drain water. Poor drainage can result in prolonged periods of moisture, leading to softening of the clay and reduced stability. It can also contribute to the development of potholes and surface deterioration in the road pavement. Adequate drainage measures, such as the inclusion of geosynthetic materials or installation of drainage systems, are necessary to mitigate these issues.
- **Construction Difficulties**: Working with clay as a fill material during road construction can be challenging. Wet clay can be cohesive and sticky, making it difficult to compact and shape. It can also cause equipment to become clogged and slow down construction progress. Proper moisture control, suitable construction techniques, and the use of additives to improve workability may be necessary to overcome these challenges.
- **Long-term maintenance**: Clay embankments may require ongoing monitoring and maintenance to address settlement, erosion, and other potential issues. Regular inspections and appropriate remedial measures should be implemented to ensure the embankment's integrity over time.

To address these challenges, it is important to conduct thorough geotechnical investigations, employ appropriate engineering techniques, and follow best practices for embankment design and construction. It's important to note that the specific design and construction considerations for clay embankments may vary depending on the project requirements, site conditions, and engineering standards to be used. Similarly, the roadway excavated clay fill materials play a significant role in maintaining cost-effectiveness and environmental sustainability of road embankment construction of a particular project. However, the plasticity property of clay fill materials plays a significant role in determining their suitability for various applications. The plasticity of clay fill materials is typically quantified using the Atterberg limits, which include the liquid limit, plastic limit, and shrinkage limit. These tests help classify the clay and provide valuable information about its behavior. The plasticity of clay fill materials is typically quantified using the Atterberg limits, which include the liquid limit, plastic limit, and shrinkage limit.

These tests help classify the clay and provide valuable information about its behavior (Dennehy, 1978).

In summary, the plasticity property of clay fill materials has both positive and negative implications for their suitability. While plasticity allows for easier compaction and improved strength, it can also lead to shrinkage, swelling, and potential stability issues. The specific requirements of the project and the potential consequences of plasticity-related behaviors should be carefully considered when evaluating the suitability of clay fill materials.

2.8 Effect of Moisture Content in Stress Distribution in Embankments

Slope of embankments tend to be finished off with a surface covering called liner. This surface covering is commonly constructed with clayey soil. The clayey soil in the unsaturated zone is subject to swelling and shrinkage resulting in fractures. Repeated extreme wetting and drying conditions promote further fracturing. Consequently, surface clay is very permeable. As water infiltrates the embankment, instability of the embankment can occur due to slope softening (Day and Axten, 1990; Kyfor and Gemme, 1994). To prevent such action the deeper layers, have to be impermeable. Intense compaction in the deeper layers can reduce the water infiltration. The water content of a compacted soil in a construction will tend to an equilibrium range. If soil is compacted at too high a water content compared to this equilibrium range, a permanent volume reduction result. This shrinkage will result in permanent wide cracks. Thus, clays are best compacted at water contents at approximately a longer-term equilibrium water content related to the average negative pore pressure at the location in the construction. In the Netherlands, a proper water content of clay liners during compaction is less than the water content at $I_C = 0.75$ and for clay in the core of embankments $I_C = 0.6$. For temperate climate conditions such as France's summer $I_C = 0.8$ (Kruse and Nieuwenhuis, 1998).

According to ERA Geotechnical Design Manual (2013), if the moisture content of a clay is closer to the plastic limit than the liquid limit, the soil is likely to be over-consolidated. This means that in the past the clay was subjected to a greater stress than now exists. Over-consolidation can be due to the weight of a natural soil deposit that has since been eroded away, the weight of a previously placed fill that is removed, loads due to structures that have been demolished, or due to desiccation. Normally consolidated soils undergo large settlements compared to over-consolidated soils.

In Ethiopia, residual soils are found in many parts of the country usually associated with basaltic rocks that can weather easily. Pore pressures in residual soil slopes often react quickly to heavy rainfall and any infiltration can eliminate soil tensions and increase positive pore pressure by raising either perched water or groundwater tables. On slopes, failure can occur either during or sometime after the rainfall ends. Along roads, any percolation from the top of cuttings can lead to softening and slope failure.

CHAPTER THREE: MATERIALS AND METHODS

This section explains the materials and methods used in the entire process of the study. It also indicates the relationship between the different work steps of the investigations and the experimental tests being carried out.

1.1. Methods

1.1.1. Standards To be Used

The soil testing was conducted at the laboratory level, following the established standards. The AASHTO and ASTM standards were interchangeably used for sampling and testing procedures, whereas the ERA Standard Technical Specifications, Flexible Pavement Design Manuals and other relevant studies were used for making interpretation and discussion of the obtained results accordingly. The previous studies, which have been reviewed and elaborated in Section-2, were used for results discussion and comparison purposes. Additionally, the Casagrande's procedure was employed for the classification of fine-grained soils in the USC system using the plasticity chart. Furthermore, Casagrande's suggestion for the estimation of shrinkage limit from the plasticity chart was used (adopted from Holtz et al, 1981).

1.1.2. Sampling Method

The non-random, systematic method of sampling technique was employed for the study. This means that representative soil samples were collected from 34 (thirty four) different lots (every lot at c/c 300 meters) along the roadway section starting from km 22+000, with one sample collected at every lot within the 10 km distance.

1.1.3. Sample Preparation

The samples for testing were prepared as air-dried samples, based on the procedures mentioned under AASHTO or ASTM soil testing manuals.

1.1.4. Experimental Program

The following soil test requirements and procedures were carried out as illustrated in the table below.

Table 10: Types of Tests to be used

Test Description	AASHTO Code	ASTM Code	Remark
Preparation of Soil Sample	T 87		
Natural Moisture Content	T-265	D-2216	
Particle Size Distribution	T-88	D-422	
Compaction	T-180	D-1557	
Liquid Limit	T-89	D-4318	
Plastic Limit	T-90	D-4318	
Shrinkage Limit	T-92	D-4943	Casagrande's Suggestion to be used instead
CBR (3- Point)	T-193	D-1883	
Classification of Soils	T 145	D-2487	D-2487 is for USCS

1.1.5. Method of Data Analysis and Interpretation

When analyzing soil laboratory test data for research purposes, there are several methods can be considered. The choice of method is depending on the specific objectives of the research, the type of soil data collected, and the statistical techniques that are comfortable with. Hence, this study followed Statistical analysis or Data Visualization technique (plotting graphs or charts) to visually represent the soil data. Visualizations can help identify patterns, outliers, and relationships that may not be apparent from raw data. The other method of data analysis was Comparative analysis to compared with established standards to identify any variations or deviations.

To employ the captioned methods of data analysis, the author used MS-Excel 2019 application software.

The suitability test was carried out on roadway-excavated samples satisfying the specification requirements set in the works contract. To easy the screening processes the following steps were followed for results for data analysis and results interpretation:

- Carry out LL test,
- If the criteria for LL ≤ 60% is satisfied then,
- Carry out 3-point CBR and Swell tests,
- If the materials satisfy the minimum CBR at 95% compaction and possess Swell not more than 2% then compute,
- Consistency Index to determine suitability in terms of NMC,
- If it passes the Suitability test in terms of Consistency Index then the material/soil can be considered OMC and used for embankment fill, replacement and at subgrade level.

1.2. Materials

1.2.1. Sample Size

The size of collected samples from each lot were as summarized in the table below.

Table 11: Sample Sizes

Test Description	Net Size Required (g)	Allowance for Wastage (g)	Total Size Required (g)	Remark
Natural Moisture Content	500	500	1000	
Particle Size Distribution	3000	2000	5000	
Compaction	30000	5000	35000	
Atterberg Limits	700	300	1000	
CBR (3- Point)	15000	5000	20000	
Total Sample Size for One Lot (gm)	48,700	12,300	61,000	

Test Description	Net Size Required (g)	Allowance for Wastage (g)	Total Size Required (g)	Remark
Total Sample Size for the Study (kg)	1,672	435	2,107	

1.2.2. Sampling Tools

The author collected undisturbed soil samples for experimental studies using sacks, shovels, spades and other hand tools as if necessary. Furthermore, the author has arranged a vehicle for the transportation of the collected representative samples to Central Laboratory for Materials Testing of Gondwana Engineering PLC.

The following flow chart of methodological approach describes the overall sequences of activities for the study.

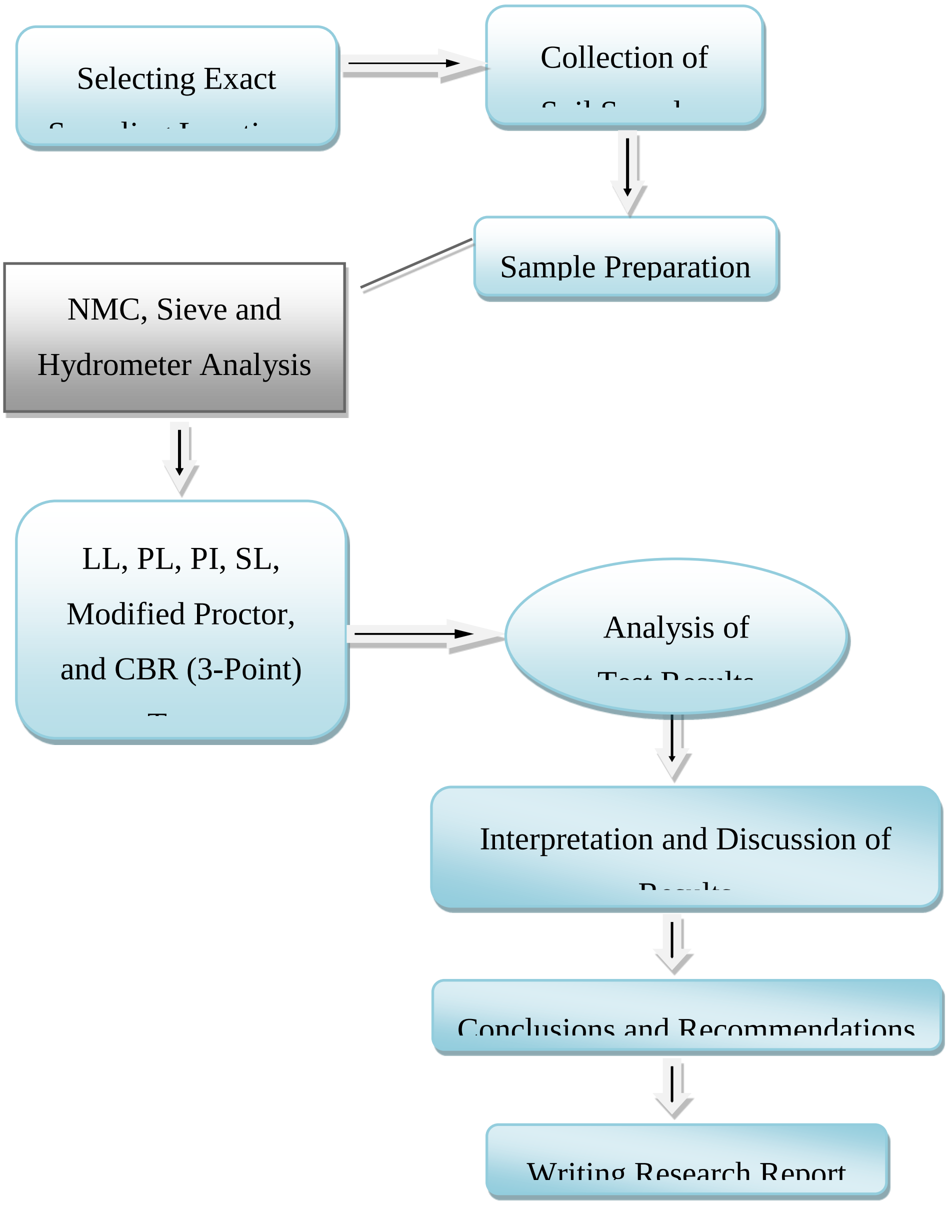

Figure 12: Flow chart of methodological approach for the study.

