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OPTMIZATION OF STABILIZATION OF HIGHWAY EMBANKMENT SLOPES USING DRIVEN PILES (PHASE II – DEVELOPMENT AND VERIFICATION)

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16. Abstract

This study examines the feasibility of using driven piles to stabilize 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 can add significant shear resistance to a slope. The outcome of phase I of this study established, that within limits, the driven piles can improve the stability of failing slopes. The original intent of this project, which was field validation of the findings of phase I was abandoned due to lack of funding. Instead, a parametric study was performed to examine the effects of slope steepness, depth of failure surface, and driven pile stiffness in the ability of driven piles to mitigate failing highway embankments due to weaknesses developed due to excess underground water during the snow melt periods.

Based on the parametric study conducted here, it was concluded that driven piles have significant capacity to mitigate failing slopes of mountain highway embankments. Driven piles are more effective is failing slope mitigation when the slopes are less steep (3:1) irrespective of depth of failure. However, for steeper slopes (2:1), the ability of driven piles to provide efficient mitigation is limited only to shallow failures.

It was also concluded that additional numerical studies are needed to examine the effects of varying material parameters, slope steepness, slope height, and failure surface depths. Finally a field validation is very important to establish whether the findings of computational studies can be used directly, or if calibration adjustments are needed.

Implementation:

Based on the results of the study it is recommended that the Colorado Department of Transportation pursue further computational studies along with a field validation study.

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EXECUTIVE SUMMARY

This report presents the findings of Study No. 094.91, “Optimization of Stabilization of Highway Embankment Slopes using Driven Piles (Phase II - Installation, Monitoring, and Modeling)”. Embankment failures of Colorado’s mountain highways are a relatively frequent problem, xxxxx by large horizontal and vertical movements of slopes and settlement of the highway surface. One method that the CDOT maintenance crews have used with reasonable success to mitigate this problem is to drive piles along the shoulder of the road. This typically has been done without significant engineering. In the first phase of this study, CDOT funded the first phase of this study aiming to perform a literature review of stabilization methods, conduct a national survey of state DOT’s, review CDOT/Consultants 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. At the outset of that study, the CSM research team identified a failing highway embankment at Muddy Pass as a desired site to mitigate using driven piles. Towards this ultimate goal, the intended approach of the current study included the following scope: a) Perform a detailed analysis of the conditions at Muddy Pass, including the geometric and material characteristics of failure, then, finalize its selection as the main site to be investigated; b) Design a detailed mitigation plan for the site using driven piles; c) Design a detailed instrumentation plan to measure the performance of the driven piles as well as their overall effect on the slope mitigation; d) Implement the designed mitigation to the Muddy Pass slope; and e) Evaluate and perform analysis of the observed behavior of the mitigating piles and the slope.

Due to unforeseen difficulties in funding, the original objectives and goals could not be pursued. It was decided that the research emphasis of this project should be shifted toward a parametric computational study to optimize the slope stabilization using driven piles. The revised goals, were to examine the ability of driven piles to mitigate highway embankment failures as a function of a)

The slope of the embankment; b) The depth of the failed zone; c) The strength of the failed zone; and d) The stiffness and spacing of the driven piles. Because the revised research objective is based on computer simulations, it has the advantage that multiple and diverse conditions can be simulated at a fraction of the cost and time. Of course, this new approach suffers from the significant disadvantage of lack of field verification.

The mitigation approach examined here was based on the commonly applied approach of driving one row of piles at the top of the failing slope, close to the edge of the embankment. In most cases examined, this approach resulted in significant reduction of road surface settlement. This performance was used as the criterion of success of this method. In some cases, the use of driven piles resulted in a successful mitigation of the road surface settlement but created a secondary slope failure below the piles. Based on the parametric study conducted here, it was concluded that driven piles have significant capacity to mitigate failing slopes of mountain highway embankments, when certain criteria are satisfied. Driven piles are more effective in failing slope mitigation when the slopes are less steep (3:1) for all the failure surface depths that were examined. However, for steeper slopes (2:1), the ability of driven piles to provide efficient mitigation is limited only to shallow failures.

The analysis of the results indicate that road surface settlements of the mitigated slopes are the outcome of two separate deformation mechanisms: a) Soil mass movement that bends the piles, thus activating their resistance; and b) Soil mass movement that occurs as flow between the piles. Reduction of the former is achieved by increase of the pile stiffness, and as long as the piles do not fail, this movement is a one time occurrence. Reduction of the latter can be achieved by pile spacing adjustments. However, this is not address in this study, which was confined on a single pile spacing of 5 ft.

Based on the outcomes of the current study the following future actions are recommended: a) Additional numerical studies should be conducted to examine the effects of varying material parameters, slope steepness, slope height, and failure surface depths. B) An experimental field validation should be performed to establish whether the findings of computational studies can be used directly, or if calibration adjustments are needed.

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1. 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 have to address often. 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, and others).

When slope failures of highway-embankments are considered, the practical remedies are more limited because 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, when capable to produce sufficient improvements, appear to be the most realistic approach to achieving stability.

Driven piles have several advantages as a ground reinforcement technique:

1. Transportation departments are familiar with pile materials and pile driving equipment;
2. The piles can be installed quickly and provide immediate strength improvements;
3. The installation of the piles does not significantly disrupt traffic flow; and
4. 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 when mitigating 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. To date there is not a widely accepted verified design method for slope stabilization using driven piles.

1.2. Phase I Research Outcome

A feasibility level, phase I, study was carried by the Colorado School of Mines, and completed on December of 2010 [1]. It was concluded that stabilizing piles can be a solution to the challenges in maintaining slope stability on Colorado's highway embankments. More specifically, the conclusions of that study are summarized as follows:

- Slope stabilizing piles improve the shear capacity of the slope by reinforcing the slip surface, while transferring part of the load of the moving slope mass below the failing surface through the fixation of the piles.

- 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.

- Under certain circumstances, slope stabilizing piles can be cost-competitive to other low impact landslide mitigation techniques.

- Slope stabilizing piles modeled using finite elements show that driven piles, under certain conditions, 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.

1.3. Objectives

The objective of this study, as originally defined, was to develop an on-field verification process of the findings of the first phase of this study. To achieve this object the goals for Phase II were set as follows:

- Select a test site, where a stability failure had been established.

- Develop a computational model to examine the causes of failure.

- Develop a driven-pile mitigation plan based on the above model.

- Develop a pile instrumentation plan for the site.

- Implement the driven-pile mitigation plan.
- Evaluate the performance of the test site over one snow-thaw period.
- Compare and calibrate results to develop design methodology.

After an extensive search for sites, during the Phase I research, the Muddy Pass site was selected. This selection was based on the following facts:

- The site provides a clear failure pattern.
- It is accessible.
- It has a significant maintenance problem where large and continuous settlements were recorded every year during the snow-thaw period.
- There exists a reasonable amount of geotechnical information.
- The expressed interest in implementing a solution.

However, due to unforeseen difficulties, the original objectives and goals could not be pursued. The main difficulty was that the funding for the demonstration of the project could not be secured. The liability and risks associated with the implementation of a not fully tested method probably added concerns and reduced the interest.

A meeting was held between the research teams of CDOT and CSM on 6/8/1012 to address the new conditions. It was concluded that the research emphasis should be shifted toward a parametric computational study to optimize the slope stabilization using driven piles.

Because the revised research objective is based on computer simulations, it has the advantage that, as opposed to the original objective, multiple and diverse conditions can be simulated at a fraction of the cost and time. Of course, this new approach suffers from the significant disadvantage of lack of field verification.

2. DEVELOPMENT OF THE NEW OBJECTIVES

2.1. Scope

The intent of the redefined scope is to optimize the use of the available time and funds to maximize the outcome of this study. Based on the literature review, and the experiences of the first phase of this study the following features were identified as critical in this study:

1. *The slope of the embankment*, which influences the failure tendencies, as well as the ability of driven piles to retrofit the stability failure.

2. *The depth of the failed zone*, which influences the ability of the driven piles to retrofit the stability failure.

3. *The strength of the failed zone*, which influences the ability of the driven piles to retrofit the stability failure.

4. *The stiffness and spacing of the driven piles*, which influences their ability to retrofit the stability failure, as well as the cost of the retrofit approach.

In order to address the above issues efficiently within the constraints of this study, the scope of the research was defined as follows:

1. Examine two embankment slopes: 3:1 and 2:1.

2. Examine three depths of failure/weakened zones: Shallow, Intermediate and Deep.

3. Consider seven driven pile stiffnesses, corresponding to HP piles: HP 12x53, HP 12x84, HP 14x89, HP14x102, HP 16x141, HP16x183, and HP 18x204.

4. Allow one weanened zone strength, which is sufficient to prevent free flow of the the clay material between piles spaced 5 feet appart.

5. To address practical field issues and common practice, only the case of pile mitigation where the piles are driven in one row, close to the slope crest has been considered.

Examples of 3:1 slopes with shallow, intermediate, and deep failure zone depths as described above are presented in Figure 1.

The selection of slopes was based on the fact that embankments on mountainous roads are often required to be relatively steep. Embankments that are less steep than 3H:1V are often on flat lands, and have lower risk of failure during the snow melt period. Embankments that are steeper than 2H:1V are often avoided because they are difficult to access and maintain. Clearly, the steeper slopes have a larger tendency to fail, and are harder to mitigate.

The selection of depth of failure was based on the general experience of the researchers on typical failure depths, within the geometric constraints of the slopes examined here. There are two important characteristics of the failure zone depths examined here: As the depth of the shear failure zones transitions from shallow to deep, the slope of the slip surface becomes less steep, but the mass above the failure zone increases. The former makes mitigation easier, however, the latter makes mitigation harder.

The stiffer piles provide larger resistance to movement, and as a result, are more effective in mitigating a failing slope. However, the stiffer piles are also heavier, harder to drive (need a bigger hammer) and more expensive.

The material parameters were selected to be similar to those reported in the geotechnical report for the Muddy Pass. The strength of the weak zone was adjusted to be such that the 5-foot spacing of the driven piles was sufficient to prevent excessive material flow between the piles. The material modeling details are discussed later in this report.

