SLOPE STABILITY EVALUATION OF NEWLY CONSTRUCTED RAILWAY EMBANKMENT ALONG LAGOS-IBADAN RAIL LINE, NIGERIA

BY

ALHASSAN, Alfa Abdulrasheed
MEng/SEET/2017/7168

DEPARTMENT OF CIVIL ENGINEERING
FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA

AUGUST, 2021

SLOPE STABILITY EVALUATION OF NEWLY CONSTRUCTED RAILWAY EMBANKMENT ALONG LAGOS-IBADAN RAIL LINE, NIGERIA

BY

ALHASSAN, Alfa Abdulrasheed
MEng/SEET/2017/7168

A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA, NIGERIA IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE OF MASTER OF ENGINEERING IN CIVIL ENGINEERING (GEOTECHNICS)

AUGUST, 2021

ABSTRACT

This research is aimed to evaluate stability of the slope of newly constructed railway embankment along Lagos-Ibadan rail line. Soils samples were collected from KM75+500, KM90+500, KM105+500 and KM116+500 at depths ranging from 0m to 5.5m and 5.5m to 10m and taken to laboratory for tests such as particle size analysis, Atterberg limits, standard Proctor and direct shear test. The slope stability assessment was done for the high filled embankments in KM75+500, KM90+500, KM105+500 and KM116+500. Geotechnical parameters were derived from the direct shear test as inputs for slope/W software used for the slope stability assessment. The results showed that the fill materials for the embankments from 0m to 5.5m was characterized as A-2-6 (clayey soil) subgroup in AASHTO classification system comprises of 78% of sand and 17% of fine particles and is considered as average soil according to International Union of Railways specifications. The subgrade materials from 5.5m to 10m consisting of 51% sand and 49% fines is classified as A-6 (clay soil), with high liquid limit of 41% and plastic index of 19% at KM116+500, and is regarded as poor soil according to railway specifications that requires stabilization. The factor of safety of the embankments according U.S. Army Corps of Engineers shows the slope is stable at KM75+500, KM90+500 and KM105+500 has factor of safety greater than 1.3 but slope at KM116+500 is unstable as the factor of safety is less than 1.3. It was also noticed that there was increases in factor of safety with decrease in embankment height. Slope protection and soil treatment be provided KM116+500 embankments.

TABLE OF CONTENTS

CHAPTER THREE

CHAPTER FOUR

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

BSM	Bishop Simplified Method
Fm	Moment equilibrium line
FS	Factor of Safety
FOS	Factor of Safety
Ft	Force equilibrium and
JGM	Jambus General Method
JSM	Jambu Simplified Method
KM	Kilometre
LEM	Limit Equilibrium Method
MP-M	Morgenstern-Price Method
SM	Spencer Method
UIC	Union of International Railway Corporation

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the Study

As the population grows, so does the demand for quick and safe transportation. The railroad sector grew rapidly as an alternative system, and it now serves as the world's largest transportation system. In addition to conventional tracks, the introduction of new Significant Speed Trains resulted in a high demand for corresponding railroads capable of tolerating heavier loads and a greater speed.

Because of a multitude of circumstances, effectively modeling slope stability difficulties is difficult. One of the most fundamental concerns is that slope behavior cannot be predicted precisely. Engineers utilize a factor of safety methodology to reduce the danger of slope failure. When dealing with fluctuating uncertainties, such as slope stability, deterministic approaches give a systematic technique. The stability of a slope has a significant impact on the area surrounding it, because when a slope collapses, it frequently endangers human lives or causes significant material damage. As a result, one of the most important areas of practical usage in soil engineering is slope stability analysis (Capper, 1976).

Nigeria's federal government has recently begun establishing standardized and broad inland transportation networks that are accessible to both passengers and enterprises. The Federal Ministry of Transport planned many high-speed rail lines around the country to achieve this goal. For economic reasons, the Lagos-Ibadan rail line extension to Apapa port from Ebute-Meta is a priority.

Subgrade is a phrase used to describe the grade line that is elevated that is ready to receive subballast in construction (Dingqing *et al*. 2015). Because it provides a strong foundation for the track superstructure, subgrade is an important feature of railway systems. It is necessary to avoid progressive shear failure, excessive swelling and shrinking, and subgrade attrition (Selig and Waters, 1994)

The Lagos-Ibadan section is part of Nigeria's 1,343-kilometer Lagos–Kano Standard Gauge Railway, which was been constructed. It is projected to connect the country's two main cities, Lagos in the southwest and Kano on the Niger border in the north, if completed. It will also connect the country's third largest city, Ibadan (also in the southwest), and the national capital Abuja. The railway will run parallel to the Cape gauge line, which was built by the British and has a lower design capacity and is in poor condition (Israel, 2014).

The conversion of single tracks to double or multiple lines, as well as the renovation of old rails to allow high-speed trains, necessitates a large number of materials. It has also become critical to ensure that the quality of such construction meets the requirements for safe, comfortable, and cost-effective train passage under specified conditions. Furthermore, with such a large building volume, finding good quality materials at a reasonable price is a difficult task.

The limit equilibrium approach is adopted to assess slope stability based on assumptions about the sliding surface geometry. The method is popular because it is easy and only requires a few parameters (geology, slope geometry, topography, geotechnical factors, static and surcharge loads and hydrogeologic conditions). The stability of natural and artificial slopes, such as road/railway embankments, hydraulically produced dams, earth dams, and so on, is a major concern in geotechnical engineering.

This research aimed to evaluate stability of the slopes of newly constructed railway embankment from KM75+500 to KM116+500 along Lagos-Ibadan rail line. Limit equilibrium approach has been used to examine the slope stability state of the railway embankment, taking into account the strength parameters of the materials of the newly constructed railway embankment located along the study region.

1.2 Problem Statement of the Research Problem

The stability of slope is a main concern where movements of existing slopes would have an effect on the safety of people and property. Although, slope analysis had been carried out during preliminary design of the embankment, but the possible of slope failure is still high after construction. The aforementioned phenomena occur as a result of either an incorrect approach to assessing their stability, or errors made during geotechnical investigations, incorrect assumptions made during the calculation phase, or improper machine placement on the slope surcharge, or material deterioration over time after construction. One of the causes of faulty slope stability assessments could be an incorrect estimation of the slope's geological structure (Das, 2011)

1.3 Justification of Study

The motivation driving toward this study is closely related to the assessing the slope stability of railway embankment and the important role slope/w plays in stability analysis of slopes in civil engineering applications and design. Furthermore, evaluating the slope stability of newly constructed railway embankment was to provide the current stability status of the embankment.

This study is intended to evaluate the slope stability of the newly constructed railway embankment along Lagos-Ibadan rail line and in the same time, propose some stabilization measures based on the field investigations and stability assessment of the slope. In other to reduce the problems that could occur after the construction of Lagos-Ibadan railway modernization project, it is essential to identify and assess those sections that could be prone to slope failure periodically.

1.4 Aim and Objectives of the Study

The aim of this research is to evaluate stability of the slope of newly constructed railway embankment along Lagos-Ibadan rail line.

This aim has to fulfill these objectives;

1. To determine the geotechnical characteristics of the embankments at KM75+500, KM90+500, KM105+500 and KM116+500
2. To evaluate the safety factor for slopes at chanaige KM75+500, KM90+500 KM105+500 and KM116+500 embankments using Slope/W 2018 from GeoStudio
3. To determine potential failure mechanisms

1.5 Scope of the study

The new railway embankments selected is from KM75+500 to KM116+500 along Lagos-Ibadan rail line. Samples were collected at KM75+500, KM90+500, KM105+500 and KM116+500 at depths ranging from 0m to 10m

To determine the factor of safety, the Slope/W software was used to evaluate the slopes of varying heights 10m, 7m and 5m and side slope ratio 1:1.5 and 1:1.75 of berm based on the limit equilibrium of the Morgenstern-Price technique.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Preamble

The purpose computer programs are used to undertake a wide range of slope stability evaluations. Soil properties, pore pressure, slip surfaces, and analytical techniques are just a few of the options and aspects to consider. Each of these options and characteristics has sub combinations that lead to hundreds of possible options and features in a computer program for complete slope stability. Obviously, sophisticated computer algorithms can't be evaluated for every possible data combination, or even a significant part of the possibilities (Duncan and Wright, 2005).

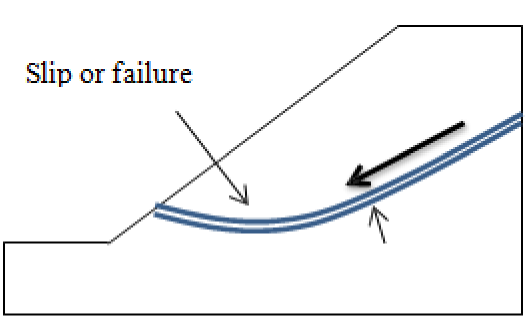
2.2 Types of Slop Failures in an Embankment

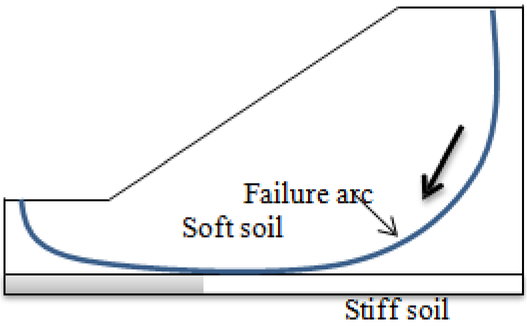
Failures of a Slope are influenced by a variety of factors, soil type, soil stratification, slope geometry, seepage and ground water (Budhu, 2000).

Figure 2.1(I) depicts a translational slide that happens when a slope falls along a weak soil zone. Translational slides are frequent in coarse-grained soils. In this condition, the sliding mass can travel a long distance before coming to a halt. A rotational slide failure type is observed in homogeneous fine-grained soils, the point of rotation on the axis parallel to the slope (Duncan and Wright, 2005).

The following are common three types of rotational failure:

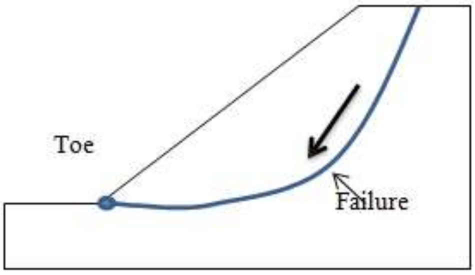
An arc enveloping the entire slope causes a base slide. The soft soil layer base sitting on a stiff soil layer can easy fail and it goes below the toe Figure 2.1(ii). In a toe slide, the failure surface passes through the slope's toe Figure 2.1(iii). The failing surface follows through the slope and above the toe in a slop slide Figure 2.1(iv)

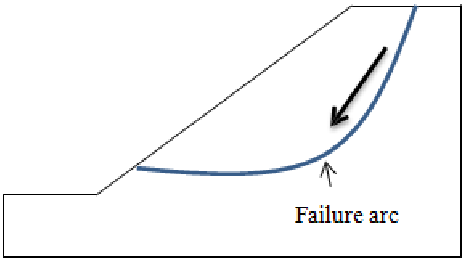




(i): Soil mass movement along a thin Layer of weak soil

(ii): Base slide





(iii): Toe slide

(iv): Slope slide.

Figure 2.1: Modes of slope failure (Duncan and Wright, 2005; Budhu, 2000

Understanding the reasons of slope instability is critical for designing, constructing, and repairing failed and damaged slopes.

In most cases, multiple factors are present at the same time. Water affects the slope in a different dimension, causing it impossible to isolate certain effect. Furthermore, clay soil has complex and unexpected behavior, whether due to softening, gradual failure, or combined.

Trying to figure out which one caused the breakdown it is difficult and also it's also incorrect (Duncan and Wright, 2005). As a result, in planning and constructing new slopes, it is necessary to consider any changes in qualities and situations that may affect the structure over its lifetime so that it stays stable in the face of any changes.

The soil's shear strength must be larger than the shear stress required for equilibrium to prevent slope failure. Two mechanisms can be used to achieve the instability condition (Duncan and Wright, 2005):

Moisture content, pore pressure, void ratio, sustained loads under creep, and weathering all reduce shear strength, or the maximum shear stress that the soil can withstand. Another process is an increase in shear stress as a result of increased water pressure generating soil saturation and a reduction in water level.

2.3 Slope Stability Principles

Engineers and researchers choose the Slice Methods of Limit Equilibrium to analyses stability of slope because they are classic and well-established. The Swedish or Ordinary method of slices was introduced by Fellenius (1936). In the 1950s, Bishop (1955) and Janbu (1954) made breakthroughs in the method. Electronic computers made it easier to manage the method's iterative procedures in the 1960s, resulting in quantitatively more explicit formulations such as those established by Spencer (1967), and Morgenstern and Price (1965).

One of the reasons for the limit equilibrium method's popularity is that solutions can be generated by hand calculations. To achieve solutions, simplifying assumptions had to be made, but the concept of numerically breaking a bigger body into smaller pieces for analytic purposes was new at the time. Through a series of parametric simulations, Rahardjo *et al*. (2007) found out the relative impact of soil characteristics, initial water table location, rainfall intensity, and slope geometry in producing instability of a homogeneous soil slope under varied rainfall. Sinha (2008) emphasizes the importance of advanced slope stability analysis for cost-effective earth embankment design and explores the concepts and theories involved in various slope stability analysis methodologies for railroad embankments. According to Abdoullah (2010), utilizing a mixed soil methodology to achieve slope load sustainability is a suitable method for slope building technology.

Slope stability is one of geotechnical engineering's oldest and, possibly, most-studied and least-understood topics. Duncan *et al*. (2014) says “In civil engineering, assessing the stability of slopes in soil is an important, intriguing, and difficult task”. In terms of theoretical, statistical, analytical, experimental and numerical techniques, there are various works in this topic, potentially beginning with Terzaghi's 1950, and they're still going strong today. Knowing what methods are available and what constraints they have becomes a crucial topic since solving slope stability problems needs comprehending the analytical methods and their application. Only after a comprehensive investigation and analysis of a slope stability problem can effective remedial therapies be employed.

Salokangas and Vepsalainen (2009) investigated the stability of a historic railway embankment built on very soft ground in a comparative study. Through a limit equilibrium technique, in southern Finland, the stability of this embankment was studied by increasing axle load and calculating and comparing safety factors. All calculations were performed using the soft layer's undrained shear strength parameters and Slope/W as the analytical program. The embankment's position over relatively soft clay ground presented a challenge for this project, making the analysis more complicated and sensitive. The study's findings had a considerable impact on the concept of embankment stability. It was said that soil strength plays the most important role in embankment stability, regardless of the method or kind of analysis utilized, and slopes with undrained shear strength of the soil are less secure than slopes with adequate strength parameters. Researchers also had to cope with a difficult problem requiring the study of limit equilibrium calculations using effective parameters. During the train loading, the pore pressure was held constant and determined at the measuring instant in limit equilibrium type modeling; nevertheless, in reality, this pressure would build during this period till failure happens. Due to the impossibility to define the excess pore pressure in fully saturated situations, slope/W was recommended as a solution.

Assefa *et al.* (2017) used three distinct stochastic methodologies to examine the stability of the new railway embankment in Ethiopia, using commercially accessible finite element and finite difference programs. Due to the solid foundation and deep groundwater table, there was no evidence of liquefaction.

Jain and Jain (2015) investigated the effects of different types of natural soils on a railway embankment in India. It has been discovered that as the height of the embankment rises, the factor of stability decreases. In addition, when the safety factor is not up to 1.4, the factor of safety for soft clay is quite low.

