SLOPE RELIABILITY

A CASE STUDY OF ALI VILLA SLOPE BAHRIA GOLF CITY

By

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ABSTRACT

This research comprises of analysis of Ali Villa slope at Bahria Golf City Islamabad, using Limit Equilibrium Method and Finite Element Modeling (FEM) incorporating SLIDE 6.0 and PHASE2.

Slopes exposed to water tends to lose its effective strength. The colluvium on the slope comprised of broken shale and mudstone had little cohesion therefore water penetrating it caused problems. During the research work we have faced numerous challenges like deformations at site are either not monitored and if they are, the data is not extensive.

Conclusions derived from the analysis of slope were that Grouted Tiebacks with Modular Block Wall as facing should be used as remedial technique along with Gabion Wall for the lower slope.

This is to certify that the

Report entitled

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Has been accepted towards the partial fulfillment

of

the requirements

for

Bachelors of Engineering in Civil Engineering

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DEDICATED

TO

OUR

PARENTS

&

TEACHERS

ACKNOWLEDGEMENT

We take this opportunity to evince our profound appreciation to our supervisor, Dr. S. Muhammad Jamil, for his guidance and encouragement. His comments reflected his experience as a counselor and supervisor, he would ask at the end of every meeting if we learned anything. His constant encouragement, support and sheer interest in our project is what kept us going.

Sincere gratitude is extended to Engr. Asim Ayaz, who took keen interest in developing and drafting initial course for our project, helped us through and through by bombarding us with loads of information and options to take.

Lastly our family members whose constant support throughout the journey lifted us and spirits to highest morale.

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CHAPTER 1

INTRODUCTION

1.1 OVERVIEW

Landslide is the phenomenon of movement of a mass of soil, debris, composite materials or rock down a slope. Various destabilizing factors or forces cause the failure of a slope in mountainous areas. For a specific set of conditions, these slopes strive to become stable by attaining a natural equilibrium and become unstable once this natural equilibrium is disturbed, resulting in landslides. Landslides are one of the potential geological hazards of mountainous areas and have been observed and recorded for several centuries worldwide. The landslides have caused extensive loss to humans life and infrastructure worldwide. Sometimes near any river they cause floods of varying ranges and thus cause a potent threat to the domestic and industrial development of any area.

Murree formation mostly consists of sandstone, mudstone and shale and due to its fragile geology and deforestation along with uncontrolled urbanization it is a serious threat to the people living in that area.

Our site was located in Bahria Golf City Islamabad under Ali Villa Road and above Hole 18. The elevation difference was 150 feet with horizontal distance of 600 feet and the slope spread to 800 feet in length. The failure initiated was under Ali villa road and the proposed road was to be stabilized in accordance with.

1.2 PROBLEM STATEMENT.

“Analysis of the existing slope at Bahria Golf City under Ali Villa with economical strengthening measures incorporated with softwares modeling”

1.3 MAIN OBJECTIVES

1. The first objective was to perform non circular analysis with different method to obtain factor of safety of unsupported slope in Slide 6.0 and Phase2.

2. The second objective was to use economical strengthening measures for the slope stability.

1.4 THESIS OUTLINE

Chapter 1 includes introduction.

Limit Equilibrium Methods and Finite Element Analysis used for Circular and non-circular analysis to determine the factor of safety are incorporated in **Chapter 2**.

Methodology and Analysis of the whole project is explained in **Chapter 3**.

Chapter 4 includes conclusions and recommendations of this study.

CHAPTER 2

LITERATURE REVIEW

2.1 SLOPE STABILITY ANALYSIS

For 2D analysis different assumptions are made to render the problem to determinate.

Following two approaches are used while analyzing slopes

- Limit Equilibrium Analysis
- Finite Element Analysis

2.1.1 LIMIT EQUILIBRIUM ANALYSIS

It is generally used in slope stability analysis,.

All limit equilibrium methods have some assumptions in common, all assume a failure surface and check whether the Mohr-Coulomb failure criteria is fulfilled or not for the particular surface.

All limit equilibrium methods of slope stability analysis have four characteristics in common, according to publication in 1980 by Duncan and Wright.

Factor of Safety

$$FOS = s/\tau$$

Where “s ” is the shear strength of soil.

“τ” is the shear stress required for equilibrium.

This method uses different assumptions in order to make the equation determinate,

2.1.1.1 Method of slices:

The method of slices divides the whole mass into number of slices, in a way to account for pore water pressure and normal forces acting along the assumed failure surface. Assumptions have been made to make the problem statically determinate, and it includes the assumptions made on side and interslice forces.

Different methods of slices are:

- Ordinary or Fellinius Method
- Simplified Bishop Method

- Spencer’s Method
- Janbu’s Method
- Morgenstern-Price Method

Forces involved in method of slices are:

- Mobilizes cohesion “ Cm ”
- Normal stress “N”
- Interslice friction (shear component) “X”
- Pore water pressure “u”
- Surface water force “ Uw ”
- Seismic or surcharge forces “k”, “Q”.

2.1.2 FINITE ELEMENT ANALYSIS

No assumption needs to be made in advance about the shape or location of the failure surface. Failure occurs “**naturally**’’ through the zones within the soil mass in which the soil shear strength is unable to sustain the applied shear stresses. If realistic soil compressibility data is available, the finite element solutions will give information about deformations at working stress levels.

2.2 GEOTECHNICAL SITE INVESTIGATION:

2.2.1 Introduction:

In order to obtain information regarding site, a detailed analysis of soil properties and information about the rock type found in the area is obtained. This investigation regarding the surrounding of the area site helps in the determination of methodology adopted for data collection and processing .this whole process is termed as site/field investigation.

Initial phase of investigation consists of detailed environmental and geological study of the area. The information gained from reviewed data helps in the chalking out exploration plan for the project. Data collection methodology includes desk study of available data.

2.2.2 Objectives:

The measure of information required through field examination relies upon the information accessible. Following objectives are required for the site examination of a geotechnical issue:

- Study of ground water levels.
- Sample collection for lab testing.
- Determination of strata properties i.e. type, nature etc.
- Determination of material/soil property.
- Data collection for plotting of map

2.2.3 Methods

Soil parameters and site conditions are the basic tools for site analysis. Few methods for soil and rock parameters and properties determination for lab use are given as under:

2.2.4 Boring

Soil boring is a process in with hand tools or drills are employed to extract soil data from below the surface. The hole resulting for this boring is further known as soil boring. The type of tool to be used depends upon the size of bore required. Depth of bore varies from tool to tool. Objectives for caring out boring are as follows:

- To collect disturb and undisturbed samples.
- To perform in-situ stress analysis.
- To get information regarding sub-surface strata.

2.2.5 Location of Bores

It is also important to select the location of the boreholes efficiently. Boring layout as explained in US UFC (2005) should also be governed y the geology of the site i.e.:

- Geological Sections: Arrange borings so that geological sections may be determined at the most useful orientations for final methodology and design. Borings in slide areas should establish the full geological section necessary for stability analyses.
- Critical Strata: Where detailed settlement, stability or seepage analyses are required, include a minimum of two borings to obtain undisturbed samples of critical strata. Provide sufficient preliminary sample borings to determine the most representative location for undisturbed sample borings.

2.3 Standard Penetration Test (SPT):

It is an in-situ dynamic penetration test used to provide an indication of the relative density of granular deposits, such as sands and gravels from which it is virtually impossible to obtain undisturbed samples. The main reason for the widespread use of this test is that it is simple to execute and inexpensive.

This test is elaborated in BS 1377-9:1990 and ASTM D1586 as:

"in this test a thick-walled sample tube is used which is driven into the ground at the bottom of a borehole by blows from a slide hammer with a weight of 63.5 kg (140 lb.) falling through a distance of 760 mm (30 in). The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm (6 in) up to a depth of 450 mm (18 in) is recorded. The sum of the number of blows for the first 6 inch penetration is neglected while that required for the second and third 6 in. of penetration is termed the "standard penetration resistance" or the "N-value". The blow count provides an indication of the density of the ground, and it is used in many empirical geotechnical engineering problems.

2.4 Laboratory Investigation Methods

The Laboratory tests are one of the important requisites for the study of landslides. The laboratory tests are performed as per requirements for the study of the landslides. The laboratory method of investigation is a detailed experimental study on the samples retrieved from the field investigations.

In the laboratory investigation methods, tests are carried out under controlled conditions and the required parameters are obtained for the study of the landslides in different scenarios (Clayton, 1995). However there are limitations of the laboratory methods of investigations. The samples carried to the laboratory for tests are mostly disturbed and the field conditions are difficult to be replicated. The common laboratory tests performed for the slope stability studies are as follows;

2.4.1 Soil Classification/Gradation

The soil classification test is the primary test for any Geotech related site. Soil classification tests are performed for the identification of materials comprising the slope body. Many soil properties affecting the soil behavior can be inferred from the soil classification results. Modern engineering classification systems are designed to allow an easy transition from field observations to basic predictions of soil engineering properties and behaviors.

There are 2 major classification systems that are in use today, namely:

2.4.1.1 Unified Soil Classification System (USCS):

The USCS has three major classification groups:

- coarse-grained soils (e.g. sands and gravels);
- fine-grained soils (e.g. silts and clays); and
- highly organic soils (peat).

The USCS further subdivides the three major soil classes for clarification.

2.4.1.2 AASHTO Soil Classification System:

The AASHTO system uses both grain-size distribution and Atterberg limits data to assign a group classification and a group index to the soil. The group classification ranges from A-1 (best soils) to A-8 (worst soils). Group index values near 0 indicate good soils, while values of 20 or more indicate very poor soils. The main drawback of this system is that it is basically designed for highway construction purposes i.e. a soil that may be "good" for use as a highway sub grade might be "very poor" for other purposes, and vice versa.

2.4.2 Density/Unit weight

This test is helpful in determination of useful engineering properties of strata comprising the slope. Density tests are the measure of degree of packing of the material. Normally high density results in higher strength and good performance of the material comprising the slope. It also helps in determining the stress conditions at various depths of the slope. Various other engineering properties and mathematical analysis or models are also obtained by correlating them to this physical property.

2.4.3 Moisture Content

Moisture content tests are conducted for the determination of the field moisture conditions of the slope forming materials. Its presence in various proportions has a direct relation with the soil strength and stability of the slopes. Moreover, the moisture content helps in working out weight volume relationships and subsequently determines mechanical properties of soil.

2.4.4 Atterberg Limits

Atterberg limits (i.e. Liquid Limit and Plastic Limit) test is helpful in soil classification and other engineering properties of the soil that comprises the slope. These limits which include liquid limit, plastic limit, shrinkage limit, plasticity index etc., enable us to determine the relationship of soil and water. Various indices calculated through these limits bear direct correlation for determining the soil properties, like shear strength, stability, swelling etc. This test is only suitable for cohesive soils to ascertain their plasticity.

