TRIBHUVAN UNIVERSITY

INSTITUTE OF ENGINEERING

PULCHOWK CAMPUS

THESIS NO.: G814/073

Stability Analysis of Road-cut slope: A Case Study of Kanti Lokpath

by

Shankar Khatri

A THESIS

SUBMITTED TO THE DEPARTMENT OF CVIL ENGINEERING

IN PARTIAL FULLFILLMENT OF THE REQUIREMENT FOR THE

DEGREE OF MASTER OF SCIENCE IN

GEOTECHNICCAL ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

LALITPUR, NEPAL

SEPTEMBER, 2020

COPYRIGHT

TRIBHUVAN UNIVERSITY

INSTITUTE OF ENGINEERING

PULCHOWK, CAMPUS

The undersigned certify that they have read, and recommended to the Institute of Engineering for acceptance, a thesis entitled "Stability Analysis of road-cut slope:A case study of Kanti Lokpath" submitted by Shankar Khatri in partial fulfillment of the requirements for the degree of Master in Geotechnical Engineering.

____

Supervisor, Dr. Indra Prasad Acharya

Associate Professor

Institute of Engineering

Pulchowk Campus, Pulchowk, Lalitpur, Nepal

____

External Examiner, Prof. Dr. Akal Bahadur Singh

Institute of Engineering

Pulchowk Campus, Pulchowk, Lalitpur, Nepal

____

Program Coordinator, Dr. Indra Prasad Acharya

Associate professor

M. Sc. Program in Geotechnical Engineering

Department of Civil Enginering

Pulchowk Campus, Pulchowk, Lalitpur, Nepal

Date ________________________

ABSTRACT

This research work presents the stability analysis of natural slope subjected to cutting for the construction of road at three different chainage along Kanti Lokpath using limiting equilibrium method by means of computer based geotechnical software slope/W(Geo-studio) 2019 and finite element method by means of software Phase2. Data required for the slope stability analysis is obtained from the laboratory test, carried out in many samples in order to determine physical and mechanical properties of soils, field survey and standard guidelines Factor of safety for the cut slopes was determined for different anticipated conditions. The result shows that stability of the slope decreases with increase in ground water level, increase in unit weight, decrease in cohesion strength and decrease in friction angle.

Two cut slopes is unstable at dry condition and one cut slope is unstable when water table rises. Reduction of ground water table, application of retaining wall and soil nailing techniques are used with different parameters as preventive measures and cut slope is analyzed to provide optimized solution.

ACKNOWLEDGEMENT

I would like to express my deep gratitude to my thesis supervisor Dr**. Indra Prasad Acharya** for his valuable guidance, expertise, encouragement and critical suggestion without whom; this dissertation would not have been possible. His constant surveillance and suggestions have helped me in the critical stage of this thesis.

My sincere thanks to Prof. Akal Bahadur Singh for his valuable suggestions during the thesis period.

My special thanks go to my friends especially Prabin Acharya, Manju Gurung and Naresh Regmi for their great help and encourage during the thesis work.

I would like to thank my colleague and senior who encouraged me to remain within the deadline.

I am thankful to all the staffs of department of Civil Engineering, Pulchowk Campus, IOE, who supported me throughout my thesis work.

Last but not the least; I would like to thank my family: my parents, my brother for their constant support, love and encouragement through my study period.

TABLE OF CONTENTS

LIST OF FIGURE

LIST OF TABLE

LIST OF ABBREVIATIONS

2-D= Two Dimensional

3-D= Three Dimensional

FEM=Finite Element Method

FHWA=Federal Highway Administration

FOS= Factor of Safety

GIS= Geographic Information system

GWT= Ground Water Table

IS= Indian Standard

kN= Kilo Newton

kPa= Kilo Pascal

LEM= Limit equilibrium Method

OMS= Ordinary Method of Slice

PVC= Polyvinyl Chloride

RDSO= Research Designs and Standards Organization

SRF= Strength Reduction Factor

MSS= Maximum shear strain

1. INTRODUCTION

1.1 Introduction

Landslides in the Himalayan region occur naturally due to neotectonic activity, earthquakes, and high rainfall, but are increasing in frequency due to large-scale human activities, such as road widening and the construction of dams, bridges, tunnels, and connections between valleys and main roads. All of these activities increase the vulnerability of rock and soil masses to failure when those masses are already subject to alarmingly high levels of natural stress. The increasing human population and its overexploitation of natural resources pose a threat to these slopes. Slope failure is the most frequent disaster faced by many road projects in Nepal, especially when relatively steep natural slopes are subjected to cutting for the development of space for carriageway. Most failures of road cut slope are caused by design errors which include geometric design i.e. slope inclination, slope height and inability to estimate the load and the soil resistance. During excavation work of natural slopes, the slope face may deform and results in the reduction of shear strength, and this can lead to slope failure.

Slope stability analysis is performed to assess safe and economic design of human made and natural slopes. The main objectives of slope stability analysis are finding the endangered areas, investigation of potential failure mechanism, determination of slope sensitivity towards different triggering factors, designing of optimal slope with regards to safety reliability and economics, designing possible remedial measures. In assessment of the slopes, engineers primarily use factor of safety values to determine how close or far slopes are from failure. When factor of safety is greater than 1, resistive shear strength is greater than driving shear stress and the slopes is considered stable. When factor of ration is close to 1, shear strength is nearly equal to shear stress and the slope is close to failure, if FOS is less than 1 the slope should have already failed.

1.2 Study Area

Study area lies in the Kanti Lokpath at the chainage of 62+300, 68+300 and 68+700 in the Lalitpur district near Bhatte Danda of Provinence -3 of Nepal as shown in figure 1.1

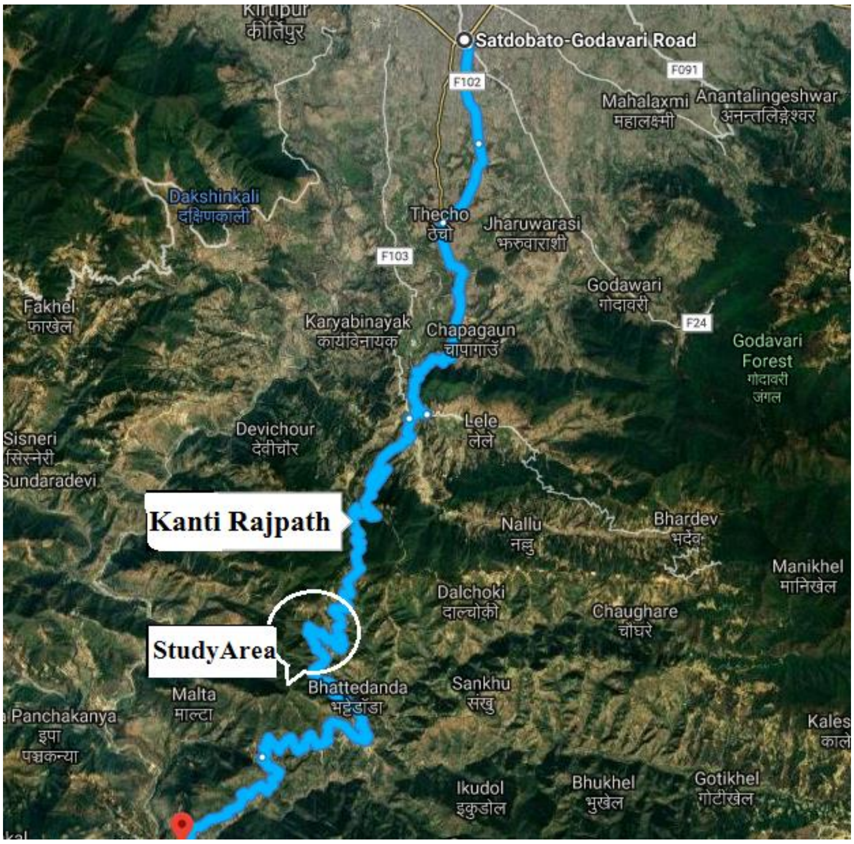


Figure 1.1: Alignment of Kanti Lokpath and study area

1.3 Objectives and Limitation of the study

1.3.1 Objectives

The main objectives of this thesis work is to carried out engineering and geotechnical study of the cut slope area of Kanti Lokpath at chainage sections 62+300, 68+300 and 68+700 in Lalitpur district near Bhatte Danda and find out respective suitable solution to make them stable.

The specific objectives of this study are to:

- Carryout the geological study of the cut slope, soil type and its role in stability of the slope.

- Generate the slope model of the cut slope using Slope/W module of Geo studio 2019 and Phase2 in static conditions.
- Assess the safety of a cut slopes in terms of its stability.
- Understand and numerically evaluate the sensitivity of stability to its geologic parameters.
- Find the probable solutions for the stabilization of each slope section according to their individual properties and situations which may be ground water reduction, retaining wall, soil nailing or other if any.
- To find out the most effective stabilization measures or combination of them with their optimization and aid in their design
- To compare and model verification of the each outcome of thesis work with the existing verified data and literature.

1.3.2. Limitation of the study

- For evaluating the cohesion and angle of internal friction better result can be obtained if undisturbed samples could have been obtained through Standard penetration test, direct cone penetration test and field vane shear test.
- Preparing 3-D model give more realistic factor of safety and closely represents the ground conditions.

1.4 Organization of thesis

The chapters in this thesis document have been organized as follows:

Chapter 1 serves as introduction of research. This chapter provides background of the research with general outlines. Study area, objectives and limitation of the discussed in this chapter.

Chapter 2 presents the overview of the literature reviewed during for this research work. This includes literature review of geotechnical parameters that affect the stability analysis, method of stability analysis and their significance. Different slope stabilization methods and their significance are also has been reviewed.

Chapter 3 describes the techniques, methods and software used in this study. All the methods applied from data collection, management, refinement and uses have been discussed in this chapter. This chapter also includes various parameters considered during the data analysis and methodologies used for stability analysis of the of slope sections.

Chapter 4 deals with the result and outcomes of the research work. In this chapter the results of static analysis of different three cut slope sections with variable parameters using slope/W and phase2 software has been presented.

Chapter 5 deals with concluding remarks with solution and recommendation for Future study.

2. LITERATURE REVIEW

2.1 Introduction

A major cause of cut slope failure is related to the reduced confining stress within the soil upon excavation. Undermining the toe of the slope or increasing the slope angle results in slope failure. The major cut slope design parameters are slope geometry, soil shear strength and predicted or measured ground water levels. For cohesion less soil, stability of a cut slope is independent of height and therefore slope angle becomes the key parameter of concern. For cohesive soils, the height of the cut becomes the critical design parameter. For c-φ and saturated soils, slope stability is dependent on both slope angle and height of cut.

2.2. Site Investigations

Before any further examination of an existing slope, or the ground onto which a slope is to be built, essential borehole information must be obtained. This information will give details of the strata, moisture content and the standing water level. Also, the presence of any particular plastic layer along which shear could more easily take place will be noted. Ground investigations also include:

- In-situ and laboratory tests
- Aerial photographs
- Study of geological maps and memoirs to indicate probable soil conditions
- Visiting and observing the slope for the study in this thesis, field investigations have been done.

2.3 Geotechnical Parameters

Before a geotechnical analysis can be performed, the parameters values needed in the analysis must be determined.

2.3.1 Unit weight

Unit weight of a soil mass is the ratio of the total weight of the soil to the total volume of the soil. Unit weight (γ) is usually determined in the laboratory by measuring the weight and volume of a relatively undisturbed soil sample obtained from the field.

2.3.2 Cohesion

Cohesion is the component of shear strength of a rock or soil that is independent of interparticle friction. In soil true cohesion is by the electrostatic forces in stiff over consolidated clays and cementing. It is also caused by negative capillary pressure and pore pressure due to loading process. Slopes having less cohesion force are less in stable. Different factors like friction, stickiness of particles, and cementation of grains by calcite or silica, manmade reinforcement, water content, repeated expansion or contraction due to wetting and drying, under cutting of slope and vibration due to earthquake or blasting affect cohesive forces. Cohesion(c) is usually determined in the laboratory from the Direct Shear Test.

2.3.3 Friction Angle

It is the angle between the normal force and the resultant force when the failure just occurs due to shearing stress. The measure of the material able to withstand any amount of shear stress. Factors which are responsible for friction angles are particle roundness, particle size and amount of quartz content in the soil. The angle of internal friction can be determined in the laboratory by the Direct Shear Test or by tri-axial test. For our analysis we will use direct shear test to determine the angle of internal friction.

2.3.4 Young's Modulus of Soil

Young's modulus of soil (E), commonly referred to as soil elastic modulus, is an elastic soil parameter and a measure of soil stiffness. It is defined as the ratio of the stress along an axis over the strain along that axis in the range of elastic soil behaviour.Young’s soil modulus (Es) may be estimated from empirical correlations, laboratory test results on undisturbed specimens and results of field tests. Laboratory test that might be used to estimate the soil modulus is the tri-axial test. For our analysis we will use values determined by Tyrens AB.

2.3.5 Type of soil

Geotechnical engineers classify soils, or more properly earth materials, for their properties relative to foundation support or use as building material. These systems are designed to predict some of the engineering properties and behavior of a soil based on a few simple laboratory or field tests

1. Sand Soil material that contains 85% or more sand; the percentage of silt plus 1.5 times the percentage of clay does not exceed 15 (CSSC; USDA).
2. Clay Soil materials that contains 40% or more clay and 40% or more silt (CSSC; USDA).
3. Silt Soil material that contains 80% or more silt and less than 12% clay (CSSC; USDA).
4. Silty clay Soil material that contains 40% or more clay and 35% or more silt (CSSC; USDA).
5. Sandy clay Soil material that contains 7 to 27% clay, 28 to 50% silt, and less than 52%.

Table2. 1: Soil classes and estimated shear strength properties

Group	Classification	Unit weight (t/m^3)	Friction angle (^{o})	Cohesion (t/m^2)
GW	Clean gravel, well graded	2.00 ±0.25	40 ±5	0
GP	Clean gravel, poor graded	1.90 ±0.30	38 ±6	0
GM	Silty gravel, little fines	2.10 ±0.25	36 ±4	0
GC	Clayey gravel, little fines	2.05 ±0.20	34 ±4	0
GM-ML	Silty gravel, many fines	2.15 ±0.25	35 ±5	0
GM-GC	Silty to clayey gravel	2.19 ±0.20	33 ±3	0.2 ±0.2
GC-CL	Clayey gravel, many fines	2.10 ±0.20	29 ±4	0.3 ±0.3
GC-CH	Clayey gravel, with high plastic fines	1.95 ±0.20	28 ±4	0.4 ±0.4
SW	Clean sand, well graded	1.96 ±0.20	38 ±5	0
SP	Clean sand, poorly graded	1.85 ±0.25	36 ±6	0
SM	Silty sand, little fines	2.00 ±0.25	34 ±3	0
SC	Clayey sand, little fines	1.96 ±0.20	32 ±4	0
SM-ML	Silty sand, many fines	2.00 ±0.20	34 ±3	0
SM-SC	Silty to clayey sand	2.10 ±0.20	31 ±3	0.5 ±0.5
SC-CL	Clayey sand, many fines	2.05 ±0.20	28 ±4	0.5 ±0.5
SC-CH	Clayey sand, with high plastic fines	1.85 ±0.20	27 ±3	1.0 ±1.0
ML	Silt	1.90 ±0.25	33 ±4	0
CL-ML	Silt to clayey silt	2.10 ±0.15	30 ±4	1.5 ±1.0
CL	Clayey silt	2.00 ±0.15	27 ±4	2.0 ±1.0
CH	Clay	1.75 ±0.15	22 ±4	2.5 ±1.0
OL	Organic clayey silt	1.20 ±0.15	25 ±4	1.0 ±0.5
OH	Organic clay	1.56 ±0.15	22 ±4	1.0 ±0.5
MH	Inorganic silt with high compressibility elastic silt	1.56 ±0.15	24 ±6	0.5 ±0.5

(Source: Adapted from Krahenbuhl and Wagner 1983)

2.4 Slope Stability Analyses

For a slope to be stable the resisting forces in the slope must be sufficiently greater than the forces causing the failure (Duncan and Wright 2005). To perform a slope stability analysis the geometry of the slope, external and internal loading, soil stratigraphy and strength parameters and variation of the ground water table all along the slope must be defined. In the current state of practice, there are many number of slope stability analysis methods available. However, the scope of this report is limited to a discussion on the limit equilibrium method and Finite element method in static and pseudo static cases. The most common slope stability analysis methods discussed as follow;

2.4.1 Limit Equilibrium method

The limit equilibrium method is the most common approach for analyzing slope stability in both two and three dimensions. This methods of analysis is a well-established method and widely used by the geotechnical engineers and engineering geologist. This method mainly provides an assessment of stability of the slope in terms of its safety factor. The Limit equilibrium analyses consider force and/or moment equilibrium of a mass of soil above the potential failure surface. The available shear strength is assumed to be mobilized at same rate at all points on the potential failure surface. Therefore, as a result the factor of safety is constant over the entire failure surface. The Limit equilibrium analysis provides only an estimate of the stability of a slope but does not provide any information regarding to the magnitude and movement of slope (Duncan & Wright, 2005).

A variety of limit equilibrium procedures have been developed to analyze the static stability of slopes. Slope that fail by translation on a planar failure surface (figure 2.1a) such as a bedding plane, rock joint, or seam of weak materials can be analyzed quite easily by the Cullman method (Taylor,1948). Slopes in which failure is likely to occur on two or three planes (figure 1b) can be analyzed by wedge methods (Perloff and Baron, 1976; Lambe and Whitman, 1969). In homogeneous slopes, the critical failure surface usually has circular (figure 2.1c) or log-spiral shape. Since the minimum factors of safety for circular and log-spiral failure surfaces are very close, homogeneous slopes are usually analyzed by methods such as the ordinary method of slices (Fellenius,1927) or Bishop`s modified method (Bishop, 1955), which assume circular failure surfaces. When sub-surface conditions are not homogeneous (e.g., when the layers with significantly different strength, high anisotropic strength, or discontinuous exists), failure surfaces are likely to be non-circular (figure 2.1d). In such cases, methods like those of Morgenstern and Price (1965), Spencer (1967), and Janbu (1968) may be used

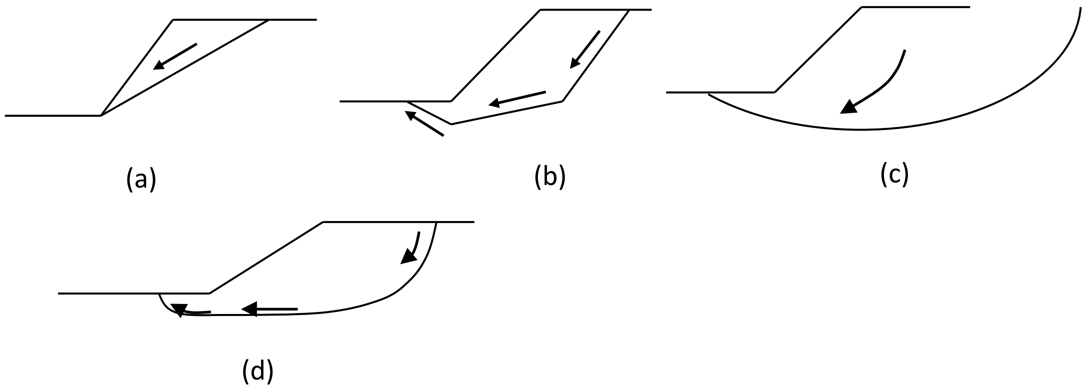


Figure 2. 1: Common failure surface geometries: (a) planner, (b) multi planer, (c) circular, (d) non-circular

2.4.2 Finite element method

The finite element method was first introduced to geotechnical engineers in 1966 Berkeley conference on stability of slopes and embankments by Clough and Woodward (1967). The finite element method considers linear and non-linear stress – strain behavior of the soil in calculating the shear stress for the analysis. In a finite element approach the slope failure occurs through zones which cannot resist the shear stresses applied. Hence, the results obtained from this analysis are considered to be more realistic compared to limit equilibrium method (Griffiths and Lane 1999).

Finite element methods are well known for the estimating the realistic deformations of the slopes and embankments. Some of the advantages of using a finite element analysis over limit equilibrium methods are,

The movement of the slopes at a particular location can be calculated. This helps in monitoring the movement of the slope. Also, soil stresses and pore water pressure responses to different external factors such as load, water level, reservoir level etc. can be calculated.

Stability of the slope during staged construction such as step by step excavation or construction of embankments, levees etc. can be calculated by performing incremental analysis.

The types of soil stress-strain relationships that can be used are linear elastic, elastoplastic, hyperbolic, Modified Cam Clay, elastoviscoplastic and multilinear elastic models. The selection of a particular stress-strain relationship depends on the state of the soil structure to be analyzed, its purpose of analysis and its laboratory and field properties available. The determination of soil properties in the field involves a large amount of uncertainty and so the application of finite element analyses imposes complexity on the stability problem (Griffiths and Lane 1999).

Traditionally, the slope stability analysis with a finite element approach is performed by Strength reduction method (SRM). In this method, the factor safety is defined as the factor by which the original shear strength parameters must be divided to bring the slope to be in failure mode (Griffiths and Lane 1999). Hence, the factor shear strength parameters (c´f and Ø´f) are shown as follows,

$c'f = c'/SRF$ Equation 2.1

$Ø'f = \arctan(\tan Ø'/ SRF)$ Equation 2.2

Where, SRF is the Strength Reduction Factor.

A systematic estimation is required for the SRF value to find out the value which will just cause the slope to fail. The SRF value, at which the slope will just to fail, is known as the factor of safety. The failure condition in this method could be when 1) the non-linear equation solver cannot achieve convergence after a few iterations, 2) sudden rate of change in displacement and 3) a failure mechanism is developed. However, this method has some limitations such as appropriate selection of constitutive model and geologic parameters, boundary conditions and defining a failure condition (Krahn 2007).

2.4.3Numerical Analysis Methods

Numerical analysis methods give reasonable approximations to the correct mathematical solution of the governing equations of the mechanics of slope stability. In comparison to the limit equilibrium methods, the numerical analysis methods are more sophisticated and complicated: they take into account deformations (strains) and not just forces (stresses) like the conventional limit equilibrium methods do. Numerical methods have been extensively used in past several decades due to advances in computing power. The numerical methods can be classified in to continuum and discontinuum methods. There are quite large number of numerical methods that have been presented in the literature to estimate the behavior of system made of geo materials (Griffith, 2001).

2.4 Slice Method

The slice methods can be categorized in to two groups: rigorous and non-rigorous. The rigorous methods satisfies both force and moment equilibrium where as non-rigorous methods satisfies either force or moment equilibrium only. The factor of safety estimated from rigorous methods is relatively intensive to the assumptions made to obtain determinacy (Duncan, 1992).Based on equilibrium equations to be satisfied the limit equilibrium methods can be classified as:

- Overall equilibrium methods,
- Force equilibrium methods
- Moment and force equilibrium methods.

The method of slices assumes a trial circular failure surface, with the slipping or mobilized soil divided into a number of vertical slices so that the failure surface is approximately linear for each slice.

For each of these slices a number of assumptions are made:

- The slices are of width b (not necessarily constant for all slices) and variable height h measured through the centre-line
- The side that is the failure surface becomes a straight line of length l, at an angle a to the horizontal
- The factor of safety for each slice is the same and equal to that of the whole sliding mass.
- The normal force acts at the centre of each slice

The forces that act on a typical slice are,

1. Total weight of the slice = W
2. Normal force acting on the base = N (consisting of two parts the effective normal force, N', and the force due to the pore pressure at the base, U).
3. Shear force on the base = T
4. Normal forces on the sides = E1, E2.
5. Shear forces on the sides = X1, X2.

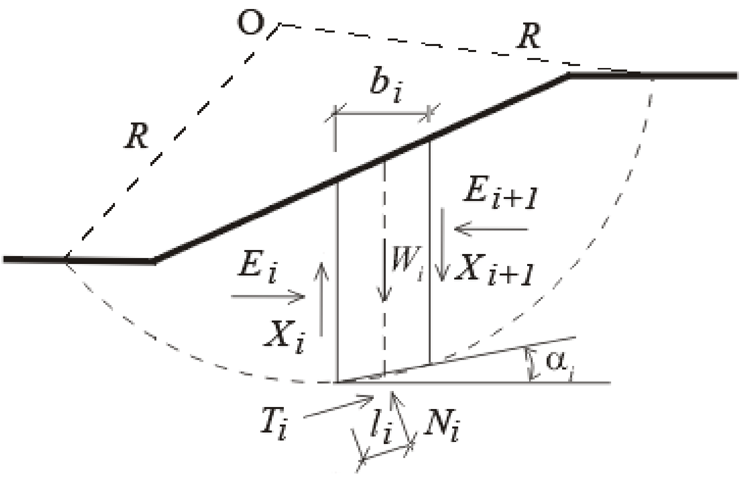


Figure 2. 2: Different forces on slice

The general form of the solution by method of slices is as follows:-

The factor of safety is taken as the ratio of the available shear strength (τ_f) to the mobilized shear stress (τ_m) on the failure plane.

Or,

$$F_s = \frac{\tau_f}{\tau_m}$$ Equation 2.3

However, the driving moment about O must be equal to the restoring,

$$\sum[W.r\sin\alpha] = \sum[T.r]$$ Equation 2.4

Note that since the inter slice forces are internal forces (no resultant moments about O), but they influence the magnitude of N and T, and T has a moment about O.

Now

$$T = \tau_m.l = \frac{\tau_f}{F_s}.l$$ Equation 2.5

$$\tau_f = c' + \sigma' \tan\phi'$$ Equation 2.6

$$\sigma' = \frac{N'}{l}$$ Equation 2.7

Thus,

$$F_s = \frac{c' L_a + \sum [N'] \tan \phi'}{\sum [W \sin \alpha]}$$

Equation 2.8

With all the forces acting simultaneously on a slice the problem is statically indeterminate – especially in evaluating N'. The value of N' is a function of the weight of a slice, the pore pressure acting at the base of the slice and the inter slice forces – from vertical equilibrium. However, solutions can be found by making simplifying assumptions regarding the inter slice forces.

For Un-drained instability,

Inter slice force assumption: Inter slice forces E and X are ignored

Normal effective force: Not required – fu = 0

Factor of safety:

$$F_s = \frac{c_u \sum [b.\sec \alpha]}{\sum [W \sin \alpha]}$$

Equation 2.9

All slice methods assumes that the assumed soil mass and failure surface can be divided in to a finite number of slice and equilibrium conditions are consider for all slices.

Table2. 2: Equilibrium Conditions Satisfied by Limit Equilibrium methods **(Source: Abramson et al., 2002)**

Methods	Force Equilibrium		Moment Equilibrium
	Vertical	Horizontal	
Ordinary Methods of Slice	No	No	Yes
Bishop Simplified	Yes	No	Yes
Janbu Simplified	Yes	Yes	No
Bishop Rigorous	Yes	Yes	Yes
Janbu Generalized	Yes	Yes	Yes
Spencer	Yes	Yes	Yes
Morgenstern-Price(M-P)	Yes	Yes	Yes

There are different types of slice analysis methods based on limit equilibrium principles (Abramson et al., 2002) such as;

2.4.1 Ordinary Method of slice (OMS)

Ordinary method of slices is also known as the Fellenius or Swedish method. Fellenius **(**1927) used the concept of slices his method and it is effect only moment equilibrium method ($\sum M = 0$).Various forces acting on a slice are.

Inter slice force assumption: $E_1 = E_2$ and $X_1 = X_2$, but they are ignored in the analysis

Normal effective force: Force equilibrium normal to the shear plane gives:

$$N' = W\cos\alpha - u.l \qquad \text{Equation 2.10}$$

Where, u = pore pressure on the base of the slice

$l = b\sec\alpha$

$$F_s = \frac{c'.L_a + \sum[W\cos\alpha - u.l]\tan\phi'}{\sum[W\sin\alpha]} \qquad \text{Equation 2.11}$$

Factor of safety:

$$F_s = \frac{c'\sum[b.\sec\alpha] + \sum[W\cos\alpha - u.l]\tan\phi'}{\sum[W\sin\alpha]} \qquad \text{Equation 2.12}$$

2.4.2 Bishop simplified method

Bishop's Method eliminates most of the shortcomings of the method of slices. Bishop **(**1955**)** suggested determining the normal force N by resolving the forces acting on any slice in the vertical direction and not in the direction normal to the base as was done by Fellenius.

Inter slice force assumption: $E_1 \neq E_2$ and $X_1 = X_2$

Normal effective force:Vertical force equilibrium gives:

$$N' = \frac{W - (c'.l/F_s)\sin\alpha - u.l.\cos\alpha}{\cos\alpha + [\tan\phi'\sin\alpha/F_s]} \qquad \text{Equation 2.13}$$

$$F_s = \frac{1}{\sum[W\sin\alpha]}\sum\left[\frac{(c'.b + (W - u.b)\tan\phi')\sec\alpha}{1 + [(\tan\alpha.\tan\phi')/F_s]}\right] \qquad \text{Equation 2.14}$$

$$F_s = \frac{1}{\sum[W\sin\alpha]}\sum\left[\frac{(c'.b + W(i - r_u)\tan\phi')\sec\alpha}{1 + [(\tan\alpha\tan\phi')/F_s]}\right]$$ Equation2.15

Where, $r_u = \frac{u}{\gamma.h} = \frac{u.b}{W}$

2.4.3 Janbu simplified method

Janbu's method (1973) is more versatile that can be used for non-homogeneous and layered soil profiles and failure surfaces of any shape circular, curved or planar. Besides, although equal slice width considerably simplifies calculations, the width of individual slices can be different. Slice widths can be chosen to fit with the structural features such as soil-rock interface/slope geometry, changes in material properties and water pressure distribution. Because of this particular advantage, this method is also better able to take into account the effect of variability of shear strength parameters along different sectors/sections of the failure surface. The method is therefore also called the General Procedure of Slices (GPS).

Inter slice force assumption: $E_1 \neq E_2$ but are ignored; $X_1 = X_2$

Factor of safety:

$$F_s = f_o\frac{\sum[c'.b + (W - u.b)\tan\phi']m_\alpha}{\sum[W\tan\alpha]}$$ Equation 2.15

$$m_\alpha = \frac{\sec^2\alpha}{1 + [(\tan\alpha\tan\phi')/F_s]}$$ Equation 2.16

f_0 = Correction factor

2.4.4 Wedge methods

Wedge analysis is useful if the failure plane is assumed to be linear or a combination of linear segments. For a single planed slip surface, the factor of safety is given by;

$$F_s = \frac{c'.L_a + (W\cos\alpha - U)\tan\phi'}{W\sin\alpha}$$ Equation 2.17

$$F_s = \frac{2c'\sin\beta}{\gamma.H\sin(\beta-\alpha)\sin\alpha} + \frac{\tan\phi'}{\tan\alpha}$$ Equation 2.18

Where,

L_a= length of the planer slip surface

W= weight of the total slipping wedge

U= total pore pressure force on the slip surface

α = angle of the slip surface to the horizontal

β = angle of the slope to the horizontal

H= Height of the slope

The analysis can be extended to include multiple planer slip surfaces.

2.4.5 Morgenstern –Price Method

This method was developed by N.R. Morgenstern, and V.E. Price which consider not only the normal and tangential equilibrium but also the moment equilibrium for each slice in circular and non-circular slip surfaces. It is solved for the factor of safety using the summation of forces tangential and normal to the base of a slice and the summation of moments about the center of the base of each slice. The equations were written for a slice of infinitesimal thickness. The force and moment equilibrium equations were combined and a modified Newton-Raphson numerical technique was used to solve for the factor of safety satisfying force and moment equilibrium. The solution required an arbitrary assumption regarding the direction of the resultant of the interslice shear and normal forces.

2.4.5 Spencer’s Method

The Spencer’s method is considered same as Morgenstern-Price method except the assumption made for inter slice forces. In this method constant inclination is assumed for inter slice forces and the FOS is computed for both equilibriums(Spencer 1967).Spencer(1967) presented slope stability analysis method that satisfies all conditions of equilibrium for circular slip surfaces .Later ,he generalized and modified his method to adopt it to general slip surfaces.

Accuracy of the computational methods available is based on the extent to which it can satisfy the equilibrium conditions and its assumption on the inclination of side forces on each slice. According to Duncan and Wright (2005), the accuracy of the different methods is described in Table 2.3.

Table2. 3: Summary of 2D Limit Equilibrium methods for Slope stability analysis (after, Duncan and Wright (2005))

Method	Accuracy and Limitations
Ordinary method of slices (Fellenius 1927)	• Gives a very low Factor of safety value in case of effective stress analyses for flat slopes with high pore water pressures. • Accurate only when Ø = 0 analyses • Accurate in case of total stress analyses with circular slip surfaces.
Modified Swedish method (Corps of Engineers 1970)	• Applicable for all types of slip surfaces • Factor of safety values are generally higher than the other methods which satisfy all the conditions of equilibrium.
Bishop's modified method (Bishop 1955)	• Applicable for all types of slip surfaces • Factor of safety values are generally higher than the other methods which satisfy all the conditions of equilibrium.
Janbu's simplified method (Janbu 1968)	• Accurate method satisfying all equilibrium conditions. • Applicable to any shape of failure surface • Results in a lower factor safety values than other methods satisfying all equilibrium equations
Spencer's method (Spencer 1967)	• Accurate method satisfying all equilibrium conditions. • Applicable to any shape of failure surface
Morgenstern and Price method (Morgenstern and Price 1965)	• Accurate method satisfying all equilibrium conditions. • Applicable to any shape of failure surface

Based on the accuracy for each method discussed, only Morgenstern-Price method is used for conducting stability analysis of this research.

2.5 Shear strength Characterization

Shear strength is the main concern in slope stability analyses. Determination of the shear strength parameter is important work and understanding the theory is a essential in order to conduct analysis successfully. The limit equilibrium methods used for the evaluating the stability of slopes require an accurate and reliable estimate of the in situ shear strength of the slope materials. However, the shear strength parameters are strongly influenced by many conditions including the in situ state of stress, drainage, loading rates and soil and rock composition (Abramson, 2001).

2.5.1 Mohr-Coulomb Model

The most common way of describing the shear strength of geotechnical materials is by Coulomb's equation which is:

$\tau = c + \sigma_n \tan \varphi$ Equation 2.19

Where:

τ = shear strength (i.e. shear at failure),

c = cohesion, σ_n = normal stress on shear plane, and

φ = angle of internal friction (phi).

This equation represents a straight line on shear strength versus normal stress plot (Figure2.3). The intercept on the shear strength axis is the cohesion (c) and the slope of the line is the angle of internal friction (φ).

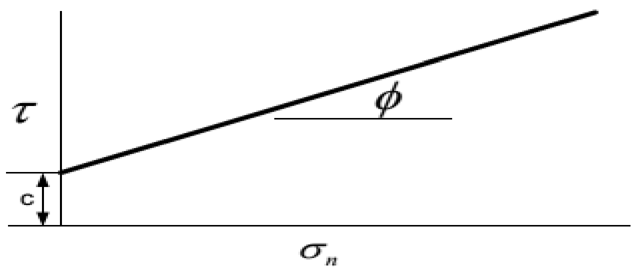


Figure 2. 3: Graphical representation of Coulomb shear strength equation

The failure envelope is often determined from tri-axial tests and the results are presented in terms of half-Mohr circles, as shown in Figure2.4, hence the failure envelope is referred to as the Mohr-Coulomb failure envelope.

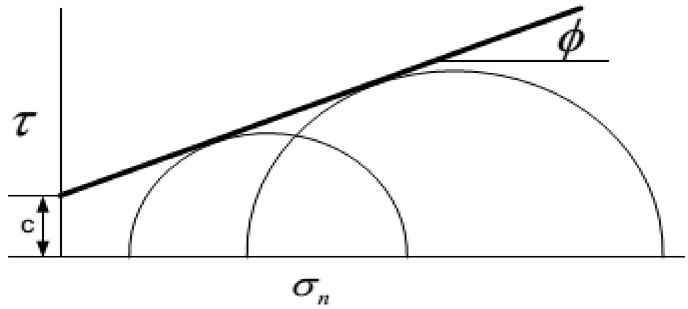


Figure 2. 4: Mohr-Coulomb failure envelope for undrained conditions

When φ is zero, the failure envelope appears as shown in Figure2.5. The soil strength then is simply described by c.

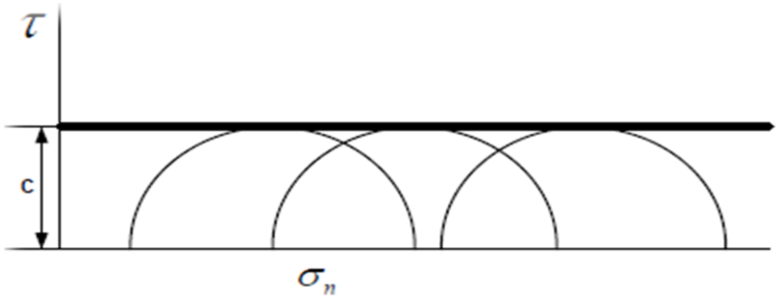


Figure 2. 5: Undrained strength envelope

The strength parameters c and φ can be total strength parameters or effective strength parameters. From a slope stability analysis point of view, effective strength parameters give the most realistic solution, particularly with respect to the position of the critical slip surface. The predicted critical slip surface position is the most realistic when we use effective strength parameters. When we use only un-drained strengths in a slope stability analysis, the position of the slip surface with the lowest factor of safety is not necessarily close to the position of the actual slip surface if the slope should fail. This is particularly true for an assumed homogeneous section.

2.6 Slope/W Software

Slope/W, developed by GEO-SLOPE international Canada, is used for slope stability analysis of soil and earth slope. The initial code was developed by Professor D.G. Fredlund at the University of Saskatchewan. This Software is based on the theories and principles of LEM methods discussed in the previous section .With SLOPE/W, both simple and complex problems can be analyzed for verities of pore water pressure conditions, slip surface shapes, analysis methods and loading conditions. SLOPE/W can model heterogeneous soil types, complex stratigraphy and slip surface geometry, variable pore-water pressure conditions using a large selection of soil models.

Varieties of options are available while modeling a slope in SLOPE/W which can be summarized in to following five steps.

- Geometry – description of the stratigraphy and shapes of potential slip surfaces.
- Soil strength - parameters used to describe the soil (material) strength.
- Pore-water pressure – means of defining the pore-water pressure conditions
- Reinforcement or soil-structure interaction – fabric, nails, anchors, piles, walls and so forth.
- Imposed loading – surcharges or dynamic earthquake loads.

The software Slope/W computes FOS for various shear surfaces, for example circular, non-circular and user-defined surfaces (Slope/W 2002, Krahn,2004).This software works on a limit equilibrium framework and includes methods such as; the Ordinary method of slice, Spencer's method, Bishop's method, Janbu's generalized method, Morgenstern-Price methods etc. For verification of the analysis illustrative examples are provided in the Slope/W manual (2019).These examples show a detailed comparison of the analysis result from Slope/W with solutions obtained from the Stability Charts developed by Bishop and Morgenstern(1960),a comparison with published results and a comparison with theoretical calculations of earth pressure. The analyses results from Slope/W prove to be the same as the values obtained from the other method, indicating that the result obtained from Slope/W program are reliable.

