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Comparison of design/analysis methods for pile reinforced slopes

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Comparison of design/analysis methods for pile reinforced slopes

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The student author and the program of study committee are solely responsible for the content of this thesis. The Graduate College will ensure this dissertation is globally accessible and will not permit alterations after a degree is conferred.

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DEDICATION

To God, my mother; Nancy González and my husband: Henry Bello.

TABLE OF CONTENTS

LISTS OF TABLES

LISTS OF FIGURES

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ABSTRACT

Potentially unstable slopes can be treated by several measurements such as geometry changes, reinforcement, or avoidance of the problem. If avoidance and/or geometry changes are not viable options, the slope may be strengthened. Strengthening of slopes can incorporate many different technologies: drilled shafts, soil nails, tieback anchors, and micropiles as reinforcing elements. Among these technologies, the use of piles have been found effective and economical. The current methods of analysis for pile-reinforced slopes are based on either limit equilibrium (LE) or geomechanical numerical modeling (finite element method, FEM, and finite difference method, FDM). Although in recent years there has been an increase in the use of geomechanical numerical modeling, designers still question the relative advantages, limitations, and accuracy of these methods compared to traditional methods.

In this study, a comparative analysis have been performed, and the results of a Deep Foundation Institute, Deep Foundations for Landslides/Slope Stabilization Committee study on Design Comparisons of Slope Stabilization Methods are reported. The evaluation was focused on comparing the current methods of advanced numerical modeling for pile reinforced slopes (LE, FEM, FDM) by analyzing three cases using different analysis approaches performing coupled and uncoupled analysis.

From the results, recommendations regarding the selection of the most beneficial method for stability analysis are given. Conclusions regarding pile optimum location, pile optimum length, key factors for each type of analysis, and lesson learned are presented.

CHAPTER 1. INTRODUCTION

A deep understanding of the factors involved in slope stability mechanism has been fundamental in the geotechnical field for several years. Ensuring the support of structures that have been designed and constructed are able to withstand soil and induced movements is a main concern for geotechnical engineers. In many areas, slope instability is a major threat disrupting infrastructures, causing casualties and economical losses. Therefore, prevention is desirable rather than mitigation (Turner and Schuster 1996). However, prevention may not be a viable option once movement has initiated and failure has started.

To evaluate if a slope will be safe enough to prevent failure, a slope stability analysis must be conducted. The basis of this analysis is determining whether or not soil strength and stresses caused by gravity through the soil are in equilibrium. Therefore, because these forces depend on soil properties, the stability analysis is a complex mechanism that remains a challenge for geotechnical engineers.

Significant research and theories have been developed to analyze slope stability. Among these theories, the limit equilibrium method (LEM) and geomechanical numerical methods (finite element method (FEM) and finite difference method (FDM)) represent the most common approaches. The limit equilibrium method has been claimed to be simple and easy to apply but limited or inaccurate (Duncan 1996, Griffins and Lane 1999, Cai and Ugai 2000, Geo-Slope International 2010) . On the other hand, the finite element and finite difference approaches are considered more useful and accurate, but time-consuming, complicated and costly (Duncan 1996, Jeong et al. 2003 ,Nian et al. 2008).

After performing the analysis, if the slope is found unstable, several measures can be taken to mitigate the stability problem such as geometry changes, reinforcement of the slope or avoidance of the problem. The selection of an adequate treatment will greatly depend on factors such as soil properties, rate of movement, surrondings's geometry, and structures involved. If avoidance and geometry changes are not viable options, the slope may be strengthened. This may be done by placing drilled shafts, soil nails, tieback anchors, micropiles, or stabilizaing the soil to resist the movements.

Among the mentioned technologies, the use of pile foundations for this purpose was found to be effective for most of the practical cases (Nian et al. 2008). Several authors have published the details of the method (Ito and Matsui 1975, Fukuoka 1977, Nethero 1982, Poulos 1995, Lee et al. 1995, Chen and Poulos 1997, Hassiotis et al. 1997, and others). The general types of analyses are chart or closed form solutions, limit equilibrium, limit analyses, and continuums method that encompasses elastic analysis, elastoplastic analysis using finite element, finite difference, discrete element or boundary elements (Carter et al. 2000). Currently, the finite element method and the finite difference method are considered the most useful and accurate ways to evaluate pile reinforced slopes (Nian et al. 2008). However, they are considered expensive and complicated. Therefore, traditional methods to evaluate slope stability are still widely applied for simplicity purposes.

When analyzing pile-reinforced slopes the complicated soil-pile interaction mechanism is added to the complex mechanism of slope stability, making the analysis of a reinforced slope difficult and uncertain. The soil resistance along the pile will depend on the pile deflection, and likewise the deflection of the pile is a function of the soil resistance; therefore, analysis of the piles must be performed to verify their resistance to the loads developed by the soil movements (Hassiotis et al. 1997).

The overall pile reinforced slope stability analysis can be done as coupled or uncoupled (Poulos 1995, S.Hassiotis et al. 1997, Jeong et al. 2003, and Won et al. 2005). A coupled analysis refers to the kind of analysis in which pile and slope are analyzed together and an uncoupled one refers to the analysis of the stabilizing pile and the slope separately. Regarding these types of analysis, numerical methods such as finite element and finite difference method allows for the characterization of both pile and slope simultaneously. On the other hand, procedures based on limit equilibrium analysis do not include the stabilizing pile responses. As a result, additional analysis of the stabilizing pile must be conducted.

No general agreement has been found as to which type of analysis (couple or uncouple) will represent the real conditions of the pile/slope system as close as possible. Authors such as Jeong et al. (2003), Won et al. (2005) and Cai and Ugai (2000), have presented comparisons between both types of analysis. However, no significant information regarding relative advantages, limitations, and accuracy of each method have been clearly reported. In addition, when performing uncoupled analysis no widely acknowledged method for analyzing and designing the stabilizing piles has yet to be presented (Chow 1996, Guo and Lee 2001, Thompson et al. 2005).

1.1 Scope of the Work

This thesis describes the basics of the current approaches to conducting pile reinforced slope stability analysis. In addition, the results from a DFI Deep Foundations for Landslides/Slope Stabilization Committee study on Design Comparisons of Slope Stabilization

Methods are reported. The comparative analysis will be performed by evaluating three cases under uncoupled and coupled conditions. For performing the uncoupled analysis, two limit equilibrium programs; SLOPE/W (Geo-Slope International 2010) and Slide 7 (Rocscience Inc. 2016), will be used in combination with LPILE 09.010 (Isenhower et al. 2016) for the analysis of laterally loaded piles. On the other hand, FLAC 2D (Itasca 2005) and Phase2 (Rocscience Inc 2017) will be used for the coupled analysis as FD and FE software. The calculations from FLAC were performed by Dr. Daniel Pradel as part of the DFI project. The strength reduction analysis will be used to compare results from the geomechanical numerical analyses to the factor of safety from limit equilibrium analyses.

1.2 Research Objectives

The two main objectives of this research are (1) investigate methods and accuracy of advanced numerical modeling for slope stabilization methods (LEM, FEM and FDM) through comparing three cases using different analysis approaches performing coupled and uncoupled analysis, and (2) to provide recommendations for the selection of a given method for stability analysis from a comparative basis regarding relative benefits when compared to traditional methods.

1.3 Study Outline

The research outlined above is presented in four chapters. Chapter 2 provides background information and reviews previous literature related to this study. Chapter 3 describes the methodology and information regarding the three analyzed cases and the summary and discussion of the results. Chapter 4 recaps the conclusions and key findings derived from the analyses to report some suggestions for future research. Supporting materials are included as appendices that follow the list of references that made possible the elaboration of this thesis.

Key Terms:

Slope stability, limit equilibrium, pile reinforced slope, numerical modeling in geotechnical engineering, constitutive models.

CHAPTER 2. LITERATURE REVIEW

The basics of slope stability are presented in this chapter as well as the current methods of analysis for stability problems. There are two (2) primary methods of analysis that are currently used to estimate the stability of unreinforced and reinforced slopes: (1) methods based on limit equilibrium analysis and (2) numerical-based analysis methods (such as finite element analysis, strength reduction finite element method, and finite difference methods). The major assumptions for each method, the advantages and disadvantages are discussed .The practical applications that have been developed are mainly based on these theories of analysis with variations and modifications that have been introduced by authors to address a specific aspect of an analysis approach.

2.1 Basics of Slope Stability

Through the years, slope failures have been responsible for immeasurable economic losses and casualties all around the world. According to Turner and Schuster (1996), landslides or mass wasting represent a major component of numerous “multiple-hazard disasters”. Hence, regarding the improvements in the prediction and mitigation of this type of events, a greater amount of effort must be focused to increase the general understanding of the assessment of the safety of slopes (natural or man-made slopes). This assessment is generally done by conducting a slope stability analysis in which the major premise is to determine whether or not the soil beneath the slope will be able to withstand loads without undergoing failure.

Commonly, stability is determined by balancing the resisting forces with driving forces. The resisting forces, the soil strength, act opposite to the movement holding the soil or rock in place while the driving forces, gravity through the soil weight, tend to pull the mass of soil or rock down the slope. Hence, because both forces depend on soil properties, stability analysis is a complex mechanism that remains a challenge for geotechnical engineers. The mechanism of failure of slopes has been extensively studied and according to Duncan (1996) is considered one of the areas of practice that has produced the most important advances in the complex behavior of soils. However, they are gaps that need to be addressed in terms of selection of an appropriate method of analysis for a particular given problem.

2.1.1 Types of Slope Failure

Stability can be defined as the safety of the earth mass against movement. Hence, slope failure or mass wasting is the vertical and/ or horizontal soil displacement down slope of the earth mass, soil, rock or debris (Turner and Schuster 1996). These movements have been subdivided into six groups regarding the characteristics of the failure as shown in Figure 1 (Cornforth 2005, following the original work of Varnes (1978) and Cruden and Varnes (1996)):

a. Falls: vertical sliding of the surface particles in a slope. This involves large areas and the movement is produced gradually between the mobile and fixed particles. The slide surface is difficult to define.
b. Topples: rotation about some axis point in a forward direction due to the action of gravity and forces applied by fluids or elements within the cracks.
c. Slides: refers to the downslope movement of a block of material, it could also be referred as the movement of the slope body. It can be rotational and translational. The slip surface penetrates deeply into the soils and can be easily defined.
d. Spreads: horizontal failure mainly related to shear and liquefaction.

e. Flows: faster movements of the natural slope such that the mechanism resembles the movement of a viscous material. The slide surface is not easy to define and it is developed in a short period of time.

f. Composites: a combination of any of the defined types. A composite failure may exhibit least two kinds of movement at once in different locations of the displaced mass.

From Figure 1, slides are generally the better defined processes of slope failure and more related to slope stability analysis.

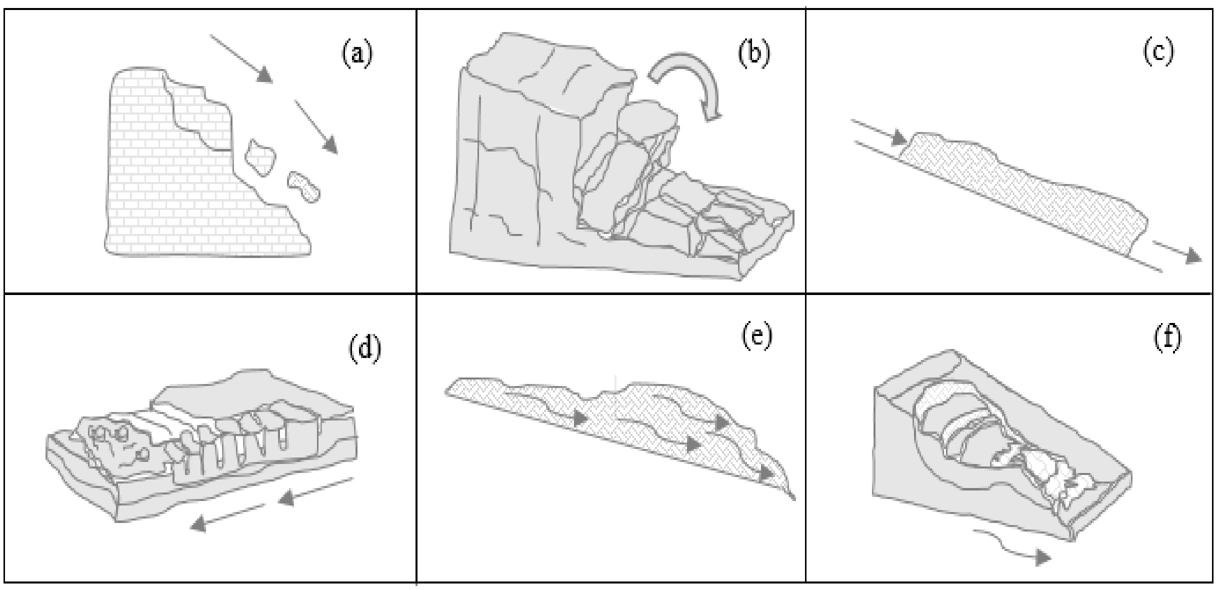


Figure 1. Slope Movements Based on Classification by Varnes (1978) and Cruden and Varnes (1996).

2.1.2 Potential Causes of Slope Failure

To remediate a problem the first step consists of understanding the factors that originated the problem. In slope stability, failure can be caused by natural or man-made factors. In addition, natural slopes have different issues than engineered and constructed slopes. Therefore, factors such as geological history, formation, stress history, climate and man-made effects will play an important role into the stability conditions of the slope.

Wright and Duncan (2005) summarized the common causes of slope failure. They stated that the general processes causing failure will lie between increasing shear stress and decreasing shear strength of the soil within the slope. Either of these mechanisms will lead to changing the equilibrium between strength soil and shear stress. The most common triggering factors for the mentioned processes are detailed as follows:

- **Increasing shear stress**: In this category, field characteristics (could be natural or constructed characteristics) and geological environment will be very important. For instance, changes in slope geometry or water level fluctuations may either increase the weight of the slope or decrease the lateral support. Moreover, water pressure development on top of the slope, increasing loads on top of the slope (vegetation type and amount may be included), and sudden shocks such as earthquakes, hurricanes, heavy traffic or any triggered sudden movement should be accounted for when analyzing the factors that can caused an increase in the shear forces that needs to be balanced by the resisting forces.

- **Decreasing the shear strength:** In this case, water level fluctuations also affect the stresses values. Rise in the ground water may decrease the effective stress by pore pressure increment. Seepage and leaching may also affect the slope equilibrium. In some types of clays, swelling, strength softening, creep and decomposition by dissolution of mineral may lead to strength loss. In general, dry clays have higher strength than wet clays. Water level rising will saturate the soil and that will be the critical condition for that kind of material. In the same way, in loose coarse grained saturated soils liquefaction may occur under cyclic loading conditions causing considerable increases in pore water pressure then decreases in effective stresses (Duncan and Wright 2005). On the other hand, continue dropping or rising in water level may cause cracking and fractures that result in strength loss on the cracking planes. Bedding planes developed naturally by previous deposition are also susceptible to water due to the planar shape that allows water to go through and reduced cohesion.

Table 1 contains the major triggering processes for slope failure divided into either natural or man-made factors as presented by Turner and Schuster (1996). It must be pointed that factors increasing stress may also decrease the strength of the soil in the slope.

Table 1. Landslide Triggering Factors after Turner and Schuster (1996).

General process	Source of the triggering	Natural factors	Anthropogenic factors
Generally increase the stress	Toe removal	Streams, river, waves and current that may erode the toe.	Excavations Drawdown of lakes or reservoirs.
	Lateral material removal	Solution in karts terrain Piping	Mining
	Addition of surcharges	Precipitation of rain or snow Flow of surface or ground water Movement of the slide Vegetation Volcanic activities	Construction loads Fill placement Waste dumps Stockpiling
	Transitory stresses	Earthquake Explosion from volcanic activity Storms	Explosions (construction related) Vibrations from pile driving or heavy traffic
	Uplift or tilting	Tectonic forces Volcanic activities Earthquake Melting of ice sheets	Cutting
Generally decrease strength	Material characteristics	Weak and sensitive soils	Disturbance due to construction may affect sensitive material
		Saturation Chemical and physical weathering Arrangement and fabric Hydration of clay that may cause loss of cohesion	Dewatering will cause water table fluctuations
	Mass characteristics	Discontinuities such as faults, fissures, fractures sheared zones	Fractures caused by construction processes

2.2 Slope Stability Analysis Methods

By understanding the potential causes of slope failure and their classification scheme, it is easy to realize that to ensure stability, it is necessary to determine whether to increase the resisting forces or decrease the driving forces. This simple concept corresponds to the basic of most methods of analysis termed “limit equilibrium”. Those basic methods are intended to determine a factor of safety involving equilibrium between the resisting and the driving forces. There are also more sophisticated methods involving numerical analysis such as finite element method and finite difference method incorporating the strength reductions technique. Each of these procedures has different basic assumptions, advantages, and disadvantages that may need to be taken into account when applied to a specific problem.

2.2.1 Limit Equilibrium

Limit plastic equilibrium is one of the simplest concepts for which several procedures have been developed to conduct stability analysis. In general, limit equilibrium can be expressed as

$$F.S = \frac{s}{\tau} \tag{1}$$

Where τ represent the shear stress in the soil, s the soil shear strength and F.S. the factor of safety to achieve a state of limit equilibrium (Huang 2014).

In limit equilibrium procedures, the shear strength is most often represented using the Mohr-Coulomb equation as

$$s = c + \sigma_n \tan \phi \tag{2}$$

Where c is cohesion, σ_n is normal stress, and ϕ is the angle of internal friction. Since c and ϕ are known parameters of the soil, the shear stress along the failure surface can be determined from Equation 2.

Although limit equilibrium based methods have been widely used due to their simplicity and easy application, the accuracy and quality of the results could be compromised due to the assumptions that need to be made to make most slope stability problems statically determined. Hence, each procedure may yield different results for the same problem while satisfying equilibrium conditions due to the different assumptions their authors have made.

The basic assumptions needed to conduct the analysis within a limit equilibrium procedure can be summarized as:

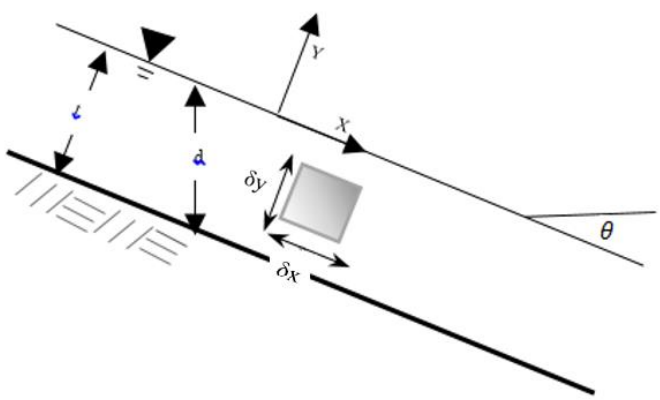
- Failure mechanism hypothesis (Assuming the shape and location of the failure surface instead of determining it).
- Soil movement is assumed as a rigid body block.
- 3-D effects are neglected.
- Uniform location of shear stresses considering they are uniformly mobilized.

Currently, there are several limit equilibrium procedures, a summary of the most well know was presented by Wright and Duncan (2005), Hopkins et al. (1975), and cited by Huang (2014). From the literature review, the presented procedures were classified as single body methods and slices methods. Moreover, from each category the most popular are presented.

2.2.1.1 Single body

Methods that consider equilibrium for a single free-body which means that do not separate the soil mass. These are relatively simple to apply and are subjected to a range of applicability. Among these methods can be found:

- **The infinite slope:** Is an exact slope stability solution in which the conditions by which a soil layer will slip along a surface may be determined. Figure 2 presents the diagram for infinite slope analysis. The general assumptions are that the slope is infinite and consequently the stresses are the same for any two planes. This method is better applicable to cohesionless slopes due to independency of the factor of safety to the depth of slide. However, the method could be used as an approximation for cohesive slopes (Lambe 1969).



$t = Thickness\ of\ the\ body$

$d = vertical\ distance$

$\theta = Slope\ angle$

δx and δy= differential elements of soil

Figure 2. Diagram for Infinite Slope Analysis (Cruikshank 2002)

- **The logarithmic spiral:** in this procedure, the shape of the slip surface is considered a spiral that has a center and an initial radius as shown in Figure 3. The radius varies with the angle of rotation about the spiral center. This method satisfied momentum equilibrium and force equilibrium. The method was presented by Wright and Duncan (2005) as the best one for the analysis of homogeneous slopes that has also been widely used in several computer programs for the design of reinforced slopes.

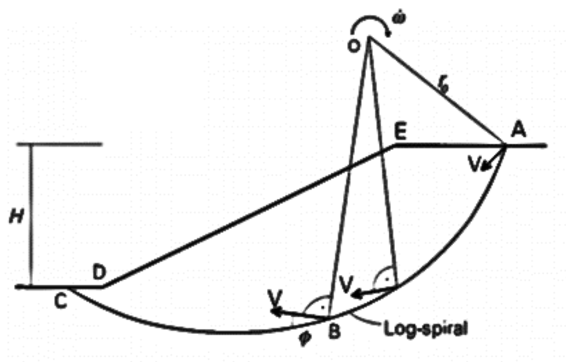


Figure 3. Diagram for Logarithmic Spiral Analysis (Michalowski 2002)

- **Swedish slip circle:** Also called the Fellenius method, this procedure assumes that the friction angle is zero and the slip surface is a circular arc (it is a singular case of the log-spiral with simpler equation). Hence the diagram for the Swedish slip circle will be very similar to Figure 3 but the friction angle will be zero. In the circular failure interface, stress and strength parameters are analyzed using circular geometry and statics. It is accurate for homogeneous and non-homogenous soils (Duncan and Wright 2005).

2.2.12 Method of Slices

As indicated by the name, these procedures divide the soil mass into a certain number of vertical slices which contain the forces and moments that will be summed to find an overall factor of safety. The failure surface bounding the slices could be circular and non-circular depending upon the assumption made to develop the given procedure. A summary table is presented below containing the main characteristics regarding equilibrium condition and slip surfaces assumptions for the most currently used methods under this category (Duncan and Wright 2005, Huang 2014, GEO-SLOPE International 2010).

Table 2 shows the different assumptions of each method and their range of applicability. In fact, methods developed based on circular slip surfaces such as Ordinary method and Simplified Bishop method are restricted to simple geotechnical problems. For instance, problems related to multilayered systems with a combination of weak and strong layers may not exhibit circular slip surface shape. Hence, soil properties and slope geometry will influence the selection of the procedure to guarantee the accuracy of the results.

On the other hand, as methods become more able to be applied to different conditions its complexity also grows. In this regard, The Bishop and Ordinary method can easily be calculated by hand while Morgenstern-Price's, considered applicable to all slopes geometries, is not feasible for hand calculation.

Many different types of software have been developed based on limit equilibrium methods such as Slide, SLOPE/W, Hydrus, SVSlope, DotSlope, Galena, GSlope, Clara-W, TSlope3, and Autoblock (for rock slopes), and specifically programmed loops that enable analysis of a given situation using different procedures.

Table 2. Assumptions Equilibrium Condition for Slices Limit Equilibrium Procedures

Method	**Equilibrium condition that satisfies**	**Interslice forces included/inclination**	**Slip surfaces shape**
Ordinary method of slices or Fellenius	Moment equilibrium (overall)	No interslice forces	Circular
Simplified Bishop	Moment equilibrium (overall)	Normal/Horizontal	Circular
Original Spencer	Moment and force equilibrium (overall)	Normal and Shear /Parallel and inclined x angle with the horizontal	Circular and Non-circular
Janbu's Simplified	Force equilibrium (each slice)	Normal/Horizontal	Circular and Non-circular
Morgenstern-Price	Moment and force equilibrium (each slice)	Parallel between them /Unknown determined through a function	Circular and Non-circular
	Force equilibrium(each slice)	Normal and Shear / Average of ground surface and Failure surface	Circular and Non-circular
Corps of Engineers (1)	Force equilibrium (each slice)	Normal and Shear / Parallel to the slope of a line from crest to toe	Circular and Non-circular
Corps of Engineers (2)		Normal and Shear / Parallel to the slope of the surface and same for all slices	
Janbu Generalized	Moment and force equilibrium (each slice)	Normal and Shear /Assumed arbitrarily	Circular and Non-circular
Sarma – vertical slices	Moment and force equilibrium (overall)	Normal and Shear (Include unknown seismic coefficient) / Function	Circular and Non-circular
Spencer	Moment and force equilibrium (each slice)	Normal and Shear interslice/Parallel and inclined x angle with the horizontal	Circular and Non-circular

2.2.2 Numerical Analyses

Although numerical approximation as a solution for complicated problems is a technique that has been around over a hundred years, in the past because of the complexity of the geotechnical materials (e.g. soil and rocks) this kind of solutions were considered very difficult to apply requiring significant computer power (Desai and Christian 1977). However, with the development of computers, the application of numerical techniques to geotechnical problems started to grow. Among the available numerical techniques, the finite element, and finite difference methods are the most common applied in the geotechnical field.

2.2.2.1 Finite Element Method

The Department of the U.S Army Corps of Engineers in their Engineering and Design Geotechnical Analysis by the Finite Element Method (Kamien 1995), defined the Finite Element Method (FEM) as "a numerical technique" that can be used in the geotechnical field to solve geotechnical problems. This technique has enhanced the capacity to perform more complex analyses such as analysis of deformations of slopes and embankments, determination of stresses and movements in excavations as well as tunneling, earth pressure structures among other complex geotechnical problems.

FEM are based on discretization of the problem which means the division of the problem's geometry into several small elements. Regarding the use of FEM in slope stability problems several authors (Carter et al. 2000, Duncan 1996, Duncan 1998, Christian 1998) have presented its advantages when compare with traditional methods as follows:

- Soil strength-stress relationship (non-linear material behavior) can be analyzed.
- Displacements can be determined and construction sequences can be modeled.

- Soil structure interaction when performing reinforced analyses can be studied.
- Changes in soil behavior can be simulated.
- No assumption of failure mechanism, interslice forces, and shape of the slip surface are needed to conduct the analysis.

Regarding the disadvantages, skills such as geotechnical engineering understanding, relative large computer storage, time investment to developed a model, and cost are crucial to achieving a successful analysis. Concerning the available software's, they are several systems designed specifically for the geotechnical field. Those are PLAXYS, SIGMA, and CRISP. Moreover, general-purpose programs such as ABAQUS, ADINA, SAP, and ANSYS can also be used for geotechnical problems and also special programs designed for particulars cases are also currently available.

2.2.2.2 Finite Difference Methods

Finite difference methods (FDM) as indicated by their name are based on finite difference formulation. In this method, the problem is analyzed by time steps, stresses and strains are then computed for each time step by either forward, backward or central differences. The FDM is the oldest numerical technique introduced in the 1930s with the solution of mathematical physics problems by means of finite difference (Thomã 2001). As in the FEM, FDM can also simulate the soil stress-strain relationship hence assumptions needed in traditional slope stability methods (LEM) are eliminated. Hence, complex problems and more realistic results are found from this kind of analysis. According to Carter et al. (2000), the FDM could be considered more user-friendly than FEM regarding the inputs for soil modeling. However, given the type of algorithms employed, the method is less efficient for linear or moderately nonlinear problems. Even though both FEM and FDM are numerical methods and both need constitutive models to represent the soil behavior under applied stresses, several differences among the methods can be mentioned. The major differences are:

- FDM does not require matrix operations for solution.
- FEM comes from mechanical and structural analysis being generalized to be applied to continuous media like soils.
- Field variables vary using specified functions in the FEM, while for FDM the field variables are defined at nodes and not in between.
- FDM use an explicit method by solving in time steps. FEM implicitly operates matrix by trial and error until the error is minimized.

Regarding the available programs, the most popular FDM based program is FLAC. The FLAC code was developed by ITASCA (1996) and consists of an explicit finite difference code that simulates soil or rock structures which experience plastic flow when their yield limit is reached (Kourdey et al. 2001).

2.2.2.3 Strength Reduction Method

The Shear Strength Reduction (SSR) method is a procedure used in FEM/FDM in which the factor of safety is obtained by weakening the soil in steps in an elastic-plastic finite element or finite difference analysis until the slope “fails” (Dawson et al. 2015, Griffins and Lane 1999, Pradel et al. 2010, Fu and Liao 2010). According to the literature, the method was first proposed by Zienkiewicz et al. (1975). In this method, the factor of safety for a slope is considered to be the ratio of the actual shear strength to the lowest shear strength of a rock or soil material that is required to maintain the slope in equilibrium. Thus, the factor of safety is considered to be the factor by which the soil strength needs to be reduced to reach failure. Numerically, the failure occurs when it is no longer possible to obtain a converged solution.

For Mohr-Coulomb material the shear strength reduced by a factor (of safety) F is determined as

$$\frac{\tau}{F} = \frac{c'}{F} + \frac{tan\varphi'}{F} \tag{3}$$

Where;

$$c^* = \frac{c'}{F} \; and \; \varphi^* = arctan\left(\frac{tan\varphi'}{F}\right) \tag{4}$$

are the reduced Mohr-Coulomb shear strength parameters.

Regarding the advantages of the application of the SSR, Rocnews (2004) has presented a summary of the major advantages as:

- It eliminates the need for a priori assumptions on failure mechanisms as in LEM (the type, shape, and location of failure surfaces). This also minimizes the expertise required in finding critical failure mechanisms.
- Elimination of arbitrary assumptions regarding the inclinations and locations of inter-slice forces.
- Monitoring of the development of failure zones from localized areas until the total slope failure is automatically achieved.
- Expected deformations at the stress levels found in slopes can be predicted.

Despite, all the mentioned advantages SSR, its adoption has been limited primarily because of inadequate experience that engineers have with the tool for slope stability analysis, and the limited published information on the quality/accuracy of its results (Hammah et al. 2005).

2.2.2.4 Modeling

Numerical modeling originates from applying numerical analyses to represent a real physical problem. A model should characterize as close as possible the real conditions that are intended to be analyzed. However, because of the inherent difficulties that embody a numerical analysis, simplifications and engineering judgment are very important factors to achieve accurate results. The objective for modeling must be clearly defined before the model creation. As expressed by Marr (1999) “Know the answer before you start” should be the guiding rule when modeling (FEM). This is because errors in models involving quite complex mechanisms are difficult to detect, so an estimation of the answer may help to evaluate comparatively the FEM or FDM results. Thus, since the process has been defined as complex some of the reasons that justify undertaking a thorough modeling effort are: make quantitative predictions, compare alternatives, identify governing parameters, understand processes, and train our thinking (Geo-Slope 2013). For instance, problems related to complex geometries and material variations will be better analyzed by modeling than by using traditional methods (such as limit equilibrium method).

Numerical modeling has been pointed out as a skill that helps to overcome many limitations related to oversimplified assumptions for soil properties and boundary conditions in the geotechnical field. Problems such as non-linear soil properties, soil-structure interaction, and seepage among other can be better analyzed. However, the necessity for understanding geology, site conditions, and the phenomena that control the construction processes to account for a realistic point of view is still needed to perform an adequate analysis. Hence, numerical modeling application does not substitute the basic requirements of a good geotechnical engineer (Duncan 1998). Therefore, using software should be seen as part of the modeling processes tied to conceptualization and interpretation of the results. It is then elemental to determine whether or not modeling will be needed for a problem, the means by which the model will be constructed, the available information to create the model, and what will be the model assessment.

If the conditions while modeling are not well-understood errors may take place. Some of the reasons for these errors could be mathematical model errors, conceptual errors, input data errors, numerical errors and interpretation errors. To prevent the occurrence of errors several factors should be taken into account when constructing a numerical model, these factors will be detailed in the following section.

Important Factors for Modeling:

A model may represent the real condition of the problem that is being analyzed in the most accurate way possible. Hence there are a series of factors that may help to achieve a successful model. Among these factors are the key aspects of the construction of the model, what kind of components should be modeled, the way the model will represent certain field conditions, the way the geomaterials will be represented by the model, and the way results will be evaluated and compared with the reality.

Model Building

Before starting to build a model, a meticulous characterization of the site or problem conditions must be performed. Understanding the mechanisms and factors involved in the problem that will be analyzed as well as estimation of the result will lead to successful application of the desired numerical analysis. For instance, water conditions, soil boundary conditions pore water pressures, and geometry are some of the factors that should be clearly visualized before starting modeling.

Regarding the geometry, generally, the common belief is that adding every single detail to a model will make it more accurate. However, that increases the complexity and possibly the inaccuracy of the results. Therefore, the important components or most significant aspects that may influence the results for which the model is being built should be selected (Geo-Slope 2013). For instance, very small layers of soils embedded into a large stratum with similar properties could be simplified as one stratum. Moreover, physical elements that do not affect the results can be simplified or eliminated from the model, for instance, a rounded structural element could be linearized.

Constitutive Models

Another factor playing a key role in model building is the characterization of the soil properties involved in the studied case. Constitutive models have been developed with the objective of representing the relationship between stress and strength, and perhaps time, of a given material for numerical analysis (Lade 2005). The quality and accuracy of any given numerical analysis such as finite element method or finite difference method applied to a geotechnical problem will greatly depend on the representation of the constitutive relationship used. Therefore, it can be said that constitutive model, its selection, and accuracy have a major responsibility in geotechnical finite element/ finite difference analysis. As presented by several authors (Desai and Asce 1985, Duncan 1996, Carter et al. 2000, Lade 2005), there are a large variety of constitutive models. However, not all types of model can be correctly applied to all types of soils and, moreover, each method has been developed over a range of stresses and strains intended to represent the real condition to which the material will be exposed.

The mathematical representation of soil properties is not an easy task because of soils' complex nature and behavior under loading conditions. In this regard, several theories of material behavior perfectly fit with other construction materials such as steel or concrete. Those two mentioned materials can easily be represented as elastic or elastic-perfectly plastic. In the soils' case, stress-strain behavior is not linear, volume change takes place during shearing and different materials will behave differently whether the range of stresses is close or far from failure and the material is loose or dense. Hence, to develop realistic models that can duplicate the important aspects of the soil stress-strain behavior subjected to different loading conditions, it is required to perform advanced experiments to study soil behavior versus loading within applied mathematical frameworks of elasticity and plasticity theory.

Duncan (1996) presented a summary of the currently available stress-strain relationships that can be selected for a numerical analysis. He highlights the association between accuracy and complexity that may be analyzed when selecting a given model. In this regard, the alternatives include, linear elastic, multilinear elastic, hyperbolic (elastic), elastoplastic, and elasto-viscoplastic behavior.

- Linear Elastic: This theory is based on Hooke's law. Immediate deformations and settlement are calculated from an elastic modulus estimated from a stress-strain curve (Lade 2005). The major advantage of this kind of model is the simplicity and the small number of parameters needed (Poisson's ratio and Young's modulus). However, in elasticity theory when an applied stress is removed, the material returns to its undeformed state. Hence, soils are fundamentally frictional materials so volume change will take place during drained shearing. Hence, this theory is not a good model in representing the actual soil behavior.