CHAPTER FOUR: RESULTS AND DISCUSSIONS

4.1 General

This chapter summarizes the obtained soil sample test results of the experimental works, which were carried out to reach the objectives of the study and planned to undergo the suitability analysis of soils from the roadway excavation at the area of study. The discussions and interpretations of the obtained soil test results were stated following the obtained results.

A detailed natural moisture content study had been conducted as stipulated hereunder.

a) A total of 34 samples have been gathered from km 20+940 to 30+940 covering the total of 10 km project length.
b) Samples were collected from the roadway excavation materials.
c) The samples were taken from the stockpiles and roadway at every 300m.

4.2 Soil Samples Laboratory Test Result Summary

The natural moisture contents (NMC) are compared with the OMC, LL and PL. The OMC is an ideal moisture content at which neither drying nor wetting is needed. It is also a perfect moisture content at which the maximum dry density (MDD) and then the highest CBR could be mobilized through compaction. The following table and graphs shown the comparison of the NMC under different scenarios.

Table 12: Grand Summary of Soil Test Results

Grand Summary of Laboratory Test Results				
Sr. No	**Description**	**Frequency of Occurrence**	**% of Occurrence**	**Remark**
1	Suitable Materials in-terms of CBR Value	34	100%	Based on project contact specification
2	Suitable Materials in-terms of LL Value	34	100%	
3	Suitable Materials in-terms of PI Value	34	100%	
4	NMC greater than OMC	34	100%	
5	NMC greater than LL	24	71%	
6	NMC greater than PL	34	100%	
7	Sensitive Soil	24	71%	Based on literature review which are not specified in the project contract specification
8	Plastic Soil	10	29%	
9	Over consolidated Soil	0	0%	

Grand Summary of Laboratory Test Results				
Sr. No	**Description**	**Frequency of Occurrence**	**% of Occurrence**	**Remark**
10	Soil at liquid state	24	71%	
11	Soil at plastic state	10	29%	
12	Soil at solid state	0	0%	
13	Soil at viscous liquid consistency stage	24	71%	
14	Soil at very soft consistency stage	0	0%	
15	Soil at soft consistency stage	4	12%	
16	Soil at medium stiff consistency stage	3	9%	
17	Soil at stiff consistency stage	3	9%	
18	Soil at very stiff consistency stage	0	0%	

Grand Summary of Laboratory Test Results				
Sr. No	**Description**	**Frequency of Occurrence**	**% of Occurrence**	**Remark**
19	Soil at hard to very hard consistency stage	0	0%	
20	UCS (less than 0 kPa)	24	71%	
21	UCS (0 to 25 kPa)	0	0%	
22	UCS (26 to 50 kPa)	4	12%	
23	UCS (51 to 100 kPa)	3	9%	
24	UCS (101 to 200 kPa)	3	9%	
25	UCS (201 to 400 kPa)	0	0%	
26	UCS (More than 400 kPa)	0	0%	

Table 13: Soil Plasticity Properties and Classification

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL										
Sr. №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**Atterberg Limits (AASHTO T-89/T-90)**			**Suitability in-terms of LL**	**Soil Classification**
						LL (%)	**PL (%)**	**PI (%)**		
	From	**To**				**Max 60**	**-**	**Max 30**		
1	Km 22+100	Km 22+400	22.3	01	Reddish clay soil	47	28	19	Suitable	A-7-6(4)
2	Km 22+400	Km 22+700	22.6	02	Brown clay soil	57	29	27	Suitable	A-7-6(5)
3	Km 22+700	Km 23+000	22.9	03	Brown clay soils	57	40	17	Suitable	A-7-5(3)
4	Km 23+000	Km 23+300	23.2	04	Red to brown silty clay soil	46	29	17	Suitable	A-7-5(1)
5	Km 23+300	Km 23+600	23.5	05	Light brown silty clay soil	45	30	15	Suitable	A-2-7(0)

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL										
Sr. №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**Atterberg Limits (AASHTO T-89/T-90)**			**Suitability in-terms of LL**	**Soil Classification**
						LL (%)	**PL (%)**	**PI (%)**		
	From	**To**				**Max 60**	**-**	**Max 30**		
6	Km 23+600	Km 23+900	23.8	06	Yellowish clay soil	55	38	16	Suitable	A-7-5(2)
7	Km 23+900	Km 24+200	24.1	07	Brown silty clay soil with gravel	39	29	9	Suitable	A-4(3)
8	Km 24+200	Km 24+500	24.4	08	Brown silty clay soil	42	31	11	Suitable	A-7-5(3)
9	Km 24+500	Km 24+800	24.7	09	Dark to brown silty clay soil	48	34	14	Suitable	A-7-5(1)
10	Km 24+800	Km 25+100	25.0	10	Yellowish silty clay with gravel	58	41	16	Suitable	A-2-7(0)

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL										
Sr. №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**Atterberg Limits (AASHTO T-89/T-90)**			**Suitability in-terms of LL**	**Soil Classification**
						LL (%)	**PL (%)**	**PI (%)**		
	From	**To**				**Max 60**	**-**	**Max 30**		
11	Km 25+100	Km 25+400	25.3	11	Brown silty clay soil	57	40	17	Suitable	A-7-5(2)
12	Km 25+400	Km 25+700	25.6	12	Brown silty clay soil	58	44	14	Suitable	A-7-5(1)
13	Km 25+700	Km 26+000	25.9	13	Red silty clay soil	58	42	16	Suitable	A-7-5(9)
14	Km 26+000	Km 26+300	26.2	14	Brown to reddish clay soil	56	36	21	Suitable	A-7-5(6)
15	Km 26+300	Km 26+600	26.5	15	Dark silty clay soil	57	40	17	Suitable	A-7-5(4)
16	Km 26+600	Km 26+900	26.8	16	Red silty clay soil	59	40	20	Suitable	A-2-7(1)

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL										
Sr. №	Representation Station (Km)		Mid-Section Station (Km)	Sample Code	Visual Material Classification	Atterberg Limits (AASHTO T-89/T-90)			Suitability in-terms of LL	Soil Classification
						LL (%)	PL (%)	PI (%)		
	From	To				Max 60	-	Max 30		
17	Km 26+900	Km 27+200	27.1	17	Brown silty clay soil	47	33	14	Suitable	A-2-7(0)
18	Km 27+200	Km 27+500	27.4	18	Brown clay soil	57	40	17	Suitable	A-7-5(3)
19	Km 27+500	Km 27+800	27.7	19	Red silty clay soil	56	46	10	Suitable	A-2-5(0)
20	Km 27+800	Km 28+100	28.0	20	Brown silty clay soil	52	35	17	Suitable	A-2-7(1)
21	Km 28+100	Km 28+400	28.3	21	Red silty clay soil	51	34	17	Suitable	A-7-5(1)
22	Km 28+400	Km 28+700	28.6	22	Red silty clay soil	56	39	17	Suitable	A-7-5(2)

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL										
Sr. №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**Atterberg Limits (AASHTO T-89/T-90)**			**Suitability in-terms of LL**	**Soil Classification**
						LL (%)	**PL (%)**	**PI (%)**		
	From	**To**				**Max 60**	**-**	**Max 30**		
23	Km 28+700	Km 29+000	28.9	23	Dark to brown silty clay soil	47	33	14	Suitable	A-7-5(3)
24	Km 29+000	Km 29+300	29.2	24	Light Brown silty clay soil	57	40	17	Suitable	A-2-7(0)
25	Km 29+300	Km 29+600	29.5	25	Brown clay soil	52	35	17	Suitable	A-7-5(2)
26	Km 29+600	Km 29+900	29.8	26	Red brown silty clay soil	53	36	17	Suitable	A-2-7(0)
27	Km 29+900	Km 30+200	30.1	27	Light grey clay material	57	36	21	Suitable	A-7-5(2)

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL											
Sr. №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**Atterberg Limits (AASHTO T-89/T-90)**			**Suitability in-terms of LL**	**Soil Classification**
						LL (%)	**PL (%)**	**PI (%)**		
	From	**To**				**Max 60**	**-**	**Max 30**		
28	Km 30+200	Km 30+500	30.4	28	Light Brown silty clay soil	46	27	19	Suitable	A-7-6(2)
29	Km 30+500	Km 30+800	30.7	29	Dark to brown silty clay soil	50	39	11	Suitable	A-7-5(0)
30	Km 30+800	Km 31+100	31.0	30	Dark to brown silty clay soil	51	37	14	Suitable	A-7-5(2)
31	Km 31+100	Km 31+400	31.3	31	Yellowish to Brown silty clay soil	55	39	16	Suitable	A-2-7(0)
32	Km 31+400	Km 31+700	31.6	32	Dark to brown clay soil	54	39	15	Suitable	A-2-7(1)

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL										
Sr. №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**Atterberg Limits (AASHTO T-89/T-90)**			**Suitability in-terms of LL**	**Soil Classification**
						LL (%)	**PL (%)**	**PI (%)**		
	From	**To**				**Max 60**	**-**	**Max 30**		
33	Km 31+700	Km 32+000	31.9	33	Dark to brown clay soil	55	38	17	Suitable	A-7-5(3)
34	Km 32+000	Km 32+300	32.2	34	Brown silty clay soil	51	37	14	Suitable	A-7-5(2)

Table 14: Soil Plasticity and Moisture-Density Relations

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL														
Sr №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**In-situ NMC (%)**	**Moisture-Density Relationship**		**NMC> OMC**	**NMC>LL (Zero Shear Strength)**	**NMC >PL**	**Project Specification for Suitability in-terms Plasticity Parameters**	
	From	**To**					**OMC (%)**	**MDD (g/cc)**				**LL < 60**	**PI < 30**
1	Km 22+100	Km 22+400	22.3	01	Reddish clay soil	51.0	19.0	1.80	1	1	1	Acceptable	Acceptable
2	Km 22+400	Km 22+700	22.6	02	Brown clay soil	38.1	19.6	1.68	1	0	1	Acceptable	Acceptable
3	Km 22+700	Km 23+000	22.9	03	Brown clay soils	44.2	24.5	1.56	1	0	1	Acceptable	Acceptable
4	Km 23+000	Km 23+300	23.2	04	Red to brown silty clay soil	49.3	27.7	1.48	1	1	1	Acceptable	Acceptable
5	Km 23+300	Km 23+600	23.5	05	Light brown silty clay soil	52.2	28.0	1.51	1	1	1	Acceptable	Acceptable
6	Km 23+600	Km 23+900	23.8	06	Yellowish clay soil	59.4	32.0	1.41	1	1	1	Acceptable	Acceptable
7	Km 23+900	Km 24+200	24.1	07	Brown silty clay soil with gravel	43.5	24.5	1.55	1	1	1	Acceptable	Acceptable

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL														
Sr №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**In-situ NMC (%)**	**Moisture-Density Relationship**		**NMC> OMC**	**NMC>LL (Zero Shear Strength)**	**NMC >PL**	**Project Specification for Suitability in-terms Plasticity Parameters**	
	From	**To**					**OMC (%)**	**MDD (g/cc)**				**LL < 60**	**PI < 30**
8	Km 24+200	Km 24+500	24.4	08	Brown silty clay soil	51.9	25.0	1.49	1	1	1	Acceptable	Acceptable
9	Km 24+500	Km 24+800	24.7	09	Dark to brown silty clay soil	50.6	26.2	1.47	1	1	1	Acceptable	Acceptable
10	Km 24+800	Km 25+100	25.0	10	Yellowish silty clay with gravel	65.0	32.0	1.43	1	1	1	Acceptable	Acceptable
11	Km 25+100	Km 25+400	25.3	11	Brown silty clay soil	50.3	30.0	1.34	1	0	1	Acceptable	Acceptable
12	Km 25+400	Km 25+700	25.6	12	Brown silty clay soil	48.4	24.5	1.38	1	0	1	Acceptable	Acceptable
13	Km 25+700	Km 26+000	25.9	13	Red silty clay soil	59.1	27.5	1.51	1	1	1	Acceptable	Acceptable
14	Km 26+000	Km 26+300	26.2	14	Brown to reddish clay soil	66.7	28.3	1.49	1	1	1	Acceptable	Acceptable