2.6 Phase2 Software

Phase2 has been widely used in geotechnical and mining engineering as a tool for the design and the analysis of tunnel, surface excavation and ore extraction and supports (Phase2 , 1999).Phase2 7.0 is an extremely versatile 2D elasto-plastic finite element stress analysis program for designing underground or surface excavations and their support systems. Phase2 7.0 can be used for rock or soil applications and includes finite element slope stability and groundwater seepage analysis.One of the major features of Phase2 is finite element slope stability analysis using the shear strength reduction method. This option is fully automated and can be used with either MohrCoulomb or Hoek-Brown strength parameters. The analysis parameters can be customized if required. Slope models can be imported from Slide and computed in Phase2 allowing easy comparison of limit equilibrium and finite element results. The Phase2 program consists of 3 program modules: model, compute and interpret. Model is the pre-processing module used for entering and editing the model boundaries, support, in-situ stresses, boundary conditions, compute for solving the program and interpret for visualize the output.

2.7 Slope Stabilization Methods

If the result of slope stability analysis indicate the roadway slope does not meet the factor of safety requirement, then it may be necessary to use slope stabilization methods. The safe FOS value for road cut slope is considered to be 1.2 (Hoek and Bray, 1989).Now days several slope stabilization methods are available to mitigate the slope failure along the road and other civil structure. Slope stabilization method can be broadly categorized as

- Preventive stabilization methods, applied to stable, but potentially unstable natural slope and slope to be cut.
- Remedial or corrective treatments applied to existing unstable, moving slopes or to failed slope.

2.7.1 Soil slope stabilization methods

According to Abramson (2001) the stability of any slope will be improved if certain actions are carried out. First of all identify the most important controlling process which affect the stability of slope and then determine the appropriate technique which can be sufficiently applied to overcome the influence of that process. A number of methods have been adopted to stabilize slopes, each of them found to be appropriate for a particular set of conditions.

- Application of Slope,
- Purpose of stabilizing,
- Time available,
- Accessibility of the site,
- Types of construction equipment, and
- The cost of repair and maintenance.

Various geotechnical, construction and environmental issues must be considered while selecting and designing stabilization measures appropriate for a site. Construction and environmental issues, which can affect the cost and schedule of the work, should also be addressed during design phase of the project. Other issues that are frequently important are equipment access, available work time during traffic closures, and disposal of waste rock and soil. The following sections provide a general description to techniques that can be used for soil slope stabilization.

2.7.2Retaining structures

Usually retaining structure is provided at the toe of a slope to stabilize it from slope failure, overturn or collapse. Some examples of retaining structure are retaining walls, sheet pile wall, basement wall and sheeting in excavations etc. A retaining structure used for supporting the soil mass laterally so that the soil can be retained at different level on two sides (Arora, 1997).

2.7.3 Gabions

Gabions are wire mesh cage or basket filled with stones. Gabion walls are simple and quicker in construction and also less expensive regarding to the other stabilization methods. Gabion provides excellent drainage facility because of their coarse fill, and can withstand foundation movement and they do not require elaborate foundation preparation. Gabion works because the friction between the individual gabions row is very high in comparison to the basal row and the soil underneath. Gabion walls built on clay should provide counterforts. The counterforts can be constructed as gabion headers extending from the front of wall to beyond the slip circle. The counter forts serve as both structural components and drains (Hutchinson, 1977).

2.7.4 Drainage techniques

Drainage of water is an effective method of increasing the stability of a slope. Water in a slope may come from two primary sources: surface water and groundwater. Water control is generally maintained through installation of surface and subsurface drainage devices within and adjacent to potentially unstable slopes. Runoff and infiltration of water along a slope face can often be reduced by planting vegetation on top of the slope to prevent or minimize erosion.

Surface Drainage Systems: Surface drains and landscape design are used to direct water away from the head and toe of cut slopes and potential landslides and to reduce infiltration and erosion in and along a potentially unstable mass.

Subsurface Drainage Systems: The main functions of sub-drains are to remove subsurface water directly from an unstable slope, to redirect adjacent groundwater sources away from the subject property and to reduce hydrostatic pressure beneath and adjacent to engineered structures. Control of subsurface drainage is generally attained by installing a network of horizontal and/or vertical sub-drains.

Drainages are the most frequently used means of stabilizing slopes. Slope failures are very often precipitated by a rise in the groundwater level and increased pore pressures. Therefore, lowering groundwater levels and reducing pore pressures is a logical means of improving stability. In addition, improving drainage is often less expensive than other methods of stabilization, and a large volume of ground can frequently be stabilized at relatively low cost. Once a system of drainage has been established, it must be maintained to keep it functional.

2.7.5 Geosynthetics reinforcement

Geosynthetics are porous, flexible, man-made fabrics which act to reinforce and increase the stability of structures such as earth fills, and thereby allow steeper cut slopes and less grading in hillside terrain. Geosynthetics of various tensile strengths are used for a variety of stability problems, with a common use being reinforcement of unpaved roads constructed on weak soils. Geosynthetics and Geosynthetics -related materials are generally classified on the basis of their manufacturing process. Geosynthetics can be knitting, woven, nonwoven or composite. Related Geosynthetics products in use are webs, mats, nets, grids, plastic sheets or composite structure. Geosynthetics have been used for filtration, drainage, separation, reinforcement, fluid barrier and protection.

2.8 Soil Nailing

It is a soil reinforcement technique that places closely spaced metal bars or rods into soil to increase the strength of the soil mass by resisting against tensile, shear, and bending stresses imposed by slope movements. Soil nails are either installed in drilled bore holes or secured with grout, or they are driven into the ground. The soil nails are generally attached to concrete facing located at the surface of the structure. The function of the facing is to prevent erosion of the surface material surrounding the soil nails, rather than providing structural support. This is a method of in situ reinforcement utilizing passive inclusions that get mobilized in case of slope movement occurs. It can be used to retain excavations and stabilize slopes by creating in situ reinforced soil retaining structures.

2.8.1 Types of soil nailing

Various types of soil nailing can be employed in the field:

1. Grouted nail – In the excavated wall/slope face holes are drilled first and nailed are placed in the - holes. Finally, cement grout is used to fill the drill hole.
2. Driven nail- Nails are mechanically driven in to the wall during excavation .In this type of soil nailing installation is very fast but it does not provide a good corrosion protection and are generally used as a temporary nailing.
3. Self-drilling soil nail- Hollow bars are driven and grout is injected simultaneously during the process of drilling through hollow bar. It exhibits more corrosion protection than driven nail and is faster than grouted nailing.
4. Jet grouted soil nail- Jet grouting is used to erode the ground and for creating the hole to install the steel bars. The grout provides god corrosion protection.
5. Launched soil nail- In this method bar is launched in to the soil with very speed using firing mechanism involving compressed air. Installation of nail is fast but to control the length of bar penetrating the ground is difficult.

2.8.2 Element of soil nail slope

Various components of grouted soil nail are as following:

1. Tendon- Tendons bars are main components of the soil nailing system. They may be solid or hollow (steel) bars .Solid bar are placed in stable drill holes and grouted in place while hollow bars are fitted with drill bit and used to drill the hole to the remain there as a the permanent nail. In response to the later movement and deformation of retained soil tensile stresses are mobilized through the tendon. Both solid and hollow tendons are typically threaded.
2. Grout- Grout usually consists of Portland cement and water. Grout is injected in the pre-drilled borehole after the nail is placed to fill up the nail bar and surrounding ground. The function of grout is to (i) transfer shear stresses

between deforming ground and the tendons ;(ii)transfer tensile stress from tendon to surrounding stable soil;(iii)provide some level of corrosion protection to the tendons. Grout pipe is used to inject the grout.

3. Centralizers-They are PVC material and fixed to the soil nail to ensure that the soil nail is centered in the drill hole.
4. Nail head-It is the threaded end of the soil nail which protrudes from the wall facing. It consists of a square shape concrete structure which includes the steel plates, steel nuts, and soil nail head reinforcement. This part of structure provides the soil nail bearing strength, and transfer bearing loads from the soil mass to soil nail.
5. Steel plate- a square shape steel plate used to transfer bearing from soil nail to the soil nail head.
6. Grout tube-Use to transfer the cement grout from grouting machine to bottom of soil nail.
7. Hex nut, washer- These are attached to the nail head and are used for connecting the soil nail to the facing.
8. Facing-Nails are connected to the slope surface by facing elements. Facing consist of temporary facing and permanent facing. Temporary/initial facing is applied on the supported excavation prior to advancement of excavation grades. It provides support to the exposed soil and also receives the bearing plate of soil nail. Permanent / final facing are provided over the temporary facing after installation of soil nails and provides structural continuity throughout the design life of soil nail .Permanent facing includes aesthetic finishing. The initial facing commonly consists of reinforced shortcrete. The reinforcement includes (i) welded wire mesh (WWM) installed over the entire excavation lift ,(ii)horizontal bars placed around the nail heads to add bending resistance to the horizontal direction and (iii) vertical bars placed at nail head to add bending resistance to the vertical direction. For soft or weathered rock other reinforcement options can be used such as steel or synthetic fibers. While permanent facing generally consists of CIP- reinforce concrete, reinforce shortcrete, or precast concrete panels.
9. Drainage System- drainage system are installed behind the wall to collect the infiltrated surface water present behind the facing and direct the collected

ground water away from the wall. Vertical geo-composite strips are installed prior to application of temporary facing for the drainage purpose.

10. Corrosion protection-High Density Polyethylene (HDPE) or Polyvinyl Chloride tube surrounding the nail bars is usually used to provide additional corrosion protection.

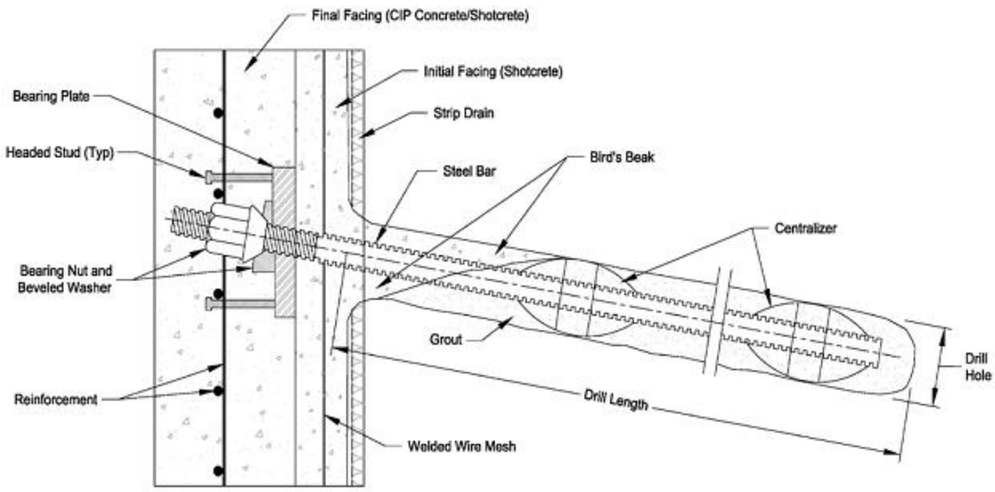


Figure 2. 6: Main components of a solid bar soil nails and facing modified after Porterfield et.al (1994).

2.8.3 Advantages of soil nailing

1. Fewer disturbances to traffic and less environmental impact than other stabilization technique.
2. Installation is relatively faster and requires less construction materials. It can be usable even at sites with remote access because of smaller and mobile construction plant.
3. Easily cope with site constraints and variation in ground condition encountered during construction by adjusting location, length of soil and inclination of soil nail.
4. Ductile failure mode of soil nail system provides warning system before failure.
5. Soil nails walls are relatively flexible and can accommodate relatively large total and differentials settlements. Soil nails walls have performed well during seismic events owing to overall system flexibility.

6. Soil nails require smaller right of way compared to ground anchors as soil nail are typically shorter.
7. It is less sensitive to undetected adverse geological features, and thus more robust and reliable than unsupported cuts. In addition, it renders higher system redundancy than unsupported cuts or anchored slopes due to the presence of a large number of soil nails (Geoguide 2007).

2.8.4 Disadvantage of soil nailing

1. The soil nailing system requires some soil deformation to mobilize resistance. For a site where very strict deformation control is required the soil nailing is not suited.
2. Soil nails are not suited for grounds with high ground water table which may lead to difficulty in drilling and excavation due to seepage of ground water table in excavation, corrosion of steel bars and change in grout water ratio.
3. The zone occupied by soil nails is sterilized and the site poses constraints to future development.
4. Soil nails are not suitable in case of cohesion less soils, because during drilling of hole, the un-grouted hole may collapse. In case of such soil casing can be provided during drilling.
5. Long soil nails are difficult to install, and thus the soil nailing technique may not be appropriate for deep-seated landslides and large slopes.
6. Soil nails are not effective in stabilizing localized steep slope profiles, back scarps, overhangs or in areas of high erosion potential. Suitable measures, e.g., local trimming, should be considered prior to soil nail installation (Geoguide 2007).
7. Construction of soil nail walls requires specialized and experienced contractors.

2.8.5 Fundamental Mechanism of soil nail system

The soil nailing technique improves the stability of slopes, retaining walls and excavations principally through the mobilization of tension in the soil nails (Geo-guide 2007).In soil nail tensile forces are developed primarily through the frictional interaction between the soil nails and the ground as well as the reactions provided by soil-nail heads/facing. The tensile forces in the soil nails reinforce the ground by directly supporting some of the applied shear loadings and by increasing the normal stresses in the soil on the potential failure surface, thereby allowing higher shearing resistance to be mobilized. Confinement effect provided by soil nail head and facing also limits the ground deformation close to normal to the slope face. The ultimate effect is to increase the mean effective stress and the shearing resistance of the soil behind the soil nail heads. They also help to prevent local failures near the surface of slope and to promote integral action of reinforced soil mass through the redistribution of forces among soil nails. The part of soil nail that is embedded in to the ground behind the potential failure surface provides the resistance against pullout failure of soil nails.

The internal stability of a soil-nailed slope is usually assed using a two-zone model, namely the active zone and passive zone (or resistance zone), and these zones are separated by a potential failure surface. The region in front of the potential failure surface is named as active zone and it has tendency to detach from the soil nailed system through potential failure surface. The passive zone is located behind the failure surface, where it remains more or less intact. The function of soil nail is to tie the active zone to passive zone. Two-zone analysis is only a simplified model for limit equilibrium analysis in which the deformation of a soil-nailed system is not accounted.

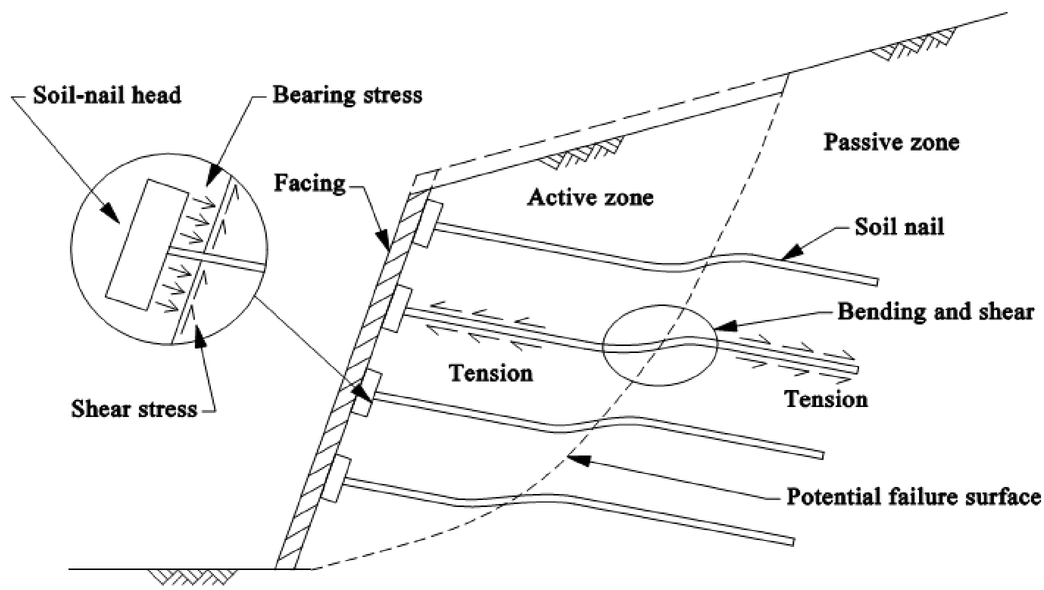
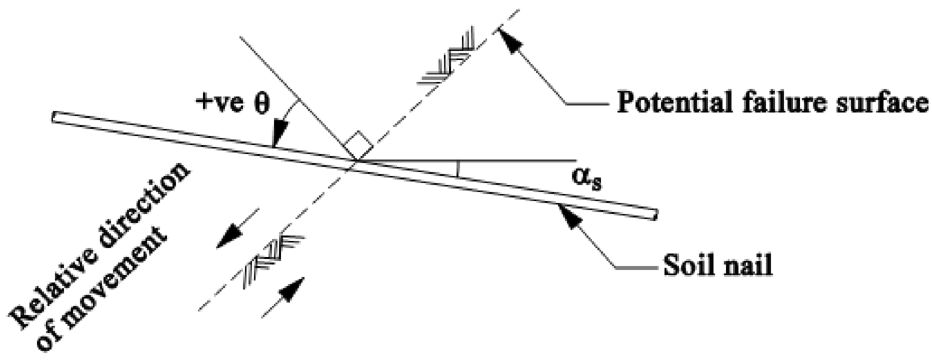


Figure 2. 7: Two-zone model for Model of Soil-nailed System. (Source: Geoguide 7)

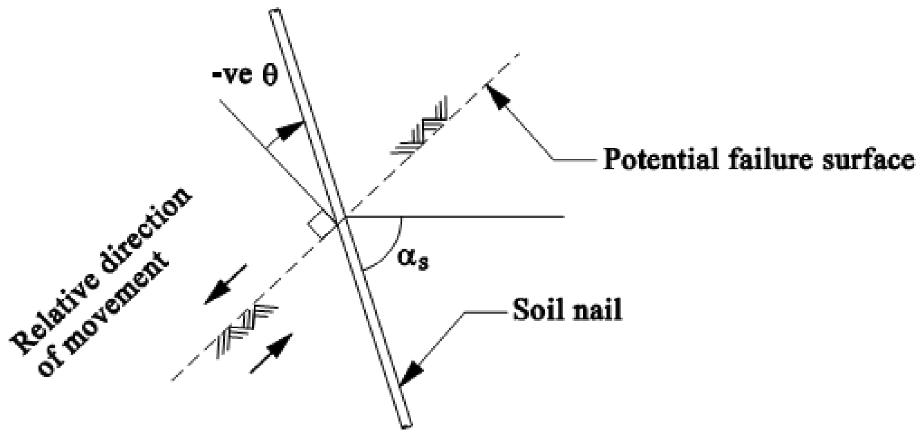
2.8.6 Nail ground interaction

Through the interaction between the grounds, the soil nails, the soil-nail heads and the slope facing forces are developed in the active zone. There are two basic mechanisms of nail-ground interaction, namely (i) nail-ground friction that leads to the development of axial tension or compression in the soil nails, and (ii) the soil bearing stress on the soil nails and the nail-ground friction on the sides of soil nails that lead to development of shear and bending moments in the soil nails.

Tension forces are developed on the soil nails if the soil nails are aligned close to the direction of the maximum tensile strain of soil. This is developed through the mechanism of nail-ground friction. Shear stresses and bending moments are developed in the soil nails through the mechanism of soil bearing stresses as well as the nail-ground friction at the sides of soil nails (Geoguide2007). In a homogeneous and isotropic soil mass, the mobilization of shear stresses and bending moments of soil nails are small under service load conditions (Jewell & Pedley, 1992). Compressive forces will be developed in the soil nails, if the soil nails are aligned in the direction of compressive strain in the soil. The development of compressive forces leads to decrease in normal stresses in the soil on the potential failure surfaces and ultimately reduces the shearing resistance of the reinforced soil mass. Inclination of the soil nail in the direction of zero axial strain leads to developments of shear and bending moments on the soil nails. However, due to relatively slender dimensions of the soil nails, these reinforcing contributions are limited by the small flexural strength, and they are usually negligible (Jewell & Pedley, 1992; FHWA, 1998).The above principles explain the effect of the soil nail inclination on the mobilization of forces in soil nails. With increase in inclination of the soil nail to the horizontal, the effectiveness of a soil nail in mobilization of tensile forces decrease. However, due to relatively slender dimensions of the soil nails, these reinforcing contributions are limited by the small flexural strength, and they are usually negligible (Jewell & Pedley, 1992; FHWA, 1998).The effectiveness of the soil nails will be reduced significantly, if the soil nails are steeply inclined as some of the soil nails may be in compression. Therefore steeply inclined soil nails should be used with caution. The following figure shows the effect of reinforcement orientation on the shear strength of reinforced soil, where α_s the inclination of soil nail to the horizontal is and θ is the orientation of soil nail with respect to the potential failure surfaces.



(a) Mobilisation of Tensile Force in a Soil Nail



(b) Mobilisation of Compressive Force in a Soil Nail

Figure 2. 8: Effect of Soil-nail inclination on the mobilization of Forces in a soil nails (Source : Geoguide 7)

In response to the ground deformation in the active zone compressive and shear strains are developed in the soil beneath a soil-nail head. The head-ground interaction will be dominantly in the form of a bearing mechanism if the resultant strain is close to the direction perpendicular to the base of soil-nail head. Combined effect of bearing and sliding is seen if the resultant strain is in a direction that deviates significantly from the normal to the base of the soil-nail head. In this case, the effectiveness of the soil-nail head in mobilizing tensile force in the soil nail will be reduced.

The soil nails and soil-nail heads/facing act together to tie active zone to the passive zone. The bearing mechanism between soil-nail heads and ground .gives rise to tensile loads at the heads of soil nails and these tensile loads are taken by the soil-nail reinforcement. With increase in size of soil-nail head or the increase in coverage of facing, the tensile forces in a soil nail increase.

The zone behind the potential failure i.e. passive zone contains the distal end of the soil nails with sufficient bond length to prevent the soil nails from being pulled out. With development of ground deformation in the active zone, pullout forces are induced in the soil nails in the passive zone. Through the mobilization of bond stresses between the ground and the cement grout sleeve, and between the cement grout sleeve and the soil-nail reinforcement, the pullout force is transferred between the soil-nail reinforcement and the ground (Geoguide 2007).The distribution of bond stress between the cement grout sleeve and the ground along a soil nail is not uniform. The following figure depicts a schematic distribution of the locus of maximum tensile forces of soil nails and the potential failure surface of a slope. The point of maximum tension in a soil nail is close to, but does not necessary occur at the point of maximum soil shear strain, i.e., the potential failure surface of a slope (FHWA, 2003). The nail-ground interaction is complex, and the forces developed in the soil nails are influenced by many factors. These factors include the mechanical properties of the soil nails (i.e., tensile strength, shear strength and bending capacity), the inclination and orientation of the soil nails, the shear strength of the ground, the relative stiffness of the soil nails and the ground, the friction between the soil nails and the ground, the size of soil-nail heads and the nature of the slope facing and designer should take into account the interaction between soil nails and the ground in the design of a soil-nailed system.

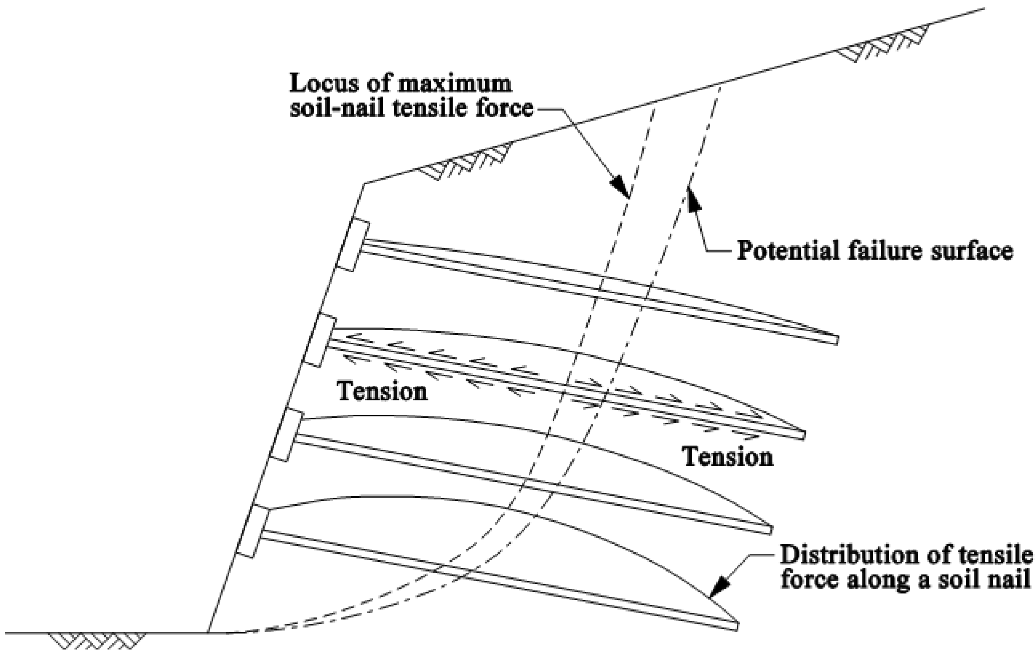


Figure 2. 9: Schematic distribution of tensile forces along the soil nails

(Source: Geoguide 7)

2.8.4 Slope Stability with Nail

The stability of nail slope is analyzed with modification of equilibrium equations incorporating the equilibrium equation .The allowable factor of safety is considered as 1.5.Only tension is considered in the present analysis as bending and shear forces of soil nail has a lesser effect in stabilization of nailed slope (Jewell and Pedeley, 1992).Nail tension (T_j) is calculated based on the available pull-out resistance of soil nail. The available pull-out resistance is equal to either bond strength between the soil and reinforcement to be obtained on the site from pull-out test or the tensile strength of the reinforcement, which is lesser.

2.9 Modes of Failure of Nailed Slope

The ability of the soil nail wall to act as a coherent gravity mass is a function of the vertical and horizontal spacing of the nails, the long-term allowable strength of the nails, the stress transfer between the reinforced soil nail and the nail, the connection strength between the nail and the facing, and flexural strength of the facing. There are different failure modes based on above mentioned parameters. Broadly these can be classified into three categories as external, internal and facing failure modes.

2.9.1 External Failure

External failure refers to the development of potential failure surfaces essentially outside the soil-nailed ground mass. The failure can be in the form of sliding, rotation, bearing, or other forms of loss of overall stability.

2.9.2 Internal Failure

Internal stability of the soil nail wall is concerned with the ability of the nails carrying tensile forces and transferring them by friction, friction and adhesion, or friction and bearing (RDSO,2010).During excavation ,while soil nail wall system deforms, it mobilizes bond strength between the grout and the surrounding soil. This bond strength is mobilized progressively along the entire soil nail and surrounding soil, due to which tensile forces are developed. Due to insufficient bond strength or inadequate tensile strength of the nail or slippage of the grout and steel bar interface failure occurs in the load transfer between the soil, the soil and grout. These failures modes are denoted as internal failure modes. Internal failures can occur in the active zone, passive zone, or in both of the two zones of a soil-nailed system.

The most common internal failure mechanisms are listed as follow:

1. Nail pull-out failure: Failure along the soil-grout interface due to insufficient intrinsic bond-strength or insufficient nail length.
2. Tensile failure of the nail: Due to inadequate tensile strength
3. Slippage of the bar grout interface
4. Bending and shear of the nails.

Use of threaded bars and relatively high- strength grout, the potential slippage between nail and grout can be avoided and therefore disregarded. In most current design methods, due to relatively ductile behavior of the mild steel reinforcements and no strength contribution assigned to grout, the shear and bending strength of the soil nails are conservatively disregarded. Generally two mechanism i.e. nail pullout failure and nail tensile failure are consider in most of the analysis.

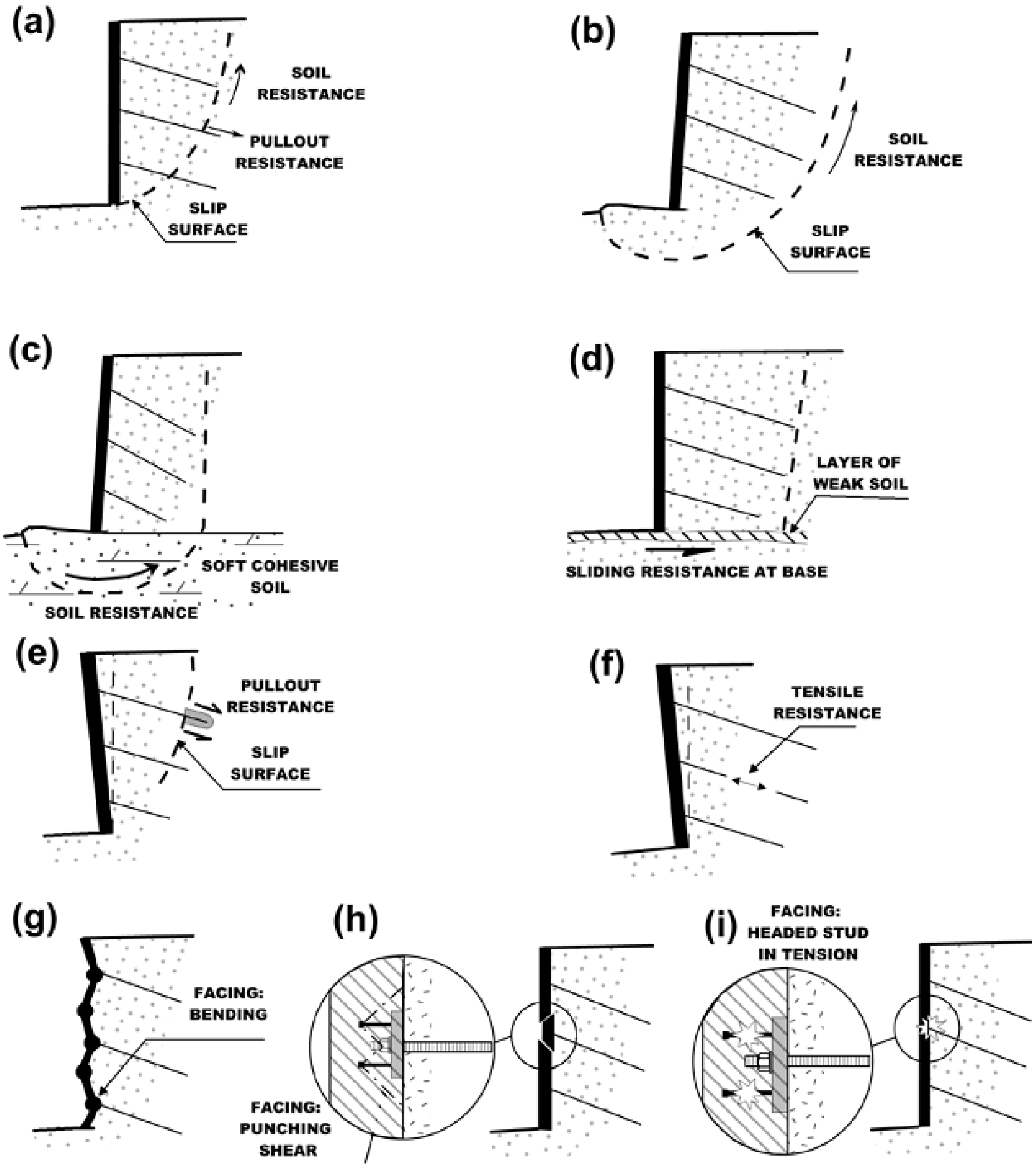


Figure 2. 10: Illustration. Potential limit states in soil nail walls

(a) External stability failure (global stability-slip surface intersecting soil and nails; due to insufficient development of reaction force by nail in failure plane(b) External failure (slip surface not intersecting soil and nails); due to dense nail but insufficient length of nail system(c) External stability(basal heave; due to low bearing capacity of foundation soil (d) geotechnical strength: lateral sliding; due to weak layer of soil between nail wall system and foundation soil (e) geotechnical strength: pullout ; due to insufficient embedment beyond potential slip surface (f)structural strength: nail in tension; due to insufficient tensile capacity of nail (g) facing structural strength: bending; due to low flexural strength of facing (h) facing structural strength: punching shear; (i) facing structural strength: headed stud in tension.(Lazarte et al.,2013)

From FHWA (2015)

Pullout resistance

The axial resistance developed at a soil by mobilization of bond (shear) stresses along its grout soil interface.

Pullout capacity (P_0) = $Q_U*L_P = \pi*q_u*d*l_p$ Equation 2.20

Where,

Q_U= Pullout capacity per unit length

q_u=Ultimate bond strength

d= Diameter of drill hole

l_p= Length of nail beyond failure surface

Tensile resistance (T_R)=$(\pi*d^2*f_y)/4$ Equation 2.21

Where

f_y = Yield strength of nail material

Nail head resistance (N_R) = $K*(q + \gamma*z)*S_H*S_V$ Equation 2.22

Where,

K= coefficient of earth pressure, q = surcharge load, γ = unit weight of soil

Z= height of soil above the nail, S_H, = horizontal spacing and

S_V = vertical spacing

2.10 Parameter affecting Soil Nailing

1. Soil Nailing Spacing

Soil nails are installed in a grid pattern. The horizontal nail spacing (S_H), is often same as vertical nail spacing (S_V) .Nail spacing in both direction generally ranges from 4t to 6 ft and occasionally up to 6.5ft, and is routinely selected at 5ft. The spacing can be checked that S_H*S_V is less than approximately 36 to 42 ft^2.

2. Soil Nail Inclination

Soil nail installed in direction of maximum tensile strain of soil gives higher FOS. Nails are not installed in upward direction as grout flows due to gravity. Practically nails are sub inclined at 5- 10 degree, as in case of horizontal nail there is formation of voids in grout.

3. Nail Length

Length of nail should be adjusted that it intersects the slip surface. Nail length higher than 20m should be avoided.

4. Nail Pattern

A staggered pattern results in more uniform earth- pressure distributions, better soil arching effects, and provides a slightly larger resistance compared to those from a square pattern.

3. METHODOLOGY

3.1 Introduction

Slope stability analysis is an important area in geotechnical engineering. Most textbooks on soil mechanics include several methods of slope stability analysis. A detailed review of equilibrium methods of slope stability analysis is presented by Duncan (Duncan, 1996).The rapid development of computer added software and hardware technology has become boon to solve the complex geotechnical problem and analysis. Slope stability problem can be solved by using various commercially available software packages such as; Geo-studio (Slope/w), Phase2, Slide, Plaxis and others. Sound knowledge of soil mechanics, rock mechanics and geology is essential for software analysis (Geostudio-2019).The accuracy of results obtained from software depend upon the correct input of geotechnical and geometrical parameters by the user.

The limit equilibrium type of analyses has been commonly used by geotechnical engineers for slope stability analysis for many decades. Slope/w software is widely used for slope stability analysis based on limit equilibrium method. The software code slope/w (Geo-slope 2019) allows geotechnical engineers to carry out limit equilibrium slope stability analysis of existing natural slope, unreinforced man-made slopes or slope with soil reinforcement (Geostudio-2019).However analysis and interpretation is a difficult task. Therefore, efforts should be made to collect the field data and the observation of failure patterns in order to understand the failure mechanism, which determines the methods applicable for slope stability analysis. Natural or cut soil and rock slopes are non-isotropic and have heterogeneous properties. The accuracy of model analysis is largely determined by boundary conditions applied. The correct materials and boundary conditions of the particular soil and rock model are the part of solution results (Geo-studio, 2019).The graphical outputs of results by Slope/w depicts the failure plane/slip surfaces and slice forces which ultimately gives the clear idea on the stability condition.

The present study deals with cut slope stability analysis based on limit equilibrium method by using Slope/w software and finite element method by phase2 software.

The stability of cut slope is analyzed both in various anticipated conditions through simplified slope geometry and input parameters.

3.2 Methodology adopted for the present study

A conceptual frame work on slope stability studies was developed through a systematic literature review. Before field different work have been done to acquire the detailed information about the area and to be well prepared for field work. Since there are several studies are carried out in the past by many researchers in the field of natural slope and man-made slope therefore as a part of literature review exhaustive review of previous studies was also undertaken. Thus, to meet our objectives of present study following systematic methodology was followed;

3.3 Collection and Reviews of available documents and Literature.

All the available previous study reports, data/ including maps drawings and related to study area are collected from different sources. All these reports, documents, data and information are studied and analyzed in depth in the context of the objectives of the study .the initial collection and review of information provided valuable information to plan the detailed study in field investigation and data analysis.

3.4 Geological and Engineering Geological Studies

The geological and engineering geological studies were carried out during the pre-field stage. At this stage, the collected maps were thoroughly studied in order to identify the geological setup of the study are. The major geological boundaries and tectonic set up together with lineaments passing through the area will be identified.

3.5 Field observation

Efforts were made to identify the different instability manifestation features present on the slope and to identify and collect data for the probable causative factors for the slope instability during the field work. The various work carried out in field observations were:

3.5.1 Field verification Preliminary Results

The chain-age Section of 62+300 to 68+300 and 68+700 of Kanti Lokpath near Bhatte Danda of Lalitpur district was thoroughly visited and focus in the area of concerned. The verification of the preliminary information obtained from different sources was carried out and necessary correction was made.

3.5.2. Geological and Engineering Geological Data Collection

During the period of field visit, engineering geological investigation was carried out. These works include collection of detailed geological information in and around the concerned area. The geological field work was performed to cover an appreciable amount of surrounding area such that the regional geological picture should be clear. From the field various information related to the spot site profile, topography, surface condition, soil deposits, cut slope condition, rock/soil distribution were collected. For the determination of Index properties and engineering properties of the soil sample were collected from road-cut slope at section of 62+300 to 68+300 and 68+700.

3.6 Lab test and Calculation

To obtained the index and engineering properties required various lab test were performed which are listed below.

- Sieve analysis
- Direct shear test
- Determination of unit weight
- Determination of specific gravity

These data and curve are used in model calculation in Geo studio 2019 and Phase2 to define the soil parameters. Sieve analysis was performed according to IS: 1498-1970 with using standard sieve of various sizes ranging from 80 mm to 75 microns. The soil shear strength parameters (cohesion and internal frictional angle) were determined from direct shear test apparatus after plotting Mohr's circle at failure condition. Standard test procedure was followed to determine index properties like unit weight, specific gravity, liquid limit and plastic limit of soil sample.

3.7 Software used for slope stability analysis

There is several computer based geotechnical software used in the slope stability analysis. Some based software based on limit equilibrium approach of analysis and some software based on finite element approach of analysis. Among them Slope/W and phase2 is commonly used software by geotechnical engineers for the stability analysis of natural and artificial slope. Slope/W based on limit equilibrium approach of stability analysis, which is developed by Geo-slope International Canada and

Phase2 is based on finite element method developed by Rockscience Inc.,Toronto, Canada.

3.7.1 General description about Slope/W software

Slope/W is software based on limit equilibrium principle for stability analysis of earth structure. It include different types of methods like Ordinary method of slice (OS), Bishop (BS),Janbu Simplified (JS),Spencer (SP),Morgenstern-Price (M-P), Corps of Engineers, Sharma methods. For present study only Morgenstern-Price (M-P) methods were only used for FOS computations. The results of slope/W can be obtained as both visuals and numbers. The very important advantage of the slope/w analysis is that it allows handling of all possible slides in a same model with corresponding factor of safety.

3.7.1.1 Procedure of slope stability analysis by Slope/W

The general procedures followed for slope stability analysis by using slope/w software are:

- Geometrical modeling (2-D) representation of selected slope through site observation and topographic review.
- Factor of safety was computed by Slope/W based on limit equilibrium principles by different methods such as: Bishop simplified Method (BS), Janbu simplified Method (JSM), Spencer Method and Morgenstern-Price Method (M-P).
- Minimum factor of safety was determined by considering several slip surfaces through the techniques provided in the software. Slip surface can be defined in terms of grid and radius, entry and exit, fully specified and auto search etc.
- Auto search option provided in the software was used to locate critical slip surface and corresponding factor of safety.
- Relative interpretation of FOS under various anticipated conditions for critical slope section was prepared in tabular and graphical format.
- Critical cases were identified based on the minimum factor of safety, specified as less than then acceptable limit.
- Critical cases were reanalyzed after adopting Soil nail in order to increase the factor of safety to acceptable limit.

