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OPTIMIZATION OF STABILIZATION OF HIGHWAY EMBANKMENT SLOPES USING DRIVEN PILES – PHASE I

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December 2010

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16. Abstract

This study determined the feasibility of using driven piles to stabilize highway embankment slopes. The activities performed under this study were a detailed literature review, a national survey of state DOTs, a review of inspection and stabilization mitigation reports, targeted field inspections, a cost comparison analysis, and a finite element study. The results of this study show that driven piles can be a cost-effective solution to stabilizing highway embankment slopes.

The literature review showed that there has been significant research done concerning the lateral capacity of piles. This research tends to be focused on different applications, but still shows that piles have significant lateral capacity. The survey conducted shows that several DOTs have used driven piles to stabilize highway embankment failures and most of these departments would recommend future use. Also three DOTs have performed similar research using plastic pins to stabilize embankments. The site visits allowed the research team to identify two sites, the Muddy Pass slide and also the Rye slide, as potential sites for investigation under Phase II of the project. These slides in particular had broad shoulders along the highway that provide better accessibility. The cost comparison analysis showed that for a particular slope, driven piles would cost $41 per linear foot of road stabilized. This was compared to drilled shafts and launched soil nails which had estimated costs of $32 and $130 per linear foot, respectively. The finite element study showed that the factor of safety for a stabilized slope could be significantly improved with pile installation.

Implementation:

Based on the results of the study it is recommended that the Colorado Department of Transportation (CDOT) pursue Phase II of the study.

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EXECUTIVE SUMMARY

This report presents the findings of Study No. 074.90, "Optimization of Stabilization of Highway Embankment Slopes Using Driven Piles (Phase I – Literature Review and Preliminary Assessments of Highway Slopes)." The stated goals of this study were to perform a literature review of stabilization methods, conduct a national survey of state DOTs, review inspection and stabilization mitigation reports, perform targeted field inspections, perform a cost comparison analysis of various stabilization methods, and analyze the accumulated data to determine when driven piles are a feasible landslide mitigation method.

Embankment failures of Colorado's mountain highways are a relatively frequent problem. Horizontal and vertical movements of slopes often cause settlement of the highway surface resulting in pavement distress and dangerous conditions for highway users. Maintenance resources are commonly used to deal with these stability issues, typically by repaving the afflicted area, and on occasion attempting some mitigation. One method that the maintenance crews have used in the past, with reasonable success, is to drive piles along the shoulder of the road, typically with little or no geotechnical engineering input. Maintenance crews have limited budgets and generally have steel shapes available. Hence driving piles is often a viable option to improve current slope factor of safety without significant engineering.

Significant research has been conducted concerning the lateral capacity of piles. However, most of this research is either purely theoretical or for significantly different applications. Several design methods, for stabilizing slopes and obtaining necessary lateral capacity, have been derived from these studies. The extension of these methods to stabilize slopes has not been studied adequately and has not been verified with field monitoring.

A survey was conducted to investigate how other State DOTs have addressed the issue of highway embankment stabilization using driven piles. The survey had an 86% response rate. Of the responding departments, 48% had previously used driven piles as a slope stabilizing method. Of those, 90% recommended the use of driven piles. Three Midwestern state DOTs have recently conducted research concerning a driven pile approach (Iowa, Wisconsin, and Missouri). Their studies address slope embankments on flat ground, and thus, their conclusions cannot be directly extended to the mountainous regions of interest to CDOT. Furthermore, these studies did not

have the same access and right of way restrictions, and piles were typically distributed throughout the slope instead of being concentrated at the shoulder of the road.

Five sites were visited during the study. These sites had slides of varying magnitudes, some of which had been previously stabilized, although not always successfully. Two of the sites visited (SH-72 and Douglas Pass) had been mitigated using a driven pile type of system. SH-72 appeared to be performing very well. Douglas Pass was performing well but had some drainage issues. The slide at Muddy Pass and also the Rye slide were identified as sites that could be further investigated in the Phase II research project. These particular slides had a relatively flat and wide shoulder on both sides of the highway that would allow access for driving equipment.

While visiting different sites it was observed that the landslides were different in size, depth of failure, three dimensional characteristics and accessibility for remediation. It is clear, therefore, that a "one solution fits all" approach is not applicable to this problem. An example cost comparison analysis was performed for the purposes of this study, for a specific slope with fixed geometry and soil characteristics. Stabilization methods based on driven piles, drilled shafts and launched soil nails, a method that was seen on one of the Douglas Pass slides, were assessed with the assumption that the conditions were ideal for each installation. The estimated costs per linear foot of road stabilized were $41, $32, and $130 respectively.

Some preliminary software development has been performed using the finite element method, to better understand the potential failure mechanisms and load transfer occurring in pile-reinforced slopes. Specifically, if calibrated to actual field observations of pile performance, the finite element method could be used to predict pile/slide performance under a wide variety of configurations and conditions. This work showed that the factor of safety could be significantly improved depending on the length and location of the installed pile.

Slope stabilizing piles have had significant success as a practical un-engineered solution. Furthermore, rudimentary analysis shows that slides can be effectively stabilized using an engineered solution. Additionally, there is a three dimensional aspect that hasn't been previously considered that may cause further improvements in the efficiency of stabilizing pile systems. Most analyses only consider a two dimensional slope to be stabilized for computational efficiency. However, most actual landslides occupy a three dimensional space where they are shallower at the edges and deeper in the middle. As the shallower portion of the slide is successfully stabilized it may force a different failure mechanism with more intense 3-D characteristics and potentially a higher factor of safety.

Based on the acquired information, it is recommended that the current project be extended to Phase II. The main goal of the research of Phase II of this study will be the instrumented mitigation of one or two (based on available budget) highway embankments using stabilizing piles.

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1.0 INTRODUCTION

1.1 Background

Slope stability is the result of balance between driving forces that promote down-slope movement and resisting forces that react to driving forces and deter movement. Slope instability results when resisting forces cannot balance the driving forces. Stabilization of slopes is an issue that geotechnical and structural engineers must often address. In general, slope stabilization methods aim to reduce the driving forces, increase the resisting forces, or suitably combine both.

The following approaches can be used to reduce the driving forces:

1. Remove mass from the crest.
2. Flatten slopes.
3. Apply slope benching.

Approaches to increase the resisting forces include:

1. Drainage to improve the shear strength of the ground.
2. Use of cement, lime, or other materials to improve the shear strength of the ground.
3. Elimination of weak layers.
4. Increasing the mass at the toe of the slide.
5. Provide a retaining structure.
6. Reinforce the ground (piles, drilled shafts, soil nailing, anchors, deep-rooted vegetation).

When slope failures of highway-embankments are considered, the practical remedies are more limited as the slope crest is commonly the road grade, and the toe is typically at or near the right-of-way boundary. In these cases, the crest cannot be modified without significant expense, additional mass cannot be added to the toe, the slope grade cannot be easily modified, and the shear strength of the ground typically cannot be improved without significant expense and traffic disruption. As such, ground reinforcement techniques appear to be the most realistic approach to achieving stability.

Driven piles have several advantages as a ground reinforcement technique. Transportation departments are familiar with pile materials and pile driving equipment. The piles can be installed quickly and provide immediate strength improvements. The installation of the piles does not significantly disrupt traffic flow, and they can be installed from the shoulder of the road without completely closing the highway. There are, however, a few significant limitations of driven piles:

1. They can only be used in smaller slides where appropriate flexural stiffness of the piles is secured and adequate penetration into an underlying stable material can be achieved.

2. They can be relatively expensive compared to other solutions for bigger slides.

3. They lose effectiveness in soils that tend to flow between the piles (e.g. soft clays or loose sands).

4. The activity of driving piles may have an adverse effect on slope stability during installation.

5. There currently is not a widely accepted verified design method for slope stabilization using driven piles.

1.2 Objectives

The objectives of this study are to:

- Research and identify the state-of-the-art and the state-of-the-practice of slope stabilizing piles.

- Identify potential sites for detailed investigation, field instrumentation, and verification and monitoring for future Phase – II research study.

- Document and analyze all available data and recommend the pursuit of Phase – II research.

The criteria, identified by the CDOT research panel responsible for this project, for recognizing whether a problem slope may be effectively stabilized using driven piles are:

- Maximum depth to the failure surface of approximately 20 ft.
- Maximum length of roadway impacted of approximately 300 feet.
- Maximum aerial extent of the slide mass of approximately 5 acres.

1.3 Approach

To fulfill the first objective of this study, identifying the state-of-the-art and state-of-the-practice of slope stabilizing piles, the following tasks (quoted from the CSM proposal) were performed:

Task 1: Perform a literature review on the stabilization of highway embankment slopes using driven piles and other methods including drilled shafts and soil nailing, to determine if there has been similar research that will aid CDOT in improving the current practice.

Task 2: Conduct a national survey of State DOTs to determine if other states have had similar problems and if so, their solutions and recommendations for driven piles and other methods.

Task 3: Detailed review of CDOT/Consultants inspection and slope stabilization mitigation reports for Colorado, and other states.

To fulfill the second objective of identifying potential sites for more detailed investigation, task 4, listed below, was performed.

Task 4: Perform targeted field inspections of approximately 20 sites in Colorado in consultation with CDOT maintenance and engineering staff.

Another related objective was to familiarize the CSM research team with typical slides that occur on Colorado's highway system. While the initial scope of work envisioned approximately 20 site visits, time and budget constraints allowed only five site visits. The research team feels that this reduced number of site visits was adequate.

The use of driven piles to increase slide resistance is not always perceived as an economic process for improving the stability of an embankment slope. An example cost study was, therefore, performed in task 5 in order to compare the performance of driven piles to other comparable deep foundation stabilization methods such as drilled shafts and soil nailing.

Task 5:Provide a cost comparison of driven piles to other deep foundation stabilization methods.

All data collected in tasks 1 through 5 were analyzed as part of task 6 to determine whether driven piles can produce reliable and economic slope stability. This analysis was performed so as to recommend Phase – II, field validation, of this study.

Task 6: Analysis will be performed of all data collected in tasks 1 through 5.

2.0 LITERATURE REVIEW

The aspect ratio of a slide or failure generally is used to classify the slope failure type. As presented in Figure 1, a rotational slide produces a failure surface with an aspect ratio in the range of 0.15 <D/L<0.33 where D is the depth of the sliding surface perpendicular to the slope face, and L is the length of the sliding surface, Abramson et al. [1].

Slope geometry, soil type, saturation, and seepage are among the factors affecting the size of shallow slope failures. Shallow slope failures often are parallel to the slope surface and usually are considered as infinite slope failures. The depth varies depending on many factors, including soil type, slope geometry, and climatic conditions. Various depths were reported in the literature

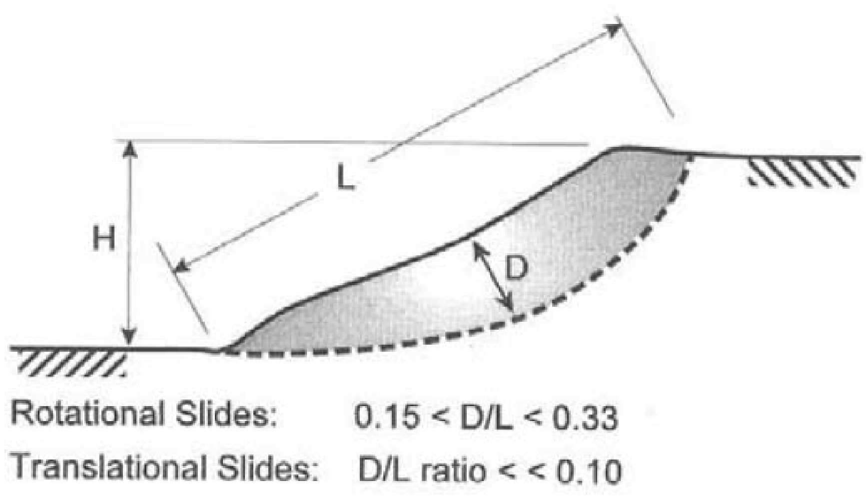

Figure 1. Aspect ratio of failure mass (Abramson et al., 2002).

based on case histories, but all studies indicated the shallow nature of surficial failures. The aspect ratio of the failure can be used to categorize whether the slide is shallow or not. In Figure 1, when the aspect ratio, D/L < 15%, or failure surface depth is less than 10 ft, the slide is characterized as shallow, Abramson et al. [1].

