

**ADDIS ABABA UNIVERSITY**  
**ADDIS ABABA INSTITUTE OF TECHNOLOGY**  
**AFRICAN RAILWAY CENTER OF EXCELLENCE**



**Investigation of the Cause and Remedial Method for Failed  
Slope Around Railway Tunnel Portal  
A Case of Awash-Kombolcha-Hara Gebaya Railway Project,  
Tunnel 09**

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A Thesis in Master of Science in Railway Engineering  
(Civil Infrastructure)

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08 July, 2019

Addis Ababa

A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science in  
Railway Engineering (Civil Infrastructure)

Investigation of the Cause and a Remedial Method for Failed Slope Around Railway  
Tunnel Portal

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### UNDERTAKING

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Mahamoud Ahmed Muhumed

## **ABSTRACT**

Ethiopia has been trying to construct a new railway route that links central Ethiopia to the northern regions. However, this route crossed mountainous terrains which demanded the construction of several tunnels. The failed slope studied in this research is around tunnel 09 entrance portal located along with the new AKH single track railway line phase II. During the excavation of this slope, some cracks were detected and within a short time, these cracks developed into full sliding. All works on the site were stopped immediately for the safety of the employees.

During the design of the above-said slope, the geotechnical ground conditions were defined as rock and 1V:1H slope geometry was selected. However, as construction progressed failure occurred on soil deposits located at the bottom of the slope, which is contrary to the expected rocky condition. The slope inevitably is due to the 1V:1H slope geometry that was used during construction.

In this research, limit equilibrium method and finite element method have been used to evaluate the stability of the slope. For LEM, SLOPE/W (GEOSTUDIO, 2012) software has been used to compute the factor of safety, whereas PLAXIS2D software has been used for the FE method. Four cases of unreinforced slopes have been analyzed using both methods. Later, slope stabilization techniques were implemented by reducing slope geometry and reinforcing nails with different inclination angles.

Three cases from the literature have been analyzed to verify and validate the output result obtained from LEM and FEM software used in this work.

Keywords: Tunnel portal slope stability analysis, LEM, FEM, Slope/w, Plaxis2D, FoS

## ACKNOWLEDGMENTS

First of all, I would like to thank my almighty God who I can do nothing without his help. Secondly, I would like to express my deep sincere to my principle supervisor Dr. Im Soo-Been for all support and encouragement he provided me. I am grateful for his guidance and professional advice during my research work that allowed me to complete my thesis successfully.

I specially want to thank African Railway Centre of Excellence (ARCE) in AAiT and World Bank for the opportunity they provided me to study this MSc program.

I would also like to thank all staffs of the ARCE, specially Dr. Abraham, Dr. Danial, Zewdie Moges, Tenagne Sheferow, and my special thanks also extend to postgraduate students in ARCE for their friendship during this program.

Finally, I would like to thank my parents for their continuous support, encouragement, advice, and prayers throughout the program.

Mahamoud Ahmed Muhumed

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July 2019

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## LIST OF SYMBOLS

|             |  |
|-------------|--|
| $u$         | Pore water pressure  |
| $l$         | Slice base length and  |
| $\alpha$    | inclination of slip surface at the middle of the slice                       |
| $c'$        | The effective cohesion   |
| $\varphi'$  | The effective angle of friction  |
| $N$         | Slice base normal force which can be obtained by summation of vertical force |
| $W$         | Slice weight   |
| $\beta$     | Slope inclination angle  |
| $\gamma_d$  | Dry unit weight of soil (KN/m)   |
| $\lambda$   | Scale factor   |
| $\nu$       | Poisson's ratio  |
| $\Psi$      | Dilatancy angle  |
| $\sigma'$   | Effective normal stress (kPa)  |
| $\sigma_1$  | Major principal stress ((kPa)  |
| $\sigma_3'$ | Effective minor principal stress (kPa)                                       |
| $\tau$      | Shear stress or mobilized shear stress (kPa)                                 |
| $\tau_f$    | Shear strength of the soil (kPa)   |

## LIST OF ABBREVIATIONS

|       |  |
|-------|--|
| AKH   | Awash-Kombolcha Hare Gabeya  |
| LEM   | Limit Equilibrium Method   |
| FEM   | Finite Element Method  |
| FoS   | Factor of safety   |
| ARCE  | African Railway Centre of Excellence                                   |
| AAiT  | Addis Ababa Institute of Technology                                    |
| T09   | Tunnel 09  |
| OMS   | Ordinary Method of Slices  |
| BSM   | Bishop's Simplified Method   |
| JSM   | Janbu's Simplified Method  |
| M-PM  | Morgenstern-Price Method   |
| SRF   | Strength Reduction Factor  |
| DDA   | Discontinuous deformation analysis                                     |
| GLE   | General limit equilibrium method                                       |
| Qc    | Undifferentiated materials with the brown non-consolidated soil matrix |
| Qvs   | Eluvial Deposits. Weathered volcanoclastic sediments                   |
| TasTV | Trachyandesite and Basic Ignimbrite/Agglomerate                        |
| CSS   | Critical slip surface  |
| GWT   | Ground water table   |

## CHAPTER ONE: INTRODUCTION

### 1.1 Background

Slope stability analysis is one of the oldest and prominent subject matter in geotechnical engineering, yet it remains one of the most active areas of study in both research and practice [1], particularly large projects (i.e. railway tunnel portal slopes, dams, etc.) where the loss of life and properties are involved. In the last decades, many researchers including Taylor [2], Morgenstern [3] and Fredlund and Krahn [4] have been assessing the stability of slopes. However, this problem still presents significant challenges to the engineers [5]

Slope instability around tunnel portals may result from the change of ground water table, stress redistribution, weathering or natural causes such as heavy rainfall and earthquake. According to Abramson et al (2002), the primary purpose of slope stability analysis is to contribute to the safe and economic design of excavation, slopes, earth dams, embankment, etc. [6]. The main interest of slope stability analysis in tunnel portals is typically to determine the FoS value against slope failure.

Currently, there are several methods used for slope stability assessment. These methods have been used over decades, and today, the advancement and availability of high computational computer-based software make possible to handle very complex slope geometry with a difficult ground condition. Limit Equilibrium method (LEM) is one of the oldest and most commonly methods used for solving slope stability problems. Bishop (1955), Janbu (1968) and Ordinary method of slices (1927) are common types of LEM. These methods of slices are based on purely the principles of statics; that is the summation of moments, vertical forces, and horizontal forces [7].

On the other hand, numerical analysis became the best alternative method for slope stability analysis. Finite element method is among numerical method which is a more advanced approach to handle slope stability evaluation and provide user's more accurate and reliable results. This method also allows evaluating stress and displacement which limit equilibrium do not. However, this method involves more complex theory and it usually requires more time for developing model parameters, performing computer analyses and interpreting the results [8].

## 1.2 Problem Statement

In fact, the stability of slopes either from natural or man-made are usually one of the challenges to any researchers and professional engineers, the field engineering [9]. Particularly, slopes located around tunnel portals that could erode gravely by surface water and develop much weathering fractures; thus, landslides may easily occur and slope's stability is hardly guaranteed [10]. Generally, slope stability depends on various factors, some of them are very difficult stress condition, deformation, water ingress, geological structure. In addition, these are not easy to find out and handle [11].

This research focusses investigation of the cause and remedial method for failed slope around T09 portal. T09 is one of the AKH Railway project located in Amhara Region, northern Ethiopia. Regarding the geological context, this slope is highly complex with material and tectonic variation along the alignment. At the entrance slope of the T09, cracks have already appeared mainly around the left slope of the portal (in increasing chainages). After a few days, these cracks developed into a full slope slide affecting the tunnel portal as figure 1.1 shows.



Figure 1. 1: (a) cracks developed tunnel portal slope (left) 1(b) tunnel portal failure(right)

## 1.3 Objectives

### 1.3.1 General objectives

The main objective of this research is to examine a failed slope around a railway tunnel, using traditional limit equilibrium and finite element method and to introduce stabilization techniques.

### 1.3.2 Specific objectives

To meet out the aforementioned objectives, the following specific objectives are developed.

- To analyze three cases from literature to verify and validate the reliability of the software used in the analysis.
- To develop a numerical model which represents the problem
- To examine the current methods used for slope stability analysis “limit equilibrium and finite element method
- Carry out a parametric study for portal slope failure mechanism and stabilization techniques

This study focusses on examination of failed slope located AKH railway project T09 which is currently under construction. All data (i.e. geological and geotechnical) presented in this work were obtained as a secondary data from AKH railway project T09 investigation.

## 1.4 Scope and limitation

The scope of this work is limited to the assessment of failed slope stability around a railway tunnel portal according to LEM and FEM. SLOPE/W and PLAXIS2D software are used for these analyses. First, three cases from the literature have been analyzed for validating both LEM and FEM software used in this work.

Secondly, four cases of reinforced slope and two cases of unreinforced slope have been analyzed. In each case, FoS and displacement are computed from these analyses. The characteristics of the slope are mainly distinguished through their geometry; the width of the slope, reinforced or non-reinforced and the corresponding inclination. All modelled slopes have similar conditions with respect to soil properties. Therefore, the soil

parameters are not changed within the analysis of the slope. The calculations in PLAXIS 2D are limited to 2D static drained analysis since this is the main focus of the thesis. This thesis does not aim to compare LEM to other computational methods or in other words FEM but to use them parallelly.

## **1.5 Significance of the study**

Slope stability is one of the defining challenges to geotechnical engineers, because of its intrinsic complexity and the high number of factors involved. Various factors contribute to the stability problem of slopes such as running water force, excavation activities, geological and hydrological ground condition, etc. Landslide, rock falls and mass movement are the result of slope failure that could cause loss of lives and properties.

Simple hand solution procedures for slope stability analysis don't properly account interslice forces, shear, and deformation. On the other hand, slope stability evaluation by using only one approach (LEM) may not accomplish the objectives and can result slope stability miss-judgement.

Many researchers have been analyzed the stability of the slopes, yet this problem remains one of the core matters to geotechnical engineers. The slope studied in this thesis which located AKH railway T09 portal, is one example of the current failed slope. Therefore, in this work, the author analyses and find a solution for the slope around T09 entrance portal instability using both limit equilibrium method and finite element method.

## **1.6 Document Structure**

This document consists of six chapters. Chapter one is all about the introduction, problem statement, objectives, scope, and limitation. Chapter two highlights the literature review related to the problem. Chapter three is about the detailed methodology of the research, case study information. Chapter four will highlight results and discussion of slope stability analysis, using the limit equilibrium method and numerical method. The factor of safety from these two methods will be compared accordingly. Chapter five discusses two methods for slope stabilization techniques and analyzing the effect of the application of these stabilization methods. Chapter six, conclusion and recommendation

## **2. CHAPTER TWO: LITERATURE REVIEW**

### **2.1 Slope Failure Mechanism**

Slope instability is one of the major problems in geotechnical engineering where loss of life and property can and do occur [12]. Slope failure has been identified as one of the most frequent natural disasters that can lead to a huge loss of human lives and property. Slope failure occurs when the downward movements of soil or rock material because of gravity and other factors, creates shear stresses that exceed the inherent shear strength of the material [13]. Geological structure (soil stratification), slope geometry, ground water, and seepage play a significant role in slope failure mechanism. Thus, in order to prevent slope failure, a greater understanding of the process through which failure occurs is required [14].

The mechanics and rates of slope movement are controlled by many factors: slope gradients, slope height, structural soil/rock properties, water content, and soil pore water pressure, and certain engineering properties of soil and weathered rock, such as cohesion and friction angle. Moreover, the trigger of a slope failure can be divided into external and internal causes. The forces with in nature like heavy rainfall leading to erosion and landslides constitute an important example of internal disruptive forces while the external forces mainly human activities such as excavation and filling of slopes have also caused the slide [15]. Furthermore, the other external factors that may lead slope failures are like tectonic forces, weathering, and erosion processes, traffic loads and compounded-anthropogenic factors in the high relief mountain system.

According to Hamed Niroumand et al [16], these causes may be summarized simply as follows:

- a) A too high slope or too steep for materials of which it is composed.
- b) Too weak materials to sustain the slope at its present profile.
- c) Too high porewater pressures, and thus adversely affect the soil strength.
- d) The materials contain weak inclusions or discontinuities.

e) Slope is affected adversely by some external influence, for example applied loads from structures or excavation at or near the toe of the slope.

William G., 2006, pointed out slope failure can a result from either rotational or translational which can be subdivided into four failure modes: planar, circular, wedge and toppling failure. Figure 2.1 shows a typical failure that can be expected in rock and soil slopes [6].

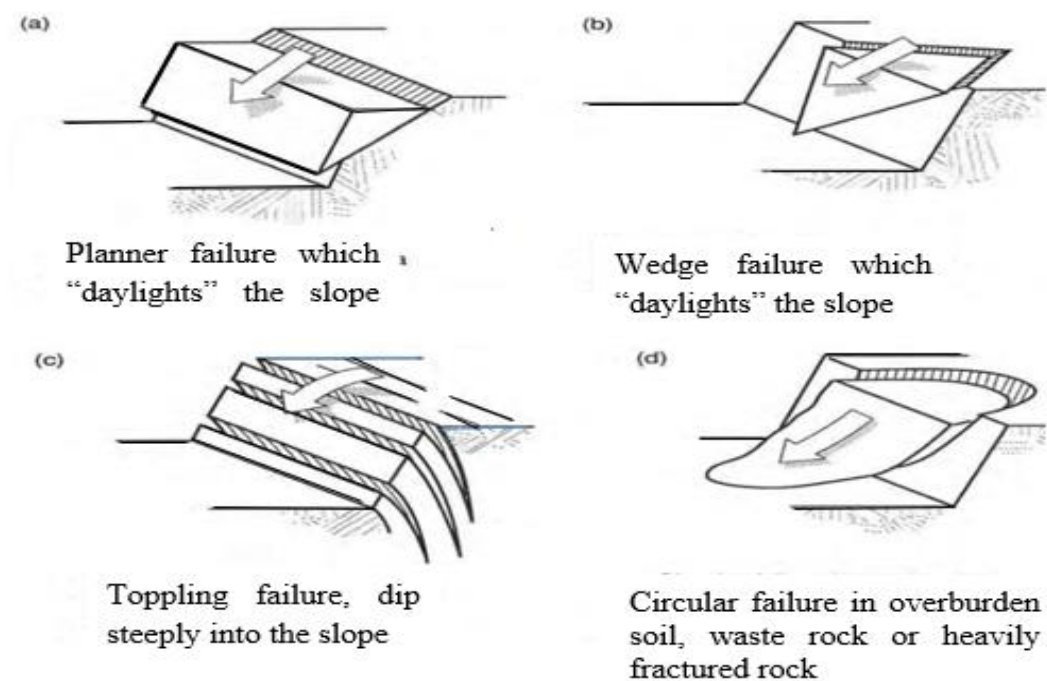


Figure 2. 1: Major types of slope failure [17]

## 2.2 Limit Equilibrium Methods for Slope Stability Analysis

Traditional limit-equilibrium methods are the most commonly-used analysis techniques for slope stability problem [7]. Limit equilibrium method is based on LE formulation that is the principle of static force and moment. Currently, most slope stability analyses involve LEM analysis due to its simplicity and accuracy [12]. These methods consist in cutting the slope into fine slices so that their base can be comparable with a straight line then to write the equilibrium equations (equilibrium of the forces and/or moments) [18]. For the simplest form of limit equilibrium analysis, the equilibrium forces acting on the slope is only required. The total forces inducing on the sliding portion of the slope is compared

with in the sum of the resistance forces and the ration between these two forces is defining factor of safety FoS. (Eq (2-1))

$$F = \frac{\sum \text{Resistance force}}{\sum \text{Driving forces}} \quad \text{Eq - 2.1}$$

Over the last decades, the analysis of slopes using limit equilibrium method has been improved by using various methods such as; Bishop (1955), Janbu (1954), Morgenstern Price (1965), and Spencer (1967). Although Limit Equilibrium methods vary with respect to the slope failure mechanism in equations (I.e. translational or rotational sliding) and the assumption adopted in order to achieve a determinate solution, all LEM share common approaches based on a comparison of resisting forces/moment mobilized and the disturbing forces/moment [10]. According to the assumptions made on the efforts between the slices and the equilibrium equations considered, many alternatives were proposed as table 2.2 presented [18].

Famous methods of the limit equilibrium methods in table 2.1 includes Fellinius (1927), Bishop’s Modified Method (Bishop,1955), Janbu’s Generalized procedure of Slices (Janbu,1968), Morgenstern & Price’s Method (Morgenstern & Price’s, 1965), and Spencer’s Method (Spencer, 1967).

In other words, the differences between these methods depending on which equations of statics are adopted and satisfied, which forces of interslice are used and what is the assumed relationship between the interslice shear and normal forces? Duncan JM pointed out that the difference between various methods was less than 6% [8]. Table 2.2 listed the common method of LEM analysis and the condition of the static equilibrium that is satisfied in determining the factor of safety (FoS).

Table 2. 1: Main Limit Equilibrium Methods

| Methods                                     | Equilibrium condition satisfied                      | Slip surface          | Use  |
|---|--|-----------------------|--|
| Ordinary Method of Slices (Fellinius, 1927) | Moment of equilibrium about the center of the circle | Circular slip surface | Applicable to non-homogenous slopes and C-Ø where slip surface can be approximated by a circle. Very convenient for hand calculation. In accurate for effective stress analysis with high porewater pressure |

|  |   |           |  |
|--|---|-----------|--|
| Bishop's Modified Method (Bishop,1955)                     | Vertical equilibrium and overall moment equilibrium | Circular  | Applicable to non-homogenous slopes and C- $\phi$ soils where slip surface can be approximated by a circle. More accurate than Ordinary Method of slices, especially for analysis with high porewater pressure. The calculation is feasible by hand or spreadsheet |
| Janbu's Generalized procedure of Slices (Janbu,1968)       | Force equilibrium (vertical and horizontal)         | Any shape | Applicable to non-circular slip surface. Also, for shallow, long planar failure surface that is not parallel to the to the ground surface  |
| Morgenstern & Price's Method (Morgenstern & Price's, 1965) | All conditions of equilibrium                       | Any shape | An accurate procedure applicable to virtually all slope geometrics and soil profiles. Rigorous, well established complete equilibrium procedure  |
| Spencer's Method (Spencer, 1967)                           | All conditions of equilibrium                       | Any shape | An accurate procedure applicable to virtually all slope geometrics and soil profiles. The simplest complete equilibrium procedure for computing factor of safety   |

Table 2. 2: Static equilibrium condition satisfied by limit equilibrium methods [6]

| Methods                              | Force Equilibrium |     | Moment Equilibrium |
|--------------------------------------|-------------------|-----|--------------------|
|                                      | X                 | Y   |                    |
| Ordinary Method of Slices (OMS)      | No                | Yes | Yes                |
| Bishop's Simplified Method of Slices | Yes               | No  | Yes                |
| Janbu's Simplified                   | Yes               | Yes | No                 |
| Morgenstern-Price                    | Yes               | Yes | Yes                |

### 2.2.1 Ordinary Method of Slices (OMS)

Fellenius (1936) introduced the first method, referred to as the Ordinary or the Swedish method, for a circular slip surface [19]. This method ignored all interslice forces and fails to satisfy force equilibrium [6]. Therefore, this method satisfied only moment equilibrium. On the other hand, Ordinary method is considered the simplest of the all method of slices since it is the only procedure that results in a linear factor of safety equation and does not require an iteration process [4]. This method is more suitable for hand calculation. Equation 2-2 and 2.3 show the formula used to compute FoS of the OMS.

$$F = \frac{\sum(c'l + N' \tan \phi')}{\sum W \sin \alpha} \quad \text{Eq 2-2}$$

$$N' = W \cos \alpha - ul \quad \text{Eq 2-3}$$

Where,

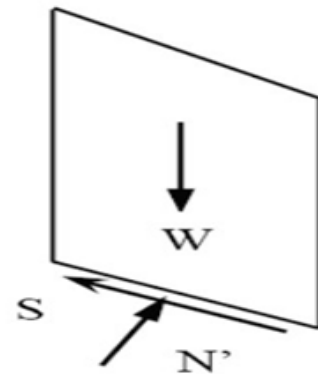
u = pore pressure,

l = slice base length and

$\alpha$  = inclination of slip surface at the middle of the slice

**In summary, OMS**

- ✚ satisfies moment equilibrium condition,
- ✚ neglects the interslice normal and shear forces,
- ✚ gives the most conservative FOS, and
- ✚ is useful only for demonstrations.



The forces considered in OM are shown in the sketch

**2.2.2 Bishop’s Simplified Methods**

The Bishop's simplified procedure is based on the equilibrium of moments about the center of a circular shear surface and equilibrium of forces in the vertical direction [20]. Bishop’s Simplified Method (BSM) is very common in practical for circular sliding surfaces. BSM considers the interslice normal forces but neglects the interslice shear forces [6]. It further satisfies vertical force equilibrium to determine the effective base normal force ( $N'$ ), is given by:

$$N' = \frac{1}{m\alpha} \sum \left( W - \frac{c'l \sin \alpha}{F} - ul \cos \alpha \right) \quad \text{Eq 2-4}$$

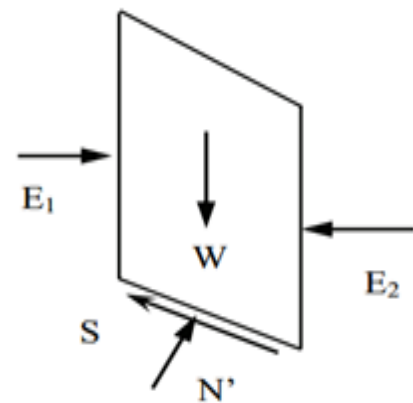
Where,

$$m\alpha = \cos \alpha \left( 1 + \tan \alpha \frac{\tan \phi'}{F} \right) \quad \text{Eq 2-5}$$

Bishop (1955) also showed how non-zero values of the resultant forces ( $X_1$ - $X_2$ ) could be introduced into the analysis but refinement has only a marginal effect on the factor of safety and the pore water pressure can be related to the total fill pressure at any point by means of dimensionless pore pressure ratio  $ru=u/\gamma h$  [6]. In addition, this method assumes a circular failure surface, FoS can be determined by using the same equation (Eq. 2.2) as the OMS. However, computation BSM requires an iterative procedure because of the nonlinear relationship as the FoS appears on both sides. The value of FoS then could be computed by first assuming an arbitrary value for FoS.

### In summary, BSM

- ✚ satisfies moment equilibrium for FOS,
- ✚ satisfies vertical force equilibrium for N,
- ✚ considers interslice normal force,
- ✚ more common in practice, and
- ✚ applies mostly for circular shear surfaces.



The forces considered in BSM are shown in the sketch.

### 2.2.3 Janbu's simplified method

The Janbu's Simplified method is similar to the Bishop's Simplified method except that the Janbu's Simplified method satisfies only overall horizontal force equilibrium, but not overall moment Equilibrium. JSM is based on a composite slip surface (i.e. non-circular) and the FoS is determined by horizontal force equilibrium. As in BSM, the method considers interslice normal forces ( $E$ ) but neglects the shear forces ( $T$ ). The a reduces to the Bishop solution of circular slip surfaces. Janbu's method satisfies vertical force equilibrium for each slice, as well as overall horizontal force equilibrium for the entire slide mass (i.e., all slices) [21]. The simplified method, generalized method and direct method developed by Janbu (1954, 1968) are very common in stability analysis of portal slopes. This method satisfies vertical force equilibrium for each slice as well as overall horizontal forces equilibrium for the entire slice (all slices) [6]. And the base normal force ( $N$ ) is determined in the same way as in BSM as Eq 2-5 shows.

$$N' = \frac{1}{m_a} [w(1 - k_v) - \frac{c \sin \alpha}{F} - u_\alpha \cos \alpha + u_\beta \cos \beta + Q \cos \delta] \quad \text{Eq 2-6}$$

Where  $m_a$  is determined by Eq 2-5, and the FOS of JSM can be than calculated as Eq 2-8

$$Ff = \frac{\sum c/l + (N - ul) \tan \phi'}{\sum W \tan \alpha + \sum \Delta E} \quad \text{Eq 2-7}$$

Where,

$\sum \Delta E = E_2 - E_1 =$  net interslice normal forces (zero if there is no horizontal force).

The reported Janbu factor of safety (FoS) value is calculated by multiplying the calculated F, value by a modification factor,  $f_0$ .

$$FoS_{Janbu} = F_{calculated} * F_0 \quad \text{Eq. 2-8}$$

This modification factor is a function of the slide geometry and the strength parameters of the soil as figure 2.1 shows. The variation of the  $f_0$  value as a function of the slope geometry (i.e. D and L) and the type of soil. The FoS, with this correction factor, can increase by 5 -12%, giving the lower range in friction only soils, i.e. the soils without cohesion and the higher range for clayey soils [6].