2.4.5 Consolidation and Swelling Tests

The odometer consolidation tests are performed for the determination of the consolidation characteristics of the samples. The same apparatus can also be used for determining the swelling index of the soil sample. The use of consolidation tests with reference to the landslides is to determine the pre-consolidation pressures and in-situ stress conditions (Anderson, 2000). The results also help in evaluating soil behavior such as volume changes/settlement under various loading conditions, such as its permeability characteristics, swell potential etc. This test gives an idea of whether the soil is normally consolidated or over-consolidated and accordingly their swelling characteristics can be understood.

2.4.6 Shear Test

Shear tests are conducted to know the shear strength of the soil. Depending upon the drainage conditions and loading patterns, there are two types of Shear Tests;

2.4.6.1 Direct Shear Test:

Direct shear test helps in determining the peak strength parameters of the slope soil. The test also helps to determine the parameters of cohesion and the angle of the internal friction under simulated conditions (Stump et al. 1995). The parameters determined from direct shear tests also help to assess other soil properties through correlation. This test does not take into account the drainage boundary conditions.

2.4.6.2 Triaxial Test:

Triaxial tests are conducted under different conditions, such as unconsolidated undrained (UU) and consolidated un-drained (CU), to simulate the different scenarios at the landslide site, for the determination of cohesion (c) and angle of internal friction (ϕ) to be used as input parameters in the analyses of the landslides. This test is far better than direct shear test as it helps in determining the behavior of soil under a wide range of moistures, stress as well as loading conditions. The results of these tests bear close relevance to the actual in-situ results as they create an environment similar to the field conditions. The Triaxial apparatus can also be used to determine the residual strength of soils.

2.4.7 Unconfined Compressive Strength Test

The unconfined compressive strength test is performed on the hard clay or rock samples obtained through field investigations. The samples for the test are retrieved from different depths of the slope material so as to get clear picture about the strength of the materials comprising the slope. The parameters obtained from the test are used as inputs in the landslide analysis. The test parameters are also used for consolidation through correlations.

2.4.8. Compaction Tests

This laboratory test is performed to determine the relationship between the moisture content and the dry density of a soil for a specified compactive effort. The compactive effort is the amount of mechanical energy that is applied to the soil mass. Several different methods are used to compact soil in the field. Proctor (1933) developed a methodology which employed tamping or impact compaction method, therefore the test is also known as the Proctor test.

2.5 GEOTECHNICAL INVESTIGATION RESULTS:

The laboratory testing of the samples bore the results of the geotechnical parameters as follows

Table 2.1: Geotechnical parameters values

Sr. No.	Parameter	Unit	Range of Values	
			Sandstone	Shale
1	Unit weight	kN/m^3	24.26 to 25.59	22.14 to 25.84
2	Natural moisture content	%	0.30 to 1.37	1.19 to 6.22
3	Specific gravity		2.74 to 2.77	2.62 to 2.72
4	Compressive strength	MPa	20.20 to 59.87	8.44 to 74.30
5	Point load strength	MPa	8.1 to 16.1	2.8 to 4.4

Table 2.2: Geotechnical Parameters Values of Rock

Parameters		Overburden	Shale/ Claystone	Sandstone
UCS (MPa)		-	26	59.5
GSI		-	28	41
D		-	0.7	0.7
Mi		-	6	17
MR		-	200	275
Ei (MPa)		-	5200	16362.5
Mb		-	0.114832	0.664617
S		-	2.94E-05	0.000193
A		-	0.525561	0.510622
Slope (H=50m)	C (MPa)	0.01	0.114846	0.34149
	Ø (deg)	20.0	23.4872	44.9008
σt (MPa)		-	-0.00665	-0.01731
σc (MPa)		-	0.107958	0.755626
σcm (MPa)		-	0.994696	6.1669
Em (MPa)		-	173.493	1008.51

CHAPTER 3

STABILITY ANALYSIS

3.1 INTRODUCTION

To calculate the factor of safety at the toe of the tunnel a number of analytical techniques were used the techniques can be broadly divided into two categories:

- Limit Equilibrium Analysis
- Finite Element Analysis

For the finite element analysis (FEA) PHASE2 software was used, model of each section of the selected slope was made and analyzed by inputting the relevant field details.

For the empirical approaches three criterions were used, Mohr-Coulomb, Hoek-Brown and Lade-Duncan. In manner similar to the FEA each section of the selected slope was analyzed and results were compiled.

3.2 LIMIT EQUILIBRIUM ANALYSIS

The 2-D slope analysis was carried out on SLIDE software. The cross-sections were drawn as in slide as mentioned earlier. The cross-sections are represented in the figure. The slope face lies parallel to the lines drawn, indicating the direction of the cross-section. Soil parameters that are used in the analysis given by the CONSULTANT, expect the unconfined compressive strength. The vales are cross check by our test as discuss earlier.

The site strata consisted of shale and Colluvium soil and embankment fill material. For shale generalized Hoek-Brown method was used with an unconfined compressive strength of 26 MPa. Other soil parameters like unit weight, geological strength index and intact rock constant were taken to be 23.4 KN/cubic meter, 28 and 6 respectively.

Soil parameters for Colluvium soil :Unit weight of soil used was21 KN/cubic meter and cohesion was taken to be zero. For both the material mohr-columb method was used for analysis purpose. Friction angle of Colluvium was taken to be 20 degree.

3.2.1 MOHR-COULOMB

The stability analysis by the Mohr- Coulomb approach is based on the Mohr-Coulomb strength criterion that is a critical combination of principle stresses causes failure. The equation derived for deformation in the plastic zone is as follows:

$$u_p = \frac{R_p^2(P \sin \emptyset + \cos \emptyset)}{2r_o G} \qquad (1)$$

This criterion takes into account the principal stresses while the influences of the intermediate stresses are completely ignored.

3.3 APPLICATION OF THE LIMIT EQUILIBRIUM

The slope was divided into number of sections based on their geology. The deformations were calculated on the toe as well as on the upslope on these divisions, the overburden pressure and the slope geometry.A detail of cross sections and their details is given as under;

SECTION 8+00

The section had FOS of 1.37 and required bond length of 27 feet and unbonded length of 60 feet in order to form the bond in competent strata. The angle with horizontal is 15 degrees in all the sections.

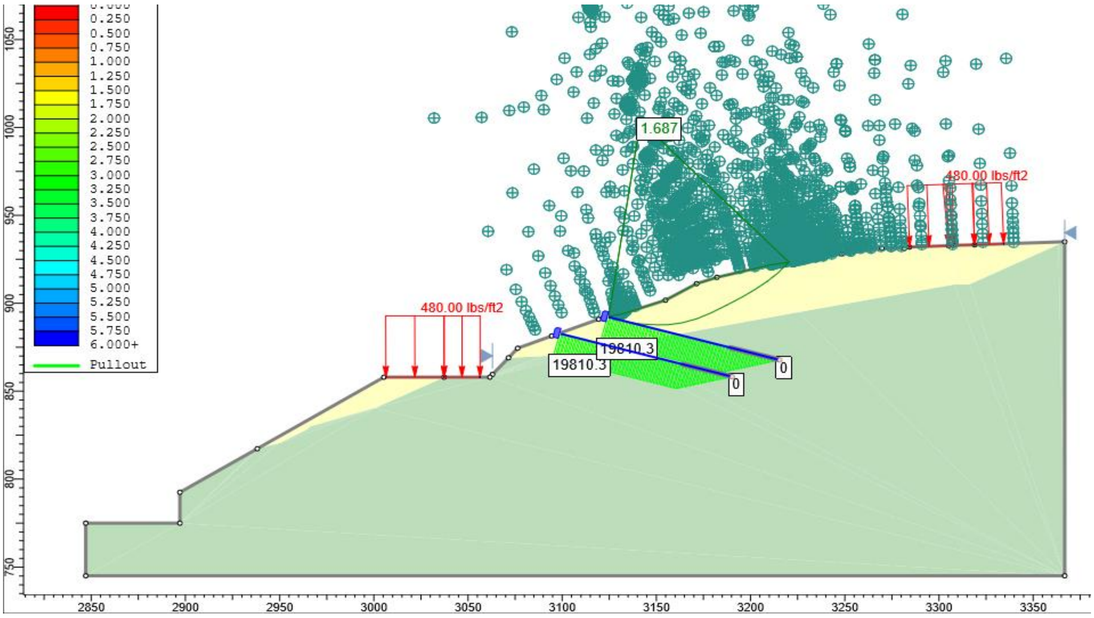

Figure 3.1: Non Circular Analysis of 8+00

Section 9+00

The section had FOS of 1.253 and required bond length of 27 feet and unbonded length of 65 feet in order to form the bond in competent strata. The angle with horizontal is 15 degrees in all the sections.

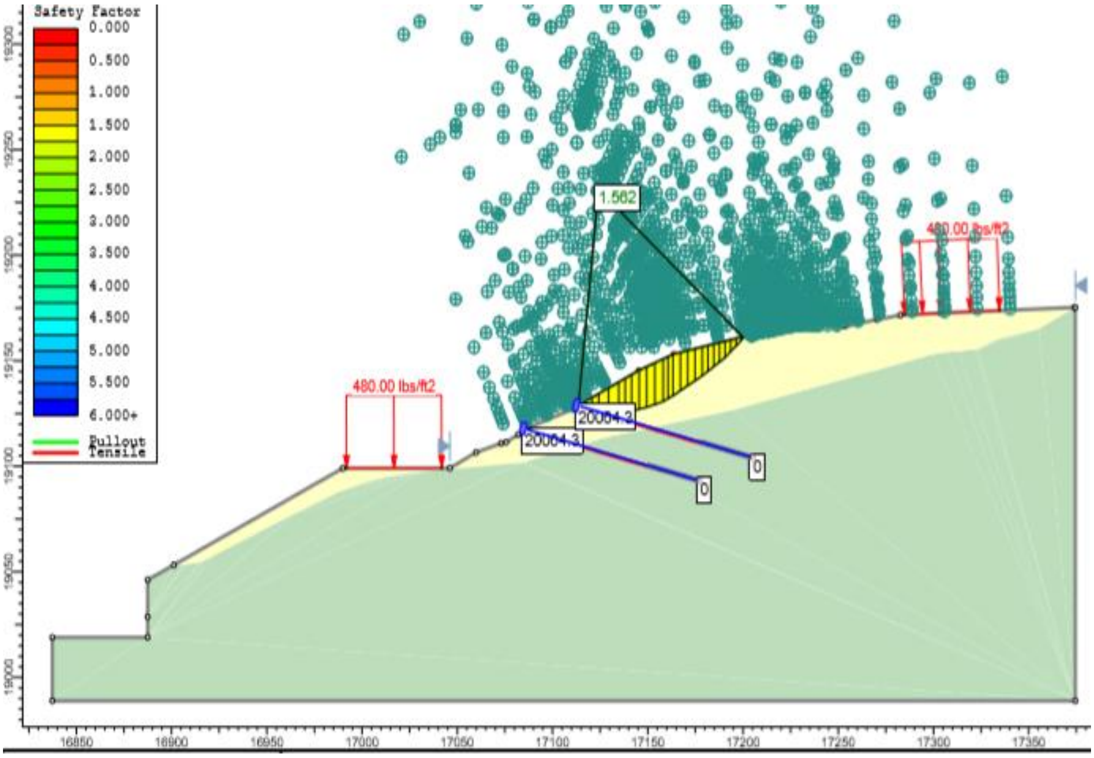

Figure 3.2: Non Circular Analysis of 9+00

Section 10+00

The section had FOS of 1.228 and required bond length of 28 feet and unbonded length of 63 feet in order to form the bond in competent strata. The angle with horizontal is 15 degrees in all the sections.