- Multilinear Elastic: In this model more than one straight line is used to model stress-strain curves, and modulus reductions. This is done with the objective of improving the accuracy of the values from the lab test (Duncan 1996).
- Hyperbolic Elastic: This method relates stress increments to strain increments (as in Hooke's law). However, in this method stresses and moduli are systematically related.
- Elastoplastic: In this kind of model, yield function and surface, plastic potential, elastic properties and surface, and hardening must be characterized. Since in this model the stress-strain relationship is more complex than in elastic models, they are expected to capture more closely the real behavior of soils under high stress levels (closest to failure). Duncan (1996) pointed that if the conditions are not close to failure no major advantage will be found in applying this model.
- Elastic-perfectly plastic: The Mohr-Coulomb model is the simplest well know elastic-perfectly plastic model. In this model, the soil stress-strain behaves linearly in the elastic range (based on Hooke's law). Hence, in that range the Poisson ratio and Young's modulus are the representing parameters. On the other hand, in the perfectly plastic zone the behavior is based on the Mohr-Coulomb failure criterion. The friction angle, ϕ, the cohesion, c, and the dilatancy angle, Ψ, are the parameters that define the failure criteria (Ti et al. 2009).

Hence, when selecting a constitutive model a key factor to be considered is the type of material that will be represented. For instance, the density of the material will be important to determine its behavior under loading. Some models represent the behavior of soft clays better than sands (frictional materials must be represented with a model developed over the non-associated flow rule (Lade 2005)). This is because in some cases, if soils are not stressed close to failure, they could be represented as linear elastic materials. Thus, models developed under elastic general assumption can be used.

Another vital factor is to understand the basis for the model's development and its shortcomings, how the relationship stress-strain was considered. For example, hyperbolic models relate stress and strain by a systematic method and the parameters values can be determined by laboratory test while linear elastic models use just two elastic parameters and they are not a good representation of the actual soil behavior. However, as complexity is also a factor to be considered some simplifications can be done regarding the type of project and the purpose of the analyses. Construction sequence is also an important detail in selecting the constitutive model (Duncan 1996). In this regard, slow a construction assumption will indicate little development of excess pore pressure, which may affect long-term stability; these effects will be different from one model to another (USACE 1995).

The number of parameters involved in the model should be also considered. Generally, the larger the number of parameters, the more flexible and versatile the model is considered. In addition, parameters can be easily determined when compared with experimental results. However, the calibration process of the model will influence the complexity and cost if the model demands non-standard and complex experiments for calibration, which will increase the cost (Lade 2005).

Finally, numerical analyses have enabled the geotechnical field to increase the ability to perform more accurate analysis. However, the effort to increase the ability to understand and represent site conditions in modeling may be excessive. This is significantly true for the constitutive models' case where complex ground conditions, inadequate site characterization, and inadequate data for appropriate constitutive models still remain as reasons for potential limitations on the applicability of FEM to geotechnical field. As expressed by Whittle (1999) the profession needs to invest more resources on validation of the current capacities and this includes current constitute models before the development of new ones.

Model Calibration and Validation

Generally speaking, calibration is defined as correlation or standardization of a modeled reading with those from known results. Hence, when talking about model calibration, the simulated data must be matched with observed values by varying the inputs to account for unknown or uncertain conditions. The main objective of calibration is to guarantee that a certain model will represent the studied conditions as closely as possible. In the same way, validation is needed to define reliable models when numerical analysis has been conducted. Reference points regarding domain of the analysis, type of model to be applied, type of tests that will be conducted to validate the model, and replicability of the results are very important to determine if the strength properties of the in situ ground are accurately represented in a model (Carter et al. 2000).

2.3 Reinforced Slopes

After the factor of safety for an unreinforced slope has been determined, several approaches for remediation can be applied if the calculated value does not meet minimum safety requirements. These remediation techniques can generally be categorized as follows; (1) avoid the problem, (2) decrease the shear stress or driving forces, or (3) increase the shear strength or resisting forces (Turner and Schuster 1996). Regarding the degree of stabilization required and the actual condition of the slope, other approaches such as maintenance, observation, and do nothing could also be followed. Maintenance and observation will be generally focused on the monitoring and temporary treatment of slow moving landslide. On the other hand, the do nothing could be plausible when a slow constant trend resulting for years of monitoring is seen. However, the understanding of the later consequences must be ensured (Cornforth 2005). A summary of approaches to mitigate slope stability problems is presented in Table 3.

Slopes containing structural elements that increase the ability of the slope to withstand movements are called reinforced slopes. Methods by which a slope can be reinforced are presented in Table 3 and will generally follow under the increasing the resisting forces approach. The resisting forces can be increase by either applying external forces or increasing the internal strength of the soil in the failure zone (Turner and Schuster 1996). The methods in the first category will focus on providing enough external restraint to prevent the movement. On the other hand, to increase internal strength, reinforced soil and in situ reinforcement has been used. The selection of a particular remediation method will vary in terms of available physical space, geometry, surrounding structures cost, desirable remediation degree and material characteristics.

Table 3. Remediation Methods for Slope Stability Problems, Turner and Schuster (1996)

Category	Procedures	Limitations
Do nothing	"No action"	Non effective if long term catastrophic failure Applicable to slow moving or stable accelerating slopes
Avoid the problem	Relocate project	Should be studied during planning Cost impact regarding location selected No cost effective if design is complete
Maintenance	Activities such as removing materials, closing areas, etc.	Limited to slow moving landslides Will be temporary then it will need to be re-done
Monitoring	Inclinometers, surface survey points, etc.	Limited to slow moving landslides
Decrease driving forces	Change the grade	Could affect sections of adjacent roadways Not always available physical space for change in geometry
	Drain surface	Will only correct surface infiltration or seepage due to the surface infiltration
	Drain subsurface	Will depends on permeability of the sliding mass
	Reduce weight	Requires right-a-way and lightweight materials Produce excavation waste that need to be handle
Increase resisting forces	Use buttress or counterweight fills; toe berms	Could not be effective in deep-seated landslides Requires right-a-way Could require a firm foundation
	Use structural system	Could not handle large deformations Should penetrate well below sliding surface
	Install anchors	Requires a firm foundation to resist the shear forces of by the anchors tension
	Drain subsurface	Will depend on permeability of the sliding mass
	Use reinforced backfill	Requires durability of reinforcement
	Install in situ reinforcement	Requires long-term durability of nails, anchors, piles, micropiles
	Use biochemical stabilization	Limited to the height of the slope Affected by climatic conditions
	Treat chemically	Long-term effectiveness is still in evaluation Could be affected and affect the environment
	Use electrosmosis	Requires maintenance and constant direct current Could be expensive
	Use thermal stabilization	Require expensive and carefully designed system Could be expensive

2.4 Pile Reinforced Slopes

Among the mentioned methods in section 2.3, this thesis will focus on the evaluation of stability of reinforced slopes by means of pile elements. Piles have been found to be an effective way to stabilized slopes (De Beer and Wallays (1970), Ito and Matsui (1975), Cai and Ugai (2000), Hassiotis et al. (1997), Lee et al. (1995), Poulos (1995), Chen and Poulos (1997)). However, evaluating the stability of pile reinforced slopes represents a major challenge in the field, not only because of the several uncertainties that take place in the phenomenon, but also because the mechanism soil-stabilizing pile is very complex. Moreover, no widely acknowledged method for analyzing and designing the stabilizing piles has yet to be presented (Chow 1996).

To evaluate the stability of pile-reinforced slopes in the same way as for unreinforced slopes, the basis of the analysis is to determine the factor of safety. The factor of safety has been defined previously as the ratio between resisting forces and the driving forces along the failure surface. However, to determine the critical failure surface in a reinforced analysis, two analyses should be conducted for the same slope (1) determine the factor of safety of the unreinforced slope, and (2) determine the factor of safety for the reinforced slope, because the failure surface is not constant and tends to vary while accounting for the pile effects (Ausilio et al. 2001, Ito and Matsui 1975, Lee et al. 1995). This means that other critical surfaces could be generated as a consequence of the pile placement. Therefore, an iterative process should be conducted.

Moreover, when analyzing pile-reinforced slopes, study of the piles must be also performed to verify the resistance of the pile to the loads developed by the soil movement (Hassiotis et al. 1997). Hence, the overall analysis can be done as coupled or uncoupled. The first one refers to the kind of analysis in which pile and slope are analyzed together and the second one refers to the analysis of stabilizing pile and slope stability separately. Regarding these types of analysis, numerical methods such as finite element and finite difference method allow for the characterization of both pile and slope stability simultaneously. On the other hand, procedures based on limit equilibrium analyses do not include the stabilizing pile responses on the analysis. As a result, additional analysis of the stabilizing pile must be conducted.

2.4.1 Analysis of Reinforcing Piles

Piles used to stabilize slopes will be subject to lateral forces developed from the movement of the surrounding soil. In general, piles subjected to lateral forces can be divided into “passive piles” and “active piles” (De Beer and Wallays 1970, Ito et al. 1979). According to Ito et al. (1979), the passive piles case is more complex than the active one due to the interaction between the piles and the soil that generates the lateral forces.

The general design approach for stabilizing piles follows a procedure presented by (Viggiani 1981) in which three main steps are followed; (1) evaluate the needed shear force to increase the safety of factor of the slope; (2) evaluate the maximum shear force provided by each pile; (3) select the number of pile and the optimum location. This will be discussed in more detail in section 2.4.2. The first step is conducted by performing a stability analysis with a target factor of safety. From the difference between the actual factor of safety (unreinforced) and the target value, the stabilizing force required can then be found.

The second step is addressed by performing a lateral response analysis. In this regard, several empirical and numerical methods have been presented. These methods can be categorized as (1) pressure-based methods (De Beer and Wallays 1970, Ito and Matsui 1975, Hassiotis et al. 1997); (2) displacement-based methods (Poulos 1995, Chen and Poulos 1997), and (3) continuum methods (Cai and Ugai 2000, Kanungo et al. 2013). The general basis of the methods for analyzing pile responses will be presented:

2.4.1.1 Pressure-based Method

The piles subjected to lateral pressures are analyzed as passive piles based on the method developed by Ito and Matsui (1975). This method was developed assuming that the soil surrounding the pile undergoes two plastic states: (1) plastic deformation and (2) plastic flow. The first stage satisfies the Mohr-Coulomb yield criterion and in the second one, the ground is considered as a visco-plastic solid. Hence, plastic deformation can be compared to hard soil layers and plastic flow could be compared to creep deformation of a soft layer. The general assumptions of the plastic deformation theory by Ito and Matsui (1975) are:

1. AEB and A'E'B' are two sliding surfaces that occur making an angle of $\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)$ with the x-axis, when the soil layer deforms as presented in Figure 4.
2. In the area AEBB'E'A' the soil is in a state of plastic equilibrium. Thus, the Mohr-Coulomb yield criterion applies.
3. The active earth pressure acts on lane AA’.
4. The soil layer is in plane-strain respect to depth direction.
5. The piles are considered rigid
6. The frictional forces on surfaces AEB and A'E' B' are neglected, when the stress distribution in the soil AEBB'E'A' is considered.

Then, assuming the Mohr-Coulomb yield criteria rules the plastic deformation of the EBB'E' and AEE'A" areas, the pressure acting on the piles per unit thickness (q) will be determined as:

$$q = A_c\left(\frac{1}{N_\varphi \tan\varphi}\left\{e^{\left[\frac{D_1-D_2}{D_2}N_\varphi \tan\varphi \tan\left(\frac{\pi}{8}+\frac{\varphi}{4}\right)\right]-2\sqrt{N_\varphi}\tan\varphi-1}\right\}+\frac{2\tan\varphi+2\sqrt{N_\varphi}+\frac{1}{\sqrt{N_\varphi}}}{\sqrt{N_\varphi}\tan\varphi+N_\varphi-1}\right)$$

$$-c\left(D_1\frac{2\tan\varphi+2\sqrt{N_\varphi}+\frac{1}{\sqrt{N_\varphi}}}{\sqrt{N_\varphi}\tan\varphi+N_\varphi-1}-2D_2\frac{1}{\sqrt{N_\varphi}}\right)$$

$$+\frac{\gamma\hat{z}}{N_\varphi}\left\{Ae^{\left[\frac{D_1-D_2}{D_2}N_\varphi \tan\varphi \tan\left(\frac{\pi}{8}+\frac{\varphi}{4}\right)\right]-D_2}\right\} \tag{5}$$

Where c is the cohesion intercept, D_1 is the center to center distance between piles, D_2 opening between piles, φ is the angle of internal friction of the soil, γ the unit weight of the soil, ẑ the depth from the ground surface and N_φ and A are:

$$N_\varphi = tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right) \tag{6}$$

$$A = D_1\left(\frac{D_1}{D_2}\right)^{\sqrt{\left[N_\varphi \tan\varphi+N_\varphi+1\right]}} \tag{7}$$

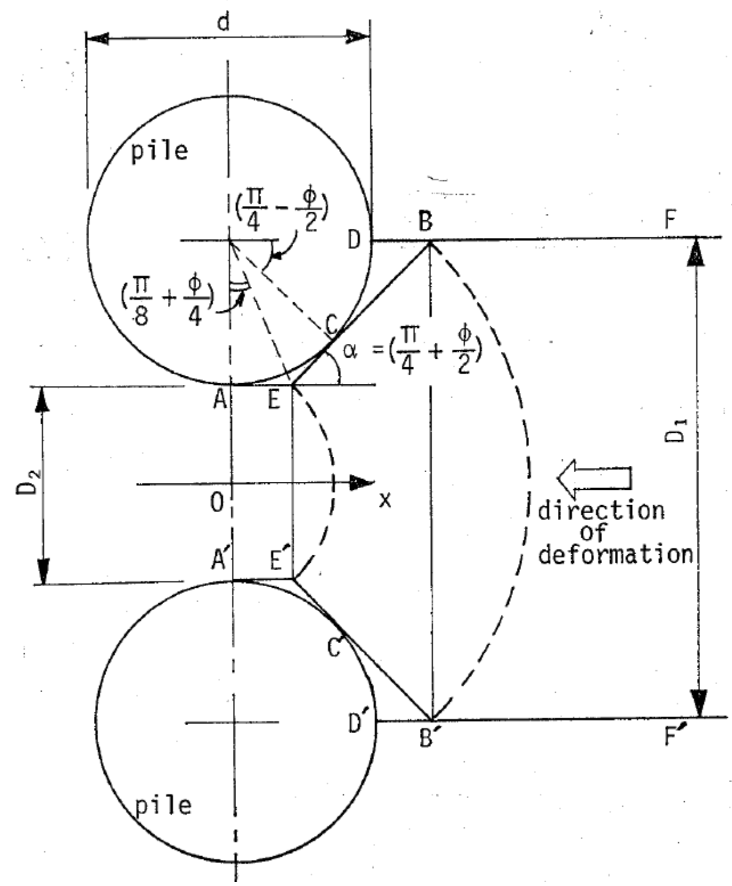


Figure 4. State of Plastic Deformation in the Ground just Around the Piles (after Ito and Matsui 1975)

Furthermore, the general assumptions of the plastic flow theory Ito and Matsui (1975) are:

1. Figure 5 shows the soil between two piles ACDFF'D'C'A' flowing at velocity v_1. It is assumed that a visco-plastic flow will occur just around the piles in the AEBB'E'A' and that the flow direction is always centripetal towards the EBB'E' zone.
2. The soil layer is considered in steady state behaving as a Bingham fluid with yield stress τ_y and plastic viscosity n_p. In addition, the flow is considered in the depth direction.

3. The forces applied on the portions GH and G'H' by small soil elements can be obtained by summing the force due to earth pressure and the viscous flow of the element through a channel with width GG'.
4. Piles were assumed to be rigid.

Hence assuming the soil flows uniformly at averages velocities $v_1, v_r\ and\ v_2$ through BB', GG' and EE' and substituting and integrating the shearing force acting on the small portions GH and G'H' and the velocity v_r the force acting on the surface is obtained. The lateral force acting on the pile per unit thickness (p) is finally determined as:

$$p = \sqrt{2m\tau_y}\left(\sqrt{1+\frac{m}{2\tau_y D_2^2}} - \sqrt{1+\frac{m}{2\tau_y D_1^2}} + \log\frac{D_1\left(1+\sqrt{1+\frac{m}{2\tau_y D_1^2}}\right)}{D_2\left(1+\sqrt{1+\frac{m}{2\tau_y D_2^2}}\right)}\right) \tag{8}$$
$$+ (D_1 - D_2)\left(\frac{[\sqrt{2}-1]\pi m}{8D_2^2} + (\sqrt{2}-1)\sqrt{\left(\frac{\pi^2 m}{8D_2^2}\right) + \frac{\pi^2 m\tau_y}{4D_2^2}} + \frac{m}{D_1 D_2} + \sqrt{2}\tau_y\right.$$
$$\left. - 2c + \gamma z\right)$$

Where c is the cohesion intercept, D_1 is the center to center distance between piles, D_2 opening between piles, τ_y is yield stress, n_p is plastic viscosity, γ the unit weight of the soil, ẑ the depth from the ground surface and m is :

$$m = 16 n_p v_1 \frac{D_1}{\pi^2}. \tag{9}$$

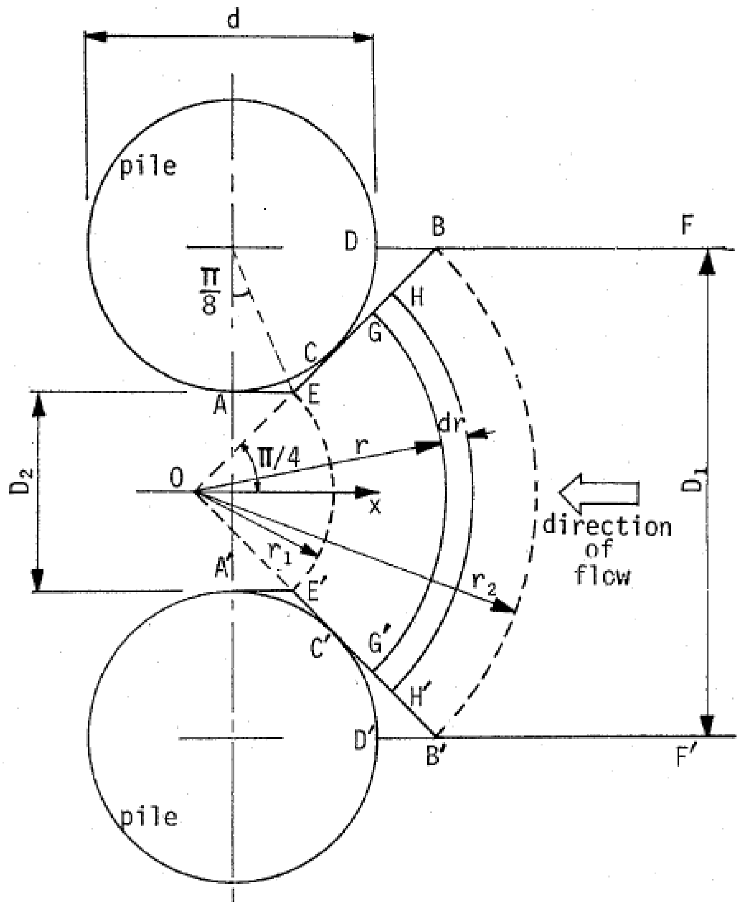


Figure 5. State of Plastic Flow in the Ground just Around the Piles (after Ito and Matsui 1975)

Because the presented method considered a fixed critical surface when in fact the critical surface will vary by the piles addition, (Hassiotis et al. 1997, Ausilio et al. 2001, Ito and Matsui 1975, Lee et al. 1995), the piles were considered to be rigid, and the soil was assumed to be able to plastically deform around the piles, the method has limited applicability by not representing the actual soil/pile interaction (Jeong et al. 2003). Ito and Matsui (1975) concluded that, in the theory of plastic deformation, the lateral force will increase as c and Ø increases. On the other hand, in the theory of plastic flow, the lateral force will increase as increasing yield stress and plastic viscosity but the change is not significant for yield stress changes. For both theories, the lateral forces agree in order of magnitude with the observed values. The lateral force should be estimated by integrating equation (5), from the theory of plastic deformation along the depth of the soil layers, when piles are top restrained

2.4.1.2 Displacement-based Method

This method is based on determining the lateral pile response to the lateral ground movements. Hence, the lateral soil movement needs to be determined for its application. The method could use either measured inclinometer data or analytical results by finite element approach to simulate the soil/pile interaction mechanism. Poulos (1973) presented a useful approach to determine laterally loaded pile responses using the soil/pile interaction analysis in which the movement of the soil through the pile is considered.

The presented approach uses a simplified boundary element where the pile is modelled as elastic beam and the soil as an elastic continuum. The lateral displacements for each pile element are associated with its bending stiffness and the horizontal pile/soil interaction stresses. A limiting lateral pile/soil stress can be specified allowing local failure of the soil to obtain a nonlinear response. This analysis must be solved by a computer program since a set of equations should be solved for incremental analysis and incremental pressures.

According to Poulos (1995) the lateral response analysis requires knowledge of lateral soil modulus distribution, free-field horizontal soil movements and distribution with depth of the limiting soil/pile pressure. The basic problem of a pile in an instable slope is presented in Figure 6. It has been assumed that the upper soil moves as a rigid body downslope direction, a small zone below is called drag zone and the stable zone is stationary. The key soil parameters involved in the approach are the Young’s modulus and the limiting lateral pressure. These parameters are mainly assessed by correlation, therefore, their values may carry some inaccuracy.

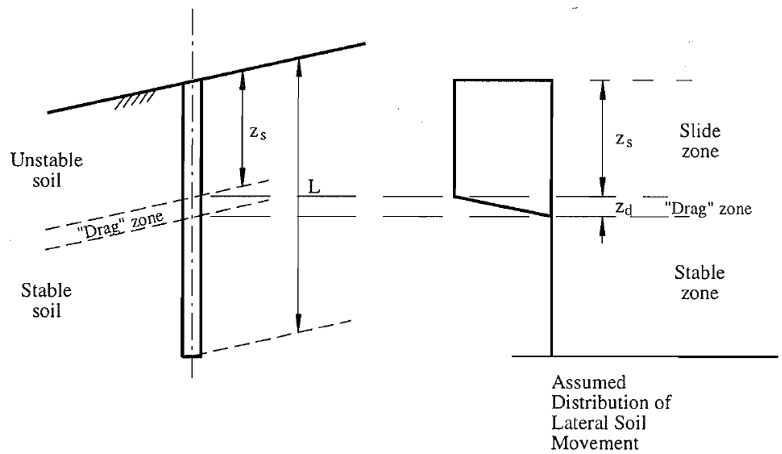


Figure 6. Basic Problem of a Pile in Unstable Slope after Poulos (1995)

The theoretical analysis from Poulos (1995) revealed the existence of four pile modes of failures; (1) flow mode; (2) short-pile mode; (3) intermediate mode; (4) long-pile failure. The first three modes correspond to modes of failure of the soil while the last one refers to the failure of the pile itself. The soil modes of failures are illustrated in Figures 7 to 9.

1. Flow mode: The critical sliding surface is shallow and the unstable soil flows around the pile.
2. Short-pile mode: The critical sliding surface is deep and the length of the pile in the unstable soil is shallow and full mobilization of the soil strength on the stable layer happens.
3. Intermediate mode: The stable and unstable soil strength is mobilized along the pile length.
4. Long-pile failure: The maximum bending moment reaches the yield moment of the pile section and consequently the pile yields.

Poulos (1995) determined that the resistance provided by the piles to stabilize the slope will be greater if the governing mode of failure is the intermediate one. Additionally, proper reinforcement of the pile is needed to avoid the long-pile failure.

Most of the available methods to analyze laterally loaded piles are based on the p-y method. This method uses finite element difference for solving the nonlinear four order differential equation of a bean-column system on an elastic foundation. Equation 10 was originally presented by Hetenyi (1946) and posteriorly modified by Poulos (1973) and Byrne et al. (1984) to analize the pasive case that is when pile are subjected to lateral movements (Suleiman et al. 2007).

$$EI\frac{d^4y}{dx^4} + Q\frac{d^2y}{dx^2} - k(y - y_s) + w = 0 \tag{10}$$

Where EI is pile flexural stiffnes, Q is axial load applied on the pile, k is soil stiffness at a depth z, ys is free-field soil movement, and w is the externally-applied distributed load. The pile-soil interaction mechanism and analytical model based on displacement-based method is illustrated in Figure 10.

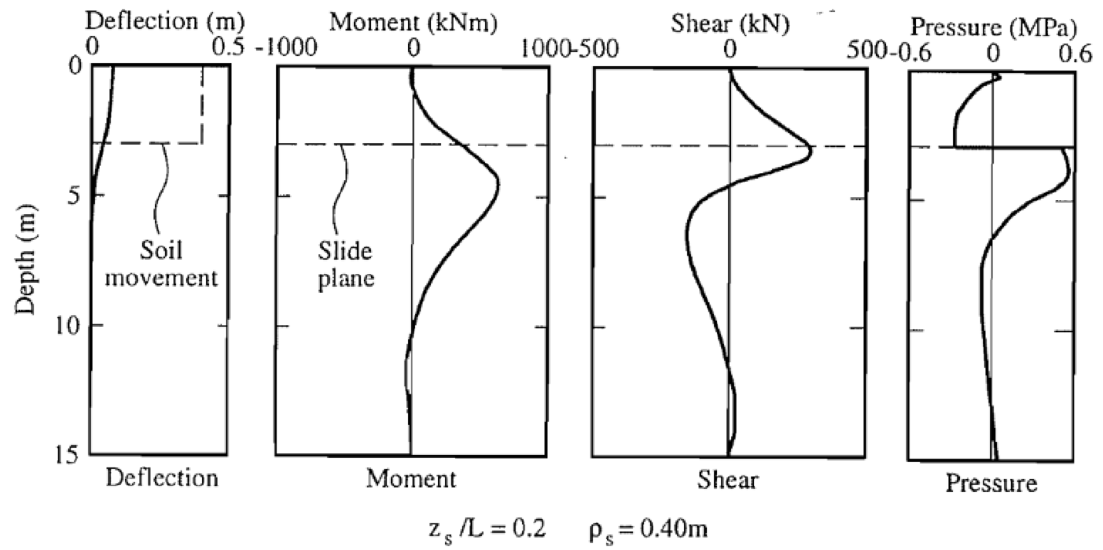


Figure 7. Flow Mode after Poulos (1995)

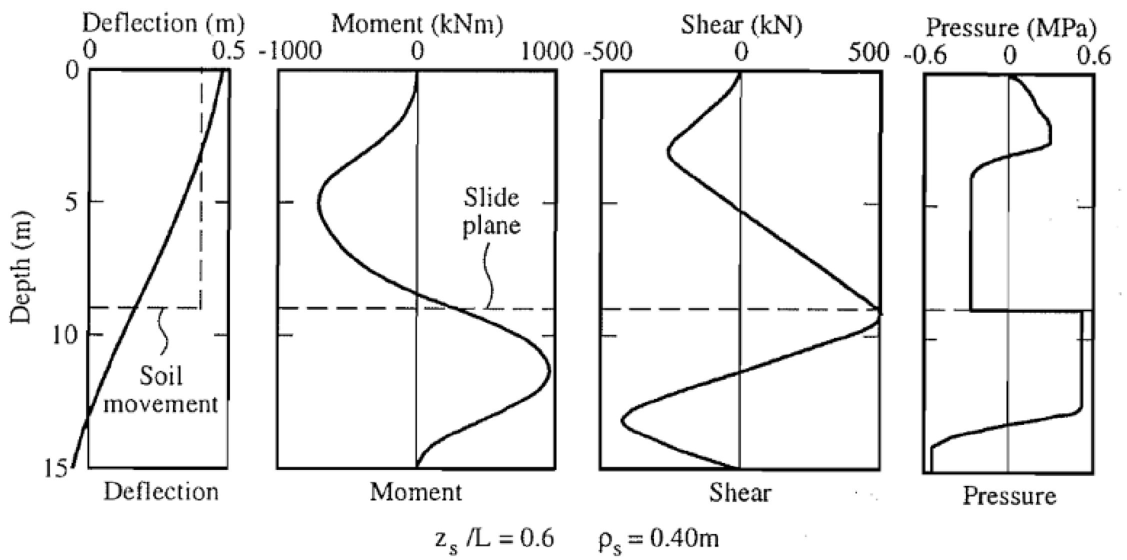


Figure 8. Intermediate Mode Poulos (1995)

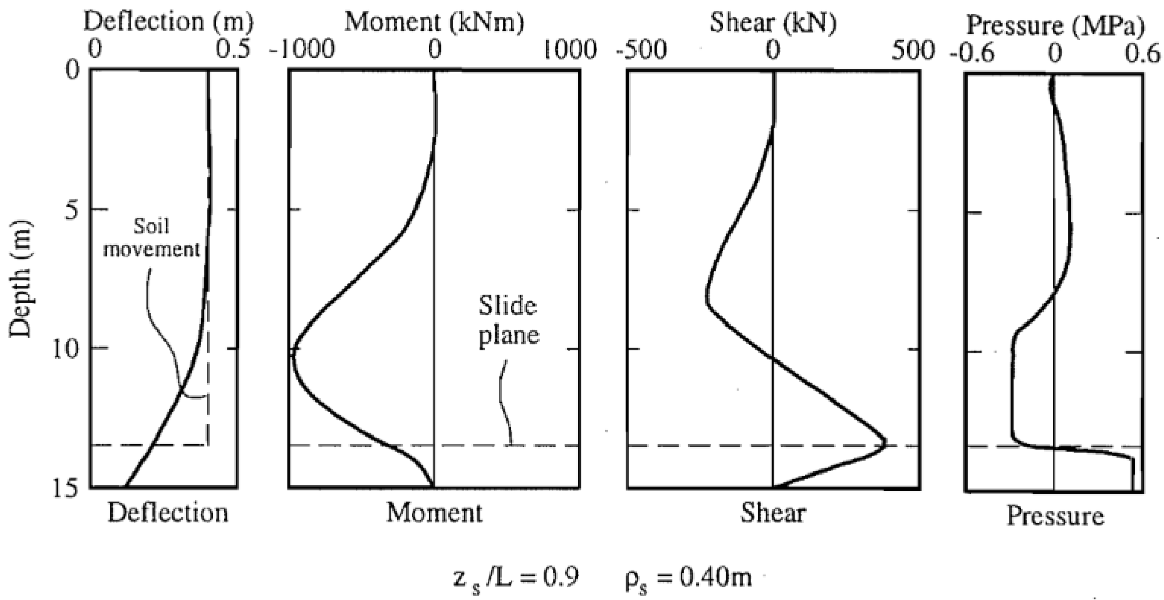


Figure 9. Short Pile Mode Poulos (1995)

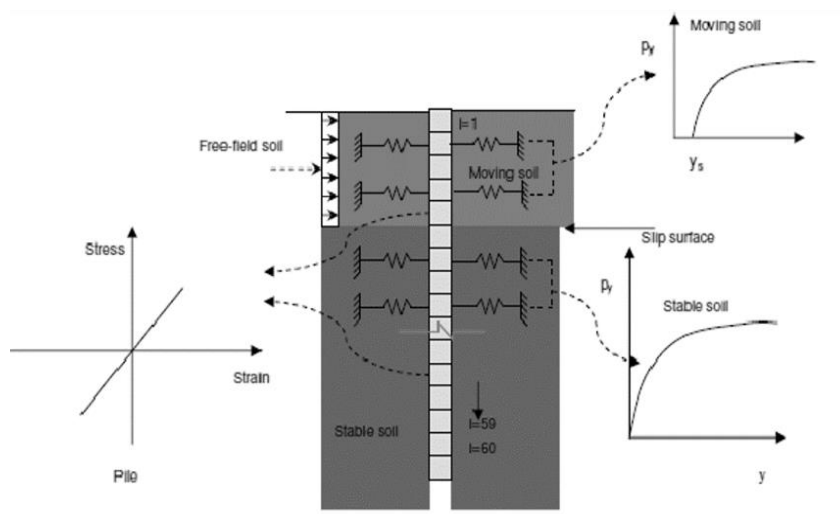


Figure 10. The 2-D Analytical Model Used to Study the Behavior of Pile in Stabilizing Slopes (from Suleiman et al. 2007).

The soil resistance is simulated as nonlinear springs, where p represents the soil pressure and y is the pile deflection. The first use of the p-y method is difficult to determine, however, some of the earliest publications correspond to Matlock (1970), Reese et al. (1974) Reese et al. (1975), and Reese and Welch (1975).

Originally, the analysis conducted by Poulos (1995) was performed by using a FORTRAN 77 computer program call ERCAP (Earth retaining capacity of piles). Currently, several programs are available to perform a laterally loaded pile analysis such as COM624P, COM624G, RSPile, and LPILE. Finite element program for general purposes can also be used as the procedure presented by Jeong et al. (2003) using the commercial finite-element package, ABAQUS, or Chen and Poulos (1997). Moreover, a spreadsheet is also available CLM2.

- COM624P and COM624G were developed by Reese et al. (1984). The COM624P succeeded the COM624G. The programs are based on the p-y curve performing iterations that represent the soil nonlinear response. From the analysis pile displacement, rotation, bending moment and shear are determined.
- RSPile is a program from Rocscience to calculate the axial load capacity of driven piles and the analysis of piles under lateral loading. The program can be used in conjunction with Slide, a slope stability program from the same group, to perform slope stability analysis. This program is also based on the p-y method for laterally loaded piles.
- The available spreadsheet CLM2, the characteristic load method (CLM) of analysis of laterally loaded piles was developed by Duncan et al. (1994) . The method is a simpler approximation of the p-y analysis results from a modification of the Evans and Duncan (1982) procedure, where dimensionless load-deflection and load-moment curves were developed for piles in cohesive and cohesionless soil.
- LPILE compute deflections, bending moments, and shear forces developed along the length of a pile under loading by a finite difference technique. In these analyses, the pile

is assumed as a beam-column and the soil is modeled with nonlinear Winkler-spring mechanisms. Hence, p-y curves are generated by the program, this mechanism can accurately predict the nonlinear response of the soil (Isenhower et al. 2016).

In this thesis, LPILE v2016 will be used to performed the pile response analysis for the uncouple slope stability method.

2.4.1.3 Continuum Analysis

In this type of analysis the pile response and slope stability are considered simultaneously (coupled analysis). Several models analyzing the stability of pile reinforced slopes by the coupled analysis have been presented. In this regard, Cai and Ugai (2000) presented a method of analysis using a three-dimensional elasto-plastic shear strength reduction finite element method by which the soil/pile interaction can be analyzed. Chen and Martin (2002) adopted a plain-strain model using the finite difference program FLAC to evaluate the pile/soil interaction and arching phenomenon. Pradel et al. (2010) presented a numerical approach for the design of drilled shafts to enhance the factor of safety of slopes by using the strength reduction technique and the program FLAC. Table 4 presents a summary of analysis methods for pile reinforced slopes regarding the type of lateral response analysis.

Table 4. Pile Reinforced Slopes Analisys Methods

Method/Paper	Soil type	Failure Type	Model characterization
Ito et al. (1979)	Cohesive Soil	Circular	Pressure based method/ Uncoupled
Poulos (1995)	Clay, Claystone and Silt stone	Circular	Displacement based/ Uncoupled
Hassiotis et al. (1997)	Cohesive Soil	Circular	Finite Difference Method/Pressure based/ Uncoupled
Lee et al. (1995)	Purely Cohesive slope	Circular	Boundary Element/ Displacement based/ Uncoupled
	Upper soft Lower stiff	Circular	
	Upper stiff Lower soft	Circular	
Cai and Ugai (2000)	C=10kpa φ=20◦	Any	3D Shear Reduction Finite Element Method/Coupled analysis
Ausilio et al. (2001)	C=4.7kpa φ=25◦	Log-spiral	Kinematic approach limit analysis/ Uncoupled
Jeong et al. (2003)	Anisotropic and non-homogeneous	Log-spiral	Displacement based and Strength Reduction Finite Element Method/ Uncoupled and Coupled
Nian et al. (2008)	γ=20.0kn/m3 C=10kpa φ=20◦	Log-spiral	ABAQUS Finite Element Method/ Uncoupled
Chen and Martin (2002)	Granular and Fined-grain soils	Any	Finite difference program FLAC /Coupled
Pradel et al. (2010)	Cohesive Soil	Any	Strength Reduction Method/Coupled

2.4.2 Primary Factors on the Analysis

From the previously presented methods, several authors concluded that among all the factors taking place on the analysis of pile reinforced slopes, there are certain parameters that will have the greater influence on the slope performance (pile and slope). Those critical parameters can be summarized as the pile location, diameter and spacing, the number of rows, pile head restraint, pile length and embedment and, type of soil and stratum (Hassiotis et al. (1997), (Jeong et al. 2003), Lee et al. (1995), Poulos (1995), (Zhang and Wang 2010)). The major findings are presented.

