A Method for Predicting Mobilization of Resistance for Micropiles Used in Slope Stabilization Applications

A report submitted to the joint ADSC/DFI Micropile Committee for review and comment

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Chapter 1 – Introduction

Use of micropiles for stabilization of earth slopes has been proven technically effective through full-scale field implementation on numerous occasions. However, implementation of the technique remains rather limited for reasons that include:

1. general lack of understanding on the part of many owners and consultants of how and why the technique works;
2. questions regarding assumptions invoked in current design procedures and lack of a widely accepted method for analysis and design of micropile stabilization schemes; and
3. perception that use of micropiles is generally cost prohibitive except for extremely challenging conditions where other slope stabilization methods cannot be employed.

These three issues are interrelated. Several alternative design and analysis procedures have been proposed. These methods have commonalities, but also distinct differences in terms of the general approach, in how forces attributed to the micropiles are incorporated into the analysis, and in terms of assumptions regarding how the forces provided by the micropiles are established for input into stability analyses. The lack of a consensus design approach heightens the general lack of comfort with the technique on the part of many owners and consultants. These two issues lead to the understandable result of designers frequently taking very conservative design postures, which results in costs that may be greater than is truly needed. Combined, these three issues result in the approach seldom being considered, and if so, only as a last resort.

Despite these issues, micropiles have been successfully used to stabilize earth slopes. Fortunately, several sites have been instrumented to help develop a better understanding of the technique and how it works. While results of instrumentation activities have been analyzed by several investigators on a case by case basis, no comprehensive evaluation and analysis of these results has been performed on the collective set of available data. Given that several cases are now available, the opportunity is present to collectively review and analyze the available data to begin to judiciously address the issues noted above and to evaluate, modify, and enhance current design methods in light of all available information. This report documents results of a project performed under direction of the joint ADSC/DFI Micropile Committee and funded by the ADSC Industry Advancement Fund.

Objective, Methodology, and Scope of Project

The primary objective of the work described in this report has been to identify a method for predicting the resistance provided by micropiles used for slope stabilization that: (1) is consistent with the available field performance data, and (2) takes advantage of the improved knowledge gained from the performance observed in previous implementations. This objective has been met through review and analysis of case histories with well documented performance measurements, analysis of stability and load transfer for these case histories, development of an improved technique for predicting the resistance provided by micropiles, evaluation of the proposed technique as compared to available field performance measurements, and, finally, use of the technique to demonstrate its application for a hypothetical case and to demonstrate the implications of the technique for design of slope stabilization schemes with micropiles.

Organization of Report

This report is organized into six different chapters describing different aspects of the project. In Chapter 2, the proposed analysis procedure is described through a simple, hypothetical “hand” example for calculating the axial and transverse resistance provided by micropiles and the stability of the example slope. Chapter 3 then describes the anticipated implementation of the technique using presently available computer software and tools. Chapter 4 describes a series of evaluations performed to evaluate the effectiveness of the proposed approach for predicting mobilization of axial and lateral resistance of micropiles and for “calibration” of appropriate p-y and t-z models for use in designing slope stabilization schemes. A series of practical implications drawn from the results of these evaluations are presented in Chapter 5 along with discussion of several issues that remain to be addressed before the full potential of micropile stabilization schemes can be realized. Finally, Chapter 6 provides a summary of this report, significant observations and conclusions derived from the project, a series of
recommendations for implementation of the technique, and future work needed to verify and/or enhance the proposed procedures.
Chapter 2 - Recommended Analysis Approach

The recommended approach for analysis of slopes with micropiles is presented in this chapter via “hand calculations” for a simple example problem to demonstrate the approach and required calculations. The example selected is similar to many applications where micropiles have been used to stabilize earth slopes and thus serves as a convenient basis for demonstration of the analysis procedure without complications that are introduced in a more rigorous evaluation. The procedure generally relies on widely used “p-y” methods for predicting the shear resistance provided by micropiles and similar “t-z” methods for predicting the axial resistance provided by the micropiles. Use of these methods provides means for appropriately considering the relative stiffness of the micropiles and surrounding soil as well as for ensuring compatibility of the established axial and lateral contributions to micropile resistance. Techniques for implementing the proposed approach in more general cases are subsequently presented in Chapter 3.