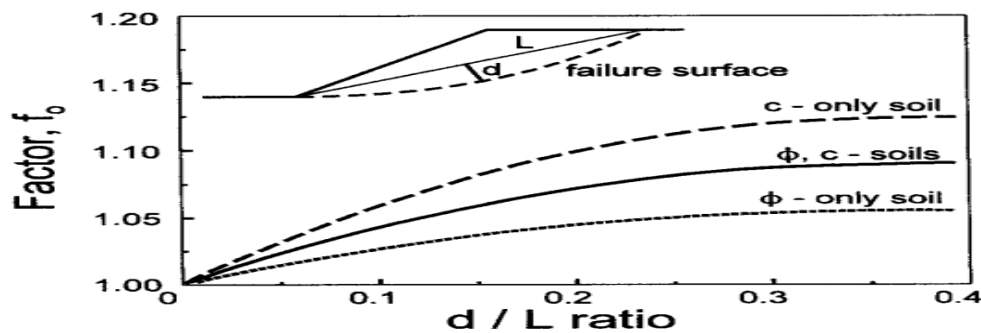


Figure 2. 2: Janbu's correction factor for the simplified method [6]

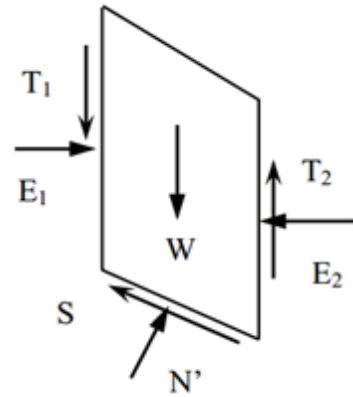
This modification factor can also be calculated according to the formula given below

$$f_0 = 1 + b_1 \left[ \frac{d}{L} - 1.4 \left( \frac{d}{L} \right)^2 \right] \quad \text{Eq-2-9}$$

where  $b_1$  varies according to the soil type: C only soil:  $b_1 = 0.69$ ,  $\phi$  only soil:  $b_1 = 0.31$ , C and  $\phi$  soil:  $b_1 = 0.50$ .

**In summary, JSM**

- ✚ Satisfies both force equilibrium
- ✚ Does not satisfy moment equilibrium
- ✚ Consider interslice normal force and
- ✚ Commonly used for composite shear Surface



The forces considered are indicated in the sketch

**2.3.4 Morgenstern-Price method (M-PM)**

The Morgenstern-Price method is a rigorous method based on limit equilibrium formulation. M-PM also satisfies both force and moment equilibrium and assumes the interslice force function. According to M-PM (1965), This method introduces mathematical arbitrary function which is used to describe the direction of the interslice forces.

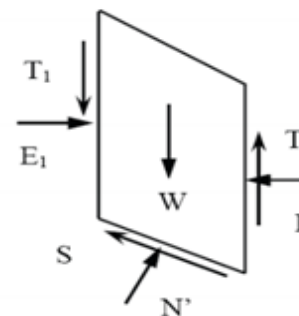
When the function is a constant, the Morgenstern-price method is the same as spencer method. On the other hand, this method is based on the summation of tangential and normal forces to the base of a slice and the summation of moments about the center of the base of each slice [6]. The relationships for the base normal force (N) and inter-slice forces (E, T) are shown in the sketch. The expression of safety factor (F) in terms of force (F<sub>f</sub>) and moment equilibrium (F<sub>m</sub>) is given as follows;

$$F_f = \frac{\sum\{c'l+(N-ul) \tan \varphi'\} \sec \alpha}{\sum\{W-(T_2-T_1)\} \tan \alpha + \sum(E_2-E_1)} \tag{Eq-2-10}$$

$$F_m = \frac{\sum(c'l+(N-ul) \tan \varphi')}{\sum W \sin \alpha} \tag{Eq-2-11}$$

**In summary, M-PM**

- ✚ considers both interslice forces,
- ✚ assumes an interslice force function, f(x),
- ✚ allows selection for interslice force function,
- ✚ computes FOS for both force and moment equilibrium.



The forces considered M-PM are shown in the sketch.

### 2.2.5 Generalized Limit Equilibrium (GLE) method

A general limit equilibrium (GLE) formulation was developed by Fredlund at the University of Saskatchewan in the 1970s [4]. This method encompasses the key elements of all of the methods of slices [22]. Moreover, GLE is an extension of Spencer and Morgenstern-Price methods where, the interslice slope,  $\tan\theta = \lambda$ .  $f(x)$  is assigned to determine the interslice forces [6]

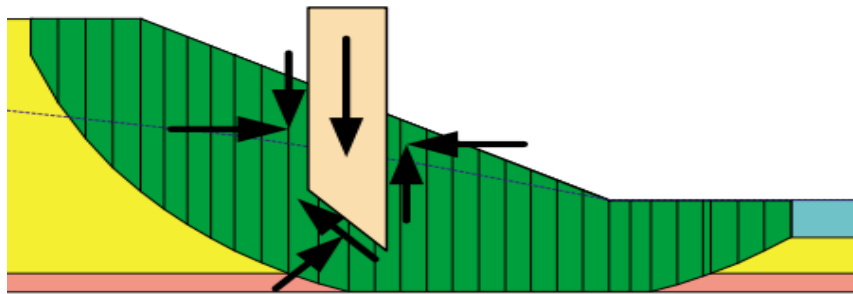


Figure 2. 3: Slices and forces in a sliding mass [23].

The GLE formulation is based on two factor of safety equations and allows for a range of interslice shear-normal force conditions. The interslice shear forces in the GLE method are handled with an equation proposed by Morgenstern and Price (1965) [3][1].  $X = E\lambda f(x)$ . Where  $f(x)$  is a function,  $\lambda$  is the percentage (in decimal form) of the function used,  $E$  is the interslice normal force, and  $X$  is the interslice shear force.

J. Karhn, 2003. introduced figure 2.10 to shows a typical half-sine function. The upper curve in this figure is the actual specified function. The lower curve is the function used. The ratio between the two curves is  $\lambda$ . In addition, Fredlund and Krahn 1977, pointed out a good procedure to compare GLE to the most common methods of a FoS versus  $\lambda$  diagram as shown in Fig. 2.9. [4].

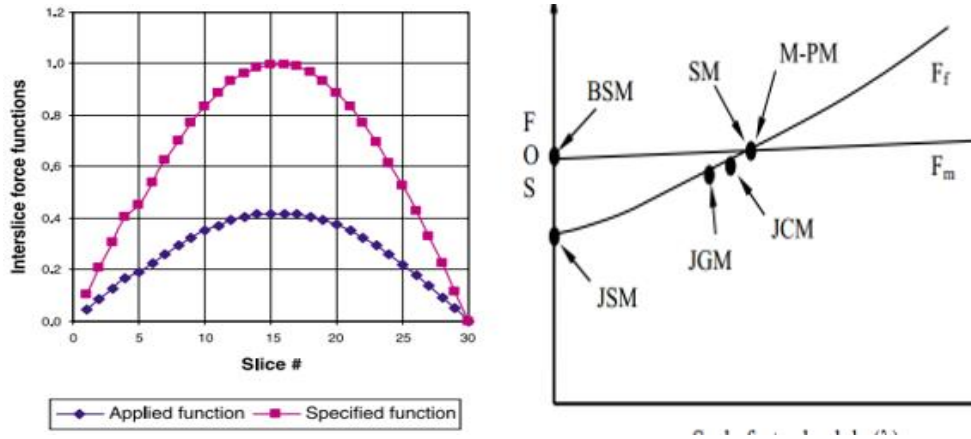


Figure 2. 4: (a) Half sine interslice force function, (b) Scale factor Lambda [22]

The GLE formulation is based on two factors of safety equations. One equation gives the factor of safety with respect to moment equilibrium,  $F_m$  (equation 2-11) while the other equation gives the factor of safety with respect to horizontal force equilibrium,  $F_f$  (equation 2-12)

The GLE factor of safety equation with respect to moment equilibrium is

$$F_m = \frac{\sum [c'\beta R + (N - u\beta)R \tan \phi']}{\sum W_x - \sum Nf \pm Dd} \quad \text{Eq-2-12}$$

The factor of safety equation with respect to horizontal force equilibrium is

$$F_f = \frac{\sum [c'\beta \cos \alpha + (N - u\beta) \tan \phi' \cos \alpha]}{\sum N \sin \alpha - D \cos w} \quad \text{Eq-2-13}$$

All though limit equilibrium is simple and quack analysis of slope stability evaluation, this method misses some basic fundamental physics of stress-strain relationships [22]. The major disadvantages of these methods are (1) they do not take into account the soil behavior and (2) the complex cases cannot be studied with precision [18]. Saha, S. 2010, listed some drawbacks of LEM and says [24]:

- (1) Whether it is an embankment or an excavation, the method of analysis is the same.
- (2) The effect of incremental construction has not been taken into consideration.
- (3) Stress strain relation of the material is not included in the calculation.
- (4) Initial stress of the slope has also to reinforce into consideration

## 2.3 Numerical Method for Slope Stability Analysis

Conventional methods for slope stability analysis are limited to simplistic problems in their scope of application, encompassing simple slope geometries and basic loading conditions and such provide little insight in to slope failure mechanism [10]. But many tunnel portal slope instabilities involve complex geometries, in-situ stresses, non-linear behavior, dynamic, and loading. Therefore, searching for an alternative and more advanced approach for slope stability evaluation has created a great demand in the field of geotechnics. Thus, the advancement of a numerical method for slope stability analysis has been taking advantage of the development of high processing computers and computer-based software's. Dr. Erik Eberhardt 2013, says, regarding the material assumption, numerical methods used to analyze slope stability can be divided in to three categories [10]:

- Continuum method
- Discontinuum method
- Hybrid method

In addition, all these three types have a capacity to model a varying structure, different types of stress states, different overburden condition, and loading applied to the ground [25]. Moreover, Rana Muhammed outlined the steps recommended for performing numerical analysis for slope stability Analysis as follow [11]:

Step 1: Define the objective of numerical analysis

Step 2: Selection of appropriate software and of 2D or 3D approach

Step 3: Conceptual drawing of the analysis layout

Step 4: Create geometry and finite element mesh

Step 5: Application of boundary condition, initial condition. and external loading

Step 6: Apply material properties

Step 7: Check the results and

Step 8: Interpretation of the results

### 2.3.1 Finite Element Method for Slope Stability Analysis

Finite Element Method is a numerical analysis technique which provides an approximate solution for engineering problems [26]. Finite element method represents a powerful alternative approach for slope stability analysis which requires fewer assumptions, especially, regarding the failure mechanism [27]. Likewise, LEM, this method can handle very difficult slope geometry and ground properties with pore water pressure and loading condition. Finite element method can also compute displacement which LEM does not. However, FEM involves more complex theory and it usually requires more time for developing model parameters, performing the computer analyses and interpreting the results [8].

Griffiths and Lane (1999) listed several advantages of FEM over the conventional limit equilibrium method [27]:

- ✚ In this method, no assumptions are needed to be made in advance about the shape or location of the failure surface. Failure occurs “naturally” through the zones within the soil mass in which the soil strength is unable to resist the applied shear stresses.
- ✚ Since there is no concept of slices in the finite element approach there is no need for assumptions about slice side forces. The finite element method preserves global equilibrium until “failure” is reached.
- ✚ If realistic soil compressibility data is available, the finite element solutions will give information about deformations at working stress levels.
- ✚ The finite element method is able to monitor progressive failure up to and including overall shear failure.

Moreover, FEM for slope stability analysis can provide a very good prediction of the behavior of soil/rock structure interaction problems if the different construction stages and the material behavior are simulated correctly and accurately in the analysis [28]. Furthermore, according to the B.H. Mule, 2011, the main concepts of the FEM are [28]:

- 1- Discretization of the region being analyzed into finite elements. These discrete elements are assumed to be interconnected only at the joints which are called nodes.

- 2- The use of interpolating polynomials to describe the variation of a field variable within an element.

In other words, the procedure of computational modelling using the FEM broadly consists of main steps: modelling of the geometry, meshing (discretization), specification of material property and specification of the boundary [29].

Duncan JM, (1996), Zienkiewicz et al, (1975), Taylor's (1937) Smith & Hobbs (1974) and other several scholars have been researched FEM for slope stability analysis and confirmed the accuracy of the use of this method. [8] [30] [2] [31]. In addition, Griffiths (1980) extended this work to show reliable slope stability results over a wide range of soil properties and geometries as compared with charts of Bishop & Morgenstern (1960). [32] [33]. The computation of FoS using FEM based software is generally performed as a two-dimensional (2D) analysis. Two-dimensional (2D) analysis is simple and the result is more accurate compared with traditional LEM. Most importantly, the two-dimensional (2D) factor of safety is generally considered to be conservative (i.e. lower than the 'true' 3D factor of safety), so practitioners are not willing to invest in the more time-consuming 3D approaches [1].

## 2.4 Factor of Safety

There are various definitions of a factor of safety. Some of these definitions are summarized in figure 2.5. However, according to Wright and Duncan (2005), the most common used definition of FoS is the ratio of ultimate shear strength to the shear stress mobilized at imminent failure [20].

$$\text{FoS} = \frac{\text{Available Shear strength}}{\text{Shear stress required for equilibrium}}$$

The total shear strength inside the soil indicates the shear strength due to the different soil/rock properties, i.e. the soil cohesion, internal friction; while the shear stress required for the balance is mainly caused by the weight of soil and water which is the downward component of the total soil water weight along the slope. One of the well-recognized function of the FoS is to account for uncertainty and to guard against ignorance about the

reliability of the slope analysis, such as strength parameters, pore pressure distribution and stratigraphy [6].

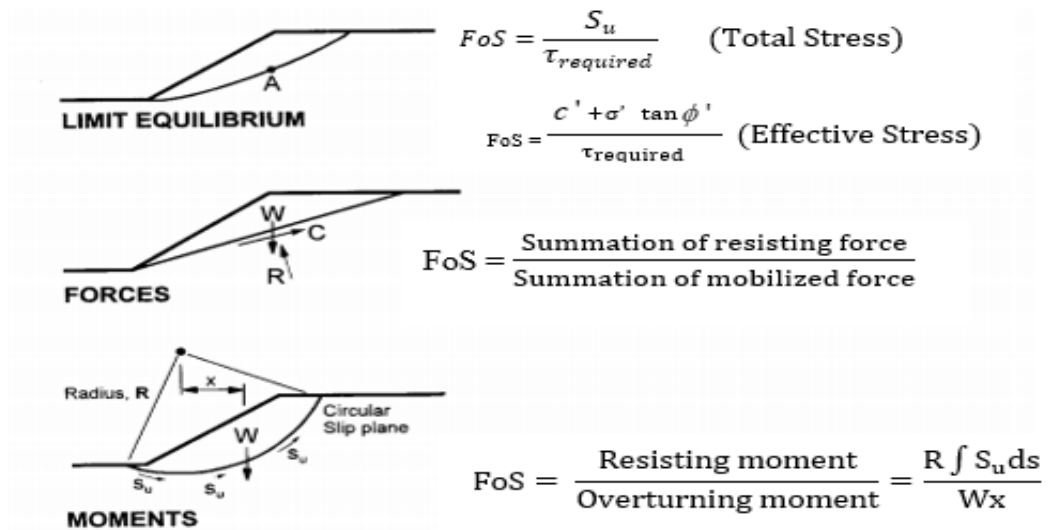


Figure 2. 5: Various definitions for FoS [3]

Omer, (2013) says the most basic purpose of slope stability analysis is to determine a factor of safety against a potential failure, or landslide [34]. On the other hand, the main assumption of the FoS in limit equilibrium is the same at all points along the slip surface. Generally, FoS less than 1.0 indicates that the slope is unstable whereas FoS is significantly greater than 1.0 is considered that slope is stable in certain condition [35]. Table 2.3 and 2.4 are presented the recommended values of FoS according to the US. Army Corps of Engineering Slope Stability Manual (2003) and Wright and Duncan (2005) respectively.

Table 2. 3: Factor of safety criteria from U.S. Army Corps of Engineers’ slope stability manual (2003)

| Type of Slope   | Required Factor of Safety |                              |                |
|---|---------------------------|------------------------------|----------------|
|   | For End of Construction   | For long-term steady seepage | Rapid drawdown |
| Cost of repair comparable to incremental cost to more conservatively designed slope | 1.3                       | 1.5                          | 1.0-1.20       |

Table 2. 4: Recommended minimum values of factor of safety (Duncan and Wright 2005)

| Cost and Consequences of Slope Failure  | Uncertainty of Analysis Condition |                |
|---|-----------------------------------|----------------|
|   | Small                             | Large          |
| Cost of repair comparable to incremental cost to more conservatively designed slope                   | 1.25                              | 1.5            |
| Cost of repair much greater than the incremental cost to construct more conservatively designed slope | 1.5                               | 2.0 or greater |

## 2.5 Slope Stabilization Method

Stabilization is a term in which the natural soil/rock is changed in order to meet the engineering purposes by means of the physical, chemical, biological and combined method of either two of them or all three [36]. Stabilizing a slope involves increasing the FoS [37].

Since slope failures are common in many parts of the world which occur due to manifold reasons and they result in huge losses to the respective locals [38]. Many researchers have been studied slope stabilization techniques and introduced different approaches such as controlling ground water with drainage, surface cover, excavating and regrading slopes, and reinforcing support structure [37].

Abramson et al (2002) described the general objectives of slope stabilization as a way to reduce driving forces and increase resistant forces or both [6]. Generally, artificial slopes adjacent to infrastructures such as road, dams, and railways can be less stable and requires to stabilize. However, the selection method for slope stabilization is a site specific [37].

According to Pun, W. and G. Urciuoli [39], the range of slope stabilization works may be categorized as follows:

- a. Surface protection and drainage
- b. Subsurface drainage
- c. Slope regrading

- d. Retaining structures
- e. Structural reinforcement
- f. Strengthening of slope-forming material
- g. Vegetation and bioengineering
- h. Special materials and technique

Walker and fell [40], pointed that, stabilization of slope involves some or all of the following

- ✚ Reducing pore pressure in the by surface and subsurface drainage
- ✚ Reducing de-stabilizing forces by removing slide material or removing material from the upper part of the slide
- ✚ Increasing stabilizing forces by adding weight to the toe of unstable or by increasing the shearing strength along the failure surface
- ✚ Supporting unstable areas by construction of retaining walls

### **2.5.1 Geometrical Method for Slope Stabilization**

Geometry method for slope stabilization is one of the most common approaches in slope remedial. Yulindasari and Nurly have studied the effect of slope angle on slope stability by using LEM and concluded that the steepness of the slope the more it susceptible to failure [41]. Cornforth also discussed a case study of slope in Pelton Dam in central Oregon, and he concluded that the lowering slope angle can significantly decrease observed failure [37]. Moreover, slope regrading for slope stabilization is simple and cheaper in cost but requires sufficient space [38]. Changing the slope angle from steep to gentle increases slope stabilization by increasing of a factor of safety. [42].

Abdoullah Namdar et al [43], studied evaluation slope stability performance in different methods, and they concluded by realizing the presence of the direct correlation between FoS and Slope geometry

### **2.5.2 Drainage Method for Slope Stabilization**

Good drainage is a key element in the satisfactory of the slope functioning [44]. Drainage is a basic way to minimize the amount of water present in the slope that could result slope

failure [37]. According to N. Mizal et al (2011), slope saturation and pore water pressure build up in subsoil as a result of ground or surface water is one of the major factors for slope failure, and thus proper design of surface and subsurface drainage system be minimizing the chance of building up pore water pressure and therefore slope stability can be increased [38]. Moreover, drainage measurement for slope stabilization is implemented to where there is an obvious source of water above the landslide back scarp that continues to channel water into the slide mass [44]:

- ✚ There are springs evident in the back scarp;
- ✚ There is suspected seepage from the landslide back scarp into the landslide debris in front of it (this may not be evident from surface inspection, but might be inferred from the drainage condition of the debris if there are no other apparent sources of water); and
- ✚ Requirements to increase soil strength through a reduction in pore water pressure, and hence an increase in effective stress

However, implementation of drainage into the railway tunnel portal slopes requires great attention to the maintenance of the drainage system in both surface and subsurface. Special consideration is required for subsurface drainage system because sometimes it is difficult to maintain it and could result improper functioning. Figure 2.7 shows typical drainage measures for slopes

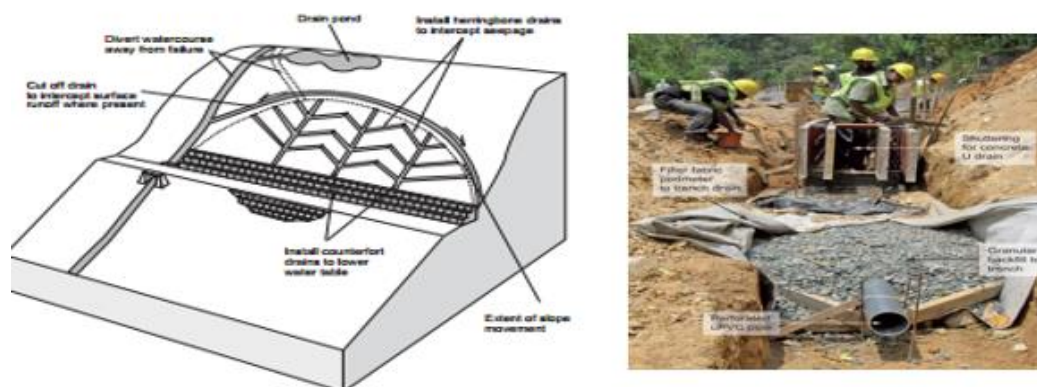


Figure 2. 6: Slope drainage system

### 2.5.3 Retaining Structures for Slope Stabilization

G.J Hearn (2011) described retaining structures as a feature of a rail or road construction in a hilly and mountainous area which can account 20% of the total construction cost [44]. Likewise, Hearn, N. Mizal et al (2011), noted that the retaining structure is highly cost and added retaining structure is always the most commonly used method for slope stabilization due to its flexibility in a constrained site [42]. Generally, this method “retaining structures” comprise [44]:

- ✚ Gravity walls, where the weight of the wall and its backfill provide most of the stabilizing force (masonry, gabion and reinforced concrete cantilever walls are typical examples);
- ✚ Embedded walls, where the soil in front of and behind the structure and anchors (if any) provide the stabilizing force (sheet pile or bored pile walls are typical examples);
- ✚ Reinforced soil, where the in-situ soil mass is reinforced with nails or dowels (usually behind a protective face);
- ✚ Reinforced fill, where steel or geosynthetic geogrids or straps are embedded into the fill during its emplacement

The main objectives for using retaining structure is to resist the downward forces of the soil or rock masses. Figure 2.8 summarizes types of retaining.

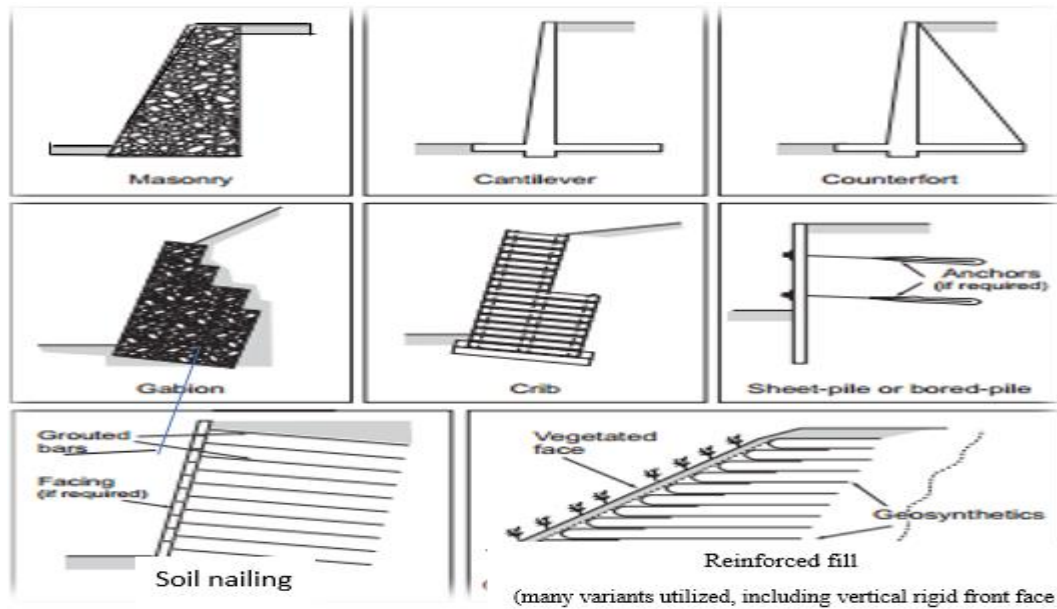


Figure 2. 7:Typical retaining structures [44]

### 2.5.4 Slope Stabilization by Nail Reinforcing

Nailing is a technique of in-situ ground reinforcement which is widely used in foundations, pit excavations and permanent slopes that can help to reinforce the slope by interacting with soil, and it can improve the stability of the slope obviously [45]. The fundamental concept of soil nailing is to reinforce the soil with closely spaced passive inclusions to create a coherent gravity structure and thereby increase the overall shear strength of the in-situ soil, restraining its displacements [46]. Nailing provides resistant forces against slope failure, and its construction process is faster than other similar methods [47].