2.3.1 Analysis of failure

The following equation can be used to calculate the failure safety factor at any slope surface:

$$F=\frac{\text{resistance shear}}{\text{causing shear}} \tag{2.1}$$

As shown in the next equation, the ultimate shear is equal to the total of the critical shearing stresses.

$$\tau = c + \sigma \tan\varphi \tag{2.2}$$

Many approaches investigated the slip surface using a specific number of slices in order to establish crucial safety criteria such:

1. Ordinary method

2. Bishop's Simplified Method

3. Janbu's Simplifies Metho.

4. Spencer method.

5. Morgenstern - Price Method (1965).

They all based their principles on tangential and normal force limit equilibrium on the slip surface, as shown in Figure 2.3.

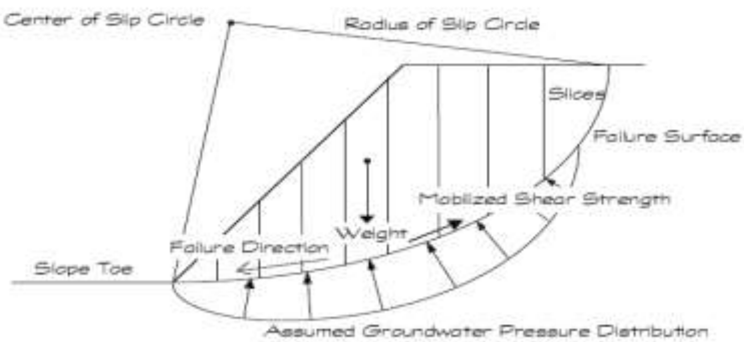


Figure 2.2: slip surface (Sinha 2008)

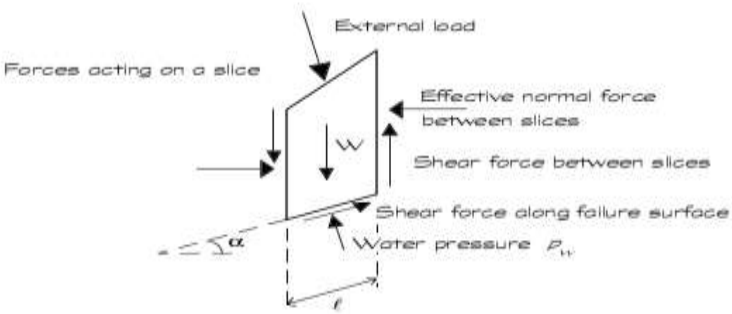


Figure 2.3: Forces on slice from slip surface (Sinha 2008)

2.4 Limit Equilibrium Methods

The Ordinary approach is only suitable for hand calculations and demonstrations, but Bishop Simplified and Janbu's Simplified methods have been frequently used for stability evaluations for many years. These approaches are widely used because the safety factor can usually be determined with accuracy sufficiently. These approaches, however, have limits in terms of meeting force and moment balance. Bishop Simplified technique, according to Abramson *et al.* (2002), should be used only for circular shear surface analysis. Similarly, Janbu's method for assessing the factor of safety for noncircular surfaces is more flexible. The main advantage is that it may be used efficiently in nonhomogeneous slopes and deteriorating surfaces. Moment equilibrium for slices, which is comparable to the Morgenstern-Price methodology, provides the entire result for force equilibrium safety factor.

For circular shear surfaces, the Bishop Simplified technique always offers a greater factor of safety than Janbu's simplified technique and is within 5% of the more stringent methods' factor of safety. When compared to the findings derived using the Spencer and Morgenstern-Price techniques, the factor of safety can change by 15% (Abramson *et al.* 2002)

2.5 Summary of Methods and their Usefulness

2.5.1 Ordinary (Fellelius) method of slices

Nonhomogeneous slopes are application and c-θ soils with a circle can be used to represent a slip surface. It ignores all interslice forces, fails to achieve force equilibrium, and assumes that the resulting inclined interslice forces angle parallel to the slice's base, which is a flaw in the approach (Abramson *et al.*, 2002).

2.5.2 Simplified Bishop's

It applicable to c-θ with non-homogeneous slopes, where the slip surface shows a circle failure pattern. For high pore pressure, it is more precise than the traditional method of slice. It predicted that interslice forces are zero and that force equilibrium and moment are satisfied (Abramson *et al*., 2002).

2.5.3 Janbu's

The interslice shear force is assumed zero. Even if the moment equilibrium is not satisfied, it satisfies both force and force equilibrium (Abramson *et al*., 2002).

2.5.4 Spencer's

It is applicable to practically all slope shapes as well as soil profiles, and it achieves both static and dynamic equilibrium if the resultant interslice force has a constant but indeterminate inclination. This is the simplest full equilibrium methodology for computing the factor of safety (Abramson *et al*., 2002).

2.5.5 Morgenstern and Price's

It's a precise approach that works on nearly all slope geometries and soil profiles. Except that the inclination of the interslice resultant force is supposed to vary according to the inclination of the interslice resultant force, the procedure is identical to Spencer's. It’s a rigorous and well-established complete equilibrium process (Abramson *et al*., 2002).

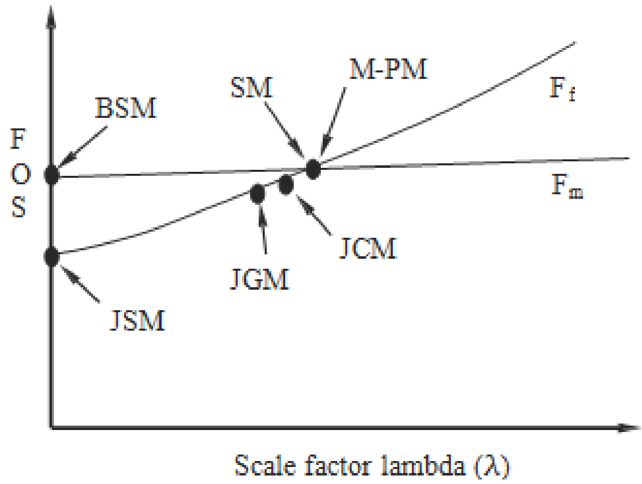


Figure 2.4: Presentation of the methods (Fredlund and Krahn 1977)

2.6 Critical Slip Surface Location

The application of any of the slicing methods and the assumption of a specific form and location for the failure surface of a slope only produce one value of the safety factor, or collapse load. It's possible that this isn't the lowest number, and hence the failure surface isn't the most crucial. As a result, numerous probable failure surfaces must be tested, as well as the calculations for each slope. The failure surface is frequently thought to pass through the slope toe; however this is not a requirement for failure (Bromhead, 1992).

Finding the local minimum of the safety factor or collapse load can be a difficult mathematical challenge. The definition of a grid in which each point represents the center of a circular arc is a frequent search approach for circular failure surfaces. For each node, different radii of the failure surface are then evaluated, and the appropriate factor of safety for each of these probable slip circles is determined. The values of the minimal safety factor in each node are contoured on the defined grid after a number of potential slip surfaces have been analyzed (Figure 2.5). A local minimum of safety has been found if closed contours are established. A broader search grid must be defined if this is not the case. The computed local minimum does not have to be the only value of the safety factor's minimum value. There may be numerous local minima, but the critical slip surface is only one. Although the gird-search method is straightforward, it can be time consuming when dealing with huge situations (Bromhead, 1992).

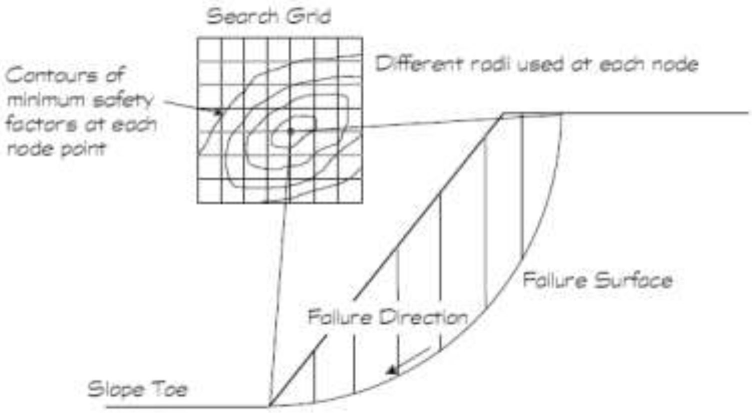


Figure 2.5: Grid search pattern to locate the critical slip surfaces (Mostyn and Small, 1987)

2.7 Factor of Safety

In slope stability and indeed in the subject of geotechnical in particular, a soil's shear strength is frequently questioned. Because the loading is usually merely the slope's self-weight, it is more precisely known. As a result, the safety factor is computed as a ratio of available shear strength to the shear strength needed to keep the slope stable. The most basic purpose of a slope stability study is to determine a safety factor for future failure. The slope is considered stable if this safety factor is high enough (safe). If it's less than 1.0, it's unsafe (Duncan and Wright, 2005). The US Army Corps of Engineers established a safety factor guideline of Slope Stability Manual Table 2.1, based on their experience

Table 2.1: Manual for slope stability of United States Army Corps of Engineers safety factor

Types of Slopes	Required factors of safety		
	For end of construction	long-term For steady seepage	for rapid drawdown
All types	1.3	1.5	1.0-1.2

Source: U. S Army Corps of Engineers manual, (2003)

2.8 Railway Embankment

Subgrade is the support foundation for everything above it, and it is generally the most changeable and weakest of track components. For this reason, if the line must be closed for remedial measures, rehabilitation, or if a derailment occurs, the failure of a railway embankment can result in significant financial losses. In the case of an embankment, the subgrade may be borrowed soil, whereas in the case of cuttings, it may be the original soil. The function of the subgrade can be performed by controlling natural soil and embankment filling settlements, as well as providing time-independent mechanical qualities under design train loads and velocity. Using locally accessible, low-cost materials with acceptable engineering qualities to make them functional and cost-effective could be a good approach (Mundrey, 1993). When an embankment's subgrade is course, the track stability is usually guaranteed, and drainage becomes the primary concern, even if the subgrade is not sensitive to moisture fluctuations.

During embankment characterization, type of soil, grain size distribution and mechanical characteristics of strength should be determined. The soil type is the most important factor in determining the subgrade of an embankment. The physical state of the soil, including its moisture content and density, is then determined. Finally, certain physical properties and engineering qualities should be explored. The least problematic subgrade is coarse-grained with mixture of clay or silt. Most sand and gravel subgrades work well with appropriate drainage and appropriate surface compaction. Coarse-grained subgrade, on the other hand, might be troublesome because to failure mechanisms like liquefaction and cyclic movement (Dingqing *et al.* 2015).

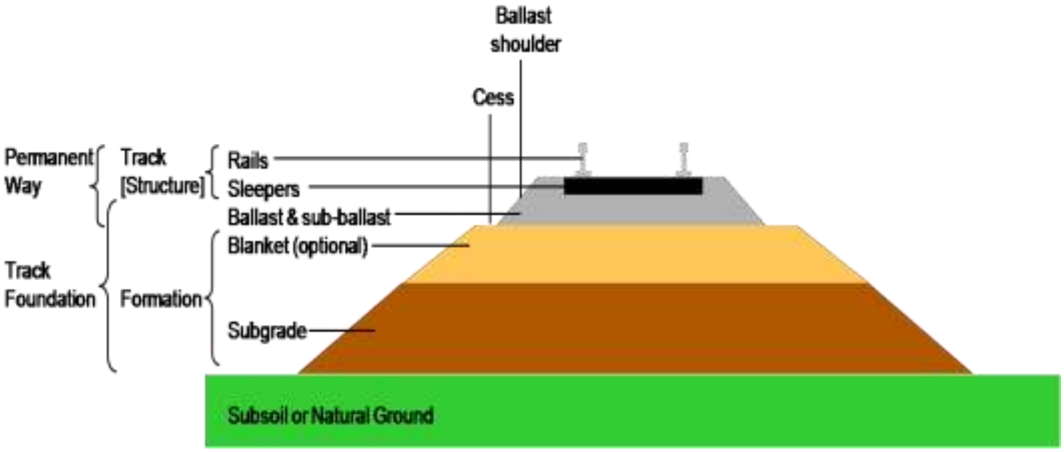


Figure 2.6: Railway embankment cross-section (Chirag *et al*, 2016)

The behavior of the subgrade can be macroscopically described and classed as follows, according to the International Union of Railways (Table 2.2): S0: ‘‘Inappropriate" Soils that need to be removed or stabilized ‘‘Poor soil," says S1. Soils with fines greater than 50%, these soils may be use but drainage should be provided. It's possible that soil improvement is required. ‘‘Average" is the S2, Soils with particles ranging from 12% to 50%. S3: ‘‘Wonderful" Soils with a fines content of less than 12% (Profillidis and Kouparoussos, 2006).

Table 2.2: Specifications for Railway Embankment (UIC)

Layer	Specification	Thickness
Embankment Fill (Top Layer subgrage)	Ip≤10,WL≤40%, fine content≤(20±5)% 25% crushed stone +75% natural soil SQ2/SQ3 Soils	As per Embankment height

Source: UIC (2016)

2.9 Railway Embankment and Surcharge Load

Railway embankment are subjected to stresses coming from traffic, own weight and other stresses due to vibration caused by railway traffic. These stresses or loads are transmitted from the axles to the embankment through the rail, the support sleepers and ballast bed, all have different material. The contact surface between the wheel and the rail has the shape of ellipsis, the load application points differing dependent upon the aligned or curve vehicle movement (Esveld, 2001). Pressure under the sleeper can be obtained from the pressure distribution diagram shown in Table 2.3, this given as:

Table 2.3: Parameters of substructure of standard gauge rail line

Parameter	value
Ballast coefficient C	5.00
Sleeper spacing t (cm)	60.00
Crushed stone ballast thickness (cm)	30.00
Rail inertia momentum Is (cm4)	3055.00
Sleeper inertia momentum It (cm4)	15033.00
Rail elastic modulus Es	2.10E105
Concrete elastic modulus Et	3.35E105
Sleeper width b (cm)	27.50
Dynamic coefficient a	2.00
Speed V (km/h)	200
Equivalent beam length L (cm)	130.00
Weight per axle G (kN)	250.00
Pressure distribution at sleeper base P (kN/m2)	53.00

Source: Esveld (2001)

Pressure, $P=\frac{G}{2b0L}$ (2.3)

Where L is the equivalent beam length

And $L = \sqrt[4]{\frac{4(EsIs+\alpha tEtIt)}{\alpha blC}}$ (2.4)

The embankment is made of fill material which is rest on a firm base. The 200km/h speed train axle loads of 250kN/axle (BS EN 1991-2:2003) will be placed along the axis of symmetry, and influence surface will be imposed to the embankment geometry, in order to affect more on the stability and safety factor.

The stress distribution uses the replacement longitudinal approach, the approach consider the rail as supported on the sleeper on a sufficient length, the track formed by the rail-sleeper is taken as a continuous beam supported on an elastic bed, through a sleeper replacement, which is continuous beam of width having statically the same effect as sleepers situated at distance, sleepers are rigidly joined to the rail and take part in stress take over (Duncan and Wright, 2005).

2.10 Geotechnical Properties of Soil

The geotechnical parameters of the subsoil at the project site must be assessed in order to generate suitable input data for foundation design and construction for the proposed structures (Oke and Amadi, 2008). The following properties are discussed:

2.10.1 Specific gravity

Specific gravity is defined as the quantity of soil solids divided by the mass of an equal volume of water. It is a major soil index attribute that is directly linked to mineralogy or chemical composition and also represents weathering history (Oyediran and Durojaiye, 2011). A higher specific gravity number suggests stronger road and foundation strength; a lower specific gravity value indicates that the soil is unsuitable for use as a construction material. It's also used to calculate the void ratio, saturation level, porosity and other soil characteristics (Prakash and Jain, 2002). Roy and Dass (2014) discovered that raising specific gravity can improve shear strength metrics (cohesion and angle of friction). Table 2.4 lists some typical specific gravity values.