2.4.2.1 Pile Location, Diameter, and Spacing

The pile location is one of the most important factors in the analysis of reinforced slopes. However, no general agreement has been found among design methods. Poulos (1995) concluded that in order to be effective the stabilizing pile should be located around the center of the critical failure surface (middle of the slope). In agreement with this theory are conclusions from Ito et al. (1979), and Cai and Ugai (2000), Jeong et al. (2003) from coupled analysis.

On the other hand, Hassiotis et al. (1997), and Jeong et al. (2003) from the uncoupled analysis presented that the optimal location will be in the upper part of the slope closer to the top. Finally, Ausilio et al. (2001), and Nian et al. (2008), found as optimal location near the toe of the slope.

Regarding the pile diameter, relatively large diameters are needed to provide the required resistance to shear and moments (Poulos 1995). In addition, the safety factor increases significantly as the pile spacing decreases. Table 5 presents a summary of the suggested pile optimum location from the reviewed literature.

Table 5. Suggested Pile Optimum Location

Method/Paper	Pile location	Model characterization
Ito et al. (1979)	Middle	Pressure based method/ Uncoupled
Poulos (1995)	Middle	Displacement based/ Uncoupled
Hassiotis et al. (1997)	Upper Top	Finite Difference Method/Pressure based/ Uncoupled
Lee et al. (1995)	Toe and Crest	Boundary Element/ Displacement based/ Uncoupled
Cai and Ugai (2000)	Middle/B and Top/UB	3D Shear Reduction Finite Element Method/Coupled analysis
Ausilio et al. (2001)	Toe	Kinematic approach limit analysis/ Uncoupled
Jeong et al. (2003)	Middle/Toe	Displacement based and Strength Reduction Finite Element Method/ Uncoupled and Coupled
Nian et al. (2008)	Toe	ABAQUS Finite Element Method/ Uncoupled
Chen and Martin (2002)	-	Finite difference program FLAC /Coupled
Pradel et al. (2010)	-	Strength Reduction Method/Coupled

2.4.2.2 Pile Head Restraint

Several authors agree that piles with restrained heads result in higher factors of safety (Ito et al. 1979, Jeong et al. 2003, Hassiotis et al. 1997). However, since all parameters are interrelated by the most effective pile condition could change as parameters such as spacing or diameter change.

2.4.2.3 The Number of Rows

Most studies have been done for a single row of piles. However, some landslides will demand more reinforcement than the one that can be provided by a single row of piles. For extensive landslides, Ito et al. (1982) presented a method from the extension of the design method of a single pile row. This type of analysis will be more complex if parameters such as pile spacing, diameter, and head condition are unknown, because the lateral forces between the rows of piles will vary along with the mentioned parameters.

The number of rows in a group will influence the ultimate lateral pressure acting upon each pile. Therefore, a group factor must be determined. Chen and Poulos (1997) presented an analysis for a group of piles consisting in two rows. The piles were arranged parallel to the direction of the soil movement. The maximum values for the pile groups were found by multiplying values from single isolated piles by the calculated group factor. From this study was concluded that the group effect should be estimated by a finite element analysis in order to obtain satisfactory prediction of the pile response.

Chen and Martin (2002) performed a study in which groups containing two rows of pile were analyzed under parallel and zigzag rearrangement as presented in Figure 11.

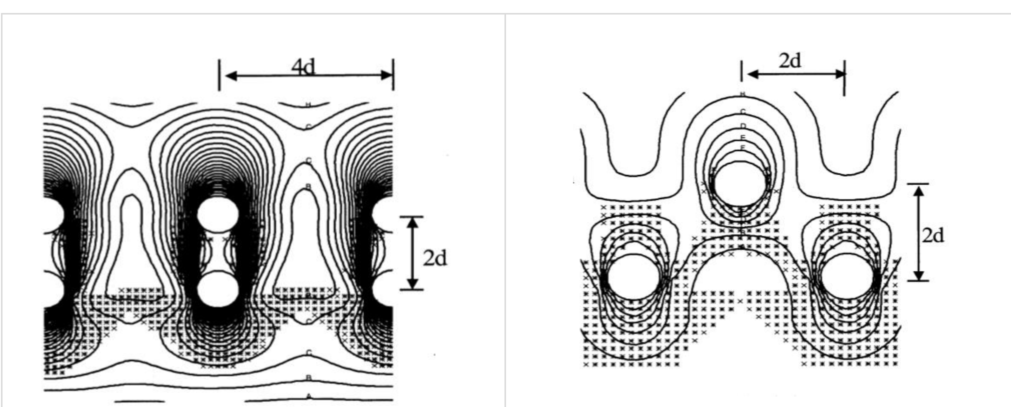


Figure 11. Parallel and Zigzag Rearrangement from Chen and Martin (2002)

From the study, the parallel arrangement was found similar to the case for single row of piles. While for the zigzag arrangement, was found to provide more resistance to soil movement due to the multiple soil arching effects that can be developed by this type of arrangement. The arching effect is considered a stabilizing mechanism that according to Chen and Martin (2002) is limited by pile spacing. For drained conditions, the lateral forces on the pile are reduced with reducing space. On the other hand, for the undrained conditions, the forces action on the pile will be higher and the group effects can be considered not significant.

2.4.2.4 Pile Length and Embedment

This parameter will control the soil failure mode as mentioned before the intermediate mode of failure will yield the maximum shear and bending moment resistance. Hence, an adequate length of pile may be assured to achieve the required embedment. Poulos (1995) suggested that the pile length must be increase if the critical failure surface is near or below the pile tips. He suggested that the optimum depth at which the maximum resistance is developed (hence, the intermediate mode of failure will develop) is when the failure surface is located at 0.6 to 0.75 times the length of the pile. In other words, the length of the pile in the stable soil must be 0.4 to 0.35 times the length of the pile to ensure the optimum mode of failure. Therefore, pile length depends on the characteristics of the analyzed slopes and will be different for each studied case. The embedment depth has been suggested to be 0.5 the ratio sliding depth / pile length which means at least half of the pile must be placed in the stable zone.

2.4.2.5 Type of Soil

The properties of the material within the slope will primarily define the characteristics of the critical surface failure. For a soil with a large cohesion and a small friction angle, the most critical failure surface will be a deep circle, while shallow circle will be the critical slide surface for a soil with a small cohesion and a large friction angle material (Huang 2014, NAVFAC (1986)). This is especially important for an uncoupled analysis where the slope stability analysis is performed based on LEM in which assumptions regarding the shape and location of the failure will be done. That assumption will consequently influence the lateral; response since pile embedment, maximum moment and shear will varies regarding the surface failure depth.

2.4.3 Advantages and Disadvantages of LEM vs Numerical Modeling

This section is focused on the comparison between performing analysis by LEM, which means an uncoupled analysis and performing geomechanical analyses (coupled approached). The advantages and disadvantaged presented in Table 6 are summarized from the literature review from several authors such as Duncan (1996), Geo-Slope International (2010), Cai and Ugai (2000), Nian et al. (2008), Pradel and Chang (2011), Griffins and Lane (1999), Jeong et al. (2003), Won et al. (2005). The uncoupled approach is separated into displacement and pressure-based because different advantages and disadvantages from each method were found.

Table 6. Compassion LEM and Numerical Modeling

LEM-Pressure based	LEM-Displacement based	Numerical modeling; FE/FDM
Advantages		
Well known-widely applied Considered of easy application and simple.	Well known-widely applied. Considered of easy application and simple. Able to directly measure accurate pile responses from the soil movement. Considered an intermediate solution between pressure-based and FE/FD modeling.	A single analysis can be performed. No assumption of the location and shape of the critical surface failure. Failure mechanism no need assumption. Sensitivity analysis to determine different design alternatives can be more easily done. Progressive failure and shear failure can be monitored.
Disadvantages		
Soil-pile interaction is not considered. The method is considered conservative. The assumptions are considered simplified. Assumption regarding the location and shape of the critical surface failure is needed. Considers a fixed critical surface Does not consider pile flexibility, saturation and arching of clays. Considers the soil able to plastically deform around the pile. Thus, does not represent the real soil/pile interaction. Two analyses should be performed (slope and pile).	Needs accurate values of displacement to be reliable. Greatly depends on the soil properties representation for the lateral analysis (*p*-*y* curves). Two analyses should be performed (slope and pile).	Complex and costly. May required lab calibration. Could require considerable time. Knowledge is required to avoid misuse and misinterpretation of the results. Highly dependent in the constitutive model for soil.

2.4.4 Step-by-Step Design Approach for Pile Reinforced Slopes

After the selection of the general analysis as either uncoupled or coupled, step-by-step procedures must be followed to ensure the correct application of the selected method of analysis.

For the uncoupled analysis several authors (Isenhower et al. 2016, Vessely et al. 2007, Reese et al. 2004, Esser and Vanden Berge 2010, among others) suggested a procedure that can be summarized as follows:

For Distributed Load Approach:

1. Determine if the slope requires reinforcement by performing an unreinforced analysis and comparing the factor of safety to a target value.
2. Determine the require loads for stabilization that will reach the target factor of safety. This must be determined by performing the stability analysis after placing the pile in the proposed position. From the analysis the location of the critical surface should be also determined.
3. Analyze the pile response with a computer program. In this study, LPILE will be used.
4. Develop *p-y* curves for the soil above and below the sliding mas (modifiers should be applied for active landslide and closely-spaced shafts).
5. Applied a distributed load to the pile that could follow: (NAVFAC (1986), Reese et al. (2004), ODOTO (2011). The distributed load must be applied from the ground surface to the sliding depth.
6. Determine the pile responses to the applied loads and compare the maximum bending moment and shear to the nominal values of the pile to verify structural integrity.

For Displacement-based Analysis:

1. Follow steps from 1 to 4 from the previous method.
2. Defined a soil movement profile along the length of the pile, the soil movement must be evaluated before (From in situ measurements, finite element analysis, correlation with case studies).
3. Determined the pile responses to the applied soil movements and follow step 6 for distributed load approach.

For Coupled Analysis:

For the coupled analyses, the soil properties and constitutive model must be selected. The slope and the pile are modeled together, thus, the most important key to the accuracy of the results is the correct representation of the materials and conditions that will be modeled. As in the uncoupled analysis, it must be determined if the slope requires reinforcement by performing an unreinforced analysis. Furthermore, if the reinforcement is required the pile must be placed at the proposed position and the new factor of safety could be evaluated. From the analysis, the maximum bending and shear in the pile should be compared with the nominal values of the pile to verify structural integrity.

For both coupled and uncoupled if the structural integrity of the pile is compromised modifications on reinforcement, geometry or pile material must be done. Additionally, before conducting the lateral response analysis it must be established if the loads from the slope stability analysis correspond to factored or unfactored loads to relate them to LRFD or ASD analysis. Currently, for the displacement-based method, the analysis can only be conducted for ASD; it must be specified if the value used for the pile analysis is ultimate or allowable.

CHAPTER 3. STUDIED PROBLEMS

3.1 General Information

In this chapter, three cases will be analyzed. Each of the cases will represent a different soil or reinforcement condition. The cases are studies previously presented by other authors with the objective of verifying, validating, or contrasting the results. For each case, description, soil properties, and results will be presented. Regarding the analyses, the unreinforced condition will be analyzed first followed by the reinforced condition. Moreover, limit equilibrium followed by finite element analyses will be presented following a general classification as uncoupled (limit equilibrium-pile analysis) and coupled (FD and FE analysis).

As stated previously, the analyses will be performed by two limit equilibrium based programs (Slope W and Slide 7.0) in conjunction with LPILE for the analysis of the laterally loaded pile followed by a 2D finite difference analysis using FLAC and the 2-dimensional elasto-plastic finite element analysis by the shear strength reduction method using Phase2. In this thesis, for the limit equilibrium analyses, the procedure presented by Poulos (1995) will be followed. The unreinforced analysis will be performed, then the shear force necessary to achieve a minimum factor of safety depending on the target value for each case will be determined.

The required stabilizing force will be used for verification in a new stability analysis and finally, the resistance developed by the piles will be verified to guarantee that the pile will provide the necessary force. The overall results will be summarized and discussed for each specific case in the following chapters.

3.2 Problem I: Layered Frictional Case

A layered slope consisted mainly of frictional materials will be studied. Figure 12 presents the general geometry of the slope. This corresponds to chainage 3225, western side, state highway 23, Newcastle, Australia presented by Poulos (1995). The soil properties of the slope are presented in Table 7. In this case, constructability issues resulted in the creation of slopes up to 8m. Hence, the stability analyses were conducted to verify the areas considered unstable and installation of pile as reinforcement was suggested.

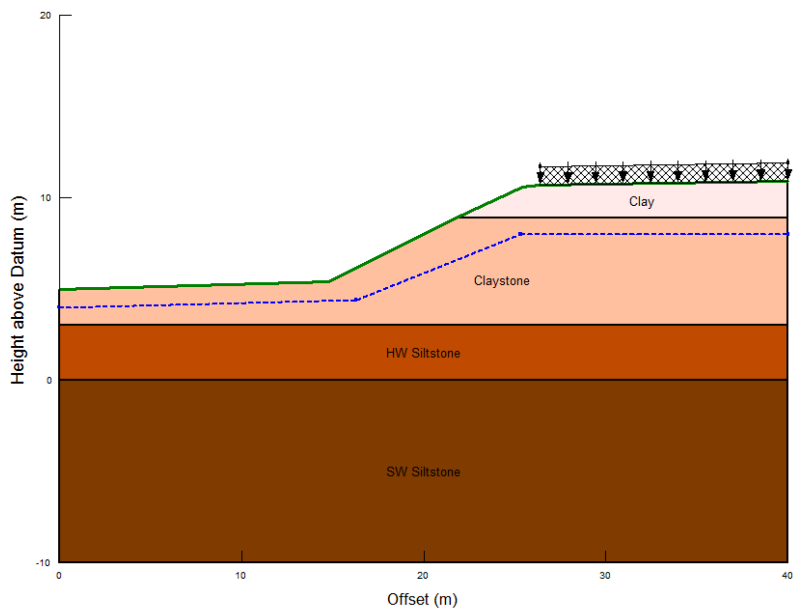


Figure 12. Geometry Probem I

The stability analysis will be performed by following the original assumptions presented by Poulos (1995); a circular slip surface and using the Bishop's simplified method. It is important to highlight that the circular assumption is just necessary for the limit equilibrium analysis. The existence of a weak stratum within the geometric configuration of the slope is the major issue to be addressed for the geotechnical design. Determining the location of a failure surface in a layered slope represents a challenge due to the variability of each soil properties.

Table 7. Soil Properties Problem I

Soil type	***c′* (kPa)**	Φ’(°)	Young’s Modulus (kPa)	Poisson’s ratio
Clay (Fill)	5	25	8,000	0.30
Claystone	5	22	20,000	0.36
HW Siltstone	0	25	33,500	0.31
SW Siltstone	20	30	53,000	0.36

3.2.1 Uncoupled Analysis

For the uncoupled analysis, slope stability and pile analysis results are presented. First, the factor of safety of the unreinforced slope is verified. If the factor of safety is less that the target value, in this case 1.5, the slope will need to be reinforced and the reinforced analysis is then conducted as uncoupled.

3.2.1.1 Unreinforced Analysis

The results for the given conditions are presented below in Figures 13 and 14. Figure 13 presents the results from one of the limit equilibrium programs (SLOPE/W). Moreover, the location of the failure surface, as well as the value of the factor of safety can be seen. Figure 14 shows the results attained from the same limit equilibrium analysis but using a different software (Slide 7.0). From the reported results, high similarity of the factors’ of safety can be appreciated. From SLOPE/W factor of safety was found to be 1.263. On the other hand, from Slide 7.0 the factor of safety value was 1.269.

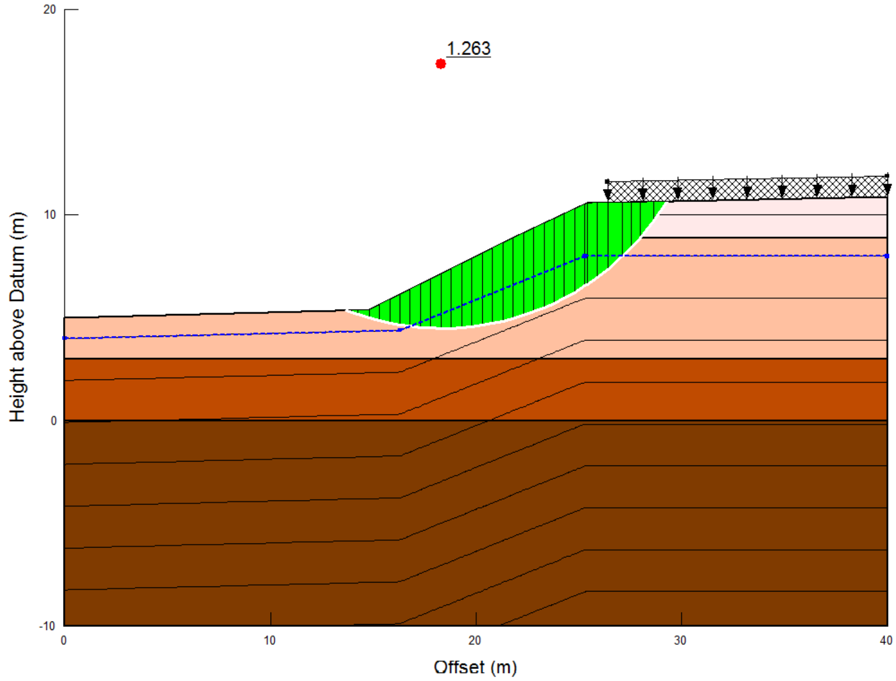


Figure 13. Unreinforced SLOPE/W

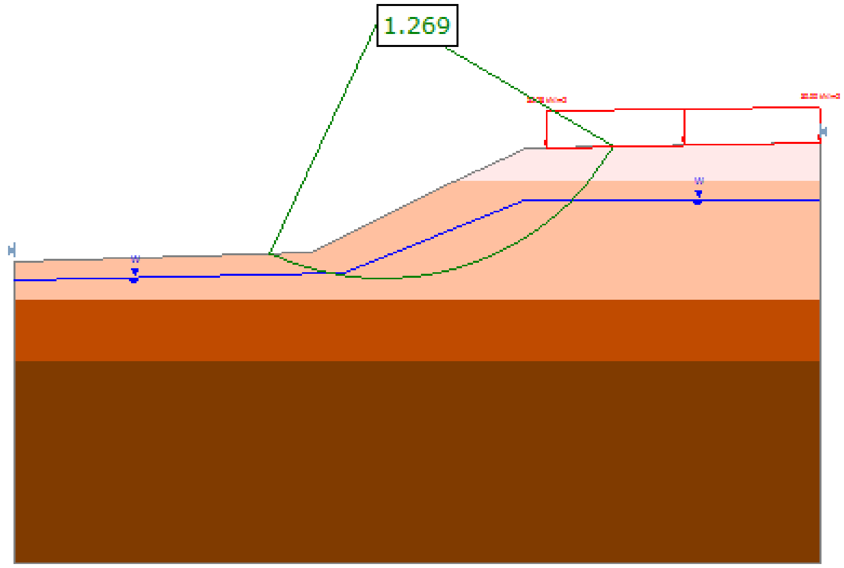


Figure 14. Unreinforced Slide 7.0

3.2.1.2 Reinforced Analysis

Reinforced analyses were conducted by placing a pile near the mid-slope. They are two (2) methods to determine the required shear force from a reinforcement to achieve the target factor of safety: (1) “Method A” and (2) “Method B” by Duncan and Wright (2005). In this study, the “Method A” will be used to compare among programs. However, “Method B” will be also used for comparison with the original case study that presumably employed the mentioned method to determine the required shear.

The “Method A” has been selected because as suggested by Duncan and Wright (2005), soil and reinforcement have different sources of uncertainties. Thus, it would be more accurate to factor the forces separately as in “Method A”, where the reinforcement forces are included in the denominator of the factor of safety equation. This means that reinforcement forces are not divided by the slope stability factor of safety.

The parameters needed it to conduct this analysis in the limit equilibrium programs are: the shear force (To be determined), pile length (9m), and pile spacing (3m) as given by Poulos (1995). For the limit equilibrium analysis, after performing the unreinforced analysis the required stabilizing force must be determined. This force can be calculated from the stability analysis as the necessary force from the reinforcement to reach the minimum factor of safety specify.

In Poulos (1995) the stabilizing force was computed as 87.5kN (per unit width of soil). However, in the present study, the stabilizing force varies among the software and the assumptions regarding its inclination. An initially reinforced analysis was performed with the value presented by Poulos (1995).Subsequently, adjustments were made until the target factor of safety was reached.

From SLOPE/W the factors of safety were found to be 1.59 and 1.58 for a force perpendicular to the reinforcement and parallel to the slip respectively. The same values were found for the corresponding analysis in Slide 7.0.

Furthermore, the values of shear forces to achieve 1.500 for “Method A” were determined. For SLOPE/W the target value was found with a 61kN/m shear force when the inclination of the force was assumed perpendicular to the reinforcement, while a slightly slower F.S. was found for the parallel case (F.S.=1.490). The same analysis carried out by Slide 7.0, yielded a shear force of 60kN/m to reach the target factor of safety in the perpendicular case and again smaller F.S. in the parallel condition (F.S.=1.490).

If the analysis is conducted by the “Method B”, the value of shear presented by Poulos (1995) achieved a 1.499 factor of safety. From the equation presented in Poulos’s study, equation (2), it could be understood that the method used to determine the reinforcement load was “Method B”. Hence by indicating “Passive” method in Slide 7.0, the values of the required shear from the present and previous study (87.5kN/m) produces very similar factors of safety. Therefore, as indicated by the reviewed literature (Duncan and Wright 2005), the “Method B” is more conservative than the “ Method A” demanding higher shear from the reinforcement to achieve same factor of safety since the force is divided by the slope factor of safety.

Table 8 presents a summary of the factors of safety from the reinforced limit equilibrium analysis. Figure 15 presents pile location, critical surface, and target factor of safety for the general case and also a comparison between the reinforced and unreinforced critical surface’s location.

Table 8. Factors of Safety Reinforced Limit Equilibrium Analyses

Method	**Required Shear (kn/m)**	**Perpendicular to the pile**		**Parallel to the slip**	
	kN/m	SLOPE/W	Slide 7.0	SLOPE/W	Slide 7.0
Method A (Allowable)	87.5	1.590	1.590	1.580	1.580
	60	-	1.500		1.490
	61	1.500	-	1.490	-
Original Poulos *	87.5	1.500			
Method B		-	1.499	-	1.494

*Poulos (1995) stability analysis was not performed with SLOPE/W or Slide 7.0.

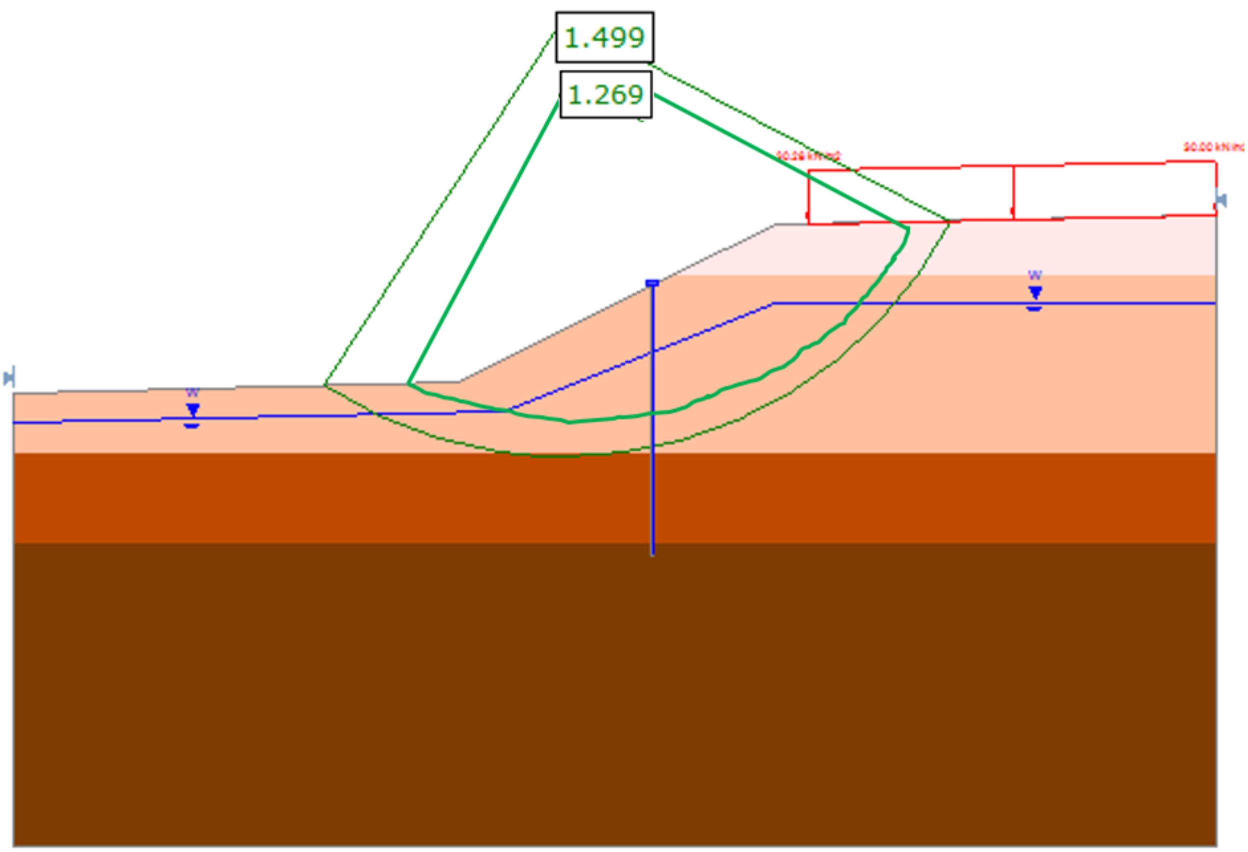


Figure 15. Unreinforced (F=1.269) and Reinforced (F=1.499) Results Problem I

3.2.1.3 Discussion of the results Unreinforced and Reinforced Analysis

From the results sections 3.2.1.1 and 3.2.1.2, the factors of safety of the slope analyzed using SLOPE/W and Slide 7.0 are compatible with the values from the study presented by Poulos (1995). For the unreinforced case, a difference of 8% between the original value presented by Poulos (1995) and the present study has been found. However, the agreement between the values found for both SLOPE/W and Slide 7.0 indicate that around 1.26-1.27 lies a reliable value for the unreinforced slope. In addition, the preliminary unreinforced analysis for all three cases suggested that the slope must be reinforced since the target value (1.5) has not been met.

Regarding the reinforced analysis, from the results summarized in Table 8, the target factor of safety is achieved with a shear force compatible with the value presented by Poulos (1995) when the analysis is performed with “Method B”. This is expected because the equation presented by Poulos (1995) considered the target factor of safety as the ratio between resisting forces and driving forces, including the reinforcement loads in the resisting forces. The difference between the factors of safety from “Method A” and “Method B” can also be appreciated by comparing the magnitude of the shear forces required to achieve the same factor of safety from each case. For “Method A” a shear of 60kN/m (allowable) will be enough to develop the 1.5 factor of safety. On the other hand, for “Method B” the value must be 87.5 kN/m (ultimate). The results are compatible with the information presented by Duncan and Wright (2005) that suggested that shear the values from “Method B” should be considered ultimate while the values from “Method A” should be treated as allowable values. Duncan and Wright (2005) pointed that the difference between the two methods could be significant and that determining what method is used for each program is essential to specify an appropriate reinforcing force. The results also suggest that assuming the inclination of the shear forces either perpendicular to the reinforcement or parallel to the critical surface does not represent significance source of variability among the results. For this case, the variability regarding the mentioned assumption does not represent more than 1%.

Figure 15 shows the changes on the critical failure position when the reinforcement have been placed as indicated by several authors from the literature review (Ausilio et al. 2001, Ito and Matsui 1975, Lee et al. 1995). Therefore, when assuming a fixed location of the sliding surface the actual soil/pile interaction mechanism is not accurately represented. The sliding depth is an important factor to determine the structural integrity of the laterally loaded pile. The difference between the presented results in terms of the required shear for the target factor of safety among methods is graphically presented in Figure 16.

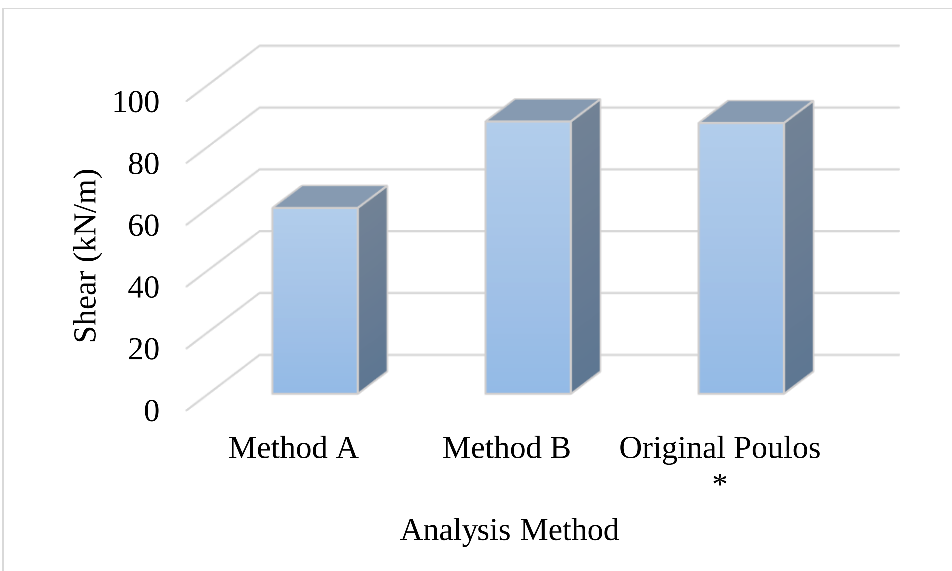


Figure 16. Shear Force to Achieve F.S.=1.5 among Methods

3.2.1.4 Sensitivity Analyses (Uncoupled)

After the stabilizing force has been determined, additional analysis were performed with the calibrated condition that matches Poulos (1995). The purpose of this analysis is to study the influence of pile length in the required reinforcing load to achieve the target factor of safety. In some cases, the values of the target factor of safety were not achieved no matter the magnitude of the shear. For those cases, the optimum value at which the factor of safety did not present any improvement has been specified. Table 9 summarizes the results from the mentioned analyses.

Table 9. Pile Lenght Influence on Required Shear Force

Length of Pile (L1, m)	Critical surface depth (L2,m)	**L2/L1	Required shear (kN/m)	F.S. achieved	Ratio Length/Shear
6*	6	1.02	>48	1.33	-
7*	7.2	1.00	>46.6	1.42	-
8*	8.09	1.01	>85	1.491	-
9	5.4	0.60	90	1.5	10%
12	5.4	0.45	90	1.5	13%
14	5.4	0.38	90	1.5	16%
18	5.4	0.30	89	1.5	20%

*Not 1.5 could be achieved.
**L2/L1, ratio sliding surface/ pile length

From the results presented by Table 9, there is an optimal length at which the required shear could be minimized to achieve the target value, 9.0m. For relatively short piles (piles considered not to be embedded on the firm strata, less than 0.6 in this case) the target factor of safety was not reached. These piles represented critical surfaces touching the pile tip. After the pile length is increased beyond the optimum length, the required shear was not decreased. The ratio length/shear represents the relationship in percentages of shear by length to achieve the indicated factor of safety. Thus, piles having between 8.0m and 9.0m will represent the most efficient alternative since 1.491 could be acceptable if the target F.S.=1.5. Piles having 8m will have a failure surface below the pile tip which indicates that the failure surface has been forced below the reinforcement in this case with an acceptable factor of safety.

Moreover, to verify the influence of pile location within the slope into the factor of safety a similar analysis was conducted placing piles with different lengths and same required shear from the previous analysis, at different locations from the toe to the top of the slope. The results are presented in Figure 17.

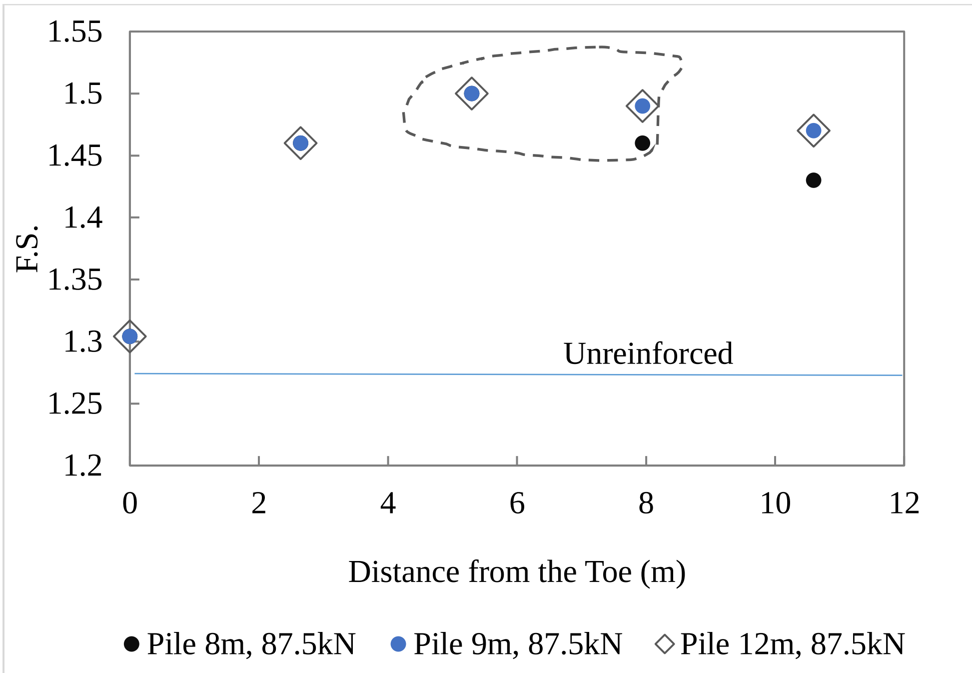


Figure 17. Factor of Safety vs Pile Location

For all piles (different length, same shear force), the highest factor of safety is found when the pile is located in the middle of the slope (5.3m). The lowest factor of safety is found when the pile is placed in the toe of the slope. After the optimum length has been achieved, increasing length will not help to increase the factor of safety for the same shear force. Piles located around the top seem to be another good alternative for the configuration of this slope. However, as the distance from the toe increases, the length of the pile may also be increased to ensure an adequate embedment. In addition, if the pile is located at the top, there is a risk that the downslope movement will continue past the pile. Finally, the findings from this analysis are compatible with the conclusion presented by Hassiotis et al. (1997) (also uncoupled analysis), in which the pile optimum location was found to be at the upper portion of the slope from the middle. For both cases, the properties of materials within the slope correspond to frictional material or combined frictional and cohesive soils (ϕ-c.). Thus, a conclusion can be drawn, the optimum pile location from an uncoupled analysis for slopes consisted of combined ϕ-c materials could most likely be the upper portion of the slope from the middle, especially if the frictional properties dominate the material properties.