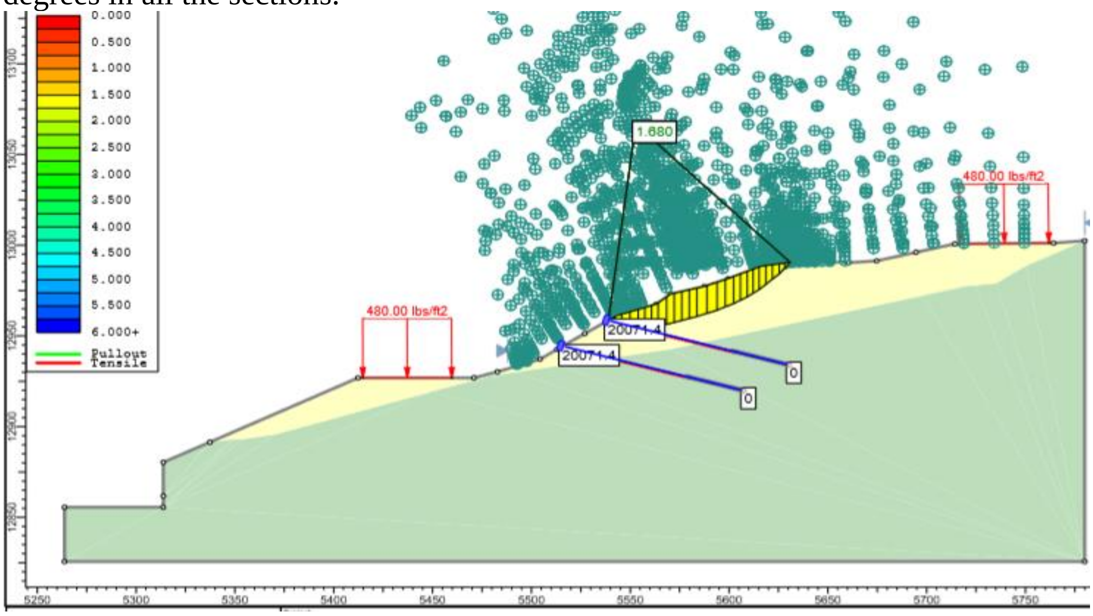

Figure 3.3: Non Circular Analysis of 10+00

Section 11+00

The section had FOS of 1.151 and required bond length of 27 feet and unbonded length of 65 feet in order to form the bond in competent strata. The angle with horizontal is 15 degrees in all the sections.

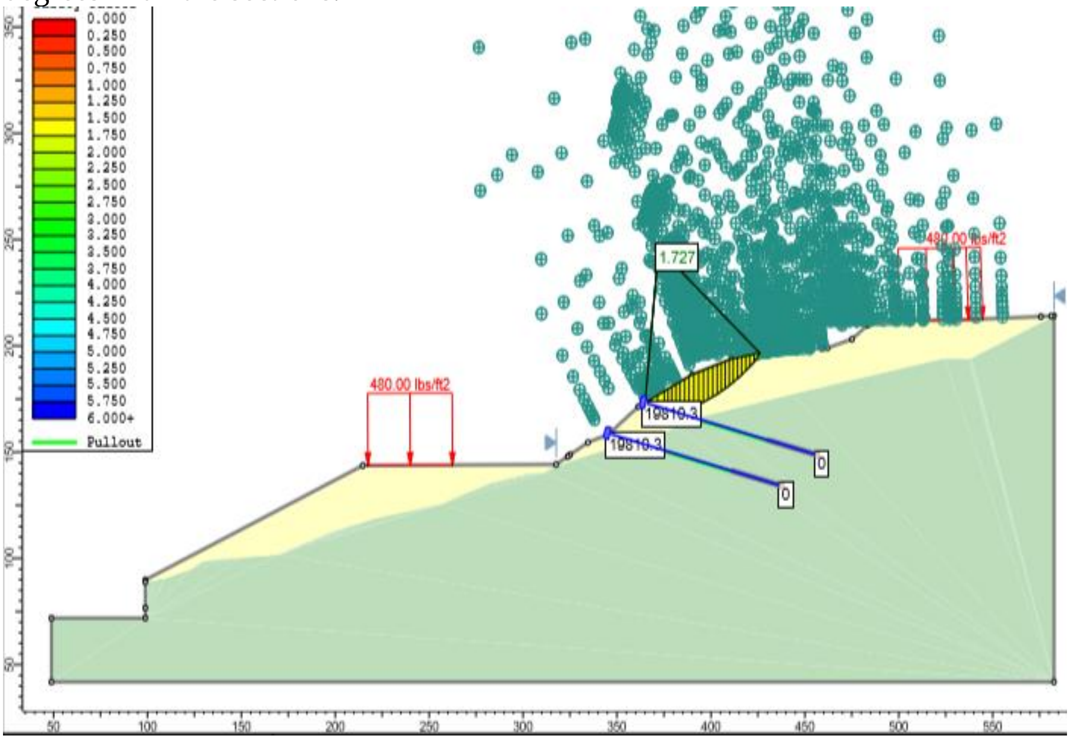

Figure 3.4: Non Circular Analysis of 11+00

Section 12+00

The section had FOS of 1.141 and required bond length of 27 feet and unbonded length of 65 feet in order to form the bond in competent strata. The angle with horizontal is 15 degrees in all the sections.

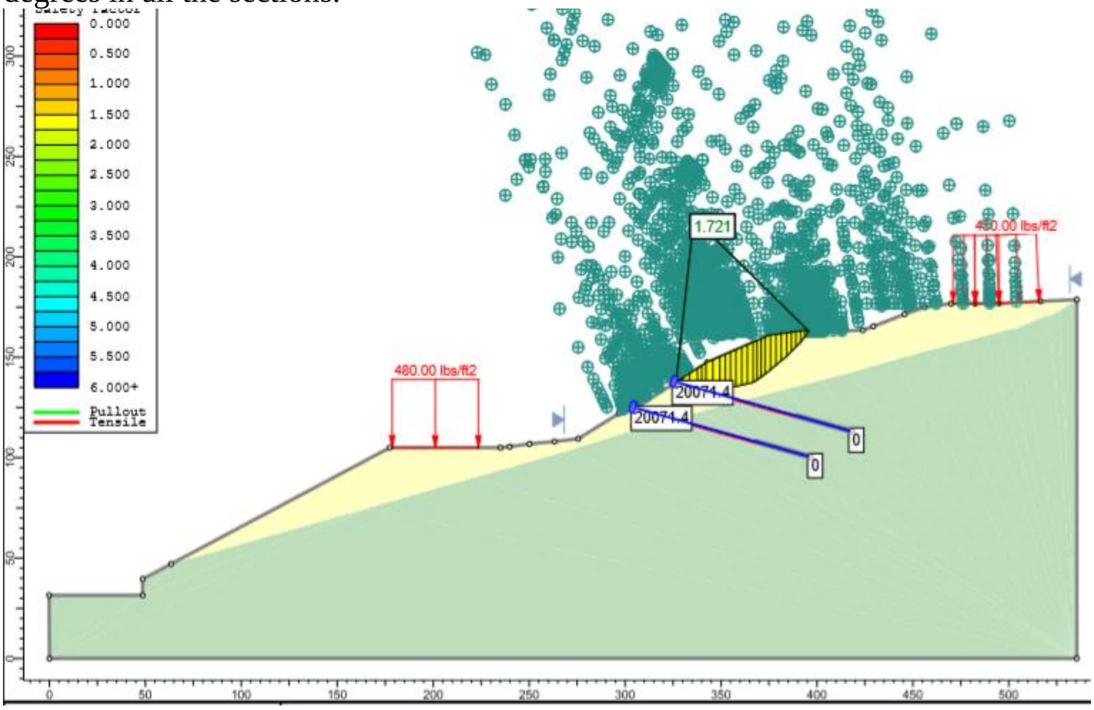

Figure 3.5: Non Circular Analysis of 12+00

Section 13+00

The section had FOS of 1.303 and required bond length of 27 feet and unbonded length of 65 feet in order to form the bond in competent strata. The angle with horizontal is 15 degrees in all the sections.