Midhula and S. Chandrakaran (2015) noted that soil nailing is an accepted, economical, top-down construction technique that increases the overall shear strength of unsupported soils in situ through the installation of closely spaced reinforcing bars (nails) into the soil/rock [48]

Md Akhtar and Ashraful Islam (2016), studied the effect of variation of nail inclination on a dry slope and then concluded [49];

- The factor of safety of a slope increase with the use of nail (slope stability was improved).

- From the result of the analysis, it is found that the variation of nail inclination has a significant influence on the factor of safety of the slope stability problem
- The optimum nail inclination improves the factor of safety by 18%.

Generally, the nails used to reinforce slopes are driven into angles of  $0^{\circ}$ ,  $15^{\circ}$  to  $30^{\circ}$  with a horizontal plane [50].

## **2.6 Software Used for Slope Stability Analysis**

Currently, there is various geotechnical software that can be used for evaluating slope stability. The software used to utilize LEM formulation has been using over decades to solve geotechnical problems. In recent decades, many professional engineers and researchers put their interest in numerical simulation software, based on constitutive law which can estimate displacement and produce more accurate results than LEM software. However, both LEM and FEM can also parallelly used to evaluate the stability of slopes. in the field of geotechnics.

### **2.6.1 SLOPE/W Software**

SLOPE/W is a geotechnical software developed by GEOSLOPE International, Canada, based on limit equilibrium principles [49]. This software is one of the modern software for LEM that make possible to deal with complex slopes, with porewater pressure. This software can be used separately or integration with SEEP/W, QUAKE/W, etc. SLOPE/W is used to calculate FoS in a various shear surface such as circular, non-circular and user defined surfaces [23] The comprehensive formulation of SLOPE/W makes it possible to easily analyze both simple and complex slope stability problems using variety methods to calculate the FoS [49]. For this presented study, SLOPE/W was adopted to evaluate the stability of the railway tunnel portal slope by computing FoS according to a different method of slices.

### **2.6.2 PLAXIS2D Software**

PLAXIS is a finite element code for geotechnical application [51]. This software is encompassed different features that enable to deal with complex geotechnical structures. PLAXIS has several FEM modeling options and four main sub-routines. These routines are; Input, Calculation, Output, and Curve plots [51]. The Factor of safety which

represent as  $\Sigma Msf$  is determined from curve plots sub-routine. The factor of safety for stability analysis in PLAXIS2D is defined as the ratio of the available strength over the strength at the failure and is denoted  $\Sigma Msf$ , see Equation 2.15 [52]

$$FoS, PLAXIS2D = \Sigma Msf = \frac{\text{Available strength}}{\text{Strength at the failure}} \quad \text{Eq 2.15}$$

In this work, Plaxis2D software was used to assess the stability of slope by computing FoS and displacement. Plaxis2D also was used to cross check the result obtained from LEM based SLOPE/W analysis. The procedure and options selected during PLAXI2D and SLOPE/W for slope stability analysis are discussed in chapter three.

### **3.0 CHAPTER THREE: RESEARCH METHODOLOGY**

#### **3.1 Introduction**

This study aims evaluation and remedial method for failed slope in a railway tunnel portal. Limit equilibrium and finite element method have been parallely used in these analyses. For the limit equilibrium method, SLOPE/W software has been used to evaluate the stability of the slope by determined FoS. SLOPE/W is GEOSTUDIO software package sub-routine. On the other hand, FEM based PLAXIS2D has been used in numerical analyses. Factor of safety is computed from both methods, while displacements are determined from FEM. Figure 3.1 summarized methodology used in this thesis.

#### **3.2 Computation of Factor of Safety by LEM.**

All types of LEM analysis based on the method of slices share common feature and limitation and each method satisfy either one or both of force equilibrium and moment equilibrium equation as table 2.1 and 2.2 summarized. In this research, the FoS is determined according to the Morgenstern-Price's procedure and then FoS of other methods of slices such as OMS, BMS JSM and JGM are computed. Morgenstern and Price method (M-PM) satisfies all requirements of the static equilibrium. Generally, this method contains a number of unknowns (i.e. forces and forces location, etc.) and the problem of computing FoS is statically indeterminate. Thus, to achieve a balance equation and unknowns' assumption are required. The steps required to provide the input data for performing the slope stability analysis include [18]

- ✚ A survey of the elevation of the ground surface on a section perpendicular to the slope.
- ✚ Estimation of ground stratigraphy from borehole logs and soil/rock properties from engineering soil/rock tests.
- ✚ The determination of ground water level from piezometer readings to estimate ground water pore-water pressures if required.

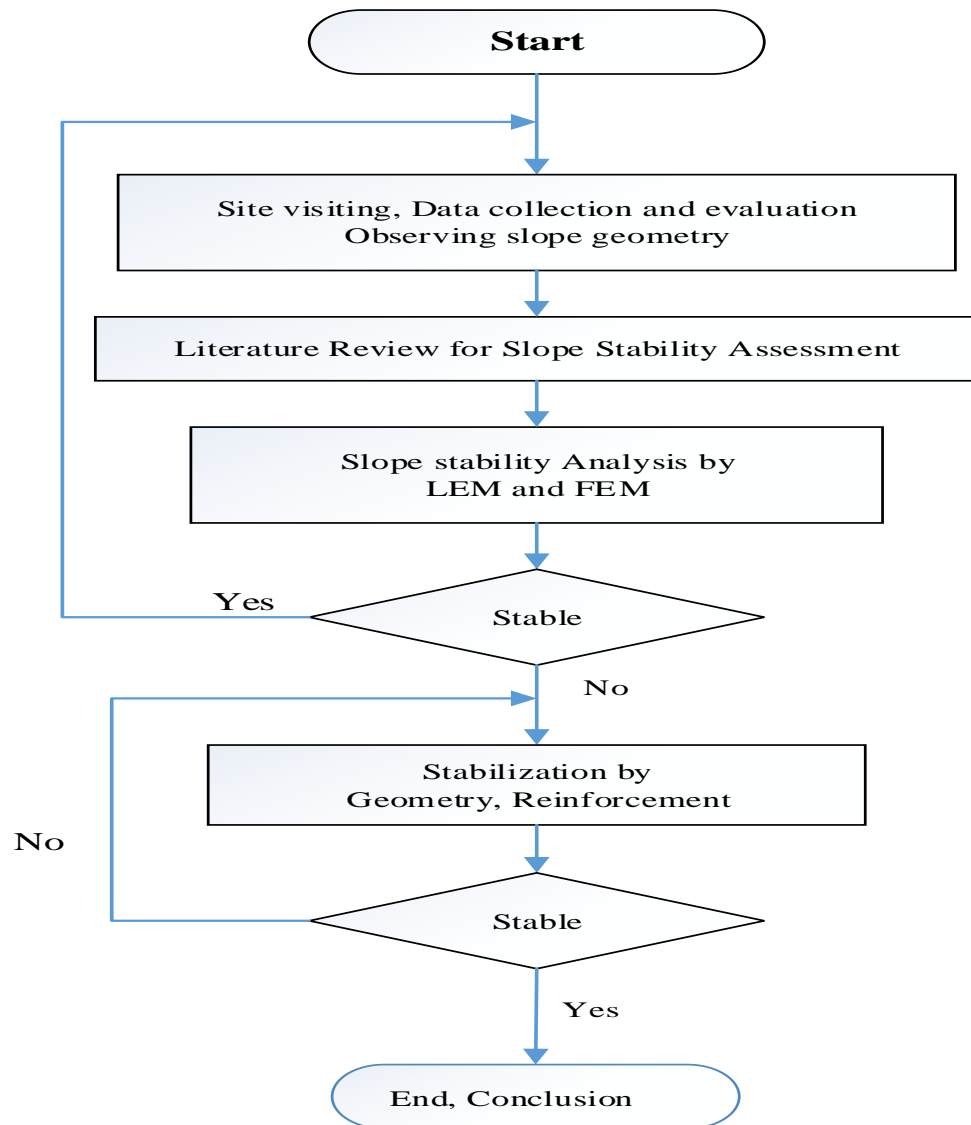


Figure 3. 1: Research methodology follow chart

### 3.2.1 Morgenstern-price procedure

As stated before, FoS is determined according to Morgenstern-Price. This method (M-PM) satisfies both force and moment equilibriums and assumes the interslice force function as discussed in chapter two. M-PM is very similar to Spencer's method; thus, Spencer's and M-PM of slices are both satisfying all equations of statics. Furthermore, these methods are very suitable for slope stability analysis by using SLOPE/W software.

In addition, for this analysis to begin with the generalized formulation, the following assumptions are made for a body of mass slope:

1. The failure surface is assumed to be circular.

2. The soil/rock is a non-homogeneous.
3. The soil/rock is an isotropic material.
4. The failure mass is a rigid body.
5. The base normal force acts at the middle of each slice.
6. The Mohr-Coulomb failure criterion is used.

The critical circular slip surface and the corresponding factor of safety are computed by using SLOPE/W software.

### 3.2.2 Modeling

Generally, there are different methods used to define the material strength of the soil/rock mass. These models include the hardening soil model, soft creep soil model, and Mohr-coulomb model. However, in this work, Mohr-coulomb model is adopted for this slope stability analysis. The equation below illustrates a graphical representation of the coulomb shear strength equation as figure 3.2 shows.

$$\tau = c + \sigma_n \tan \phi \quad \text{Eq- 3.1}$$

Where,  $\tau$  is shear strength (i.e., shear at failure),  $c$  is cohesion,  $\sigma_n$  is normal stress on shear plane, and  $\phi$  is angle of internal friction. The equation 3.1 represents a straight line on shear strength versus normal stress plot (Figure 3.2). The intercept on the shear strength axis is the cohesion  $c$  and the slope of the line is the angle of internal friction  $\phi$ .

In addition, the intercept on the shear strength axis is the cohesion ( $c$ ) and the slope of the line is the angle of internal friction.

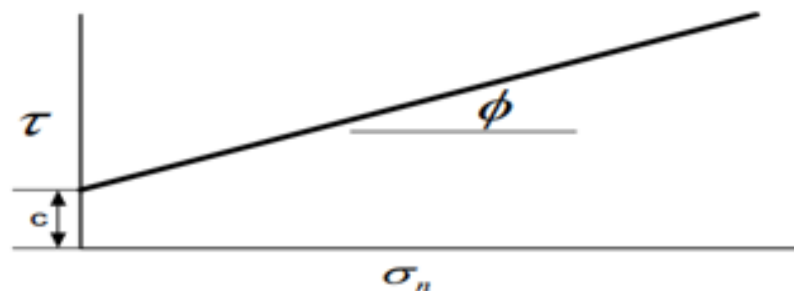


Figure 3. 2: Graphical representation of coulomb shear strength equation

The failure envelope is often determined from tri-axial tests and the results are presented in terms of half Mohr-circles, as shown in figure 3.3 and figure 3.4, hence the failure envelope is referred to as the Mohr-Coulomb failure envelope.

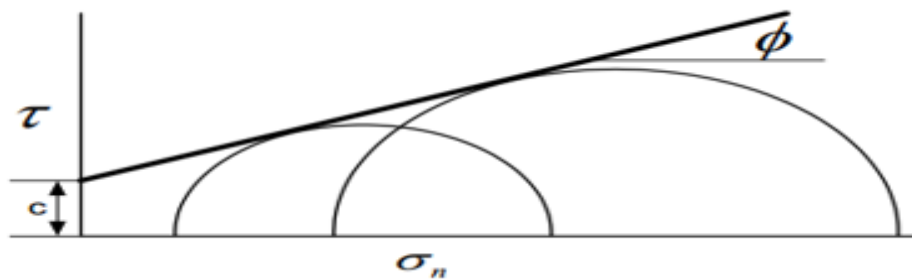


Figure 3. 3: Mohor-Coulomb failure envelope

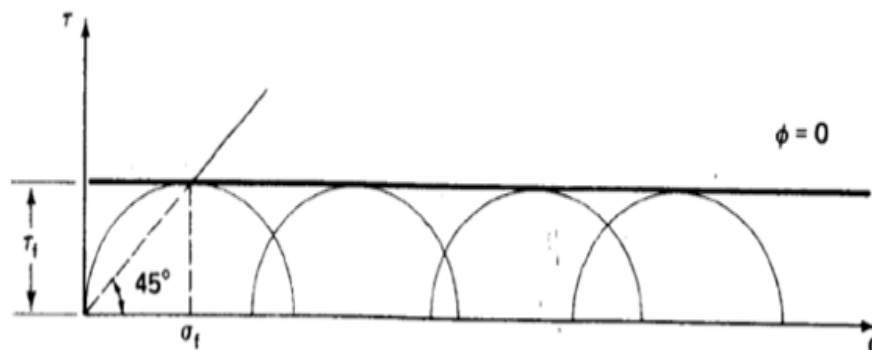


Figure 3. 4 Undrained strength envelope

### 3.3 Slope stability Analysis by FEM

For the finite element method, PLAXIS2D program has been used for this assessment. This program has been used by professional geotechnical engineers and researchers over the last decades. The software was first developed by The Technical University of Delft in 1987 to analyze soft soils of the low lands of Holland and then was extended to cover all aspects and applications of geotechnical engineering simulation using a user-friendly interface with the power of finite element Method [23].

### 3.3.1 Computation of Factor of Safety (FoS) by PLAXIS2D

Generally, using FEM, slope stability can be analyzed either increasing gravity load or reducing the strength characteristics of the soil mass. For this work, the reduction of soil strength parameters was used to compute factor of safety. This method (Phi-c reduction) is based on the reduction of the shear strength ( $c$ ) and the friction angle ( $\tan\phi$ ) of the soil. The strength parameters are reduced each step successively until all the steps have been performed and soil/rock mass fails. Equations 3.3 to 3.6 are the common mathematical expressions used phi-c reduction model.

$$\tau = \sigma_n \tan \phi + c \quad \text{Eq-3.1}$$

where:  $\tau$  = shear strength of soil material on a certain failure plane,  $\sigma_n$  = normal stress on the failure plane,  $\phi$  = angle of internal friction of soil material, and  $c$  = cohesion intercept of soil material.

The shear strength of the sliding surface is denoted by  $\tau_f$  and expressed as follows

$$\tau_f = \sigma_n \tan \phi_f + c_f \quad \text{Eq-3.2}$$

Where  $c_f$  and  $\phi_f$  are the factored shear strength parameters and they can be given as follows:

$$C_f = \frac{c}{SRF} \quad \text{Eq- 3.3}$$

$$\phi_f = \frac{\phi}{SRF} \quad \text{Eq- 3.4}$$

In addition, PLAXIS 2D uses a factor to relate the reduction in the parameters during the calculation at any stage with the input parameters, see equation 3.7 [12].

$$SRF = \frac{\tan \phi_{\text{input}}}{\tan \phi_{\text{reduced}}} = \frac{C_{\text{input}}}{C_{\text{reduced}}} \quad \text{Eq-3.5}$$

Where, SRF= stress reduction factor at any stage during the calculation,  $\tan\phi_{\text{input}}$ , and  $C_{\text{input}}$  are input parameter of the soil/rock;  $\tan\phi_{\text{reduced}}$  and  $C_{\text{reduced}}$  are reduced parameters calculated by the program. The Phi-C reduction model, load advancement number of steps procedure followed and the incremental multiplier 'Msf' is used to specify the increment of strength reduction calculation steps. Finally, PLAXIS computes the FoS as

the ratio of the available shear strength to the strength at the failure by summing up the incremental multiplier (Msf) as defined by:

$$FoS = \frac{\text{Available Strength}}{\text{Strength at failure}} = \text{Value of } \sum \text{Msf at failure} \quad \text{Eq-3.6}$$

### 3.3.2 Material Model

In PLAXIS2D, several different material models are available as discussed before. Among these models are; Mohr-coulomb, linear elastic, hardening soil, and joint rock model. In this analysis Mohr-coulomb law (elastic perfectly plastic) was selected.

### 3.3.3 Mohr-coulomb Model

Mohr-coulomb (perfectly plastic) model is a constitutive model with a fixed yield surface i.e. a yield surface that is fully defined by model parameter and not effected by (plastic) straining [51]. Furthermore, M-C model is based on the elastic-perfectly plastic theory of soil mechanics. This model involves five parameters, namely young 's modulus, (E), Poisson 's ratio, ( $\nu$ ), the cohesion, (C), the friction angle, ( $\phi$ ), and the dilatancy angle,  $\psi$ .

Usually, the formulation of Mohr-coulomb model encompasses six yield function and six plastic functions. Equation 3.7 and 3.8 are illustrating one example of yielding function and one example of plastic function respectively.

$$f_{1a} = \frac{1}{2}(\sigma'_2 - \sigma'_3) + (\sigma'_2 + \sigma'_3) \sin \phi - c \cos \phi \leq 0 \quad \text{Eq-3.7}$$

$$g_{1a} = \frac{1}{2}(\sigma'_2 - \sigma'_3) + (\sigma'_2 + \sigma'_3) \sin \psi \quad \text{Eq-3.8}$$

## 3.4 Case Study information

In this section, the author discussed the general information about the case study, the sliding slope around a railway tunnel portal. This section provides basic information of the geological and geotechnical investigation which has conducted along tunnel 09, particularly entrance portal slope in which this thesis deals with.

### 3.4.1 General Description of the Tunnel Route and Geometry

The slope studied in this work is located Kombolcha and it is part of Awash-Kombolcha-Hara Gebaya railway project. Awash–Kombolcha–Hara Gebaya Railway Project consists

in a 391 km long single-track line connecting northern Ethiopia (Hara Gebaya) with the central region (Awash city). The Client (Employer) is Ethiopian Railway Corporation (ERC) and the Design & Built Contractor is Yapı Merkezi (YM).

Awash–Kombolcha–Hara Gebaya Railway Project will consist of two phases. Both phases (Phase 1 and Phase 2) include 7 tunnels: Tunnel T01 to Tunnel T07 in Phase 1 and Tunnel T08 to Tunnel T14 in Phase 2. Tunnel 09 “in which the studied slope is located” is 1840 m long, located between chainage 276+660 and 278+500. The portal slope of the tunnel 09, particularly the Entrance Portal slope, soil type materials (colluvium and eluvial deposits) was observed. Figure 3.4 and 3.5 show the general layout and Geologic Map along the tunnel 09

### **3.4.2. Site Investigation**

Generally, in order to estimate geological and geotechnical properties of rock and soil mass along the tunnel, a detailed field investigations and measurements include borehole drilled and laboratory test were conducted in the entrance, exit and middle sections of tunnel route. According to the Yapı Merkezi (2017) for T09 Report, the geological and geotechnical evaluation and description of the ground properties are on the basis of Borehole logs, in situ investigation and Laboratory test. Eight number of boreholes were conducted from chainage 276+238 to 278+494. The first three boreholes are located in the entrance portal slope as table 3.1 illustrated.

# Investigation of the Cause and a Remedial Method for Failed Slope Around Railway Tunnel Portal

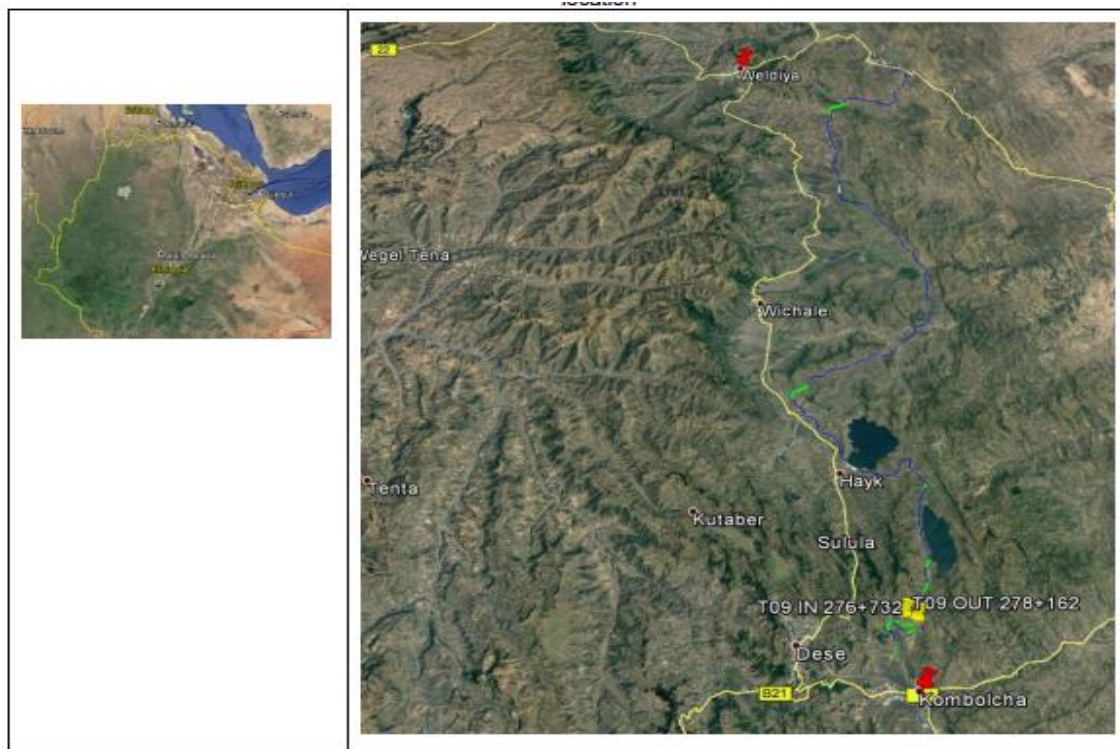


Figure 3. 5: General plan of Awash-Kombolcha-Hara Gebaya single track railway line and Tunnel T09 location

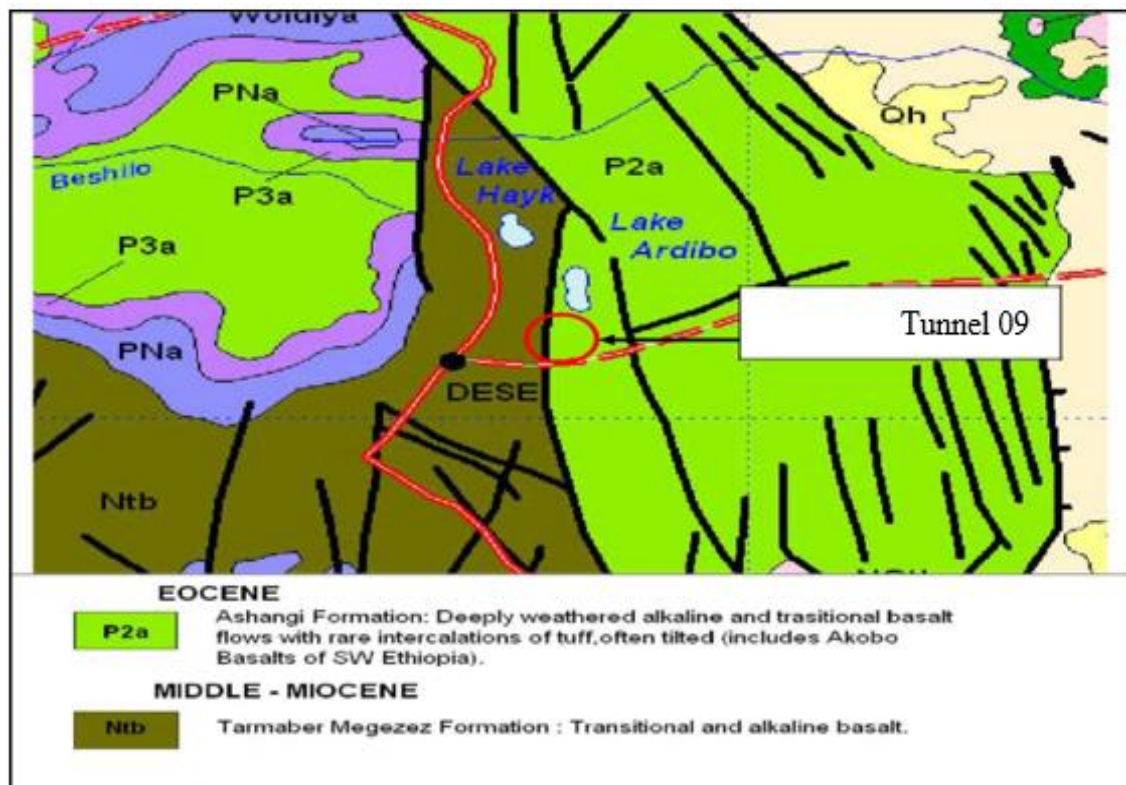


Figure 3. 6: Geological Map of Ethiopian at Tunnel 09 Location

Table 3. 1: Borehole's main features, executed within the context of ground investigation

| Borehole No | Chainage [Km] | UTM Coordinates |            | Structure Type | Elevation [m] | Depth [m] |
|-------------|---------------|-----------------|------------|----------------|---------------|-----------|
|             |               | y [East]        | X[North]   |                |               |           |
| T-09/BH-01  | 276+238       | 580063,99       | 1232351,90 | Portal         | 2098,13       | 25        |
| T-09/BH-02  | 276+456       | 580229.00       | 1232489.00 | Portal         | 2097,85       | 20        |
| T-09/BH-03  | 276+647       | 580299.00       | 1232664.00 | Portal         | 2017.05       | 25        |
| T-09/BH-04  | 276+778       | 580297.00       | 1232793.00 | Tunnel         | 2119.94       | 35        |
| T-09/BH-05  | 276+971       | 580218,71       | 1232967,63 | Tunnel         | 2150,71       | 55        |
| T-09/BH-06  | 277+178       | 580055,86       | 1233090,97 | Tunnel         | 2165,70       | 70        |
| T-09/BH-07  | 277+344       | 579904,51       | 1233159,12 | Tunnel         | 2181,22       | 80        |
| T-09/BH-08  | 278+494       | 578866,65       | 1233648,72 | Portal         | 2141,59       | 35        |

### 3.4.4 Entrance Portal Slope Geotechnical Condition

The Entrance Portal slope is located on the eastern portion of the tunnel 09. The excavation of the portal slope begins, according to the borehole, within the contact zone between the colluvial (Qc) and acid eluvial deposits (Qvs) as well as within acid (TaVS) and basic (TasTV) rock masses. The materials which compose the Unit Qvs are particularly unstable as indicate two big landslides observed nearby the tunnel alignment which mainly involves them, as well as an interpreted third landslide which will be affected by the entrance portal excavation. In addition, at the toe of these all landslides, some points of water upwelling are observed, that may be made the entrance portal excavation operations and slope stability measurements more complex.