Example Problem

The example considered is illustrated in Figure 2.1. The slope is composed of a stiff silty clay residual soil overlying a shale formation with a weathered surface. An existing failure has occurred along the surface indicated, with the scarp outcropping at the ground surface about 25 ft from the crest of the slope. The silty clay has effective stress shear strength parameters of $c' = 300$ psf, $\phi' = 26^\circ$, and unit weight, $\gamma = 120$pcf. Groundwater is located approximately at the surface of the weathered shale.

![Figure 2.1. Diagram of example problem.](image)

Limiting force equilibrium analyses of a simple two wedge model are performed with slices A and B shown in Figure 2.1. The shear strength parameters used are as follows:

- For the silty clay, $c'=0, \phi' = 26^\circ$ on the failure surface where cohesion is taken as zero due to existing slip surface
- For the weathered shale, $c'=0, \phi' = 16.9^\circ$ from back-calculations for a factor of safety = 1.0 on the existing failure surface

Plan for Stabilization Using Micropiles

A micropile scheme is tentatively planned for stabilization of the slope as indicated in Figure 2.2. The micropiles are tentatively assumed to be installed at $45^\circ$ from vertical with alternating piles battered upslope and downslope. As a preliminary plan, the micropiles are assumed to consist of 7 inch o.d., 80 ksi steel pipe with 0.45 inch thick wall installed into a minimum 9 inch diameter hole that is filled with 4000 psi grout.
As slide movement occurs, the micropiles will contribute to stability by mobilizing axial resistance parallel to the pile axis and shearing resistance perpendicular to the pile axis as shown on Figure 2.3. The forces shown are:

- $P_{u,m}$ = mobilized axial force on the upslope pile, anticipated to be in tension as indicated
- $V_{u,m}$ = mobilized shear force on the upslope pile
- $P_{d,m}$ = mobilized axial force on the downslope pile, anticipated to be in compression as indicated
- $V_{d,m}$ = mobilized shear force on the downslope pile

Since micropiles are passive elements, these forces will be mobilized as downslope movement occurs. As the sliding mass moves downslope, the micropiles will tend to deform as illustrated in Figure 2.4.

**Evaluation of Micropile Forces**

In order to evaluate the stabilizing effect of the micropiles, it is necessary to estimate the resistance mobilized as a function of downslope soil movement. Axial and transverse shear components of these forces must be evaluated to determine the magnitude of the components at compatible movement levels and at magnitudes that do not exceed the structural or geotechnical capacity of the pile.

**Analysis of Shear Resistance**

The shear resistance mobilized transverse to the pile axis is evaluated using the p-y approach, implemented via the computer code LPILE (Ensoft, 2006) in this example. P-y curves represent the soil as a series of nonlinear springs, which are established using correlations of stiffness and limit pressure with soil properties. The pile is modeled using the stiffness of an equivalent elastic beam. The p-y curves above the failure surface are offset by some specified displacement to simulate movement of the sliding
soil mass. The pile is thus loaded by the soil above the failure surface and transfers load through flexure to the soil or rock below the failure surface.

\[ M_{\text{max}} = \frac{f_y I}{c} = 1,142 \text{ in - kips} \]

where:
- \( f_y \) = yield strength of steel = 80 ksi
- \( I \) = moment of inertia of the steel pipe = 50 in.\(^4\)
- \( c \) = distance from neutral axis to extreme fiber = 3.5 inches

Data plotted in Figure 2.6 indicate that the pile would begin to yield in flexure at a soil movement just over 1.5 inches transverse to the pile axis with a load transfer in shear of around 60 kips. Note also that the stiffness in shear is 60 kips/inch at a displacement up to 0.5 inches (and shear up to 30 kips) and 44 kips/inch at displacements of around 1 inch (and shear up to 44 kips).

**Mobilization of Shear with Respect to Downslope Soil Movement**

A consideration of the deflected shapes of the piles shown in Figure 2.5b provides a means for computing the amount of axial and transverse pile movement associated with the general soil mass movement. The data suggest that pile deflections occur over a distance of approximately 5 to 6 feet. Using the 5 ft distance as representative of soil movements in the 0.5 to 1 inch range, consider the kinematics of displacement of a pile battered in the upslope and downslope directions as shown in Figures 2.7 and 2.8.
Figure 2.5  Computed pile response from L-Pile analyses for several different values of soil movement: (a) input soil movement, (b) pile deflection, (c) pile shear force, and (d) pile bending moment.
Figure 2.6  Mobilized moment and shear vs transverse soil movement

Figure 2.7  Kinematics of upslope pile displacement.