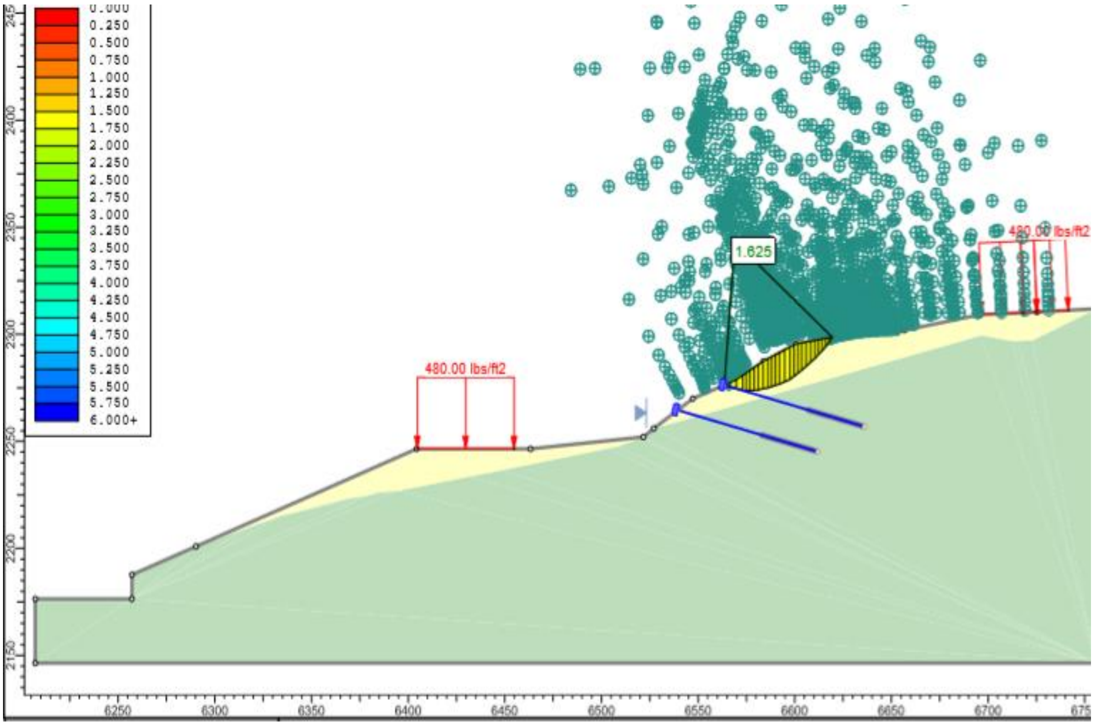

Figure 3.6: Non Circular Analysis of 13+00

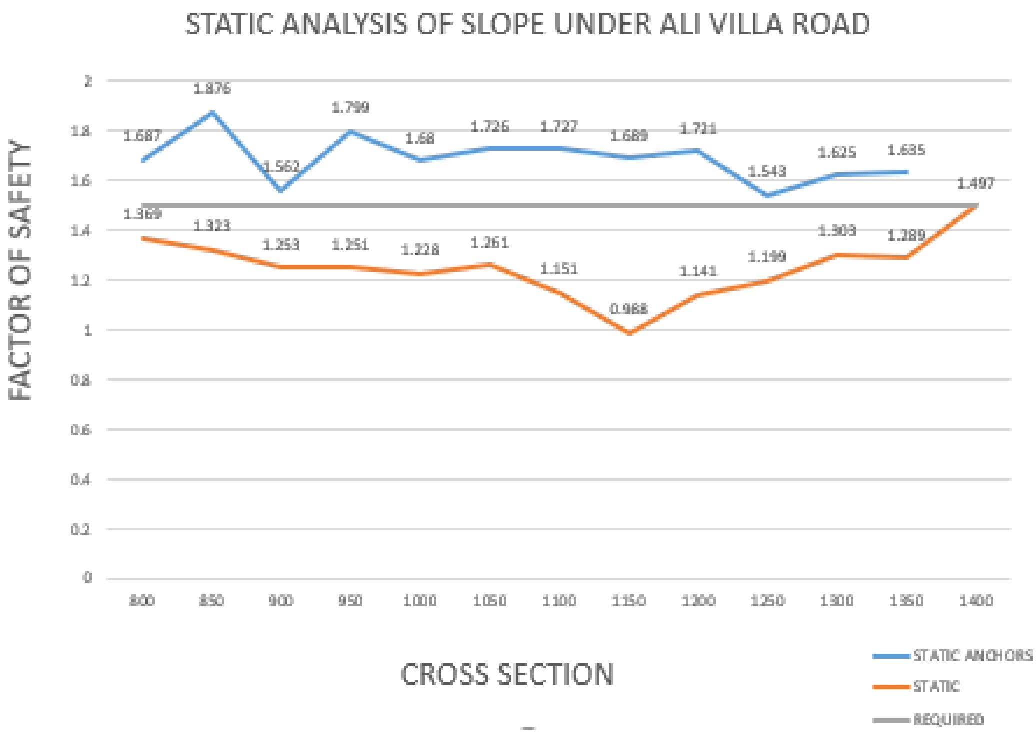

Figure 3.7: Variation of FOS of supported and unsupported slope

3.4 FINITE ELEMENT ANALYSIS

Section 8+00

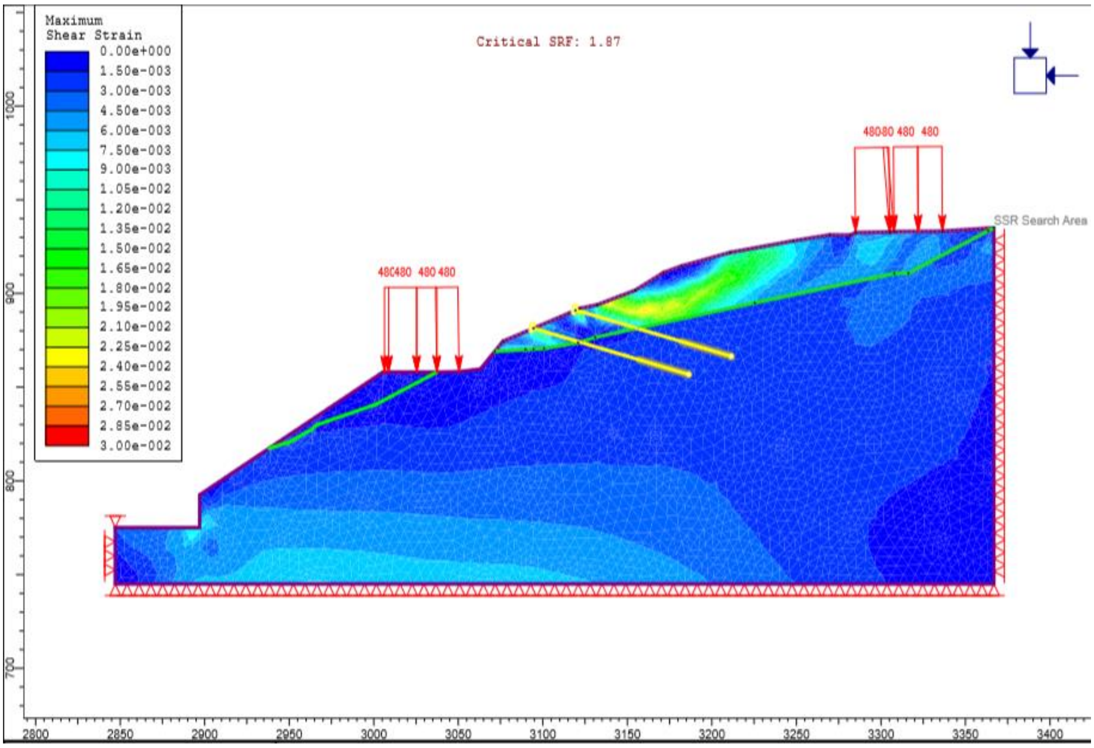

Figure 3.8: SRF Analysis of 8+00

Section 9+00

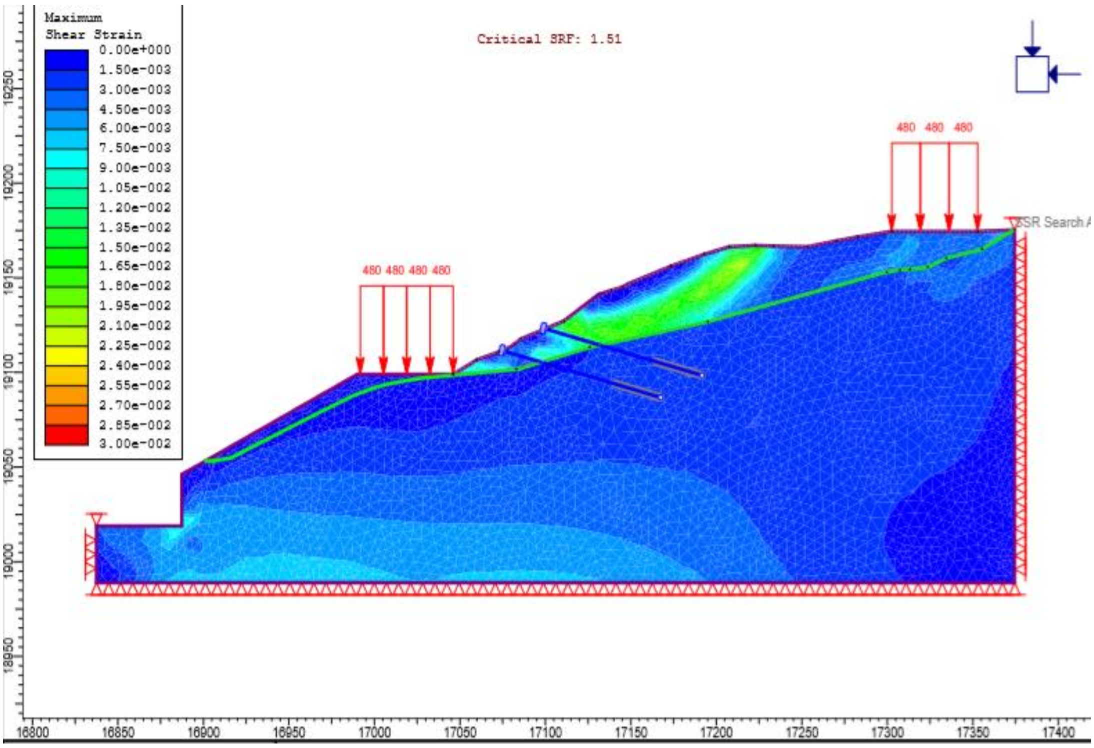

Figure 3.9: SRF Analysis of 9+00

Section 10+00

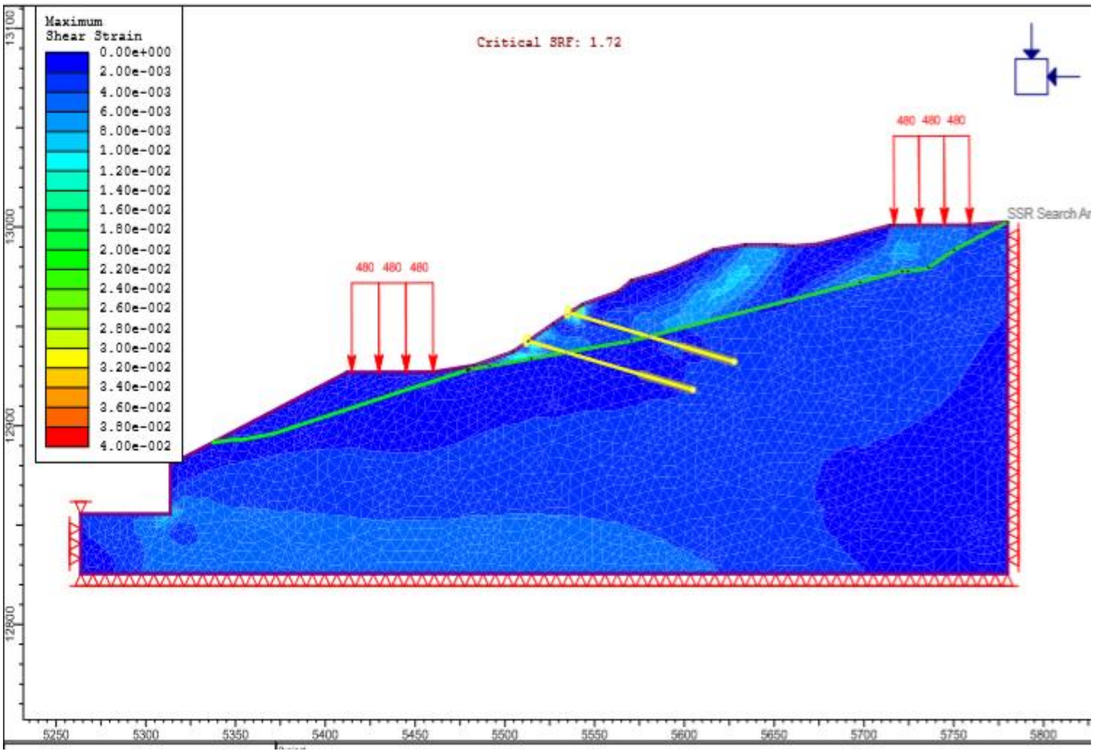

Figure 3.10: SRF Analysis of 10+00

Section 11+00

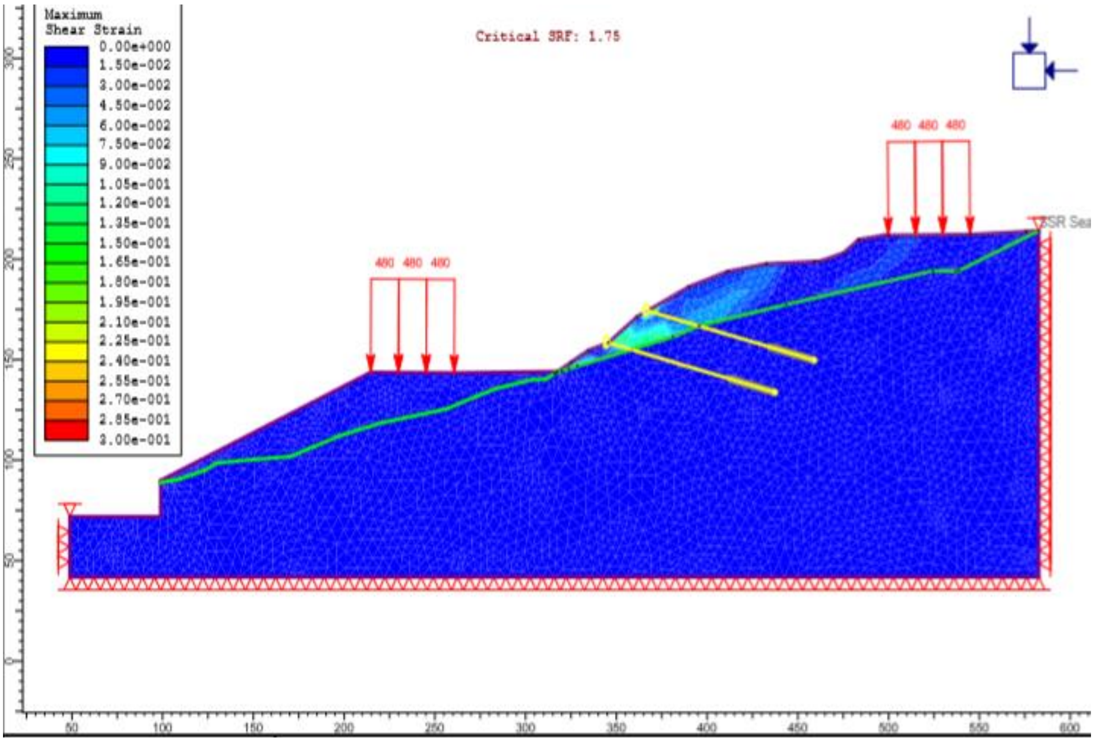

Figure 3.11: SRF Analysis of 11+00

Section 12+00

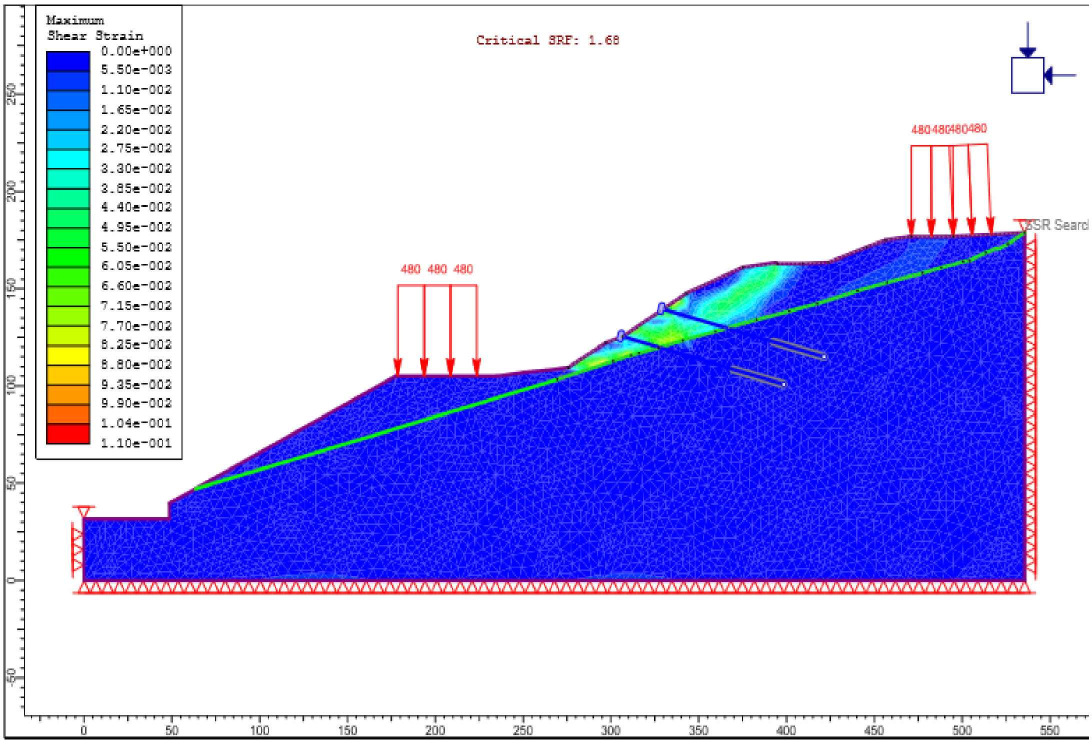

Figure 3.12: SRF Analysis of 12+00

All the results can be summarized as

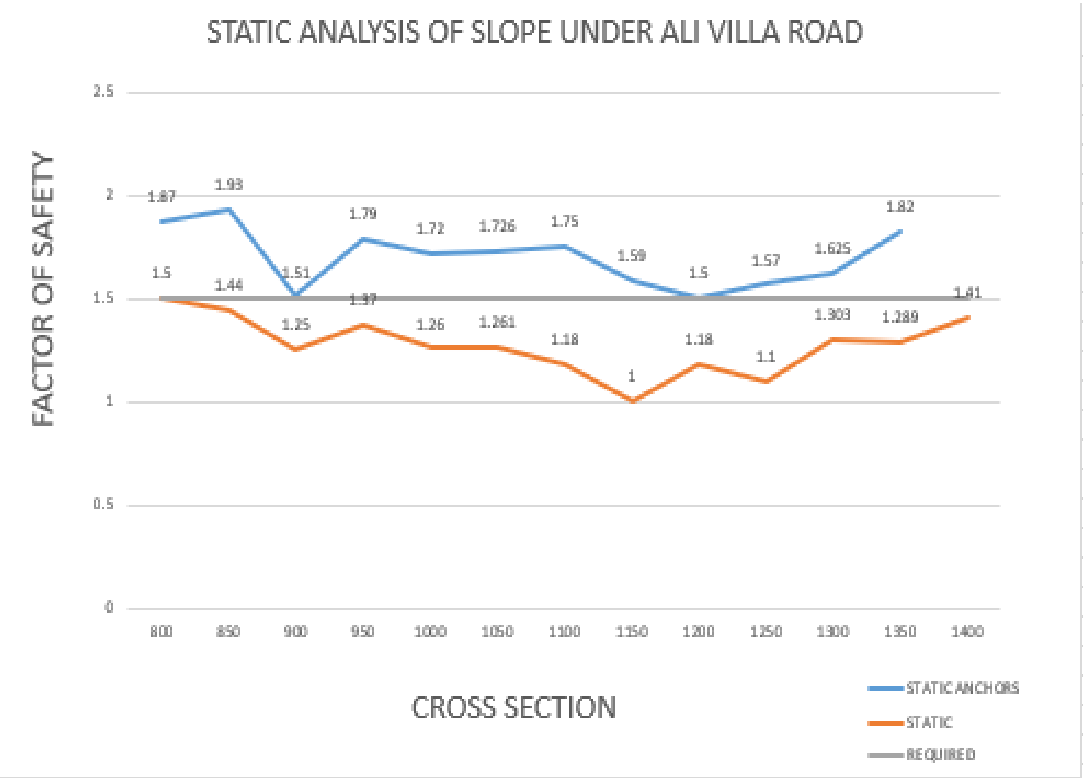

Figure 3.13: Variation of SRF of Supported and unsupported slope

3.5 GABION WALL

"It is the cage, cylinder or box filled with rocks, concrete, sand or soil which is use in civil engineering."

3.5.1 USES

- To stabilize shorelines, stream banks
- Prevent erosion
- Used as RETAINING WALL
- Noise barrier
- Temporary flood walls
- Silt filtration from runoff
- Channel lining
- Temporary or permanent dam
- To protect sappers, infantry, artillerymen

3.5.2 CHECKS

Like other retaining wall gabion wall must pass the following checks to sustain

- Overturning
- Sliding
- Soil bearing
- Eccentricity
- Seismic

3.5.2.1 OVERTURNING CHECK

- The ratio between the resisting moment to overturning moment
- It should be greater than 2.5

Resisting moment is moment of vertical force from the toe of wall, because of weight of gabion rock and overburden over the rock