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL													
Sr №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**In-situ NMC (%)**	**Moisture-Density Relationship**		**NMC>OMC**	**NMC>LL (Zero Shear Strength)**	**NMC >PL**	**Project Specification for Suitability in-terms Plasticity Parameters**	
	From	**To**					**OMC (%)**	**MDD (g/cc)**				**LL < 60**	**PI < 30**
15	Km 26+300	Km 26+600	26.5	15	Dark silty clay soil	63.4	25.5	1.47	1	1	1	Acceptable	Acceptable
16	Km 26+600	Km 26+900	26.8	16	Red silty clay soil	44.4	27.2	1.55	1	0	1	Acceptable	Acceptable
17	Km 26+900	Km 27+200	27.1	17	Brown silty clay soil	51.8	30.0	1.45	1	1	1	Acceptable	Acceptable
18	Km 27+200	Km 27+500	27.4	18	Brown clay soil	48.1	26.3	1.47	1	0	1	Acceptable	Acceptable
19	Km 27+500	Km 27+800	27.7	19	Red silty clay soil	61.0	30.2	1.49	1	1	1	Acceptable	Acceptable
20	Km 27+800	Km 28+100	28.0	20	Brown silty clay soil	58.6	23.6	1.56	1	1	1	Acceptable	Acceptable
21	Km 28+100	Km 28+400	28.3	21	Red silty clay soil	58.3	23.6	1.57	1	1	1	Acceptable	Acceptable

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL													
Sr №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**In-situ NMC (%)**	**Moisture-Density Relationship**		**NMC>OMC**	**NMC>LL (Zero Shear Strength)**	**NMC>PL**	**Project Specification for Suitability in-terms Plasticity Parameters**	
	From	**To**					**OMC (%)**	**MDD (g/cc)**				**LL < 60**	**PI < 30**
22	Km 28+400	Km 28+700	28.6	22	Red silty clay soil	57.2	26.5	1.54	1	1	1	Acceptable	Acceptable
23	Km 28+700	Km 29+000	28.9	23	Dark to brown silty clay soil	52.5	20.2	1.66	1	1	1	Acceptable	Acceptable
24	Km 29+000	Km 29+300	29.2	24	Light Brown silty clay soil	42.8	22.6	1.45	1	0	1	Acceptable	Acceptable
25	Km 29+300	Km 29+600	29.5	25	Brown clay soil	57.8	22.5	1.57	1	1	1	Acceptable	Acceptable
26	Km 29+600	Km 29+900	29.8	26	Red brown silty clay soil	55.2	22.7	1.51	1	1	1	Acceptable	Acceptable
27	Km 29+900	Km 30+200	30.1	27	Light grey clay material	59.4	30.1	1.31	1	1	1	Acceptable	Acceptable
28	Km 30+200	Km 30+500	30.4	28	Light Brown silty clay soil	53.1	26.6	1.51	1	1	1	Acceptable	Acceptable

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL														
Sr №	**Representation Station (Km)**		**Mid-Section Station (Km)**	**Sample Code**	**Visual Material Classification**	**In-situ NMC (%)**	**Moisture-Density Relationship**		**NMC> OMC**	**NMC>LL (Zero Shear Strength)**	**NMC >PL**	**Project Specification for Suitability in-terms Plasticity Parameters**	
	From	**To**					**OMC (%)**	**MDD (g/cc)**				**LL < 60**	**PI < 30**
29	Km 30+500	Km 30+800	30.7	29	Dark to brown silty clay soil	58.2	25.0	1.39	1	1	1	Acceptable	Acceptable
30	Km 30+800	Km 31+100	31.0	30	Dark to brown silty clay soil	54.5	40.4	1.20	1	1	1	Acceptable	Acceptable
31	Km 31+100	Km 31+400	31.3	31	Yellowish to Brown silty clay soil	47.8	27.5	1.42	1	0	1	Acceptable	Acceptable
32	Km 31+400	Km 31+700	31.6	32	Dark to brown clay soil	49.0	24.0	1.50	1	0	1	Acceptable	Acceptable
33	Km 31+700	Km 32+000	31.9	33	Dark to brown clay soil	48.5	24.0	1.50	1	0	1	Acceptable	Acceptable
34	Km 32+000	Km 32+300	32.2	34	Brown silty clay soil	54.6	27.0	1.37	1	1	1	Acceptable	Acceptable

Table 15: CBR Values and Analysis

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																		
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**3-Point CBR (%)**												**Suitability in-terms of CBR**
						10 blows			**30 blows**			**65 blows**			**Avg. Swell (%)**	**95% of MDD**	**CBR at 95% MDD**	
	From	**To**				**Density (g/cc)**	**CBR**	**Swell (%)**	**Density (g/cc)**	**CBR**	**Swell (%)**	**Density (g/cc)**	**CBR**	**Swell (%)**				
1	Km 22+100	Km 22+400	Reddish clay soil	01	22.25	1.53	2	0.9	1.72	10	0.8	1.83	13	0.7	0.8	1.71	9	Suitable
2	Km 22+400	Km 22+700	Brown clay soil	02	22.55	1.53	4	2.6	1.62	5	2.1	1.71	6	1.8	2.2	1.60	5	Suitable
3	Km 22+700	Km 23+000	Brown clay soils	03	22.85	1.41	4	2.2	1.53	6	1.9	1.58	9	1.5	1.9	1.48	5	Suitable
4	Km 23+000	Km 23+300	Red to brown silty clay soil	04	23.15	1.33	6	1.3	1.43	9	1.2	1.50	16	0.8	1.1	1.40	8	Suitable
5	Km 23+300	Km 23+600	Light brown silty clay soil	05	23.45	1.30	7	0.9	1.43	16	0.5	1.53	22	0.3	0.6	1.43	15	Suitable
6	Km 23+600	Km 23+900	Yellowish clay soil	06	23.75	1.27	3	1.5	1.37	5	0.7	1.43	6	0.3	0.8	1.34	4	Suitable
7	Km 23+900	Km 24+200	Brown silty clay soil with gravel	07	24.05	1.39	15	1.3	1.52	36	0.3	1.57	43	0.2	0.6	1.47	29	Suitable
8	Km 24+200	Km 24+500	Brown silty clay soil	08	24.35	1.32	9	0.7	1.43	19	0.6	1.50	24	0.3	0.5	1.42	18	Suitable

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																			
Sr. No	Station (km)		Visual Material Classification	Sample Code	Mid-Section Station (Km)	3-Point CBR (%)													Suitability in-terms of CBR
						10 blows			30 blows			65 blows			Avg. Swell (%)	95% of MDD	CBR at 95% MDD		
	From	To				Density (g/cc)	CBR	Swell (%)	Density (g/cc)	CBR	Swell (%)	Density (g/cc)	CBR	Swell (%)					
9	Km 24+500	Km 24+800	Dark to brown silty clay soil	09	24.65	1.27	7	0.9	1.41	13	0.8	1.48	18	0.6	0.8	1.40	12	Suitable
10	Km 24+800	Km 25+100	Yellowish silty clay with gravel	10	24.95	1.23	11	0.7	1.37	30	0.4	1.44	38	0.1	0.4	1.36	29	Suitable
11	Km 25+100	Km 25+400	Brown silty clay soil	11	25.25	1.19	9	0.9	1.32	18	0.8	1.37	25	0.5	0.7	1.27	15	Suitable
12	Km 25+400	Km 25+700	Brown silty clay soil	12	25.55	1.23	15	0.5	1.34	24	0.3	1.39	37	0.3	0.4	1.31	22	Suitable
13	Km 25+700	Km 26+000	Red silty clay soil	13	25.85	1.32	14	0.8	1.44	22	0.6	1.52	32	0.2	0.5	1.43	22	Suitable
14	Km 26+000	Km 26+300	Brown to reddish clay soil	14	26.15	1.25	13	0.8	1.41	17	0.5	1.51	23	0.1	0.5	1.42	18	Suitable
15	Km 26+300	Km 26+600	Dark silty clay soil	15	26.45	1.31	6	3.1	1.41	18	2.1	1.50	21	1.8	2.3	1.40	17	Suitable
16	Km 26+600	Km 26+900	Red silty clay soil	16	26.75	1.37	10	0.7	1.48	19	0.4	1.57	23	0.2	0.4	1.47	19	Suitable
17	Km 26+900	Km 27+200	Brown silty clay soil	17	27.05	1.30	7	0.8	1.40	13	0.7	1.47	29	0.4	0.6	1.38	11	Suitable
18	Km 27+200	Km 27+500	Brown clay soil	18	27.35	1.28	11	0.6	1.43	21	0.5	1.50	32	0.3	0.5	1.39	18	Suitable

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																			
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**3-Point CBR (%)**													**Suitability in-terms of CBR**
						10 blows			**30 blows**			**65 blows**			**Avg. Swell (%)**	**95% of MDD**	**CBR at 95% MDD**		
	From	**To**				**Density (g/cc)**	**CBR**	**Swell (%)**	**Density (g/cc)**	**CBR**	**Swell (%)**	**Density (g/cc)**	**CBR**	**Swell (%)**					
19	Km 27+500	Km 27+800	Red silty clay soil	19	27.65	1.31	9	0.9	1.42	16	0.6	1.50	18	0.2	0.6	1.41	15	Suitable	
20	Km 27+800	Km 28+100	Brown silty clay soil	20	27.95	1.39	11	0.5	1.49	19	0.3	1.58	22	0.1	0.3	1.48	18	Suitable	
21	Km 28+100	Km 28+400	Red silty clay soil	21	28.25	1.38	12	0.8	1.52	25	0.5	1.60	32	0.2	0.5	1.49	22	Suitable	
22	Km 28+400	Km 28+700	Red silty clay soil	22	28.55	1.32	8	0.7	1.48	26	0.4	1.55	34	0.2	0.5	1.46	24	Suitable	
23	Km 28+700	Km 29+000	Dark to brown silty clay soil	23	28.85	1.46	7	0.5	1.60	28	0.4	1.69	35	0.2	0.4	1.58	25	Suitable	
24	Km 29+000	Km 29+300	Light Brown silty clay soil	24	29.15	1.29	9	0.6	1.39	20	0.5	1.47	25	0.3	0.5	1.38	19	Suitable	
25	Km 29+300	Km 29+600	Brown clay soil	25	29.45	1.36	7	0.9	1.50	18	0.7	1.58	28	0.5	0.7	1.49	18	Suitable	
26	Km 29+600	Km 29+900	Red brown silty clay soil	26	29.75	1.29	15	0.8	1.45	20	0.7	1.53	26	0.6	0.7	1.44	20	Suitable	
27	Km 29+900	Km 30+200	Light grey clay material	27	30.05	1.15	15	1.4	1.26	23	1.0	1.31	27	0.4	0.9	1.24	22	Suitable	
28	Km 30+200	Km 30+500	Light Brown silty clay soil	28	30.35	1.34	12	0.4	1.46	29	0.3	1.51	34	0.1	0.2	1.43	26	Suitable	