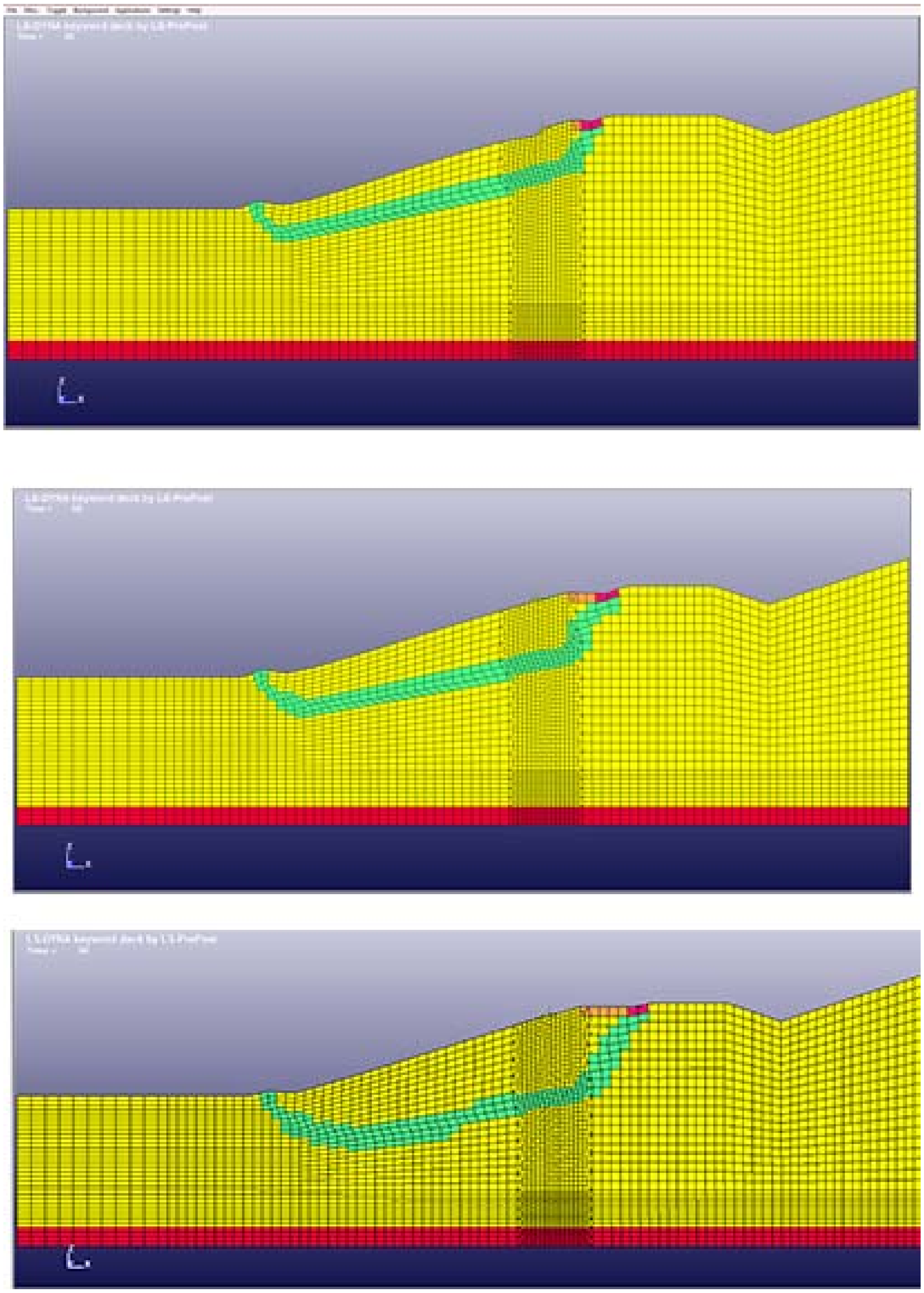


Figure 1: Embankments with 3:1 slope and varying depths of the failure zone

3. LITERATURE REVIEW

The subject of slope stabilization using driven piles has been studied to some extent in the literature, although studies that address the difficult conditions of an embankment on a mountainous sloped area are less common.

Typically, a slope failure is classified based on the aspect ratio of a slide or failure. As presented in Figure 2, 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. [2].

Slope geometry, soil type, degree of saturation, and level of seepage are among the factors influencing the size of shallow slope failures. Shallow slope failures often are parallel to the slope surface and are commonly analyzed as infinite slope failures. The depth varies depending soil type, slope geometry, and climatic conditions. Various depths were reported in the literature based on

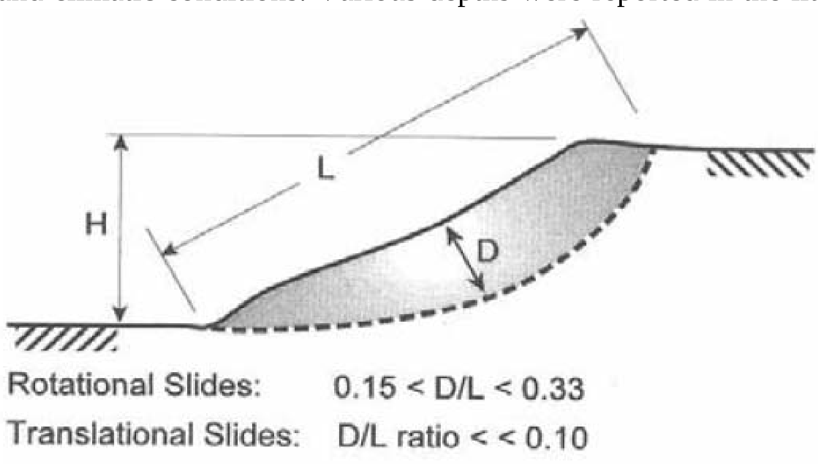


Figure 2. Aspect ratio of failure mass (Abramson et al., 2002).

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 2, 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. [2].

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 [2].

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 3.

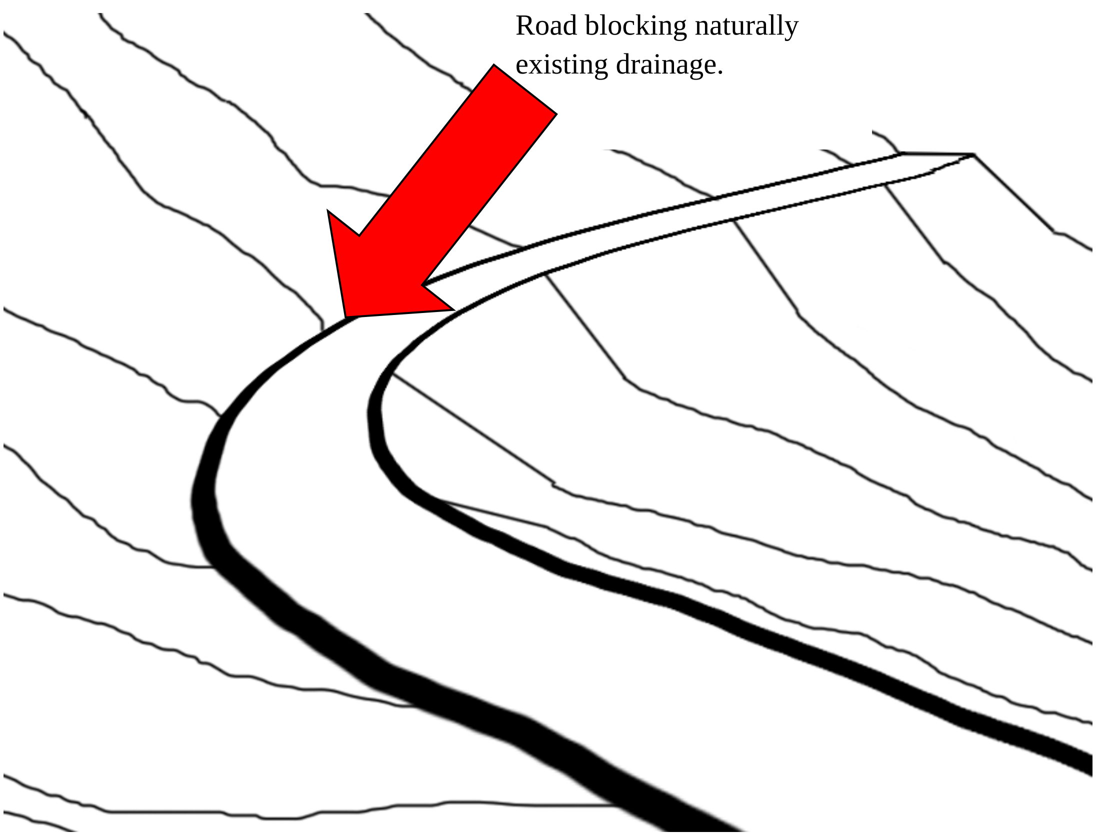


Figure 3. Naturally existing drainage blocked by road.

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 [3, 4] 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 their 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 [5] 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 [6] 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. [7] 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 [8]. The method presented by Hassiotis et al. [7] takes advantage of an extension of the force distribution calculated by Ito and Matsui [5] and Ito et al. [9] 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 [5] 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 [5,9], and, thus in the work of Hassiotis and co-workers [7,8]. 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 [10] 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. [11] 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 of a rebar or steel pipe embedded in a concrete shaft. The pins are installed by drilling. Pearlman et al. [11] 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 [5]. 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 [12] 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 [3,4], 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.

4. MODELING

4.1. Geometry modeling

Based on the outcomes of the first phase of this study, a three dimensional model was built and tested. The geometric model, examples of which are presented in Figure 1, resembles physically the Muddy Pass in its original unmitigated stage, which failed under its own weight due to the development of weakness zones in wet periods (Figures 4 and 5). A more detailed discussion of the Muddy Pass unmitigated stage is discussed in the final report of the Phase I of this study [1].

Figure 4: Aerial extents of the Muddy Pass slide

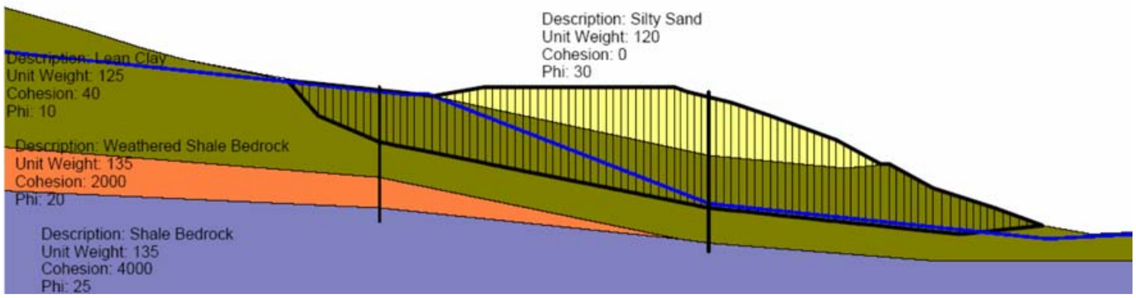


Figure 5: Soil stratification of the Muddy Pass Slide

The approach to evaluate the capacity of a driven pile system to mitigate the selected site is performed in two stages:

1. A model of the "as is" state of the slope is developed to establish a failure zone under self-weight including a weak, wet, remolded zone.
2. The retrofitted state is simulated, where driven piles have been added, and their effectiveness is examined in preventing failure, while the weak zone has already been developed.

To simulate a retrofitting action rather than a failure-preventing action, the failure zone is modeled as a remolded weakened zone. Thus, the retrofitting action of driven piles has to contend with an already weakened slope. This is a departure from past studies which have addressed strengthening a slope BEFORE failed remolded zones have developed.

4.2. Material modeling

The geometric model that has been selected in this study (Figure 1) consists of three materials:

1. A homogenous clay that forms the main body of the slope.
2. A remolded clay that forms the weakened zone that forms the slip plane of failure.
3. A hard base material (such as rock), which underlies the clay.

The geotechnical parameters for each material are presented in Table 1.

Table 1: Geotechnical Material Parameters

Material Name	Material Type	C	ϕ	G	K
		psf (kPa)	degrees	ksf (MPa)	ksf (MPa)
Homogeneous Clay	Elastoplastic	119 (5.7)	19.50	100 (4.788)	500 (23940)
Weakened Clay	Elastoplastic	60 (2.9)	7.50	21 (1.005)	500 (23940)
Strong Base Material	Elastic	N/A	N/A	50000 (2394)	100000 (4788)

c = cohesion

ϕ = friction angle

G = Shear modulus

K = Bulk modulus

To simulate material behavior in the finite element analysis, an elastoplastic model has been selected, which is briefly discussed here.

The material is simulated as elastic, perfectly plastic. The yield function F is defined as

$$F = J_{2D} - (a_0 + a_1 p + a_2 p^2) = 0 \tag{1}$$

where,

J_{2D}= the second invariant of the deviatoric stress defined as

$$J_{2D} = \frac{1}{6}\left[\left(\sigma_x - \sigma_y\right)^2 + \left(\sigma_y - \sigma_z\right)^2 + (\sigma_z - \sigma_x)^2\right] + \tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2 \tag{2}$$

p = the average pressure at a point, defined as

$$p = -\frac{\sigma_x + \sigma_y + \sigma_z}{3} \tag{3}$$

The negative sign in the above equation is such that makes p positive if it is compressive.