Table 2.4: specific gravity values

Type of soil	Specific gravity
Sand	2.65 to 2.67
Silty sand	2.67 to 2.70
Inorganic clay	2.70 to 2.80
Soil with mica or iron	2.75 to 3.00
Organic soil	1.00 to 2.60

Source: Bowles (2012)

2.10.2 Particle size analysis

Sieve analysis is adopted to determine the proportion of different sizes of soil particles coarser than 75u, while hydrometer analysis is used to assess the percentage of different sizes of soil particles finer than 75u. Particle size distribution curves are plotted based on the particle size analysis. It indicates the gradation of the soil, allowing one to assess whether the soil is properly or poorly graded. As a result, knowing the grain size distribution of each soil is essential for proportioning the selected soils (Prakash and Jain, 2002).

The particle size of sands and silts, according to Raj (2012), has some utility in the construction of filters as well as the assessment of permeability, Frost susceptibility and capillarity. Grain size curves can show a lot of useful information, such as (a) the overall percentage of course or fine than a specific size, and (b) the grain size uniformity or range. The shearing strength of soil is controlled by the sand shape, whether it is changeable or precise (Dafalla, 2013).

2.10.3 Atterberg limits

The content of moisture of a fine-grained soil affects the Atterberg limits of the soil's furthest reaches. A fine-grained soil slurry passes from a fluid state to a plastic state, then to a semi-solid state, and finally to a solid state when the water content decreases. For different soils, the water components at these state transitions are different (Kaniraj, 1988).

Table 2.5: plasticity index based on soils Types

Plasticity index (%)	Type of Soil	Stage of plasticity	Stage of cohesiveness
0	Sand	Non-plastics	Non-cohesive
Less than 7	Silt	Low plastic	partly cohesive
7-17	Silt	clay Medium	plastic Cohesive
Less than 17	Clay	High plastic	cohesive

Source: Prakash and Jain (2012)

Sand has no plastic limit, however fine sand displays slight versatility, (Table 2.5). Soil is supposed to be in the plastic reach when it has water content in the middle of fluid cutoff. The scope of the plastic state is given by the contrast between fluid cutoff and plastic breaking point and is characterized as the versatility list (Prakash and Jain 2012).

Laskar and Pal (2012) found that versatility relies upon grain size of soil. Expansive soils are those that shrink and swell with time. This group includes Indian black cotton soils (Raj, 2012).

2.10.4 Shear strength

Friction and particle bonding, as well as possible cementation or bonding at particle contacts, contribute to soil's shear resistance. The cohesiveness and friction angle of soils are the shear strength parameters. The shear strength of soil is determined by the effective stress, drainage conditions, particle density, strain rate, and strain direction. Consequently, the Shearing strength is influenced by material consistency, mineralogy, grain size distribution, particle form, initial void ratio, and characteristics like as layers, joints, fissures, and cementation (Poulos, 1989).

According to Akayuli *et al*. (2013), a sandy soil has a higher friction angle than its cohesiveness, while a clayey soil has a lower friction angle. In their research, Shanyoug et al. (2009) found that clay content increases overall cohesiveness. Clay particles fill the vacuum spaces between the sand particles as more clay is put into the sandy materials, inducing the sand with integument. More clay is introduced into sandy materials, and the clay particles begin to fill the vacuum areas between the sand particles, causing the sand to interlock. As a result, clayey sand soils are expected to have low cohesion, whereas cohesion increases as clay concentration increases.

Cohesion is mostly owing to the intermolecular link between the adsorbed water around each grain, according to Murthy (2002) and El-Maksoud (2006), especially in fine-grained soils. Soils with high plasticity, such as clayey soils, have higher cohesiveness and lower angle of shearing resistance, according to Mollahasani *et al*. (2011). In contrast, as the grain size of the soil rises, such as in sands, the soil cohesiveness decreases.

2.11 Treatment Methods

Treatment of an embankment fill is defined as a change or modification to one or more of the subgrade's qualities utilizing a variety of approaches. Existing Subgrade, Treatment Methods Chemical additives, mechanical compaction, and biological microorganisms can all be used to stabilize the subgrade. The degree of stabilization is highly dependent on the type of structure to be constructed as well as the soil's technical properties. According to Thomas et al. (2002), Cement and lime were utilized by Otoko and Blessing (2014) to increase the strength and compaction behavior of marine clay. When the soil was compacted, the maximum dry density increased as the amount of cement or lime rose, with a corresponding fall in the optimal moisture content. Furthermore, Soltani-Jigheh and Jafari (2012) discovered that adding gravel to the clay had an effect on both the optimal moisture content and the maximum dry density, with an increase in gravel content the maximum dry density increases while the optimal moisture content reduces.

Slope stabilization and repair methods come in a variety of shapes and sizes, and method selection is site-specific. Shear strength is often increased when groundwater and drainage are managed. Slopes are protected from erosion by surface cover. Excavation and re-grading reduce failure-causing forces. The addition of structural reinforcement to the slope material increases direct supporting forces. Eleven general stabilizing techniques were discovered through research. The stabilizing methods investigated are summarized in Table 2.6, along with a source of background material proving each method's use.

Other treatment options include;

2.11.1 Compaction grouting

Compaction grouting is a technique for forming a grout bulb that displaces the soil around it, increasing the strength of the grout. Compaction grout is a stiff, low-slump mixture of cement, sand, clay, and water that is pumped. Compaction grout columns can transfer stress to a stronger stratum in addition to densifying the soil (Dingqing *et al.*, 2015).

2.11.2 Penetration grouting

Penetration grouting, also known as permeation grouting, involves injecting Portland cement or chemical grout into the soil to increase soil strength while also lowering permeability. Because of the relatively wide holes and high permeability, this approach is ideal for granular soil. Some low-viscosity compounds, such as certain urethanes, can, however, penetrate fine-grained dirt (Dingqing *et al.*, 2015).

2.11.3 Soil mixing

Soil mixing is a technique for improving soil by mechanically mixing it with cementitious binding material. This strategy is often utilized with soft soil that has a lot of moisture. The soil mixing paddles, or augers, are advanced into the soil first, followed by the injection of the binder material, which mixes with the soil as the drill is retracted. The bearing capacity of the soil–cement columns is increased (Dingqing *et al.*, 2015).

2.11.4 Slurry injection

For many years, slurry injection, a type of penetration grouting, has been utilized on railways with varied success. Lime or cement slurry is injected into the subgrade using this method, which involves pushing the material through injection rods or pipes as they are installed (Dingqing *et al*., 2015).

Table 2.6: Slope stabilization methods researched

Stabilization Method	Source of Background Information
Drainage pipes, wells, and channels	Cornforth (2005)
Dewatering	Coduto *et al.* (2011)
Vegetation	Abramson *et al.* (2002)
Buttressing / rip-rip	Abramson *et al.* (2002)
Geosynthetics	Gee (2015)
Remove and replace	Duncan & Wright (2005)
Re-grading and benching	Cornforth (2005)
Lightweight fill	Abramson *et al.* (2002
Retaining walls	Cornforth (2005)
Soil nails / rock bolts / tieback anchors	Abramson *et al*. (2002)
Mechanically stabilized earth embankments	Abramson *et al*. (2002)

2.12 SLOPE/W Analysis

SLOPE/W evaluates the value of Factor of Safety (FOS) for various shear surfaces and it can produce more than one angle of slopes and identify different type of soils. The ratio of shear strength to shear stress along a critical failure surface is known as the factor of safety (FOS). When using SLOPE/W software, analyses must be based on a model that appropriately represents site subsurface conditions and ground behavior. The high embankment slope stability program is analyzed using the traditional Limit Equilibrium approach. Geo-Studio (SLOPE/W) was developed using the moment and force equilibrium factor from safety equations. The Analysis calculates a factor of safety, which is defined as the ratio of available shear resistance (capacity) to the amount necessary for equilibrium (GEO-SLOPE 2018), (Rahman, 2012).

Appendix C shows the guideline for limit equilibrium of slope, which represents the condition of slope based on its FOS value. These values of FOS will be compared with FOS value obtained from SLOPE/W to get the condition of slope. If the value of FOS is 1.0, the slope is in a state of impending failure. The required factor of safety ranges between 1.25-1.5. In general, the value of FOS with respect to strength that is acceptable for the design of a stable slope is 1.5.

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Materials

3.1.1 Field Sampling

Eight samples were collected by sampling procedure in conformity with BS 1377 (1990) at depth ranging from 0m to 10m on each chainage. These samples were collected at chainages KM75+500 KM90+500, KM105+500 and KM116+500 at their moist condition using plastic bags. To prevent moisture loss, the plastic bags were knotted together. Moisture content in situ was established right away. For each test, the samples were delivered to the laboratory and tested at 105°C in the oven. Each sample was dried in the oven until the weight remained constant after continuous weighing.

3.1.2 Case Study Location

This study was carried out from chainage KM75+500 to KM116+500 along Lagos-Ibadan railway line (Figure 1.1). The topography of the area below the embankment is dominated by denuded plains and residual hills and the terrain is slightly undulating. The ground elevation is between 52 to 145m above sea level. The vegetation is developed with shrub and grass mainly. The stratigraphic lithology and engineering geologic characteristics are silty clay of brownish yellow, hard plastic, Gneiss which is fully weathered brownish yellow and Gneiss heavily weathered brownish yellow but surface water is not developed.

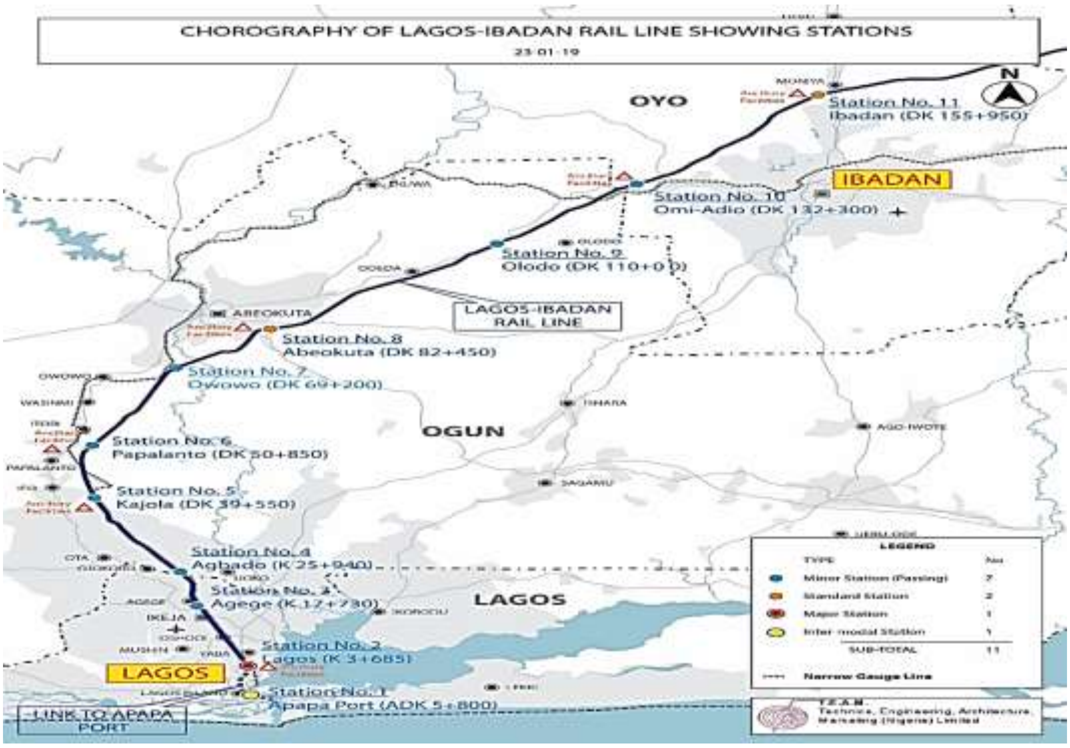


Figure 1.1: Lagos – Ibadan Railway Project Map (TEAM Nig, 2019)

3.2 Methods

3.2.1 Laboratory test preparation

Moisture content, particle size analysis, Atterberg limits, specific gravity, unit weight of soil, and geotechnical characteristics were all determined from soil samples obtained from eight test pits at varying depths. These qualities were then utilized to classify soil samples and determine their strength. The tests were carried out in accordance with ASTM D-1883 (2010) and BS 1377 (2003) standards.

Here are the technical details of the calculating methods and software that were used to achieve the objectives. The following laboratory tests were carried out in accordance with BS 1377 (2003).

3.2.2 Moisture content determination (MC)

The natural moisture content test was determined according to the standard As soon as the dirt was retrieved from the borrow pit, it was placed in an airtight polythene leather and immediately transported to the laboratory. We cleaned and weighed three empty cans. Each container received around 30g of broken soil. The containers and their contents were weighed to the nearest 0.01g and then baked for 24 hours at 1000 degrees Celsius. The containers were taken out of the oven and left to cool after drying. Moisture content is defined as the amount of water in the voids divided by the mass of solids (typically stated as a percentage).

$$Moisture\ content = Mw\ /Ms \times 100\% \tag{3.1}$$

where: Mw is the Mass of Water and Ms is the Mass of Solid

3.2.2.1 Particle size distribution

The diameter of the soil particles that make up the soil mass was determined using a sieve test. After washing and oven-drying, a representative sample of approximately 500g was used for the test. Using an automatic shaker and a set of sieves, the sifting was done mechanically.

3.2.2.2 Atterberg's limits determination

On cohesive soils, Atterberg's limit tests are often used for categorization and correlation. The percentage of moisture based on dry weight at which each change in consistency occurs is determined by testing the portion of samples that pass a (ASTM 1969) standard sieve No.40 (0.425mm).

3.2.2.3 Liquid limit determination:

It is the difference in water content between the liquid and plastic states of the soil. It is defined as the lowest water content at which the soil, while still liquid, has low shear strength against flowing water.

A soil sample weighing 200g and passing through a 425m sieve was combined with water to make a thin homogenous paste. The paste was gathered in the Casangrade apparatus cup, a grove was made, and the number of blows it took to close it was recorded. Moisture content was also determined.

3.2.2.4 Plastic limit determination:

This limit lies between the plastic and semi-solid state of the soil defined in BS 1377 (1990).

This boundary exists between the soil's plastic and semi-solid states. It is determined by rolling out a thread of soil on a non-porous flat surface. It's the water content at which the soil starts to disintegrate and roll into a 3mm diameter thread. Plastic limit is denoted by PL.

A 200g soil sample was taken from the material that passed the 425m test screen and combined with water until it was homogeneous and pliable enough to be formed into a ball. The dirt ball was rolled on a glass plate until the thread cracked at a diameter of around 3mm. As a result, the moisture content was calculated.

The plasticity Index (PI) is the range of water content over the soil is in the plastic condition. Plasticity Index (PI) = LL – PL (3.2)

3.2.2.5 Specific gravity

The number of times a material is heavier than water is represented by its specific gravity. In simplest terms, it is the ratio of the mass of any defined volume substance divided by the mass of an equal volume of water. The number of times the soil solids are heavier than a given amount of water is the specific gravity of soils.

3.2.2.6 Compaction test

Empty mold with base plate was weighed. 3.0kg of air dried soil passing BS sieve aperture of 2.0mm was also weighed and placed on a flat trey about 1.0m X 1.0m. About 7% of water by weight of the dried soil was added immediately and mixed thoroughly until the whole mix formed a uniform paste. The mixture was then compacted in the mold with the extension collar connected by ramming it into three layers and giving each layer 25 blows with a 2.5kg rammer released from 30cm above the soil level. The blows have been dispersed evenly. The collar was removed and the soil shaved off even with the top of the mold after the third layer was compacted. The mold and the soil were then weighed. Two representative samples for moisture content determination were collected from bottom and top of the compacted soil.