3.7.1.2 Numerical model of cut slope in Slope/W

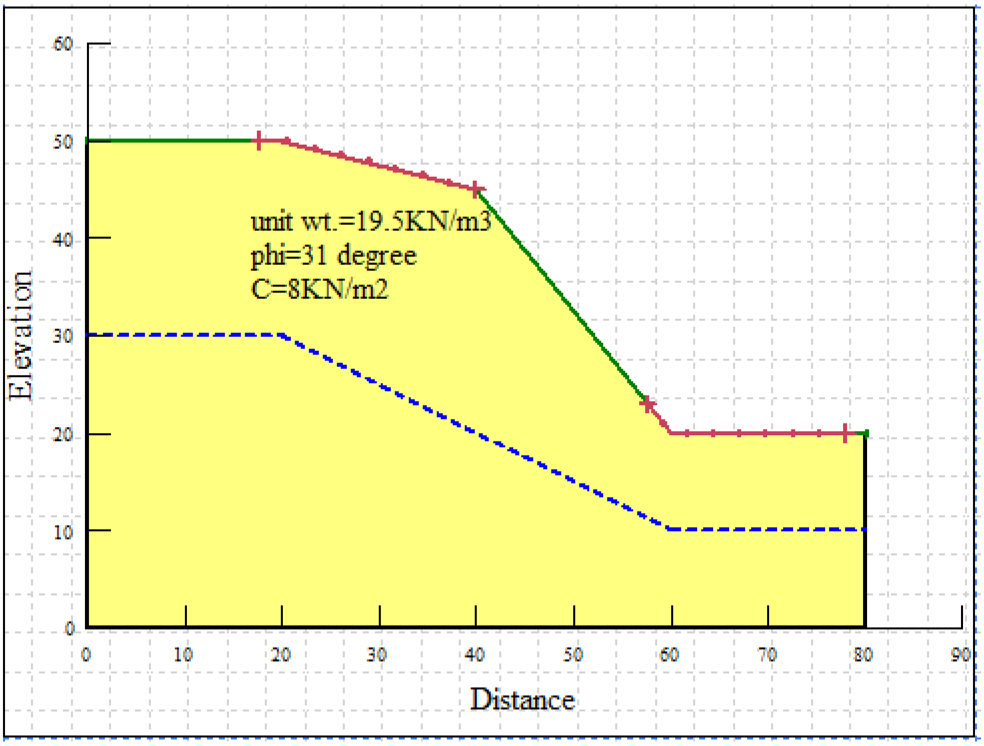


Figure 3. 1: Schematic diagram of Problem statement at chainage 62+300

An 25 m high slope made up of c- φ soil and overlying a same foundation material has been analyzed for instability due to wide range of cohesion (c) and angle of internal friction (φ) using Slope/W of Geo-studio 2019 utilizing limit equilibrium methods such as Morgenstern-Price method.

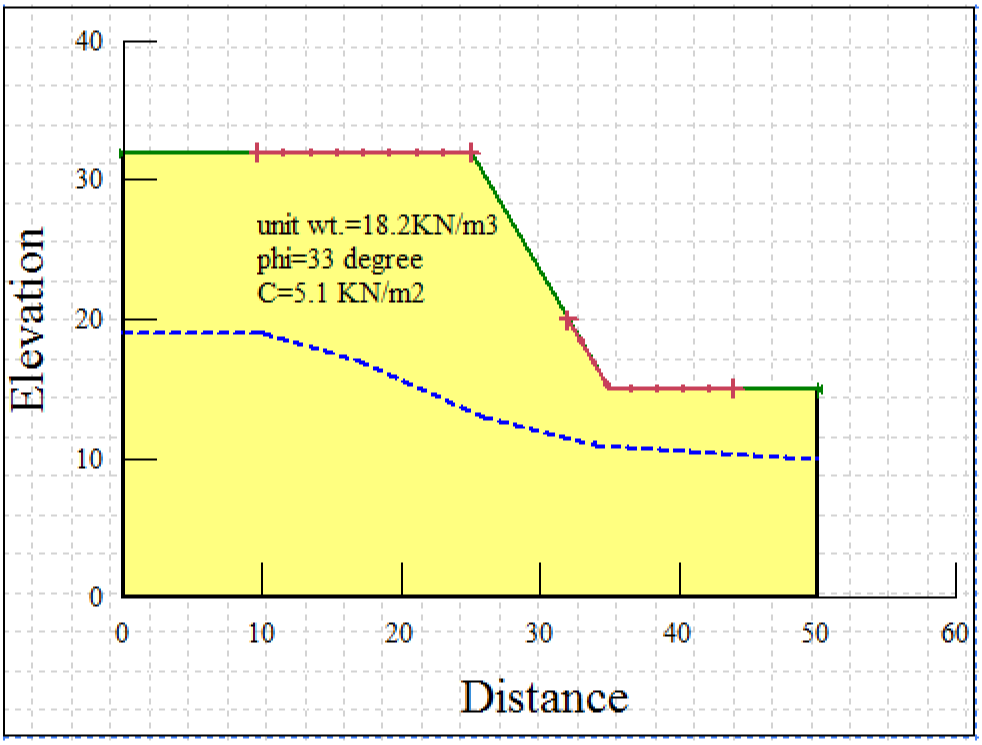


Figure 3. 2: Schematic diagram of Problem statement at chainage 68+300

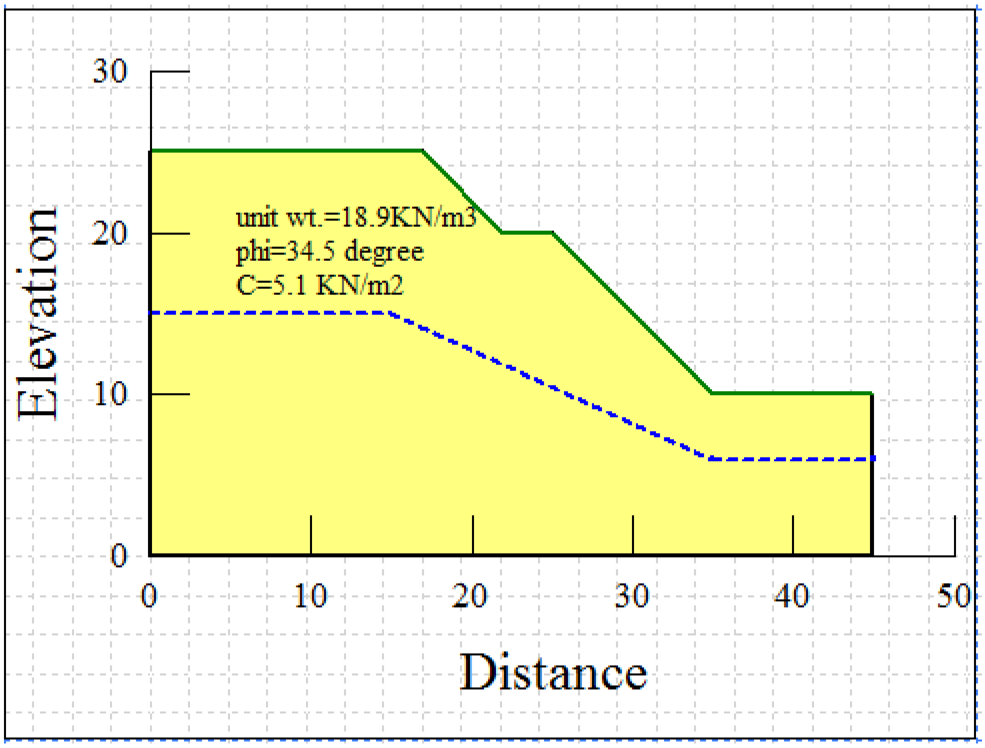


Figure 3. 3: Schematic diagram of Problem statement at chainage 68+700

And also 17m and 15m high slope made up of c- φ soil and overlying a same foundation material has been analyzed for instability due to wide range of cohesion (c) and angle of internal friction (φ) using Slope/W of Geo-studio 2019 utilizing limit equilibrium methods such as Morgenstern-Price. Determining the position of the critical slip surface with the lowest FOS remains one of the key issues in a stability analysis. There are many different ways for defining the positions of trial slip surfaces in Slope/W, namely grid and radius method, entry and exit method, fully specified slip surfaces and auto search of critical slip surface. In the present analysis, auto search option was used to locate critical slip surface. The most realistic position of the critical slip surface is obtained when effective strength parameters are used in the analysis. Effective strength parameters, however, are only meaningful when they are used in conjunction with pore-water pressures. There are different ways to specify the pore pressure conditions in Slope/W 2019, namely single piezometric line, multiple piezometric lines, pore-water pressure head with spatial function. In the present analysis, single piezometric line method has been used. The variation of water table has been considered by specifying height and inclination of the piezometric line. The Mohr-Coulomb's material model was used while assigning the strength parameters

3.8 Preparation of input parameter for slope stability analysis

Shear strength parameters; cohesion and angle of shearing resistance are one of the important inputs for any slope stability analysis. The critical slope section of the present study area is mainly composed of c-φ soil. From the critical slope section samples were collected for laboratory test. The shear strength parameters obtained from laboratory were utilized material model definition in the analysis. Computations were performed using slope/W software and were based on the strength parameters on the determination of the factor of safety.

For present study attempts were made to estimate the shear strength parameters through direct shear test which was conducted in the lab. The laboratory results are presented in Table 4.1

4 RESULT AND DISCUSSION

4.1 Geotechnical Findings

The three numbers of soil samples were collected at the different location of the cut slope and laboratory testing was carried out to evaluate the index and the strength properties of cut slope materials. The test result is summarized in the following table and related graphs are presented in annex.

Table 4. 1**:** Geotechnical properties of cut slope materials at different chainage section

Chainage	Material	Unit weight (kN/m^3)	Cohesion (kN/m^2)	Internal Friction angle (^{0})
62+300	c- φ soil	19.5	8	31
38+300	c- φ soil	18.2	6	33
68+700	c- φ soil	18.9	5.1	34.5

4.2 Static analysis of cut slope using Slope/W and Phase2 at chainage 62+300

Static stability analysis was performed by neglecting the effect of earthquake. Cut slope model was prepared in suitable scale and geotechnical parameters are assigned according to laboratory test. Slope stability analysis was performed under static dry condition .In this case ground water table level is considered to be much below the influence zone i.e. the ground water level is believed below the possible failure surfaces.

The FOS was computed by Slope/W and Phase2 for static dry condition results obtained are presented in Table 4.2.Persual of table 4.2 clearly indicates that the FOS values by each method is less than 1. Thus, it may be concluded that the slope section is unstable for static dry condition and Slope stabilization measures are necessary to stabilize the cut slope. The critical slip surface and possible slip surfaces are shown in figure 4.1.The figure clearly indicates that the shape of slip surface is circular.

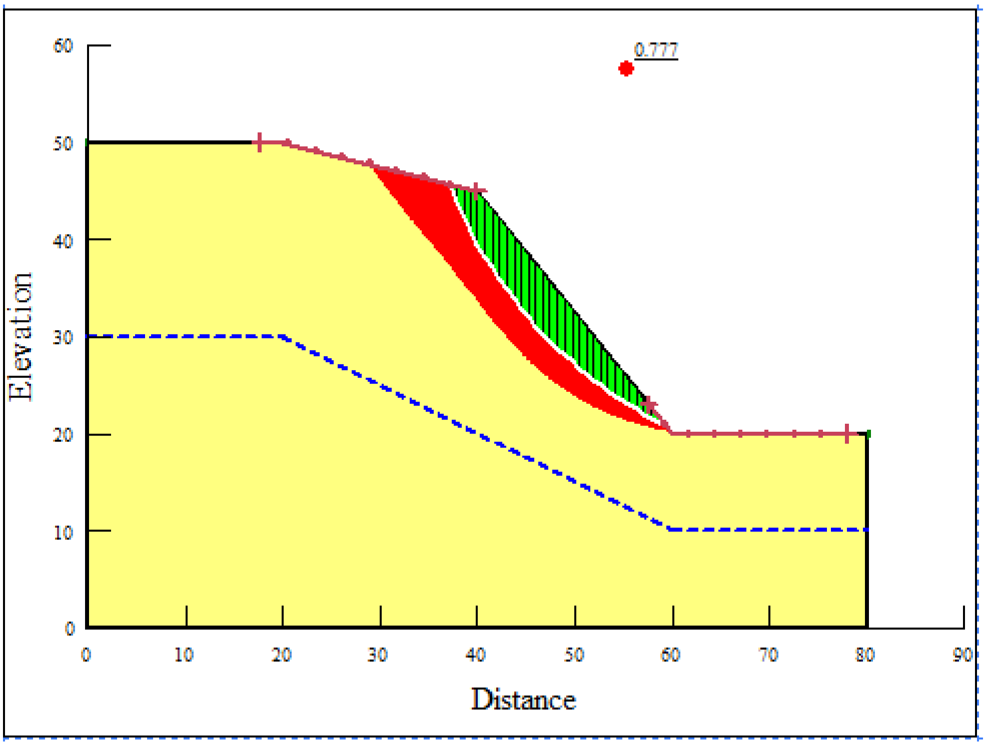


Figure 4. 1: The Critical Slip surface and possible slips surfaces during static condition at 62+300 slope section by slope/W

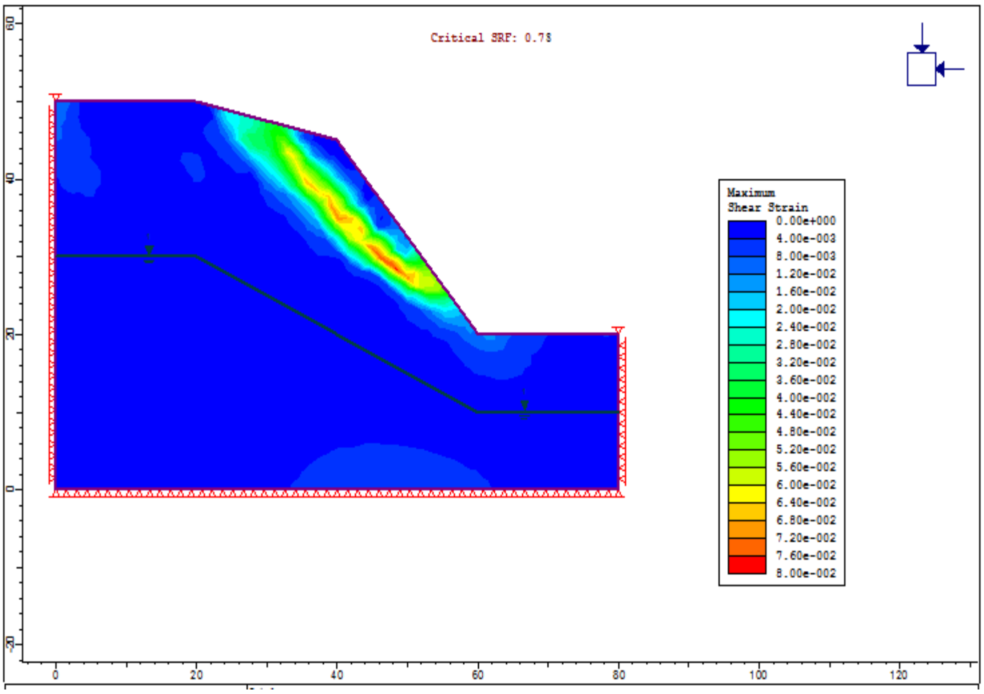


Figure 4. 2: Total displacement diagram during static condition at 62+300 slope section by Phase2 with critical SRF 0.78

Table 4. 2:Soil parameters and FOS for Static analysis in dry condition (62+300)

Unit Weight(kN/m^3)	Cohesion(kPa)	Friction angle(^{0})	FOS	
			Slope/w	Phase2
19.5	8	31	0.777	0.78

4.2.1 Factor of safety determination for different cases of pore water pressure.

Pore water pressure is important factor to be considered in slope stability analysis. Porewater pressure directly affects the shear strength parameters of soil.

Table 4. 3:Variation of FoS with depth of water table from top of the slope (62+300).

Soil Parameters

unit weight (γ)= 19.5 kN/m^3,cohesion(c)= 8 kPa ,friction angle (φ)= 31^0

Case	Depth of water table from top(m)	FOS by M.P(Slope/W)
1	1	0.217
2	2	0.462
3	4	0.54
4	6	0.656
5	8	0.749
6	10	0.77

The variation of factor of safety with variation water table is show in figure 4.3.The factor of safety increased with increased in depth water table from top of cut slope. Almost a linear relationship exists between two parameters.

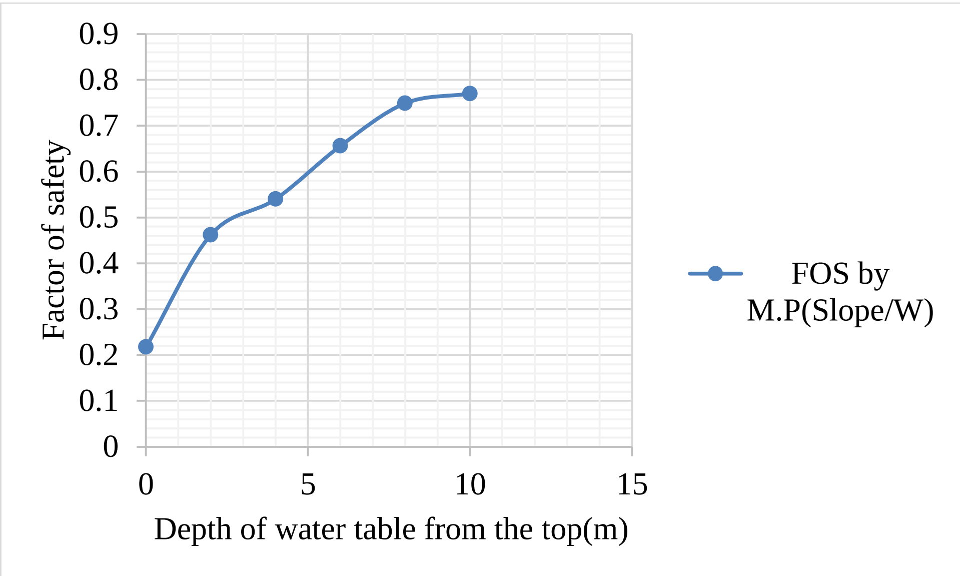


Figure 4. 3: Variation of factor of safety with the depth of water table from top(62+300).

4.2.2 Influence of Cohesion of cut slope material in stability of slope.

Cohesion of cut slope material is other import parameters for stability of cut slope. Effect of variation of cohesion of cut slope material is analyzed by varying cohesion of slope material from 0 kPa to 25 kPa and keeping other parameters same as static dry condition.

Table 4. 4:Variation of Fos with change in cohesion of the slope material (62+300)

Soil Parameters

unit weight (γ)= 19.5 kN/m^3,cohesion(c)= 0-25 kPa ,friction angle (φ)= 31^0

Case	Cohesion of Soil (kPa)	FOS by M-P(SlopeW)
1	0	0.5
2	5	0.705
3	10	0.831
4	15	0.926
5	20	1.018
6	25	1.1

The variation of factor of safety with variation of cohesion of the slope materials are show in figure 4.4.The factor of safety increased with increased in cohesive nature of the slope materials. For variation of cohesion from 0 kPa to 25 kPa of the slope materials the factor of safety increased from 0.5 to 1.1 for Morgenstern and Price method.

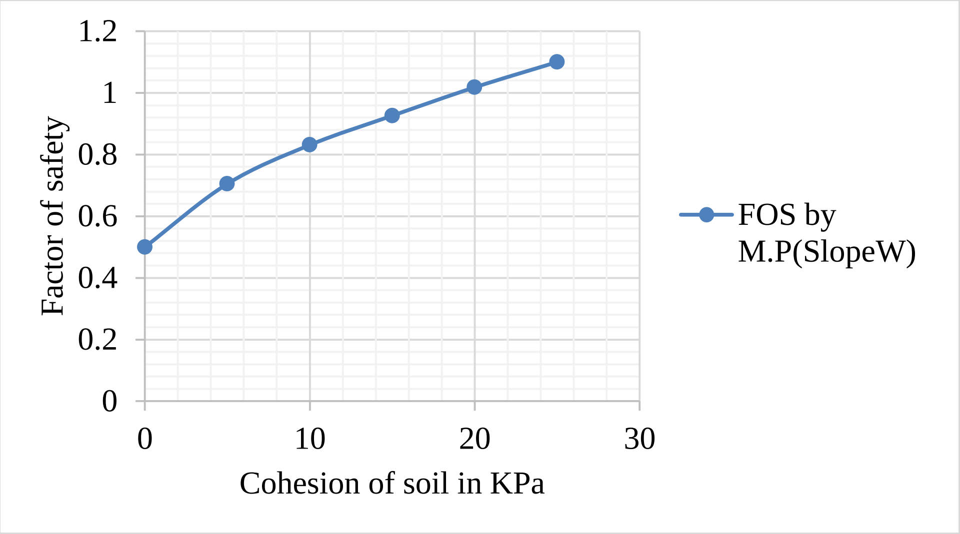


Figure 4. 4: Relationship between Cohesion and Factor of safety (62+300).

4.2.3 Influence of Internal Frictional angle of slope material Factor of safety

Table 4. 5: Influence of Internal Frictional angle of slope material Factor of safety (62+300)

Soil Parameters

unit weight (γ)= 19.5 kN/m^3,cohesion(c)= 8kPa ,friction angle (φ)= 10^0-35^0

Case	Friction Angle	FOS by M.P
1	10	0.33
2	15	0.438
3	20	0.540
4	25	0.646
5	30	0.754
6	35	0.873

The variation of factor of safety with variation of internal frictional angle of the slope materials are show in figure 4.5. The factor of safety increased with increased in internal friction angle of the slope materials for all methods of analysis. For variation of friction angle from 10^0 to 35^0 of the slope materials the factor of safety increased from 0.33 to 0.873.for Morgenstern and Price method.

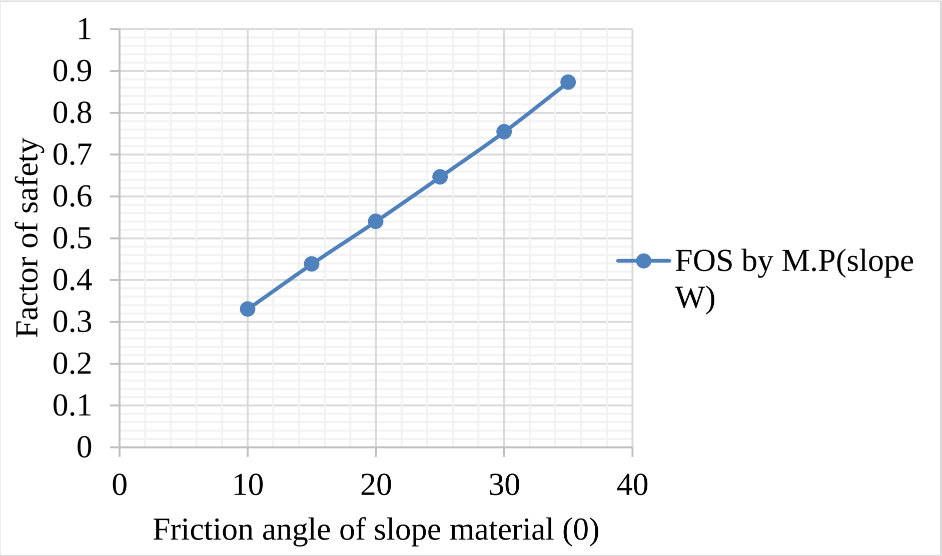


Figure 4. 5:Relation between Internal friction angle and factor of safety (62+300)

4.2.4 Influence of Unit weight of slope material on Factor of safety

Table 4. 6: Influence of unit wt. of slope material Factor of safety(62+300)

Soil Parameters

unit weight (γ)= (10-25) kN/m^3,cohesion(c)= 8kPa ,friction angle (φ)= 31^0

Case	Unit weight(kN/m^3)	FOS by M.P(slope W)
1	10	0.973
2	15	0.834
3	20	0.772
4	25	0.734

The variation of factor of safety with variation of unit weight of the slope materials are show in figure 4.6.

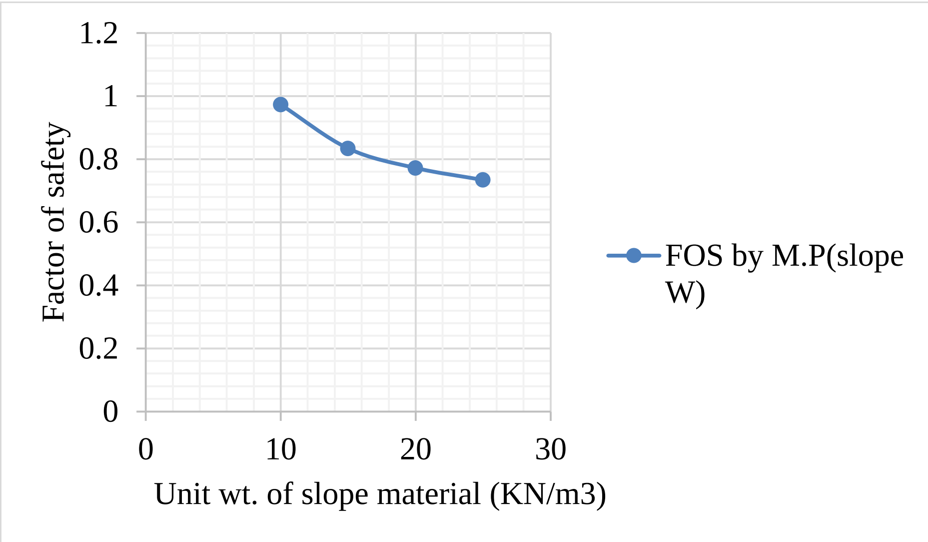


Figure 4. 6: Influence of unit weight of slope material on Factor of safety (62+300)

4.3 Static analysis of cut slope using Slope/W and Phase2 at chainage 68+300

The FOS was computed by Slope/W and Phase2 for static dry condition results obtained are presented in Table 4.7.Persual of table 4.7 clearly indicates that the FOS values by each method is less than 1. Thus, it may be concluded that the slope section is unstable for static dry condition and Slope stabilization measures are necessary to stabilize the cut slope. The critical slip surface with safety map are shown in figure 4.7.The figure clearly indicates that the shape of slip surface is circular

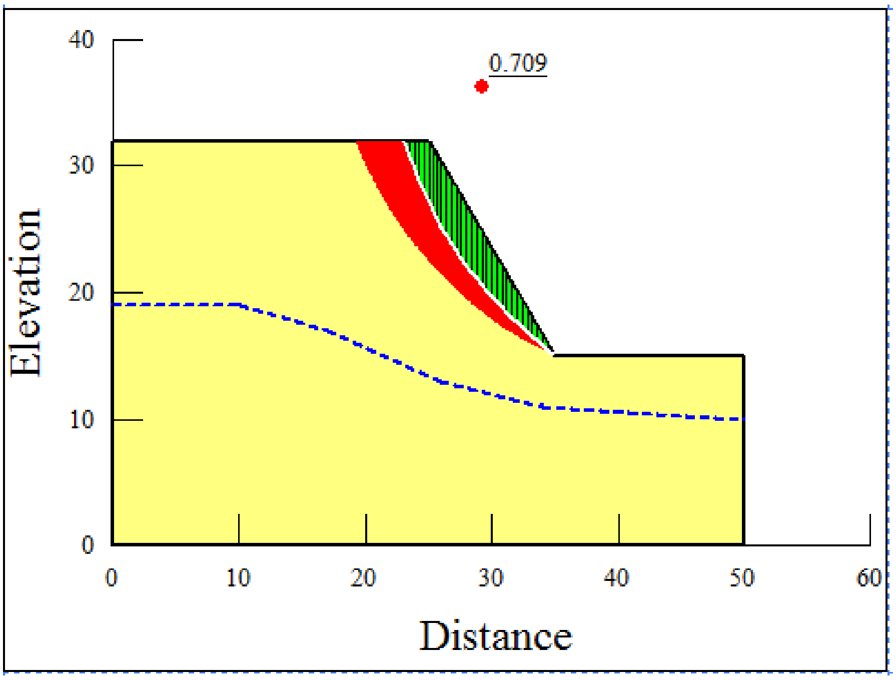


Figure 4. 7:The Critical Slip surface and possible slips surfaces during static condition at 68+300 slope section by slope/W

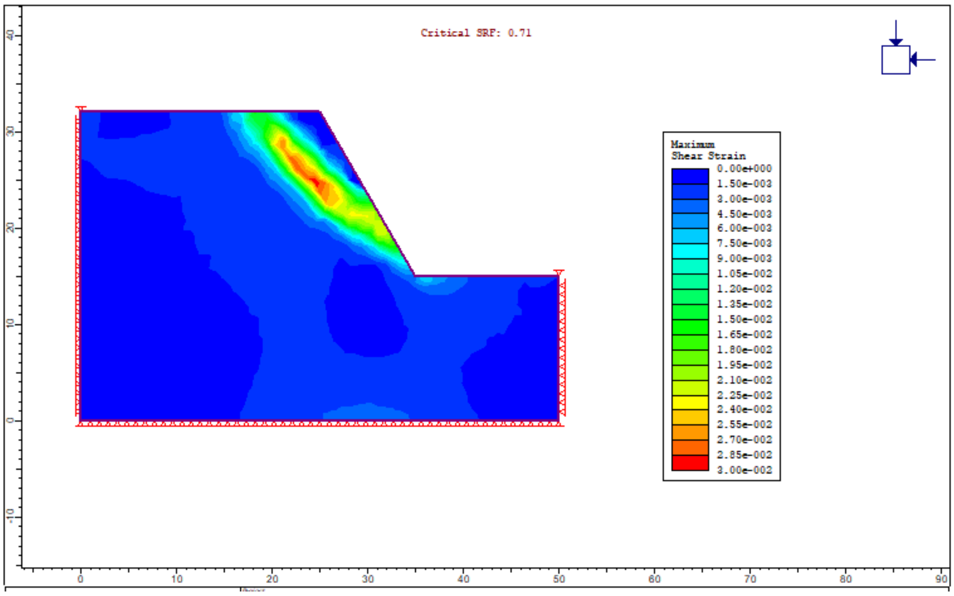


Figure 4. 8:Maximum shear strain diagram during static condition at 68+300 slope section by Phase2 with critical SRF 0.71

Table 4. 7:Soil parameters and FOS for Static analysis in dry condition (68+300)

Unit Weight(kN/m^3)	Cohesion(kPa)	Friction angle(^{0})	FOS	
			Slope/w	Phase2
18.2	6	33	0.709	0.71

4.3.1 Factor of safety determination for different cases of pore water pressure.

Pore water pressure is important factor to be considered in slope stability analysis. Porewater pressure directly affects the shear strength parameters of soil.

Table 4. 8: Variation of FoS with depth of water table from top of the slope(68+300).

Soil Parameters

unit weight (γ)= 18.2 kN/m^3,cohesion(c)= 6 kPa ,friction angle (φ)= 33^0

Case	Depth of water table from top(m)	FOS by M.P(Slope/W)
1	0	0.121
2	2	0.341
3	4	0.532
4	6	0.676
5	8	0.709
6	10	0.709

The variation of factor of safety with variation water table is show in figure 4.9.The factor of safety increased with increased in depth water table from top of cut slope for all methods of analysis. Almost a linear relationship exists between two parameters.

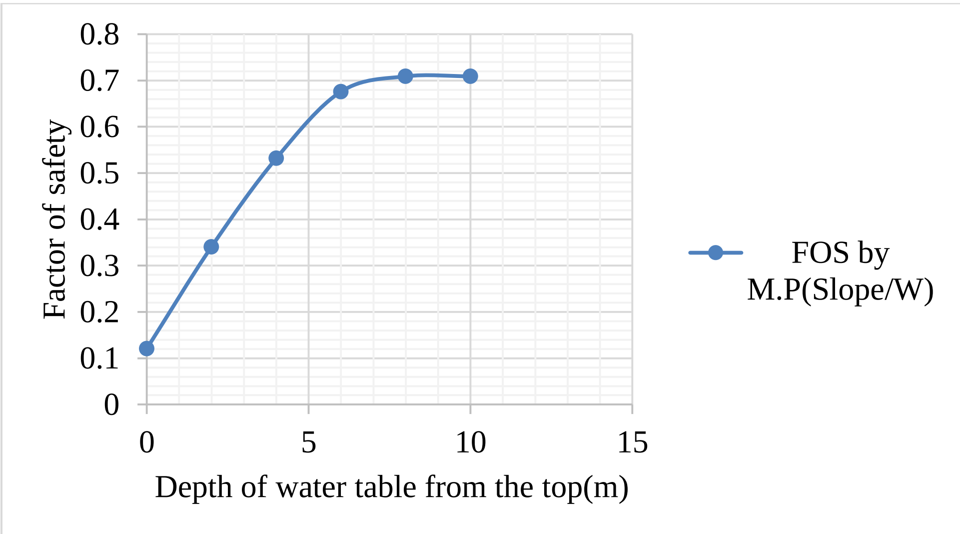


Figure 4. 9: Variation of factor of safety with the depth of water table from top(68+300).

4.3.2 Influence of Cohesion of cut slope material in stability of slope.

Cohesion of cut slope material is other import parameters for stability of cut slope. Effect of variation of cohesion of cut slope material is analyzed by varying cohesion of slope material from 0 kPa to 25 kPa and keeping other parameters same as static dry condition.

Table 4. 9:Variation of FoS with change in cohesion of the slope material(68+300)

Soil Parameters

unit weight (γ)= 18.2 kN/m^3,cohesion(c)= 0-25 kPa ,friction angle (φ)= 33^0

Case	Cohesion of Soil (kPa)	FOS by M-P(SlopeW)
1	0	0.448
2	5	0.672
3	10	0.863
4	15	0.970
5	20	1.093
6	25	1.205

The variation of factor of safety with variation of cohesion of the slope materials are show in figure4.10.The factor of safety increased with increased in cohesive nature of the slope materials for all methods of analysis. For variation of cohesion from 0 kPa to 25 kPa of the slope materials the factor of safety increased from 0.448 to 1.205 for Morgenstern and Price method.

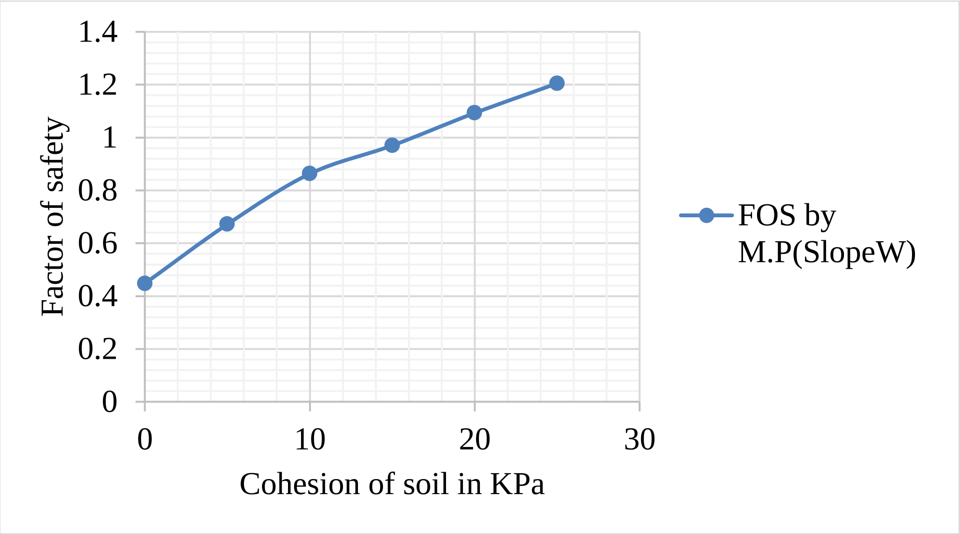


Figure 4. 10: Relationship between Cohesion and Factor of safety (68+300).

4.3.3 Influence of Internal Frictional angle of slope material Factor of safety

Table 4. 10: Influence of Internal Frictional angle of slope material FoS values 68+300)

Soil Parameters

unit weight (γ)= 18.2 kN/m^3,cohesion(c)= 6kPa ,friction angle (φ)= 10^0-35^0

Case	Friction Angle	FOS by M.P
1	10	0.309
2	15	0.396
3	20	0.481
4	25	0.567
5	30	0.654
6	35	0.747

The variation of factor of safety with variation of internal frictional angle of the slope materials are show in figure 4.11.The factor of safety increased with increased in internal friction angle of the slope materials for all methods of analysis. For variation of friction angle from 10^0 to 35^0 of the slope materials the factor of safety increased from 0.309 to 0.747.for Morgenstern and Price method.

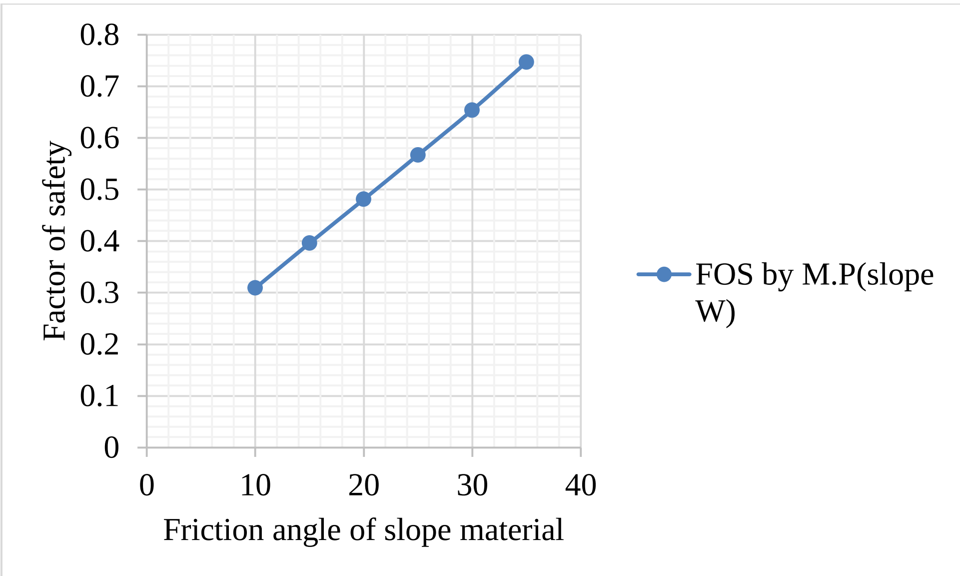


Figure 4. 11:Relation between Internal friction angle and factor of safety (68+300)

4.3.4 Influence of Unit weight of slope material on Factor of safety

Table 4. 11: Influence unit wt. of slope material FoS values (68+300)

Soil Parameters

unit weight (γ)= (10-25) kN/m^3,cohesion(c)= 6kPa ,friction angle (φ)= 33^0

Case	Unit weight(kN/m^3)	FOS by M.P(slopeW)
1	10	0.863
2	15	0.755
3	20	0.689
4	25	0.649

The variation of factor of safety with variation of unit weight of the slope materials are show in figure 4.12.