Figure 2.8  Kinematics of downslope pile displacement.
For a downslope soil movement of 1 inch, the corresponding soil displacement component, \( \delta \), transverse to the pile axis for the upslope and downslope piles is:

\[
\delta_{\text{lat-up}} = (\Delta x) \sin(54.5^\circ) = 0.81 \text{ in}
\]

\[
\delta_{\text{lat-down}} = (\Delta x) \sin(35.5^\circ) = 0.58 \text{ in}
\]

Since the pile is mobilizing shear at 44 kips/in of transverse soil movement, the pile will actually mobilize shear force at

\[
K_{t-up} = (0.81)(44) \text{ kips/in of downslope soil movement}.
\]

\[
K_{t-down} = (0.58)(44) \text{ kips/in of downslope soil movement}.
\]

**Axial Pile Extension with Respect to Downslope Soil Movement**

For the upslope pile, the length of pile within the zone of movement, \( L_o \) and distance \( x_o \) is:

\[
L_o = \frac{60 \text{ in}}{\sin(54.5^\circ)} = 73.70 \text{ in}
\]

\[
x_o = \frac{60 \text{ in}}{\tan(54.5^\circ)} = 42.80 \text{ in}
\]

So, for a soil mass movement downslope, \( \Delta x \), of 1 inch,

\[
L_o + \Delta L_{up} \approx \left[60^2 + (x_o + \Delta x)^2\right]^{1/2} = 74.29 \text{ in}
\]

and the pile is “pulled” through an axial displacement of \( \Delta L \) for 1 inch of downslope soil mass movement:

\[
\Delta L_{up} \approx 74.29 \text{ in} - 73.70 \text{ in} = 0.59 \text{ in}
\]

Similarly for the downslope pile, the length of pile within the zone of movement, \( L_o \) and distance \( x_o \) is:

\[
L_o = \frac{60 \text{ in}}{\sin(35.5^\circ)} = 103.32 \text{ in}
\]

\[
x_o = \frac{60 \text{ in}}{\tan(35.5^\circ)} = 84.12 \text{ in}
\]

So, for a downslope soil mass movement, \( \Delta x \), of 1 inch,

\[
L_o - \Delta L_{down} \approx \left[60^2 + (x_o - \Delta x)^2\right]^{1/2} = 102.51 \text{ in}
\]

and the pile is “pushed” through an axial displacement of \( \Delta L \) for 1 inch of downslope soil mass movement:

\[
\Delta L_{down} \approx 103.32 \text{ in} - 102.51 \text{ in} = 0.81 \text{ in}
\]

From these values, it is necessary to consider the axial stiffness and capacity of the pile in order to compute the rate at which axial force develops with downslope soil movement and the maximum axial force that can be mobilized.

**Axial Capacity**

Axial capacity calculations are performed first to determine the axial resistance above the failure surface. In this case, the micropile is designed to engage axial resistance above the failure surface
through side shear without reliance on the pile cap. For the example problem, the stiff silty clay is anticipated to provide an average unit side shearing resistance of 1.8 ksf over the 35 ft length of pile through this zone, thus for a pile with 9 inch (3/4 ft) minimum diameter:

$$P_{u-w} = (1.8 \text{ ksf})(0.75 \text{ ft})\pi(35 \text{ ft}) = 150 \text{ kips}$$

This magnitude of axial force represents the maximum axial force that can be mobilized to resist sliding if potential contributions from the pile cap are neglected.

The socket below the failure surface should be sized to provide a factor of safety (generally taken as 2.0) over the resistance cited above. For the example problem, the shale below the failure surface is anticipated to provide a unit side shearing resistance of at least 6 ksf. To provide a socket with at least 2.0 times the 150 kips maximum axial force above, the socket length should be:

$$L_{u-s} \geq \left(\frac{150 \text{ kips}}{6 \text{ ksf}}\right)\left(\frac{2.0}{0.75 \text{ ft}}\right) = 22 \text{ ft}$$

In general, field load tests should be performed to verify axial resistance values.

**Axial Stiffness**

Axial displacement occurs both below and above the failure surface as pile axial force is engaged, and displacement occurs both to mobilize side shear at the pile/soil interface and through elastic stretching of the pile itself. An analysis of the pile load versus displacement response above and below the pile axis can be performed using the t-z approach as described in Chapter 3. With calibration to load test data, this approach provides the most reliable estimate; however, load tests should include creep measurements to estimate the long term load-displacement response.