3.2.1.5 Designing Resisting Pile

After the target factor of safety has been achieved, the pile analysis must be conducted to ensure that the piles will develop the required resistance. In the original analysis, Poulos (1995) used a simplified boundary element program in which the soil was modeled as an elastic continuum and the pile as a simple elastic beam. In this study, since LPILE will be used, the basics of the analysis (*p-y* model for soil) consider the soil as a series of discrete resistances and the pile as non-linear and elastic beam.

From the stability analisys, the sliding surface will be taken to be at 5.4m below the pile top. Hence, the moving soil will consist on the claystone, while the stable soil will be the slightly and the highly weathered siltstone, as presented in Figure 18.

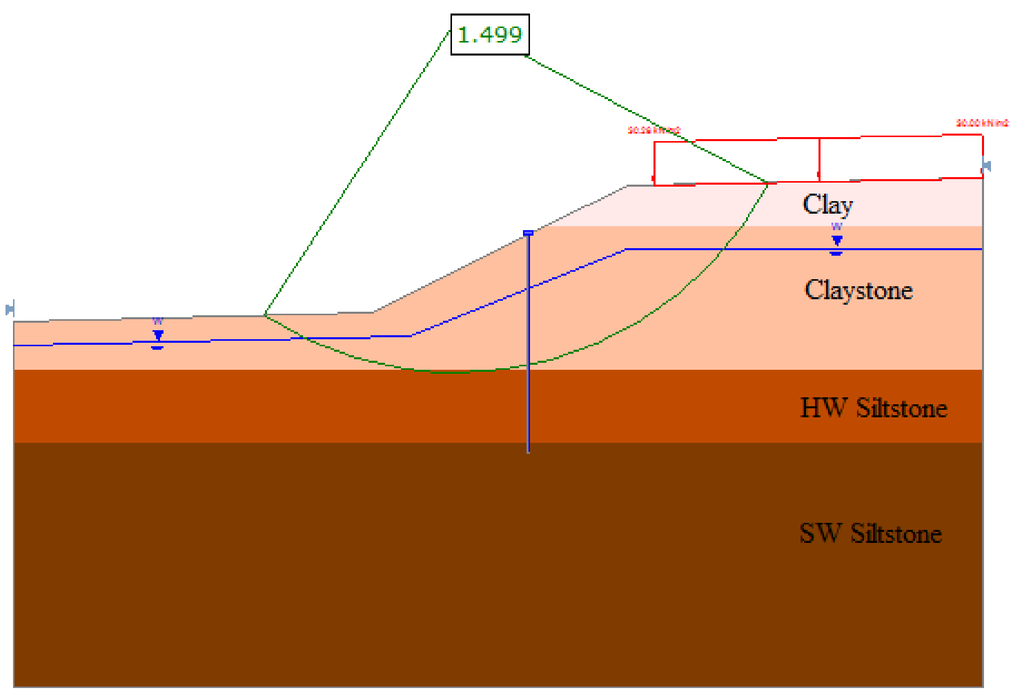


Figure 18. Critical Circle after Stabilization

The parameters used for the analysis are presented in Table 10. The soil properties presented by Poulos (1995) will be used and correlated when needed, to develop the *p*-*y* curves as user defined, following the procedure presented by Reese et al. (2004) and Isenhower et al. (2016). To generate the *p*-*y* curves for the weathered rocks, the stiff clay model has been used as presented by Gabr et al. (2002)for a possible approach for the generation of *p*-*y* curves in weathered rocks (following from Reese et al. (1975)).

The *p*-*y* curves for this problem are presented in Figure 19. Since the top of the pile is located on the mid-slope, the first curve for the analysis will be for the claystone strata. The maximum moment capacity for the section was found to be 3,393kN-m unfactored. This value is compatible with the value presented by Poulos (1995), approximately 3,100kN-m.

Table 10. Summary of Parameters for Pile Lateral Analysis

Pile diameter (Bored concrete pile)	1.2m
Reinforcement equivalent area	2.5% of Ac
Yield stress of reinforcement	260Mpa
Young’s modulus of the soil	Varying from 5Mpa at a rate of 3Mpa/m
Unstable soil limiting lateral pressure	As presented by (Reese et al. 2004)
Stable soil limiting lateral pressure	4xHorizontal earth coefficient

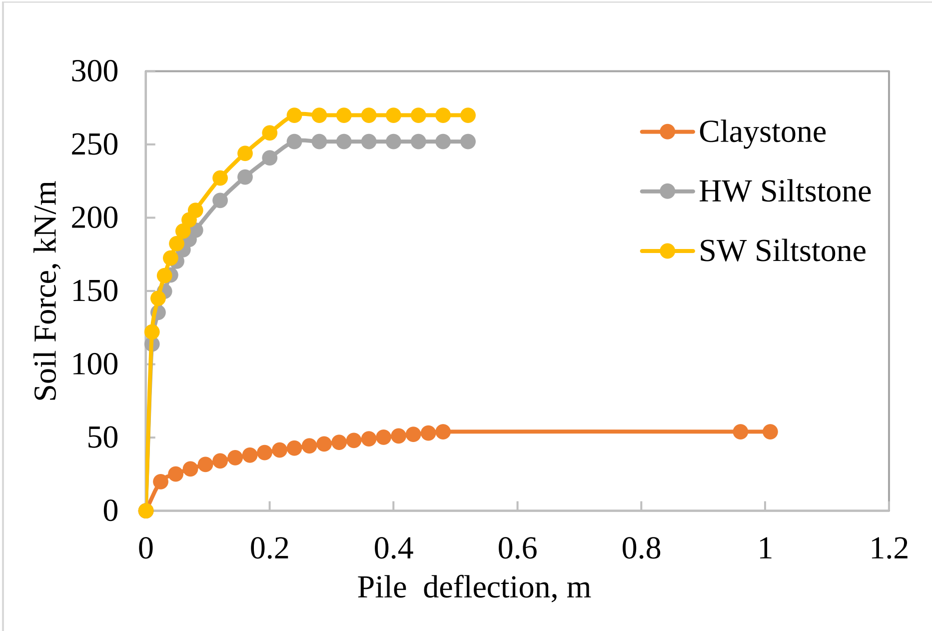


Figure 19. ***p-y*** **cuves for Laterally Loaded Pile Analysis**

To perform the displacement-based method, the best way to estimate the lateral soil movement as indicated by Jeong et al. (2003) is from reliable in situ measurements. However, approximation of the movement can be done by correlations or using finite element programs, when field test data is not available. In this study, the displacement used comes from the computations in FLAC.

Because the spacing between the piles is 3m (less than 3.5 diameters), modification should be applied to the soil resistance. Reese et al. (2007) suggested that for a single row of pile placed side by side, the *p*-modifier should be calculated as:

$$\beta_2 = 0.64\left(\frac{S}{D}\right)^{0.34} \tag{11}$$

Where β_2 is the modifier for soil resistance, S is spacing and D pile diameter. In the present case $\beta_2 = 0.87$.

An additional analysis will be conducted, for this analysis, the shear force from the LE analyses will be applied to the pile as a distributed load. The analysis follows the procedure presented by The Ohio Department of Transportation (ODOTO 2011). Figure 20 presents the distribution of the lateral load on the pile. In this analysis, because the results from FLAC and Phase2 do not factor the shear force by the slope stability factor of safety, the required shear will be considered allowable and the value from “Mathod A” will be used as unfactored load.

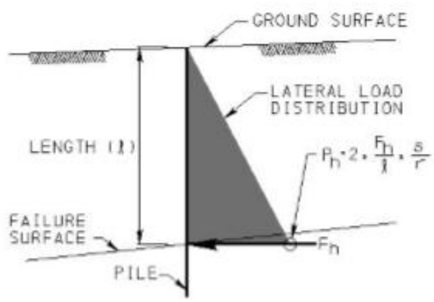
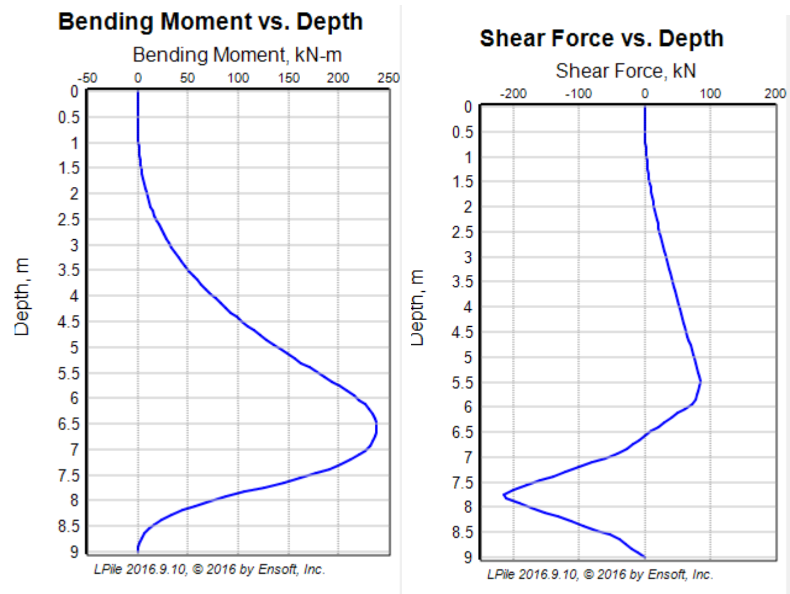


Figure 20. Distribution of Lateral Load on a Pile as Suggested by ODOTO (2011)

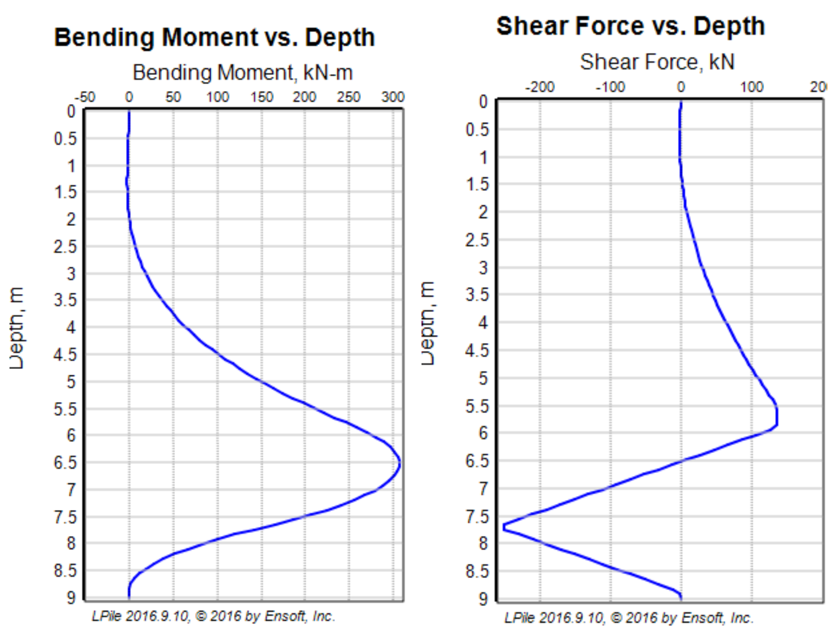
Where;

F_h is the lateral load per unit length of embankment required by slope stability analysis, s is center-to-center pile spacing, r is number of rows of piles, l is distance from ground surface to failure plane, P is distributed lateral load at the failure h plane for use in lateral pile analyses. The distributed load is presented in Table 11.

The results from the lateral analysis are presented in Figure 21. It should be pointed that the results from the displacement-based analysis greatly depend on the input displacements, for small soil movements smaller stabilizing forces will be achieved.



Displacement-based Method*Unfactored values



Distributed Load Method* Unfactored values

Figure 21. Bending Moment and Shear from Lateral Response Analyses

Table 11. Distribution of Lateral Loads for Pile Design Problem I

Variable	Value
F_h	61kN (per width of soil, Unfactored)
s	3m
r	1 rows
l	5.4m
P_h	61kN/m per pile

From Figure 21, the results from the displacement-based approach and the ones applying a uniform load from the stability analysis to the pile are similar. The maximum values for both conditions are located around the same depth. The summary of the results is presented in Table 12. To verify the pile structural integrity, the results are compared to the nominal ultimate bending moment and shear for the reinforced concrete section. The factors of safety should be >1.67 for flexure and >1.5 for shear. The shear at the sliding depth for this case does not correspond to the maximum shear. Thus, to verify the structural integrity the maximum shear should be compared with the factored shear resistance of the pile no matter the location.

Table 12. Lateral Analisys Results (Unfactored loads)

Parameter	Distributed load approach	Displacement based (90mm)
Unfactored nominal capacity of pile	3,393 kN.m	3,393 kN.m
Maximum applied moment	306 kN.m	237 kN.m
Unfactored resisting shear force	1018 kN	1018 kN
Maximum shear force at critical circle	138 kN	87 kN

3.2.2 Coupled Analysis

For this analysis, the unreinforced analysis will also be performed to verify if the target factor of safety is reached (as presented in section 3.2.1.1). The reinforced analysis will then be conducted after placing the reinforcing pile.

3.2.2.1 Unreinforced Analysis

The unreinforced factor of safety for this condition has been determined from the 2-D elasto-plastic finite element analysis by the shear strength reduction method using Phase2. Additional results are also presented from the analysis in the finite difference program FLAC.

The factor of safety is calculated using the Shear Strength Reduction method (SSR). To represent the soil behavior the Mohr-Coulomb failure criterion has been selected. This is because this criterion has been found simple and accurate in representing the soil behavior. In addition, its parameters has physical meaning and it is generally accepted in the geotechnical area (Labuz and Zang 2012).

Figure 22 presents the results from the FE SSR analysis performed using Phase2. The contours of maximum solid displacement at failure computed by the SSR method with the LE critical failure circle superposition shows a good agreement. Table 11 shows a summary for a general comparison of the unreinforced analyses from LE, FE and FD programs.

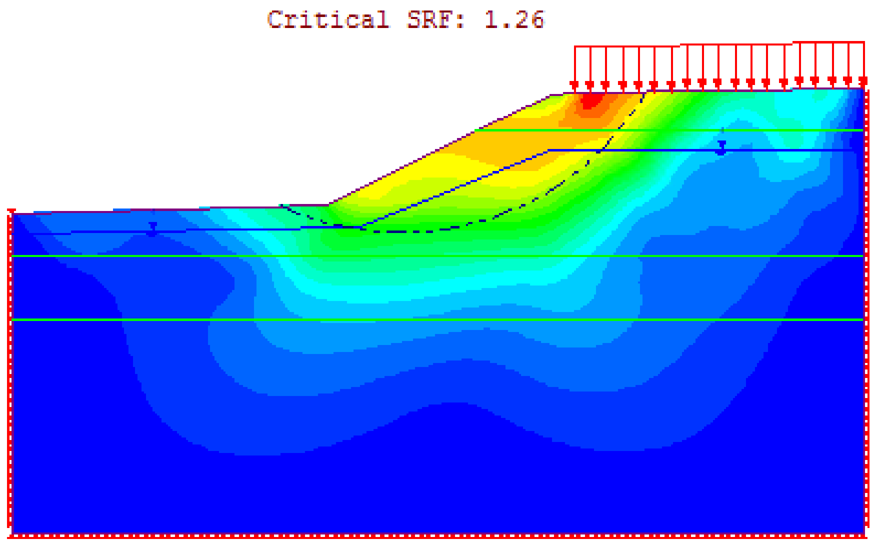


Figure 22. SSR Unreinforced Analisys with Phase2.

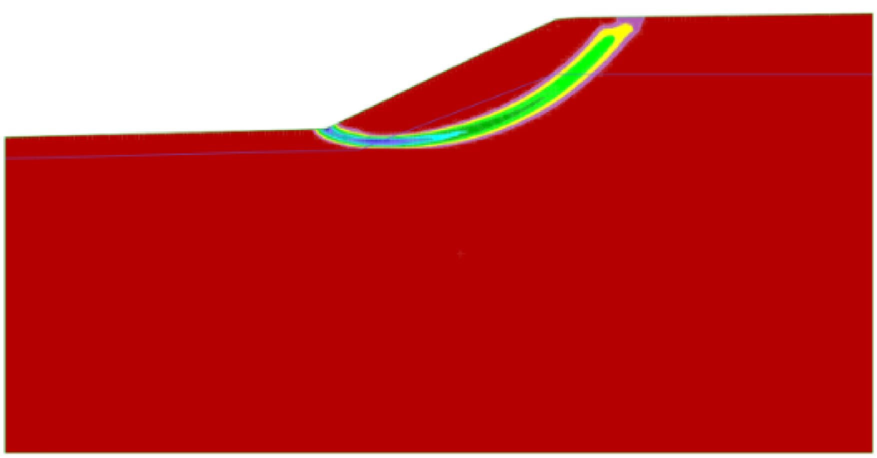


Figure 23. SSR Unreinforced Analisis with FLAC SRF=1.26

Table 13. Summary Unreinforced Analyses Problem I

Programs	SLOPE/W	Slide 7.0	Phase2	FLAC
Unreinforced F.S.	1.263	1.269	1.260	1.26

The summary from the unreinforced analysis clearly indicates that results from LE analyses can be reproduced by SSR FE and FD analyses with high agreement. In this case, layered slope with different soil properties, the difference among results is less than 1%, thus all methods are relatively accurate and reliable.

3.2.2.2 Reinforced Analysis

In Phase2, the piles were modeled using a structural interface with a joint on either side of the reinforcement (liner) allowing slip to occur between the reinforcement and the soil (Rocscience Inc. 2017). The pile properties are presented in Table 14 and the soil properties were previously presented in Table 6. These properties come from the limit equilibrium analysis with additional correlations for elastic modulus and Poisson's ratio values.

Table 14. Pile Properties SSR Analysis Phase2 and FLAC

Pile length	9m
Pile spacing	3m
Pile diameter	1.2m
Young's modulus	40,000,000
Poisson's ratio	0.2

The results from the coupled analyses are presented in Figure 24 and 25. The location of the critical failure surface from the LE is superimposed to the results from Phase2 and a good agreement is found between the two failure mechanisms. Hence, from coupled analysis the critical failure can easily be determined and no assumption needs to be made regarding the shape of the critical slide.

To verify the accuracy between Phase2 and the LE programs, the value for the maximum shear force from the FE analysis was used to perform a new LE analysis. The results are presented in Figure 26.

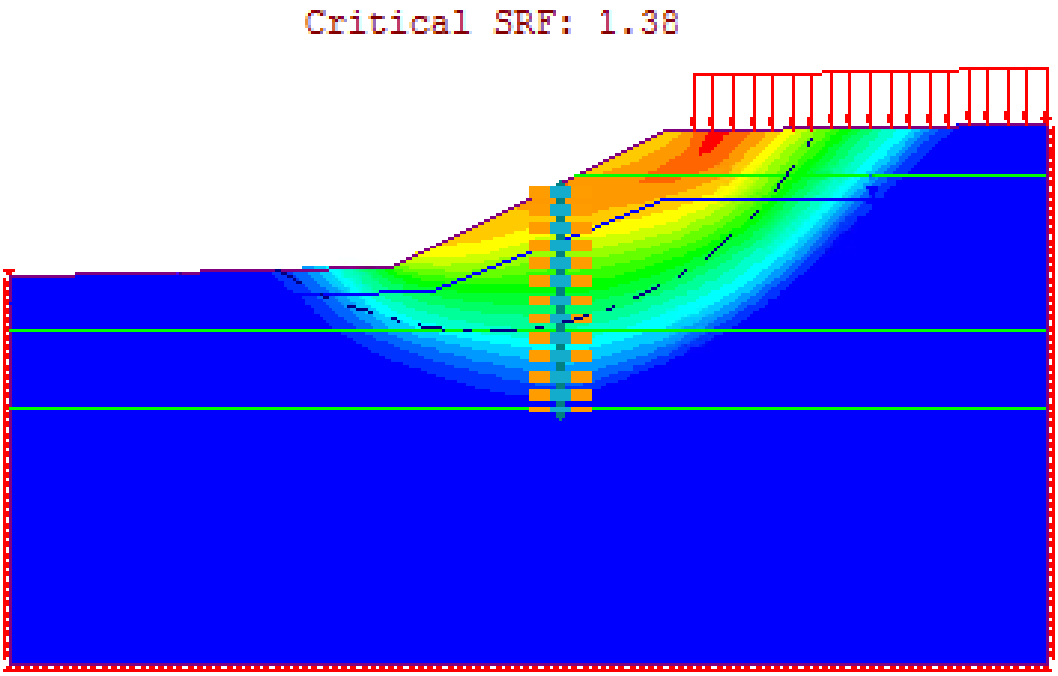


Figure 24. SSR Reinforced Analisys with Phase2

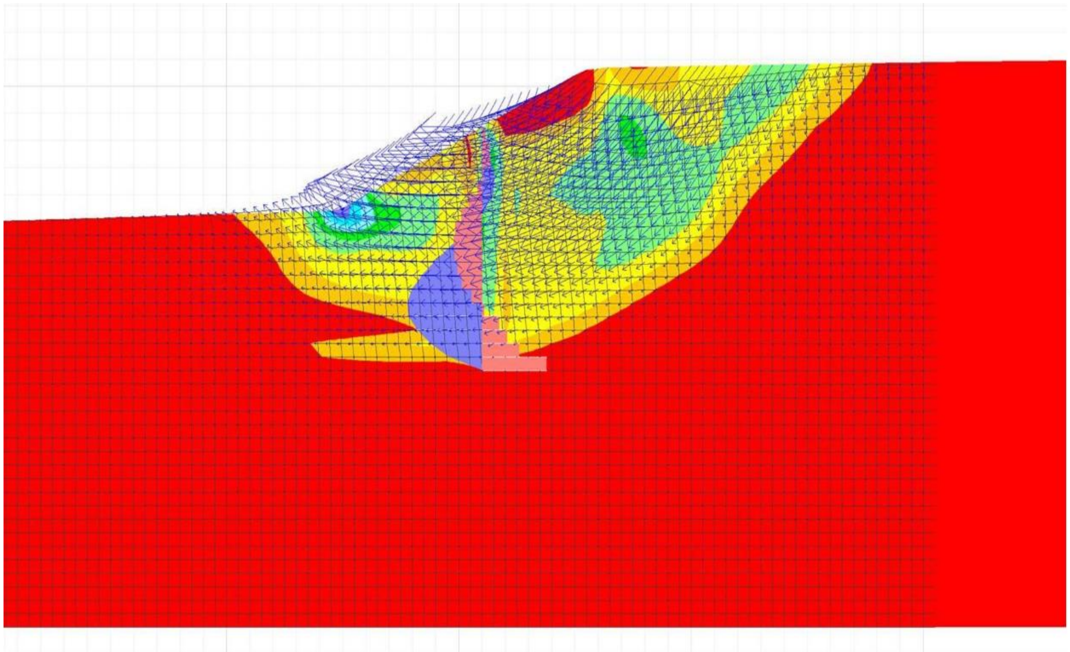


Figure 25. SSR Reinforced Analisys with FLAC (FS=1.37)

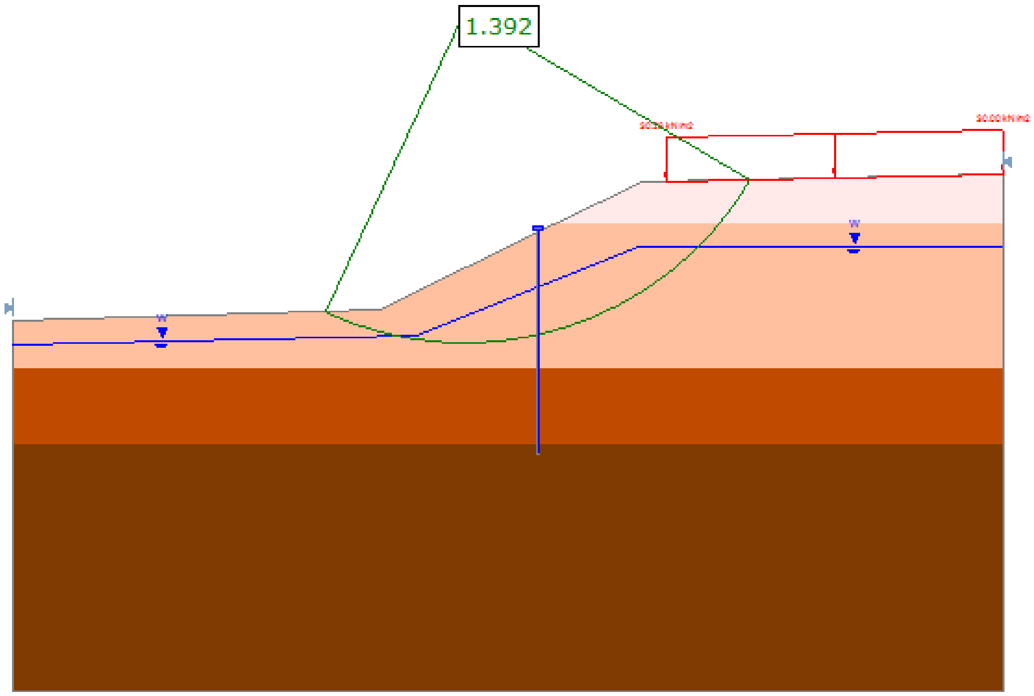


Figure 26. Factor of Safety from Slide 7.0 for Shear Force from Phase2

From the results, for the same shear force both methods LE (Slide 7.0) and FE (Phase2) produced very similar factors of safety, 1.392 From Slide 7.0 versus 1.38 from Phase2. This indicates that if both analysis result in very similar factor of safety for the same shear, then from the FE analysis, the shear resistance developed by the reinforcement under the analyzed configuration (9m, location and diameter), is not enough to attain the target factor of safety. From the reinforced analysis, section 3.2.2.2 the shear necessary to achieve 1.5 for a 9m long pile was found to be 61kN (allowable and 87.5kN, ultimate). However, from this analysis (coupled) the distribution of forces around the reinforcement indicates that modification in pile geometry (length, diameter), pile location within the slope, or pile reinforcement is needed to increase the shear resistance for achieving the desirable factor of safety.

3.2.2.3 Sensitivity Analyses (Coupled)

To verify the influence of pile location within the slope into the factor of safety, a similar analysis to that described in section 3.2.1.4 was performed in Phase2. Piles with different lengths were placing at different locations from the toe to the top of the slope. The results are presented in Figure 27.

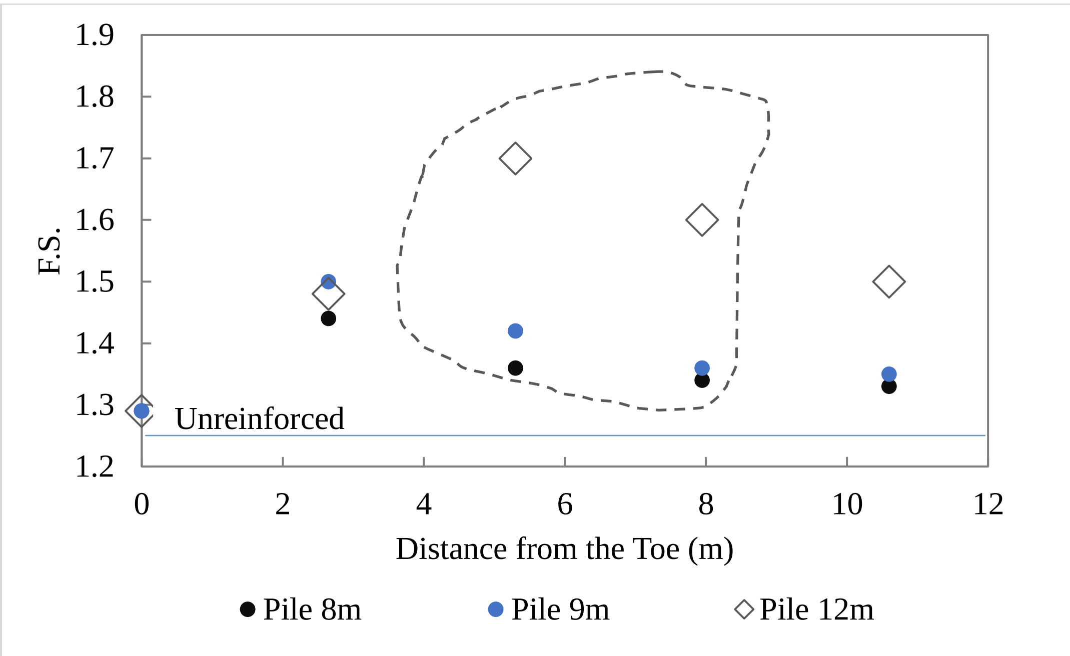


Figure 27. Factor of Safety vs Pile Location Original Condition

From a coupled analysis as presented before by Jeong et al. (2003), Won et al. (2005),and Cai and Ugai (2000), the optimum pile location from SSR FE analysis was reported to be in the middle of the slope. However, from Figure 27, in this case, the optimum location is found at a distance Le/4 (quarter of the slope from the toe) for the smaller piles. As the pile length increases, the optimum location changes to the middle of the slope. Comparing the results from Sections 3.2.1.4 and 3.2.2.3, the results suggest that from uncoupled analysis the pile optimum location is in the middle and around the top as presented by Hassiotis et al. (1997). On the other hand, for the same slope, the coupled analysis suggested to place the pile around middle being slightly higher the factor of safety of the piles located around the quarter of the slope (for shorter piles). Hence, the common findings among methods for the properties of the material of this slope (frictional) are: piles located at the toe do not represent a favorable condition, and placing piles around the middle will always increase the factor of safety especially when an adequate embedment depth is assured.

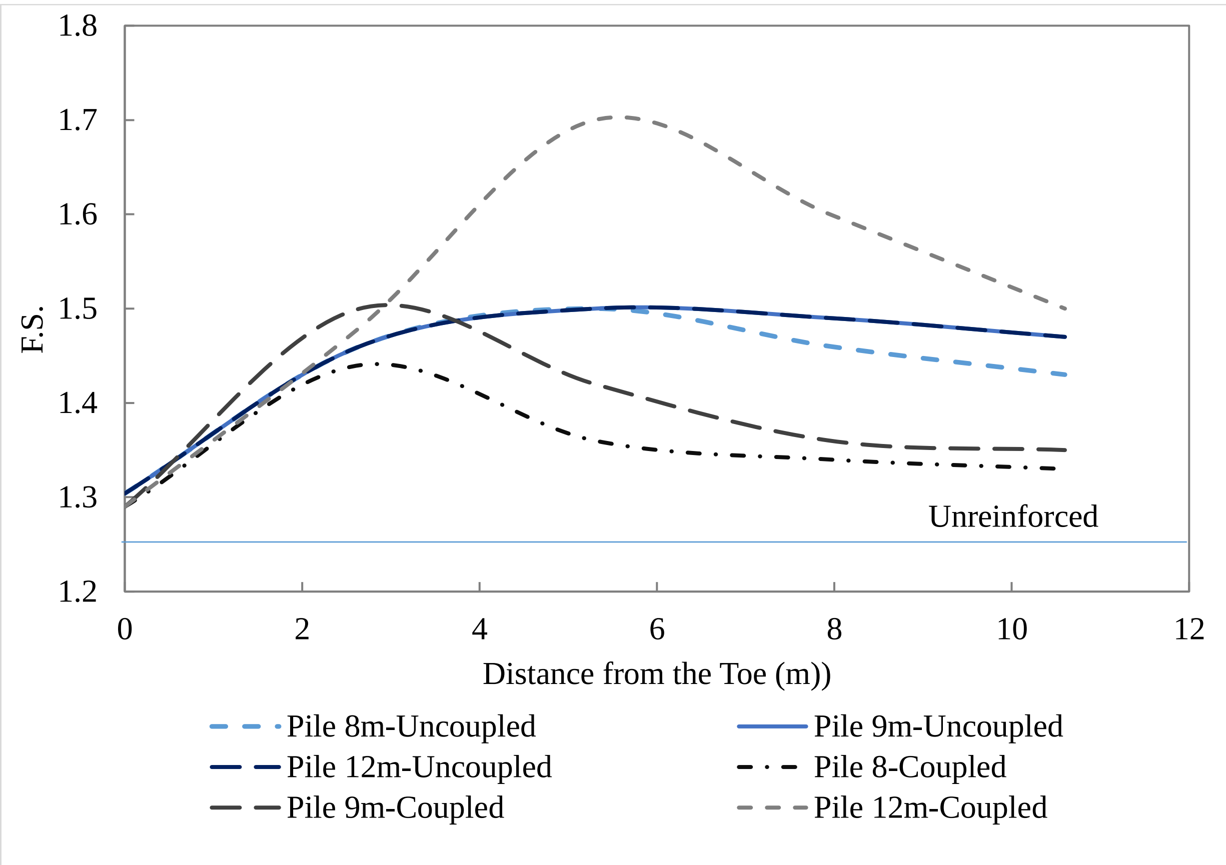


Figure 28. Optimun Location Coupled and Uncoupled Analyses

3.2.3 Summary Results Coupled and Uncoupled Analyses Problem I

An overall summary table from the performed analyses is presented below. Table 15 presents the comparison for unreinforced analysis from the fourth computer programs and the original value presented by (Poulos 1995). For the LE programs the results correspond to the values perpendicular to the reinforcement. The allowable values will be used to compare among programs (Method A), because the values from Phase2 and FLAC correspond to unfactored values. In addition, the results are presented for piles normalized at 1m since the results from Phase2 originally assume the third dimension equal to 1m.

Table 15. General Summary Problem I

Condition	**Unreinforced F.S.**	**Reinforced F.S.**	****Shear Force (kN/m)**	*****Shear at the Sliding depth (kN)**	*****Max. Shear (kN)**	**Max. Moment (kN.m)**	**Ultimate F.S.** (*)
*Poulos 1995	1.18	1.5	87.5	73	73	460	1.46
Slide 7	1.269	1.499	60	(1)43	(1)84	(1)103	(1)1.44
SLOPE/W	1.263	1.5	61	(2)29	(2)71	(2)79	(2)1.38
Phase2	1.26	1.38	-	31	80	88	1.38
FLAC	1.256	1.37	-	28	67	77	1.37

*The values presented correspond to ultimate values hence they most likely are factored by 1.50
**The values presented come from LE programs
***The values presented come from either LPILE for uncoupled analysis or FE, FD programs.
(1) Distributed load method (2) displacement-based method

(*) Ultimate factor of safety refers to performing a limit equilibrium analysis with the developed shear resistance from the lateral analysis (shear from LPILE) to verify the factor of safety with the true resistance that will be developed by the pile under the studied conditions. For Phase2 and FLAC the F.S. equal to the ultimate.

For the unreinforced analyses, all methods of analysis achieve around the same factor of safety. Thus, LE, FE, or FD programs are comparatively recommended regarding accuracy and reliability.

