ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

GEOTECHNICAL INVESTIGATION OF SOME FAILED SECTIONS ALONG OMO RIVER-TERCHA ROAD PROJECT

A Thesis in Geotechnical Engineering

By

Biniam Yihun

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Dr.-Ing. Tensay Gebremedhin

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

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By: Biniam Yihun Anteneh

The signatories have examined the thesis entitled **'Geotechnical Investigation of some Failed Sections along Omo River-Tercha Road Project'** presented by Biniam Yihun Anteneh, a candidate for the degree of Master of Science and hereby certify that it is worthy of acceptance.

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UNDERTAKING

I attest that research work titled "Geotechnical Investigation of some Failed Sections along Omo River-Tercha Road Project" is my work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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Biniam Yihun Anteneh

ABSTRACT

This thesis investigates the geotechnical failures within the Omo River-Tercha Road Project, specifically focused on unanticipated premature failures emerged shortly after opening the road for traffic. The study initiated by examining relevant project documents and conducting comprehensive on-site surveys to identify and record signs of geotechnical failures along the project road. Among these, five major geotechnical failures were identified as significant for further detailed investigation: saturation of recently placed pavement materials and surface pumping, erosion on the surface of roadway cut slopes, depression and several cracks in newly placed pavement, failure of roadway cut slopes and failure of roadway side fill slopes.

After conducting extensive assessments of each failure, which involved a thorough analysis of available project design and construction data, a detailed on-site investigation to detect signs of failure, soil sampling for laboratory testing to examine soil properties and modeling of road cross-sections at locations of slope failure to identify slip surfaces and calculate factors of safety. As a result, the study revealed significant factors contributing to these premature failures, including inadequate subsurface drainage, removal of vegetation covers, problematic subgrade soil, steep geometry of cut and fill slopes, challenges associated with Design-Build contracts and limited engagement of geotechnical engineers.

In response to these findings, the thesis has proposed remedial measures to this newly completed road project. These measures include the establishment of longitudinal interceptor drains for subsurface drainage, re-vegetation with the use of soil erosion blankets, installation of vertical moisture barriers to encapsulate problematic soil, flattening of slopes and integration of geo-grids into the construction of fill slopes within confined spaces. Prompt action is recommended to implement these measures, preventing further damage, minimizing maintenance costs and ensuring infrastructure longevity. The findings provide valuable insights for future road projects, emphasizing proactive geotechnical considerations in design, construction and maintenance.

Key words: Premature, Drainage, Erosion, Subgrade, Slope, Modeling, Geo-grids

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ABREVATIONS

AASHTO – American Association of State Highway and Transportation Officials

ASTM – American Society for Testing and Materials

CBR – California Bearing Ratio

DCP – Dynamic Cone Penetrometer

EGS – Ethiopian Geological Survey

ERA – Ethiopian Road Authority

FS – Factor of Safety

FWD – Falling Weight Deflector Meter

IS – Indian Standard

LEM - Limit Equilibrium Method

LHS – Left Hand Side

MDD – Maximum Dry Density

OMC – Optimum Moisture Content

PGA – Peak Ground Acceleration

RHS – Right Hand Side

USCS – Unified Soil Classification System

CHAPTER 1 INTRODUCTION

Road infrastructure plays a crucial role in promoting economic development, connecting communities, and facilitating efficient transportation systems. However, geotechnical challenges can pose significant obstacles to the successful construction and long-term functionality of roads. In particular, the recently completed Omo River-Tercha road project has encountered geotechnical issues that demand thorough evaluation and effective solutions.

Due to the predominantly mountainous and escarpment terrains along the alignment of this road project, characterized by transversal slopes which are ranging from 25% to above 50%, the construction process has been confronted with substantial challenges. These challenges arise from extensive slope excavation and the presence of intricate geological conditions, as outlined in the (ERA, Geometric Design Manual, 2013) manual. Consequently, the smooth progress and safety of the construction schedule are frequently compromised.

The primary objective of this thesis project is to comprehensively assess and address the geotechnical challenges that have arisen in the Omo River - Tercha road project. By thoroughly observing and recognizing signs of geotechnical failures that have occurred along the road, a detailed investigation will be conducted to determine the probable causes behind these failures. Subsequently, an assessment will be made of the impact these geotechnical challenges have had on the road's overall performance.

To ensure the preservation and sustainability of the road, the thesis project will propose appropriate diagnostic measures that effectively address the identified geotechnical challenges. These recommendations will be based on a thorough analysis of the observed failures, aiming to enhance the road's stability and performance in the face of geotechnical issues.

The significance of this thesis project lies in its potential to provide valuable insights and practical solutions for the Omo River - Tercha road project. By identifying, analyzing and proposing effective diagnostic measures for geotechnical challenges, this research aims to contribute to the improvement of road infrastructure.

Overall, the thesis project endeavors to bridge the gap between theoretical knowledge and practical implementation by evaluating and addressing the geotechnical challenges encountered in the Omo River - Tercha road project. Through a comprehensive analysis and recommended solutions, this research aims to enhance the road's performance, durability, and safety, ultimately benefiting the communities and economic activities that rely on this crucial transportation link.

1.1 Background of the Thesis

The background of the thesis project is shaped by the observed geotechnical problems that have emerged soon after the construction of the Omo River - Tercha road project. This road project, which traverses challenging mountainous and escarpment terrains, has encountered multiple significant geotechnical failures that have resulted in premature pavement failures of this recently completed road project. These failures are:

Failure of roadway cut slope:

One of the primary geotechnical issues observed is the failure of road cut slopes. Many sections of the road project have steep cut slopes that have either already failed or are on the verge of failure. These failures manifest as landslides, rock falls and debris flow, posing hazards to vehicles and pedestrians. The affected areas exhibit deformations such as scarps, bulges, side cracks, visible crack movements, toe erosion and frequent debris flow.

Failure of roadway fill slope:

Additionally, failures in road side fill slopes have also been observed. Over-steep fill slopes, particularly at specific locations along the road are prone to instability, sliding, collapsing and eroding. These failures lead to the loss of soil, damage to the road and becoming potential hazards for road users and pedestrians.

Soil erosion from the surface of roadway cut slope:

Road cutting side slope erosion is another issue that has been identified. The gradual removal of soil and other materials from the side slopes of road cuts has resulted in the accumulation of eroded material near the carriageway, impacting drainage and allowing surface water to flow on the pavement surface.

Uncontrolled seepage:

The occurrence of uncontrolled seepage from the side slope into the pavement materials presents a significant geotechnical challenge. This phenomenon involves the unplanned and unregulated flow of water, which, over time, can weaken the pavement structure and compromise its stability. The detrimental effects of uncontrolled seepage become particularly pronounced under the stress of heavy vehicle loads. Addressing this issue is crucial for ensuring the long-term durability and reliability of the road infrastructure.

Problematic subgrade soil:

Problematic or unsuitable subgrade soils have also been observed in certain locations along the road project. These soils are incapable of furnishing sufficient upkeep to the road pavement structure, leading to settlement, deformation, and safety hazards.

To sum up, the background of the thesis project is framed by the presence of geotechnical failures observed in the Omo River - Tercha road project. These failures include failed road cut slopes, failed side fill slopes, erosion of road cut slopes, uncontrolled seepage from side slopes to the pavement structure, and problematic subgrade soils. These geotechnical challenges have resulted in premature pavement failures, emphasizing the need for a comprehensive evaluation and viable remedies to tackle these problems and guarantee the enduring stability and functionality of the road.

1.2 Statement of the Problem

The occurrence of geotechnical challenges in a recently completed road project. These challenges encompass undetected signs of progressive geotechnical failures, indeterminate causes of the failures, the adverse effects of these failures on the road's performance and the absence of appropriate remedial measures for the road.

1.3 Research Questions

This study will primarily seek to answer the following research questions:

- What are geotechnical failures that arisen in the recently completed road project?
- What are the probable causes of the geotechnical failures that were not identified during the project's design and construction phases?
- How have these geotechnical failures impacted the performance of this newly constructed road project?
- What remedial measures can be recommended to effectively preserve the recently completed road project and prevent further geotechnical issues?

1.4 Research Objectives

1.4.1 General Objectives

The general objective of the thesis project is to assess and address geotechnical challenges in this recently completed road project by identifying geotechnical failures that have been occurred, investigating their probably causes, assessing their impact on the road's performance and proposing appropriate remedial measures.

1.4.2 Specific Objectives

The research will have the following specific objectives to achieve the main goal:

- Recognizing the presence of each geotechnical failure along the project road
- Conducting detailed investigation to determine the likely causes of each geotechnical failures
- Detecting noteworthy geotechnical issues that were unforeseen and overlooked during the design and construction stages of this road project and assessing how these issues have affected the performance of the recently finished road project.
- Recommending appropriate remedial measures to preserve this recently completed road project.

1.5 Scope of the Research

The scope of the thesis project encompasses a comprehensive examination of geotechnical challenges within a recently completed road project. The study will involve the identification and documentation of signs and indicators associated with geotechnical failures along the project road. Moreover, a thorough investigation will be conducted to determine the likely causes of these geotechnical failures. The research will also evaluate the effect of these failures on the road's performance and functionality. Based on the findings, appropriate diagnostic measures will be developed and recommended to address the identified geotechnical challenges, ensuring the preservation and longevity of the road. It is important to note that the scope of this project is limited to the geotechnical features of the road, focusing specifically on the detection, analysis and recommendations related to the geotechnical failures encountered during the design and construction phases.

1.6 Methodology of the Research

The geotechnical investigation in the Omo River-Tercha Road Project aims to address unexpected premature failures shortly after the road's opening. This research employs a systematic methodology, covering failed road cut and fill slopes, uncontrolled seepage, problematic subgrade soils and road cutting side slope erosion. The investigation integrates literature review, initial site surveys, field inspections, laboratory testing and data analysis, utilizing software tools for slope stability assessment. The study seeks to identify root causes and propose effective remedial measures to preserve and enhance the newly constructed road infrastructure.

- The methodology for investigating failed road cut and fill slopes involved a systematic approach. Literature review and document analysis provided insights into the project's history. Initial site investigation and field surveys located failed sections, documented deformations and identified signs of instability. Detailed field investigations collected samples for laboratory testing, analyzing mechanical properties and shear strength. Data analysis and slope stability analysis were performed using geo-studio software.
- For uncontrolled seepage, a detailed site investigation examined geological and hydrological conditions. Laboratory testing determined soil hydraulic conductivity. Data analysis identified seepage sources and pathways, assessing pavement failure.
- The investigation of problematic subgrade soil included site investigation, sampling and testing. Soil samples underwent laboratory testing for physical and engineering properties. Data analysis compared results with standards, identifying deviations and forwarding remedial measures.
- For road cutting side slope erosion, visual inspection during site investigation identified erosion signs. Rainfall and climate data were analyzed to understand erosion triggers. Remedial measures will propose to mitigate and prevent further erosion, addressing underlying causes to preserve pavement performance.

1.7 Significance of the Research

The thesis project holds significant importance for several reasons:

- Practical implications: By addressing geotechnical challenges in a recently completed road project, the findings and recommendations of this research can directly benefit for road projects development and maintenance practices. The diagnostic measures proposed can assist in mitigating geotechnical failures and ensuring the long-term stability and performance of roads.
- Cost reduction: Geotechnical failures in road projects can lead to expensive repairs and maintenance efforts. By identifying the causes and proposing appropriate diagnostic measures, this research can help optimize resources and minimize future costs associated with geotechnical issues.
- Practical guidance: The research outcomes can serve as a valuable reference for engineers, designers, and project managers involved in road construction and maintenance. The recommended diagnostic measures can guide decision-making processes, ensuring geotechnical considerations are appropriately addressed in similar projects.
- Improved road safety: Geotechnical disasters can pose serious risks to road users, including accidents, disruptions and potential damage to vehicles. By understanding and addressing these failures, this thesis project can contribute to enhancing road safety and reducing hazards for commuters.
- Knowledge advancement: The thesis project contributes to the body of knowledge in geotechnical engineering by investigating geotechnical failures in a real-world road project scenario. The findings can expand understanding of geotechnical challenges, their impacts, and effective diagnostic measures, providing valuable insights for future research and development in the field.

CHAPTER 2 LITERATURE REVIEW

Geotechnical engineering plays an important role in the design and construction of road projects. Here are some of the specific roles (Christopher, 2006):

- Site investigation: geotechnical engineers conduct site investigations to determine the physical and engineering properties of the soil and rock at the site. This information is used to assess the appropriateness of the site for road construction and to design the road foundation.
- Design of road foundation: foundation of a road is critical to its long-term stability and durability. Geotechnical engineers use the information gathered from site investigations to design the road foundation, including the thickness and type of material used.
- Slope stability analysis: geotechnical engineers also assess the stability of slopes near the road to prevent slope failure that can cause damage to the road or endanger users.
- Drainage design: proper drainage is essential for the longevity of the road. Geotechnical engineers design drainage systems to ensure that water does not accumulate on the road surface, causing damage to the road structure or making it unsafe for use.
- Material testing and quality control: geotechnical engineers conduct material testing to ensure that the materials used in the road construction meet the required specifications. They also provide quality control during construction to ensure that the road is built according to the design.

2.1 Geotechnical Features of Pavements

There are four major components of a pavement and these are (Brockenbrough, 2009); subgrade which is the top surface of a roadbed upon which the pavement structure and shoulders are constructed, subbase refers to one or more layers of designated or chosen materials with a designed thickness, positioned on a subgrade to provide support for a base course. The base is a layer or layers of specified or selected material with a designed thickness, placed on a subbase or subgrade to support a surface course. The surface course comprises one or more layers in a pavement structure intended to withstand traffic loads, with the top layer resisting skidding, traffic abrasion and the deteriorating effects of weather.

The geotechnical elements within a pavement system encompass the unbound granular base, unbound granular subbase, the subgrade or roadbed, aggregates and geo-synthetics employed in drainage systems. Additionally, graded granular aggregate and geo-synthetics serve as separation and filtration layers, along with the foundation of the roadway embankment.

2.2 Geotechnical Issues in Pavement Design and Performance

Satisfactory pavement performance depends upon the proper design and functioning of all of the key components of the pavement system. These include (Christopher, 2006):

- A surface layer that delivers adequate smoothness, friction resistance and either sealing or drainage of surface water.
- Structural layers with binding materials (such as asphalt or Portland cement concrete) that offer sufficient load-carrying capacity and act as barriers against water intrusion into the underlying unbound materials.
- Unbound base and subbase layers providing additional strength, especially for flexible pavement systems, and resisting moisture-induced deterioration (including swelling and freeze/thaw) and other forms of degradation (e.g., erodibility, intrusion of fines).
- A subgrade ensuring a uniform, sufficiently stiff, strong and stable foundation for the layers above.
- Drainage systems that swiftly remove water from the pavement system before it compromises the properties of the unbound layers and subgrade.
- In certain cases, remedial measures such as soil improvement/stabilization or the use of geo-synthetics to enhance strength, stiffness and/or drainage characteristics of various layers, or to prevent fines contamination by providing separation between layers.

In summary, geotechnical engineering plays a crucial role in ensuring the secure, efficient, and cost-effective construction of road projects. It guarantees that roads are designed and constructed to withstand the stresses and environmental factors they will encounter throughout their lifetime.

2.2.1 Basic Concepts of Pavement Design

Pavements are structured systems created to fulfill the following objectives in accordance with (Garber, 2009):

- to offer a tough framework for bearing the loads imposed by traffic (structural capacity).
- to present a smooth surface for vehicular travel (ride quality).
- to give a skid-resistant surface for enhanced safety.

Moreover, the system should possess ample durability to prevent premature deterioration caused by environmental factors such as water, oxidation, and temperature effects. The unbound soil layers within a pavement contribute significantly to the overall structural capacity of the system, particularly in the case of flexible pavements, often exceeding 50 percent. As shown in Figure 2-1, the stresses generated in a pavement system due to traffic loads are most pronounced in the upper layers and gradually decrease with depth. As a result, the upper layers of all pavement systems utilize higher quality and typically costly materials, while the deeper layers of the pavement employ lower quality and more affordable materials, as shown in Figure 2-2. This optimization of material usage minimizes construction costs and maximizes the ability to use locally available materials.

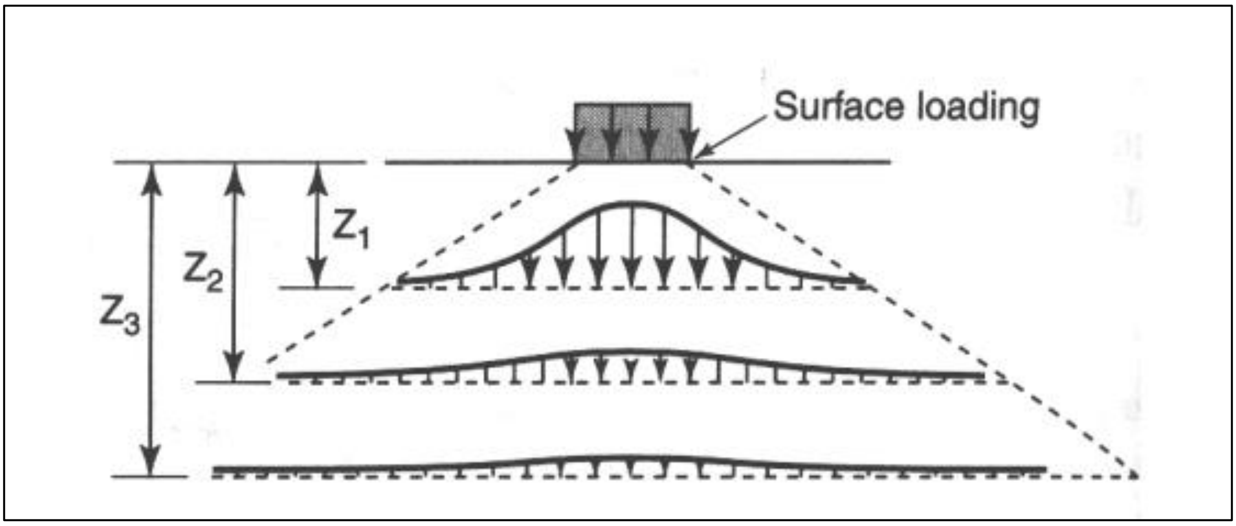

Figure 2-1: Reduction of load-induced stresses with depth ((Christopher, 2006)

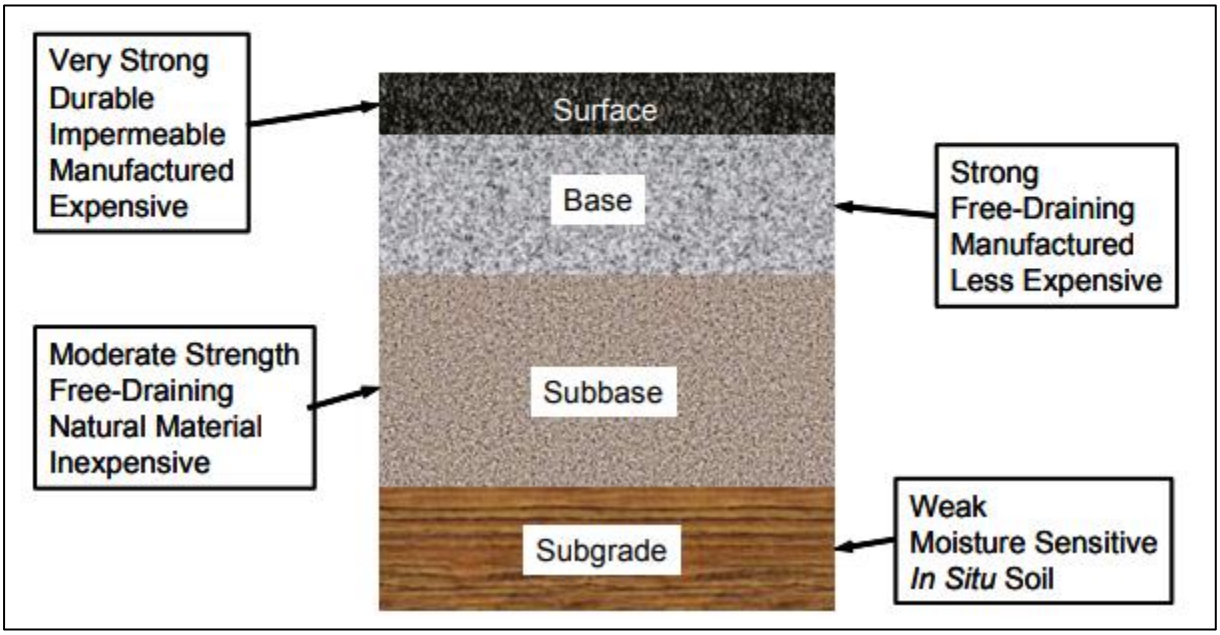

Figure 2-2: Difference of material quality with depth in a pavement system with ideal drainage features ((Christopher, 2006)

Like any geotechnical structures, pavements are significantly impacted by moisture and various environmental elements. Water infiltrates the pavement structure through a combination of surface infiltration, such as through cracks in the surface layer, inflows from the edges (resulting from poorly drained side ditches or insufficient shoulders), and from the underlying groundwater table, often through capillary potential in fine-grained foundation soils.

According to (Moulton, 1980) the presence of moisture within the pavement system almost always adversely affects pavement performance. It diminishes the strength and stiffness of unbound pavement materials, encourages contamination of coarse granular material through fines migration, and may lead to swelling, such as frost heave or soil expansion, followed by consolidation. Moisture introduces significant variability in pavement properties and performance, resulting in local distresses like potholes or more widespread issues such as excessive roughness. Therefore, the design of geotechnical aspects for pavements should prioritize the selection of moisture-insensitive free-draining base and subbase materials, stabilization of moisture-sensitive subgrade soils, and effective drainage to manage infiltrating water within the pavement system.

2.2.2 Key Geotechnical Issues

The geotechnical issues in pavement design can be organized into two categories according to (G.H.McNally, 2003): (i) fundamental issues that establish the overall framework for the design, such as the distinction between new and rehabilitation designs; and (ii) particular technical concerns, such as determining subgrade stiffness and strength, are briefly introduced in the following subsections within each of these categories according to (G.H.McNally, 2003).

2.2.2.1 General Issues

Distinguishing between New Construction, Rehabilitation and Reconstruction significantly impacts various critical geotechnical aspects of pavement design. New construction mandates comprehensive site characterization, involving examinations of geological and soil maps, boring programs, laboratory testing of borehole samples, and geophysical subsurface exploration. The lack of prior knowledge about soil profiles and properties along the new alignment necessitates an extensive exploration and material characterization program. Limited accessibility due to adverse terrain conditions adds to the challenges.

In contrast, rehabilitation projects benefit from original design documents and as-built construction records, offering substantial background information about subsurface conditions along the project alignment. Material properties, such as subbase stiffness, determined during the initial design may become irrelevant due to contamination from subgrade fines. Therefore, new tests may be necessary, either through laboratory tests on samples extracted from borings through the existing pavement or in-situ tests like the Dynamic Cone Penetrometer (DCP) via boreholes through the existing pavement structure. Nondestructive evaluations, such as falling weight deflector-meters (FWD), are commonly used to determine in-place material properties for rehabilitation design. Forensic evaluation of distresses in the existing pavement aids in identifying deficiencies in the underlying unbound layers. However, as the underlying unbound layers are not exposed or removed in typical rehabilitation projects, any deficiencies in these layers must be compensated for by increased structural capacity in the added surface layers.

Natural Subgrade vs. Cut vs. Fill: Pavement construction on a natural subgrade represents the conventional approach in pavement design. The subsurface profile, including the depth to bedrock and the groundwater table, is directly determined through a subsurface exploration program. Subgrade properties required for design can be obtained from tests on the natural foundation soil in both its in-situ condition and its compacted state, particularly if the upper foundation layer undergoes processing, re-compaction, or removal and replacement during construction.

However, in most highway projects, the alignment doesn't always conform to the site topography, leading to the need for various cuts and fills. Geotechnical design in cut and fill areas necessitates special considerations, with attention to transition zones, such as between a cut and an at-grade section, due to potential non-uniform pavement support and subsurface water flow.

For cut sections, drainage becomes a primary concern as the surrounding site slopes toward the pavement structure, and the groundwater table is generally closer to the bottom of the pavement section in cuts. Stabilization of moisture-sensitive natural foundation soils might be necessary. Stability of the cut slopes adjacent to the pavement is a crucial design issue, typically treated separately from pavement design itself.

In contrast, embankments for fill sections are constructed from well-compacted material, often resulting in a higher-quality subgrade than the natural foundation soil. Drainage and groundwater issues are generally less critical for pavements on embankments, although erosion of side slopes from pavement runoff and long-term water infiltration can pose challenges. The primary concerns for pavements in fill sections include the stability of embankment slopes and settlements, either due to compression of the embankment itself or consolidation of soft foundation soils beneath the embankment, typically evaluated by the geotechnical unit as part of roadway embankment design.

2.2.2.2 Environmental Effects

Environmental conditions play a crucial role in influencing the performance of both flexible and rigid pavements. Specifically, moisture and temperature are key environmental variables that can significantly impact the properties of pavement layers and subgrades, consequently affecting pavement performance. The effects of the environment on pavement materials encompass the following aspects:

Moisture-Induced Stiffness Changes in Unbound Materials:

The stiffness of unbound materials tends to decrease with an increase in moisture content. Moisture affects stress states through factors like suction or pore water pressure. Both coarse-grained and fine-grained materials may experience a substantial increase in modulus as they dry, with cohesive soils being influenced by intricate clay-waterelectrolyte interactions. Additionally, moisture can alter the soil structure by disrupting the cementation between soil particles.

Impact on Bound Materials:

Bound materials, such as asphalt mixtures and cement-bound materials, are not directly affected by the presence of moisture. However, excessive moisture can lead to stripping in asphalt mixtures and have long-term consequences for the structural integrity of cement-bound materials.

Damage to Cement-Bound Materials during Environmental Cycles:

Cement-bound materials may incur damage during freeze-thaw and wet-dry cycles, resulting in reduced modulus and increased deflections over time. These environmental cycles can have a lasting impact on the structural performance of cement-bound materials used in pavements.

2.2.2.3 Specific Issues

Material types and properties: The material properties relevant to pavement design can be categorized into various classes, encompassing:

- Physical Properties: Such as soil classification, density, and water content.
- Stiffness and/or Strength: Including resilient modulus, modulus of subgrade reaction, and California Bearing Ratio (CBR).
- Thermo-Hydraulic Properties: Encompassing drainage coefficients, permeability, and the coefficient of thermal expansion.
- Performance-Related Properties: Such as characteristics related to repeated load permanent deformation.

Drainage Considerations: Recognizing the detrimental impact of excess moisture on pavement longevity, it is crucial to address drainage concerns. The combination of heavy traffic and moisture-sensitive materials, especially when subjected to freezing, can accelerate performance deterioration.

Potential sources of moisture in the subgrade and pavement structure include upward seepage from a high groundwater table, lateral flow from pavement edges and shoulder ditches, and infiltration through surface defects. Effectively controlling moisture is essential to minimize related problems.

Based on (Christopher, 2006), a key objective in pavement design is to prevent paving materials from becoming saturated or exposed to constant high moisture levels. Three primary approaches to control or reduce moisture-related issues include:

Prevent Moisture Entry:

- Ensure adequate cross slopes and longitudinal slopes for swift surface water runoff.
- Seal cracks, joints, and discontinuities to minimize surface water infiltration.

Use Moisture-Insensitive Materials and Design Features:

- Utilize materials insensitive to moisture effects, such as granular materials with few fines, cement-stabilized and lean concrete bases, and asphalt-stabilized base materials.
- Incorporate design features like dowel bars and widened slabs for rigid pavements, and full-width paving for flexible pavements.

Efficiently Remove Moisture:

- Implement drainage features like underdrains and ditches to permanently lower the water table.
- Utilize permeable bases and edge drains to remove surface infiltration water promptly.

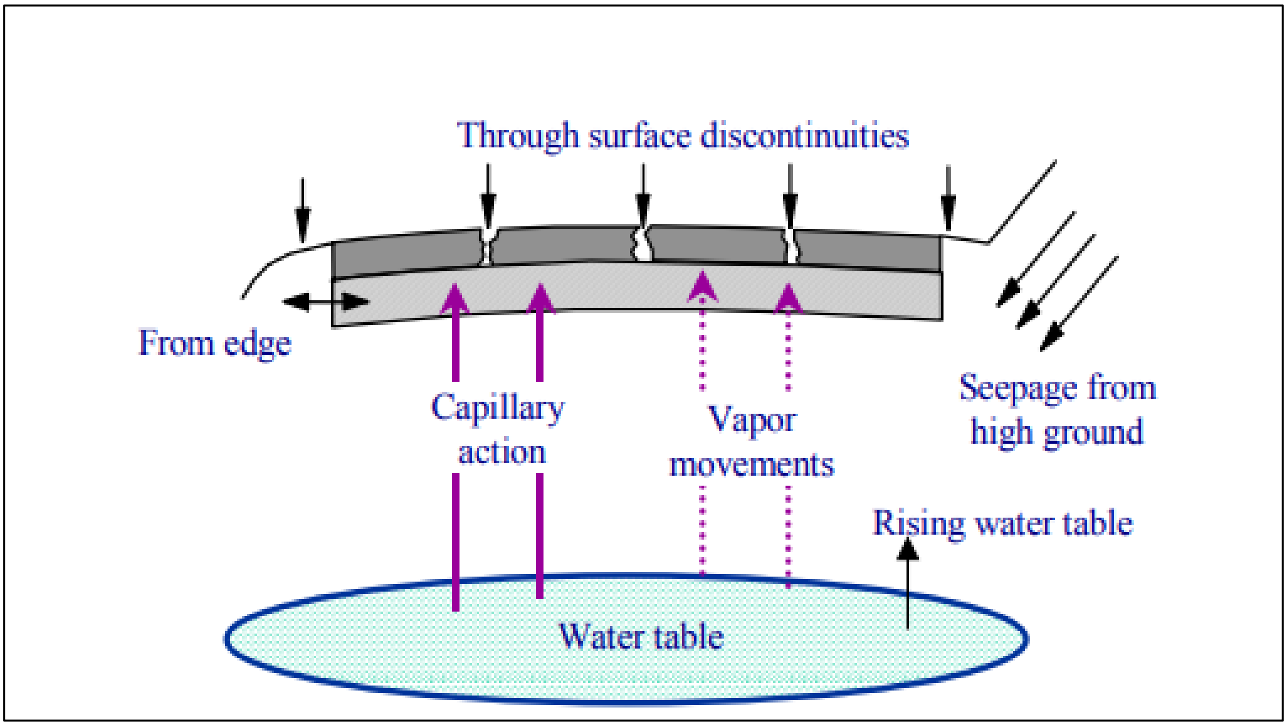

Figure 2-3: Sources of moisture for pavement structure (Look, 2007)

2.3 Geotechnical Inputs for Pavement Design

As per (Nikolaides, 2015)geotechnical properties are typically required during pavement design and construction. These include standard properties required for soil identification and classification, compaction control and field quality control and quality assurance.

2.3.1 Physical Properties

The fundamental characteristics of unbound materials are outlined by physical properties, offering a foundational description. These properties frequently serve as key components in establishing correlations for more advanced engineering attributes like stiffness or permeability. The primary physical properties under consideration encompass the specific gravity of solids, water content, unit weight (density), gradation characteristics, plasticity (Atterberg limits), classification and compaction characteristics.

Laboratory and field methods (where appropriate) (Das, SOIL MECHANICS LABORATORY MANUAL, 2002)for determining the physical properties of unbound materials in pavement systems are described in the following subsections.

Specific gravity of soil and aggregate solids

- Description: The specific gravity of soil solids (Gs) is defined as the ratio of the weight of a certain volume of soil solids at a specific temperature to the weight of an equivalent volume of distilled water at the same temperature.
- Uses in pavements: Utilized in determining soil unit weight, void ratio, and other volumetric properties. Also applied in the analysis of hydrometer tests for particle distribution in fine-grained soils.
- Laboratory determination: AASHTO T 100 or ASTM D 854.

Moisture content

- Description: the moisture content expresses the amount of water present in a quantity of soil. The gravimetric moisture or water content w is defined in terms of soil weight as $w = W_W/W_S$ = where Ww is the weight of water and Ws is the weight of the soil solids in the sample.
- Uses in pavements: employed to ascertain the total unit weight of soil, void ratio, and various volumetric properties, and to establish correlations with soil behavior and other pertinent soil properties.
- Laboratory determination: drying of the soil in a conventional (temperature of 110±5°C) or microwave oven to a constant weight (AASHTO T 265, ASTM D 2216/conventional oven, or ASTM D 4643/microwave).

Unit weight

- Description: The unit weight represents the ratio of the total weight to the total volume of a soil sample.
- Uses in pavements: Calculation of in-situ stresses, correlations with soil behavior and other properties, and for compaction control.
- Laboratory determination: In the laboratory, the unit weight of undisturbed fine-grained soil samples is determined by weighing a portion of the sample and dividing it by its volume, typically using thin-walled tube (Shelby) samples. In cases where undisturbed samples are unavailable, such as with coarse-grained soils, the unit weight is assessed through weight-volume relationships.

Compaction characteristics

- Description: compaction characteristics are depicted by the relationship between equivalent dry unit weight and moisture content for a soil at a specific compaction energy level. Of particular significance are the maximum equivalent dry unit weight and the corresponding optimum moisture content at the designated compaction energy level.

- Uses in pavements: used in combination with other tests (e.g., CBR), this test determines the impact of soil density on engineering properties. It serves as a tool for field quality control and assurance during the compaction of natural subgrade, placed subbase and base layers and embankment fills.
- Laboratory determination: two sets of test protocols are most commonly used: AASHTO T 99 (Standard Proctor), T 180 (Modified Proctor) and ASTM D 698 (Standard Proctor), D 1557 (Modified Proctor).

Grain size distribution of particles (coarse)

- Description: the grain size distribution represents the percentage of soil finer than a specified size versus grain size. Coarse particles are those exceeding 0.075 mm (No. 200 sieve).
- Uses in pavements: soil classification and correlations with other engineering properties
- Laboratory determination: the grain size distribution of coarse particles is determined from a mechanical washed sieve analysis (AASHTO T 88, ASTM D 422).

Grain size distribution of fine particles (hydrometer analysis)

- Description: the grain size distribution is the percentage of soil finer than a given size vs. grain size. Fine particles are defined as smaller than 0.075 mm (No. 200 sieve).
- Uses in pavements: soil classification and correlations with other engineering properties
- Laboratory determination: the grain size distribution of fine particles is determined from a hydrometer analysis (AASHTO T 88, ASTM D 422).

Plasticity of fine-grained soils

- Description: plasticity describes the response of a soil to changes in moisture content. Plasticity is quantified by Atterberg limits.
- Uses in pavements: soil classification and correlations with other engineering properties

- Laboratory determination: Atterberg limits are ascertained through testing procedures outlined in AASHTO T89 (liquid limit), AASHTO T90 (plastic limit), AASHTO T 92 (shrinkage limit), ASTM D 4318 (liquid and plastic limits), and ASTM D 427 (shrinkage limit). A representative sample is extracted from the soil portion passing through the No. 40 sieve, and the moisture content is adjusted to recognize three stages of soil behavior concerning consistency.

2.3.2 Mechanical Properties

Stiffness stands out as the pivotal mechanical attribute of unbound materials in pavements, determining the allocation of stresses and strains across the pavement system, (Kézdi, 1988).

The subsequent section elaborate on laboratory and field techniques employed to ascertain the stiffness and other pertinent mechanical properties of unbound materials within pavement systems.

California Bearing Ratio (CBR)

- Description: The California Bearing Ratio or CBR is an indirect measure of soil strength based on resistance to penetration.
- Uses in pavements: directly incorporated into certain empirical pavement design methodologies and correlated with resilient modulus and other engineering properties
- Laboratory determination: CBR is determined by evaluating the resistance to penetration from a standardized piston moving at a fixed rate for a specified penetration distance. Test methods are AASTHO T 193 or ASTM D 1883.

2.4 Geotechnical Investigation of Failures along Roadway

Geotechnical failures of roads refer to the failure of the road or its supporting structures due to issues related to the soil, rocks, or other geotechnical factors. These failures can lead to significant damage, inconvenience, and even danger for road users.
Some common examples of geotechnical failures of roads include according to ((ERA), Geotechnical Investigation Manual , 2013) are the followings.

- Slope failures: this occurs when a road is constructed on a hill or a slope, and the soil or rocks supporting the road give way. It can be caused by heavy rainfall, seismic activity, or erosion.
- Settlement: settlement happens when the soil below the road compresses and shifts, leading to the road sinking or unevenly settling. This can be caused by poor soil conditions, inadequate compaction during construction, or changes in the moisture content of the soil.
- Lateral spreading: lateral spreading happens when the soil supporting the road moves horizontally, causing the road to crack, buckle, or shift. It can be triggered by earthquakes, liquefaction, or landslides.
- Foundation failure: foundation failure occurs when the supporting structures beneath the road, such as retaining walls or piles, fail. This can be caused by poor design, inadequate construction, or changes in the soil conditions.
- Erosion: erosion occurs when soil and rocks are removed from the road surface or supporting structures, causing instability and damage. This can be initiated by natural factors like heavy rainfall or man-made factors like inadequate drainage.

To prevent geotechnical failures of roads, engineers and construction professionals must conduct detailed geotechnical surveys to assess soil and rock conditions, design structures that can withstand potential hazards, and use proper construction techniques and materials. Regular conservation and observing of the road and its supporting structures are also essential to detect and address any potential issues before they lead to failure.

2.4.1 Geotechnical Investigation of Slope Failures

According to (ABRAMSON, 2002) gravitational forces are always acting on a mass of soil or rock beneath a slope. Slope failure and movement, such as creep, falls, slides, avalanches, or flows, occur when there is an imbalance of forces. Failure transpires when the driving forces surpass the resisting forces. The initiation of a landslide takes place when the tangential (shearing) stresses within a soil mass exceed its capacity to resist. The augmentation of active shearing forces may arise from the construction of an engineering structure on the slope, an increase in the weight of the soil mass, steeper gradients, etc. Conversely, a decrease in resisting forces may occur with the removal of lateral support, such as during the excavation of a cut across the slope.

Slope failures such as landslides can occur in almost any hilly or mountainous terrain. Types of slope failures are listed in Table 2-1.

Table 2-1: Types of slope failures ((ABRAMSON, 2002)

Type	Form	Definition
Falls	Free fall	Sudden dislodgement of single or multiple blocks of soil or rock which fall in free descent.
	Topple	Overturning of a rock block about a pivot point located below its center of gravity.
Slides	Rotational or slump	Relatively slow movement of an essentially coherent block (or blocks) of soil, rock, or soil-rock mixtures along some well-defined arc-shaped failure surface. Rupture failure may appear as a series of consecutive shears occurring along load-induced lines of sliding. Since the shear line is curved, the displacement of a slump occurs followed by rotation of the moving slump about one or several successive instantaneous centers.
	Planar or translational	Slow to rapid movement of an essentially coherent block (or blocks) of soil or rock along some well-defined planar failure surface. Unlike a rotational shear failure, it demonstrates absence of a surface of sliding dissecting the soil mass of the slope. The surface of sliding is substituted by surfaces of bed rocks comprising the slope.
Avalanches	Rock or debris	Rapid to very rapid movement of an incoherent mass of rock or soil-rock debris wherein the original structure of the formation is no longer discernible, occurring along an ill-defined surface.
Flows		Soil or soil-rock debris moving as a viscous fluid or slurry, usually terminating at distances far beyond the failure zone; resulting from excessive pore pressure. It occurs locally where ground water is discharged to the surface of the slope, say, as spring.
Creep		Slow, imperceptible down slope movement of soil or soil-rock mixtures.

2.4.2 Causes of Slope Failures

Slope failure occurs either due to a decrease in soil strength or an increase in stress (Duncan, 2014):

Typical situations conducive to land sliding induced by proposed cuts or fills:

- Constriction of the flow of groundwater due to side hill fill.
- Imposition of excess load on a relatively fragile underlying soil layer through the addition of fill.

- Application of heavy side hill fill leading to the overloading of sloping bedding planes.
- Steepening of cuts in unstable rock or soil beyond a safe limit.
- Elimination of a thick mantle or porous soil layer by cuts, especially if it serves as a natural restraining blanket over a soft core.
- Augmentation of seepage pressure resulting from changes in the direction and nature of groundwater flow caused by cuts or fills.
- Exposure of stiff, fissured clay through cutting, which may undergo softening and swelling upon contact with surface water.
- Stripping away of a wet soil mantle by side hill cuts, potentially compromising toe support and triggering sliding along the interface with stable bedrock.

According to (Look, 2007), the macro factors causing slope failures are listed Table 2-2:

Table 2-2: Macro factors causing slope movements

Macro factor	Effects
Tectonics	Increased height that results in an angle change.
Weathering	Chemical and physical processes resulting in disintegration and break down of material. Subsequent removal of the material by water.
Weathering	Removes material, either in a small-scale surface erosion or major undercutting of cliffs and gullies. Aided by wind and gravity. Water increases dead weight of material and /or increased internal pressure to dislodge the material.
Weathering	Downward movements of material due to its dead weight.
Dynamic	Due to natural vibrations such as earthquakes, waves or man-made such as piling and blasting.

2.4.3 Slope Failure Development Stages

The stages of the slope failure are as per (Hunt, 2005):

- Initial stage: the early stage is characterized by slow, often discontinuous movements, tension cracks, and tilted and bent trees. Recent movements are characterized by tilted but straight trunks; older, continuing movements result in bent trunks.
- Intermediate stage: significant movement occurs, often still discontinuous, accompanied by large displacements. This stage is characteristic of slides,

especially in the progressive mode. It is during the intermediate stage that the soils may be considered as colluvium, although the displaced blocks are often intact.

- Final stage: total displacement has occurred, leaving a prominent failure scar, resulting in a mature colluvium deposit. Additional movement can still occur. Normally only falls, avalanches, and flows constitute the final stage because failure is usually sudden

2.4.4 Failures of Side Hill Fills

As per (Cheney, 1989), when side fills are placed on moderately steep to steep slopes of residual or colluvium soils they are frequently prone to stability problem unless seepage is properly controlled or the embankment is supported by a retaining structure. The construction of the embankment with impervious material can be expected to block the natural drainage and evaporation. As seepage pressures increase, particularly at the toe, the embankment strains and concentric tension cracks form. The movements develop finally into a rotational failure with slumping at the fill toe.

It is also advised to check if there is overloading of relatively weak underground soil layer by the fill. It is also important to check if the fill materials are properly compacted. When fill materials are not compacted adequately, they normally absorb water and develop pore water pressure during rainy season. Due of the pore pressure and an increase in weight, tension cracks may occur which will be followed by settlements.

2.4.5 Remedial Measures for Slope Failures

The following measures can be taken for treating a road side fill slope failures (Hunt, 2005):

- Route relocation, geologic conditions often differ from one side of a valley or mountain slope to the other, even where the materials are similar. For example, on one side of the valley or mountain slope dipping beds may incline out of a cut slope, whereas on the opposite side the dip of the beds is into the slope.
- Where colluvium soils exist they often are thickest and most unstable along the lower slope elevations and at times can be avoided by locating an alignment at higher elevations up slope. Residual soils can also be found with higher strengths and less exposure to changes in seepage conditions upslope.
- Removal of the land slide entirely or partially at the toe and placing drains to intercept seepage. The area is then backfilled with appropriate material.

- Bridging, whereby the land slide area is avoided by a bridge between the two solid extremities of the moving area.
- Cementation of loose material, the material to be cemented should be permeable. Cement grout is injected into the moving area. It produces a material that has higher shear resistance. In cohesive soils vertical columns are obtained and their effect is that of a system of piles.

According to (Moulton L. , 1980) the following remedial measures can be taken in order to mitigate failed highway fill slope.

Providing internal drainage

Internal seepage forces are removed by drainage systems, including upslope trenches, wells, horizontal and vertical drains and trenches at the toe of slope (toe drains). The purpose of the drains is to lower the piezo-metric head below the potential sliding surface.

- Sub horizontal drains are installed at a slight angle upslope to provide for gravity flow. They consist of perforated pipes forced into a predrilled hole.
- Trenches at the toe of slope relieve pore pressures where high seepage forces in the toe area are the major causes of instability.

Removal of material at the top

The quickest expedient to stabilize a moving slope mass where it is not excessively large is often to remove material from the head of the sliding area by excavation and, if space allows, to place it at the toe to increase sliding resistance.

Reduction of slope inclination

Existing natural slope angles represent the maximum stable slope angle. The maximum slope angle and the stable slope height reflect the strength parameters of cohesion and friction and the ground water conditions. The first approach to stabilization when a cut is to be made is the determination of the angle at which the cut will remain stable without support.

Sealing of cracks on the surface of slope

It is a slope treatment that prevents the ingress of surface water into the slide mass and minimizes erosion. Individual cracks may be sealed by hand filling with clay, bituminous materials etc.

Vegetation

Establishing dense native vegetation on the slope contributes to fortifying the superficial soils through robust root systems. This planting practice not only acts as a deterrent against erosion but also hinders infiltration, thereby reducing seepage pressures. Vegetation stands out as a crucial factor in stabilizing slopes.

Vegetation discourages desiccation which causes fissuring. The presence of profound fissures creates pathways for rainwater to infiltrate the sliding mass, amplifying seepage pressure within the mass while exerting hydrostatic pressure against the walls of the fissure or crack.

The slops which are susceptible to failure by sliding can be improved and made usable and safe. Various methods are used to stabilize the slopes. The methods generally involve one or more of the following measures, which either reduce the mass which may cause sliding or improve the shear strength of the soil in the failure zone (Arora, 2004).