Overturning moment is moment of active force of soil from the toe of wall, acting horizontal to slide the gabion wall

Coefficient of active earth pressure is calculated by

RANKIENE METHOD

$$K_{ab} = \tan^2\left(45 - \frac{\phi'_b}{2}\right)$$

OR

$$K_{ab} = \frac{\sin^2(\theta + \phi'_b)}{\Gamma \sin^2\theta \sin(\theta - \delta)} \qquad (4\text{-}2)$$

where:

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_b + \delta)\sin(\phi'_b - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2 \qquad (4\text{-}3)$$

β = Nominal slope of backfill behind wall (deg)

δ = Angle of friction between retained backfill and reinforced soil, set equal to β (deg)

ϕ'_b = effective friction angle of retained backfill (deg)

θ = 90^{o} for vertical, or near ($< 10^{\circ}$) vertical, wall (deg)

3.5.2.2 SLIDING CHECK

- The ratio of product of friction coefficient and active force to vertical force due to weight of rock and overburden over rock.
- Factor of safety should be greater than 1.5

FOS = (Cf * Pa)/ Rvt

3.5.2.3 SOIL BEARING

The capacity of soil to support the loads applied to the ground. The bearing capacity of soil is the maximum average contact pressure between the foundation and the soil which should not produce shear failure in the soil.

- It varies from site to site.
- In our site soil capacity was 3 Tsf / 6.67 Ksf so the pressure that the gabion wall exert should be less than mention value.

3.5.2.4 ECCENTRICITY CHECK

Eccentricity is the distance between resultant foundation load and the center of reinforced zone.