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																				
Sr. No	Station (km)		Visual Material Classification	Sample Code	Mid-Section Station (Km)	3-Point CBR (%)														Suitability in-terms of CBR
						10 blows			30 blows			65 blows			Avg. Swell (%)	95% of MDD	CBR at 95% MDD			
	From	To				Density (g/cc)	CBR	Swell (%)	Density (g/cc)	CBR	Swell (%)	Density (g/cc)	CBR	Swell (%)						
29	Km 30+500	Km 30+800	Dark to brown silty clay soil	29	30.65	1.26	14	0.5	1.34	29	0.5	1.40	50	0.4	0.5	1.32	24			Suitable
30	Km 30+800	Km 31+100	Dark to brown silty clay soil	30	30.95	1.05	9	0.4	1.17	11	0.3	1.21	20	0.1	0.3	1.14	10			Suitable
31	Km 31+100	Km 31+400	Yellowish to Brown silty clay soil	31	31.25	1.26	6	0.6	1.37	11	0.4	1.43	16	0.2	0.4	1.35	10			Suitable
32	Km 31+400	Km 31+700	Dark to brown clay soil	32	31.55	1.34	13	0.4	1.43	17	0.2	1.50	23	0.2	0.3	1.42	17			Suitable
33	Km 31+700	Km 32+000	Dark to brown clay soil	33	31.85	1.32	15	0.6	1.42	28	0.5	1.52	38	0.4	0.5	1.42	29			Suitable
34	Km 32+000	Km 32+300	Brown silty clay soil	34	32.15	1.23	10	0.7	1.34	22	0.4	1.41	27	0.2	0.4	1.30	17			Suitable

Table 16: Liquidity and Consistency Indices of soil

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**LL (%)**	**PL (%)**	**PI (%)**	**AASHTO Soil Classification**	**Moisture-Density Relation**		**In-situ NMC (%)**	**Liquidity Index, I_L**	**Consistency Index, I_c**	**Suitability in-terms of I_L**	**Suitability in-terms of I_c**
	From	**To**								**OMC (%)**	**MDD (g/cc)**					
1	Km 22+100	Km 22+400	Reddish clay soil	01	22.25	47	28	19	A-7-6(4)	19	1.80	51.0	1.21	-0.21	Sensitive Soil	Sensitive Soil
2	Km 22+400	Km 22+700	Brown clay soil	02	22.55	57	29	27	A-7-6(5)	20	1.68	38.1	0.33	0.67	Plastic Soil	Plastic Soil
3	Km 22+700	Km 23+000	Brown clay soils	03	22.85	57	40	17	A-7-5(3)	25	1.56	44.2	0.24	0.76	Plastic Soil	Plastic Soil
4	Km 23+000	Km 23+300	Red to brown silty clay soil	04	23.15	46	29	17	A-7-5(1)	28	1.48	49.3	1.19	-0.19	Sensitive Soil	Sensitive Soil
5	Km 23+300	Km 23+600	Light brown silty clay soil	05	23.45	45	30	15	A-2-7(0)	28	1.51	52.2	1.47	-0.47	Sensitive Soil	Sensitive Soil
6	Km 23+600	Km 23+900	Yellowish clay soil	06	23.75	55	38	16	A-7-5(2)	32	1.41	59.4	1.30	-0.30	Sensitive Soil	Sensitive Soil
7	Km 23+900	Km 24+200	Brown silty clay soil with gravel	07	24.05	39	29	9	A-4(3)	25	1.55	43.5	1.53	-0.53	Sensitive Soil	Sensitive Soil
8	Km 24+200	Km 24+500	Brown silty clay soil	08	24.35	42	31	11	A-7-5(3)	25	1.49	51.9	1.86	-0.86	Sensitive Soil	Sensitive Soil

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																	
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**LL (%)**	**PL (%)**	**PI (%)**	**AASHTO Soil Classification**	**Moisture-Density Relation**		**In-situ NMC (%)**	**Liquidity Index, I_L**	**Consistency Index, I_c**	**Suitability in-terms of I_L**	**Suitability in-terms of I_c**
	From	**To**								**OMC (%)**	**MDD (g/cc)**					
9	Km 24+500	Km 24+800	Dark to brown silty clay soil	09	24.65	48	34	14	A-7-5(1)	26	1.47	50.6	1.19	-0.19	Sensitive Soil	Sensitive Soil
10	Km 24+800	Km 25+100	Yellowish silty clay with gravel	10	24.95	58	41	16	A-2-7(0)	32	1.43	65.0	1.46	-0.46	Sensitive Soil	Sensitive Soil
11	Km 25+100	Km 25+400	Brown silty clay soil	11	25.25	57	40	17	A-7-5(2)	30	1.34	50.3	0.63	0.37	Plastic Soil	Plastic Soil
12	Km 25+400	Km 25+700	Brown silty clay soil	12	25.55	58	44	14	A-7-5(1)	25	1.38	48.4	0.34	0.66	Plastic Soil	Plastic Soil
13	Km 25+700	Km 26+000	Red silty clay soil	13	25.85	58	42	16	A-7-5(9)	28	1.51	59.1	1.07	-0.07	Sensitive Soil	Sensitive Soil
14	Km 26+000	Km 26+300	Brown to reddish clay soil	14	26.15	56	36	21	A-7-5(6)	28	1.49	66.7	1.52	-0.52	Sensitive Soil	Sensitive Soil
15	Km 26+300	Km 26+600	Dark silty clay soil	15	26.45	57	40	17	A-7-5(4)	26	1.47	63.4	1.39	-0.39	Sensitive Soil	Sensitive Soil
16	Km 26+600	Km 26+900	Red silty clay soil	16	26.75	59	40	20	A-2-7(1)	27	1.55	44.4	0.24	0.76	Plastic Soil	Plastic Soil

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**LL (%)**	**PL (%)**	**PI (%)**	**AASHTO Soil Classification**	**Moisture-Density Relation**		**In-situ NMC (%)**	**Liquidity Index, I_L**	**Consistency Index, I_c**	**Suitability in-terms of I_L**	**Suitability in-terms of I_c**
	From	**To**								**OMC (%)**	**MDD (g/cc)**					
17	Km 26+900	Km 27+200	Brown silty clay soil	17	27.05	47	33	14	A-2-7(0)	30	1.45	51.8	1.35	-0.35	Sensitive Soil	Sensitive Soil
18	Km 27+200	Km 27+500	Brown clay soil	18	27.35	57	40	17	A-7-5(3)	26	1.47	48.1	0.47	0.53	Plastic Soil	Plastic Soil
19	Km 27+500	Km 27+800	Red silty clay soil	19	27.65	56	46	10	A-2-5(0)	30	1.49	61.0	1.51	-0.51	Sensitive Soil	Sensitive Soil
20	Km 27+800	Km 28+100	Brown silty clay soil	20	27.95	52	35	17	A-2-7(1)	24	1.56	58.6	1.39	-0.39	Sensitive Soil	Sensitive Soil
21	Km 28+100	Km 28+400	Red silty clay soil	21	28.25	51	34	17	A-7-5(1)	24	1.57	58.3	1.41	-0.41	Sensitive Soil	Sensitive Soil
22	Km 28+400	Km 28+700	Red silty clay soil	22	28.55	56	39	17	A-7-5(2)	27	1.54	57.2	1.09	-0.09	Sensitive Soil	Sensitive Soil
23	Km 28+700	Km 29+000	Dark to brown silty clay soil	23	28.85	47	33	14	A-7-5(3)	20	1.66	52.5	1.38	-0.38	Sensitive Soil	Sensitive Soil
24	Km 29+000	Km 29+300	Light Brown silty clay soil	24	29.15	57	40	17	A-2-7(0)	23	1.45	42.8	0.15	0.85	Plastic Soil	Plastic Soil

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																	
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**LL (%)**	**PL (%)**	**PI (%)**	**AASHTO Soil Classification**	**Moisture-Density Relation**		**In-situ NMC (%)**	**Liquidity Index, I_L**	**Consistency Index, I_c**	**Suitability in-terms of I_L**	**Suitability in-terms of I_c**
	From	**To**								**OMC (%)**	**MDD (g/cc)**					
25	Km 29+300	Km 29+600	Brown clay soil	25	29.45	52	35	17	A-7-5(2)	23	1.57	57.8	1.35	-0.35	Sensitive Soil	Sensitive Soil
26	Km 29+600	Km 29+900	Red brown silty clay soil	26	29.75	53	36	17	A-2-7(0)	23	1.51	55.2	1.13	-0.13	Sensitive Soil	Sensitive Soil
27	Km 29+900	Km 30+200	Light grey clay material	27	30.05	57	36	21	A-7-5(2)	30	1.31	59.4	1.11	-0.11	Sensitive Soil	Sensitive Soil
28	Km 30+200	Km 30+500	Light Brown silty clay soil	28	30.35	46	27	19	A-7-6(2)	27	1.51	53.1	1.38	-0.38	Sensitive Soil	Sensitive Soil
29	Km 30+500	Km 30+800	Dark to brown silty clay soil	29	30.65	50	39	11	A-7-5(0)	25	1.39	58.2	1.78	-0.78	Sensitive Soil	Sensitive Soil
30	Km 30+800	Km 31+100	Dark to brown silty clay soil	30	30.95	51	37	14	A-7-5(2)	40	1.20	54.5	1.26	-0.26	Sensitive Soil	Sensitive Soil
31	Km 31+100	Km 31+400	Yellowish to Brown silty clay soil	31	31.25	55	39	16	A-2-7(0)	28	1.42	47.8	0.54	0.46	Plastic Soil	Plastic Soil
32	Km 31+400	Km 31+700	Dark to brown clay soil	32	31.55	54	39	15	A-2-7(1)	24	1.50	49.0	0.70	0.30	Plastic Soil	Plastic Soil

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**LL (%)**	**PL (%)**	**PI (%)**	**AASHTO Soil Classification**	**Moisture-Density Relation**		**In-situ NMC (%)**	**Liquidity Index, I_L**	**Consistency Index, I_c**	**Suitability in-terms of I_L**	**Suitability in-terms of I_c**
	From	**To**								**OMC (%)**	**MDD (g/cc)**					
33	Km 31+700	Km 32+000	Dark to brown clay soil	33	31.85	55	38	17	A-7-5(3)	24	1.50	48.5	0.61	0.39	Plastic Soil	Plastic Soil
34	Km 32+000	Km 32+300	Brown silty clay soil	34	32.15	51	37	14	A-7-5(2)	27	1.37	54.6	1.30	-0.30	Sensitive Soil	Sensitive Soil

Table 17: State and consistency of soil Analysis

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																	
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**Liquidity Index, I_L**	**Consistency Index, I_c**	**Sate of Material**			**Consistency Stage**						**UCS (kPa)**
								Liquid	**Plastic**	**Solid**	**Viscous Liquid**	**Very Soft**	**Soft**	**Medium Stiff**	**Stiff**	**Very Stiff**	
	From	**To**						**(1=Yes, 0=No)**									
1	Km 22+100	Km 22+400	Reddish clay soil	01	22.25	1.21	-0.21	1	0	0	1	0	0	0	0	0	<0
2	Km 22+400	Km 22+700	Brown clay soil	02	22.55	0.33	0.67	0	1	0	0	0	0	1	0	0	51-100
3	Km 22+700	Km 23+000	Brown clay soils	03	22.85	0.24	0.76	0	1	0	0	0	0	0	1	0	101-200
4	Km 23+000	Km 23+300	Red to brown silty clay soil	04	23.15	1.19	-0.19	1	0	0	1	0	0	0	0	0	<0
5	Km 23+300	Km 23+600	Light brown silty clay soil	05	23.45	1.47	-0.47	1	0	0	1	0	0	0	0	0	<0
6	Km 23+600	Km 23+900	Yellowish clay soil	06	23.75	1.30	-0.30	1	0	0	1	0	0	0	0	0	<0
7	Km 23+900	Km 24+200	Brown silty clay soil with gravel	07	24.05	1.53	-0.53	1	0	0	1	0	0	0	0	0	<0