Shallow slope failures often occur during or after periods of heavy rainfall. Rapid snowmelt resulting from sudden increases in temperature can also lead to surficial instabilities in slopes and embankments. Many cases of surficial instabilities of slopes are attributed to prolonged-rainfall events, particularly during the spring thaw (snowmelt).

Shallow slope failures commonly occur when the rainfall intensity is larger than the soil infiltration rate and the rainfall lasts long enough to saturate the slope up to a certain depth, which leads to the buildup of pore water pressure [1].

Snowmelt creates a continuous source of water that infiltrates soil for longer time periods. Therefore, snowmelt may result in rising water levels as water perches on drainage barriers, consequently raising pore water pressures that trigger slope failures.

Additionally, roads are occasionally constructed over naturally occurring drainage such as chutes, ravines, or gullies; increasing the degree of saturation and reducing the factor of safety in these areas, Figure 2.

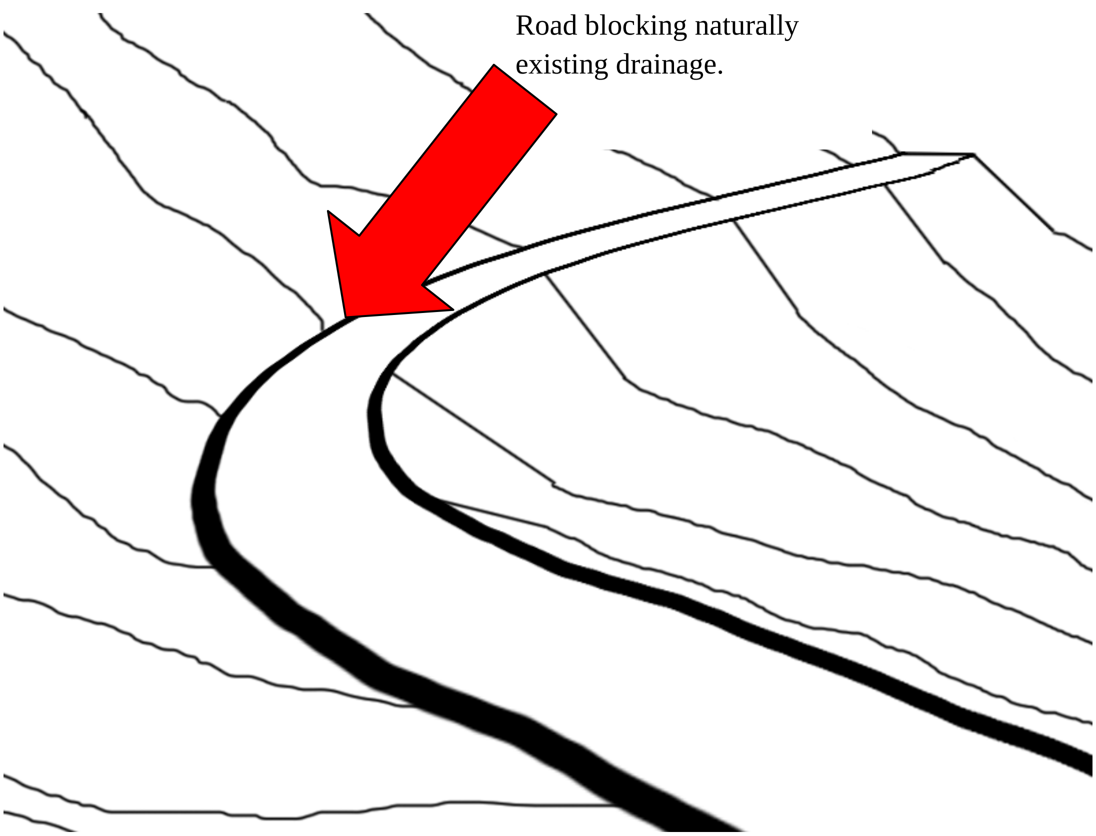

Figure 2. Naturally existing drainage blocked by road.

Recent methods for repairing shallow slope failures include the use of driven or bored short vertical structural members. This technology has been successfully used in other states such as Missouri. In this methodology, the failed soil is pushed back in place and the structural members are installed vertically into the ground. These members will resist the forces driving the slope failure. A variety of materials can be used to make these structural members, including wood, metal, recycled plastic, and other cost-effective materials. The importance of the subject has led to a number of research studies, as summarized below:

Broms [2, 3] developed methods for calculating the ultimate lateral resistance and lateral deflections for piles driven into cohesive and cohesionless soils. Broms identified two different pile configurations; free-headed piles which are free to rotate about its top end, and fixed-headed piles, which may be restrained by a pile cap or a bracing system. Broms found two dominant failure modes: a) structural failure by development of a plastic hinge, or plastic hinges in the fixed-headed piles, in the pile section and b) geotechnical failure by exceeding bearing capacity of the supporting soil.

Ito and Matsui [4] developed a procedure for identifying the loads acting on landslide resisting piles that has become the dominant means for calculating these loads. They calculated the loads assuming plastic deformation and plastic flow for hard and soft soils respectively, and perfectly rigid piles. Flow resistance is increased by the soil arching mechanism. The developed theory was then tested on laterally loading piles, where the measured load distribution was compared to the predicted load distribution.

Poulos [5] performed a theoretical analysis on a single pile subjected to lateral soil movement. Poulos used a finite difference method to calculate the displacements and lateral pressures for a specified horizontal soil movement. This method was used to determine the effect of several parameters such as pile relative stiffness, the influence of fixed-headed piles, and pile diameter. The specified soil movement is estimated using either elastic theory or finite element analysis. The theoretical results were compared with existing field measurements.

Hassiotis et al. [6] produced a design method for stabilizing piles. The safety factor of the slope is determined based on the ratio of the pile diameter to spacing, and the distance from the toe of the slope to the pile. The relationship of these was determined in an earlier study conducted by

Hassiotis and Chameau [7]. The method presented by Hassiotis et al. [6] takes advantage of an extension of the force distribution calculated by Ito and Matsui [4] and Ito et al. [8] to calculate the forces acting on a semi-rigid pile above the slip surface. Below the slip surface, finite differences were used to calculate the response. It was concluded that piles driven in the upper middle part of the slide mass are more effective and result in overall larger factor of safety.

The design methods reviewed above have some limitations that reduce their applicability to the types of stability problems often encountered on Colorado's mountainous highways. For example, Ito and Matsui [4] make a number of assumptions about how soil will move between piles that may not reflect actual conditions. Also, they do not consider the lateral resistance of the soil/rock adjacent to the lower part of the pile that acts to resist pile deflection. Some of these assumptions are carried through to Ito and Matsui's later papers (1981 and 1982 use the same reference system here with the number of the publication), and, thus in the work of Hassiotis and co-workers [6,7] These methods do not appear to adequately consider the overall performance of the soil/pile/slope system. While Ito and Matsui's original work was based on actual pile installations in active landslides, the field conditions are not discussed in their papers. Since their work was performed in Japan over 30 years ago, it would be difficult to make the necessary comparison between their field conditions and those commonly present along Colorado highways. In their discussion of Hassiotis' results, Hull and Poulos [9] state that "analysis of the influence of piles on the stability of slopes ... has attracted the interest of engineers for many years, but it still remains a problem with no definitive approach that has found universal approval."

Pearlman et al. [24] analyzed several case studies involving the use of Type “A” INSERT (In-Site Earth Reinforcement Technique) walls and developed a preliminary design procedure. Type “A” INSERT walls are composed of combinations of vertical, and near-vertical pins that extend beneath the slide plane. The pins are connected together with a concrete cap just underneath the ground surface. The pins are composed or a rebar or steel pipe embedded in a concrete shaft. The pins are installed by drilling. Pearlman et al. [24] documented seven different cases in which Type “A” insert pins had been used in stabilization attempts, however only two of these cases are discussed. Both the applications discussed are for slides of about 25’ in depth. The pins were able to successfully stabilize one slide, and significantly reduce the movement of the other. The design method produced is based on the theory developed by Ito and Matsui [4]. The design method simplifies the developed theory by providing charts that directly compare ultimate horizontal stress transfer with the undrained shear strength and the angle of friction for different pin depths, spacing and diameters.

El Sawwaf [10] performed a series of laboratory model tests concerning the behavior of a strip footing above a reinforced embankment. In this study he inspected the influence of pile diameter, pile length, pile spacing, and pile location on a bearing capacity improvement factor. The bearing capacity improvement factor represents the percent change in bearing capacity from an unstabilized condition. The pile spacing had the most significant influence on the bearing capacity. When a normalized spacing of 2.5 was reduced to 0.5 there was a 65% improvement in the slopes bearing capacity. The observed optimal pile location, from a bearing capacity standpoint, was at the crest of the slope. Another observation was that sheet piling further increased bearing capacity. This is typically not a practical solution however, as sheet piling inhibits drainage.

Based on the reviewed literature, the following observations are emphasized:

Most research published on this subject addresses drilled circular shafts used to stabilize slopes rather than driven H- or similar piles. Broms' work [2,3], which explicitly addresses driven piles, is not concerned with slope soil movement. Instead, it studies the problem of a driven pile loaded by a horizontal force at its top.

The pile-slope stabilization problem has not been addressed in the literature as a "repair" method. All analysis and design approaches examine the increase of factor of safety against slide of a slope due to stabilizing piles. In such approach, it has always been concluded that the pile-stabilized slope fails at a different failure circle than the non-stabilized slope. In many practical problems however, stabilization is required after slope instability has been initiated. In these cases, a remolded-material failure zone has been created, and as a result, the same failure circle may still be critical. This issue has not been addressed in the published literature.

3.0 SURVEY

Beginning 13 March 2009, a survey was mailed electronically to all state Departments of Transportation and the Federal Highway Administration. The survey was prepared with the help of a web-based facility, www.SurveyMonkey.com. The survey questions are as follows:

- Which options has your department considered when remediating small landslides, with a maximum depth of failure surface of 20 ft?
- Has your department ever used driven piles to mitigate a small landslide that is adjacent to a road?
- If so, how well have they performed? Would you recommend their continued use?
- Has your department ever performed research concerning the lateral strength of driven and drilled piles?
- If so, how can this research be accessed?

Forty three (86%) departments have completed the survey. Twenty (48%) of the responding departments have attempted to use piles to stabilize slopes. Seventeen (85%) of these twenty departments recommend the use of piles given certain criteria. The most significant disadvantages cited were the cost and the poor performance in very moist locations. Of the responding departments, 14 have performed, or are performing research concerning the lateral strength of piles. Four DOT reports have been obtained from Iowa, Missouri, Wisconsin and Tennessee. The reports from Iowa, Missouri and Wisconsin explicitly investigate the use of vertical members to stabilize slopes. The results of the survey are presented in a concise table in Appendix A.

3.1 Iowa Research

A research project supported by the Iowa DOT, titled “Innovative Solutions for Slope Stability Reinforcement and Characterization,” was conducted by White [11] at Iowa State University. One of the projects goals was to develop a slope remediation method using micropile reinforcement. This report found that the use of pile elements offered considerable lateral movement resistance offering an improvement factor typically between 1.2 and 6.6.

Furthermore, the relative soil-pile displacement at the surface was shown to be indicative of the pile behavior. This would allow maintenance crews to determine how the slope was performing. A design method was proposed that uses displacement-based lateral response analysis methods (soil *p-y* curves), which were found to accurately predict the deflection and bending moment of the piles. While this research shows the promise of pile stabilized slopes, it requires multiple rows of piles.

