Case Studies of Landslide Stabilization with Deep Foundation Elements

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ABSTRACT: Slope stabilization with deep foundations is not widely used due to a lack of published design procedures. This article illustrates 2 case studies involving the installation of vertical micropiles and drilled shafts to stabilize significant mass movements. The first case study describes the stabilization of an ancient landslide (in soil) by using a total of 75 drilled shafts embedded into bedrock. The second case study describes the emergency stabilization of a rockslide with more than 110 uncased micropiles. The article presents the encountered geotechnical conditions, design aspects (global stability, p-y analyses, required embedment for passive resistance, group effects, structural capacity, etc.), and construction observations. These 2 cases are compared since the principles of design are completely different. For the rockslide stabilized with micropiles, the controlling failure criterion was shear at a defined joint system, while for the soil landslide flexural stresses on the drilled shafts controlled the design. The proposed design methodologies depending on the particular geotechnical conditions are discussed for application on future projects.

KEY WORDS: Deep foundations, Landslide Stabilization, Micropiles, Drilled Shafts.

1 Introduction

While effective, slope stabilization with deep foundations, such as drilled shafts or micropiles, has not been used widely used due to a lack of published design procedures and familiarity. There are many complexities that often discourage designers from considering deep foundations (e.g., micropiles and drilled shafts) for the stabilization of slopes. Specifically, the predicted resistance of the pile system based on the likely failure modes (i.e., shear, axial or flexural) and the interaction between the soil/rock and the piles are common hurdles in design of this system. In general, structural failure mechanisms (shear vs. bending) vary by slope conditions and must be designed for each specific project. For example, when the failure mechanism consists of an intact sliding rock block, the resistance from the structural element will be controlled by shear. On the other hand, structural elements that reinforce soil slopes are generally controlled by bending (compressive and tensile stresses).

The orientation of the piles (i.e., vertical versus battered) affects the failure modes that must be considered, in that battered piles will take lateral loads axially while vertical piles will resist lateral loads by bending or shear mechanisms. Furthermore, the geotechnical resistance may be a potential failure mechanism, where it could be argued that active, at rest or passive resistance of the soil in front of the deep foundation element could control the design. However, it is commonly accepted that even passive failure of the soil above the sliding plane is not a factor since the structural element is designed to take that load. The geotechnical resistance below the failure plane should be evaluated to ensure enough fixity of the reinforcing element is provided.

2 General Design Considerations

Typically the necessary pile resistance and “shear force” required to obtain an adequate overall factor of safety (FS), generally 1.3 to 1.5, can be calculated using a slope stability computer program. The lateral deflection analyses of the deep foundations can be performed using p-y curves, which can be used to predict the lateral deformation of the elements, calculate the required embedment below the failure plane, and perform structural capacity checks (e.g., bending and shear stresses).

The spacing of the deep foundation elements must be optimized to maximize the available passive resistance on piles. The optimum element spacing is generally within 3 to 5 pile diameters of the proposed pile/shaft diagonal. Pile spacing of less than 3 pile diameter often requires a reduction in passive resistance due to the overlapping effect of soil arching.

This article presents 2 very different cases, controlled by different variables, where successful slope stabilization was achieved by using deep foundation elements.

3 Case Study of Stabilization in Soil: Bending Controlling Case

This project was located in the State of Ohio (USA) and involved the installation of a pipeline at the toe of an existing 110 ft (33.5 m) tall embankment constructed within a known ancient landslide. Due to previous landslides, large pockets of weak colluvium were prevalent across the project site. The 110 ft (33.5 m) tall embankment was the direct result of needing additional working space at the top where it ties into existing flatter ground. The space would later be used for various buildings and equipment.

Months after construction of the 110 ft (33.5 m) tall embankment it was necessary to install a new pipeline at its toe. Construction for the project required excavation to a depth of about 8 ft (2.4 m) for the pipeline in addition to a 50 ft (15.2 m) wide bench for construction access near the existing toe. To accommodate the bench width, an additional 45 ft (13.7 m) tall embankment below the existing 110 ft (33.5 m) tall embankment needed to be constructed. The upper 110 feet existing embankment had a 2H:1V slope while the proposed lower stabilized slope/embankment had a slope of 1.3H:1V. A preliminary design considered installing a rock embankment below the existing slope for the construction bench; however, the global stability FS was less than 1.5 (minimum allowed). Drilled shafts were proposed to resist and minimize disturbance to the existing soils. See Figure 1 for an overview of the site.