A simplified approach to estimating axial load may be made by estimating that the load transfer in side shear is mobilized at about 0.1 inches of relative pile/soil movement. With a uniform distribution of side shear, the load in the pile will vary uniformly from a maximum at the sliding surface to zero at the far end. Thus, the movement required to fully mobilize the axial resistance above the failure surface may be computed as:

$$\Delta_{up} = 0.1 \text{ in} + \frac{1}{2} \left(\frac{PL}{AE}\right)$$

Where:

- $P = \text{axial force in the pile} = 150 \text{ kips maximum}$
- $L = \text{length of pile} = 35 \text{ ft} = 420 \text{ inches}$
- $A = \text{pile cross-sectional area} = 9.26 \text{ in}^2$ for the pipe only
- $E = \text{modulus of steel} = 29,000 \text{ ksi}$

Under these assumptions, the soil movement required to fully mobilize the axial resistance of 150 kips above the failure surface, $\Delta_{up} = 0.22$ inches.

The axial displacement below the failure surface must be added to the value above to obtain the complete extension of the pile with respect to axial tension. Since the socket length is twice that required for an axial resistance of 150 kips, the estimated stiffness should use a value of $L$ equal to $\frac{1}{2}$ the socket length. A similar analysis as above for the socket below the failure surface yields a value for $\Delta_{down} = 0.13$ inches. Thus, a total pile extension of 0.35 inches is estimated for an axial tensile force of 150 kips and thus the axial stiffness of this pile is approximately $150/0.35 = 425 \text{ k/in}$.

**Mobilization of Axial Force with Respect to Downslope Soil Movement**

Since axial displacements of 0.59 inch and 0.81 inch are computed for the upslope and downslope piles, respectively, for a downslope soil movement of 1 inch, the axial forces in the piles are mobilized at a rate of:
Computation of Compatible Pile Forces for Stability Analysis

For a given magnitude of downslope soil movement, $\Delta x$, the forces from the micropiles acting on the sliding soil mass may thus be computed as:

$$V_{u-m} = K_{t-up} (\Delta x) = 35 \cdot \Delta x \text{ kips}$$

$$P_{u-m} = K_{ax-up} (\Delta x) = 251 \cdot \Delta x \text{ kips}$$

$$V_{d-m} = K_{t-down} (\Delta x) = 25 \cdot \Delta x \text{ kips}$$

$$P_{d-m} = K_{ax-down} (\Delta x) = 344 \cdot \Delta x \text{ kips}$$

Stability Analysis Including Micropiles

The equations of force equilibrium can be developed for the soil wedge A (Figure 2.1) but including the pile forces as illustrated on Figure 2.9. Forces from soil wedge B are included by considering a mobilized lateral force, $P_A$, in the equilibrium equations. For this simple hand solution, a target factor of safety for stability is selected and the pile forces computed.

<table>
<thead>
<tr>
<th>$P_{d,m}$</th>
<th>$P_{u,m}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_{d,m}$</td>
<td>$V_{u,m}$</td>
</tr>
<tr>
<td>$N \tan \varphi'$</td>
<td>$W$</td>
</tr>
<tr>
<td>$54.5^\circ$</td>
<td>$35.5^\circ$</td>
</tr>
<tr>
<td>$9.5^\circ$</td>
<td>$35.5^\circ$</td>
</tr>
</tbody>
</table>

Figure 2.9 Forces on wedge including micropile forces

Summation of forces perpendicular to the failure surface:

$$\sum F_\perp = 0 = N - W \cos(9.5) + P_A \sin(9.5) + \frac{V_{u-m} \cos(54.5) - P_{u-m} \sin(54.5)}{s_{up}} + \frac{-V_{d-m} \cos(35.5) + P_{d-m} \sin(35.5)}{s_{down}}$$

where $s_{up}$ and $s_{down}$ are the spacing on piles in the upslope and downslope directions, respectively.