According to the investigation, a thick soil layer of the Unit Qc appears on the surface in the portal slope. Under this layer, a thinner one composed of eluvial deposits (Unit Qvs) was expected. Below this soil lays the rock unit TasTV. Thus, the material made the expected slope excavation sequence was:

**Unit Qc:** Colluvium. Undifferentiated materials with a brown non-consolidated soil matrix

**Unit Qvs:** Eluvial Deposits. Weathered volcanoclastic sediments

**Unit TasTV:** Trachyandesite and Basic Ignimbrite/Agglomerate.

### 3.4.5 Data Collection and Evaluation

The geological and geotechnical data used for this research is the site investigation conducted by the YAPI MERKEZI “design & built contractor” which encompassed borehole drilled, in-situ and laboratory test. The main geological and geotechnical parameters of the material found along the entrance portal slope of tunnel 09 is summarized in table 3.2. These parameters are used to assess and evaluate the stability of the slope according to limit equilibrium method and numerical method by using SLOPE/W and PLAXIS2D software respectively. Figure 3.7a, b, c, and d shows the idealized of four slope geometries which are analyzed in this thesis. The slopes are non-homogenous 81 m height with three layers of different properties.

On the other hand, there is no ground water table (GWT) found during the investigation, the slope is considered as dry slope and the analysis input parameters used for both LEM and numerical approach are shown in table 3.2.

**Table 3. 2:** Geotechnical characterization for the tunnel 09 entrance portal slope area

| Layer  | $\gamma$ [kN/m <sup>3</sup> ] | C [kPa] | $\Phi$ [°] | E[kPa] | $\nu$ | k [m/s]           |
|--------|-------------------------------|---------|------------|--------|-------|-------------------|
| Upper  | 20                            | 16      | 32         | 23000  | 0.3   | $2 \cdot 10^{-5}$ |
| Middle | 20                            | 23      | 30         | 31000  | 0.3   | $1 \cdot 10^{-7}$ |
| Lower  | 25                            | 110     | 50         | 227000 | 0.25  | $1 \cdot 10^{-6}$ |

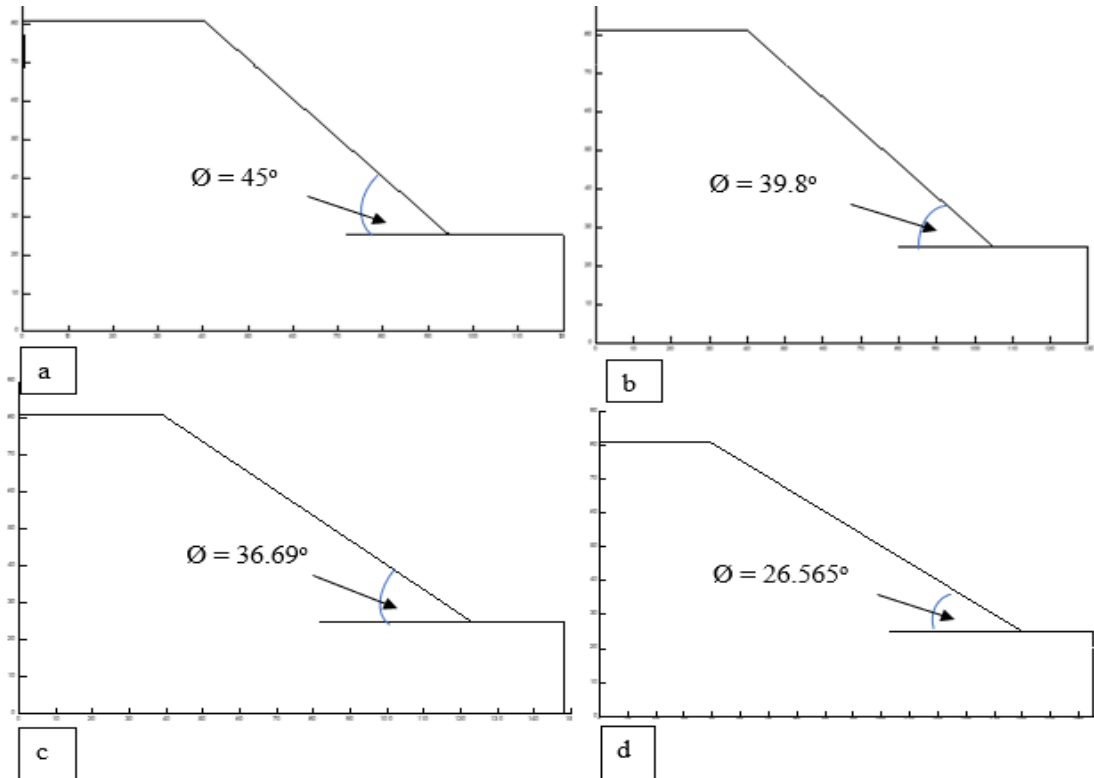


Figure 3. 7: four different cases of slope geometries that have been analyzed in this work. The dimensions and corresponding slope angles of above figure are fully discussed through chapter four and five

## CHAPTER FOUR: ANALYSIS RESULTS AND DISCUSSIONS

### 4.1 Validation of the software used these analyses

Three cases from the literature review have been analyzed to compare and confirm the validity of the software used in this work. Case one follows the analysis performed by GRIFFITHS and LANE [27], whereas case two adopted from B.H. MAULA and L.ZHANG [28]. And case three is captured from the verification manual of Slide 7.0 (Verification #1) [53]

#### 4.1.1 Case One Results from Analysis

The idealized homogenous slope geometry is shown in figure 5.20. According to the Griffiths and Lane, the adopted parameters are based on  $c'/\gamma H = 0.05$ , and the slope is  $26.565^\circ$ . The slope height is assumed to be 40 m as table 4.1 shows.

Table 4. 1: Slope material properties

| c (kPa) | $\phi$ ( $^\circ$ ) | $\gamma$ (kN/m <sup>2</sup> ) | H (m) |
|---------|---------------------|-------------------------------|-------|
| 40      | 20                  | 20                            | 40    |

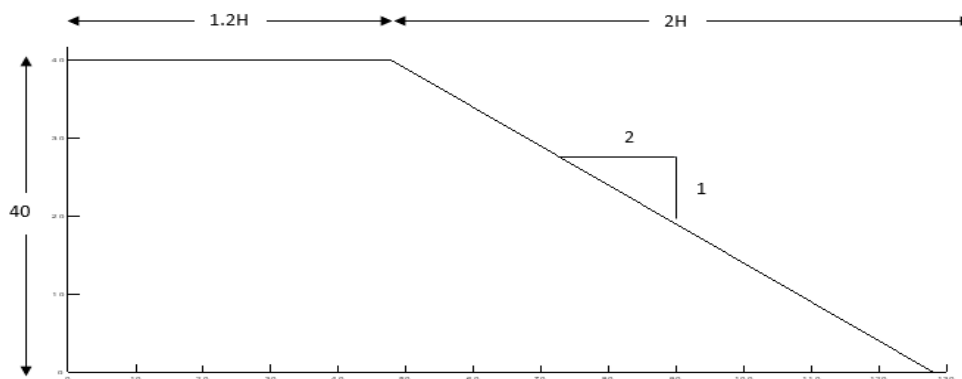


Figure 4. 1: Idealized slope geometry

For Slope/w, Morgenstern and Price Method with the help of half sine was used to solve the FoS and slip surfaces. For plaxis2D, Ph-c reduction procedure was adopted. Figure 4.2 and 4.3 show the computed FoS calculated from SLOPE/W and PLAXIS2D respectively.

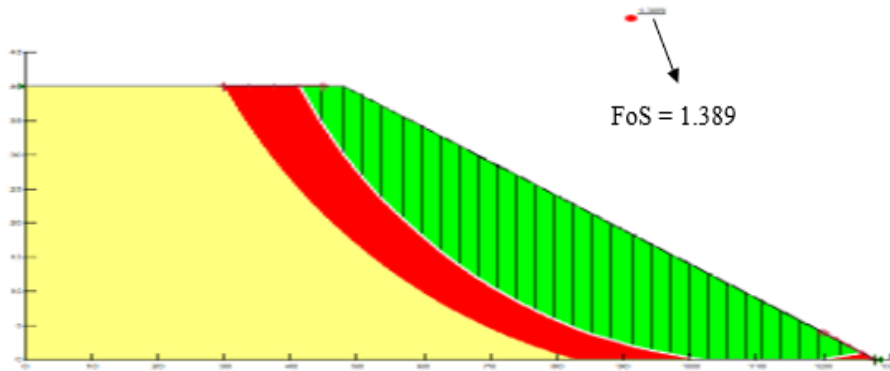


Figure 4. 2: Explains FoS computed from SLOPE/W

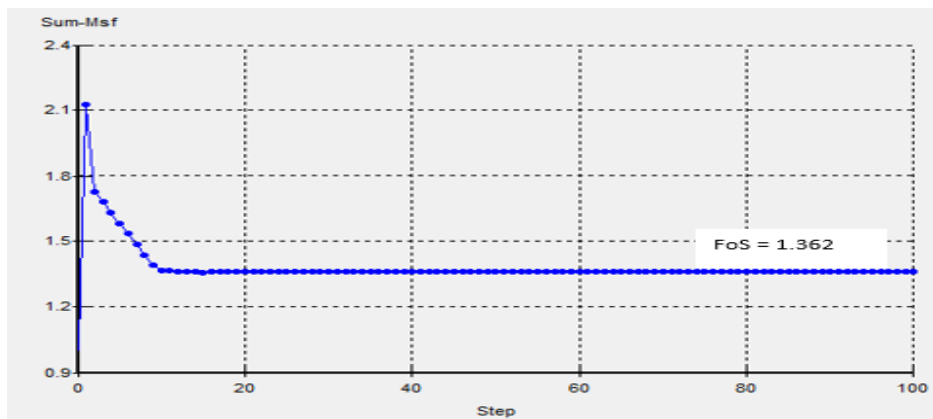


Figure 4. 3: Explains FoS computed from PLAXIS2D

#### 4.1.2 Case two Results from Analysis

The idealized homogenous slope geometry is shown in figure 4.4. According to the B.H. Maula and L. Zhang (2011), the adopted parameters of case three are listed in table 4.2

The slope angle is  $45^\circ$  and its height is assumed to be 10 m as table 4.2 shows.

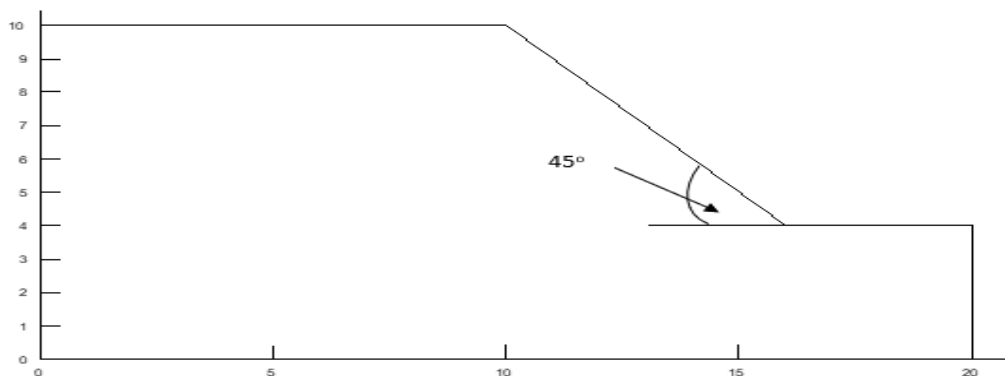


Figure 4. 4: idealized slope geometry

Table 4. 2: Slope material properties

| c (kPa) | $\phi$ (°) | $\gamma$ (kN/m <sup>2</sup> ) | H (m) |
|---------|------------|-------------------------------|-------|
| 5       | 45         | 20                            | 10    |

The procedure for using slope/w and plaxis2D is similar to previous sections.

Figure 4.5 and 4.6 illustrated the computed FoS from LE and FEM respectively

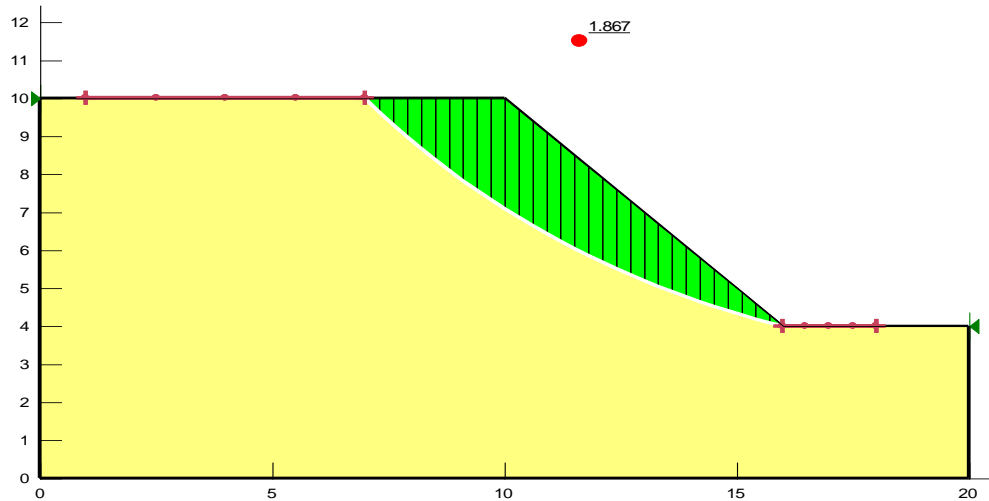


Figure 4. 5: Explains FoS computed from SLOPE/W

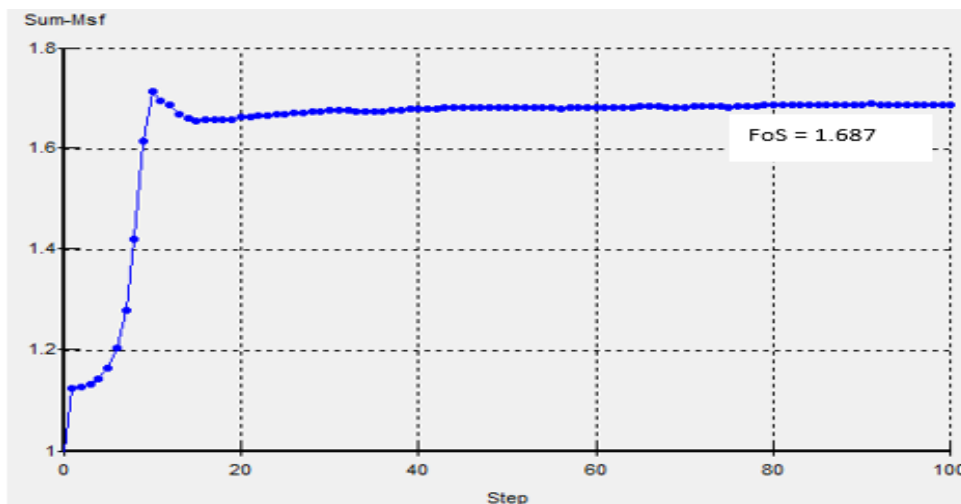


Figure 4. 6: Explains FoS computed from PLAXIS2D

### 4.1.3 Case three Result from Analysis

In 1988 a set of 5 basic slope stability problems, together with 5 variants, was distributed both in the Australian Geomechanics profession and overseas as part of a survey sponsored by ACADS [54].

This problem is captured from the verification manual of Slide 7.0 (Verification #1). This problem was distributed to the Australian Geomechanics and professions and overseas in 1988 to verify the efficiency and accuracy of the slide7.0. Figure 4.7 and table 4.3 show slope geometry model and the corresponding material properties respectively.

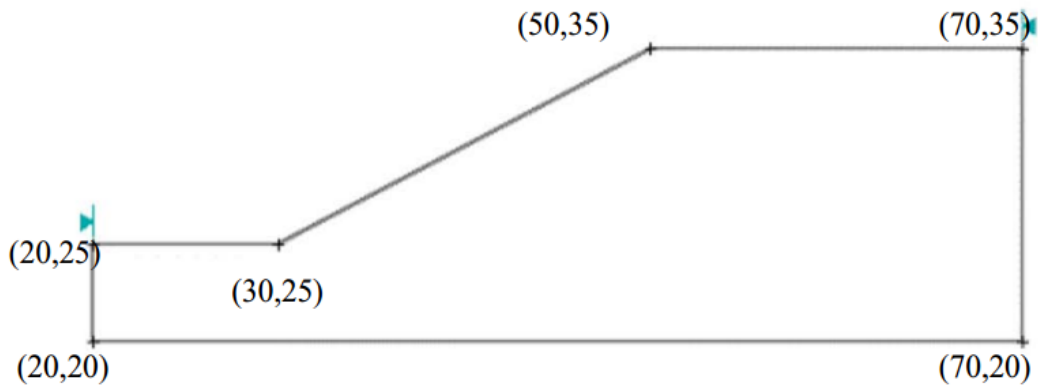


Figure 4. 7: Slope Model Geometry from Slide 7 [53]

Table 4. 3: Slope Dimensions and Material Properties for ACAD Problem

| C (kPa) | $\phi$ (°) | $\gamma$ (kN/m <sup>2</sup> ) | v   | E(kN/m <sup>2</sup> ) |
|---------|------------|-------------------------------|-----|-----------------------|
| 40      | 20         | 20                            | 0.3 | 10000                 |

Figure 4.8 and figure 4.9 are shown FoS computed from LEM and FEM respectively

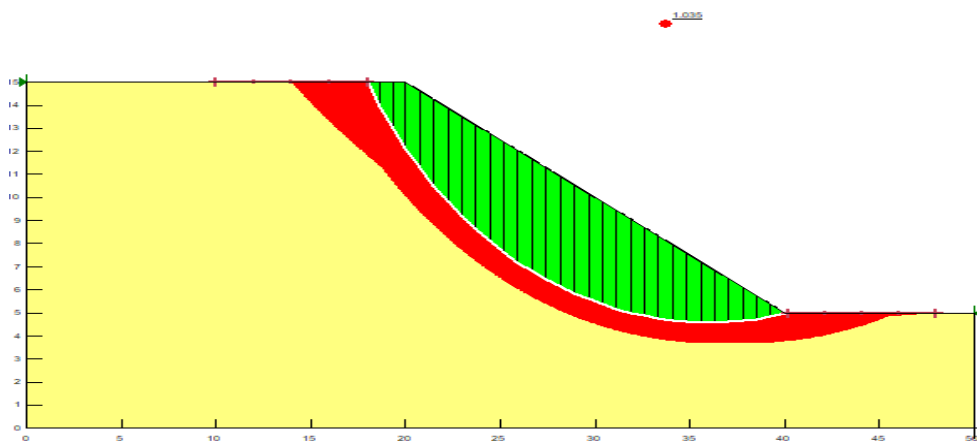


Figure 4. 8: FoS computed from SLOPE/W (FoS = 1.035)

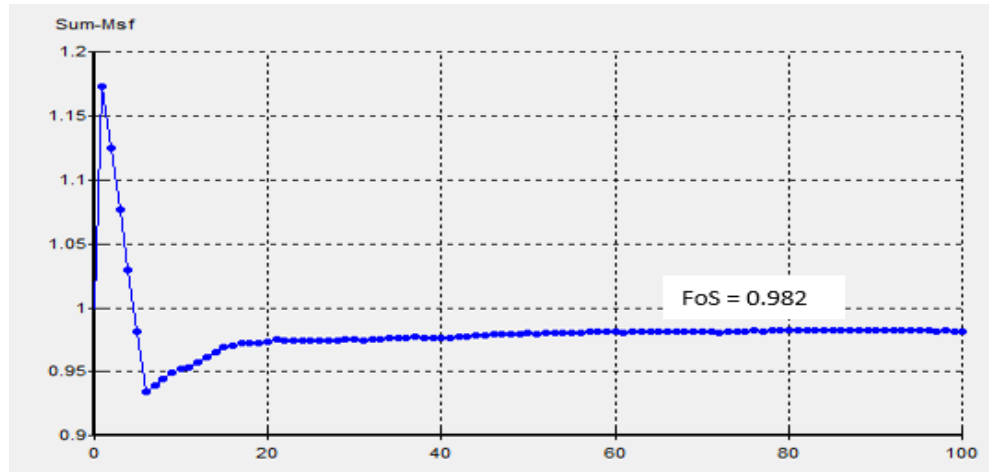


Figure 4. 9: FoS computed from PLAXIS2D

#### 4.1.4 Comparison of the Computed Results to the the original results

Table 4.4 summarizes the comparison of the result obtained the three cases from literature with the original results. This table indicates the reliability of the software used in these analyses.

Table 4. 4: Comparison of the computed result to the original result

| Slopes | Original results from Griffiths and Lane |          | Result obtained from Analysis |          | Percentage Error % |          |
|--------|--|----------|-------------------------------|----------|--------------------|----------|
|        | Slope/w                                  | Plaxis3D | Slope/w                       | Plaxis2D | Slope/w            | Plaxis2D |
| Case 1 |  |          |                               |          |                    |          |
| FoS    | 1.380                                    | 1.4      | 1.389                         | 1.362    | -0.65              | 2.7      |
| Case 2 | Original result from Maula and L. Zhang  |          | Result obtained from Analysis |          | Percentage Error % |          |
|        | Slope/w                                  | Plaxis2D | Slope/w                       | Plaxis3D | Slope/w            | Plaxis2D |
| FoS    | 1.86                                     | 1.68     | 1.867                         | 1.685    | -0.38              | -0.3     |
| Case 3 | Original result from varication manual   |          | Result obtained from Analysis |          | Percentage Error % |          |
|        | Slide (Rocscience)                       |          | Slope/w                       | Plaxis2D | Slope/w            | Plaxis2D |
| FoS    | 0.986                                    |          | 1.035                         | 0.982    | 4.96               | 0.4      |

## 4.2 Slope Stability Analyses

After confirming the validity and reliability of both SLOPE/W and PLAXIS2D software, slope stability analyses were conducted as follows.

### 4.2.1 Case one: Non-homogenous 45° unreinforced dry slope

SLOPE/W software has been used in this work to evaluate slope stability around a railway tunnel portal. The Morgenstern and Price Method (M-PM) was selected to compute FoS of the slopes. Mohr-coulomb model within a half-sine function without considering of tension was selected to compute the interslice forces. The geometry and material properties of the slope model are shown in figure 3.7 and table 3.2 respectively. The minimum value of factor of safety calculated from SLOPE/W and critical slip surface are shown in figure 4.10.