- Flattening the slope diminishes the mass's weight that may lead to sliding. It can be used wherever possible.
- Enhancing resistance to movement is achieved by installing a berm below the slope's toe, particularly beneficial in scenarios where there's a risk of base failure.
- Drainage helps in reducing the seepage forces and hence increases the stability.
- The zone of subsurface water is lowered and infiltration of the surface water is prevented.
- Consolidation by surcharging, electro-osmosis or other methods helps in increasing the stability of slopes in cohesive soils.
- Injecting cement or other compounds into targeted areas through grouting aids in enhancing slope stability.
- Lateral support and increased stability can be achieved by implementing sheet piles and retaining walls; nevertheless, this approach tends to be costly.
- Enhancing soil stability contributes to the overall increase in slope stability.
- To maintain cost-effectiveness, economical methods like slope flattening and drainage control are typically favored.

2.5 Drainage in Pavements

According to (White, 1994), providing adequate drainage to a pavement system has been regarded as a critical design consideration in order to avoid premature failures caused by water-related issues such as pumping action, loss of support, and rutting, among others. Rainfall infiltration into unsaturated pavement layers, through joints, cracks, shoulder edges and several other absences, accounts for the majority of water in pavements, particularly in older deteriorated pavements. Water may also seep upward from a high groundwater table as a result of capillary suction or vapor movements, or it may flow laterally from pavement edges and side ditches.

2.5.1 Pavement Subsurface Drainage Systems

Paved and unpaved roads are subjected to problems associated with excess water within the foundation structure of the roadway. The excess water arises from infiltration along the road surface, groundwater seepage from higher elevations, elevated water levels in road ditches, or the upward movement of groundwater beneath the roadway. The heightened moisture content in the road foundation results in a deterioration of its strength, ultimately leading to the failure of the surface, whether it is paved or unpaved. The economic impact of pavement damage due to excessive water is estimated to reach tens of millions of Birrs annually at the national level, (ERA, Drainage Design Manual , 2013)

The necessity and effectiveness of subsurface drainage are critical considerations in meeting pavement performance expectations and protecting public investments. In a drained pavement structure, there is usually an underlying layer designed to rapidly remove water, thereby reducing or eliminating adverse impacts on moisture-sensitive subbase or subgrade materials. The design, constructability, and performance of a drainage layer heavily rely on the aggregate gradation and associated hydraulic conductivity. This layer can be built with or without an asphalt or cement stabilizer and typically necessitates a separation layer to prevent clogging caused by subgrade fines, (Kathleen T. Hall, 2022). An excess of water in a pavement structure can cause or exacerbate pavement distress, slope instability alongside the roadway, and loss of pavement strength, stability, and durability. Pavement subsurface drainage systems are intended primarily to remove water that infiltrates pavement structures through joints and cracks in the pavement surface or through lateral groundwater seepage, fluctuations in groundwater levels, and capillary action. The two types of subsurface drainage systems used to remove infiltrated water are those with a permeable base layer, longitudinal edge drains, and transverse outlets to the roadside ditch, and those with a permeable base layer day-lighted to the roadside ditch.

Pavement subsurface drainage systems benefit pavement performance most when they are used at locations where they are warranted by climate and soil conditions and when they are designed, constructed, and maintained to ensure timely and unobstructed water outflow. (Kathleen T. Hall, 2022)

- Subsurface drainage, a system employed to intercept, gather, and expel water that permeates into the pavement section;
- Surface infiltration, the entry of water into the pavement section through longitudinal and transverse joints, cracks between travel lanes, and the curbs or shoulders;
- Subsurface infiltration, water penetrating the pavement section via lateral groundwater seepage, changes in groundwater levels, or capillary action;
- Drainage layers, a porous aggregate layer within the pavement section facilitating the removal of infiltrated water. This layer can be un-stabilized, asphalt-stabilized, or cement-stabilized, and is often denoted as a permeable, open-graded, or free-draining layer;
- Edge drain, an underdrain located along a longitudinal edge of the pavement;
- Filter layer, a stratum beneath the drainage layer intended to allow water drainage while preventing soil or aggregate particles in the underlying layer (subbase or subgrade/embankment) from infiltrating into the drainage layer;
- Underdrain, a conduit designed to eliminate water from a drainage layer;

2.5.2 Types of Highways Sub-Drainage Systems

According to (Nicholas J. Garber and Lester A. Hoel, 2009), the categorization of subsurface drainage systems typically falls into five broad classifications:

- Longitudinal drains
- Transverse drains
- Horizontal drains
- Drainage blankets
- Well systems

Longitudinal Drains

Longitudinal drains beneath the surface typically involve the placement of pipes in trenches within the pavement structure, running parallel to the highway's centerline. These drains serve the purpose of either lowering the water table beneath the pavement structure or extracting water that infiltrates into the pavement. In instances where the water table is exceptionally high and the highway is wide, employing more than two rows of longitudinal drains may be necessary to attain the needed reduction of the water table beneath the pavement structure.

Transverse Drains

Below the pavement, transverse drains are positioned typically perpendicular to the center line, though they may be arranged in a skewed manner to create a herringbone configuration. These drains are employed to alleviate groundwater infiltration through the pavement joints. One drawback of transverse drains is their potential to induce pavement unevenness in regions prone to frost action, where overall frost heaving occurs. This unevenness arises from the general heaving of the entire pavement, except at the locations of the transverse drains.

Horizontal Drains

Horizontal drains are used to relieve pore pressures at slopes of cuts and embankments on the highway. They usually consist of small diameter, perforated pipes inserted into the slopes of the cut or fill. The subsurface water is collected by the pipes and is then discharged at the face of the slope through paved spillways to longitudinal ditches.

Drainage Blankets

A drainage blanket refers to a layer of material with an exceptionally high coefficient of permeability, typically exceeding 9 m/day. It is positioned beneath or within the pavement structure, with its width and length extending significantly more in the flow direction than its thickness. The coefficient of permeability represents the constant that relates the flow velocity to the hydraulic gradient between two points within the material. Drainage blankets serve the purpose of directing subsurface water away from the pavement and enabling the flow of groundwater that has permeated through cracks into the pavement structure, as well as subsurface water from artesian sources.

Well Systems

A well system consists of a series of vertical wells, drilled into the ground, into which ground water flows, thereby reducing the water table and releasing the pore pressure. When used as a temporary measure for construction, the water collected in the wells is continuously pumped out, or else it may be left to overflow. A more common construction, however, includes a drainage layer either at the top or bottom of the wells to facilitate the flow of water collected.

2.5.3 Effect of Inadequate Subsurface Drainage

Inadequate sub-drainage on a highway will result in the accumulation of uncontrolled subsurface water within the pavement structure and/or right of way, which can result in poor performance of the highway or outright failure of sections of the highway.

The effects of inadequate sub-drainage fall into two classes: poor pavement performance and instability of slopes (Nicholas J. Garber and Lester A. Hoel, 2009).

Pavement performance: - if the pavement structure and subgrade are saturated with underground water, the pavement's ability to resist traffic load is considerably reduced, resulting in one or more of several problems, which can lead to premature destruction of the pavement if remedial actions are not taken in time.

Slope Stability: - the presence of subsurface water in an embankment or cut can cause an increase of the stress to be resisted and a reduction of the shear strength of the soil forming the embankment or cut. This can lead to a condition where the stress to be resisted is greater than the strength of the soil, resulting in sections of the slope crumbling down or a complete failure of the slope

Water enters the pavement structure in many ways, such as through cracks, joints or pavement infiltration, or as groundwater from an interrupted aquifer, high water table, or localized spring. Effects of this water (when trapped within the pavement structure) on pavement include as per (AASHTO, 1993).

- Diminish the strength of unbound granular materials.
- Weaken the strength of roadbed soils.
- Induce the pumping of fines in the aggregate base under flexible pavements, leading to a subsequent loss of support.

Less frequently noticed problems due to entrapped water include:

- Stripping of asphalt concrete
- Differential heave on swelling soils

Free water in the base, subbase, and subgrade is of particular concern because it can decrease the strength in the following ways (RIDGEWAY, 1982):

- Reducing the apparent cohesion by lowering the capillary forces;
- Reducing the friction by reducing the effective mass of the materials below the water table; and

- For quickly applied loads, possibly reducing the strength by the development of increased and / or oscillating pore pressure.

When high pore pressures are developed in a base or subbase material, its load transfer properties are altered considerably so that the stresses applied to the subgrade are not reduced to their expected level.

2.5.4 Groundwater and Seepage Problems on Road

According to (Nijland, 2005), groundwater is an important consideration in pavement design and construction because it will have significant impacts on the performance and longer life span of pavements. When water accumulates under pavement, it can cause soil saturation that leads to pavement subgrade instability, deformation, and eventually failure. Therefore, it is essential to understand the effects of groundwater and seepage on pavements.

Groundwater can affect pavement in several ways. One of the primary effects of groundwater is the weakening of the pavement structure due to soil saturation. When the soil under the pavement becomes saturated, it loses its strength, causing the pavement to sink or deform. This can lead to unevenness on the pavement surface, cracking, and potholes. The accumulation of water under the pavement can also cause erosion of the underlying soil, leading to further pavement damage.

Another effect of groundwater on pavements is related to the freeze-thaw cycle. In colder climates, water can enter into pavement cracks and joints and freeze during the winter months. As the water freezes, it will expand and exerts pressure on the pavement, leading to cracking and other forms of damage. When the ice thaws, the water can penetrate further into the pavement, elevate the damage.

Seepage can also have an impact on pavement performance. Seepage occurs when water flows through the pavement layers, carrying away fines and weakening the soil structure. This can lead to pavement subgrade instability, unevenness, and ultimately, failure. Seepage can also lead to the formation of potholes and cracks as the pavement layers shift and settle.

To mitigate the effects of groundwater and seepage on pavements, designers and engineers often include drainage systems in the pavement design. These drainage systems can include subgrade drainage layers, edge drains, and other features that help to remove water from the pavement structure. Properly designed drainage system can prevent water from accumulating under the pavement, reducing the risk of damage and extending the pavement's service life.

Generally, groundwater and seepage can have significant impacts on pavement performance and longevity. It is essential to consider these factors when designing and constructing pavements to ensure that they can withstand the effects of water infiltration and drainage. Proper drainage systems can mitigate the effects of groundwater and seepage, helping to ensure that pavements remain safe, functional, and durable for several years.

Groundwater and seepage control is required for pavement design and construction for the following situations (Hunt, 2005): -

- For pavements to provide protection against pumping and frost heave
- For slopes to provide stabilization in either natural or cut conditions

Groundwater is the water that is found beneath the earth's surface in underground reservoirs, known as aquifers. In the context of pavements, groundwater can be a significant factor in the occurrence of two primary issues: pumping and frost heave.

Pumping occurs when water is forced out of the ground due to the repeated loading and unloading of the pavement. This can happen when vehicles drive over the pavement, causing the underlying soil to compress and forcing water to the surface. Over time, this can cause erosion of the soil beneath the pavement, leading to instability and ultimately, pavement failure.

Frost heave occurs when water in the soil freezes and expands, causing the soil to push up and lift the pavement. This can occur when water is present in the soil during freezing temperatures and can result in significant pavement damage.

To protect against these issues, several mechanisms can be used to groundwater and seepage control. One common approach is to install a layer of aggregate or gravel beneath the pavement to allow water to drain away from the pavement's surface. This can help reduce the potential for pumping and frost heave by allowing water to move freely through the soil.

In addition, a subsurface drainage system can be installed to collect and remove groundwater from beneath the pavement. This can be achieved through the installation of perforated pipes or porous pavements that allow water to drain through to a collection system. Properly designed subsurface drainage systems can effectively reduce the potential for pumping and frost heave, improving pavement performance and longer life.

Generally, controlling groundwater and seepage is critical to protect pavements from issues such as pumping and frost heave. By using appropriate drainage systems and materials, pavement designers and engineers can reduce the potential for these problems, ensuring long-lasting and safe pavement infrastructure.
Seepage flow is the movement of water through soil or rock under the influence of gravity. When a sloping ground is present, the flow of water can be affected and may flow towards a pavement structure. This can cause problems for the stability and durability of the pavement.

2.5.5 Seepage Flow from Sloping Ground

The condition for seepage flow to occur is the presence of a slope in the ground, where the water table is higher on the upslope side and lower on the downslope side. This creates a pressure gradient, and water flows downhill along the path of least resistance, which can be towards a pavement structure (ROCHESTER, 1996).
The hydraulic conductivity of soils depends on several factors: fluid viscosity, pore-size distribution, grain-size distribution, void ratio, roughness of mineral particles, and degree of soil saturation (Das, 2014).
The controlling mechanism for seepage flow from a sloping ground to a pavement structure is the design and construction of a proper drainage system. This drainage system can consist of a combination of surface drainage (such as gutters or channels), sub-surface drainage (such as pipes or trenches), and proper grading of the pavement surface.
The surface drainage system should be designed to divert surface water away from the pavement structure and towards suitable collection points. The sub-surface drainage system should be designed to intercept and collect any water that seeps through the soil and prevent it from reaching the pavement structure.
In addition to drainage, proper grading of the pavement surface is also essential to ensure that water does not accumulate and cause excess water or flooding. The pavement surface must be sloped away from the pavement structure and towards the drainage system to allow for the efficient flow of water.
Overall, a well-designed drainage system and proper grading can effectively control seepage flow from a sloping ground to a pavement structure and prevent any potential damage.
It is important to design and construct an appropriate drainage system to control seepage flow from sloping ground to pavement structures. Seepage flow refers to the movement of

water through soil or porous material, which can be particularly problematic for pavements as it can lead to structural damage and erosion.

To prevent seepage flow from damaging pavements, a drainage system must be put in place. This system must consist of several components which including surface drainage, sub-surface drainage, and proper grading of the pavement surface according to (C.A. Oflaherty, 2002).

- Surface drainage refers to the use of gutters or channels to direct water away from the pavement surface. These features can be installed alongside the pavement or integrated into its design and are effective at diverting water that would otherwise seep into the ground beneath the pavement.
- Sub-surface drainage involves the use of pipes or trenches to collect and channel water away from the pavement structure. This is particularly important in areas where the soil has a high-water table, or where there is a risk of groundwater seepage. Sub-surface drainage systems can also help to prevent the buildup of hydrostatic pressure, which can cause structural damage to the pavement.
- Proper grading of the pavement surface is also essential for controlling seepage flow. The surface should be sloped in such a way that water is directed towards the surface drainage features and away from the pavement structure. This will help to ensure that water does not accumulate on the surface or penetrate the ground beneath the pavement.

So, the controlling mechanism for seepage flow from sloping ground to pavement structures is a proper drainage system. This system must include a combination of surface drainage, sub-surface drainage and proper grading of the pavement surface to effectively divert water away from the pavement structure and prevent seepage-related damage.

2.6 Roadway Subsidence

Roadway subsidence failure refers to the sinking or settling of a roadway due to the failure or instability of the underlying soil or rock. This can occur for a variety of reasons and can result in serious safety hazards, as well as damage to vehicles and infrastructure. There are several causes of roadway subsidence, including (Molenaar, 1996):

- Soil consolidation: soil consolidation is a natural process that occurs when soil particles settle and compact over time. This can lead to subsidence if the soil beneath a roadway is not properly prepared or if the road is built on top of poorly compacted fill material.

- Sinkholes: sinkholes are depressions in the ground that can be caused by natural processes such as erosion or by human activities such as mining or drilling. If a sinkhole forms beneath a roadway, it can cause the roadway to sink or collapse.
- Soil erosion: soil erosion can occur due to natural processes such as rainfall or due to human activities such as construction. When soil erodes beneath a roadway, it can cause subsidence and structural failure.

Remedial measures for roadway subsidence depend on the underlying cause. Some possible solutions include (Horner, 1988):

- Grouting: - grouting involves injecting a fluid material such as cement or polyurethane into the soil beneath the roadway to fill voids and stabilize the soil.
- Soil stabilization: - soil stabilization techniques such as soil cement or lime stabilization can be used to improve the strength and stability of the soil beneath the roadway.
- Geo-synthetics: - geo-synthetics such as geotextiles and geo-grids can be used to reinforce the soil beneath the roadway and prevent subsidence.
- Drainage: - improving drainage can help prevent subsidence by reducing the amount of water that accumulates in the soil beneath the roadway.
- Reconstruction: - in some cases, it may be necessary to reconstruct the roadway to address subsidence issues. This can involve removing and replacing the affected section of roadway, or constructing a new roadway on top of stable soil or rock.

CHAPTER 3 RESEARCH METHODOLGY

This research methodology outlines the steps taken to investigate and identify geotechnical problems that caused the premature failure of a road project. It also provides recommendations for remedial measures to prevent such failures in the future. The methodology aims to offer a comprehensive and systematic approach to geotechnical investigations for road construction projects, ensuring their long-term success and sustainability.

3.1 Project Description

The construction of the Omo River-Tercha road project, spanning approximately 81 km, commenced in July 2015. This project implemented a Design-Build contract approach, where the contractor submitted designs for approval by the project consultant. Despite the original construction period being set at 4 years, the completion of the road project extended to around 8 years. Shortly after the road opened for traffic, several geotechnical failures emerged, challenging the intended design life of the road. This resulted in additional costs for investigating and addressing these premature failures, leading to the emergence of contract claims.

The project road traverses mountainous and escarpment terrains, featuring numerous high fill and cut sections. The region experiences an extended rainy season with heavy rainfall, multiple sources of groundwater and areas sensitive to moisture. Unfortunately, these factors were not adequately considered and the involvement of geotechnical engineers was minimal. Their engagement was limited to instances when failures occurred, neglecting proactive consideration of these geotechnical challenges during the design and construction phases.

3.2 Project Location

The project area is located in the southern part of Ethiopia and found in the South West Ethiopia Peoples' Region, Dawro Zone as shown Figure 3-1. The project area is accessible by asphalt road from Addis Ababa – Butajira – Worabe – Alaba – Wolayita Sodo. After Wolayita Sodo the road bifurcates and goes to Beli town passing through Omo River (Gilgel Gibe III) to Gessa – Tercha Road project which is all weather roads.

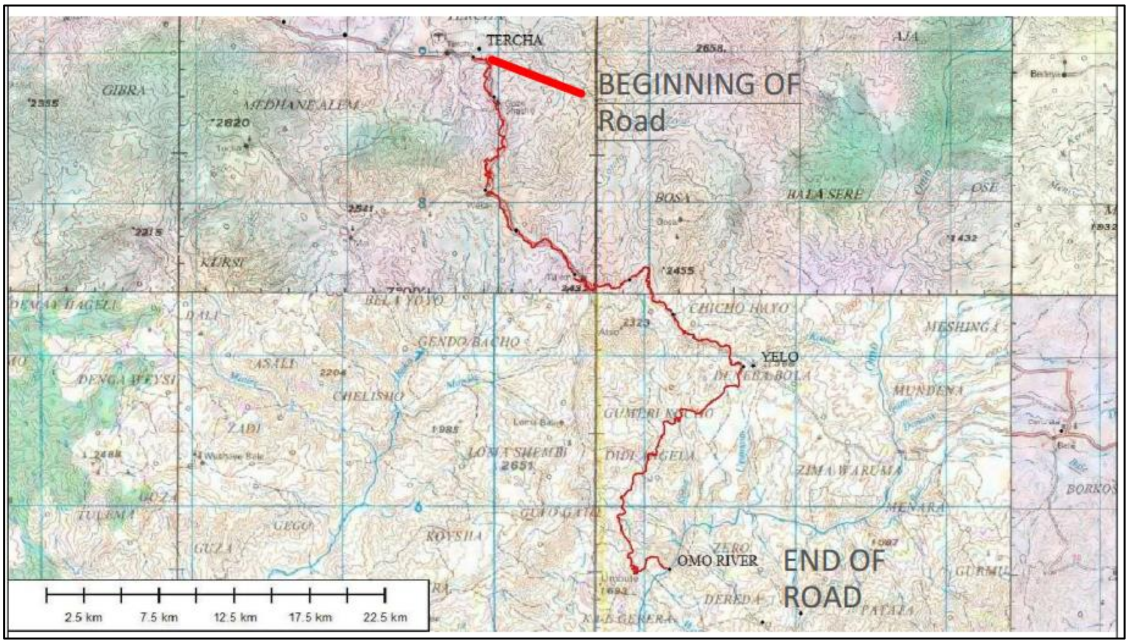

Figure 3-1: Project location map

3.3 Details of Observed Geotechnical Problems

In this road project there are significant geotechnical problems that contribute to the failure of the road section. Some observed significant geotechnical problems that are the major concerns and further analysis of this research of failed sections of prematurely failed road include the followings:

Failed road cut slope

- This is a situation where a section of a slope that has been cut away to create a road or highway and has progressively collapsing and resulting in instability and potential danger to roadway and surrounding area. If this section of the roadway is not maintained or if there will not be any diagnostic measures, this geotechnical failure can cause road closure, traffic disruption and pose a significant risk to public safety.
- The failure has resulted in slope movements such as sliding and toppling, leads to slope deformation and loss of material strength as shown in Figure 3-2.
- The lack of proper slope stabilization measures such as retaining walls, anchors, drainage systems or vegetation cover are progressively exacerbating the failures which is impacting the roadway.

- Geotechnical investigations, monitoring and remedial measures are essential to prevent, mitigate or manage the risks associated with failed road cut slopes.

Figure 3-2: A) Slumping of soil mass B) Accumulation of displaced soil mass from the surface of cut slope @ km 100+300 RHS

Failed road side fill/embankment

- This is the situation where a man-made (artificial) earthen structure supporting a roadway has experienced instability and deformation as shown by Figure 3-3.
- This failure of road side fill or embankment is progressively leading to a significant road damage.
- The failure also has resulted the collapse or sliding of the embankment.

- This failed section has created a traffic disruption by a lane closure which adjacent to the fill and leading to traffic congestion.
- It is also creating hazards due to a potential for land slide.

Figure 3-3: A) Visible significant cracks on the surface of fill slope B) Bulging & deformation on the surface of fill slope @ km 126+000 RHS

Uncontrolled seepage from sloping ground to the pavement structure

- This is the condition where the groundwater infiltrates and saturated the soil beneath the pavement which is progressively leading to instability and damage the road structure.
- This seepage is occurred when the slope above the road is saturated and progressively causing the water to flow downward and infiltrate to the soil beneath the road surface as the presence of moisture is shown in Figure 3-5.

- This uncontrolled seepage has caused several problems for the pavement structure such as softening of the subgrade (the soil has become saturated with water and lost its strength and stiffness, leading to deformation of the road surface), rutting and cracking of the road surface, stripping asphalt aggregates and pumping out of water to the surface of the road as shown in Figure 3-4.
- When a vehicle drives over a section of the pavement, the weight of the vehicle causes the pavement to deflect, creating a low-pressure zone underneath the pavement.
- This pumping out of water is progressively leading to the following problems like reducing skid resistance (water on the pavement surface is reducing the friction between the tires of a vehicle and the pavement surface, making it more difficult to stop or steer the vehicle), rutting (the movement of water through the pavement can cause it to deform, leading to ruts in the surface of the pavement) and surface damage (the repeated pumping of water through the pavement can cause damage to the surface, including cracking and potholes).
- Generally, the following major problems were observed during site investigation: softening of the pavement layers and subgrade by becoming saturated and remaining so for prolonged periods, degradation of the quality of pavement and subgrade material due to interaction with moisture and loss of bonding between pavement layers due to saturation with moisture.

Figure 3-4: Pumping of water through new placed asphalt concrete road @ km 105+000

Figure 3-5: A) Wet patches at the surface of back slope B) Visible outflows from high ground to the pavement @ Km 105+000 RHS

Problematic/unsuitable subgrade soil

- The subgrade/native soil provides the foundation on which a road is built, and its properties play a crucial role for the strength and durability of the road.
- Due to the subgrade soil is unsuitable or problematic this is progressively leading to a range of issues that are negatively affecting the road's performance and longevity.
- These soils have higher plasticity character than the specified limit to be serve as a subgrade soil and lower load bearing strength (California bearing ratio) than the minimum value.
- Progressive geotechnical failures are observed at some stretches along the roadway due to problematic subgrade soils such as reduced load bearing capacity (due to low strength is leading to a weaker foundation for the pavement structure),

progressively increasing settlement which is resulting differential settlement where one part of the road settles more than the other this in turn leads to uneven pavement, roughness and cracking as shown in Figure 3-6.

Figure 3-6: A) Cracks and uneven surface on newly placed asphalt B) Problematic/Unsuitable subgrade soil @ km 114+100 LHS

Road cutting slope erosion

- Progressive erosion and loss of soil from the slope due to rainfall has been occurred at some sections along the project road as shown in Figure 3-7.
- This erosion of soil from the road side cut slope has resulting the following significant problems like reducing stability (this erosion is weakening the slope and increase the risk of slope failure), it is creating sedimentation (the eroded soil is accumulating in side ditches) this in turn results overflow of surface water on the

pavement, damaging the pavement structure (this side slope erosion is damaging the pavement structure particularly at the edge of the pavement adjacent to the slope) and increasing maintenance cost specially for repairing drainage systems as shown in Figure 3-7.

Figure 3-7: A) Accumulation of eroded soil in to the side ditch of the road B) Formation of gullies and channels on the surface of cut slope @ km 134+760 RHS

3.4 Climate, Seismicity and Geology of the Project Area

Geotechnical investigation of a prematurely failed asphalt concrete road requires a thorough understanding of the climate, topography and geology of the project area. These factors can have a significant impact on the behavior of the underlying soil and rock, which in turn can affect the performance of the asphalt concrete pavement.

Climate pertains to the extended weather conditions observed in a specific area, encompassing elements like temperature, precipitation, and wind. The geotechnical investigation may be influenced by the climate of the project area in various ways. For example, high levels of precipitation can increase the moisture content of the soil, which can affect its strength and stiffness properties. Similarly, extreme temperature fluctuations can cause soil expansion and contraction, which can lead to differential settlement and cracking in the pavement. Wind can also play a role in soil erosion, which can affect the stability of the pavement subgrade.

Topography refers to the physical features of the land surface, including its elevation, slope, and drainage patterns. The topography of the project area can affect the geotechnical investigation by influencing the depth and thickness of the soil layers, as well as the type of soil and rock formations present. For example, a hilly or mountainous terrain may have steep slopes that require additional stabilization measures, while a low-lying area may have poor drainage that can lead to soil saturation and instability.

Geology refers to the composition, structure and properties of the rocks and soils in a given area. The geology of the project area can have a significant impact on the behavior of the pavement subgrade and underlying soil layers. For example, soft or weak soils may require additional stabilization measures, while rock formations may require specialized drilling and excavation techniques. Similarly, the presence of certain minerals or chemicals in the soil or rock can affect the durability and performance of the asphalt concrete pavement.

3.4.1 Climate

Temperature

As it can be understood from (ERA, Drainage Design Manual , 2013), the daily mean average

temperature of the Project area is about between 17.5°C and 22.5°C. The average maximum daily temperature of the project area is in between 22.5°C & 27.5°C while the average minimum daily temperature is in between 12.5°C & 17.5°C.

Rainfall

According to the map shown on (ERA, Drainage Design Manual , 2013), the Project area is located in B1 & B2 rainfall regions which receive high annual rainfall. It lies in one of the wettest zones of Ethiopia and the mean annual rainfall of these regions is between 1201 – 1599 mm per year.

3.4.2 Seismicity

According to ES EN 1998-1:2015, Table D2 outlines the Seismic Hazard Zonation for Selected Towns. Given that the project road is situated between Jimma and Sodo towns, referring to Table D1 for Bedrock Acceleration Ratio reveals that Jimma town falls under Zone 1, while Sodo Town is categorized under Zone 4. Additionally, consulting Figure D2, Ethiopia's Seismic Hazard Map in terms of peak ground acceleration (PGA), indicates three PGA values as displayed in Figure 3-8, along the project road: 0.055 – 0.07, 0.07 – 0.085, and 0.085 – 0.1. Considering the seismicity zonation, the project road is identified as more susceptible to seismic effects. Therefore, in the slope stability analysis, the impact of seismicity on both cut and fill slopes will be taken into account.

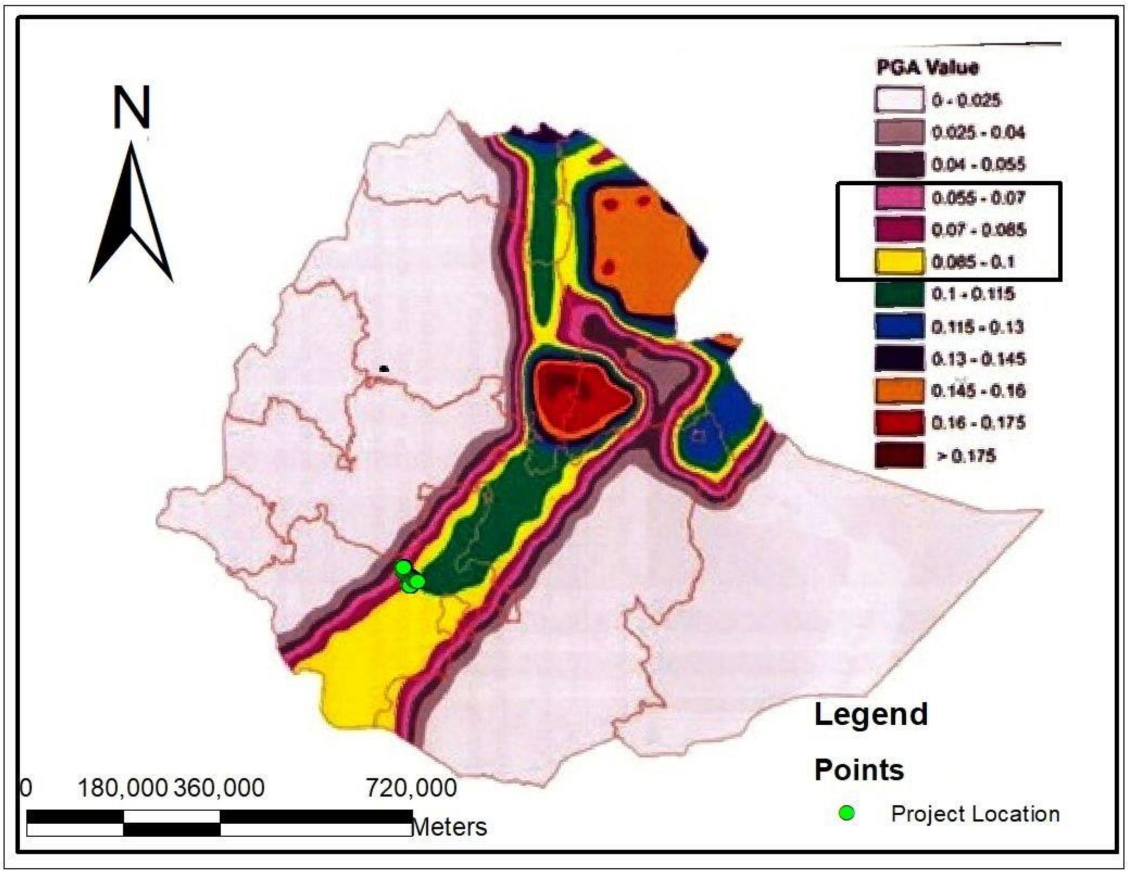

Figure 3-8: Seismicity of project area (ES EN 1998-1:2015)

3.4.3 Geology

As per the geological map of the project (EGS, 2012) shown in Figure 3-9, the project road traverse through the following major geological formations Tv5 middle basalt flows, Tv6 middle trachyte flow and Tv3 lower trachyte flows.

Tv3 lower trachyte flow

This well stratification is seen exposed on the road to Waka, east of Tarcha. Here 10 to 15-meter-thick trachyte flows are often separated by paleosoil. The light grey is often weathering color and when fresh it is dark grey with secondary zeolite and primary sanidine. This trachyte has an approximate area of 2197 sq. km. To the east of Omo, this trachyte forms cliff exposing older caldera for about 1 km. thick. It is slightly columnar. East of Tarcha, and west of Gojab town, the unit is mixed with ignimbrite. The igrimbrite is weathered and shows light grey and medium grained texture. It shows flow layering and often tilts to the east (60_0) due probably faulting. There are two layers of ignimbrite separated by paleosoil. Thin section study of the ignimbrite sample near Gojeb town depict rock fragments of pumice 10%, sanidine phenocrysts 3% and quartz 2%. The groundmass mainly contains oxidized glass/ ash 55%, glass shards 10%, and finer rock fragments 15%. The texture of ignimbrite is often chaotic.

Tv5 middle basalt flow

To the east of Gibe, this unit directly overlies the lower pyroclast. In this area fine grained, dark grey basalts are separated by paleosoil. Horizontally layered basalts form 5 (five) lava layers. At places occurs intercalation of basaltic pyroclasts. The pyroclast contains boulders, blocks, cobbles & gravels cemented by fine grained material. Some of the fragments are scoriaceous basalts. They represent near source facies. These are exposed in Warwarsa river. Often they are cut by basalt dikes. The middle basalt has an area of 3109 sq. km. The major exposure of TV_5 is around Waka to Chida and to the Gojeb river and continue to Bonga. They also form basalts around Jima & Kiltu Wolka areas. There are minor exposures of TV5to the extreme southwestern part of the map, southwest of Chiri.

Tv6 middle trachyte flow

The middle trachyte covers many areas of the map sheet. It is exposed west of Agaro northeast of Jima, southeast of Jima, in utmost areas of Gera and Belete Chaka, south of Bonga & Delbi Moye to Debo areas. It also forms the highlands of Waka and Wolde Hane and to the extreme northeastern part, east of Deneba and north of Gilgel Gibe dam site. In most of its exposure the TV_6 is light grey, fine to medium grained trachyte. It often contains clear sanding and at places soda pyroxene occur. The rock is often massive, but at places show fractures. The color often changer to dark grey or greenish to grey. The trachyte is at placer porphyritic with sanidine phenocrysts. The middle trachyte covers wide area of about 5750 sq. km.

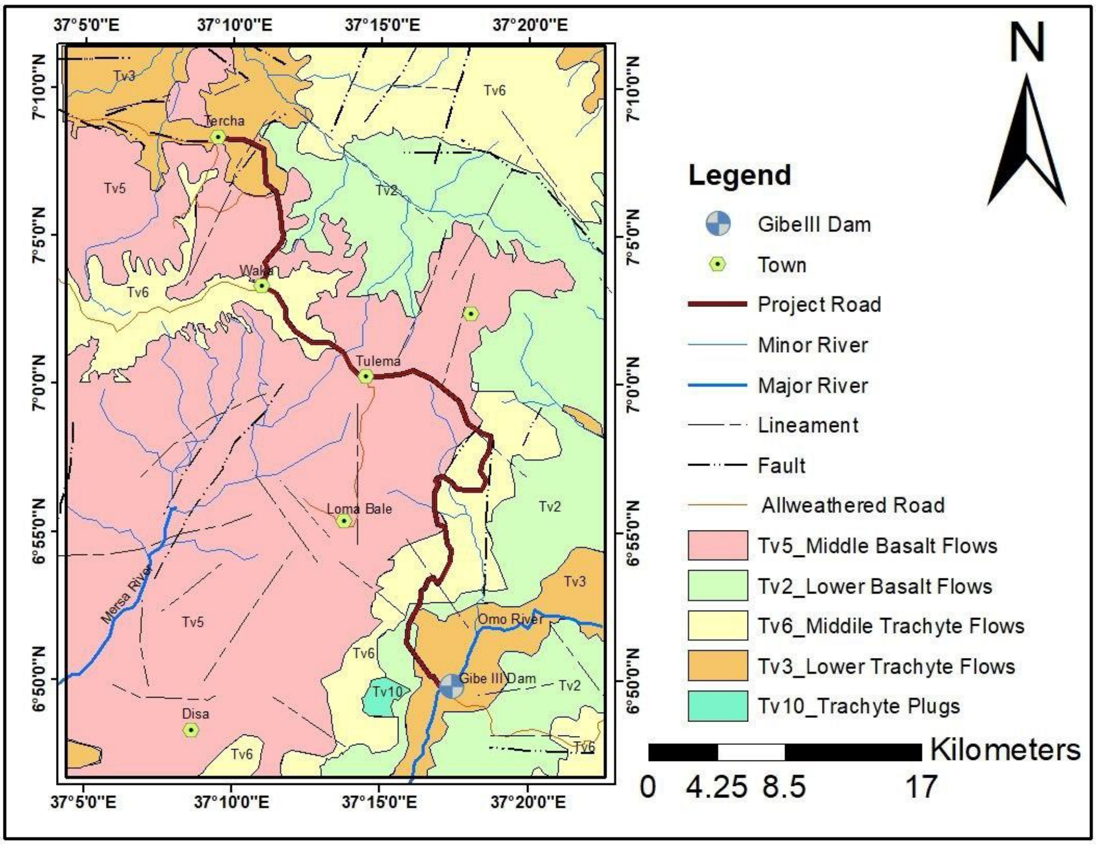

Figure 3-9: Geological map of the project road

3.4.4 Methodology for Failed Road Side Cut and Fill Slopes

Geotechnical investigation of failed road cut and road side fill slopes along this road project has involved the following activities to determine the root causes of these failures and to develop appropriate remedial measures. These methods include visual inspection of the failed road cut and fill slopes to find out different signs of slope (both cut and fill) failures, taking samples for required laboratory testing and analysis of geotechnical data using geo-studio software to determine the slip surface and so as to determine the cause of failures and to recommend diagnostic measures.

So, to achieve the goals of this study the following systematic methodology was followed for road side cut slope and road side fill slope: -

- Review of literatures and project related available documents were used to get firsthand information about the project area and construction history of the project road.
- Establishment of initial site investigation/reconnaissance survey program to locate failed road cut slopes and road side fill slopes along the roadway. The locations of

these failed sections were noted by station/geological locations and coordinates with representative photos.

- Conduct detailed field surveys to identify areas with failed road cut and fill slopes, documenting the magnitude of the failures, deformations and signs of instability.
- Detail field investigation of these failed sections to collect samples for respective laboratory tests to determine engineering properties of the slope materials.
- Laboratory testing, laboratory testing was carried out to determine the mechanical/shear strength properties of the slope material.
- Data analysis/slope stability analysis, the data obtained from the field and laboratory testing is analyzed to assess the stability of the slope and identify the factors that led to its failure.
- Slope stabilization measures, based on the findings of the investigation, appropriate slope stabilization measures are recommended.

3.4.5 Methodology for Uncontrolled Seepage

The investigation of uncontrolled seepage from the road cut slope/high ground to the pavement materials along the project road involves the following methodology:

- Site investigation, the first step is to conduct a detailed site investigation to understand the geological and hydrological conditions of the area. Examination of features influencing the subsurface movement of water in the project area. The site investigation involves any subsurface water sources that are responsible for saturation of pavement layers and pumping out on the surface of the road.
- Laboratory testing, based on the results of the field tests, laboratory testing is carried out to determine the hydraulic conductivity of the soil.
- Data analysis, the data obtained from the field and laboratory testing is analyzed to identify the sources and pathways of the seepage and to assess the pavement failure due to the seepage.
- Remedial measures, based on the findings of the investigation, appropriate remedial measures are recommended.

3.4.6 Methodology for Problematic Subgrade Soil

The research methodology for investigating problematic subgrade soils along the project road at failed section due unsuitability of pavement foundation material involves the following steps:

- Site investigation: the first step is to conduct a detailed site investigation to identify unsuitable/problematic subgrade visually and its consequence on the performance of the road along the project route.
- Sampling and testing: the next step is to collect soil samples from the subgrade soils along the failed road. The soil samples are then tested to determine their physical and engineering properties. The laboratory testing is conducted following the relevant ASTM or AASHTO standards.
- Data analysis: the data obtained from the laboratory testing is analyzed to identify the problematic soil properties that contributed to the failure of the road. The analysis includes comparing the test results with relevant specifications and standards and identifying any deviation from these standards. Remedial measures: based on the results of the investigation, appropriate remedial measures will recommend.

3.4.7 Methodology for Road Cutting Side Slope Erosion

The research methodology for investigating soil erosion from the slope of a roadway cut will involve the following producers:

- Visual inspection: during the site investigation at the selected station along the roadway it will be conducted a comprehensive visual inspection of the slope to identify any visible signs of erosion, such as gullies, rills, exposed roots, cracks, or sediment accumulation.
- Rainfall and climate data collection: collecting rainfall data and analyze climate patterns to understand the relationship between precipitation, runoff, and soil erosion. This data can help identify critical rainfall thresholds or storm events triggering erosion.
- Based on the outcomes of the site investigations, suitable remedial measures will be proposed to mitigate and prevent further erosion of the soil on the roadway side slope. These measures aim to address the underlying causes of erosion and prevent any adverse effects on the pavement performance.

CHAPTER 4 ANALYSIS AND FINDINGS

In this chapter, an investigation of selected locations along the roadway will be presented. The study is anticipated to identify significant geotechnical failures that have occurred, including road cutting slope failure at km 100+300 RHS, road side fill slope failure at km 126+000 RHS, road cutting side slope erosion at km 134+760 RHS, uncontrolled seepage into pavement materials at km 105+000 RHS and problematic/unsuitable subgrade soil at km 114+000. The discussions primarily revolve around recognizing the presence of each problem by observing signs of each geotechnical failure along the route. Additionally, an analysis is led to determine the likely causes of each failure. Special attention is given to addressing unforeseen geotechnical issues that may rise during the design and construction of the road project, as well as the impact of each geotechnical failure on the newly constructed asphalt concrete road. Lastly, appropriate diagnostic measures are recommended to ensure the maintenance and preservation of the recently completed road project.

4.1 Uncontrolled Seepage from the Side Slope

At station km 105+000 RHS, notable issues have been observed. Firstly, there is significant pumping, indicated by water emerging through the newly laid asphalt concrete in the middle of the driving lane. Additionally, there are premature pavement failures characterized by extensive cracking. On the adjacent side of this station, a slope with an estimated height of 5m was excavated during construction. Signs of slope soil saturation and a high groundwater table are evident on the surface of this cut slope, such as wet patches, visible outflows and the presence of hydrophilic plant species (plants that have a natural affinity for water and are adapted to thrive in moist or wet environments).

The uncontrolled seepage of water into the pavement materials is an ongoing problem that greatly affects the performance of the recently constructed road project. Unfortunately, neither during the design stage nor during the construction phases was this condition identified, nor were any mechanisms installed to regulate the water seepage into the pavement materials.

4.1.1 General

Both paved and unpaved roads can encounter issues related to an excess of water within the pavement materials of the roadway. This surplus water can come from various sources, including infiltration from the road surface, seepage of groundwater from higher areas, elevated water levels in roadside ditches or the upward movement of groundwater from beneath the road. The excess moisture content in the road foundation results in a deterioration of the road foundation, ultimately leading to the failure of the surface, whether it is paved or unpaved. Sub-surface drainage of highway pavements comprises the measures incorporated in the design in order to control levels of groundwater and drain the road foundation. Subsurface drainage is normally necessary in order to remove any water which may permeate through the pavement layers of roads in both cut and fill situations.

Subsurface drainage in excavated areas should not only ensure proper drainage of pavement layers but also effectively eliminate any groundwater present in the excavation to a sufficient depth.

Accumulation of moisture introduced into the pavement subgrade from any of the sources can adversely affect pavement performance, leading to accelerated pavement deterioration. Pavement problems associated with infiltrated water may fall into three categories:

- Softening of the pavement layers and subgrade by becoming saturated and remaining so for prolonged periods;
- Degradation of the quality of pavement and subgrade material due to interaction with moisture; and
- Loss of attachment between pavement layers due to saturation with moisture.

4.1.2 Groundwater Investigation

To address the groundwater problem, the initial step involved acknowledging the existence of the issue. Detailed and comprehensive field observations were conducted to identify this potential problem. During the field investigation at the selected station along the roadway (Station km 105+000 RHS), the following significant conditions were observed as presented in Figure 4-1 A.

- *At the selected station, typical indicators of wet patches and visible outflows on the side of a road cut were observed.*

Wet patches on road cut faces are areas or sections of the exposed inclined surfaces of a road cut where moisture or water is visibly present. These patches typically indicate the presence of groundwater or water seepage and high ground water table within the soil of the road cut face:

- The infiltration of water into the road cut face, caused by a high-water table or groundwater flow in the vicinity, has resulted in the gradual formation of wet patches.
- Due to the high permeability of the soils in the road cut face, water can easily pass through, resulting in the formation of wet patch.

Figure 4-1: A) Wet patches exposed on the surface of a road cut B) Out flow and seeping of water from high ground in the pavement and side ditch

The presence of visible outflows on the side of a road cut indicates the active flow or seepage of water or moisture from the exposed inclined surfaces of the cut as shown in Figure 4-1 B. These outflows manifest as flowing rivulets, signifying a continuous discharge of water from within the road cut.

- This visible outflow typically originates from subsurface water within the road cut. The presence of outflows suggests a higher water pressure or hydraulic gradient within the road cut, leading to water actively escaping through openings.
- The flow range from a gentle trickle or seepage to a more forceful or concentrated stream as shown in Figure 4-2 A & B.
- Visible outflows highlight the importance of effective subsurface drainage measures in road cut design and construction.

Figure 4-2: A) Visible outflows on the side of road cut B) Presence of hydrophilic vegetation on the surface of high ground

It is crucial to assess and manage visible outflows on road cuts to ensure the safety, longevity and functionality of the road infrastructure.

- *The road cut surface displays variations in color and the presence of vigorous growth or hydrophilic vegetation.*

The presence of robust and thriving hydrophilic vegetation on the road cut face at station km 105+000 RHS as shown Figure 4-3, along the roadway indicates the availability of moisture and favorable conditions for plant growth.

Hydrophilic vegetation are plant species that thrive in wet or water-rich environments. This vegetation is well-adapted to areas with ample moisture, such as the seepage or drainage zones within the road cut at the specified station.

The variations in color and progressive growth of vegetation also provide insights into the hydrological dynamics within the road cut. They serve as indicators of localized water sources, such as seepage from higher ground. These signs are valuable in assessing the potential for water accumulation and the necessity for appropriate subsurface drainage measures to prevent the infiltration of water into the pavement structure at the investigated station along the roadway.