For the reinforced analyses, to compare the results from coupled and uncoupled analysis, the shear resistance developed by the pile and the factor of safety can be compared. On one hand, the shear resistances from LPILE (for uncoupled condition) are similar to the values from the Phase2 and FLAC. This indicates that the ultimate factor of safety of an uncoupled analysis will be lower than the target factor since the resistance of the pile will not be developed a hundred percent (as in a superposition case where the applied load will be equal to the resistance). Hence, results strongly indicate that coupled analyses are able to predict a more realistic force distribution along the reinforcement (for the uncouple analyses, the user is able to manually insert different shear without taking into account pile properties or whether or not the selected pile will develop the necessary shear).

For the displacement-based method (2), the prediction of the factor of safety was found to be similar to the results from the coupled analysis performed by FLAC. For the same displacement (90mm from FLAC analysis), the displacement-based analysis from LPILE yields slightly higher values for both bending moment and shear resistance. The values from the distributed load approach, on the other hand, where found higher for all parameters. The findings are consistent with the one described by Jeong et al. (2003) and Won et al. (2005), indicating that the displacement-based method is intermediate in theoretical accuracy between the coupled analysis and the uncoupled analysis (based in distributed load and Bishop's method), being the results from an uncoupled analysis more conservative.

Overall, the ultimate factors of safety are similar. Thus, performing a coupled or uncoupled pile reinforced slope analysis will yield similar results. Then, it should be questioned if the degree of effort from performing two separate analyses (slope and pile) for uncoupled condition is indeed smaller than the degree of effort for the inputs parameters needed to perform a coupled analysis. On one hand, to perform an uncoupled analysis, assumptions regarding the soil representation need to be made in order to developed the *p-y* curves, for combined ϕ-c materials (silt like soil) and weathered rocks equivalent models should be used since no widely known model to developed *p-y* curves for materials having friction and cohesion has been published (Gabr et al. 2002).

In addition, duality of the factors (geotechnical factor of safety and structural factor of safety) is most likely to occur if the balance and unbalance forces are not clearly defined. Moreover, as mention previously, assuming the pile will reach a desired shear without taking pile properties into account (just equilibrium) does not represent the real soil/structure interaction. Also, iterations will be needed to match pile resistance, slip surface and pile stiffness for some of the original uncouple approaches (Pradel et al. 2010).

In contrast, to perform the coupled analysis, the effort just needs to be focused on determining the required parameters for the model. The coupled analysis requires considerably more parameters that a limit equilibrium one. However, no assumptions on failure mechanism, empirical models (*p-y* curves), or modifiers will be needed. Additional parameters can be seen as a better way to represent the soil instead of an increasing in input effort.

Regarding the lateral analysis (this is for both coupled and uncoupled), the pile structural integrity is verified by comparing the maximum values to the nominal ones. In this case, to compare among programs, the lateral analysis was performed with the unfactored loads from the

LE analysis and the results from FLAC and Phase2. In general, the strength limit state (factored loads), the serviceability limit state (unfactored loads) and the geotechnical limit state (factored loads- overall stability) should be verify performing LRFD design as stated by Brown et al. (2010) in FHWA NHI-10-016.

For the optimum pile location, by comparing the results from Sections 3.2.2.4 and 3.2.3.3, the coupled and uncoupled conditions produce similar results. For soils in which friction is predominant, the optimum location will be around the middle and the upper portion of the slope if adequate embedment depth is assured. For relatively, short piles the optimum location will be around the quarter of the slow with approximated 5% difference between the factors of safety in the middle and the quarter of the slope (because the length should be increased). To assure that the embedment is adequate a sensitivity analysis should be conducted. Piles located from the middle to upper portion will require larger lengths to maintain the embedment length. In a coupled analysis, the critical modes of failure are easily determined since displacement increases with the reduction of the strength (approximation to SSF). For the uncoupled analysis, this should be verified from iterations to achieve the optimum pair, length/shear force and then verifying the ratio sliding depth and pile length to ensure that the mode of failure at which the resistance will be maximum will be operative, as presented by Poulos (1995).

The location of the maximum shear indicates that the pile length must be extended to provide the shear force required for overall stability. This can also be seen from the sensitivity analysis (coupled section) in which great improvement in the factor of safety is achieved when the pile length is increased from 9m to 12m.

3.3 Problem II –Homogeneous Cohesive Slope

A slope consisting of a uniform soil strata is presented in Figure 29. This correspond to a generic case presented by the Dept. of the Navy, Naval Facilities Engineering Command, NAVFAC (1986) to evaluate the influence of the reinforcement location in the value of the factor of safety. The soil properties are presented in Table 16.

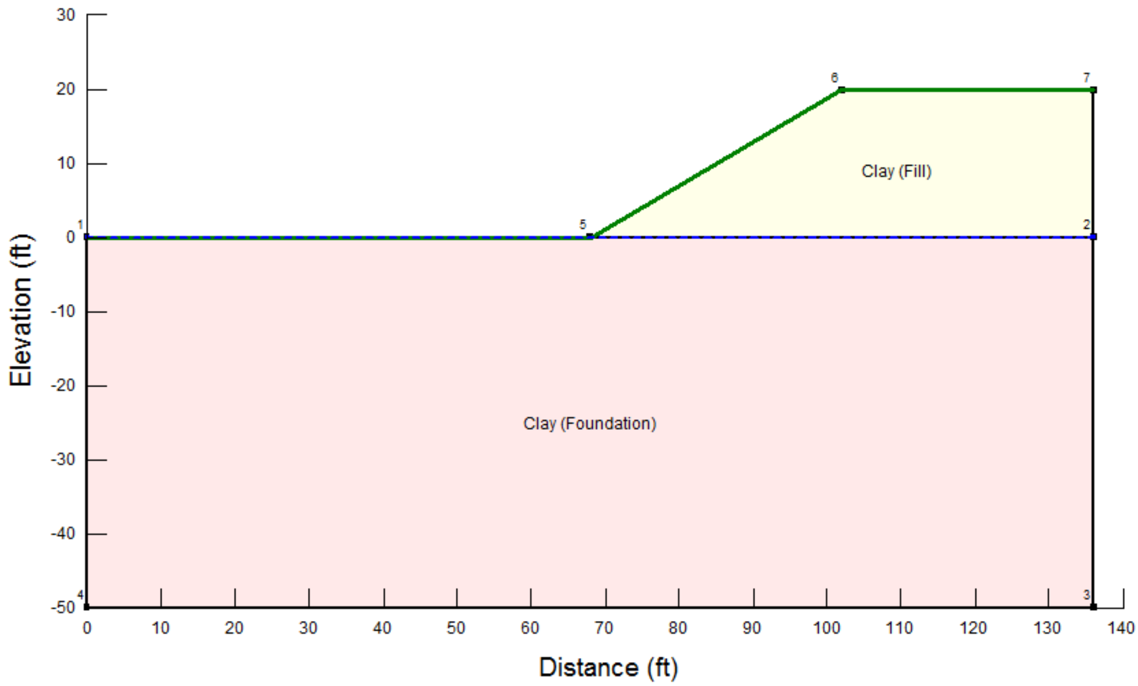


Figure 29. Geometry Probem II

The angle of inclination of the slope was determined to be 30 degrees. The final angle used to evaluate the reinforced case was the inclination at which the unreinforced factor of safety was found < 1.40 as suggested by Dept. of the Navy, Naval Facilities Engineering Command in NAVFAC (1986) the publication.

Table 16. Soil Properties Problem II.

Soil type	Unit Weight (pcf)	c' (psf)	Φ'(°)	Young's Modulus (psf)	Y
Clay (Fill)	114.38	200	17	208,854	0.3
Clay	112.40	200	15	208,854	0.4

3.3.1 Uncoupled analysis

For the uncoupled analysis, slope stability and pile analysis are presented. First, the factor of safety of the unreinforced slope is verified. If the factor of safety is less than 1.4 the slope will need to be reinforced and the reinforced analysis is conducted as uncoupled. Two different conditions will be verified for this case (1) the factor of safety will be determined by assuming the location of the slip surface, and (2) a general analysis will be conducted without restricting the location of the failure surface.

3.3.1.1 Unreinforced Analysis

For the first condition, the failure surface will be constrained to an approximated value. The failure surface according to NAVFAC (1986) would be around 9ft below the ground surface. The factors of safety for this condition were found to be 1.354 and 1.357 in SLOPE/W and Slide 7, respectively, as shown in Figures 30 and 31. Since both factors of safety were found <1.40, the slope must be reinforced.

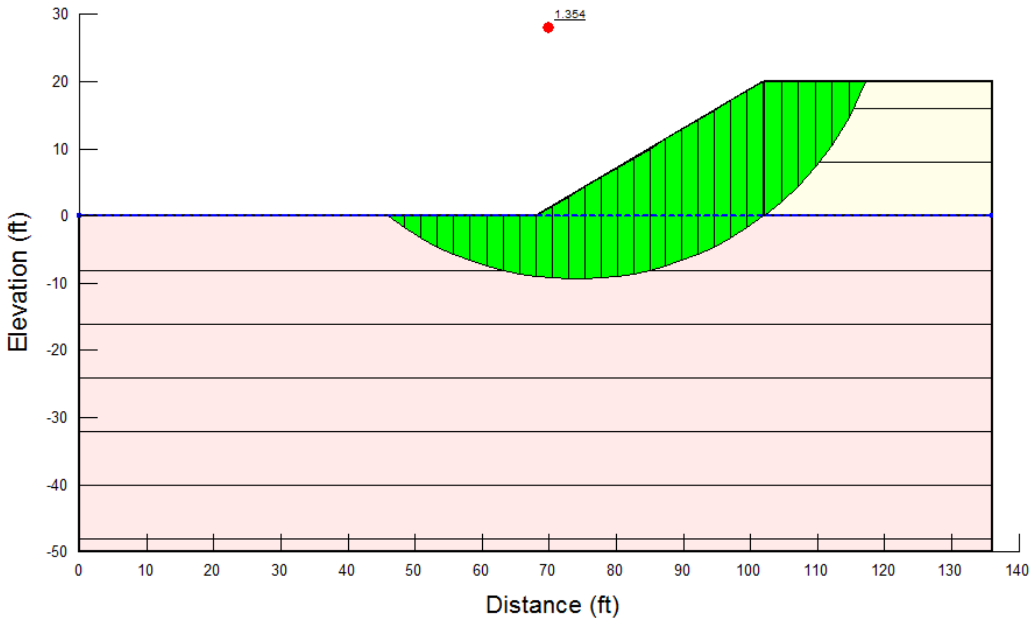


Figure 30. Unreinforced failure surface constrained-SLOPE/W

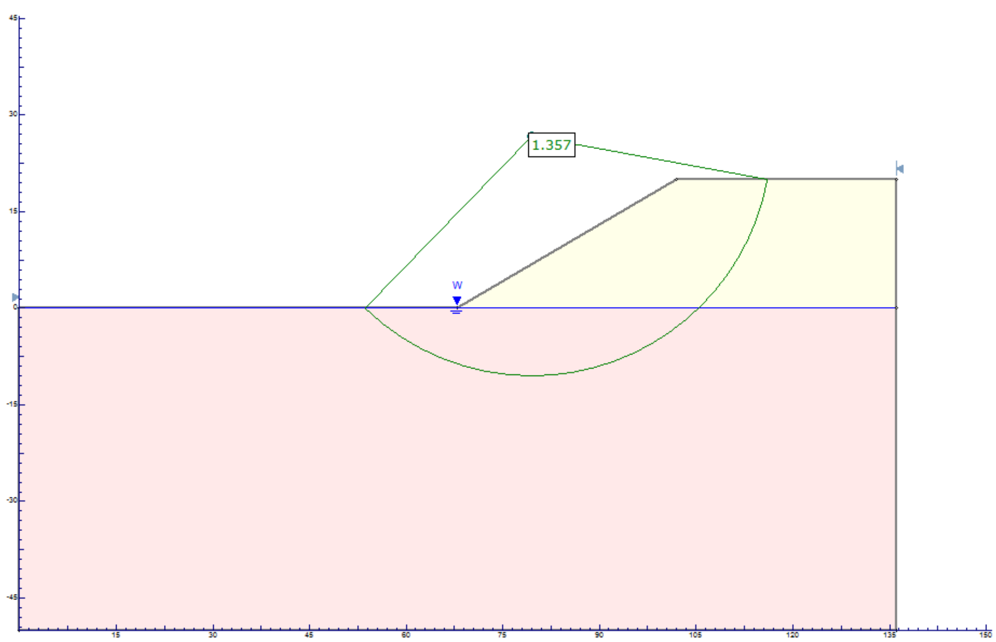


Figure 31. Unreinforced failure surface constrained-Slide 7

On the other hand, when conducting a general analysis, the location of the critical failure surface was found to be 6.34ft and 6.42ft in SLOPE/W and Slide 7 respectively. In addition, the factors of safety were found to be 1.316and 1.311 in SLOPE/W and Slide 7, respectively. Results are presented in Figure 32 and 33.

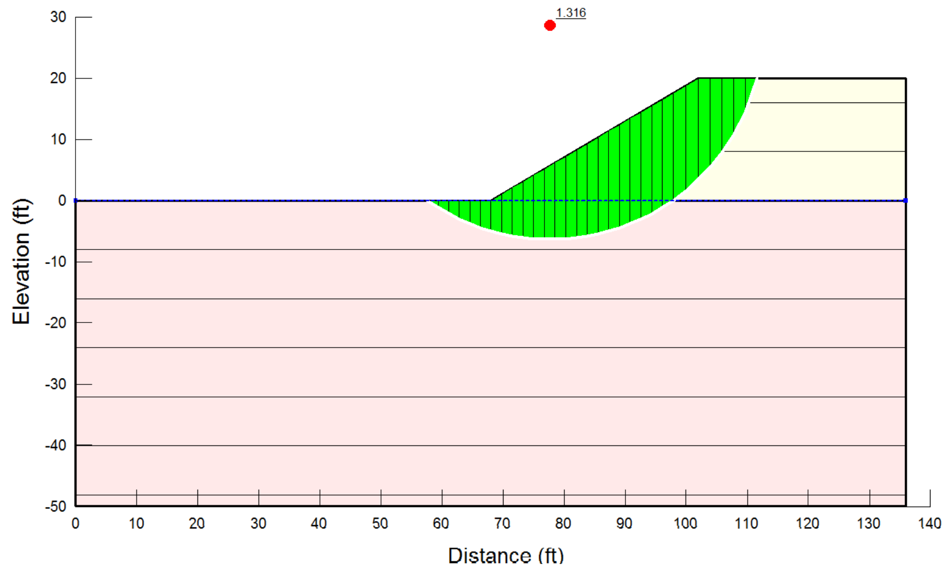


Figure 32. Unreinforced unconstrained –SLOPE/W

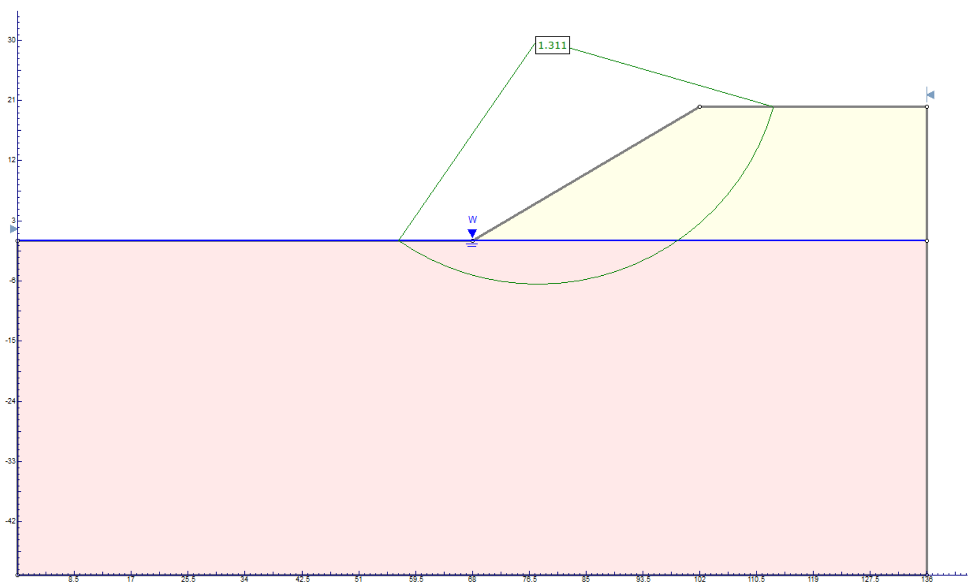


Figure 33. Unreinforced unconstrained-Slide 7.0

For the unconstrained condition, reinforcement of the slope will also be needed. Table 17 summarizes the results from the unreinforced analyses for conditions one and two as presented above. The results support the reviews from the literature that if the failure surface is assumed in a fixed position, the results will be inaccurate. However, if the assumption is carefully done the variation among the results could be minimized, at least for the unreinforced case.

Table 17. Summary Unreinforced Factors of Safety

Condition	Critical surface depth (ft)	F.S.
Constrained (SLOPE/W)	9.10	1.354
Constrained (Slide 7)		1.357
Unconstrained (SLOPE/W)	6.34	1.316
Unconstrained (Slide 7)	6.42	1.311

3.3.1.2 Reinforced Analysis

Both conditions, constrained and unconstrained, will be verify by placing the pile at the location suggested by NAVFAC (1986). Then, the pile shear force needed to increase the factor of safety to 1.4 will be determined. The pile is located at the toe of the slope assuming full mobilization of soil shear strength along the failure surface. The necessary parameters to conduct this analysis in the limit equilibrium programs are the shear force (to be determined), pile length 24.8ft from the original case, and pile spacing (4.5ft) as given by NAVFAC (1986). For the uncoupled analysis, after performing the unreinforced analysis, the required stabilizing force must be determined. This force can be calculated from the stability analysis as the necessary force from the reinforcement to reach the minimum factor of safety specified. From SLOPE/W and Slide 7 in the constrained condition, the factors of safety were reached after applying shear forces perpendicular to the reinforcing pile of 6,720lb for both analyses.

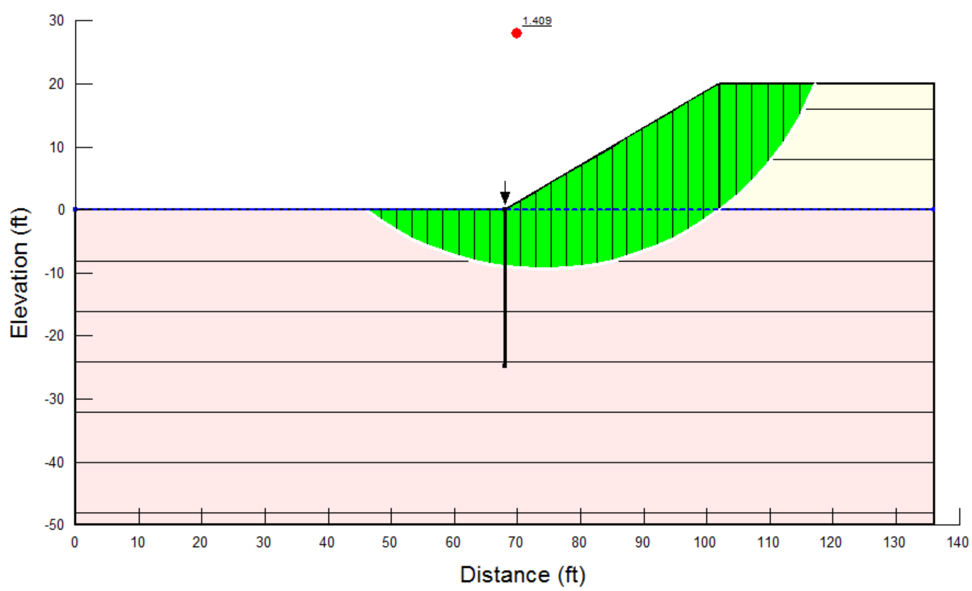


Figure 34. Reinforced Failure Surface Constrained SLOPE/W

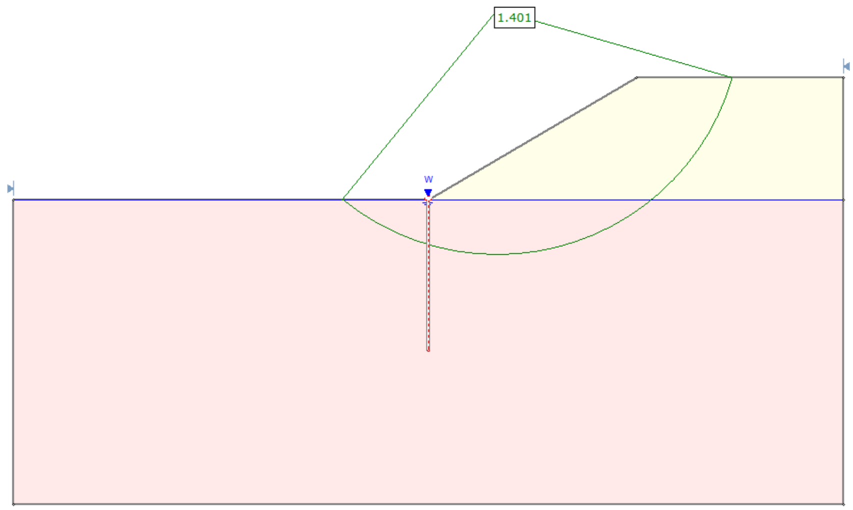


Figure 35. Reinforced Failure Surface Constrained Slide 7

On the other hand, when performing the unconstrained analyses, the factors of safety did not increase after applying values of shear resistance to the piles greater than 10 kips. This indicates that the location of the pile under this condition could not correspond to the optimal location. Figures 36 and 37 present the changes in the critical surface when the pile is located at the toe.

Additional analyses were conducted, and the pile location was varied from the toe to the middle of the slope for the unconstrained condition. The length of the pile was increased to account for the sloping ground to 30ft. Under the mentioned circumstances, the target value (F.S.=1.4) was achieved. The new critical failure surfaces from placing the pile at this location are presented in Figure 38 and 39. Table 18 summarizes the results from the reinforced analyses for both constrained and unconstrained conditions.

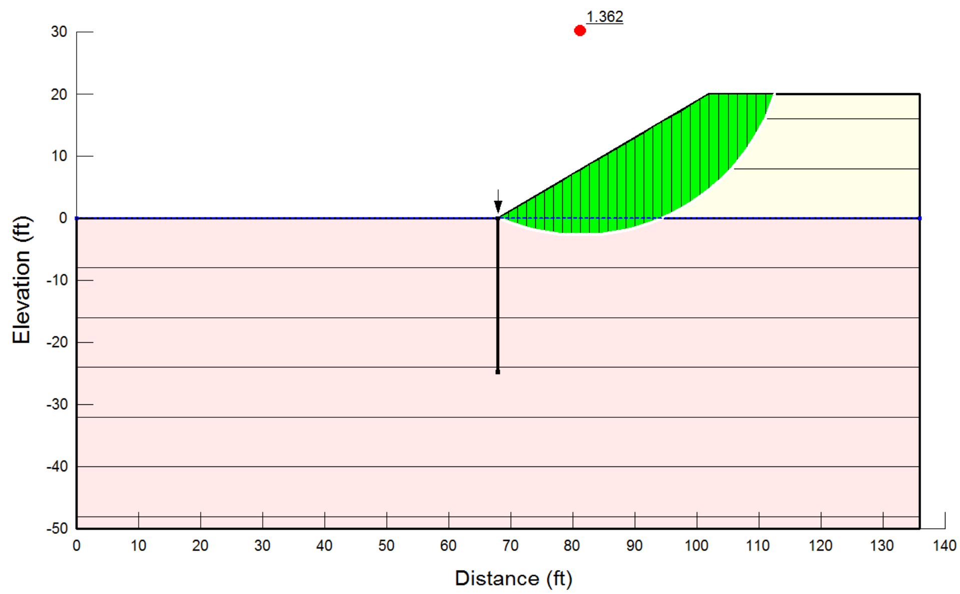


Figure 36. Reinforced failure surface unconstrained, pile at toe-SLOPE/W

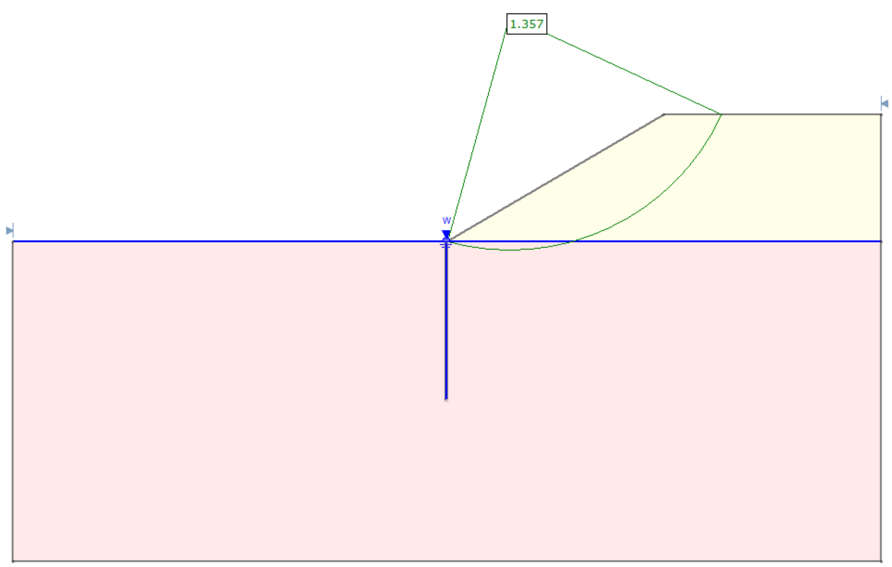


Figure 37. Reinforced failure surface unconstrained, pile at toe-Slide 7

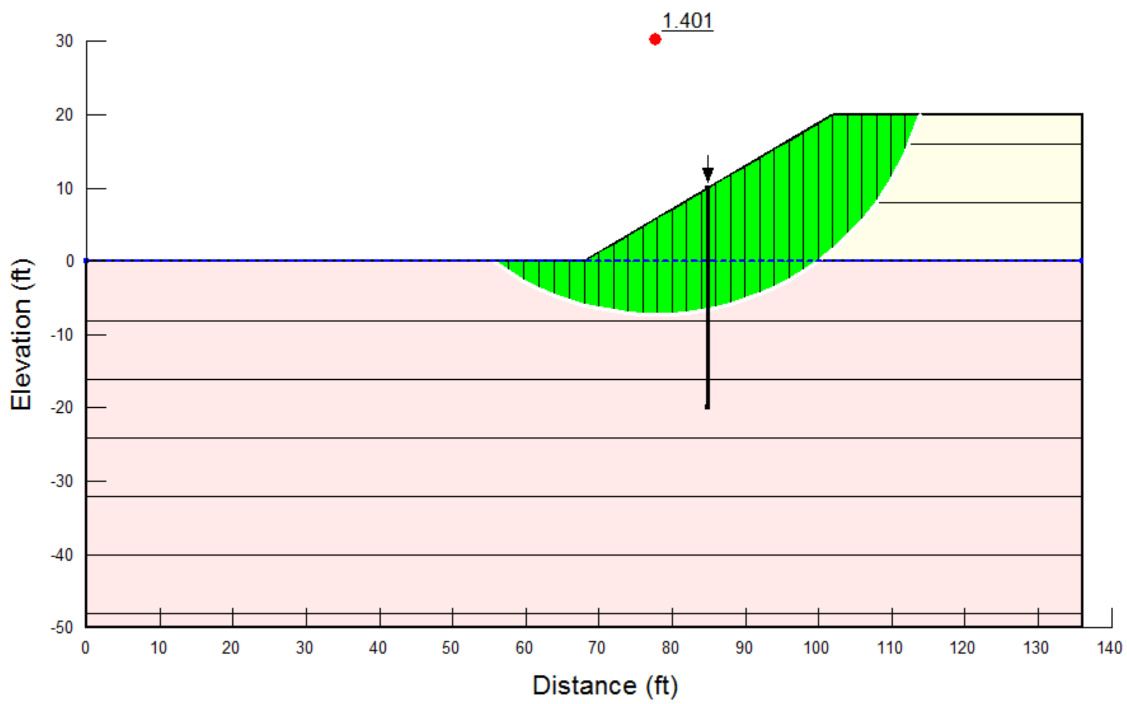


Figure 38. Reinforced Failure Surface Unconstrained, Pile in the Middle SLOPE/W

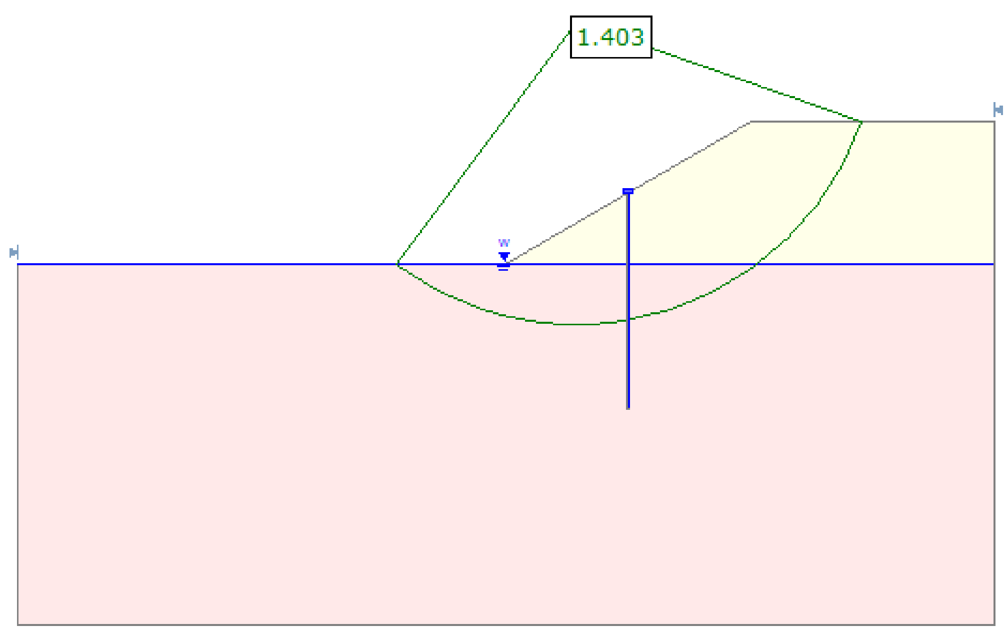


Figure 39. Reinforced Failure Surface Unconstrained, Pile in the Middle Reinforced Slide 7

The final location of the critical surface from the unconstrained reinforced condition was 15.8ft below the top of the pile (approximately 5.8ft from the ground surface). The results show the difference between the assumed sliding depth (9ft) and the value achieved after placing the piles. The magnitude of the stabilizing forces is significantly influence by the depth of the sliding surface.

Table 18. Summary of results reinforced analises to achived F.S. = 1.4

Condition	Shear force (lbs)	Shear force Unfactored (Method A) (lbs)	Pile location
Constrained (SLOPE/W)	6,720	4800	Toe
Constrained (Slide 7)	6,720	4800	
Unconstrained (SLOPE/W)	F.S. not achieved		Toe
Unconstrained (Slide 7)			
Unconstrained (SLOPE/W)	8,540	6100	Middle
Unconstrained (Slide 7)	8,775	6268	

In the next section the optimal pile location and depth will be determined based on the required shear to stabilize the slope to the target factor of safety.

3.3.1.3 Sensitivity Analysis Pile Location and Length (Uncoupled)

Before designing the reinforcing pile, a sensitivity analysis was conducted to verify the optimum location and the optimum length of the pile within the slope. The objective of the analysis is to understand the influence of both parameters in the development of the resistance to achieve the target factor of safety. The analysis was conducted by placing the pile along the length of the slope at different locations from the toe to the top. At those locations the length of the pile was varied from a minimum value to around the total depth of the soil stratum. A diagram of the physical representation of the variable described above is presented in Figure 38. The optimum pair could be determined as the combination location/length requesting the lowest shear force from the reinforcement to achieve the 1.4 factor of safety. However, since the relationship between the stabilizing force/factor of safety is not linear, the analysis was also performed for a 1.5 factor of safety. The summary of the results is presented in Tables 19 and 20.

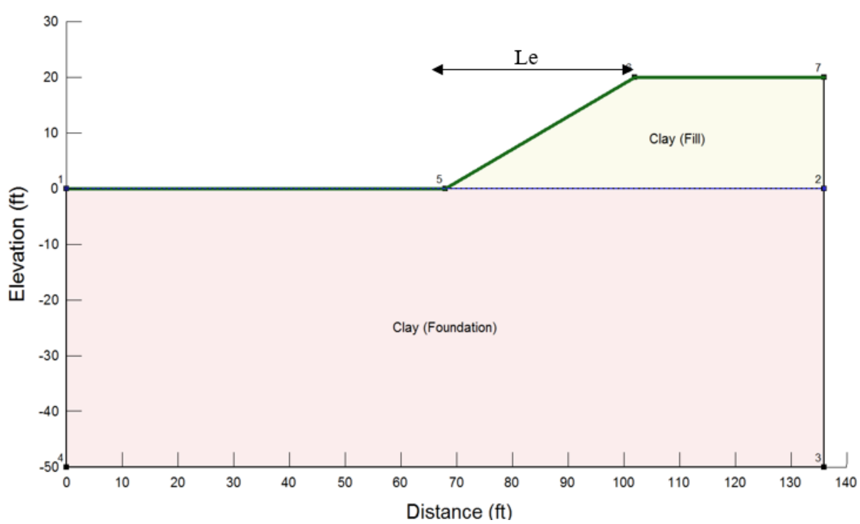


Figure 40. Length of the slope "Le"

Table 19. Summary Required Shear (lbs) for achiving F.S.=1.4

Location	Pile length (ft)							
	16	20	24	26	28	30	35	40
Toe	Pile in the toe does not reach F.S.=1.4							
Le/4	NR*	8,700	8,700	8,700	8,700	8,700	8,700	8,700
Le/2	NR	NR	NR	8,750	8,750	8,750	8,750	8,750
(3/4)Le	NR	NR	NR	NR	10,500	10,500	10,500	10,500
Top	NR	NR	NR	NR	11,000	11,000	11,000	11,000

*NR means F.S. not reached
*Ultimate

Table 20. Summary Required Shear for achiving F.S.=1.5

Location	Pile length (ft)							
	16	20	24	26	28	30	35	40
Toe	Pile in the toe does not reach F.S.=1.5							
Le/4	NR*	NR	NR	13,600	13,600	13,600	13,600	13,600
Le/2	NR	NR	NR	NR	NR	13,500	13,500	13,500
(3/4)Le	NR	NR	NR	NR	NR	NR	14,400	14,400
Top	Pile in the top does not reach F.S.=1.5							

*NR means F.S. not reached
*Allowable

From the results presented in Table 19 and 20, locating the pile at the toe does not represent a favorable condition since the target factors of safety could not be reached even at higher shear values. In the same way, the results also show that for relative higher improvement in the F.S., locating the pile at the top also represents an unfavorable condition. The highest F.S. at the top for high shear values was 1.43. Pile lengths with embedment depth below 20ft (from the horizontal ground to the pile tip, ratio sliding depth/pile length below 0.3 for F.S.=1.4 and 0.45 for F.S.=1.5) also fail in achieving the target F.S.s. The results indicate that every pile length will reach a maximum factor of safety no matter the shear capacity. Thus, to increase the factor of safety in that situation the total length should be also increased.