The parameters a_o, a_1, and a_2 are selected such that the material satisfies the values of c and ϕ as calculated in triaxial tests.

The theoretical relation of the major and minor stresses of triaxial tests base on the Mohr-Coulomb failure criterion is as follows:

$$-\sigma_1 = -\sigma_3 \cdot K_p + 2c \cdot \sqrt{K_p} \tag{4}$$

where $K_p = \tan^2\left(45 + \frac{\phi}{2}\right)$, is the passive pressure coefficient.

Relationship (1) for a triaxial test becomes:

$$F = \frac{\sigma_3 - \sigma_1}{3} - \left(a_0 - a_1 \frac{(\sigma_1 + 2\sigma_3)}{3} + a_2 \frac{(\sigma_1 + 2\sigma_3)^2}{3^2}\right) = 0 \tag{5}$$

Parameters a_0, a_1, and a_2 are evaluated and presented in Table 2 so that both equations (4) and (5) are valid for the c and ϕ values of Table 1.

Table 2: Plasticity Model Material Parameters				
Material Name	Material Type	a_0	a_1	a_2
		psf^2	psf	
Homogeneous Clay	Elastoplastic	21241.5	126.42	0.188
Weakened Clay	Elastoplastic	5157.2	22.632	0.0248
Strong Base Material	Elastic	N/A	N/A	

4.3. Load Modeling

4.3.1. Original State

A three dimensional state is defined, where the far field is fixed to zero displacement. This instigates variation of stresses in the long direction and can lead to three-dimensional failure. Load is applied as a time-ramp function until failure. When failure is established, the weak remolded clay material is defined. An example of the outcome of this process is presented in Figure 6, including contours of vertical displacements. The color code of deformation is a transition from the most positive (upward) movement (red) to the most negative (downward) movement (dark blue).

4.3.2. Mitigated State

With the remolded weak-clay area established, piles are placed at 5-ft interval along the embankment crest. The gravity is applied in a similar ramp function as in the case of the original failure. This is an inaccurate simulation, in the sense that gravity should have been applied BEFORE the placement of the piles. Unfortunately the Finite Element model does not allow the addition of the driven piles after gravity has been applied. The effect of this process is that the gravity movements are somewhat hindered by the soil adherence to the piles.

At the end of gravity application, the deformations of the road pavement area are measured. It should be pointed out that driven piles contribute to the mitigation of a site by activation of their bending stiffness. This is a "passive" load development, generated by pile deformation. It is thus expected that some road settlement which activates the piles and results in a stable slope. The activation settlement is evaluated as the difference between the settlement of the far field of the embankment, which is due to gravity, and the settlement of the field close to the piles, which is both due to gravity, and due to the slope movement. Additional surface load is then applied to account for the activation settlement which is expected to be an additional repair to complete the mitigation of the road surface settlement. For example, if the activation settlement is 5 inches, than in the zone where the activation settlements take place, a surface load of 5 inches of pavement weight is applied. An example of the outcome of this process is presented in Figure 7.

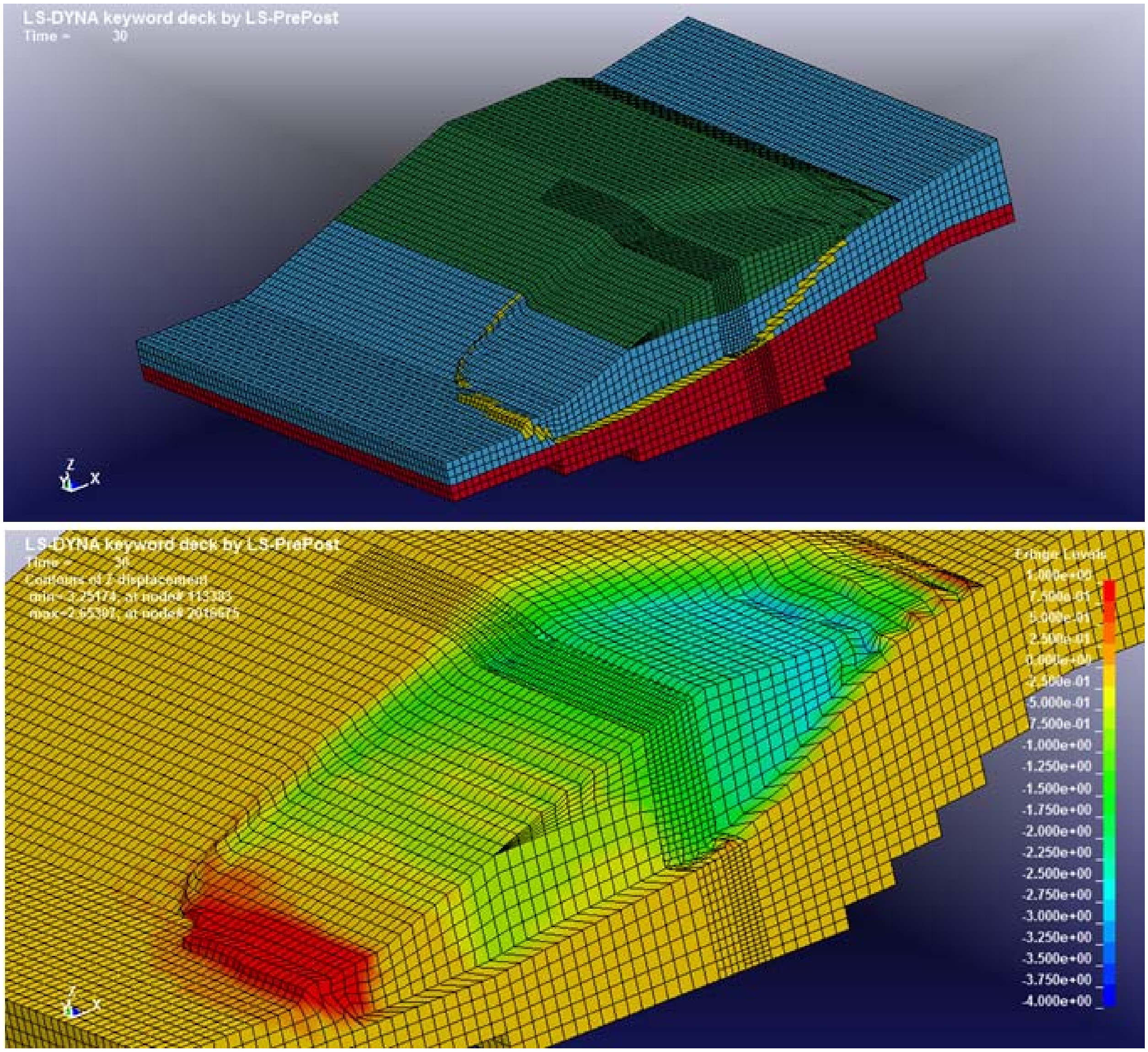


Figure 6: Failed 3-D slope before mitigation and contours of vertical deformation

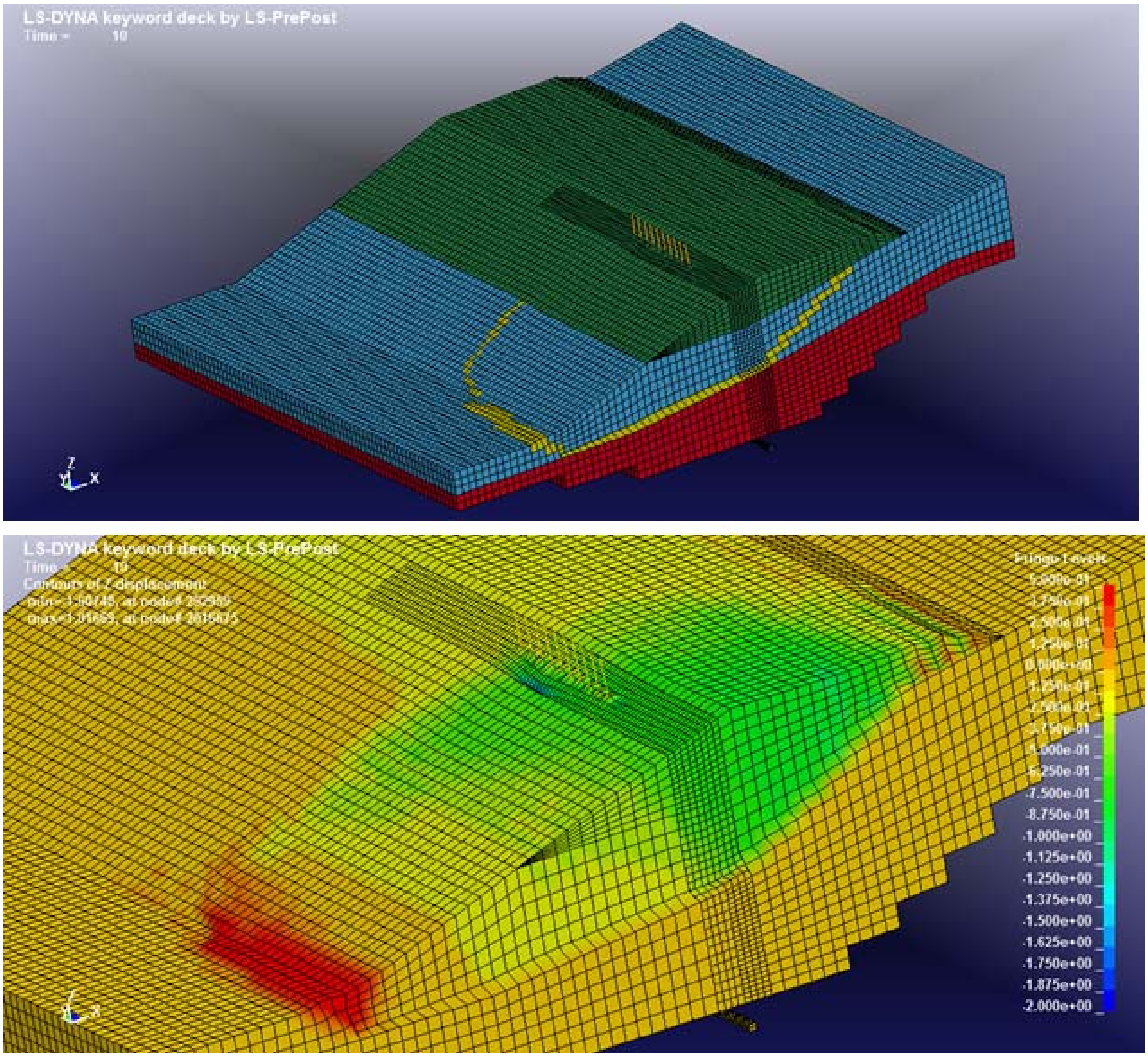


Figure 7: Mitigated slope with the use of driven piles at the failure edges and contours of vertical deformation