Figure 1. Overview of the site.

3.1 Geotechnical Model

A significant amount of colluvial material, approximately 10 to 20 ft (3.0 to 6.1 m) in thickness, was present along the slope overlying residuum and bedrock. Depending on location, the depth to bedrock was approximately 16 to 30 ft (4.9 to 9.1 m) and depth to water was relatively shallow below the ground surface. The colluvial material consisted mainly of clays and silts with varying amounts of sand and gravel. The relative density of the coarse-grained soils varied between medium-dense and very dense, while that of fine-grained soils varied between medium and very stiff. Bedrock consisted mainly of interbedded claystone and siltstone with relatively low rock quality designation (RQD) but with high core run recoveries. Bedrock was described as soft and generally slightly-to-highly weathered.

Based on borings and lab testing, unit weights of 120 to 135 pcf (18.9 to 21.2 kN/cu m), friction angle of 24 to 45 degrees, and cohesion of 0 to 100 psf (0 to 4.8 kPa) were utilized for the different soil materials. A unit weight of 145 pcf (22.8 kN/cu m), friction angle of 0 degrees, and a shear strength of 10,000 psf (480 kPa) for competent bedrock/rock mass were also used. For the lateral analysis, using LPILE, the soil was modeled with a modulus of subgrade reaction between 60 and 125 pci (16,290 to 33,945 MN/cu m) depending on its relative density. Bedrock was modeled with an unconfined compression strength of 2,930 psi (20.2 MPa).

Design for the proposed slope retrofit included analyses for global stability of the proposed treatment (verify geotechnical stability) and lateral analysis of the proposed drilled shafts (structural concerns). The two analyses were performed sequentially to ensure that the proposed layout satisfied geotechnical and structural requirements. A sketch of the proposed retrofit is included in Figure 2.