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																	
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**Liquidity Index, I_L**	**Consistency Index, I_c**	**Sate of Material**			**Consistency Stage**						**UCS (kPa)**
								Liquid	**Plastic**	**Solid**	**Viscous Liquid**	**Very Soft**	**Soft**	**Medium Stiff**	**Stiff**	**Very Stiff**	
	From	**To**						**(1=Yes, 0=No)**									
8	Km 24+200	Km 24+500	Brown silty clay soil	08	24.35	1.86	-0.86	1	0	0	1	0	0	0	0	0	<0
9	Km 24+500	Km 24+800	Dark to brown silty clay soil	09	24.65	1.19	-0.19	1	0	0	1	0	0	0	0	0	<0
10	Km 24+800	Km 25+100	Yellowish silty clay with gravel	10	24.95	1.46	-0.46	1	0	0	1	0	0	0	0	0	<0
11	Km 25+100	Km 25+400	Brown silty clay soil	11	25.25	0.63	0.37	0	1	0	0	0	1	0	0	0	26-50
12	Km 25+400	Km 25+700	Brown silty clay soil	12	25.55	0.34	0.66	0	1	0	0	0	0	1	0	0	51-100
13	Km 25+700	Km 26+000	Red silty clay soil	13	25.85	1.07	-0.07	1	0	0	1	0	0	0	0	0	<0
14	Km 26+000	Km 26+300	Brown to reddish clay soil	14	26.15	1.52	-0.52	1	0	0	1	0	0	0	0	0	<0
15	Km 26+300	Km 26+600	Dark silty clay soil	15	26.45	1.39	-0.39	1	0	0	1	0	0	0	0	0	<0

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																		
Sr. No	Station (km)		Visual Material Classification	Sample Code	Mid-Section Station (Km)	Liquidity Index, I_L	Consistency Index, I_c	Sate of Material			Consistency Stage							UCS (kPa)
								Liquid	Plastic	Solid	Viscous Liquid	Very Soft	Soft	Medium Stiff	Stiff	Very Stiff	
	From	To						(1=Yes, 0=No)										
16	Km 26+600	Km 26+900	Red silty clay soil	16	26.75	0.24	0.76	0	1	0	0	0	0	0	1	0	101-200
17	Km 26+900	Km 27+200	Brown silty clay soil	17	27.05	1.35	-0.35	1	0	0	1	0	0	0	0	0	<0
18	Km 27+200	Km 27+500	Brown clay soil	18	27.35	0.47	0.53	0	1	0	0	0	0	1	0	0	51-100
19	Km 27+500	Km 27+800	Red silty clay soil	19	27.65	1.51	-0.51	1	0	0	1	0	0	0	0	0	<0
20	Km 27+800	Km 28+100	Brown silty clay soil	20	27.95	1.39	-0.39	1	0	0	1	0	0	0	0	0	<0
21	Km 28+100	Km 28+400	Red silty clay soil	21	28.25	1.41	-0.41	1	0	0	1	0	0	0	0	0	<0
22	Km 28+400	Km 28+700	Red silty clay soil	22	28.55	1.09	-0.09	1	0	0	1	0	0	0	0	0	<0
23	Km 28+700	Km 29+000	Dark to brown silty clay soil	23	28.85	1.38	-0.38	1	0	0	1	0	0	0	0	0	<0

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																		
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**Liquidity Index, I_L**	**Consistency Index, I_c**	**Sate of Material**			**Consistency Stage**							**UCS (kPa)**
	From	**To**						**Liquid**	**Plastic**	**Solid**	**Viscous Liquid**	**Very Soft**	**Soft**	**Medium Stiff**	**Stiff**	**Very Stiff**		
								(1=Yes, 0=No)										
24	Km 29+000	Km 29+300	Light Brown silty clay soil	24	29.15	0.15	0.85	0	1	0	0	0	0	0	1	0	101-200
25	Km 29+300	Km 29+600	Brown clay soil	25	29.45	1.35	-0.35	1	0	0	1	0	0	0	0	0	<0
26	Km 29+600	Km 29+900	Red brown silty clay soil	26	29.75	1.13	-0.13	1	0	0	1	0	0	0	0	0	<0
27	Km 29+900	Km 30+200	Light grey clay material	27	30.05	1.11	-0.11	1	0	0	1	0	0	0	0	0	<0
28	Km 30+200	Km 30+500	Light Brown silty clay soil	28	30.35	1.38	-0.38	1	0	0	1	0	0	0	0	0	<0
29	Km 30+500	Km 30+800	Dark to brown silty clay soil	29	30.65	1.78	-0.78	1	0	0	1	0	0	0	0	0	<0
30	Km 30+800	Km 31+100	Dark to brown silty clay soil	30	30.95	1.26	-0.26	1	0	0	1	0	0	0	0	0	<0

SUMMARY OF LABORATORY TEST RESULTS OF ROADWAY EXCAVATED MATERIAL																	
Sr. No	**Station (km)**		**Visual Material Classification**	**Sample Code**	**Mid-Section Station (Km)**	**Liquidity Index, I_L**	**Consistency Index, I_c**	**Sate of Material**			**Consistency Stage**						**UCS (kPa)**
								Liquid	**Plastic**	**Solid**	**Viscous Liquid**	**Very Soft**	**Soft**	**Medium Stiff**	**Stiff**	**Very Stiff**	
	From	**To**						**(1=Yes, 0=No)**									
31	Km 31+100	Km 31+400	Yellowish to Brown silty clay soil	31	31.25	0.54	0.46	0	1	0	0	0	1	0	0	0	26-50
32	Km 31+400	Km 31+700	Dark to brown clay soil	32	31.55	0.70	0.30	0	1	0	0	0	1	0	0	0	26-50
33	Km 31+700	Km 32+000	Dark to brown clay soil	33	31.85	0.61	0.39	0	1	0	0	0	1	0	0	0	26-50
34	Km 32+000	Km 32+300	Brown silty clay soil	34	32.15	1.30	-0.30	1	0	0	1	0	0	0	0	0	<0