4.3.3. Simplification of the process

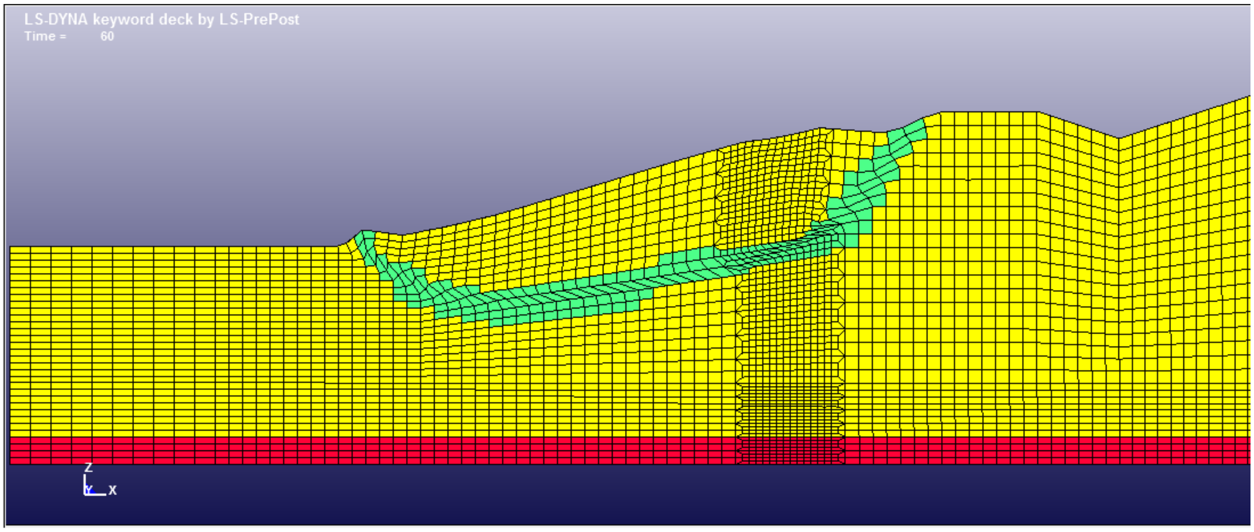
Computer runs based on the process described above typically last approximately 72 hours or more on parallel runs on a Windows based i7 Intel processor. To improve the solution efficiency, and expedite the analysis, the far-field effects of the three dimensionality of the failure surface were ignored. The analysis is still three dimensional as soil stresses and movements vary between piles. The failure surface, however, is quasi-2D as it ignores the variation of the failure depth at the ends of the sliding mass. This allows the modeling of a narrower strip of material which reduces significantly the size of the finite elements system.

For the scope of this project, all driven piles are placed at 5 foot spacing. This avoids excessive soil flowing between the piles, while maintaining a practical pile spacing.

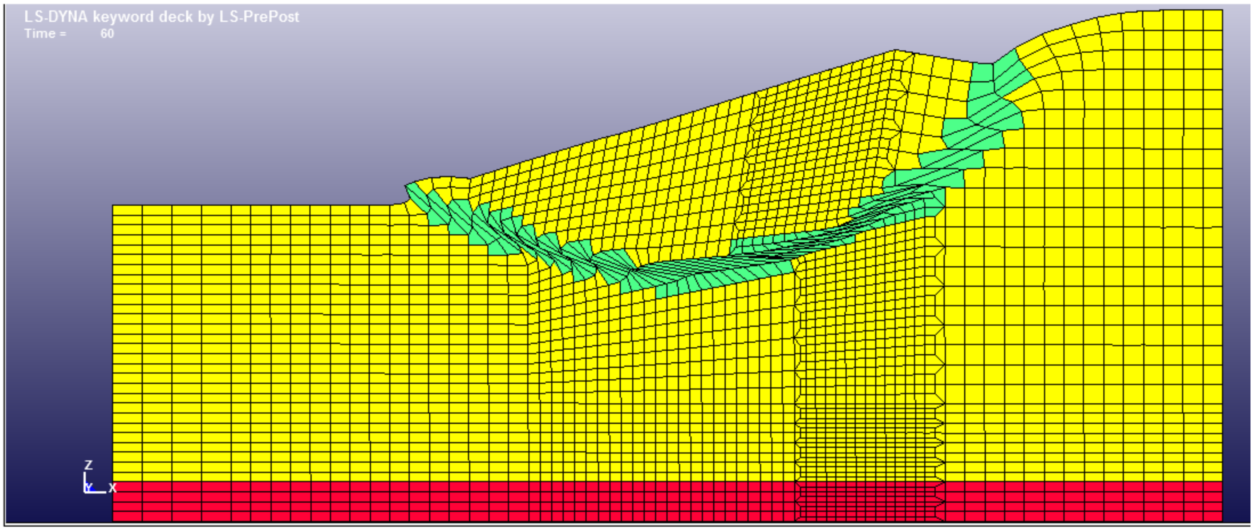
5. ANALYSIS OUTCOME

5.1. Results of Analysis

Typical examples of calculated slope failures are presented in Figures 8-14. Mitigation of these, and all other slopes considered in this study was pursued with the use of H-piles spaced at 5 ft. intervals. An example of unmitigated state is presented in Figure 8, while examples of mitigated slopes are presented in Figures 9 through 15. The deformations patterns are often complicated, especially in the zones that are down-slope from the row of driven piles. Understanding these deformation patterns is significant in developing a better understanding of the mechanisms for failure and the role of the driven piles in the mitigation process. However, from the practical perspective, one should remember that the goal is to mitigate the settlements of the road surface.

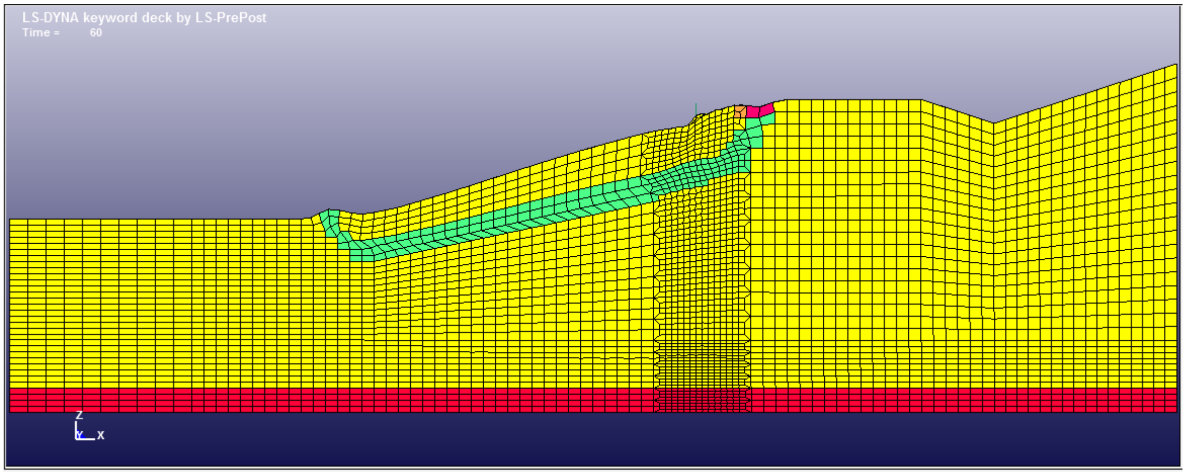


(A): Unmitigated failure of a 3:1 slope embankment

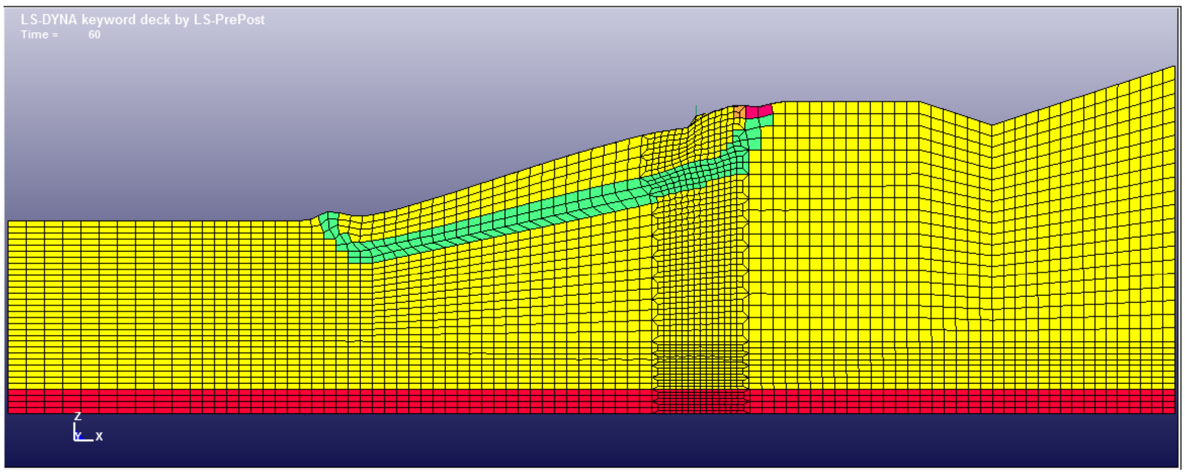


(B): Unmitigated failure of a 2:1 slope embankment

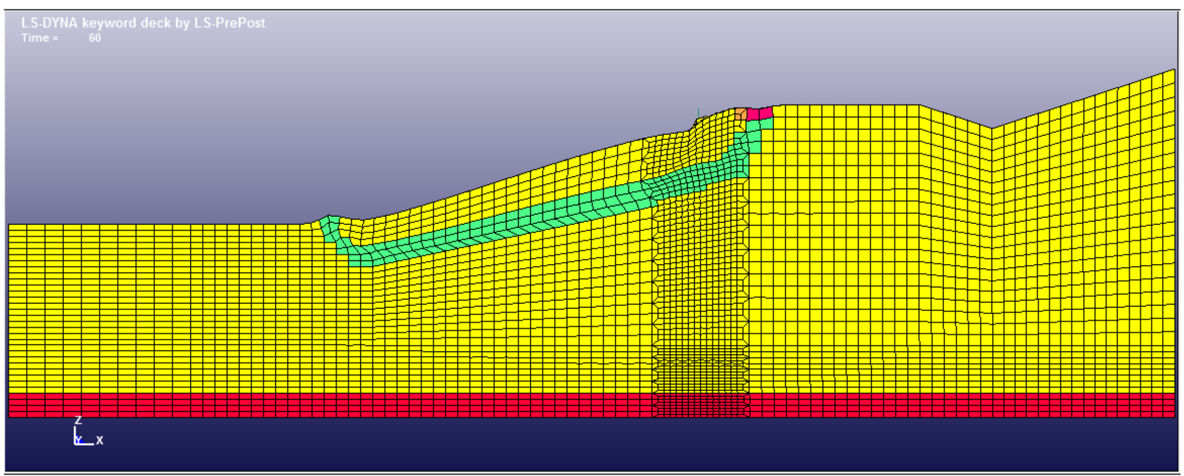
Figure 8: Unmitigated failed states of 3:1 and 2:1 slope embankments



(A): Mitigation based on HP 12x53 driven piles spaced at 5 ft.

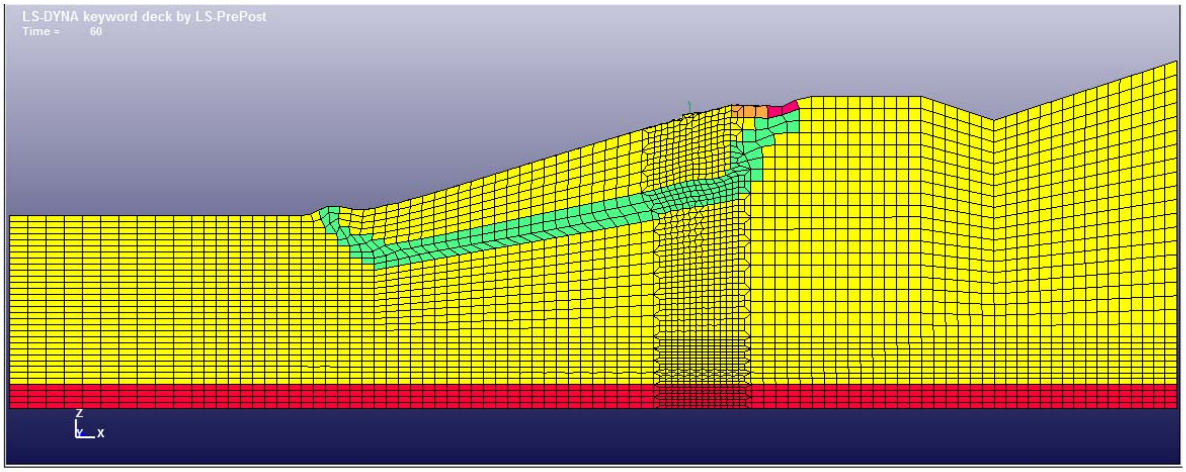


(B): Mitigation based on HP 14x102 driven piles spaced at 5 ft.