3.2 Missouri Research

Research sponsored by the Missouri DOT, conducted by Loehr and Bowders [12] at the University of Missouri, studied the use of driving plastic pins as a method of earth reinforcement. The plastic pins are a structural variety of the composite plastics commonly found at hardware stores specifically for the use of outdoor decking. The pins were installed very close together in multiple rows to achieve resistance against shallow slides (depth to failure surface less than 10 ft). When right of way access is available, this method provides an efficient solution to slope instability.

3.3 Wisconsin Research

An "Investigation of Vertical Members to Resist Surficial Slope Instabilities" was performed by Titi and Helwany [13] of the University of Wisconsin. This study was mostly a continuation of the study performed by Loehr and Bowders [12] at Missouri. This study compared the plastic pin method with other methods such as soil nail launching and replacing the plastic pins with lumber. It was determined that lumber and plastic pins can provide cost-effective stabilization.

3.4 Comments

Most studies that have been published on reinforcing slopes to prevent or repair failures of highway embankments, address similar causal issues to the ones found in Colorado such as slope instabilities during the spring when snow melt increases soil wetness. The main difference however, is that these studies address slope failure concerns of Midwestern states (Wisconsin, Missouri, Iowa) such as embankments on flat ground (Figure 3A). In contrast, the problems on mountainous Colorado highways are described closer by the schematic in Figure 3B.

It is clearly demonstrated that failures in mountainous Colorado highways tend to be deeper and more often extend into the foundation ground, thus making stabilization with driven piles more challenging, due to the larger depth to the failure surface.

Figure 3. Differences between embankment failures on flat ground and mountainous terrain.

4.0 SITE VISITS

A number of site visits were conducted in 2009. The site visits were performed to identify the types of slides that commonly occur on Colorado highways. During these visits eight different slide areas were investigated, some of which had been previously stabilized. The locations of the slide areas are shown in Figure 4.

Figure 4. Site visit locations.

4.1 State Highway 72

The site visit was performed March 6, 2009 near mile marker 25 along State Highway 72 (Steve Laudeman, Aziz Khan, Russel Cox, Alan Lisowy, John Hart, Panos D. Kiousis, Jared Stewart). The slide had been previously stabilized using steel guard rail sections. Type 3 guardrail posts were used as piles spaced at six ft and three inches in this case, a typical section is shown in Figure 5. Additionally, guardrail railing was used to provide lagging between the piles, Figure 6. This slide did not show any signs of recent movement.

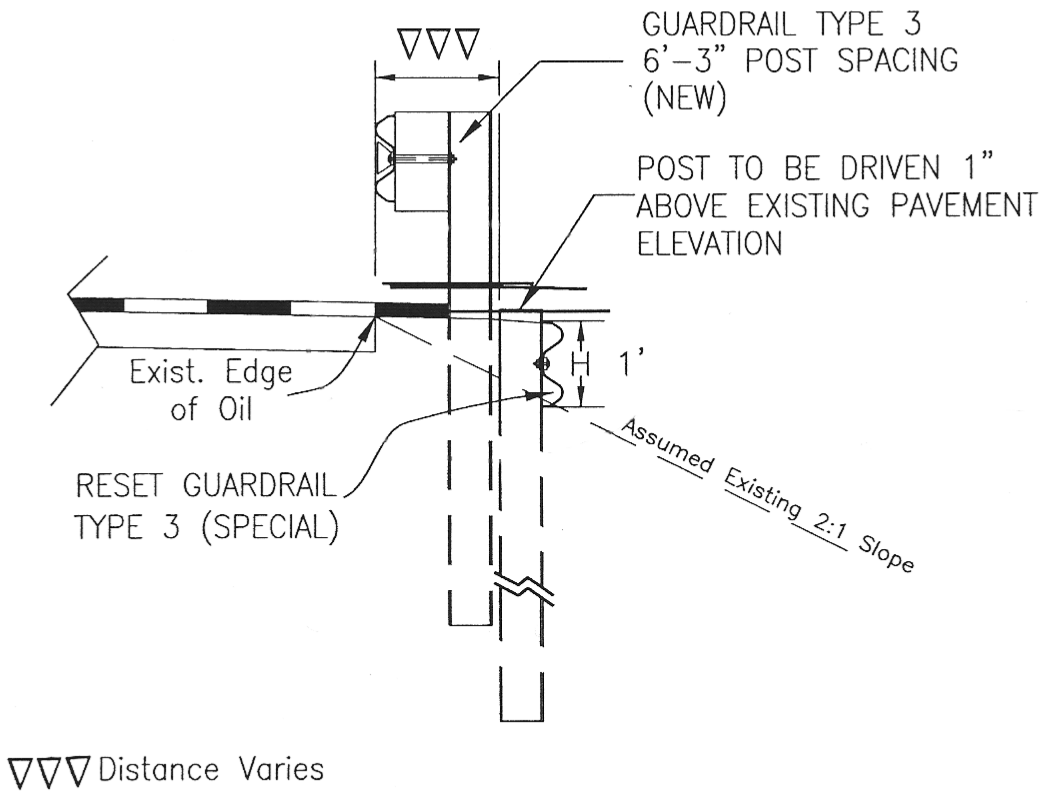

Figure 5. Typical section SH-72 stabilization system.

Figure 6. Guardrail stabilization system along SH-72.

4.2 Rye Slide

Two main areas of distress were identified at the Rye slide (State Highway 165, mile marker 26) during the visit (Aziz Khan and Panos D. Kiousis) on May 11, 2009. The first area is identified in the 2006 report (Figure 7) and again during the May 11, 2009 visit (Figure 7 white arrow). This distress appears to be the result of a deep seated slip failure, which, according to the 2006 report, had its toe 50 to 100 ft within the private property that borders the northern “right-of-way” boundary. This is verified by close examination of Figures 1 and 4 of the 2006 “Rye Slide Interim Report” [14] and the inclinometer data of borehole I103 (up to 3-4-2008) provided by Mr. Laudeman, where a shear band at a depth between 34 and 36 ft (at the interface of clay and claystone layers) has been developed.

Figure 7. Pavement distress at Rye (2006).

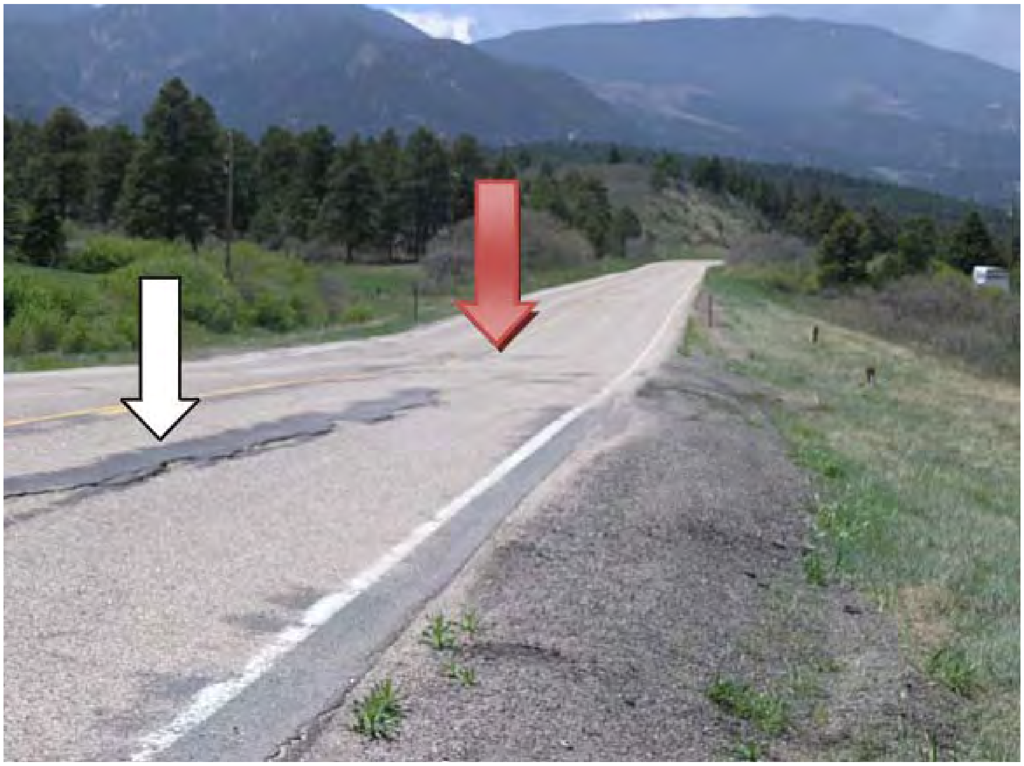

Figure 8. Pavement distress at Rye (2009).

The second area of distress is approximately 100 ft west of the aforementioned area (Figure 8 – red arrow) and it appears to be one where the predominant movement is settlement. Directly underneath this area, at the base of the embankment, there is a 24 inch diameter culvert which clearly sags approximately 9 inches in the area under the pavement (Figure 9 – the opening at the opposite end is barely visible due to sag). This indicates that a significant part of the settlement occurs below the fill, into the organic clay and/or clay layer beneath it. The inclinometers of boreholes I101 and I104 are located in this area, on the south and north sides of the road respectively. No shear failure is indicated in these inclinometers. Instead, a gradual displacement, almost linearly outwards (with respect to the road) in the case of I104 is observed. It is not clear if the displacements recorded by the inclinometer in borehole I101 are inward or outward with respect to the road.

Figure 9. Photo through culvert showing sag.

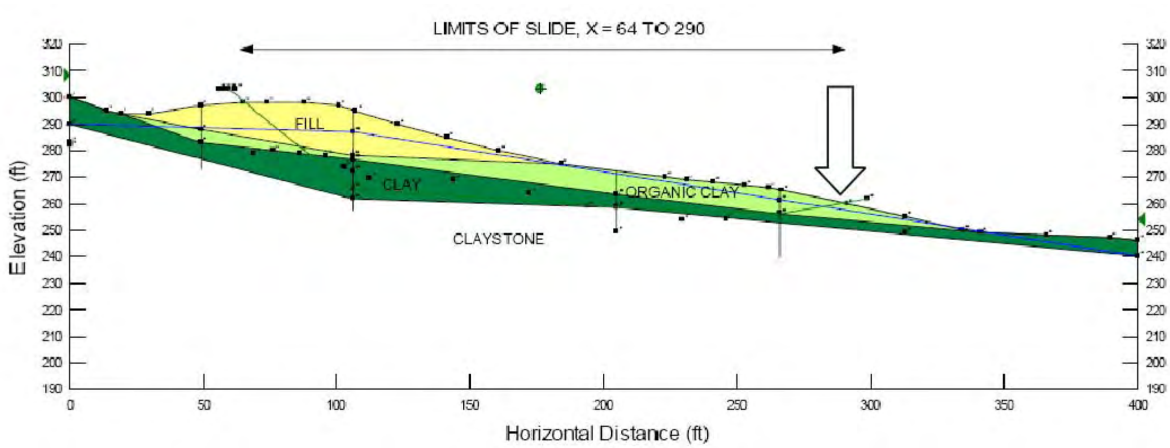

Figure 10. Soil stratification of the Rye slide area.

Based on the stratification of Figure 10 (from 2006 report- arrow indicates approximate location of right-of-way border), the effectiveness of driven piles to mitigate this site cannot be decided without further study. Piles driven next to the road may need to be longer than 30 ft to capture the deep seated instability of this problem. Stabilization of the slope failure close to the toe (at the northern boundary of the right-of-way) using driven piles requires lengths in the order of 15 ft. Nevertheless, the Rye slide may be a good test site due to the three dimensional characteristics of the observed progressive slide as will be discussed in recommendations. The drains that were installed in the past (only two work currently) could aid in the mitigation of the problem. It appears that the currently installed drains were designed to predominately aid the drainage of the sandy clay fill material. Whereas this is necessary, drainage and/or other techniques to stabilize the organic clay and clay layers, especially under the fill may also prove to be necessary.