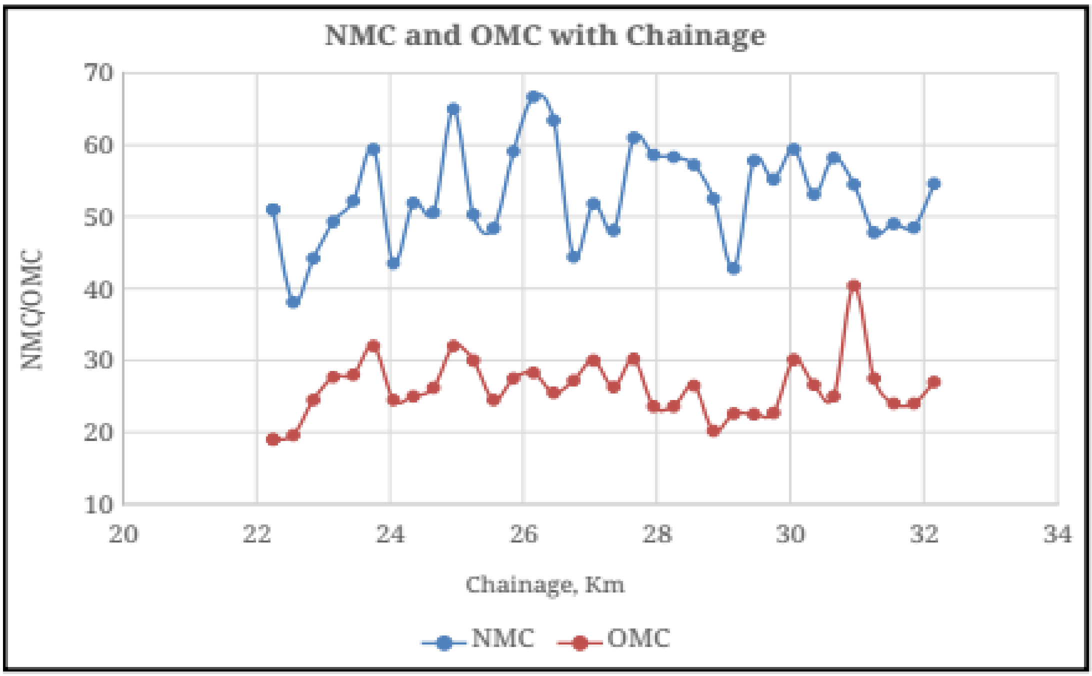

Figure 13: Relation between NMC vs OMC with Chainage

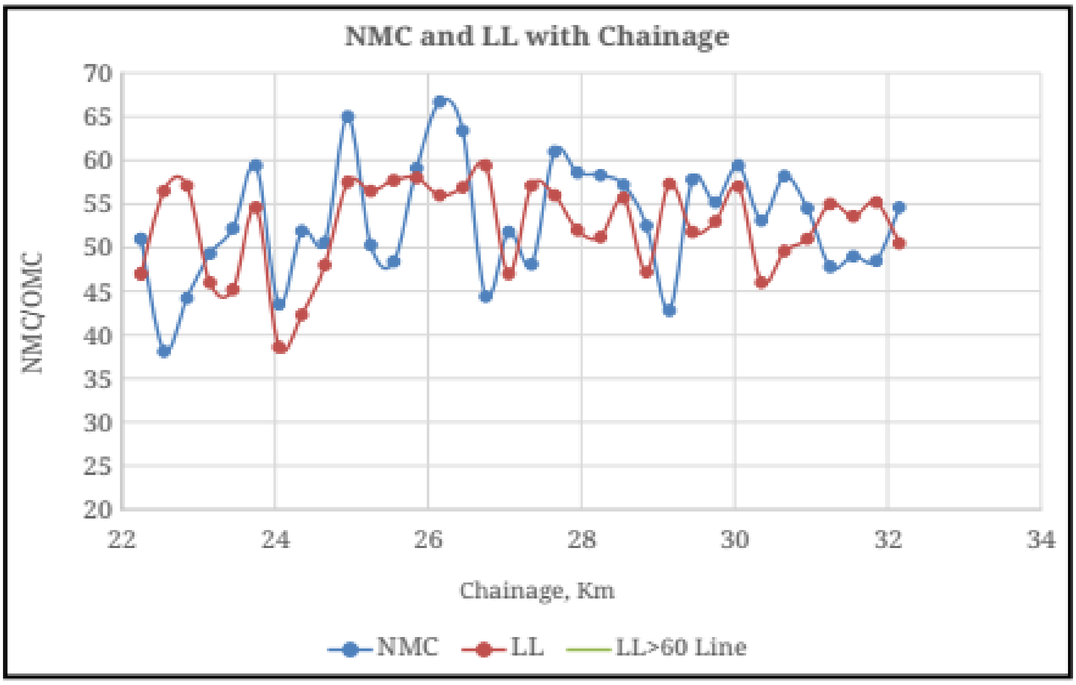

Figure 14: Relation between NMC vs LL with Chainage

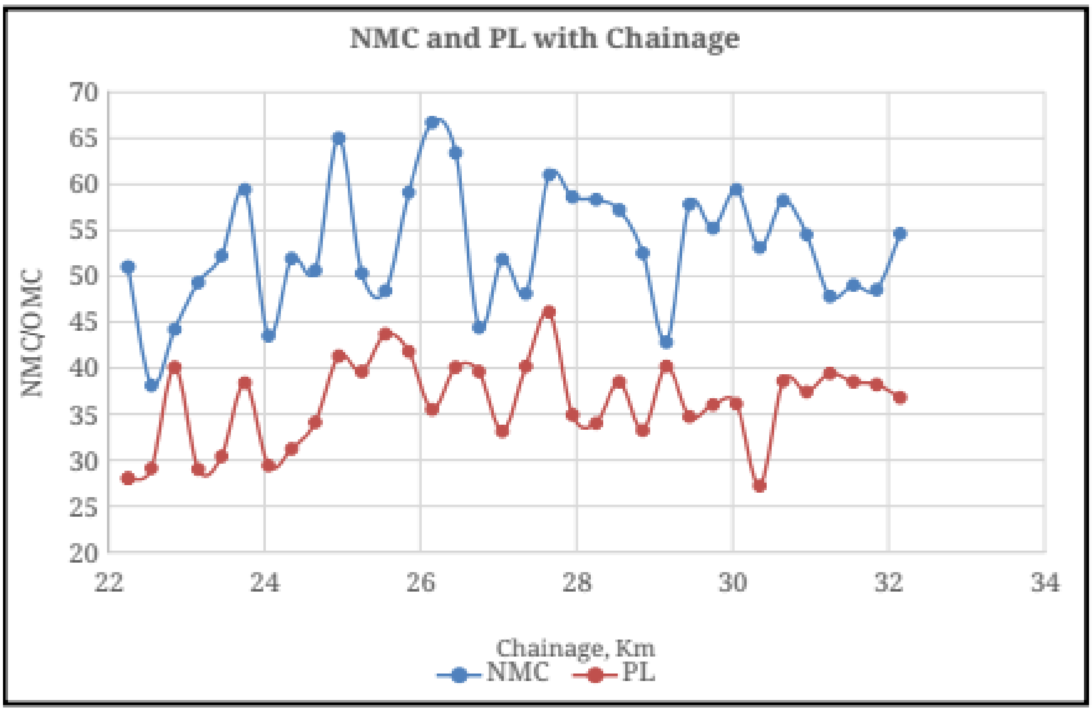

Figure 15: Relation between NMC vs PL with Chainage

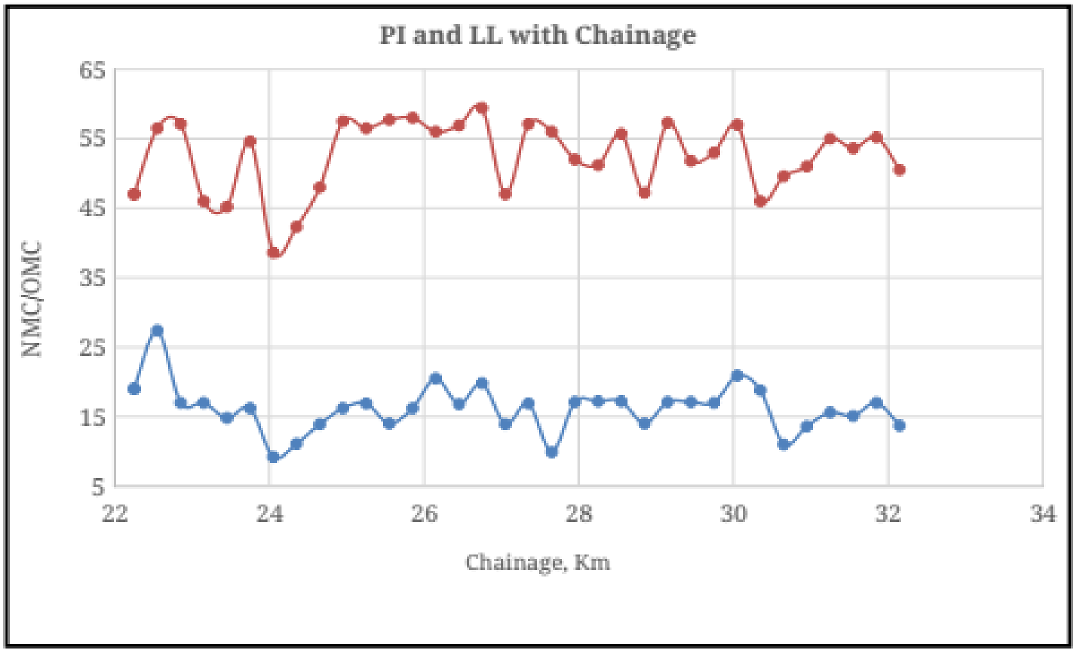

Figure 16: Relation between LL vs PI with Chainage

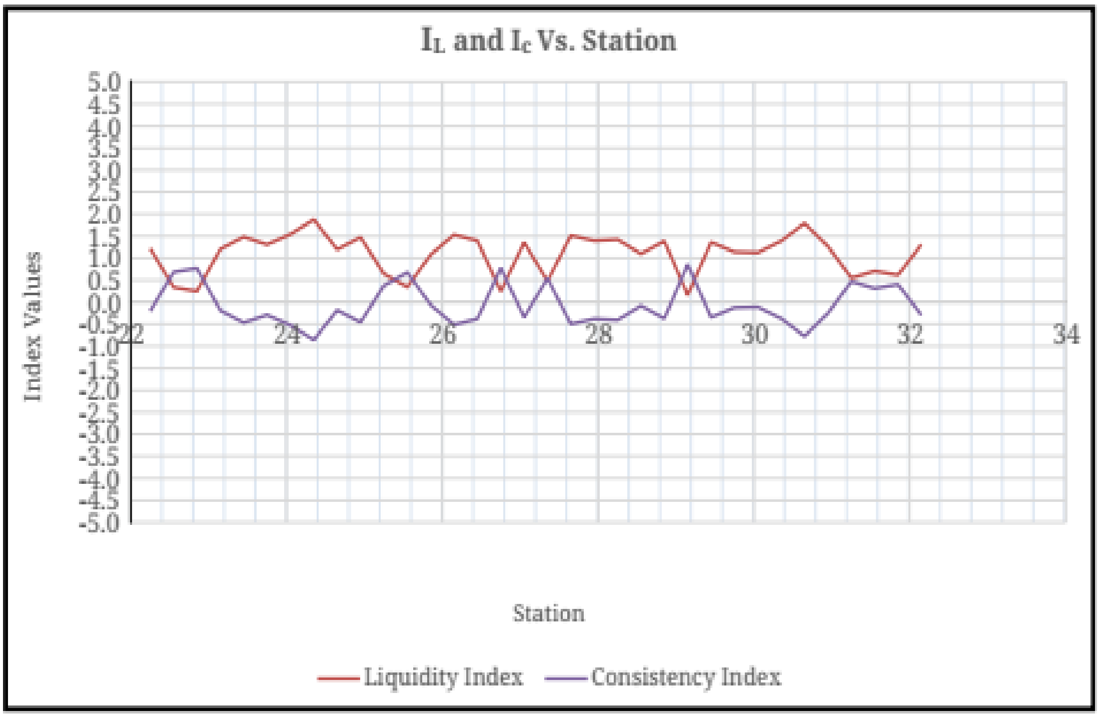

Figure 17: Variations of I_c and I_L with Chainage

4.3 Results Discussion

The soil samples collected for the project were classified as dark brown to red clay and light yellowish to yellowish silty clay with gravel soils. Based on the project specification, all 34 samples were found suitable for embankment fill in terms of CBR, LL, and PI values. However, when analyzed based on the author's specifications from different literatures, 24 out of the 34 samples were deemed unsuitable due to NMC being greater than LL, and 24 samples were identified as sensitive soils based on the criteria of Ic < 0 or $I_L > 1$.

Further testing revealed that 10 samples were classified as plastic soils, while none were identified as over consolidated may be directly used as embankment fill material. The majority of the samples (24 out of 34) were found to be sensitive soils at a liquid state consistency stage, indicating low strength and stability. Only 4 samples were classified as having a soft consistency stage, while 3 samples each were found to be at medium stiff and stiff consistency stages. No samples were identified as very stiff or hard to very hard. In terms of unconfined compressive strength (UCS), the majority of the samples (24 out of 34) had a UCS of less than 0 kPa, with only a few samples falling within the range of 26 to 200 kPa.

These results indicate that while the soil samples meet the project's specification for embankment fill material in terms of CBR, LL, and PI values, they exhibit significant workability and plasticity issues, as well as low consistency and strength. This suggests potential challenges in achieving stability in embankment construction using these soil samples. Therefore, further assessment and mitigation measures should be considered to address these issues before proceeding with construction.

Based on the data provided in the project specification and the author's specification from different literatures, the soil samples can be characterized as follows:

Soil Classification:

The majority of the soil samples are classified as dark brown to red clay and light yellowish to yellowish silty clay with gravel soils. This indicates that the soil is predominantly composed of fine-grained materials, such as clay and silt, with some gravel content.

Workability/Plasticity:

According to the project specification, the soil samples have a Plasticity Index (PI) greater than 30 and a Liquid Limit (LL) greater than 60, which suggests that the soil has a high plasticity and is considered unsuitable for use as embankment fill. However, the remaining soil samples, which account for 100% of the total, can be used as fill material for embankment construction.

The implications of having high plasticity and viscous liquid consistency in the soil samples are as follows:

i) Workability and Compaction: Soils with high plasticity (PI > 30) and Liquid Limit (LL > 60) are generally difficult to work with and compact effectively. They have a narrow range of moisture content where they can be compacted. The viscous liquid consistency indicates that the soil is in a highly fluid state, making it challenging to achieve the desired compaction and density during the embankment construction process.

ii) Stability and Settlement: Highly plastic and viscous liquid soils are prone to significant volume changes and settlement under loading. They can experience large deformations and are susceptible to slope instability, especially in embankment construction. The lack of shear strength and cohesion in these types of soils can lead to issues with the overall stability and long-term performance of the embankment.

iii) Drainage and Erosion: Viscous liquid soils have poor drainage characteristics, which can lead to the accumulation of water within the embankment structure. This can increase the risk of erosion, surface runoff, and potential failure of the embankment.

iv) Construction Challenges: Handling and placing highly plastic and viscous liquid soils during construction can be extremely challenging. They require specialized techniques, equipment, and strict moisture control to ensure proper compaction and performance. The construction process may be more time-consuming and require additional resources to address the challenges posed by these soil characteristics.

Consistency:

The test results based on the literature review show that 71% of the soil samples are at the viscous liquid consistency stage, indicating a very soft and highly sensitive soil. The remaining 29% of the samples are at the plastic state. None of the samples are in the solid state.

Strength and Stability:

The Unconfined Compressive Strength (UCS) of the soil samples varies significantly. Approximately 71% of the samples have a UCS less than 0 kPa, indicating a very weak and unstable soil. Another 12% of the samples have a UCS ranging from 26 to 50 kPa, which is considered soft. The remaining 17% of the samples have a UCS ranging from 51 to 200 kPa, which can be classified as medium stiff to stiff.

In the context of the provided data, the high plasticity and viscous liquid consistency of the majority of the soil samples (71%) indicate that they are not suitable for use as embankment fill material without significant additional treatment or modification. Measures such as soil stabilization, replacement with more suitable materials, or the use of specialized construction techniques may be necessary to mitigate the risks associated with these soil characteristics.

Overall, the majority of the soil samples are classified as high plasticity, viscous liquid, and very weak, which suggests that they are not suitable for use as embankment fill.

However, a significant portion of the samples (100% based on the project specification) can be used as fill material, as they have the required characteristics in terms of CBR value, Liquid Limit, and Plasticity Index.

In general, when soil's NMC is equal to its PL then the value of the Liquidity Index is Zero. On the other hand, if soil's NMC increases to its LL, Liquidity Index value increases from Zero to One. Further increase in the NMC will lead to Liquidity Index above 1 which is indicative of Liquefaction or Quick Potential. It can also be noticed that when a soil is in Plastic State then its Liquidity Index varies from 0 to 1. At water content lower than Plastic Limit, soil is relatively harder and brittle in nature. Here Liquidity Index of the soil will be negative. In general, with the increase in water content liquidity Index increases and firmness of a soil decreases. When a soil's NMC is at the PL Consistency Index value is 1 so Liquidity Index value is zero and at LL Consistency Index value is zero so Liquidity Index value is 1.