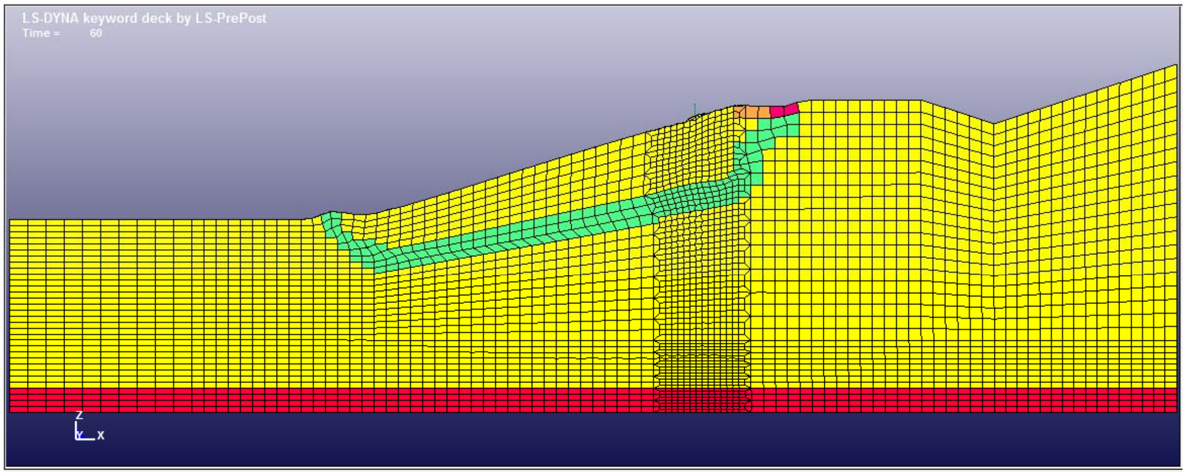


(C): Mitigation based on HP 18x204 driven piles spaced at 5 ft.

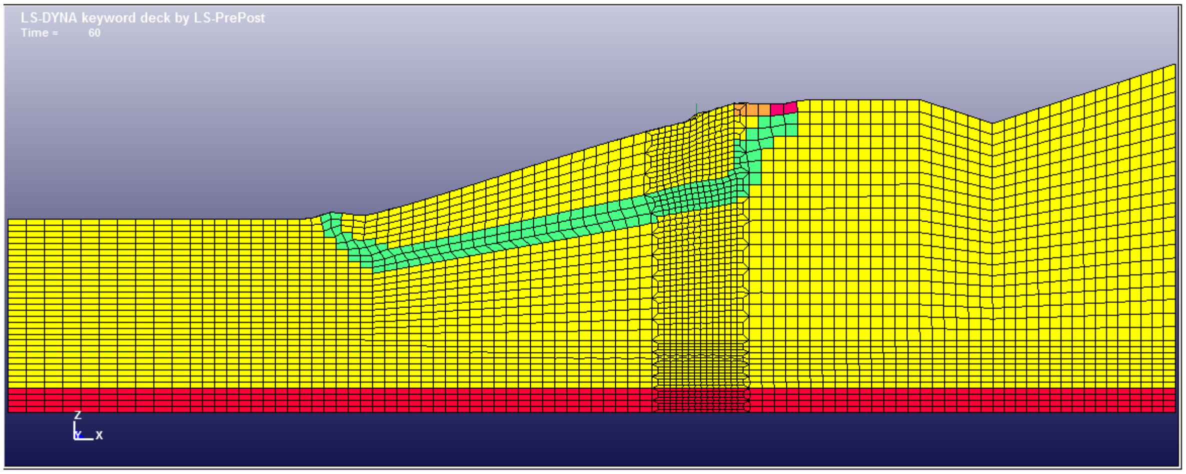
Figure 9: Examples of mitigation of 3:1 slope with a shallow depth of failure



(A): Mitigation based on HP 12x53 driven piles spaced at 5 ft.

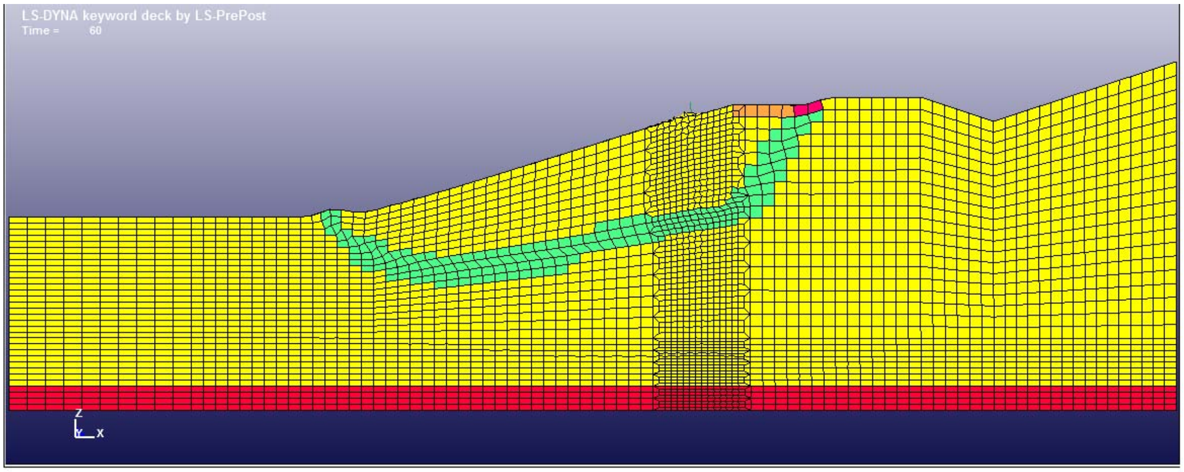


(B): Mitigation based on HP 14x102 driven piles spaced at 5 ft.

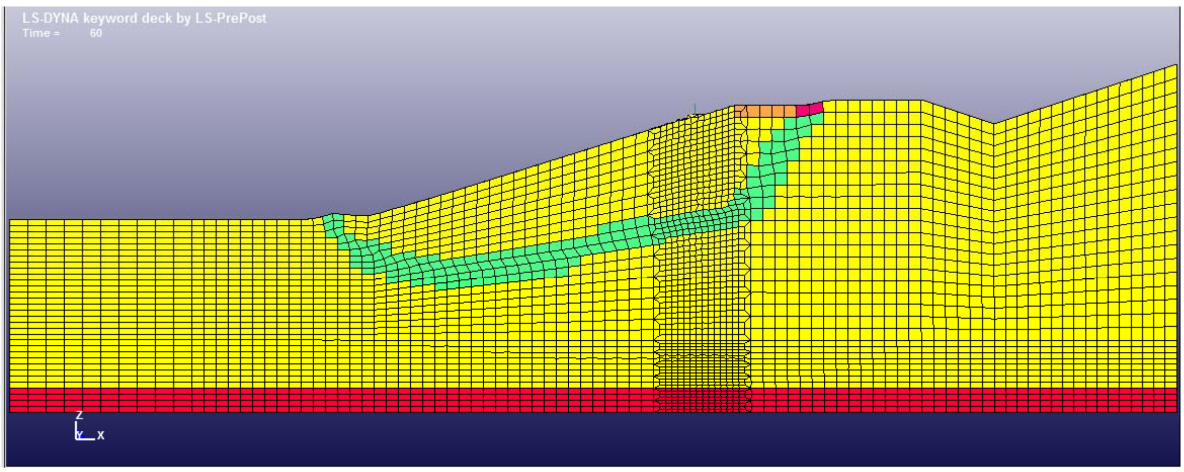


(C): Mitigation based on HP 18x204 driven piles spaced at 5 ft.

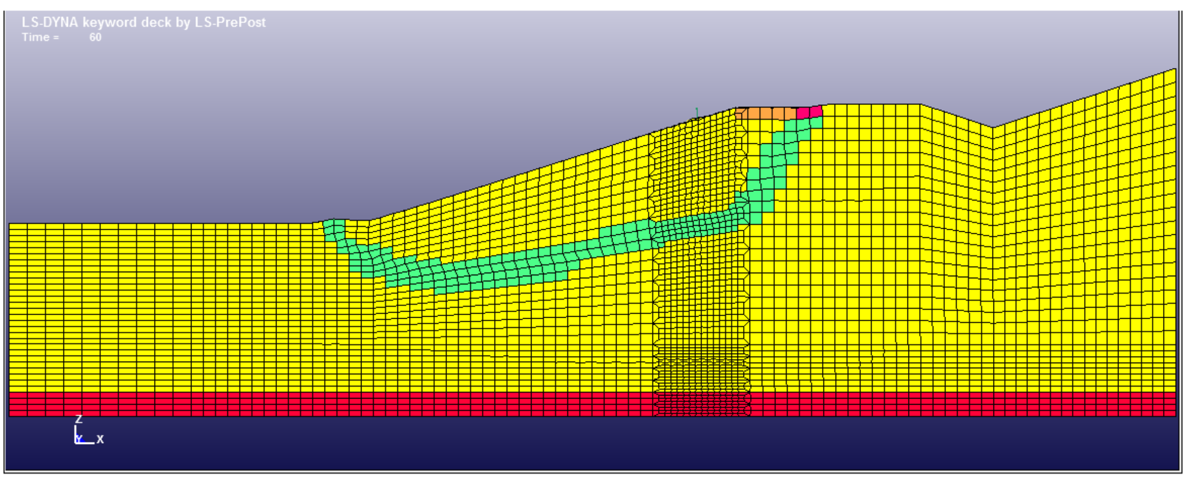
Figure 10: Examples of mitigation of 3:1 slope with an intermediate depth of failure



(A): Mitigation based on HP 12x53 driven piles spaced at 5 ft.

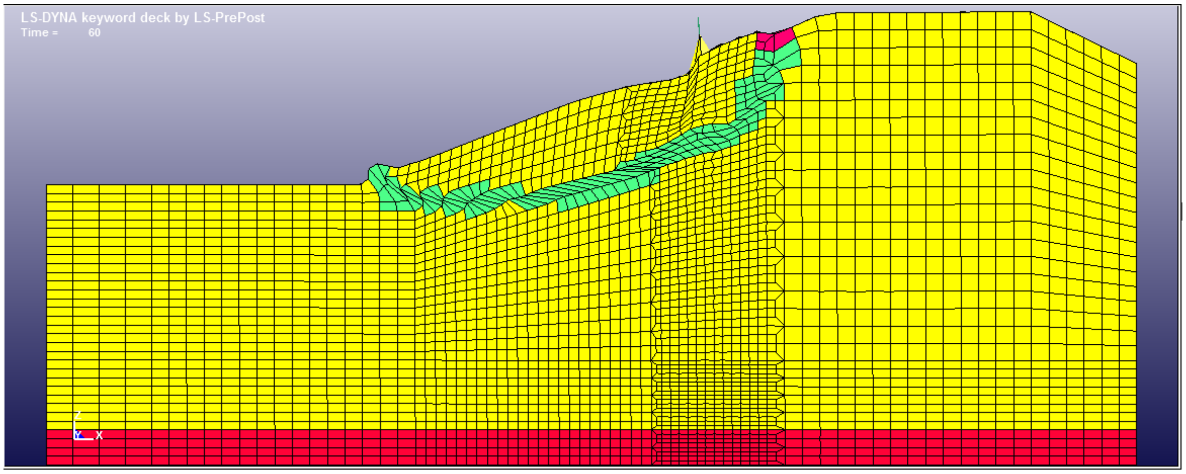


(B): Mitigation based on HP 14x102 driven piles spaced at 5 ft.