Furthermore, piles located around Le/4 and Le/2 with embedment length equal or greater that 20ft yields the most favorable results for both cases. However, the minimum required shear force from the reinforcement was found when the pile is located at Le/4 for pile length equal or larger than 20ft (for F.S.=1.4). The same configuration did not meet the requirements for F.S.=1.5 indicating again that an appropriate embedment length must be assured to increase the factor of safety. When piles are located from the toe to the top, the total length must be increased so that the minimum embedment depth is assured. Thus, the optimum combination for both factors of safety is considerably different. On one hand, the optimum position will be at a distance around the quarter of the slope, with very small difference with the values from the middle of the slope, when the shear wants to be minimized at smaller pile lengths. On the other hand, the optimal location of the pile would be in the middle of the slope when higher factors of safety are targeted.

To graphically understand the relationship between F.S. and shear loads to select the condition that maximize factor of safety and minimize the shear loads, the optimum pile lengths from the results in Tables 19 and 20 are used to determine several F.S./loads and have been plotted presented in Figure 41. Moreover, when the optimum length of a pile for x condition has been reached, increasing the embedment length will no produced any favorable effect on the required shear to achieve a desirable factor of safety (if the pile length is increased beyond x value but the shear is kept constant).

From combining Tables 19 and 20 with Figure 41 the following trends can be seen, for small improvement on the factor of safety, all pile locations will yield favorable results. For instance, From Table 19 no shear force was enough to achieve the target factor of safety (1.4). This is because the maximum improvement that could be reached at that location is a F.S. around 1.357 that is very close to the unreinforced value. Furthermore, when a larger improvement on the factor of safety is desirable, the toe and the top are not good candidates regarding pile location. According to the results, piles located around the Le/4 and Le/2 will optimize the shear factor of safety relationship. For the characteristic of this problem (clay with friction and cohesion), the best option is L/4 since a smaller pile length will help to achieve the desirable value almost at the same rate of improvement of the piles located at L/2 (the middle of the slope).

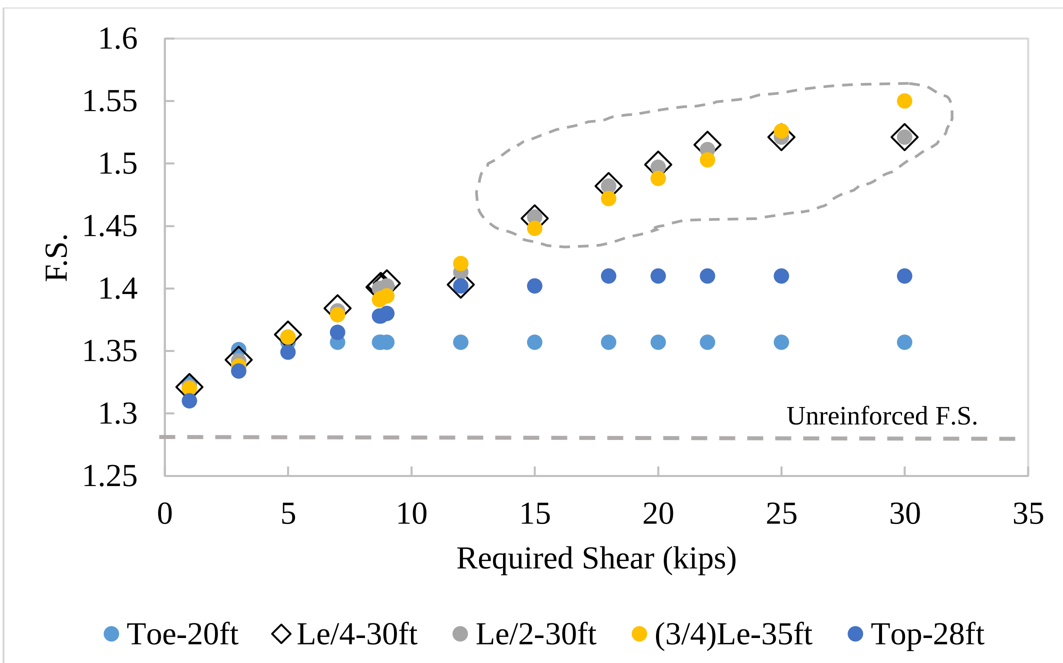


Figure 41. Factor of Safety vs Required Shear

Assuming piles with different length and same shear, Figure 42 has been plotted. The results indicate the most favorable location of the pile within the slope is as presented in Figure 41, but summarizing the pile location influence in the factor of safety. Different lengths with the same shear producing the same factor of safety indicate that the shear must be increased to increase the factor of safety, not increasing the pile length will affect the F.S. beyond that length as presented by Figure 42, for the pile 30ft length, the required shear should be increased to improve the factor of safety. Then, at x length after x shear value no improvement in the F.S. will be achieved, in the same way, at x shear after x length no improvement in the F.S.

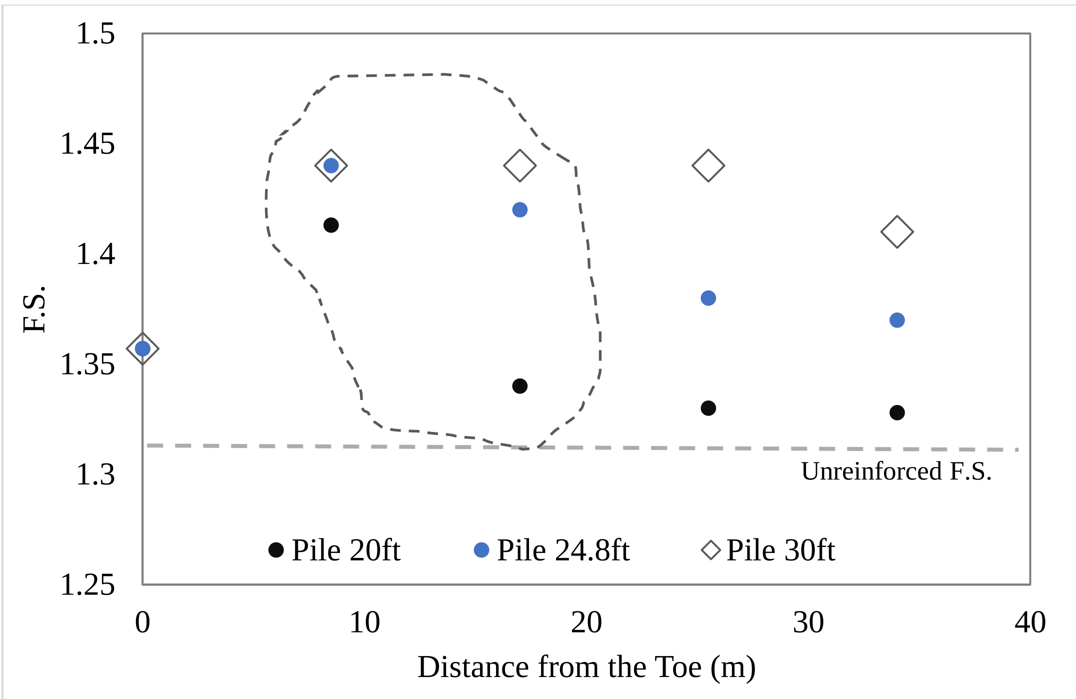


Figure 42. Factor of safety vs Pile Location

The same analysis was conducted assuming the following conditions; (1) the soil stratum is homogenous with the foundation clay properties, (2) condition one but increasing the cohesion to 400psf, (3) condition one, cohesion decreased to zero (cohesionless), (4) original configuration, purely cohesive (zero friction angle), (5) original properties increasing both cohesion and friction angle, and (6) original properties increasing friction angle. Results are presented in Figure 43.

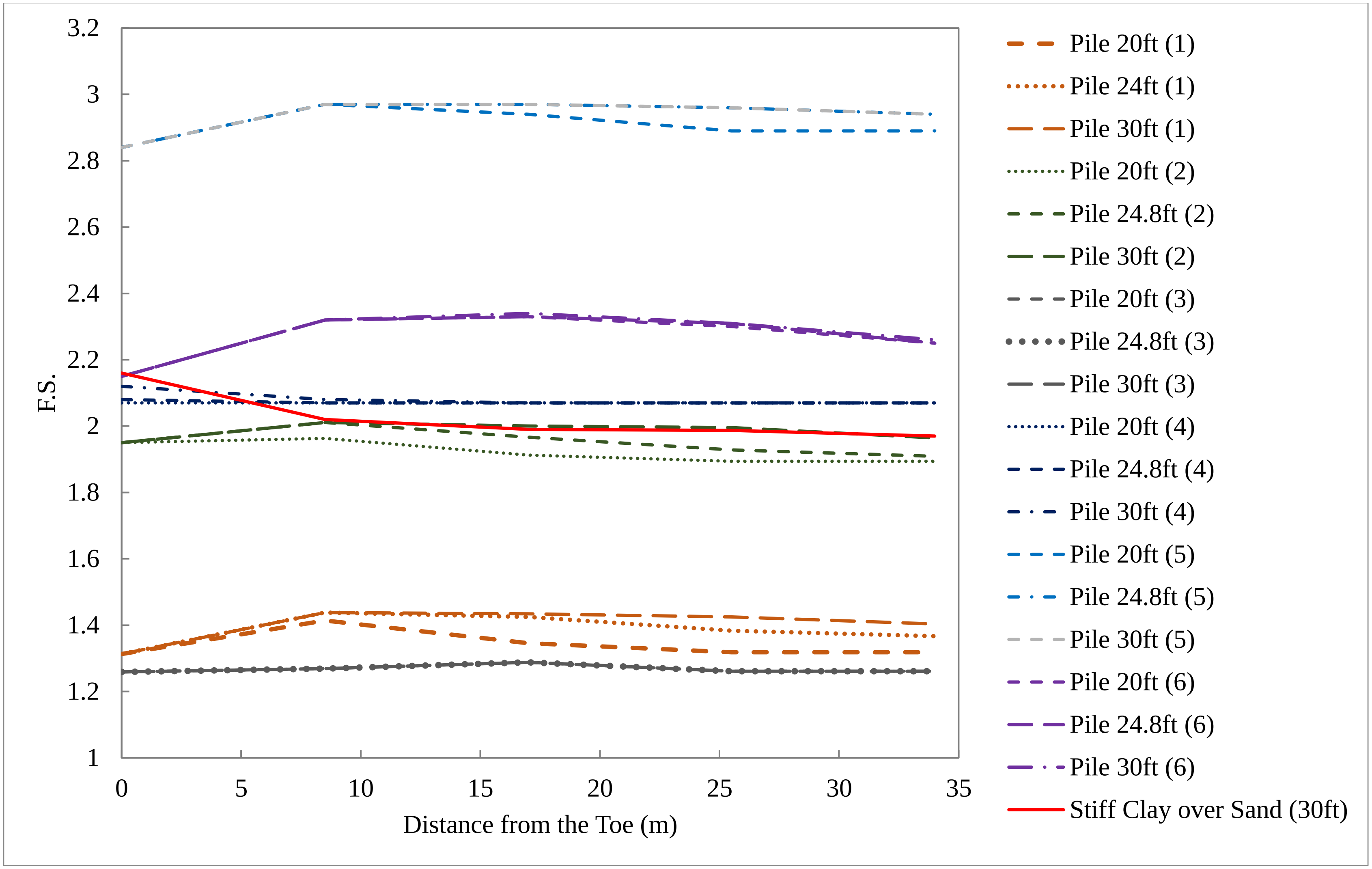


Figure 43. Pile Location vs Factor of Safety with variation in Soil Properties (Uncoupled)

The results strongly indicate that the optimum location of a pile within a slope depends on the properties of the materials. For soils predominantly cohesive, the difference in the factors of safety for piles at different locations along the slope length is considered minimal. For purely cohesive soils the same trend is followed with a slightly higher factor of safety for the pile located at the toe (4 and clay over sand cases). For soils predominantly cohesionless, the optimum location changes to around the quarter and the middle going to the upper portion of the slope as the friction increases. In addition, the differences in the factors of safety for the various locations become significant as the friction angle increases and smaller as the cohesion increases. Thus, for combined materials (with friction and cohesion) the differences in the factors of safety will be controlled by the dominant parameter. This could be related to the strength of the materials since frictional materials are generally stronger than cohesive materials.

As mention in section 2.4.2, for a soil with a large cohesion and a small friction angle, the most critical failure surface will be a deep circle, while shallow circle will be the critical slide surface for a soil with a small cohesion and a large friction angle material (Huang 2014, NAVFAC (1986). Hence, as cohesion increases the critical circle moves deeper. In contrast, as friction angle increases, the critical circle moves upward making certain places of the slope (like the toe) less suitable for the pile location.

The results show that if the material within the slope is sandy soil the optimum pile location is clearly the middle (a shallow critical surface is developed). In contrast, for the opposite condition the optimum location will be the toe (slightly higher factor of safety than the other locations). For a slope with sand and cohesive foundation, the critical circle depends on the consistency of the cohesive material. Sand over soft clay will develop a deep critical circle and sand over stiff clay will most likely develop a shallow sliding surface. Thus, for the first case the optimum location will be around the toe and for the second one from the middle to the top. This is consistent with the findings for a slope with stiff clay over soft clay and vice versa. The deepest critical circle generated through the weakest soil layer will generally require the piles to be located around the toe.

For the uncoupled analysis, it can be said that because most soils in nature are combined, the weakest material will govern the failure surface position. Small values of cohesion in a material may suggest that the dominant behavior will be frictional and that the pile optimum location will range around the quarter and the middle moving upslope with higher values of friction angle. Cases 1, 2, and 5 show that increments of 100psf in cohesion reduces the gap between factors of safety for piles a different distances from the toe, suggesting that for cohesive soils a pile could be located in any place, with the toe a slightly better location. This allows for constructability freedom where the optimum pile location could be selected in terms of the convenient location regarding access, equipment, previous experience of contractors, among others. The presence of stiff cohesive materials changes the optimum location of a pile regarding the location of the stronger material (stiff material in the top, the pile should be placed around the toe, stiff clay in the foundation, pile location around the middle and upper portion).

A summary regarding the key findings from the sensitivity analysis is presented in Table 21. The trends suggest that overall the optimum location of a pile depends on the elevation of the sliding depth which at the same time depends upon the consistency of the materials within the slope. For the case of cohesive materials, the improvement on the F.S. among locations is considerably small and engineering judgement must be used to determine the most convenient location in terms of constructability.

Table 21. Summary Optimun Pile Location

Slope characteristics	Critical surface	Pile location
Soft clay over stiff clay Soft clay over sand	Shallow critical circle	From the quarter to the upper portion
Stiff clay over soft clay Sand over soft clay	Deep critical circle	Slightly better the toe and the top.
Purely cohesive (homogeneous)	Deep critical circle	Any location (slightly better the toe)
Purely frictional (homogeneous) Sand over stiff clay	Shallow critical circle	Middle
Friction and cohesion (predominantly frictional properties)	Intermedia (verify the weakest layer)	Generally from the quarter to the upper portion

3.3.1.4 Designing Resisting Pile

To analyze the resisting pile two approaches will be followed, the shear force from the stability analysis is applied to the pile as distributed load following the procedure by ODOTO (2011) described in section 3.2.2.5, for (1) the constrained and (2) the unconstrained conditions. To replicate the original conditions, the pile will be design for the constrained analysis assuming the sliding depth at 9ft and then, the results will be compared to the unconstrained analysis at which the pile was located in the middle of the slope.

The soil properties presented in Table 17 will be used to develop the p-y curves and correlated when needed. The soft clay (Matlock) model will be used to represent the clay stratum for this problem. The pile is modeled as a round concrete shaft with permanent casing (bored pile). The pile properties are presented in Table 22.

From the stability analisys, the sliding surface will be taken to be around 9ft below the pile top. Hence, the moving soil will consist on both the clay fill and the clay (foundation) as presented previously in Figures 34 and 35.

Table 22. Properties Concrete Pile LPILE Analisys Problem II

Pile diameter	1.5ft
Pile length	30ft
Pile spacing	4.5ft
Yield Stress (Reinforcement)	60,000psi
Elastic Modulus (Reinforcement)	29,000,000psi
Reinforcement equivalent area	2.48% of Ac

For the lateral analysis, the unfactored loads from the LE analyses will be used. The results from this analysis are presented in Table 24 along the results from the second approach.

For the second approach the shear force will be used to determine the distributed lateral load at the failure plane (P_h) to perform the lateral pile analyses as presented by Zicko et al. (2011) from a method provide by The Ohio Department of Transportation (ODOTO 2011).

Table 23 presents the information from the stability analysis and the corresponding values for P_h.

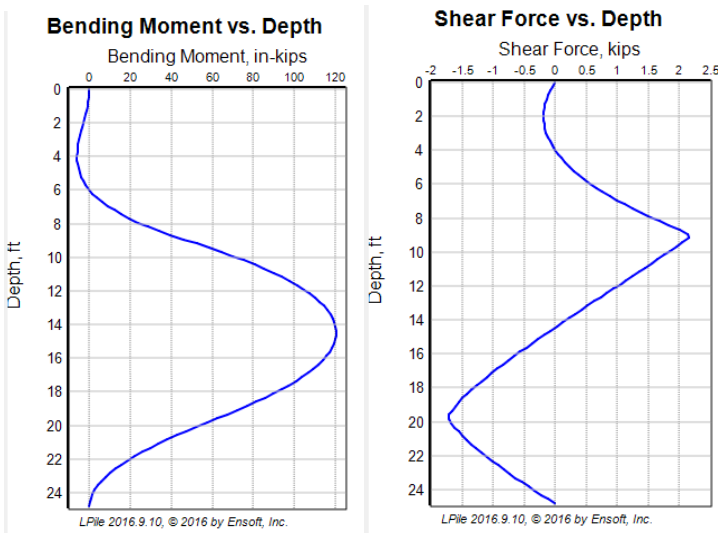
Table 23. Distribution of Lateral Loads for Pile Design Problem III

Variable	Value	Value
F_h	1.1kips	1.4kips
S	4.5ft	4.5ft
R	1 rows	1 rows
L	9ft	15.8ft
P_h	92lbs/in	69lbs/in

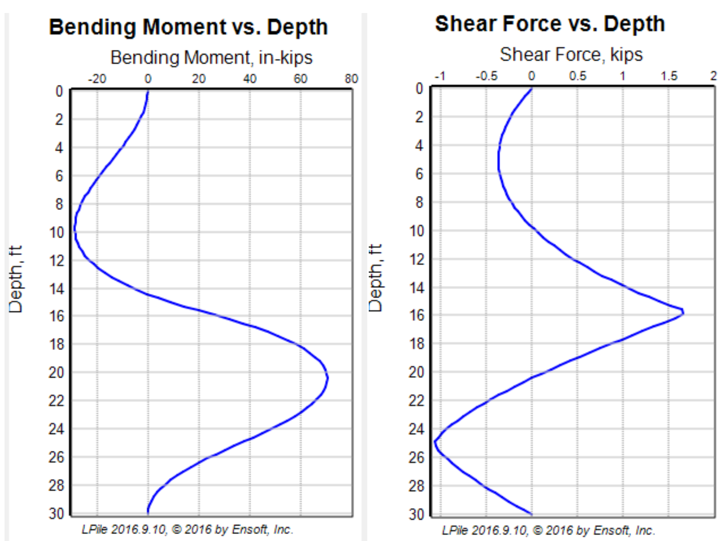
Table 24 presents the results of pile analysis for both approaches. The pile structural integrity was verified from the results presented below. The shear and moment diagrams from the lateral stability analyses are presented in Figure 44.

Table 24. Results of Pile Analyses for Strength Limit State

Parameter	Distributed load (Restrained to 9.0ft)	Distributed load (Unrestrained)
Nominal resisting moment of pile	2,321kip· in	2,321kip· in
Maximum applied moment	121kip.in	70.5 kip.in
Resisting shear force	21.3kips	21.3kips
Maximum shear force	2.2kips	1.7kips



Pile at the Toe-failure surface 9ft



Pile at the middle-Failure surface 15.8ft

Figure 44. Bending Moment and Shear from Lateral Analysis

3.3.2 Coupled analysis

For this analysis, the unreinforced analysis will also be performed to verify if the target factor of safety is reached (as presented in section 3.3.1.1). The reinforced analysis will then be conducted after placing the reinforcing pile in both positions the toe and the middle of the slope as in the uncouple analysis.

3.3.2.1 Unreinforced Analysis

The unreinforced factor of safety for this condition has been determined from the 2-dimensional elasto-plastic finite element analysis by the shear strength reduction method using Phase2. Additional results are also presented from the analysis in the finite difference program FLAC.

Figure 45 presents the factor of safety and the location of the critical failure surface from the LE analysis in superposition with the critical circle from the LE analysis performed in Slide 7.0 presented in section 3.3.1.1. The results show high compatibility between the location of the critical circle and the factors of safety. For the LE analysis in Slide 7.0, F.S.=1.311 while from Phase2 the SSR was 1.29.

Furthermore, the results from the FD analysis performed in FLAC are presented in Figure 48. The deformed mesh indicates that the location of the critical circle is also consistent with the results from Phase2.

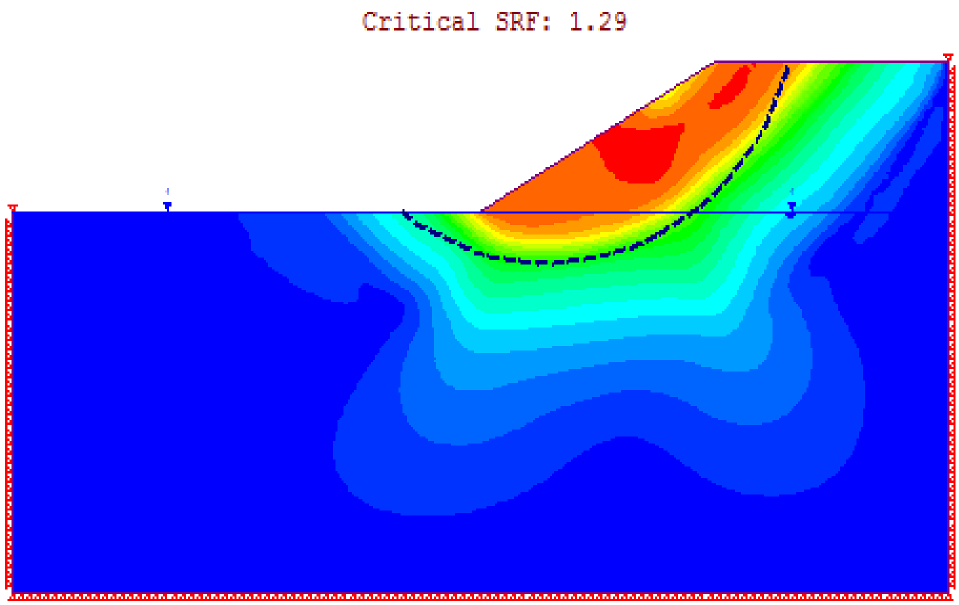


Figure 45. Superposition Unreinforced Analysis Phase2 and Slide 7.0

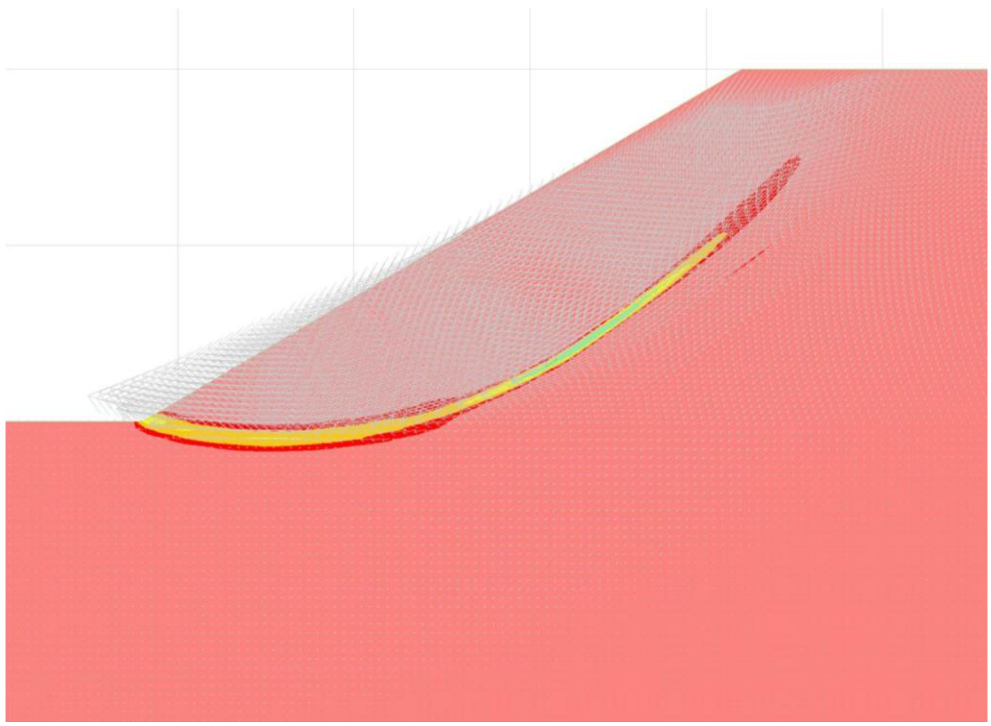


Figure 46. Unreinforced Analysis FLAC (F.S.=1.33)

3.3.2.2 Reinforced Analysis

In phase2, the piles were modeled using structural interfaces with a joint on either side of the reinforcement (liner)allowing slip to occur between the reinforcement and the soil (Rocscience Inc. 2017). Pile properties for both FE and FD analysis are presented in Table 25. The soil properties are the same from the LE analysis.

Table 25. Properties SSR Analysis Phase2 and FLAC

Parameter	Magnitude
Pile length	30ft
Pile spacing	4.5ft
Pile diameter	1.5ft
Young's modulus	3605(ksi)
Poisson's ratio	0.15
Yield Strength (f'c)	4000psi
Tensile strength	350psi

The results are presented in Figures 47 and 48 for the pile placed at the toe. To verify which one would be the optimum pile location a sensitivity analysis will be presented in the next section. However, Figures 49 and 50 present the results for the pile placed in the middle. A considerable difference is found between the results from FLAC and Phase2, additional analysis should be done to determine if the boundary conditions when the pile is located at the middle of the slope for both analysis have been considered the same (either free in both sides of the slope or restricted below or above the pile).

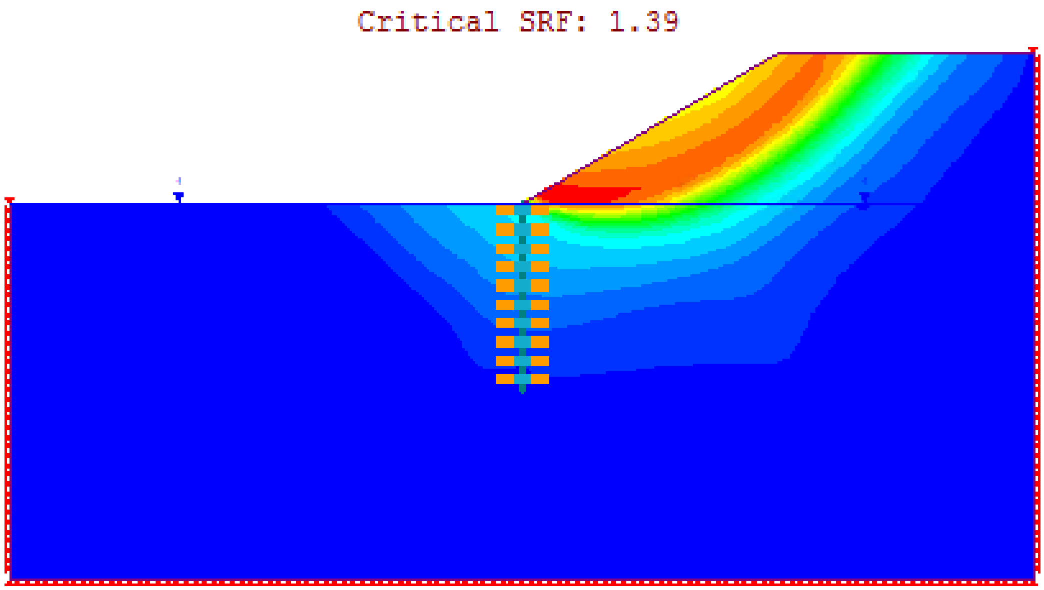


Figure 47. Reinforced Analysis Phase2 Pile in the Toe

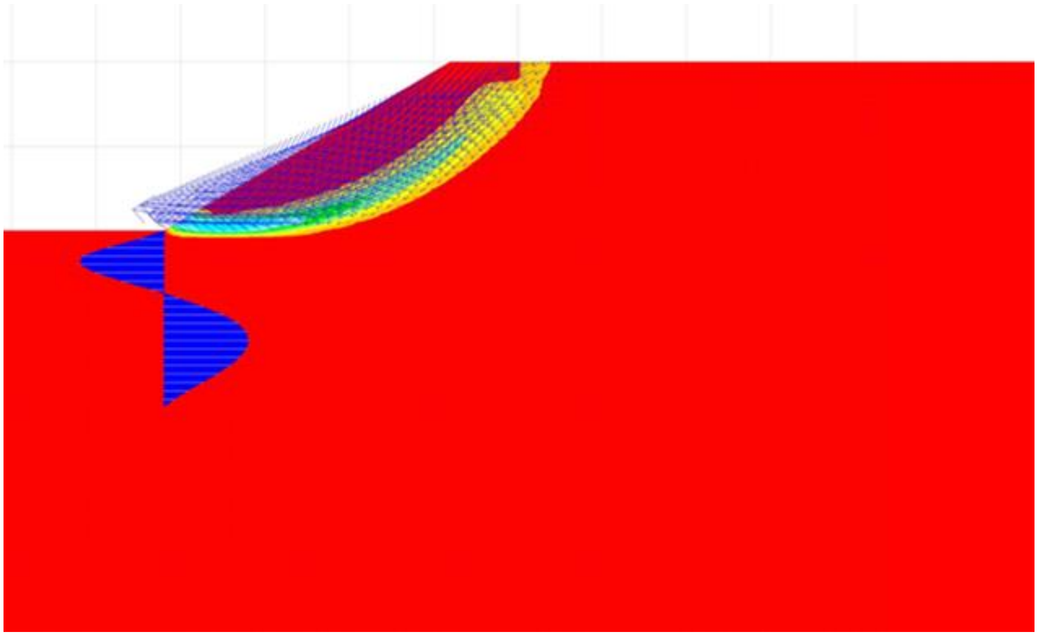


Figure 48. Reinforced Analysis FLAC Pile in the Toe (F.S.=1.35)

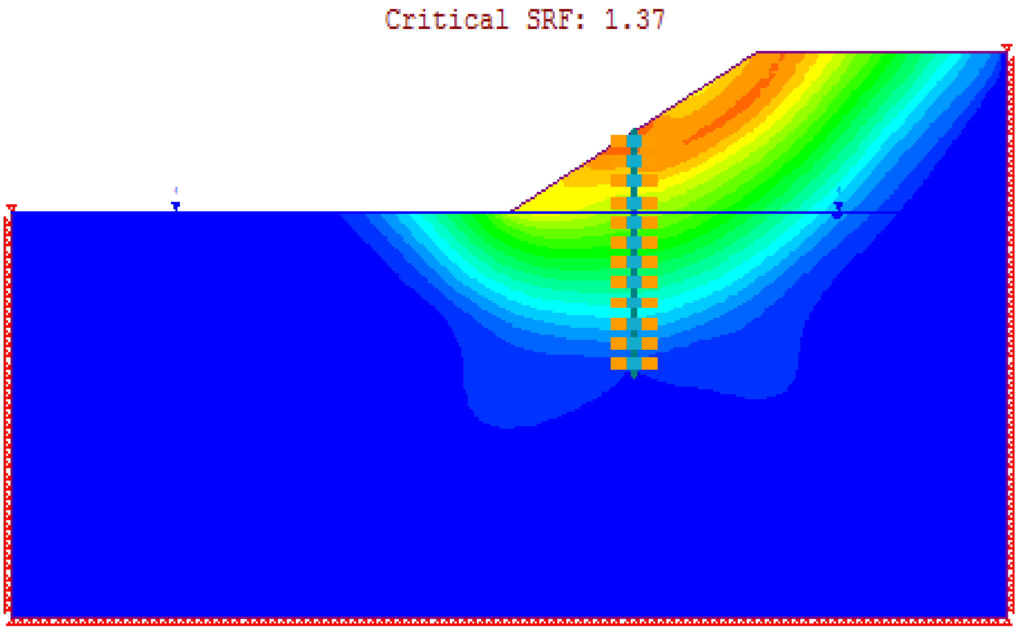


Figure 49. Reinforced Analysis Phase2 Pile in the Middle

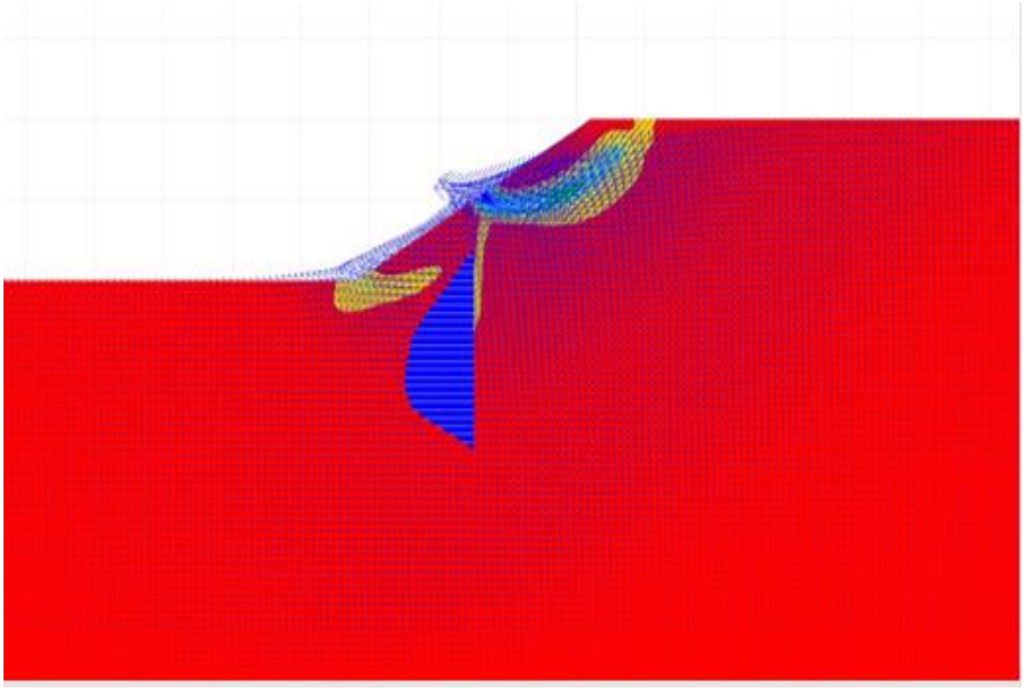


Figure 50. Reinforced Analysis FLAC Pile in the Toe (F.S.=1.99)

3.3.2.3 Sensitivity Analysis (Coupled)

As in the uncoupled analysis, for the coupled one, the pile will also be located at different locations within the slope to verify the optimum location from FE analysis. Conditions considered include: (1) the soil stratum is homogenous with the foundation clay properties, (2) condition one but increasing the cohesion to 400psf, (3) condition one, cohesion decreased to zero, (4) original configuration and purely cohesive and (5) original properties increasing both cohesion and friction angle, and (6) original properties increasing friction angle, will also be analyzed. The results are presented in Figure 51 for the original problem and Figure 52 for the additional conditions.