In what State will soils be at their Optimum Moisture Contents (OMC)? This has been performed by replacing the NMC by the OMC. Their Consistency Index is always above 1 indicating that soils compacted at their OMC are either in Solid or Semi Solid States. To princely predict in which State, Solid or Semi Solid their Shrinkage Limit (moisture content below which they are in Solid State) is needed. Shrinkage Limit test, however, is not present in the Laboratory Test Results. Shrinkage Limit test is difficult to carry out by taking selected representative samples to undertake volumetric shrinkage tests as the test results vary according to the test method and initial moisture content. This is more pronounced when the soils are remolded or disturbed to undertake the test.

CHAPTER FIVE: CONCLUSION AND RECOMMENDATIONS

5.1 CONCLUSION

The roadway excavated materials when are suitable they shall be used in fill sections and areas of subgrade improvement. The remaining balance of the roadway excavation that are suitable and those that don't satisfy NMC specification requirements (i.e. NMC>LL) shall be taken cut to spoil.

Nevertheless, most of the roadway excavations satisfying the project specification requirements are characterized by very high in-situ moisture requiring drying for their further use for embankment fill and replacement of unsuitable subgrade soils. This is what has been specified in the Works Contract under Clause 2408-d. It is witnessed during the progress of the roadway excavations that in-situ moisture of the excavated materials are much above their optimum moisture content (OMC) in many instances. This has been observed to impose difficulties during construction activity and the contractor has repeatedly complained that the progress of the project would be significantly slowed if the excavated materials are kept to dry until their moisture are reduced to that of the OMC.

Due to prevalent wet weather condition it requires considerable time to dry the excavated material to make it suitable both for embankment fill and as subgrade for direct pavement support. And yet the project corridor is one of the areas with high amount of annual precipitation or rainfall in excess of 1500mm distributed over a longer period of the year. The length of season suitable for construction is not more that 6-months at large often spanning from October to March. Not only this, the mean annual temperature is well under 20°C. So far, the roadway excavated materials shall be spoiled and embankments are constructed from suitable, borrow material sources.

This analysis of study is therefore prepared to offer a recommendation for the consideration of the Employer regarding roadway excavations that are suitable in terms of specification requirements but unpractical for their use through drying in account of the prevailing weather condition and workability of the materials themselves owing to their high in-situ moisture content.

Accordingly, an attempt was made to set some limiting NMC above which they become difficult to handle and drying may not be practical as far as the weather condition and reasonable construction progress are concerned. Furthermore, analysis of the moisture contents of those samples recorded as suitable but replaced due to very high in-situ moisture contents fail to yield some consistent relation with the OMC.

Finally, the concept of Liquidity and Consistency Indices that are related with the firmness and softness of soils has been introduced. This has been discovered in depth in future studies as well. An interesting relationship has been observed between the NMC, LL, PL and SL with workability of soils (softness and firmness). Liquidity and Consistency Indices are related to each other and their sum is always Unity. Their connections to the state of soils, liquid, plastic, semi-solid and solid states is also well illustrated. Remarkably, soils at their OMC are predominantly in a Solid State. On the contrary, most of the soils from roadway excavations are in liquid or plastic state. Therefore, these soils need to be brought out of their liquid or plastic state through drying and then passed their Semi-Solid State to reach to their OMC at which the MDD and Hence the Maximum bearing Strength would be mobilized through compaction in their Solid State.

5.2 RECOMMENDATION

To demonstrate these soils of excavated roadway sections satisfying specification requirement only in terms LL have been computed for three cases of Consistency Indices as discussed in section 2. As the Consistency Index is reduced below 1 more and more sections become suitable in terms of in-situ moisture content. However, this is by allowing higher and higher NMC becoming more unstable and demanding more and more time for drying. A Consistency Index of Zero corresponds to Liquidity Index of One above which soils start to behave like liquid. Between Consistency Index of 0 and 1 soils stay in their Plastic States. In account of this, soils with Consistency Index of 1 and above are recommended as suitable in terms of In-situ/NMC and those below 1 are proposed to be unsuitable. Accordingly, the following formula is finally recommended to determine suitability of those suitable soils satisfying all other specification requirements.

$I_c = \frac{LL - NMC}{PI} < 1$, Unsuitable---------------------------------- Equation 5.1

$I_c = \frac{LL - NMC}{PI} \geq 1$, Suitable-- Equation 5.2

The suitability Test shall be carried out on all roadway excavated samples satisfying the specification requirements set in the Works Contract.

To easy the Screening Process the following steps may be followed,

i) Carry out LL test,
ii) If the criteria for LL ≤ 60% is satisfied then,
iii) Carry out 3-point CBR and Swell tests,
iv) If the materials satisfy the minimum CBR at 95% compaction and possess Swell not more than 2% then compute,
v) Consistency Index to determine suitability in terms of NMC,
vi) If it passes the Suitability test in terms of Consistency Index then the material/soil can be considered the OMC and used for embankment fill, replacement and at subgrade level.

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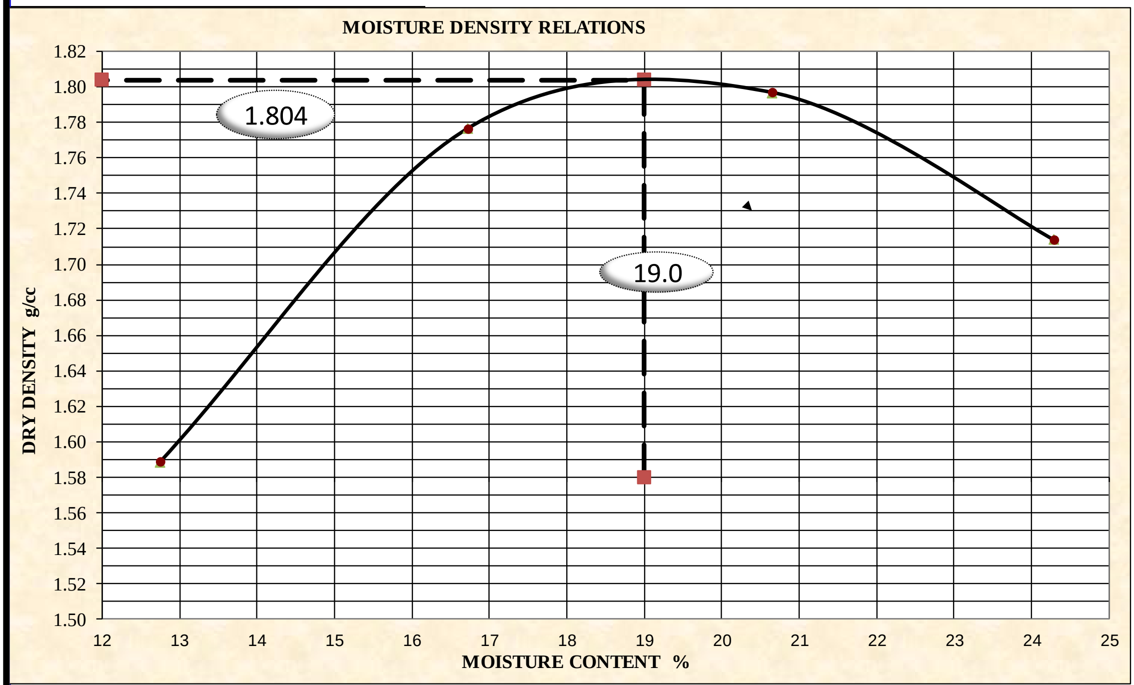
APPENDICES

Appendix – A: Test Log Pits

MOISTURE DENSITY RELATIONSHIP OF SOIL (TEST METHOD: AASHTO T-180 METHOD D)								
Representing Section	**km 22+100-22+300**		**Visual Descrip.**		**Reddish clay soils**			
Sampling Station			**Sampling Date**					
Material Type			**Testing Date**					
Depth (M)			**Lab #**					
	TRIAL NUMBER		**1**	**2**	**3**	**4**	**5**	
DENSITY	Water %		400	600	800	1000		
	WEIGHT OF SOIL + MOLD (g)	W_1	8,800	9,400	9,600	9,520		
	WEIGHT OF MOLD (g)	W_2	4995	4995	4995	4995		
	VOLUME OF MOLD (Cm^3)	V	2124	2124	2124	2124		
	WEIGHT OF WET SOIL (g)	$W_3 = W_1 - W_2$	3,805	4,405	4,605	4,525		
	WET DENSITY OF SOIL (g/Cm^3)	$W_d = W_3/V$	1.79	2.07	2.17	2.13		
MOISTURE	CONTAINER NUMBER		70	36	D	7		NMC
	WET SOIL + CONTAINER (g)	a	361.6	393.8	364.9	373.0		389.3
	DRY SOIL + CONTAINER (g)	b	325.6	341.9	307.4	308.5		377.0
	WEIGHT OF CONTAINER (g)	c	43.2	31.60	29.0	43.0		39.0
	WEIGHT OF WATER (g)	e = a-b	36.0	51.9	57.5	64.5		12.3
	WEIGHT OF DRY SOIL (g)	d =b-c	282.4	310.3	278.4	265.5		338.0
	MOISTURE CONTENT (%)	**m= (e/d)*100**	12.75	16.7	20.7	24.3		3.6
DRY DENSITY OF SOIL	**(g/Cm3)**	**$D_d = W_d/(100+m)*100$**	1.59	1.78	1.80	1.71		

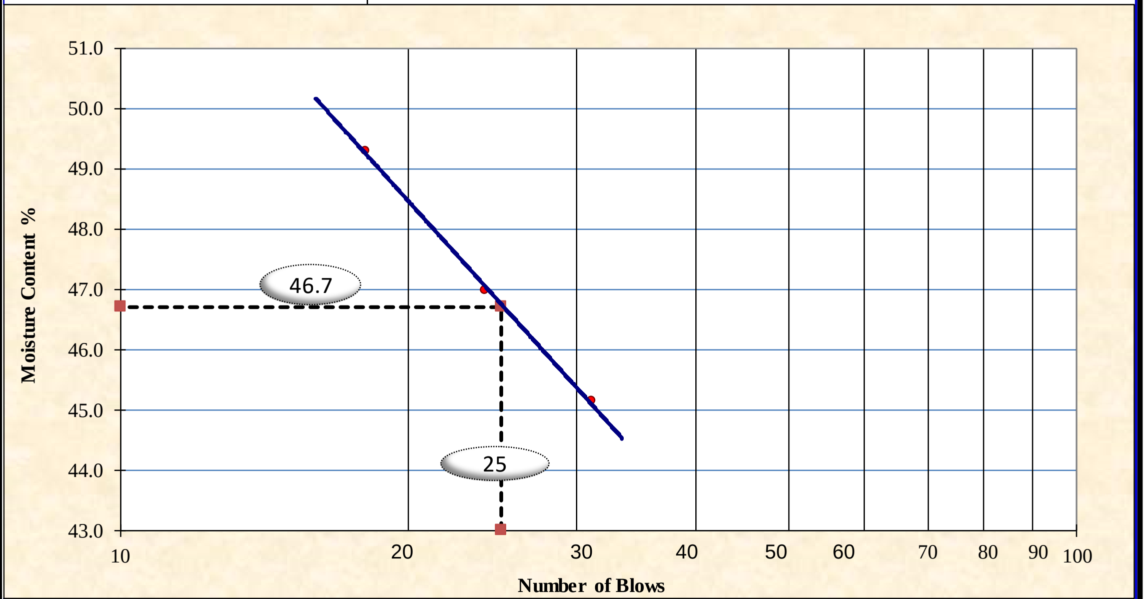


MDD :	**1.804** g/cm^3
OMC :	**19.00 %**

REMARK:

SOIL CONSISTENCY TEST RESULT (TEST METHOD : AASHTO T-89, & T-90)