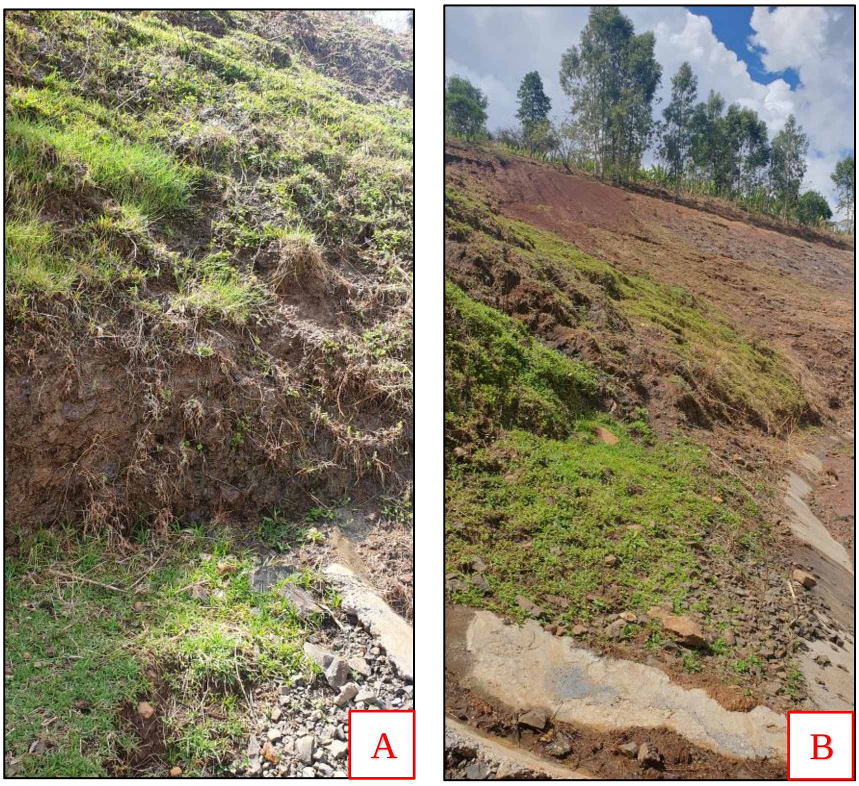

Figure 4-3: A) The road face cut exhibits the growth of hydrophilic vegetation B) Wet patches

- *At the selected station, road surface failures including pumping and tension cracks have been observed. These failures can be attributed to the absence of a subsurface drainage system in place for this particular road section.*

The occurrence of pumping, characterized by the upward movement of water through the pavement layers due to vehicular traffic, is progressively happening as subsurface water infiltrates the pavement layers and saturates the underlying base and subgrade materials. As vehicles traverse the saturated layers, the water is forced upward, leading to the pumping effect on the driving lanes' surface. This phenomenon has resulted in the softening of the road surface, leading to rutting and deterioration. Such observations have been made at the selected station along the carriageway as shown in Figure 4-4 B.

Additionally, tension cracks have been observed at the same station as shown in Figure 4-4 A. These cracks are typically longitudinal and develop on the road surface due to the stress and strain imposed by external forces. When subsurface water seeps into the pavement layers, it causes a loss of support and uneven expansion or contraction of the layers. This differential movement generates tensile stresses, leading to the formation of tension cracks on the road surface. These findings have been noted at station km 105+000 along the roadway.

Figure 4-4: A) Road surface pre-mature failures B) Pumping out of water on the surface of the pavement

4.1.3 Effects of Excessive Water in Pavement Structures

Water that is trapped in a pavement structural section can have many different effects on the materials in the pavement materials. The subgrade, subbase, or base can become saturated, lowering the effective weight and strength of the materials, and free water can fill the interstices between layers of different materials and be displaced under dynamic load impacts. The movement of water under load impacts can erode the materials of the structural section. Also, there is evidence that the pulsating surges of pore pressures building up and diminishing under wheel loads may disintegrate stabilized bases.

Design formulas Asphalt Concrete pavements make the assumption that pavement and base layers are able to spread the loads through successively lower layers in such a manner that the strain or pressure at any level does not exceed the strain or compressive strength allowed for the particular material at that level. If water accumulates in structural sections to the point where free water is present, the normal load spreading assumptions are no longer valid as inter-granular pressures are largely nullified. As a consequence, external traffic loads on structural sections in which excessive water is present introduce deviations. One of the most significant deviations to normal design assumptions occurs when the pavement structural section components contain excess water in joints, cracks and void spaces. If water is permitted to infiltrate into the structural section and no effective drainage is provided, then the base and subbase materials act as reservoirs and sources of water supply to prolong the detrimental actions that occur.

The subsequent paragraphs outline the primary actions that occur in structural sections of pavement layers when they are saturated with excessive water.

- As the pavement remains unloaded, the water infiltrating from the adjacent road cut slope hill saturates the base and subbase. Over time, all the voids within these layers become filled with water as the air within the voids is displaced.
- When heavy wheel loads (dynamic) are imposed on the pavement surface, the loading is transmitted through the successive layers of the structural section until it reaches the foundation subgrade soil (saturated). The foundation soil will tend to compress slightly under load; this compression will reduce the void spaces, slightly increasing the density of the soil. Minute amounts of water are forced out and temporarily become free water. The saturated subgrade material becomes temporarily supersaturated and high pore pressures can be developed.
- The moving pressure wave created by the dynamic action of the wheel loads on the pavement surface can create large hydrostatic forces within the structural section. Application of a force causes either a movement or a reaction. At least two different kinds of actions may occur when a pavement is subjected to dynamic pressure waves of heavy wheel loads. The most probable is forceful displacement of water within the structural section. The displaced water will often transport fines within the structural section and subgrade, affecting the gradation characteristics of various layers supporting the wheel loads. often the void spaces in granular bases or subbases become filled with mud; consequently, the basic assumptions used in the design of the pavement system are modified because of the loss in supporting power.

- Another type of action occurs at well-defined interfaces between subgrades and bases or subbases when the adhesion at the interfaces is negligible. It occurs only when free water· is present to act like a wedge to create a "slick" layer, similar to that found in rock faults and which is highly susceptible to disintegration. Even though the thickness of the water layer may be only a small fraction of an inch, its rapid movement under pressure often amplifies the deflection of the pavement surface under dynamic loading.
- Since water is practically incompressible, large hydrostatic pressures can develop where there is no escape available. Hydrostatic pressures are transmitted equally in all directions; the only reaction to the vertical pressure thrust is the weight of the pavement section above a given level. Any increase in deflection that occurs when the pavement loses support because of erosion of bases or rapid water displacement certainly is a potential factor in decreasing the useful life of the pavement system through larger magnitudes of stresses and accelerated fatigue accompanying them.

The repetitive occurrence of this phenomenon can result in the stripping of asphalt from aggregates that have been stabilized with asphalt cement. It causes the loss of cohesion and contributes to the development of shrinkage cracking in Asphalt Concrete pavements. These effects have been observed progressively in the selected section of the road project as clearly shown in Figure 4-5 A, B &C.

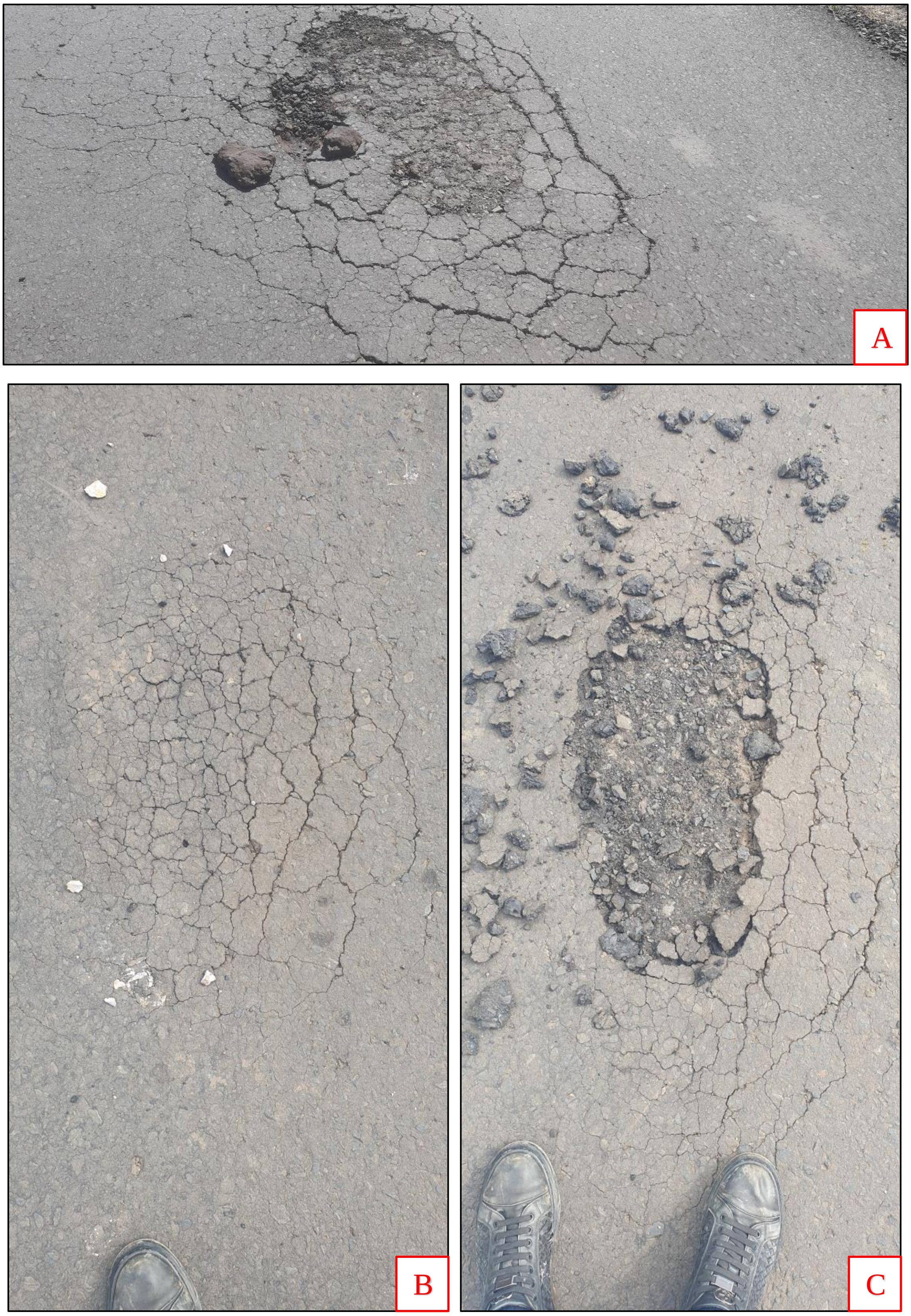

Figure 4-5: A&B) Cracking on newly placed asphalt C) Stripping of asphalt from aggregates

4.1.4 Analysis of Groundwater Problems

Estimation of discharge (q)

An estimation of amount of discharge from the slope/high ground soil in to the newly constructed pavement road has done based on available data of slope soil's particle size distribution and the cross-section of the road at this specified station.

Samples were taken from the neighboring hillside slope where underground water consistently emerges and two specific laboratory tests were carried out on these samples: the particle size distribution test and the natural moisture content test. The former aimed to estimate the permeability of the slope soil, while the latter assessed the soil's moisture content. Utilizing the AASHTO T-88 standard, a particle size distribution laboratory test employing wet sieving was performed on samples gathered from station km 105+000 of the adjacent hillside slope and its result has shown in Figure 4-6. The intention behind this test was to estimate the coefficient of permeability.

The result of wet sieve analysis is as follows:

- Coarse gravel (75 mm-19 mm): 0%
- Fine gravel (19 mm-4.75 mm): 0%
- Coarse sand (4.75 mm-2.0 mm): 16.23%
- Medium sand (2.0 mm-0.425 mm): 24.41%
- Fine sand (0.425 mm-0.075 mm): 56.45%
- Silt and clay (< 0.075 mm): 2.50%

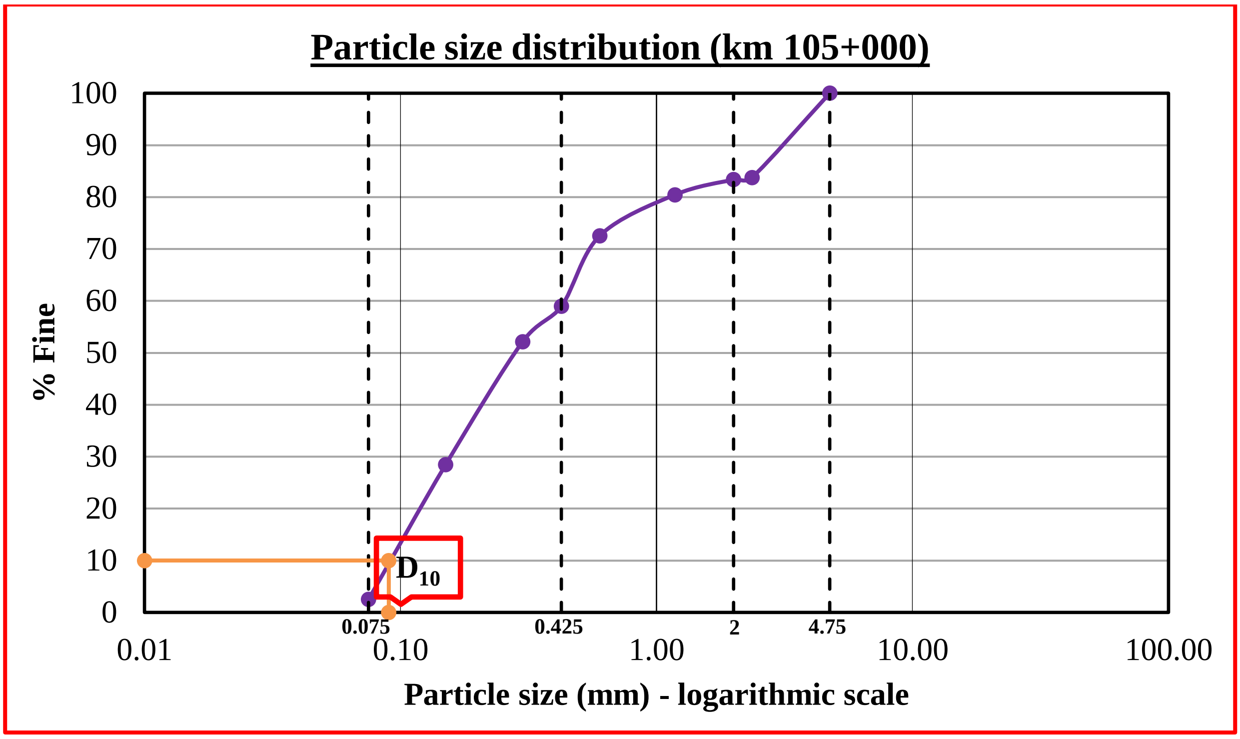

Figure 4-6: Particle size distribution (km 105+000)

Considering these findings, the soil appears to have a mixture primarily composed of fine to medium sand particles, with a smaller presence of coarse sand and minimal silt/clay content. This composition suggests a moderately permeable soil, with good potential for water movement due to the dominance of sand particles, particularly in the fine sand range. The value of the coefficient of permeability of a soil can be estimated by using indirect method, one of the method is by using Allen Hazen's formula,

$$k = \mathrm{CD}_{10}^2$$

Where k = coefficient of permeability (m/s)

D_{10} = the diameter corresponding to percent finer of 10%.

C = constant, with a value between 0.01 and 0.00667 where k is expressed in m/s and D_{10} in mm.

As read from the particle size distribution graph the value of D_{10} which corresponds to the percent finer of 10% is approximately 0.09mm. Having this value based on the above equation;

$k = \mathrm{CD}_{10}^2$,

k = 0.01*0.09*0.09

k = 0.000081 m/s

k = 8.1*10^{-5}m/s

Once the infiltration rate has been estimated, the inflow or quantity of water entering the road surface is determined by applying Darcy's Law as follows:

$Q = kiA$

Where: Q = quantity of water entering the surface (m^3/s)

k = permeability or infiltration rate (m/s)

i = hydraulic gradient, i.e. head of water divided by length of drainage path

The hydraulic gradient from the above cross-section of the road at km 105+000, was taken as an average of the two consecutive back slopes.

$i_1 = 8.3/6.9 = 1.2$ and $i_2 = 6.05/17.9 = 0.34$; the average the two hydraulic gradients equals to 0.77.

The area of this hillside slope per meter is equals to (10.8*1) + (18.6*1) = 29.4 m^2

Finally, $Q = kiA$

$Q = 8.1*10^{-5}$ m/s * 0.77 * 29.4 m^2 = **0.001833 m^3/s = 1.833 l/s**

As a result of this, a consistent inflow of 0.001833 m^3/s is entering the newly constructed pavement from the adjacent hillside slope. This continuous inflow saturates the layers of the pavement and has caused premature pavement failures.

Following a comprehensive on-site and laboratory investigations at the chosen station and considering the topographic features of the road cross-section at station km 105+000, implementing an adequate subsurface drainage system will efficiently manage the seepage flow from the nearby road cut slope to the pavement materials. This will be achieved by intercepting the seepage flow prior to it reaching the pavement materials. To accomplish this, it is recommended to install longitudinal interceptor drains as the suitable subsurface drainage system in this scenario.

Interception drains, also known as intercept drains or interceptor drains, are a type of subsurface drainage system designed to intercept and redirect water away from areas where its presence could cause potential damage or compromise the stability of pavement structures. These drains are typically installed below the ground surface and are strategically positioned to collect and convey subsurface water to designated outlets or disposal points.

Interception drains are a vital component of subsurface drainage systems, particularly in areas where water seepage or accumulation can pose risks or challenges. They play a crucial role in maintaining the integrity and performance of roadways by managing and redirecting water away from vulnerable areas.

Placing an interceptor drain up gradient from the ditch as indicated in Figure 4-7, or beneath the ditch itself, can help to control the hill slope seepage and decrease or even eliminate the flow beneath the roadway, thus removing the source of water from entering into the pavement foundation.

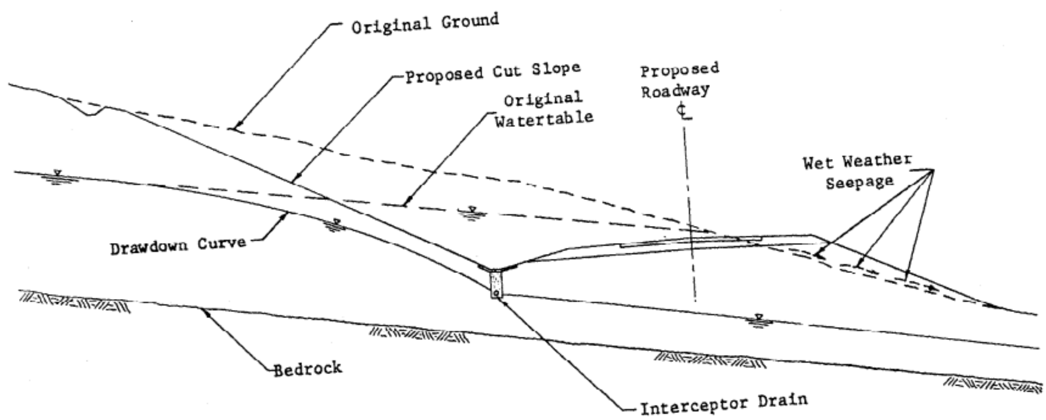

Figure 4-7: Longitudinal interceptor drain used to halt seepage and lower the groundwater table. (Moulton L. K., 1998).

Components of longitudinal interceptor drain

There are three major components of longitudinal interceptor drain. These are underdrain pipes (perforated), aggregate filters and geotextiles. Here's the breakdown of the purposes of each component in a longitudinal interceptor drain:

Underdrain pipes (perforated): -

These pipes are perforated to facilitate the collection and transport of water. They serve as conduits for draining water away from the constructed pavement layers. This is essential as excessive moisture can compromise the pavement's structural integrity by weakening the pavement layers and contributing to premature deterioration.

Aggregate filters

Positioned around the perforated pipes, aggregate filters prevent soil particles from clogging the pipes. They allow water to pass through while blocking fine soil particles, maintaining the drainage system's effectiveness over time.

Within the context of flexible asphalt concrete pavements, aggregate filters around the perforated pipes play a critical role in preserving the drain's functionality. By preventing the intrusion of fine particles from surrounding soils into the drain system, they ensure that the pipes remain clear and effective in draining water. This safeguards against potential clogging that could hinder the drain's performance.

Geotextiles

Geotextiles in this context serve as a protective barrier, shielding the drain system from soil infiltration while enabling water to permeate through. Specifically, for flexible asphalt concrete pavements, geotextiles help maintain the stability and functionality of the drain by preventing soil fines from entering the drain system and causing blockages. Additionally, they contribute to the overall structural stability of the pavement system by providing support and separation between soil and aggregate layers.

Filter material design

When the backfill is specified so that it serves as a filter, the aggregate prevents the movement of particles of the soil being drained, is permeable enough to allow free water to enter the pipe, and it is coarse enough not to migrate into open joint and perforations of the pipe. Backfill should be designed to maintain progressive, greater outflow capabilities in the direction of flow It must also carry any vehicle loading without allowing the pipe to be damaged.

Aggregates utilized in developing the filters and permeable media should consist of sound, clean granular materials with zero plasticity. The gradation should be selected to protect against the loss of soil fines by hydraulic piping while still providing sufficient drainage capacity.

As per (AASHTO, DESIGN OF PAVEMENT STRUCTURES, VOLUME 2, 1986) the gradation specification for the filler material should be as per Table 4-1:

Table 4-1: Gradation specification for the filter material

Sieve size (mm)	U.S. sieve no.	Percentage of passing by weight
63.5	$2\frac{1}{2}$ in.	100
50.8	2 in.	900 - 100
38.1	$1\frac{1}{2}$ in.	35 - 70
25.4	1 in.	0 - 15
12.7	$\frac{1}{2}$ in.	0 - 5

Establishing the free-water surface in the subgrade

Recommendations on the minimum depth to the free-water surface from the pavement surface vary. As per (BROWN, 1974), a minimum specified height that the final grade shall be above the water table for different states: Massachusetts, 2.1 m; Michigan and

Minnesota, 1.5m; Nebraska's criterion is that the subgrade shall be 0.9 to 1.2 m above the water table in granular materials and 2.1 m above the water table in cohesive soils. Also investigators in Germany concluded that a critical depth for moisture is 2 m below the pavement surface as per (Halsey, 1978).

The geotextile shall be made from a strong, tough, porous nylon, polypropylene or other rot-proof polymeric fibers formed into a fabric of the woven or non-woven type. Storage and handling of geotextiles shall be in accordance with the manufacture's recommendations.

Details of proposed longitudinal subsurface drainage

- Depth of the trench drain: 2 m
- Width of the trench drain: 0.45 m
- Diameter of the perforated pipe: 0.15 – 0.3 m
- Sides and bottom of the trench will be lined with geotextile

To sum up, the issue involves uncontrolled water seepage from a roadside slope affecting the pavement at station km 105+000. Visible signs like wet patches, outflows and hydrophilic vegetation indicate excessive moisture. This water infiltration weakens the pavement, causing failures like pumping and tension cracks. Such issues arise when excess water affects the road foundation, leading to degradation and loss of pavement strength. Groundwater investigation suggests an inflow of 1.833 l/s into the pavement. To manage this, installing longitudinal interceptor drains is recommended. These drains consist of perforated pipes surrounded by aggregates and geotextiles, effectively redirecting water away from vulnerable pavement areas.

4.2 Soil Erosions from Cut Slope

During the site investigation conducted at station km 134+760 RHS, several common signs were observed, indicating progressive soil erosions from the slopes of the roadway cut. These signs serve as indicators of the ongoing erosion processes taking place at this specific location. By identifying and documenting these signs, it becomes evident that the soil erosion issue is not only present but also occurring in a continuous manner. These observations play a crucial role in understanding the severity and extent of the erosional processes happening along the slopes of the roadway cut at station km 134+760 RHS.

4.2.1 General

During the comprehensive site investigation, a detailed assessment of the road vicinity was conducted, revealing the presence of progressive soil erosion from the slopes of the roadway cut at the station of km 134+760 on the right-hand side (RHS). Numerous notable indicators were observed, providing evidence that soil erosion is occurring from the slope surface, infiltrating the side ditch and impacting the performance of the pavement layers. To prevent erosion-induced defects from affecting the pavement and to prolong the road's service life, it is crucial to conduct a thorough diagnosis of the uncontrolled soil erosion occurring from the surface of the roadway cut slopes.

4.2.2 Investigation of Soil Erosion

The following signs were observed during the site investigation, providing evidence of ongoing soil erosion from the surface of a roadway cut slope into the pavement system materials:

- *Formations of rills and gullies on the surface of a slope*

At the designated station along the roadway, the presence of rills and gullies on the slope surface has been observed. These formations range from small channels (rills) to larger, deeper channels (gullies) as shown in Figure 4-8 A & B. The existence of these channels indicates the concentrated flow of water, which is progressively causing erosion and soil removal Figure 4-8 C.

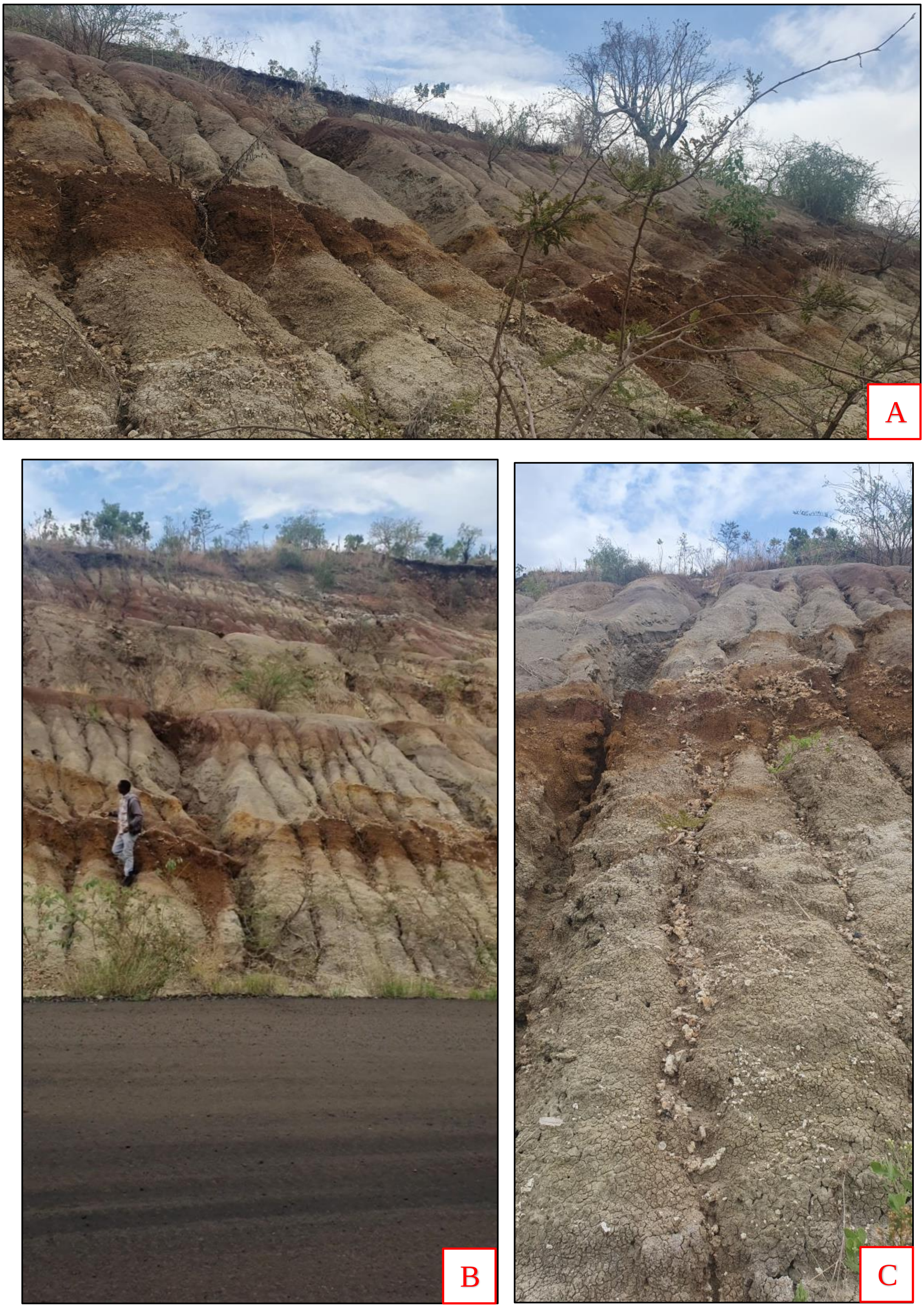

Figure 4-8: A & B) Formations of rills and gullies on the surface of slope C) Crack and fissures on the surface of slope soil

- *Exposed plant roots and vegetation loss on the surface a slope*

At this specific station, a significant number of exposed plant roots have been observed. These roots were initially covered by soil but have become exposed as a result of soil erosion along the surface of the roadway side cut slope. This condition progressively compromises the stability of the slope and exacerbates the erosion process. And also it has been observed that there is progressive loss of vegetation cover on the slope, including the disappearance of grass, shrubs, or trees, due to erosion washing away the topsoil and affecting the growth and stability of vegetation as shown in Figure 4-9 A & B.

Figure 4-9: A) Exposed plant roots and vegetation loss on the surface of a slope B) Disappearance of grass, shrubs and trees due to erosion washing away the topsoil

- *Accumulation of sediments and tiling of constructed structures*

Another prominent indication of soil erosion from the cut slope of the roadway at the selected station is the accumulation of sediment. This accumulation is the progressive buildup of sediment at the base of the slope, including the presence of debris or sediment deposits near the roadway and within drainage channels. These accumulations serve as clear evidence that the soil is being transported downslope due to erosion processes.

At the site it has been observed as shown in Figure 4-10 A & B, that the ongoing soil erosion along the side slope is adversely affecting the drainage patterns surrounding the gabion wall constructed at the base of the slope. The uncontrolled buildup of sediment and debris behind the wall is obstructing the natural flow of water, resulting in pooling and potentially compromising the foundation of the wall. It has been also observed that leaning and tilting of the constructed gabion wall due to erosion-induced instability.

Figure 4-10: A) Accumulation of sediments B) Tilting of adjacent structure (gabion wall)

- *Formation of cracks, fissures and soils on the slope becoming lose, crumble and saturated.*

At the specified station, additional indicators of soil erosion from the sloping surface include the formation of cracks or fissures on the slope as shown in Figure 4-11. These cracks or fissures have occurred due to the drying and shrinking of eroded soil. Additionally, the soil on the slope may become loose, crumbly, or saturated, which are clear signs of erosion and a reduction in soil stability.

Figure 4-11: Formation of cracks, fissures, looseness and crumbliness of soils on the slope surface

4.2.3 Effects of Uncontrolled Erosion

The uncontrolled erosion occurring on the side slope of the roadway at station km 134+760 on the right-hand side (RHS) cut slope has led to several impacts on the recently constructed flexible asphalt concrete pavement. Some of these effects comprise:

- Loss of support: The ongoing soil erosion is causing a gradual loss of support for the pavement structure. As the slope erodes, the stability of the underlying soil progressively diminishes, resulting in a weakened foundation for the pavement. Consequently, this leads to settlement and unevenness on the pavement surface over time.
- Surface drainage problems: The persistent erosion from the side slope is interfering with the inherent drainage system of the pavement. This uncontrolled erosion is causing depressions and unevenness on the recently built pavement surface, impeding the smooth flow of water. As a result, water accumulates on the

pavement, posing a heightened risk of hydroplaning and diminishing the skid resistance.

- Escalated maintenance demands: The unregulated erosion occurring at the location is substantially increasing the maintenance needs for the pavement. The erosion process necessitates more frequent interventions, such as patching and resurfacing, to rectify the resulting damage and uphold a secure and operational pavement surface. Consequently, this is resulting in augmented maintenance expenses and heightened disruption to traffic flow.
- Safety risks: The erosion-induced deterioration of the pavement is generating hazards for individuals using the road, as observed during the site investigation. Cracks, potholes, and uneven surfaces pose a threat to vehicle stability, diminish tire traction and escalate the likelihood of accidents.

4.2.4 Laboratory Tests Conducted (km 134+760 RHS)

Several laboratory tests were conducted on samples collected from a precarious road-side cut slope situated at the specified location along the highway as listed in Table 4-2. These tests were undertaken with the objective of ascertaining the root causes attributable to the distinctive soil properties responsible for the persistent erosion phenomenon. The ultimate aim of this rigorous investigation is to facilitate an accurate diagnosis of erosion causality and the development of efficacious solutions for the ongoing erosion, thereby ensuring the preservation of the recently constructed road infrastructure. Summary of laboratory test results are depicted under Table 4-3.

Table 4-2: Laboratory tests conducted and their test method (134+760, RHS)

S. No.	Laboratory test	Test method
1	Atterberg limit tests (LL & PI)	AASHTO T-89 & 90
2	Particle size distribution	AASHTO T-88
3	Compaction characteristic (MDD & OMC)	AASHTO T-180 D
4	Density of soil in place	AASHTO T-191
5	Natural moisture content	AASHTO T-265
6	Dispersive characteristic of soil (double hydrometer (%))	ASTM D4221

Table 4-3: Summary of laboratory test results (134+760)

S. No.	Laboratory test		Test result
1	Atterberg limit	LL	51.6%
		PL	30.3%
		PI	21.3%
2	Soil classification	AASHTO	A-2-7 (4)
		UCS	SP (poorly graded sand with little fines)
3	Compaction characteristic	MDD	1.61g/cm3
		OMC	17.30%
4	Density of soil in place		1.51g/cm3
5	Natural moisture content		41.39%
6	Dispersive characteristic of soil (double hydrometer (%))		53%

4.2.5 Analysis of test results (km 134+760 RHS)

I. Particle size distribution

The influence of soil texture on the susceptibility of slope soil to erosion can be understood as follows: Larger particles resist transportation due to the increased force needed to mobilize them, while finer particles resist detachment owing to their cohesive nature. The least resistance particles are silts and fine sands (Morgan, 2005). Thus soils with a silt content above 40% are highly erodible (Richter, 1977). And (Evans, 2020), prefers to examine erodibility in terms of clay content, indicating that soils with a clay content between 9 and 30% are the most susceptible to erosion.

The use of the clay content as an indicator of erodibility is theoretically more satisfying because the clay particles combine with organic matter to form soil aggregates or clods and it is the stability of these that determines the resistance of the soil (Morgan, 2005).

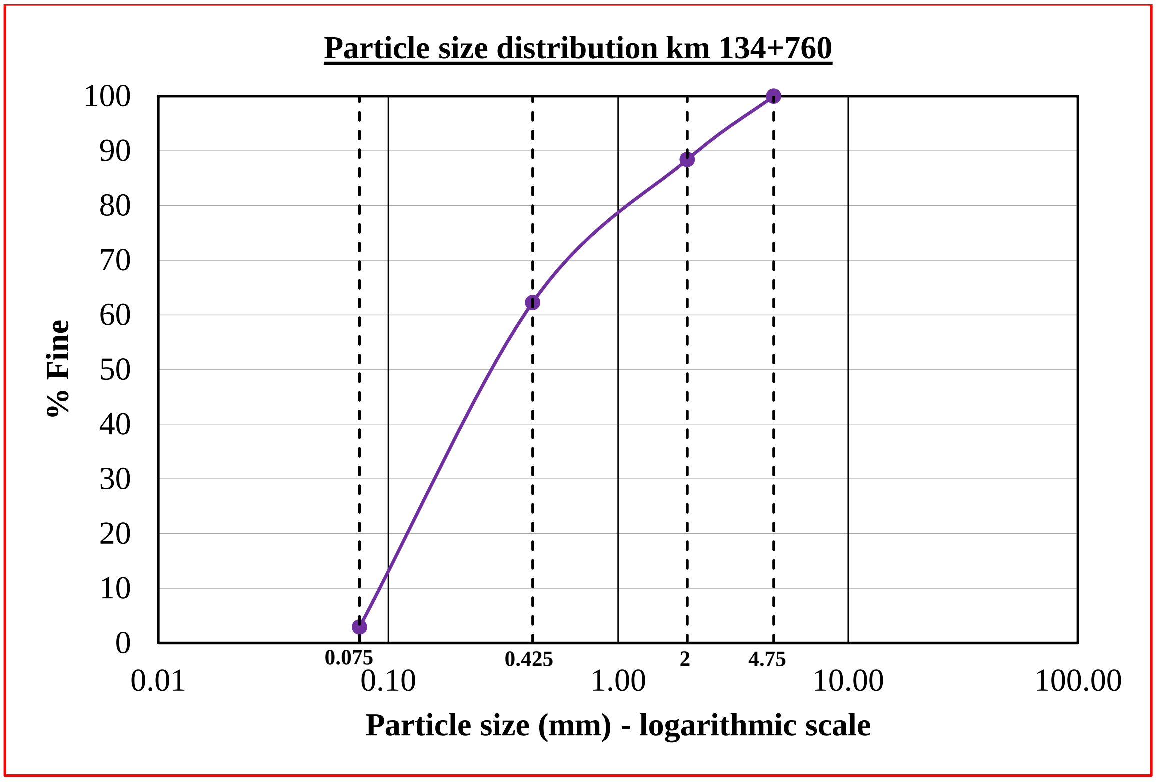

Figure 4-12: Particle size distribution km 134+760

From the particle size distribution analysis obtained through a wet sieving laboratory test on samples collected from the right-hand side of the roadway cut slope at kilometer 134+760, following the ASTM D2478 standard as shown Figure 4-12, the grain size distribution is as follows:

- Coarse gravel (75 mm-19 mm): 0%
- Fine gravel (19 mm-4.75 mm): 0%
- Coarse sand (4.75 mm-2.0 mm): 11.6%
- Medium sand (2.0 mm-0.425 mm): 26.11%
- Fine sand (0.425 mm-0.075 mm): 59.38%
- Silt and clay (< 0.075 mm): 2.90%

The grain size analysis test results provide valuable insights into the erodibility of the soil from a roadway cut slope.

- Proportion of fine particles: A high percentage of fine sand (59.38%) in the soil suggests that a significant portion of the soil consists of particles that are smaller in size. Fine particles are typically more susceptible to erosion compared to larger ones.

- Coarse sand content: While there is coarse sand present (11.6%), it is not the dominant fraction. Coarse sand is relatively more resistant to erosion compared to fine sand.
- Silt and clay fraction: The presence of silt and clay (2.90%) indicates that there is a cohesive component in the soil. Cohesive soils can resist erosion to some extent.
- Absence of coarse gravel: The absence of coarse gravel and fine gravel indicates that larger, more stable particles are not a significant component of the soil.

Based on the laboratory tests that involved wet sieving and the determination of Atterberg limits, which are typically conducted to classify soil, the soil sample taken from the slope can be categorized under different soil classification systems as follows:

- According to the AASHTO classification (134+760), it falls into the A-2-7(0) category.
- Under the Unified Classification System (134+760), it is classified as SP, indicating a poorly graded sand.

Furthermore, a report by the (National Academies of Science, 2019) provides a practical method for estimating the erosion resistance of soil using the Unified Soil Classification System (USCS). This approach involves the creation of distinct zones on erosion charts, each associated with different soil types defined by the USCS categories.

In this report, it is emphasized that the erodibility of coarse-grained soils is primarily influenced by gravitational forces, which are, in turn, related to the grain size. Since the USCS soil classification for coarse-grained soils primarily hinges on grain size distribution, it serves as a valuable tool for distinguishing erosion categories among such soils.

Given that the soil on the slope is of a coarse-grained nature, referencing these erosion charts is particularly useful. As per these erosion charts, the roadway cut slope soil at 134+760, categorized as SP (poorly graded sand) under the USCS, falls into the high to very high erodibility category as shown in Figure 4-13. This conclusion is supported by the information provided in the Figure 4-14 below:

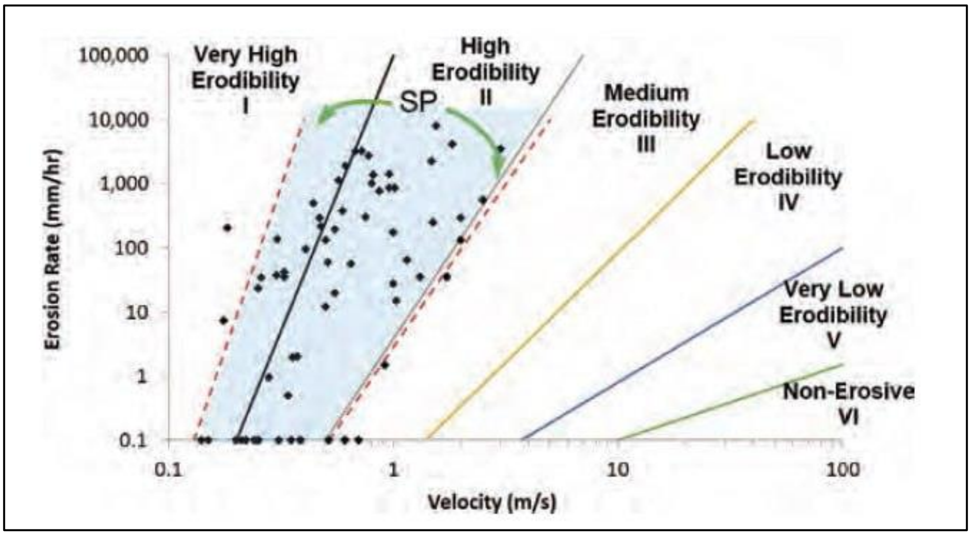

Figure 4-13: Velocity-erosion rate plot for poorly graded sand (SP) soils, (National Academies of Science, 2019)

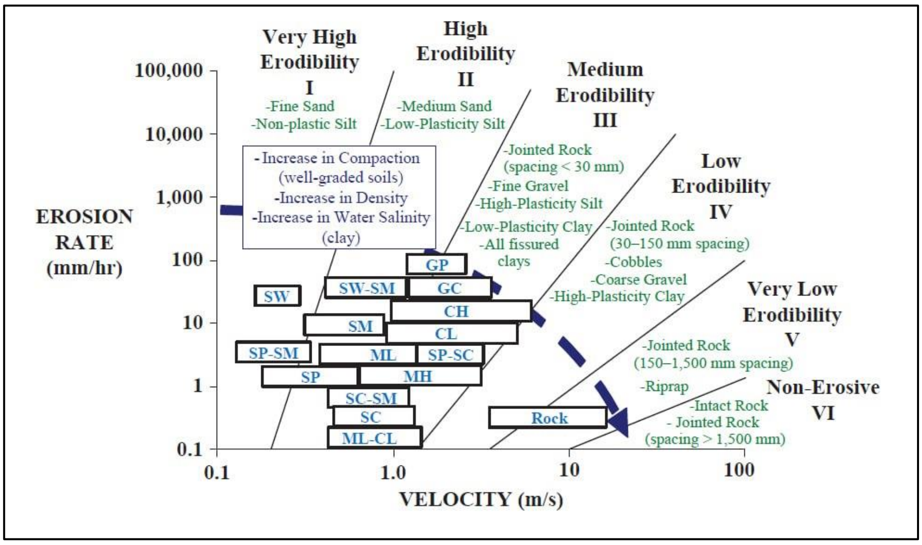

Figure 4-14: Erosion category chart with USCS symbols, (National Academies of Science, 2019)

It is evident from these charts that poorly graded sand (SP) is inherently susceptible to erosion in the high to very high erodibility category as shown in Figure 4-14. When considering the extended slope length and the removal of vegetation cover during construction without adequate protective measures, this soil erosion from the roadway cut slope into the pavement has occurred uncontrollably. These factors have had significant adverse effects on the recently constructed road project.

Atterberg limit test results, Atterberg limit test were also conducted in order to understand the slope soil erodibility, the test results are:

- Liquid limit (LL) = 51.6%
- Plastic limit (PL) = 30.3%
- Plasticity index (PI) = 21.3%

These values suggest the following, as per the conclusions of (National Academies of Science, 2019):

An increase in the plasticity index (PI) in general leads to an increase in the erosion resistance in both coarse-grained and fine-grained soils, the plasticity index value of this soil is 21.3% which lower since the soil type is poorly graded sand so that the soil will not get a resistance for erosion.

So from the Atterberg limit test results we can conclude that the soil by nature have no a character of erosion resistance due to lower values of Atterberg limit tests, which further increases erosion susceptibility of the roadway cut slope soil.

The Atterberg limit tests were carried out to assess the erodibility of the slope soil, yielding the following results:

- Liquid limit (LL) = 51.6%
- Plastic limit (PL) = 30.3%
- Plasticity index (PI) = 21.3%

These findings align with the conclusions drawn in the report by the (National Academies of Science, 2019). According to their analysis: Typically, an increase in the plasticity index (PI) tends to enhance erosion resistance, whether in coarse-grained or fine-grained soils. However, the PI value for this soil is 21.3%, which is relatively low. This is primarily due to the soil being categorized as poorly graded sand, implying that it lacks inherent resistance to erosion.

Therefore, based on the results of the Atterberg limit tests, it can be inferred that the soil, by its natural characteristics, does not possess inherent erosion-resistant qualities due to its low Atterberg limit values. This, in turn, heightens the susceptibility of the roadway cut slope soil to erosion.

The other tests conducted on samples collected at station were targeted to assess the density of the soil, modified proctor compaction and in-place density of soil by sand replacement method to relate them to the erodibility of the soil collected from the roadway cut slope.

II. Compaction characteristic test results

- Maximum Dry Density: 1.61 g/cm³
- Optimum Moisture Content: 17.30%

III. Sand replacement (in-place) density test result

- In-place density of soil: 1.51 g/cm³

The connection between soil density and resistance to erosion can be understood as follows: Increased soil density generally leads to enhanced cohesion and stability, rendering the soil more resilient against erosive forces. In simpler terms, denser soils are less prone to being readily displaced by water.