As a first estimate, assume a required factor of safety of 1.4 and spacing of 3 ft on center between piles with alternating upslope and downslope piles. Thus, the spacing between piles in each upslope and downslope row is 6 ft.

$$W = \frac{1}{2} (120 \text{ ft}) (40 \text{ ft}) (0.120 \text{ kips/ft}) = 288 \text{ kips/ft}$$
Compute $P_d$ using $\phi'_m = \tan^{-1}\left(\frac{\tan(26)}{1.4}\right) = 19.2^\circ$:

$$P_d = \frac{1}{2} \gamma h^2 K_a = 48.47 \text{kips/ft}$$

Solve for $N$:

$$N = \left(288 \frac{\text{kip}}{\text{ft}}\right) \cos(9.5) - \left(48.47 \frac{\text{kip}}{\text{ft}}\right) \sin(9.5) - \frac{[35\Delta x \cos(54.5) - 251\Delta x \sin(54.5)]}{6}
- \frac{[-25\Delta x \cos(35.5) + 344\Delta x \sin(35.5)]}{6}
= 276.05 + 0.77\Delta x \text{kip/ft}$$

Summation of forces parallel to the failure surface:

$$\sum F_{\beta} = 0 = \frac{N \tan \phi'_r}{F} - W \sin(9.5) - P_d \cos(9.5) + \frac{V_{u-m} \sin(54.5) + P_{u-m} \cos(54.5)}{s_{up}}
+ \frac{V_{d-m} \sin(35.5) + P_{d-m} \cos(35.5)}{s_{down}}$$

For a target factor of safety of $F = 1.4$, and $\phi'_r = 16.9^\circ$ as computed previously,

$$\sum F_{\beta} = 0 = \frac{(276.05 + 0.77\Delta x) \tan(16.9)}{1.4} - 288 \sin(9.5) - 48.47 \cos(9.5)
+ \frac{35\Delta x \sin(54.5) + 251\Delta x \cos(54.5) + 25\Delta x \sin(35.5) + 344\Delta x \cos(35.5)}{6}
= -35.43 + 78.30\Delta x$$

$\Delta x = 0.45 \text{in}$

The pile forces can thus be computed as:

$$V_{u-m} = K_{t-up} (\Delta x) = 35 \cdot \Delta x = 16 \text{kips/pile}$$

$$P_{u-m} = K_{ax-up} (\Delta x) = 251 \cdot \Delta x = 114 \text{kips/pile}$$

$$V_{d-m} = K_{t-down} (\Delta x) = 25 \cdot \Delta x = 11 \text{kips/pile}$$

$$P_{d-m} = K_{ax-down} (\Delta x) = 344 \cdot \Delta x = 156 \text{kips/pile}$$

The major contribution to stability from the micropiles results from the axial load component in this case. Note that the computed pile forces are associated with a global factor of safety of 1.4. With the exception of $P_{d-m}$, these pile forces are below the structural and geotechnical capacity of the piles. The fact that the computed value for $P_{d-m}$ exceeds the axial geotechnical capacity of the pile above the sliding surface (150 kips) suggests that the actual factor of safety will be slightly less than 1.4 since the 156 kips required to achieve $F = 1.4$ is not available. However, it is worth noting that the remaining contributions to stability (lateral components for both the upslope and downslope piles and axial component from the upslope pile) are well below structural and geotechnical limit states suggesting that additional resistance can be mobilized with additional deformation. Any additional force mobilized will contribute to a greater factor of safety. One could consider decreasing the pile spacing to decrease the required axial force in the downslope piles. Alternatively, considering the small difference between the geotechnical capacity and required axial force for a factor of safety of 1.4, one could consider that the additional load could be
transferred to the pile cap, transferred to the upslope piles via the pile cap, or simply accept the result as being acceptably close.

**Computation of a Resultant Pile System Force**

As an alternative to the stability calculations presented in the previous section, a resultant force from the pile system can be determined and used in stability calculations since the magnitude of stiffness and the direction of each component pile force is known. Use of a single resultant force can greatly simplify subsequent computations and allow for the effect of the micropile system to be more easily evaluated. To demonstrate, consider the pile force components illustrated in Figure 2.10 along with the resultant force, $R_m$, that is equivalent to the micropile forces.