Based on Geo-Slope 2012, the factor of safety was determined according to Bishop's simplified method, (BSM), Ordinary Method (OM), Janbu's simplified Method (JSM) and Janbu's generalized method. The factor of safety computed according to JSM is the least FoS computed from LEM as figure 4.10 illustrated. Figure 1 in appendix B shows all computed slice failure surfaces, whereas table 4.5 listed the five most critical surface of failure are selected from hundreds of analyzed surfaces. The red line in figure 4.10 indicated number of slips which have a similar or nearly similar factor of safety

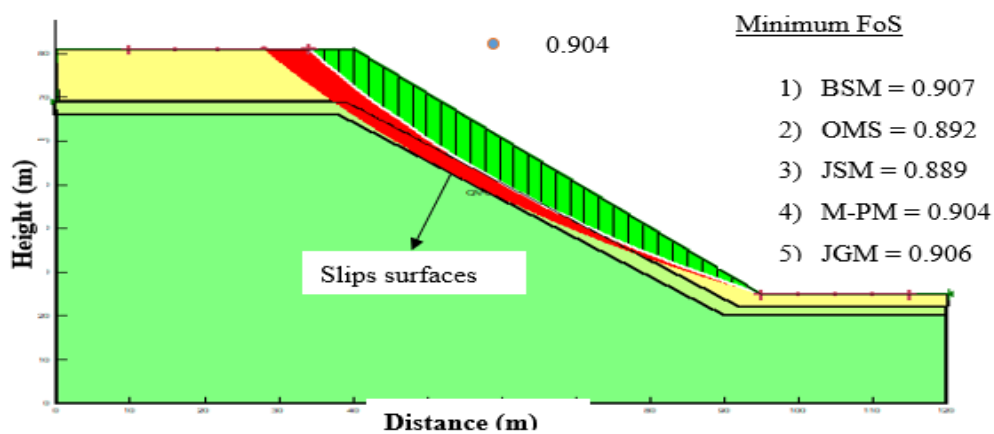


Figure 4. 10: Explains slope/w FoS output

Table 4. 5: 5 Most critical failure surfaces computed from slope/w

| Failure Surface No | Factor of safety |
|--------------------|------------------|
| 1                  | 0.904            |
| 2                  | 0.909            |
| 3                  | 1.007            |
| 4                  | 1.010            |
| 5                  | 1.109            |

## 4.2.2 PLAXIS2D Analysis Results

### 4.2.2.1 Case one: Non-homogenous 45° unreinforced dry slope

PLAXIS2D is a FEM based software which has been used to solve geotechnical problems include slope stability evaluation. This code is an automated mesh generation tool that makes the cretration of soil/rock model easy. In this work, slope stability has been analyzed as a plane strain model with 15 nodal triangular, where displacement and strains in the z-direction are assumed to be zero. The idealized slope geometry is the same as the SLOPE/W model in figure 3.7a show. The Mohr-coulomb material model option was selected with drained material type. Factor of safety was computed by Phi-C reduction /Strength Reduction Method (SRM). Figure 4.11 shows the deformed model whereas figure 5 in appendix B shows total extreme displacement. The computed FoS from PLAXIS2D is also illustrated in figure 4.12.

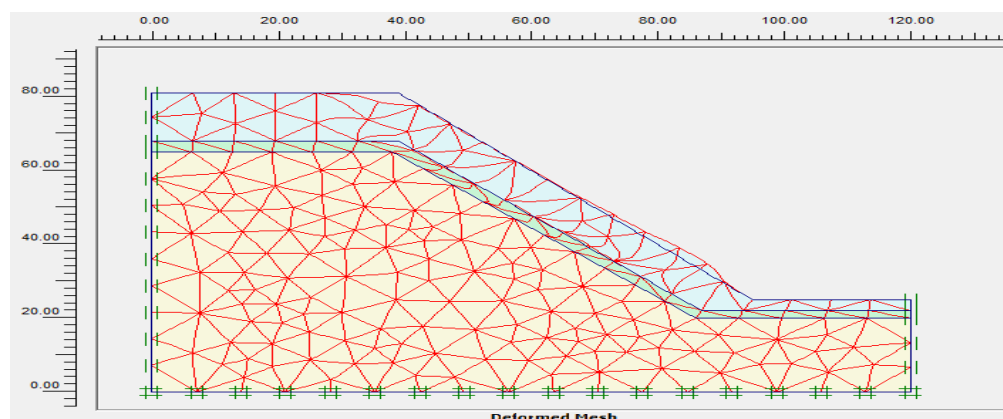


Figure 4. 11: Deformed mesh from Plaxis2D

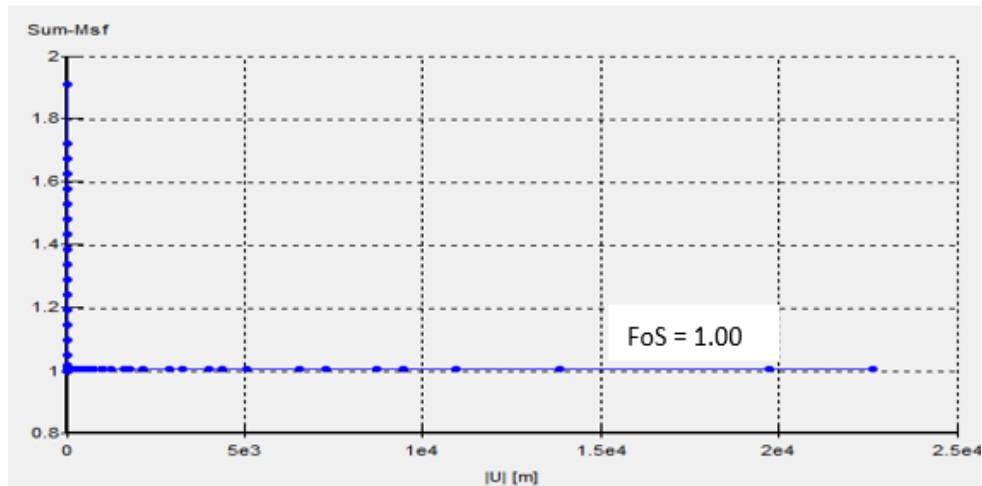


Figure 4. 12:  $\sum Msf$  vs the U

### 4.3 Discussion of Results from Non-homogeneous 45° Slope Analysis

The slope analyzed above sections represents the real slope geometry observed from T09 portal which is part of the railway lane project located in Kombolcha. This slope is a dry non-homogenous with three layers of different properties. This case is analyzed as the slope was before it failed. LEM and FEM have been used to evaluate slope stability by computing factor of safety and displacement. The computed factor of safety of the slope from LEM and FEM analyses are presented in figure 4.10 and 4.12 respectively. As we observed these figures, this slope is unstable and it requires an application of slope stabilization techniques.

Generally, slope is considered stable if the factor of safety is greater than 1.0. But from these analyses, the computed FoS obtained from LE methods are less than 1, whereas, FoS calculated from FEM analysis is 1 as the above figures illustrate. Therefore, according to the results obtained from analyses, slope is considered unstable. In other words, these results confirm the exact ground condition. because this slope has made cracks and within a short time, cracks are developed into full slides. Therefore, there is a need for the application of slope stabilization method to ensure the safety of the slope and the tunnel portal.

## **CHAPTER FIVE: SLOPE STABILIZATION**

This work presented two methods of slope stabilization techniques; slope geometry (slope angle reduction) and slope reinforcement (nailing). These two methods have been used over the years and they are good for improving slope stability within economic satisfactory. First, slope was stabilized by reducing slope angle constantly until the computed FoS surpassed both the FoS against slope stability and overall slope stability, 1.4 and 1.5 respectively regarding the FHWA 2003. Secondly, two cases of a reinforced slope using nails were analyzed. For case one, the original slope, as observed from the site, was reinforced. The second case, the slope was reinforced with the help of slope angle reduction. The analyzed has been conducted by using both LEM and numerical approach.

### **5.1 Slope stabilization by Slope Geometry – Slope angle reduction**

Due to the slope steepness, the AKH tunnel portal slope instability may result from slope angle which is 1V:1H, thus slope angle has been reduced successively until stable slope has achieved. First slope angle was reduced from 1:1 to 1:1.2, and then it further reduced 1:1.5 and 1:2, case one, case two and three respectively. The slope stability analysis procedure for the stabilized slopes are the same as we discussed in chapter four. Traditional LEM and FEM have also been used for the assessment of the above mentioned three cases of slopes. The general input parameter, as used before in table 3.2 and the slope geometry is shown in figure 3.7 b, 3.7c, and 3.7d were applied in these analyses.

#### **5.1.2 Case one (1V:1.2H): Slope Stability Analysis by LEM, Slope/w**

The Morgenstern and Price's method was used to solve the factor of safety as we did before, case one in chapter four. The Mohr-coulomb model together with the half-sine function was selected. First, Morgenstern-Price Method was selected and then other methods of slices such as BSM, OMS, JSM, JGM, and M-PM were performed as figure 5.1 shows. All failure surface slices from SLOPE/W are illustrated figure 2 in appendix B. Finally, the five most critical failure surfaces which have the least FoS were listed table 5.1

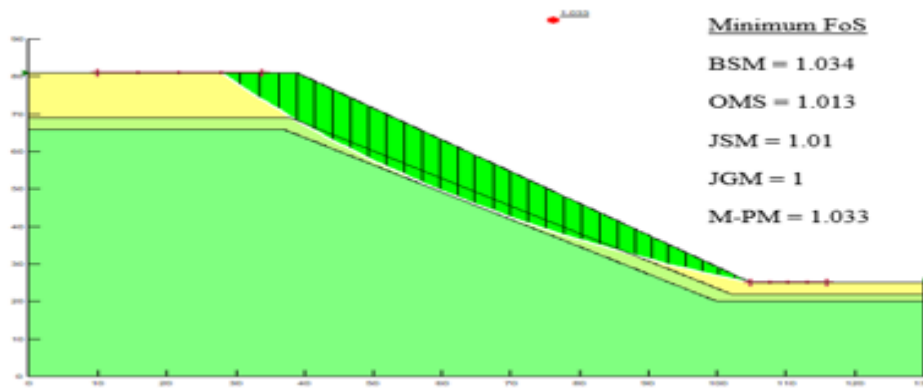


Figure 5. 1: Computed FoS computed from slope/w

Table 5. 1: Five most critical failure surfaces computed from slope/w

| Failure Surface No | Factor of safety |
|--------------------|------------------|
| 1                  | 1.033            |
| 2                  | 1.038            |
| 3                  | 1.163            |
| 4                  | 1.178            |
| 5                  | 1.181            |

### 5.1.3 Case two (1V:1.5H): Slope Stability Analysis by LEM, Slope/w

The calculated FoS from different methods of LE were illustrated in figure 5.2, where figure 3 appendix B shows all failure surfaces resulted from SLOPE/W analysis. Table 5.2 listed the five most critical slices which have the least computed FoS from SLOPE/W.

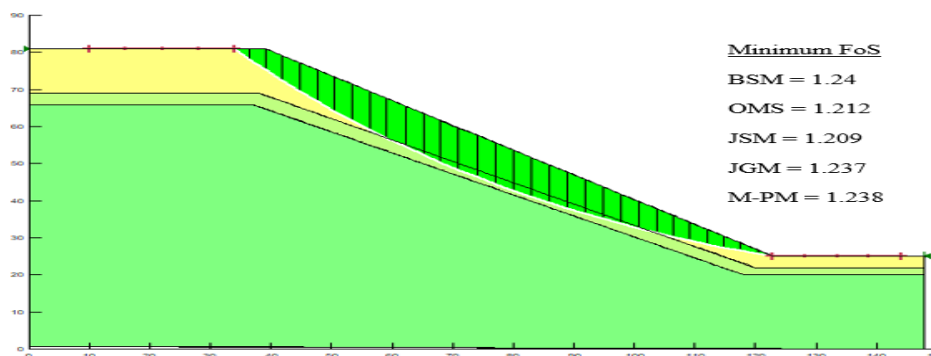


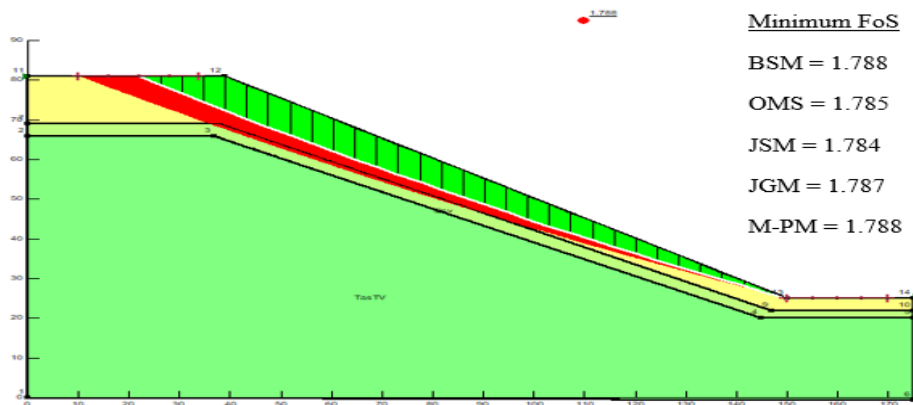
Figure 5. 2: Computed FoS from slope/w

Table 5. 2: Five most critical computed from slope/w

| Failure Surface No | Factor of safety |
|--------------------|------------------|
| 1                  | 1.238            |
| 2                  | 1.413            |
| 3                  | 1.423            |
| 4                  | 1.442            |
| 5                  | 1.451            |

### 5.1.4 Case three (1V:2H): Slope Stability Analysis by LEM, Slope/w

The computed FoS and five most critical failure surfaces which have the least or minimum FoS resulted from Tradition LEM by using the SLOPE/W were illustrated figure 5.3, and table 5.3 respectively. All failure surfaces on the slices are illustrated in figure 4 appendix B



**Figure 5. 3:** Computed FoS from slope/w

Table 5. 3: Five most critical failure surfaces from slope/w

| Failure Surface No | Factor of safety |
|--------------------|------------------|
| 1                  | 1.787            |
| 2                  | 1.792            |
| 3                  | 1.804            |
| 4                  | 1.832            |

## 5.2 FEM Analysis Result by using PLAXIS2D

The procedure of the plaxis2D analysis for these cases are similar as we applied in section 4.2. Thus, Mohr-coulomb material model with drained type was adopted. Fine mesh option was selected. The Strength Reduction Method was also selected to compute FoS and total displacement.

### 5.2.1 Case one (1V:1.2H): Result from FEM Analysis, by PLAXIS2D

The analysis results of case one from PLAXIS2D software were presented in this section. Figure 5.4 shows the deformed mesh of the model while 5.5 and figure 6 in appendix B shown FoS and total displacement respectively. The output results indicate that the slope stability has improved and the factor of safety increased by 12.9% compared to the 1V:1V slope geometry.

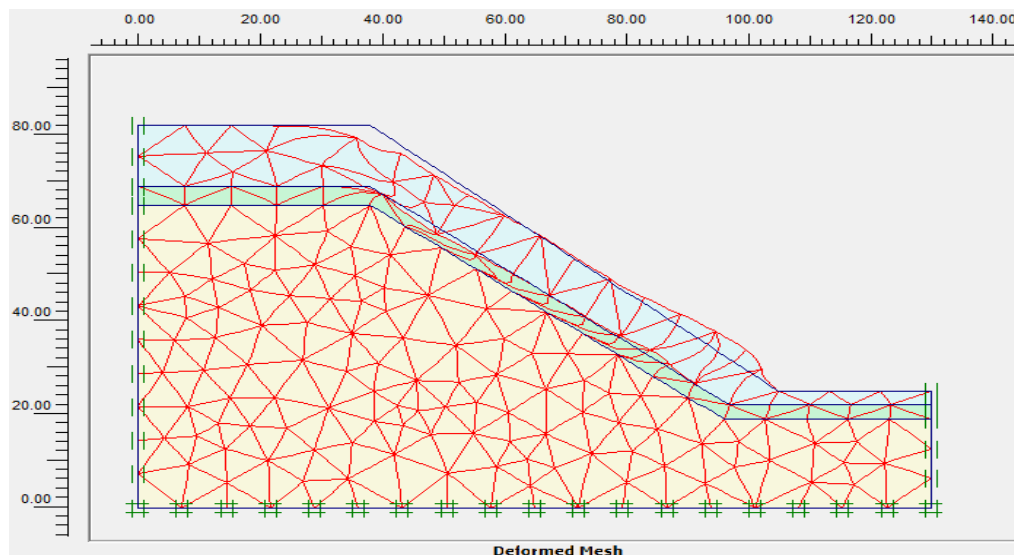


Figure 5. 4: Deformed mesh of the model from plaxis2D

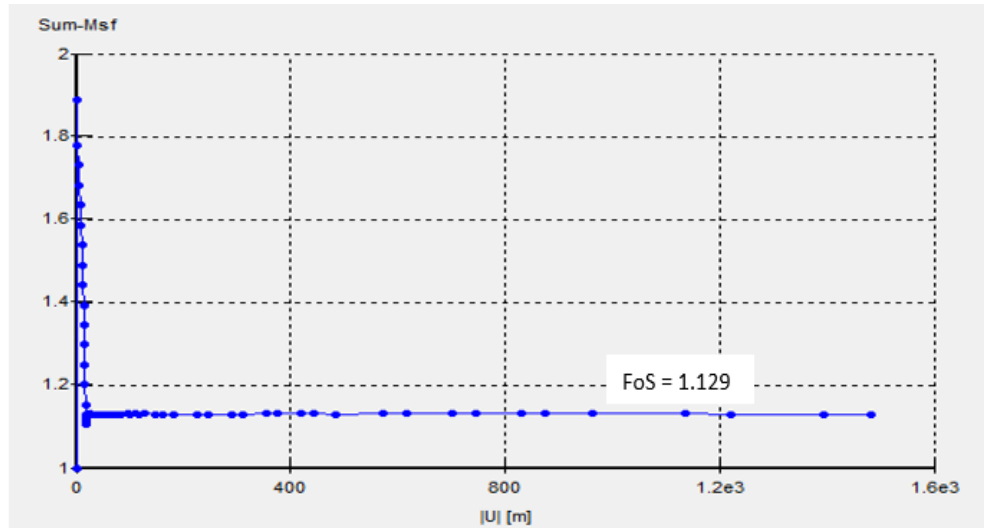


Figure 5. 5: Sum-Msf vs U

### 5.2.2 Case Two (1V:1.5H): Result from FEM Analysis, by PLAXIS2D

Case two outputs from the PLAXIS2D analysis were illustrated in figures bellow. The total deformed mesh is illustrated in figure 5.6 and the total displacement of the model is shown in figure 7 in appendix B and FoS computed from plaxis2D is illustrated in figure 5.7.

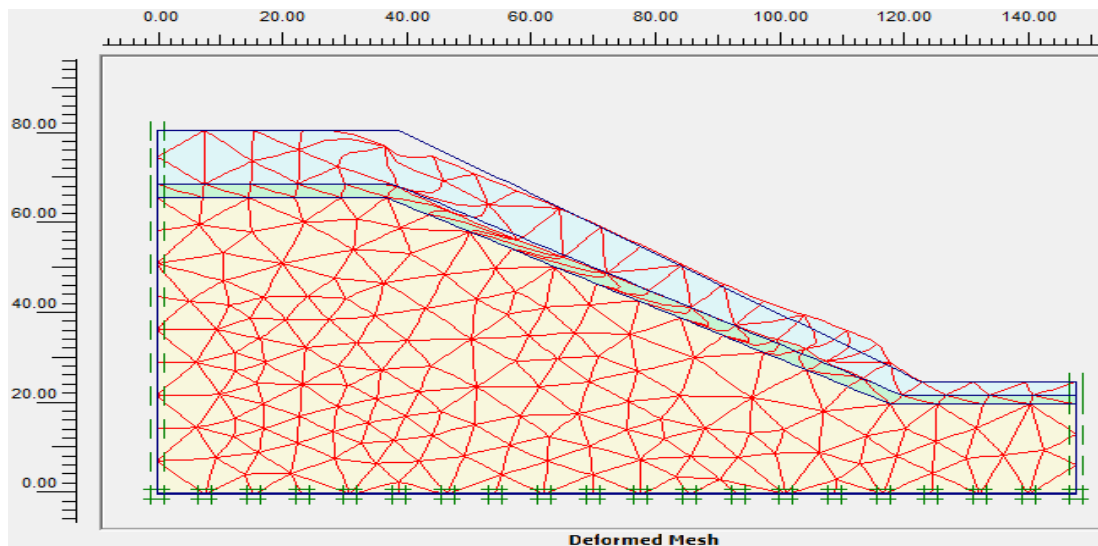


Figure 5. 6: Deformed mesh of the model from plaxis2D

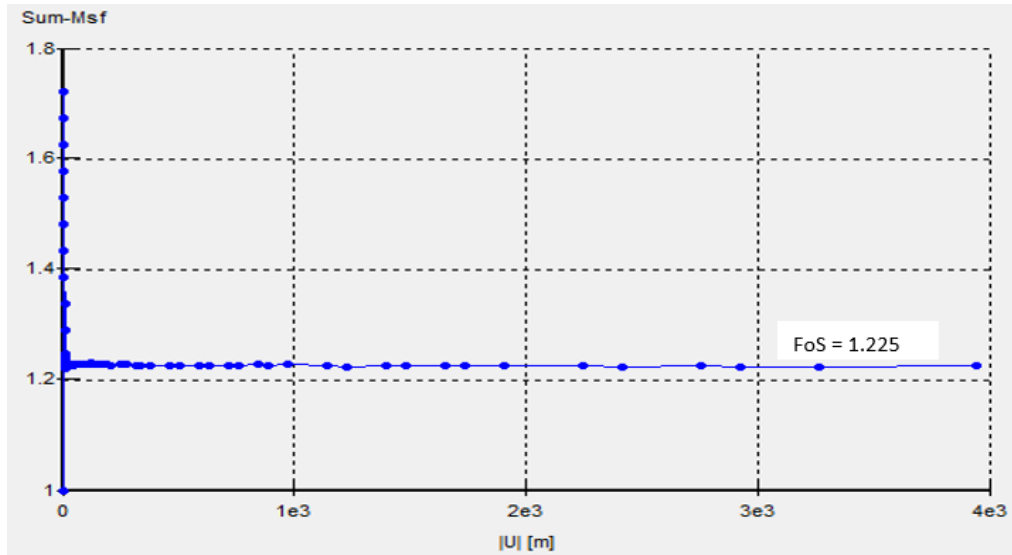


Figure 5. 7: Sum-Msf vs U

### 5.2.3 Case three: PLAXIS2D Result (1V:2H)

As well as case one and two the computed output of the case three is presented in below figures. The deformed mesh of the model is shown in figure 5.8, while figure 5.9 illustrate FoS plot from the PLAXIS analysis respectively.

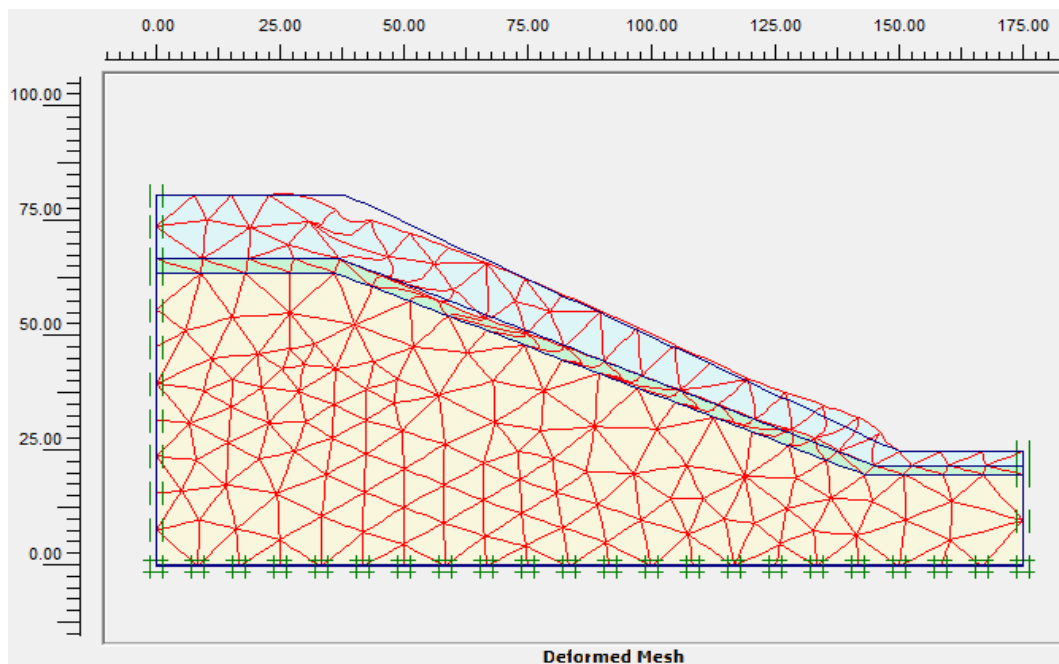


Figure 5. 8: Deformed mesh of the model from plaxis2D

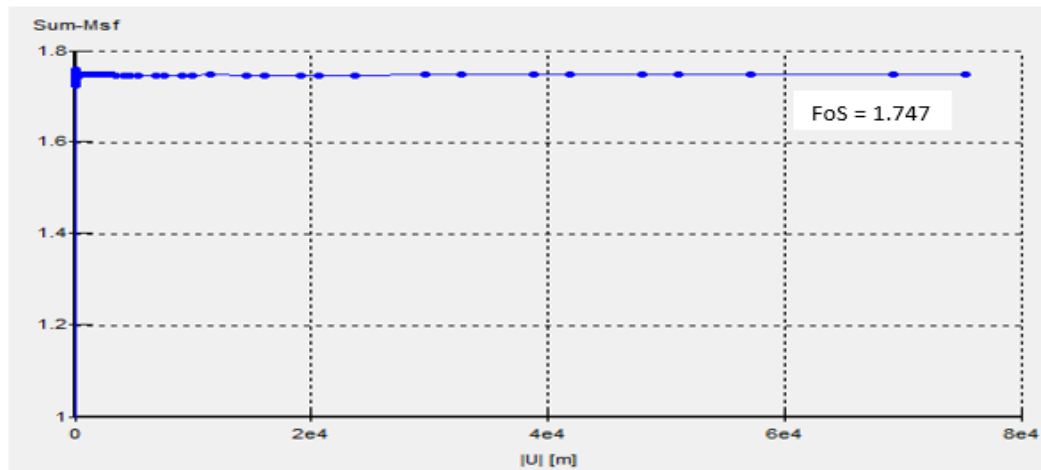


Figure 5. 9: Sum-Msf vs U

### 5.3 Summary from the analyses

From the first analysis in chapter four, we have seen that the analyzed slope is unstable as discussed in section 4.4. Thus, slope stabilization techniques were implemented to improve the stability of the slope structure and to eliminate any possibility of human impairment or economic loss. First, slope was stabilized by reducing slope angle successively. In case one, slope angel has reduced from 1:1 to 1:1.2, and then slope angle has further reduced 1:1.5 and 1:2 respectively. Factor of safety and displacement of these slopes have computed by using LEM and FEM.