4.3 Hoosier Pass

A slide occurring at Hoosier Pass on State Highway 9 at mile marker 74.8 was inspected June 8, 2009 (Steve Laudeman, Aziz Khan, Alan Lisowy, John Hart, D. V. Griffiths, Panos D. Kiousis, Xiaoxia Zhou, Jared Stewart). The tension cracks at this slide cross the entire width of the roadway and the toe was found approximately 250 ft below the crest of the slope as shown in Figure 11. This slide presented a deep failure over a sloped underlying base.

Figure 11. Pavement distress at Hoosier Pass.

4.4 Muddy Pass

As part of a two-day site visit, July 30 and 31, 2009, Muddy Pass Slide (located on US 40 at approximately mile marker 157) was the first site investigated (Steve Laudeman, Aziz Khan, Del French, Rex Goodrich, John Hart, Alan Lisowy, Panos D. Kiousis, Jared Stewart). The slide was initially observed in May 2006 after a realignment of US-40. The slide begins about 50 ft south of the highway and is roughly 125 ft wide on the south shoulder and 200 ft wide on the north

Figure 12. Aerial extents of the Muddy Pass slide.

shoulder, as shown in Figure 12. The toe is not apparent; it is believed to be near, or in, Muddy Pass Lake. Signs of distress were observed on the north side of the highway where the ground appears to have sunken as much as two ft in some locations. Tension cracks were found surrounding the slide mass and descending towards the lake.

The slide was first investigated by the CDOT Geotechnical Program shortly after it was first observed in May 2006. Four borings were taken using a hollow stem auger. The investigation determined material types, depth to bedrock, and depth to groundwater. The approximate locations and the depth to bedrock are shown in Figure 13. The materials consist of 13 to 17 ft of stiff to very stiff clay above weathered claystone and shale bedrock on the south side of the highway; and 12 ft of medium dense, silty sand were found over 27 ft of the native clay overlying the shale bedrock on the north. Figure 13 provides a generalized cross section of the slide. Two inclinometers were also installed at this time. The data from the inclinometer shows the slip surface largely parallel to the bedrock. The slip surface, as calculated by the CDOT Geotechnical Program, is shown in Figure 13.

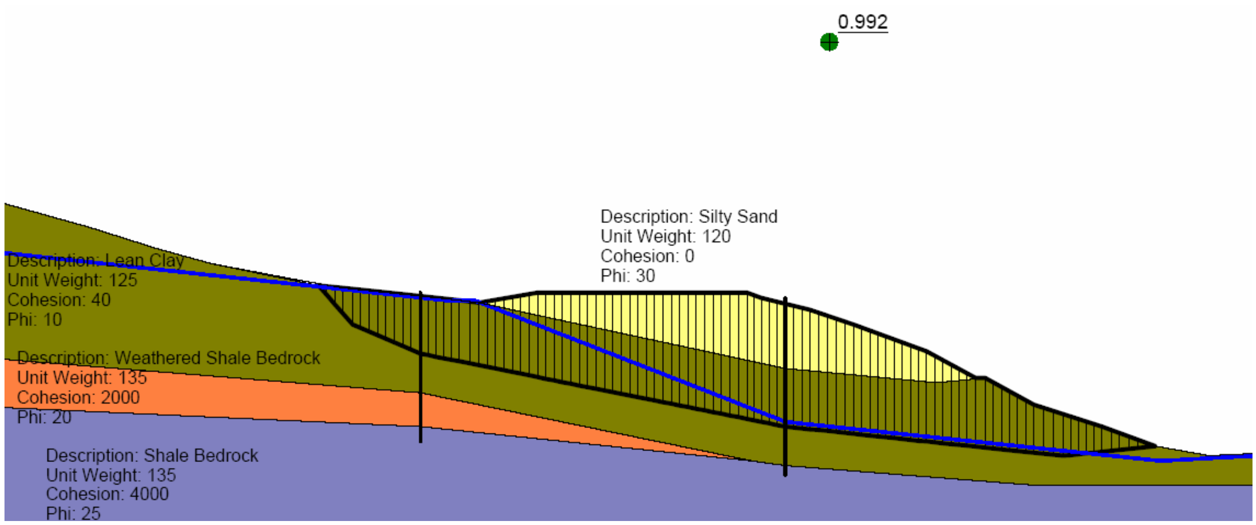

Figure 13. Soil stratification of the Muddy Pass slide.

Muddy Pass is potentially an appropriate site to investigate in the Phase II of this research project. It is a well documented slide with existing subsurface exploration and three years of inclinometer data. Furthermore, the slide is flat enough on either side of the highway to accommodate a pile driving truck. The depth of the slip surface is a concern. However, as in the case of the Rye site, the three dimensional nature of the slide may be advantageous for a more comprehensive study as will be discussed in the recommendations section of this report.

4.5 Douglas Pass

Three sites were visited along State Highway 139 going south from Rangely towards Grand Junction. The first site visited here (located at approximately mile marker 36) had previously been stabilized unsuccessfully before using driven piles. The first attempt used small box-section piles that were driven into the failing slope, but later failed (Figure 14). The second attempt made use of 12 inch deep H-Piles spaced at 6 ft. Lagging, composed of guardrail posts, was also installed to a depth of 10 ft as shown in the same figure. The piles were installed in 2004 and appear to be working well. The stiffness of the lagging is sufficient to provide support even at 12 foot spacing as is shown in Figure 15 where it is clear that the lagging support by the H-piles does not occur at the intended 6 foot interval. Nevertheless, it appears that the runoff has caused significant washout behind the lagging and piles, and may soon become a problem at this site.

Figure 14. Stabilized site at Douglas Pass, showing failed first attempt.

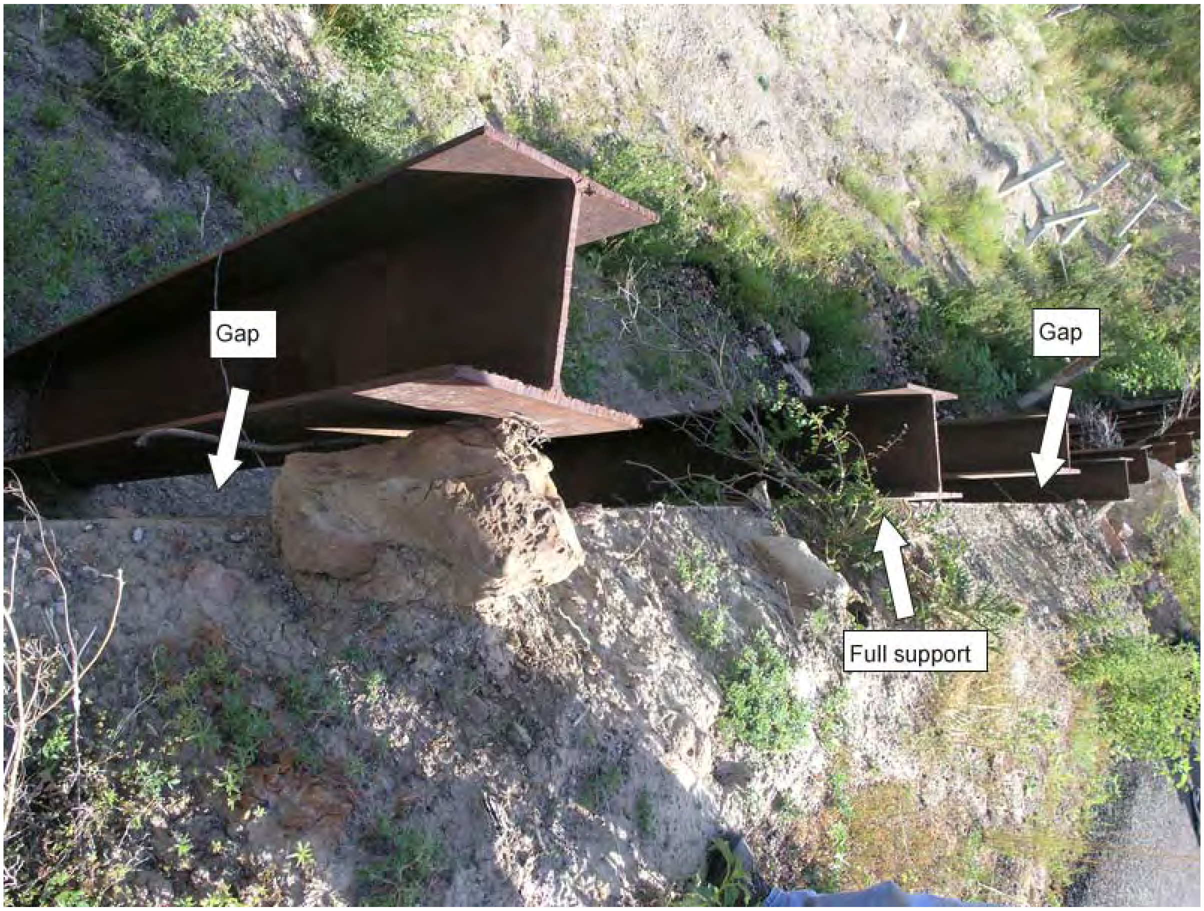

Figure 15: Stabilized site at Douglas Pass, showing incomplete lagging support.

The second slide visited at Douglas Pass was located approximately 1 mile south of the mile marker 36 slide. The sliding at this slide had occurred a decade or two earlier. The slide mass was so large that it could be made apparent only when viewed from some distance. The slide had started close to the ridge of the mountain and had slid all the way through the base, damming up a creek in the lower valley. The third slide visited was a very small surficial slide that had been previously stabilized using launched soil nails. Figure 16 shows an approximate schematic of the installation at Douglas Pass. Actual installation details are unknown.

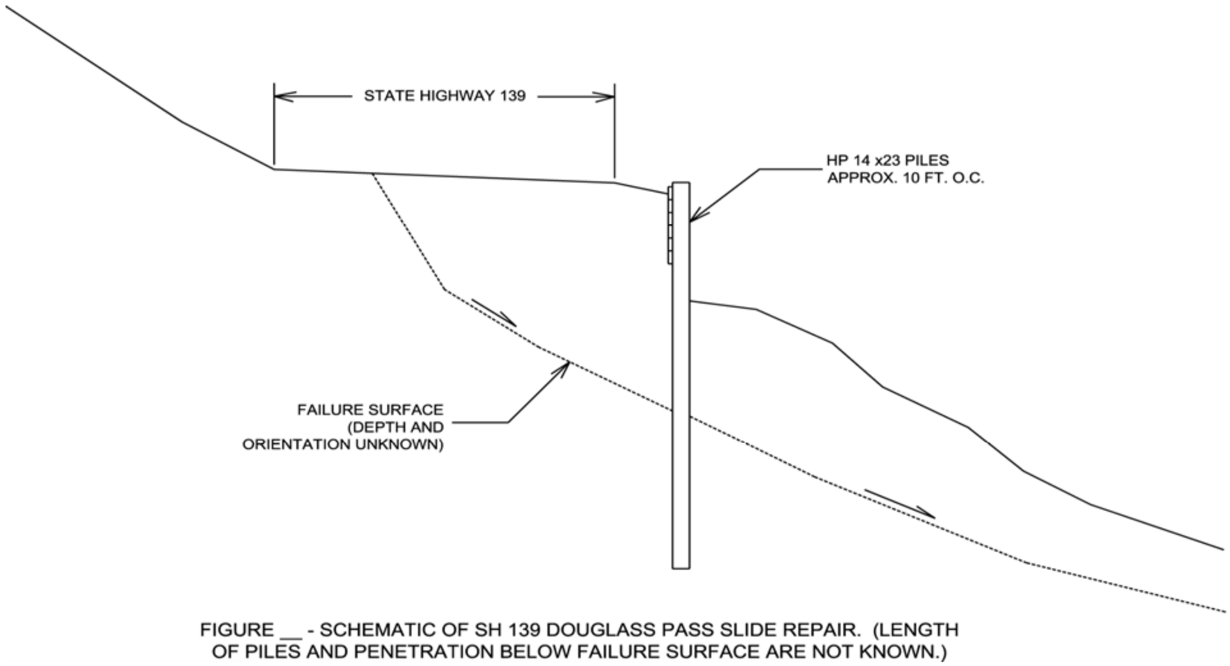

Figure 16. Approximate schematic of SH-139 Douglas Pass slide repair. Courtesy of Steve Laudeman.