Representing Section	km 22+100-22+300		**Visual Description**		**Reddish clay soils**	
Sampling Station			**Sampling Date**			
Material Type			**Testing Date**			
Depth			**Lab.#**			
	Liquid Limit				**Plastic Limit**	
No. of Blows	31	24	18			
Container Number	G	6	D		B1	3
Wt. of Container + Wet Soil (g) = (W_1)	38.635	38.695	38.172		28.42	28.374
Wt. of Container + Dry Soil (g) = (W_2)	34.03	33.832	33.311		27.428	27.380
Wt. of Container (g) = (W_3)	23.82	23.486	23.45		23.834	23.816
Weight of Moisture (g) = (W_1 - W_2) = A	4.61	4.863	4.861		0.99	0.99
Weight of Dry Soil (g) = (W_2 - W_3) = B	10.21	10.35	9.86		3.59	3.56
Moisture Content (%) = (A / B)x 100	45.16	47.00	49.3		27.60	27.89
				AV. Plas. Lim.	27.7	



LIQUIDLIMIT	LL	46.7		**AASHTO Classification:**		A-7-6(4)
PLASTIC LIMIT	PL	27.7		Wt of Before Washing (gm)		734.5
PLASTICITY INDEX	(LL-PL)=PI	19.0	Sieve sizes(mm	Wt Retained(gm)	% Retained(gm)	Cum.% Pass
Remark			4.75	0	0.00	100.0
			2	44.6	6.07	93.93
			0.425	121.9	16.60	77.33
			0.075	270.0	36.76	40.57
			pan			

CALIFORNIA BEARING RATIO WORK SHEET (TEST METHOD AASHTO T-193)

Representing Section	**km 22+100-22+300**	**Visual Descrip.**	**Reddish clay soils**
Sampling Station		**Sampling Date**	
Material Type		**Soaking Date**	
Lab #		**Testing Date**	

D E N S I T Y D E T E R M I N A T I O N

SOAKING CONDITION			**10 Blows**		**30 Blows**		**65 Blows**	
			BEFORE	**AFTER**	**BEFORE**	**AFTER**	**BEFORE**	**AFTER**
MOLD NUMBER			**D6**		**D9**		**D5**	
WEIGHT OF SOIL + MOLD	(g)	**W_1**	11090	11433	11600	11768	11790	11899
WEIGHT OF MOLD	(g)	**W_2**	6340	6340	6320	6320	6180	6180
VOLUME OF MOLD	(Cm^3)	**V**	2580	2580	2576	2576	2574	2574
WEIGHT OF WET SOIL	(g)	**$W_3 = W_1 - W_2$**	4750	5093	5280	5448	5610	5719
WET DENSITY OF SOIL	(g/cm3)	**$W_d = (W_3/V)$**	1.84	1.97	2.05	2.11	2.18	2.22
DRY DENSITY OF SOIL	(g/cm3)	**$D_d = W_d/(100+m)*100$**	1.53	1.52	1.72	1.65	1.831	1.75

M O I S T U R E D E T E R M I N A T I O N

SOAKING CONDITION		**10 Blows**			**30 Blows**			**65 Blows**		
		BEFORE	**AFTER TOP 1 in.**		**BEFORE**	**AFTER TOP 1 in.**		**BEFORE**	**AFTER TOP 1 in.**	
CONTAINER NUMBER		9	6A		39	21B		11	I	
WET SOIL + CONTAINER	a	348.10	330.00		362.20	310.00		389.20	290.00	
DRY SOIL + CONTAINER	b	295.00	262.20		308.50	252.60		334.30	235.00	
WEIGHT OF CONTAINER	c	32.40	34.60		29.00	46.60		45.70	29.20	
WEIGHT OF WATER	d = a - b	53.1	67.8		53.7	57.4		54.90	55.0	
WEIGHT OF DRY SOIL	e = b - c	262.6	227.60		279.5	206		288.60	205.8	
MOISTURE CONTENT	m =(d/e)*100	20.22	29.79		19.21	27.86		19.02	26.72	
AVG. MOIST. CONTENT (%)										

P E N E T R A T I O N T E S T D A T A

PENETRATION	**10 Blows**			**30 Blows**			**65 Blows**		
(mm)	DIAL RDG	COR. LOAD (kn)	**CBR %**	DIAL RDG	COR. LOAD (kn))	**CBR %**	DIAL RDG	COR LOAD (kn))	**CBR %**
0	0	0.00		0	0		0	0.00	
0.64	7.0	0.15		28.0	0.60		37.0	0.79	
1.27	9.0	0.19		43.0	0.92		58.0	1.23	
1.96	12.0	0.26		54.0	1.15		73.0	1.55	
2.54	14.0	0.30	**2.23**	60.0	1.28	**9.56**	84.0	1.79	**13.39**
3.18	16.0	0.34		67.0	1.43		95.0	2.02	
3.81	17.0	0.36		72.0	1.53		103.0	2.19	
4.45	19.0	0.40		77.0	1.64		113.0	2.40	
5.08	20.0	0.43	**2.14**	83.0	1.77	**8.86**	121.0	2.57	**12.92**
7.62	22.0	0.47		90.0	1.92		130.0	2.77	
10.16									
12.7									

SWELL			
No. OF BLOWS	**10**	**30**	**65**
RDG (BEFORE SOAKING)	1.85	1.15	0.81
RDG (AFTER) SOAKING)	2.94	2.09	1.57
PERCENT SWELL	0.94	0.81	0.65
AVERAGE PERCENT SWELL	**0.80**		

Ring Factor (N/div)= **21.28**

`

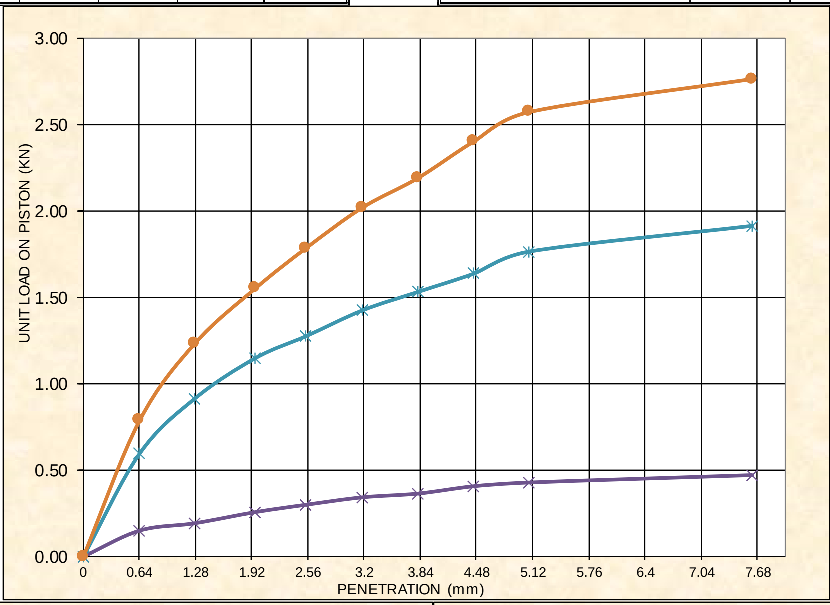
MDD (gm/cc)	1.804
95 % of MDD	1.714
C.B.R.at 95%of MDD	**9.20**

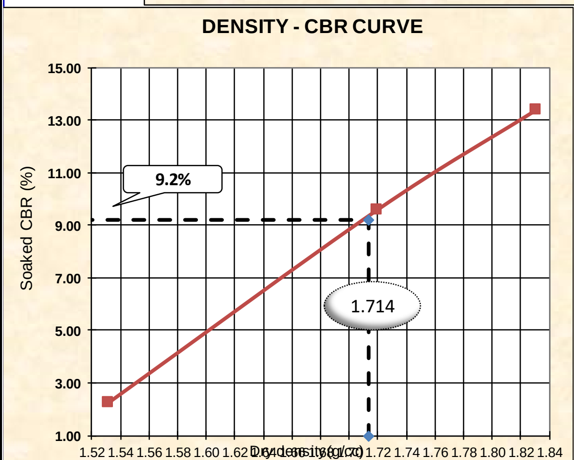
REMARK:

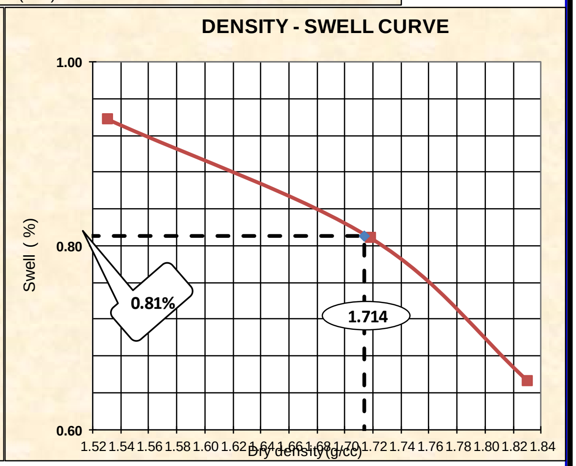
CALIFORNIA BEARING RATIO TEST (TEST METHOD AASHTO T-193)			
Representing Section	**km 22+100-22+300**	**Visual Descrip.**	**Reddish clay soils**
Sampling Station		**Sampling Date**	
Material Type		**Soaking Date**	
Lab #		**Testing Date**	

BLOWS	**LOAD (KN)**		**CBR(%)**		**SWELL**
	2.54mm	5.08mm	2.54mm	5.08mm	**%**
10	0.30	0.43	2.2	2.1	0.94
30	1.28	1.77	9.6	8.9	0.81
65	1.79	2.57	13.4	12.9	0.65

DRY DENSITY Vs SOCKED C.B.R.			
No # OF BLOWS	**10**	**30**	**65**
Swell	0.94	0.81	0.65
DRY DENSITY	1.531	1.719	1.831
SOCKED C.B.R.	2.2	9.6	13.4







MODIFIED PROCTOR : T 180	
MDD (g/cc) :	1.804
OMC (%) :	19.00
95% of MDD(g/cc)	1.714
CBR at 95% of MDD	9.2
Swell at 95% of MDD	0.8

REMARKS:

<table>
<tr><td colspan="5">Determination of Natural Moisture Content of SOIL (TEST METHOD: AASHTO T-265)</td></tr>
<tr><td colspan="2">Representing Section</td><td>km 22+100-22+300</td><td colspan="2">Visual Descrip.</td><td>Reddish clay soils</td></tr>
<tr><td colspan="2">Sampling Station</td><td></td><td colspan="2">Sampling Date</td><td></td></tr>
<tr><td colspan="2">Material Type</td><td></td><td colspan="2">Testing Date</td><td></td></tr>
<tr><td colspan="2">Depth (M)</td><td></td><td>Lab #</td><td></td><td></td></tr>
<tr><td rowspan="8">NATURAL MOISTURE CONTENT OF SOILS BEFORE SUN LIGHT DRY</td><td colspan="2">CONTAINER NUMBER</td><td>12</td><td>5</td><td rowspan="8"></td></tr>
<tr><td colspan="2">WET SOIL + CONTAINER (g) a</td><td>339.4</td><td>388.4</td></tr>
<tr><td colspan="2">DRY SOIL + CONTAINER (g) b</td><td>235.4</td><td>266.2</td></tr>
<tr><td colspan="2">WEIGHT OF CONTAINER (g) c</td><td>36</td><td>28.60</td></tr>
<tr><td colspan="2">WEIGHT OF WATER (g) e = a-b</td><td>104.0</td><td>122.2</td></tr>
<tr><td colspan="2">WEIGHT OF DRY SOIL (g) d =b-c</td><td>199.4</td><td>237.6</td></tr>
<tr><td colspan="2">MOISTURE CONTENT (%) m= (e/d)*100</td><td>52.2</td><td>51.4</td></tr>
<tr><td colspan="2">Average (%)</td><td colspan="2">51.8</td></tr>
</table>

Appendix – B: Sampling and Testing Activities


CAT


UTEST


TOYOTA


UTEST