#### 5.3.1 Results obtained from LEM, SLOPE/W

From the above analyzed slopes, FoS is computed according to LEM and FEM. For LEM, the result obtained from SLOPE/W shows that the slope angle has a great impact on slope stability. For example, in case one the computed FoS has increased from 0.904 “original slope as discussed chapter four” to 1.033 which realized an increase of 14.27% while the slope angle has reduced from 1:1 to 1:1.2. From case two and case three slope stability had significantly improved and the FoS computed from case three has surpassed both FoS against slope stability and overall stability which are 1.4 and 1.5 respectively regarding the FHWA 2003. Moreover, table 5.4 summarizes the computed FoS from the above analysis using different options of LEM

Table 5. 4: Summary of computed FoS from Slope/w with different slope angle

| Slope angle | Factor of Safety from LEM |       |       |       |       |
|-------------|---------------------------|-------|-------|-------|-------|
|             | BSM                       | OMS   | JSM   | JGM   | M-PM  |
| 1V:1H       | 0.907                     | 0.892 | 0.889 | 0.906 | 0.904 |
| 1V: 1.2H    | 1.034                     | 1.013 | 1.01  | 1.00  | 1.033 |
| 1V:1.5H     | 1.24                      | 1.212 | 1.209 | 1.237 | 1.238 |
| 1V:2H       | 1.788                     | 1.785 | 1.784 | 1.787 | 1.788 |

### 5.3.2 Results Obtained from FEM, PLAXIS2D

The results obtained from FEM by using PLAXIS2D are presented in this section. Factor of safety and displacement of the above mentioned three cases with varied slope angle are computed. The deformed mesh and the factor of safety from the analyzed slopes are presented in figure 5.1 to 5.9 respectively. In addition, all output results from FEM are summarized in table 5.5. Likewise, LEM results, the output results from the FEM shows reducing slope angle has a significant effect on slope stability. From table 5.5, the computed FoS from case three was increased up to 1.747 which means both LEM and FEM confirmed that the slope stability has been improved by reducing slope angle from steep to gentle.

Table 5. 5: Summary of computed FoS from Plaxis2D

| Parameter                  | Origin<br>(1V:1H) | Case 1<br>(1:1.2) | Case 2<br>(1:1.5) | Case 3 (1:2) |
|----------------------------|-------------------|-------------------|-------------------|--------------|
| FoS                        | 1                 | 1.129             | 1.225             | 1.747        |
| Total displacement<br>(mm) | 196               | 111.4             | 53.3              | 32           |

### 5.4 Slope Stabilization by Reinforcement (nailing)

Nailing is one of the major techniques that has been used to improve the stability of natural and men made slopes. Slope reinforced by nailing can be more advantageous for increasing

slope stability within economic satisfactory. In this study, the reinforced slope was analyzed by using LEM and numerical method.

#### 5.4.1 Evaluation of the Reinforced slope by LEM, SLOPE/W

In this research, nails are used as a reinforced material of the dry slope. By using SLOPE/W (GEO-SLOPE/W, 2012), the effect of nailing on the slope stability was studied. First, the slope was reinforced and analyzed as to its origin geometry, 45° degree. Secondly, slope geometry was reduced 1:2 together with slope reinforcement.

The procedure used for LEM analysis using SLOPE/W is similar to the previous analysis, which means Morgenstern and Price Method (M-PM) with the help of the half-sine and Mohr-coulomb expression was adopted. The installation of nails was performed within three different nail angles to determine optimum nail inclination. These nail angles used for this analysis are 0°, 15° and 30° respectively. The idealized geometry of the reinforced slopes is illustrated in figure 3.6 a and d respectively as discussed in earlier. The general slope input parameters are not changed as in our previous analysis, and the properties of the nail are listed in table 5.6. Whereas figure 5.16 illustrates numerical model of reinforced slope by using SLOPE/W.

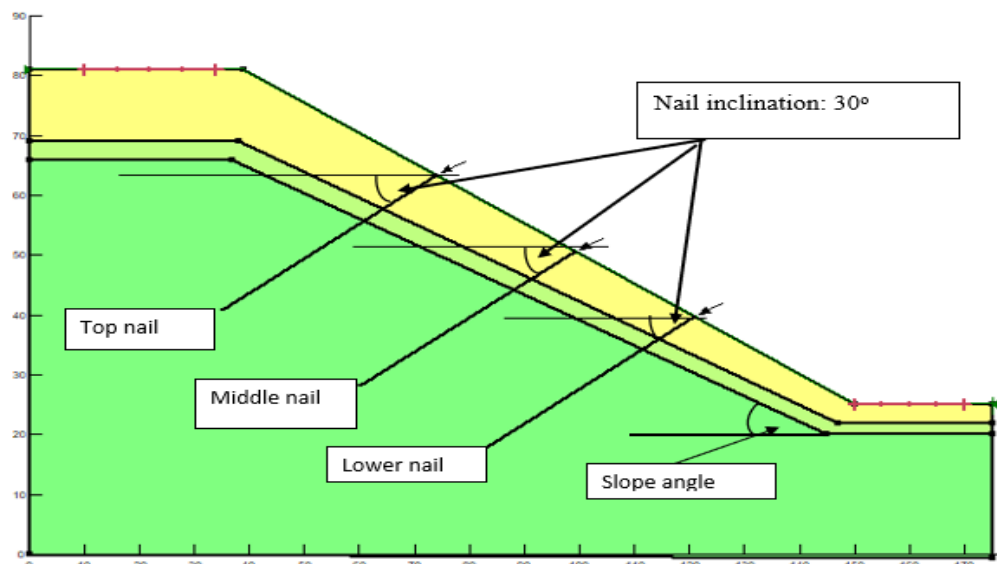


Figure 5. 10: Numerical model of reinforced slope by using slope/w

Table 5. 6: Nail properties [23]

| The | Property                    | Value   |
|-----|-----------------------------|---------|
|     | Nail length                 | 18 m    |
|     | Bond Diameter               | 0.318 m |
|     | Bond Safety Factor          | 1.5     |
|     | Bond Skin Friction (F/Area) | 100 KPa |
|     | Bar Capacity                | 300 KN  |
|     | Bar Safety Factor           | 1.5     |
|     | Nail spacing                | 1.5 m   |

results from LEM by using SLOPE/W for the 45° reinforced slope within different nail inclination is shown in figure 5.17. From this analysis, FoS is maximum when nail inclination is zero and a further increase of nail inclination reduced the computed FoS as table 5.7 illustrated.

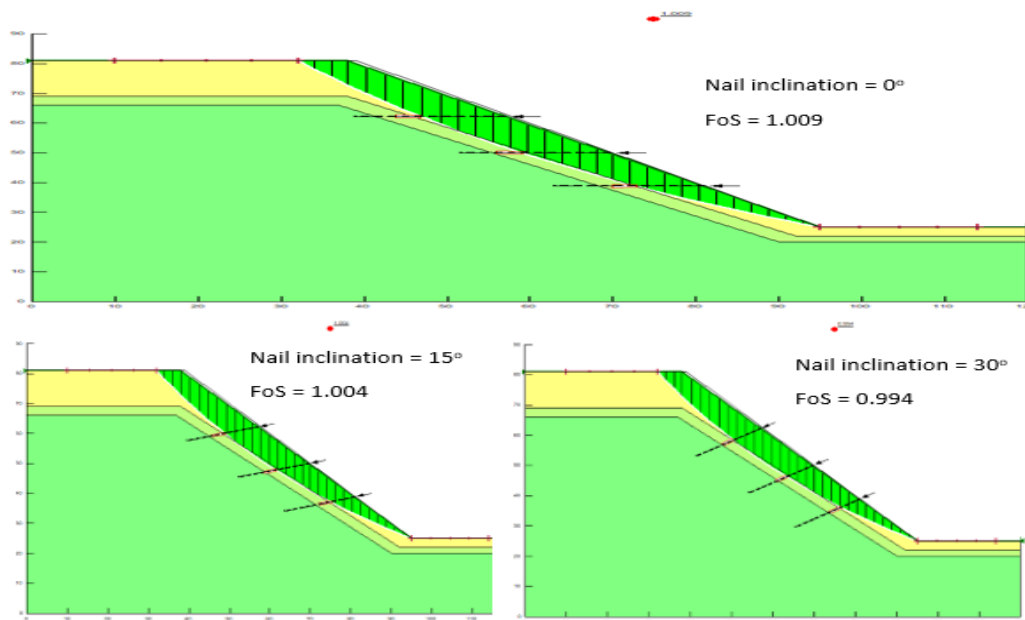


Figure 5. 11: Computed FoS and slip surface of 45o reinforced slope by slope/w

For case two, the slope was reinforced together with slope angle reduction. The slope angle was reduced to 1:2, and as well as case one nails used as reinforcement material were applied in three different inclination angles.

Table 5. 7: Nail inclination angle vs FoS case one reinforced slope (Slope angle 45°)

| Case            | Nail angle (Degree) | FoS   |
|-----------------|---------------------|-------|
| Without nailing | —                   | 0.904 |
| I               | 0°                  | 1.009 |
| II              | 15°                 | 1.004 |
| III             | 30°                 | 0.994 |

Limit equilibrium method analysis results determined from case two reinforced 26.565° slope shows that the factor of safety is increased significantly and slope stability is improved as figure 5.12 presented. However, for case two reinforced slope the computed FoS is reached it is maximum when nail inclination is 30° and further increase or decrease of nail inclination angle reduces the computed FoS, see table 5.8.

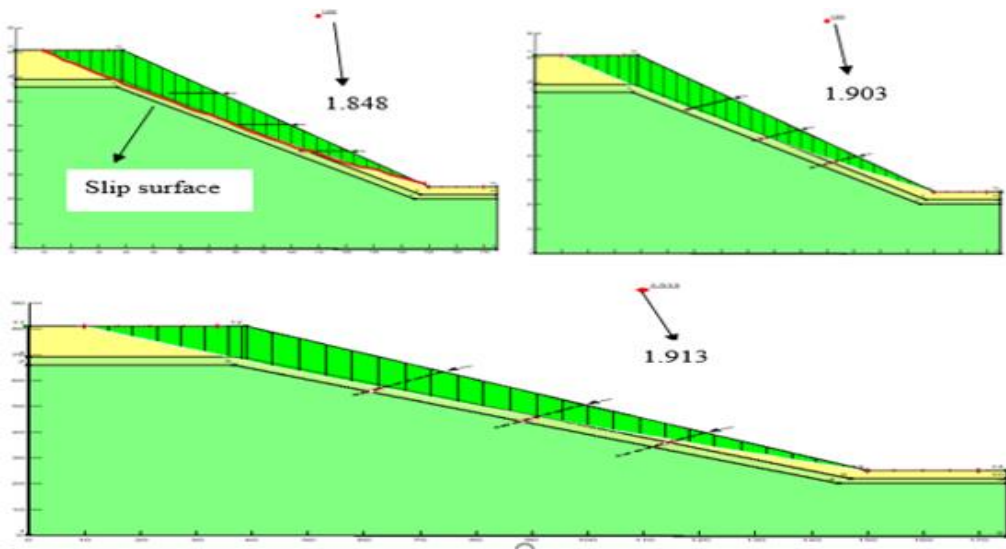


Figure 5. 12: Computed FoS and slip surface of 45° reinforced slope by SLOPE/W

Table 5. 8: Nail inclination angle vs FoS of case one reinforced slope (Slope angle 26.565°)

| Case            | Nail angle (Degree) | FoS   |
|-----------------|---------------------|-------|
| Without nailing | —                   | 1.788 |
| I               | 0°                  | 1.848 |
| II              | 15°                 | 1.903 |
| III             | 30°                 | 1.913 |

#### 5.4.2 Evaluation of the Reinforced Slope Analysis by FEM, Plaxis2D

The reinforced slopes have also been analyzed by using FEM based PLAXIS2D software. Similar to the previous analysis, plain strain with 15 nodal triangulations were selected to analyze the reinforced slopes. As well as section 4.3 and 5.2, Mohr-coulomb model, perfectly plastic constitutive was selected from material option with drained material type.

The idealized slope geometries of the model and the material properties are the same as the reinforced slope/w model. The standard fixities were selected to simulate the actual boundary condition of the model. FEM based Plaxis2D package also provide opportunity to simulate the model of finned, medium and coarse mesh generation. Finned and medium mesh generation yielded more accurate results than very coarse mesh generation, thus this presented study, medium mesh generation was selected from mesh generation setup.

The simulated nail input parameters used for this reinforced slope is summarized by table 5.9. The axial and bending stiffness are fundamental input value for nail simulation of plane elements. The axial stiffness is calculated for correct simulation of the nails. Babu et al (50) formulation was adopted to compute the equivalent modulus of elasticity of the modelled nails as per equ-5.1. The axial stiffness also is given by Equ-5.2. The equivalent bending stiffness and the equivalent plate diameter of the nail are calculated from equ-5.3 and equ-5.4. However, the equivalent plate diameter is directly calculated by plaxis.

$$E_{eq} = E_n \left( \frac{A_n}{A} \right) + E_g \left( \frac{A_g}{A} \right) \quad \text{Eq-5.1}$$

$$\text{Axial stiffness } EA \left[ \frac{\text{KN}}{\text{m}} \right] = \frac{E_{eq}}{S_h} \left( \frac{\pi D^2 D_H}{4} \right) \quad \text{Eq-5.2}$$

$$\text{Bending stiffness } EA \left[ \frac{\text{kNm}^2}{\text{m}} \right] = \frac{E_{eq}}{S_h} \left( \frac{\pi D^2 D_H}{64} \right) \quad \text{Eq-5.3}$$

$$d_{eq} = \sqrt{12 \left( \frac{EI}{EA} \right)} \quad \text{Eq -5.4}$$

These four equations are used to calculate nail input parameters for these reinforced slopes. The Phi-C reduction method (SRM) calculation type was selected and the incremental multiplier value ( $\sum M_{sf}$ ) is used as a factor of safety of the reinforced slopes. Figure 5.19 illustrates numerical modelling of the reinforced slope by plaxis2D.

Table 5. 9: Nail properties [55]

| Parameter              | Value   | Unit                |
|------------------------|---------|---------------------|
| Nail element           | Elastic | —                   |
| Nail type              | Plate   | —                   |
| Axial Stiffness (EA)   | 2.128E6 | kN/m                |
| Flexural rigidity (EI) | 1.330E5 | kNm <sup>2</sup> /m |
| Diameter               | 30      | Mm                  |

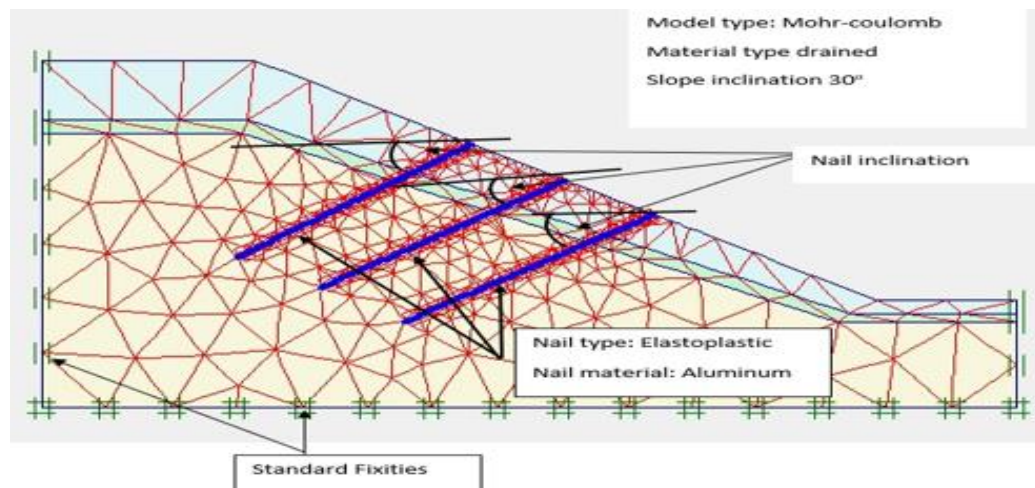


Figure 5. 13: Numerical model for reinforced slope by using plaxis2D

### 5.4.3 PLAXIS2D Results from Reinforced Slope Analysis

For both cases of reinforced slopes, the computed FoS from plaxis2D reached its maximum at nail inclination of 30 degrees, and thus further increase or decrease of nail angle result to decrease of computed FoS. Table 5.10 summarizes the calculated FoS of both reinforced slopes.

Table 5. 10: FoS computed from reinforced slopes

| Slope angle with horizontal (°) | Nail inclination with horizontal |       |       |       |
|---------------------------------|----------------------------------|-------|-------|-------|
|                                 | FEM                              | 0°    | 15°   | 30°   |
| FoS,                            |                                  | 1.022 | 1.042 | 1.205 |

|         |           |       |       |       |
|---------|-----------|-------|-------|-------|
| 45°     | Disp (mm) | 148.4 | 48.6  | 36.8  |
| 26.565° | FoS       | 1.883 | 1.818 | 1.935 |
|         | Disp (mm) | 52.58 | 57.2  | 58.4  |

## 5.6 Discussion of the Result from the Analyses

### 5.6.1 Slope Stability Investigation and Remedial method by LEM and FEM

Stability of slope in a railway tunnel portal has been assessed in this study. First, traditional limit equilibrium method has been used to analyze both reinforced and unreinforced slopes with different slope geometry. This method is considered as a simple and effective analysis, and it has been played a key role in the evaluation of slope stability over decades. Traditional LEM can easily compute FoS and CSS of natural and men made slopes within minimum input parameters.

However, according to John Krahn (2003), LEM is missing some fundamental physics of stress-strain relationships, and therefore this method couldn't perform a realistic stress and strain distribution [22]. Nevertheless, LEM remains more usable due to its simplicity and reliable output results, and currently many professional engineers and researcher's around the world are practicing this method.

From the late 20<sup>th</sup> century, FEM (one of numerical method) has been improved and became one of the best alternatives of traditional LEM to evaluate slope stability and other geotechnical and structural fields. FEM is an advanced complex procedure and requires more knowledge, input parameters, and time. The results obtained from FEM are more accurate and reliable compared to the LEM. Finite element method can deal a difficult slope geometry with static, dynamic and pore water pressure. This method also has a capacity to compute stress-strain relationship and displacement within realistic stress distribution, which LEM is missed.

### 5.6.2 Comparison of the results from LEM and FEM

One of the biggest differences between limit equilibrium method and finite element method is that of their basic fundamental principles. LEM is based on limit equilibrium

formulation which is the principle of static forces and moments as discussed before. Whereas, FEM is based on principles of stress-strain relationship. On the other hand, the computed FoS from these two methods revealed on their difference. For LEM, FoS has been computed on the basis of the method of slices, for example, M-PM, OMS, BSM which are all relied on statics moment and interslice forces of equilibrium formulation, while FEM based Plaxis2D is used stress-strain relationship procedure. In plaxis2D, phi-c reduction method was selected to calculate FoS and total stress and strain displacements. Another big difference between LEM and FEM is the searching of critical slip surfaces. For FEM, the critical slip surface is automatically generated by plaxis2D, while LEM does not, the user is required to define slip surfaces.

In this research, nine cases of reinforced and unreinforced slope have been analyzed by using both LEM and FEM. First, FoS and displacement of four cases of an unreinforced slope with varied slope angle were computed. Factor of safety was determined from both LEM and FEM whereas displacements were computed from numerical analysis, FEM. Table 5.11 compares FoS obtained from LEM and FEM analyses of unreinforced slopes.

In this research, the result obtained from LEM are conservative. However, in some areas FoS computed from LEM are slightly greater than FEM results. Because LEM refers to a simple procedure and thus missing some important parameters (i.e.  $E$ ,  $K_o$ ,  $\Psi$ ) that have an effect on the output result.

For the traditional LEM analysis, the convergence of the results obtained from Morgenstern-Price Method used SLOPE/W model was utilized by using lambda ( $\lambda$ ) value. The computed FoS from all cases of slopes are matched the convergence results, see appendix A

Two cases of a reinforced slope were analyzed by using both LEM and FEM. Nails within different inclination angles are used as a reinforcement material as discussed in previous sections. Figure 5.10 and 5.13 show Slope/w and plaxis2D models of the 1:2 reinforced slope with 30° nail inclination respectively. For LEM analysis, the maximum FoS of the reinforced 45° slope is computed at zero-degree nail inclination, and a further increase of nail angle reduced FoS. whereas case two reinforced slope the computed FoS is maximum at 30-degree nail inclination.

However, for FEM using plaxis2D software, the analysis shows that the computed FoS from reinforced slopes are maximum when nail inclinations are 30° and further increase or decrease reduced the computed FoS, see table 5.12.

In this presented study, also three cases from the literature have been analyzed to compare and validate the reliability of the LEM and FEM software's used in this analysis. These slopes were modeled by using geometries and parameters defined in the literature. The output results were compared to the original FoS computed by Griffiths and Lane (1999), B.H. Maula and L. Zhang (2011) and the slide7.0 verification manual respectively. The original and computed results are very similar since the maximum error is less than 5% as table 4.4 show.

Table 5. 11: Comparison of the FoS computed from unreinforced analyses by LEM and FEM

| Case (V:H) | LEM, SLOPE/W | FEM, PLAXIS2D FoS | % FoS of LEM > FEM |
|------------|--------------|-------------------|--------------------|
| Case 1:1   | 0.904        | 1                 | 10.6               |
| Case 1:1.2 | 1.033        | 1.129             | 9.29               |
| Case 1:1.5 | 1.238        | 1.225             | 1                  |
| Case 1:2   | 1.788        | 1.747             | 2.3                |

Table 5. 12: Comparison of FoS computed from LEM and FEM

| Cases (V:H) | LEM, SLOPE/W |       | FEM, PLAXIS2D FoS | % FoS of LEM > FEM |
|-------------|--------------|-------|-------------------|--------------------|
|             | Degree       | FoS   |                   |                    |
| Case 1:1    | 0°           | 1.009 | 1.022             | 1.29               |
|             | 15°          | 1.004 | 1.042             | 3.78               |
|             | 30°          | 0.994 | 1.205             | 21                 |
| Case 1:2    | 0°           | 1.846 | 1.883             | 2                  |
|             | 15°          | 1.905 | 1.817             | 4.6                |
|             | 30°          | 1.913 | 1.935             | 1.15               |

## CHAPTER SIX: CONCLUSIONS AND RECOMMENDATIONS

Investigation of the cause and remedial method for failed slope around railway tunnel portal using limit equilibrium method and finite element method has been fully discussed in this research. For limit equilibrium, SLOPE/W (GEOSTUDIO, 2012) software has been used to compute FoS and critical slip surfaces. On the other hand, FEM based PLAXIS2D was cast-off to evaluate slope stability determining FoS and displacement.