5.0 COST COMPARISON ANALYSIS

The challenge of performing a cost comparison on different shallow slide mitigation methods is that the cost of mitigation methods is site specific. Where one variety of mitigation may be more economic at one particular slide, another method may be better at another site. The reasons for this vary but are generally dependent on the material present, slope geometries, the location of the slide, and the availability of repair materials. A coarse cost analysis for several stabilization methods in mountainous Colorado areas, limited to slides 10 to 20 feet deep, is shown in Table 1 [15]. The comparison stems from costs per square foot of reinforcement provided.

Table 1. Rudimentary cost comparison of several earth retention systems.

Installation Method	Low Cost	High Cost
Soil Nail	25 $^{\$}/_{SF}$	40 $^{\$}/_{SF}$
Soldier Beams (with Drilled Shafts) and Lagging, no Tieback	35 $^{\$}/_{SF}$	45 $^{\$}/_{SF}$
Soldier Beam (with Drilled Shafts) and Lagging, Tiebacks	25 $^{\$}/_{SF}$	35 $^{\$}/_{SF}$
Drilled Shaft Wall/Soldier Caisson	60 $^{\$}/_{SF}$	100 $^{\$}/_{SF}$
Driven Pile and Lagging	15 $^{\$}/_{SF}$	20 $^{\$}/_{SF}$
Launched Soil Nail Walls	10 $^{\$}/_{SF}$	15 $^{\$}/_{SF}$

A more detailed cost analysis for three of these methods (launched soil nails, driven piles, and drilled shafts) follows. The costs estimated here come from the *RS Means: Building Construction Cost Data 2010* [16] and the *Application Guide for Launched Soil Nails: Volume I* [17]. To insure that the costs are comparable across the systems investigated, each system is designed for the same slope. The driven piles and drilled shafts are designed as cantilever beams for a depth equal to the slide plane depth at the shoulder of the road. The launched soil nails are designed using a method detailed in New York State DOT report, *Geotechnical Design Procedure Manual: Design Procedure for Launched Soil Nail Shallow Slough Treatment* [18].

The slope designed for is shown in Figure 17 and has a 2:1 slope. The material parameters used are; γ = 120 pcf, ϕ’ = 20˚, and c’ = 30 psf. The factor of safety was found to be equal to 1.05 using Bishop’s method. The slide is assumed to have no three dimensional characteristics and represents a plane strain case.

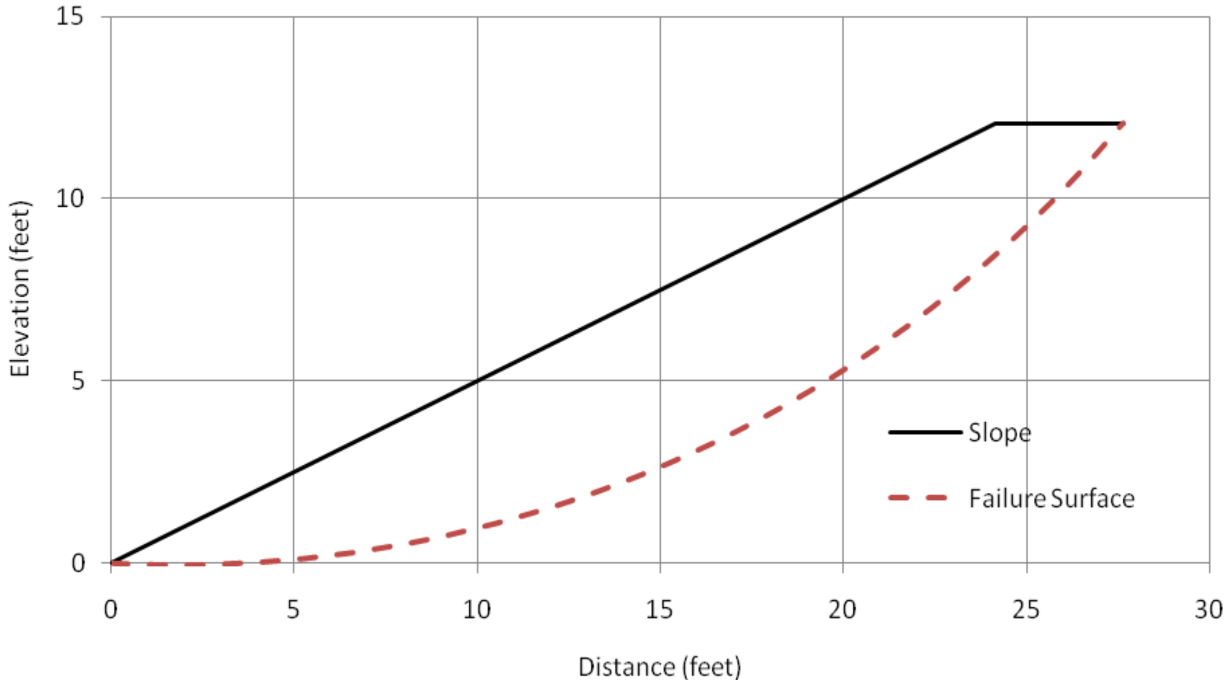

Figure 17. Failing slope section detail.

5.1 Driven Pile and Drilled Shaft Design

Driven pile and drilled shaft designs were both performed in a similar method. In this method, it was assumed that the piles and shafts behaved as a cantilever wall at the slopes crest. This doesn't accurately reflect the real forces that these reinforcement systems would need to resist and only serves as a coarse approximation. The piles were assumed to be inserted at the crest of the slope, which approximates the shoulder of a road above the slope. The depth to the slip surface at this location on Figure 17 is 3.62 ft. Using the cantilever assumption, the peak shear and moment values and required installation length are determined for piles at different spacing and pile diameters as shown in Table 2. . The moments and shear acting on the installed piles and shafts is based on the diagram shown in Figure 18 from the California Trenching and Shoring Manual [25].

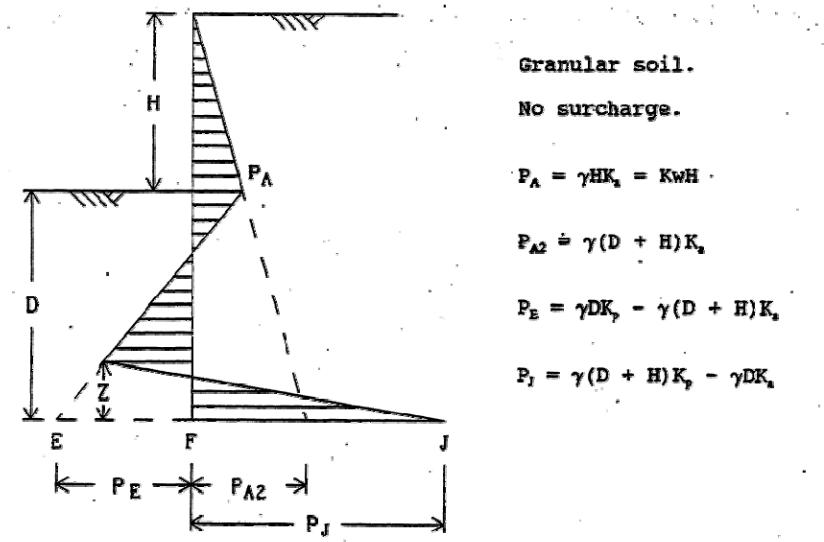

Figure 18. Cantilever pile loading, basis for driven pile and drilled shaft design, from California Trenching and Shoring Manual [25].

Table 2. Peak moments and shears for various pile spacings and diameters.

Spacing (feet)	**Diameter** (inches)	**M_{peak}** (k feet)	**M_{peak} Location** (feet)	**V_{peak}** (k)	**V_{peak} location** (feet)	**Installation Length** (feet)
6	10	12.1	8.18	4.65	10.6	12.3
6	12	11.7	7.94	4.94	10.2	11.8
6	14	11.4	7.75	5.21	9.88	11.3
6	16	11.2	7.60	5.47	9.64	11.0
6	18	11.0	7.47	5.72	9.44	10.7
8	10	17.2	8.62	5.65	11.3	13.3
8	12	16.5	8.34	5.99	10.8	12.6
8	14	16.0	8.12	6.30	10.5	12.2
8	16	15.6	7.94	6.59	10.2	11.8
8	18	15.3	7.79	6.86	9.95	11.4
10	10	22.7	9.39	6.60	12.4	14.1
10	12	21.7	8.69	6.98	11.4	13.4
10	14	20.9	8.44	7.32	11.0	12.9
10	16	20.3	8.24	7.64	10.7	12.4
10	18	19.9	8.08	7.94	10.4	12.1
12	10	28.6	9.76	7.55	13.1	14.9
12	12	27.2	9.01	7.93	11.9	14.1
12	14	26.2	8.73	8.30	11.5	13.5
12	16	25.4	8.52	8.65	11.0	13.1
12	18	24.7	8.34	8.98	10.8	12.6

The driven piles were selected from the set of H-Piles shown in Table 3. Table 3 also shows section properties and cost per vertical linear foot, as provided in *RS Means: Building Construction Cost Data 2010* [16]. The piles are then selected for the loads provided in Table 2, and the optimal pile for each spacing is shown in Table 4.

Table 3. Section properties and costs.

Section	**Z** (in^3)	**b_f** (in)	**D** (in)	**t_w** (in)	**A_w** (in^2)	**Cost (Depending on Market)** ($^{\$}/_{VLF}$)
HP10X42	48.3	10.1	9.70	0.415	4.03	$ 32.50
HP10X57	66.5	10.2	9.99	0.565	5.64	$ 40.00
HP12X53	74	12.0	11.8	0.435	5.13	$ 38.50
HP12X74	105	12.1	12.1	0.605	7.32	$ 50.50
HP14X73	118	14.6	13.6	0.505	6.87	$ 51.00
HP14X89	146	14.7	13.8	0.615	8.49	$ 60.00
HP14X102	169	14.8	14.0	0.705	9.87	$ 67.00
HP14X117	194	14.9	14.2	0.805	11.43	$ 75.00

Table 4. Selected sections and costs for driven piles.

Spacing (feet)	**Width** (inches)	**M_u** (k feet)	**V_u** (k)	**Section**	**Installation Length** (feet)	**Cost/Pile** ($)	**Cost/LF** ($)
6	10	19.4	7.44	HP10X42	12.3	$400	$67
8	10	27.5	9.04	HP10X42	13.3	$432	$54
10	10	36.3	10.6	HP10X42	14.1	$459	$46
12	10	45.8	12.1	HP10X42	14.9	$485	$41

The driven piles have been designed according to Load and Resistance Factor Design in the *Specification for Structural Steel Buildings* [19], where $\phi_B = 0.90$ for bending, and $\phi_V = 1.00$ for shear. The drilled shafts were designed in accordance with the *Building Code Requirements for Structural Concrete* [20], using the column interaction charts provided in *Design of Reinforced Concrete: ACI 318-05 Code Edition* [21]. The shafts were selected to be of 10, 12, 14, 16, or 18 inches in diameter. The costs associated with each shaft diameter are shown in Table 5. The drilled shafts were designed using Load and Resistance Factor Design. Table 6 shows the required amount of reinforcement for a pile under each condition. As the concrete is cast against, and permanently exposed to the earth, the required cover of the reinforcing bars is 3 inches. For practical purposes this eliminates piles of 10 and 12 inch diameters.