The soil compacted were crumbled, remixed with the remaining soil and 3.5% of water by weight of the dried soil was added to the mixture and the process repeated. The compaction soil was repeated each time increasing the water content until drop in weight of mold plus weight of soil. The result was presented in terms of dry densities using the expression:

$$\rho d = \frac{100\rho}{100 \times \omega} \qquad (3.3)$$

where,

ρd = dry density of the mix (Mg/m^3)

ρ = Bulk density of the mix (Mg/m^3)

ω = Moisture content of the mix (%)

3.2.3 Geotechnical parameters

The unit weights and shear strength envelopes are the most important laboratory data used in slope stability assessments.

3.2.3.1 Direct shear

The direct shear test is the most basic and oldest type of shear test available. In a metal shear box, a soil sample is put and subjected to a horizontal force. The soil fails by shearing along a plane when the force is applied. The test can be carried out in a stress- or strain-controlled setting. On cohesionless soils, the test is often performed as a consolidated-drained test. The Agency's testing technique is described in AASHTO T236, (2008).

The samples were subjected to a number of direct shear tests, with specimens measuring 60 mm x 60 mm. With a shearing rate of 1.0 mm/min, direct shear tests were performed at three normal stresses of 120, 180, and 240 kPa.

3.2.4 Embankment geometry

Figure 3.1 shows a Characteristic slope of KM75+500 to KM116+500 with a slope varying heights 10m, 7m and 5m and side slope ratio 1:1.5 and 1:1.75 of berm. The slope was evaluated using the limit equilibrium method: Morgenstem and Price.

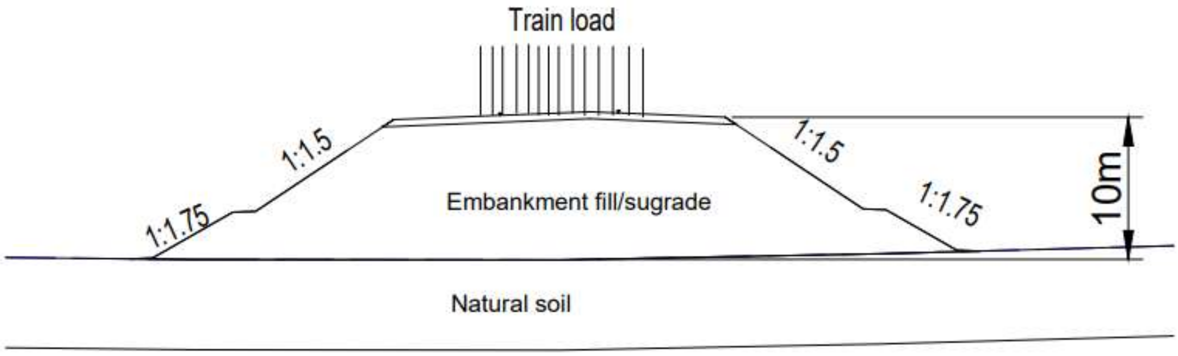


Figure 3.1: Characteristic slope of K75+500 to Km116+500 embankments

3.2.5 Slope modeling methodology

Geostudio (slope/W) was used to analyze the slope stability, with laboratory data (geotechnical parameters) as inputs. With the use of the Computer Aided Drafting, CAD-like ability, the SLOPE/W program provided a strong design tool to graphically represent the real-world soil conditions and features of the case study. The "entry-exit" strategy was employed to search the slip surface. Morgenstern-Price Method (M-PM), which is used in SLOPE/W to assess the model and identify the factor of safety, is the most popular Limit Equilibrium (LE) based method employed. The following were the stages involved in creating the modeled railway embankments.

The working area, scale, and grid spacing were all set up at the start, and the files were saved.

I. I. The embankment geometry and 1:1.5 slopes were sketched and then drawn on the SLOPE/W graphic interface.

II. Limit equilibrium was chosen. Morgenstern-Price analysis method, no pore-water pressure, deterministic approach, entry and exit surface option, and right to left movement direction were chosen in this part.

III. For the Mohr-Coulomb strength model, the soil parameters of each layer of the embankment were defined. For the fundamental parameters, these were the input parameters for cohesion, unit weight, and internal friction angle.

IV. The entry and exit lines for the slip surface were then determined and sketched.

Based on previously established analytic methodologies, the SLOPE/W software generates a network of safety criteria. The value of the minimum factor of safety is then calculated. The analyzed slip surface was then displayed, along with the minimum factor of safety.

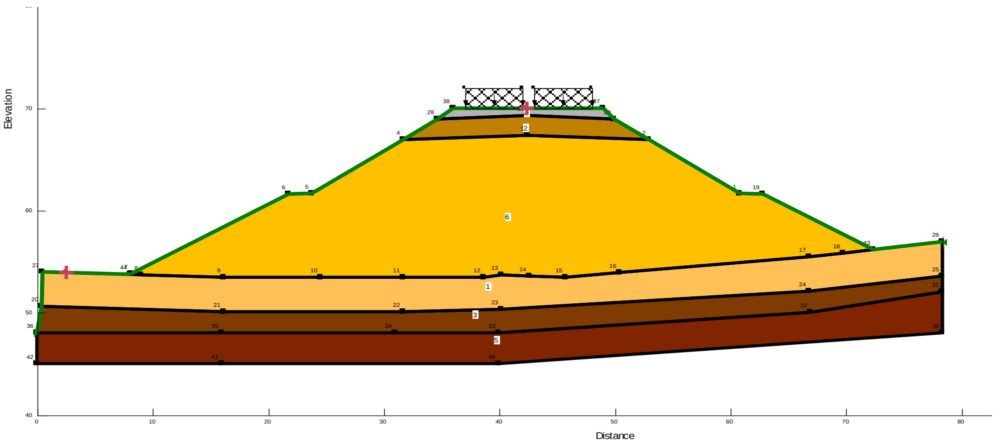


Figure 3.2: Slope/W model of KM75+500 to KM116+500 Embankments

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1.1 Soil moisture content

This test verifies the natural moisture status of embankment sections in the research region. The test was performed on total of eight soil samples at KM75+500, KM90+500, KM105+500 and KM116+500 at depths ranging from 0m to 10m. The average natural moisture content from 0m to 5.5m and 5.5m to 10m of the selected section Figure 4.1

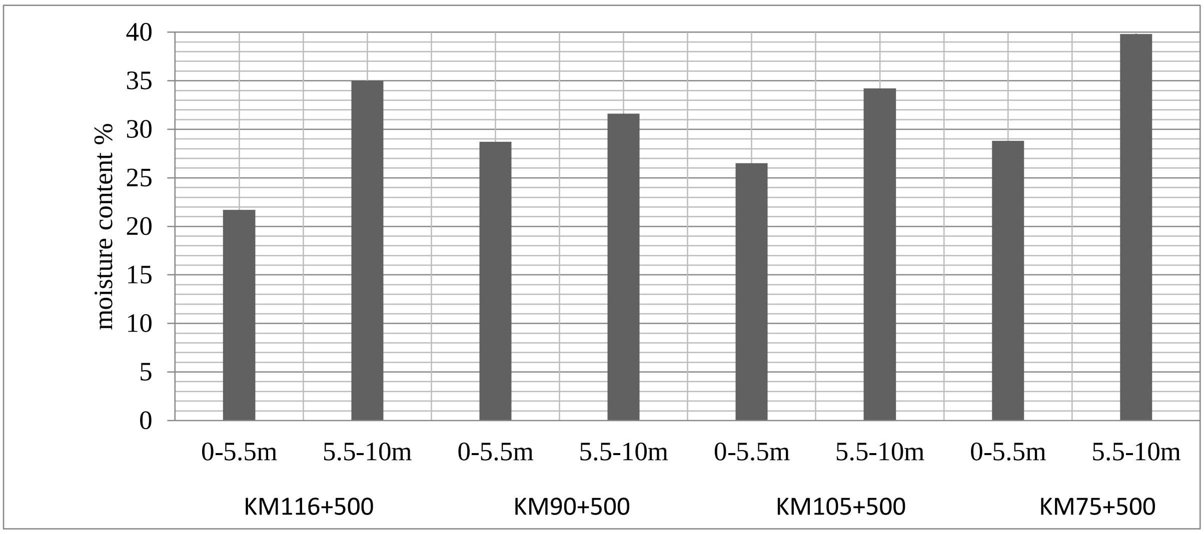


Figure 4.1: Natural moisture content of the embankments at varied depths for each location

Figure 4.1 shows that natural moisture content at KM75+500 increased from 21% at 0m-5.5m depth to 35% at 5.5m to 10m depth. Other selected chainages have similar moisture content profile. This means that the subgrade in this section have low to high moisture content with respect to depth in all the selected locations. The high content of clay soil with rise in ground water Table could be responsible for such characteristics.

4.1.2 Particle size analysis

The particle size conducted for the four selected embankments along the study area are presented in Figure 4.2 and Figure 4.3

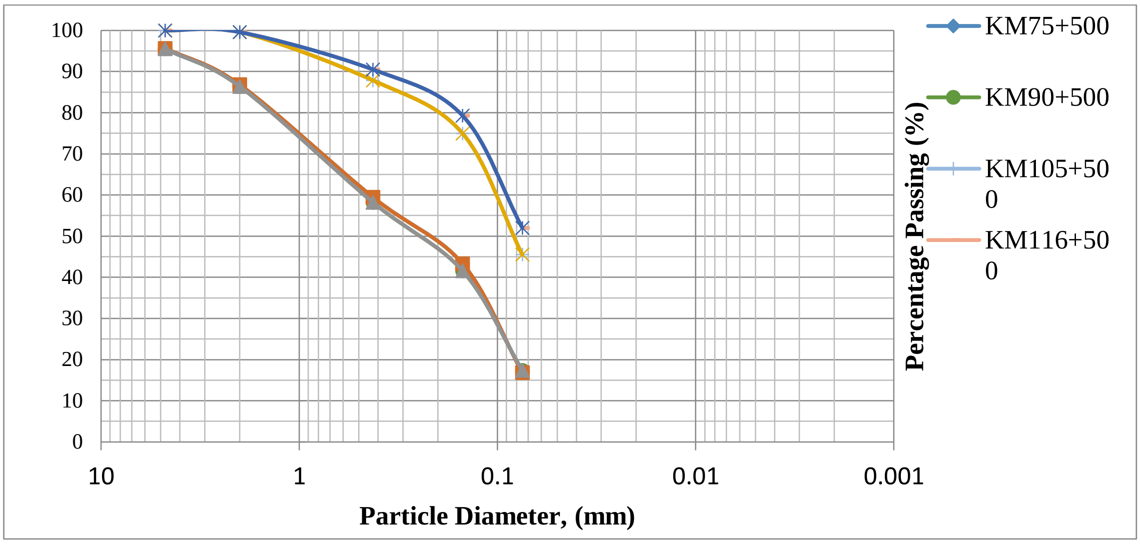


Figure 4.4: Particle size Distribution curves of subgrades at 0m to 5.5m depth

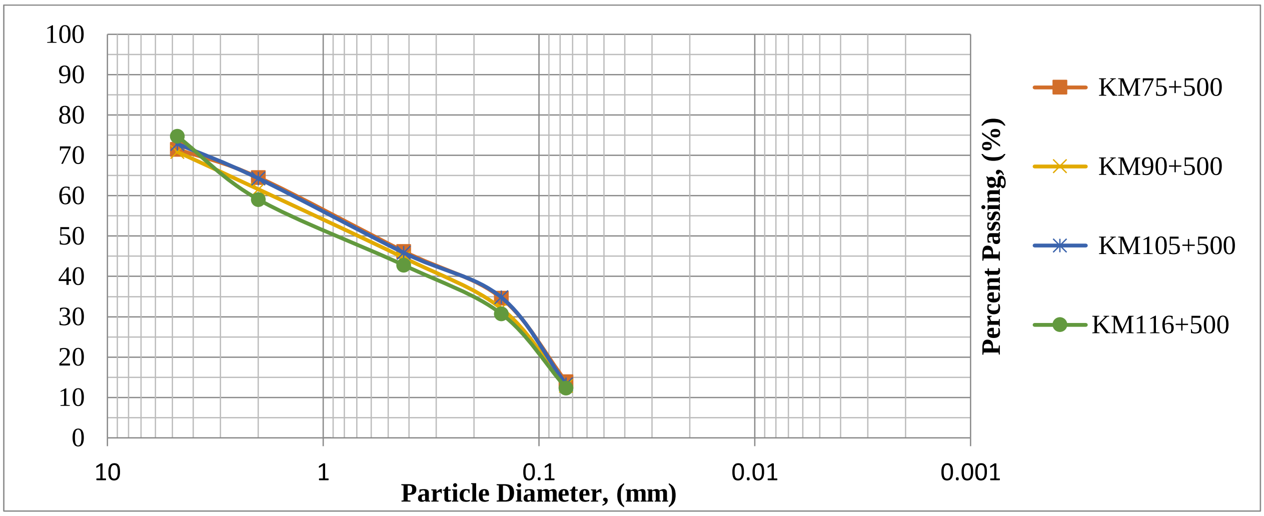


Figure 4.5: Particle size distribution curves of subgrades at 5.5m to 10m depth

Particle size distribution curves which is presented in Figure 4.4 and Figure 4.5 shows that the embankment soils from the four locations at 5.5m to 10m depth have fine fractions that ranged from 40-50%, 0-3% coarse and 50-60% of sand which is classified as clayed soil while at 0m to 5.5m depth the subgrade consist of 15-20% sand, 4-6% coarse particles and 70-80% fines which is classified as clayed sand. The sample with the highest clay content of the four soils is KM116+500 samples which has 52% fines content. The International Union of Railways (UIC) categorizes this soil grade as a poor soil for subgrade construction.

4.1.3 Specific gravity of soil

Using distilled water at a given temperature, the specific gravity of collected soil samples in each depth of the embankment was measured. Table 4.1 shows that the specific gravity of soils ranges from 2.62 to 2.65. These specific gravities matched the findings of Tuncer and Lohnes (1977), who found that clayed and silty soils had specific gravities ranging from 2.60 to 2.90. Rendenouer (1970) also stated those materials with a specific gravity less than 2.65 could decay in engineering sites, owing to their weakness and durability. Therefore, the selected subgrade section may fail with time if not treated or improved.