### 6.1 Conclusion

Currently, the two methods of slope stability analyses, one based on limit equilibrium (LEM) formulations and the other based on finite element (FEM) principles are widely used in practice. The basic physics of stress-strain relationship, which is missing in LEM, has been well covered by the FEM. As a result, FEM can simulate displacement, which has been experienced problematic in LEM. This has been one of the advantages of FE calculations. On the other hand, LEM has been used over the decades. Thus, they are common in practice due to their well-establishment, user-friendliness, and simplicity.

In chapter 4 and chapter 5, the stability of 2D static slope around the tunnel portal was examined. In addition, the analyses of three case studies were carried out in this thesis to verify and validate the results obtained from the software. The major conclusion from the research is the investigation of the cause and remedial method for the failed slope using both LE model and FE model. In most of the cases, LEM gives a lower estimate of FoS (high risk) than FEM models, while the other two cases gave higher FoS, (low risk). Because, LEM are simplistic in their approach and ignore some important parameters like the horizontal to vertical stress ratio (K ratio), young's modulus (E), to name a few, are not included in the LE analysis that have an effect on slope stability. The effects of these parameters tend to make the slope more stable than the results obtained from the LE analysis.

First, slope was analyzed as it before cracks were developed (failed), and thus, FoS and displacement were computed from LEM and FEM. In this case, the results obtained from both methods (LEM & FEM) show that the slope is unstable and risk to failure. In other words, this result confirms the real condition of the slope as the slope has failed.

From the analyses in chapter five, it found that the steepness of the slope the more it susceptible to the failure. For case one, the results from the analyses indicated slope is highly prone to failure as discussed earlier. However, after application of the slope stabilization techniques “slope angle reduction” slope stability has significantly increased. For example, a case four where slope angle is 1:2, FoS of 1.788 and 1.747 are computed from LEM and FEM respectively. In addition, the other finding is that the application of slope angle reduction together with nailing made the slope more stable and reliable. This means slope stability has greatly improved; thus, we concluded the major problem of slope instability was triggered due to the 1H:1V slope geometry.

It is noted that conducting such as comprehensive slope stability analysis, requires a full set of site investigation data. However, the limited data received from Yapi Merkezi, “the design and built contractor” made possible to conduct this thesis. But, as a result of limited data, the work carried out this thesis is limited to 2D static slope analysis using LEM and FE method.

## **6.2 Recommendation**

This thesis was carried out a real failed slope which located Awash-Kombolcha-Hara Gebaya T09. As a result of slope instability, there is a need for slope stability investigation and application of an appropriate remedial method.

Due to observed instability, lower parameters than initially envisaged, and lack of rocky conditions, and in order to have a stable slope, at the same time removing the slide material, and to start with a 2/1 slope eliminating the 1/1 slope geometry is recommended.

Three dimensional with seismic analysis is recommended for future work to find out a detailed cause of the slope stability problem.

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## APPENDICES

### APPENDIX A: Limit equilibrium lambda utilize convergence results from M-PM, SLOPE/W

Figure A1: Unreinforced slope case 1H:1V convergence result from M-PM using SLOPE/W and utilized by lambda

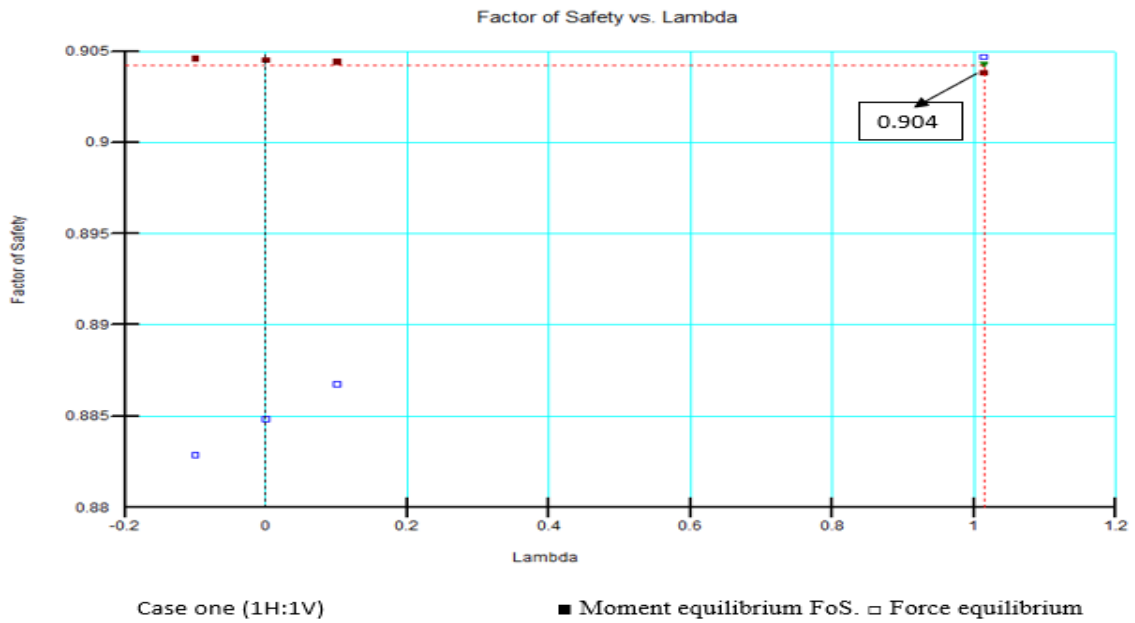


Figure A2: Unreinforced slope case 1.2H:1V convergence result from M-PM using SLOPE/W and utilized by lambda

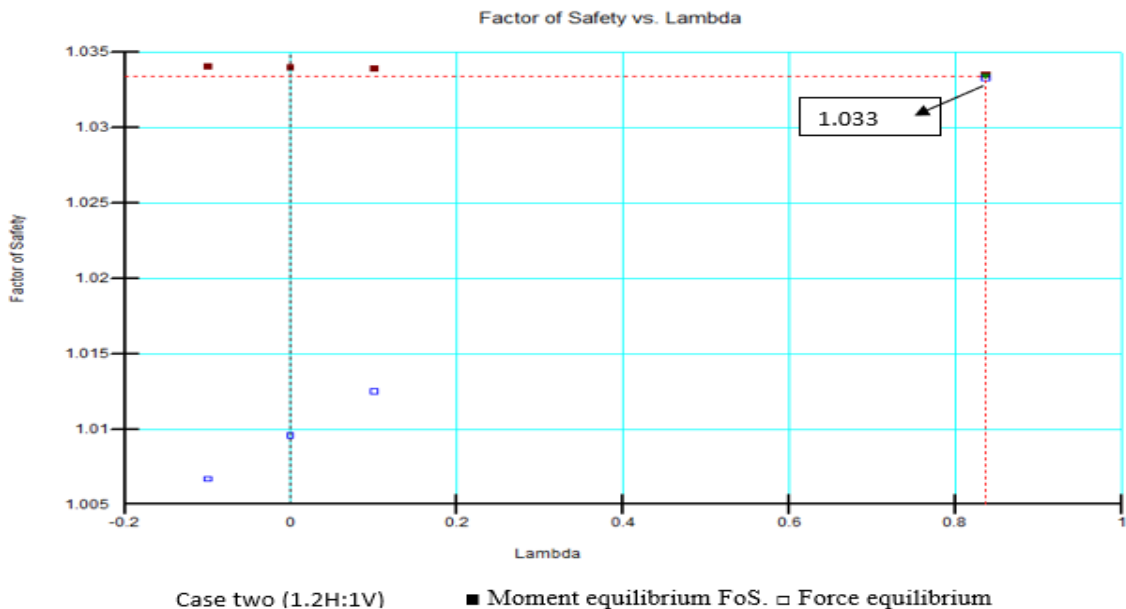


Figure A3: Unreinforced slope case 1.5H:1V convergence result from M-PM using SLOPE/W and utilized by lambda

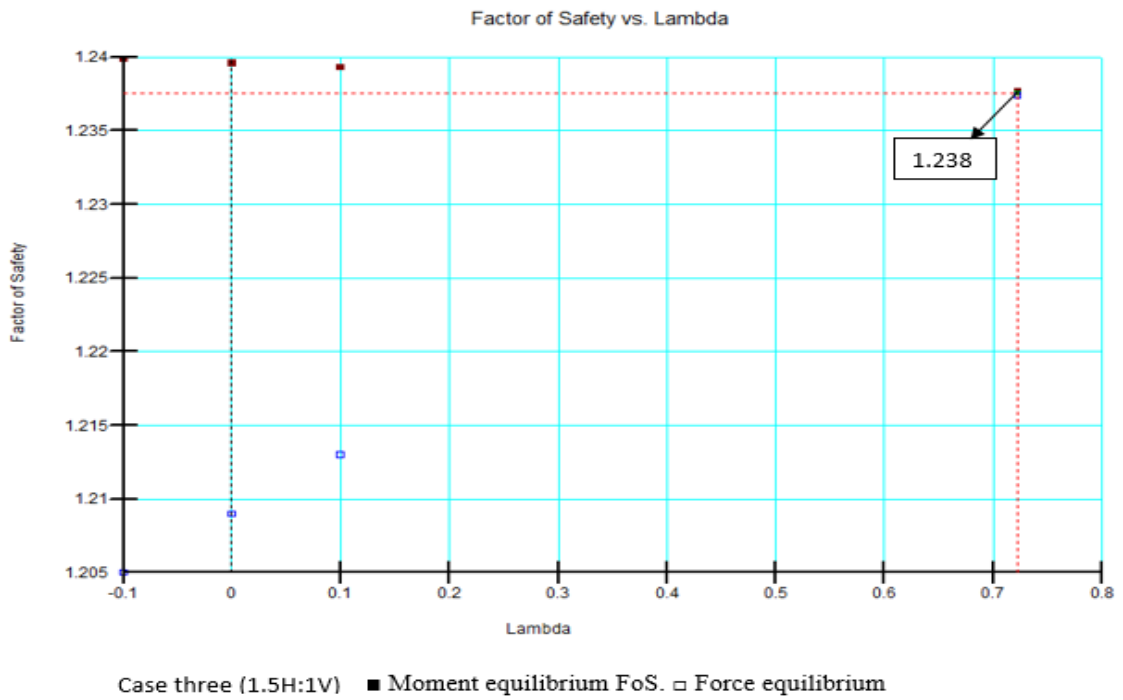


Figure A4: Unreinforced slope case 1.5H:1V convergence result from M-PM using SLOPE/W and utilized by lambda

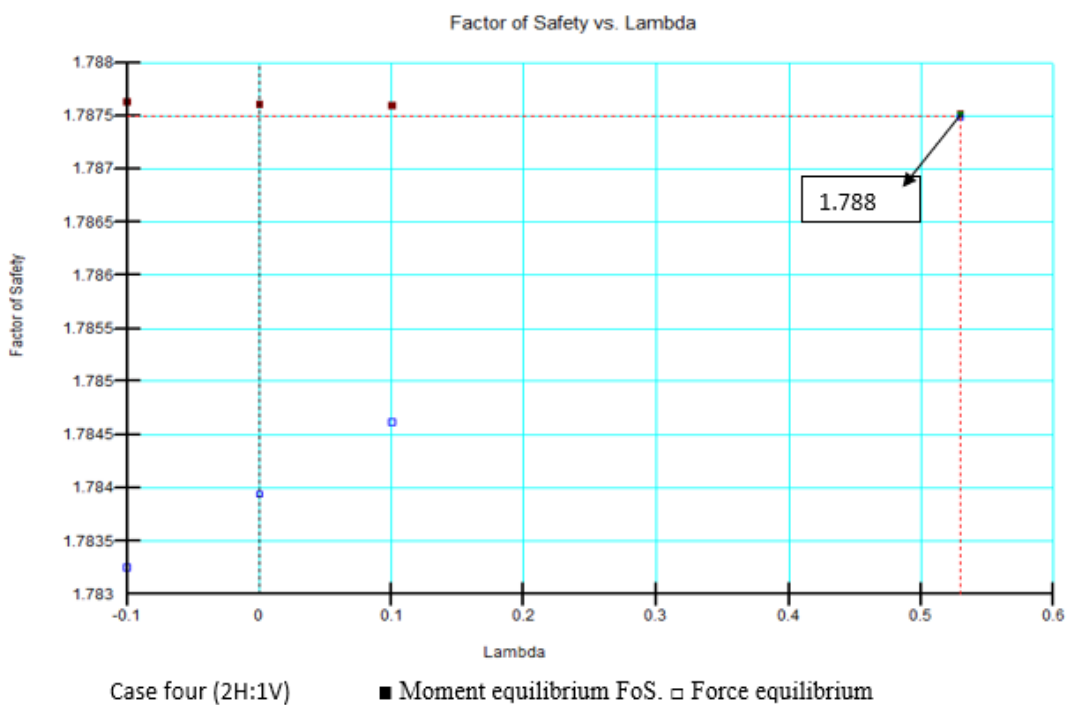


Figure A5: Reinforced 45° slope convergence result from M-PM using SLOPE/W with utilized by lambda

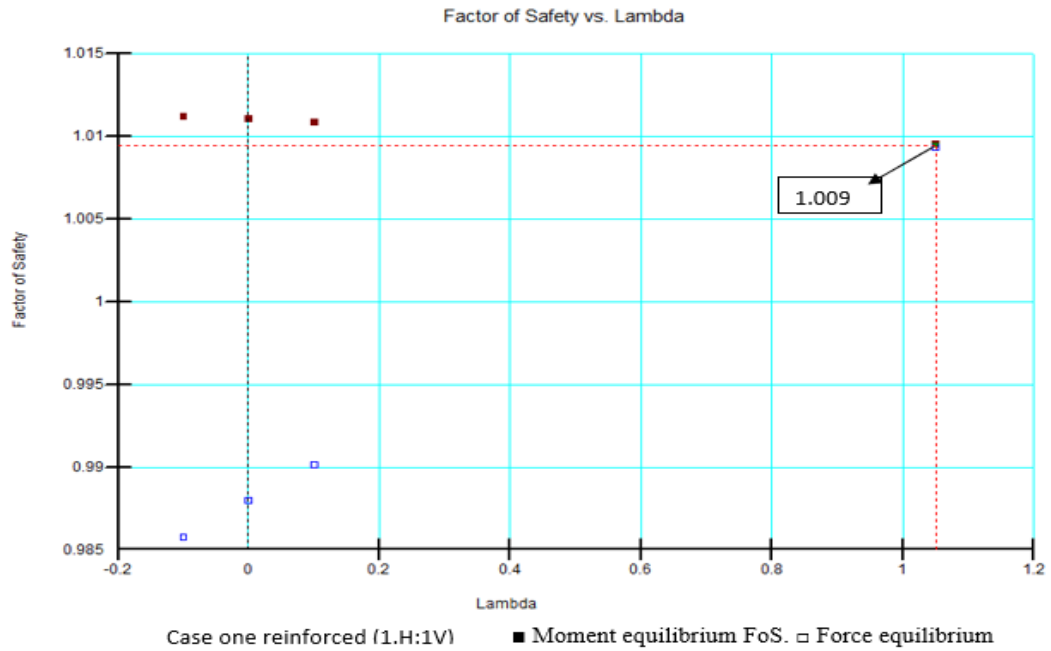
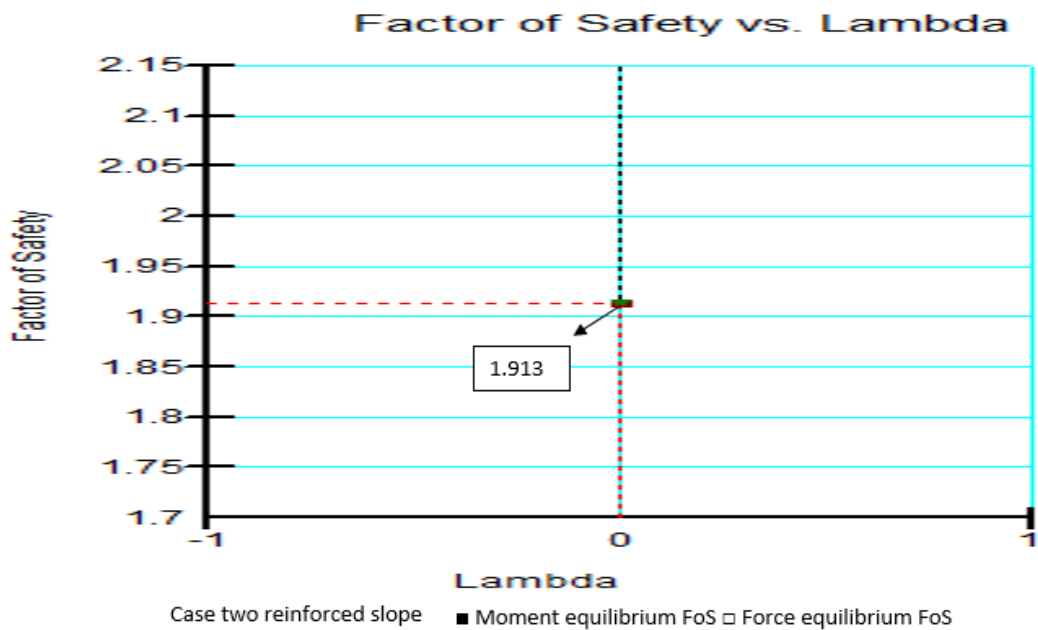


Figure A6: Reinforced 26.565° slope convergence result from M-PM using SLOPE/W with utilized by lambda