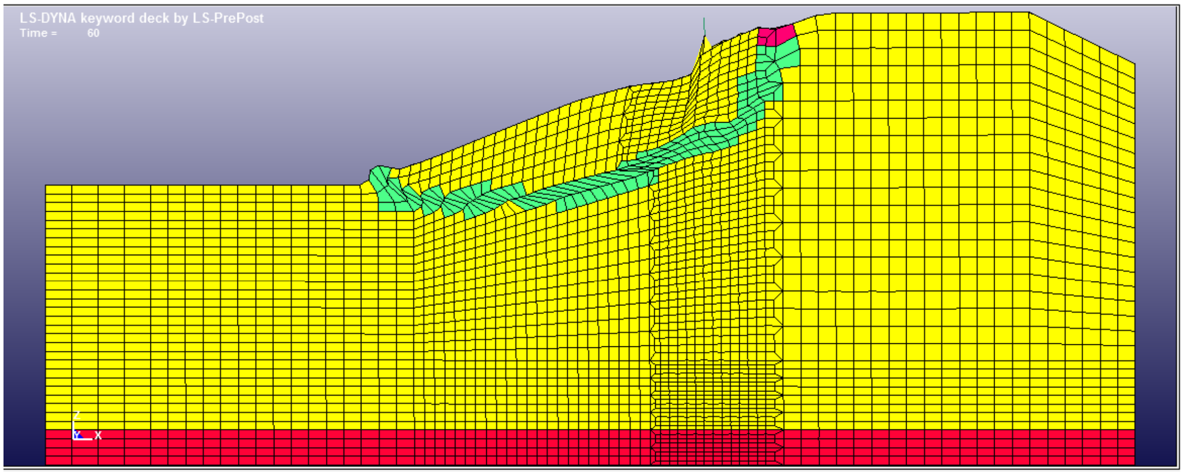


(C): Mitigation based on HP 18x204 driven piles spaced at 5 ft.

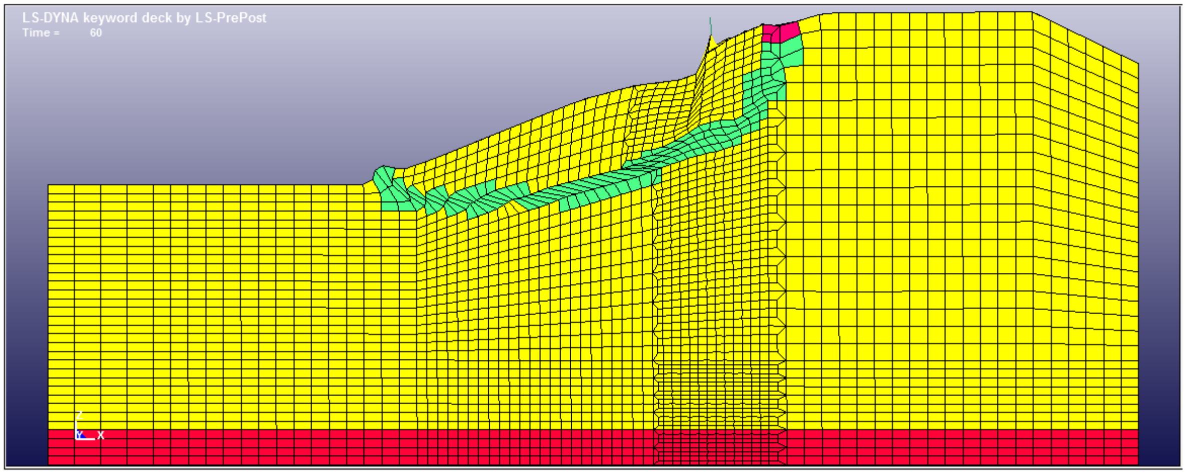
Figure 11: Examples of mitigation of 3:1 slope with a larger depth of failure



(A): Mitigation based on HP 12x53 driven piles spaced at 5 ft.

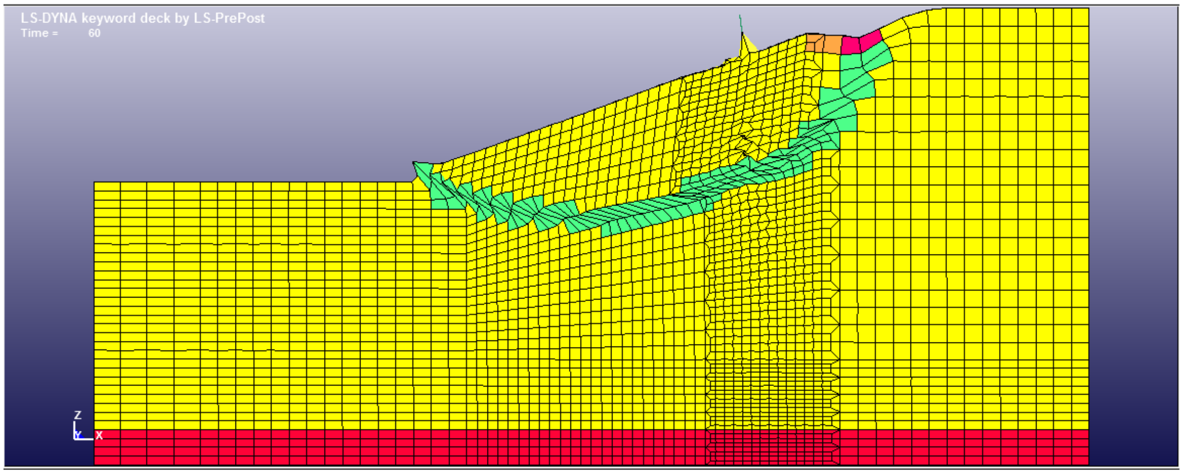


(B): Mitigation based on HP 14x102 driven piles spaced at 5 ft.

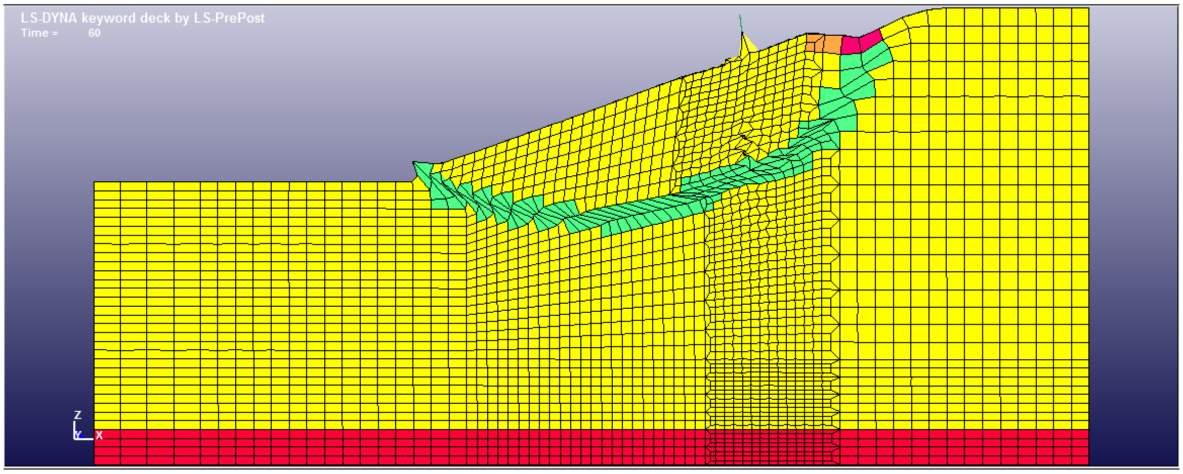


(C): Mitigation based on HP 18x204 driven piles spaced at 5 ft.

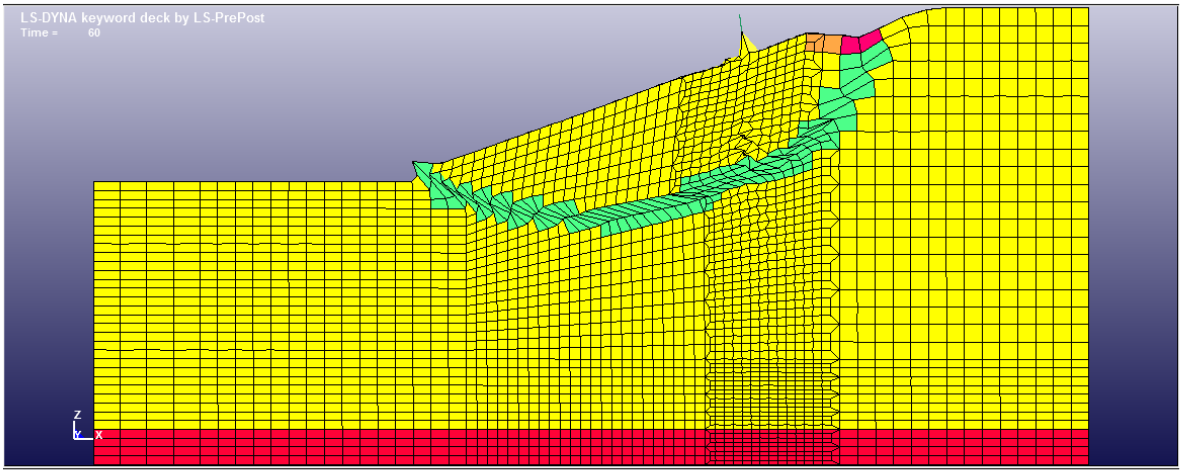
Figure 12: Examples of mitigation of 2:1 slope with a shallow depth of failure



(A): Mitigation based on HP 12x53 driven piles spaced at 5 ft.

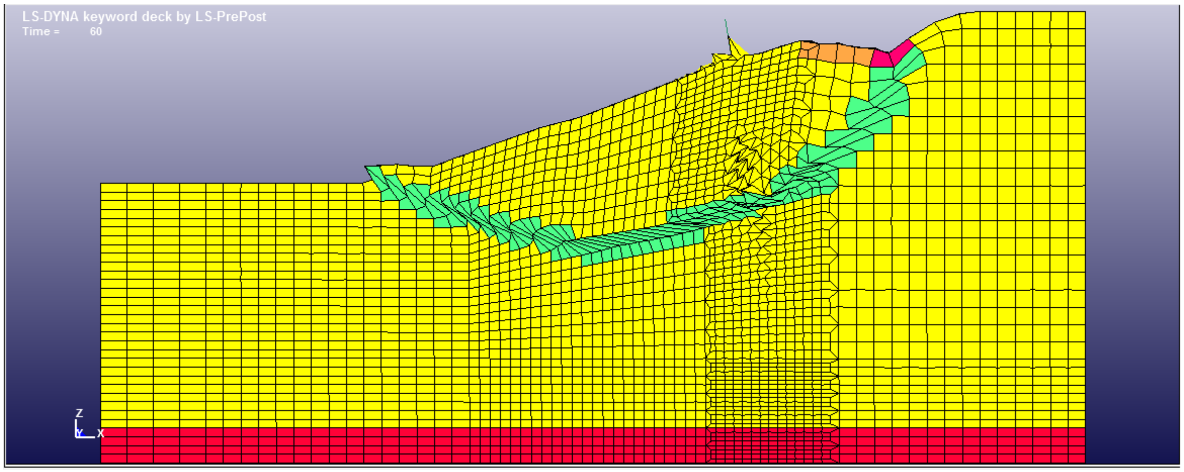


(B): Mitigation based on HP 14x102 driven piles spaced at 5 ft.