![Figure 2.10 Micropile forces and directions.](image)

**Horizontal component**

From horizontal equilibrium, the horizontal component of the result force is

$$R_m \cos(\theta) = \frac{V_{u,m} \cos(45) + P_{u,m} \cos(45)}{s_{up}} + \frac{V_{d,m} \cos(45) + P_{d,m} \cos(45)}{s_{down}}$$

With 6 ft center to center spacing for both upslope and downslope piles and using the stiffness values computed previously:

$$R_m \cos(\theta) = \frac{35\Delta x \cos(45) + 251\Delta x \cos(45)}{6} + \frac{25\Delta x \cos(45) + 344\Delta x \cos(45)}{6}$$

$$= 77\Delta x$$

**Vertical component**

Similarly, the vertical component of the result force can be derived from vertical force equilibrium:

$$R_m \sin(\theta) = \frac{V_{u,m} \sin(45) - P_{u,m} \sin(45)}{s_{up}} + \frac{-V_{d,m} \sin(45) + P_{d,m} \sin(45)}{s_{down}}$$

Substituting the appropriate values yields

$$R_m \sin(\theta) = \frac{35\Delta x \sin(45) - 251\Delta x \sin(45)}{6} + \frac{-25\Delta x \sin(45) + 344\Delta x \sin(45)}{6}$$

$$= 12.1\Delta x$$

**Resultant**

The ratio of the vertical to horizontal components of the resultant is:
Thus, \( \theta_k = 8.9^\circ \) and
\[
R_m = \frac{77\Delta x}{\cos(\theta_k)} = 78\Delta x \text{kip/ft}
\]
Since the failure surface has a dip angle of 9.5°, the resultant force from the micropile system acts at an angle 0.6° below (clockwise from) the failure surface. Thus, the resultant force is nearly parallel to the failure surface.

The effects of alternative pile spacing can readily be evaluated because the resultant pile system force is directly proportional. For instance, a pile spacing of 5 ft center to center (2.5 ft between alternating piles) would produce a resultant force of
\[
R_m = \left(\frac{6}{5}\right) 78\Delta x = 94\Delta x \text{kip/ft}
\]
The maximum force available for resistance could be conservatively set as the force at which any pile in the system reaches a limit state. For this example, the limiting condition would be the pile with a stiffness of 344\(\Delta x\), which would reach the limiting axial force of 150 kips at
\[
\Delta x = \frac{150}{344} = 0.44 \text{ in}
\]
For a \(\Delta x\) of 0.44 inches, the maximum shear is \(35\Delta x = 15.4\) kips, which is within the magnitude of shear that is below the yield moment in the pile and within the range for the transverse stiffness value that was used.

Now it is possible to use the restraining force provided by the pile group at the maximum level of soil mass movement established above and evaluate stability by computing a global factor of safety associated with these pile forces. For a soil mass movement of 0.44 inches,
\[
R_m = 78\Delta x = 78(0.44) = 34.3 \text{ kip/ft}
\]
and \(R_m\) acts at an angle 0.6° below (clockwise from) the failure surface.

Summation of forces perpendicular to the failure surface:
\[
\sum F_\perp = 0 = N - W \cos(9.5) + P_A \sin(9.5) - R_m \sin(0.6)
\]
Recalling \(W = 288\) kips/ft, and \(P_A = 48.47\) kips/ft and solving for \(N\):
\[
N = 276.41
\]
Summation of forces parallel to the failure surface:
\[
\sum F_\parallel = 0 = \frac{N \tan(\phi_r)}{F} - W \sin(9.5) - P_A \cos(9.5) + R_m \cos(0.6)
\]
Solving for the factor of safety using \(\phi_r = 16.9^\circ\) as computed previously,
\[
\sum F_\parallel = 0 = \frac{276.41 \tan(16.9)}{F} - 288 \sin(9.5) - 48.47 \cos(9.5) + 34.3 \cos(0.6)
\]
\[
F = 1.38
\]
which is just slightly below 1.4 as previously deduced because the downslope micropile force has been limited to 150 kips.

**Alternative Micropile Designs**

Alternative designs can be evaluated following a similar approach. For example, consider the use of upslope piles battered at 30° from horizontal (flatter angle), downslope piles battered at 60° from horizontal (steeper angle), and that two upslope piles are installed for each downslope pile. With piles at 3 ft center to center spacing and a 2:1 ratio of upslope to downslope piles, the effective spacing of upslope piles is 4.5 ft (two per 9 ft) and the effective spacing of downslope piles is 9 ft. The basic geometry for this design is shown on Figure 2.11.

![Figure 2.11 Micropile forces and directions for alternative design](image)

Assuming that the geometry does not change so much that the LPILE and axial analyses differ significantly (they could be repeated if necessary), the major differences will result from the differing kinematics as shown in Figures 2.7 and 2.8.