Table 5. Drilled shaft estimated cost (depending on market).

Cast in place drilled shafts no casing or reinforcing	
12" Diameter	23.00 $^{\$}/_{V.L.F.}$
18" Diameter	44.50 $^{\$}/_{V.L.F.}$
Add Reinforcing Steel	0.80 $^{\$}/_{Lb.}$

Table 6. Resisting pile design and costs.

Spacing (feet)	Diameter (inches)	M_u (k feet)	Installation Length (feet)	ϕ	Rn = $^{Mu}/_{f'c\ Ag\ h}$	ρ	A_s (in^2)	$Cost_{concrete+labor}$ ($)	$Cost_{steel+labor}$ ($)	Cost/Pile ($)	Cost/LF ($)
6	12	11.7	11.8	0.9	0.026	0.01	1.13	$271	$45	$316	$53
6	18	11.0	10.7	0.9	0.007	0.01	2.54	$476	$93	$569	$95
8	12	16.5	12.6	0.9	0.036	0.01	1.13	$289	$48	$337	$42
8	18	15.3	11.4	0.9	0.010	0.01	2.54	$507	$99	$606	$76
10	12	21.7	13.4	0.9	0.048	0.01	1.13	$308	$52	$360	$36
10	18	19.9	12.1	0.9	0.013	0.01	2.54	$538	$105	$643	$64
12	12	27.2	14.1	0.9	0.060	0.01	1.13	$324	$54	$378	$32
12	18	24.7	12.6	0.9	0.016	0.01	2.54	$561	$109	$670	$56

5.2 Launched Soil Nail Design

Launched soil nail design is based on design charts provided in *Application Guide for Launched Soil Nails: Volume I* [17] and *Geotechnical Design Procedure Manual: Design Procedure for Launched Soil Nail Shallow Slough Treatment* [18]. The design charts were developed using the simplified wedge analysis method. Figure 19 shows the appropriate design chart for the slope shown in Figure 17.

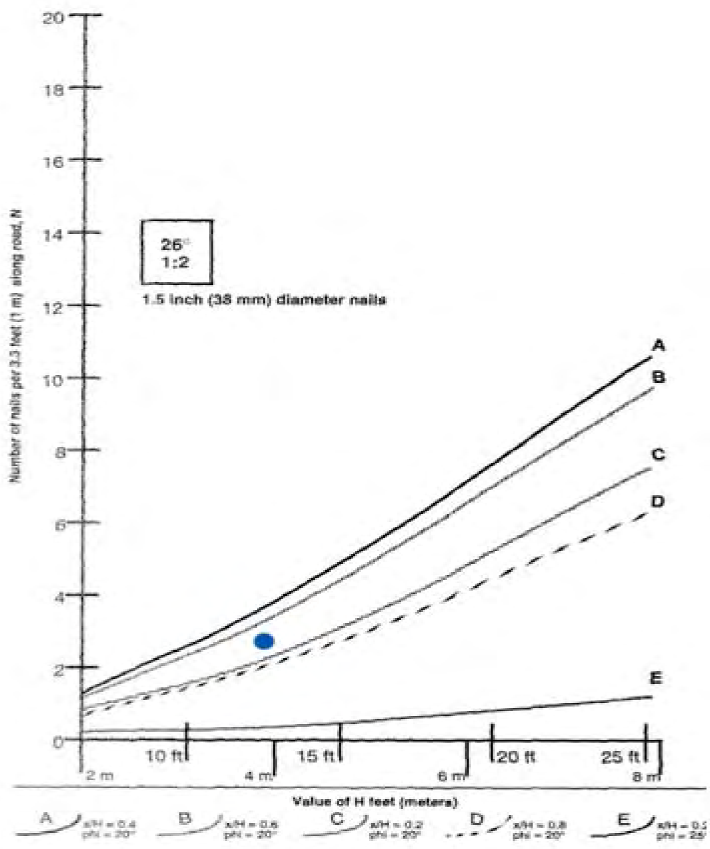

Figure 19. Design chart for a 2H:1V slope.

The slope shown in Figure 17 has a ratio X/H = 0.2899, where X is 3.5 feet and is the distance from the slope crest to the crack, and H is 12.07 feet which represents the vertical height of the failed region. This is a different ratio than the aspect ratio, D/L, defined in Section 2. The blue dot in Figure 19 is the point which represents these values. This point shows that 3 nails are needed per 3.3 feet of road stabilized. The price given by Application Guide for Launched Soil Nails: Volume I is between \$80 and \$135 per nail [17]. Incorporating a 2″ thick shotcrete surfacing over the stabilized area adds \$65 per nail [16]. This gives a cost between \$130 and \$180 per linear foot of this slide stabilized.

6.0 FINITE ELEMENT ANALYSIS

Some preliminary software development has been performed, using the finite element method, to better understand the potential failure mechanisms and load transfer occurring in pile-reinforced slopes. Specifically, if calibrated to actual field observations of pile performance, the finite element method could be used to predict pile/slide performance under a wide variety of configurations and conditions. This work led to a publication by Griffiths *et al.* [22]. The developments involved making modifications to existing elasto-plastic finite element slope stability codes (e.g. Smith and Griffiths [23]) in order to assess the influence of pile length and location on the slope factor of safety for a variety of soil types. The methodology shows great promise as a diagnostic tool for proposed pile reinforced slope configurations. The full paper is included in Appendix B.

7.0 SUMMARY AND CONCLUSIONS

Stabilizing piles appear to be a possible solution to the challenges in maintaining slope stability on Colorado's highway embankments. The following conclusions have been drawn about the performance of slope stabilizing piles:

- Slope stabilizing piles improve the shear capacity of the slope by reinforcing the slip surface.
- Slope stabilizing piles can provide effective solutions to slope stabilization problems where space and access restrictions that typically occur in highway embankments render alternate approaches unfeasible.
- Slope stabilizing piles have not been thoroughly researched, and, while they show significant benefits over the current status-quo, they are not fully understood.
- Slope stabilizing piles have a cost similar to other low impact landslide mitigation techniques.
- Slope stabilizing piles modeled using finite elements show that piles can provide significant improvements to the factor of safety of a slope. This improvement depends upon the location and length of the installed pile. The improvement forced the slip surface deeper – so as to avoid the pile. This improvement was shown to continue up until the point at which the slide transferred to a shallower location, circumventing the pile reinforcement entirely.

Slope stabilizing piles show significant promise however, not enough is currently understood about their behavior to make them an engineered solution.

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8.0 RECOMMENDATIONS

Given the current state-of-the-art, and the rather sparse experimental observations on slope stabilization for problems of significance to Colorado mountain highways, it is recommended that the current project be extended to Phase II.

The main goal of the research of Phase II of this study will be the instrumented mitigation of one or two (based on available budget) highway embankments with clearly defined three-dimensional existing slope failures (Figure 20). Piles will be driven along the length of the highway to cover the entire length of failure as shown. Important issues of study are the following:

1. Piles close to the edges of the failure plane (section C-C) are charged to prevent a shallower failure than those in the middle of the failure plane (section A-A).

2. The performance of piles is expected to be different as we transition from edge piles to middle piles. More specifically it is expected that piles at the edge will be more effective than piles in the middle. By properly instrumenting representative piles in various regions (sections A-A, B-B, and C-C), we shall develop a better understanding on their ability to prevent slides based on slide depth. Such observations can provide us with clear guidelines of the performance of pile stabilization of slopes as a function of failure depth.

3. Depending on budget, additional numerical and physical tests will be performed on failing slopes to examine the effects of pile spacing, pile size, pile position within the slide profile, preexisting slip lines, etc. The effect of preexisting slip lines on the effectiveness of pile stabilization has not been examined in the past. It will be examined here numerically as follows:

    a. In the analysis of a failed site, use the finite element method (see Appendix B) to reproduce the observed failure mechanism.

    b. Introduce reduced (remolded) strength along the slip line.

c. Examine the effects of driven piles. Determine the required amount of strength reduction on step b to force failure along the same slip surface after the pile installation.

The outcome of this study will be the enhancement of the state-of-the-art on slope stabilization using piles, and a set of guidelines to determine when this method of stabilization is expected to be effective and economically feasible, leading to a slope stabilization design method that can be used by CDOT Maintenance and Engineering staff.

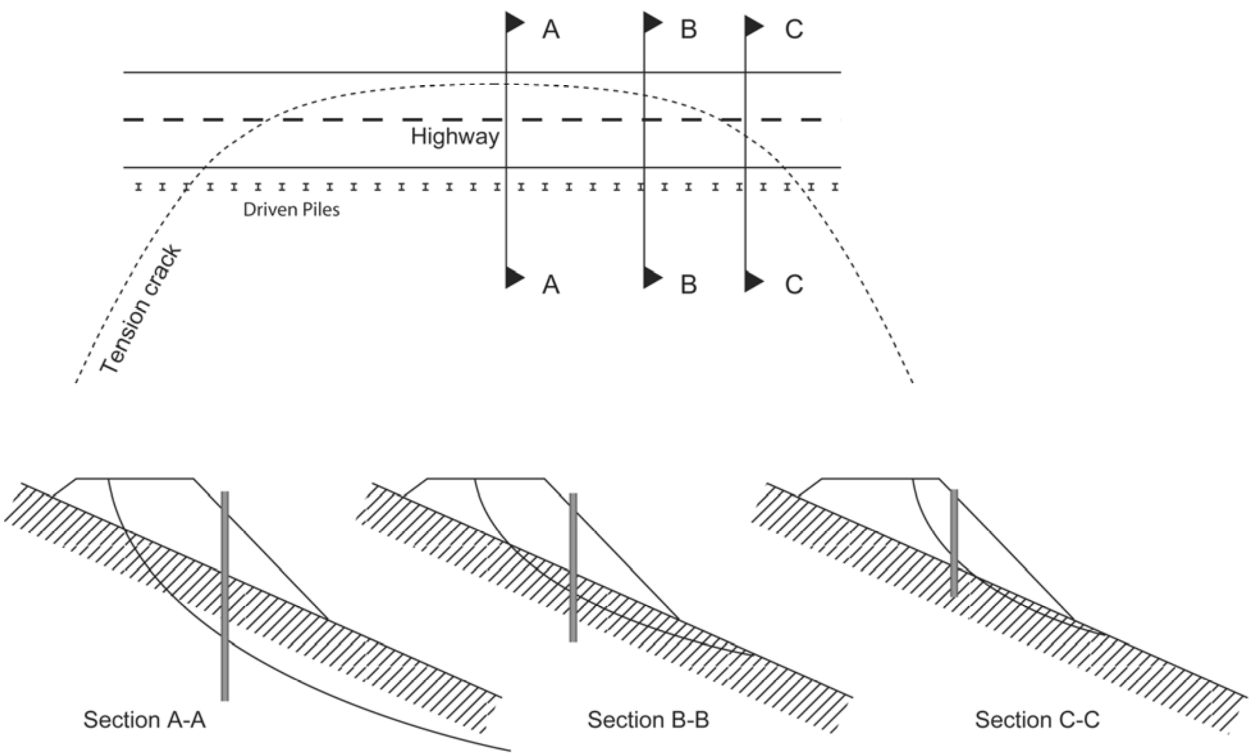

Figure 20. Proposed approach.