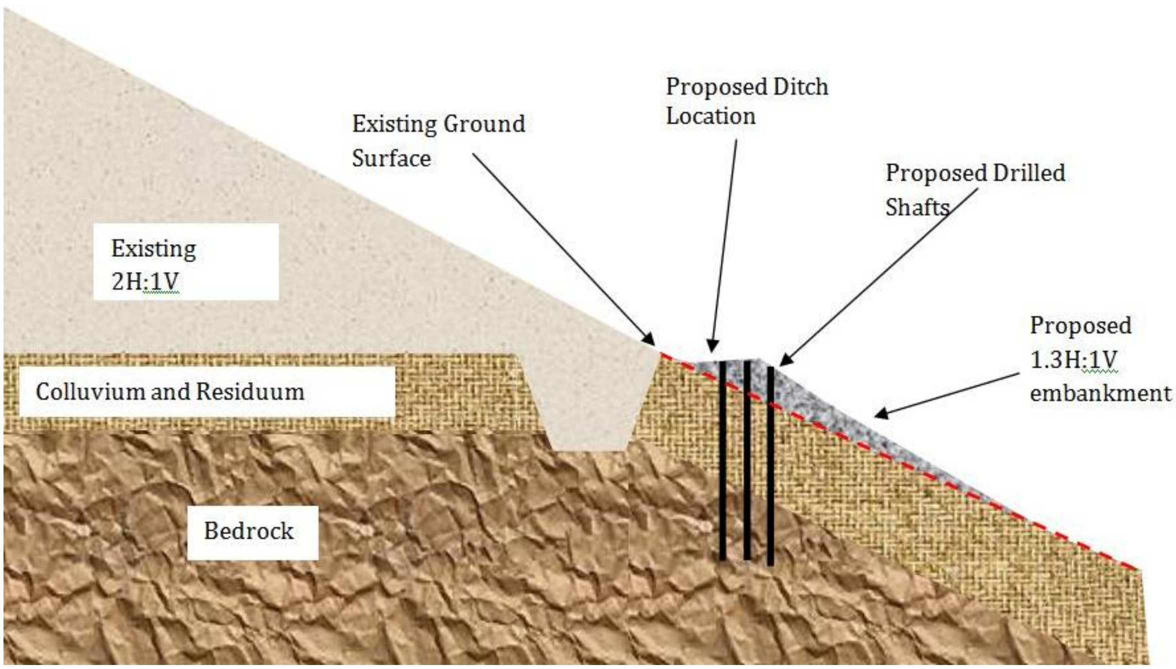

Figure 2. Schematic Description of Proposed Drilled Shaft Retrofit

3.2 Global Stability Analyses

2D stability analyses were performed using SLIDE to analyze the upper slope (existing embankment above the pipeline) and lower slope (proposed treatment region below the pipeline). The slope treatments were designed to satisfy a FS of 1.3 for the temporary condition and 1.5 for the permanent condition. Based on required resistance and a minimum FS of 1.5, the drilled shafts were modeled in SLIDE as a resisting force to determine an optimized pile layout. Multiple analyses were performed to check local and global stability for temporary and final conditions.

3.3 Lateral Deflection (P-Y curve) Analyses and Minimum Required Embedment

Once the required resistance per shaft was determined, lateral analyses (p-y modeling using LPILE) were then performed to size the drilled shafts and to ensure they extended deep enough to satisfy fixity. As the drilled shafts were to consist of W-shaped steel sections encased in concrete, bending resistance of the 50 ksi (345 MPa) W sections only was considered and any resistance from concrete was conservatively neglected.

The analyses assumed that the colluvial layer would move 1 in (25 mm) due to soil relaxation and would engage resistance of the underlying layers. This deformation was assumed to occur from the ground surface down to the failure plane determined from the global stability analyses. The analyses considered a drilled shaft spacing of 3 times diameter along each row to maximize passive resistance from soil arching. The drilled shafts were spaced about 7.5 ft (2.3 m) or 3 times diameter apart and the rows were spaced 15 ft (4.6 m) or 6 times diameter apart and were staggered approximately 4 ft (1.2 m) to avoid "shadowing" and group effects that could potentially reduce the shaft resistance.

The depth of the piles was extended in the model until fixity was achieved, which was defined as the second point of zero deflection. The global stability analyses followed a FS approach, however, the LPILE analyses considered LRFD methodology (i.e., resistance factor = 1/FS).

3.4 Structural Checks

A W18x106 (W460x158) section for the drilled shaft reinforcement was selected based on structural capacity checks. The maximum moment and total stress on the piles was checked against the plastic moment and yield strength of different section sizes to adequately size the reinforcement. The section was sized so that the moment and stress from the output did not exceed 0.67 times the plastic moment and yield strength of the section (FS of 1.5).

Two separate FS were used in the computations: FS=1.5 for required resisting force from the slope stability analyses and FS=1.5 to reduce the ultimate strength of the reinforcing elements. The structural pile was checked against shear, but bending was found to control the design.

3.5 Construction of the Drilled Shafts and Pipeline Installation

Ultimately, the design mandated the installation of 75 drilled shafts (30 in [762 mm] in diameter and 40 ft [12.2 m] deep) in three rows along the embankment embedded into bedrock. The drilled shafts were constructed using vertically placed W18x106 (W460x158), 50 ksi (345 MPa) steel beams and concrete with a 28-day compressive strength of 3,000 psi (20.7 MPa). The drilled shafts were installed approximately 10 ft (3 m) into competent bedrock, resulting in an overall length between 30 and 40 ft (9.1 and 12.2 m).

Reconstruction of the slope including the construction bench and installation of the pipeline was successfully completed within four weeks. No significant observations (i.e., tension cracks, differing subsurface conditions, etc.) were encountered during construction that required deviations from the proposed design. See Figures 3 and 4 for photographs during construction.

Figure 3. Terminated Drilled Shaft/Steel Section after Construction

Figure 4. Pipeline Installation After Construction of the 75 Drilled Shafts

4 Case Study of Stabilization in Rock: Shear Controlling Case

This case study considers a landslide in an existing rock cut located uphill from a shopping plaza in Pennsylvania (USA). The landslide created tension cracking along the existing rock cut, and during a period of weeks the tension cracks expanded from inches to feet (mm to meters). Survey readings indicated that portions of the slide were moving at a rate of approximately 2 in/day (5 cm/day). In addition to the shopping plaza, high voltage electric lines and an underground gas line ran along the base of the rock cut, further prioritizing the need to mitigate the failure as quickly as possible. See Figure 5 for a picture of the rockslide after initial movement.