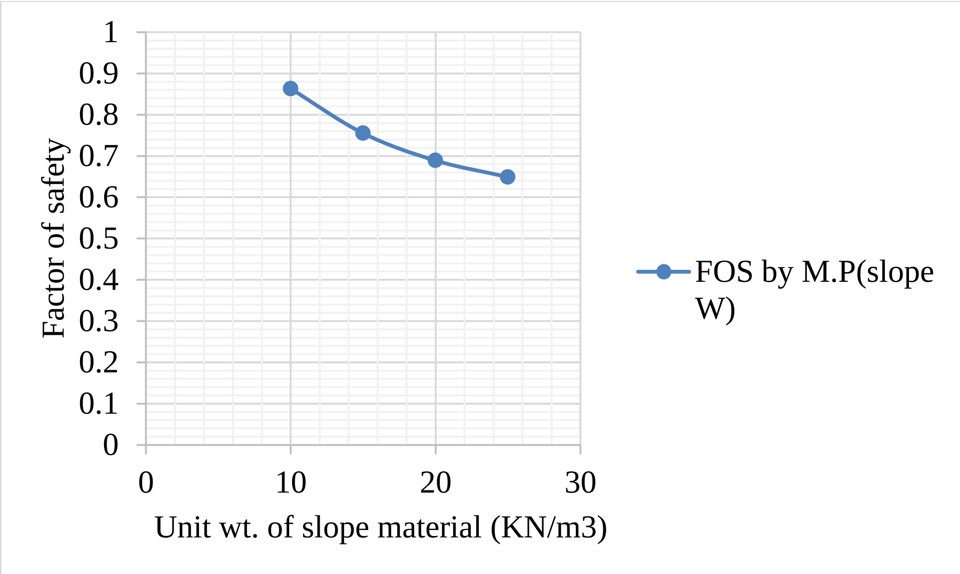


Figure 4. 12: Influence of unit weight of slope material on Factor of safety (68+300)

4.4 Static analysis of cut slope using Slope/W and Phase2 at chainage 68+700

The FOS was computed by Slope/W and Phase2 for static dry condition results obtained are presented in Table 4.12.Persual of table 4.12 clearly indicates that the FOS values by each method is more than 1 in static dry condition. But when there is increase in ground water table it willbe unstable so that the stabilization measures are necessary to stabilize the cut slope. The critical slip surface with safety map are shown in figure 4.13.

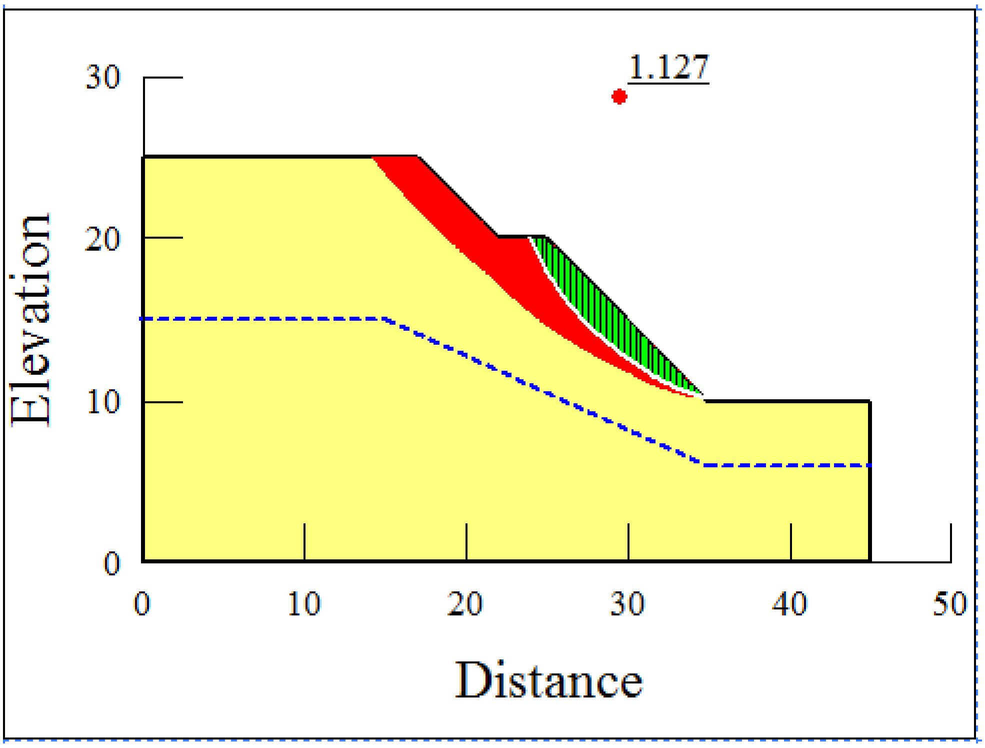


Figure 4. 13: The Critical Slip surface and possible slips surfaces during static condition at 68+700 slope section by slope/W

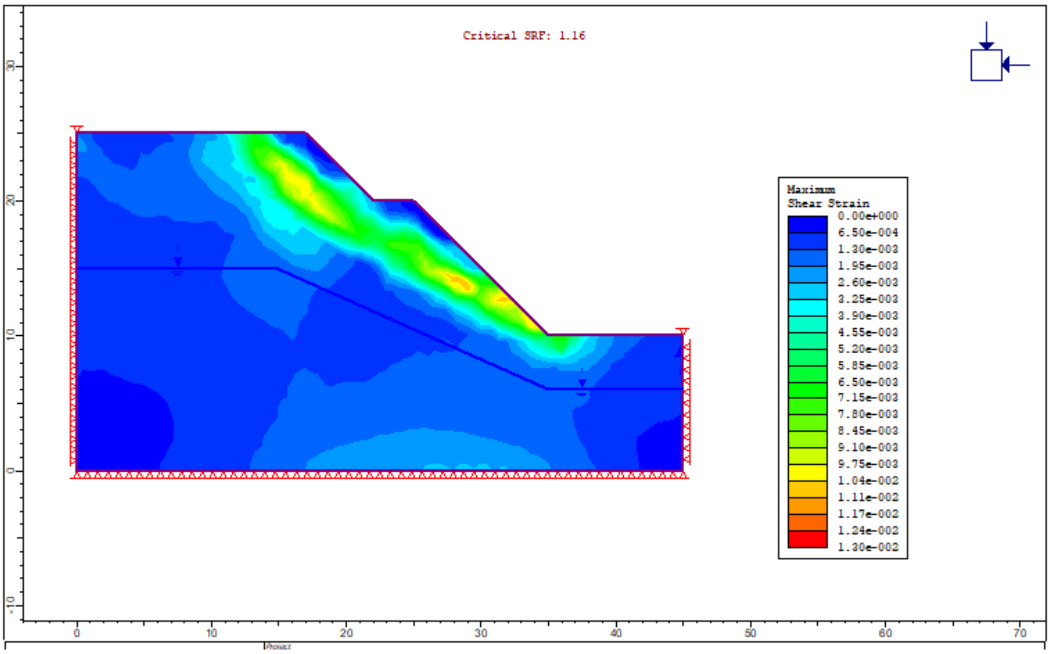


Figure 4. 14: Maximum shear strain diagram during static condition at 68+700 slope section by Phase2 with critical SRF 1.16

Table 4. 12:Soil parameters and FOS for Static analysis in dry condition (68+700)

Unit Weight(kN/m^3)	Cohesion(kPa)	Friction angle(^{0})	FOS	
			Slope/w	Phase2
18.9	5.1	34.5	1.127	1.16

4.4.1 Factor of safety determination for different cases of pore water pressure.

Pore water pressure is important factor to be considered in slope stability analysis. Porewater pressure directly affects the shear strength parameters of soil.

Table 4. 13: Variation of FoS with depth of water table of the slope (68+700).

Soil Parameters

unit weight (γ)= 18.9 kN/m^3,cohesion(c)= 5.1 kPa ,friction angle (φ)= 34.5^0

Case	Depth of water table from top(m)	FOS by M.P(Slope/W)
1	0	0.443
2	2	0.835
3	4	1.11
4	6	1.127
5	8	1.127
6	10	1.127

The variation of factor of safety with variation water table is show in figure 4.15. The factor of safety increased with increased in depth water table from top of cut slope for all methods of analysis. Almost a linear relationship exists between two parameters.

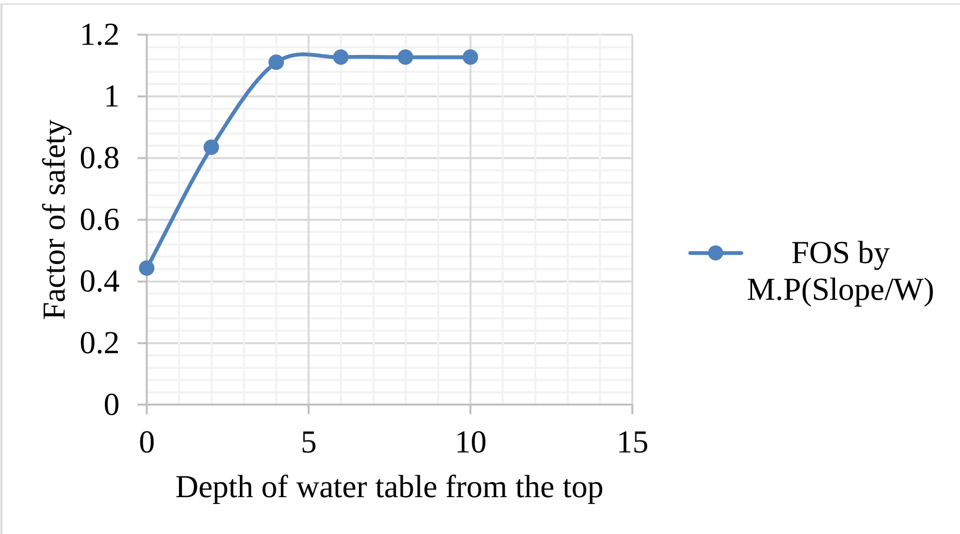


Figure 4. 15: Variation of factor of safety with the depth of water table from top (68+700).

4.4.2 Influence of Cohesion of cut slope material in stability of slope.

Cohesion of cut slope material is other import parameters for stability of cut slope. Effect of variation of cohesion of cut slope material is analyzed by varying cohesion of slope material from 0 kPa to 25 kPa and keeping other parameters same as static dry condition.

Table 4. 14:Variation of FoS values with change in cohesion of the slope material (68+700)

Soil Parameters

unit weight (γ)= 18.9 kN/m^3,cohesion(c)= 0-25 kPa ,friction angle (φ)= 34.5^0

Case	Cohesion of Soil (kPa)	FOS by M-P(SlopeW)
1	0	0.75
2	5	1.121
3	10	1.271
4	15	1.567
5	20	1.718
6	25	1.869

The variation of factor of safety with variation of cohesion of the slope materials are show in figure 4.16.The factor of safety increased with increased in cohesive nature of the slope materials for all methods of analysis. For variation of cohesion from 0 kPa to 25 kPa of the slope materials the factor of safety increased from 0.75 to 1.869 for Morgenstern and Price method.

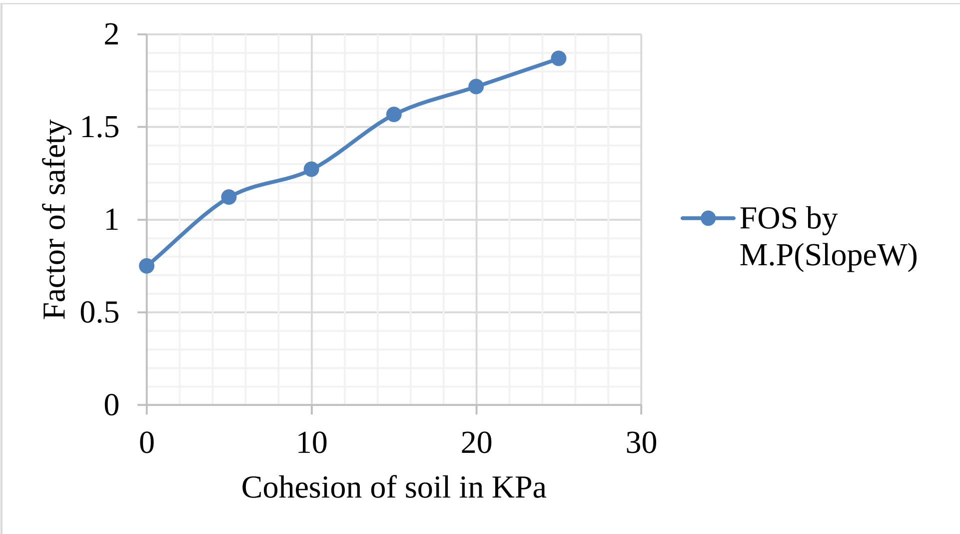


Figure 4. 16: Relationship between Cohesion and Factor of safety (68+700).

4.4.3 Influence of Internal Frictional angle of slope material Factor of safety

Table 4. 15:Influence of Internal Frictional angle of slope material Factor of safety(68+700)

Soil Parameters
unit weight (γ)= 18.9 kN/m^3,cohesion(c)= 5.1kPa ,friction angle (φ)= 10^0-35^0

Case	Friction Angle	FOS by M.P
1	10	0.443
2	15	0.588
3	20	0.728
4	25	0.86
5	30	0.994
6	35	1.142

The variation of factor of safety with variation of internal frictional angle of the slope materials are show in figure 4.17.The factor of safety increased with increased in internal friction angle of the slope materials for all methods of analysis. For variation of friction angle from 10^0 to 35^0 of the slope materials the factor of safety increased from 0.443 to 1.142 for Morgenstern and Price method.

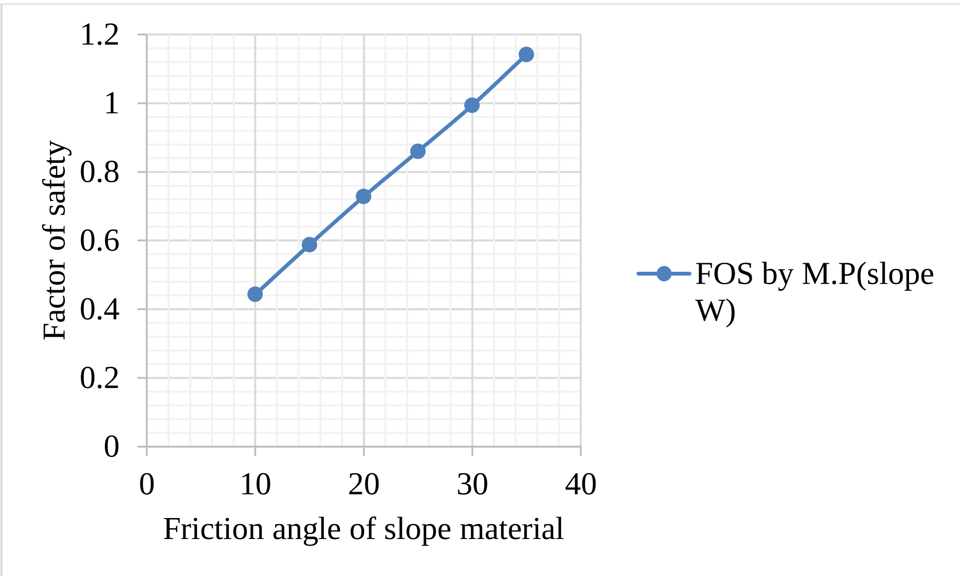


Figure 4. 17:Relation between Internal friction angle and factor of safety (68+700)

4.4.4 Influence of Unit weight of slope material on Factor of safety

Table 4. 16: Influence of unit wt. of slope material Factor of safety (68+700)

Soil Parameters

unit weight (γ)= (10-25) kN/m^3,cohesion(c)= 5.1kPa ,friction angle (φ)= 34.5^0

Case	Unit weight(kN/m^3)	FOS by M.P(slopeW)
1	10	1.375
2	15	1.204
3	20	1.11
4	25	1.05

The variation of factor of safety with variation of unit weight of the slope materials are show in figure 4.18.

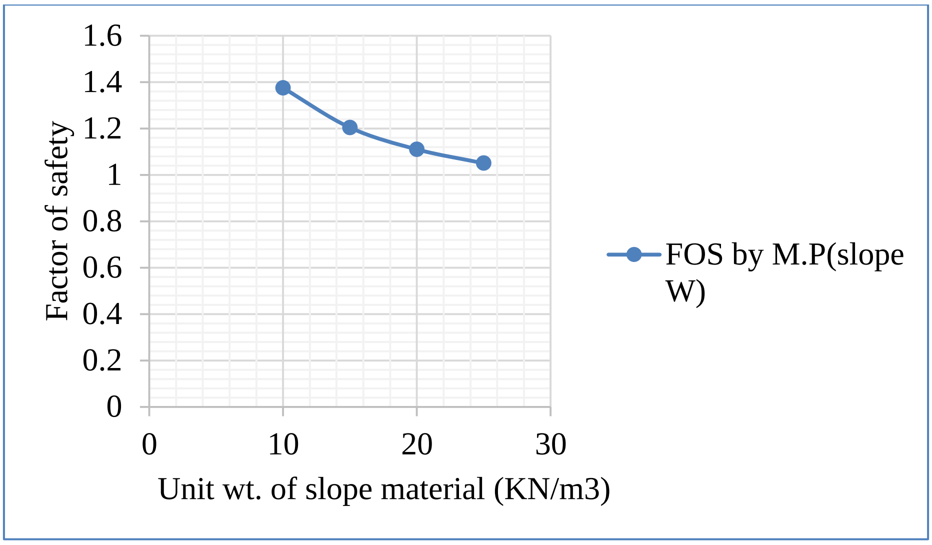


Figure 4. 18:Influence of unit weight of slope material on Factor of safety (68+700)

4.5 Combined effect of parametric variation

4.5.1 Combined effect of Cohesion and Unit Weight on Factor of Safety

The effect of cohesion together with the unit weight of the soil on the factor of safety was studied in this section. Here, cohesion and the unit weight of the soil were increased together, while their ratio remained constant for all slope section

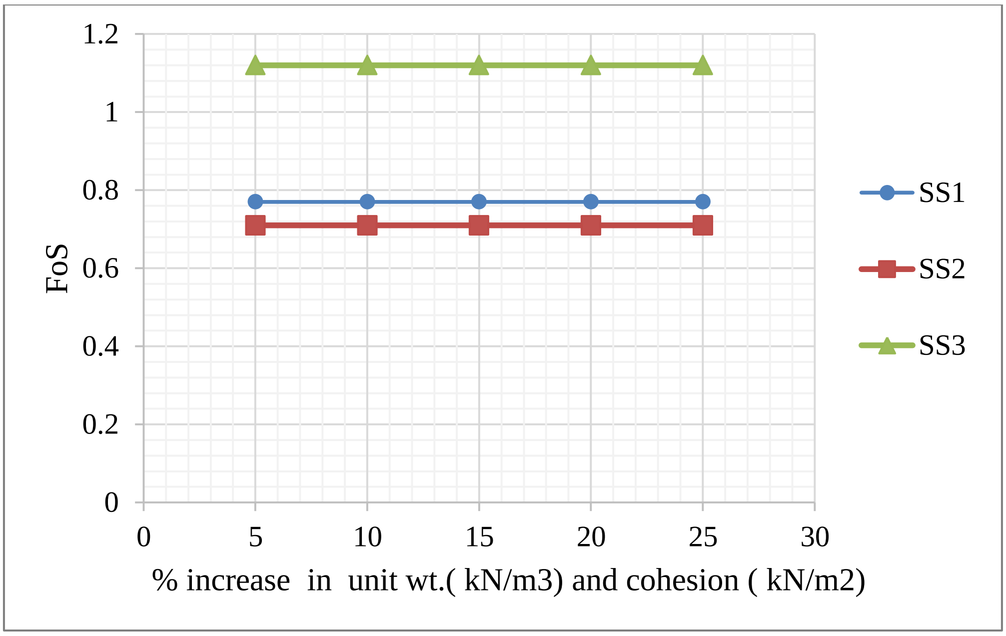


Figure 4. 19: Combined effect of cohesion and unit weight on factor of safety

Figure 4.19 indicates that factor of safety remains constant while increasing cohesion and unit weight of soil in same ratio.

4.5.2 Combined Effect of Internal Friction and the Unit Weight on FoS

In this section values of unit weight and internal frictional angles are increases keeping cohesion constant. Figure 4.20 indicates the slightly increase on factor of values on increasing the internal frictional angle and unit weight of soil.

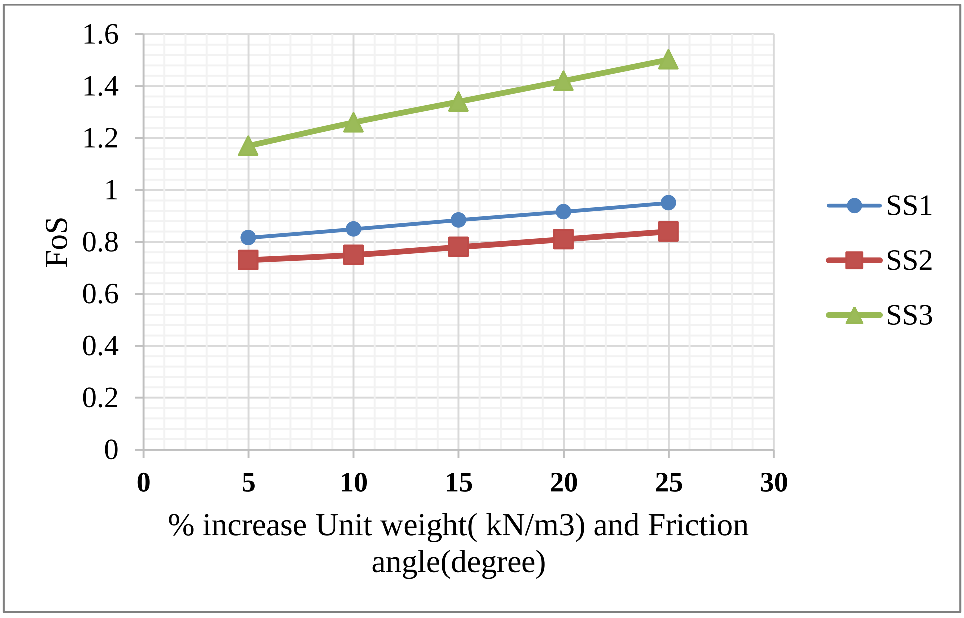


Figure 4. 20: Combined effect of internal frictional angle and unit weight on FoS

4.5.3 Combined Effect of Internal Friction and Cohesion on the FoS

In this part, since the potential failure surface is anticipated to be affected by the combination of cohesion and internal friction angle values, the relation between the factor of safety ,cohesion and internal friction angle is shown in Figure 4.21. Since both of this shear strength parameters are resisting forces, increasing these two value leads to an increase in the value of factor of safety.

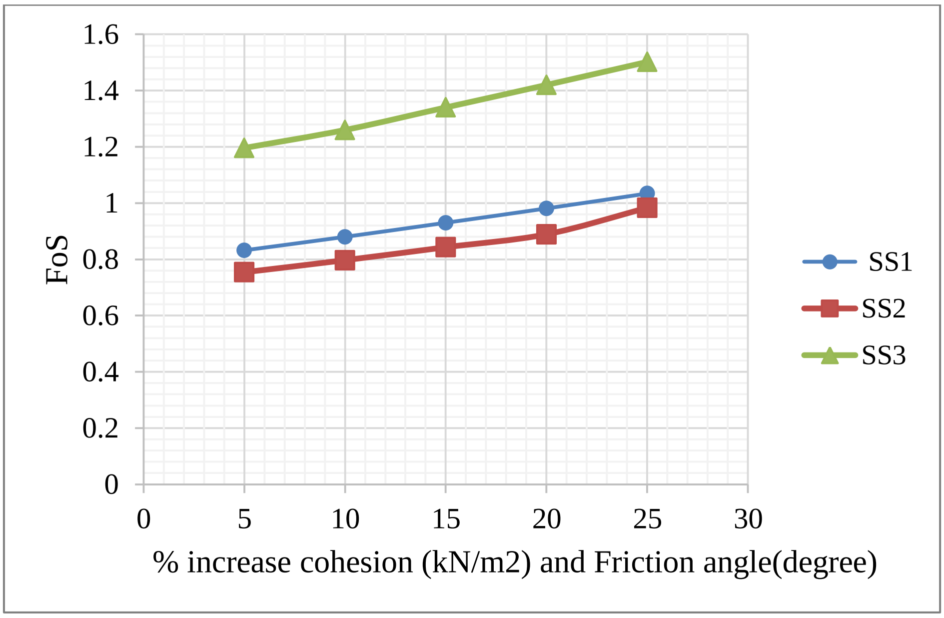


Figure 4. 21: Combined effect of cohesion and internal frictional angle on FoS

4.5.4 Combined Effect of Internal Friction and Cohesion and unit wt. on FoS

In this section values of unit weight and internal frictional angle and cohesive strength are increased. Figure 4.22 indicates the slightly increase on factor of values on increasing the internal frictional angle and unit weight and cohesive strength of soil.

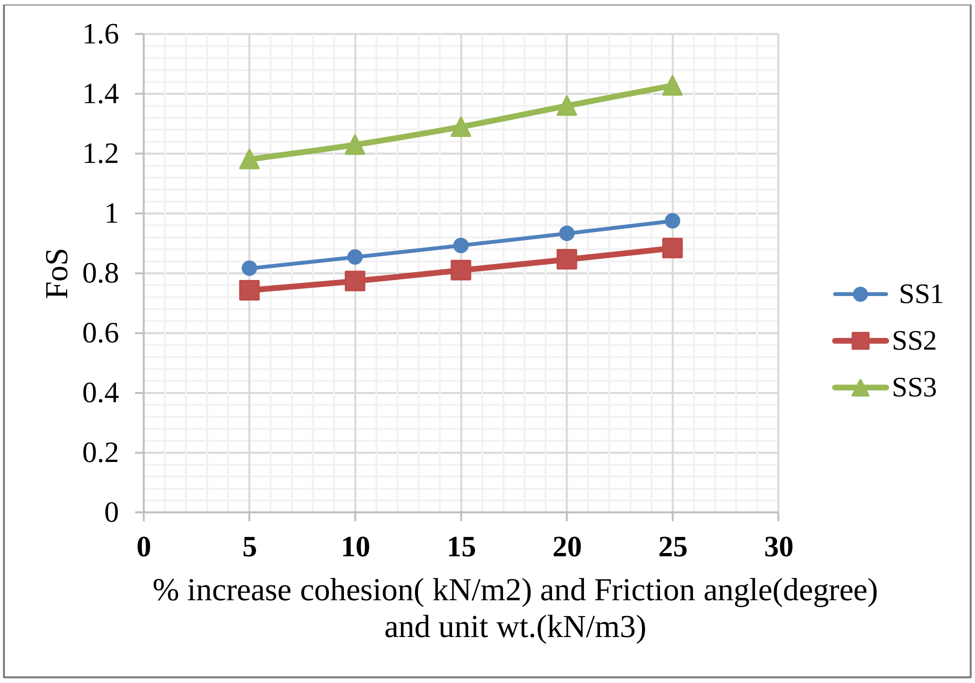


Figure 4. 22: Combined effect of cohesion and internal frictional angle and unit wt. on FoS

4.6 Remedial and preventive measures

If result of the stability analysis indicates the road cut slope does not meet the minimum factor of safety requirement, then it may be necessary to use slope stabilization methods. Various slope stabilization technologies are available in market to mitigate cut slope failure along the road. In general, slope stabilization methods can be categorizes in to two groups as:

- Preventive stabilization methods, applied to stable, but potentially unstable natural slopes, slope to be cut or embankment to be constructed.
- Corrective or remedial stabilization methods applied to existing unstable, moving slopes or to failed slope.

The stability of any slope will be improved if certain actions are carried out (Abramson, 2001).While stabilizing the any slope first of all one must identify the most important controlling process that is governing the stability of the slope; second one must determine the appropriate technique to be sufficiently applied to reduce the influence of that process.

The road slope instability disaster has not only created the enormous direct economic loss, but also the indirect economic loss which is difficult to estimate and the bad social impact because of disaster interrupting the traffic (Li et al.2009).Therefore, finding appropriate remedial and preventive measures for critical slope will be very helpful to reduce economic and social problems in the study area, particularly in rainy season. In this present study area, combinations of different factors (rainfall, engineering properties of soil, geology, topography, and others) are responsible for triggering cut slope instability. Different mitigations measures can be applicable in this situation. The remedial measure should be cost effective and feasible. Different factors such as slope geometry, surface and subsurface ground water conditions strength of slope materials and reason of stabilization affect the applicability of remedial measures. A number of techniques such as; modification of slope geometry, proper managements of drainage, retaining structures, internal slope reinforcement have been developed to stabilize slopes considering the above mention conditions. In this present study the slope is reinforced with soil nail to improve its stability.

4.6.1 Slope stabilization using retaining wall.

The safety factor of two cut slopes SS1 and SS2 is less than 1 in general dry condition, and for cut slope SS3 is also below 1 in case of rise of water table which shows that slope is unstable, and it must be supported. So one of the supporting structure is retaining wall. Retaining walls of different sizes are applied in the toe of the cut slope and stability of slope with structures is evaluated. The maximum height of gravity retaining wall applied is for numerical modeling 10 m with maintainable ground water table as shown in figure below.

Table 4. 17: Factor of Safety with retaining wall at slope section SS1 (62+300)

Retainging wall height(m)	Factor of Safety/SRF	
	Slope/W	Phase2
8	0.902	0.9
10	0.958	0.95

Table 4. 18: Factor of Safety with retaining wall at slope section SS2 (68+300)

Retainging wall height(m)	Factor of Safety/SRF	
	Slope/W	Phase2
8	0.831	084

Table 4. 19: Factor of Safety with retaining wall at slope section SS2 (68+300)

Retainging wall height(m)	Factor of Safety/SRF	
	Slope/W	Phase2
4	1.359	1.39
5	1.538	1.56

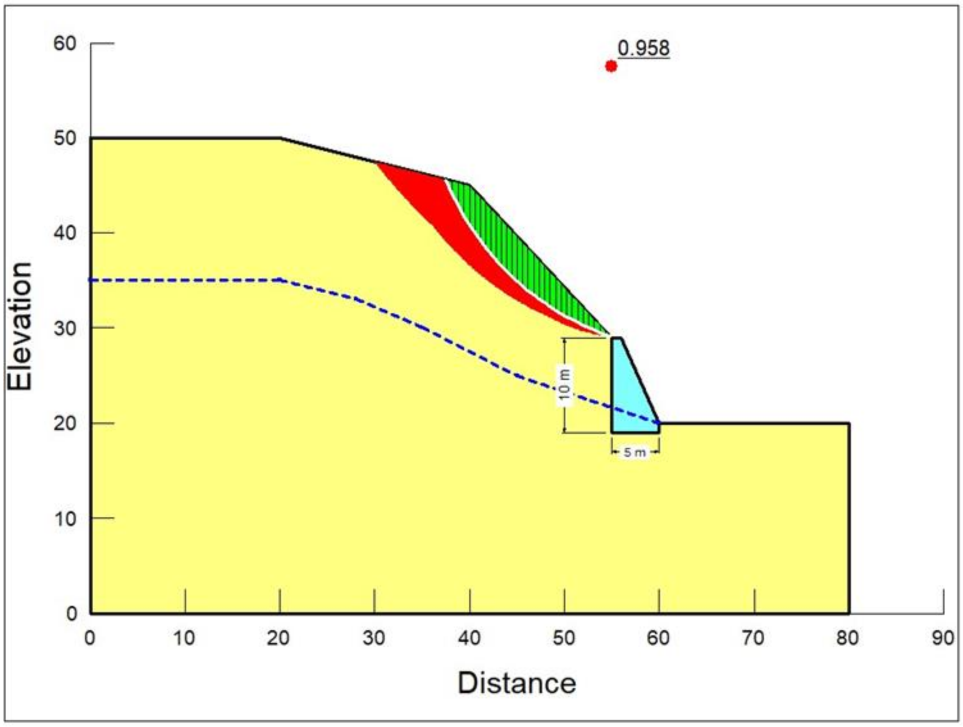


Figure 4.23: Result showing FoS with 10 m retaining wall in (slope/W) software (62+300)

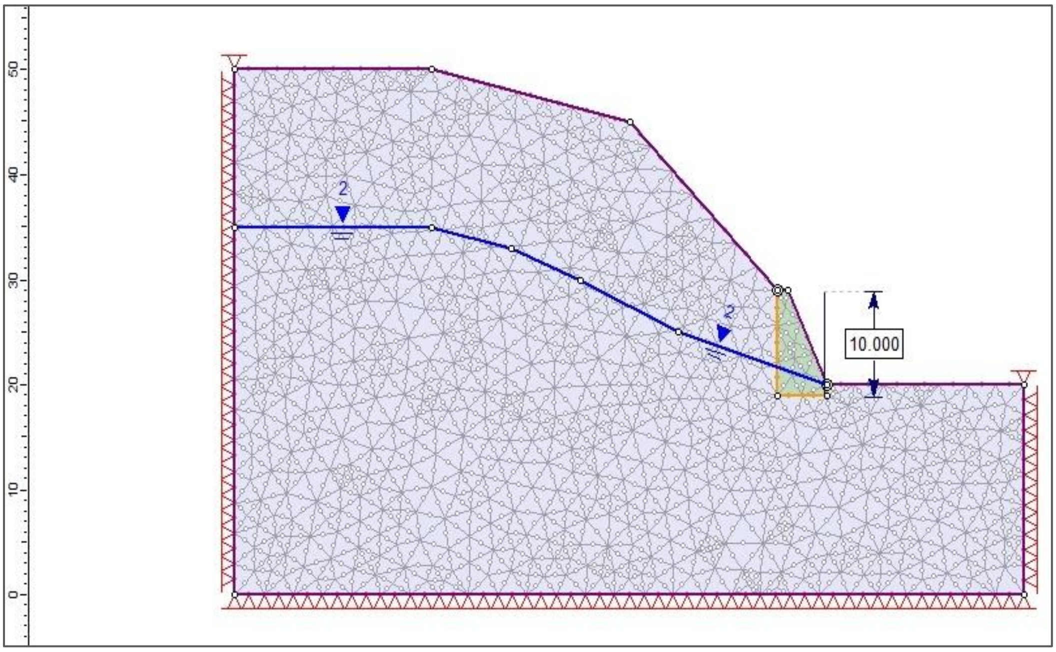


Figure 4. 24:Input model with 10 m retaining wall in (Phase 2) software (62+300)

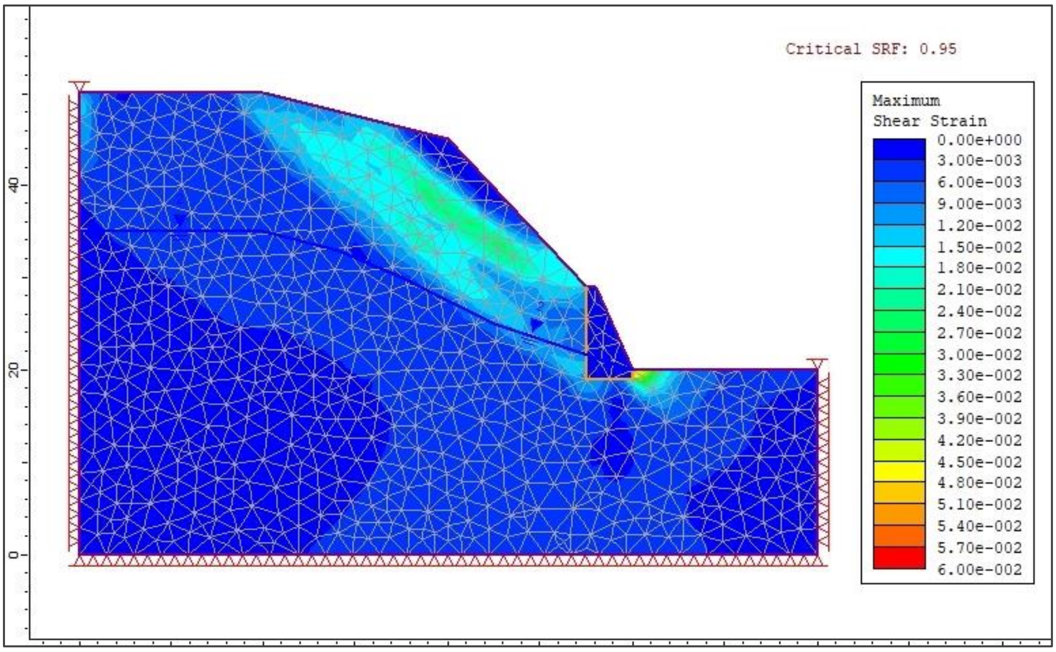


Figure 4. 25:Result showing MSS diagram with critical SRF with 10 m retaining wall in (Phase 2) software (62+300)

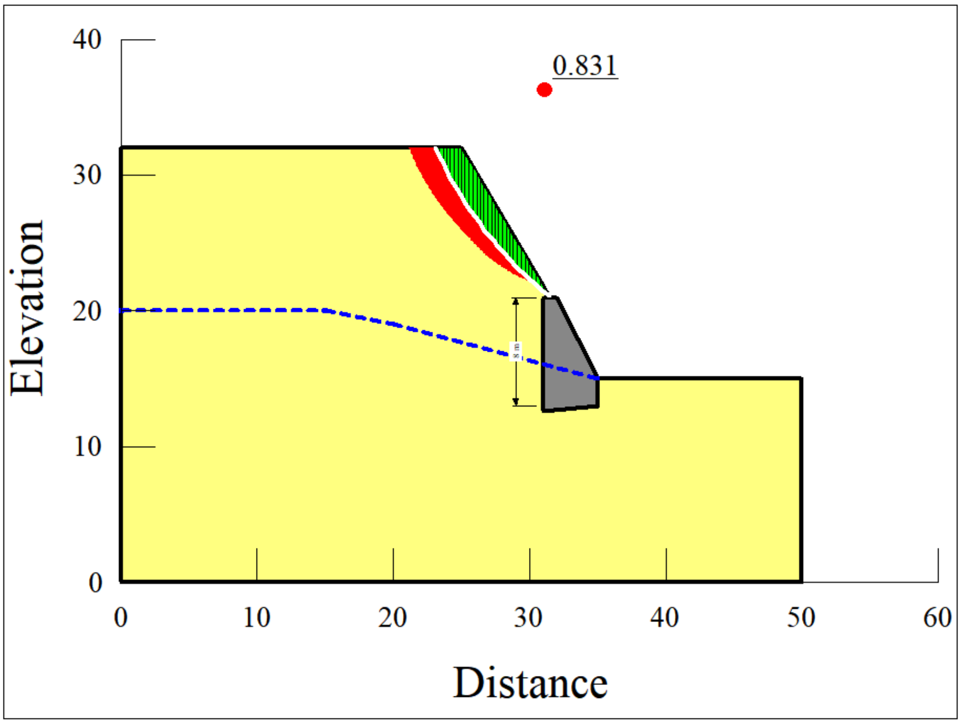


Figure 4. 26: Result showing FoS with 8 m retaining wall in (slope/W) software (68+300)