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APPENDIX A: SURVEY RESULTS

Department	Responded	Who	Used	Success	Research
Alabama					
Alaska					
Arizona	12/9/2009	Norman Wetz	No		No
Arkansas	3/16/2009	David Ross	No		No
California	7/27/2009	Mohammed Islam	No		Yes
Connecticut	7/23/2009	Leo Fontaine	No		No
Delaware					
Florida					
Georgia	3/16/2009	Thomas Scruggs	Yes	Some success, very good at times	No
Hawaii	3/19/2009	Herbert Chu	No		No
Idaho	7/23/2009	Tri Buu	No		No
Illinois	3/25/2009	Bill Kramer	Yes	Yes, when the slide is shallow and the underlying soil is penetrable but strong	Yes
Indiana					
Iowa	11/17/2009	Bob Stanley	Yes	Yes	Yes
Kansas	7/30/2009	James Brennan	Yes	Good performance, but expensive	Yes
Kentucky	3/13/2009	Bart Ascher	Yes	No	No
Louisiana	7/30/2009	Gavin Gautreau	Yes	Good performance, but expensive	Yes
Maine	3/16/2009	Kitty Breskin	Yes	Very successful	No
Maryland	7/24/2009	Xin Chen	No		No
Massachusetts	7/31/2009	Peter Connors	No		Yes
Michigan	3/20/2009	Robert Endres	No		No
Minnesota	8/18/2009	Gary Person	No		No
Mississippi	11/18/2009	James Williams	Yes	Very successful, pricey	No
Missouri	4/16/2009	Thomas W. Fennessey	Yes	Plastic Pins have worked well	Yes
Montana	3/16/2009	Richard Jackson	Yes	Did not perform well; additional ROW generally available	No
Nebraska	3/18/2009	Omar Qudus	No		No
Nevada	7/29/2009	J. Mark Salazar	No		Yes
New Hampshire	3/25/2009	Charles Dusseault	No		No
New Jersey	12/10/2009	Kuang-Yu Yang	No		No

New Mexico	7/23/2009	Bob Meyers	No		No
New York	3/16/2009	Bob Burnett	Yes	Performed well in tight quarters; too expensive to use often	No
North Carolina					
North Dakota	3/13/2009	Jon Ketterl	Yes	Somewhat	No
Ohio	3/16/2009	Monique Evans	No Response		No Response
Oklahoma					
Oregon	3/18/2009	Matthew Mabey	No Response		No Response
Pennsylvania	11/20/2009	Bonnie Fields	No Response		No Response
Rhode Island	11/24/2009	Robert Snyder	No		No
South Carolina	4/9/2009	Jeff Sizemore	Yes	Ok in non-critical applications	Yes
South Dakota	7/29/2009	Kevin Griese	Yes		No
Tennessee	7/23/2009	Len Oliver	Yes	Adequate, not always the best option	Yes
Texas	3/17/2009	Mark McLelland	Yes	Poorly, mudflow type failures	No
Utah	9/2/2009	Darin Sjoblom	No		Yes
Vermont	7/23/2009	Christopher Benda	Yes	Very well	No
Virginia	7/29/2009	Stanley L. Hite	Yes	Good, with the exception of high moisture sites	Yes
Washington	7/29/2009	Steve Lowell	No		No
West Virginia	3/16/2009	Donald Williams	Yes	Very few failures	Yes
Wisconsin	7/28/2009	Bob Arndorfer	No		Yes
Wyoming	3/13/2009	Jim Coffin	Yes	Yes, below 25' to failure	No
FHWA	7/23/2009	Matthew DeMarco	No		No

APPENDIX B: FINITE ELEMENT ANALYSIS OF PILE REINFORCED SLOPES

A comparison of numerical algorithms in the analysis of pile reinforced slopes

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Abstract

The paper describes the influence of pile reinforcement on the stability of slopes through numerical analysis. Included in the paper is some discussion of the modifications made to include pile reinforcement in an existing finite element slope stability program that uses the strength reduction method. Then the finite element program developed is compared for accuracy in the solution of the piled slope problem with a popular proprietary code that uses the finite difference method. Finally, parametric studies are presented to assess the influence of pile location and length on the slope stability.

1 Introduction

Piles have been used in geotechnical engineering to stabilize slope for many years and the methodology has been accompanied by a significant bibliography (e.g. Ito and Matsui 1975; Jeong et al. 2003; Won et al. 2005; Chow 1996; Hassiotis et al. 1997; Harry 1995; Ito et al. 1981; Poulos and Chen 1997). In the past, methods of analysis of pile-reinforced slopes have often used limit equilibrium methods, where soil–pile interaction was not properly considered (e.g. Won et al. 2005). Recently, with rapid development of computer techniques, numerical methods using either finite element or finite difference methods have been widely applied in slope engineering, and have been shown to offer many advantages over limit equilibrium method (Griffiths and Lane, 1999), such as the ability to develop the critical failure surface automatically with fewer assumptions.

In this paper, we will make some modifications for an existing finite element slope stability program that uses the strength reduction method, to include pile reinforcement. Results obtained using the developed finite element program are then compared for accuracy in the solution of the piled slope problem with a popular proprietary code that uses the finite difference method. Finally, parametric studies are presented to assess the influence of pile location and length on slope stability and the factor of safety.

2 Finite element slope stability program including pile reinforcement

The programs used in this paper are based on Program 6.3 in the text by Smith and Griffiths (2004), and have been modified to include the pile reinforcement in slope to form a new program (named p63_s). The program is for two-dimensional plane strain analysis of elastic perfectly plastic soils with a Mohr-Coulomb failure criterion utilizing eight-node quadrilateral elements with reduced integration (four Gauss points per element) in the gravity loads generation, the stiffness matrix generation and the stress redistribution phases of the algorithm. The soil is initially assumed to be elastic and the model generates normal and shear stresses at all Gauss points within the mesh. These stresses are then compared with the Mohr-Coulomb failure criterion. If the stresses at a particular Gauss point lie within the Mohr-Coulomb failure envelope, then that location is assumed to remain elastic. If the stresses lie on or outside the failure envelope, then that location is assumed to be yielding.

The pile is simulated by a beam-rod element, based on Program 4.3 in the text by Smith and Griffiths (2004) which contains three degrees of freedom for each node (two translational and one rotational). The beam-rod element stiffness matrix is formed by superposing the beam and rod stiffness matrices and can sustain axial and transverse loads in addition to moments.

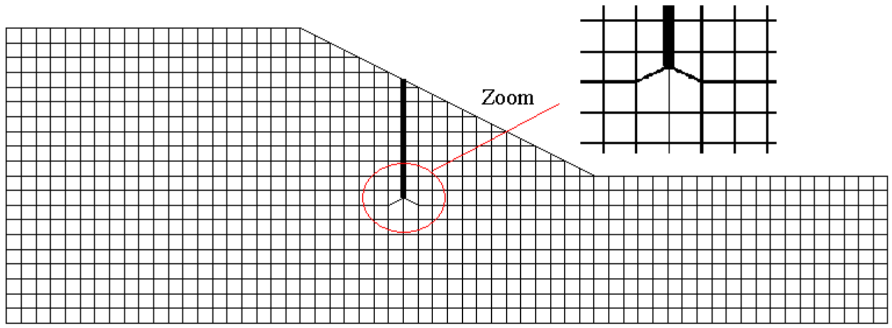

Fig.1 Numerical model for slope with pile reinforcement

In order to add a pile element to the slope, the following modifications were made,

(1) the coordinates of the mesh were adjusted to accommodate the lateral location and length of the pile as shown in Figure 1 ;

(2) the soil stiffness matrix `km` of elements adjacent to the pile were augmented by the pile element stiffness matrix `p_km`. Each slope element will usually be adjacent to two pile elements. For example as shown in Figure 2, `km` for slope element `iel` is augmented in its upper part.

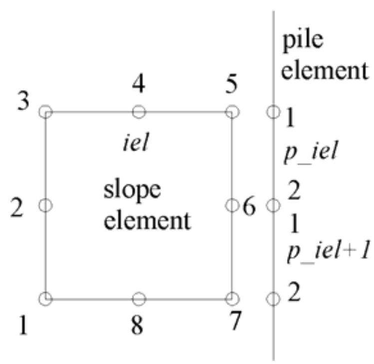

Fig.2 Local node numbering for soil and pile elements.

3 Validation for the program

3.1 Slope model

In order to validate the program p63_s, its calculated results are compared with those obtained using FLAC2D. Firstly, the same homogenous slopes are formed by two programs (p63_s and FLAC2D) as shown in Figures 3 and 4. The height of the slope is 10m, with a slope angle of 26.56° (2:1 gradient). Parameters of the slope are 20.0 kN/m^3 for unit weight, 1×10^5 kPa for elastic modulus, 0.3 for Poisson' ratio, 15.0kPa for cohesion, and 20.0° for friction angle. Parameters of pile are 0.62m for diameter D and 25×10^6kPa for elastic modulus E. Then axial rigidity EA and bending stiffness EI for the beam-rod elements can be formed by,

$$EA = E \cdot \frac{1}{4}\pi D^2 = 7.55\times10^6 \text{ kN}$$

$$EI = E \cdot \frac{\pi D^4}{64} = 1.81\times10^5 \text{ kNm}^2$$

In the actual situation, piles are driven periodically in the third direction, in which case the equivalent pile properties for plane strain analysis can be scaled as suggested by Donovan et al. (1984).

The slope model is fixed on the bottom boundary with vertical rollers on the side boundaries. The factor of safety (F) of a soil slope is defined as the number by which the original shear strength parameters must be divided in order to bring the slope to the point of failure. This method is referred to as the 'shear strength reduction technique' (Zienkiewicz et al. 1975, Griffiths 1980, Matsui and San (1992), Ugai and Leshchinsky (1995), Griffiths and Lane 1999).

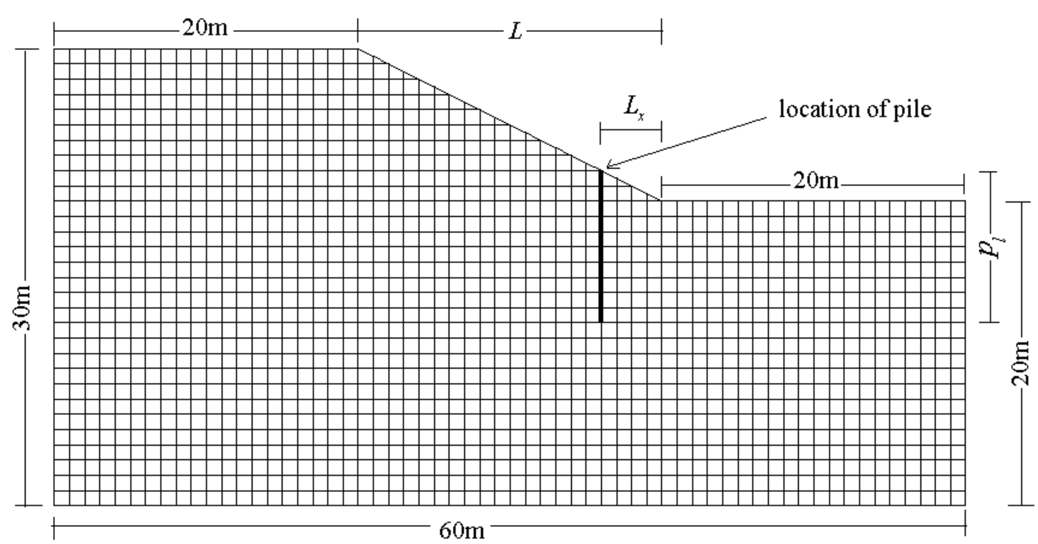

Fig.3 FE model for p63_s with 1510 elements and 4711 nodes.