Table 4.1: The average specific gravity at chosen locations

Subgrade profile	Labels	Depth of sample taken (m)	Average specific gravity
KM75+500	K4012	0-5.5	2.65
	K4013	5.5-10.0	2.63
Km90+500	K4022	0-5.5	2.65
	k4023	5.5-10.0	2.63
Km105+500	K4032	0-5.5	2.65
	k4033	5.5-10.0	2.63
Km116+500	K4042	0-5.5	2.62
	K4043	5.5-10.0	2.64

4.1.4 Bulk and dry densities of soil samples

The materials were subjected to the standard proctor test to assess their bulk and dry densities. This test was carried out on soil samples collected at various depth intervals throughout the study region. The results of the experiments are shown in Figures 4.6 and 4.7

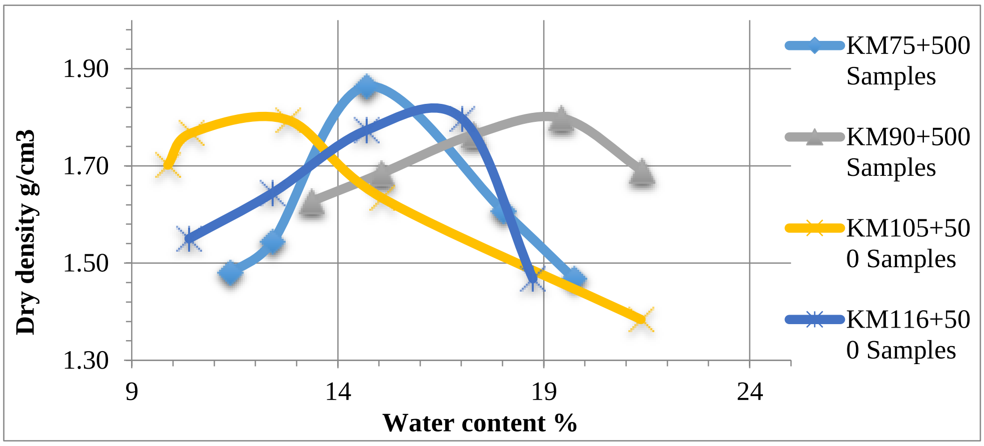


Figure 4.6: Compaction curves of subgrade at 0m to 5.5m depth

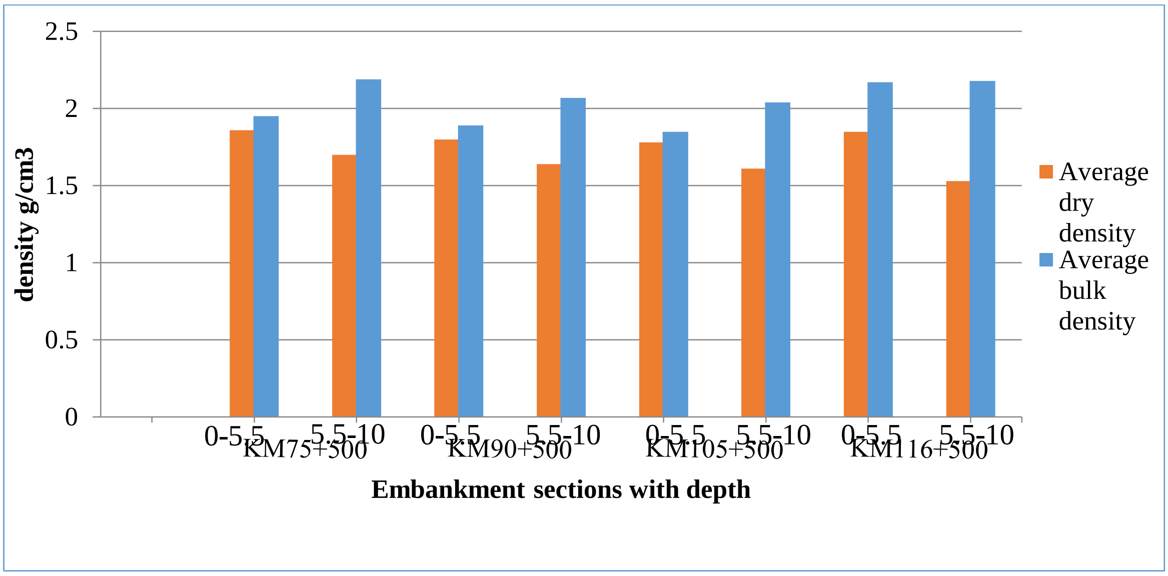


Figure 4.7: The average bulk density and dry density

Figure 4.6 and 4.7 the bulk density of the embankment section with respect to depth, ranges between 1.78-2.15 g/cm^3. Osmotic swelling occurs in clay with a bulk density less than 2.45 g/cm^3, according to Seedman (1986) and Hong *et al* (2012), and the bulk density values of the selected embankment are within the range of swelling clays. Because of the high water absorption, the embankment's natural moisture content may rise, causing instability. As illustrated in Figure 4.3.1, the optimum moisture content ranged between 14 and 17 percent, and the maximum dry density was between 1.53 and 1.86g/cm^3, as reported in Table 4.3. Clay with high plasticity has a very low dry density and high optimal moisture content, according to Arora (2008), which is compatible with the theory that high optimum moisture content and low maximum dry density indicate inferior subgrade materials.

4.1.5 Atterberg's limits of subgrade

The structural strength of the soils is established through Atterberg's limits. Liquid limit tests were performed on the soils to determine the moisture content required for them to lose their cohesion and flow as a liquid. Plastic limit tests, on the other hand, were used to assess the moisture content required before the soils split or crumbled. The plasticity index was derived using liquid and plastic limits to determine the range of plasticity in the research area's soils before deformation. Figure 4.8 shows the results in a comparative bar chart for liquid limit, plastic limit, and plasticity index.

The average liquid limit LL and plastic limit PL of the selected subgrade materials are 30.5 to 41 percent and 10 to 19.8 percent, respectively.

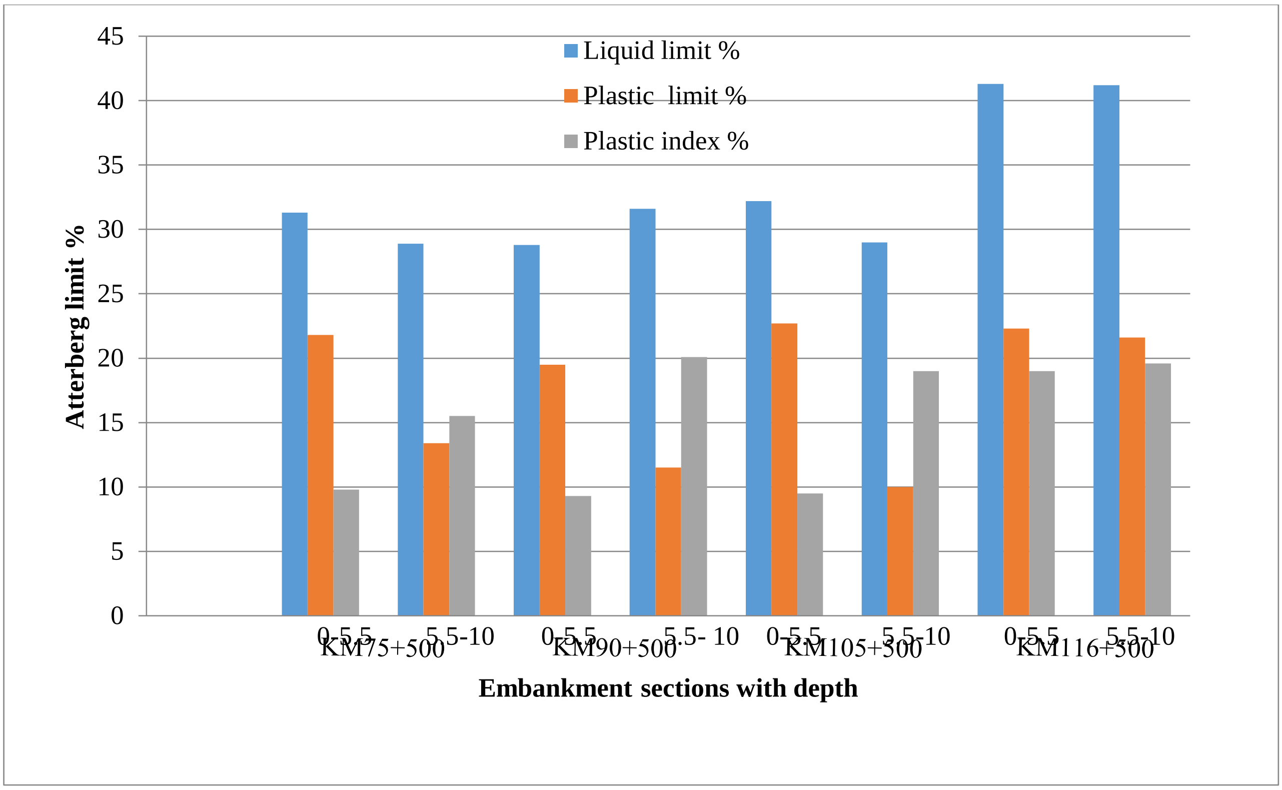


Figure 4.8: Atterberg's limits of the embankment soils at ranging depths

From Figure 4.8 it is observed that at KM116+500 soil specimens have high liquid limits of +41.3% and 41.2% at 0-5.5m and 5.5-10m depths respectively. According to Specifications for Railway Formation (Table 2.1) plastic index PI should be less than 12%. Soils with plastic index PI higher than 20% are classified as having very high potential for volume change (Holtz and Gibbs, 1956).Therefore, subgrade soils at this location need special treatment or improvement in order to meet the UIC criteria.

Similarly, the KM75+500, KM90+500 and KM105+500 subgrade soils have moderate liquid limits which varied from 28.8 to 33.3% and average plastic index PI range of 15% to 19%. This is still more than 12% plastic index requirement for a construction of railway formation. Therefore, subgrade soils at these locations need treatment or improvement. The summary of the embankment is presented in Table 4.2

Table 4.2: Summary of the soil characteristic of embankment

Properties	KM75+500		KM90+500		KM105+500		KM116+500	
Soil samples	0-5.5m	5.5-10m	0-5.5m	5.5-10m	0-5.5m	5.5-10m	0-5.5m	5.5-10m
Moisture content %	14.5	17.3	15.7	17.5	15.9	18.5	15.0	21.3
Dry unit weight kN/m^3	18.6	17.0	18.0	16.4	18.3	16.1	19.5	15.5
Specific gravity Gs	2.65	2.63	2.65	2.63	2.65	2.63	2.62	2.64
Liquid limit %	33.30	28.90	28.80	31.60	32.20	29.00	41.3	41.2
Plastic limit %	21.90	13.40	19.50	11.50	22.70	10.00	22.3	31.6
Plastic index %	9.80	15.50	9.30	20.10	9.50	19.00	19.0	9.6
Percentage gravel %	4.49	0.00	6.49	0.00	5.35	0.00	4.70	2.40
Percentage sand %	78.13	51.10	77.13	53.96	79.46	59.70	70.3	42.00
Percentage Fines	17.38	48.90	16.38	46.04	15.19	40.30	19.2	52.50
AASHTO soil Classification	A-2-6 Clayey sand	A-6 Clay soil	A-2-6 Clayey sand	A-6 Clay soil	A-2-6 Clayey sand	A-6 Clay soil	A-2-6 Clayey sand	A-6 Clay soil
UIC classification	S1	S0	S1	S0	S1	S0	S1	S0

4.2 Slope Stability Analysis

The shear strength parameters such as cohesion and angle of friction were measured using a direct shear box test. Table 4.3 shows an overview of the findings of laboratory tests.

Table 4.3: Summary Shear strength parameter of soils from high embankments

Chainage	Depth (m)	Cohesion (kN/m^2)	Angle of internal friction (Ø)
KM75+500	0-5.5	20	25
	5.5-10	40	18
KM90+500	0-5.5	20	25
	5.5-10	33	15
KM105+500	0-5.5	20	25
	5.5-10	34	12
KM116+500	0-5.5	20	25
	5.5-10	35	10

4.2.1 Slope/W embankment simulations at KM75+500

After simulating the critical slope in the program of a high filled slope of KM75+500, the potential slip surface and the safety factor in critical surfaces, also the type of slippage, may be determined. This was done using SLOPE/W in accordance with the principle of limit equilibrium, and the results are displayed in Figure 4.9.

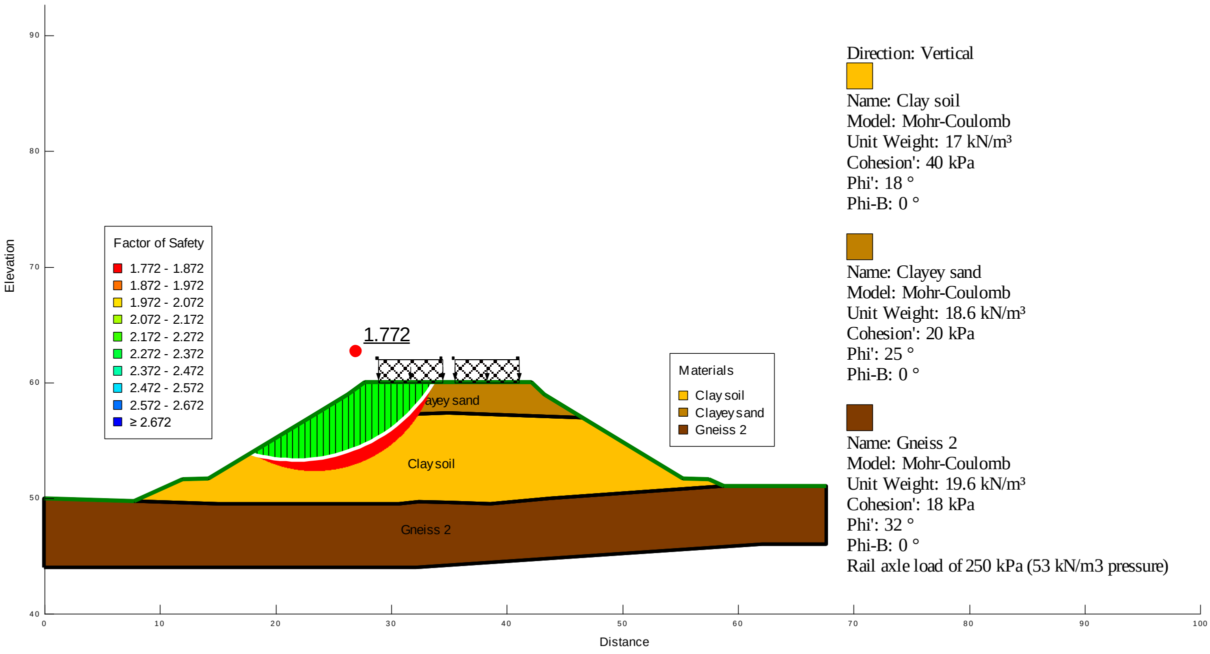


Figure 4.9: The safety factor and slip surface for 10m high KM75+500 Embankment

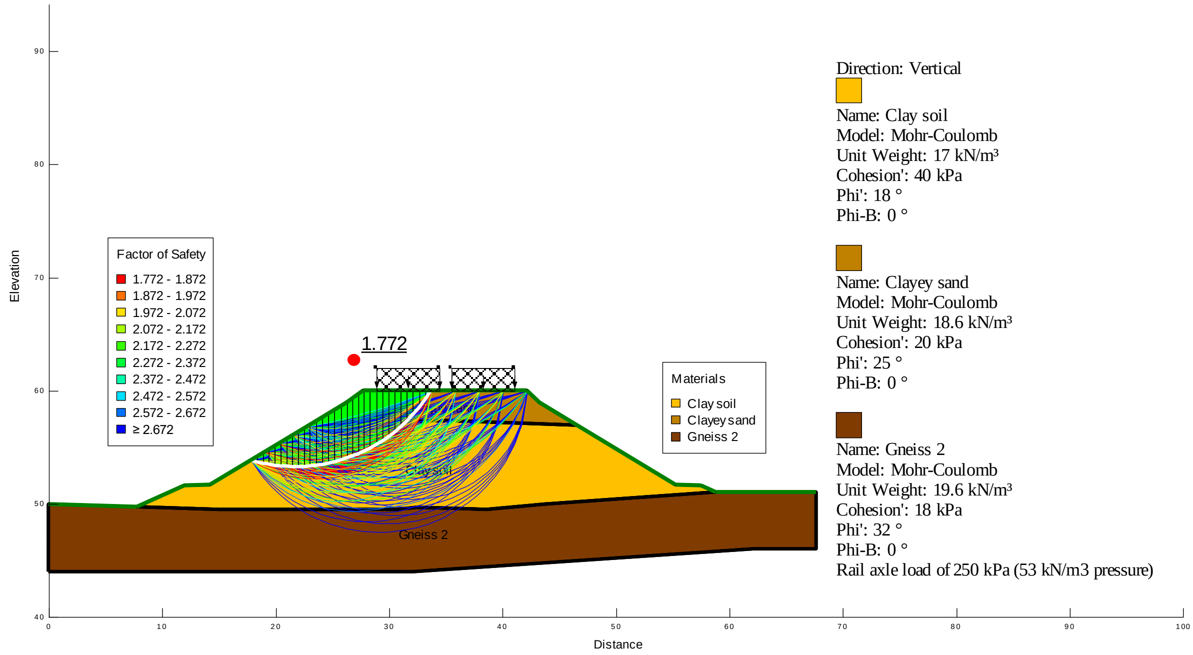


Figure 4.10: Factor of Safety (FOS) for potential slip surfaces on the KM75+500 slopes