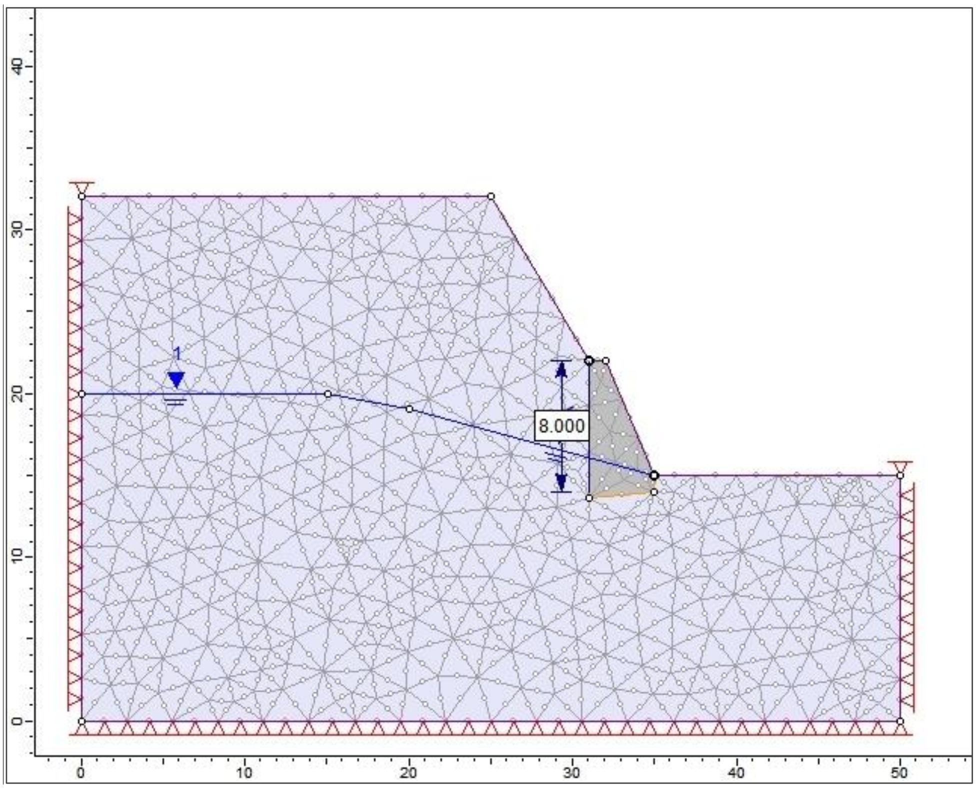


Figure 4. 27: Input model with 8 m retaining wall Phase 2 software (68+300)

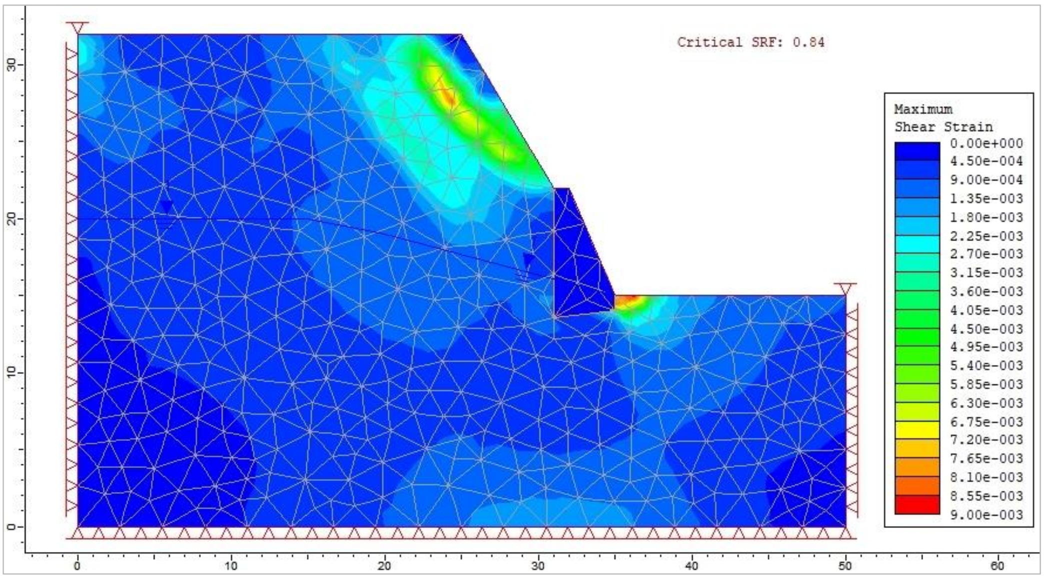


Figure 4. 28:Result showing MSS diagram with critical SRF with 8 m retaining wall in (Phase 2) software (62+300)

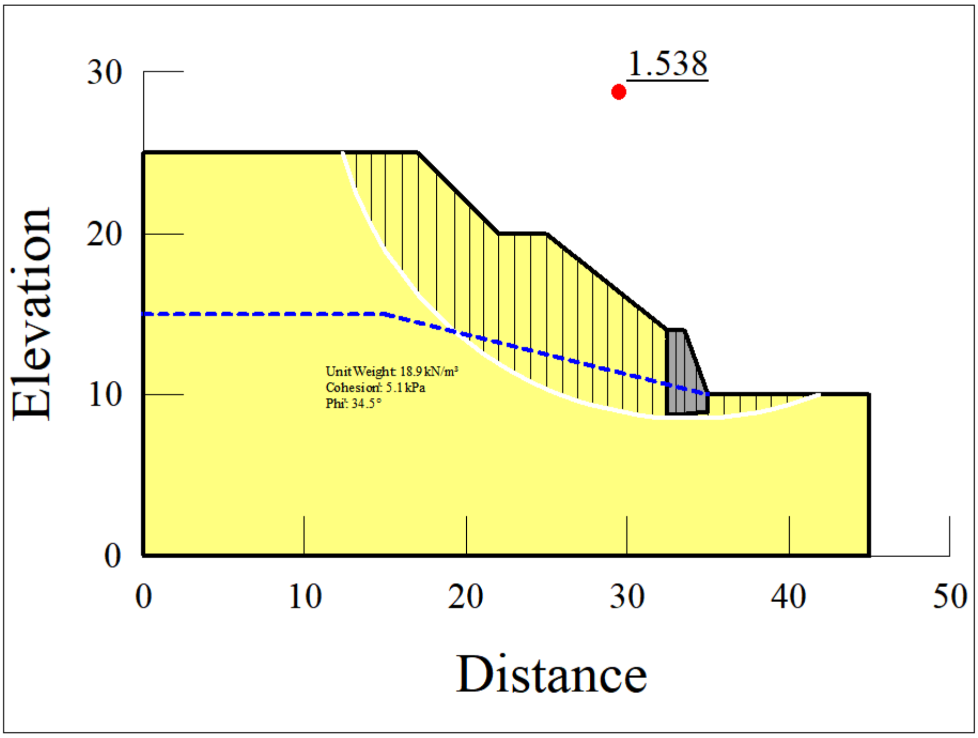


Figure 4. 29: Result showing FoS with 5 m retaining wall in (slope/W) software ((68+700)

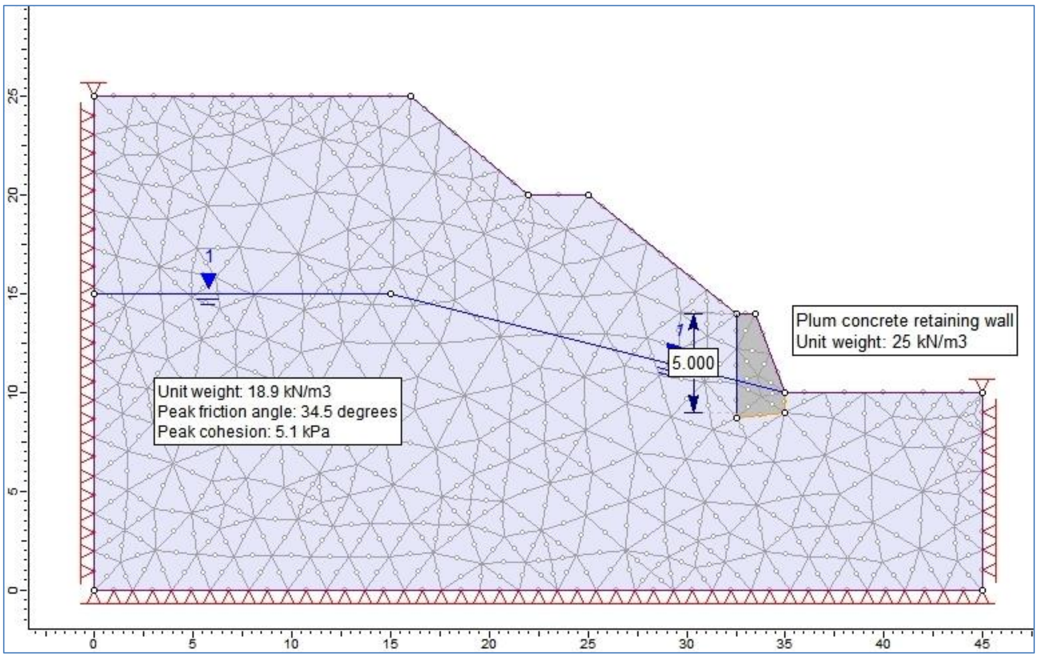


Figure 4. 30:   6 noded uniformly discretized input modele with 5 m ret. Wall wall in Phase 2 software (68+700)

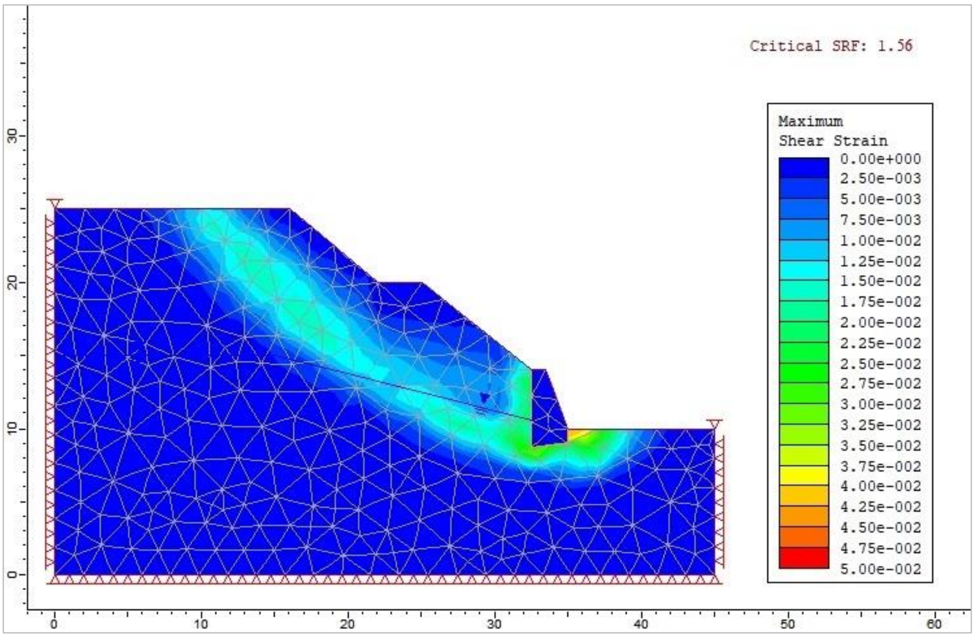


Figure 4. 31:Result showing MSS diagram with critical SRF with 5 m retaining wall in (Phase 2) software (68+700)

From above analysis it is clearly shown that with the use of retaining wall also Factor of Safety of first two slopes SS1 and SS2 are below 1 and Factor of Safety of slope section SS3 is only above 1.5.

4.6.2 Stabilization using soil nailing at Slope section SS1 (62+300)

In order to find out optimum inclination and length of soil nail for required safety factor soil nail deep angles are adjusted 0^0,5^0 ,10^0,15^0, 20^0, 25^0, 30^0,35^0,40^0 ,45^0, 50^0, 55^0, 60^0 with variable soil nail length of 10 m, 12 m,14 m and 16 m while other parameters of soil nail remains same mentioned below.

Pullout Resistance = 100 kPa, Tensile capacity = 200 kN, Bond diameter = 25 cm

Face anchorage = Yes, vertical spacing =1.56m horizontal spacing = 1m

The FoS Values and their relationship with different nail length for variable dip angle of soil nail is shown in Table 4.20 and Figure 4.28

Table 4. 20: Factor of Safety for different nail length with variable nail inclination below horizontal at SS1 (62+300)

nail inclination	nail length				nail inclination	nail length			
	10 m	12 m	14 m	16 m		10 m	12 m	14m	16 m
0	1.083	1.143	1.205	1.284	35	1.327	1.412	1.488	1.551
5	1.145	1.203	1.28	1.335	40	1.334	1.416	1.488	1.5
10	1.188	1.254	1.345	1.409	45	1.334	1.413	1.447	1.447
15	1.223	1.301	1.395	1.452	50	1.322	1.379	1.379	1.379
20	1.261	1.343	1.431	1.488	55	1.296	1.296	1.296	1.296
25	1.288	1.373	1.459	1.52	60	1.206	1.206	1.206	1.206
30	1.309	1.398	1.478	1.544					

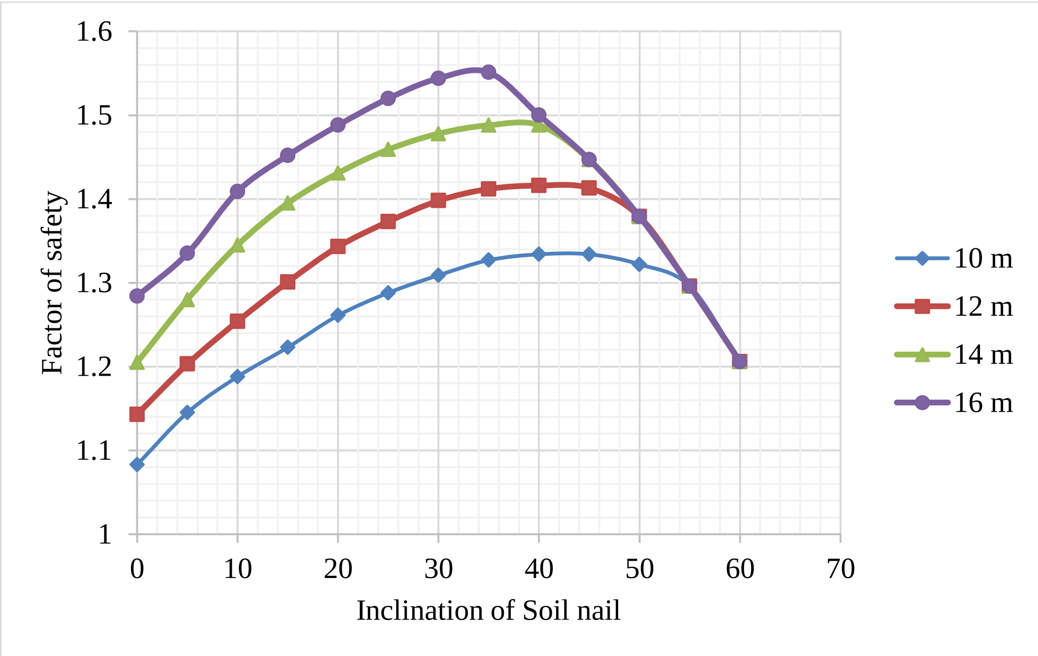


Figure 4. 32: Plot of Factor of Safety vs inclination of soil nail with horizontal for different soil nail length (62+300)

From above analysis 16 m long soil nail is tried for solution as preventive measures and analysis was done with variable soil nail parameters.

4.5.2.1. Influence of Soil Nail Dip Angle

In order to analyze the influence of soil nail dip angle on the safety factor, the soil nail deep angles are adjusted 10^0,15^0, 20^0, 25^0, 30^0,35^0,40^0 ,45^0, 50^0,55^0,60^0while other parameters of soil nail remains same as those in stability analysis of soil nail. The relationship between the safety factor and the dip angle of soil nail is shown in Figure 4.29.

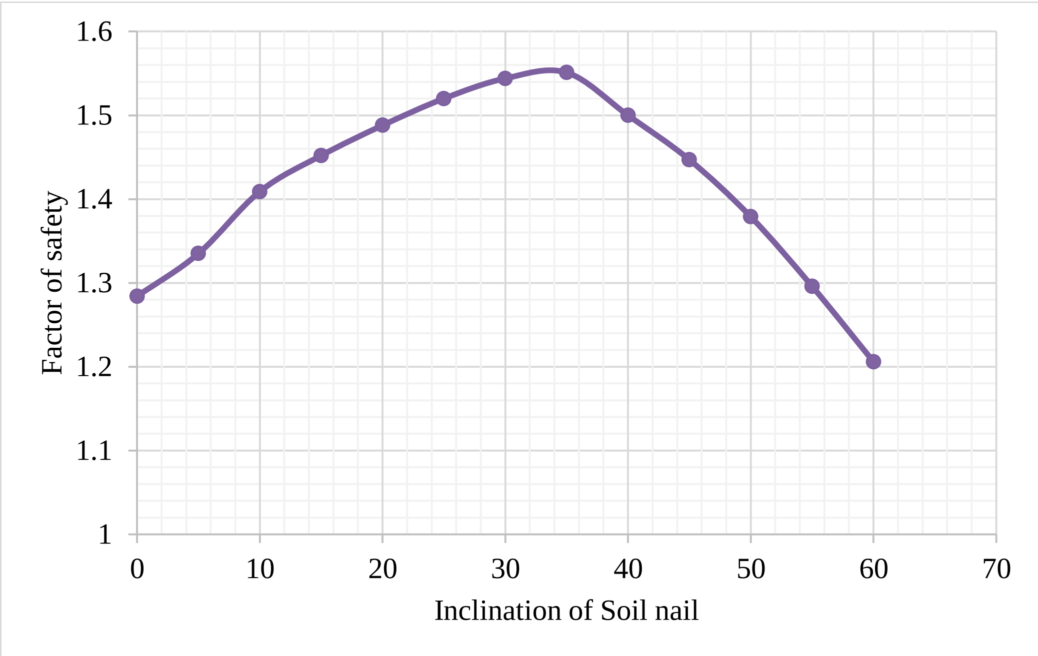


Figure 4. 33: Plot o factor of safety verses inclination soil nail with horizontal (62+300)

The perusal of Figure 4.29 clearly indicates that factor of safety increases with increase in nail inclination to the horizontal first and reach to optimum value and then start to decrease. Here the optimum angle of soil nail inclination is found to be 35^0 from the horizontal.

4.5.2.2 Influence of Soil Nail Length.

In order to analyze the influence of soil-nail length on the safety factor, the soil-nail lengths are varied from 10m to 20m while other parameters remain the same.

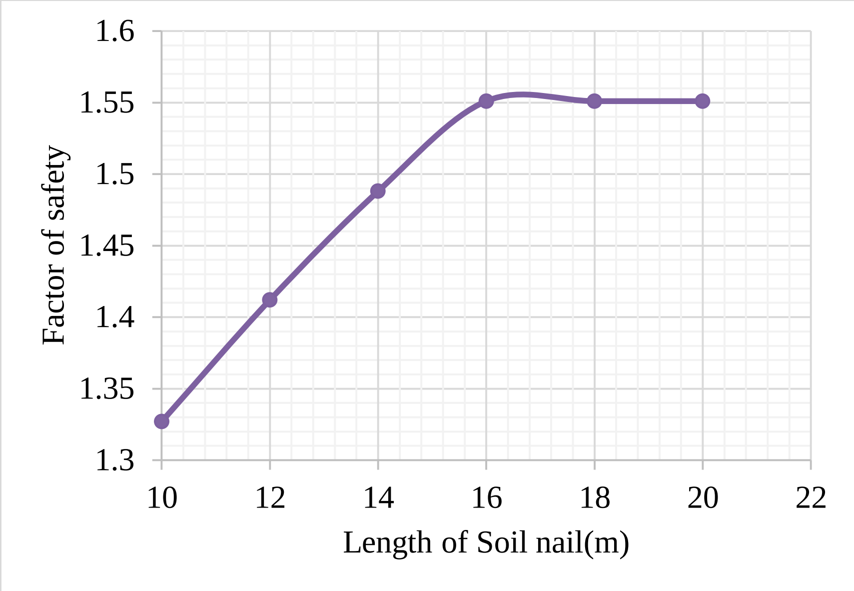


Figure 4. 34: Influence of soil nail length on safety factor (62+300)

As can be seen from Figure 4.30 the soil nail has a significant impact on the stability of the slope. The factor of safety increases with the increase in of the soil nail length, but after some point even with increase in nail length factor of safety remains constant keeping other parameter same.

4.5.2..3 Influence of Soil Nail Spacing

In order to analyze the influence of soil-nail spacing on the safety factor, the soil-nail vertical spacing are varied from 0.5m to 5 m while other parameters remain the same as in stability analysis of nail slope. As can be seen from Figure 4.31 the factor of safety decrease with increasing the vertical spacing of the soil nail and highest factor of safety is obtained for 0.5m spacing of soil nail.

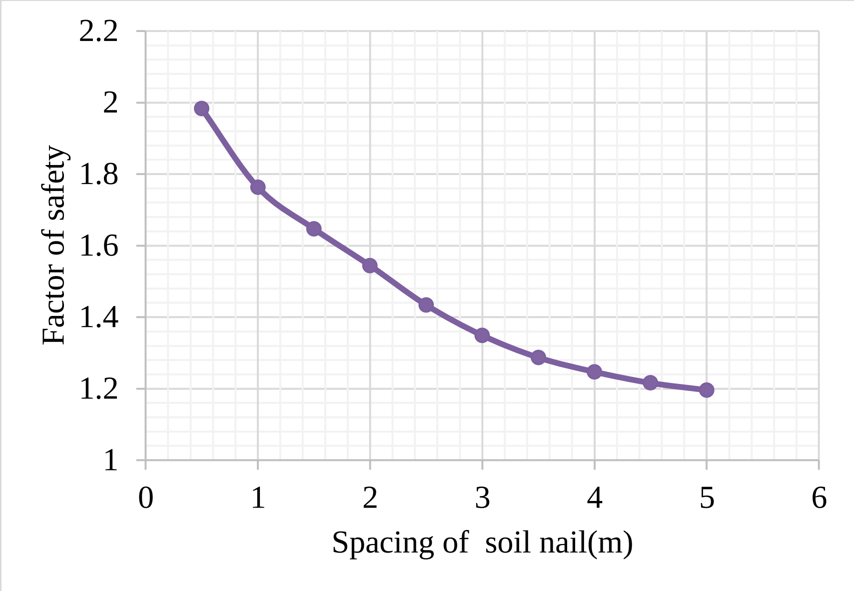


Figure 4. 35: Influence of soil nails spacing on the safety factor (62+300)

4.5.2.4 Influence of Friction Angle of Soil on Stability of Soil-Nailed Slope

In order to analyze the influence of friction angle of slope soil on the safety factor of soil-nailed slope, the friction angle of slope soil varied from 10^0 to 40^0 while other parameters remain the same. As can be seen from Figure 4.32 the factor of safety increase with increasing angle of friction of slope soil.

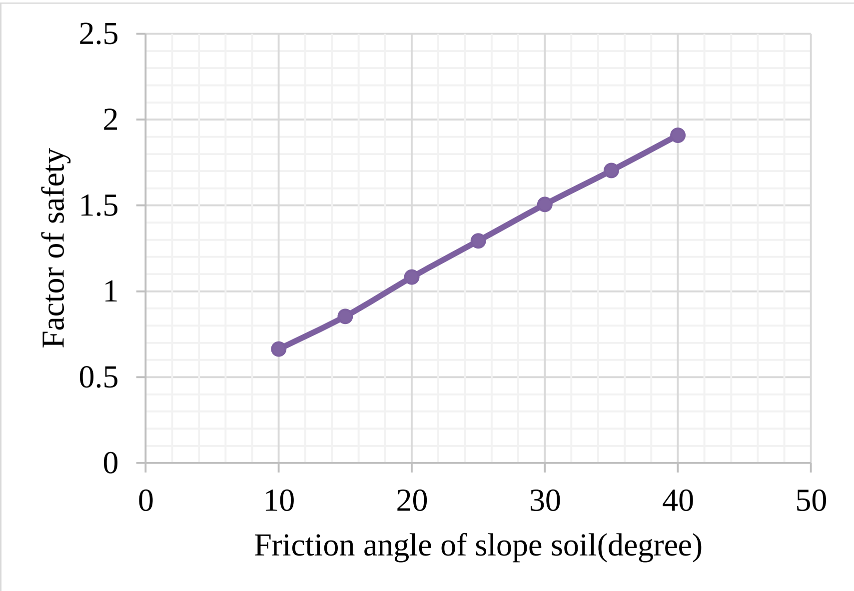


Figure 4. 36: Influence of friction angle of soil on safety factor of soil nailed slope(62+300)

4.5.2.5 Influence of Water Table on Stability of Soil-Nailed Slope

In order to analyze the influence variation of water table on the safety factor of soil-nailed slope, the water table is varied from fully saturated condition to 22 m below the top of the slope while other parameters remain the same as in stability analysis of nail slope. As can be seen from Figure 4.33 the factor of safety increase with increasing depth of water table from top.

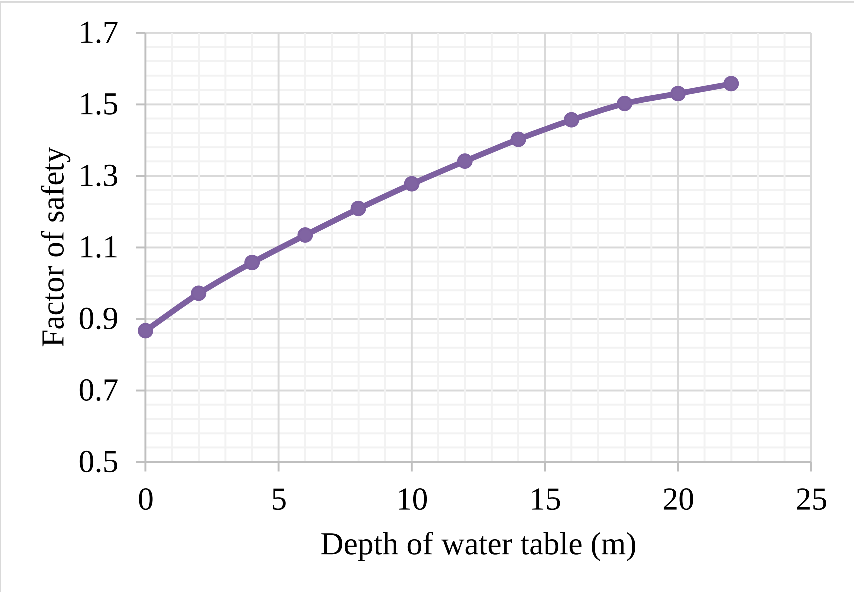


Figure 4. 37: Influence of water table depth on safety factor of soil-nailed slope

4.5.2.6 Influence of Unit Weight of Soil on Stability of Soil-Nailed Slope

In order to analyze the influence of unit weight of slope soil on the safety factor of soil-nailed slope, the unit weight is varied from 14kN/m^{3}to 22kN/m^3 while other parameters remain the. From Figure 4.34, the factor of safety decrease with increasing unit weight of slope soil.

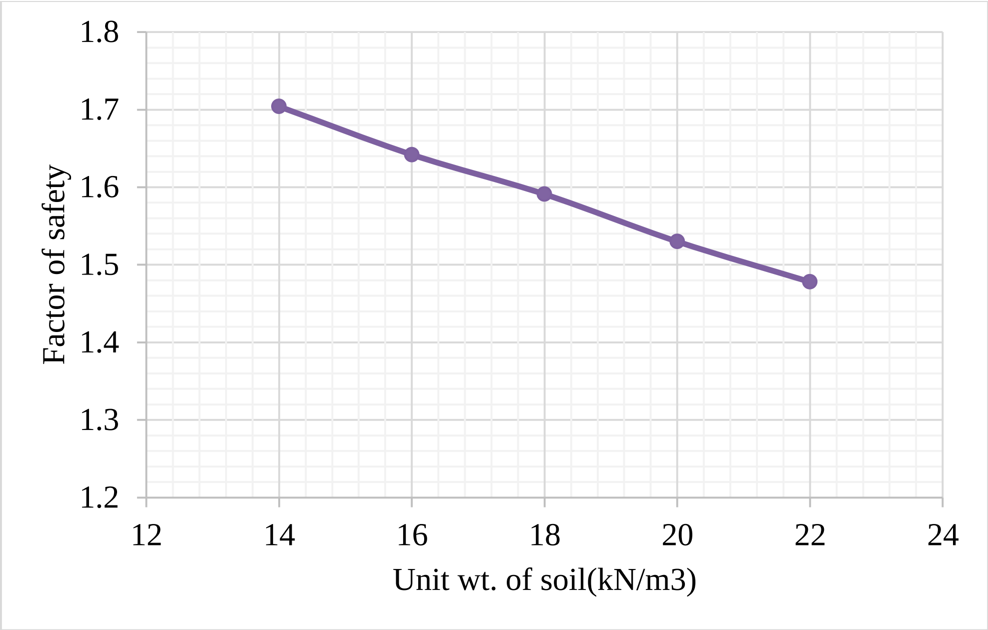


Figure 4. 38: Influence of unit weight of slope soil on safety factor of soil-nailed slope (62+300)

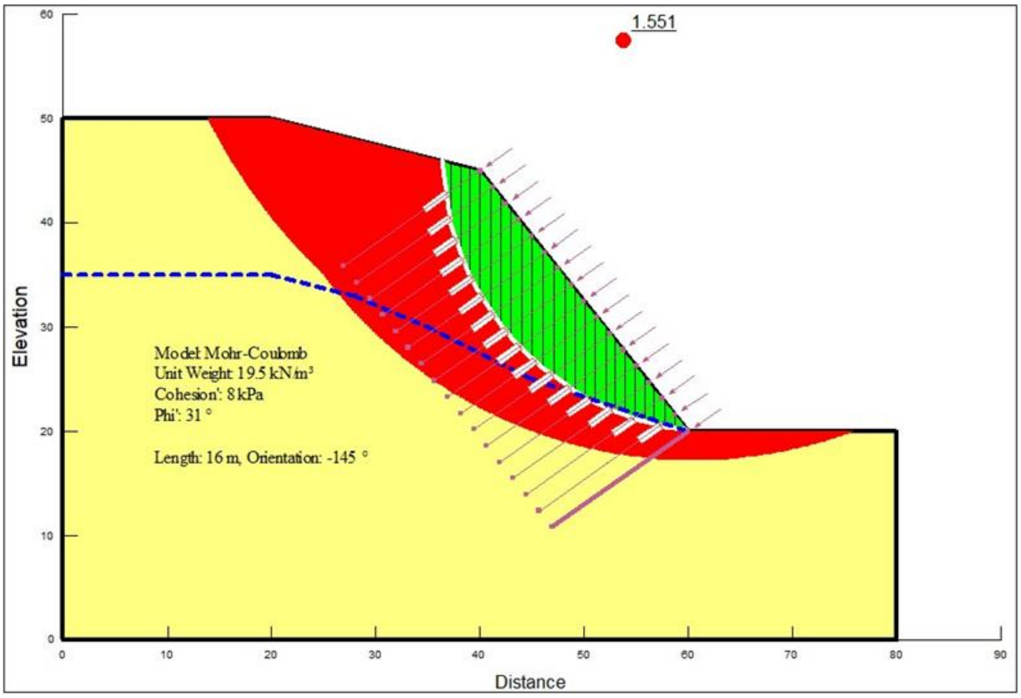


Figure 4. 39: Result showing FoS with safety map with 16 m nail at inclination 35 degree from horizontal at SS1(62+300)

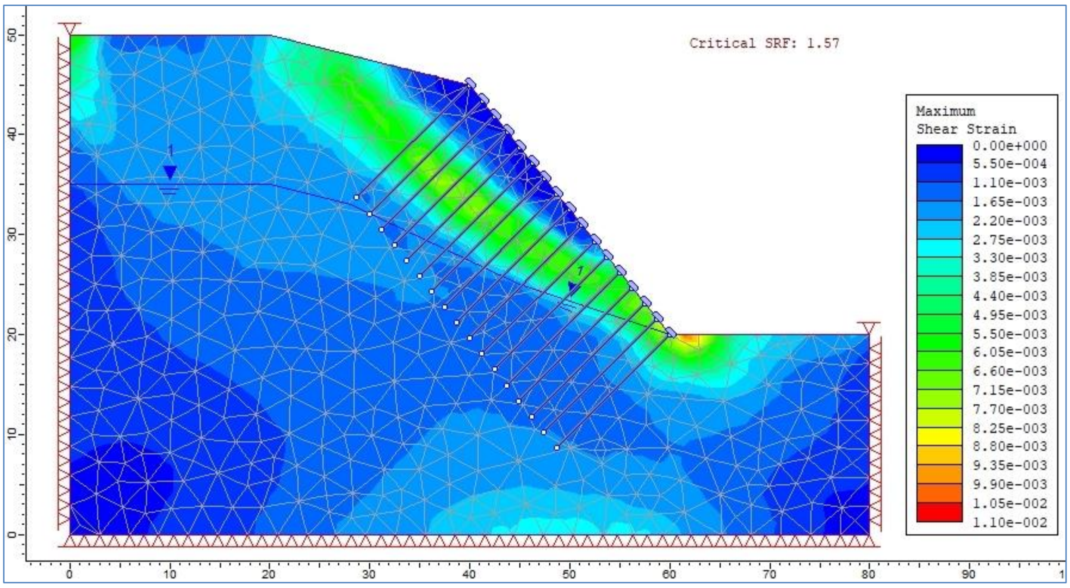


Figure 4. 40: Result showing Critical SRF with MSS diagram with 16 m nail at inclination 35 degree from horizontal at SS1 (62+300)

4.6.3 Stabilization using soil nailing at Slope section SS2 (68+300)

In order to find out optimum inclination and length of soil nail for required safety factor soil nail deep angles are adjusted 0^0,5^0 ,10^0,15^0, 20^0, 25^0, 30^0,35^0,40^0 ,45^0, 50^0, with variable soil nail length of 8 m, 10 m, and 11 m while other parameters of soil nail remains same mentioned below.

Pullout Resistance = 100 kPa, Tensile capacity = 200 kN, Bond diameter = 25 cm

Face anchorage = Yes, vertical spacing =2 m horizontal spacing = 1.5 m

The FoS Values and their relationship with different nail length for variable dip angle of soil nail is shown in Table 4.21 and Figure 4.37

Table 4. 21: Factor of Safety for different nail length with variable nail inclination below horizontal at SS1 (68+300)

	Length of nail (m)		
nail inclination	8 m	10 m	11 m
0	1.167	1.276	1.367
5	1.231	1.352	1.42
10	1.262	1.411	1.464
15	1.301	1.475	1.5
20	1.346	1.484	1.507
25	1.361	1.476	1.476
30	1.37	1.443	1.443
35	1.354	1.393	1.393
40	1.336	1.336	1.336
45	1.276	1.276	1.276
50	1.212	1.212	1.212

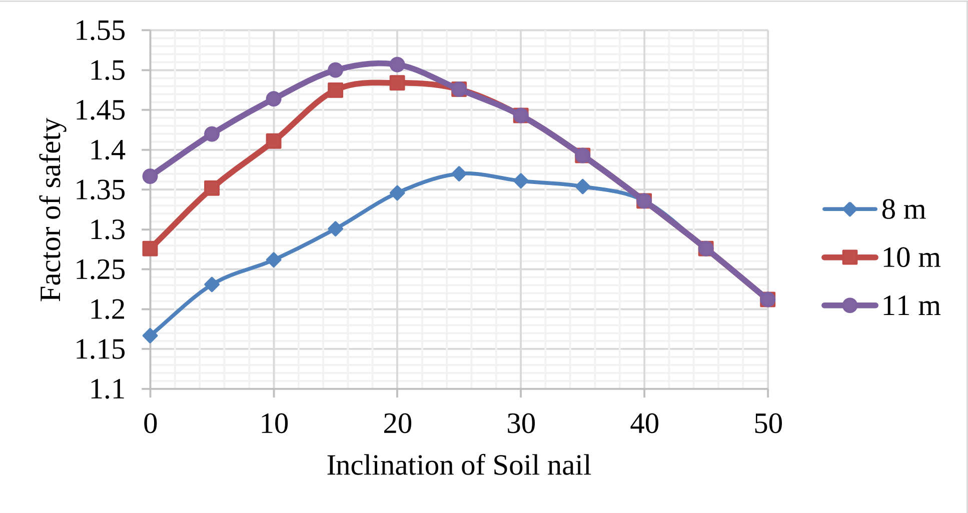


Figure 4. 41: Plot of Factor of Safety vs inclination of soil nail with horizontal for different soil nail length (68+300)

From above analysis 11 m long soil nail is tried for solution as preventive measures and analysis was done with variable soil nail parameters

4.5.3.1. Influence of Soil Nail Deep Angle

In order to analyze the influence of soil nail dip angle on the safety factor, the soil nail deep angles are adjusted 0^0, 5^0, 10^0,15^0, 20^0, 25^0, 30^0, 35^0,40^0 ,45^0, 50^0, while other parameters of soil nail remains same as those in stability analysis of soil nail. The relationship between the safety factor and the dip angle of soil nail is shown in Figure 4.38.

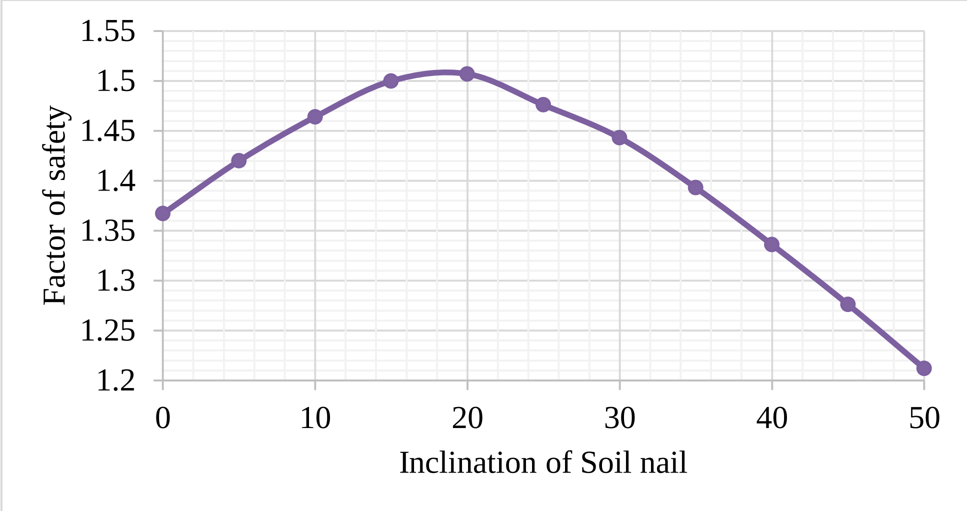


Figure 4. 42: Plot factor of safety vrs. inclination soil nail with horizontal (68+300)

The perusal of Figure 4.38 clearly indicates that factor of safety increases with increase in nail inclination to the horizontal first and reach to optimum value and then start to decrease. Here the optimum angle of soil nail inclination is found to be 20^0 from the horizontal.

4.5.3.2 Influence of Soil Nail Length.

In order to analyze the influence of soil-nail length on the safety factor, the soil-nail lengths are varied from 8 m to 14 m while other parameters remain the same.