This relationship is exemplified by the outcomes of laboratory tests, specifically the modified compaction test, which yielded a density of 1.61 g/cm^3. This density is relatively low and the in-place density of 1.51 g/cm^3 is even lower. Consequently, soils with lower in-place density are more susceptible to erosion.

The reason behind this vulnerability lies in the increased presence of air voids in soils with low density, which, in turn, makes it easier for water to displace soil particles.

Regarding to the moisture content test result, which stands at 41.39%, this level of water content is notably higher for the specific soil type categorized as poorly graded sand. According to the findings in the (National Academies of Science, 2019), moisture content has a detrimental impact on the erosion resistance of coarse-grained soil. The higher the moisture content, the lower the resistance to erosion and this factor further contributes to the ongoing soil erosion issue.

IV. Dispersive characteristic of soil (double hydrometer (%))

According to a (Knodel, 1991), the erodibility of soils is often assessed through a widely used test known as the double hydrometer test. This test, conducted following the guidelines outlined in ASTM D4221 “Standard Test Method for Dispersive Characteristics of Clay Soil by Double Hydrometer,” provides valuable insights into soil erodibility based on its results. As per (Knodel, 1991) the criteria for evaluating degree of dispersion using results from the double hydrometer test are listed under Table 4-4.

Table 4-4: Criteria for evaluating degree of dispersion/erodibility as per (Knodel, 1991)

Percent dispersion	Degree of dispersion/erodibility
<30	Nondispersive
30 to 50	Intermediate
>50	Dispersive

The standard test method, ASTM D4221 stated that dispersion-prone clays are those that typically undergo deflocculating when exposed to low-salt concentration water, in contrast to aggregated clays that would maintain flocculation in the same soil-water system. Generally, dispersive clays are highly erosive, possibly subject to high shrink-swell potential, may have lower shear strength, and have lower permeability rates than aggregated clays.

Based on the fundamental assessment criteria mentioned earlier, the outcome of the double hydrometer test, carried out following ASTM D4221 standards, shows a result of 53%. This places it within the dispersive category, signifying that the soil present on the road's cut slope is highly susceptible to erosion.

4.2.6 Topography and slope characteristic

In addition to evaluating the inherent soil properties of the roadway cut slope, an analysis of the slope's topography and characteristics was conducted to investigate other contributing factors to the ongoing soil erosion issue. This assessment aimed to identify additional key elements causing erosion and devise appropriate solutions for remediation.

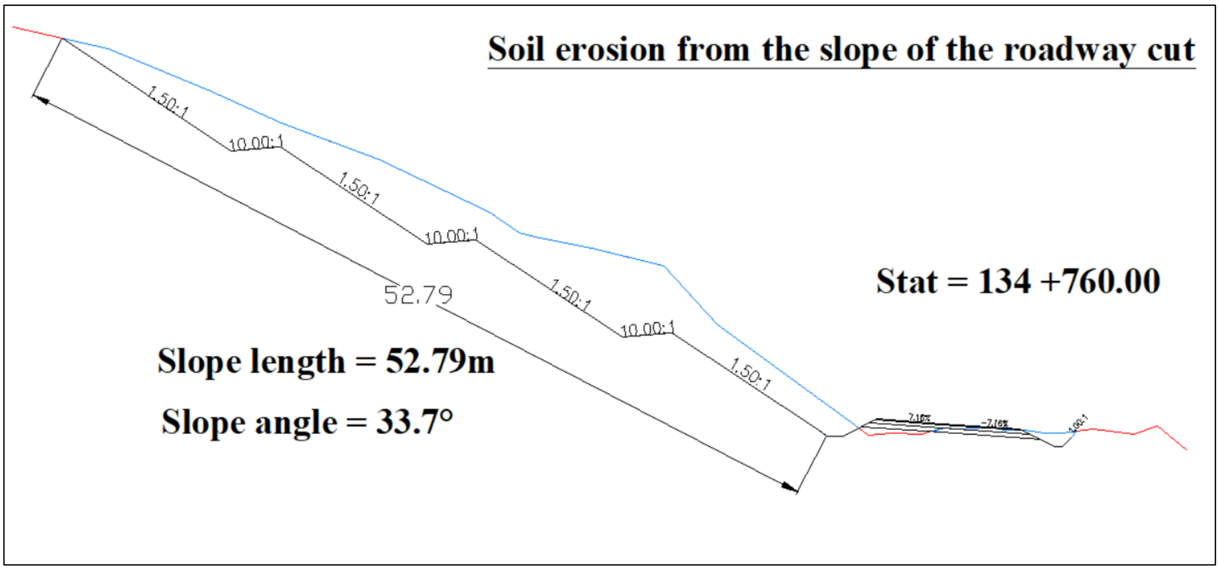

Figure 4-15: Cross-section of the road at km 134+760

Topographical features: as observed in the preceding cross-section as shown in Figure 4-15 of the road at station 134+760 RHS, the finished level exhibits a series of sequential slopes (comprising four in total) with an approximate incline of 1.5H:1V, equivalent to 33.7° from the horizontal plane. Between these consecutive slopes, there are three intervening benches, each measuring 3 meters in length, characterized by an incline of approximately 10H:1V or 5.7°. The collective length of the entire roadside cut spans 52.79 meters, maintaining an approximate 33.7°. This configuration signifies the presence of conditions conducive to soil erosion induced by rainfall. Notably, the extended length of the slope and its elevated slope angle serve as significant factors contributing to the ongoing erosion of the soil on the roadway cut slope.

Vegetation: in this specific location, as evident from the road's cross-section, the original ground level has been altered to achieve the intended finished road level. This alteration has resulted in the removal of nearly all natural vegetation covering the soil, thus exacerbating the continuous erosion of the soil along the roadside cut slope.

Rainfall intensity: the twenty-year rainfall data obtained from a neighboring station reveals that the region experiences relatively intense and heavy rainfall. This heightened rainfall intensity stands as another significant contributing factor to the ongoing soil erosion along the roadside cut slope.

4.2.7 History and Extent of Erodible Area

The historical evolution of the topography and vegetation cover in this susceptible area along the roadway has been examined using Google Earth's historical imagery tool. The analysis revealed that the construction of the road has led to the removal of existing vegetation cover, leaving the erodible soil exposed to rainfall without any remedial measures in place. The accompanying figures Figure 4-16 and Figure 4-17 in this section depict the changes in topography over time at kilometer 134+760.

The erosion spans from station 137+700 to 134+835, covering approximately 135 meters. The length of the sloping ground with erodible soil measures approximately 57.2 meters, as measured from the affected area on Google Earth.

Figure 4-16: Google earth historical imagery dated February, 2021 @ km 134+760 RHS

Figure 4-17: Google earth historical imagery dated October, 2014 @ km134+760 RHS

Key findings from the analysis include:

- Particle size distribution: the soil predominantly consists of fine sand with a low percentage of coarser materials, indicating a higher susceptibility to erosion, especially considering the abundance of fine particles.
- Soil classification: categorized as “A-2-7 (4)” by AASHTO and “SP (poorly graded sand with little fines)” in the Unified Classification System, both classifications suggest high erodibility for this soil type.
- Compaction characteristics: the soil exhibited a relatively low maximum dry density and an optimum moisture content, indicating increased vulnerability to erosion due to higher air voids within.
- In-place density: the in-place density determined by the sand replacement method is relatively low, signifying a predisposition to erosion.
- Moisture content: the soil's moisture content is notably high, a factor detrimental to erosion resistance according to the ((National Academies of Science, 2019)).
- Topographical features: the slope's design, including its length, steepness, and bench structures, contributes significantly to erosion induced by rainfall.
- Vegetation: the removal of natural vegetation during construction has worsened soil erosion along the roadside cut slope.
- Rainfall intensity: the region experiences intense and heavy rainfall, further heightening the risk of soil erosion.

This comprehensive laboratory analysis serves to uncover multiple contributing factors to ongoing soil erosion, providing crucial insights to guide effective remedial measures for preserving the integrity of the newly constructed roadway.

4.3 Monitoring and Addressing Soil Erosion

Timely monitoring and addressing of erosion issues from side slopes are essential to mitigate the aforementioned effects on flexible asphalt concrete pavement. By taking proactive measures to prevent erosion, we can preserve the integrity and longevity of this recently constructed pavement.

As per (AASHTO, Highway Drainage Guidelines, 2007), the general principles for developing control plan for ongoing soil erosion are design slopes consistent with soil properties, limit the area of unprotected soil exposure, minimize the duration of unprotected soil exposure, protect soil with vegetative cover, mulch, or erosion-resistant material, control concentrations of runoff and retard runoff with planned engineering works.

The proposed measures for this particular station aim to halt the continuous and unregulated erosion of soil from the surface of the road's cut slope, which is gradually impacting the recently completed road construction project.

The first task when beginning maintenance on this road section should involve the removal of accumulated eroded soils from the side ditch and the driving lane's surface. This will ensure that both the driving lane and side ditch function effectively according to their intended purposes.

To address the unanticipated and persistent geotechnical issue, remedial measures will be implemented to control the erosion of the roadway cut slope. According to (AASHTO, A Policy on Geometric Design of Highways and Streets, 2004) typically, there are two primary categories of soil erosion control methods: non-engineering or bio-engineering approaches, which involve techniques like vegetation, soil erosion control blankets, silt fences and geotextiles and engineering techniques such as diversion drains and lattice structures.

Out of all the erosion control methods available, vegetation cover is likely the most commonly employed technique for managing this ongoing erosion on roadside cut slopes. This preference arises from the fact that vegetation cover serves multiple purposes, including intercepting rainfall, enhancing water infiltration, stabilizing the soil through root systems that bind soil particles, and dissipating the force exerted by water. Implementing a process of vegetation establishment on this exposed roadside cut slope will provide a sustainable and long-term solution for soil erosion control.

The effectiveness of vegetation covers in controlling erosion becomes significant once the vegetation is established and mature. However, it takes time for the vegetation to become fully effective in controlling ongoing erosion and stabilizing the slope. This means that during the period when there is no vegetation or during the immature stage, soil erosion will continue, making it difficult to establish vegetation without immediate and adequate protection.

Moreover, the absence of initial binding material on the roadside cut slope can result in poor vegetation cover. To address this, soil erosion control blankets/geotextiles are used as short-term vegetation covers to provide immediate soil protection.

Soil erosion blankets help reduce runoff and soil erosion by improving soil quality and promoting vegetation growth, offering a permanent erosion control solution. Geotextiles, on the other hand, control rain splash and runoff while creating a favorable microclimate for subsequent vegetation growth. They are applied on bare slopes after spreading a seed mixture for long-term erosion control.

These erosion control geotextiles can be made from natural or synthetic materials. Synthetic geotextiles, such as silt fences, are commonly used in highway and construction projects to provide temporary sediment control. Silt fences help reduce runoff velocity and filter sediments, enhancing sedimentation. They are preferred due to their cost-effectiveness and ease of installation. Natural geotextiles, constructed from organic materials, are more efficient in controlling soil erosion as they adhere to the surface's micro topography and maintain close contact with the soil. They are also environmentally friendly, being made of biodegradable material.

In summary, silt fences are specific erosion control measures utilized to manage soil erosion on the surface of roadway cut slopes. They act as barriers, slowing down water flow and trapping sediment to prevent erosion and the migration of sediment into nearby water bodies. Regular maintenance and inspection are necessary to ensure their effectiveness.

While engineering techniques like diversion drains and lattice structures are also effective in reducing erosion on roadside slopes by diverting runoff and intercepting it, they do not provide a protective layer on the slope surface. Thus, soil detachment from direct rainfall impact can still occur. Combining engineering techniques with vegetation measures can offer an effective approach to reduce runoff and direct rainfall impact, ultimately mitigating soil loss on roadside cut slopes.

Based on the above discussion, re-vegetation is considered the most efficient and cost-effective strategy to control ongoing and uncontrolled soil erosion on the surface of roadway cut slopes. Vegetation cover provides long-term erosion control, requires less maintenance compared to complex engineering structures, and enhances the aesthetic appeal of the landscape. Therefore, establishing a dense vegetation cover is a priority for restoring this roadside slope and controlling the ongoing soil erosion.

4.4 Problematic/Unsuitable Subgrade Soil

During the detailed site investigation stage, another significant geotechnical issue was discovered at the station ranging from km 114+000 to km 114+500. This issue involved problematic or unsuitable subgrade soil, which was identified based on the distresses observed on the newly constructed asphalt concrete road. The observed problems included surface unevenness and settlements accompanied by alligator cracks.

The subgrade soil in the vicinity of the road's adjacent areas exhibited undesirable characteristics. It appeared as yellowish to whitish fine-grained soil with properties such as reduced strength upon contact with water, higher plasticity, and lower load-bearing capacity as shown in Figure 4-18. Unfortunately, neither during the design stage nor during the construction stage of this project was the presence of this problematic subgrade soil identified and no treatment strategy was implemented.

As a consequence of these overlooked issues, premature pavement failures occurred shortly after the project was completed and opened for traffic.

Figure 4-18: Problematic/unsuitable subgrade soil (Yellowish to whitish silty clay soil)

4.4.1 General

The strength and stiffness of subgrade soil play a crucial role in determining the thickness of pavement layers in conjunction with traffic loadings. When the native subgrade soil exhibits higher strength, the required thickness of the pavement layers above it is reduced. Conversely, if the subgrade soil has lower strength, the thickness of the layers above it needs to be increased. This highlights the significant impact of subgrade strength on the overall pavement design.

In our country, as well as globally, the strength of subgrade soil is typically evaluated using the California Bearing Ratio (CBR) test, which assesses its load-bearing capacity. The plasticity characteristics of the soil are examined through the Atterberg Limit test, while gradation tests are conducted on soil samples. These test results are crucial in determining the suitability of the subgrade soil as a foundation for the road project.

The assessment of subgrade soil suitability as a pavement foundation occurs during both the design and construction phases. This ensures that the pavement layers above it are supported by appropriate material. However, if the test results, upon comparison with the standard specifications, indicate unsuitability, it becomes necessary to recommend and implement treatment measures for the existing native subgrade soil during the construction stage, before any pavement layers are added.

4.4.2 Identifying the Existence of Problematic Subgrade Soil

At the chosen location, noticeable signs of failure are becoming increasingly apparent in the asphalt concrete road due to the presence of problematic or unsuitable subgrade soil. The newly constructed pavement is being affected and undermined by the native subgrade soil, as evidenced by the following significant distress indicators:

- *The newly constructed asphalt concrete road exhibits surface cracking:*

The surface of the recently built asphalt concrete road is showing progressive development of cracks, attributed to inadequate subgrade support. These cracks include both longitudinal cracks, which run parallel to the direction of traffic and alligator cracks, characterized by a pattern resembling the skin of an alligator as shown in Figure 4-19 B. Currently, these cracks are facilitating water infiltration, leading to additional deterioration of the pavement.

- *The pavement surface displays settlements and surface deformations:*

The in situ subgrade soil’s insufficient supporting strength leads to ongoing settlements, resulting in an uneven pavement surface with varying elevations in adjacent driving lanes. The surface deformations, such as bumps, waves, and undulations, are evident on the road surface as shown in Figure 4-19 A & C. These deformations are caused by the subgrade soil’s inadequate supporting capacity, creating an uncomfortable and uneven driving experience for road users.

Figure 4-19: A) Surface undulation B) Surface cracking C) Settlement and depression

4.4.3 Problems due to Problematic Subgrade Soil

The reasons behind the formation of surface deformations or subsidence due to poor subgrade soil are as follows:

- The subgrade soil's quality is crucial for providing necessary support and stability to the road structure. However, when the subgrade soil is of poor quality, it often lacks sufficient load-bearing capacity.
- In a flexible pavement asphalt concrete road, the load transfer mechanism is engineered to distribute the weight of traffic over a wide surface area. This is achieved by transmitting the load through the various layers of the road structure, including the asphalt layer, base course, and subbase. Eventually, the load is transferred to the subgrade soil. It is crucial for the subgrade soil to possess the necessary capability to support the traffic load effectively.
- Insufficient support from the subgrade results in poor subgrade support, causing the road layers above to lack adequate support. As a consequence, the subgrade soil compresses under the load, resulting in excessive deformation and settlement. This leads to the sinking or formation of depressions on the road surface.
- The high plasticity of poor subgrade soil causes it to deform and change shape under load. When heavy traffic loads traverse such soils, they experience plastic deformation, which leads to the formation of depressions on the road surface.

4.4.4 Laboratory tests conducted

A series of laboratory tests as listed under Table 4-5, were carried out on specimens gathered from this site to evaluate the appropriateness of the in-situ subgrade soil for use as a foundation for pavement. These tests were specifically aimed at determining the inherent characteristics of the subgrade soil in terms of its strength and its behavior when exposed to water in the most unfavorable conditions. The results of laboratory tests are depicted under Table 4-6.

Table 4-5: Lists of laboratory tests conducted and their test method (114+000)

S. No.	Laboratory test	Test method
1	Atterberg limit tests (LL & PI)	AASHTO T-89 & 90
2	Particle size distribution	AASHTO T-88
3	Compaction characteristic (MDD & OMC)	AASHTO T-180 D
4	California bearing ratio (CBR)	AASHTO T-193
5	Density of soil in place	AASHTO T-191
6	Natural moisture content	AASHTO T-265
7	Free swell	IS-2720
8	Linear shrinkage	BS 1377-2

Table 4-6: Summary of laboratory test results (114+000)

S. No.	Laboratory test		Test result
1	Atterberg limit	LL	72.6%
		PL	39.1%
		PI	33.5%
2	Soil classification	AASHTO	A-7-5 (17)
		UCS	MH (elastic silt)
3	Compaction characteristic	MDD	1.416g/cm3
		OMC	26.42%
4	Density of soil in place		1.40g/cm3
5	California bearing ratio (CBR) @95% of MDD		0.53%
6	CBR swell		7.18%
7	Natural moisture content		49.97%
8	Linear shrinkage		14.61%
9	Free swell		170%

4.4.5 Analysis of test results

I. Atterberg limit test results

Atterberg limit test results indicated a high susceptibility of the subgrade soil to volume changes with variations in moisture content. This high sensitivity to moisture conditions is a key characteristic of problematic highway subgrade soils and is a major contributor to many of the failures observed when these soils become wet.

The specific results of the Atterberg limit test are as follows:

- Liquid limit: 72.6%
- Plastic limit: 39.1%
- Plasticity index: 33.5%

The high liquid limit of 72.6% indicates that the soil has a significant water-holding capacity and is prone to undergo significant volume changes with variations in moisture content. This high liquid limit suggests that the soil becomes very plastic when wet, which can lead to shrinkage and swelling as moisture content fluctuates. This characteristic makes the soil highly susceptible to changes in moisture conditions, which is a concern for subgrade soils in highway construction.

The plastic limit of 39.1% indicates the moisture content at which the soil begins to behave in a plastic or deformable manner. A plasticity index of 33.5% highlights the range of moisture content within which the soil exhibits plastic behavior. Soils with a high plasticity index are known to undergo significant changes in volume when they dry out or become wet.

According to (ERA, Standard Technical Specification and Method of Measurement, 2013), unsuitable material is defined as a material having a liquid limit (LL) exceeding 60; or a plasticity index (PI) exceeding 30. Therefore, based on the project specifications, the subgrade material is identified as unsuitable for serving as a pavement foundation, necessitating special treatment.

II. Soil classification

- Soil classification as per AASHTO (114+000): the soil is classified as **A-7-5(17)**
- Soil classification as per USCS (114+000): Soil group symbol is **MH (Elastic silt)**

Using plasticity chart as shown in Figure 4-20:

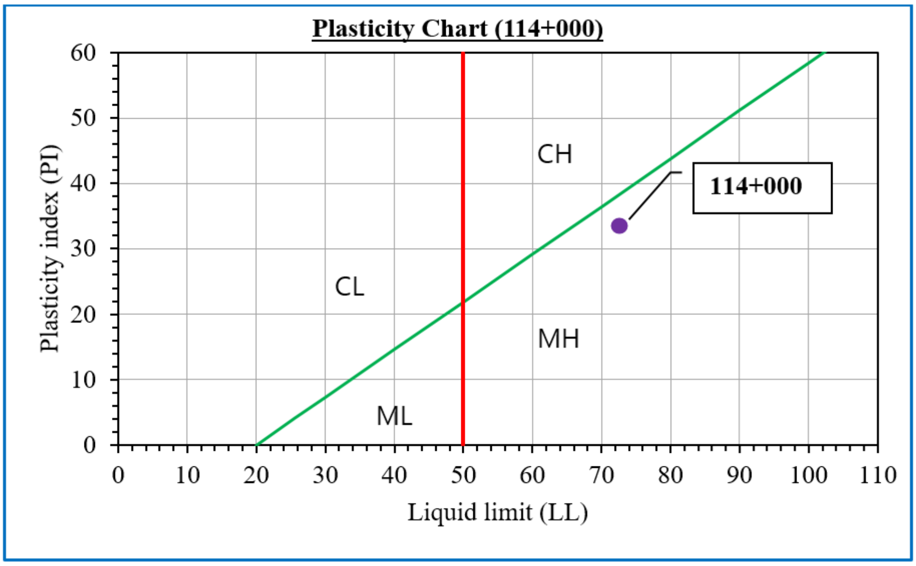

Figure 4-20: Plasticity chart (114+000)

The soil is designated as A-7-5(17) according to AASHTO highway subgrade soil classification system. This classification implies that the subgrade's suitability as a pavement foundation is deemed "POOR," necessitating special treatment.

The soil group symbol is MH, indicating it falls under elastic silt.

The unsuitable subgrade soil, when plotted on the plasticity chart using the liquid limit and plasticity index values, aligns with the MH (elastic silt) classification. According to the USCS, this places the soil under the category of elastic silt, suggesting potential issues with volumetric changes.

Both classification systems highlight concerns, especially when the soil interacts with water. This soil demands special attention and remedial measures to mitigate and minimize its impact on pavement performance.

III. Particle size distribution of the soil

By conducting a wet sieving laboratory test on the subgrade soil sample collected at km114+00, in accordance with the ASTM D2478 standard, the particle size distribution analysis as shown in Figure 4-21, yields the following results:

- Percent of coarse gravel (75mm-19mm): 0%
- Percent of fine gravel (19mm-4.75mm): 0%
- Percent of coarse sand (4.75mm-2.0mm): 1.50%
- Percent of medium sand (2.0mm-0.425mm): 4.20%
- Percent of fine sand (0.425mm-0.075mm): 9.10%
- Percent of fine-grained soil (<0.075mm): 85.20%

The particle size distribution results paint a pretty clear picture. The high percentage of silt and clay, which are fine-grained soils, indicates poor drainage and susceptibility to swelling when in contact with water. These soils tend to absorb water and expand, leading to swelling and shrinking behavior. This swelling can exert pressure on the flexible asphalt road, causing it to fail prematurely.

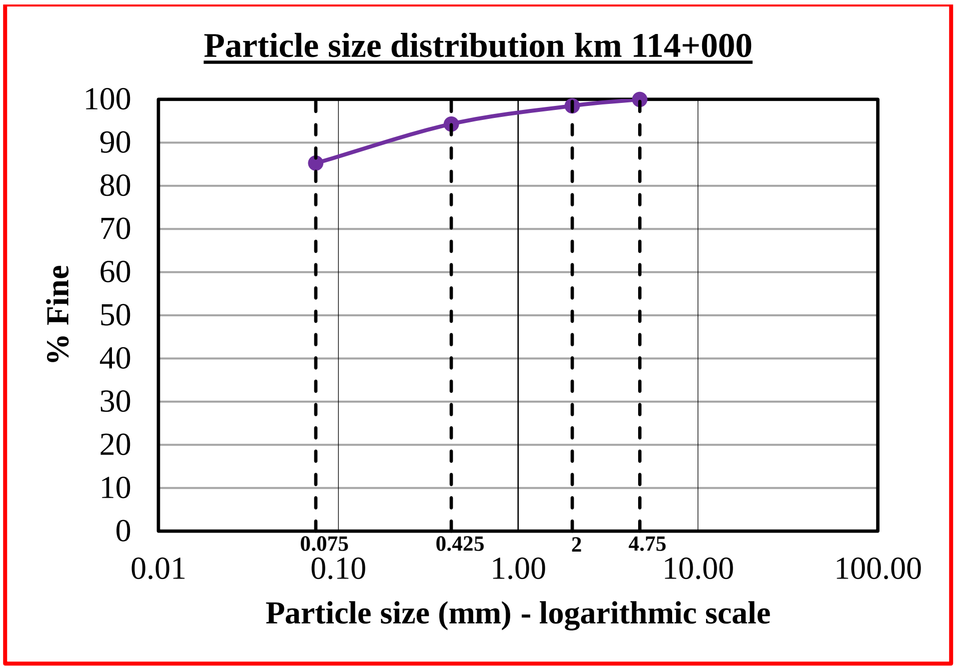

Figure 4-21: Particle size distribution km114+000

IV. Compaction characteristic

The modified proctor compaction test conducted on this subgrade soil results the following values;

- Maximum dry density of the soil: 1.41 g/cm^3
- Optimum moisture content: 26.40%

The lower maximum dry density (MDD) suggests that, even under optimal laboratory compaction conditions, the soil has limitations in achieving a high density. This can result in reduced load-bearing capacity, making the subgrade less capable of supporting the traffic loads on the road.

Achieving a high in-place density during construction may be challenging due to the inherent properties of the soil, such as the high percentage of silt and clay.

The lack of coarse-grained particles and the dominance of fines further contribute to poor structural support. Coarse particles contribute to the overall stability and strength of the soil.

The combination of lower densities and poor structural support increases the risk of deformations and cracks in the road surface. The subgrade's inability to provide sufficient support can result in distresses that manifest as uneven surfaces and structural failures.

One of the common test that will conduct on the samples of highway subgrade soils to assess their strength and load carrying capacity is a well know test of California Bearing

Ration (CBR) test, based on a three point CBR test was conducted on this subgrade soil to assess it load bearing character this test also includes the CBR-swell value within two surcharge loads on the prepared test specimen. Based on this below are the test result:

V. CBR and CBR Swell value

- CBR: 0.53%
- CBR Swell value: 7.18%

As can we understand from the test results, a CBR value of 0.5% indicates a very low strength of the subgrade soil. Typically, soils with CBR value less than 3% at 95% of modified AASHTO compaction (AASHTO method T-180) after 4 days soaking as mention in the (ERA, Standard Technical Specification and Method of Measurement, 2013) are considered unsuitable for road construction without significant improvement measures.

The lower CBR value suggest that the subgrade soil lacks the necessary strength to support the overlying pavement structure, leading to deformation and failure under traffic loading. Also this soil has extremely higher value of CBR Swell value with two surcharge rings which shows a very higher potential of volume change (swelling) of the soil when the test specimen is subjected to the worst scenario of moisture condition of 4 days soaking with two surcharge rings, by this the higher CBR Swell value has led to volume change to the subgrade soil, causing heaving and settlement issues this can further contribute to pavement distress and premature failure. This swelling pressure exert significant pressure on the overlying pavement structure, leading to cracking, rutting and other observed distresses.

Implications on the road performance

- The lower CBR suggests that the subgrade lacks the strength required to support the road pavement, this can result in deformation, rutting and other structural failures. This is exactly seen on the newly constructed road project, the road surface shows different deformations, surface distresses and structural failures.
- The extremely higher value of CBR Swell indicates that the subgrade soil is prone to volume change, upon wetting which can lead to heaving and settlement issues, this poses a risk to the stability and integrity of the road.

To sum up, the California Bearing Ratio (CBR) test on a highway subgrade soil revealed a low CBR value of 0.5%, indicating insufficient strength for pavement support. A CBR

Swell value of 14% suggests a high swelling potential, leading to heaving and settlement. The subgrade lacks the needed strength, resulting in deformations and structural failures observed in the newly constructed road. The significant swelling pressure poses a risk to road stability and integrity, emphasizing the need for remedial measures.

VI. Free swell and linear shrinkage test result values

- Free swell: 170%
- Linear shrinkage: 14.6%

The obtained values from the free swell and linear shrinkage tests conducted on samples from the problematic subgrade soil aimed to assess the impact of moisture changes on its performance, particularly during wetting and drying conditions.

The free swell test measures the potential volume increase of the subgrade soil upon water absorption. With a high free swell value of 170%, it indicates the soil's significant expansion capacity when in contact with water. This expansion tendency has resulted in structural issues and instability in the newly constructed road or pavement layers built upon it.

On the other hand, the linear shrinkage test measures the volume decrease of the soil when it loses moisture. The recorded 14.6% suggests substantial shrinkage in the subgrade soil upon drying out. This shrinkage has caused settlement beneath the pavement, leading to surface unevenness and cracking evident in the failed road section.

To summarize, the combination of a high free swell (170%) and significant (more than the project specification) linear shrinkage (14.6%) indicates the contrasting behavior of the subgrade soil regarding moisture variations. It expands significantly when wet and contracts considerably when dry, posing significant problems for road construction. These properties manifest as cracking, rutting, unevenness and potentially premature failure, as observed in the newly constructed road project.

VII. Natural moisture content (NMC) test result value

- Natural moisture content (NMC): 49.97%

A natural moisture content of 49.5% in this subgrade soil is quite high and indicates that this subgrade soil is already at near saturation. Here's how this moisture content indication impacts the performance of the subgrade soil.

- This moisture content level signifies that the soil is almost at its maximum capacity for holding water naturally. Near saturation indicates that the soil is already highly susceptible to volume changes due to additional moisture.
- It reinforces the findings of the free swell test, suggesting that the soil is already close to its maximum swelling potential. Any additional water infiltration or rainfall can exacerbate this swelling behavior, leading to further expansion and instability.
- High natural moisture content significantly impacts the stability and strength of this subgrade soil. It reduces the soil's load-bearing capacity and increases its susceptibility to deformation under traffic loads.
- Additionally, high moisture content has compromised the soil's cohesion and internal friction, further contributing to its instability.
- This level of moisture content exacerbates the soil's tendency to swell and shrink, leading to further distress in the road surface, such as increased cracking, rutting and unevenness which has exactly revealed on this newly constructed asphalt concrete road.

Generally, a series of tests evaluated subgrade soil for road foundation suitability. Results revealed high sensitivity to moisture, notably with a high liquid limit of 72.6% and plastic limit of 39.1%, indicating excessive water retention and plastic behavior when wet, unsuitable for pavement use.

Soil classification as A-7-5(17) and elastic silt flagged poor foundation suitability. Particle size distribution showed high fine-grained soil content, leading to poor drainage and swelling when wet, detrimental to road surfaces.

Compaction tests indicated challenges in achieving optimal density during construction, potentially reducing load-bearing capacity.

The California Bearing Ratio (CBR) test exhibited low strength of 0.5%, inadequate for pavement support and a significant CBR Swell value of 14%, signaling potential volume changes and pavement distress.

The free swell and linear shrinkage tests highlighted significant soil expansion and contraction due to moisture, leading to observed road failures.

High natural moisture content of 49.97% indicated near saturation and vulnerability to further expansion upon water exposure. Overall, the results underscore the soil's unsuitability for road construction due to high moisture sensitivity and inadequate strength, causing observed road distress and failures.

4.4.6 Remedial Strategies

The cumulative findings from the laboratory tests indicate that the soil possesses high plasticity, notable shrink-swell tendencies, limited bearing capacity and higher moisture content. By this the remedial measures are targeted to counter all these undesirable properties of this problematic subgrade soil. Most of the time the followings are major remedial measures that will applied for problematic highway subgrade soils such as soil stabilization, excavation and replacement, soil reinforcement, compaction and drainage improvement.

As per (ERA, Flexiable Pavement Design Manual Volume I, 2013), no allowance for CBRs below 3% has been made because, from both a technical and economic perspective, it would normally be inappropriate to lay a pavement on soils of such poor bearing capacity. Moreover, the measurement of the bearing strength of such soft soils is generally most uncertain and CBRs below 2% are of little significance. For such materials, special treatment is required.

With a CBR value of 0.5% indicating severe weakness in this soil, the logical step would have been to avoid placing pavement layers directly on this inadequate subgrade. However, in this particular road project, the decision was made to lay new asphalt concrete layers without any treatment on this problematic soil. As a result, premature failures emerged shortly after the road was opened to traffic.

According to (ERA, Design Manual for Low Volume Roads Part D, 2011), the followings are traditional countermeasures which includes the followings:

- Placing an un compacted pioneer layer(s) of sand, gravel or rock fill over the clay and wetting up, either naturally by precipitation or by irrigation;
- Pre-wetting (2-3 months) to induce attainment of the equilibrium moisture content before constructing the pavement;
- Partially or completely removing the expansive soil and replacement with inert material;
- Modifying or stabilizing the expansive soil with lime to change its properties;
- Increasing the height of the fill (surcharge) to suppress heave;

- Minimizing or preventing moisture change using waterproofing membranes and/ or vertical moisture barriers.

The issue with the subgrade soil stems from fluctuations in moisture. When it encounters water, it swells and loses its strength, while drying causes it to shrink. This variability in the soil's behavior has led to premature failures in newly constructed pavement. To address this, maintaining consistent moisture levels in the soil throughout the road's design period appears to be the most effective solution to ensure the subgrade performs its role effectively.

To do this the most effective remedial measure will be construction of vertical moisture barriers. A vertical moisture barrier serves to inhibit the seasonal lateral movement of moisture to and from the subgrade beneath the pavement. This, in turn, helps prevent the expansion of the subgrade during wet periods and its contraction during dry periods, (Picornell, 1986).

Construction of this vertical moisture barriers will maintain a relatively constant moisture regime beneath the pavement and reduce the development of pavement roughness (MCMANUS, 2002).

In a recent exceptionally dry period, it was demonstrated that the vertical moisture barriers retained higher levels of moisture beneath the pavement seal. This resulted in a substantial decrease in cracking and the associated depressions compared to control sections without barriers, (Evans R. P., 1994). The ideal depth for a cost-effective vertical moisture barrier is one that minimizes seasonal moisture fluctuations. This depth aims to control pavement edge shifts and maintain an acceptable level of smoothness, effectively preventing related cracks.

Moisture can be controlled by preventing moisture changes, either wetting or drying, through encapsulation. A partial encapsulation can be used to minimize the post construction problems encountered in expansive soils. The geo-membrane is used as a moisture barrier and is placed in horizontal, vertical, or in both directions. Preventing moisture from entering or leaving expansive soils minimizes or prevents pavement distresses, formation of cracks and slab distortion (Schaefer, 2017). By this in order to diagnose this section of the road from km 114+000 – km 1147+500, the above method which is partial encapsulation on both sides of the road (RHS and LHS) of this swelling subgrade soil will be executed as per the following construction step:

- Excavation of a trench on both sides of the roadway in the area to be encapsulated i.e. from station km 114+000 to km 114+500.
- The width and depth of the trench are 0.25 m and 2 m, respectively.
- The geo-membrane is laid into a trench. The geo-membrane is placed on the shoulder side of the trench rather than the pavement side.
- Flowable fill, consisting of medium-graded sand, a small percentage of cement, and a large portion of high fly ash and water, is used to backfill the trench so that the geo-membrane will not be damaged.
- The geo-membranes from both sides are then extended over the subgrade surface for the required distance (km 114+000 – km 114+500), ideally over the entire surface between the trench lines and seamed at the centerline of the roadway.

Materials

Most standard geo-membranes can be used including Chlorosulfonated Polyethylene (CSPE), Ethylene Interpolymer Alloy (EIA), Ethylene Propylene Diene Terpolymer (EPDM), High-Density Polyethylene (HDPE), Linear Low-Density Polyethylene (LLDPE), Polypropylene (PP), or Polyvinyl Chloride (PVC) as well as others.

The ideal backfilling material for a vertical moisture barrier is one that can be easily placed into a deep and narrow trench. It ought to be of low permeability and be self-compacting. It also needs to be readily available and relatively inexpensive.

A slurry type material, which is commonly known as flowable fill or controlled low-strength material, was selected. It had the characteristics of a flowable nature that provided an easy and rapid placement, no compaction requirements and a relatively low permeability. Flowable fill typically consists of fine- to medium-grained sand, a low cement content, a high water content, and a high proportion of fly ash to achieve the required flowability. It could be poured directly into a very narrow trench with minimum labor requirements, and due to its flowability it is self-compacting. It also displaces any water that has entered the trench, e.g., by seepage or surface runoff during a thunderstorm.

Generally, a section of a road between km 114+000 to km 114+500 faced premature pavement failures due to problematic subgrade soil. This soil, with poor load-bearing capacity, high plasticity and moisture sensitivity, wasn't adequately assessed before or during construction. The soil's instability caused surface cracking, settlements, and deformations in the newly constructed asphalt road. Lab tests highlighted its unsuitability, leading to recommendations for remedial measures such as vertical moisture barriers using geo-membranes and flowable fill to mitigate moisture-related soil fluctuations, stabilize the subgrade and prevent further pavement distress.

4.5 Roadway Cut Slope Failure

During the construction of this road project, the challenging topography and alignment necessitated the excavation of soils or rocks from the side hills and mountains to create a flat surface for road construction. This involved cutting slopes along the roadway at various stations. It was crucial to ensure that the cut slopes were positioned below a predetermined slip surface to maintain stability.

Unfortunately, many of the cut slope sections have steep angles, either on the brink of failure or already experiencing failures. A specific instance of this can be observed at station km 100+300 on the right-hand side (RHS). The deformation or failures of the slope at this station can be attributed to the steepness of the excavation angle.

4.5.1 General

Excavating a roadway slope at a steep angle may lower excavation costs but also significantly raises the risk of slope failure due to potential instability of the soil or rock. The reduced stability of the cut slope results from forces exceeding the shear strength of the soil, leading to slope failure and collapse. Therefore, careful consideration of slope steepness is essential to ensure the safety and durability of the road infrastructure. Implementing appropriate engineering measures and adhering to recommended slope angles can mitigate the risk of slope failure and contribute to the long-term stability of the excavated roadway.

4.5.2 Observation on Failed Roadway Cut Slope

The followings are significant failures that are raised as a consequence of steep slope angles:

- *Slumping*

At the specified location along the roadway, clear signs of soil slumping have been observed on the surface of the cut slope as shown in Figure 4-22 A. This movement of the soil mass downward indicates instability in the roadside cut slope. The evidence is further supported by the presence of displaced soil and fragments of rock materials at the base of the slope.

Figure 4-22: A) Slumping B) Accumulation of displaced soil from the surface of slope

- *Land Slides*

At a particular station at km 100+300 on the right-hand side (RHS) that has been monitored, it has been observed that there is a significant loss of stability in the slope, disrupting the natural equilibrium between the forces acting on the slope and the strength of the soil mass. This instability has resulted in both movement and collapse of the slope as shown in Figure 4-22 B. The failure mechanism at this specific location appears to be a slump failure, where a considerable portion of the slope has slid down. As a consequence of this slope failure, a section of the driving lane has been covered by the sliding material and the side ditch adjacent to the roadway has been filled.

Figure 4-23: A) The side ditch adjacent to the road has been filled by sliding material B) A section of driving lane has been covered by the sliding material @ km 100+300 RHS

- *Debris fall*

The debris fall observed in the roadway cut is a consequence of excavating a steep slope as shown in Figure 4-24. It involves the downward movement and deposition of materials from the failed slope surface onto the roadway and its surroundings. The debris comprises various geotechnical components such as soils, vegetation and loose materials that were originally part of the failed slope.

The amount of debris deposited on the roadway indicates localized deposition, likely due to the steep slope angle facilitating a higher velocity of flow. As a result, the falling debris has covered a significant portion of the right driving lane, posing potential traffic obstructions and risks to road users.

Figure 4-24: Debris flow near the toe of the failed slope @ km 100+300 RHS

4.5.3 Effects of Failed Roadway Cut Slope

The failure of a roadway cut slope has significant and extensive impacts on the performance of a newly constructed asphalt concrete road. The following are the major consequences of this slope failure for the road:

- Roadway disruption: roadway disruption has occurred as a result of the failure of the roadway cut slope, causing the slope material to collapse onto the road surface. This has resulted in the obstruction of a significant portion of the driving lane, impeding the flow of traffic. The disruption has caused inconvenience and difficulties for road users, impacting their ability to travel smoothly and safely.
- Safety hazards: the failure of the cut slope at the specified location has given rise to significant safety hazards for drivers and road users. The collapse of the slope increases the risk of accidents, while the presence of falling debris from the failed slope poses a continuous danger to those using the road.
- The failure of the roadside fill slope has caused damage to the recently constructed asphalt concrete road, resulting in a decrease in riding quality for drivers. Additionally, this damage has led to increased wear and tear on vehicles using the road.
- The failure of the roadside cut slope has caused a disruption in the drainage system of the recently constructed asphalt concrete road. The sliding material from the

failed slope has filled the adjacent side ditch, preventing proper water flow and resulting in water accumulation on the road surface. This accumulation of water poses various problems, including potential damage to the newly placed asphalt concrete layer due to prolonged exposure to moisture.

- Increased maintenance and repair cost: the failure of the cut slope necessitates substantial investment in repair and stabilization efforts, resulting in increased maintenance and repair costs. Addressing this issue will require allocating additional resources and budget to restore the stability of the failed slope and ensure the long-term viability of the newly constructed road project.

4.5.4 Analysis of Failed Roadway Cut Slope

At this station, where the roadway's slope has experienced gradual failures, a set of laboratory experiments were carried out on samples taken from both the upper and lower sections of the slope. This was necessitated by the presence of distinct soil layers, the upper layer comprises reddish-brown silty clay soil, while the lower layer consists of highly weathered rock. Consequently, separate samples were gathered and subjected to laboratory tests to classify the soil types and ascertain the shear strength characteristics of these layers. This information was crucial for modeling purposes, aiding in the determination of safe slope angles and identifying potential slip surfaces.

Laboratory tests and their corresponding test methods conducted on both upper and lower layers of the slope are outlined in the Table 4-7 below.

Table 4-7: Lists of laboratory tests conducted and their test method (km 100+300)

S. No.	Laboratory test	Test method
1	Atterberg limit tests (LL & PI)	AASHTO T-89 & 90
2	Particle size distribution	AASHTO T-88
3	Density of soil in place	AASHTO T-191
4	Natural moisture content	AASHTO T-265
5	Direct shear test	ASTM D3080

Here are the laboratory test outcomes for both the upper (0 – 9 m) and lower (9 m – 16 m) layers of the slope, shown in Table 4-8:

Table 4-8: Laboratory test results (100+300 RHS)

S. No.	Laboratory test		Test result	
			Upper layer	Lower layer
1	Atterberg limit	LL	60.8%	50.6%
		PL	36.5%	31.5%
		PI	24.3%	19.1%
2	Soil classification	AASHTO	A-7-5 (3)	A-7-5 (1)
		UCS	MH (Gravelly elastic silt with sand)	MH (Gravelly elastic silt with sand)
3	Density of soil in place		1.40g/cm3	1.54g/cm3
4	Natural moisture content		49.97%	29.35%
5	Direct shear test	Φ (°)	24	26
		C (Kpa)	34	30

The dimensions, slope and general form of the slope that has experienced failure are depicted in Figure 4-25.

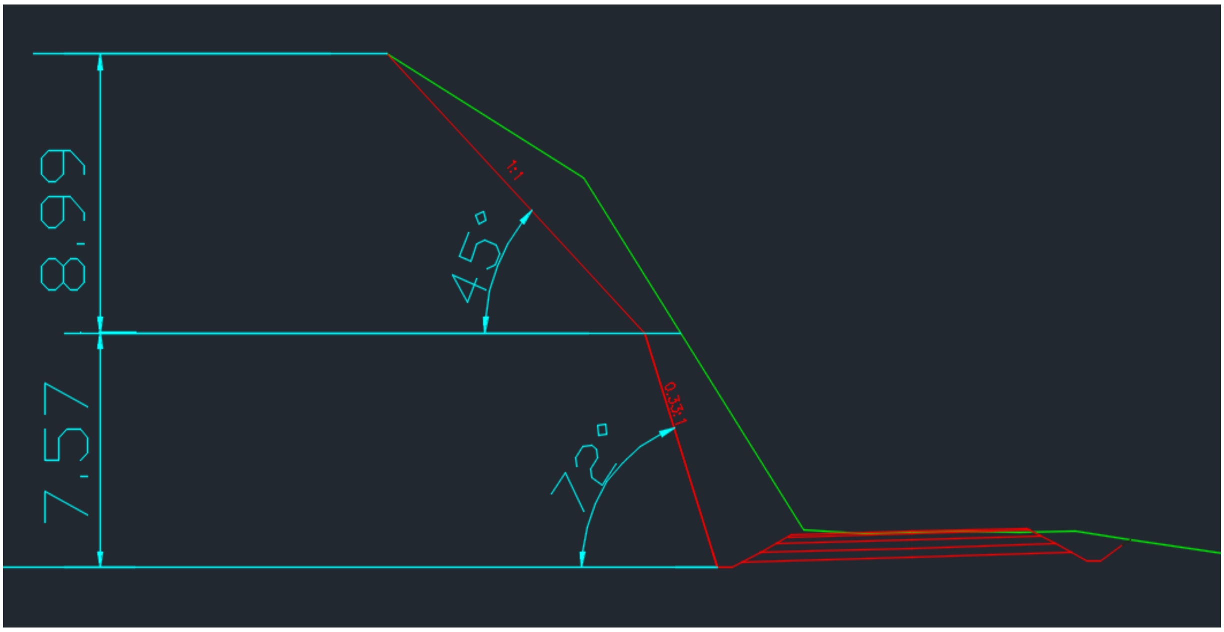

Figure 4-25: Cross-section of the road at station km 100+300 (Source: Project Consultant)

Examining the road's cross-section, the original ground level is represented by the green line, while the finished level of the cut slope is denoted by the red line as shown in Figure 4-25. During the site investigation, two distinct soil formations were identified: the upper layer (0.0 m - 8.99 m) and the lower layer (8.99 m – 16.56 m). Based on these formations, the roadway cutting implemented two slope angles, 72° angle (0.33H:1V) for the lower layer and a 45° angle (1H:1V) for the upper part as shown in Figure 4-26. Unfortunately, the slope of the roadway cut has experienced failure, leading to debris covering the adjacent surface side drainage and nearly half of the driving lane.