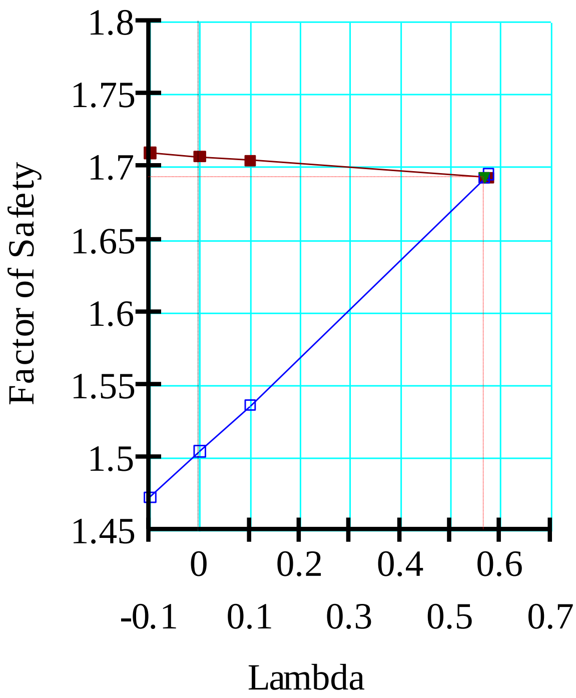


Figure 4.11: Various Factor of safety for 10m high KM75+500 embankments

From Table 4.2 and Table 4.3, the geotechnical parameters of embankment are unit weight of 18.6 kN/m^3, the internal frictional angle of 25° and cohesion of 20 kN/m^2 for clayey soil layer (0-5.5m), 17.0 kN/m^3, internal frictional angle of 15° and cohesion is 33 kN/m^2 for clay soil layer (5.5-10.0m) is applied for embankment 10m, 7m, 5m. After the simulation, the factor of safety is 1.772, 1.852, and 1.927, respectively. It also indicates that changing the slope's geometric shape generates a relative change in the resistive and shear forces, resulting in varied degrees of safety stability factor. As can be observed, as the slope height increased, the factor of safety declined. It should be emphasized that the software calculates and draws all feasible slip surfaces to establish the factor of safety for a particular scenario, with the smallest possible value being considered the critical factor of safety (Figure 4.10).

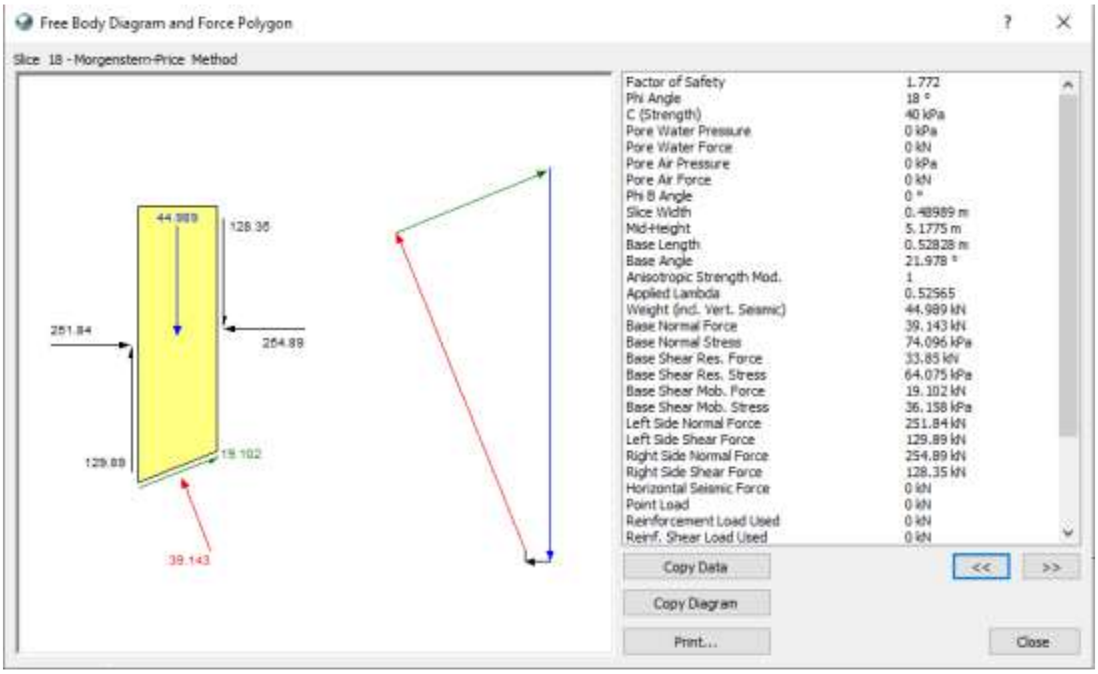


Figure 4.12: Forces on slice number 18 from Morgenstern-Price at KM75+500 embankment

For a 10m high embankment, the Bishop 1.715, Janbu 1.455, and Morgenstern–Price and Spencer 1.772 factors of safety calculated using limit equilibrium methods are shown in

Figure 4.11. These results coincide with Fredlund and Krahn (1977), as shown in Figure 2.4, where Janbu assumes zero interslice shear force and Bishop assumes zero interslice forces.

Slope height of 10m, Figure 4.10 shows changes in factor of safety (Fs) for all feasible slip surfaces, with the value of the factor of safety against sliding (Fs) for critical slip surface being 1.772. This condition's sliding wedge is divided into 30 slices. Diagram showing force polygon on 18th slice is shown in Figure 4.12.

4.2.2 Slope/W embankment simulations at KM90+500

This was carried out in accordance with the principle of limit equilibrium via SLOPE/W and the analyses of the result are presented as shown Figure 4.13. The potential slip surface and the factor of safety in the critical surfaces, also the type of slip can be observed for KM90+500 embankments.

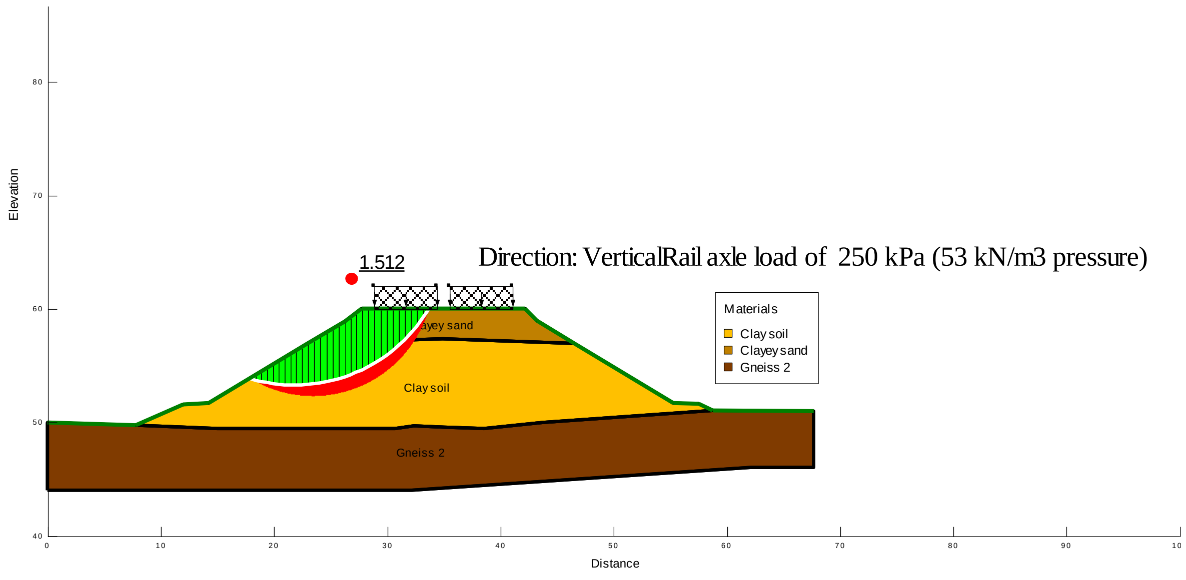


Figure 4.13: The safety factor and slip surface for 10m high KM90+500 Embankment

The geotechnical parameters of the embankment are unit weight of 18.0 kN/m^3, angle of internal friction (Ø) 25° and cohesion (C) of 20 kN/m^2 for clayey soil layer (0-5.5m), unit weight of 16.4 kN/m^3, internal frictional angle (Ø) 18° and cohesion (C) of 40 kN/m^2 for clay soil layer (5.5-10.0m), respectively, as shown in Tables 4.2 and 4.3. After modeling for embankment 5, 7, 10m, respectively, the factors of safety are 1.512, 1.676, and 1.862, respectively, indicating that the embankments are moderately stable. It also shows that changing the geometric shape of the slope leads to change in the resistive and opposing forces, resulting in a range of safety factors. As observed, the safety factor has dropped as the slope height has increased. It should be emphasized that the software evaluate and draws all feasible slip surfaces to establish the safety factor for a particular scenario, with the smallest possible value being considered the critical factor of safety (Figure 4.15).

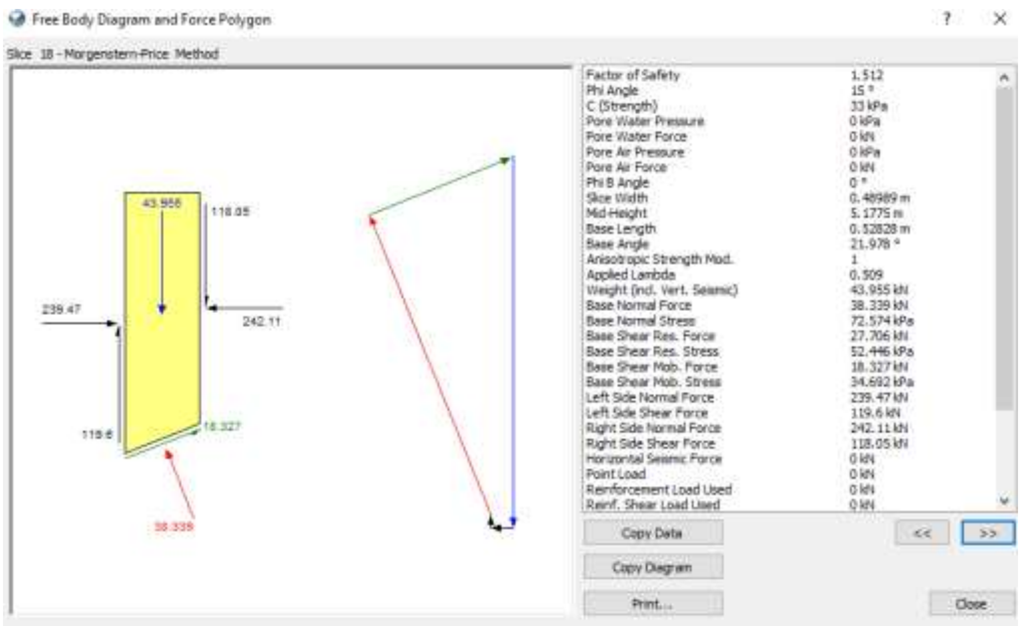


Figure 4.14: Forces on slice number 18 from Morgenstern-Price for KM90+000 slope

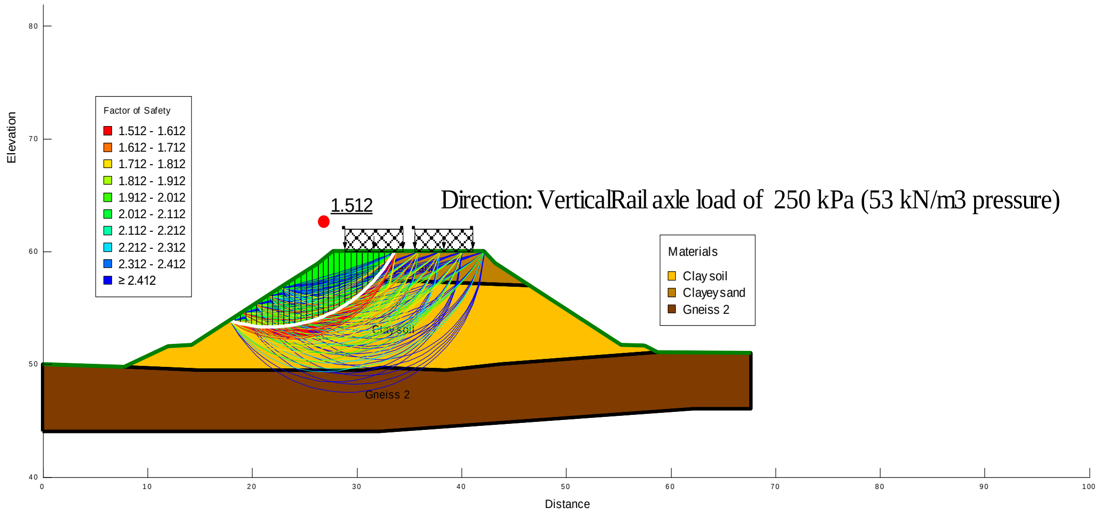


Figure 4.15: Factor of Safety (FOS) for potential slip surfaces on the KM90+500 slopes

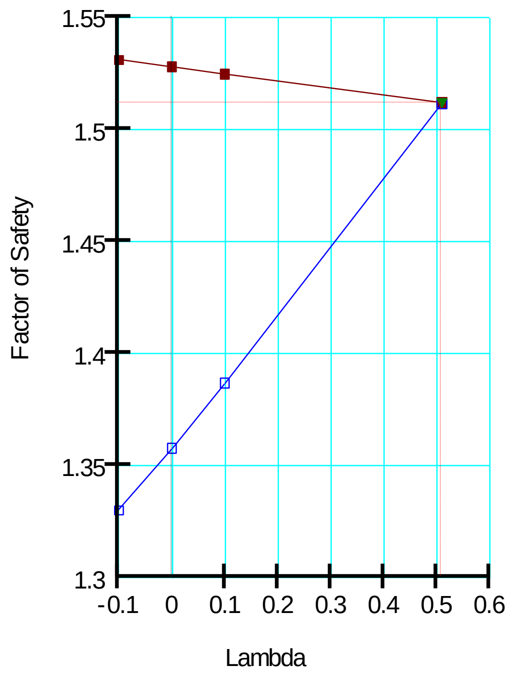


Figure 4.16: Various Factor of safety for 10m high KM90+500 embankments

Figure 4.16 illustrates the Bishop 1.535, Janbu 1.346, and Morgenstern–Price and Spencer 1.512 factors of safety computed from limit equilibrium methods for a 10m high embankment. These results are consistent with Fredlund and Krahn (1977), as seen in

Figure 2.4, where Janbu assumes zero interslice shear force and Bishop assumes zero interslice forces.

4.2.3 Slope/W embankment simulations at KM116+500

The geotechnical parameters of the embankment are specific weight of 19.5 kN/m^3, internal frictional angle (Ø) 25° and cohesion (C) of 20 kN/m^2 for clayey soil layer (0-5.5m), 15.5 kN/m^3, internal frictional angle (Ø) 10° and cohesion (C) of 35 kN/m^2 for clay soil layer (5.5-10.0m), respectively, as shown in Tables 4.2 and 4.3. The slope is consider stable because the factor of safety after simulation is 1.378, 1.495, and 1.626, after modeling for embankment 5, 7, 10m, respectively respectively.