**Appendix B:** Plot of all failure surfaces computed from LEM, SLOPE

Figure B1: All computed slice failure surfaces from 1:1 slope geometry

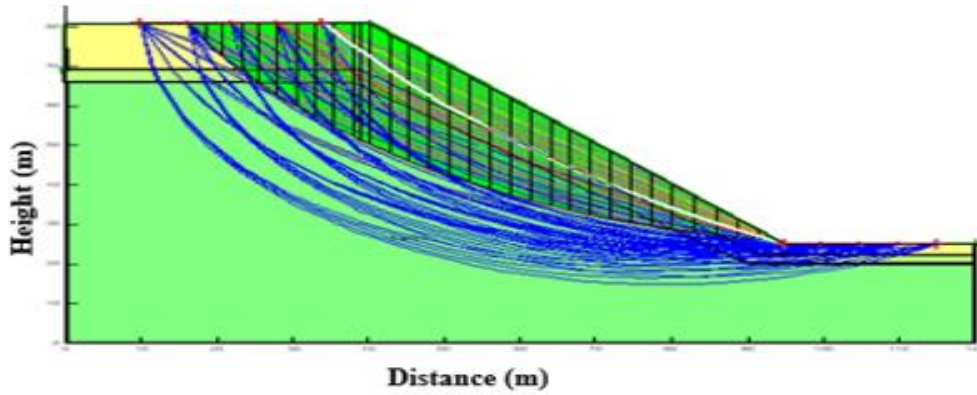


Figure B2: All computed slice failure surfaces from 1:1.2 slope geometry

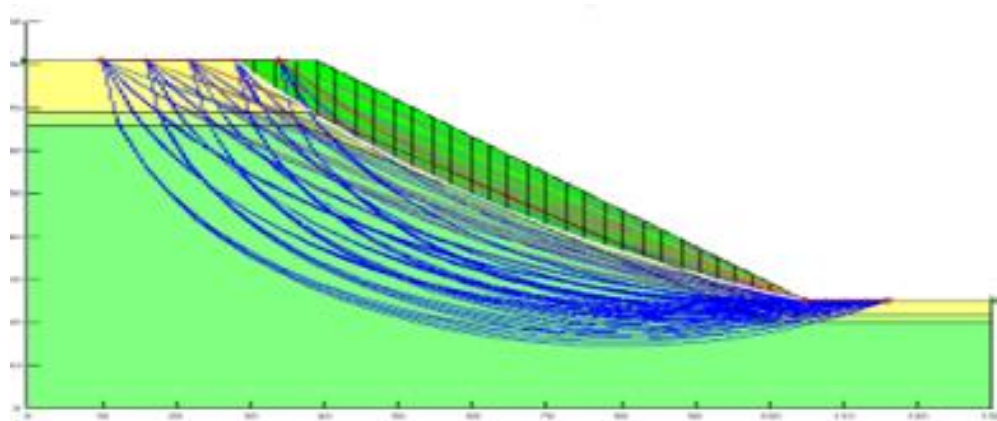


Figure B3: Plot of all failure surfaces computed from 1:1.5 slope geometry

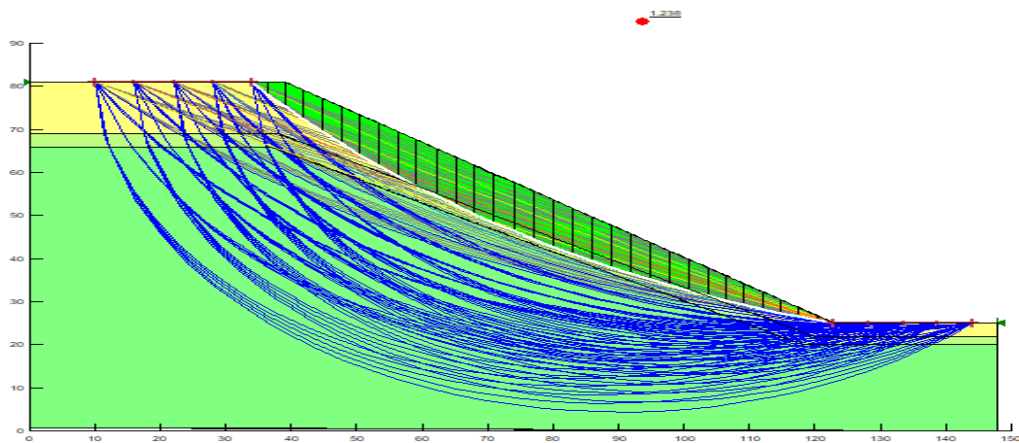


Figure B4: Plot of all failure surfaces computed from 1:2 slope geometry

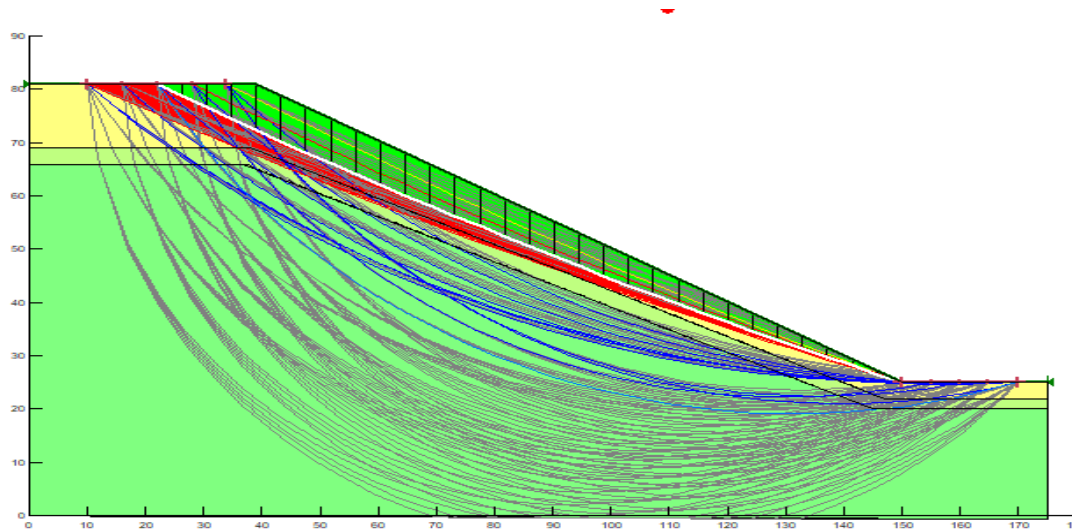


Figure 5B: Total shaded displacement from 1:1 slope geometry, FEM

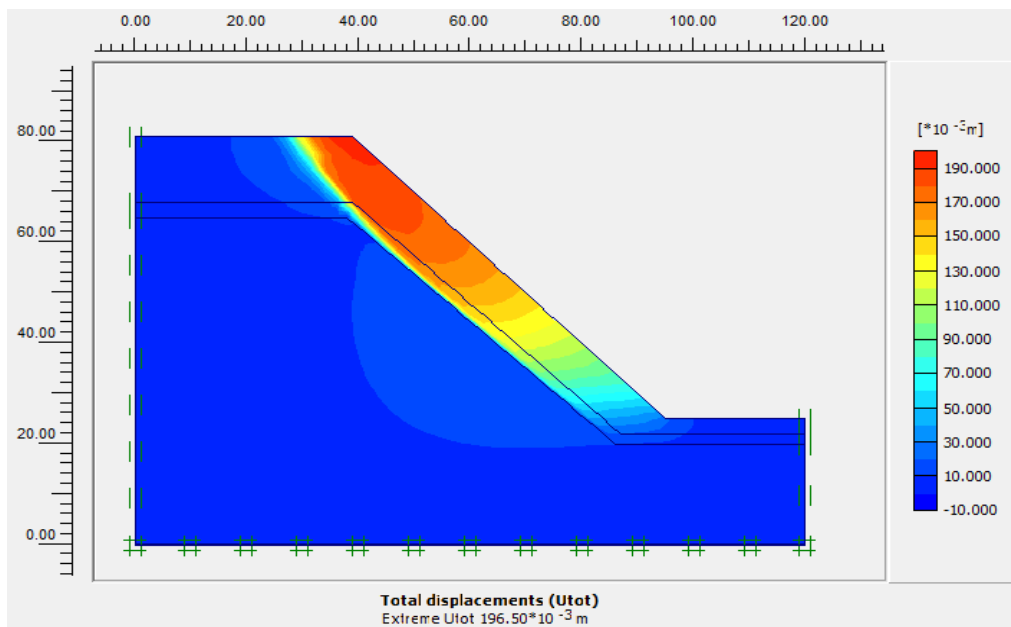


Figure 6B: Total shaded displacement from 1:1.2 slope geometry, FEM

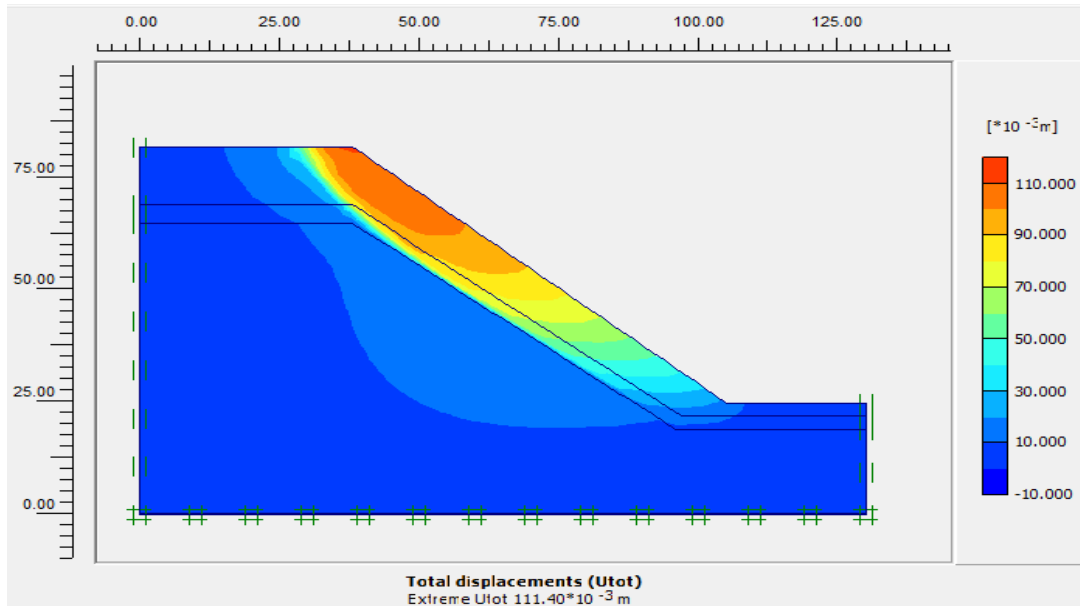


Figure 7B: Total shaded displacement from 1:1.5 slope geometry, FEM

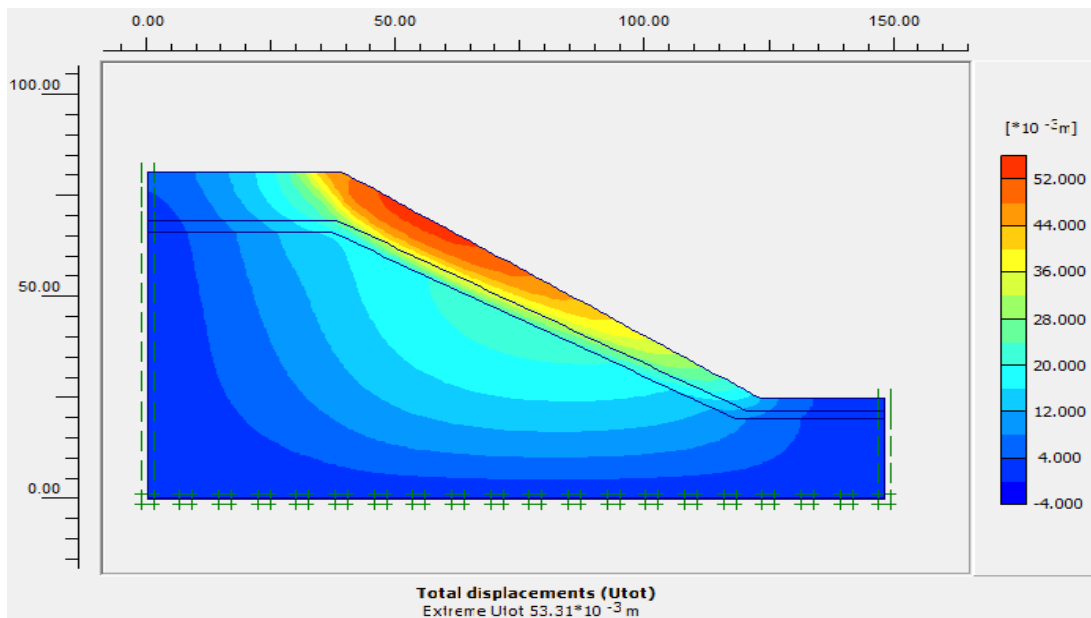
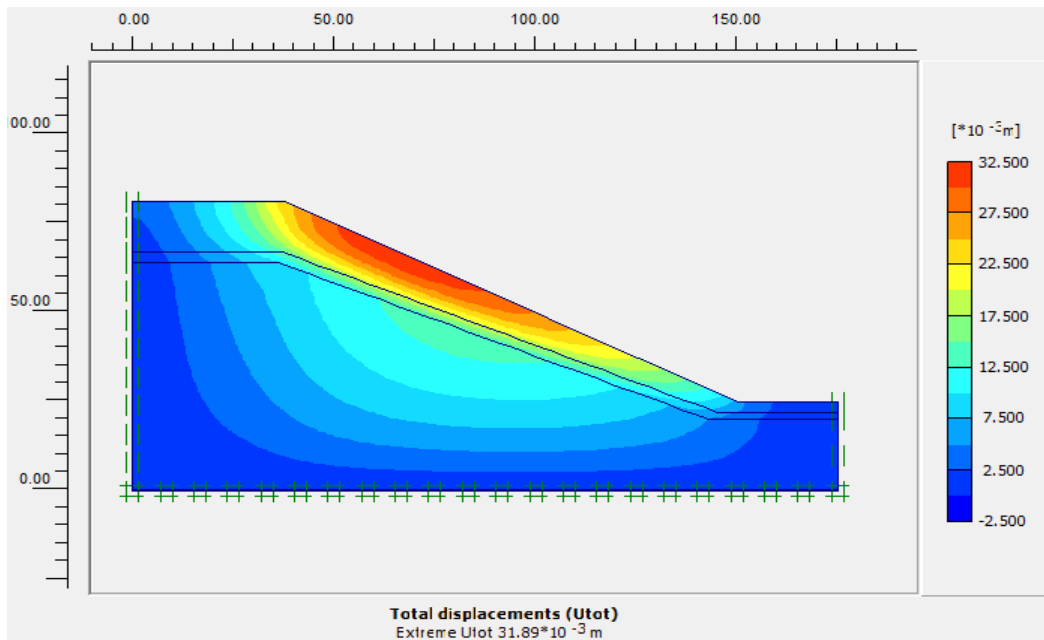
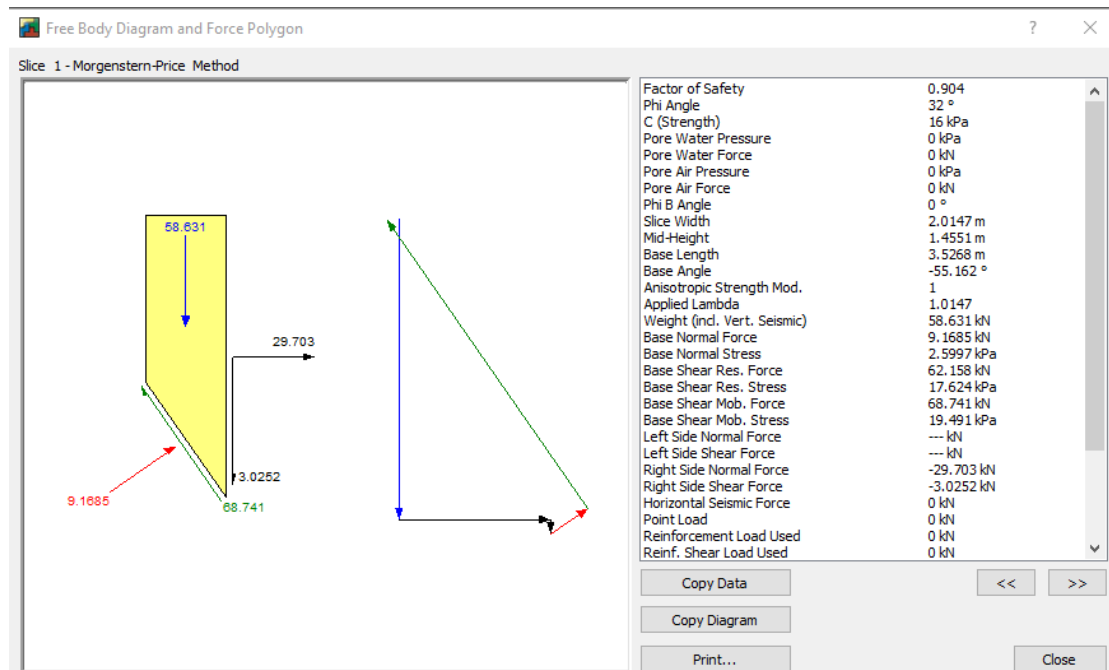


Figure 8B: Total shaded displacement from 1:2 slope geometry, FEM



## Appendix C: LEM Analysis slice information and report

Figure C.1: Slice information case one



## Slope Stability

Report generated using GeoStudio 2012. Copyright © 1991-2015 GEO-SLOPE International Ltd.

## File Information

File Version: 8.15  
Title: Case one  
Created By: Mahamoud Ahmed Muhumed  
Last Edited By: User  
Revision Number: 33  
Date: 6/8/2019  
Time: 11:45:28 AM  
Tool Version: 8.15.1.11236  
File Name: C1M-PM.gsz  
Directory: C:\Users\User\Desktop\SlopeW\  
Last Solved Date: 6/8/2019  
Last Solved Time: 12:05:37 PM

## Project Setting

Length(L) Units: Meters  
Time(t) Units: Seconds  
Force(F) Units: Kilonewtons  
Pressure(p) Units: kPa  
Strength Units: kPa  
Unit Weight of Water: 9.807 kN/m<sup>3</sup>  
View: 2D  
Element Thickness: 1

## Analysis Setting

### Slope Stability

Description: Slope stability Analysis Case one  
Kind: SLOPE/W  
Method: Morgenstern-Price  
Settings  
    Side Function  
        Interslice force function option: Half-Sine  
    PWP Conditions Source: (none)  
Slip Surface  
    Direction of movement: Left to Right  
    Use Passive Mode: No  
    Slip Surface Option: Entry and Exit  
    Critical slip surfaces saved: 1  
    Resisting Side Maximum Convex Angle: 1 °  
    Driving Side Maximum Convex Angle: 5 °  
    Optimize Critical Slip Surface Location: No  
    Tension Crack  
        Tension Crack Option: (none)  
F of S Distribution  
    F of S Calculation Option: Constant  
Advanced

Number of Slices: 30  
F of S Tolerance: 0.001  
Minimum Slip Surface Depth: 0.1 m  
Search Method: Root Finder  
Tolerable difference between starting and converged F of S: 3  
Maximum iterations to calculate converged lambda: 20  
Max Absolute Lambda: 2

## Materials

### Upper layer

Model: Mohr-Coulomb  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion': 16 kPa  
Phi': 32 °  
Phi-B: 0 °

### Middle layer

Model: Mohr-Coulomb  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion': 23 kPa  
Phi': 30 °  
Phi-B: 0 °

### Lower layer

Model: Mohr-Coulomb  
Unit Weight: 25 kN/m<sup>3</sup>  
Cohesion': 110 kPa  
Phi': 50 °  
Phi-B: 0 °

### Slip Surface Entry and Exit

Left Projection: Range  
Left-Zone Left Coordinate: (7, 80.85358) m  
Left-Zone Right Coordinate: (32, 81.04547) m  
Left-Zone Increment: 4  
Right Projection: Range  
Right-Zone Left Coordinate: (94.94776, 25) m  
Right-Zone Right Coordinate: (115.02289, 25) m  
Right-Zone Increment: 4  
Radius Increments: 4

### Slip Surface Limits

Left Coordinate: (0, 69) m  
Right Coordinate: (120.16327, 25.02615) m

Points

|          | X (m)     | Y (m)    |
|----------|-----------|----------|
| Point 1  | 0.19077   | -0.14266 |
| Point 2  | 0         | 66       |
| Point 3  | 38        | 66       |
| Point 4  | 90        | 20       |
| Point 5  | 120       | 20       |
| Point 6  | 120       | 0        |
| Point 7  | 0         | 69       |
| Point 8  | 39        | 69       |
| Point 9  | 92        | 22       |
| Point 10 | 120       | 22       |
| Point 11 | 0.10666   | 80.80067 |
| Point 12 | 40.05878  | 81.10732 |
| Point 13 | 95.04717  | 24.89838 |
| Point 14 | 120.16327 | 25.02615 |

Regions

|          | Material     | Points               | Area (m <sup>2</sup> ) |
|----------|--------------|----------------------|------------------------|
| Region 1 | Lower layer  | 1,2,3,4,5,6          | 5,346.3                |
| Region 2 | Middle layer | 2,7,8,9,10,5,4,3     | 374.5                  |
| Region 3 | Upper layer  | 7,11,12,13,14,10,9,8 | 1,061.4                |

**Current Slip Surface**

Slip Surface: 102  
 F of S: 0.904  
 Volume: 520.435 m<sup>3</sup>  
 Weight: 10,408.7 kN  
 Resisting Moment: 1,091,685.3 kN-m  
 Activating Moment: 1,207,878 kN-m  
 Resisting Force: 4,675.1777 kN  
 Activating Force: 5,167.8266 kN  
 F of S Rank (Analysis): 1 of 125 slip surfaces  
 F of S Rank (Query): 1 of 10 slip surfaces  
 Exit: (94.947758, 24.999998) m  
 Entry: (32, 81.045465) m  
 Radius: 173.40694 m  
 Center: (175.32793, 178.65219) m

## Appendix D: FEM analysis output from reinforced slopes

Figure 1D: Case one 30-degree reinforced slope deformed mesh from plaxis2D

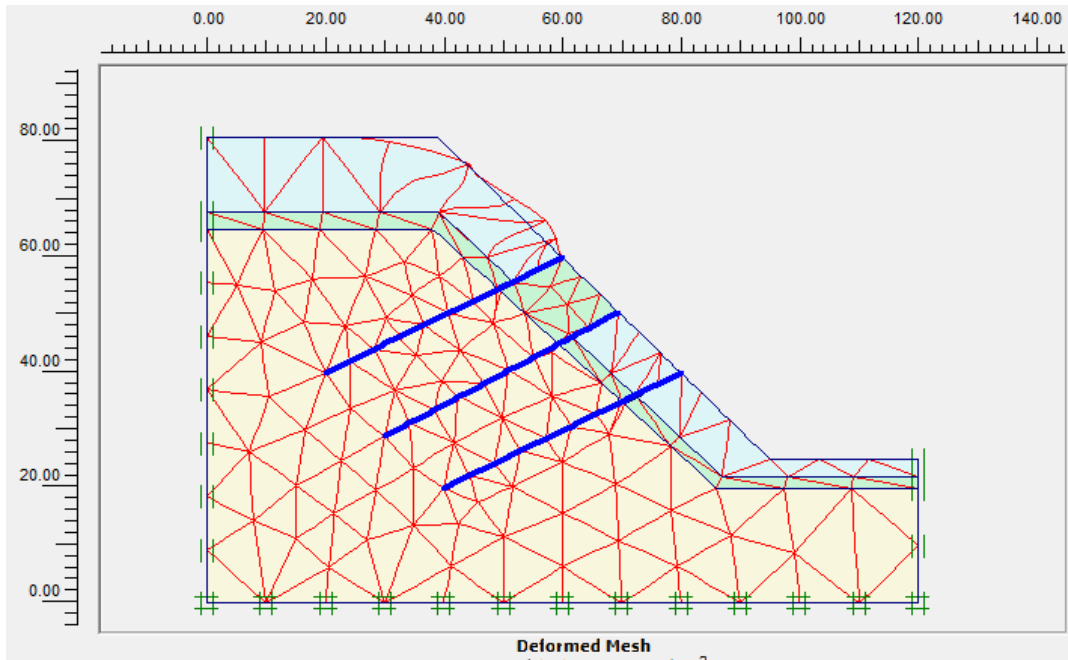


Figure 2D: Factor of safety from 0°, 15°, and 30°. case one reinforced slope

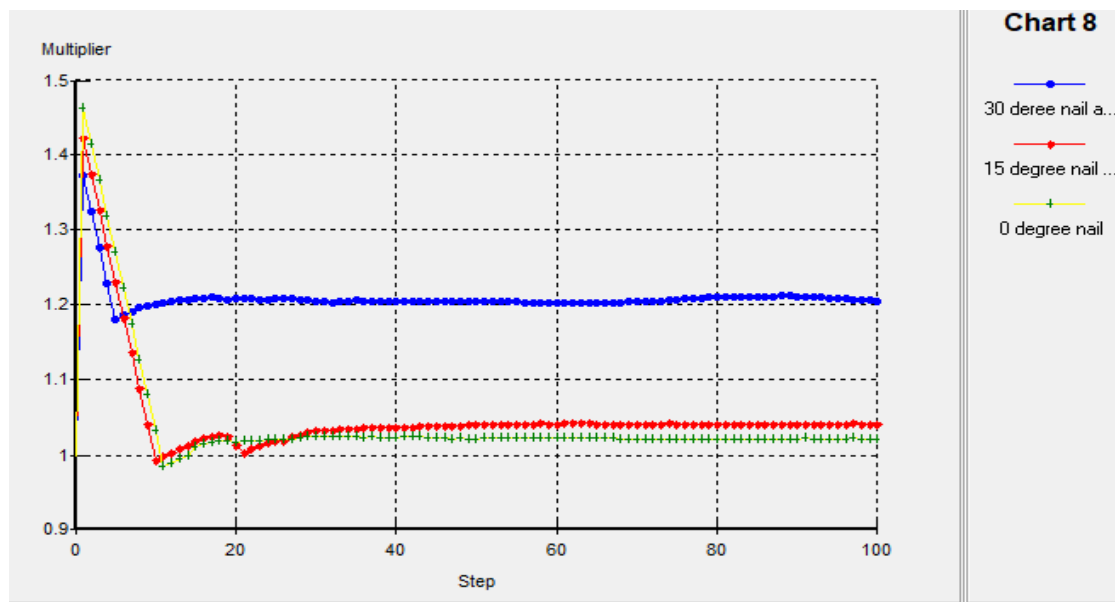


Figure 3D: Case two zero-degree reinforced slope deformed mesh from plaxis2D

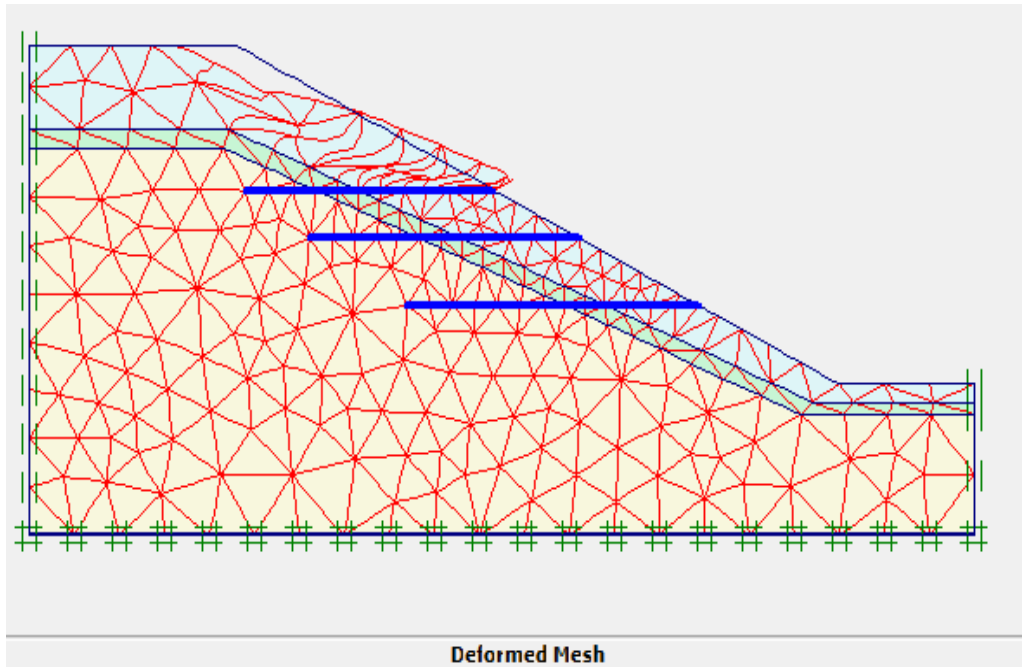


Figure 4D: Factor of safety from 0°, 15°, and 30°. case two reinforced slope

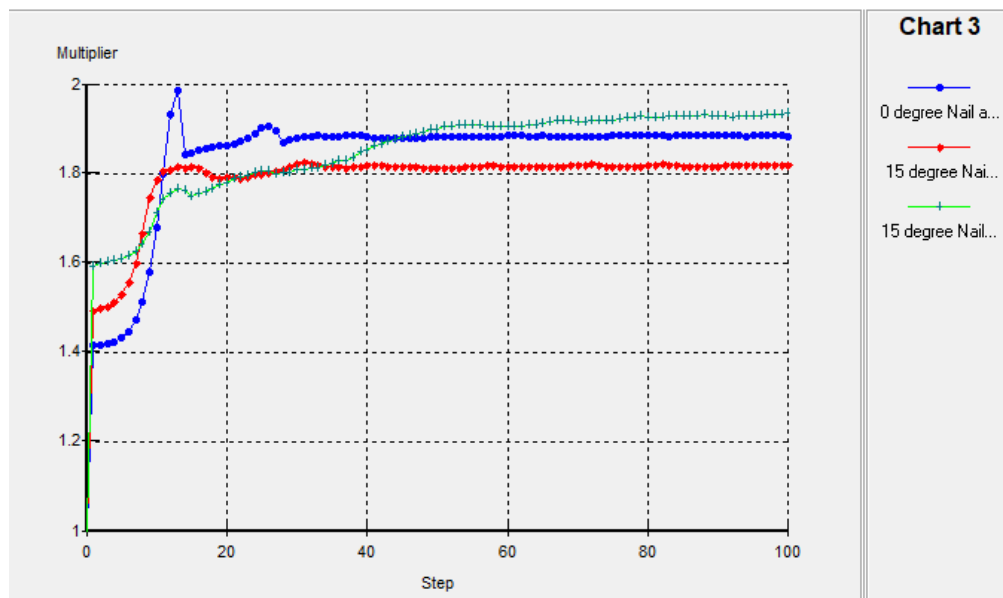


Table 1C: Summary of the displacement from all cases

| <b>Unreinforced</b> |     | <b>Total displacement(mm)</b> | <b>Horizontal displacement(mm)</b> | <b>Vertical displacement(mm)</b> |
|---------------------|-----|-------------------------------|------------------------------------|----------------------------------|
| Case 1H:1V          |     | 196.5                         | 147.9                              | -135.58                          |
| Case 1.2H:1V        |     | 111.4                         | 89.3                               | -69.47                           |
| Case 1.5H:1V        |     | 53.31                         | 50.97                              | -20.56                           |
| Case 2H:1V          |     | 31.89                         | 31.23                              | -7.31                            |
| <b>Reinforced</b>   |     | <b>Total displacement(mm)</b> | <b>Horizontal displacement(mm)</b> | <b>Vertical displacement(mm)</b> |
| Case<br>1H:1V       | 0°  | 148.37                        | 124.23                             | -96.98                           |
|                     | 15° | 48.26                         | 42.96                              | -22.4                            |
|                     | 30° | 36.52                         | 34.39                              | -16.04                           |
| Case<br>2H:1V       | 0°  | 52.58                         | 51.76                              | -12.55                           |
|                     | 15° | 57.72                         | 56.66                              | -16.12                           |
|                     | 30° | 58.96                         | 57.44                              | -20.68                           |