Following a similar procedure, the transverse and axial stiffness terms are found to be:

\[
K_{t-up} = (0.64)(44\,\text{in}) = 28\,\text{in} \quad \text{of downslope soil movement.}
\]

\[
K_{t-down} = (0.77)(44\,\text{in}) = 34\,\text{in} \quad \text{of downslope soil movement.}
\]

\[
K_{ax-up} = (0.77)(425\,\text{in}) = 327\,\text{in} \quad \text{of downslope soil movement, tension}
\]

\[
K_{ax-down} = (0.63)(425\,\text{in}) = 268\,\text{in} \quad \text{of downslope soil movement, compression}
\]

and the resulting forces are:

\[
V_{u-m} = K_{t-up}(\Delta x) = 28\Delta x \text{ kips}
\]

\[
P_{u-m} = K_{ax-up}(\Delta x) = 327\Delta x \text{ kips}
\]

\[
V_{d-m} = K_{t-down}(\Delta x) = 34\Delta x \text{ kips}
\]

\[
P_{d-m} = K_{ax-down}(\Delta x) = 268\Delta x \text{ kips}
\]

Table 2.1 shows a comparison of these values with the values determined previously for the originally assumed design. The table shows that the upslope pile in the alternative design will mobilize axial load more quickly (with less displacement) but transverse load less quickly than for the original design. The converse is true for the downslope pile, with transverse load being mobilized more quickly in the alternative design but axial load being mobilized more quickly in the original design. These differences are attributed to the orientation of the piles alone because the same pile section is being considered and...
changes in geometry (e.g. embedment depth) are neglected. In other words, these differences are exclusively due to the different relative orientation of soil movement with respect to pile orientation.

Continuing the analysis for the alternative design, the resultant force, \( R_m \), is determined to be

\[
R_m = 84.49 \Delta x \text{ kips/ft}
\]

with an orientation, \( \theta_R \), of

\[
\theta_R = -4.78^\circ
\]

with respect to the horizontal (negative meaning clockwise from horizontal). Again, limiting deformation to the deformation at which the force in any pile in the system reaches a limit state, the limiting deformation is computed to be

\[
\Delta x = \frac{150 \text{ kips}}{327 \text{ kips/in}} = 0.46 \text{ in}
\]

and, thus,

\[
R_m = 84.49 \Delta x \text{ kips/ft} = (84.49)(0.46) = 38.89 \text{ kips/ft}
\]

which acts at an angle of 14.3° from the sliding surface (9.5+4.8). Note that the magnitude of the resultant force has increased as a result of changing the design. Substituting this into the equilibrium equations as before, the computed factor of safety is found to be

\[
F = 1.50
\]

which is greater than the factor of safety for the original design \( F \approx 1.38 \). Note that this arrangement is more effective than the previous, since the same number of piles achieve a higher computed factor of safety (although some piles may need to be longer due to geometry). The higher factor of safety is achieved at almost the same computed soil movement as for the previous design (0.46 inch vs. 0.44 inch).

### Table 2.1 Summary of micropile force components for original and alternative designs.

<table>
<thead>
<tr>
<th>Pile</th>
<th>Force Component</th>
<th>Force Magnitude per unit of soil displacement (kip/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Original Design</td>
</tr>
<tr>
<td>Upslope Pile</td>
<td>( V_{u-m} )</td>
<td>35( \Delta x )</td>
</tr>
<tr>
<td></td>
<td>( P_{u-m} )</td>
<td>251( \Delta x )</td>
</tr>
<tr>
<td>Downslope Pile</td>
<td>( V_{d-m} )</td>
<td>25( \Delta x )</td>
</tr>
<tr>
<td></td>
<td>( P_{d-m} )</td>
<td>344( \Delta x )</td>
</tr>
</tbody>
</table>

### Summary

In this chapter, an approach is outlined for computing the contribution of a micropile group to global stability of a sliding mass via hand calculations supplemented with limited L-Pile computer analysis. The forces in the micropile are computed in a way that accounts for the effect of soil mass movement on the mobilization of axial and shear forces and that maintains compatibility between shear and axial force as a function of displacement. The approach provides a model to incorporate the pile forces into stability calculations, including the effect of pile forces on the resultant shear and normal forces at the sliding surface. However, because it is a “hand calculation”, it requires several simplifications and approximations that may not be appropriate for general cases. Techniques for implementing the approach for more general cases are described in Chapter 3.
Chapter 3 – General Implementation of Recommended Analysis Procedure