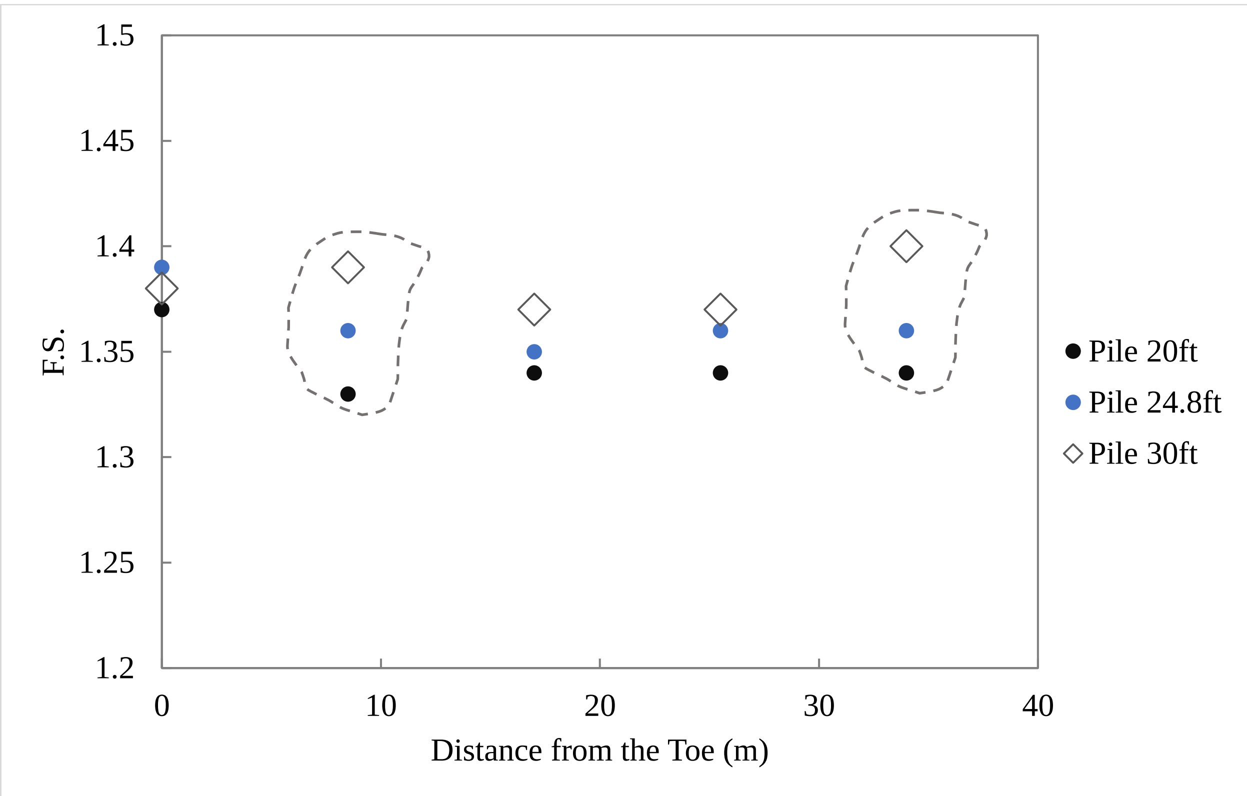


Figure 51. Pile Location Versus Factor of safety

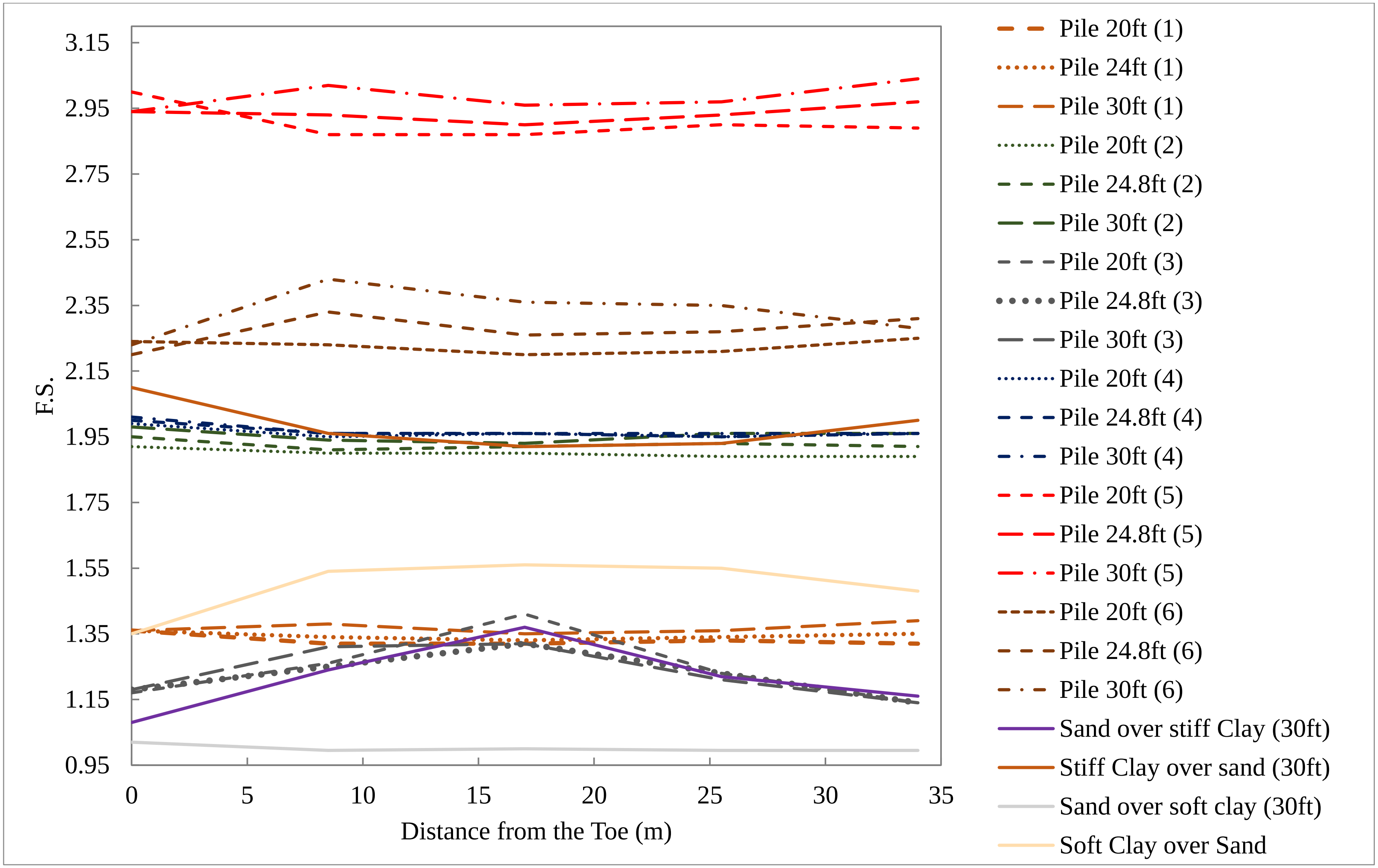


Figure 52. Pile Location vs Factor of Safety with variation in Soil Properties (Coupled)

The results from Figure 51 indicate that the factor of safety variation with pile location is considerably small being the around the toe and the top the slightly better locations and the quarter of the slope for larger piles. The findings are partially consistent with the results from section 3.3.1.3, indicating that from FE analysis the material is predominately cohesive. From the additional analyses, the same trends of the uncoupled analysis are seen. For purely frictional slopes the optimum location will be around the middle, for purely cohesive slopes the pile could be located at any place being the toe the slightly better condition. For materials with friction and cohesion, the optimum location will be from the quarter of the slope to the upper portion if frictions govern and any location of the slope if cohesion increases.

The results from both analyses, coupled and uncoupled, relate with the previously published conclusions from several authors presented in Table 5. The authors specified different pile locations regarding the case they were presenting. Comparing the individual results to the present study, the relationship between pile location and soil type become evident. For the cases where the soil correspond to combined with friction and cohesion (with dominant frictional behavior) the optimum location was reported mostly from the middle to the upper portion, and for cases were the soils where predominately cohesive several authors consistently recommended the pile to be located at either the toe or the top. Most authors indicate the middle of the slope as the optimum location because of the characteristics of the materials within the slope they were studying (generally intermediate materials with relatively lower cohesion and average friction angle).

Additional research must be conducted to verify how the factor of safety responses to the simultaneous increment of cohesion and friction can be independently characterized. In addition, the results from the present study should be validated from parametric studies conducted in others finite element or finite difference programs.

3.3.3 Summary Results Coupled and Uncoupled Analyses Problem II

An overall summary is presented in Table 26. The ultimate factor of safety refers to the value after performing stability analysis with the shear values from LPILE (for uncoupled analysis). Thus, for the coupled analysis, the value will be original reinforced factor of safety. The results are normalized by 1ft width of soil.

Table 26. General Summary Problem II

Condition	Unreinforced F.S.	Reinforced F.S.	**Required Shear (kips)	*Shear at the Sliding depth (kips)	Max. Moment (kips.in)	Ultimate F.S.
SLOPE/W-9ft	1.354	1.4	1.1	0.4	89	1.37
Slide 7-9ft	1.357		1.1			
SLOPE/W-Middle	1.316	1.4	1.4	0.4	70.5	1.34
Slide 7-Middle	1.311		1.4			
Phase2 (Toe)	1.29	1.39	-	1.2	146.4	1.39
Phase2 (Middle)		1.37	-	1.3	80.1	1.37
FLAC (Toe)	1.33	1.35	-	0.1	31	1.35
FLAC (Middle)		1.99***	-	3.6	248	1.99

**The values presented come from LE programs
***The values presented come from either LPILE for uncoupled analysis or FE, FD programs
*** Should be verified

For the unreinforced analyses, all methods of analysis achieved around the same factor of safety. Thus, LE, FE, or FD programs are comparatively recommended regarding accuracy and reliability. The influence of assuming a fixed critical surface can be seen. However, the difference between the constrained and unconstrained values is practically negligible with the F.S. from the assumed depth slightly more conservative.

For the reinforced analyses, the shear resistance developed by the pile and the factor of safety can be compared. On one hand, the shear resistances from LPILE (for uncoupled condition) are considerably different from the values from the Phase2 and FLAC. However, the ultimate factors of safety from coupled and uncoupled analysis are similar. As in section 3.2.3, the ultimate factor of safety of an uncoupled analysis was found lower than the target factor since the resistance of the pile was not developed a hundred percent. The results from FLAC when the pile is placed in the middle of the slope must be investigated in more detail because of the larger difference between shear, moment and factors of safety with the other conditions.

Overall, the ultimate factors of safety are similar. Thus, performing a coupled or uncoupled pile reinforced slope analysis will yield similar results.

For the optimum pile location, by comparing the results from Sections 3.3.2.3 and 3.3.3.3, the coupled and uncoupled conditions produce similar results. Piles must be located with consideration of the critical sliding depth. For soils in which friction is predominant, the optimum location will be around the middle and the upper portion of the slope if adequate embedment depth is assured. To assure that the embedment is adequate a sensitivity analysis should be conducted. Piles located from the middle to upper portion will require larger lengths to maintain the embedment length.

3.4 Problem III-The Mill Creek Landslide (Mitigation)

This case corresponds to a landslide developed along the lower portion of the southbound embankment of State Route 15 contiguous to the Tioga Reservoir in north central Pennsylvania. The Pennsylvania Department of Transportation observed the development of scars after heavy rains and snow melting in spring 2011 (Zicko et al. 2011). Because of long-term stability concerns to a section of the roadway, several alternatives were studied to stabilize the slide, with driven steel H-piles selected as the best alternative, based on construction costs and site impacts.

The report by Zicko et al. (2011) presents all the information regarding the site history, the surface conditions, performed lab testing, stability analysis and assessment of the alternatives. According to this information, the soil in the area corresponded to mainly alluvial and colluvial sands with the potential glacial lake sediments. Moreover, the embankment material was determined to be silty sand and gravel with density from medium to very dense. A cross section of station 1303+00 is presented by Figure 53.

As part of the exploration program, seventeen borings were drilled, inclinometers were installed in ten of those boring logs, and also a subsurface geophysical survey was performed to determine the extension of the soft glacial lakes deposits encountered.

To determine the reinforcement alternative, Zicko et al. (2011) conducted the analyses as uncoupled. The slope stability was performed by using the computer program GSTABL7 with STDwin (Gregory 2005) and the pile design was evaluated using LPILE v5.0 (Ensoft 2005). The preliminary slope stability analyses were conducted with a target factor of safety of 1.0 to determine the soil parameters from a back-calculation process.

The factor of safety after reinforcement with the steel H piles was then targeted it to 1.3 to determine the shear force needed from each pile. In this study, the parameters presented by

Zicko et al. (2011) will be used to performed a coupled and uncoupled analysis. A summary of the soil properties is presented in Table 27.

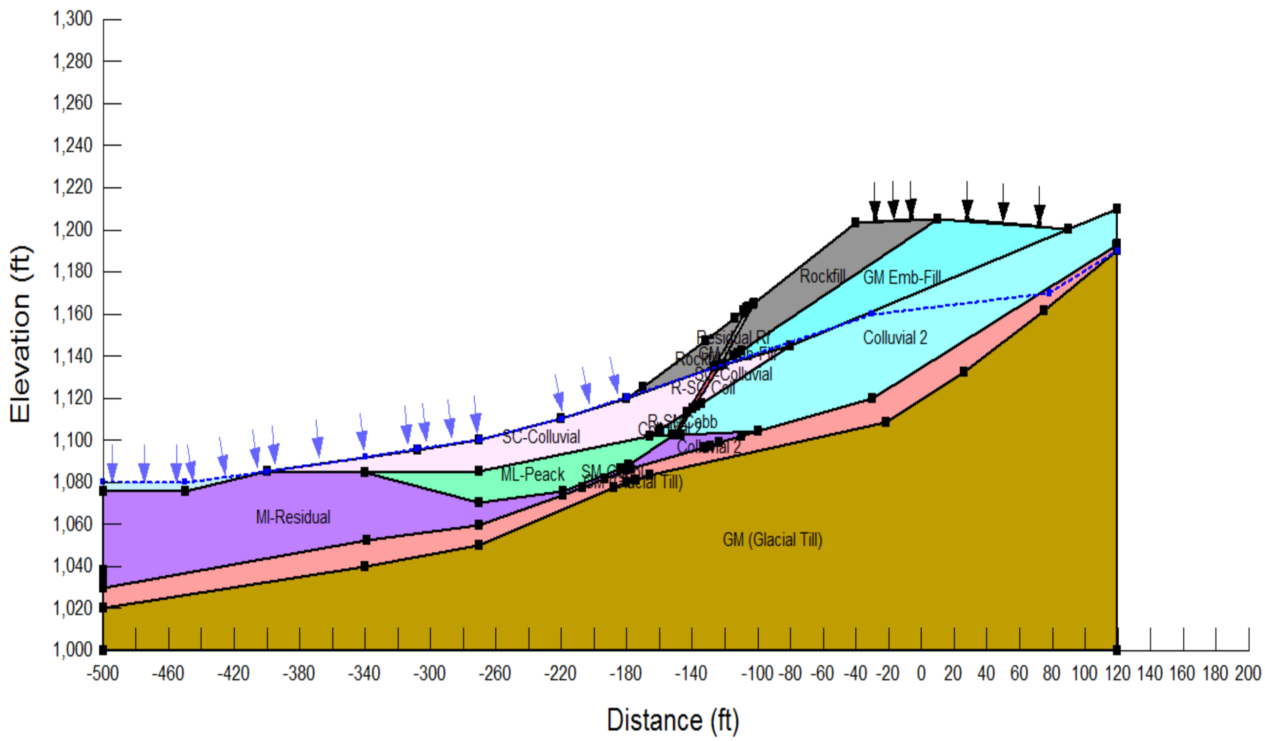


Figure 53. Subsurface Profile along the Critical cross-section at Station 1303+00 after Zicko et al. (2011)

From the laboratory testing, the glacial lake deposit was found to have residual and peak strengths that were fixed to determine the additional soil properties from the back analysis.

Table 27. Soil Properties Mill Creek Landslide after Zicko et al. (2011)

Soil type	Total Unit Weight (pcf)	*c'* (kPa)	Φ' (°)
Embankment Fill	120	0	32
Colluvial	120	0	30
Peak ML	120	0	25
SM cobbl	125	0	36
Residual ML	120	0	16
Glacial	130	0	38
Rockfill	130	0	45
Colluvial 2	120	0	34
Residual Colluvial	120	0	28
Residual SM cobbl	125	0	30
Residual Rockfill	130	0	38

3.4.1 Uncoupled Analysis

For the uncoupled analysis, slope stability and pile analysis are presented. First, the factor of safety of the unreinforced slope is verified to be 1.0 since the soil parameters come from back analysis calculations. Two different analyses will be performed by (1) analyzing the unconstrained slope to verify the potential location of the failure surface, and (2) analyzing the failure surface constrained to the location of the in situ scars (as conducted in the report). The factor of safety will then be targeted to 1.3 following the procedure by Zicko et al. (2011) to determine the necessary shear force from the reinforcing pile. Furthermore, the pile analysis will be conducted.

3.4.1.1 Unreinforced Analysis

As stated before, the first unreinforced analysis will be done to verify the potential location of the failure surface under the given soil conditions, if no in situ information of the scar would have been available. Results from SLOPE/W and Slide 7.0 are presented in Figures 54 and 55 respectively. It can be seen that from both results the values are practically the same. Moreover, the critical slide surface does not follow the in situ scar that is modelled as a small region of residual soils.

Additional information is presented by Figures 55 and 56, where the location of the 1.0 unconstrained failure is shown, since from the general analyses F.S. smaller than 1.0 were found as critical. The location of the failure surface with F.S. approximately one, for both analyses were found to follow the pattern of the constrained analysis.

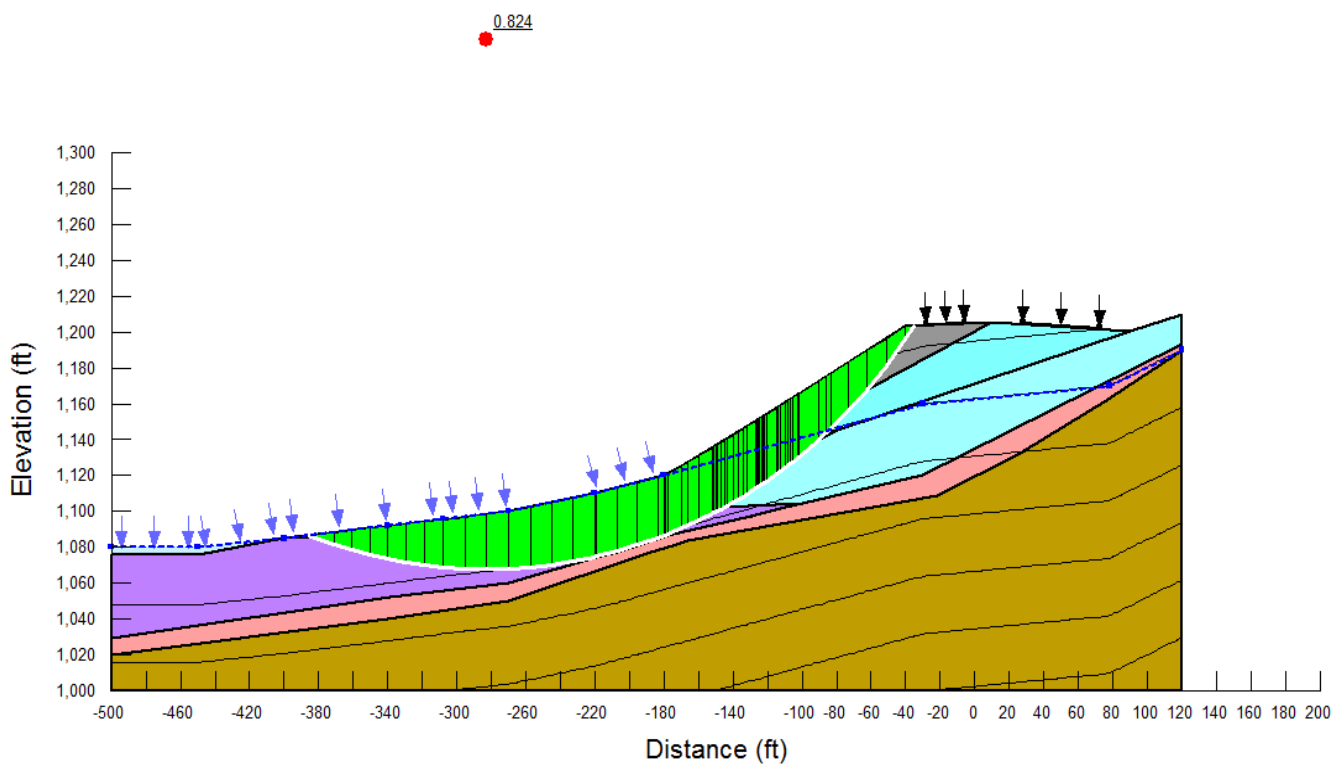


Figure 54. Factor of Safety of Unconstrained Analysis SLOPE/W

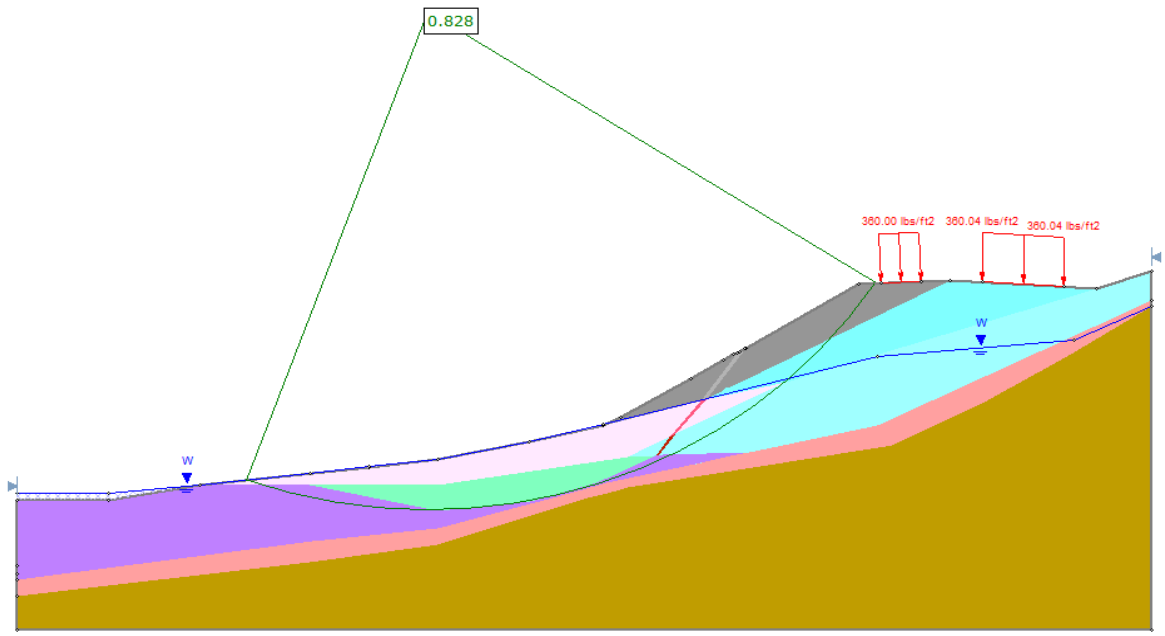


Figure 55. Factor of Safety of Unconstrained Analysis Slide 7.0

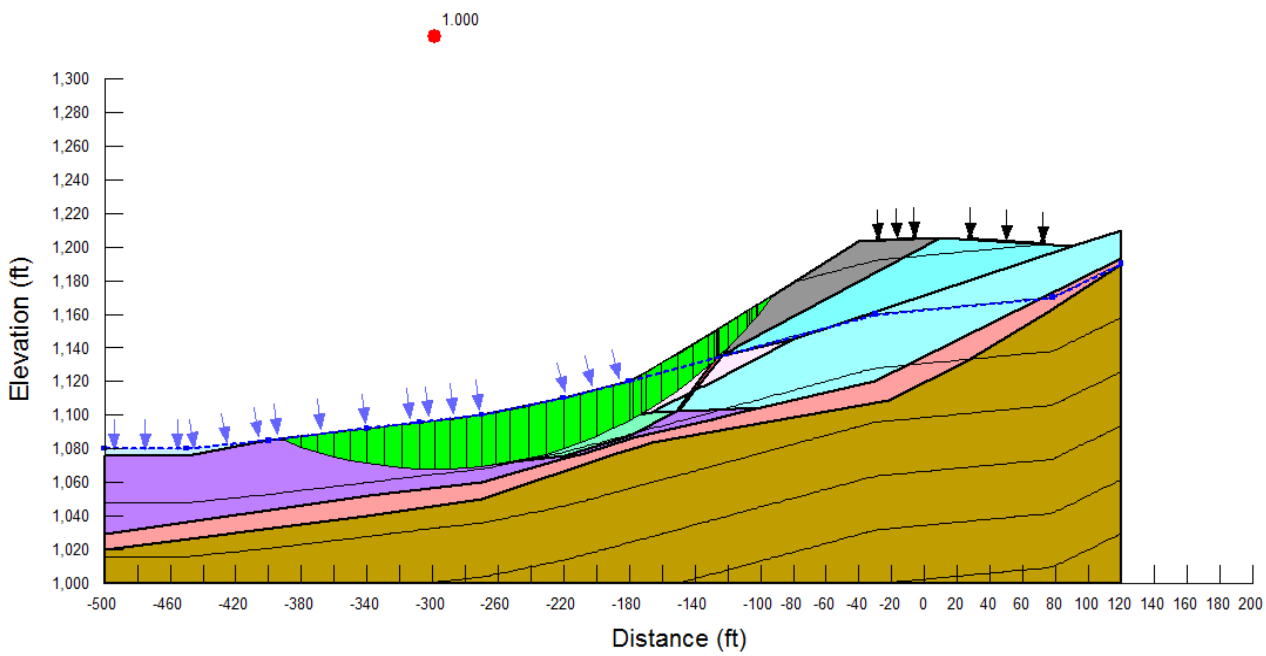


Figure 56. F.S.=1.0 of Unconstrained Analysis SLOPE/W

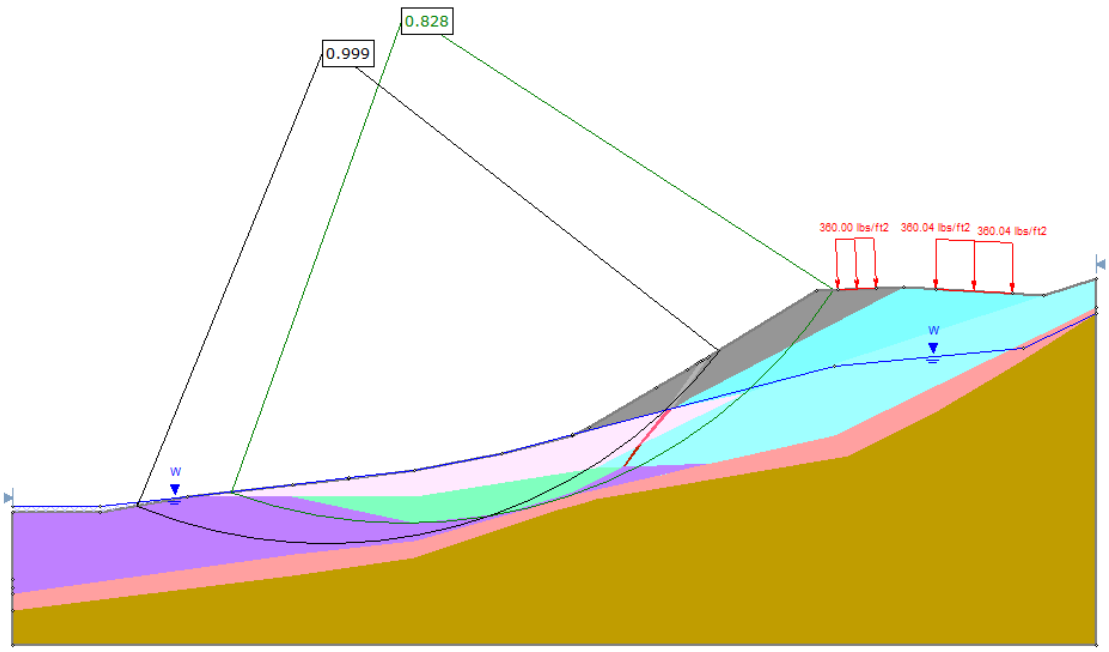


Figure 57. F.S.=1.0 of Unconstrained Analysis Slide 7.0

On the other hand, the results from the constrained analysis are presented in Figures 58 and 59. The target value F.S. equal to 1.0 is achieved by following the scar developed at the time of failure.

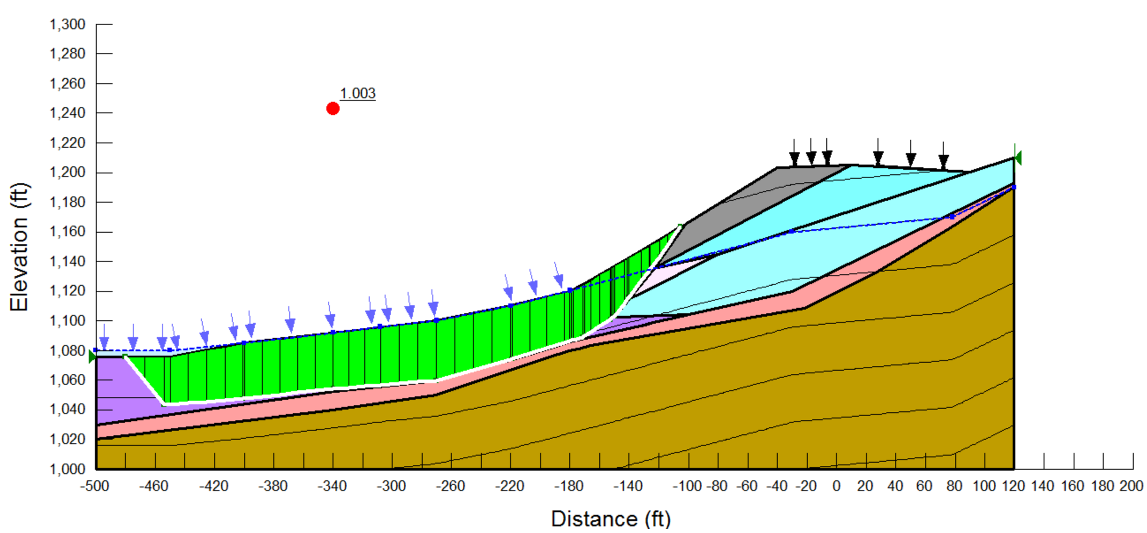


Figure 58. Factor of Safety of Unconstrained Analysis SLOPE/W

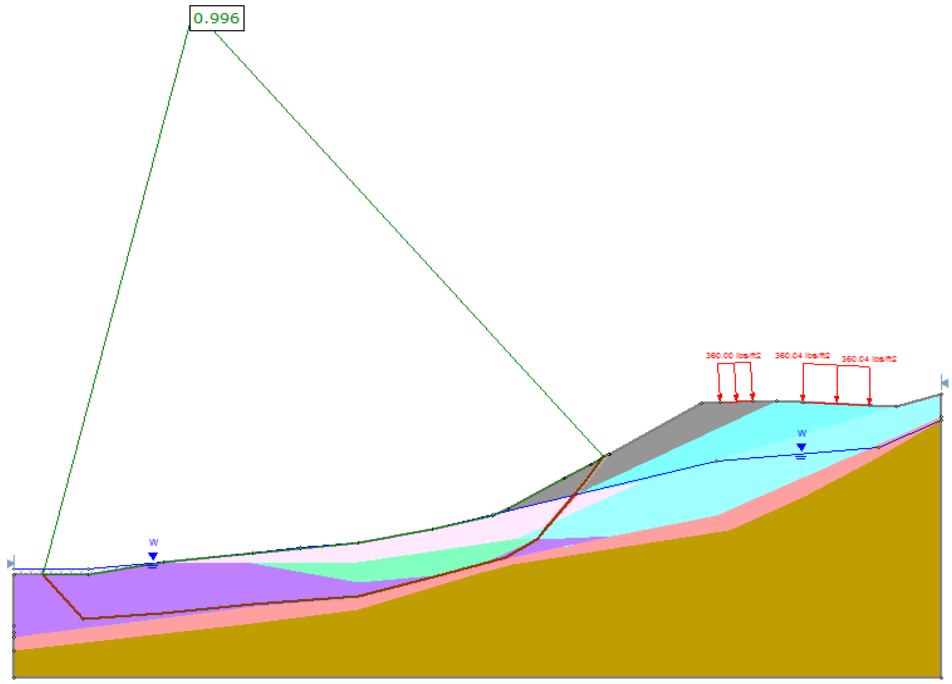


Figure 59. Factor of Safety of Constrained Analysis Slide 7.0

3.4.1.2 Reinforced Analysis

After the unreinforced analysis is conducted, a reinforced analysis with the calibrated to 1.0 factor of safety failure surface is conducted. The target factor of safety is 1.30. The section determined as the most viable option (H-Piles 12x53) was evaluated and the shear forces from the current analyses are compared with the original values. Because the original program **(**GSTABL7 by Gregory (2005)) includes the stabilizing force in the numerator to calculate the factor of safety, the method that will be used to determine the reinforcement load will be "Method B". The target value was achieved for 65 kips for both SLOPE/W and Slide 7.0. The factor of safety was also evaluated for the shear presented by Zicko et al. (2011) 75 kips. The resulting value was 1.32 and because both values are very similar, the design of the pile will be conducted with the previously presented shear (for comparison purposes). Table 28 summarizes the results from the reinforced analyses.

Table 28. Summary of Shear Forces and FOS Problem III

Condition	**Shear force (Kips)**	FOS
Constrained (SLOPE/W)	75	1.310
Constrained (Slide 7)	75	1.305
Original report (**GSTABL7)**	75	1.300

3.4.1.3 Designing Resisting Pile

To analyze the resisting pile two approaches will be followed: (1) the original approach from Zicko et al. (2011) in which the shear force from the stability analysis is applied to the pile as distributed load, and (2) a displacement based approach, applying the soil displacement registered by the installed inclinometers.

For the first approach, the corresponding shear force will be used to determine the distributed lateral load at the failure plane (P_h) to perform the lateral pile analyses as presented by Zicko et al. (2011) from a method provide by The Ohio Department of Transportation (ODOTO 2011) as performed in sections 3.2.1.5 and 3.3.1.4.The pile design was based on load and resistance factor design (LRFD) considering the loads from the stability analysis as factored loads for the strength limit state.

Table 29. Distribution of Lateral Loads for Pile Design Problem III

Variable	Value (Original report)
F_h	75kips
s	6ft
r	4rows
l	35ft
P_h	536lbs/in

The lateral resistance of the pile using p-y models was reduced by p-multipliers to account for the installation of the piles within an active landslide. The p-multiplier value was 0.231 as suggested by Zicko et al. (2011).

Table 30 presents the results from the pile analyses for strength limit state performed in LPILE for both the original study and the current one. The pile structural integrity was verified from the results presented below. The factored resisting moment of pile was found to be 2,106 kip· in, while the maximum applied moment on pile was 1,788.4kip.in. On the other hand, the factored resisting shear force of the pile was given as 98 kips and the maximum applied shear load on pile was 44.0 kips. The shear and moment diagrams from the lateral stability analysis are presented in Figures 60 and 61.