Figure 5. Tension Crack at top of the Rockslide

4.1 Geotechnical Conditions

The site investigation revealed that bedrock along the cut consisted of sandstone with interbedded shale with a bedding dip of about 16 degrees. The orientation of the dip was downslope along and towards the cut. A drainage ditch was present upslope of the failure scarp, and was intended to divert water from wetlands upslope. Water flow within the ditch seemed to “disappear” in the region closer to the scarp. It was believed that surface runoff had been flowing into the slide rather than following the path of the constructed drainage ditch. Water could be heard flowing within the tension cracks of the slope failure, and seepage was observed along the slope. See Figure 6 for an overview of the rock slide, and a closer view of the failure plane.

Figure 6. Overview of the Rockslide and Closer View of the Failure Plane.

The mode of failure was due to weathering and degradation of thin shale layers within sandstone. These shale layers weathered over time due to groundwater infiltration, and produced a slide surface for the bedrock above. The size of the sliding mass (interbedded sandstone and shale block) was approximately 400 ft (122 m) wide by 150 ft (46 m) upslope by 20 ft (6 m) thick.

Based on surveying of control points, it was determined that the sliding block was moving at a rate of approximately 2 in/day (5 cm/day). Locations above the tension cracks indicated no movement further upslope.

4.2 Rockslide Remediation Design

Uncased micropiles, also known as “shear pins”, were selected as a cost effective solution due accessibility of equipment and time of construction. The shear pins were installed above the existing scarp while the failed mass was removed. The shear pins would be used to prevent additional tension cracks from forming after the removal of the existing failure mass. See Figure 7 for a schematic description of the proposed concept.

The sizing and spacing of the shear pins were determined based on the assumption that a 20 ft (6.1 m) tall by 150 ft (46 m) long (upslope) mass would fail along the slide surface at a dip angle of 16 degrees. These dimensions were estimated based on the size of the sliding block failure present at the time of design (i.e., it was assumed that a future failure would be similar in size to the failure that already occurred for which the stabilization was required). The design of the shear pins assumed that the material in front of the block would not provide any resistance.