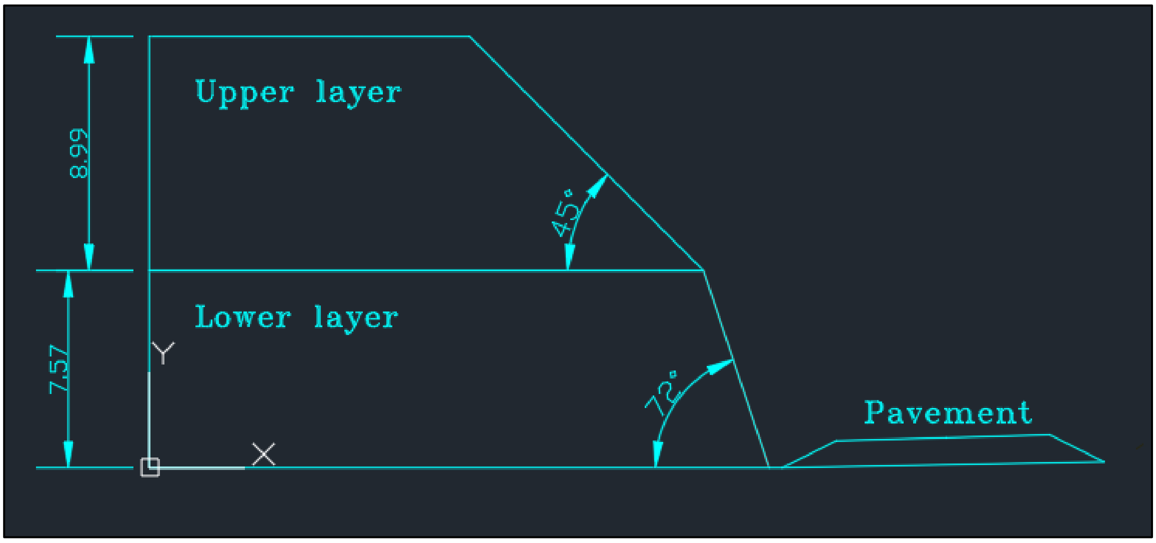

Figure 4-26: Geometry of failed roadside cut slope

Having laboratory test results and the geometry of the cut slope, the finished slope has modeled using a geotechnical software of GeoStudio to determine the factor of safety and slip surface.

From multiple possible slip surface results, the one with a similar failure plane (slip surface) has been selected. As observed during detail site investigation the length of the slumped soil from the top was estimated around 1.5 m, by this the selected slip surface shown that a length of 1.3352. By this an initial factor of safety of 0.98 by Bishop method.

The factor of safety being less than 1 indicates that the driving forces causing the failure were greater than the resisting forces holding the slope in place.

This factor of safety which is less than 1 suggests that the slope was unable to withstand the applied loads or environmental conditions, resulting in a failure. This failure has occurred due to mainly steep slope angle.

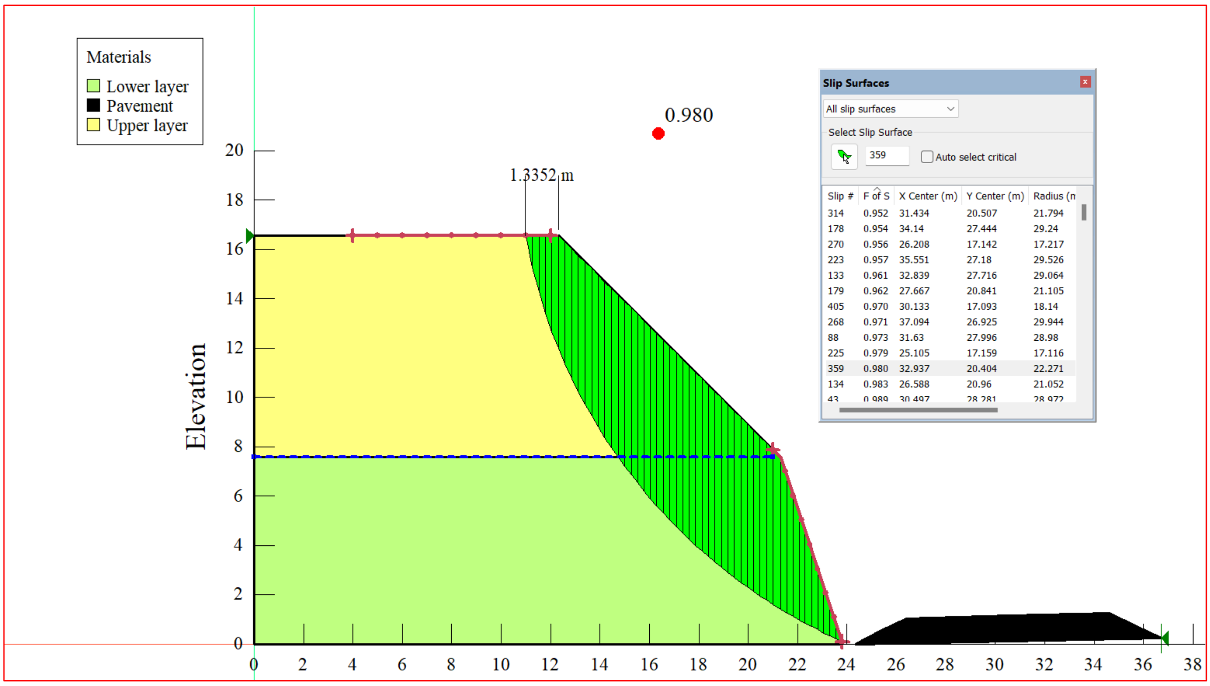

Figure 4-27: Slope stability analysis result (Bishop)

The subsequent phase involves determining potential geometries for the failed roadside cut slope, aiming to achieve a minimum factor of safety of 1.5. Numerous iterations of slope geometries are being explored to ascertain an economically viable cut slope configuration as shown Figure 4-28.

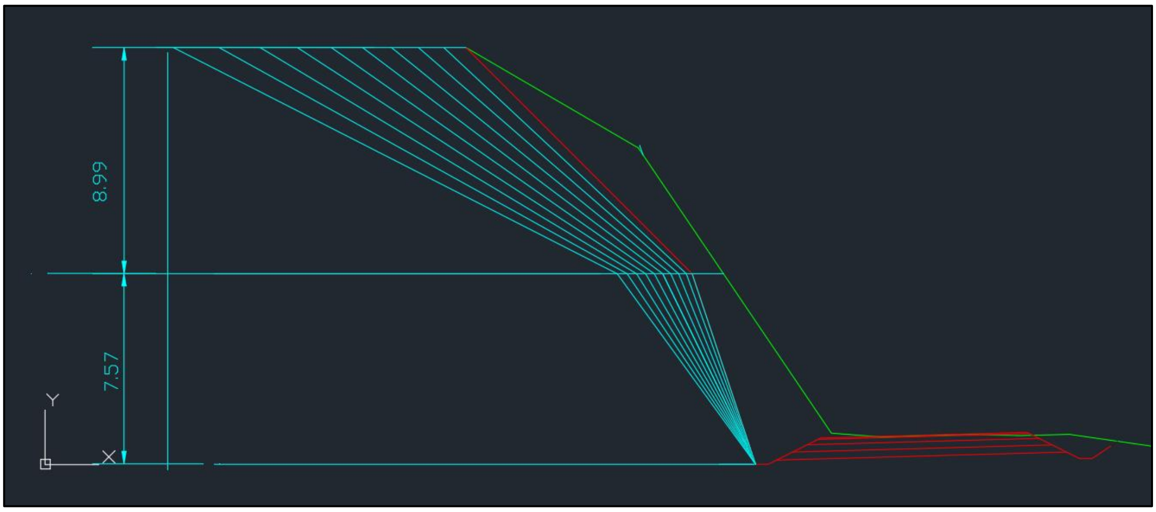

Figure 4-28: Trial slope geometries

The initial cut slopes for both the upper and lower layers were initially steeper than desired. The initial cut slope angle for the lower layer was 72° and over ten consecutive cut slopes, it was reduced by 2° each to achieve a flatter initial geometry. Similarly, for the upper layer, the initial slope was 45° and it was also reduced by 2° for ten consecutive cut slopes.

However, the overall cut slope, when subjected to slope stability analysis, proved economically impractical due to achieving a factor of safety of 1.5. Consequently, alternative geometries, including the introduction of a bench for the slope, were explored for further cut slope stability analysis.

According to the authoritative design manual on geotechnical issues ((ERA), Geotechnical Investigation Manual , 2013), it is recommended that cut slopes exceeding a height of 6 meters should incorporate a bench of at least 3 meters with an inward slope of not less than 10%. In light of this significant guideline for cut slopes, an attempt has been made to incorporate a bench in this particular slope and analyze its stability.

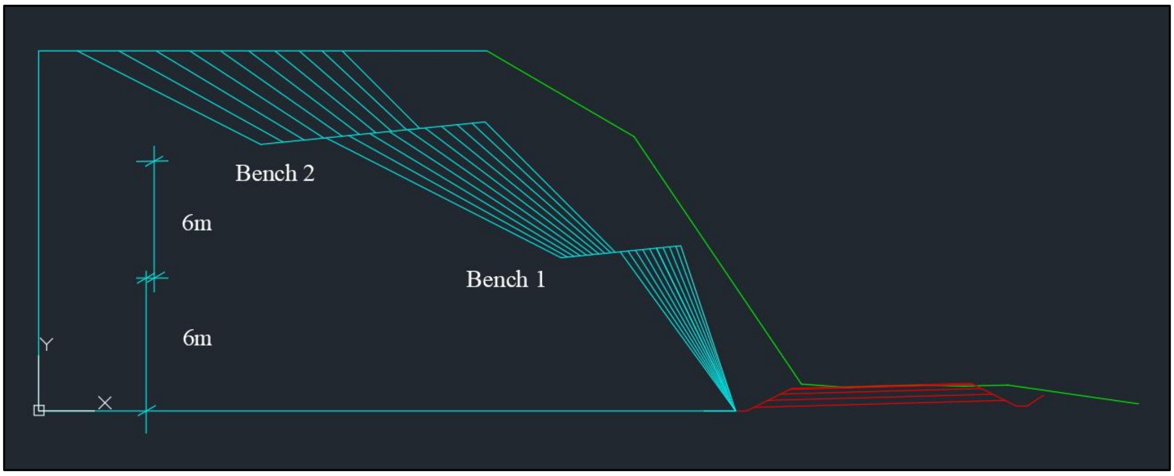

Figure 4-29: Trail cut slope geometry with two benches

As a result, two consecutive benches were incorporated for every 6-meter increment in the cut slope height as shown in Figure 4-29. Subsequent slope stability analyses were conducted using the resultant cut slope geometry. Through multiple iterations, an economical cut slope geometry was determined, achieving a factor of 1.5, as depicted in the Figure 4-30 and Figure 4-31.

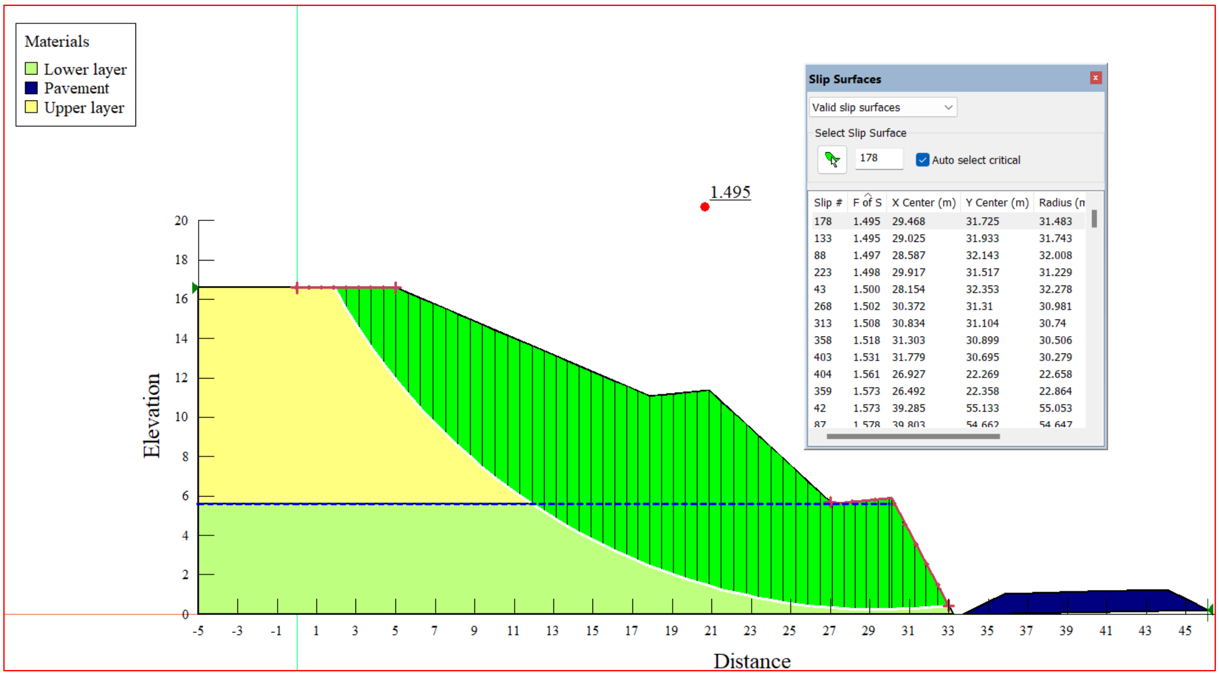

Figure 4-30: Slope stability analysis result (Morgensten-Price)

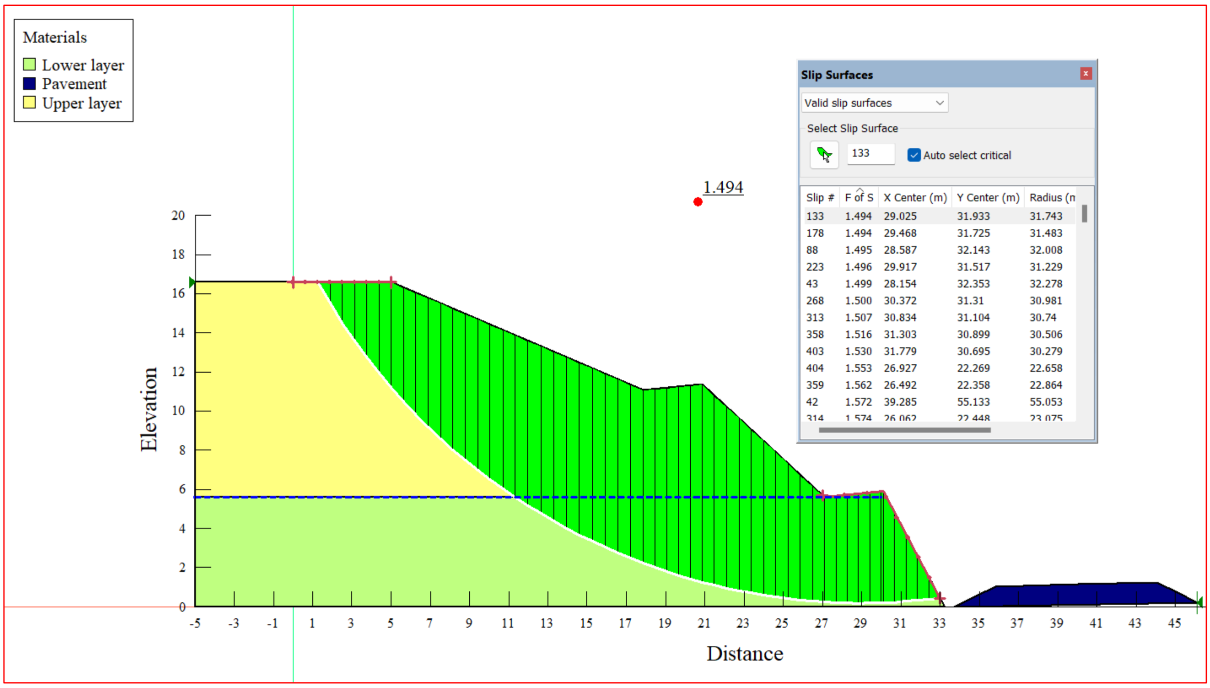

Figure 4-31: Slope stability analysis (Bishop)

In conclusion, the primary factor contributing to the failure of the roadway cut was identified as the steep geometry during construction. Numerous iterations of new cut slope geometries were then analyzed for slope stability. The objective was to implement a flatter slope, reducing shear stress. Following the recommendations from manuals, a slope geometry with two benches, as depicted in the image above, was determined to be a safe and viable solution for reconstructing and restoring the failed roadway cut slope as shown Figure 4-34.

This introducing of benches for this high roadway cut slope enhances stability by reducing steepness and shear stress, mitigating the risk of failures. These benches control erosion, facilitate maintenance and improve safety for personnel. They create platforms for vegetation, promoting slope resilience. The design aligns with standards, ensuring adherence to sound engineering practices. Aesthetically, benches complement the landscape. Overall, this approach addresses slope stability, safety and environmental considerations in geotechnical engineering and road construction.

4.5.5 History and Extent of Failed Road Roadway Cut Slope

The topography of the compromised roadway cut slope was examined using images from Google Earth's historical imagery. As depicted in Figure 4-32, the failure of this road side cut slope spans from station km 100+060 to 100+360, covering approximately 300 meters. Contrarily, Figure 4-33 shows that the road side slope appears stable but with a reduced road width.

Figure 4-32: Google earth historical imagery dated November 2020 at km 100+300 (RHS)

Figure 4-33: Google earth historical imagery dated January 2009 at km 100+300 (RHS)

4.5.6 Remedial Measure for Failed Roadway Cut Slope

Among several remedial measures considered within the project's context, the most effective and economical solution for addressing the ongoing slope failure involves implementing a geometrical method. This approach is not only straightforward and cost-efficient but also facilitates an easy enhancement of slope safety by transforming steeper slopes into gentler ones. The application of this method entails trimming the slope to a geometry where the factor of safety equals 1.5. inventory as shown in Figure 4-35.

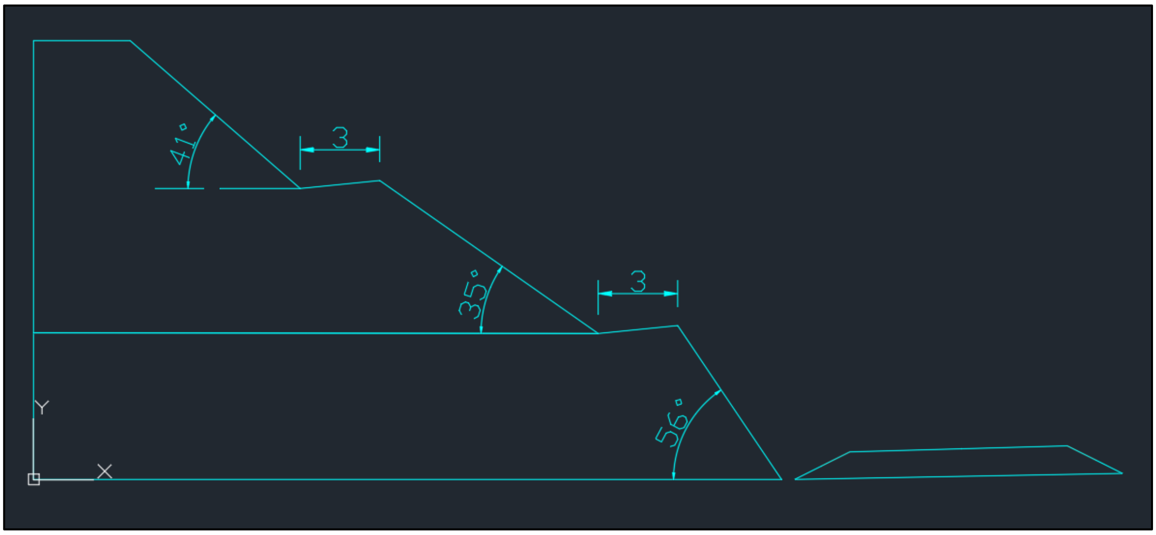

Figure 4-34: Geometry of cut slope with two benches and FS of 1.5.

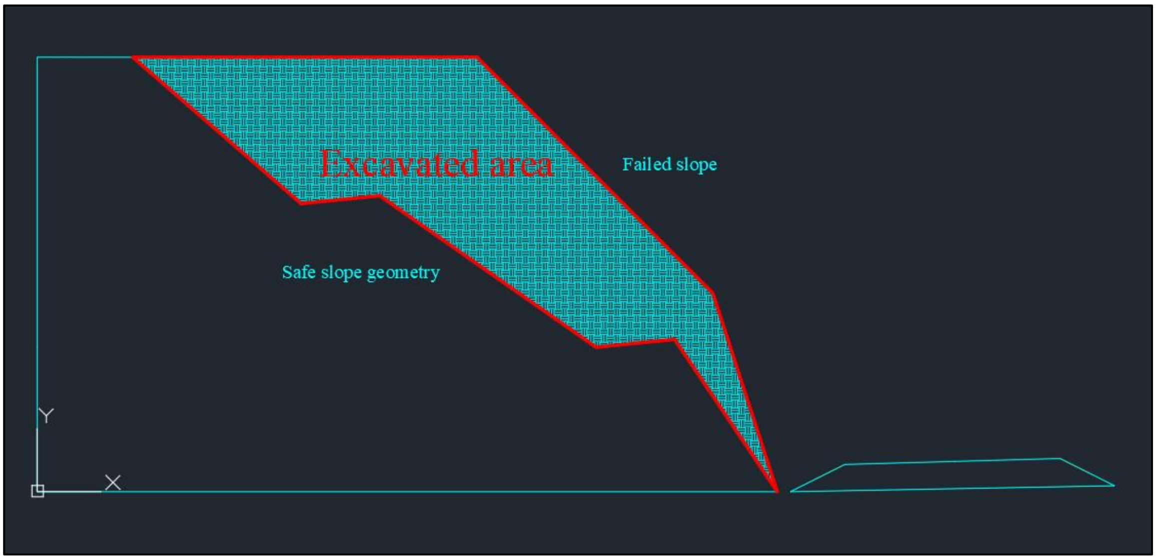

Figure 4-35: Slope flattering

In summary, the geotechnical investigation conducted on the compromised section yielded crucial insights into the factors leading to the failure of the roadway cut slope. The analysis pinpointed the heightened driving shear stress resulting from the steep geometry as a predominant factor in the failure. Employing comprehensive geotechnical techniques, including extensive sampling, shear strength parameter determination and Geostudio modeling, a revised slope geometry has been proposed to alleviate the risks associated with the initial failure. This involves mitigating the impact of the sheared soils that have influenced into the side ditch and portions of the driving lane, extending up to the revised, flatter slope to reduce the steepness of the existing roadside cut slope.

The recommended remedial strategy encompasses the implementation of slope flattening, strategically adjusting the slope angle to align with the newly derived geometry. This adjustment ensures enhanced stability, minimizing the potential for future failures and mitigating the driving shear forces at play. To further fortify slope stability, it is advisable to introduce additional measures, such as grassing. Incorporating vegetation, particularly grass cover, not only enhances the visual appeal but also facilitates root reinforcement, contributing significantly to overall slope stability

Implementing slope flattening and grassing on an actual site will involves the following steps to ensure effectiveness and long-term stability:

- Conduct a topographic survey to obtain accurate elevation data and create a detailed contour map of the area.
- Use appropriate excavation and grading equipment to cut and reshape the slope according to the revised design.
- Choose grass species suitable for the local climate, soil conditions and maintenance requirements.

4.6 Roadway Fill Slope Failure

In the vicinity where roadway cut slopes are frequently encountered along the road alignment, there are also numerous roadside fill slopes. These fill slopes are constructed alongside the road to provide support and stability to the road structure. Typically composed of artificially placed soil and weathered rocks, they are designed to create a smooth transition between the road surface and the adjacent terrain. During the construction of these roadside fill slopes, it is crucial to consider factors such as slope angles, compaction, drainage and erosion controls. By paying attention to these aspects, the constructed roadside fill slopes can have a longer service life.

However, despite these considerations, a significant roadside fill slope failure has been observed along the roadway at station 126+000 on the right-hand side (RHS). This failure is clearly visible along the road and has resulted in the closure of the RHS driving lane. Consequently, when two vehicles traveling in opposite directions meet at this station, they cannot simultaneously use this section of the road. Instead, one vehicle must wait for the other to pass before proceeding.

4.6.1 General

Roadside fill slopes are constructed artificially on the side of roads by placing and compacting soil and weathered rock materials. Their primary purpose is to provide support and stability to the road structure, as they modify the natural terrain to create a stable platform. Additionally, these slopes are designed to achieve a smooth transition between the road surface and the surrounding terrain. This transition zone ensures a gradual change in elevation, avoiding sudden shifts that could be uncomfortable or pose safety hazards for road users.

By incorporating specific slope angles, determined based on geotechnical factors such as soil strength, cohesion and angle of internal friction, roadside fill slopes are optimized for stability. The careful design of slope angles helps prevent excessive movement and maintains long-term stability, thus ensuring the safe and reliable functioning of the road.

4.6.2 Observations on the Failed Roadside Fill Slope

During the detailed site investigation conducted at the specified location along the roadway, several significant geotechnical issues have been observed as a result of the construction of a steep fill slope. These issues are directly linked to the progressively failing condition of the recently constructed roadside fill slope.

The following are prominent indicators of failure in a roadside fill slope: -

- *Visible significant cracks on the slope*

At the specified location, the presence of prominent and visible cracks on the surface of the fill slope is a clear indication of internal movement and instability. These cracks are observed as shown in Figure 4-36 A, in close proximity to the road surface, signaling potential issues with the slope’s stability.

Figure 4-36: A) Visible cracks on the surface of roadside fill slope B) Bulging on the surface of fill slope

- *The roadside fill slope exhibits noticeable bulging and deformation on its surface*

The observed bulging and deformations on the surface of roadside fill slopes as shown in Figure 4-36 B are additional prominent indicators of excessive movement and progressive failure of the slope. These deformations manifest in specific locations as areas of uneven or irregular slope surfaces, further exacerbating the slope's instability.

Figure 4-37: A) Deformations on the surface of fill slope B) Bulging on the surface of roadside fill slope

- *The surface of the road pavement shows notable signs of extensive cracking and settlement.*

At the specified station along the roadway as shown Figure 4-38, prominent cracks and settlements have been observed on the surface of the road pavement in close proximity to the roadside fill slope. These visible indications signify that the underlying slope has undergone movement or instability. The irregularities observed on the road pavement are a direct consequence of the ongoing failure of the roadside fill slope, which is progressively impacting the structural integrity of the road.

Figure 4-38: Significant signs of extensive cracking and settlements on the surface of road

- *Vegetation disturbance and slope debris accumulations*

At the specified location along the roadway, the surface of the roadside fill slope displays unusual patterns of vegetation growth and damages, including uprooted or leaning trees as shown in Figure 4-39 A. These signs indicate slope movement and progressive failure, suggesting displacement of the roadside fill slope.

Additionally, a substantial amount of debris accumulation has been observed at the base of the roadside fill slope Figure 4-39. Loose soil, weathered rocks and debris have accumulated at the site, indicating ongoing slope failure and recent slope movement.

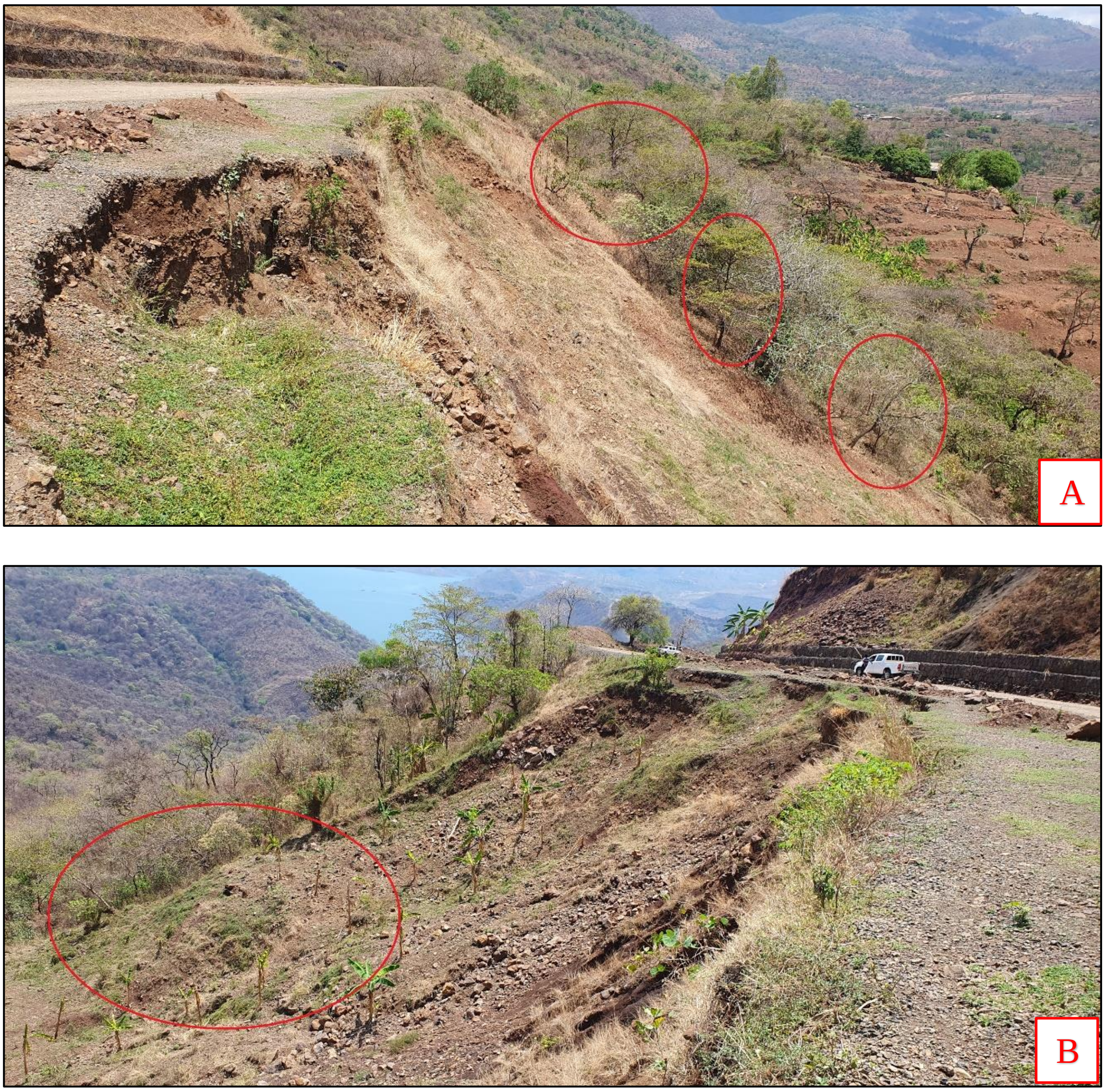

Figure 4-39: A) Leaning of tress B) Accumulation of debris

4.6.3 Effects of Failed Roadside Fill Slope

The failed roadside fill slope has significant effects on the performance of the project road, including the following:

- Roadway disruption: the failed roadside fill slope at station km 126+000 RHS along the roadway has resulted in a disruption of the normal roadway operations. As a consequence, the RHS driving lane of the carriageway has been partially closed, preventing simultaneous passage of two vehicles traveling in opposite directions. This disruption has caused traffic delays and inconveniences for road users, requiring them to wait for each other to pass through this section of the road.

- Safety hazards: the failure of the roadside fill slope presents significant safety hazards for both drivers and pedestrians. There is a risk of the slope collapsing onto the road, which can lead to accidents, injuries, or even fatalities. Additionally, the presence of falling debris or sliding materials from the failed slope poses an ongoing danger to road users.
- Road surface damaged: the movement and displacement of the roadside fill slope have resulted in significant damage to the newly constructed asphalt concrete road surface. This damage includes cracking, settlement, and deformations of the pavement, leading to an uneven driving surface. As a consequence, there has been an increase in vehicle wear and tear, as well as a reduction in riding quality for road users.
- Cost of repair and maintenance: the repair and maintenance of this failed roadside fill slope come at a substantial cost. Restoring the stability of the fill slope requires significant resources, including measures such as slope stability analysis, slope reconstruction and slope protection. The timing of this failure, occurring just after the completion of the road project, exacerbates the financial burden associated with the necessary repairs and maintenance.

4.6.4 Analysis of Failed Roadside Fill Slope

Laboratory tests were conducted on samples collected from embankment material. The tests were focused on evaluating the quality of the fill material used in constructing the embankment and its associated shear strength parameters. The following are lists of laboratory tests as depicted in Table 4-9, performed on samples taken from the fill material utilized in constructing the embankment:

Table 4-9: Lists of laboratory tests and their test method (126+000, fill material)

S. No.	Laboratory test	Test method
1	Atterberg limit tests (LL & PI)	AASHTO T-89 & 90
2	Particle size distribution	AASHTO T-88
3	Compaction characteristic (MDD & OMC)	AASHTO T-180 D
4	California bearing ratio (CBR)	AASHTO T-193
5	Density of soil in place	AASHTO T-191
6	Natural moisture content	AASHTO T-265
7	Direct shear test	ASTM D3080

From the laboratory tests assessing the material quality and shear strength parameters of the fill material, here are the obtained test results shown in Table 4-10.

Table 4-10: Laboratory test results (126+000, fill material)

S. No.	Laboratory test		Test result
1	Atterberg limit	LL	44.3%
		PL	26.7%
		PI	17.6%
2	Soil classification	AASHTO	A-2-7 (0)
		UCS	SM (Silty sand)
3	Compaction characteristic	MDD	1.820g/cm3
		OMC	16.35%
4	Density of soil in place		1.60g/cm3
5	California bearing ratio (CBR) @95% of MDD		12.18%
6	CBR swell		3.23%
7	Natural moisture content		20.01%
8	Direct shear test	Φ (°)	33.01
		C (Kpa)	10.6

4.6.4.1 Assessment of quality of the fill material

As per (TRL, 1999), the material quality requirement for embankment construction are, the material when placed and suitably compacted must be capable of standing at the appropriate designed slope both in the short and long term. Also when the fill material placed the material must be capable of resisting erosion by rainfall and surface run-off. Shear strength parameters, moisture-density relationship/compaction control, grading and plasticity are the controlling factors.

Also as per the road project technical specification ((ERA), 2014), the material requirements for the construction of embankments are as per the following specifications.

- Particle size

The material shall not contain any fragments with a maximum dimension exceeding 500mm after initial breakdown and the material shall be well graded with a maximum particle size of less than 150 mm after breakdown with a grid roller so that it can be readily placed within a 200 mm layer.

The result of the wet sieve analysis of the embankment material is as follows:

- Percent of coarse gravel (75 mm-19 mm): 0%
- Percent of fine gravel (19 mm-4.75 mm): 0%
- Percent of coarse sand (4.75 mm-2.0 mm): 39.90%

- Percent of medium sand (2.0 mm-0.425 mm): 16.60%
- Percent of fine sand (0.425 mm-0.075 mm): 16.20%
- Percent of fine-grained soil (<0.075 mm): 27.30%

The embankment material's sieve analysis shows a lack of larger gravel particles and a significant presence of sand and fine-grained soil. While sand offers some stability, the absence of coarse gravel and the high fine-grained soil content might affect drainage and load-bearing capacity. According to the project specifications, the fill material meets the necessary grading requirements.

- California bearing ratio (CBR)

Also as per the project material specification the fill material shall have a minimum soaked CBR of not less than 5% and a swell value not more than 2% (with surcharge rings) when determined in accordance with AASHTO T-193 at a modified AASHTO density of 95% of the maximum dry density determined in accordance with the requirements of AASHTO T-180 method D.

The result of the CBR and CBR swell test of the fill material as per the above mentioned test methods is as follows:

- CBR @ 95% of MDD 12.18%
- CBR swell with two surcharge weights 3.2%

Based on the specifications provided, the laboratory test results for the fill material exceed the minimum requirements for CBR but slightly surpass the maximum allowed CBR swell value. The CBR value of 12.18% is above the specified minimum of 5%, indicating good strength. However, the CBR swell value of 3.2% slightly exceeds the specified maximum of 2%, indicating a higher potential for volume change.

- Liquid limit (LL) and plasticity index (PI)

The project specifications sets that the fill material shall have a liquid limit not exceeding 60 and a plasticity index not exceeding 30 when determined in accordance with the requirements of AASHTO T-89 and T-90.

As per the recommend test methods the results of LL and PI are: -

- Liquid limit (LL) 44.3%
- Plasticity index (PI) 17.6%

The test results for the fill material fall within the project specifications for the liquid limit (LL) and plasticity index (PI). The LL value of 44.3% is below the specified maximum limit of 60% and the PI value of 17.6% is also within the specified limit of 30%. Hence, the fill material meets the project requirements for LL and PI as per the specified standards.

- Compaction

Lower layers of the embankment fill shall have minimum densities of 93% of modified AASHTO density and upper layer of the embankment fill shall be compacted to a minimum of 95% of modified AASHTO density, as stated by the project specification ((ERA), Standard Technical Specification and Method of Measurement, 2014).

Below are the laboratory test results: -

- Maximum dry density (MDD): 1.820g/cm^3 (95% of MDD = 1.729g/cm^3)
- In-place density: 1.60g/cm^3

The in-place density achieved for the embankment fill material is 1.60g/cm^3, which is lower than the specified minimum compaction requirements of 93% or 95% of the modified AASHTO density. Based on the laboratory results, the achieved density does not meet the specified minimum compaction standards for both the lower and upper layers of the embankment fill.

The lower achieved compaction density in the embankment material indicates that the road's foundation might lack the necessary stability and load-bearing capacity. This insufficiency in compaction could lead to premature failure of the road due to inadequate support.

Generally, the embankment fill material quality assessment shows:

- Particle size: lacks large gravel, high in sand and fine soil. Might hinder drainage and strength despite meeting grading standards.
- CBR test: exceeds minimum CBR at 12% but slightly surpasses max swell at 3.2%, hinting at possible volume change.
- LL and PI: meets liquid limit (LL) and plasticity index (PI) criteria.
- Compaction: achieved density below specified minimums, indicating potential instability and insufficient support, possibly contributing to premature road failure.

4.6.4.2 Assessment of stability of the constructed embankment

As depicted in Figure 4-40, the constructed side fill embankment at the specified station has a height of 9.43m above the original ground level, featuring a steep fill slope of 62° from the horizontal. Upon obtaining the shear strength parameters for the fill material, this fill slope geometry was simulated for slope stability analysis. (The green line indicates the fill material finished level and the red line indicates that the original profile of the ground). Based on the topographic characteristics of the roadside fill slope mentioned above, a stability analysis was conducted using the material properties of each section. A slip surface resembling the actual failed roadside fill slope was chosen, guided by the measured length of failed fill material. Detailed site investigations revealed that the carriageway had become narrower due to the fill material's failure, with around 2 meters of the original fill slope collapsing. Utilizing data from multiple slip surfaces, the one with a failed length of 1.81 meters at the top was selected for analysis.

The analysis revealed a factor of safety below the critical minimum value of 1.0. A factor of safety of 0.949 by Bishop limit equilibrium analysis as shown in Figure 4-41.

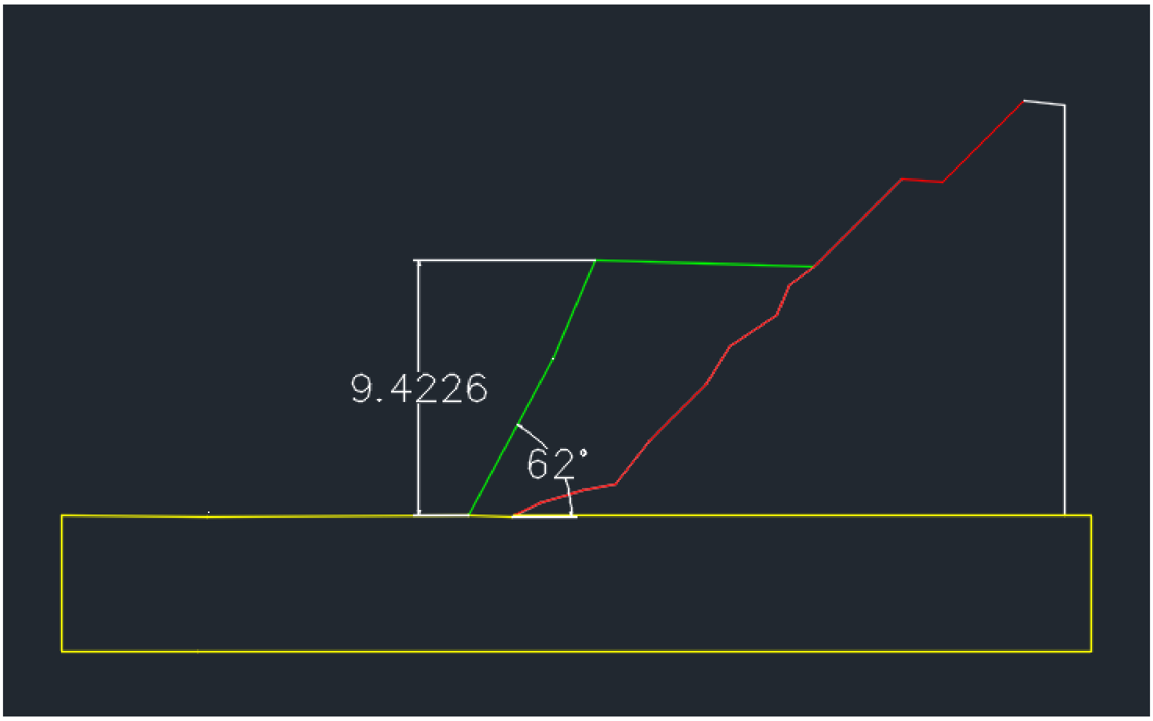

Figure 4-40: Geometry of roadside fill slope (km 126+000 LHS)

In both cases, the factors of safety are less than 1.0, signaling that the current slope conditions do not provide an adequate margin of safety against failure. It's crucial to reevaluate the slope design, consider potential contributing factors and implement appropriate remedial measures to ensure the stability of the fill slope.

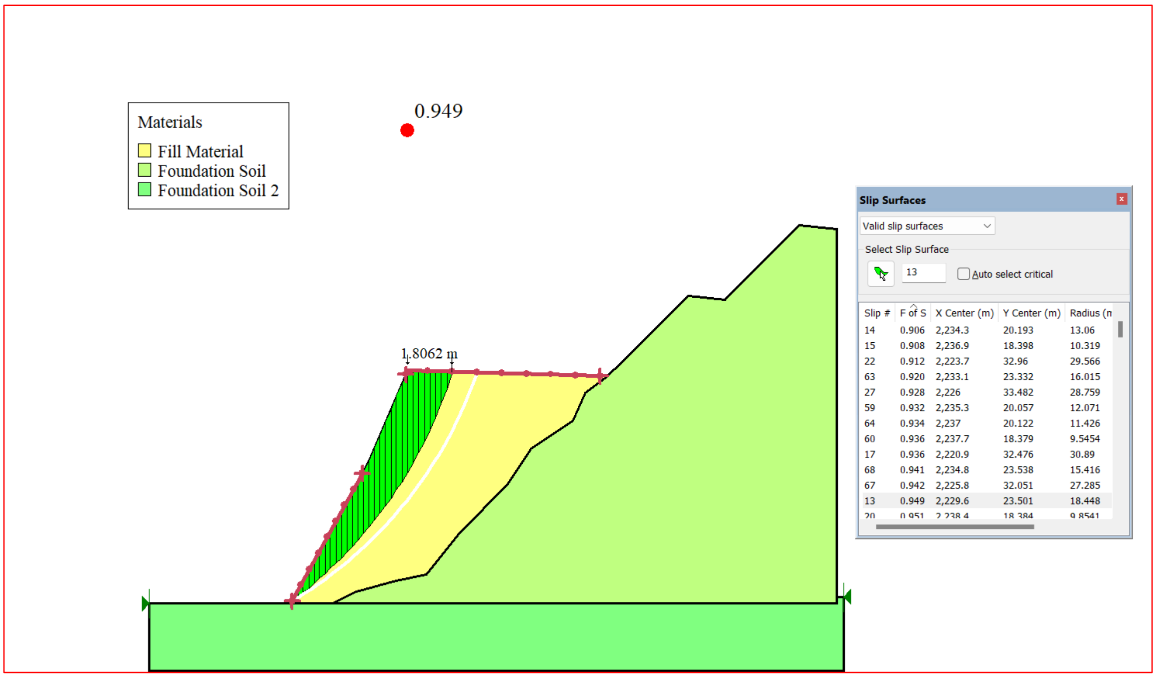

Figure 4-41: Fill slope stability analysis (Bishop)

4.7 History and Extent of Side Fill Slope Failure

The analysis of the history of the failed roadside fill slope aimed to understand the previous topographical characteristics of the road at this particular station. Figure 4-42, depicts a seemingly stable road side fill slope, while Figure 4-43, illustrates the failure of the road side fill attributed to the steep angle of the fill slope. The extent of this fill slope failure spans from station km 126+090 to 125+980, covering approximately 110 meters.

Currently, no remedial measures have been implemented, rendering it impossible for two vehicles to use the road simultaneously. The narrowed road width, resulting from the failure of the side fill, necessitates one vehicle to wait for the other to pass.

Figure 4-42: Google earth historical imagery dated February, 2021 at km 126+000 (LHS)

Figure 4-43: Google earth historical imagery dated October, 2014 at km 126+000 (LHS)

4.8 Remedial Measures for Failed Roadside Fill Slope

To address the concerns with the fill slope geometry, the initial slope of 62° will be adjusted to a gentler inclination by reducing it by 3° as shown in Figure 4-44. Each modified fill slope geometry will undergo a slope stability analysis and the one achieving a factor of safety of 1.5 will be considered the safest configuration for the roadside fill in this road.