Where eccentricity "e" equals

$$\mathbf{e} = \frac{\sum \mathbf{M_D} - \sum \mathbf{M_R}}{\sum \mathbf{V}}$$

- M_D is overturning moment about the bottom center of base
- M_R is resisting moment about the bottom center of base
- V is vertical load

3.5.2.5 SEISMIC CHECK

It analyze the interaction of earthquake with the civil infrastructure. For seismic analysis, we have to calculate coefficient of seismic earth pressure. Using Mononobe Okabe Method, we can have calculate coefficient of seismic earth pressure.

$$K_{AE} = \frac{\sin^2(90+\theta-\phi)}{\cos\theta \sin^2 90 \sin(90+\theta+\phi/2) \left[1+\sqrt{\frac{\sin 1.5\phi \sin(\phi-\theta-\beta)}{\sin(90+\phi/2+\theta)\sin(90-\beta)}}\right]^2}$$

“Table 3.1: Inputs for Stability Check for Gabion Wall

PARAMETER	***SYMBOL***	**VALUE**
Wall height	H	Depends on cross section
Wall angle	α	85
Soil-wall friction angle	δ	19
Backfill angle	β	19
Backfill specific weight	ϒb	105
Soil internal friction angle	ϕ	30
Specific weight of rock	ϒr	130
Co-efficient in horizontal direction	Kh	Through graph, depends on severity of site
Co-efficient in vertical direction	Kv	0
Theta	Θ’	Tan^-1 (kh/(1-kv))

3.6 METHODOLOGY

For gabion wall we had calculated all five checks using

- Excel
- Software

3.6.1 EXCEL

The excel sheet provided us with the cross section of appropriate height that would fit in all the required checks to stable the backfill and traffic surcharge.

“Table 3.2: Heights of Gabion Wall at Cross Sections

CROSS SECTION	HEIGHT (FEET)
8+00	21
8+50	15
9+00	18
9+50	21
10+00	33
10+50	36
11+00	33
11+50	36
12+00	33
12+50	30
13+00	27
13+50	24
14+00	15

Figure 3.14: Cross section imported from AutoCAD

We have to support the proposed road which is at an elevation which can be witnessed in above figure.

We had to provide the base to gabion wall along with the corresponding height mentioned above, such that it satisfy all five checks. One such example given below, is of cross section 8+00 where the required height to support gabion wall on ground is 21 feet. We used the gabion block unit of 3*3*3 as it is only available in market. Therefore the height and base are multiple of 3.Below tables are tables copied from MICROSOFT EXCEL to display the results of cross section 8+00.

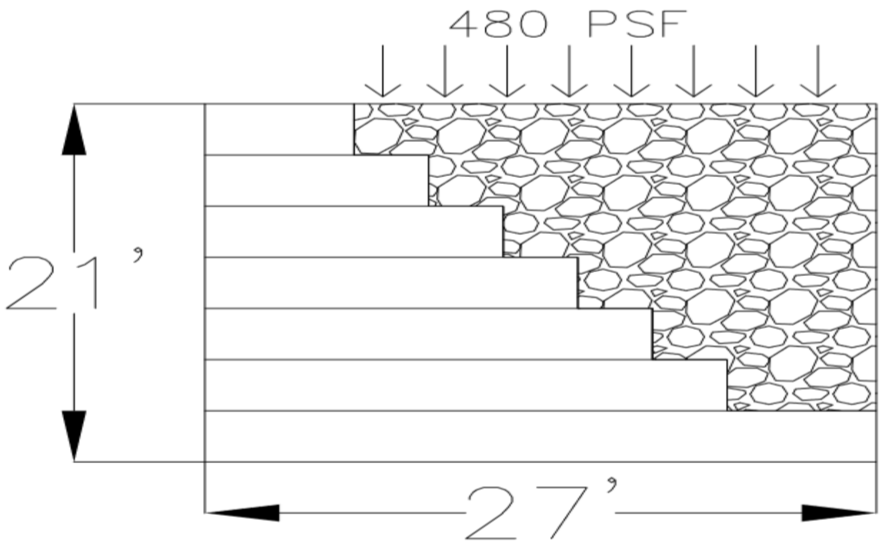

Figure 3.15: Gabion Wall Characteristics at 8+00

3.6.2 CALCULATION OF GABION MASS

3.6.2.1 CALCULATION OF BACKFILL

BASE	***HEIGHT***	***AREA***	***ϒb***	***R_v***	***MOMRNT ARM***	***MOMENT***
21	3	63	105	6615	16.5	109147.5
18	3	54	105	5670	18	102060
15	3	45	105	4725	19.5	92137.5
12	3	36	105	3780	21	79380
9	3	27	105	2835	22.5	63787.5
6	3	18	105	1890	24	45360
				∑ **V=** 25515		∑ **M** =491872.5

3.6.2.2 OVERTURNING CHECK

VARIABLE	***VALUE***	***UNITS***
HEIGHT	21	ft.
Υ_b	105	lb./ft^3
LIVE LOAD	480	lb./ft^2
H_{eq}	4.5714286	lb./ft^3
α	5	DEGREE
P_{a1}	5199.5569	lb.
P_{a2}	9807.7187	lb.
P_a	15007.276	ft.
VERTICAL LOAD	67635	lb.
MR	877972.5	lb. ft.
OTM	135134.08	lb. ft.
FOS	***6.50***	

3.6.2.3 BEARING CHECK

VARIABLE	VALUE	UNITS	DESRIPTION
BASE	***27***	ft.	
VERTICAL LOAD	67635	lb.	
AREA	27	ft^2	
M_R	877972	lb.	
OTM	135134	lb.	
ECCENTRICITY	***2.52***	***ft.***	***< 4.5***
Q_{max}	***-1.103***	***Ksf***	***< 6.67***
Q_{min}	***-3.91***	***Ksf***	***< 6.67***

3.6.2.4 SLIDING CHECK

VARIABLE	***VALUE***	***UNITS***	***DESRIPTION***
ϕ_b	20	DEGREE	
C_f	0.3639702	UNITLESS	
P_a	15007.276	kips	
VERTICAL LOAD	67635	kips	
FOS	***1.64***		***>1.5***

3.6.2.5 SEISMIC CHECK

VARIABLE	***VALUE***
K_v	0
K_{ae}	0.926
P_{ae}	21439.22
FOS	***1.15***

- Here it can be witnessed that every CHECK is above par as FACTORY OF SAFETY is the above required value.

Similarly, every cross section was solved using excel sheet. In this way, we got the required cross section of gabion wall for the corresponding cross section. Below table shows the cross section, height and base.

"Table 3.3: Heights and Bases of Gabion Wall at all Sections

CROSS SECTION	HEIGHT	BASE
8+00	21	27
8+50	15	24
9+00	18	27
9+50	21	27
10+00	33	42
10+50	36	45
11+00	33	42
11+50	36	45
12+00	33	42
12+50	30	39
13+00	27	33
13+50	24	33
14+00	15	24

- Using above mentioned height and base at the matching section we acquired the following gabion wall section
- Below figure give us **EIGHT** gabion wall section at **THIRTEEN** different positions as few of cross section position requires similar height e.g.
  1. 8+00 and 9+00 requires 21 feet height to support proposed road
  2. 10+00, 11+00 and 12+00 requires 33 feet height to support proposed road
- Gabion wall are at the batter of 5 degrees

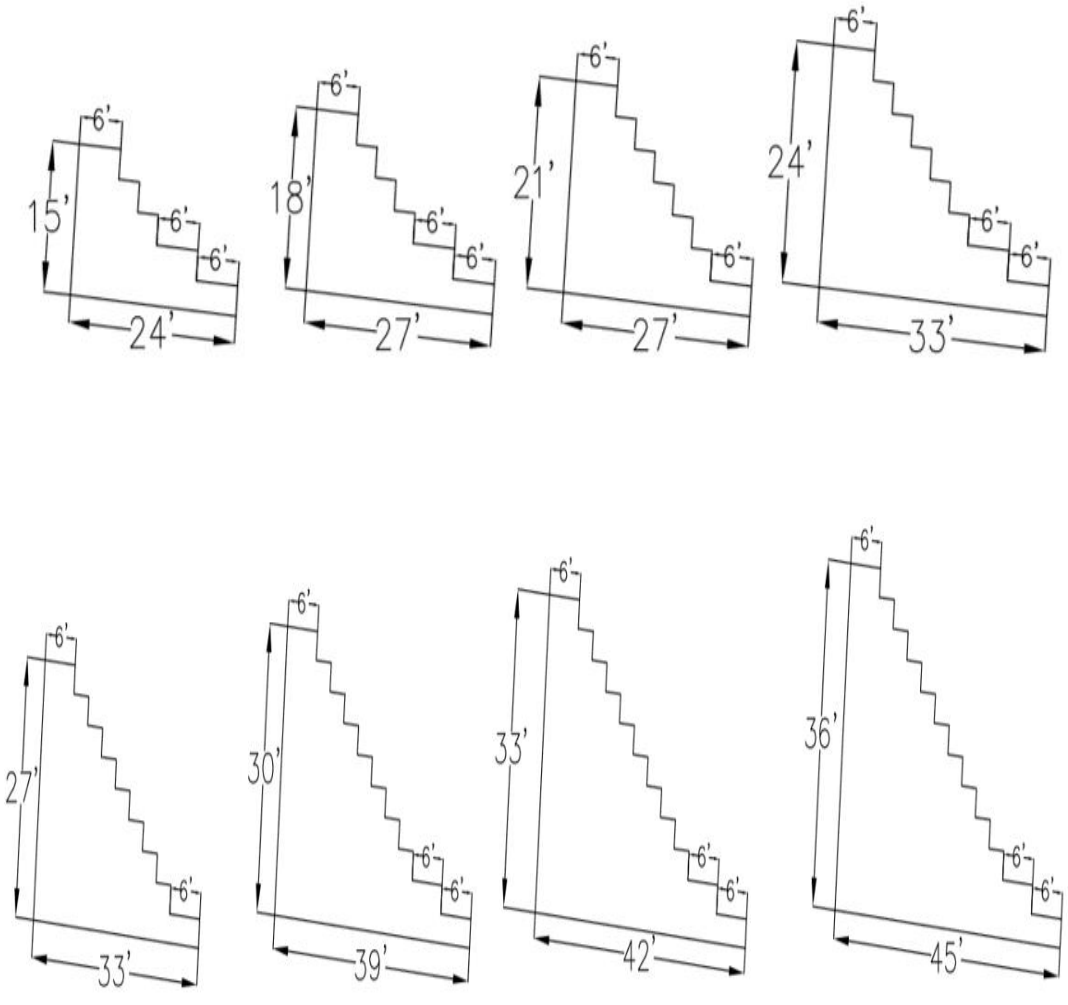

Figure 3.16: Illustrations of all Cross Sections

Using above mentioned **HEIGHT, BASE and BATTER ANGLE** at matching cross section we got **FACTOR OF SAFETY** for every cross section.