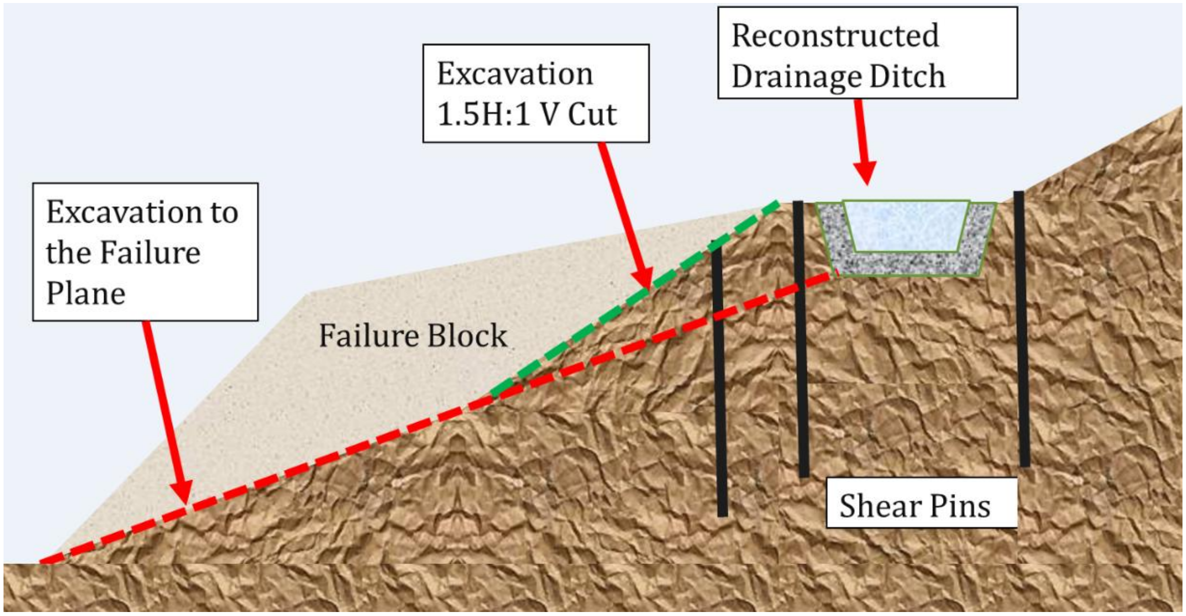

Figure 7. Schematic Description of the Rockslide Remediation

4.3 Global Stability Analyses

The size, spacing and embedment depth of the shear pins were computed using the commercially available program SLIDE such that a minimum factor of safety (FS) of 1.5 was provided. The friction angle of the slide material was back calculated using SLIDE to determine the shear strength of the material that would result in FS = 1.0 (i.e., the existing condition). Based on the analysis, the friction angle of the sliding surface was estimated to be approximately 16 degrees. Since this friction angle was for the region that had already failed, this value considered to be the residual shear strength of the degraded shale layer below the area where tension cracks had developed.

As the shear pins were to be designed for the region above (upslope of) the tension cracks where no readily noticeable signs of failure were observed, a fully softened friction angle of 20 degrees was assigned (and was supported by laboratory testing) to the weathered shale. The fully softened shear strength was meant to account for weathering of the shale layer that resulted in a weakened shear strength condition compared to that of peak strength (unweathered condition).

The shear strength of the proposed steel reinforcement bars was considered and modeled in SLIDE to properly size the shear pins, to stabilize the failure block and to satisfy a FS = 1.5 using a fully softened shear strength of the rock. An additional check was done to verify that the design solution provided a FS $\geq$ 1.1 for the residual shear strength condition. Both analyses included a tension crack filled with water at the back of the slope. The shear pins extended 10 ft (3 m) below the potential failure plane to provide resistance against a passive failure in rock and to provide fixity at the pin tip with an additional FS = 2.0. The total bar length was 30 ft (9.1 m).

4.4 Structural Considerations

A #20 (No. 64), 75 ksi (517 MPa) steel epoxy coated, reinforcing bar was considered for analysis. Based on the SLIDE analysis, the maximum center-to-center spacing for a #20 bar was approximately 8 ft (2.44 m) within a row, with a minimum of two rows. Conservatively, a third row of nails was used in the critical areas. Although the shear pins were to consist of #20 steel reinforcing bars placed in a 6 in (152 mm) diameter hole and encased with 3 ksi (20.7 MPa) grout, the shear strength calculations (conservatively) considered only the shear resistance from the steel reinforcing bar and neglected the shear resistance provided by the grout. A FS = 1.5 was applied separately to the shear strength of the bar. The final design of the slope required 110 shear pins along the slope.

For this particular application, the structural capacity of the micropiles was the controlling factor in the design. In particular, the shear strength of the reinforcing bar was the key variable. Bending stresses are not expected to control since the failure plane is a very thin discontinuity in between hard rock blocks.

4.5 Construction of the Micropiles and Field Observations

The three rows of shear pins were installed over a span of approximately three weeks without any significant inconvenience. Figure 8 shows the installation of the first micropile at the site.

Figure 8. Installation of First Micropile at the Site

Once the 110 micropiles were installed, removal of the failed mass was completed and the upper channel was reconstructed. Figure 9 shows a view of the site after completion of the project.

Figure 9. Installation of First Micropile at the Site

5 Conclusion: When to use What?

Slope stabilization with deep foundations is not widely used due to a lack of published design procedures. This article included the different types of analyses that have been used in successfully completed projects. In general, the structural elements are expected to control the design and not the geotechnical aspects.

For slides where the failure plane is constrained to a thin zone between competent blocks, such as rock slides, the controlling failure criterion is expected to be shear of the deep foundation element. For slides where this condition is not present, and the mass above the failure plane is not expected to be rigid such as in soil slides, bending of the deep foundation element is expected to control the design.

In addition to the structural aspects, the main geotechnical aspects to verify are the necessary spacing of the deep foundation elements, along the row and in between rows, and the required minimum embedment below the failure plane.

The use of these systems is expected to increase as more projects are completed using the proposed methodologies.