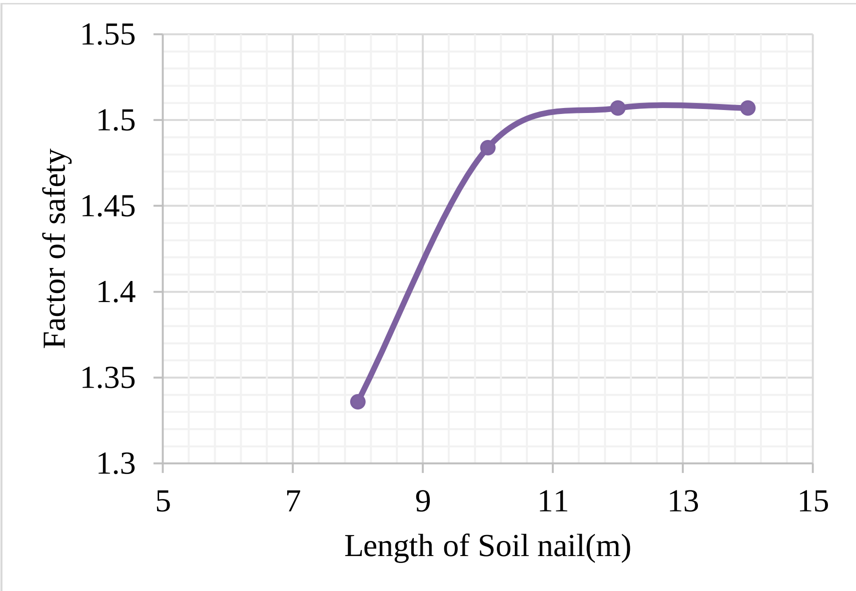


Figure 4. 43: Influence of soil nail length on safety factor (68+300)

As can be seen from Figure 4.39 the soil nail has a significant impact on the stability of the slope. The factor of safety increases with the increase in of the soil nail length, but after some point even with increase in nail length factor of safety remains constant keeping other parameter same.

4.5.3.3 Influence of Soil Nail Spacing

In order to analyze the influence of soil-nail spacing on the safety factor, the soil-nail vertical spacing are varied from 0.5 m to 4 m while other parameters remain the same as in stability analysis of nail slope. As can be seen from Figure 4.40 the factor of safety decrease with increasing the vertical spacing of the soil nail and highest factor of safety is obtained for 0.5 m spacing of soil nail.

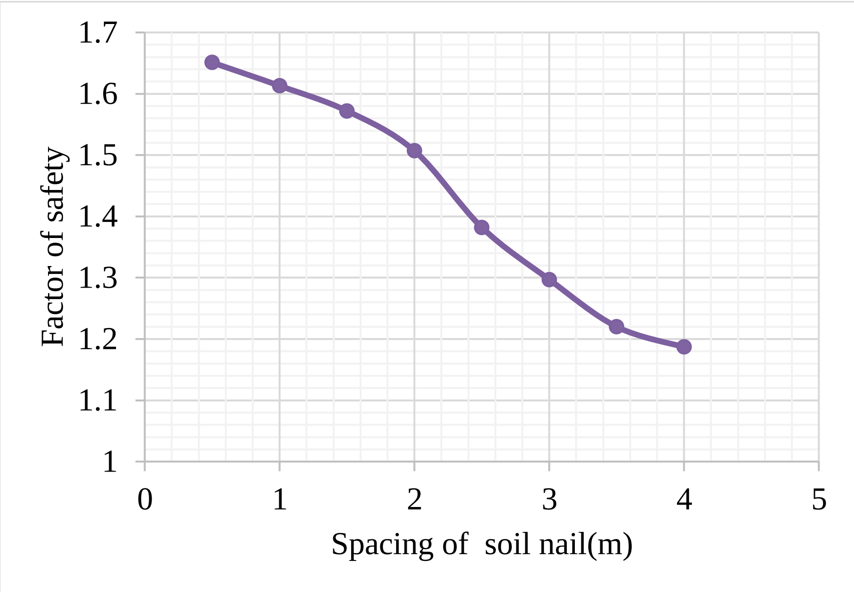


Figure 4. 44: Influence of soil nails spacing on the safety factor (68+300)

4.5.3.4 Influence of Friction Angle of Soil on Stability of Soil-Nailed Slope

In order to analyze the influence of friction angle of slope soil on the safety factor of soil-nailed slope, the friction angle of slope soil varied from 10^0 to 40^0 while other parameters remain the same. As can be seen from Figure 4.41 the factor of safety increase with increasing angle of friction of slope soil.

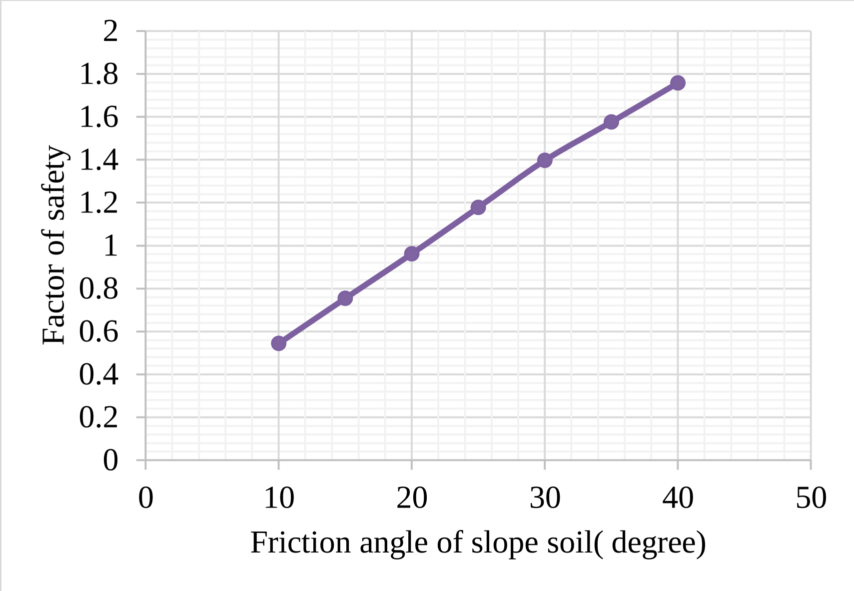


Figure 4. 45: Influence of friction angle of soil on safety factor of soil nailed slope(68+300)

4.5.3.5 Influence of Water Table on Stability of Soil-Nailed Slope

In order to analyze the influence variation of water table on the safety factor of soil-nailed slope, the water table is varied from fully saturated condition to 12 m below the top of the slope while other parameters remain the same as in stability analysis of nail slope. As can be seen from Figure 4.42 the factor of safety increase with increasing depth of water table from top.

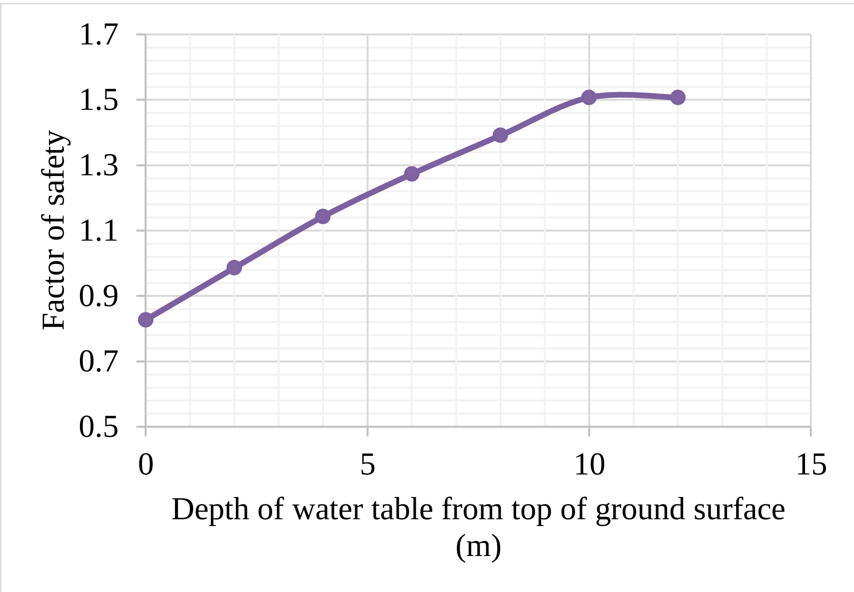


Figure 4. 46: Influence of water table depth on safety factor of soil-nailed slope (68+300)

4.5.3.6 Influence of Unit Weight of Soil on Stability of Soil-Nailed Slope

In order to analyze the influence of unit weight of slope soil on the safety factor of soil-nailed slope, the unit weight is varied from 14kN/$m^3$to 24 kN/m^3 while other parameters remain the. From Figure 4.43, the factor of safety decrease with increasing unit weight of slope soil.

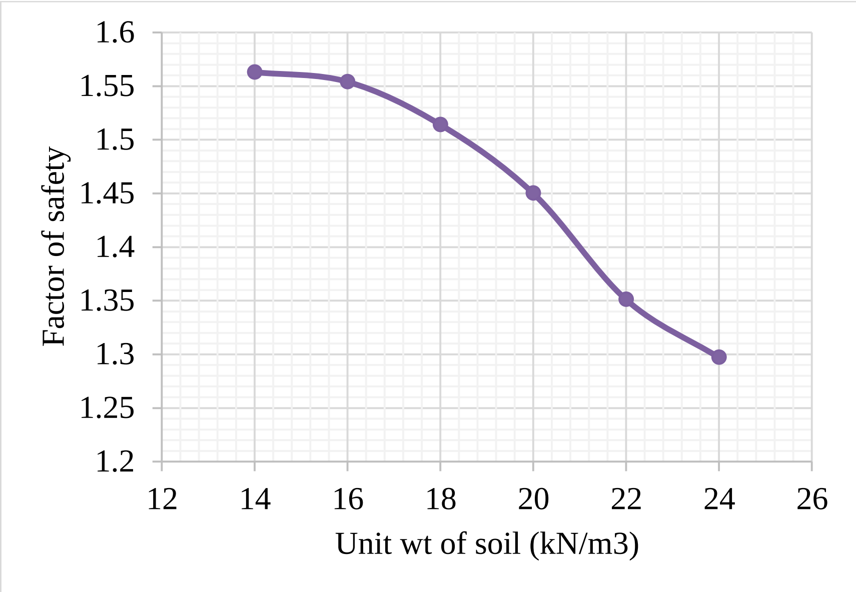


Figure 4. 47: Influence of unit weight of slope soil on safety factor of soil-nailed slope (68+300)

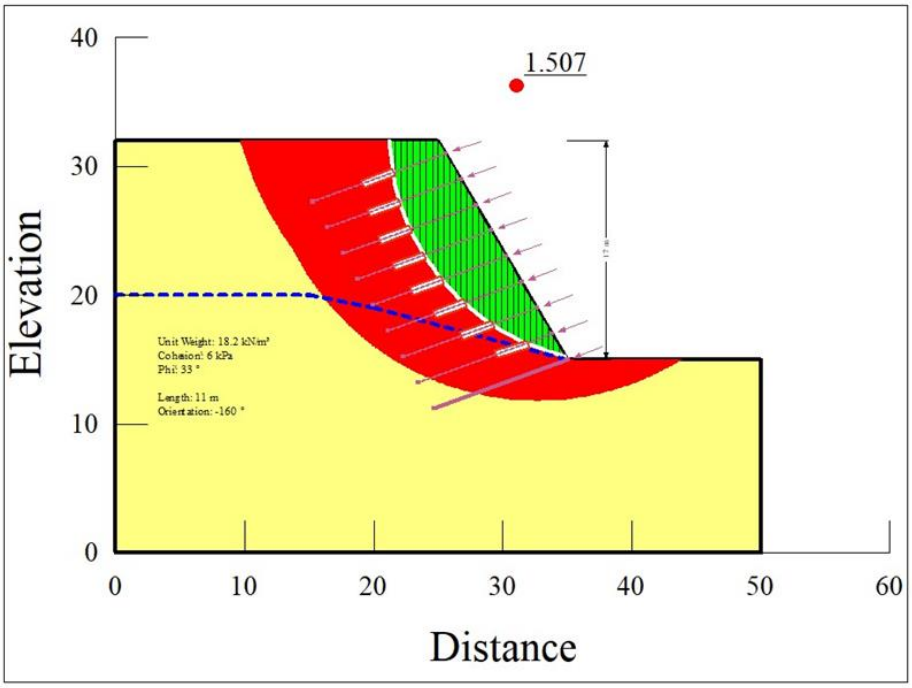


Figure 4. 48: Result showing FoS with safety map with 16 m nail at inclination 20 degree from horizontal at SS2 (68+300)

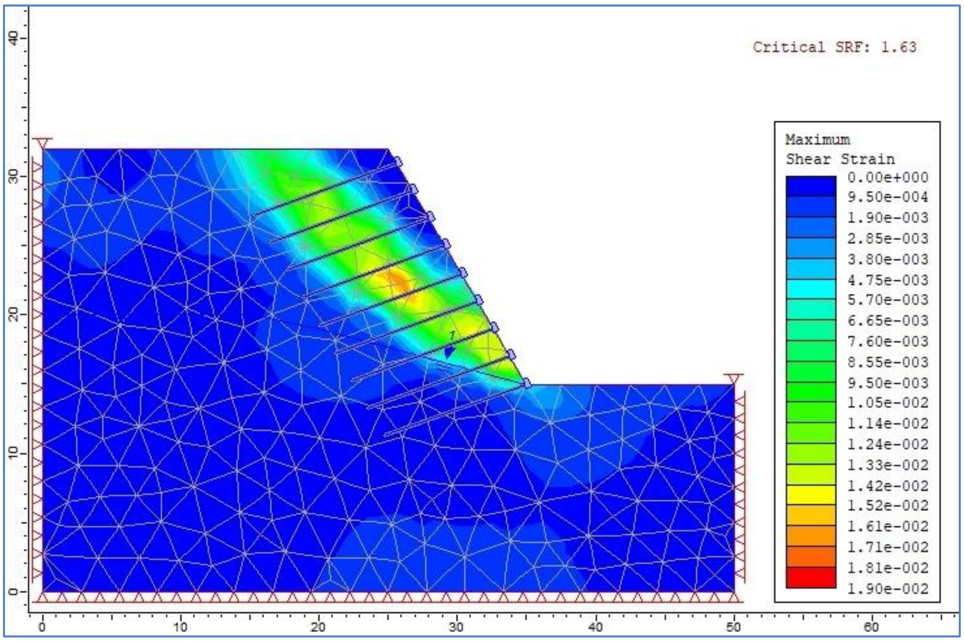


Figure 4. 49: Result showing Critical SRF with MSS diagram with 11 m nail at inclination 20 degree from horizontal at SS2 (68+300)

4.7 Validation of Analysis.

4.7.1 Verification through Literature

For verification of modal output a research work carried by Ammar Rouaiguia, Mohammed A. Dahim of Najran University is taken as reference. In their research work, they studied a 17 m height slope with two different layers of materials. The upper layer is 6m thick with unit weight 15kN/m^3, cohesion= 20 kPa, frictional angle=30^{0}and lower soli is 11m height with unit weight=17 kN/m^3, cohesion= 15kN/m^2, frictional angle = 25^0.The slope is cut in two materials 2 :1 (H:V).The total height of cut is 12m and bed rock exist 5m below the base of the cut. They also used Slope/W (Geoslope- 2007).

1. For variation of cohesion of soil

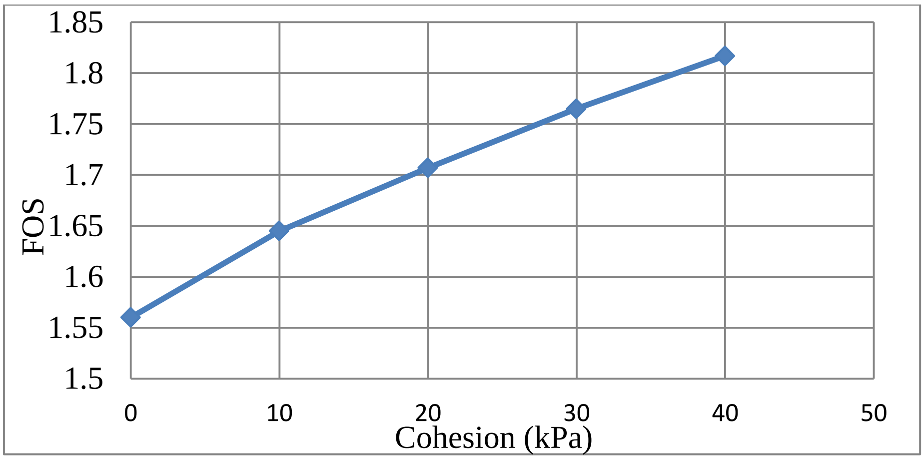


Figure 4. 50: FOS verses cohesion by Ammar Rouaiguia, Mohammed A.Dahim, (2013)

Above graph shows that factor of safety increase with increase in cohesion of soil, which is similar to the result obtained from author.

2. For variation of frictional angle of soil.

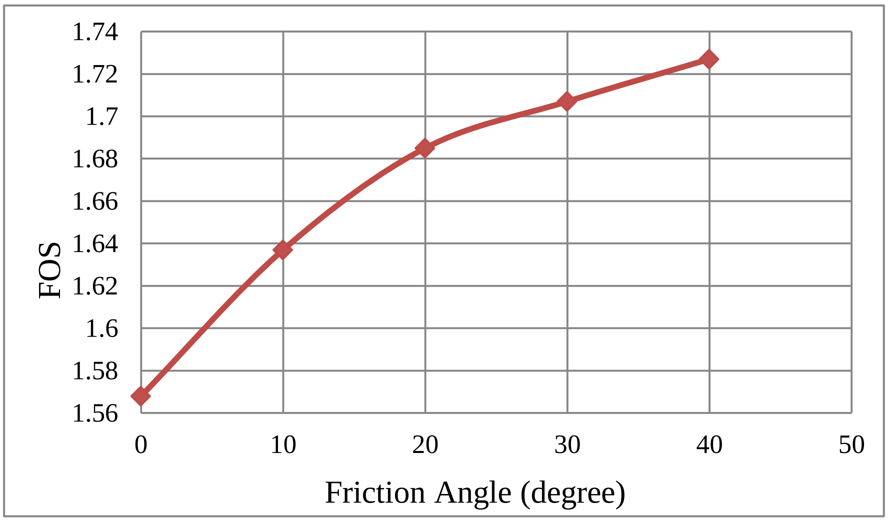


Figure 4. 51: Plot fos verses friction angle by Ammar Rouaiguia, Mohammed A.Dahim, (2013)

Above graph shows that factor of safety increase with increase in friction angle of soil, which is similar to the result obtained from author.

3. For variation of unit weight of soil.

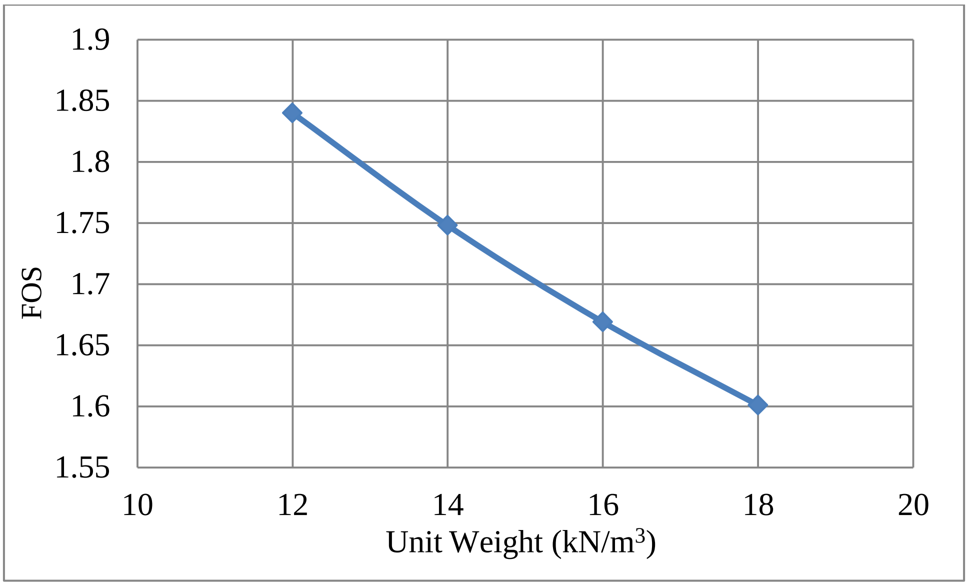


Figure 4. 52: Plot FoS verses unit weight by Ammar Rouaiguia, Mohammed A.Dahim, (2013)

Above graph shows that factor of safety decreases with increase in unit weight of soil, which is similar to the result obtained from author.

4.8 Modal Validation for Soil-Nailed Reinforced Slope

For validation of reinforced slope analytical calculation performed. During calculation planar slip surface (Sheahan and Oral, 2002) is assumed and stability of slope is analyzed.

Table 4. 22: Comparison of factor of safety obtained from Slope/W with analytical calculation for reinforced slope at 62+300

Analysis method	Factor of Safety	Variation
Slope/W	1.551	
Phase2	1.57	1.22%
Hand Calculation	1.56	0.96%

Table 4. 23: Comparison of factor of safety obtained from Slope/W with analytical calculation for reinforced slope at 68+300

Analysis method	Factor of Safety	Variation
Slope/W	1.507	
Phase2	1.63	8.63%
Hand Calculation	1.55	2.85%

From Table 4.22 and 4.23, comparing the results obtained with analytical calculation shows that the variation of the FOS is considerable. The variation in obtained may be the shape of slip surface and method of analysis. The shape of slip surface assumed in analysis is ellipsoid and in analytical calculation is linear (wedged shaped).

4.8.1 Validation of Parameter Variation of Soil Nail

For this purpose a research work carried by Ou-Ling Tang and Qing-Ming-Jiang (2015), Chengdu University of Technology is taken as reference. In their research work, they studied a 9m height slope with unit weight 18kN/m^3, cohesion 14 kPa, frictional angle 26^0.

1. Variation of soil nail dip angle

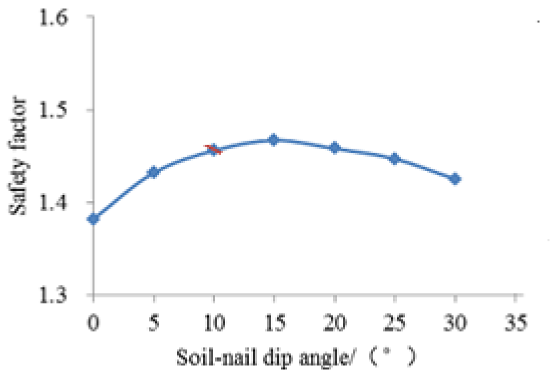


Figure 4. 53: Fos verses soil –nail dip angle plot by Tang et.al.,(2015)

The outcomes of software for various nail inclination with horizontal is somehow similar with research work of Tang et.al.,(2015).Tang et.al.,(2015) found that with increase in inclination of nail from horizontal the factor of safety increases first and reach to a optimum value and then decrease, similar results are obtained from this thesis works.

2. Variation of soil nail length

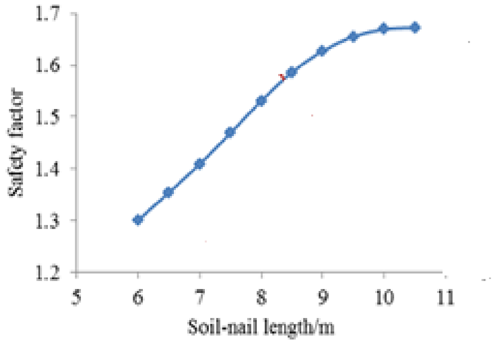


Figure 4. 54: Fos verses Soil –nail length plot by Tang et.al.,(2015)

For variation of nail length, they found that factor of safety increases with increase in nail length, but its increases is limited when soil nail length is approximately to the height of the slope. Similar results are obtained by author.

3. Variation of soil nails spacing

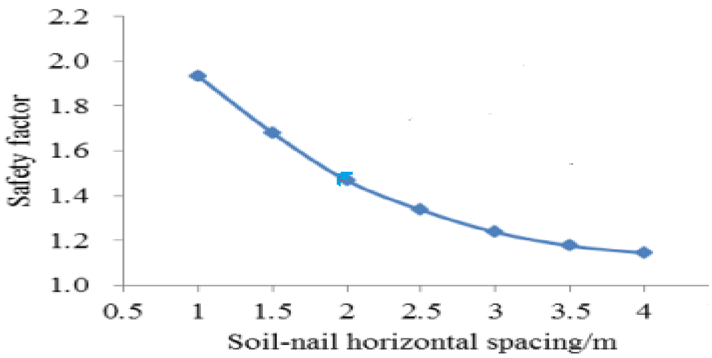


Figure 4. 55: Fos verses soil –nail spacing by Tang et.al.,(2015)

The above graph shows factor of safety decrease with increasing the soil nail spacing, which is similar to the result obtained by author.

4.9 Analytical solution of Stability

1. Stability Analysis of Cut slope

The stability analysis of cut slope was performed using Bishop simplified Method. The trail slip surface was assumed and the slip circle was divided into number of slice and factor of safety was calculated. The slip surface corresponding to minimum factor of safety was considered the critical slip surface. The adjoining figure represents the graphical representation of the stability analysis of unreinforced slope by Bishop

Method. Graphical calculations are employed to estimate the values. In this section, factor of safety calculation of critical slip surface is presented For Critical Slip Surface.

4.9.1For slope section SS1 (62+300)

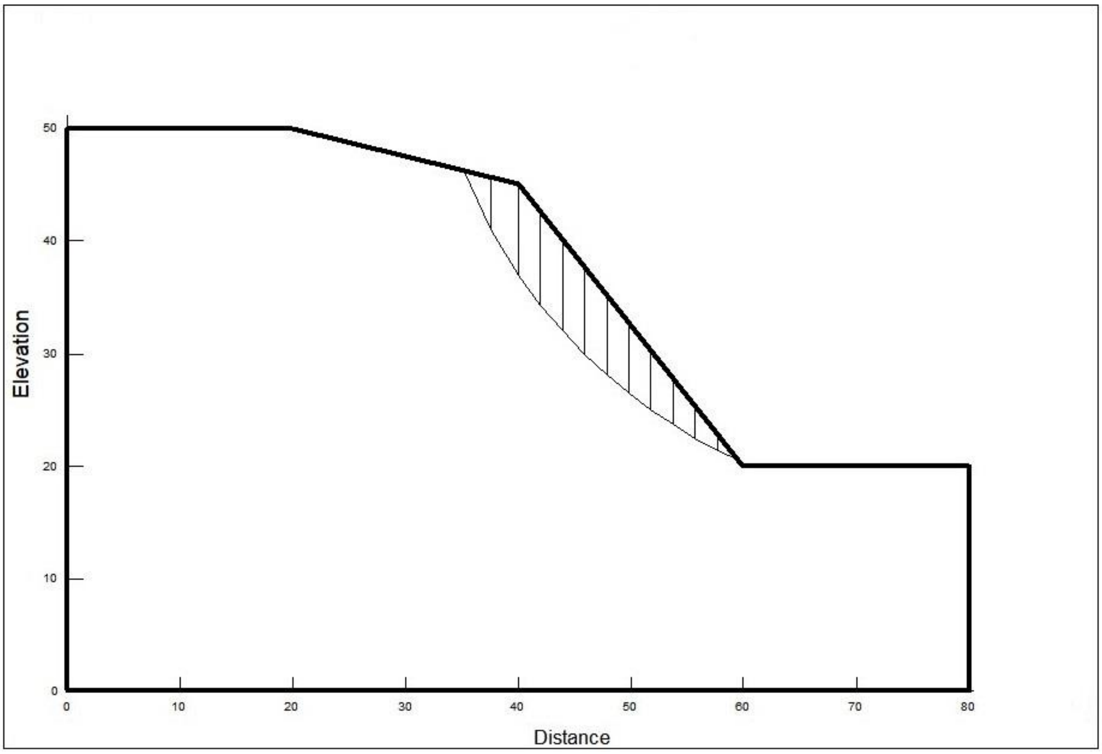


Figure 4. 56: problem statement for hand caulation at chainage 62+300

Radius of Slip Surface(R) = 45.25 m

Co-ordinate of center of slip surface(x, y) = (79.718, 61,256), (Here radius of slip surface and Co-ordinate are measured using Autocad-2007)

Height of cut slope(H) =25m

The calculations are done in tabular form. The value of m_α has been computed using equation

m_α=(1+tanΦtanα/F_s)*cosα Equation 4.1

For first trail F_s is assumed to be 1. The factor of Safety is calculated by using following formula.

$F_s = \frac{\sum 1/m\alpha[cb+(W-ub)\tan\Phi]}{W\sin\alpha}$ Equation 4.2

Here, Cohesion of soil(c) = 8 kPa

Internal Friction angle = 31^0

Assumed value of FOS= 1

Table 4. 24: Calculation of FOS when assumed value of FOS = 1 at 62+300

Slice no.	Witdth	Mid height	Base length (b)	Weight (KN)	Base angle (degree)	m_α	W*sin α	c*b	(c*b+(W-ub)*tan φ)/ m_α
1	1.79	1.87	4.56	65.49	66.79	0.95	60.19	36.5	80.13
2	1.79	5.18	3.79	181.5	61.7	1.00	159.81	30.3	138.93
3	2	6.92	3.69	270.07	57.13	1.05	226.83	29.5	183.08
4	2	7.29	3.31	284.45	52.86	1.08	226.75	26.5	182.33
5	2	7.26	3.05	283.29	48.98	1.11	213.74	24.4	175.36
6	2	6.92	2.85	270.16	48.37	1.11	201.93	22.8	166.25
7	2	6.34	2.69	247.3	42	1.15	165.48	21.5	148.54
8	2	5.54	2.56	216.27	38.79	1.16	135.49	20.5	130.14
9	2	4.56	2.46	178.17	35.72	1.16	104.02	19.7	109.00
10	2	3.43	2.37	133.82	32.76	1.17	72.41	19.0	85.21
11	2	2.15	2.30	83.85	29.9	1.17	41.80	18.4	58.97
12	2	0.73	2.24	28.77	27.12	1.16	13.11	17.9	30.25
						Sum	1621.5	Sum	1488.21

Calculated FOS= 0.918

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 0.918

Table 4. 25: Calculation of FOS when assumed value of FOS = 0.918 at 62+300

Slice no.	Base length(b)	Weight	Base angle	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	4.56	65.49	66.79	1.00	60.19	36.48	76.16
2	3.788	181.5	61.7	1.05	159.81	30.3	132.68
3	3.686	270.07	57.13	1.09	226.83	29.49	175.53
4	3.313	284.45	52.86	1.13	226.75	26.5	175.40
5	3.047	283.29	48.98	1.15	213.74	24.38	169.19
6	2.847	270.16	48.37	1.15	201.93	22.78	160.47
7	2.69	247.3	42	1.18	165.48	21.52	144.03
8	2.56	216.27	38.79	1.19	135.49	20.48	126.46
9	2.46	178.17	35.72	1.19	104.02	19.68	106.14
10	2.37	133.82	32.76	1.20	72.41	18.96	83.14
11	2.3	83.85	29.9	1.19	41.80	18.4	57.65
12	2.24	28.77	27.12	1.19	13.11	17.92	29.62
				Sum	1621.5	Sum	1436.47

Calculated FOS= 0.866

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 0.866

Table 4. 26: Calculation of FOS when assumed value of FOS = 0.86 at 62+300

Slice no.	Base length(b)	Weight	Base angle(degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	4.56	65.49	66.79	1.02	60.19	36.48	74.53
2	3.788	181.5	61.7	1.08	159.81	30.3	128.44
3	3.686	270.07	57.13	1.13	226.83	29.49	170.38
4	3.313	284.45	52.86	1.16	226.75	26.5	170.65
5	3.047	283.29	48.98	1.18	213.74	24.38	164.94
6	2.847	270.16	48.37	1.18	201.93	22.78	156.48
7	2.69	247.3	42	1.21	165.48	21.52	140.89
8	2.56	216.27	38.79	1.21	135.49	20.48	123.90
9	2.46	178.17	35.72	1.22	104.02	19.68	104.14
10	2.37	133.82	32.76	1.22	72.41	18.96	81.69
11	2.3	83.85	29.9	1.21	41.80	18.4	56.72
12	2.24	28.77	27.12	1.21	13.11	17.92	29.18
				Sum	1621.56	Sum	1401.95

Calculated FOS= 0.86

The assumed value of factor of safety is equal to calculated FoS

Hence, the factor of safety is 0.86

2. Stability analysis of reinforced slope

Here analytical calculations only for critical failure plane are presented other calculations are attached in Annex section.

Initial consideration

Height of cut slope= 25m

Face batters (α)=-39^0 and β=-14^0

Nail type= Grouted nail

Nail spacing S_v=1.562, S_h=1m

Soil nail inclination (i)=35^0

Soil nail material=Grade Fe415,f_y=415kN/m^2

Soil Parameters

Cohesion(c)	Internal friction angle	Unit weight(γ)
8 kPa	31^0	19.5 kN/m^3

Preliminary Design

A) Determination of maximum axial force T_{max}

Maximum axial tensile force T_{max} developed is given by

T_{max}(kN)= $k_a(q_s+\gamma H)$* S_v*S_h Equation 4.3

Here, there is no surcharge so $q_{s=0,}$

$$K_a = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha.\cos(\alpha+\beta)\left[1+\sqrt{\frac{\sin(\phi-\beta).\sin(\phi+\beta)}{\cos(\alpha+\beta).\cos(\beta-\alpha)}}\right]^2} \quad \text{Equation 4.4}$$

K_a = 0.095

T_{max}(kN)=0.095 (0+19.5*25)*1.562*1=72.79 kN

B) Determination of maximum nails length (L) and nail diameter (d)

Factor of safety against nail tensile failure FS_T=1.8

The required cross sectional area of nail bar can be determined as:

$A_t(mm^2)= (T_{max}* FS_T)/f_y$=(72.79*1000*1.8)/415=315.71 mm^2

Selecting reinforcement bar of diameter of 25 mm with cross sectional area of A_t=490.87 mm^2(>315.71 mm^2)

Adopt soil nail length

L= 16 m > 0.6*H

Summary adopt driven soil nail of 25 mm dia. and 16 m length.

C) Check for important failure modes

1) Global stability

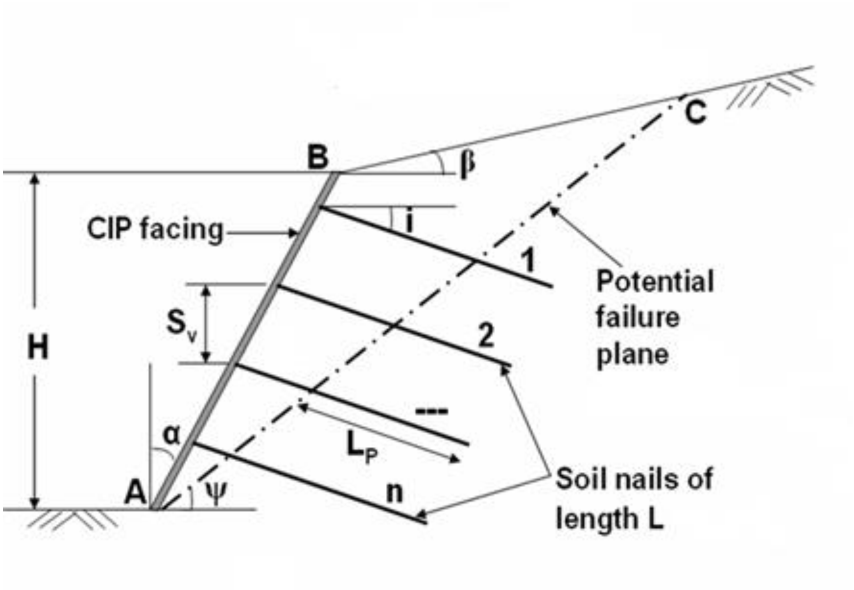


Figure 4. 57: Global stability failure of soil nail slope

Determination of equivalent nail force T_{eq}

R_p(kN)=Nail pullout capacity=πd L_P q_u Equation 4.5

q_u is ultimate bond strength which is obtained from in-situ pullout test R_p can be calculated as

R_p(kN)=($c+\sigma_v \tan\theta$) πd* L_P Equation 4.6

Where L_P = nail length beyond failure surface

θ = mobilized soil interface angle = Φ*2/3 and Φ is soil effective angle of internal friction.

For jth nail, $\sigma_v = \gamma h_j + Q$ Equation 4.7

Where, h_j= Depth of midpoint of jth nail from ground surface.

γ=unit weight of soil

Q=Surcharge acting on slope

Overburden pressure (σ_v) may be calculated at midpoint of whole nail length instead of middle of L_P.

The L_P =L-((H-Z)cos($\psi+\alpha$)/(cosα*sin(ψ+i)) Equation 4.8

Where, i= nail inclination and ψ= 37 (For critical failure plane)

Here ψ= 37

R_T(kN)=Nail Tensile capacity=(π*d^2*f_y/4000)=200 kN

The allowable axial force carrying capacity T_{all}(kN) of nail embed at depth z is the minimum of R_p and R_T. The allowable axial force carrying capacity of nails at different level can be calculated in following table.