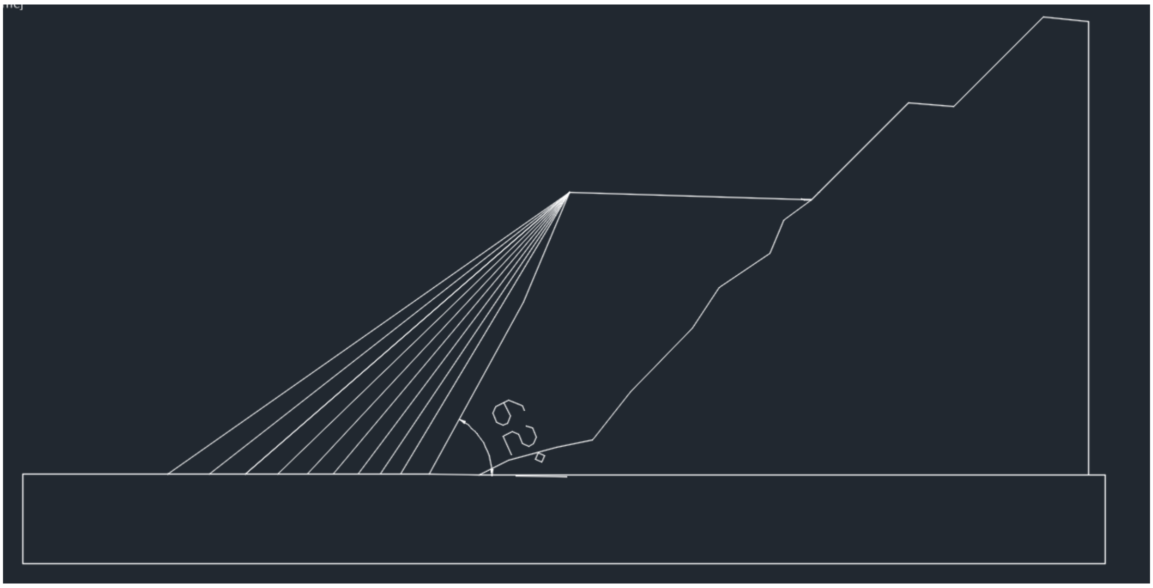

Figure 4-44: Trail fill slope geometries

Numerous trial slope configurations underwent a stability analysis for the fill slope, aiming to identify a safe geometry through the process of slope flattening.

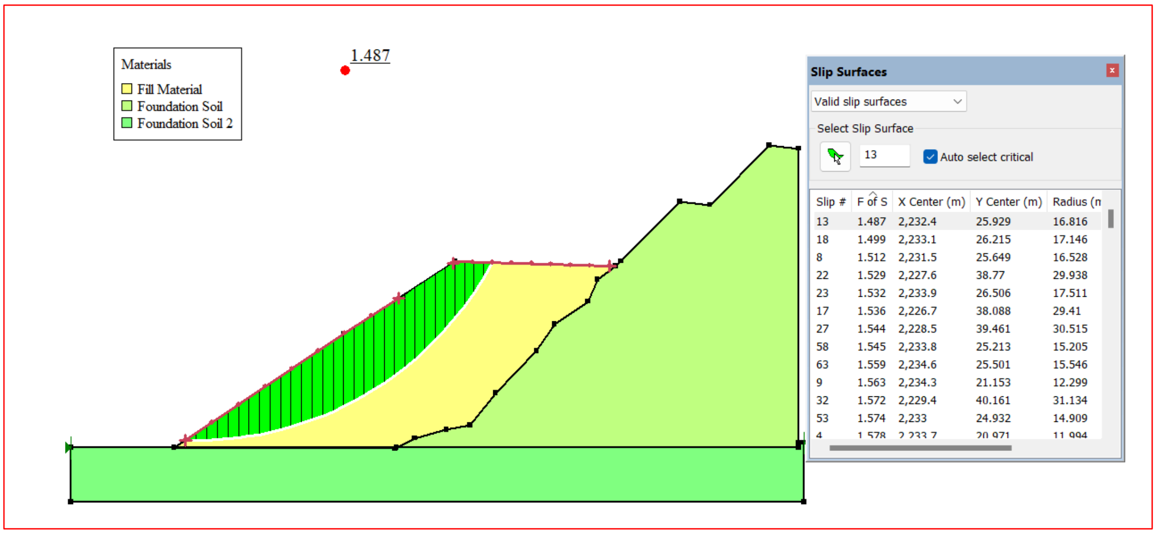

Figure 4-45: Safe fill slope geometry (Bishop)

The fill slope geometry, aiming for a factor of safety of 1.5, is illustrated below with a fill slope angle of 33° from the horizontal. Bishop analyses indicate factors of safety of 1.487 as shown in Figure 4-45, signifying a safe condition. Consequently, the recommended geometry for this roadside fill is 33° from the horizontal.

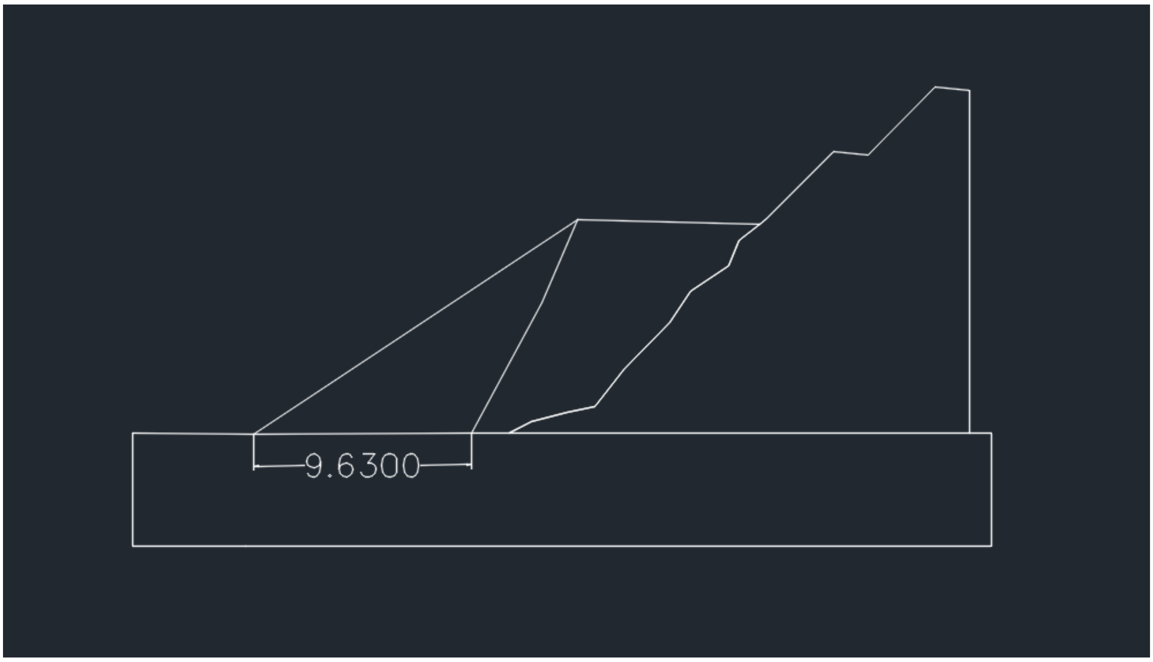

Figure 4-46: Safe fill slope geometry with additional space

Examining the cross section of the roadway fill slope above, the initial angle of 62° proved unsafe, resulting in failure. The slope stability analysis identified 33° as the safest angle for the fill slope. However, adopting this gentler slope requires additional space, approximately 9.63 m as shown in Figure 4-46. Considering the right-of-way limitations, stakeholders assert that the initially provided slope angle was the maximum feasible within the restricted area. Hence, an alternative approach is necessary to ensure the safety of the fill slope.

In areas constrained by right-of-way limitations for fill slope or embankment construction, the need to build the fill slope within confined space may necessitate adopting a steep slope angle. However, constructing the fill slope with a steeper inclination poses stability risks. To address this, reinforcing the fill soil emerges as the optimal solution. This approach ensures stability even with a steeper slope, making it the preferred remedial measure for the identified failed section, as detailed in the subsequent analysis.

After trying with various configurations of geo-grids to rebuild the failed roadside fill slope using the available fill material, a specific arrangement emerged. This involved employing a set of six geo-grids with a vertical spacing of 1.5 meters and variable lengths. The stability analysis results a factor of safety of 1.544 by Morgenstern-Price analysis method as shown in Figure 4-47 and 1.500 by Bishop analysis method as shown in Figure 4-48.

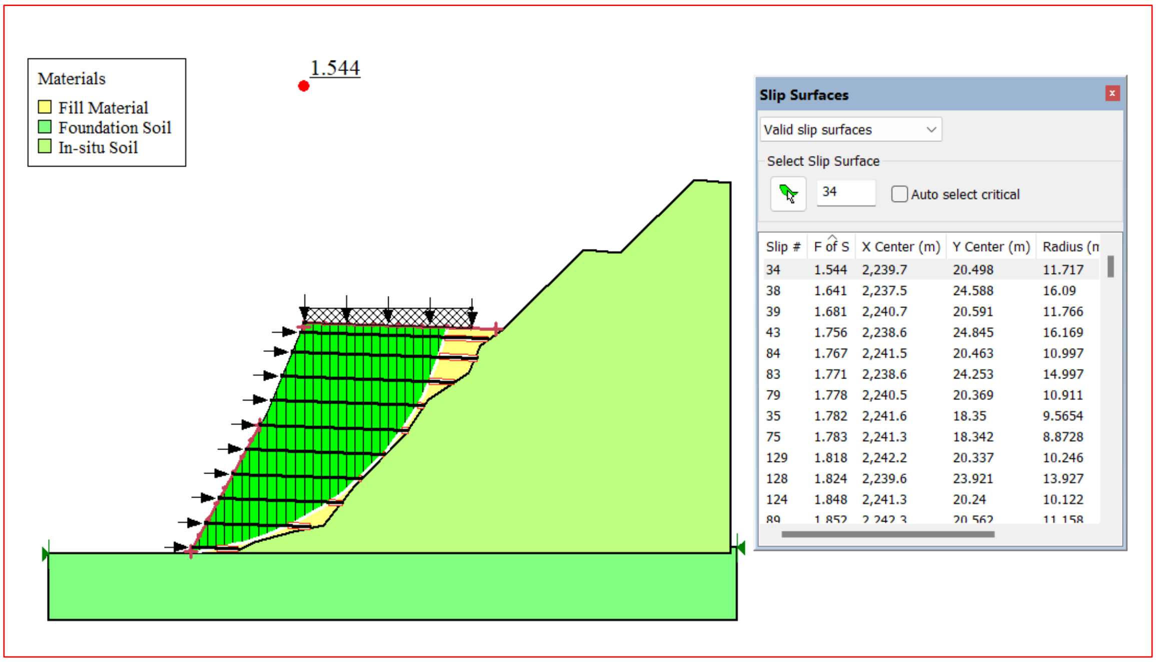

Figure 4-47: Stability of the fill slope with geo-grids (Morgenstern-Price)

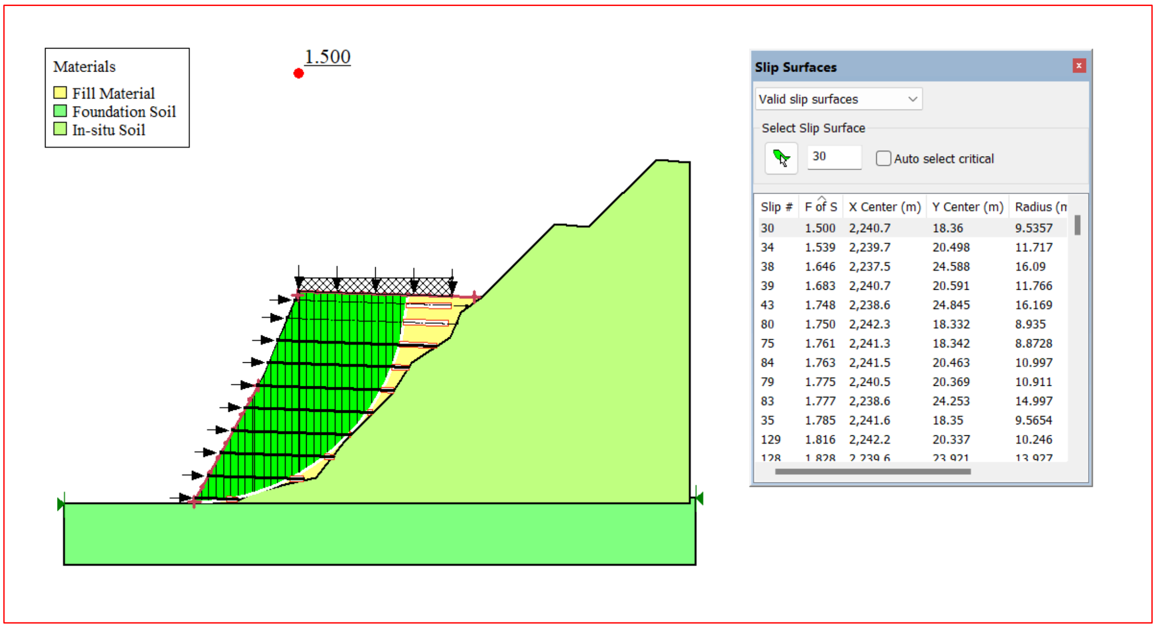

Figure 4-48: Stability of the fill slope with geo-grids (Bishop)

4.8.1.1 Details of geo-grids

Geo-grids form a distinct category of geo-synthetics designed for reinforcement. These products are distinguished by their relatively high tensile strength and a consistently distributed pattern of large apertures, which are openings between the longitudinal and transverse elements. The openings facilitate direct contact between soil particles on both sides of the installed sheet, enhancing the interaction between the geo-grid and certain soils. Additionally, these apertures ensure vertical drainage in a reinforced free-draining soil. Material specification of geo-grid (side fill soil reinforcement):

From HUESKER manufacturer PLC, selecting Fortrac T soil reinforcement grid which is made of high modulus, low creep polyester yarns with a polymeric protective coating. Fortrac T has been the standard in geosynthetic raw materials for more than 30 years. From the Fortrac T data table selecting Fortrac 150T type of geogrid material with ultimate tensile strength of 150 kN/m and mesh size of 25x25 mm. The details of these geo-grids is presented under bellow Table 4-11.

Table 4-11: Details of geogrid reinforced road side fill slope

Fortrac 150T geo-grid no.	**Length (m)**	**Spacing (m)**	**Direction (°)**	**Tensile capacity (kN/m)(Fortrac 150T)**
1	1.8	1.0	178°	150 kN/m
2	4.2	1.0	178°	150 kN/m
3	5.0	1.0	178°	150 kN/m
4	5.2	1.0	178°	150 kN/m
5	5.6	1.0	178°	150 kN/m
6	6.0	1.0	178°	150 kN/m
7	6.2	1.0	178°	150 kN/m
8	7.0	1.0	178°	150 kN/m
9	7.5	1.0	178°	150 kN/m
10	7.6	1.0	178°	150 kN/m

Table 4-12: Fortrac T geo-grids manufacturer specification

Fortrac T - Datentabelle

Type of Material	Raw Material	Ultimate Tensile Strength kN/m		Strain at Nominal Tensile Strength (%)		Standard Roll Dimensions (m)		Meshsize (mm)
		MD	CMD	MD	CMD	Width	Length	
Fortrac 35T	PET	≥ 35	≥ 20	≤ 10	≤ 10	5,0	200	25x25
Fortrac 55T	PET	≥ 55	≥ 20	≤ 10	≤ 10	5,0	200	25x25
Fortrac 65T	PET	≥ 65	≥ 20	≤ 10	≤ 10	5,0	200	25x25
Fortrac 80T	PET	≥ 80	≥ 20	≤ 10	≤ 10	5,0	200	25x25
Fortrac 150T	PET	≥ 150	≥ 20	≤ 10	≤ 10	5,0	200	25x25
Fortrac 200T	PET	≥ 200	≥ 20	≤ 10	≤ 10	5,0	200	25x25
Fortrac R 300/50-30T	PET	≥ 300	≥ 50	≤ 10	≤ 10	5,0	100	30x30
Fortrac R 400/50-30T	PET	≥ 400	≥ 50	≤ 10	≤ 10	5,0	100	30x30
Fortrac R 600/50-30T	PET	≥ 600	≥ 50	≤ 10	≤ 10	5,0	100	30x30

To sum up, address concerns regarding the failed roadside fill slope, a remedial measure has been proposed. The initial slope of 62° will be adjusted to a gentler inclination by reducing it by 3°. Each modified fill slope geometry undergoes a stability analysis, with the safest configuration achieving a factor of safety of 1.5.

Various trial slope configurations were analyzed to identify a safe geometry through slope flattening. The recommended safe fill slope geometry, with a 33° angle from the horizontal, has been verified by both Morgenstern-Price and Bishop analyses, yielding factors of safety of 1.544 and 1.500, respectively.

Considering the right-of-way limitations, stakeholders suggest that the initially provided slope angle was the maximum feasible within the restricted area. In lieu of a gentler slope, reinforcing the fill soil with geo-grids emerges as the optimal solution, ensuring stability even with a steeper inclination.

After trying with different configurations, a specific arrangement of ten geo-grids with a vertical spacing of 1.0 meter and variable lengths was found effective. The selected Fortrac T geo-grid, made of high modulus, low creep polyester yarns, with an ultimate tensile strength of 150kN/m and mesh size of 25x25mm, is recommended for reinforcing the roadside fill slope.

Table 4-13: General summary of analyses and findings

S. No.	Station	General summary
1	105+000 (RHS)	Identified geotechnical problem: • Uncontrolled seepage from the side slope in the pavement. Observed failure signs: • wet patches and visible outflows. • presence of vigorous growth or hydrophilic vegetation. • pumping and tension cracks. Failure causes: • Seepage coming from high ground surface in to the pavement layers, high permeability of slope soil. Proposed remedial measure: • Installing longitudinal interceptor drains along RHS of the road. Drains consist of perforated pipes surrounded by aggregates and geotextiles.
2	134+760 (RHS)	Identified geotechnical problem: • Soil erosions from the slope of the roadway cut in the pavement Observed failure signs: • Formations of rills and gullies on the surface of a slope. • Exposed plant roots and vegetation loss on the surface a slope. • Accumulation of sediments and tiling of constructed structures. • Formation of cracks, fissures and soils on the slope becoming lose, crumble and saturated. Failure causes: • Bare slope surface (lack of vegetation), longer slope length, steeper slope, heavy rainfall of the area and slope soil nature (erodible soil) Proposed remedial measure:

		• Re-vegetation and placement of erosion control blankets/geotextiles on the slope surface
3	114+000 – 114+500 (FW)	Identified geotechnical problem: • Unsuitable/problematic highway subgrade soil. Observed failure signs: • Newly constructed asphalt concrete road exhibits surface cracking. • Pavement surface displays settlements and surface deformations. Failure causes: • Untreated highway subgrade soil. • Lack of moisture barrier for subgrade soil. • Moisture sensitivity of the subgrade soil. • Saturation of subgrade soil. • Very low load bearing capacity of the subgrade soil. • Higher free swell and significant shrinkage character of the in-situ subgrade soil. • Exposing the subgrade soil for moisture fluctuation. Proposed remedial measure: • Maintaining consistent moisture levels in the soil throughout the road's design period by construction of vertical moisture barriers.
4	100+300 (RHS)	Identified geotechnical problem: • Roadway cut slope failure. Observed failure signs: • Slumping • Land slide • Debris fall Failure causes: • Steeper geometry of cut slope during construction.

<table>
<tr><td></td><td></td><td>Proposed remedial measure:
Slope flattering with introduction of benches and grassing.</td></tr>
<tr><td>5</td><td>126+000
(RHS)</td><td>Identified geotechnical problem:
Roadway side fill slope failure.
Observed failure signs:
Visible significant cracks on the fill slope surface.
Roadside fill slope exhibits noticeable bulging and deformation on its surface.
The surface of the road pavement shows notable signs of extensive cracking and settlement.
Vegetation disturbance and slope debris accumulations.
Failure causes:
Insufficient compaction of the fill material, its density does not meet the specified minimum compaction standards.
Steeper fill slope construction due to limited space of ROW.
Proposed remedial measure:
Geo-grid reinforced fill slope.</td></tr>
</table>

CHAPTER 5 CONCLUSIONS AND RECCOMENDATIONS

5.1 Conclusions

In conclusion, the Omo River-Tercha Road Project has experienced unexpected premature failures shortly after its opening for traffic use, contradicting the initially planned 20-years design life. The in-depth investigation has identified the major causes for these premature failures along the road corridor, which include:

- Lack of subsurface drainage: Despite clear indicators of subsurface water seepage from nearby hillsides toward the pavement system, no measures were taken to intercept this seepage before reaching into the pavement materials.
- Vegetation cover removal: During roadway side cutting, removal of existing vegetation cover exposed the face of the roadway cut slope to heavy rainfall, causing continuous erosion. Compounded by the extended length, steeper slope and the nature of the soil on the roadway cut slope, this erosion is further accelerated.
- Lack of thorough investigation into subgrade soil suitability: Failure to recognize and address problematic highway subgrade soil with characteristics such as high erratic, sensitivity to moisture and extremely low load-bearing capacity.
- Steep slope construction: Throughout the project route, the majority of slopes (both cut and fill) have been built with steep geometry, leading to considerable and progressive slope instability.
- Design-Build contract type issues: The contractor submitted designs that overlooked geotechnical issues and included steeper slopes, likely driven by a desire for cost reduction.
- Minimal involvement of geotechnical engineers: Limited engagement of geotechnical engineers in both design and construction supervision stages has contributed to project issues, including premature failures, cost overruns, delays and claims.

5.2 Recommendations

To maintain the newly built road, it is recommended to address the identified geotechnical issues promptly. Applying the proposed solutions as soon as possible is important to prevent further damage to the road and minimize maintenance costs. The following are the recommendations:

- Develop subsurface drainage: Establish longitudinal interceptor drains for redirecting seepage from the adjacent hill, preventing it from reaching the pavement system.
- Re-vegetation and soil erosion blankets: These measures prevent exposed soil on the cut slope from eroding and stabilize it, reducing the impact of rainfall and enhancing slope integrity.
- Prevent contact between problematic subgrade soil with moisture: Providing longitudinal vertical moisture barriers on both sides (LHS & RHS) to prevent contact between the subgrade soil and moisture, ensuring consistent moisture content in the subgrade soil.
- Apply slope flattering: For steeper roadway cut slope, in order to decrease the driving mass and to make it stable slope it is recommend to flatter the slope by removing the soil up to a geometry with acceptable factor of safety against stability.
- Incorporate geo-grids into the construction of roadside fill slopes: Integrate geo-grids into roadside fill slope construction to stabilize and reconstruct it within limited space.

Finally, all stakeholders involved in this road project as well as researchers conducting further studies are recommended to conduct detailed investigations to address premature failures.

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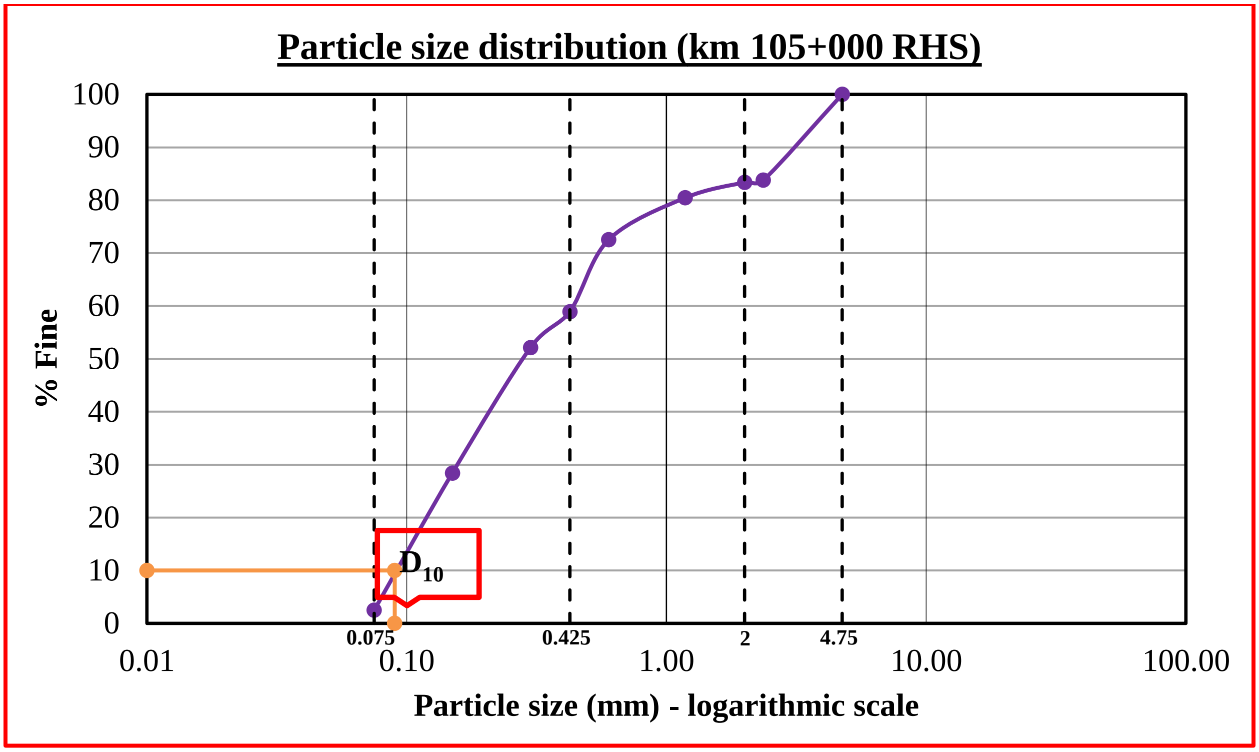
APPENDIX A LABORATORY TEST RESULTS

Station: 105+000 RHS

Particle size distribution		
Description of soil: Reddish brown silty sand soil		
Location: km 105+000 RHS	Coordinate	N: 6.97243°
Depth: 3.0m		E: 37.308457°
Test method: AASHTO T-88		
Date: 18/10/2023		

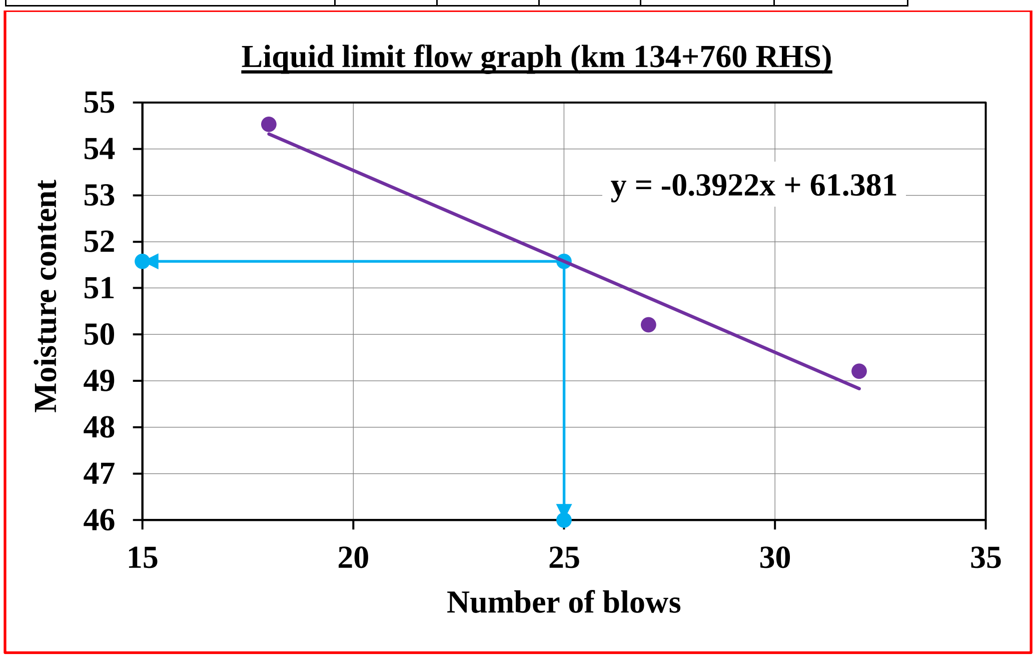
A	B	C	D		E	F	G	H
Sieve no.	Opening (mm)	Mass retained (grams) Mr (Trial1)	Mass retained (grams) Mr (Trial2)	Mass retained (grams) Mr (Trial3)	Mass retained (grams) Mr (Average)	% Retained (100*Mr/Mt)	∑ (% Retained) (∑ column D)	% Finer (100 - column G)
4	4.75	0	0	0	0.0	0.00	0.00	100.00
8	2.36	20	49	48	39.0	16.23	16.23	83.77
10	2	1	1	1	1.0	0.42	16.64	83.36
16	1.18	6	8	7	7.0	2.91	19.56	80.44
30	0.6	23	22	12	19.0	7.91	27.46	72.54
40	0.425	37	37	24	32.7	13.59	41.05	58.95
50	0.3	21	14	14	16.3	6.80	47.85	52.15
100	0.15	56	61	54	57.0	23.72	71.57	28.43
200	0.075	61	68	58	62.3	25.94	97.50	2.50
	Pan	7	3	8	6.0	2.50	100.00	0.00
	SUM	232	263	226	240.3			
	M_t =	232	263	226	240.3			


Particle size distribution (km 105+000 RHS)
% Fine
100
90
80
70
60
50
40
30
20
10
0
D10
0.075
0.425
2
4.75
0.01
0.10
1.00
10.00
100.00
Particle size (mm) - logarithmic scale

Station: km 134+760 RHS

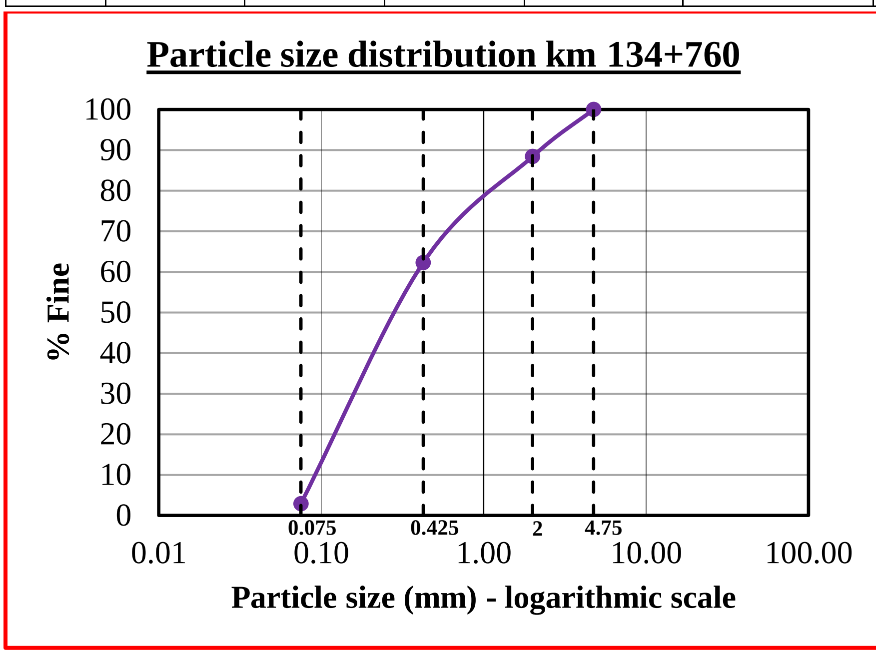
Atterberg limit test		
Description of soil: Greyish silty sand soil		
Location: km 134+760 RHS	Coordinate	N: 6.824012°
Depth: 1.0m		E: 37.297353°
Test method: AASHTO T-89 & 90		
Date: 10/10/2023		

	Liquid limit			Plastic limit	
Number of Blows	32	27	18		
Can Number	25A	29	17	B5	12
Wet Weight + Can	47.41	49.61	47.95	14.99	14.61
Dry Weight + Can	34.99	36.22	34.53	13.64	13.43
Weight of Can	9.75	9.55	9.92	9.18	9.53
Weight of Water	12.42	13.39	13.42	1.35	1.18
Dry Weight	25.24	26.67	24.61	4.46	3.90
Moisture Content	49.21	50.21	54.53	30.27	30.26



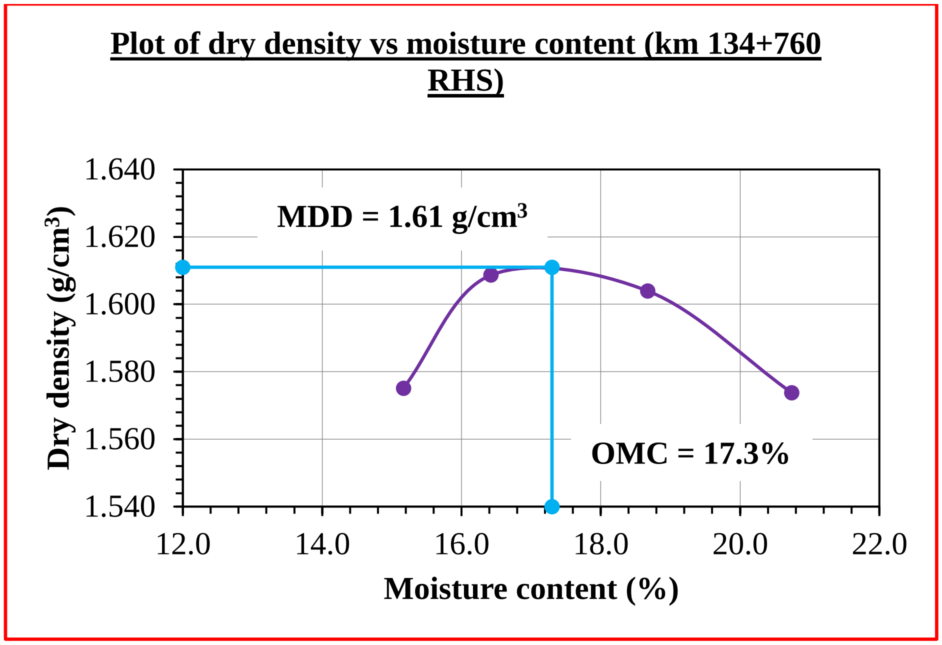
Liquid limit (%)	**51.6**
Plastic limit (%)	**30.3**
Plasticity index (%)	**21.3**

Particle size distribution							
Description of soil: Greyish silty sand soil							
Location: km 134+760 RHS					Coordinate	N: 6.824012°	
Depth: 1.0m						E: 37.297353°	
Test method: AASHTO T-88							
Date: 10/10/2023							
A	B	C	D	E	F	G	H
Sieve no.	Opening (mm)	Mass retained (grams) M_r (Trial 1)	Mass retained (grams) M_r (Trial 2)	Mass retained (grams) M_r (Average)	% Retained ($100*M_r/M_t$)	∑ (% Retained) (∑ column D)	% Finer (100 - column G)
4	4.750	0	0	0.0	0.00	0.00	100.00
10	2.000	31	21	26.0	11.61	11.61	88.39
40	0.425	55	62	58.5	26.12	37.72	62.28
200	0.075	128	138	133.0	59.38	97.10	2.90
Pan		7	6	6.5	2.90	100.00	0.00
	SUM	221	227	224.0	100.0		
	M_t =	221	227	224.0			



Modified proctor		
Description of soil: Greyish silty sand soil		
Location: km 134+760 RHS	Coordinate	N: 6.824012°
Volume of mold: 944 cm^3		E: 37.297353°
Weight of hammer: 4.54 kg	Test method: AASHTO T-180 (Method-D)	
Number of blows/layer: 56	Number of layer: 5	

Density	Trail no.	**1**	**2**	**3**	**4**	
	Added water cc	4%	6%	8%	10%	
	Weight of soil + mold (g) W_1	8,600	8,725	8,790	8,783	
	Weight of mold (g) W_2	4747	4747	4747	4747	
	Volume of mold (cm^3) V	2124	2124	2124	2124	
	Weight of wet soil (g) $W_3 = W_1\text{-}W_2$	3,853	3,978	4,043	4,036	
	Wet density of soil (g/cm^3) $W_d = W_3/V$	1.814	1.873	1.903	1.900	NMC
Moisture	Container no.	A18	C3	A8	31	E5
	Wet soil + container (g) a	264.18	289.87	288.39	266.98	266.88
	Dry soil + container (g) b	234.21	254.39	248.86	227.60	245.78
	Weight of container (g) c	36.61	38.38	37.20	37.75	37.24
	Weight of water (g) e = a-b	30.0	35.5	39.5	39.4	21.1
	Weight of dry soil (g) d =b-c	197.6	216.0	211.7	189.9	208.5
	Moisture content (%) m= (e/d)*100	15.17	16.43	18.68	20.74	10.12
Dry density of soil (g/Cm3) $D_d = W_d/(100+m)*100$		1.575	1.609	1.604	1.574	



MDD = 1.61 g/cm^3

OMC = 17.3%

Density of soil in place		
Description of soil: Greyish silty sand soil		
Location: km 134+760 RHS		
Depth: 1.0m	Coordinate	N: 6.824012°
Test method: AASHTO T-191		E: 37.297353°
Density of sand, g/cm^3: 1.316		
Date: 10/10/2023		

Depth of hole, cm	14.5
A. Total weight of sand in container, g	7761
B. Sand remain in container, g	3135
C. Weight of used sand (A-B), g	4626
D. Sand in cone, g	931.67
E. Sand in hole (C-D), g	3694
F. Density of sand, g/cm^3	1.316
G. Volume of hole, (E/F) cm^3	2807
H. Weight of wet soil, g	4250
I. Wet density of soil (H/G), g/cm^3	**1.51**

Determination of water content		
Description of soil: Greyish silty sand soil		
Location: km 134+760 RHS		
Depth: 1.0m	Coordinate	N: 6.824012°
Test method: AASHTO T-265		E: 37.297353°
Date: 10/10/2023		

Item	Test no.	
	1	2
Can No.	14	16
Mass of can, W_1 (g)	40.2	37.43
Mass of can + wet soil, W_2 (g)	336.54	333.63
Mass of can + dry soil, W_3 (g)	249.22	247.49
Mass of moisture, W_2 - W_3 (g)	87.32	86.14
Mass of dry soil, W_3 - W_1 (g)	209.02	210.06
Moisture content, $w(\%) = \left(\frac{W_2 - W_3}{W_3 - W_1}\right)x100$	41.78	41.01
Average moisture content, w = 41.39%		

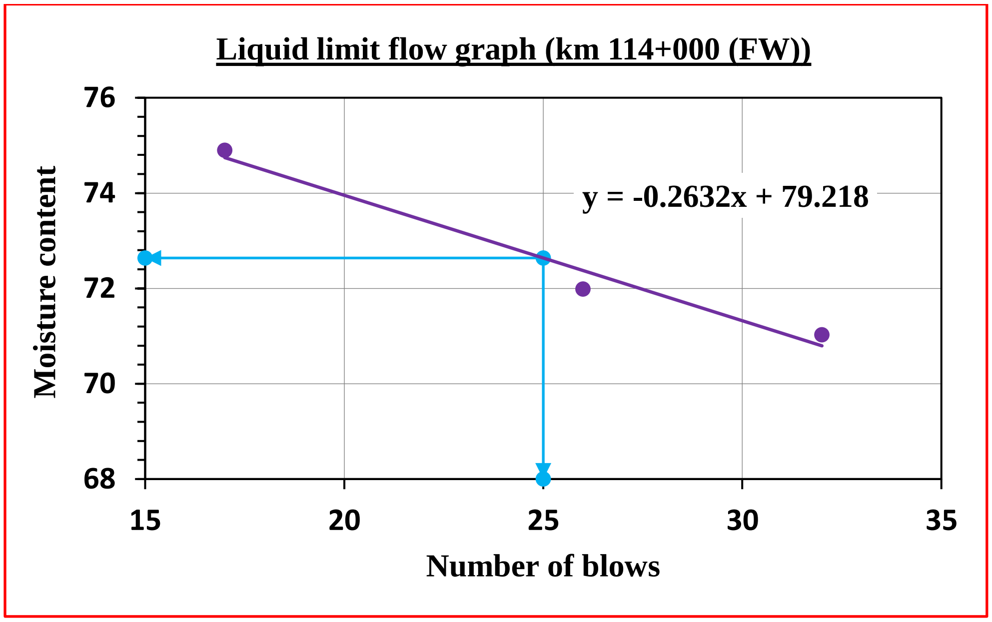
Dispersive characteristic of soil		
Description of soil: Greyish silty sand soil		
Location: km 134+760 RHS		
Depth: 1.0m	Coordinate	N: 6.824012°
Test method: ASTM D-4221		E: 37.297353°
Date: 10/10/2023		

Station	Sample Depth (m)	Field Data		ASTM D3080	
		NMC (%)	In-place density (g/cm^3)	C (kPa)	ϕ (°)
100+300 (Upper layer)	5	32.53	1.4	34	24
100+300 (Lower layer)	10	29.65	1.54	30	26
126+ 000 (Fill material)	4	20.1	1.6	33	10.6
134+760 (RHS)	**1**	**41.39**	**1.51**	**ASTM D4221**	
				53%	

Station: km 114+000 (FW)

Atterberg limit test		
Description of soil: Light yellowish silty clay soil		
Location: km 114+000 FW	Coordinate	N: 6.924824°
Depth: 1.0m		E: 37.303474°
Test method: AASHTO T-89 & 90		
Date: 12/10/2023		

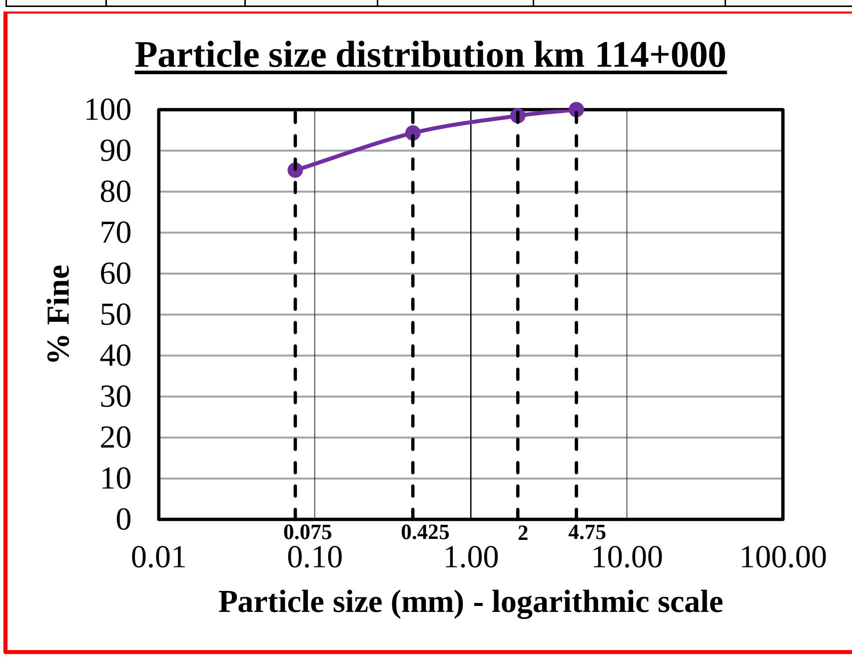
	Liquid limit			Plastic limit	
Number of Blows	32	26	17		
Can Number	40	8	17	31	20
Wet Weight + Can	47.41	48.72	48.63	14.62	14.24
Dry Weight + Can	31.72	32.35	32.04	13.19	12.93
Weight of Can	9.63	9.61	9.89	9.55	9.57
Weight of Water	15.69	16.37	16.59	1.43	1.31
Dry Weight	22.09	22.74	22.15	3.64	3.36
Moisture Content	71.03	71.99	74.90	39.29	38.99



Liquid limit	**72.6%**
Plastic limit	**39.1%**
Plasticity index	**33.5%**

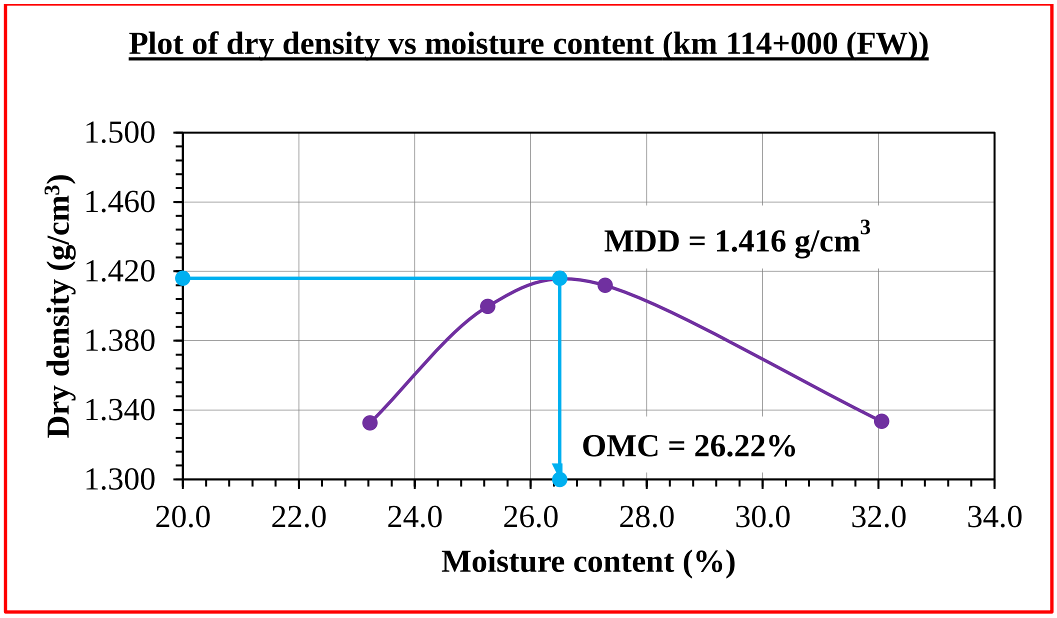
Particle size distribution		
Description of soil: Light yellowish silty clay soil		
Location: km 114+000 FW	Coordinate	N: 6.924824°
Depth: 1.0m		E: 37.303474°
Test method: AASHTO T-88		
Date: 12/10/2023		