- We have compared results of slope under proposed road without support (using ROCK SCIENCE SLIDE 2.0) and Microsoft excel with support (GABION WALL)

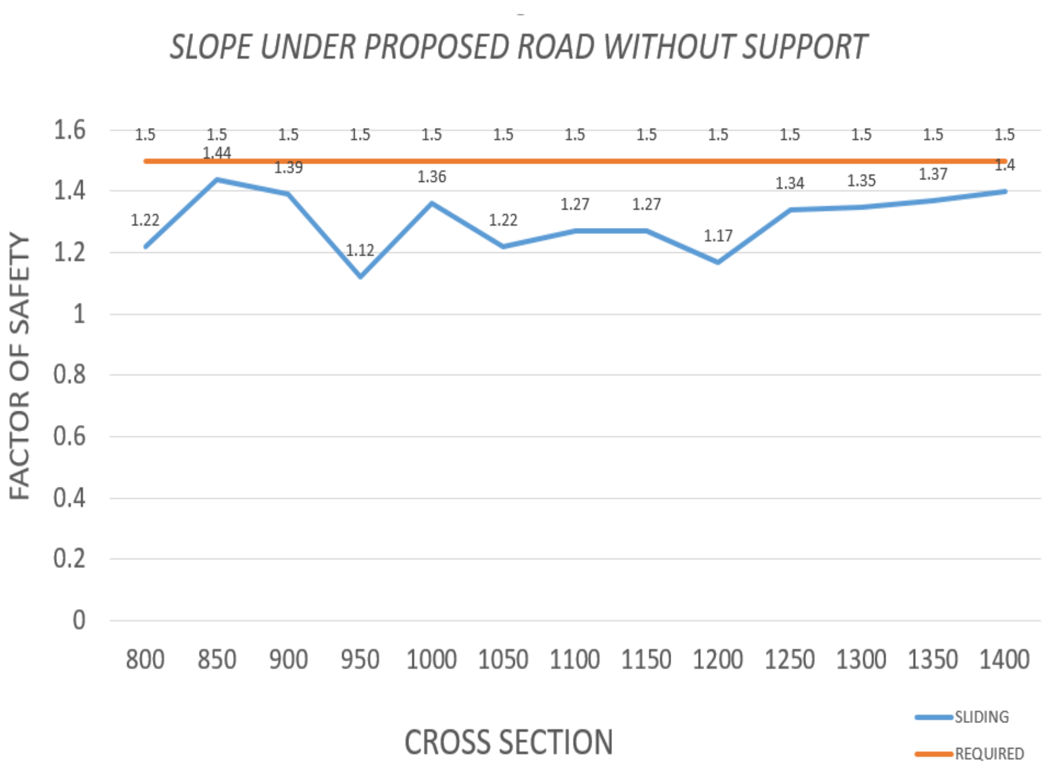

Figure 3.17: Variation of FOS

- Here we can observe that sliding FACTOR OF SAFETY for SLIDING (lower line) is quiet below the required value i.e. 1.5
- Quiet evident that failing in sliding in static analysis will fail in seismic analysis too

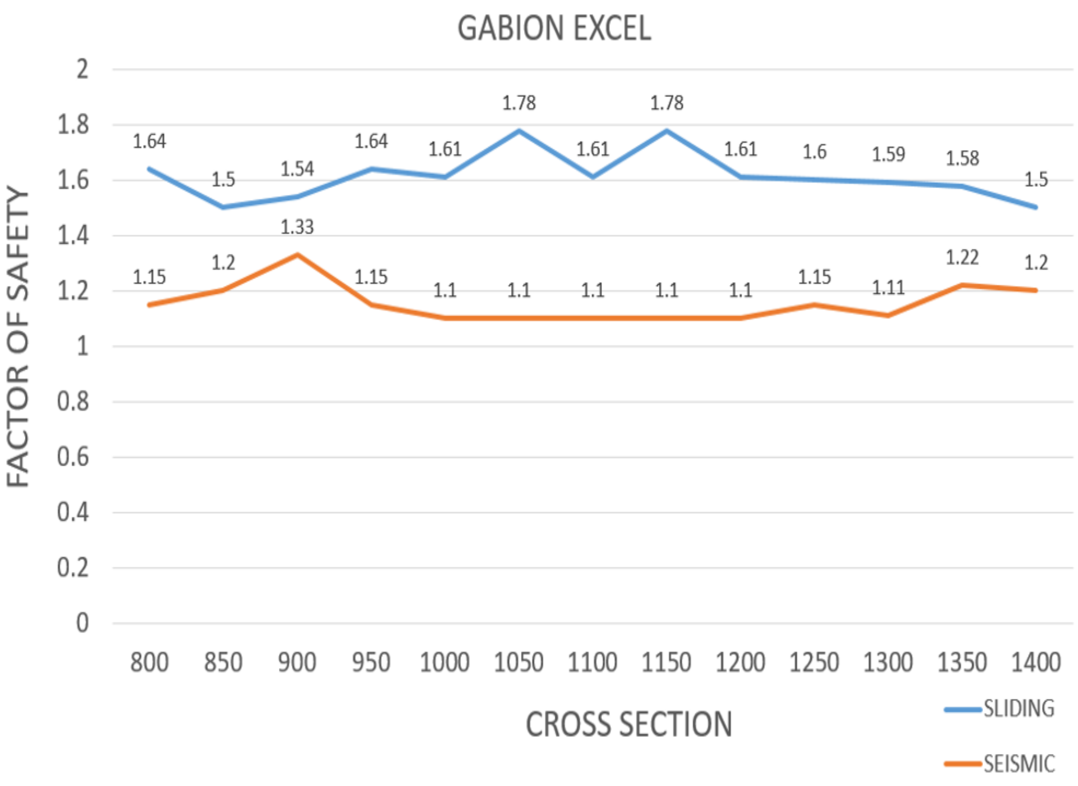

Figure 3.18: Variation of FOS of Supported and Unsupported Sections

- This figure shows the result of MICROSOFT EXCEL calculation
- Static sliding FOS (upper line) is representing value equal and greater than 1.5
- Seismic sliding FOS (lower line) is representing value equal and greater than 1.1

3.6.3 SLIDE RESULTS

- We were provided with the cross section in AUTOCAD file, we converted the cross section in DXF file and imported it into the slide.
- Defining layer and material is the second step. The parameters that were our input are mentioned before.
- For foundation rock, we used generalized HOEK AND BROWN method.
- Our proposed road was at an elevation, having no contact with the ground. For this, we gave a slope to the soil under the road maximum of 19^0 as our Φ_b was 19^0
- But the slope was without support, the results in slide shown above therefor less than 1.5