Table 4. 27: Allowable axial force carrying capacity of nails at different level

Nail No. j (from top)	Depth of nail z(m)	Effective pullout length L_P (m)	Nail pullout capacity R_p (kN)	Nail Tensile capacity R_T (kN)	Allowable axial force carrying capacity T_{all}(kN)
1	6.22	7.58	76.97	200	76.97
2	7.5	8.14	98.61	200	98.61
3	8.7	8.67	120.97	200	120.97
4	9.9	9.2	145.27	200	145.27
5	11.24	9.79	174.68	200	174.68
6	12.42	10.25	201.41	200	200.00
7	12.8	10.78	218.10	200	200.00
8	12.8	11.3	228.62	200	200.00
9	12.8	11.83	239.34	200	200.00
10	12.8	12.36	250.07	200	200.00
11	12.8	12.88	260.59	200	200.00
12	12.8	s13.41	271.31	200	200.00
13	12.8	13.94	282.03	200	200.00
14	12.8	14.416	291.66	200	200.00
15	12.8	14.99	303.28	200	200.00
16	12.8	15.52	314.00	200	200.00
17	12.8	16	323.71	200	200.00
				Sum(T_{all})	3016.51

Now,T_{eq}= Sum(T_{all})/S_h Equation 4.9

T_{eq}=3016.51 /1=3016.51kN

b) Determination of weight of failure wedge

Length of failure surface(L_F) = 50 m

Area of failure wedge = 193.84 m^2 (Here ,length and area measured using Autocad-2007)

W(kN) = Area of failure wedge x Unit weight of soil

W(kN) = 193.84x19.5= 3779.88kN/m

Now Global factor of safety FS_G under static condition is given by

FS_G= Resisting force /Driving force

Resting force= $cL_F + T_{eq}\cos(\psi - i) + (W^*\cos\psi + T_{eq}\sin(\psi - i))\tan\Phi$= 4585.94 kN

Driving force= $W^*\sin\psi$ = 2937.51 kN

FS = 4585.94 /2937.51 = 1.561

External stability analysis of reinforced slope

Analysis is done for external stability of soil nailed slope as shown in fig

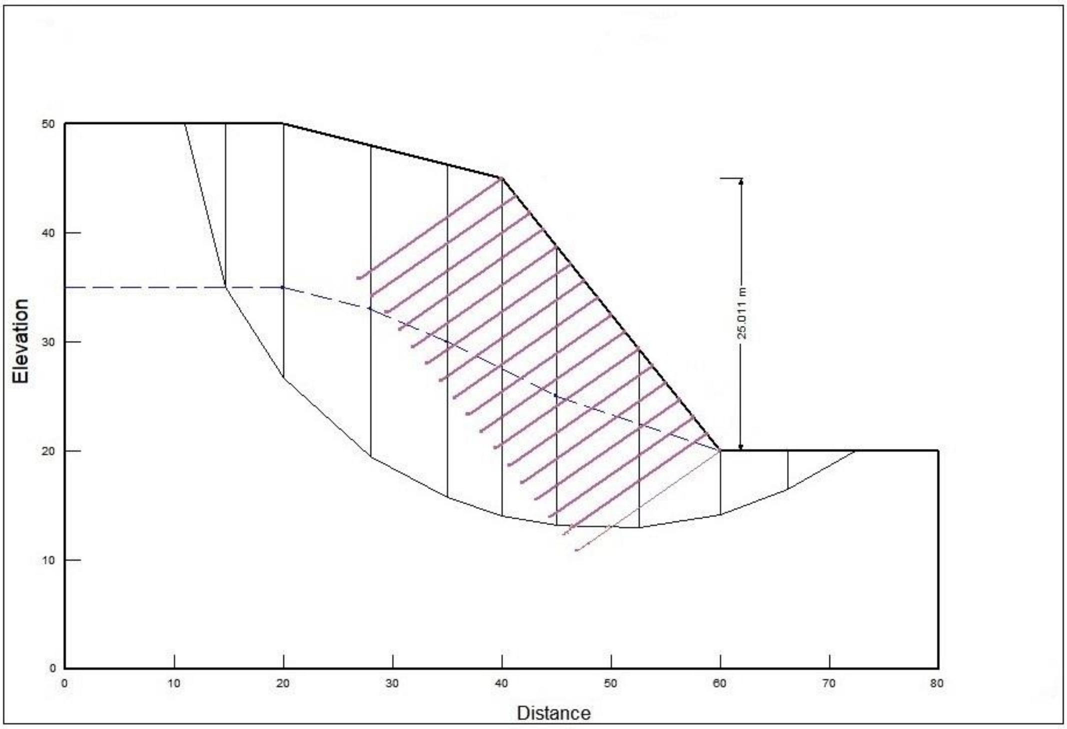


Figure 4. 58: problem statement for external stability check of nailed slope chainage 62+300

Radius of Slip Surface(R) = 38.941 m

Co-ordinate of center of slip surface(x, y) = (49.80, 51.77), (Here radius of slip surface and Co-ordinate are measured using Autocad-2007)

Height of cut slope (H) =25m

The calculations are done in tabular form. The value of m_α has been computed using equation 4.1

For first trail F_s is assumed to be 1. The factor of Safety is calculated by using

formula. 4.2

Here, Cohesion of soil(c) = 8 kPa

Internal Friction angle = 31^0

Assumed value of FOS= 1

Table 4. 28: Calculation of FOS when assumed value of FOS = 1 at 62+300

Assumed FoS 1

Slice no.	Witdth	Mid height	Base length (b)	Wt. (kN)	Base angle (degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	2.55	4.6	1.46	549	75.93	0.83	532.53	11.7	413.53
2	2.55	11	9.85	1993	57.21	1.05	1675.4	78.8	1219.39
3	2.55	14.13	10.76	4038	41.96	1.15	2699.8	86.1	2193.52
4	2.5	16.29	7.94	4025	28.19	1.17	1901.4	63.5	2130.04
5	2.5	17.86	5.25	2993	18.46	1.14	947.71	42.0	1616.07
6	2.5	18.97	5.09	2756	10.83	1.10	517.84	40.7	1549.37
7	2.5	17.58	7.50	3076	1.55	1.02	83.20	60.0	1878.41
8	2.5	13.73	7.60	1627	-9.55	0.89	-269.93	60.8	1171.41
9	2.5	9.5	6.55	562	-20	0.73	-192.22	52.4	531.32
10	2.5	5.1	7.11	214	-30	0.57	-107.00	56.9	327.91
						Sum	7788.8		13030.98
calculated FoS									1.67

Calculated FOS= 1.67

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 1.67

Table 4. 29: Calculation of FOS when assumed value of FOS = 1.67 at 62+300

Slice no.	Base length(b)	Weight	Base angle(degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	1.46	549	75.93	0.59	532.53	11.7	576.84
2	9.85	1993	57.21	0.84	1675.44	78.8	1512.18
3	10.76	4038	41.96	0.98	2699.86	86.1	2552.76
4	7.94	4025	28.19	1.05	1901.40	63.5	2360.76
5	5.25	2993	18.46	1.06	947.71	42.0	1732.17
6	5.09	2756	10.83	1.05	517.84	40.7	1616.22
7	7.50	3076	1.55	1.01	83.20	60.0	1890.54
8	7.60	1627	-9.55	0.93	-269.93	60.8	1120.84
9	6.55	562	-20	0.82	-192.22	52.4	477.67
10	7.11	214	-30	0.69	-107.00	56.9	270.31
				Sum	7788.84	Sum	14110.29
calculated FoS							1.81

Calculated FOS= 1.81

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 1.81

Table 4. 30: Calculation of FOS when assumed value of FOS = 1.81 at 62+300

Slice no.	Base length (b)	Weight	Base angle (degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	1.46	549	75.93	0.57	532.53	11.7	604.40
2	9.85	1993	57.21	0.82	1675.44	78.8	1555.29
3	10.76	4038	41.96	0.97	2699.86	86.1	2601.95
4	7.94	4025	28.19	1.04	1901.40	63.5	2390.65
5	5.25	2993	18.46	1.05	947.71	42.0	1746.66
6	5.09	2756	10.83	1.04	517.84	40.7	1624.31
7	7.50	3076	1.55	1.01	83.20	60.0	1891.96
8	7.60	1627	-9.55	0.93	-269.93	60.8	1115.29
9	6.55	562	-20	0.83	-192.22	52.4	472.17
10	7.11	214	-30	0.70	-107.00	56.9	264.93
				Sum	7788.84	Sum	14267.60
calculated FoS							1.83

The assumed value of factor of safety is not equal to calculated FoS

The process is repeated after taking FOS = 1.83

Table 4. 31: Calculation of FOS when assumed value of FOS = 1.83 at 62+300

Slice no.	Base length(b)	Weight	Base angle(degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	1.46	549	75.93	0.56	532.53	11.7	608.19
2	9.85	1993	57.21	0.82	1675.44	78.8	1561.09
3	10.76	4038	41.96	0.96	2699.86	86.1	2608.50
4	7.94	4025	28.19	1.04	1901.40	63.5	2394.61
5	5.25	2993	18.46	1.05	947.71	42.0	1748.56
6	5.09	2756	10.83	1.04	517.84	40.7	1625.37
7	7.50	3076	1.55	1.01	83.20	60.0	1892.14
8	7.60	1627	-9.55	0.93	-269.93	60.8	1114.56
9	6.55	562	-20	0.83	-192.22	52.4	471.46
10	7.11	214	-30	0.70	-107.00	56.9	264.25
				Sum	7788.84	Sum	14288.73
calculated FoS							1.83

Here calculated FoS is equal to assumed FoS, So correct FoS is 1.83

4.9.2 For slope section SS2 (68+300)

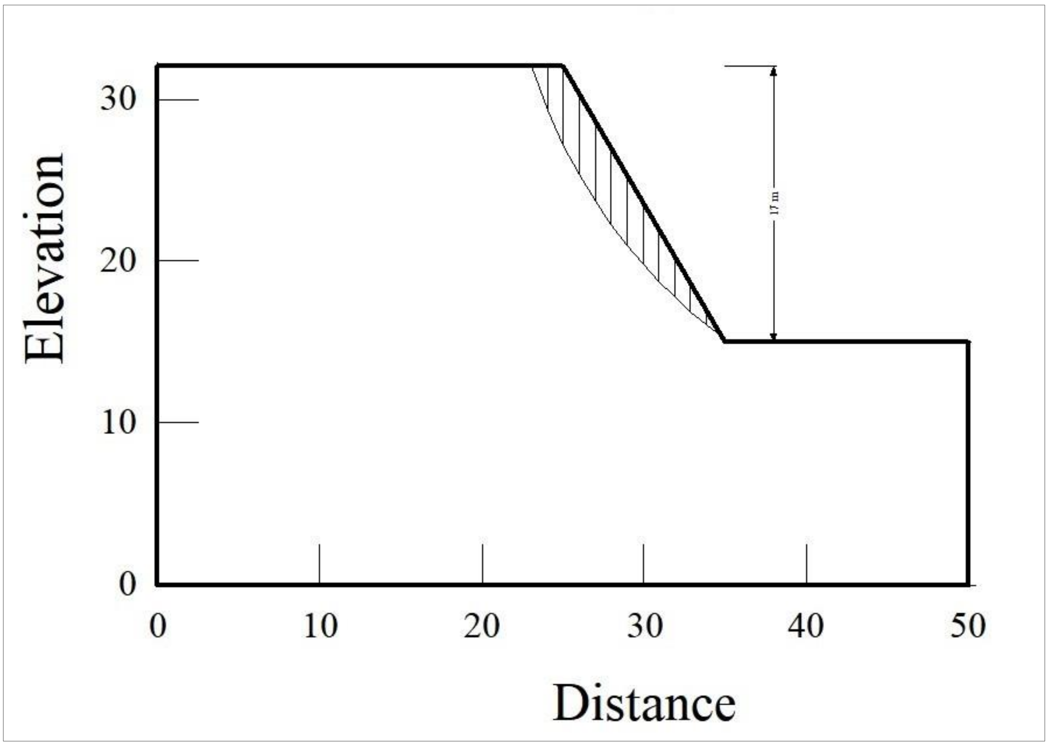


Figure 4. 59: problem statement for hand caulation at chainage 68+300

Radius of Slip Surface(R) = 33.18 m

Co-ordinate of center of slip surface(x, y) = (54.7, 41.7), (Here radius of slip surface and Co-ordinate are measured using Autocad-2007)

Height of cut slope (H) =17m

The calculations are done in tabular form. The value of m_{α} has been computed using equation 4.1

For first trail F_s is assumed to be 1. The factor of Safety is calculated by using equation 4.2

Here, Cohesion of soil(c) = 6 kPa

Internal Friction angle = 33^0

Assumed value of FOS= 1

Table 4. 32: Calculation of FOS when assumed value of FOS = 1 at (68+300)

Slice no.	Width (m)	Mid height	Base length (b)	Weight (kN)	Base angle (degree)	m_α	W*sin α	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	0.95	1.34	2.63	23.51	70.39	0.95	22.15	15.8	32.16
2	0.95	1.76	2.21	65.75	65.89	1.00	60.01	13.3	54.28
3	0.98	4.92	2.00	88.17	62.04	1.04	77.88	12.0	64.36
4	0.98	4.98	1.55	89.25	58.59	1.08	76.17	9.3	60.51
5	0.98	4.83	1.43	86.53	55.46	1.10	71.28	8.6	56.86
6	0.98	4.51	1.40	80.89	52.56	1.12	64.23	8.4	52.46
7	0.98	4.07	1.31	72.9	49.84	1.14	55.71	7.9	46.80
8	0.98	3.51	1.25	62.92	47.27	1.16	46.22	7.5	40.51
9	0.98	2.86	1.18	51.27	44.81	1.17	36.13	7.1	33.52
10	0.98	2.13	1.33	38.14	42.46	1.18	25.75	8.0	27.05
11	0.98	1.32	1.28	23.7	40.19	1.18	15.29	7.7	19.01
12	0.98	0.45	1.04	8.1	38.02	1.19	4.99	6.2	9.51
					Sum		555.81		497.04
Calculated FoS									0.89

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 0.89

Table 4. 33: Calculation of FOS when assumed value of FOS = 0.918 at (68+300)

Slice no.	Base length (b)	Weight	Base angle (degree)	m_α	W*sinα	c*b	(c*b+W*tanφ)/ m_α
1	2.63	23.51	70.39	1.02	22.15	15.8	29.79
2	2.21	65.75	65.89	1.07	60.01	13.3	50.58
3	2.00	88.17	62.04	1.11	77.88	12.0	60.26
4	1.55	89.25	58.59	1.14	76.17	9.3	56.88
5	1.43	86.53	55.46	1.17	71.28	8.6	53.64
6	1.40	80.89	52.56	1.19	64.23	8.4	49.65
7	1.31	72.9	49.84	1.20	55.71	7.9	44.42
8	1.25	62.92	47.27	1.21	46.22	7.5	38.55
9	1.18	51.27	44.81	1.22	36.13	7.1	31.97
10	1.33	38.14	42.46	1.23	25.75	8.0	25.86
11	1.28	23.7	40.19	1.23	15.29	7.7	18.21
12	1.04	8.1	38.02	1.24	4.99	6.2	9.13
				Sum	555.81	Sum	468.93
calculated FoS							0.84

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 0.84

Table 4. 34: Calculation of FOS when assumed value of FOS = 0.84 at (68+300)

Slice no.	Base length (b)	Weight	Base angle (degree)	m_α	W*sinα	c*b	(c*b+W*tanφ)/ m_α
1	2.63	23.51	70.39	1.06	22.15	15.8	28.64
2	2.21	65.75	65.89	1.11	60.01	13.3	48.78
3	2.00	88.17	62.04	1.15	77.88	12.0	58.26
4	1.55	89.25	58.59	1.18	76.17	9.3	55.10
5	1.43	86.53	55.46	1.20	71.28	8.6	52.04
6	1.40	80.89	52.56	1.22	64.23	8.4	48.25
7	1.31	72.9	49.84	1.24	55.71	7.9	43.22
8	1.25	62.92	47.27	1.25	46.22	7.5	37.56
9	1.18	51.27	44.81	1.25	36.13	7.1	31.19
10	1.33	38.14	42.46	1.26	25.75	8.0	25.26
11	1.28	23.7	40.19	1.26	15.29	7.7	17.81
12	1.04	8.1	38.02	1.26	4.99	6.2	8.94
				Sum	555.81	Sum	455.04
Calculated FoS							0.82

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 0.82

Table 4. 35: Calculation of FOS when assumed value of FOS = 0.82 at (68+300)

Slice no.	Base length (b)	Weight	Base angle (degree)	m_α	W*sinα	c*b	(c*b+W*tanφ)/ m_α
1	2.63	23.51	70.39	1.08	22.15	15.8	28.17
2	2.21	65.75	65.89	1.13	60.01	13.3	48.04
3	2.00	88.17	62.04	1.17	77.88	12.0	57.43
4	1.55	89.25	58.59	1.20	76.17	9.3	54.36
5	1.43	86.53	55.46	1.22	71.28	8.6	51.38
6	1.40	80.89	52.56	1.24	64.23	8.4	47.66
7	1.31	72.9	49.84	1.25	55.71	7.9	42.72
8	1.25	62.92	47.27	1.26	46.22	7.5	37.15
9	1.18	51.27	44.81	1.27	36.13	7.1	30.86
10	1.33	38.14	42.46	1.27	25.75	8.0	25.00
11	1.28	23.7	40.19	1.27	15.29	7.7	17.64
12	1.04	8.1	38.02	1.28	4.99	6.2	8.86
				Sum	555.81	Sum	449.26
calculated FoS							0.80

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 0.82

Table 4. 36: Calculation of FOS when assumed value of FOS = 0.80 at (68+300)

Slice no.	Base length (b)	Weight	Base angle (degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	2.63	23.51	70.39	1.10	22.15	15.8	27.69
2	2.21	65.75	65.89	1.15	60.01	13.3	47.28
3	2.00	88.17	62.04	1.19	77.88	12.0	56.58
4	1.55	89.25	58.59	1.21	76.17	9.3	53.60
5	1.43	86.53	55.46	1.24	71.28	8.6	50.70
6	1.40	80.89	52.56	1.25	64.23	8.4	47.06
7	1.31	72.9	49.84	1.27	55.71	7.9	42.21
8	1.25	62.92	47.27	1.27	46.22	7.5	36.72
9	1.18	51.27	44.81	1.28	36.13	7.1	30.52
10	1.33	38.14	42.46	1.29	25.75	8.0	24.74
11	1.28	23.7	40.19	1.29	15.29	7.7	17.46
12	1.04	8.1	38.02	1.29	4.99	6.2	8.78
				Sum	555.81	Sum	443.36
calculated FoS							0.80

Hence, the factor of safety is 0.80

Stability analysis of reinforced slope

Here analytical calculations only for critical failure plane are presented other calculations are attached in Annex section.

Initial consideration

Height of cut slope= 17 m

Face batters (α)=-30^0 and β= 0^0

Nail type= Grouted nail

Nail spacing S_v=2, S_h=1.5 m

Soil nail inclination (i)=20 ^{0}

Soil nail material=Grade Fe415,f_y=415kN/m^2

Soil Parameters

Cohesion(c)	Internal friction angle	Unit weight(γ)
6 kPa	33^0	18.2 kN/m^3

Preliminary Design

a) Determination of maximum axial force T_{max}

Maximum axial tensile force T_{max} developed is given by

K_a = 0.106 (using equation 4.4)

T_{max}(kN)=0.106 (0+18.2*17)*2*1.5= 65.59 kN

B) Determination of maximum nails length (L) and nail diameter (d)

Factor of safety against nail tensile failure FS_T=1.8

The required cross sectional area of nail bar can be determined as:

$A_t(mm^2)= (T_{max} * FS_T)/f_y=(65.59*1000*1.8)/415=284.48\ mm^2$

Selecting reinforcement bar of diameter of 25 mm with cross sectional area of A_t=490.87 mm^2(>284.48 mm^2)

Adopt soil nail length

L= 11 m > 0.6*H

Summary adopt driven soil nail of 25 mm dia. and 11 m length.

C) Check for important failure modes

1) Inernal stability

i= 20 degree and ψ= 43 degree (For critical failure plane)

Here ψ= 43 degree, nail length = 11 m

R_T(kN)=Nail Tensile capacity=($\pi*d^2*f_y/4000$)=200 kN

The allowable axial force carrying capacity T_{all}(kN) of nail embed at depth z is the minimum of R_p and R_T. The allowable axial force carrying capacity of nails at different level can be calculated in following table.

Table 4. 37: Allowable axial force carrying capacity of nails at different level at (68+300)

Nail No. j (from top)	Depth of nail z(m)	Effective pullout length L_P (m)	Nail pullout capacity R_p (kN)	Nail Tensile capacity R_T (kN)	Allowable axial force carrying capacity T_{all}(kN)
1	2.87	4.13	18.89	200	18.89
2	4.87	4.90	36.42	200	36.42
3	6.88	5.89	60.70	200	60.70
4	8.87	6.75	88.76	200	88.76
5	10.61	7.50	117.28	200	117.28
6	10.61	3.38	52.85	200	52.85
7	10.61	9.23	144.33	200	144.33
8	10.61	10.00	156.37	200	156.37
9	10.61	11.00	172.01	200	172.01
				Sum(T_{all})	847.63

T_{eq}= 847.63 kN

b) Determination of weight of failure wedge

Length of failure surface(L_F) = 26 m

Area of failure wedge = 82.05 m^2 (Here ,length and area measured using Autocad-2007)

W(kN) = Area of failure wedge x Unit weight of soil

W(kN) = 82.05x18.2 = 1493.31kN/m

Now Global factor of safety FS_G under static condition is given by

FS_G= Resisting force /Driving force

Resting force= $cL_F + T_{eq}\cos(\psi\text{-}i)+(W^*\cos\psi+ T_{eq}\sin(\psi\text{-}i))\tan\Phi$= 1525.25 kN

Driving force= $W^*\sin\psi$ = 979.69 kN

FS = 1525.25 /979.69 = 1.55

External stability check

Analysis is done for external stability of soil nailed slope as shown in fig

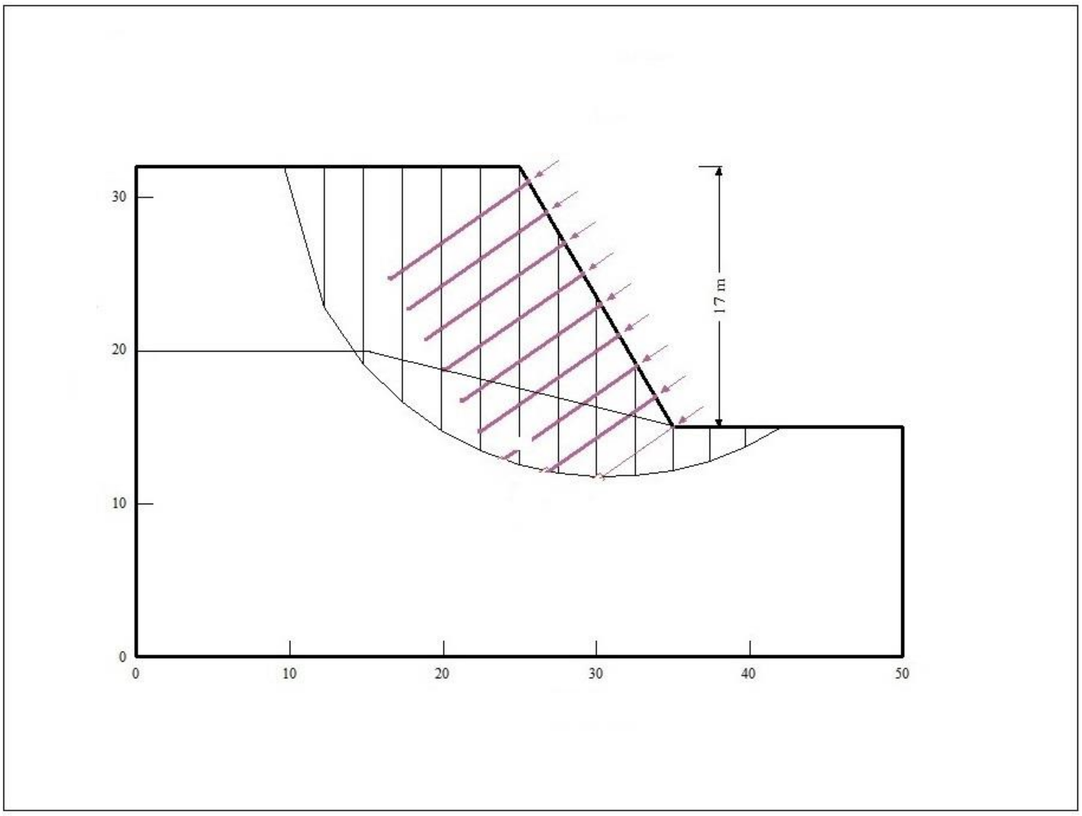


Figure 4. 60: External stability check by hand calculation of nailed slope chainage 68+300

Radius of Slip Surface(R) = 21.2 m

Co-ordinate of center of slip surface(x, y) = (30.84, 32.95), (Here radius of slip surface and Co-ordinate are measured using Autocad-2007)

Height of cut slope(H) =17m

The calculations are done in tabular form. The value of m_{α} has been computed using equation 4.1

For first trail F_s is assumed to be 1. The factor of Safety is calculated by using formula. 4.2

Here, Cohesion of soil(c) = 6 kPa

Internal Friction angle = 33^0

Assumed value of FOS= 1

Table 4. 38: Calculation of FOS when assumed value of FOS = 1 at (68+300)

Slice no.	Width	Mid height	Base length (b)	Wt. (kN)	Base angle (degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	2.55	4.6	9.53	213	74.44	0.89	205.19	57.2	218.72
2	2.55	11	4.44	513	55.36	1.10	422.07	26.6	326.27
3	2.55	14.13	3.50	657	44.43	1.17	459.93	21.0	383.04
4	2.5	16.29	3.10	758	35.53	1.19	440.50	18.6	428.85
5	2.5	17.86	2.80	831	27.27	1.19	380.75	16.8	469.03
6	2.5	18.97	2.70	883	19.69	1.16	297.51	16.2	508.15
7	2.5	17.58	2.50	800	12.53	1.12	173.56	15.0	478.51
8	2.5	13.73	2.40	625	5.67	1.06	61.75	14.4	396.77
9	2.5	9.5	2.50	436	-1	0.99	-7.61	15.0	301.82
10	2.5	5.1	2.50	234	-7.88	0.90	-32.08	15.0	185.19
11	2.3	2.53	2.45	109	-14.62	0.80	-27.69	14.7	106.92
12	2.3	1.75	2.55	76.0	-21.39	0.69	-27.75	15.3	93.21
13	2.3	0.64	2.57	27.9	-28.5	0.57	-13.34	15.4	59.02
						Sum	2346.1		3896.48
calculated FoS									1.66

Calculated FOS= 1.66

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 1.66

Table 4. 39: Calculation of FOS when assumed value of FOS = 1.66 at (68+300)

Slice no.	Base length(b)	Weight	Base angle(degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	9.53	213	74.44	0.65	205.19	57.2	303.05
2	4.44	513	55.36	0.89	422.07	26.6	404.13
3	3.50	657	44.43	0.99	459.93	21.0	453.11
4	3.10	758	35.53	1.04	440.50	18.6	490.66
5	2.80	831	27.27	1.07	380.75	16.8	520.98
6	2.70	883	19.69	1.07	297.51	16.2	549.34
7	2.50	800	12.53	1.06	173.56	15.0	503.77
8	2.40	625	5.67	1.03	61.75	14.4	406.56
9	2.50	436.3	-1	0.99	-7.61	15.0	300.45
10	2.50	234	-7.88	0.94	-32.08	15.0	178.19
11	2.45	109.7	-14.62	0.87	-27.69	14.7	98.90
12	2.55	76.09	-21.39	0.79	-27.75	15.3	82.08
13	2.57	27.96	-28.5	0.69	-13.34	15.4	48.51
				Sum	2346.12	Sum	4291.21
calculated FoS							1.83

Calculated FOS= 1.83

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 1.83

Table 4. 40: Calculation of FOS when assumed value of FOS = 1.83 at (68+300)

Slice no.	Base length (b)	Weight	Base angle (degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	1.46	549	75.93	0.57	532.53	11.7	604.40
2	9.85	1993	57.21	0.82	1675.44	78.8	1555.29
3	10.76	4038	41.96	0.97	2699.86	86.1	2601.95
4	7.94	4025	28.19	1.04	1901.40	63.5	2390.65
5	5.25	2993	18.46	1.05	947.71	42.0	1746.66
6	5.09	2756	10.83	1.04	517.84	40.7	1624.31
7	7.50	3076	1.55	1.01	83.20	60.0	1891.96
8	7.60	1627	-9.55	0.93	-269.93	60.8	1115.29
9	6.55	562	-20	0.83	-192.22	52.4	472.17
10	7.11	214	-30	0.70	-107.00	56.9	264.93
				Sum	7788.84	Sum	14267.60
calculated FoS							1.83

The assumed value of factor of safety is not equal to calculated FoS

The process is repeated after taking FOS = 1.83

Table 4. 41: Calculation of FOS when assumed value of FOS = 1.83 at (68+300)

Slice no.	Base length (b)	Weight	Base angle (degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	9.53	213	74.44	0.61	205.19	57.2	320.44
2	4.44	513	55.36	0.86	422.07	26.6	418.17
3	3.50	657	44.43	0.96	459.93	21.0	465.09
4	3.10	758	35.53	1.02	440.50	18.6	500.82
5	2.80	831	27.27	1.05	380.75	16.8	529.23
6	2.70	883	19.69	1.06	297.51	16.2	555.68
7	2.50	800	12.53	1.05	173.56	15.0	507.54
8	2.40	625	5.67	1.03	61.75	14.4	407.97
9	2.50	436.3	-1	0.99	-7.61	15.0	300.26
10	2.50	234	-7.88	0.94	-32.08	15.0	177.25
11	2.45	109.7	-14.62	0.88	-27.69	14.7	97.87

12	2.55	76.09	-21.39	0.80	-27.75	15.3	80.72
13	2.57	27.96	-28.5	0.71	-13.34	15.4	47.33
				Sum	2346.12	Sum	4361.04
calculated FoS							1.86

The assumed value of factor of safety is not correct. The process is repeated after taking FOS = 1.86

Table 4. 42: Calculation of FOS when assumed value of FOS = 1.86 at (68+300)

Slice no.	Base length(b)	Weight	Base angle (degree)	m_α	W*sinα	c*b	(c*b+(W-ub)*tanφ)/ m_α
1	9.53	213	74.44	0.60	205.19	57.2	323.36
2	4.44	513	55.36	0.86	422.07	26.6	420.47
3	3.50	657	44.43	0.96	459.93	21.0	467.03
4	3.10	758	35.53	1.02	440.50	18.6	502.46
5	2.80	831	27.27	1.05	380.75	16.8	530.55
6	2.70	883	19.69	1.06	297.51	16.2	556.69
7	2.50	800	12.53	1.05	173.56	15.0	508.14
8	2.40	625	5.67	1.03	61.75	14.4	408.20
9	2.50	436.3	-1	0.99	-7.61	15.0	300.23
10	2.50	234	-7.88	0.94	-32.08	15.0	177.10
11	2.45	109.7	-14.62	0.88	-27.69	14.7	97.71
12	2.55	76.09	-21.39	0.80	-27.75	15.3	80.51
13	2.57	27.96	-28.5	0.71	-13.34	15.4	47.14
				Sum	2346.12	Sum	4372.45
calculated FoS							1.86

Here assumed FoS and calculated FoS is equal. Hence, the factor of safety is 1.86

5. CONCLUSION AND RECOMMENDATION

5.1 Conclusions

The following conclusion can be extracted from the cut slope study of Kanti Lokpath at three different slope sections.

- According to Peck et. al. (1998) classification the slope based on factor of safety first two slope section are of very high land slide susceptibility (FOS < 1) and third slope section is of high landslide susceptibility (1.25> FOS > 1) so requires stabilization measures to mitigate the problem of instability of cut slope.
- From the results of numerical calculations, it is found that the four parameters studied have significant influence on the stability of cut slope specially when there is increase in ground water table highly reduce the factor of safety.
- As stabilization measures retaining wall is not effective solution for steep and high slope but may be appropriate for less steep and low height soil slope. For steep and high slope soil nail with sufficient length, strength and inclination is more effective.
- From the results obtained from the analysis and assessment carried out in this study, it can be concluded that the soil nails inclination has significant effect on the stability of the soil slope. The inclination of soil nails depends on the slope angle, slope geometry and strength parameter of soil and nail. Besides that, optimum nail inclination for a steep slope should be lesser than the angle of soil nails of a gentle slope.
- Optimum angle of inclination of soil nail is found 35 to 40 degree for slope section SS1 and 20 to 25 degree from horizontal for slope section SS2

5.2 Recommendation for Future Study

This study can further be elaborated in respect of the following areas for further research

1. In order to evaluate better influence of critical factors for slope instability, further detailed geotechnical investigations, geophysical and topographic survey should be performed.

2. Dynamic stability analysis can be performed by incorporating seismic parameters for both unreinforced and reinforced cut slope.
3. The external load factors (tress and others loads, surface loads, line loads can be used in slope stability analysis.

REFERENCES

1. Abramson LW, Lee TS, Sharma S, Boyce GM (2001). Slope Stability and Stabilization Methods, John Wiley and Sons.
2. Abramson, L. W., Lee, T. S., Sharma, S., and Boyce, G. M. (2002). Slope Stability Concepts. Slope Stabilization and Stabilization Methods, Second edition, published by John Willey & Sons, Inc., pp. 329-461.
3. Arora, K. R., 1997.Soil Mechanics and Foundation Engineering. Standard Publishers Distributers, Delhi, India.
4. Bishop, A. W. (1955). The use of slip circles in stability analysis of slopes. Geotechnique, 5(1), 7-17.
5. D.G. Fredlund. (2001) The relationship between Limit Equilibrium Slope Stability Methods, Department of Civil Engineering, University of Saskatchewan, Saskatoon, Saskatchewan, Canada pp 1-8.
6. Duncan, J. M. (1996): State of the Art: Limit Equilibrium and Finite Element Analysis in Slopes. Journal of Geotechnical Engineering, Vol. 122 (7), pp. 577-96.
7. Duncan, J.M. and Wright S.G. (2005). Soil Strength and Slope Stability. John Wiley & Sons, Inc. Hoboken, New Jersey. Earth science data interface, Global land cover facility. URL: http://glcfpp.glcf.umd.edu:8080/esdi/index.jsp.
8. Geo-studio, 2010. Stability Modeling with SLOPE/W 2007 Version, Geo-Slope International Ltd., Calgary, Alberta, Canada.
9. Hoek E., and Bray J.W., (1981).Rock slope engineering, 3rd ed., Institution of Mining & Metal., London, 402pp.
10. Janbu, N. (1954). Application of Composite Slip Surface for Stability Analysis. European Conference on Stability of Earth Slopes, Stockholm.
11. Janbu, N. (1973). Slope Stability Computations. Embankment Dam Engineering, Casagrande Volume, pp. 47-86.
12. Janbu, N. (1996). Slope Stability Evaluations in engineering practice.7th International Symposium on Landslides, Trondheim, Norway, Vol. 1 pp. 17-34.
13. Krahn, J. (2004). Stability Modeling with SLOPE/W. An Engineering Methodology, Published by Geo-Slope International.

14. Morgenstern, N., & Price, V. (1965). The Analysis of the Stability of General Slip Surfaces. Geotechnique, Vol. 15 (No. 1), pp. 77-93.
15. Poulami Ghosh. 2012. Effect of reinforcement on stability of slopes using geoslope, International Conference on Benchmarks in Engineering Science and Technology ICBEST 2012 Proceedings published by International Journal of Computer Applications® (IJCA).
16. Rouaiguia, A. and Dahim, M.A. 2013. Numerical Modeling of Slope Stability Analysis. International Journal of Engineering Science and Innovative Technology. 2(3), 533-542.
17. Sarma, S. K. (1973). Stability Analysis of Embankment and Slopes. Geotechnique, Vol. 23 (3), pp.423-33.
18. Spencer, E. (1967). A method of Analysis of the Stability of Embankments, Assuming Parallel Interslice Forces. Geotechnique, Vol. 17, pp. 11-26.

APPENDIXES

APPENDIX A

Lab Reports

Field density of soil		
Volume of soil	0.000965	m3
Wt. of soil	1.883	Kg
Density	1950.73	kg/m3

1. Shear test result of chainage 62+300

Horizontal dial Reading

(0.01mm)

Horizontal Displacement

(mm)

Load Dial Reading (0.17

kg)

Horizontal Shear Force (kg)

Shear Stress =2 kg/cm2

S.N	Hz. Dial Reading (0.01mm)	Hz. Displacement (mm)	Normal Stress (kg/cm^2)	Load Dial Reading	Calibration Factor (kg/div)	Shear Force	Shear Stress (kg/cm^2)
1	20	0.2	0.2	8	0.17	1.36	0.054
2	40	0.4	0.2	11	0.17	1.87	0.075
3	60	0.6	0.2	12	0.17	2.04	0.082
4	80	0.8	0.2	11	0.17	1.87	0.075
5	100	1	0.2	14	0.17	2.38	0.095
6	120	1.2	0.2	18	0.17	3.06	0.122
7	140	1.4	0.2	20	0.17	3.4	0.136
8	160	1.6	0.2	23	0.17	3.91	0.156
9	180	1.8	0.2	27	0.17	4.59	0.184
10	200	2	0.2	29.5	0.17	5.015	0.201
11	220	2.2	0.2	29.5	0.17	5.015	0.201
12	240	2.4	0.2	29.5	0.17	5.015	0.201
13	260	2.6	0.2	29.5	0.17	5.015	0.201
14	280	2.8	0.2	29.5	0.17	5.015	0.201
15	300	3	0.2	23.5	0.17	3.995	0.160
16	320	3.2	0.2	21.2	0.17	3.604	0.144
17	340	3.4	0.2	19	0.17	3.23	0.129
18	360	3.6	0.2	18.75	0.17	3.18	0.128

Horizontal dial Reading (0.01mm)

Horizontal Displacement(mm)

Load Dial Reading (0.17 kg)

Horizontal Shear Force (kg)

Shear Stress =4 kg/cm2

S.N	Hz. Dial Reading (0.01mm)	Hz. Displacement (mm)	Normal Stress (kg/cm^2)	Load Dial Reading	Calibration Factor (kg/div)	Shear Force	Shear Stress (kg/cm^2)
1	20	0.2	0.4	12	0.17	2.04	0.082
2	40	0.4	0.4	14	0.17	2.38	0.095
3	60	0.6	0.4	15	0.17	2.55	0.102
4	80	0.8	0.4	16	0.17	2.72	0.109
5	100	1	0.4	17	0.17	2.89	0.116
6	120	1.2	0.4	18	0.17	3.06	0.122
7	140	1.4	0.4	20	0.17	3.4	0.136
8	160	1.6	0.4	22	0.17	3.74	0.150
9	180	1.8	0.4	23	0.17	3.91	0.156
10	200	2	0.4	24	0.17	4.08	0.163
11	220	2.2	0.4	26	0.17	4.42	0.177
12	240	2.4	0.4	31	0.17	5.27	0.211
13	260	2.6	0.4	33	0.17	5.61	0.224
14	280	2.8	0.4	34	0.17	5.78	0.231
15	300	3	0.4	36	0.17	6.12	0.245
16	320	3.2	0.4	38	0.17	6.46	0.258
17	340	3.4	0.4	38.5	0.17	6.545	0.262
18	360	3.6	0.4	38.5	0.17	6.545	0.262
19	380	3.8	0.4	38.5	0.17	6.545	0.262
20	400	4	0.4	39	0.17	6.63	0.265
21	420	4.2	0.4	41	0.17	6.97	0.279
22	440	4.4	0.4	41	0.17	6.97	0.279
23	460	4.6	0.4	41	0.17	6.97	0.279
24	480	4.8	0.4	42	0.17	7.14	0.286
25	500	5	0.4	45	0.17	7.65	0.306
26	520	5.2	0.4	46	0.17	7.82	0.313
27	540	5.4	0.4	46	0.17	7.82	0.313
28	560	5.6	0.4	46	0.17	7.82	0.313
29	580	5.8	0.4	46	0.17	7.82	0.313
30	600	6	0.4	45.5	0.17	7.735	0.309
31	620	6.2	0.4	45	0.17	7.65	0.306
32	640	6.4	0.4	45	0.17	7.65	0.306
33	660	6.6	0.4	45	0.17	7.65	0.306
34	680	6.8	0.4	45	0.17	7.65	0.306
35	700	7	0.4	45	0.17	7.65	0.306
36	720	7.2	0.4	45	0.17	7.65	0.306

Horizontal dial Reading (0.01mm)

Horizontal Displacement (mm)

Load Dial Reading (0.17 kg)

Horizontal Shear Force (kg)

Shear Stress =6 kg/cm2

S.N	Hz. Dial Reading (0.01mm)	Hz. Displacement (mm)	Normal Stress (kg/cm^2)	Load Dial Reading	Proving Calibration Factor (kg/div)	Shear Force	Shear Stress (kg/cm^2)
1	20	0.2	0.6	15	0.17	2.55	0.102
2	40	0.4	0.6	22	0.17	3.74	0.150
3	60	0.6	0.6	27	0.17	4.59	0.184
4	80	0.8	0.6	32	0.17	5.44	0.218
5	100	1	0.6	37	0.17	6.29	0.252
6	120	1.2	0.6	42	0.17	7.14	0.286
7	140	1.4	0.6	45	0.17	7.65	0.306
8	160	1.6	0.6	47	0.17	7.99	0.320
9	180	1.8	0.6	50	0.17	8.50	0.340
10	200	2	0.6	52.5	0.17	8.93	0.357
11	220	2.2	0.6	56	0.17	9.52	0.381
12	240	2.4	0.6	57	0.17	9.69	0.388
13	260	2.6	0.6	59	0.17	10.03	0.401
14	280	2.8	0.6	60	0.17	10.20	0.408
15	300	3	0.6	61	0.17	10.37	0.415
16	320	3.2	0.6	61.5	0.17	10.46	0.418
17	340	3.4	0.6	62	0.17	10.54	0.422
18	360	3.6	0.6	63	0.17	10.71	0.428
19	380	3.8	0.6	63	0.17	10.71	0.428
20	400	4	0.6	63.5	0.17	10.80	0.432
21	420	4.2	0.6	64	0.17	10.88	0.435
22	440	4.4	0.6	64.25	0.17	10.92	0.437
23	460	4.6	0.6	64.25	0.17	10.92	0.437
24	480	4.8	0.6	65	0.17	11.05	0.442
25	500	5	0.6	65	0.17	11.05	0.442
26	520	5.2	0.6	65	0.17	11.05	0.442
27	540	5.4	0.6	65	0.17	11.05	0.442
28	560	5.6	0.6	65	0.17	11.05	0.442
29	580	5.8	0.6	65	0.17	11.05	0.442