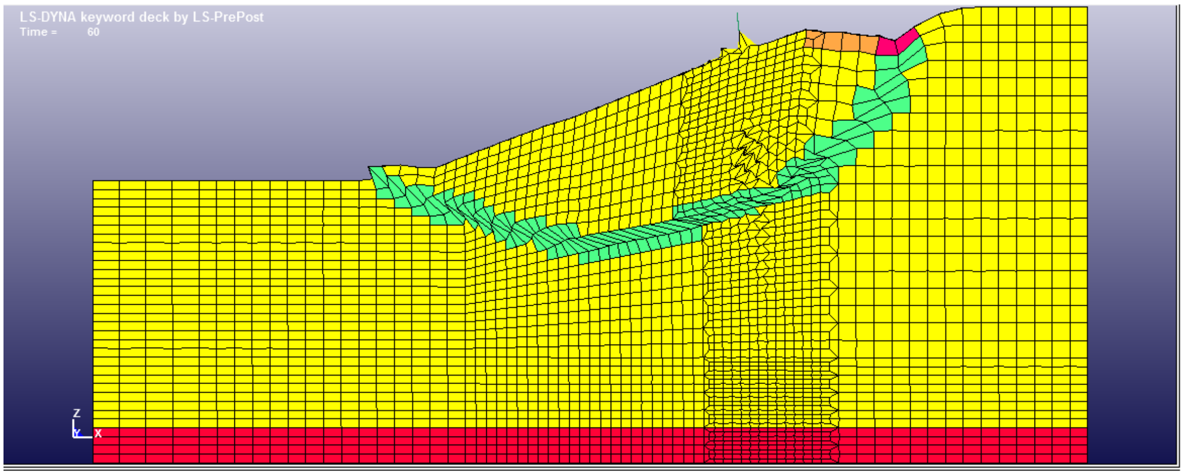


(C): Mitigation based on HP 18x204 driven piles spaced at 5 ft.

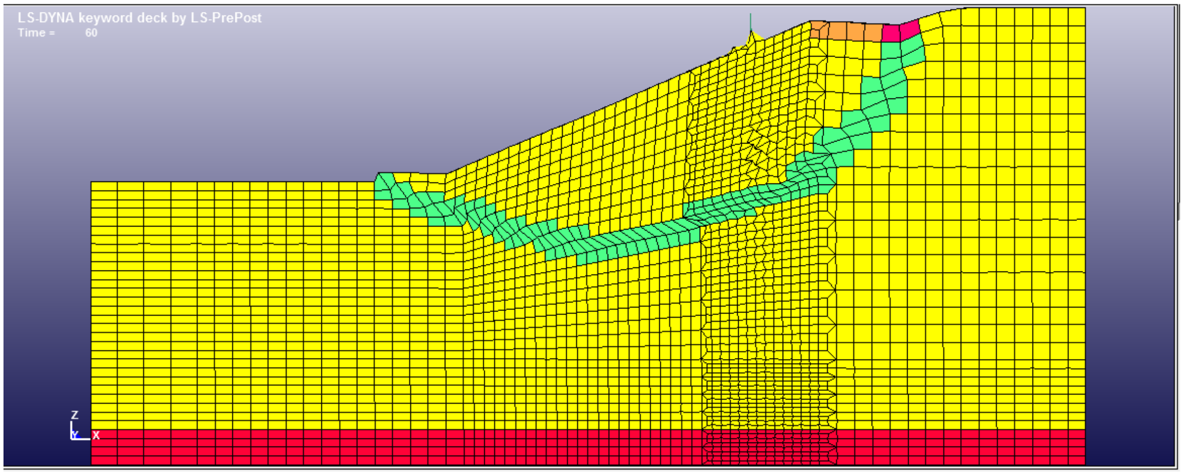
Figure 13: Examples of mitigation of 2:1 slope with a intermediate depth of failure



(A): Mitigation based on HP 12x53 driven piles spaced at 5 ft.



(B): Mitigation based on HP 14x102 driven piles spaced at 5 ft.



(C): Mitigation based on HP 18x204 driven piles spaced at 5 ft.

Figure 14: Examples of mitigation of 2:1 slope with a larger depth of failure

A review of the graphical presentation of the analysis results leads to the following observations:

a. As the size of the HP pile increases, the settlement of the highway pavement decreases.

b. Mitigation based on driven piles is more efficient when used on 3:1 slopes than when used on 2:1 slopes.

c. In many mitigated cases, slope instability occurs downslope from the line of the driven piles. Thus, whereas stiffer piles result in reduction of deformations upslope from the row of installation of the driven piles, there may exist a significant difference in behavior on the downslope side.

The calculated average settlements of the part of the top of each embankment, which is within the failure zone are presented in Figures 15 and 16 for the 3:1 and 2:1 slopes.

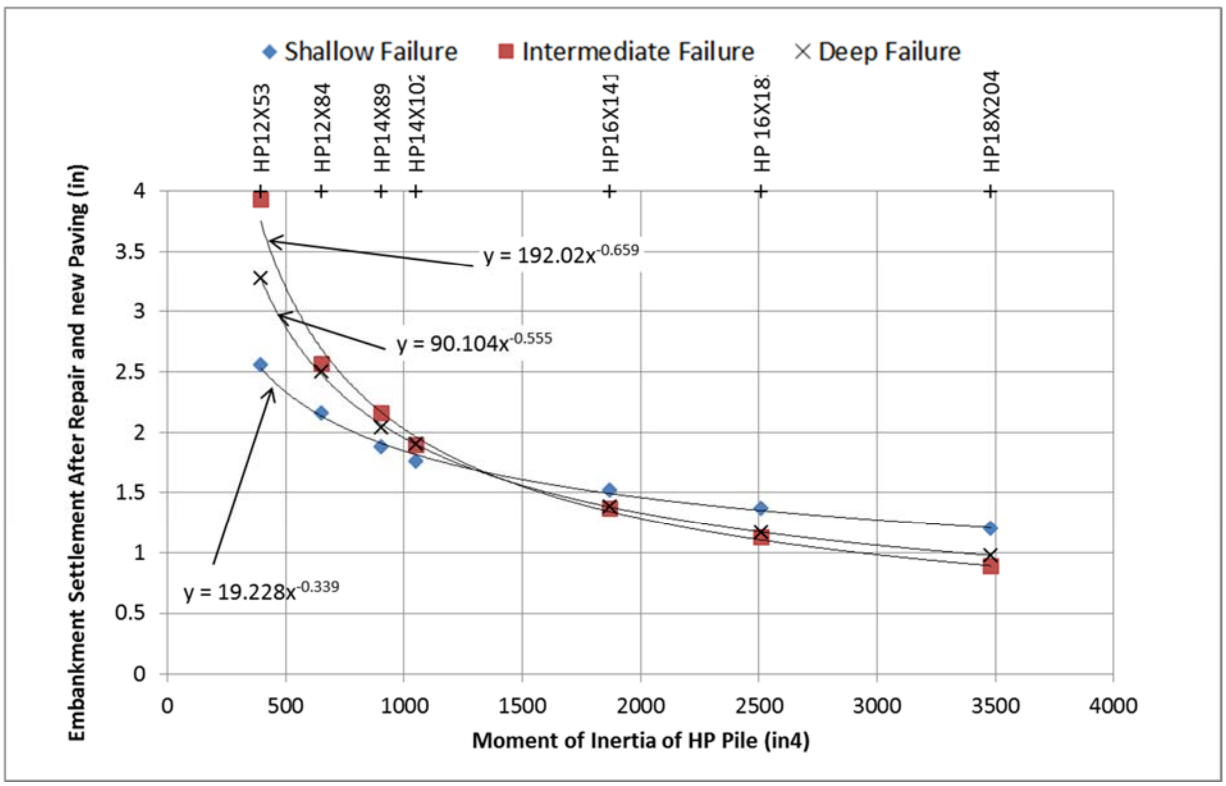


Figure 15: Summary outcome of the mitigation of the 3:1 slope embankments

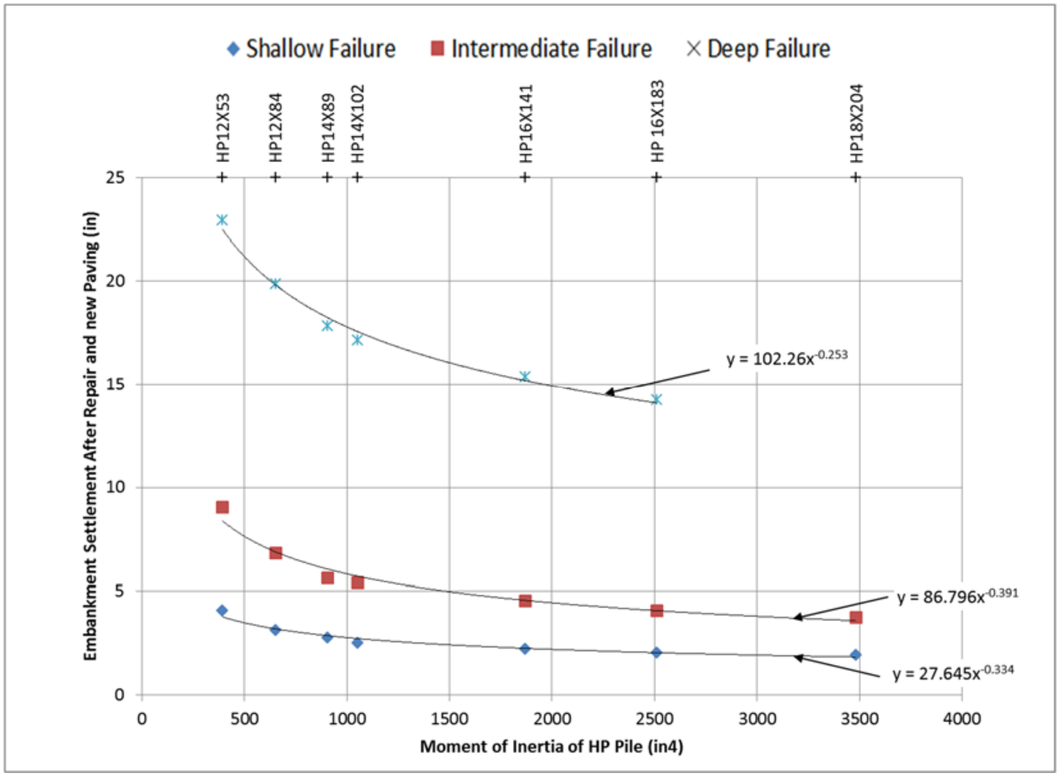


Figure 16: Summary outcome of the mitigation of the 2:1 slope embankments

5.2. Discussion of the results

The summary outcome of the mitigation of the 3:1 slopes presents results that require further explanation. Note, for example, that for the relatively light pile reinforcement of HP 12x53, the embankment crest settlements are highest for the intermediate depth of failure, followed by the deep failure, and then followed by the shallow failure. In all cases settlements decrease as the pile stiffness increases. However as the HP stiffness increases, we observe a reversal in relative behavior, where the shallow failure exhibits the largest settlements, and the intermediate failure depth has the smallest settlement.

To understand this behavior of the 3:1 slopes, the causes of the crest settlement must be examined in more detail.

Embankment settlement is caused by the down-and-out movement of the soil, which occurs because:

(a) The piles deform (Figures 17 and 18);

(b) The soil flows around the piles (Figure 19).