It also demonstrates that a change in the slope's geometric shape creates a change in the resisting and shear forces, and values of safety factors. As can be observed, the factor of safety has decreases as the slope height has increased (Figure 4.21). The software expressed and draws all potential slip surfaces to expressed the safety factor for a given scenario, with the least possible value being regarded the most important factor of safety (Figure 4.18).

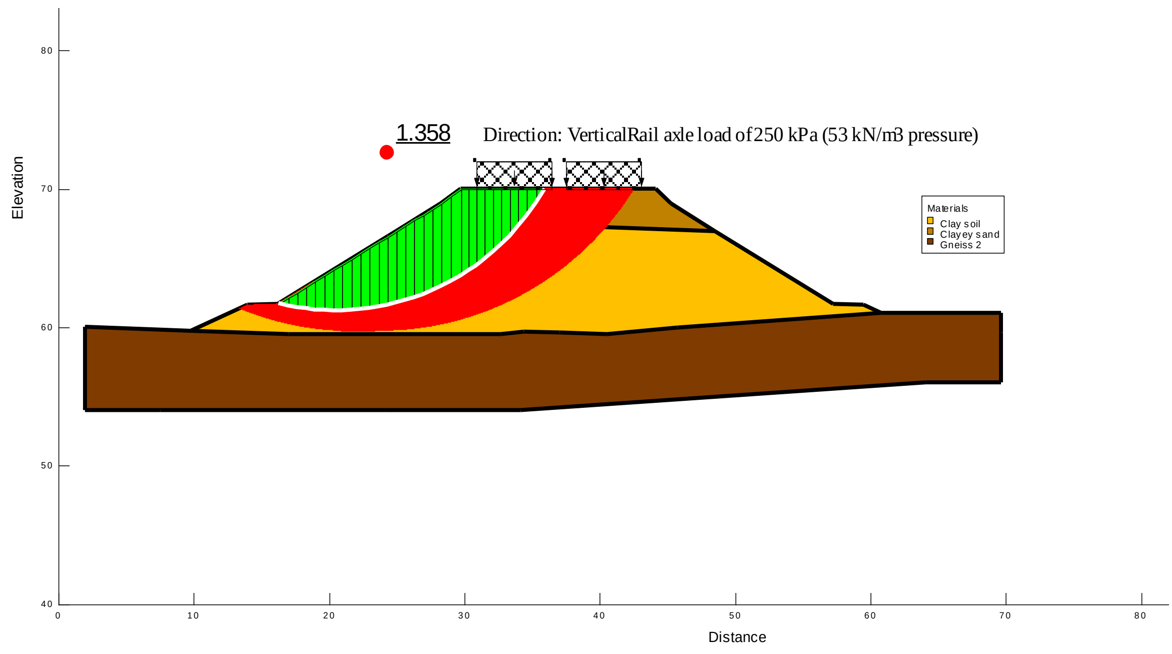


Figure 4.17: The safety factor and slip surface for 10m high KM116+500

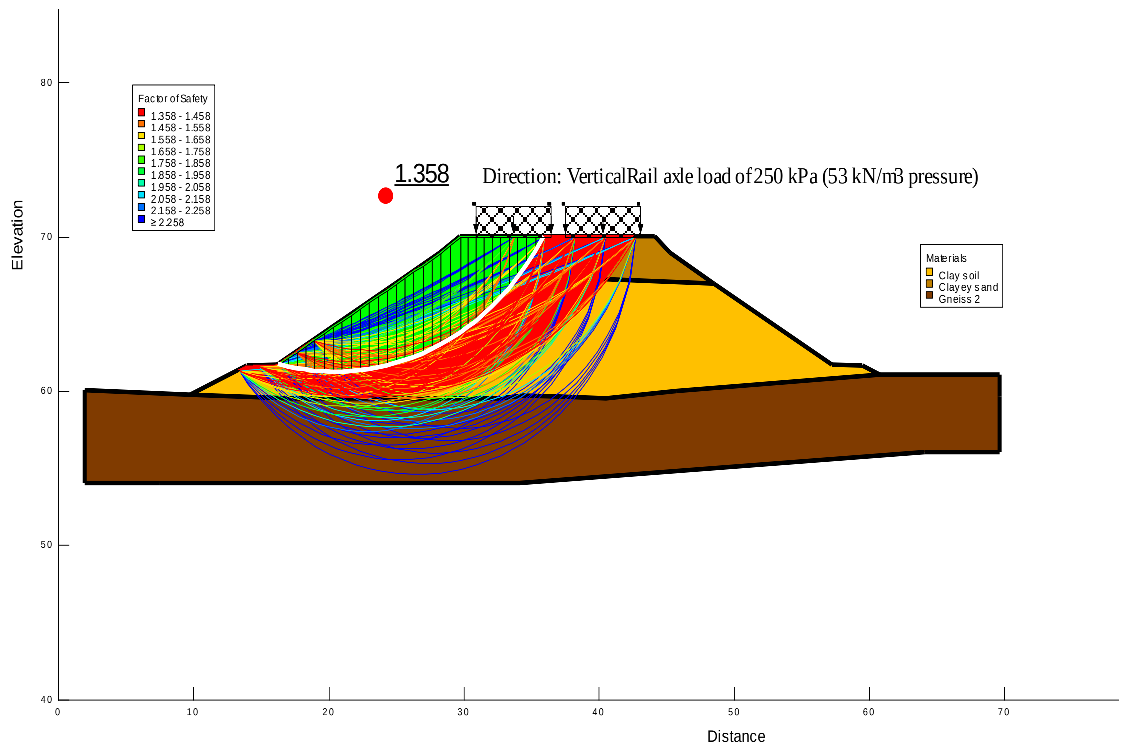


Figure 4.18: factor of Safety (FOS) for potential slip surfaces on the KM116+500

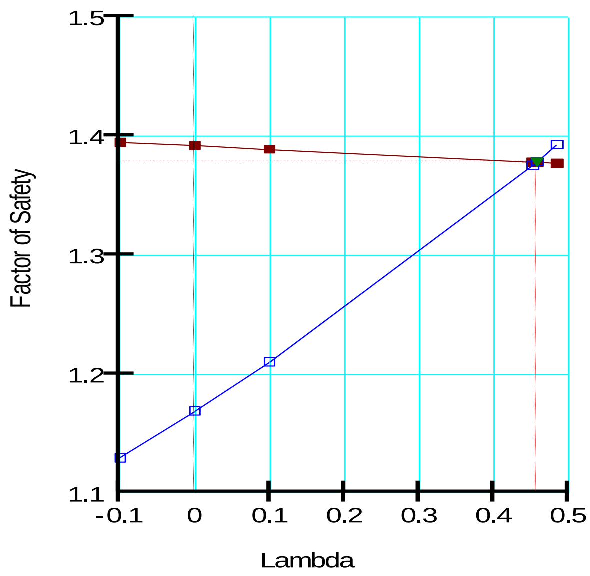


Figure 4.19: Various Factor of safety for 10m high KM116+500 embankments

Figure 4.19 illustrates the Bishop 1.395, Janbu 1.125, and Morgenstern–Price and Spencer 1.378 factors of safety computed using limit equilibrium methods for a 10m high embankment. These results are consistent with Fredlund and Krahn (1977), as seen in Figure 2.4, where Janbu assumes zero interslice shear force and Bishop assumes zero interslice forces.

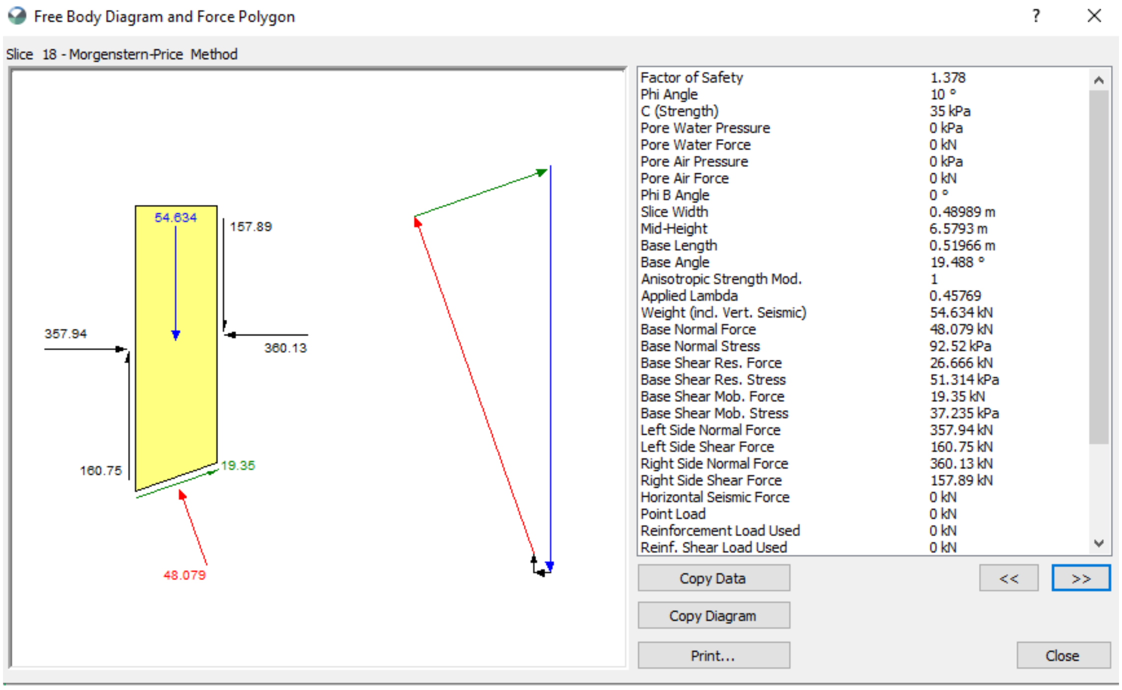


Figure 4.20: Forces on slice number 18 from Morgenstern-Price for KM116+500

4.3 Possible Failure Mechanisms of the Slopes

The results of the slope stability analyses show that the slopes are consider stable when the shear strength parameter values obtained from direct shear test are used under varying height conditions (Tables 4.4).

Rotational slide failure was observed as the potential failure mechanism for the entire slopes, is the failure where the arc of the rupture surface goes through the slope and above the toe. This failure mechanism or mode of failure was the result of the embankment material being clay soil and had a reduced shear strength parameters as well as high ratio of pore water pressure.

Also from Figure 4.22, when the internal friction angle increase, the safety factor of the slope also increases, this correspond to the increase the resistance forces. Therefore, at depth ranging from 5.5-10.0m in KM116+500 the friction angle is 10^{o}. This contributed in the lower factor of safety at that embankment.

Table 4.4: slope stability analysis results for the study slopes at different height

Location	Embankment height (m)	Factor of Safety	Remark
KM75+500	5	1.927	Stable
	7	1.852	Stable
	10	1.772	Stable
KM90+500	5	1.862	Stable
	7	1.676	Stable
	10	1.512	Stable
KM105+500	5	1.548	Stable
	7	1.437	Stable
	10	1.358	Stable
KM116+500	5	1.626	Stable
	7	1.495	Stable
	10	1.378	Stable

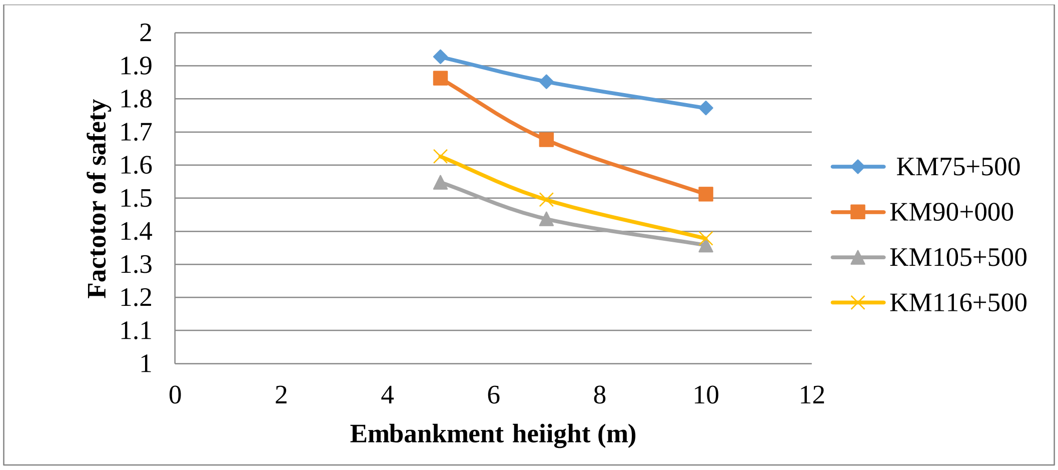


Figure 4.21: Variation Factor of safety with Embankment height

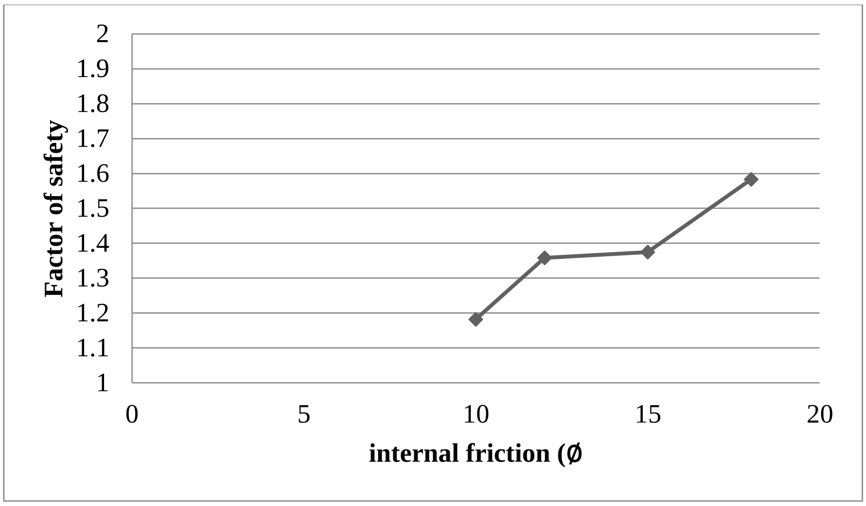


Figure 4.22: Variation of factor of safety with friction angle

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Soil investigation was conducted on four locations namely KM75+500, KM90+500, KM105+500 and KM116+500 along Lagos-Ibadan railway modernization. Samples were collected from the four locations each at depths ranging 0 to 5.5m and 5.5 to 10m. The basic properties of the soil samples were measured following procedures outlined in ASTM D-1883 (2010) and BS 1377 (1990). The index properties of the subgrades at 0-5.5m and 5.5-10.0m depths in each location is characterize as clayey sand and clay soil respectively, which is categorized as poor subgrade soils according to railway specification. The plastic index PI of the embankments soils in four locations at depth 0-5.5m and 5.5-10.0m varied from 10 to 19.8%, which exceed specification of railway formation (plastic index PI less than 12%). Therefore these subgrade layers/formations of the embankment require treatment.

It also shows that the soils of the embankment have similar characteristics with respect to depth.

The Embankment height of 10, 7 and 5m has an overall stability with safety factors more than 1.3 at all the slopes, which confirm its stability against failure according to US Army corps of engineers. Slope assessment shows that factor of safety increases with decrease in embankment height.

It also indicates that the slopes of the embankment have a rotational slide failure mechanism as a potential failure patterns which occurs above the berm.

5.2 Recommendations

However, slope stabilization and protection system is recommended installation of soil nailing, vegetation and well drainage sytems were recommended as stabilization measure to ensure the long term stability of the failure slopes and cement slurry injection grouting at KM116+500 embankment was recommended due to high liquid limit of underlying subgrades. The installation of prefabricated horizontal drains is required in order to increase the consolidation rate.