A	B	C	D	E	F	G	H
Sieve no.	Opening (mm)	Mass retained (grams) M_r (Trial1)	Mass retained (grams) M_r (Trial2)	Mass retained (grams) M_r (Average)	% Retained ($100*M_r/M_t$)	∑ (% Retained) (∑ column D)	% Finer (100 - column G)
4	4.750	0	0	0.0	0.00	0.00	100.00
10	2.000	8	7	7.5	1.50	1.50	98.50
40	0.425	23	19	21.0	4.20	5.70	94.30
200	0.075	49	42	45.5	9.10	14.80	85.20
Pan	Pass	420	432	426.0	85.20	100.00	0.00
	SUM	500	500	500.0	100.0		
	M_t =	500	500	500.0			



Modified proctor		
Description of soil: Light yellowish silty clay soil		
Location: km 114+000 FW	Coordinate	N: 6.924824°
Volume of mold: 944 cm^3		E: 37.303474°
Weight of hammer: 4.54 kg	Test method: AASHTO T-180 (Method-D)	
Number of blows/layer: 56	Number of layer: 5	

Density	Trail no.		**1**	**2**	**3**	**4**	
	Added water cc		4%	6%	8%	10%	
	Weight of soil + mold (g)	W_1	8,234	8,470	8,563	8,486	
	Weight of mold (g)	W_2	4746	4746	4746	4746	
	Volume of mold (cm^3)	V	2124	2124	2124	2124	
	Weight of wet soil (g)	$W_3 = W_1 - W_2$	3,488	3,724	3,817	3,740	
	Wet density of soil (g/cm^3)	$W_d = W_3/V$	1.642	1.753	1.797	1.761	NMC
Moisture	Container no.		A9	C3	C9	C1	A14
	Wet soil + container (g)	a	278.92	265.18	274.14	264.29	270.43
	Dry soil + container (g)	b	233.59	219.44	223.42	209.37	234.05
	Weight of container (g)	c	38.45	38.35	37.51	38.02	38.26
	Weight of water (g)	e = a-b	45.3	45.7	50.7	54.9	36.4
	Weight of dry soil (g)	d =b-c	195.1	181.1	185.9	171.4	195.8
	Moisture content (%)	m= (e/d)*100	23.23	25.26	27.28	32.05	18.58
Dry density of soil (g/cm^3) $D_d = W_d/(100+m)*100$			1.333	1.400	1.412	1.333	



MDD = 1.416 g/cm³

OMC = 26.22%

California bearing ratio (CBR)		
Description of soil: Light yellowish silty clay soil		
Location: km 114+000 FW		
Depth: 1.5m	Coordinate	N: 6.924824°
Test method: AASHTO T-193		E: 37.303474°
Date: 12/10/2023		

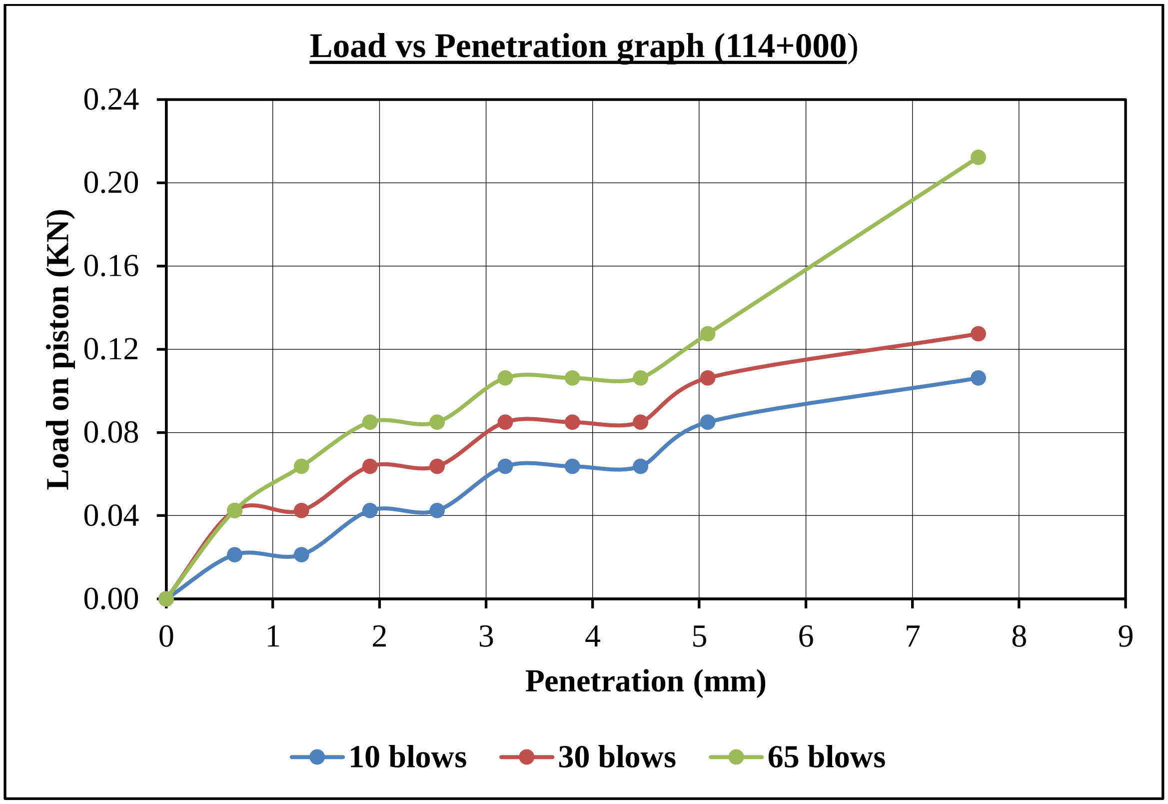
Soaking condition		10 Blows		30 Blows		65 Blows	
		Before	After	Before	After	Before	After
Mold no.		A1		F2		B2	
Weight of soil + mold (g)	W_1	9524	10265	10489	10867	9688	10179
Weight of mold (g)	W_2	6259	6259	6780	6780	5733	5733
Volume of mold (cm^3)	V	2124	2124	2124	2124	2124	2124
Weight of wet soil (g)	$W_3 = W_1 - W_2$	3265	4006	3709	4087	3955	4446
Wet density of soil (g/cm3)	$W_d = (W_3/V)$	1.54	1.89	1.75	1.92	1.86	2.09
Dry density of soil (g/cm3)	$D_d = W_d/(100+m)*100$	1.21	1.21	1.38	1.44	1.45	1.59

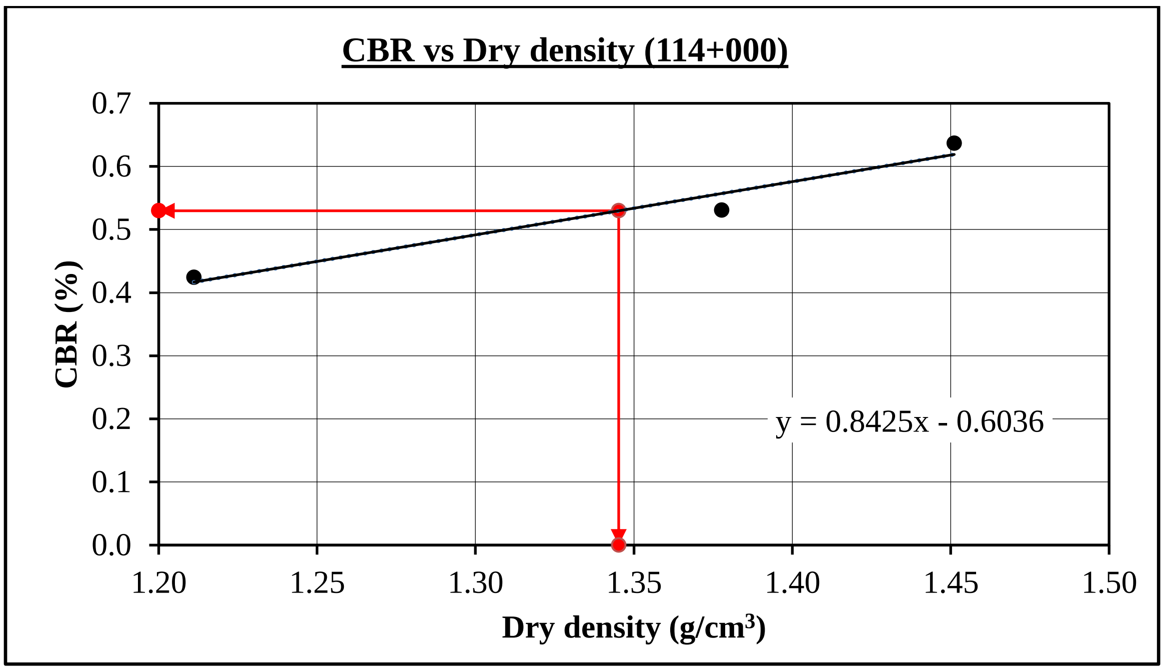
Moisture determination										
Soaking condition		**10 Blows**			**30 Blows**			**65 Blows**		
		Before	After		Before	After		Before	After	
			TOP 1 in.	Avg.		TOP 1 in.	Avg.		TOP 1 in.	Avg.
Container no.		C5	A14		A17	14		B15	W54	
Wet soil + container (g)	a	254.75	268.50		259.80	256.04		262.79	329.96	
Dry soil + container (g)	b	208.68	186.30		212.55	200.58		213.20	259.19	
Weight of container (g)	c	37.58	38.20		35.93	37.24		38.08	38.21	
Weight of water (g)	d = a - b	46.07	82.20		47.25	55.46		49.59	70.77	
Weight of dry soil (g)	e = b - c	171.10	148.10		176.62	163.34		175.12	220.98	
Moisture content (%)	m =(d/e)*100	26.93	55.50	41.21	26.75	33.95	30.35	28.32	32.03	30.17
Avg. mois. content. (%)										

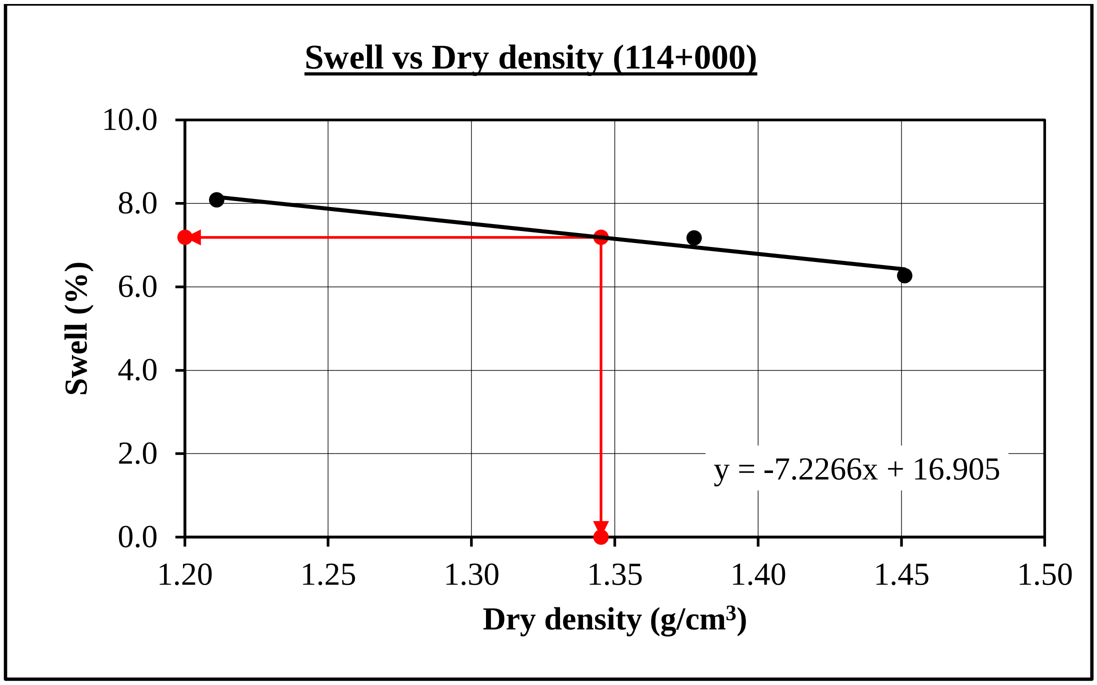
Penetration test data													
Penetration		**10 Blows**				**30 Blows**				**65 Blows**			
(mm)	**(Div.)**	Dial rdg	Load (kN)	Cor. load (kN)	**CBR %**	Dial rdg	Load (kN)	Cor. load (kN)	**CBR %**	Dial rdg	Load (kN)	Cor. load (kN)	**CBR %**
0	0.00	0	0			0	0			0	0		
0.64	0.25	0.1	0.021			0.20	0.042			0.20	0.042		
1.27	0.50	0.1	0.021			0.20	0.042			0.30	0.064		
1.91	0.75	0.2	0.042			0.30	0.064			0.40	0.085		
2.54	1.00	0.2	0.042	0.04	0.32	0.30	0.064	0.06	0.48	0.40	0.085	0.08	0.63
3.18	1.25	0.3	0.064			0.40	0.085			0.50	0.106		
3.81	1.50	0.3	0.064			0.40	0.085			0.50	0.106		
4.45	1.75	0.3	0.064			0.40	0.085			0.50	0.106		
5.08	2.00	0.4	0.085	0.08	0.42	0.50	0.106	0.11	0.53	0.60	0.127	0.13	0.64
7.62	3.00	0.5	0.106			0.60	0.127			1.00	0.212		
10.16	4.00												
12.7	5.00												

Swell				Ring factor		MDD (gm/cc)	1.42
No. of blows	**10**	**30**	**65**	212.359	N/Divis.		
Rdg (before soaking)	0.00	0.00	0.00			95 % of MDD	1.35
Rdg (after soaking)	9.46	8.39	7.33				
Percent swell	8.09	7.17	6.26			**C.B.R.at 95% of MDD**	**0.53**
Average percent swell	7.17						

Blows	Load (kN)		CBR(%)		Swell (%)	Dry density Vs soaked CBR			
	2.54mm	5.08mm	2.54mm	5.08mm					
10	0.04	0.08	0.3	0.4	**8.09**	No. blows	**10**	**30**	**65**
30	0.06	0.11	0.48	0.53	**7.17**	Dry density	1.21	1.38	1.45
65	0.08	0.13	0.6	0.6	**6.26**	Soaked CBR	0.4	0.53	0.6
MDD (g/cm^3)	1.416	**OMC (%)**			26.22	Moisture content	26.93	26.75	28.32


Load vs Penetration graph (114+000)
Load on piston (KN)
0.24
0.20
0.16
0.12
0.08
0.04
0.00
0
1
2
3
4
5
6
7
8
9
Penetration (mm)
10 blows
30 blows
65 blows


CBR vs Dry density (114+000)
CBR (%)
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1.20
1.25
1.30
1.35
1.40
1.45
1.50
y = 0.8425x - 0.6036
Dry density (g/cm³)



MDD (g/cm^3)	1.42
95% MDD (g/cm^3)	1.35
OMC (%)	26.22
CBR @ 95% MDD (%)	**0.53**
Swell @ 95% MDD (%)	**7.18**

Density of soil in place		
Description of soil: Light yellowish silty clay soil		
Location: km 114+000 FW		
Depth: 1.0m	Coordinate	N: 6.924824°
Test method: AASHTO T-191		E: 37.303474°
Density of sand, g/cm^3: 1.316		
Date: 12/10/2023		

Depth of hole, cm	15.0
A. Total weight of sand in container, g	7763
B. Sand remain in container, g	3022
C. Weight of used sand (A-B), g	4741
D. Sand in cone, g	931.67
E. Sand in hole (C-D), g	3809
F. Density of sand, g/cm^3	1.316
G. Volume of hole, (E/F) cm^3	2895
H. Weight of wet soil, g	4041
I. Wet density of soil (H/G), g/cm^3	1.40

Determination of water content		
Description of soil: Light yellowish silty clay soil		
Location: km 114+000 FW		
Depth: 1.0m	Coordinate	N: 6.924824°
Test method: AASHTO T-265		E: 37.303474°
Date: 12/10/2023		

Item	Test no.		
	1	2	3
Can No.	A5	A	6
Mass of can, W_1 (g)	39.2	36.92	36.96
Mass of can + wet soil, W_2 (g)	363.84	335.18	343.12
Mass of can + dry soil, W_3 (g)	255.53	235.63	241.43
Mass of moisture, W_2 - W_3 (g)	108.31	99.55	101.69
Mass of dry soil, W_3 - W_1 (g)	216.33	198.71	204.47
Moisture content, $w(\%) = \left(\frac{W_2 - W_3}{W_3 - W_1}\right) x100$	50.07	50.10	49.73
Average moisture content, *w* = 49.97%			

Determination of linear shrinkage		
Description of soil: Light yellowish silty clay soil		
Location: km 114+000 FW		
Depth: 1.0m	Coordinate	N: 6.924824°
Test method: BS 1377-2		E: 37.303474°
Date: 12/10/2023		

Sample ID	Length before drying, L0 (cm)	Length after drying, Lf (cm)	Linear shrinkage (%) = $\left(\frac{L_0 - L_f}{L_0}\right) * 100$
A	14	12.13	13.36
B	14	11.78	15.86
Average linear shrinkage = **14.61%**			

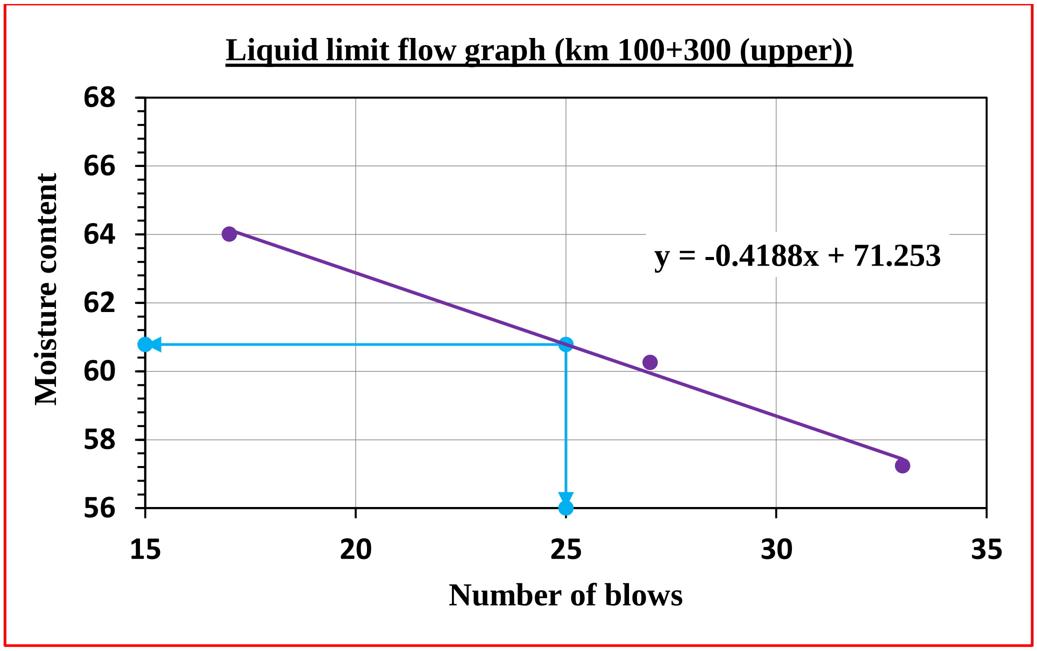
Free swell test		
Description of soil: Light yellowish silty clay soil		
Location: km 114+000 FW		
Depth: 1.0m	Coordinate	N: 6.924824°
Test method: IS-2720		E: 37.303474°
Date: 12/10/2023		

Free swell initial reading (F_i), ml	Free swell final reading (F_f), ml	Free swell (%) $\left(\frac{F_f - F_i}{F_i}\right)x100$
10	27	170

Station: km 100+300 (upper layer)

Atterberg limit test		
Description of soil: Reddish brown sandy silt with few gravels		
Location: km 100+300 RHS (upper layer)	Coordinate	N: 6.99583°
Depth: 5.0m		E: 37.283838°
Test method: AASHTO T-89 & 90		
Date: 14/10/2023		

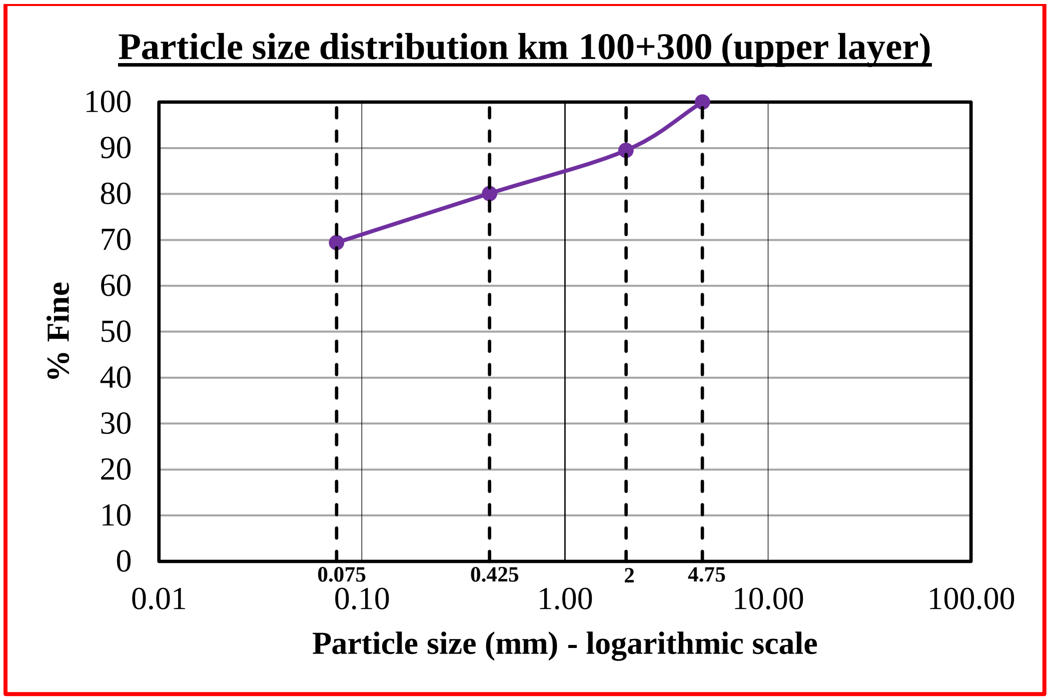
	Liquid limit			Plastic limit	
Number of Blows	33	27	17		
Can Number	37	A	6	9	34
Wet Weight + Can	48.32	48.49	48.91	14.50	14.76
Dry Weight + Can	34.20	33.90	33.70	13.10	13.36
Weight of Can	9.53	9.69	9.94	9.33	9.45
Weight of Water	14.12	14.59	15.21	1.40	1.40
Dry Weight	24.67	24.21	23.76	3.77	3.91
Moisture Content	57.24	60.26	64.02	37.14	35.81



Liquid limit	60.8%
Plastic limit	36.5%
Plasticity index	24.3%

Particle size distribution		
Description of soil: Reddish brown sandy silt with few gravels		
Location: km 100+300 RHS (upper layer)	Coordinate	N: 6.99583°
Depth: 5.0m		E: 37.283838°
Test method: AASHTO T-88		
Date: 10/10/2023		

A	B	C	D	E	F	G	H
Sieve no.	Opening (mm)	Mass retained (grams) M_r (Trial1)	Mass retained (grams) M_r (Trial2)	Mass retained (grams) M_r (Average)	% Retained ($100*M_r/M_t$)	∑ (% Retained) (∑ column D)	% Finer (100 - column G)
4	4.750	0	0	0.0	0.00	0.00	100.00
10	2.000	8	7	7.5	1.50	1.50	98.50
40	0.425	23	19	21.0	4.20	5.70	94.30
200	0.075	49	42	45.5	9.10	14.80	85.20
Pan		420	432	426.0	85.20		
	SUM	500	500	500.0	100.0		
	M_t =	500	500	500.0			



Density of soil in place		
Description of soil: Reddish brown silty sand with few gravels		
Location: km 100+300 RHS (upper layer)		
Depth: 5.0m	Coordinate	N: 6.99583°
Test method: AASHTO T-191		E: 37.283838°
Density of sand, g/cm^3: 1.316		
Date: 14/10/2023		

Depth of hole, cm	15.0
A. Total weight of sand in container, g	7763
B. Sand remain in container, g	3022
C. Weight of used sand (A-B), g	4741
D. Sand in cone, g	931.67
E. Sand in hole (C-D), g	3809
F. Density of sand, g/cm^3	1.316
G. Volume of hole, (E/F) cm^3	2895
H. Weight of wet soil, g	4041
I. Wet density of soil (H/G), g/cm^3	1.40

Determination of water content		
Description of soil: Reddish brown silty sand with few gravels		
Location: km 100+300 RHS (upper layer)		
Depth: 5.0m	Coordinate	N: 6.99583°
Test method: AASHTO T-265		E: 37.283838°
Date: 14/10/2023		

Item	Test no.		
	1	2	3
Can No.	A1	A17	A10
Mass of can, W_1 (g)	37.27	35.91	37.26
Mass of can + wet soil, W_2 (g)	414.47	422.04	409.70
Mass of can + dry soil, W_3 (g)	321.76	327.44	318.23
Mass of moisture, W_2 - W_3 (g)	92.71	94.60	91.47
Mass of dry soil, W_3 - W_1 (g)	284.49	291.53	280.97
Moisture content, $w(\%) = \left(\frac{W_2 - W_3}{W_3 - W_1}\right) x100$	32.59	32.45	32.56
Average moisture content, *w* = 32.53%			

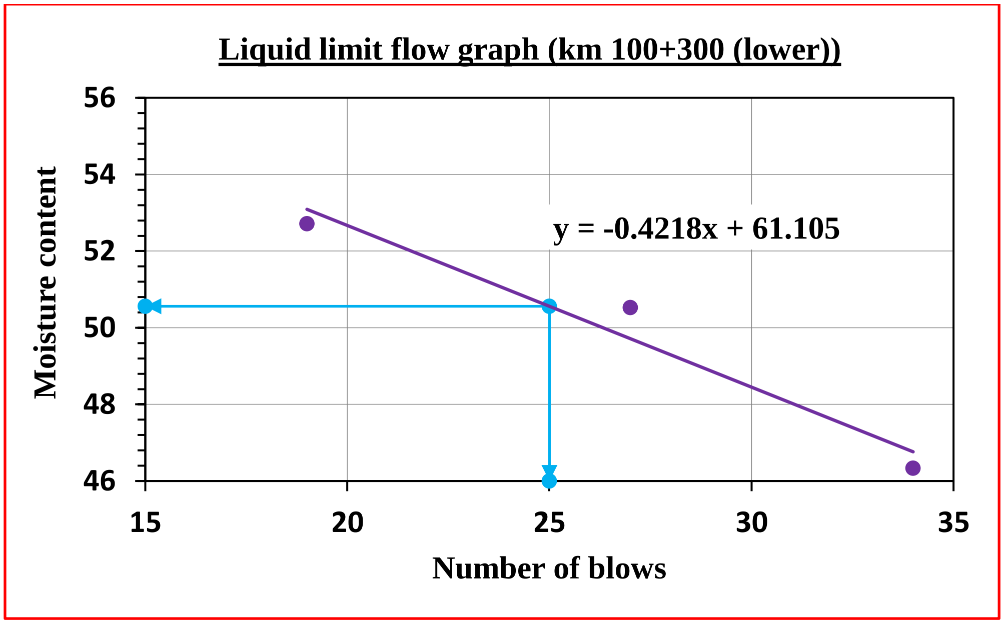
Direct shear test		
Description of soil: Reddish brown silty sand with few gravels		
Location: km 100+300 RHS (upper layer)		
Depth: 5.0m	Coordinate	N: 6.99583°
Test method: ASTM D3080		E: 37.283838°
Date: 14/10/2023		

Station	Sample Depth (m)	Field Data		ASTM D3080	
		NMC (%)	In-place density (g/cm^3)	C (kPa)	ϕ (°)
100+300 (Upper layer)	**5**	**32.53**	**1.4**	**34**	**24**
100+300 (Lower layer)	10	29.65	1.54	30	26
126+ 000 (Fill material)	4	20.1	1.6	33	10.6
134+760 (RHS)	1	41.39	1.51	ASTM D4221	
				53%	

Station: km 100+300 (lower layer)

Atterberg limit test		
Description of soil: Reddish brown sandy silt with few gravels		
Location: km 100+300 RHS (lower layer)	Coordinate	N: 6.99583°
Depth: 10.0m		E: 37.283838°
Test method: AASHTO T-89 & 90		
Date: 14/10/2023		

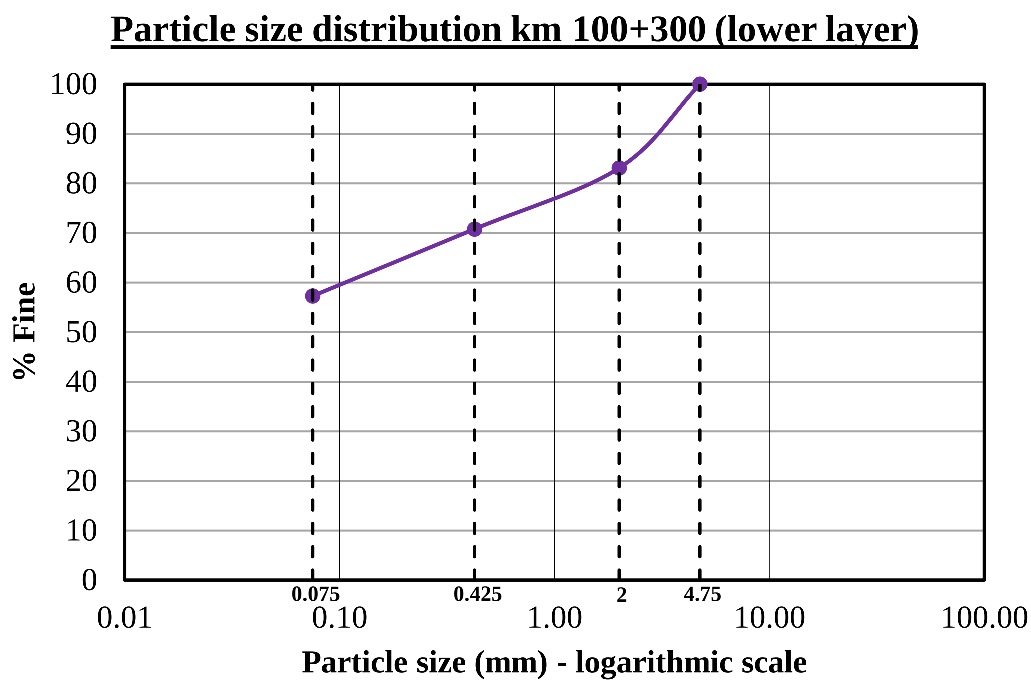
Liquid limit				Plastic limit	
Number of Blows	34	27	19		
Can Number	12	17	22	19	9
Wet Weight + Can	46.82	48.60	48.26	16.28	16.22
Dry Weight + Can	35.01	35.62	34.95	14.66	14.77
Weight of Can	9.52	9.93	9.70	9.58	10.10
Weight of Water	11.81	12.98	13.31	1.62	1.45
Dry Weight	25.49	25.69	25.25	5.08	4.67
Moisture Content	46.33	50.53	52.71	31.89	31.05



Liquid limit	**50.6%**
Plastic limit	**31.5%**
Plasticity index	**19.1%**

Particle size distribution			
Description of soil: Reddish brown silty sand with few gravels			
Location: km 100+300 RHS (lower layer)	Coordinate	N: 6.99583°	
Depth: 10.0m		E: 37.283838°	
Test method: AASHTO T-88			
Date: 14/10/2023			

A	B	C	D	E	F	G	H
Sieve no.	Opening (mm)	Mass retained (grams) M_r (Trial1)	Mass retained (grams) M_r (Trial2)	Mass retained (grams) M_r (Average)	% Retained ($100*M_r/M_t$)	∑ (% Retained) (∑ column D)	% Finer (100 - column G)
4	4.750	0	0	0.0	0.00	0.00	100.00
10	2.000	80	89	84.5	16.90	16.90	83.10
40	0.425	64	59	61.5	12.30	29.20	70.80
200	0.075	74	61	67.5	13.50	42.70	57.30
Pan	Pass	282	291	286.5	57.30	100.00	0.00
	SUM	500	500	500.0	100.0		
	M_t =	500	500	500.0			



Density of soil in place		
Description of soil: Reddish brown sandy silt with few gravels		
Location: km 100+300 RHS (lower layer)		
Depth: 10.0m	Coordinate	N: 6.99583°
Test method: AASHTO T-191		E: 37.283838°
Density of sand, g/cm^3: 1.316		
Date: 14/10/2023		

Depth of hole, cm	15.0
A. Total weight of sand in container, g	7486
B. Sand remain in container, g	3063
C. Weight of used sand (A-B), g	4423
D. Sand in cone, g	931.67
E. Sand in hole (C-D), g	3491
F. Density of sand, g/cm^3	1.316
G. Volume of hole, (E/F) cm^3	2653
H. Weight of wet soil, g	4092
I. Wet density of soil (H/G), g/cm^3	1.54

Determination of water content		
Description of soil: Reddish brown sandy silt with few gravels		
Location: km 100+300 RHS (lower layer)		
Depth: 10.0m	Coordinate	N: 6.99583°
Test method: AASHTO T-265		E: 37.283838°
Date: 14/10/2023		

Item	Test no.		
	1	2	3
Can No.	E5	14	31
Mass of can, W_1 (g)	37.23	37.22	37.74
Mass of can + wet soil, W_2 (g)	397.21	412.52	380.42
Mass of can + dry soil, W_3 (g)	315.22	326.62	301.78
Mass of moisture, W_2 - W_3 (g)	81.99	85.90	78.64
Mass of dry soil, W_3 - W_1 (g)	277.99	289.40	264.04
Moisture content, $w(\%) = \left(\frac{W_2 - W_3}{W_3 - W_1}\right) x100$	29.49	29.68	29.78
Average moisture content, *w* = 29.65%			

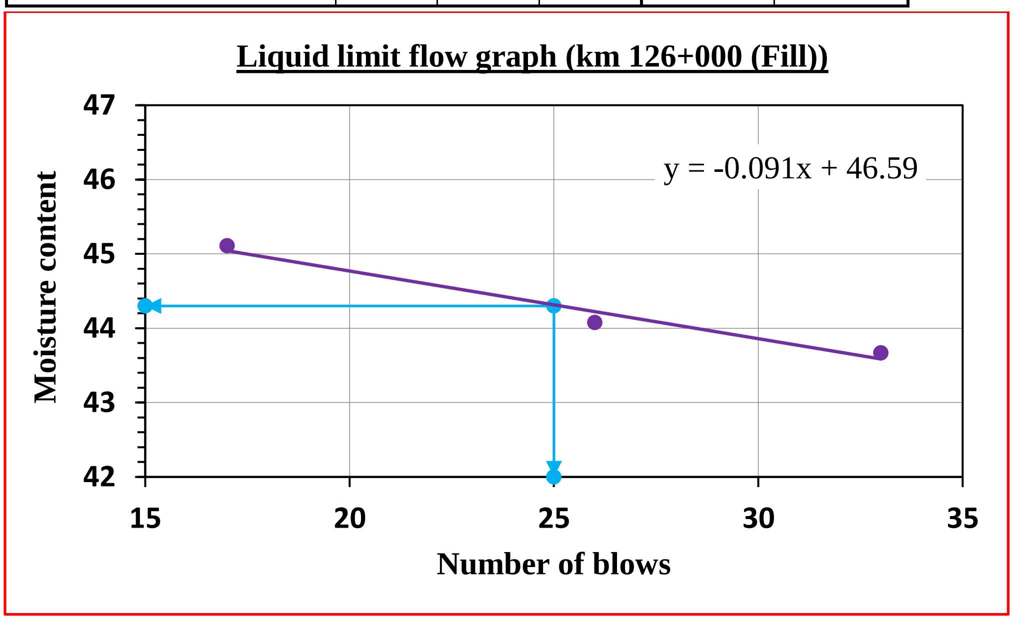
Direct shear test		
Description of soil: Reddish brown silty sand with few gravels		
Location: km 100+300 RHS (lower layer)		
Depth: 10.0m	Coordinate	N: 6.99583°
Test method: ASTM D3080		E: 37.283838°
Date: 14/10/2023		

Station	Sample Depth (m)	Field Data		ASTM D3080	
		NMC (%)	In-place density (g/cm^3)	C (kPa)	ϕ (°)
100+300 (Upper layer)	5	32.53	1.4	34	24
100+300 (Lower layer)	**10**	**29.65**	**1.54**	**30**	**26**
126+ 000 (Fill material)	4	20.1	1.6	33	10.6
134+760 (RHS)	1	41.39	1.51	ASTM D4221	
				53%	

Station: km 126+000 (fill material)

Atterberg limit test		
Description of soil: Reddish brown silty sand		
Location: km 126+000 (fill material)	Coordinate	N: 6.852265°
Depth: 4.0m		E: 37.266199°
Test method: AASHTO T-89 & 90		
Date: 18/10/2023		

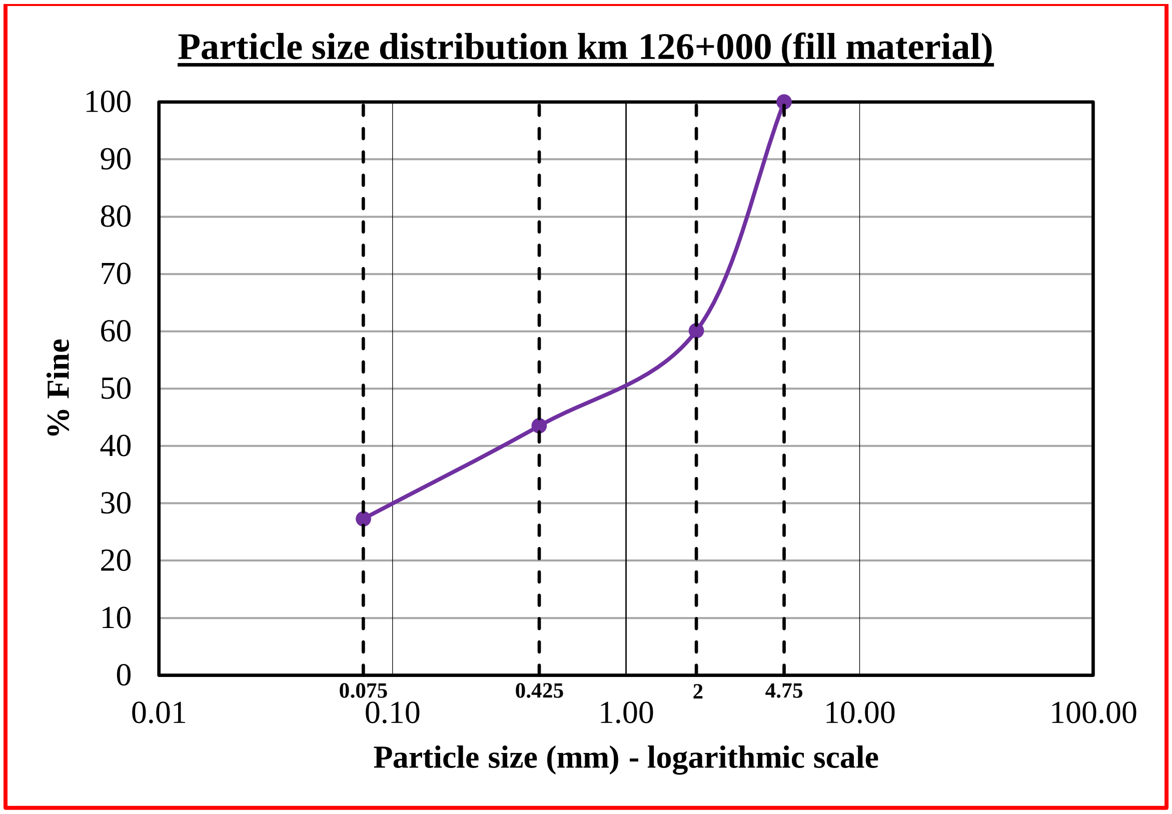
	Liquid limit			Plastic limit	
Number of Blows	33	26	17		
Can Number	29	19	36	3X	18
Wet Weight + Can	54.34	52.45	52.13	16.02	16.34
Dry Weight + Can	40.75	39.28	38.90	14.68	14.90
Weight of Can	9.63	9.40	9.57	9.67	9.51
Weight of Water	13.59	13.17	13.23	1.34	1.44
Dry Weight	31.12	29.88	29.33	5.01	5.39
Moisture Content	43.67	44.08	45.11	26.75	26.72



Liquid limit	**44.3%**
Plastic limit	**26.7%**
Plasticity index	**17.6%**

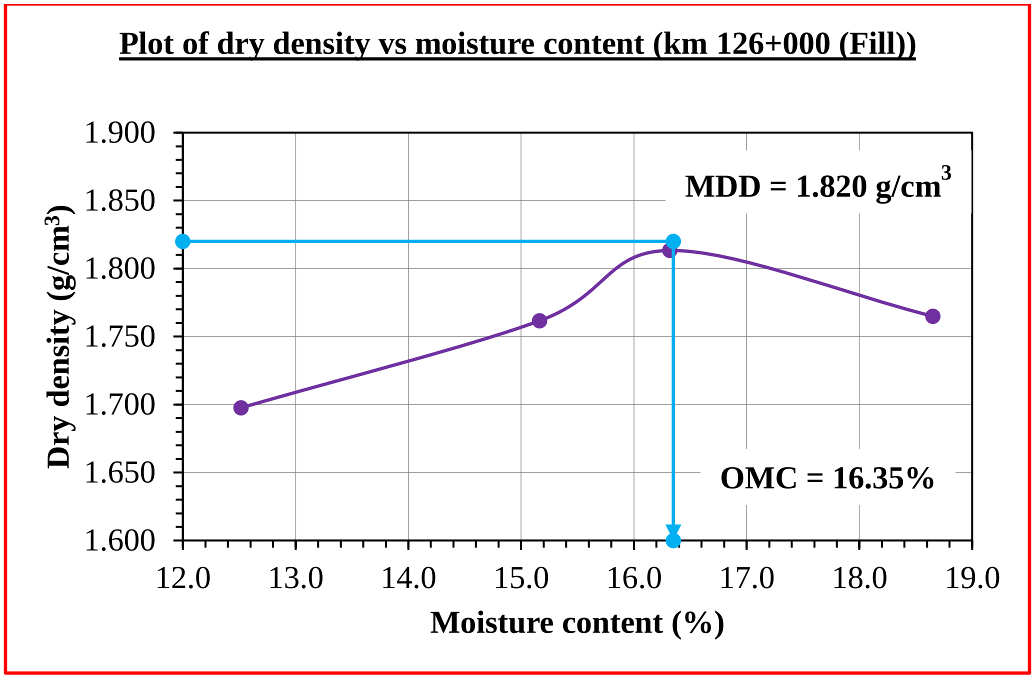
Particle size distribution		
Description of soil: Reddish brown silty sand		
Location: km 126+000 (fill material)	Coordinate	N: 6.852265°
Depth: 4.0m		E: 37.266199°
Test method: AASHTO T-89 & 90		
Date: 18/10/2023		

A	B	C	D	E	F	G	H
Sieve no.	Opening (mm)	Mass retained (grams) M_r (Trial1)	Mass retained (grams) M_r (Trial2)	Mass retained (grams) M_r (Average)	% Retained (100*M_r/M_t)	∑ (% Retained) (∑ column D)	% Finer (100 - column G)
4	4.750	0	0	0.0	0.00	0.00	100.00
10	2.000	211	188	199.5	39.90	39.90	60.10
40	0.425	86	80	83.0	16.60	56.50	43.50
200	0.075	81	81	81.0	16.20	72.70	27.30
Pan	Pass	122	151	136.5	27.30	100.00	0.00
	SUM	500	500	500.0	100.0		
	M_t =	500	500	500.0			


Particle size distribution km 126+000 (fill material)
% Fine
100
90
80
70
60
50
40
30
20
10
0
0.075
0.425
2
4.75
0.01
0.10
1.00
10.00
100.00
Particle size (mm) - logarithmic scale

Modified proctor		
Description of soil: Reddish brown silty sand		
Location: km 126+000 (fill material)	Coordinate	N: 6.852265°
Volume of mold: 944 cm^3		E: 37.266199°
Weight of hammer: 4.54 kg	Test method: AASHTO T-180 (Method-D)	
Number of blows/layer: 56	Number of layer: 5	

				1	**2**	**3**	**4**	
Density	Trail no.			**1**	**2**	**3**	**4**	
	Added water cc			2%	4%	6%	8%	
	Weight of soil + mold	(g)	W_1	8,798	9,050	9,221	9,189	
	Weight of mold	(g)	W_2	4741	4741	4741	4741	
	Volume of mold	(cm^3)	V	2124	2124	2124	2124	
	Weight of wet soil	(g)	$W_3 = W_1 - W_2$	4,057	4,309	4,480	4,448	
	Wet density of soil	(g/cm^3)	$W_d = W_3/V$	1.910	2.029	2.109	2.094	NMC
Moisture	Container no.			C19	A14	C3	C1	A9
	Wet soil + container	(g)	a	291.01	294.07	294.74	302.35	288.57
	Dry soil + container	(g)	b	262.81	260.38	258.77	260.80	265.69
	Weight of container	(g)	c	37.51	38.20	38.34	38.02	38.43
	Weight of water	(g)	e = a-b	28.2	33.7	36.0	41.6	22.9
	Weight of dry soil	(g)	d =b-c	225.3	222.2	220.4	222.8	227.3
	Moisture content	(%)	m= (e/d)*100	12.52	15.16	16.32	18.65	10.07
Dry density of soil (g/Cm3) $D_d = W_d/(100+m)*100$				1.698	1.762	1.813	1.765	