SECTION 8+00

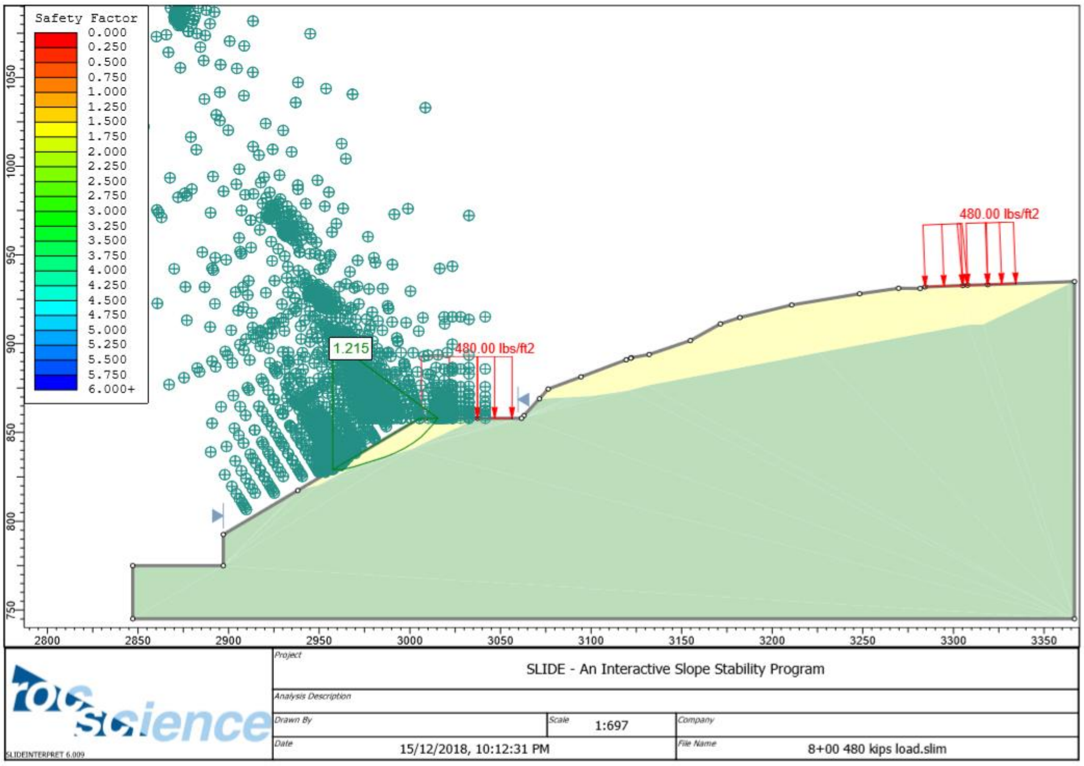

Figure 3.19: Non Circular Analysis of 8+00

SECTION 9+00

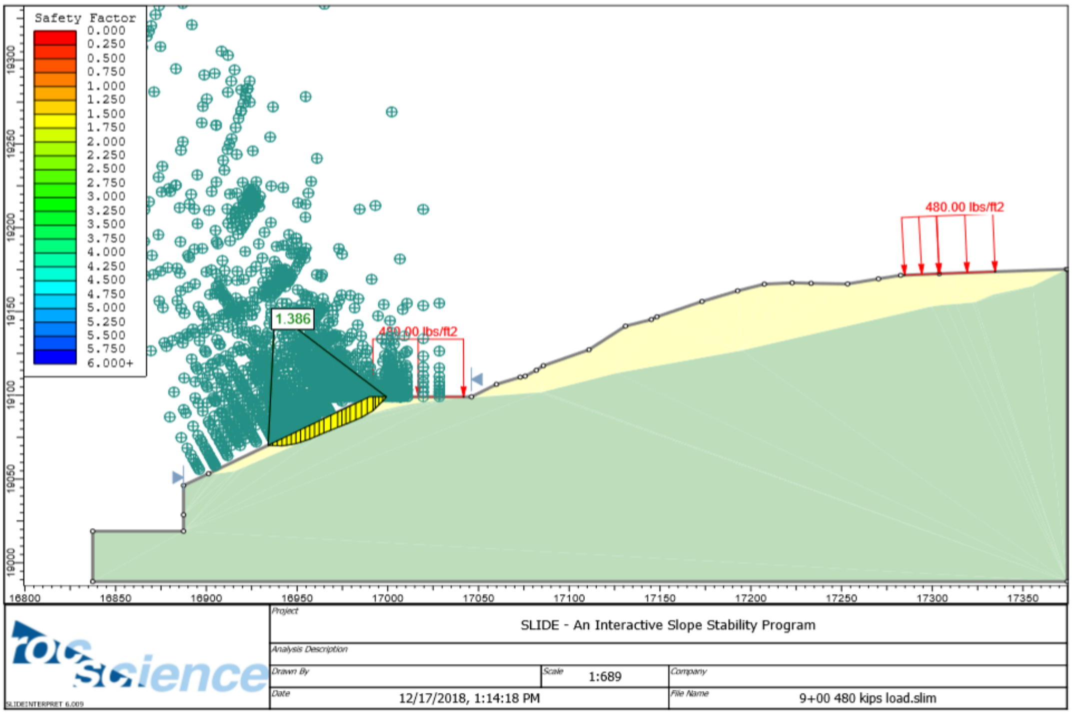

Figure 3.20: Non Circular Analysis of 9+00

SECTION 10+00

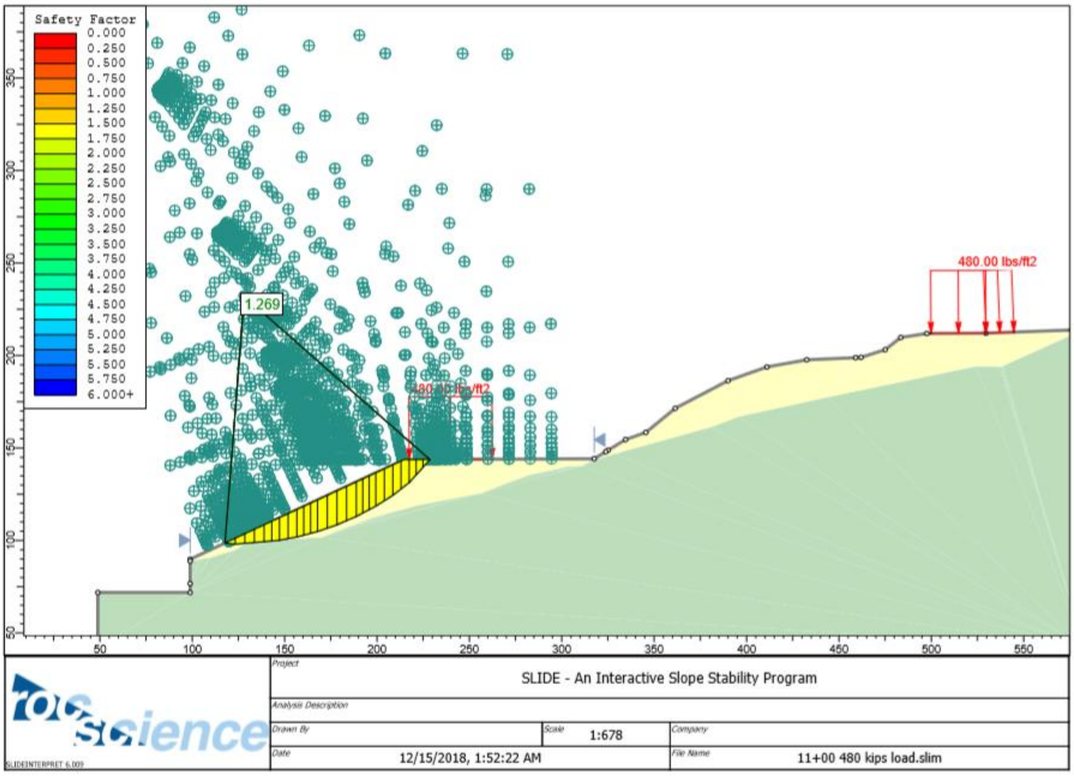

Figure 3.21: Non Circular Analysis of 10+00

SECTION 11+50

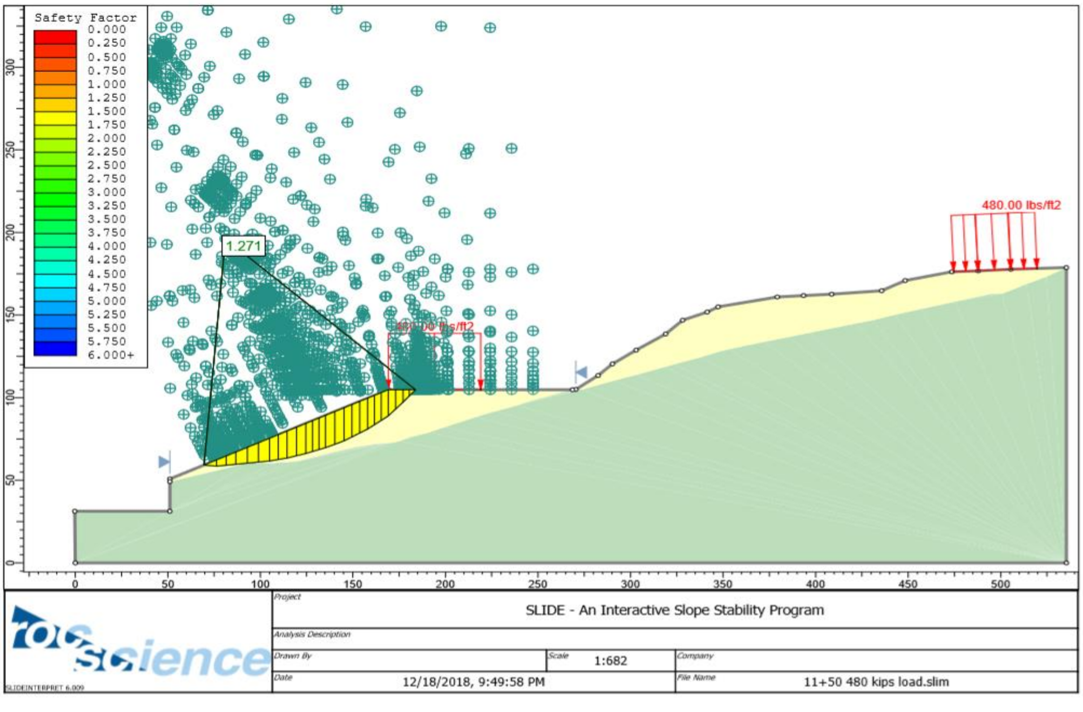

Figure 3.22: Non Circular Analysis of 11+50

SECTION 12+50

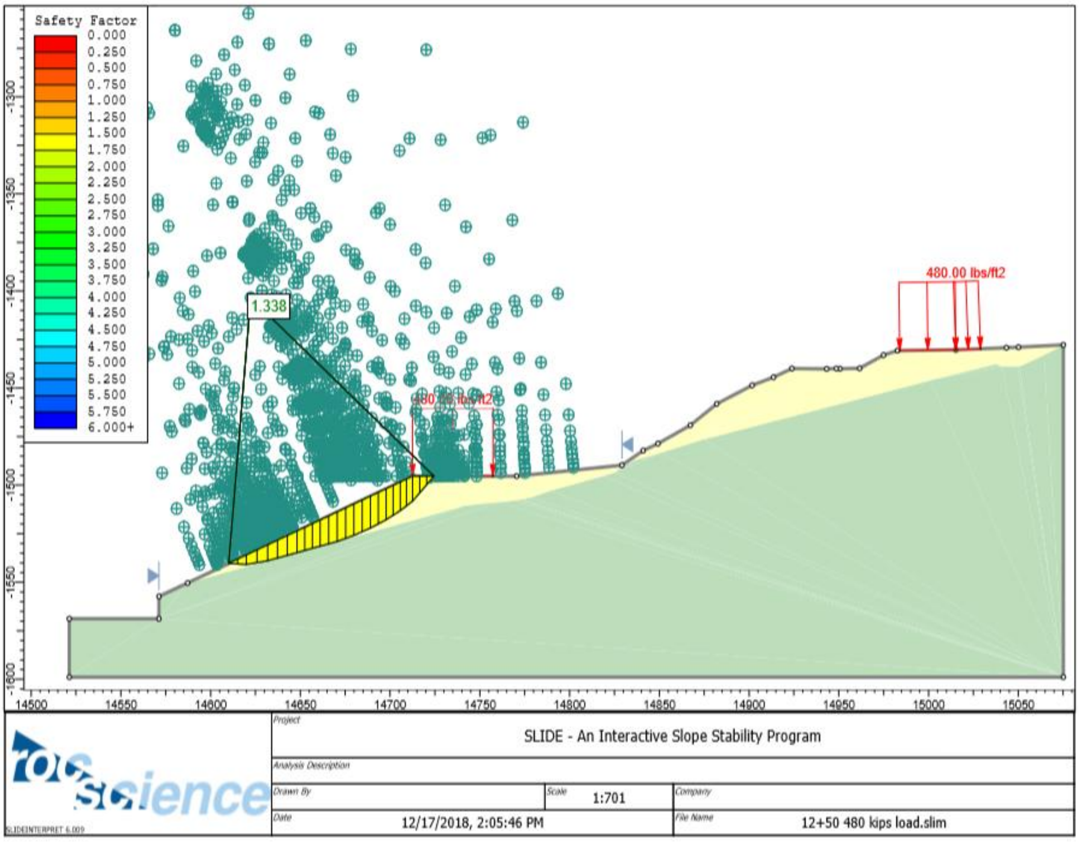

Figure 3.23: Non Circular Analysis of 12+50

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

4.1 REVIEW

The objectives of this project were the following:

- Analysis of slope deformation of selected cross sections using:
    1) Limit Equilibrium Analysis
    2) Finite Element Modelling.

4.2 CONCLUSIONS

The following conclusions were made from this project:

- Limit Equilibrium results show similar trend to FEM results, with values generally lower than FEM results.

4.3 RECOMMENDATIONS

Based on the conclusions following recommendations were made:

- Field observation of deformations should be recommended for future projects.
- Monitoring of deformation should be done extensively in a number of different locations as to provide a comprehensive data set. Larger data set results in better training of the model which in turn results in better prediction of deformation.

4.4 REFERENCES

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

Cornforth, H.Derek, (2005).Landslides In Practices. John Wiley & Sons, Inc. Hoboken, New Jersey.

Joseph E. Bowles, (1997). Foundation Analysis and Design 5th Edition. The McGraw-Hill Companies, Inc. Singapore.

P. K. Woodward, "Earth pressure coefficients based on the Lade–duncan failure criterion," Engineering Structures, vol. 19, no. 9, pp. 733–737, 1997

Bieniawski, Z.T. 1976. Rock mass classification in rock engineering. In Exploration for rock engineering, proc. of the symp., (ed. Z.T. Bieniawski) 1, 97-106. Cape Town: Balkema.

The Canadian Foundation Engineering Manual 4th Edition (2006).