Future research should be conducted periodically on settlement analysis of the new railway embankments.

5.3 Contributions to Knowledge

The research has assessed the slopes of newly constructed railway embankment of varying heights 5, 7 and 10m along Lagos-Ibadan rail line; which has not been carried out since the completion of construction. Samples were collected from KM75+500, KM90+500, KM105+500 and KM116+500, each at depths ranging 0m to 10m, characterized as clayey sand and clay soil at specific depths. Using SLOPE/W, the factor of safety increases with decrease in embankment height with overall stability of safety factors at all the slopes is greater than 1.3, which confirmed its stability against failure after construction. The research help in resolving the problem raised by Das (2011) about an incorrect estimation of the slope's geological structure assessments. Also help in establishing the current states of slopes of newly constructed railway embankment incase of construction error.

REFERENCES

AASHTO, T236-08. (2008). Standard Method of Test for Direct Shear Test of Soils Under Consolidated Drained Condition (ASTM Designation:D 3080-72 (2003))

Abdoullah N, (2010). Analysis of Slope Stability Using Limit Equilibrium, Universitatea Tehnică” GheorgheAsachi” din IaşiTomul LVI (LX), Fasc. 2

Abramson, L.W., Lee, T.S., Sharma, S., and Boyce, G.M. (2002). *Slope Stability and Stabilization Methods.* John Wiley & Sons Inc

Akayuli, C., Ofosu, B., Nyako, S.O. and Opuni, K.O. (2013). The influence of observed clay content on shear strength and compressibility of residual sandy soils., Int J Eng Res Appl., 3 (4), Jul-Aug, 2538-2542

Arora, K.R. (2008). *Soil Mechanics and Foundation Engineering (Geotechnical Engineering),* Standard Publishers Distributors, Delhi, 2008.

Assefa, E. ,Lin, L J, Sachpazis, C I. Feng, D H, Sun X S, and Xiaoliang X (2017) “Slope Stability Evaluation for the New Railway Embankment using Stochastic Finite Element and Finite difference Methods” *Electronic Journal of Geotechnical Engineering*, (22.01), pp 51-79. Available at ejge.com

ASTM D-1883, (2010). *Standard test methods for liquid limit, plastic limit, and plasticity index of soils*. West Conshohocken, PA: ASTM International.

ASTM, (1969). *Special Procedure for Testing Soils and Rocks for Engineering Purpose*. Technical Publication No. 479, 5th Edition.

B.S 1377 (1990) Method of testing soil for civil engineering purposes, British standard institution. London. UK

B.S 1377 (2003). Part 2: Traffic loads on bridges. Available at: http://www.standardsuk.com/ (Accessed: 5 December 2019).

B.S, EN. 1991-2 (2003). Eurocode 1 - Actions on Structures - Part 1-3: General Actions

Bishop, A. W. (1955). *The Use of Slip Circles in Stability Analysis of Slopes*. Geotechnique, Vol. 5,

Bowles, J. E. (2012). *Engineering Properties of Soils and their Measurements*. 4th edition, McGraw Hill Education (India) Private Limited, New Delhi.

Bromhead, E.N. (1992) The Stability of Slopes, 2nd edition, Blackie Academic & Professional, Glasgow.

Budhu, M. (2000). *Soil Mechanics and Foundations*, First Edition, Johan Wiley & Sons, Inc, University of Arizona, AZ

Capper., L. P. (1976). *The Mechanics of Engineering Soils*. John Wiley & Sons 6th Edition

Chirag D., Momin I, Patoliya B. and Jigar M, (2016). *Ground Improvement Technique for Railway Embankment, proceedings on Recent Advances in Civil Engineering for Global Sustainability*. Journal of Global Research and Development, e-ISSN: 2455-5703.

Dafalla, M.A., (2013). *Effects of clay and moisture content on direct shear tests for clay-sand mixtures.* Adv. Mater. Sci.Eng., Volume 2013, http://dx.doi.org/10.1155/2013/562726, 1-8

Das, G. (2011). Analysis of Slope Stability for Waste Dumps in a Mine. National Institute of Technology Rourkela: Bachelor Thesis

Dingqing, L., James, H., Ted, S. and Steven, C. (2015). *Substructure from: Railway Geotechnics.* CRC Press Boca Raton, Florida. Accessed on: 06 Oct 2020

Duncan, J.M. and Wright, S.G. (2005). *Soil Strength and Slope Stability*, John Wiley & Sons, Inc., Hoboken, NJ

Duncan, J.M., Wright S.G., and Brandon, T. L. (2014). *Soil Strength and Slope Stability*. Hoboken, NJ: Wiley

El-Maksoud, M.A.F. (2006). Laboratory determining of soil strength parameters in calcareous soils and their effect on chiseling draft prediction, Proc. Energy Efficiency and Agricultural Engineering, Int. Conf., Rousse, Bulgaria

Esveld, C. (2001). *Modern Railway Track (2nd ed.).* MRT Productions, Zaltboomel

Fellunius, W. (1936). *Calculations of the Stability of Earth Dams*. Proceedings of the Second Congress of Large Dams. Vol. 4, pp. 445-63, , Washington D. C.

Fredlund, D. G. and Krahn, J. (1977). *Comparison of slope stability methods of analysis.* Canadian Geotechnical Journal, Vol. 14, pp. 429-39.

GEO-SLOPE International Ltd. (2018), *Stability Modeling with SLOPE/W 2018*, GEO-SLOPE International Ltd., Calgary, Alberta, Canada.

Holtz, W. G. and Gibbs H. J. (1956). *Engineering Properties of Expansive Soils*. Transactions of ASCE, Vol. 121, pp. 641-679

Hong, J. C.; Jong K.M.and Li S. J. (2012). A study of decreasing behavior of strength and elastic parameters due to water infiltration in rock cores. J korea Geotech Soc 28(1):204-211

Israel, O. (2014). *Railway: The return of the economic live wire*. Vanguard. https://newsrand.com/2020/12/07/lagosibadan-rail-begins-operation

Jain P. and Jain, k. (2015). "Analysis of Railway Embankment under varying Sub Soil Conditions for Different Height of Embankments**"** International Journal for Scientific Research & Development Vol. 3, Issue 08, ISSN (online): 2321-0613

Janbu, N., (1954). *Stability analysis of Slopes with Dimensionless Parameters*. Thesis for the Doctor of Science in the Field of Civil Engineering, Harvard University Soil MechanicsSeries, No. 46.

Kaniraj, S.R. (1988). *Design Aids in Soil Mechanics and Foundation Engineering*. McGraw Hill Education (India) Private Limited, New Delhi

Laskar, A. and Pal, S.K., (2012). *Geotechnical characteristics of two different soils and their mixture and relationships between parameters*. EJGE, 17, 2821-2832.

Mollahasani, A., Alavi, A.H., Gandomi, A.H. and Rashed, A. (2011). Nonlinear neural-based modeling of soil cohesion intercept., KSCE J CIV ENG., 15(5), 831-840

Morgenstern, N. R. and Price, V. E. (1965). *The Analysis of the Stability of General Slip Surfaces.* Geotechnique, Vol. 15,No. 1 pp. 77-93.

Mostyn, G. and Small, J.C. (1987). Methods of Stability Analysis.In Slope stability and Stabilization. Rotterdam: A.A.Balkema

Mundrey, J.S. (1993). *Railway Track Engineering*. Tata McGraw Hill Publishing Company Ltd., New Delhi, pp 187-192

Murthy, V.N.S. (2002). Principles of Soil Mechanics and Foundation Engineering, UBS Publishers' Distributors Ltd., New Delhi,

Oke, S.A. and Amadi, A.N. (2008). A*n assessment of the geotechnical properties of the sub-soil of parts of Federal University of Technology, Minna, Gidan Kwano Campus, for foundation design and construction.,* J Sci Educ Technol., 1 (2), 87 – 102.

Otoko, G. R., and Blessing, O. C. (2014). *Cement and lime stabilization of a Nigerian deltaic marine clay (chikoko)*. European Inter. J. Sci. Tech. 3(4), 53–60.

Oyediran, I A., and Durojaiye, H. F. (2011). Variability in the Geotechnical properties of some residual clay soils from southwestern Nigeria. International Journal of Scientific & Engineering Research. Vol. 2, Issue 9 http://www.ijser.org

Poulos, S.J. (1989). Liquefaction Related Phenomena; In: Advanced Dam Engineering for Design Construction and Rehabilitation Van Nostrand Reinhold (ed.) Jansen R. B., pp. 292–320

Prakash, S., and Jain, P.K. (2002). *Engineering Soil Testing*, Nem Chand & Bros, Roorkee.

Profillidis, V., and Kouparoussos, A. (2006). *Mechanical Behavior of the Railway Subgrade*. KEDE, Scient. Bulletin of the Ministry of Public Works of Greece, Vol. 3-4, Athens.

Rahardjo, H., ong, T.H., Rezaur, R.B., and Leong, E. C. (2007). *Factors Controlling Instability of Homogeneous Soil Slopes under Rainfall.* Journal on Geotech. Geoenviron. Eng., 133(12), 1532-1543,

Rahman, M. (2012). *Slope Stability analysis and road safety evaluation, case study on two roads located close to the Petia river*. sikfors : Nystrand

Raj, P.P. (2012). S*oil Mechanics and Foundation Engineering.* Dorling Kindersley (India) Pvt. Ltd., New Delhi, 2012.

Rendenouer, D.R. (1970). *Characteristics of sediments and their dispersal systems along the shelf and slope of an active forearc margin eastern Hokkaido, northern Japan*. Sedimentary Geol 201(3-4):341-364

Roy, S. and Dass, G. (2014). *Statistical models for the prediction of shear strength parameters at Sirsa.* India, I. Journal of Civil and Structural Engineering., 4(4), 483-498.

Salokangas J. P. and Vepsalainen P., (2009). *Stability modeling of old railway embankments on very soft ground*. In Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical engineering, Alexandria, Egypt, 5-9 October, pp. 1614-1617

Seedman, R. (1986). The behavior of clay shales in water. Canadian Geotech. J 23(1):18-22

Selig, E.T. and Waters J.M. (1994). Track Geotechnology and sub- structure management. New York: Thomas Telford Services Ltd

Shanyoug, W., Chan, D., Lam, K.C. (2009). Experimental study of the fines content on dynamic compaction grouting in completely decomposed granite of Hong Kong., CONSTR BUILD MATER., 23, 1249 -1264)

Sinha, B.N. (2008). *Advance Methods of Slope-Stability Analysis for Earth Embankment with Seismic and Water Force.* International Association for Computer Methods and Advances in Geomechanics (IACMAG) 1-6 October, Goa, India

Soltani-Jigheh, H. and Jafari K. (2012). Volume Change And Shear Behavior Of Compacted Clay sand/Gravel Mixtures International Journal of Engineering & Applied Sciences (IJEAS) Vol.4, Issue 3;152-166

Spencer, E., (1967). *A method of Analysis of the Stability of Embankments, Assuming Parallel Interslice Forces*. Geotechnique, Vol. 17, pp. 11-26

TEAM (2019). Lagos – Ibadan Railway Modernization Project archive.

Thomas, M. Petry, and Dallas N, Little P.E. FASCE, (2002), Review of stabilization of clays and expansive soils and lightly loaded structures –history, practice and future.

Tuncer, O.D, and Lohnes C. G. (1977). Specific gravity of soils. J Geol. 75:432-534

U.S. Army Corps of Engineers, (2003). Engineer Manual. [Online] Available: http://www.usace.army.mil/publications/eng-manuals, (Accessed 02/15/2020)

UIC (2016). UIC, The worldwide railway organisation. Retrieved from http://www.uic.org/highspeed - General-definitions-of-highspeed

APPENDICES

1. Appendix A; AASHTO classification Table

AASHTO CLASSIFICATION OF HIGHWAY SUBGRADE MATERIALS
(with suggested subgroups)

General Classification	Granular Materials (35% or less passing #200)							Silt-Clay Materials (more than 35% passing #200)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis Percent Passing:											
# 10	0-50										
#40	0-30	0-50	51-100								
#200	0-15	0-25	0-10	0-35	0-35	0-35	0-35	36-100	36-100	36-100	36-100
Characteristics of Fraction Passing #40:											
Liquid Limit				0-40	41+	0-40	41+	0-40	41+	0-40	41+
Plasticity Index	0-6		N.P.	0-10	0-10	11+	11+	0-10	0-10	11+	11+
Group Index	0		0	0		0-4		0-8	0-12	0-16	0-20
Usual Types of Significant Constituent Materials	Stone Fragments Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good					Fair to Poor					

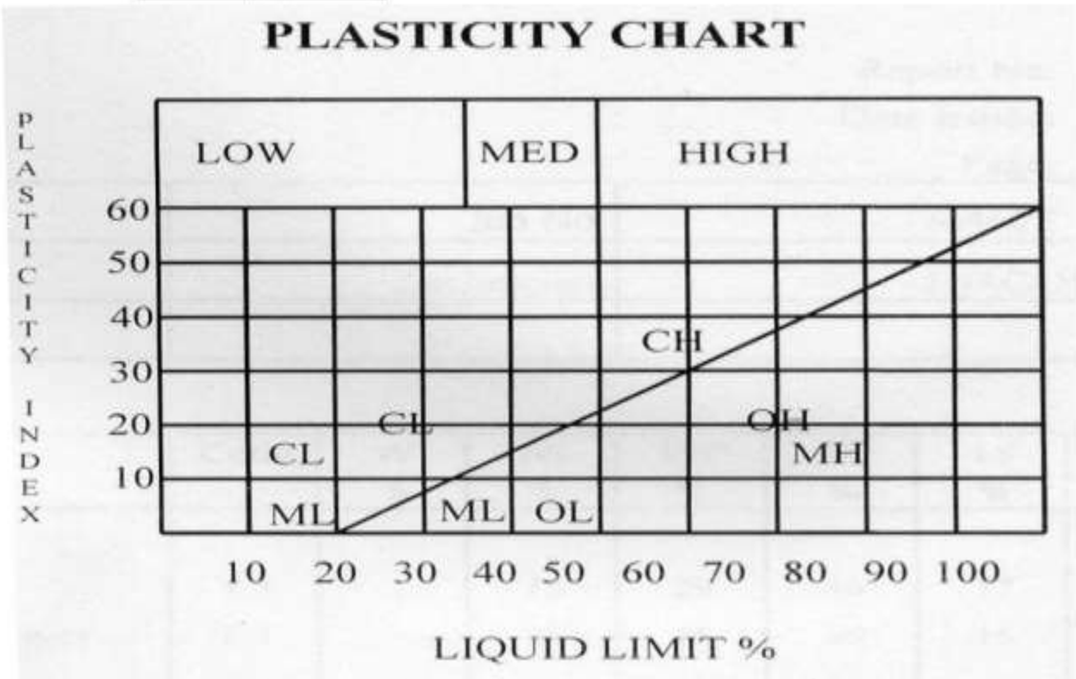
2. Appendix B; Part of Unified Classification Tables

First and/or second letters

Symbol	Definition
G	gravel
S	sand
M	silt
C	clay
O	organic

Second letter

Letter	Definition
P	poorly graded (uniform particle sizes)
W	well graded (diversified particle sizes)
H	high plasticity
L	low plasticity



3. Appendix C; Guideline for limit equilibrium of a slope

Factor of Safety	Details of Slope
<1.0	Unsafe
1.0 – 1.25	Questionable safety
1.25 – 1.4	Satisfactory for routine cuts and fills, Questionable for dams or where failure would be catastrophic.
>1.4	Satisfactory for dams / slope.

Source: Geo-slope (2018)