Plot of dry density vs moisture content (km 126+000 (Fill))
MDD = 1.820 g/cm3
OMC = 16.35%
Dry density (g/cm3)
Moisture content (%)
1.900
1.850
1.800
1.750
1.700
1.650
1.600
12.0
13.0
14.0
15.0
16.0
17.0
18.0
19.0

California bearing ratio (CBR)		
Description of soil: Reddish brown silty sand		
Location: km 126+000 (fill material)		
Depth: 4.0m	Coordinate	N: 6.852265°
Test method: AASHTO T-193		E: 37.266199°
Date: 12/10/2023		

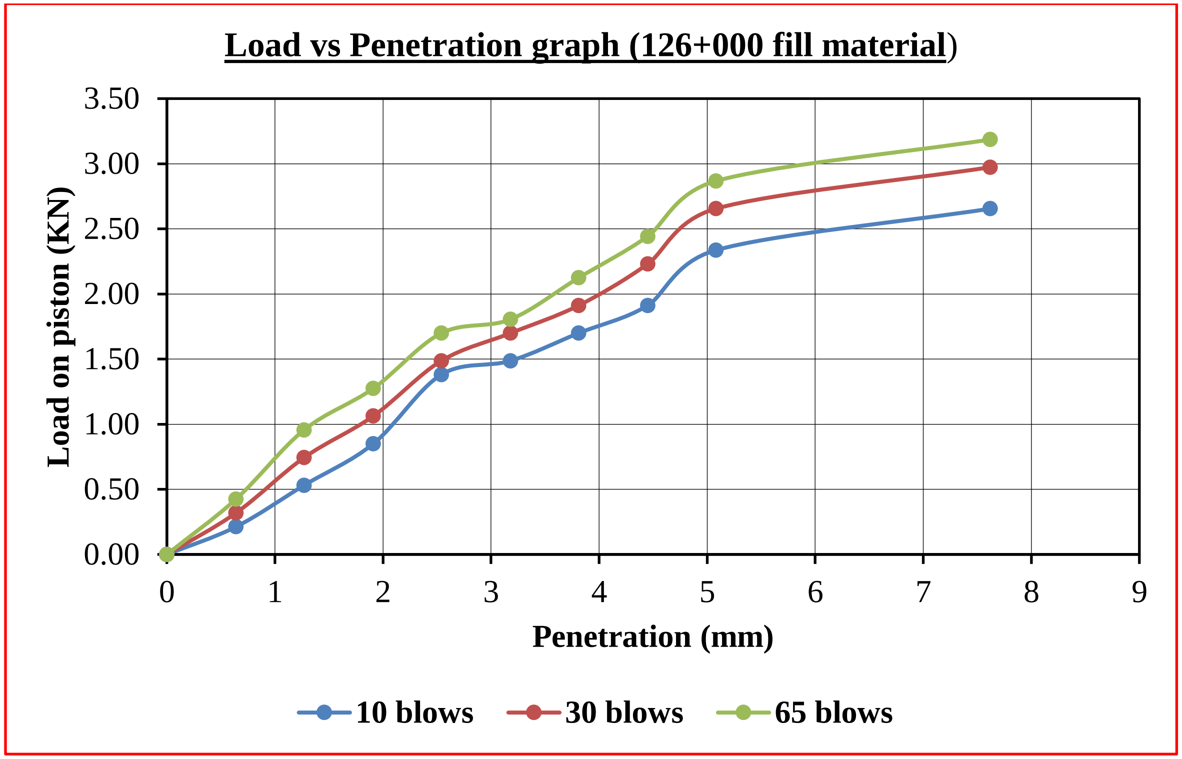
Soaking condition		**10 Blows**		**30 Blows**		**65 Blows**	
		Before	After	Before	After	Before	After
Mold no.		A2		F2		B2	
Weight of soil + mold (g)	W_1	10323	10650	11007	11231	11365	10179
Weight of mold (g)	W_2	6126	6126	6596	6596	6882	6882
Volume of mold (cm^3)	V	2124	2124	2124	2124	2124	2124
Weight of wet soil (g)	$W_3 = W_1 - W_2$	4197	4524	4411	4635	4483	3297
Wet density of soil (g/cm^3)	$W_d = (W_3/V)$	1.98	2.13	2.08	2.18	2.11	1.55
Dry density of soil (g/cm^3)	$D_d = W_d/(100+m)*100$	1.70	1.72	1.78	1.82	1.83	1.29

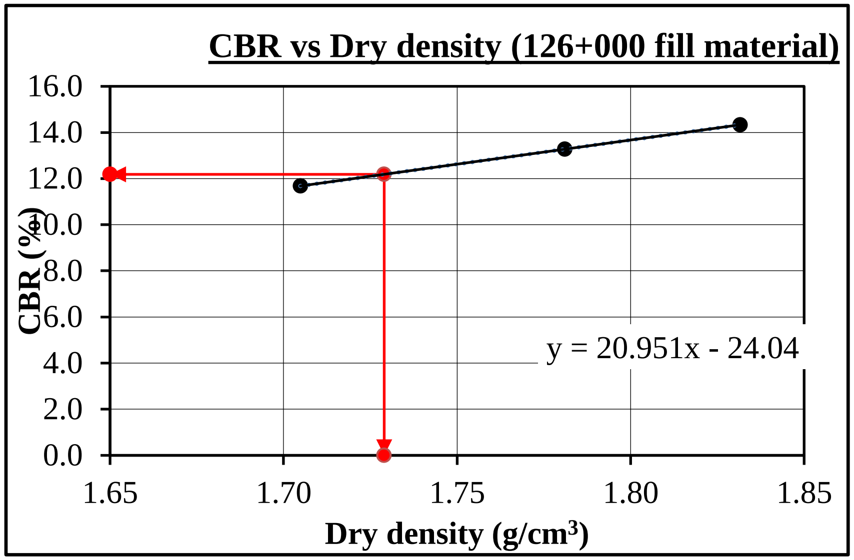
Moisture determination										
Soaking condition		**10 Blows**			**30 Blows**			**65 Blows**		
		Before	After		Before	After		Before	After	
			TOP 1 in.	Avg.		TOP 1 in.	Avg.		TOP 1 in.	Avg.
Container no.		A	W54		C10	14		A5	A14	
Wet soil + container (g)	a	291.08	268.31		264.54	286.45		284.06	243.13	
Dry soil + container (g)	b	256.08	224.51		232.28	245.42		251.65	208.36	
Weight of container (g)	c	36.03	38.22		38.00	37.00		39.00	38.00	
Weight of water (g)	d = a - b	35.00	43.80		32.26	41.03		32.41	34.77	
Weight of dry soil (g)	e = b - c	220.05	186.29		194.28	208.42		212.65	170.36	
Moisture content (%)	m =(d/e)*100	15.91	23.51	19.71	16.60	19.69	18.15	15.24	20.41	17.83
Avg. mois. content. (%)										

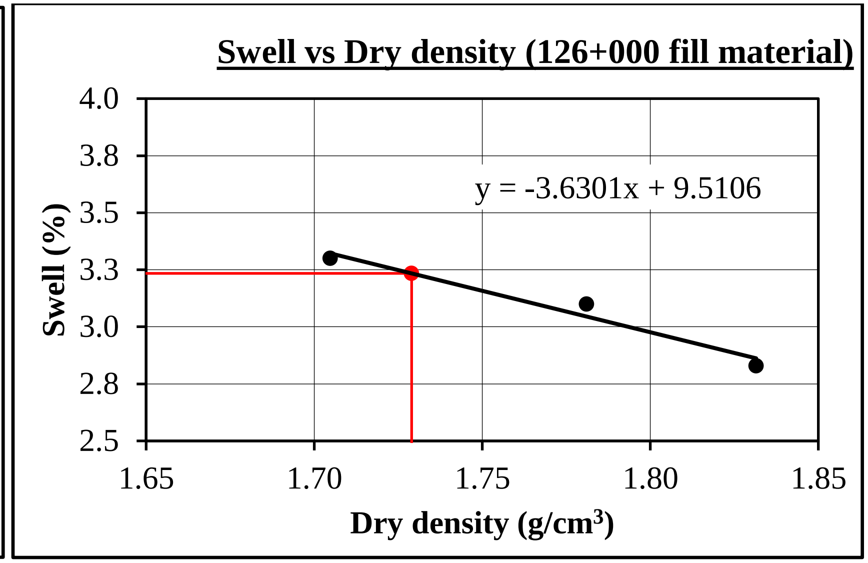
Penetration test data													
Penetration		**10 Blows**				**30 Blows**				**65 Blows**			
(mm)	**(Div.)**	Dial rdg	Load (kN)	Cor. load (kN)	**CBR %**	Dial rdg	Load (kN)	Cor. load (kN)	**CBR %**	Dial rdg	Load (kN)	Cor. load (kN)	**CBR %**
0	0.00	0	0			0	0			0	0		
0.64	0.25	1.0	0.212			1.50	0.319			2.00	0.425		
1.27	0.50	2.5	0.531			3.50	0.743			4.50	0.956		
1.91	0.75	4.0	0.849			5.00	1.062			6.00	1.274		
2.54	1.00	6.5	1.380	1.38	10.30	7.00	1.487	1.49	11.09	8.00	1.699	1.70	12.68
3.18	1.25	7.0	1.487			8.00	1.699			8.50	1.805		
3.81	1.50	8.0	1.699			9.00	1.911			10.00	2.124		
4.45	1.75	9.0	1.911			10.50	2.230			11.50	2.442		
5.08	2.00	11.0	2.336	2.34	11.68	12.50	2.654	2.65	13.27	13.50	2.867	2.87	14.33
7.62	3.00	12.5	2.654			14.00	2.973			15.00	3.185		

Swell				**Ring factor**		MDD (gm/cc)	1.82
No. of blows	**10**	**30**	**65**	212.359	N/Divis.		
Rdg (before soaking)	0.00	0.00	0.00			95 % of MDD	1.73
Rdg (after soaking)	4.10	3.81	3.31				
Percent swell	3.30	3.10	2.83			**C.B.R.at 95% of MDD**	**12.18**
Average percent swell	3.08						

Blows	Load (kN)		CBR(%)		Swell (%)	Dry density Vs soaked CBR			
	2.54mm	5.08mm	2.54mm	5.08mm					
10	1.38	2.34	10.3	11.7	**3.30**	No. blows	**10**	**30**	**65**
30	1.49	2.65	11.09	13.27	**3.10**	Dry density	1.70	1.78	1.83
65	1.70	2.87	12.7	14.3	**2.83**	Soaked CBR	11.7	13.27	14.3
MDD (g/cm^3)	1.82	**OMC (%)**			16.35	Moisture content	15.91	16.60	15.24







MDD (g/cm^3)	1.82
95% MDD (g/cm^3)	1.73
OMC (%)	16.35
CBR @ 95% MDD (%)	**12.18**
Swell @ 95% MDD (%)	**3.23**

Determination of water content		
Description of soil: Reddish brown silty sand		
Location: km 126+000 (fill material)		
Depth: 4.0m	Coordinate	N: 6.852265°
Test method: AASHTO T-265		E: 37.266199°
Date: 12/10/2023		

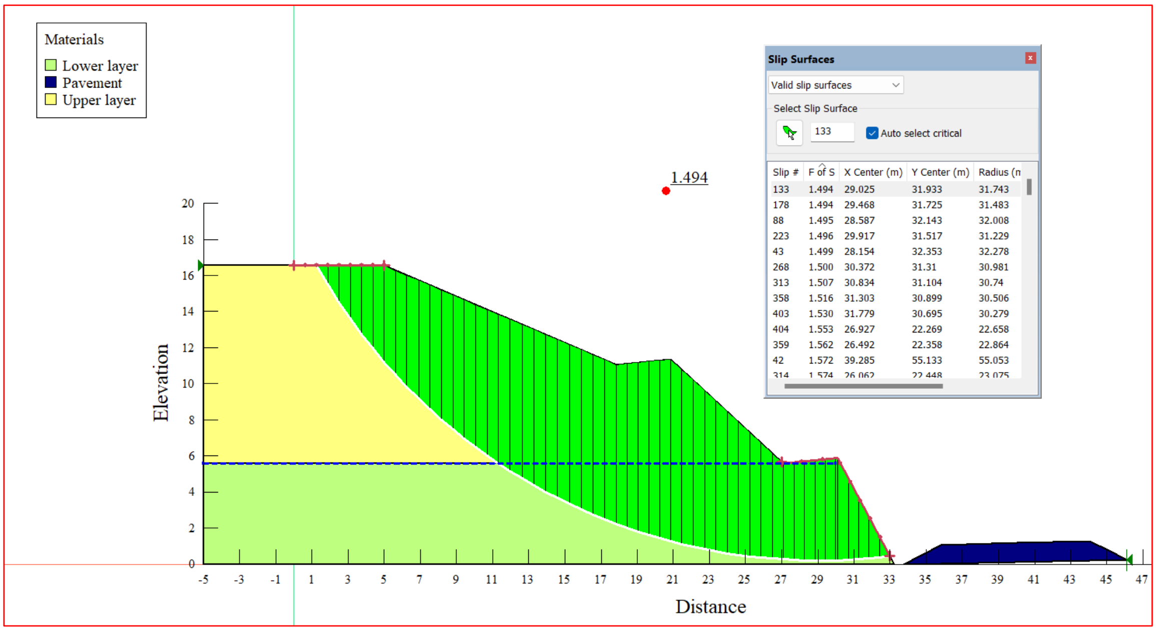
Item	Test no.	
	1	2
Can No.	A14	C23
Mass of can, W_1 (g)	38.00	39.22
Mass of can + wet soil, W_2 (g)	408.23	408.62
Mass of can + dry soil, W_3 (g)	343.36	350.21
Mass of moisture, W_2 - W_3 (g)	64.87	58.41
Mass of dry soil, W_3 - W_1 (g)	305.36	310.99
Moisture content, $w(\%) = \left(\frac{W_2 - W_3}{W_3 - W_1}\right) x100$	21.24	18.78
Average moisture content, *w* = 20.01%		

Direct shear test		
Description of soil: Reddish brown silty sand		
Location: km 126+000 (fill material)		
Depth: 4.0m	Coordinate	N: 6.852265°
Test method: ASTM D3080		E: 37.266199°
Date: 14/10/2023		

Station	Sample Depth (m)	Field Data		ASTM D3080	
		NMC (%)	In-place density (g/cm^3)	C (kPa)	ϕ (°)
100+300 (Upper layer)	5	32.53	1.4	34	24
100+300 (Lower layer)	10	29.65	1.54	30	26
126+ 000 (Fill material)	**4**	**20.1**	**1.6**	**33**	**10.6**
134+760 (RHS)	1	41.39	1.51	ASTM D4221	
				53%	

APPENDIX B SOFTWARE OUTPUTS

Limit equilibrium slope stability analysis cut slope (100+300 RHS), **F.S = 1.494**



Analysis Settings

Slope Stability

- Description: LE analsysis (100+300) safe cut slope geometry
- Kind: SLOPE/W
- Method: Bishop
- Settings
  - PWP Conditions from: Piezometric Line
    - Apply Phreatic Correction: No
    - Use Staged Rapid Drawdown: No
  - Unit Weight of Water: 9.807 kN/m³
- Slip Surface
  - Direction of movement: Left to Right
  - Use Passive Mode: No
  - Slip Surface Option: Entry and Exit
  - Critical slip surfaces saved: 1
  - Optimize Critical Slip Surface Location: No
  - Tension Crack Option: (none)
- Distribution
  - F of S Calculation Option: Constant
- Advanced
  - Geometry Settings
    - Minimum Slip Surface Depth: 0.1 m
    - Number of Slices: 50
  - Factor of Safety Convergence Settings
    - Maximum Number of Iterations: 100
    - Tolerable difference in F of S: 0.001

Materials

Upper layer

- Model: Mohr-Coulomb
- Unit Weight: 16 kN/m³
- Cohesion': 34 kPa
- Phi': 24 °
- Phi-B: 0 °
- Pore Water Pressure
  - Piezometric Line: 1

Lower layer

- Model: Mohr-Coulomb
- Unit Weight: 16 kN/m³
- Cohesion': 30 kPa
- Phi': 26 °
- Phi-B: 0 °
- Pore Water Pressure
  - Piezometric Line: 1

Pavement

- Model: Mohr-Coulomb
- Unit Weight: 20 kN/m³
- Cohesion': 30 kPa
- Phi': 30 °
- Phi-B: 0 °
- Pore Water Pressure
  - Piezometric Line: 1

Slip Surface Entry and Exit

Left Type: Range
Left-Zone Left Coordinate: (0.00095, 16.564525) m
Left-Zone Right Coordinate: (5, 16.563385) m
Left-Zone Increment: 8
Right Type: Range
Right-Zone Left Coordinate: (27, 5.68314) m
Right-Zone Right Coordinate: (33, 0.440465) m
Right-Zone Increment: 8
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (-5, 16.56528) m
Right Coordinate: (46.10442, 0.22722) m

Piezometric Lines

Piezometric Line 1

Coordinates

	X	Y
Coordinate 1	-5 m	5.587 m
Coordinate 2	30 m	5.587 m

Seismic Coefficients

Horz Seismic Coef.: 0.1
Vert Seismic Coef.: 0.05

Points

	X	Y
Point 1	-5 m	5.58698 m
Point 2	-5 m	0 m
Point 3	33.2342 m	0 m
Point 4	30.10403 m	5.88698 m
Point 5	27.10403 m	5.58698 m
Point 6	20.85617 m	11.3622 m
Point 7	17.85617 m	11.0622 m
Point 8	4.9991 m	16.56377 m
Point 9	-5 m	16.56528 m
Point 10	33.7342 m	0 m
Point 11	46.10442 m	0.22722 m
Point 12	45.10442 m	0.72722 m
Point 13	44.02442 m	1.26722 m
Point 14	40.02442 m	1.15652 m
Point 15	35.8142 m	1.04 m
Point 16	34.7342 m	0.5 m

Regions

	Material	Points	Area
Region 1	Lower layer	1,2,3,4,5	205.79 m^2
Region 2	Upper layer	6,7,8,9,1,5	250.44 m^2
Region 3	Pavement	10,11,12,13,14,15,16	10.702 m^2

Slip Results

Slip Surfaces Analysed: 345 of 405 converged

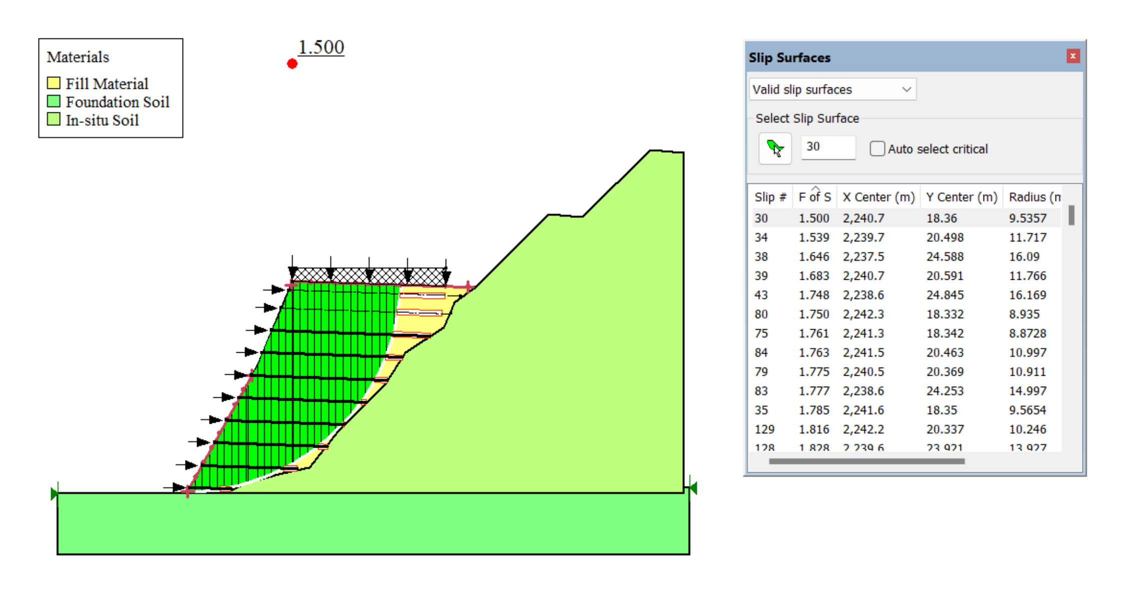
Current Slip Surface

Slip Surface: 133
Factor of Safety: 1.494
Volume: 214.74132 m^3
Weight: 3,435.8612 kN
Resisting Moment: 76,356.286 kN·m
Activating Moment: 51,115.935 kN·m
Slip Rank: 1 of 405 slip surfaces
Exit: (33, 0.44046503) m
Entry: (1.2507322, 16.564336) m
Radius: 31.74261 m
Center: (29.024684, 31.933165) m

Slip Slices

	X	Y	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength	Suction Strength	Base Material
Slice 1	1.5630962 m	16.025112 m	-102.36656 kPa	-19.957986 kPa	-8.8858677 kPa	34 kPa	0 kPa	Upper layer
Slice 2	2.1878242 m	14.991091 m	-92.225924 kPa	-6.5337926 kPa	-2.9090319 kPa	34 kPa	0 kPa	Upper layer
Slice 3	2.8125521 m	14.038738 m	-82.886196 kPa	6.3299194 kPa	2.8182617 kPa	34 kPa	0 kPa	Upper layer
Slice 4	3.4372801 m	13.155537 m	-74.224642 kPa	18.67214 kPa	8.3133722 kPa	34 kPa	0 kPa	Upper layer
Slice 5	4.0620081 m	12.332086 m	-66.149061 kPa	30.528779 kPa	13.592288 kPa	34 kPa	0 kPa	Upper layer
Slice 6	4.686736 m	11.561107 m	-58.588065 kPa	41.932118 kPa	18.669382 kPa	34 kPa	0 kPa	Upper layer
Slice 7	5.3151209 m	10.832834 m	-51.445898 kPa	51.270893 kPa	22.827272 kPa	34 kPa	0 kPa	Upper layer
Slice 8	5.9471628 m	10.143043 m	-44.681112 kPa	58.492735 kPa	26.042643 kPa	34 kPa	0 kPa	Upper layer
Slice 9	6.5792046 m	9.4922654 m	-38.298938 kPa	65.237878 kPa	29.045774 kPa	34 kPa	0 kPa	Upper layer
Slice 10	7.2112465 m	8.8771901 m	-32.266895 kPa	71.534884 kPa	31.849382 kPa	34 kPa	0 kPa	Upper layer
Slice 11	7.8432883 m	8.2950239 m	-26.55759 kPa	77.408634 kPa	34.464544 kPa	34 kPa	0 kPa	Upper layer
Slice 12	8.4753302 m	7.7433859 m	-21.147676 kPa	82.880925 kPa	36.900965 kPa	34 kPa	0 kPa	Upper layer
Slice 13	9.107372 m	7.2202283 m	-16.01707 kPa	87.970955 kPa	39.167193 kPa	34 kPa	0 kPa	Upper layer
Slice 14	9.7394139 m	6.7237751 m	-11.148354 kPa	92.695708 kPa	41.270788 kPa	34 kPa	0 kPa	Upper layer
Slice 15	10.371456 m	6.2524755 m	-6.5263185 kPa	97.070263 kPa	43.218466 kPa	34 kPa	0 kPa	Upper layer
Slice 16	11.003498 m	5.8049665 m	-2.1375975 kPa	101.10806 kPa	45.016209 kPa	34 kPa	0 kPa	Upper layer
Slice 17	11.646364 m	5.3731435 m	2.0972908 kPa	105.08302 kPa	50.229497 kPa	30 kPa	0 kPa	Lower layer
Slice 18	12.300042 m	4.9566613 m	6.1817319 kPa	109.21478 kPa	50.252577 kPa	30 kPa	0 kPa	Lower layer
Slice 19	12.953704 m	4.5621357 m	10.050844 kPa	112.93504 kPa	50.179974 kPa	30 kPa	0 kPa	Lower layer
Slice 20	13.607366 m	4.1886197 m	13.713915 kPa	116.26078 kPa	50.015448 kPa	30 kPa	0 kPa	Lower layer
Slice 21	14.261028 m	3.8352746 m	17.179171 kPa	119.20706 kPa	49.762327 kPa	30 kPa	0 kPa	Lower layer
Slice 22	14.91469 m	3.5013476 m	20.453993 kPa	121.7873 kPa	49.423558 kPa	30 kPa	0 kPa	Lower layer
Slice 23	15.568352 m	3.1861615 m	23.545023 kPa	124.01351 kPa	49.001756 kPa	30 kPa	0 kPa	Lower layer
Slice 24	16.222015 m	2.8891058 m	26.458248 kPa	125.89642 kPa	48.499238 kPa	30 kPa	0 kPa	Lower layer
Slice 25	16.875677 m	2.609629 m	29.199077 kPa	127.44565 kPa	47.918056 kPa	30 kPa	0 kPa	Lower layer
Slice 26	17.529339 m	2.3472327 m	31.772398 kPa	128.66981 kPa	47.260026 kPa	30 kPa	0 kPa	Lower layer
Slice 27	18.15617 m	2.1109125 m	34.08999 kPa	131.92859 kPa	47.719073 kPa	30 kPa	0 kPa	Lower layer
Slice 28	18.75617 m	1.8990112 m	36.168106 kPa	137.30169 kPa	49.326143 kPa	30 kPa	0 kPa	Lower layer
Slice 29	19.35617 m	1.7005018 m	38.114888 kPa	142.48013 kPa	50.902331 kPa	30 kPa	0 kPa	Lower layer
Slice 30	19.95617 m	1.5151219 m	39.932909 kPa	147.46765 kPa	52.448198 kPa	30 kPa	0 kPa	Lower layer
Slice 31	20.55617 m	1.3426327 m	41.62451 kPa	152.26766 kPa	53.964269 kPa	30 kPa	0 kPa	Lower layer
Slice 32	21.168562 m	1.1797801 m	43.221605 kPa	152.0123 kPa	53.06077 kPa	30 kPa	0 kPa	Lower layer
Slice 33	21.793346 m	1.0268856 m	44.721042 kPa	146.60052 kPa	49.689943 kPa	30 kPa	0 kPa	Lower layer
Slice 34	22.41813 m	0.88731628 m	46.089798 kPa	140.86585 kPa	46.225368 kPa	30 kPa	0 kPa	Lower layer
Slice 35	23.042913 m	0.76089319 m	47.32963 kPa	134.81092 kPa	42.667477 kPa	30 kPa	0 kPa	Lower layer
Slice 36	23.667697 m	0.64745685 m	48.4421 kPa	128.43782 kPa	39.016521 kPa	30 kPa	0 kPa	Lower layer
Slice 37	24.292481 m	0.5468664 m	49.42859 kPa	121.74807 kPa	35.272566 kPa	30 kPa	0 kPa	Lower layer
Slice 38	24.917265 m	0.45899863 m	50.290309 kPa	114.74264 kPa	31.435502 kPa	30 kPa	0 kPa	Lower layer
Slice 39	25.542049 m	0.38374718 m	51.0283 kPa	107.42199 kPa	27.505038 kPa	30 kPa	0 kPa	Lower layer
Slice 40	26.166833 m	0.32102193 m	51.643447 kPa	99.786029 kPa	23.480706 kPa	30 kPa	0 kPa	Lower layer
Slice 41	26.791627 m	0.27074773 m	52.136486 kPa	91.834018 kPa	19.36178 kPa	30 kPa	0 kPa	Lower layer
Slice 42	27.393707 m	0.2338106 m	52.498728 kPa	88.777509 kPa	17.694343 kPa	30 kPa	0 kPa	Lower layer
Slice 43	27.972961 m	0.20930612 m	52.739044 kPa	90.727951 kPa	18.528428 kPa	30 kPa	0 kPa	Lower layer
Slice 44	28.552115 m	0.1953942 m	52.875478 kPa	92.519765 kPa	19.335811 kPa	30 kPa	0 kPa	Lower layer
Slice 45	29.131269 m	0.19205485 m	52.908227 kPa	94.153138 kPa	20.116487 kPa	30 kPa	0 kPa	Lower layer
Slice 46	29.710423 m	0.19928472 m	52.837324 kPa	95.627742 kPa	20.870281 kPa	30 kPa	0 kPa	Lower layer
Slice 47	30.052015 m	0.20722652 m	0 kPa	97.008061 kPa	47.313993 kPa	30 kPa	0 kPa	Lower layer
Slice 48	30.393627 m	0.22141226 m	0 kPa	88.14019 kPa	42.988843 kPa	30 kPa	0 kPa	Lower layer
Slice 49	30.972821 m	0.2517214 m	0 kPa	69.858841 kPa	34.072433 kPa	30 kPa	0 kPa	Lower layer
Slice 50	31.552015 m	0.29266112 m	0 kPa	51.171918 kPa	24.958212 kPa	30 kPa	0 kPa	Lower layer
Slice 51	32.131209 m	0.34427274 m	0 kPa	32.070585 kPa	15.641869 kPa	30 kPa	0 kPa	Lower layer
Slice 52	32.710403 m	0.40660871 m	0 kPa	12.545049 kPa	6.118629 kPa	30 kPa	0 kPa	Lower layer

Limit equilibrium slope stability analysis reinforced fill slope (126+000), **F.S = 1.500**



Project Settings

Unit System: International System of Units (SI)

Analysis Settings

Slope Stability

- Kind: SLOPE/W
- Method: Bishop
- Settings
  - PWP Conditions from: (none)
  - Unit Weight of Water: 9.807 kN/m³
- Slip Surface
  - Direction of movement: Right to Left
  - Use Passive Mode: No
  - Slip Surface Option: Entry and Exit
  - Critical slip surfaces saved: 1
  - Optimize Critical Slip Surface Location: No
  - Tension Crack Option: (none)
- Distribution
  - F of S Calculation Option: Constant
- Advanced
  - Geometry Settings
    - Minimum Slip Surface Depth: 0.1 m
    - Number of Slices: 30
  - Factor of Safety Convergence Settings
    - Maximum Number of Iterations: 100
    - Tolerable difference in F of S: 0.001

Materials

Fill Material

- Model: Mohr-Coulomb
- Unit Weight: 16 kN/m³
- Cohesion': 10.6 kPa
- Phi': 31.01 °
- Phi-B: 0 °

In-situ Soil

- Model: Mohr-Coulomb
- Unit Weight: 16 kN/m³
- Cohesion': 45 kPa
- Phi': 26 °
- Phi-B: 0 °

Foundation Soil

- Model: Mohr-Coulomb
- Unit Weight: 16 kN/m³
- Cohesion': 30 kPa
- Phi': 28 °
- Phi-B: 0 °

Slip Surface Entry and Exit

Left Type: Range
Left-Zone Left Coordinate: (2,240.7571, 8.825172) m
Left-Zone Right Coordinate: (2,243.5842, 14.000073) m
Left-Zone Increment: 8
Right Type: Range
Right-Zone Left Coordinate: (2,245.3347, 17.999889) m
Right-Zone Right Coordinate: (2,253.1818, 17.930551) m
Right-Zone Increment: 8
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (2,235, 8.73182) m
Right Coordinate: (2,263, 9) m

Seismic Coefficients

Horz Seismic Coef.: 0.1
Vert Seismic Coef.: 0.1

Reinforcements

Reinforcement 1

Type: Geosynthetic
Outside Point: (2,240.8526, 9) m
Inside Point: (2,242.6515, 8.937181) m
Slip Surface Intersection: (2,242.2173, 8.9523447) m
Length: 1.7999965 m
Direction: 178 °
F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes
Pullout Resistance: 75 kPa
Resistance Reduction Factor: 1.5
Tensile Capacity: 150 kN
Reduction Factor: 1.5
Factored Pullout Resistance: 50 kN/m
Max. Pullout Force: 89.999826 kN
Pullout Force: 14.487435 kN
Pullout Force per Length: 33.343126 kN/m
Available Length: 0.4344954 m
Required Length: 0.4344954 m
Governing Component: Pullout Resistance

Reinforcement 2

Type: Geosynthetic
Outside Point: (2,241.3989, 10) m
Inside Point: (2,245.5963, 9.853423) m
Slip Surface Intersection: (2,245.0166, 9.8736668) m
Length: 4.1999585 m
Direction: 178 °
F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes
Pullout Resistance: 75 kPa
Resistance Reduction Factor: 1.5
Tensile Capacity: 150 kN
Reduction Factor: 1.5
Factored Pullout Resistance: 50 kN/m
Max. Pullout Force: 100 kN
Pullout Force: 19.340919 kN
Pullout Force per Length: 33.343126 kN/m
Available Length: 0.58005718 m
Required Length: 0.58005718 m
Governing Component: Pullout Resistance

Reinforcement 3

Type: Geosynthetic
Outside Point: (2,241.9452, 11) m
Inside Point: (2,246.9421, 10.823743) m
Slip Surface Intersection: (2,246.5284, 10.838336) m
Length: 5.0000076 m
Direction: 177.98 °
F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes
Pullout Resistance: 75 kPa
Resistance Reduction Factor: 1.5
Tensile Capacity: 150 kN
Reduction Factor: 1.5
Factored Pullout Resistance: 50 kN/m
Max. Pullout Force: 100 kN
Pullout Force: 13.802732 kN
Pullout Force per Length: 33.343126 kN/m
Available Length: 0.41396036 m
Required Length: 0.41396036 m
Governing Component: Pullout Resistance

Reinforcement 4

Type: Geosynthetic
Outside Point: (2,242.4915, 12) m
Inside Point: (2,247.6883, 11.816797) m
Slip Surface Intersection: (2,247.6086, 11.819605) m
Length: 5.2000282 m
Direction: 177.98 °
F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes
Pullout Resistance: 75 kPa
Resistance Reduction Factor: 1.5
Tensile Capacity: 150 kN
Reduction Factor: 1.5
Factored Pullout Resistance: 50 kN/m
Max. Pullout Force: 100 kN
Pullout Force: 2.6577383 kN
Pullout Force per Length: 33.343126 kN/m
Available Length: 0.079708734 m
Required Length: 0.079708734 m
Governing Component: Pullout Resistance

Reinforcement 5

Type: Geosynthetic
Outside Point: (2,243.0378, 13) m
Inside Point: (2,248.6344, 12.804561) m
Slip Surface Intersection: (2,248.4217, 12.81199) m
Length: 5.6000114 m
Direction: 178 °
F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes
Pullout Resistance: 75 kPa
Resistance Reduction Factor: 1.5
Tensile Capacity: 150 kN
Reduction Factor: 1.5
Factored Pullout Resistance: 50 kN/m
Max. Pullout Force: 100 kN
Pullout Force: 7.0973323 kN
Pullout Force per Length: 33.343126 kN/m
Available Length: 0.21285744 m
Required Length: 0.21285744 m
Governing Component: Pullout Resistance

Reinforcement 6

Type: Geosynthetic
Outside Point: (2,243.5842, 14) m
Inside Point: (2,249.5805, 13.790601) m
Slip Surface Intersection: (2,249.0443, 13.809327) m
Length: 5.9999551 m
Direction: 178 °
F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes
Pullout Resistance: 75 kPa
Resistance Reduction Factor: 1.5
Tensile Capacity: 150 kN
Reduction Factor: 1.5
Factored Pullout Resistance: 50 kN/m
Max. Pullout Force: 100 kN
Pullout Force: 17.890576 kN
Pullout Force per Length: 33.343126 kN/m
Available Length: 0.53655964 m
Required Length: 0.53655964 m
Governing Component: Pullout Resistance

Reinforcement 7

Type: Geosynthetic
Outside Point: (2,244.0668, 15) m
Inside Point: (2,250.2631, 14.787113) m
Slip Surface Intersection: (2,249.5182, 14.812704) m
Length: 6.199956 m
Direction: 178.03 °
F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes
Pullout Resistance: 75 kPa
Resistance Reduction Factor: 1.5
Tensile Capacity: 150 kN
Reduction Factor: 1.5
Factored Pullout Resistance: 50 kN/m
Max. Pullout Force: 100 kN
Pullout Force: 24.850487 kN
Pullout Force per Length: 33.343126 kN/m
Available Length: 0.74529567 m
Required Length: 0.74529567 m
Governing Component: Pullout Resistance

Reinforcement 8

Type: Geosynthetic
Outside Point: (2,244.4895, 16) m
Inside Point: (2,251.4852, 15.755702) m
Slip Surface Intersection: (2,249.8562, 15.81259) m
Length: 6.9999643 m
Direction: 178 °
F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes
Pullout Resistance: 75 kPa
Resistance Reduction Factor: 1.5
Tensile Capacity: 150 kN
Reduction Factor: 1.5
Factored Pullout Resistance: 50 kN/m
Max. Pullout Force: 100 kN
Pullout Force: 54.350127 kN
Pullout Force per Length: 33.343126 kN/m
Available Length: 1.630025 m
Required Length: 1.630025 m
Governing Component: Pullout Resistance

Reinforcement 9

Type: Geosynthetic
Outside Point: (2,244.9121, 17) m
Inside Point: (2,252.4076, 16.74001) m
Slip Surface Intersection: (2,250.0214, 16.822779) m
Length: 7.5000077 m
Direction: 178.01 °
F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes
Pullout Resistance: 75 kPa
Resistance Reduction Factor: 1.5
Tensile Capacity: 150 kN
Reduction Factor: 1.5
Factored Pullout Resistance: 50 kN/m
Max. Pullout Force: 100 kN
Pullout Force: 66.686251 kN
Pullout Force per Length: 33.343126 kN/m
Available Length: 2.3876725 m
Required Length: 2 m
Governing Component: Tensile Capacity

Reinforcement 10

Type: Geosynthetic
Outside Point: (2,245.2372, 17.769099) m
Inside Point: (2,252.8326, 17.503863) m
Slip Surface Intersection: (2,250.1379, 17.597964) m
Length: 7.6000297 m
Direction: 178 °
F of S Dependent: Yes
Force Distribution: Distributed
Face Anchorage: Yes
Pullout Resistance: 75 kPa
Resistance Reduction Factor: 1.5
Tensile Capacity: 150 kN
Reduction Factor: 1.5
Factored Pullout Resistance: 50 kN/m
Max. Pullout Force: 100 kN
Pullout Force: 66.686251 kN
Pullout Force per Length: 33.343126 kN/m
Available Length: 2.6963683 m
Required Length: 2 m
Governing Component: Tensile Capacity

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 12 kN/m³
Direction: Vertical

Coordinates

	X	Y
	2,245.4 m	18.15439 m
	2,245.4027 m	18.749105 m
	2,252.1946 m	18.749105 m

Points

	X	Y
Point 1	2,245.4 m	18.15439 m
Point 2	2,253.485 m	17.92183 m
Point 3	2,252.5639 m	17.2394 m
Point 4	2,252.0798 m	16.12676 m
Point 5	2,250.3854 m	14.98993 m
Point 6	2,249.4899 m	13.61122 m
Point 7	2,247.4325 m	11.50687 m
Point 8	2,246.1497 m	9.88629 m
Point 9	2,244.9394 m	9.64441 m
Point 10	2,243.3419 m	9.20903 m
Point 11	2,242.3496 m	8.70109 m
Point 12	2,240.7061 m	8.73182 m
Point 13	2,243.8493 m	14.48533 m
Point 14	2,242.4096 m	8.73182 m
Point 15	2,253.735 m	18.17183 m
Point 16	2,256.735 m	21.17183 m
Point 17	2,258.235 m	21.02183 m
Point 18	2,261.235 m	24.02183 m
Point 19	2,262.735 m	23.87183 m
Point 20	2,262.735 m	8.73182 m
Point 21	2,235 m	8.73182 m
Point 22	2,235 m	6 m
Point 23	2,263 m	6 m
Point 24	2,263 m	9 m
Point 25	2,262.735 m	9 m

Regions

	Material	Points	Area
Region 1	Fill Material	1,13,12,14,10,9,8,7,6,5,4,3,2	54.519 m^2
Region 2	In-situ Soil	14,20,25,19,18,17,16,15,2,3,4,5,6,7,8,9,10	159.42 m^2
Region 3	Foundation Soil	12,21,22,23,24,25,20,14	76.562 m^2

Slip Results

Slip Surfaces Analysed: 396 of 405 converged

Current Slip Surface

Slip Surface: 30
Factor of Safety: 1.500
Volume: 45.257995 m^3
Weight: 724.12792 kN
Resisting Moment: 7,168.5667 kN·m
Activating Moment: 4,780.4484 kN·m
Slip Rank: 1 of 405 slip surfaces
Exit: (2,240.7571, 8.8251724) m
Entry: (2,250.2008, 18.016299) m
Radius: 9.5356794 m
Center: (2,240.6713, 18.360466) m

Slip Slices

	X	Y	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength	Suction Strength	Base Material
Slice 1	2,240.9179 m	8.8293332 m	0 kPa	4.8738559 kPa	2.9296659 kPa	10.6 kPa	0 kPa	Fill Material
Slice 2	2,241.2395 m	8.8430948 m	0 kPa	14.841926 kPa	8.921455 kPa	10.6 kPa	0 kPa	Fill Material
Slice 3	2,241.5611 m	8.8677692 m	0 kPa	24.150816 kPa	14.517012 kPa	10.6 kPa	0 kPa	Fill Material
Slice 4	2,241.8827 m	8.9034419 m	0 kPa	32.861354 kPa	19.7529 kPa	10.6 kPa	0 kPa	Fill Material
Slice 5	2,242.2043 m	8.9502383 m	0 kPa	41.241577 kPa	24.790237 kPa	10.6 kPa	0 kPa	Fill Material
Slice 6	2,242.5259 m	9.0083254 m	0 kPa	48.754819 kPa	29.306434 kPa	10.6 kPa	0 kPa	Fill Material
Slice 7	2,242.8476 m	9.0779155 m	0 kPa	56.243137 kPa	33.807648 kPa	10.6 kPa	0 kPa	Fill Material
Slice 8	2,243.1751 m	9.1610058 m	0 kPa	63.423429 kPa	38.123708 kPa	10.6 kPa	0 kPa	Fill Material
Slice 9	2,243.3938 m	9.2218586 m	0 kPa	63.858084 kPa	31.145669 kPa	45 kPa	0 kPa	In-situ Soil
Slice 10	2,243.6475 m	9.303635 m	0 kPa	73.019838 kPa	43.892093 kPa	10.6 kPa	0 kPa	Fill Material
Slice 11	2,244.0044 m	9.4278105 m	0 kPa	81.150532 kPa	48.779438 kPa	10.6 kPa	0 kPa	Fill Material
Slice 12	2,244.3145 m	9.5497942 m	0 kPa	89.148033 kPa	53.586721 kPa	10.6 kPa	0 kPa	Fill Material
Slice 13	2,244.6247 m	9.6845771 m	0 kPa	96.536826 kPa	58.028111 kPa	10.6 kPa	0 kPa	Fill Material
Slice 14	2,244.9348 m	9.832767 m	0 kPa	103.73476 kPa	62.354777 kPa	10.6 kPa	0 kPa	Fill Material
Slice 15	2,245.2449 m	9.9950778 m	0 kPa	110.32989 kPa	66.319099 kPa	10.6 kPa	0 kPa	Fill Material
Slice 16	2,245.4014 m	10.080631 m	0 kPa	273.3701 kPa	164.32227 kPa	10.6 kPa	0 kPa	Fill Material
Slice 17	2,245.5626 m	10.176997 m	0 kPa	116.9204 kPa	70.280639 kPa	10.6 kPa	0 kPa	Fill Material
Slice 18	2,245.8825 m	10.37699 m	0 kPa	111.73694 kPa	67.164872 kPa	10.6 kPa	0 kPa	Fill Material
Slice 19	2,246.2024 m	10.595309 m	0 kPa	106.30602 kPa	63.900355 kPa	10.6 kPa	0 kPa	Fill Material
Slice 20	2,246.5222 m	10.833554 m	0 kPa	100.60625 kPa	60.474237 kPa	10.6 kPa	0 kPa	Fill Material
Slice 21	2,246.8421 m	11.093692 m	0 kPa	94.540874 kPa	56.828348 kPa	10.6 kPa	0 kPa	Fill Material
Slice 22	2,247.162 m	11.378181 m	0 kPa	88.221588 kPa	53.029837 kPa	10.6 kPa	0 kPa	Fill Material
Slice 23	2,247.4819 m	11.690154 m	0 kPa	81.536107 kPa	49.011206 kPa	10.6 kPa	0 kPa	Fill Material
Slice 24	2,247.8017 m	12.033703 m	0 kPa	74.421455 kPa	44.734602 kPa	10.6 kPa	0 kPa	Fill Material
Slice 25	2,248.1216 m	12.414348 m	0 kPa	66.834137 kPa	40.173879 kPa	10.6 kPa	0 kPa	Fill Material
Slice 26	2,248.4415 m	12.839848 m	0 kPa	58.668946 kPa	35.265797 kPa	10.6 kPa	0 kPa	Fill Material
Slice 27	2,248.7613 m	13.321742 m	0 kPa	49.764885 kPa	29.913582 kPa	10.6 kPa	0 kPa	Fill Material
Slice 28	2,249.0812 m	13.87864 m	0 kPa	39.96498 kPa	24.022877 kPa	10.6 kPa	0 kPa	Fill Material
Slice 29	2,249.4011 m	14.544648 m	0 kPa	28.812937 kPa	17.319405 kPa	10.6 kPa	0 kPa	Fill Material
Slice 30	2,249.7209 m	15.399331 m	0 kPa	15.533258 kPa	9.337013 kPa	10.6 kPa	0 kPa	Fill Material
Slice 31	2,250.0408 m	16.952207 m	0 kPa	-5.0715077 kPa	-3.0484741 kPa	10.6 kPa	0 kPa	Fill Material