S.N.	Max. Shear Stress (kg/cm^2)	Normal Stress (kg/cm^2)
1	0.20	0.2
2	0.31	0.4
3	0.44	0.6

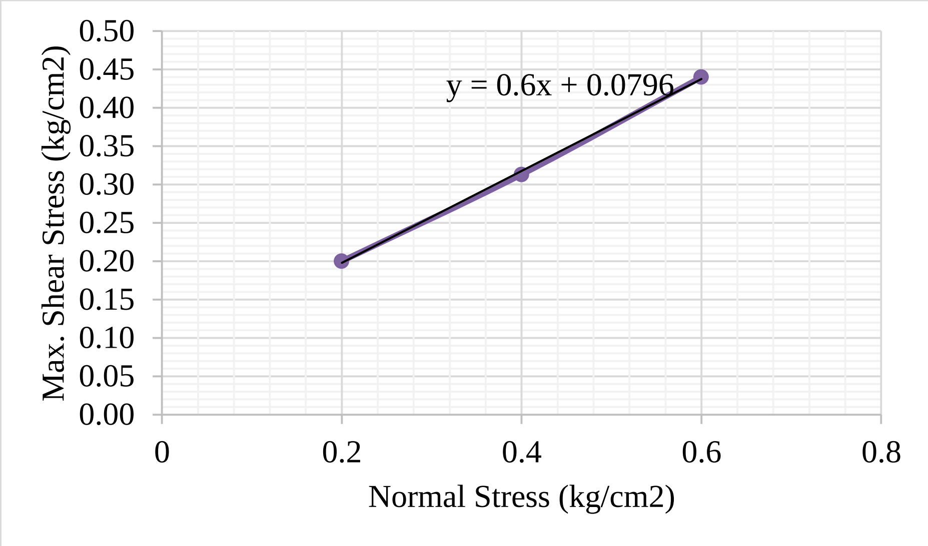


Figure A. 1 Maximum shear stress vs. Normal stress diagram of soil sample (67+300)

cohesion (kg/cm2)	0.08
Friction angle(degree)	31

2. Shear test result of chainage 68+300

Field density of soil		
Volume of soil	0.00097	m3
Wt. of soil	1.757	kg
Density	1820.00	kg/m3

Horizontal dial Reading (0.01mm)

Horizontal Displacement (mm)

Load Dial Reading (0.17 kg)

Horizontal Shear Force (kg)

Shear Stress (kg/cm2)

Normal stress= 0.2kg/cm2

SN	Hoz. Dial Reading (0.01mm)	Hoz. Displacement (mm)	Normal Stress (kg/cm^2)	Load Dial Reading	Proving Calibration Factor (kg/div)	Shear Force	Shear Stress (kg/cm^2)
1	20	0.2	0.2	8	0.17	1.36	0.054
2	40	0.4	0.2	13	0.17	2.21	0.088
3	60	0.6	0.2	15	0.17	2.55	0.102
4	80	0.8	0.2	16	0.17	2.72	0.109
5	100	1	0.2	19	0.17	3.23	0.129
6	120	1.2	0.2	21	0.17	3.57	0.143
7	140	1.4	0.2	23	0.17	3.91	0.156
8	160	1.6	0.2	25.32	0.17	4.30	0.172
9	180	1.8	0.2	28.6	0.17	4.86	0.194
10	200	2	0.2	28.6	0.17	4.86	0.194
11	220	2.2	0.2	28.6	0.17	4.86	0.194
12	240	2.4	0.2	28.6	0.17	4.86	0.194
13	260	2.6	0.2	28.6	0.17	4.86	0.194
14	280	2.8	0.2	28.6	0.17	4.86	0.194
15	300	3	0.2	28.6	0.17	4.86	0.194
16	320	3.2	0.2	27.2	0.17	4.62	0.185
17	340	3.4	0.2	25	0.17	4.25	0.170
18	360	3.6	0.2	24.75	0.17	4.20	0.168

Horizontal dial Reading (0.01mm)

Horizontal Displacement (mm)

Load Dial Reading (0.17 kg)

Horizontal Shear Force (kg)

Shear Stress (kg/cm2)

Normal stress=0.4kg/cm2

S.N	Hz. Dial Reading (0.01mm)	Hz. Displacement (mm)	Normal Stress (kg/cm^2)	Load Dial Reading	Proving Calibration Factor (kg/div)	Shear Force	Shear Stress (kg/cm^2)
1	20	0.2	0.4	12	0.17	2.04	0.082
2	40	0.4	0.4	18	0.17	3.06	0.122
3	60	0.6	0.4	20	0.17	3.4	0.136
4	80	0.8	0.4	22	0.17	3.74	0.150
5	100	1	0.4	23	0.17	3.91	0.156
6	120	1.2	0.4	24	0.17	4.08	0.163
7	140	1.4	0.4	26	0.17	4.42	0.177
8	160	1.6	0.4	28	0.17	4.76	0.190
9	180	1.8	0.4	32	0.17	5.44	0.218
10	200	2	0.4	34	0.17	5.78	0.231
11	220	2.2	0.4	36	0.17	6.12	0.245
12	240	2.4	0.4	37	0.17	6.29	0.252
13	260	2.6	0.4	39	0.17	6.63	0.265
14	280	2.8	0.4	40	0.17	6.8	0.272
15	300	3	0.4	42	0.17	7.14	0.286
16	320	3.2	0.4	42	0.17	7.14	0.286
17	340	3.4	0.4	42.5	0.17	7.225	0.289
18	360	3.6	0.4	42.5	0.17	7.225	0.289
19	380	3.8	0.4	42.5	0.17	7.225	0.289
20	400	4	0.4	43	0.17	7.31	0.292
21	420	4.2	0.4	45	0.17	7.65	0.306
22	440	4.4	0.4	45	0.17	7.65	0.306
23	460	4.6	0.4	45	0.17	7.65	0.306
24	480	4.8	0.4	46	0.17	7.82	0.313
25	500	5	0.4	49	0.17	8.33	0.333
26	520	5.2	0.4	50.5	0.17	8.58	0.343
27	540	5.4	0.4	50.5	0.17	8.58	0.343
28	560	5.6	0.4	50.5	0.17	8.58	0.343
29	580	5.8	0.4	50.5	0.17	8.58	0.343
30	600	6	0.4	49.5	0.17	8.41	0.337
31	620	6.2	0.4	49	0.17	8.3	0.333
32	640	6.4	0.4	49	0.17	8.33	0.333
33	660	6.6	0.4	49	0.17	8.33	0.333
34	680	6.8	0.4	49	0.17	8.33	0.333
35	700	7	0.4	49	0.17	8.33	0.333
36	720	7.2	0.4	49	0.17	8.33	0.333

Horizontal dial Reading (0.01mm)

Horizontal Displacement

(mm)

Load Dial Reading (0.17 kg)

Horizontal Shear Force (kg)

Shear Stress (kg/cm2)

Normal stress=0.6kg/cm2

S.N	Hz. Dial Reading (0.01mm)	Hz. Displacement (mm)	Normal Stress (kg/cm^2)	Load Dial Reading	Proving Calibration Factor (kg/div)	Shear Force	Shear Stress (kg/cm^2)
1	20	0.2	0.6	16	0.17	2.72	0.109
2	40	0.4	0.6	23	0.17	3.91	0.156
3	60	0.6	0.6	28	0.17	4.76	0.190
4	80	0.8	0.6	33	0.17	5.61	0.224
5	100	1	0.6	38	0.17	6.46	0.258
6	120	1.2	0.6	43	0.17	7.31	0.292
7	140	1.4	0.6	48	0.17	8.16	0.326
8	160	1.6	0.6	50	0.17	8.50	0.340
9	180	1.8	0.6	53	0.17	9.01	0.360
10	200	2	0.6	55.5	0.17	9.44	0.377
11	220	2.2	0.6	59	0.17	10.03	0.401
12	240	2.4	0.6	62	0.17	10.54	0.422
13	260	2.6	0.6	63	0.17	10.71	0.428
14	280	2.8	0.6	63	0.17	10.71	0.428
15	300	3	0.6	64.5	0.17	10.97	0.439
16	320	3.2	0.6	65	0.17	11.05	0.442
17	340	3.4	0.6	66	0.17	11.22	0.449
18	360	3.6	0.6	67	0.17	11.39	0.456
19	380	3.8	0.6	68	0.17	11.56	0.462
20	400	4	0.6	68.5	0.17	11.65	0.466
21	420	4.2	0.6	69	0.17	11.73	0.469
22	440	4.4	0.6	69.25	0.17	11.77	0.471
23	460	4.6	0.6	69.25	0.17	11.77	0.471
24	480	4.8	0.6	69.5	0.17	11.82	0.473
25	500	5	0.6	69.5	0.17	11.82	0.473
26	520	5.2	0.6	69.5	0.17	11.82	0.473
27	540	5.4	0.6	69.5	0.17	11.82	0.473
28	560	5.6	0.6	69.5	0.17	11.82	0.473
29	580	5.8	0.6	69.5	0.17	11.82	0.473

S.N.	Max. Shear Stress (kg/cm^2)	Normal Stress (kg/cm^2)
1	0.194	0.2
2	0.343	0.4
3	0.473	0.6

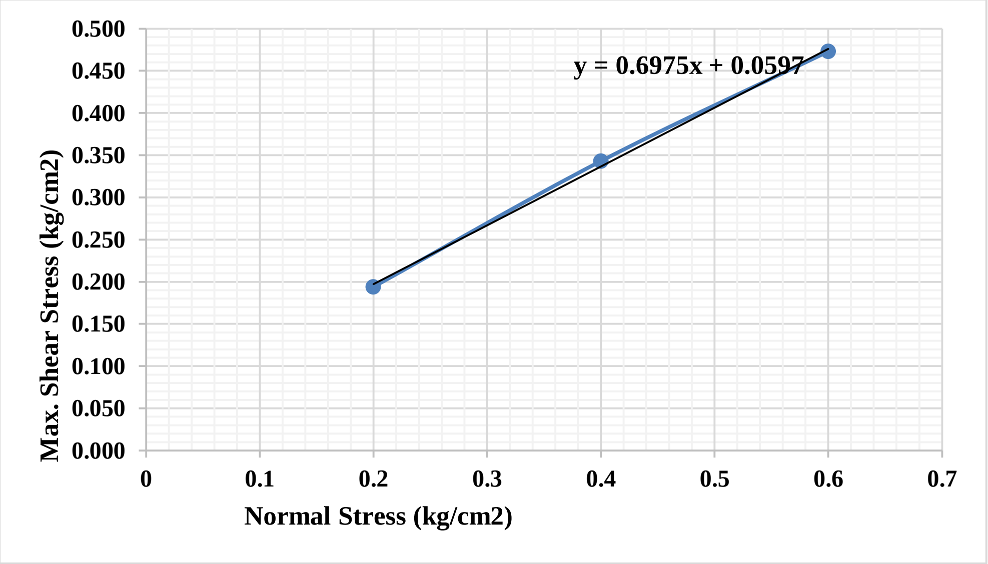


Figure A. 2 Maximum shear stress vs. Normal stress diagram of soil sample (62+300)

Cohesion	0.06	kg/cm2
Friction angle	33.00	Degree

3. Shear test result of chainage 68+700

Field density of soil		
Volume of soil	0.00097	m3
Wt. of soil	1.825	kg
Density	1890.64	kg/m3

Horizontal dial Reading (0.01mm)

Horizontal Displacement (mm)

Load Dial Reading (0.17 kg)

Horizontal Shear Force (kg)

Shear Stress (kg/cm2)

Normal stress 0.2kg/m2

S.N	Hz. Dial Reading (0.01mm)	Hz. Displacement (mm)	Normal Stress (kg/cm^2)	Load Dial Reading	Proving Calibration Factor (kg/div)	Shear Force	Shear Stress (kg/cm^2)
1	20	0.2	0.2	8	0.17	1.36	0.054
2	40	0.4	0.2	13	0.17	2.21	0.088
3	60	0.6	0.2	15	0.17	2.55	0.102
4	80	0.8	0.2	16	0.17	2.72	0.109
5	100	1	0.2	19	0.17	3.23	0.129
6	120	1.2	0.2	21	0.17	3.57	0.143
7	140	1.4	0.2	23	0.17	3.91	0.156
8	160	1.6	0.2	25.32	0.17	4.304	0.172
9	180	1.8	0.2	27.9	0.17	4.743	0.190
10	200	2	0.2	27.9	0.17	4.743	0.190
11	220	2.2	0.2	27.9	0.17	4.743	0.190
12	240	2.4	0.2	27.9	0.17	4.743	0.190
13	260	2.6	0.2	27.9	0.17	4.743	0.190
14	280	2.8	0.2	27.9	0.17	4.743	0.190
15	300	3	0.2	27.9	0.17	4.743	0.190
16	320	3.2	0.2	26.2	0.17	4.454	0.178
17	340	3.4	0.2	24	0.17	4.08	0.163
18	360	3.6	0.2	23.75	0.17	4.037	0.162

Horizontal dial Reading (0.01mm)

Horizontal Displacement (mm)

Load Dial Reading (0.17 kg)

Horizontal Shear Force (kg)

Normal stress 0.4kg/m2

S.N	Hz. Dial Reading (0.01mm)	Horizontal Displacement (mm)	Normal Stress (kg/cm^2)	Load Dial Reading	Proving Calibration Factor (kg/div)	Shear Force	Shear Stress (kg/cm^2)
1	20	0.2	0.4	11	0.17	1.87	0.075
2	40	0.4	0.4	17	0.17	2.89	0.116
3	60	0.6	0.4	19	0.17	3.23	0.129
4	80	0.8	0.4	21	0.17	3.57	0.143
5	100	1	0.4	22	0.17	3.74	0.150
6	120	1.2	0.4	23	0.17	3.91	0.156
7	140	1.4	0.4	25	0.17	4.25	0.170
8	160	1.6	0.4	27	0.17	4.59	0.184
9	180	1.8	0.4	31	0.17	5.27	0.211
10	200	2	0.4	33	0.17	5.61	0.224
11	220	2.2	0.4	35	0.17	5.95	0.238
12	240	2.4	0.4	36	0.17	6.12	0.245
13	260	2.6	0.4	38	0.17	6.46	0.258
14	280	2.8	0.4	39	0.17	6.63	0.265
15	300	3	0.4	41	0.17	6.97	0.279
16	320	3.2	0.4	41	0.17	6.97	0.279
17	340	3.4	0.4	41.5	0.17	7.05	0.282
18	360	3.6	0.4	41.5	0.17	7.05	0.282
19	380	3.8	0.4	41.5	0.17	7.05	0.282
20	400	4	0.4	42	0.17	7.14	0.286
21	420	4.2	0.4	44	0.17	7.48	0.299
22	440	4.4	0.4	44	0.17	7.48	0.299
23	460	4.6	0.4	44	0.17	7.48	0.299
24	480	4.8	0.4	45	0.17	7.65	0.306
25	500	5	0.4	48	0.17	8.16	0.326
26	520	5.2	0.4	49.5	0.17	8.41	0.337
27	540	5.4	0.4	49.5	0.17	8.41	0.337
28	560	5.6	0.4	49.5	0.17	8.41	0.337
29	580	5.8	0.4	49.5	0.17	8.41	0.337
30	600	6	0.4	48.5	0.17	8.24	0.330
31	620	6.2	0.4	48	0.17	8.16	0.326
32	640	6.4	0.4	48	0.17	8.16	0.326
33	660	6.6	0.4	48	0.17	8.16	0.326
34	680	6.8	0.4	48	0.17	8.16	0.326
35	700	7	0.4	48	0.17	8.16	0.326
36	720	7.2	0.4	48	0.17	8.16	0.326

Horizontal dial Reading (0.01mm)

Horizontal Displacement (mm)

Load Dial Reading (0.17 kg)

Horizontal Shear Force (kg)

Shear Stress (kg/cm2)

Normal stress 0.6kg/m2

S. N	Hz. Dial Reading (0.01mm)	Hz. Displacement (mm)	Normal Stress (kg/cm^2)	Load Dial Reading	Proving Calibration Factor (kg/div)	Shear Force	Shear Stress (kg/cm^2)
1	20	0.2	0.6	17	0.17	2.89	0.116
2	40	0.4	0.6	24	0.17	4.08	0.163
3	60	0.6	0.6	29	0.17	4.93	0.197
4	80	0.8	0.6	34	0.17	5.78	0.231
5	100	1	0.6	39	0.17	6.63	0.265
6	120	1.2	0.6	44	0.17	7.48	0.299
7	140	1.4	0.6	49	0.17	8.33	0.333
8	160	1.6	0.6	51	0.17	8.67	0.347
9	180	1.8	0.6	54	0.17	9.18	0.367
10	200	2	0.6	56.5	0.17	9.61	0.384
11	220	2.2	0.6	60	0.17	10.20	0.408
12	240	2.4	0.6	63	0.17	10.71	0.428
13	260	2.6	0.6	64	0.17	10.88	0.435
14	280	2.8	0.6	64	0.17	10.88	0.435
15	300	3	0.6	65.5	0.17	11.14	0.445
16	320	3.2	0.6	66	0.17	11.22	0.449
17	340	3.4	0.6	67	0.17	11.39	0.456
18	360	3.6	0.6	68	0.17	11.56	0.462
19	380	3.8	0.6	68	0.17	11.56	0.462
20	400	4	0.6	69	0.17	11.73	0.469
21	420	4.2	0.6	69	0.17	11.73	0.469
22	440	4.4	0.6	69.25	0.17	11.77	0.471
23	460	4.6	0.6	69.25	0.17	11.77	0.471
24	480	4.8	0.6	70	0.17	11.90	0.476
25	500	5	0.6	70	0.17	11.90	0.476
26	520	5.2	0.6	70	0.17	11.90	0.476
27	540	5.4	0.6	70	0.17	11.90	0.476
28	560	5.6	0.6	70	0.17	11.90	0.476
29	580	5.8	0.6	70	0.17	11.90	0.476

S.N.	Max. Shear Stress (kg/cm^2)	Normal Stress (kg/cm^2)
1	0.190	0.2
2	0.337	0.4
3	0.476	0.6

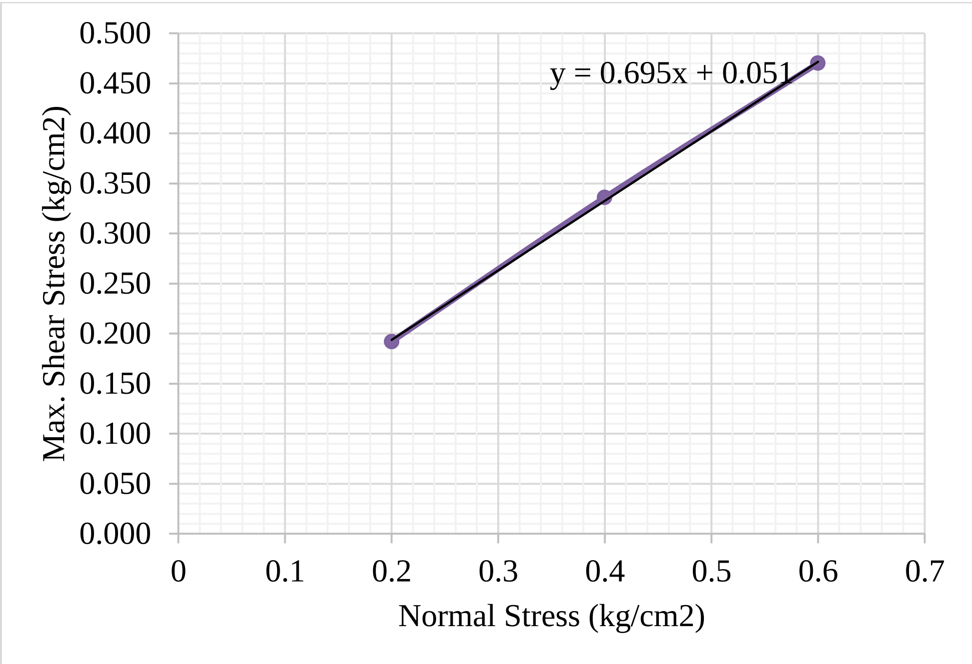


Figure A. 3: Maximum shear stress vs. Normal stress diagram of soil sample (68+700)

Cohesion	0.051	kg/cm2
Friction angle	34.500	Degree

Some major Analysis figures with results

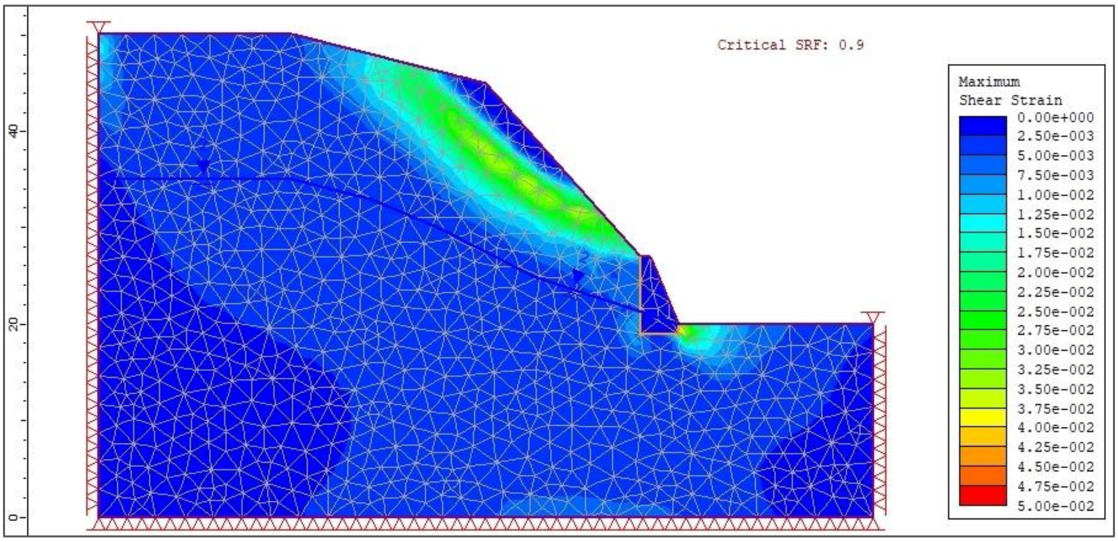


Figure A. 4: MSS diagram with critical SRF obtained from FEM using 8 m retaining wall at slope section SS1(62+300)

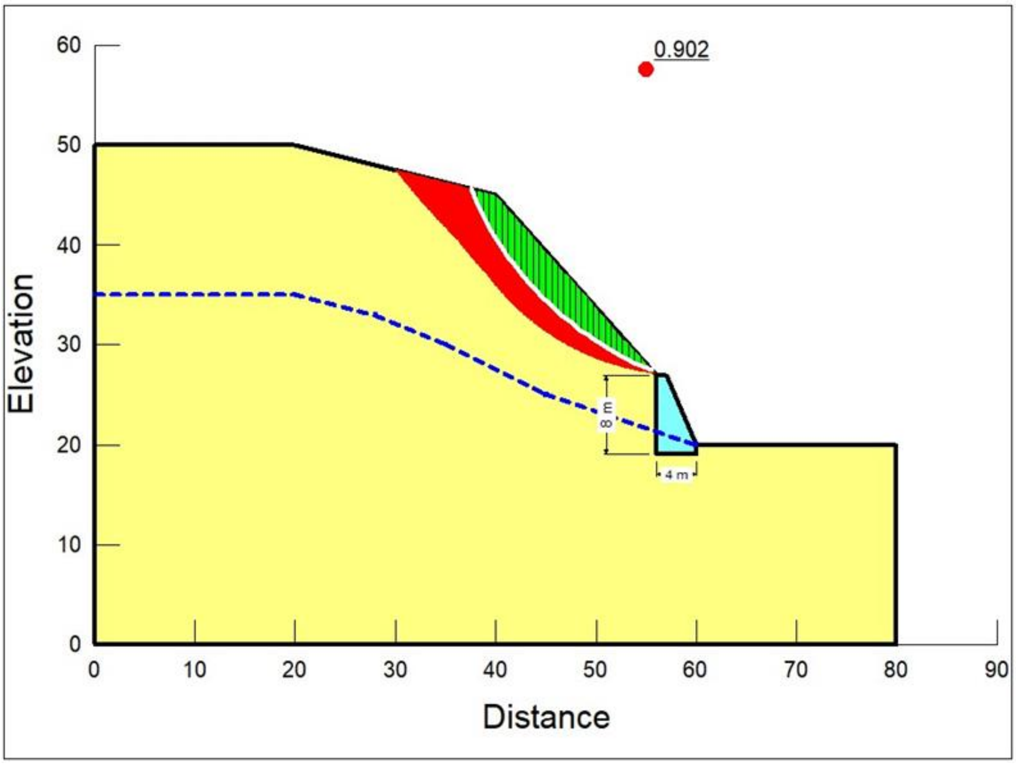


Figure A. 5: Result Showing Showing Factro of Safety using 8 m retaining wall at SS1(62+300)

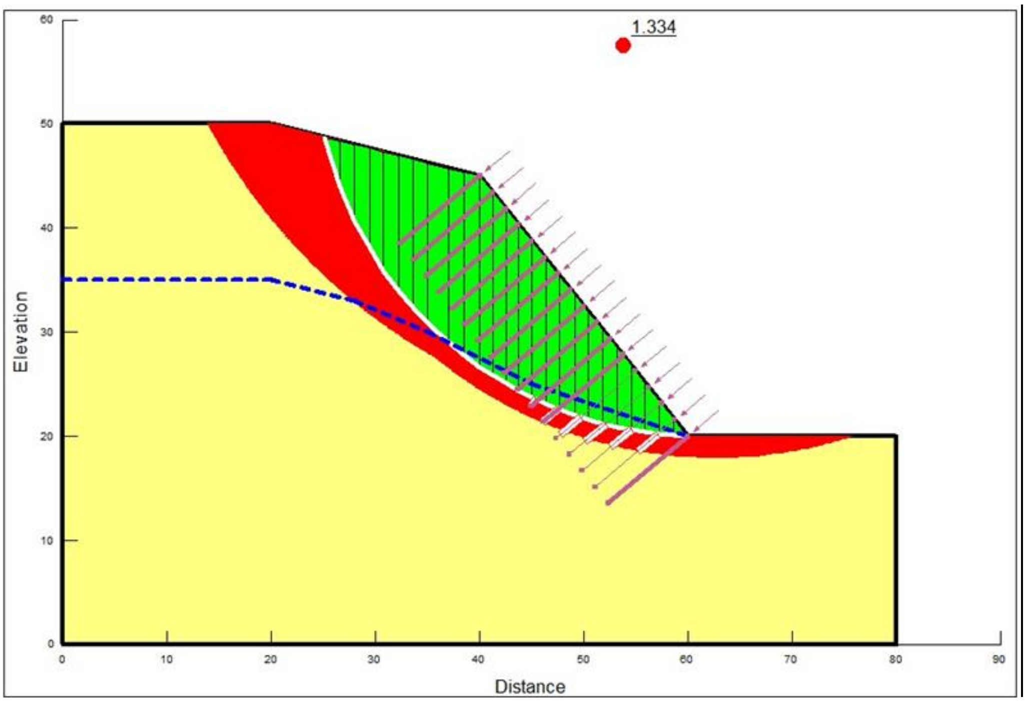


Figure A. 6: Result with safety map from Slope/W software using 120 m nail for slope section SS1(62+300)

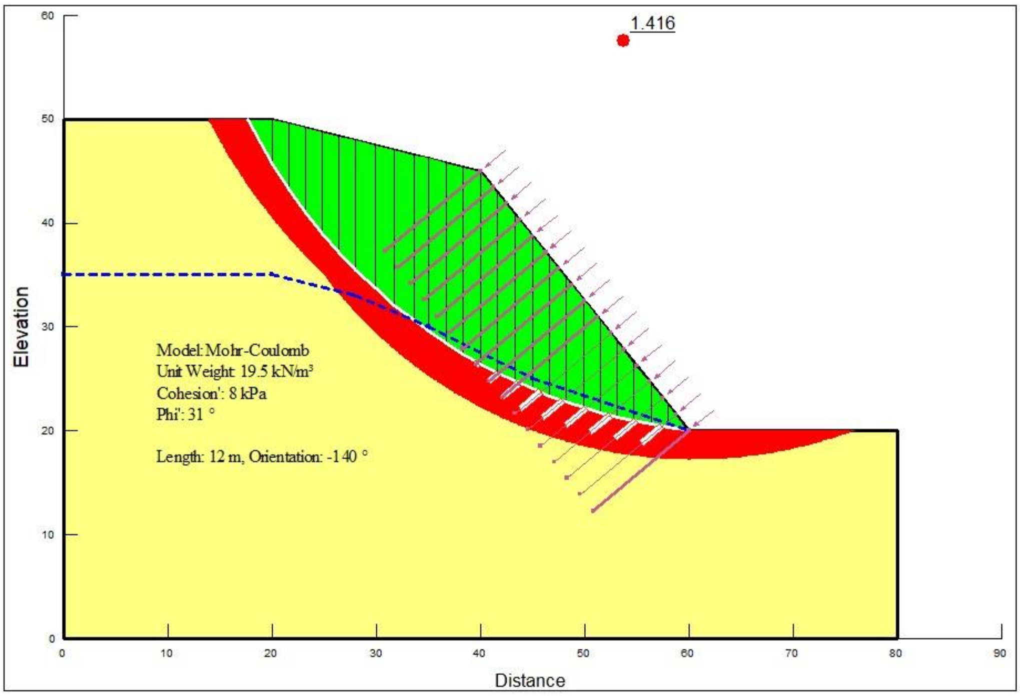


Figure A. 7: Result with safety map from Slope/W software using 12 m nail for slope section SS1(62+300)

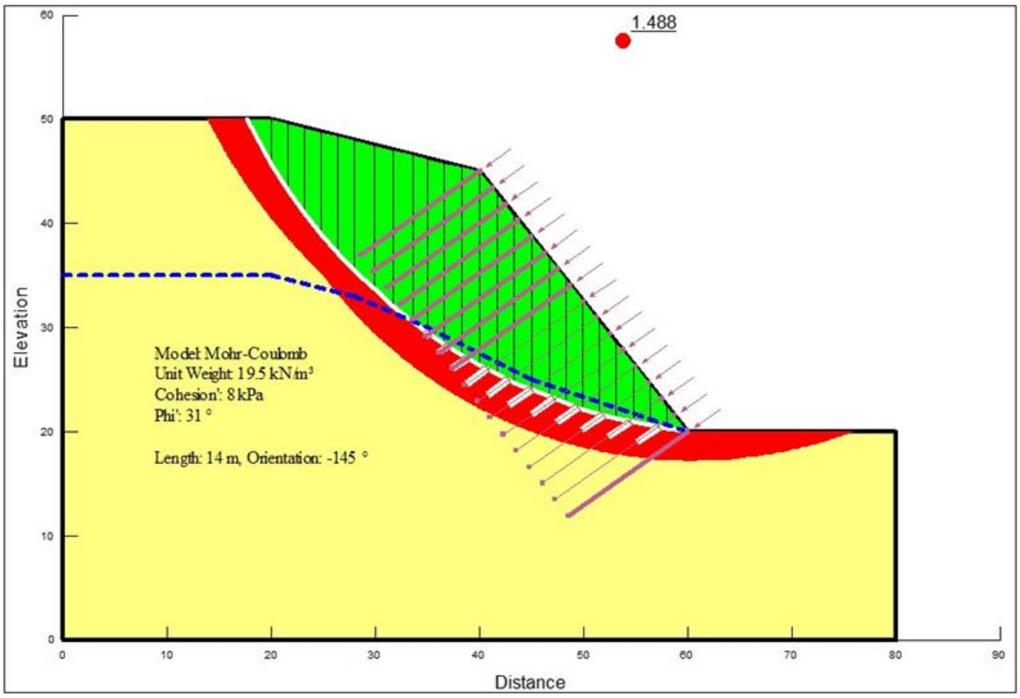


Figure A. 8: Result with safety map from Slope/W software using 14 m nail for slope section SS1(62+300)

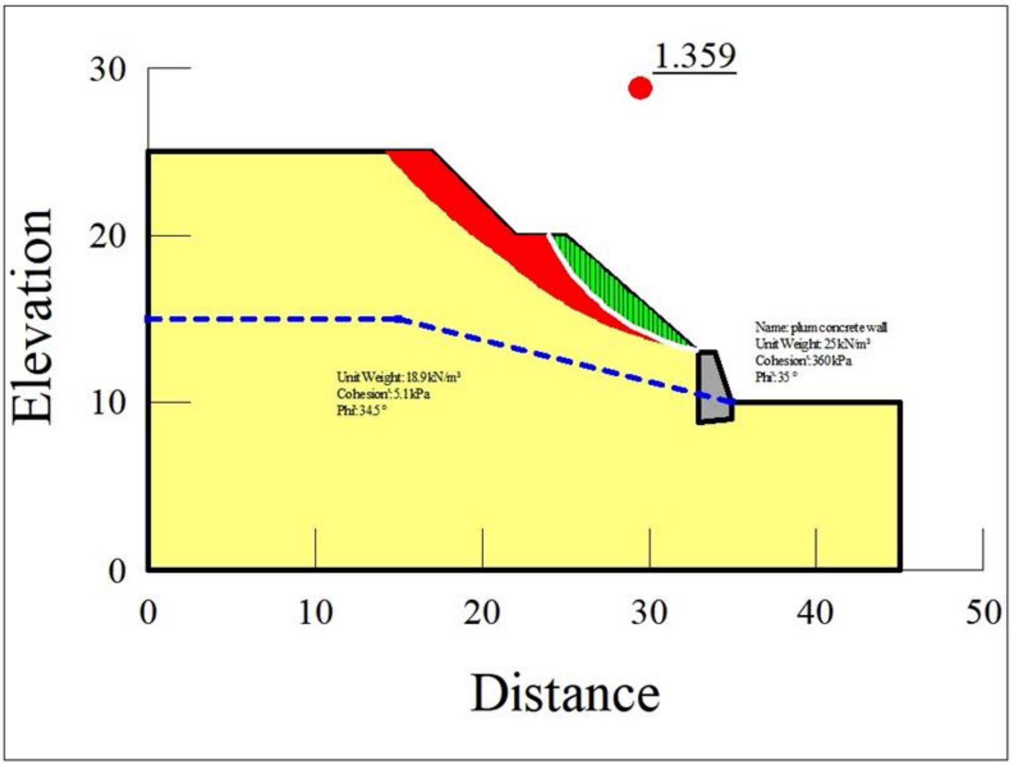


Figure A. 9: Result with safety map from Slope/W software using 4 m retaining wall for slope section SS3(68+700)

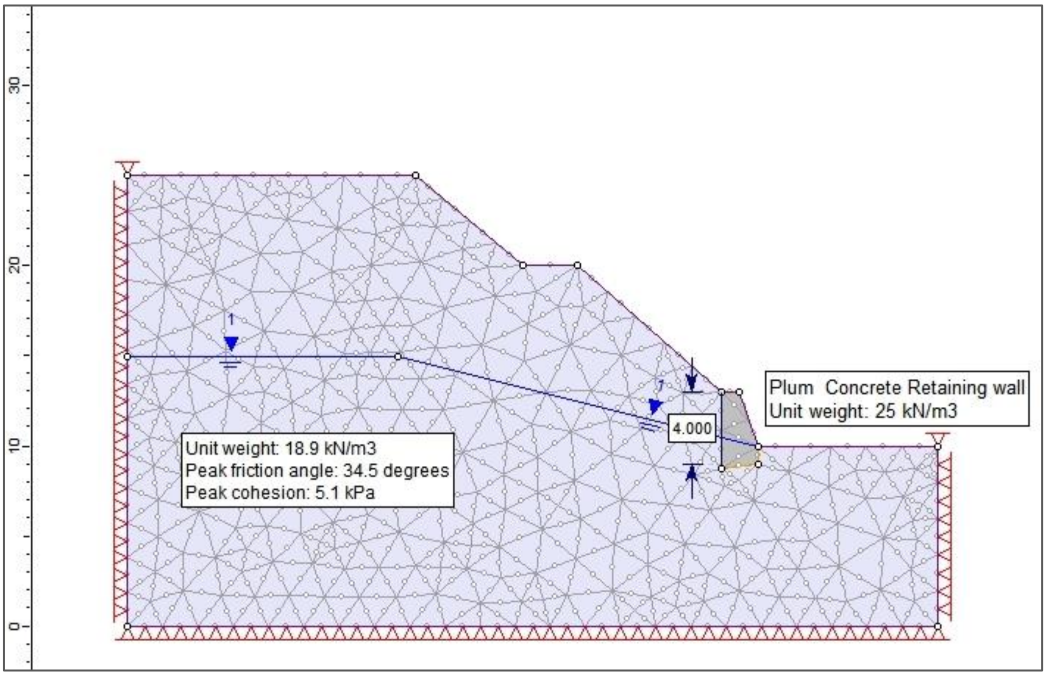


Figure A. 10: Input model with discretized mesh in phase 2 with 4 m retaining wall in phase 2 for SS3 (68+700)

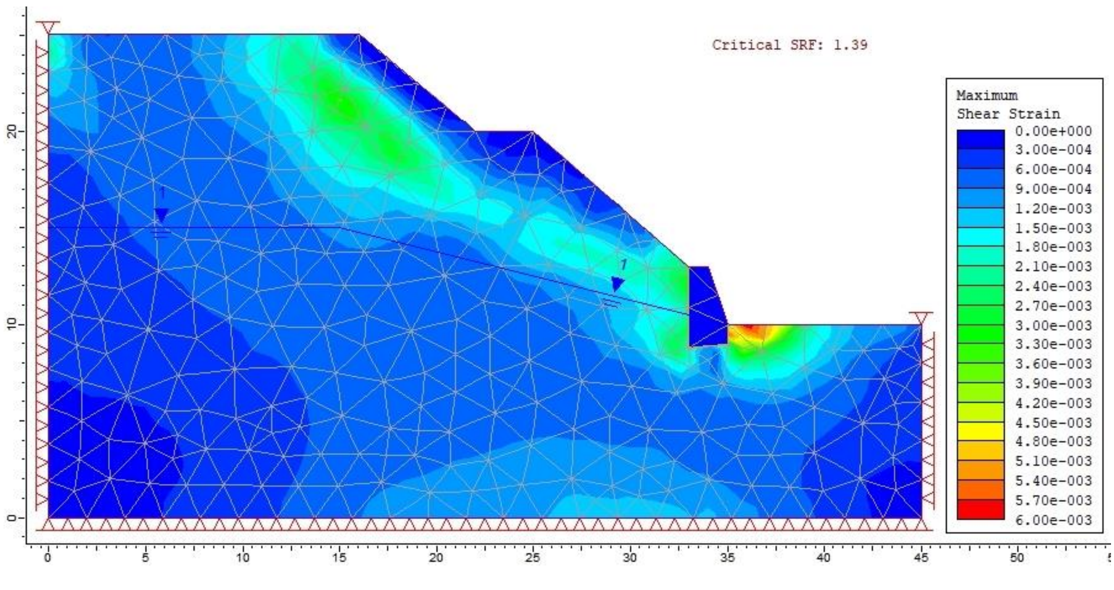


Figure A. 11: : MSS diagram with critical SRF obtained from FEM using 4 m retaining wall at slope section SS3(68+700)