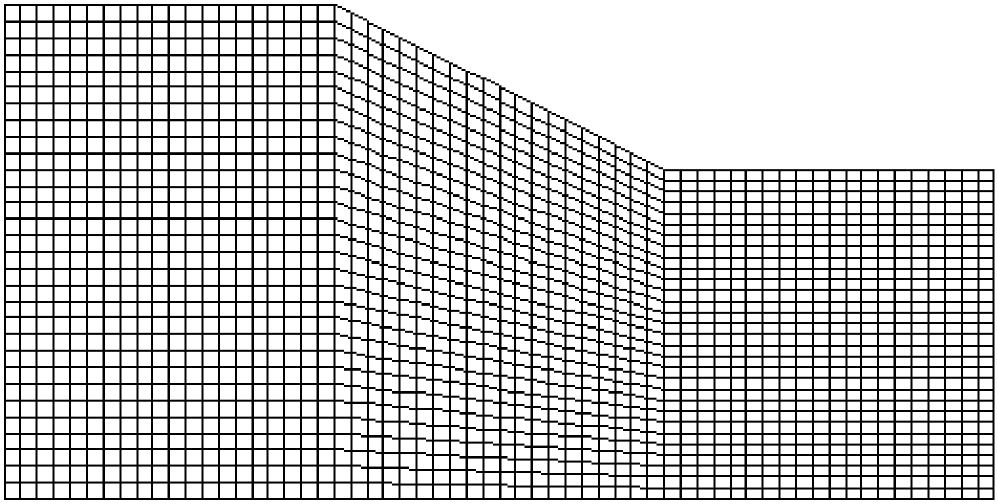

Fig.4 FD model for FLAC2D with 1800 zones and 1891 grid points.

3.2 Comparison

Comparisons are done for slopes reinforced by the pile with maximum length, results are shown in Tables 1, where L_x is the horizontal distance between pile location and the slope toe. It can be seen that the factor of safety *F* values from p63_s are similar to those from FLAC2D with p63_s giving slightly lower (conservative) values. When taking into consideration the CPU time required by each of the models on the same computer, both p63_s and FLAC2D take about 3 minutes per run.

Table 1. Comparison of results obtained by p63_s and FLAC2D for a slope reinforced by pile with maximum length $\left(p_l = 25\text{ m}\right)$

L_x/L	FLAC2D F_1	P63_s F_2	$(F_1 - F_2)/F_2 \times 100\%$
No pile	1.61	1.58	1.898
0.0	1.64	1.59	3.145
0.1	1.72	1.67	2.994
0.2	1.83	1.78	2.809
0.3	1.97	1.89	4.233
0.4	2.16	2.06	4.854
0.5	2.41	2.28	5.702
0.6	2.23	2.19	1.826
0.7	2.03	2.00	1.500
0.8	1.89	1.86	1.613
0.9	1.78	1.75	1.714
1.0	1.68	1.67	0.599

4 Parametric study

Initial studies indicated that the soil elastic modulus, pile elastic modulus and diameter had little effect on computed slope factor of safety so long as the pile elements were significantly stiffer than the soil modeling an essentially “rigid” pile.

Parametric studies were performed to assess the influence of pile location and length. The pile was assumed to be driven at varying distances from the slope toe, with L_x / L varied from 0 to 1, with the pile length varied from 6 m to 16 m at each location. The calculation model is the same as Figure 3.

The computed slope factor of safety by program p63_s are plotted in Figure 5 indicating that as L_x / L increases, the factor of safety initially rises and then falls. For shorter piles, e.g.

$6\text{m} \le p_l \le 8\text{m}$, the slope factor of safety reached its maximum value at $L_x/L \approx 0.3$ which is in the lower part of slope surface. For longer piles, e.g. $p_l \ge 10\text{m}$, the slope factor of safety reached its maximum value at $L_x/L \approx 0.5$ which is in the middle of slope surface. Ideally it appears the most effective location for the pile would be in the lower half of the slope, although this may not be a practical location for access.

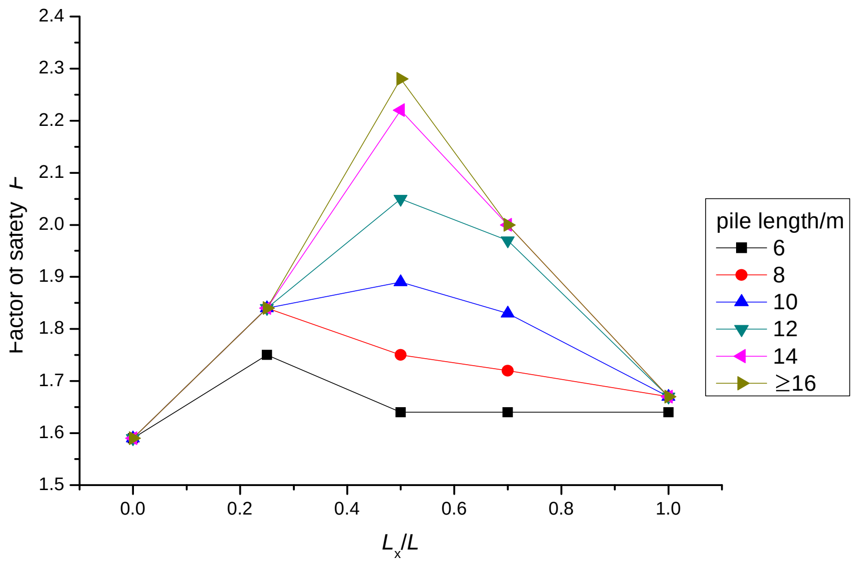

Fig.5 Effect of pile location and length on the slope factor of safety

The influence of pile length depends on its location. For the case considered, if the pile is driven at the slope vertex or toe its length has little effect on the slope factor of safety. If the pile is driven at the middle of slope surface ($L_x/L = 0.5$) however, its length has a considerable influence as shown in Figure 6. For pile lengths over a critical value (e.g. $p_l \geq 16\text{m}$), the factor of safety will remain constant,

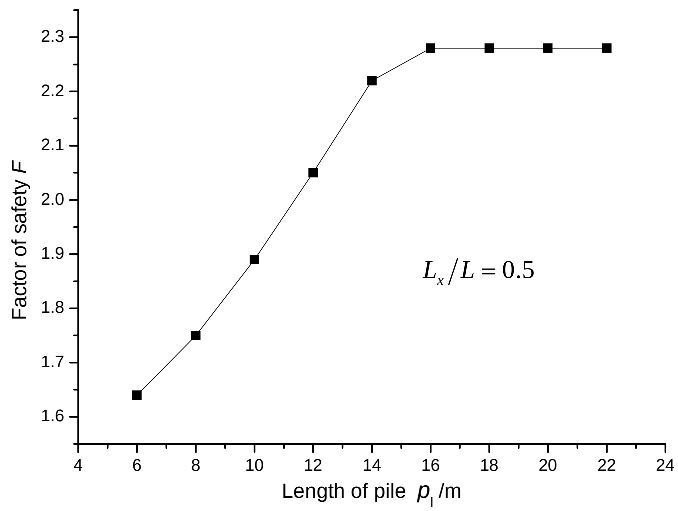

Fig.6 Relationship between slope factor of safety and pile length $\left(L_x/L = 0.5\right)$

In order to further study the effect of pile length on the potential slip plane when it is driven in the middle of slope surface, we obtained the potential slope slip surface from the graphical output of displacement vectors from p63_s as shown in Figure 7. The effect of pile length on the potential slip surface is shown in Figure 8 indicating how the surface is forced to run beneath the bottom of the pile. With no pile at all, the surface corresponds to a classical “toe” failure mechanism, but as the pile length is increased, the surface is forced ever deeper into the soil mass, with a corresponding increase in the factor of safety. When the pile length is greater than 14 m however, the potential slope failure surface radically relocates to a very shallow location just uphill of the pile tip. The change in location presumably occurs because the shallow mechanism requires less energy to develop than the much longer path navigating its way beneath the pile.

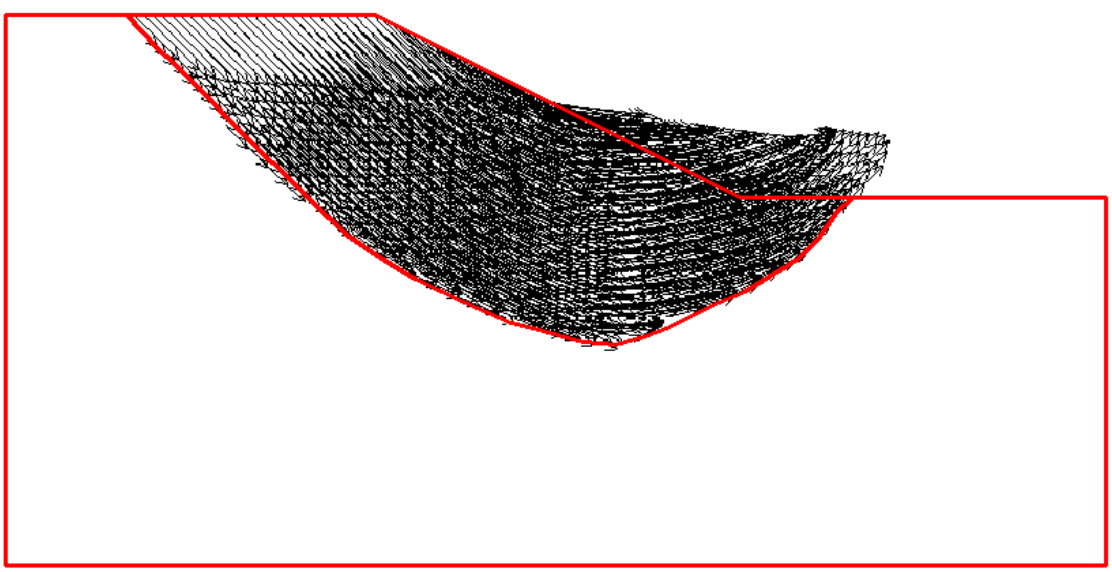

Fig.7 Displacement vectors of slope at failure.

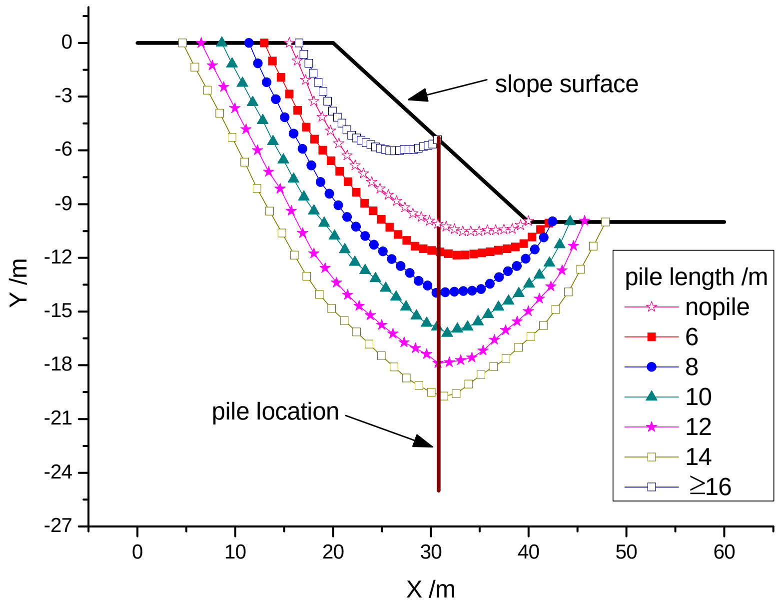

Fig. 8 Effect of pile length on the location of potential slip surfaces $\left(L_x/L=0.5\right)$

5. Conclusions

Parametric studies were performed to assess the influence of pile location and length on slope factor of safety. Although not necessarily a practical location for installation purposes, the optimal location of the pile was increased slope stability was found to be approximately half way down the slope. For a pile at this optimal location, it was observed that the factor of safety increased almost linearly with pile length until a critical depth was reached after which the factor of safety remained constant. This result was explained by studying the failure surface locations for different pile lengths. As the pile length was increased, the surface took an ever longer path as it passed below the pile tip causing the factor of safety to increase. A point was reached however as the pile length was further increased, when the energy required for the failure surface to pass below the pile tip became excessive, at which point the surface rapidly transformed to a much shallower location. Once this happened, further lengthening of the pile had not influence. Using strength reduction, a brief comparison between analyses performed using an FE program developed by the authors from the Smith and Griffiths (2004) system called p63_3 and FLAC2D indicated broadly similar results and run-times.

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