Table 30. Results of Lateral Analyses For Strength Limit State

Parameter	Previous study (LPILE v5.0)	Present study (LPILE v09.010)
Resisting moment of pile*	1,983kip· in	2106kip· in
Maximum applied moment	1,740.0kip.in	1,740.4kip.in
Resisting shear force*	98kips	98kips
Maximum shear force	41.0kips	41.8kips

*Determination of nominal values is presented in Appendix II.

For the second approach, the readings from the inclinometer installed at borehole 8 have been used to perform allowable strength design (ASD). This is because in LPILE 09.010 the analysis of loading by soil movements is only currently possible by conventional analysis. Hence, the out of balance shear force (non-factored load) will be 46.8Kips.

A summary of the results is presented in Table 31 while the shear and moment diagrams from the lateral stability analysis are presented in Figure 62. A comparison between maximum and allowable values is made to verify that the structural factors of safety meet the specifications. The factor of safety for elements under shear forces is 1.50 while the factor of safety for the allowable moment capacity is 1.67 according to American Institute of Steel Construction (2016).

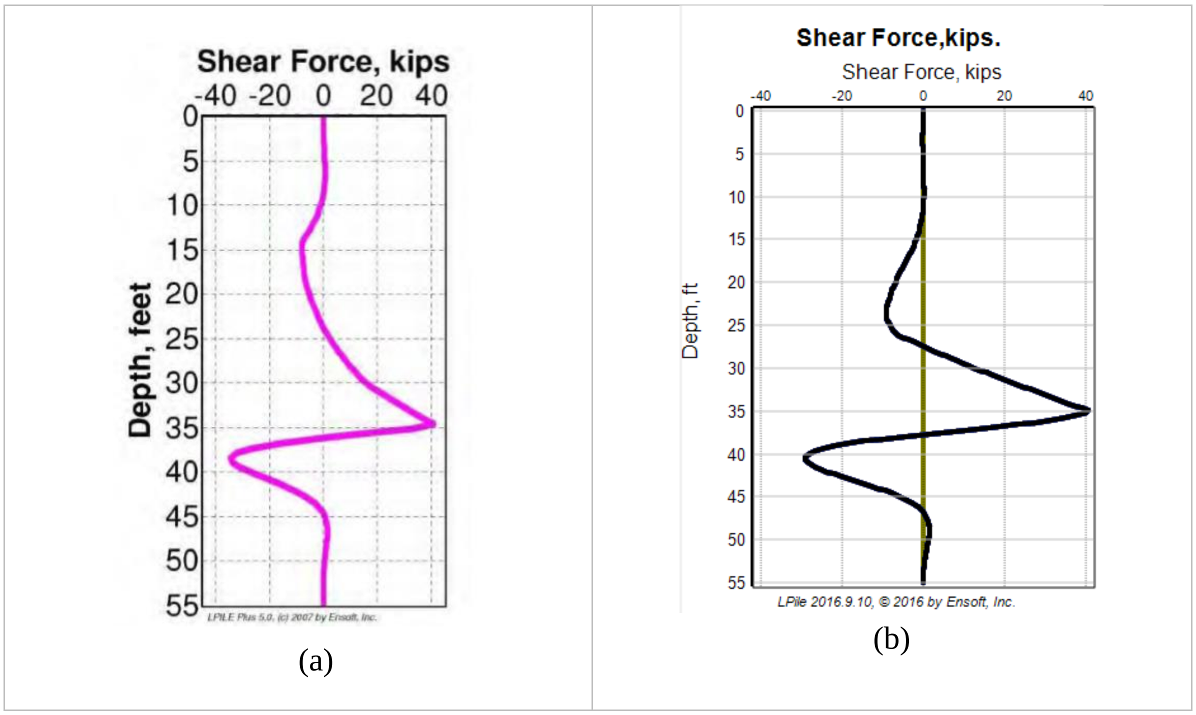
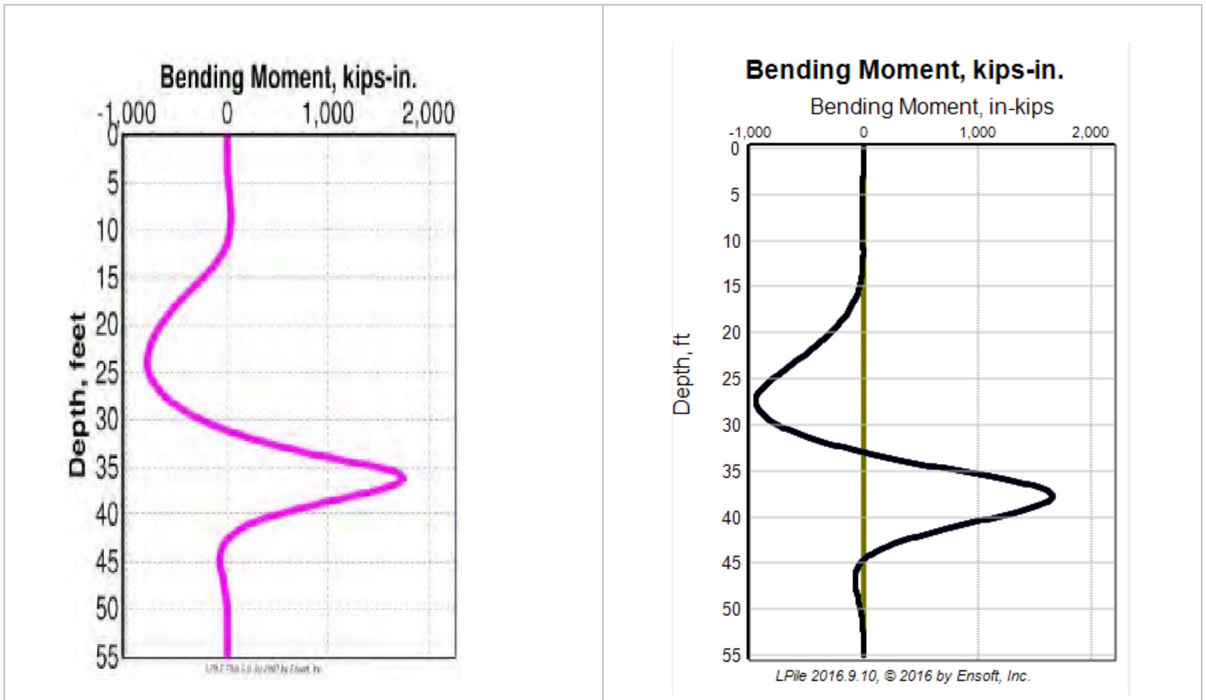


Figure 60. Shear Diagrams for Lateral Analyses (a) Original and (b) Present study



(b)

Figure 61. Moment Diagrams for Lateral Analyses (a) Original and (b) Present study

The maximum values applied to the pile under the given conditions were found below the allowable values, then requirements were met for the ASD analysis. See Appendix II for nominal values calculations.

Table 31. Results for Displacement-based Analyses

Parameter	Present study
Allowable resisting moment of pile	1,401 kip· in
Maximum applied moment	1,084.27 kip.in
Allowable shear force	72.6 kip
Maximum applied shear force	50.17 kips

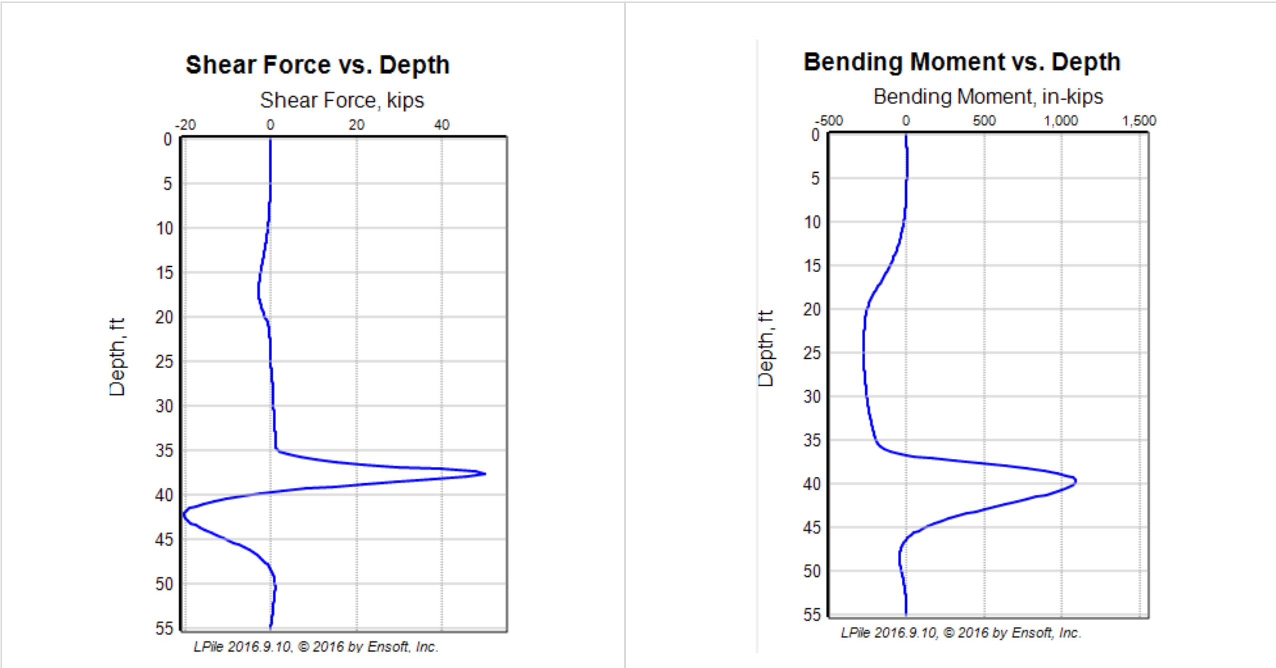


Figure 62. Shear and Moment diagrams for Lateral Analyses Displacement-based

The maximum values for both, the displacement-based analysis and the based on distributed load are located around the same area. However, the maximum shear for the displacement based analysis is not located at 35ft, the sliding depth from the LE calculation. The maximum value is located around 37.6ft. The magnitude of moments and shear forces between both approaches are not been directly compare because both are different designs methods (LRFD and ASD).

3.4.2 Coupled Analysis

For this analysis, the unreinforced analysis will also be performed to verify the value of the minimum factor of safety for the given soil conditions. Furthermore, the reinforced analysis will then be conducted after placing the reinforcing pile as in the uncouple analysis.

3.4.2.1 Unreinforced Analysis

The unreinforced factor of safety for this condition is determined from the 2-dimensional elasto-plastic finite element analysis by the shear strength reduction method using Phase2. However, because the soil parameters used for this analysis come from the calibration of the failure surface to 1.0, the minimum SSR was found smaller than 1.0. This indicates that the slope is instable and no reduction of the shear strength could actually be done. Figure 63 presents the critical SSR as well as the fitted critical slide surface from the LE analysis performed with Slide 7.0 in the uncoupled analysis. The critical region with the highest amount of soil movement is represented by the orange-red zone and matches for both the Slide 7.0 and the Phase2 results.

If the critical slide for the factor of safety equal to 1.0 is superposed to Phase2's results for SSR equal to 1.0, agreement on the location of the critical zones in terms of soil movement is also found as presented in Figure 64. The results indicate that the model should be calibrated to 1.0 in the FE program, hence, the soil properties from the uncoupled condition should vary until the target F.S. is achieved.

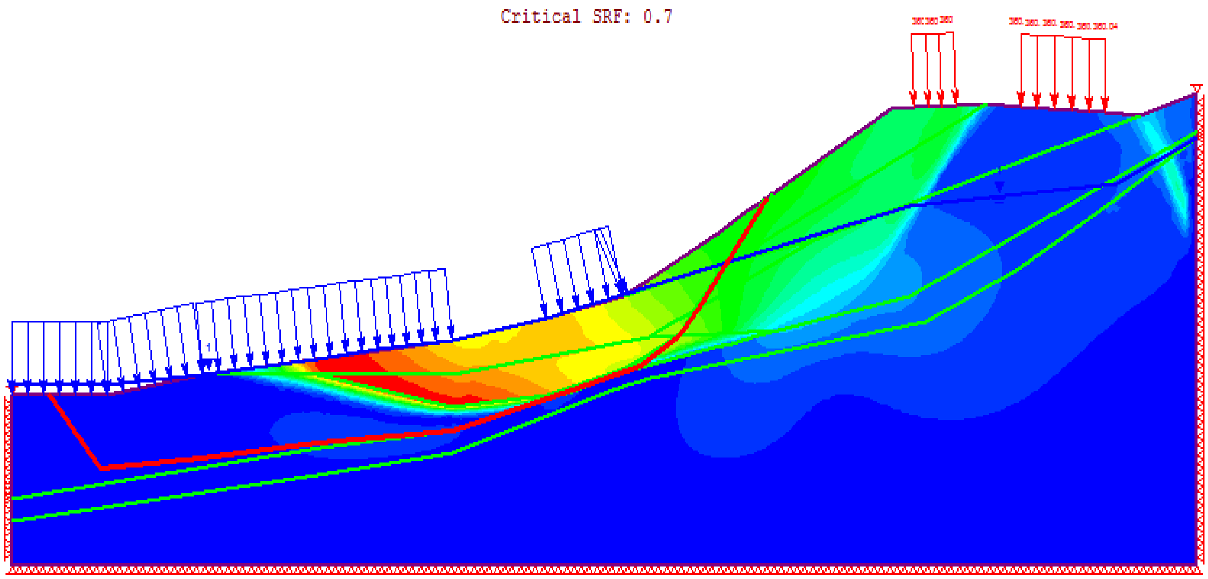


Figure 63. Unreinforced Analysis Phase2

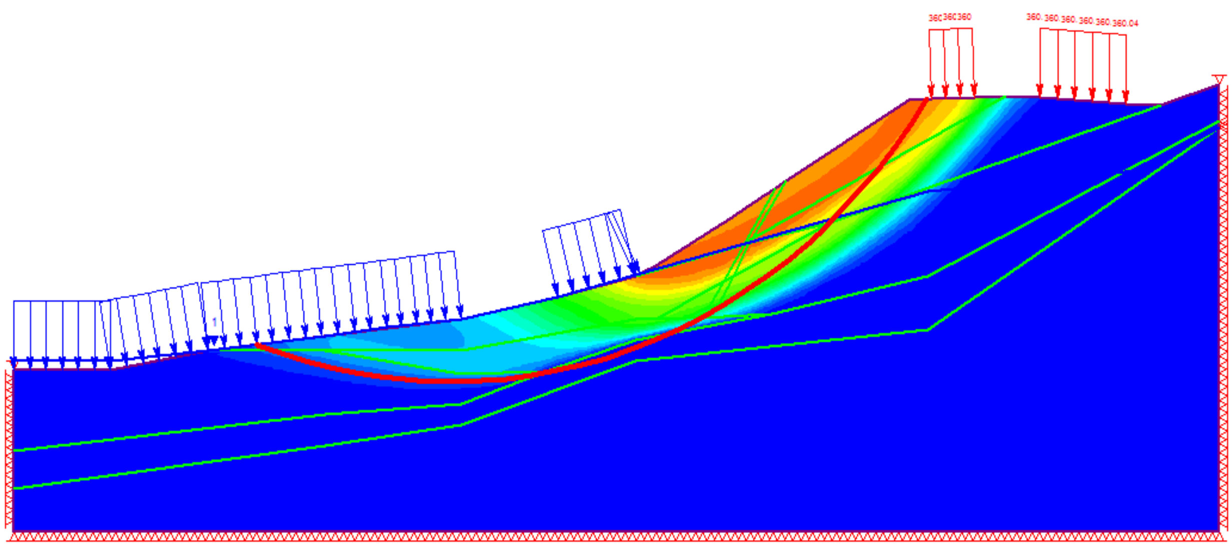


Figure 64. Unreinforced Analysis Phase2 for SSR equal to 1.0 and F.S. equal to 1.0 from Slide 7.0

3.4.2.2 Reinforced Analysis

Because this model corresponds to an active failure, analysis in Phase2 found not recommended. As presented in Figure 65, the value of the critical SSR from the unreinforced to the reinforced analysis remains almost the same after the reinforcement is been placed. Results from FLAC will be used as referenced of the couple analysis to compare with the uncoupled section in future research.

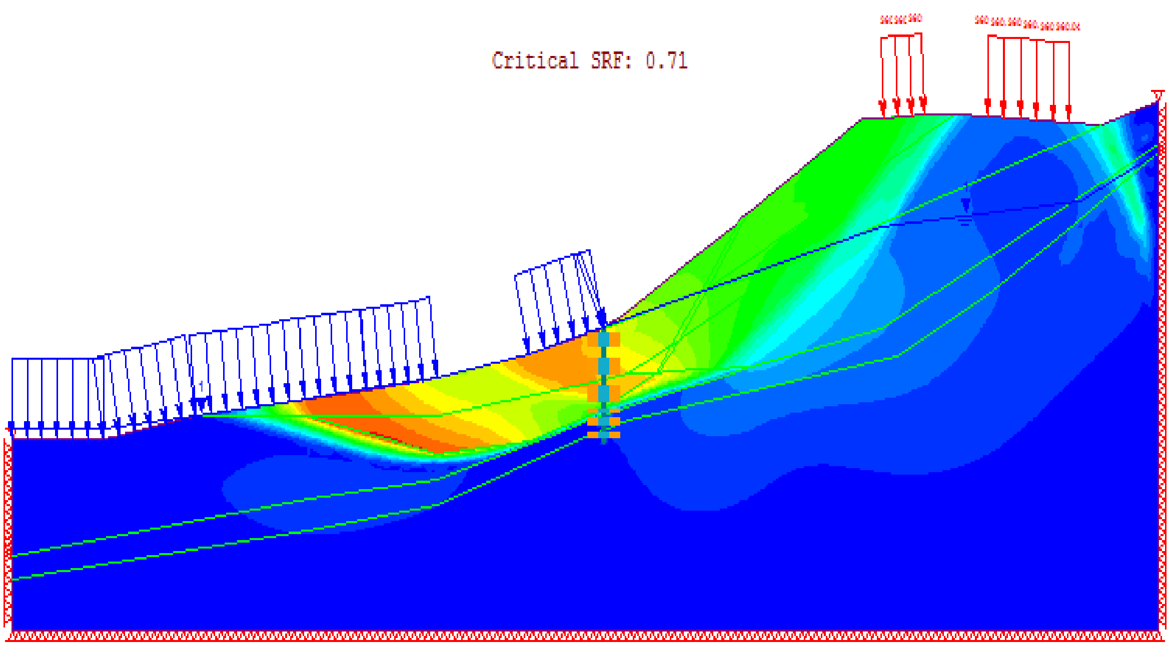


Figure 65. Reinforced Analysis Phase2

3.4.3 Summary Results Coupled and Uncoupled Analyses Problem III

The overall results from this problem are presented in Table 32. The results indicate that uncoupled analysis could be more advantageous when piles are located in an active failure zone. This is because the assumptions of the location of the sliding surface are eliminated. The displacement-based method when inclinometer data is available offers a reliable way of verifying the results for pile structural analysis validating with the distributed load method.

Table 32. General Summary Problem III

<table>
<tr><th>Condition</th><th>Unreinforced F.S.</th><th>Reinforced F.S.</th><th>Shear Force (kN/m)</th><th>Max. Shear from LPILE (kN/m)</th><th>Max. Moment (kN.m)</th></tr>
<tr><td>Zicko et al. (2011)</td><td>1.0</td><td>1.30</td><td>75</td><td>48</td><td>1,740.0</td></tr>
<tr><td>Slide 7</td><td>1.0</td><td>1.305</td><td rowspan="2">75</td><td rowspan="2">48.1(1)
50.1(2)</td><td rowspan="2">1,740.4(1)
1,084.3(2)</td></tr>
<tr><td>SLOPE/W</td><td>1.0</td><td>1.31</td></tr>
<tr><td>Phase2</td><td>0.7</td><td>-</td><td>-</td><td>-</td><td>-</td></tr>
</table>

(1) Distributed load method
(2) Displacement-based

The accuracy of coupled analysis in active landslide should be investigated in detail with different FE or FD programs.

CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

In this study, comparative analyses has been conducted in order to investigate the relative advantages, limitations, and accuracy of the current approaches for pile-reinforced slopes. The analyses have been focused on determining the most advantageous option between coupled (pile and slope in a sole analysis) and uncoupled (pile and slope in two different analysis) approach. In addition, several sensitivity analyses were conducted for understanding the primary factors when slopes reinforced by the means of pile elements are analyzed/design. From the results several conclusions can be drawn:

(1) For unreinforced slopes, coupled and uncoupled analyses are equally recommended regarding accuracy and reliability of the results.

(2) For reinforced slopes, the uncoupled analysis yields marginally more conservative results when performed by the distributed load approach than by the displacement-based method. However, the displacement-based method is better recommended when in situ information is available. Overall, coupled and uncoupled analyses produced similar factors of safety. In this study, the coupled approach is suggested as the most beneficial. This is because several assumptions regarding location of sliding depth, empirical representation of the soil (*p-y* curves for lateral analysis), introduction of modifiers, and duplication of geotechnical and structural factors of safety can be avoided. Moreover, when performing coupled analysis a unique analysis is performed, iterations to determined optimum length and pile location are easily done without distressing about assumptions of pile shear.

(3) In this study, the uncoupled approached is recommended when piles are analyzed as mitigation alternative (active landslide), especially if the slope is instrumented and displacement information is available.

(4) The coupled analysis required more parameters than the uncoupled. However, taking into account the degree of effort to perform two separate analyses and to classify the soil to develop *p*-*y* curves for the lateral analysis, the two approaches could be considered equivalent or even more, if simple FE or FD models are studied, relative little effort in terms of time consumption for model development is needed.

(5) The key factor to ensure accurate results from a coupled analysis is to represent the studied condition as close as possible. This includes the soil conditions, the constitutive model, and reliable values of the soil properties.

(6) If repeatability of the results is pursued, a coupled analysis can be performed and either the shear resistance or soil displacement can be used into a limit equilibrium or laterally loaded pile analysis program to verify the similarity of the results.

(7) Regarding the important factors when analyzing a pile-reinforced, because most authors have previously presented consistent results regarding pile spacing, pile head condition, pile length and number of rows, in this study the focus of the sensitivity analysis was to determine the optimum pile location.

(8) The optimum location was found to depend on the critical surface location which at the same time depends on the material within the slope. The weakest material determine the location of the critical surface, if the sliding depth is considerably shallow the piles must be located from the quarter to the upper portion of the slope varying regarding the frictional characteristic of the materials (pure friction; middle,

friction and cohesion; generally from the quarter to the top, pure cohesion; generally the toe but not very significant the factor of safety improvement with location). The combination weak and soft material was found to produce similar effects on the sliding depth for soft clay-stiff clay, soft clay-sand and sand-stiff clay.

(9) The recommended technique to determine optimum pile length is by performing sensitivity analysis since the optimum length greatly depends on the slope configuration. If an uncoupled analysis is conducted the optimum length should be taken as the smaller depth at which the required shear is minimized and the factor of safety is improved. In this study, to achieve a 1.4 factor of safety the optimum ratio sliding depth/ pile length was found around 0.3 and around 0.45 to achieve a 1.5 factor of safety. From the coupled analysis, the optimum length could be seen from the sensitivity analysis lying in a range where a sudden amount of shear resistance is developed by the pile.

4.2 Recommendations for Future Research

The following recommendations are proposed to enhance the analytical work:

(1) To verify the equivalency of coupled and uncoupled factors of safety additional case studies should be analyzed.

(2) To validate the results regarding optimum pile location several analyses must be conducted using different computer programs, especially for combined materials (with friction and cohesion) to separate the responses of the factor of safety to variations in friction and cohesion.

(3) The efficiency of the coupled approach to analyze mitigation alternatives for an active landslide must be studied to compare with the results of the uncoupled approach.

(4) The group effect on piles, 3D effects, and soil arching by the coupled approach should be studied since the investigations conducted in this analysis were mainly focused on the analysis of piles in row and 2D analyses.

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APPENDIX I. MAXIMUM BENDING, SHEAR AND DISPLACEMENT, PROBLEM I

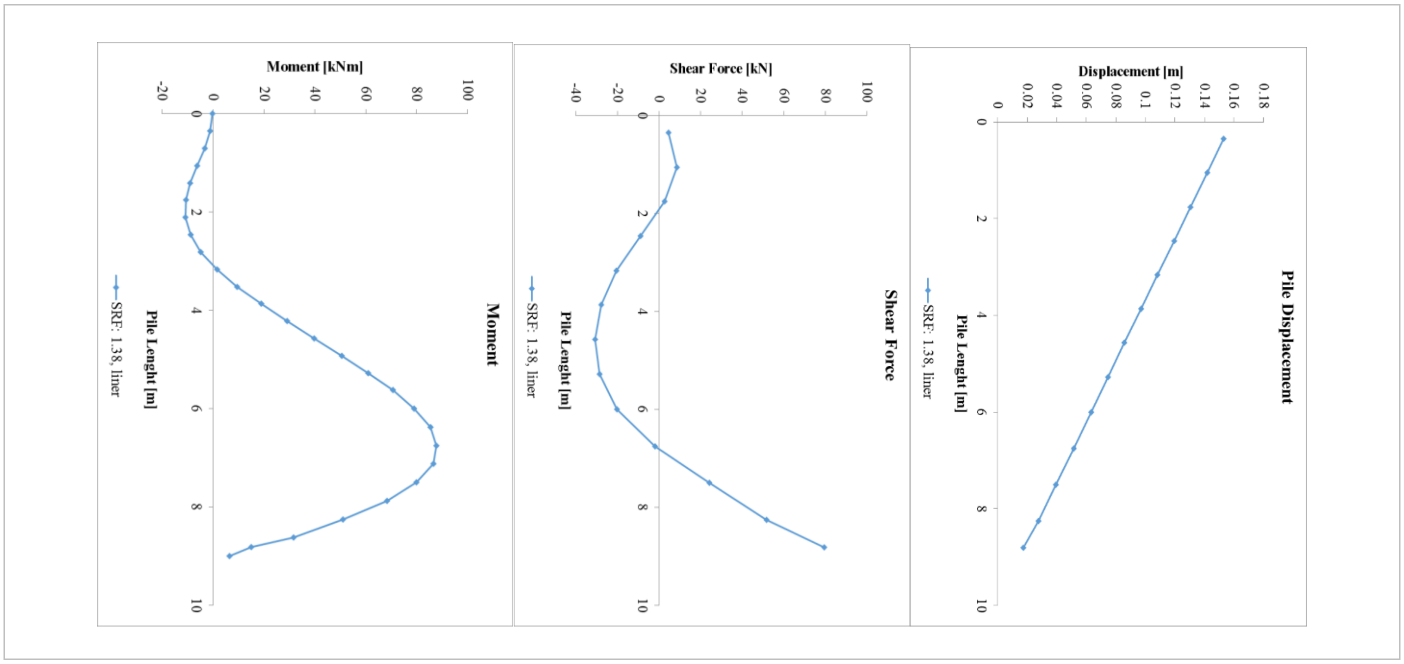

Figure 66. Shear, Moment, and Displacement Diagrams Phase2 Problem I per Pile (1m)

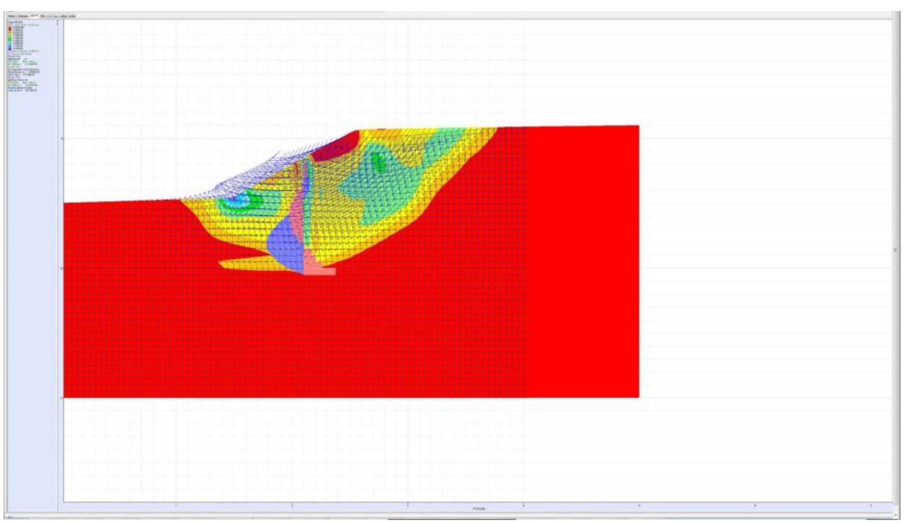

Figure 67. Shear, Moment, and Displacement Diagrams FLAC Problem I per Pile (3m)

APPENDIX II. NOMINAL BENDING AND SHEAR STRENGTH MILL CREEK LANDSLIDE

The properties of the reduced section are presented in Figure 63. The nominal parameters were calculated by following American Institute of Steel Construction (2016) for non-compact section suggested to shear and flexural stresses as follows:

The Manual of Steel Construction by American Institute of Steel Construction (2016) requires that steel structures and structural elements be design so that the strength limit state is not exceeded when subjected to all required factored load combinations. For the strength limit state, axial, flexural, shear strength are investigated.

- **Bending strength**

In flexure design, the basic requirement is expressed as:

For LRFD design

$$\boldsymbol{M_u \leq \emptyset M_n = M_r}$$

For ASD design

$$\boldsymbol{M_u \leq \frac{M_n}{\Omega} = M_a}$$

M_u *is the factored moment of the section and* M_r *is the factored flexural resistance*

M_u *is the moment of the section and* M_a *is the alloable flexural resistance*

LRFD resistance factor =0.90	**ASD factor of safety=1.67**

Because the HP section that have been analyzed is a non-compact section the nominal moment strength should be the smallest value between lateral torsional buckling and the limit state of yielding. However, because no axial compression load takes place and the bending moment is considered to occur about the strong axis and loaded at the symmetry plane, the nominal moment strength of the section will be:

$$\boldsymbol{M_n = f_y S_x}$$

$$\boldsymbol{M_n = 50ksi(46.798) = 2,340\ kips - in}$$

Where f_y the yield is stress of the section and S_x is the elastic modulus of the section in in^3.

Hence, the factored flexural resistance will be:

$$M_r = (\mathbf{0.9})\ \mathbf{2,340 = 2106}\ \boldsymbol{kips - in},$$

And the allowable bending moment;

$$M_r = \frac{\mathbf{2340}}{\mathbf{1.67}} = \mathbf{1,401}\ \boldsymbol{kips - in}$$

- **Shear strength**

As in the bending case, the design shear strength, $\emptyset \boldsymbol{V_n}$, and the allowable shear strength, $\frac{\boldsymbol{V_n}}{\Omega}$, will be found from determining the nominal shear strength as follows for I-shaped members and channels:

LRFD resistance factor =0.90	**ASD factor of safety=1.5**

$$V_n = 0.6 f_y A_w C_{vw}$$

$$V_n = 0.6(50ksi)(11.66\ in x 0.31\ in)(1) = 108.4 kips$$

Where A_w is the web area, the total depth (d) by the web thickness and C_{vw} is the web shear coefficient that according to Section G2.a, American Institute of Steel Construction (2016) is equal to 1 for all the current HP shapes.

Hence, the factored shear strength will be:

$$\boldsymbol{\emptyset V_n = (0.9)\ 108.4 = 98\ kips}$$

And the allowable shear strength;

$$\boldsymbol{\frac{V_n}{\Omega} = \frac{108.4}{1.5} = 72.2 kips}$$

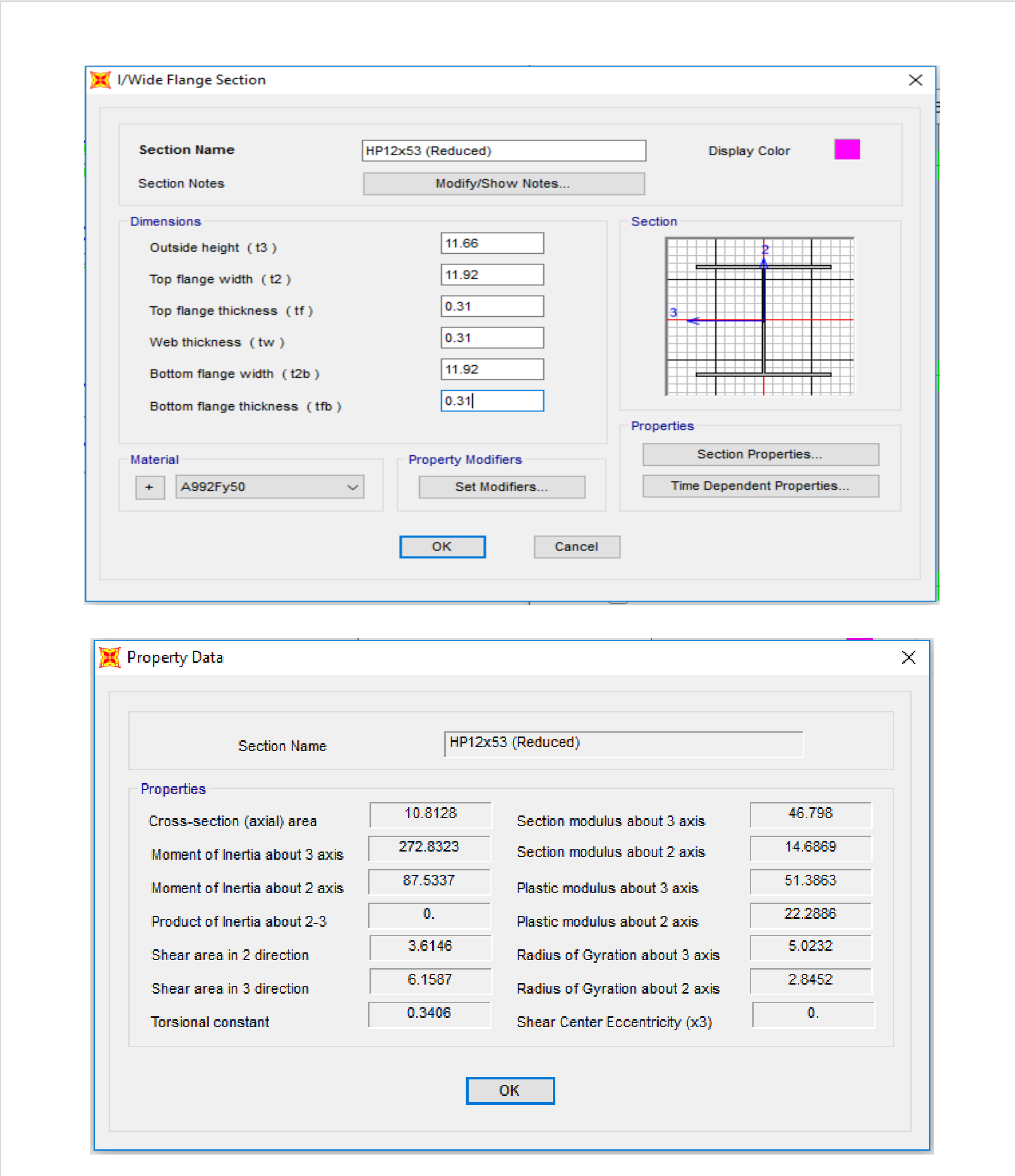

Figure 68. Geometry and properties reduced HP12x53 section from SAP200 v 18.1.1 by CSI (2016)