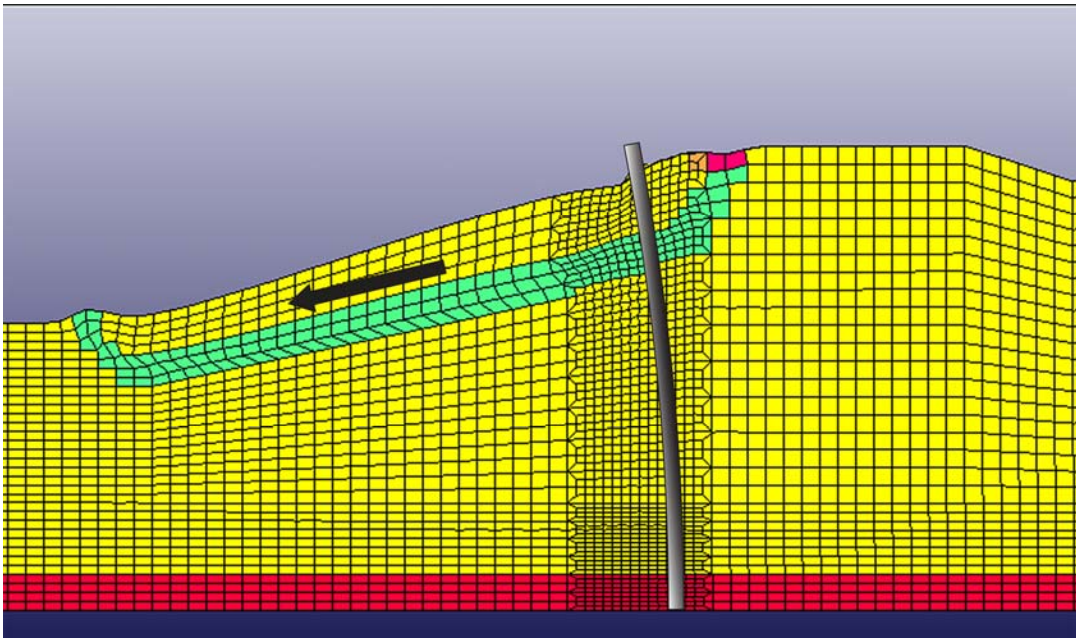


Figure 17: Deformed pile in 3:1 slope with shallow failure

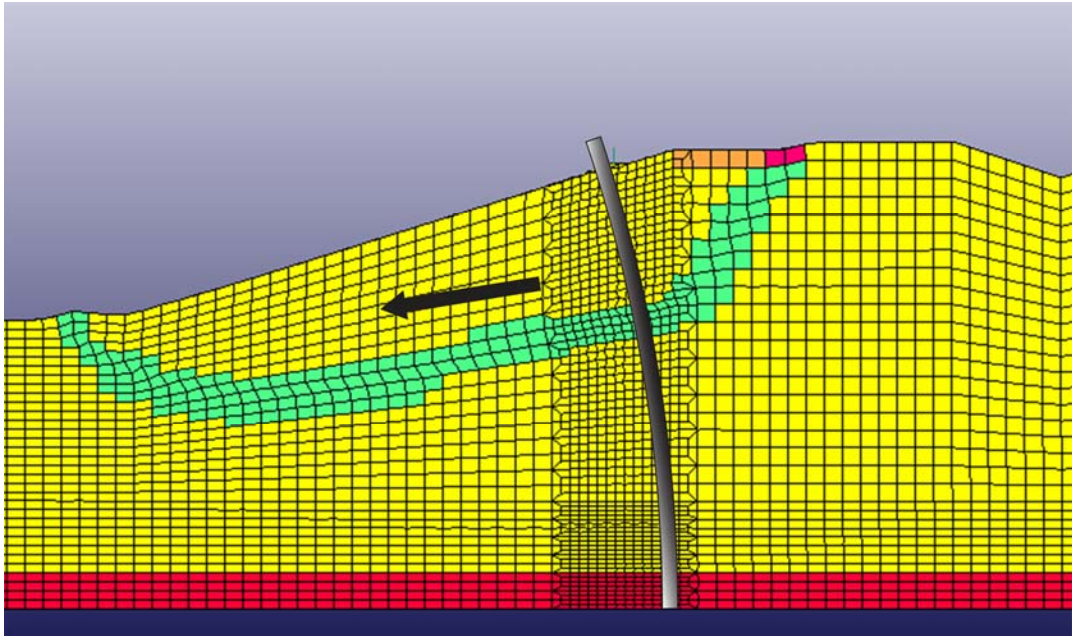


Figure 18: Deformed pile in 3:1 slope with deep failure

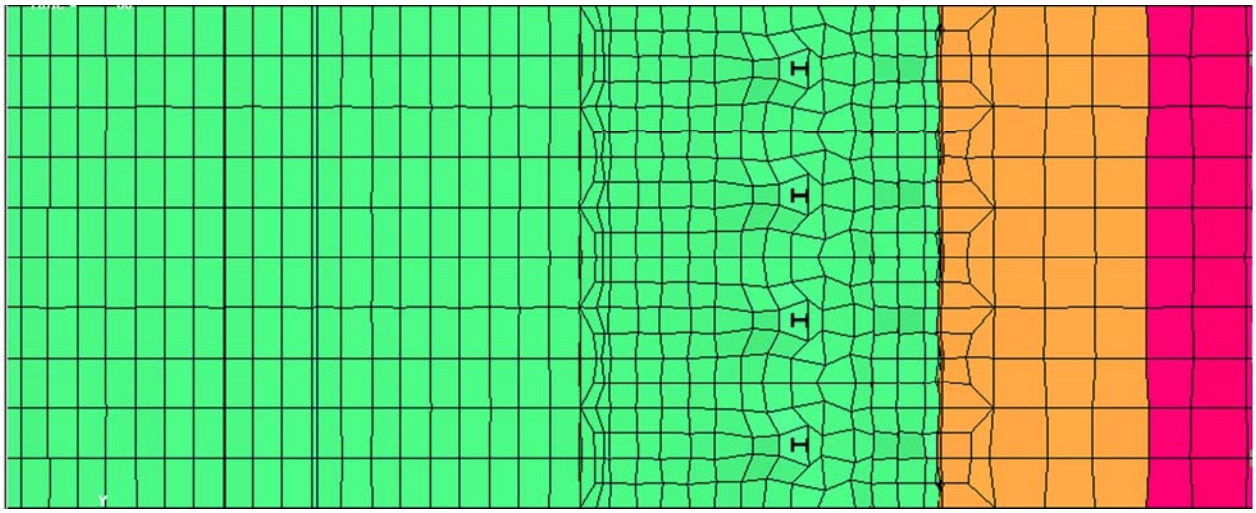


Figure 19: Soil flow between driven piles

Note in Figures 17 and 18 that as the depth of slide failure increases, the moving mass increases, but the slope of the sliding surface decreases. Thus, two opposite tendencies of deformation develop:

a) Deformation of piles increases in the transition from shallow failure to deeper failure, because a bigger mass, applied over a bigger part of the pile loads the pile.

b) Soil flow around the piles decreases in the transition from shallow failure to deeper failure due to the decreased slope of failure.

As the beams become stiffer, crest settlements due to pile bending becomes smaller. However, soil flow around the pile is influenced less, and thus, settlements due to soil flow around the pile become, in relative terms, more significant. Clearly, a deep failure that does not become flat or a steeper slope where the beam stiffness would not be as effective, can result in different behavior.

The effects of the above processes are clearly demonstrated in Figures 15 (3:1 slopes) and 16 (2:1 slopes).

In both Figures 15 and 16, the deformation for large HP piles (18x204) is dominated by soil flow through piles, while, for small HP piles (12x53), the deformation is dominated by the deformations of the pile.

The analysis of the results of the 3:1 slope (Figure 15) indicates that crest settlements due to pile deflection are larger for the deep failure, while settlements due to soil flow between the piles are larger for the shallow failure. Thus, as the pile bending deformations are eliminated with the use of very stiff piles, the tendency for settlements reverses.

It is reasonable to conclude from the above, that the use of HP12x53 at 2.5 ft. spacing is expected to perform better than the use of HP 14x89 at 5 ft. spacing. Note that the HP 14x89 has approximately twice the stiffness of HP12x53. Thus, using twice as many HP12x53 results in approximately the same stiffness, and thus similar settlements due to the pile bending. However, the reduced spacing of the 2.5 ft. spaced HP12x53 piles is expected to result in significant reduction of settlements due to soil flow between the piles.

The analysis of the results of the 2:1 slope (Figure 16), indicates a different tendency. Here, both deformations due to pile bending and soil flow are larger for the deeper failure. 2:1 slopes exhibit a much larger tendency for mass movement and thus both pile bending and soil flow between piles follow the same tendency.

It is reminded here that the above observations are all based on 5-ft spacing of piles. The movement triggers would not necessarily be the same if pile spacing, and thus soil flow, were smaller. For the material properties addressed in this study, pile spacing that exceeds 5 feet result in a perpetual movement of the slope, as long as the remolded material remains weak (snow melt period) and the crest of the highway embankments is continuously repaired (i.e. the gravity load remains constant). This is actually the case in Muddy Pass and other mountain highways that experience similar problems.

6. SUMMARY AND CONCLUSIONS

Based on the research conducted in the first phase of this study, and the parametric finite element analysis conducted here, it is concluded that stabilizing driven piles can be a credible solution to the challenges of maintaining slope stability on Colorado’s highway embankments.

General observations and conclusions can be summarized as follows

- Driven piles improve the shear capacity of the slope by reinforcing the slip surface.

- Driven piles can provide effective solutions to slope stabilization problems where space and access restrictions, which often occur in mountain highway embankments, render alternate approaches unfeasible.

- Based on earlier research during the first phase, it was concluded that slope stabilizing piles have a cost similar to other low impact landslide mitigation techniques.

More specific observations and conclusions, based on the finite elements parametric analysis conducted here, can be summarized as follows:

- Driven piles can improve the behavior of a failing embankment as long as:
    - The piles are sufficiently stiff to carry the load of the sliding soil ABOVE the failure surface.
    - The piles penetrate sufficiently into the ground below the failure surface to secure fixation.
    - The piles are spaced sufficiently close to prevent excessive flow around them. The required proximity depends on the residual strength of the weakened layer.

- Driven piles are more effective in slope stabilization when applied to milder slopes than when applied to steeper slopes.

- For mild slopes (3:1), driven piles showed similar effectiveness for failures of shallow depth as for failures of larger depth.

- For steep slopes (2:1), driven piles were more effective when applied to mitigation of shallow failures. They proved to be less effective in the mitigation of deep failures, mainly because at a spacing of 5 ft. they could not prevent soil flow between them.

- The main limitations of this study are:
    - The range of embankment heights examined is limited. This limitation is more important in cohesive soils than in frictional soils.

- The range of soils strengths of the weak soils examined is limited. It was found however, that the strength of the slope soils, other than the remolded clay, is not very influential in the outcome of the analysis.

- Lack of field validation. As was discussed earlier, the original intent of this study was to provide a field validation to the observations and conclusions of the first phase of this study. Instead, the lack of funding resulted in an altered scope, where a parametric numerical study was performed. This study has significant merits, as it provides insight on the behavior of the pile-reinforced mass and provides information on the effects of multiple parameters, such as variation of embankment slope, depth of slide failure, and driven pile stiffness. However, this study suffers from the drawback that it has not been field verified.

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7. 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 a future field verification be pursued.

It is also recommended that further computational studies be pursued, where other significant parameters be examined, such as:

- Relation between strength of the weakened remolded material and the spacing of piles to prevent soil flow between piles.
- Relation between strength of the weakened remolded material and the steepest slope that can be effectively mitigated using this technique.
- Methods to recognize depth of failure based on observed failure characteristics, such as the location of failure at the crest, the location of toe failure, and the amount of observed movement.

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