The calculations described in Chapter 2 illustrate the recommended approach to predict contributions from micropiles in earth slopes. This approach provides a means to account for the relative stiffness of micropiles and surrounding soil and to ensure compatibility of axial and transverse contributions to micropile resistance. Such “hand” calculations are useful for demonstrating the general approach, and may be useful as a check of more rigorous computer analyses or in cases where one can establish the location of the critical sliding surface by inspection alone. However, more common and realistic cases require more rigorous evaluation of stability to include consideration of more complex stratigraphy, more precise consideration of the non-linear nature of the pile-soil response, searches for the critical sliding surface, and other factors that cannot easily be included in hand calculations. As such, this chapter provides recommendations for more general implementation of the proposed approach. These recommendations generally follow the same logic as presented in Chapter 2, but with consideration of the need to account for more complex site conditions, to perform multiple calculations for various alternatives, and to perform more rigorous searches for the critical sliding surface.

Capabilities of Commercial Slope Stability Analysis Software

Most modern slope stability analysis computer programs (e.g. UTEXAS4, Slide 5.0, Slope/W, etc.) provide means for incorporating reinforcement forces due to piles, drilled shafts, micropiles, soil nails, and other similar members into slope stability analysis computations. Common implementations generally require that users input the location of the reinforcement and the magnitude and direction of resisting forces (or equivalent axial and lateral components) for the reinforcement to be used in the computations. Specific details of how such forces are input and, to a lesser extent, how they are incorporated into stability calculations vary from program to program. However, all programs generally adopt the perspective that the input forces are known values for the stability computations and, as such, reinforcement forces are treated in a manner similar to other applied forces such as surcharge loads.

Given the magnitude and direction (or perpendicular components) of the resisting force provided by slope reinforcement, commercial slope stability analysis programs generally incorporate the effect of the resisting force(s) in one of two ways. For more approximate methods of stability analysis such as Bishop’s Simplified Procedure or Janbu’s Simplified Procedure, most commercial programs simply add the resisting force provided by the reinforcement to the resistance provided by the soil along the sliding surface (i.e. adding the reinforcement resistance to the numerator in calculating the factor of safety). Some programs may alternatively choose to subtract the resisting force (or moment due to the resisting force) from the “driving forces” (i.e. subtracting the reinforcement resistance from the denominator in calculating the factor of safety). At least one commercial program has capabilities for calculating the factor of safety using either of these two methods depending upon user assigned preferences. Alternatively, for more rigorous “complete equilibrium” methods of stability analysis such as Spencer’s method, Morgenstern and Price’s method, and the Generalized Limit Equilibrium method, most programs simply include the magnitude and direction of the resisting force in the equilibrium equations for slices where the reinforcement intersects the sliding surface. This method is the most straightforward and robust means for incorporating reinforcement forces into slope stability computations as it enables both direct (e.g. from direct resistance to sliding) and indirect (e.g. from the reinforcement forces changing the normal stress on the sliding surface) contributions from reinforcement forces to be appropriately considered. Since indirect contributions from reinforcement forces can have either positive or negative effects on stability, the latter approach is strongly preferred over the technique commonly used with simplified analysis techniques.

In general, the resistance provided by micropiles and other forms of reinforcement varies with the location where the critical sliding surface intersects the micropile. In cases where an automated search for the critical sliding surface is to be performed, it is therefore necessary to establish a resistance function that describes how the resisting force and direction, or the resisting force components, vary with location as illustrated in Figure 3.1. Furthermore, axial and transverse resistance components, or the resultant magnitude and direction, may vary differently, which necessitates that two separate functions (either axial and transverse components, or magnitude and direction) be input to appropriately model the variation in micropile resistance. Resistance functions are generally input as a sequence of data points at discrete
positions (depths) along the micropile. The software then generally interpolates between the data points to establish the appropriate resistance at intermediate positions.

![Figure 3.1 Example distributions of axial and transverse resisting forces as function of sliding depth illustrating variation of resisting force for different sliding depths.](image)

How the axial and transverse resistance functions are input varies with the specific software being used, with varying degrees of generality. For example, UTEXAS4 allows the user to directly input independent axial and transverse components of resistance as a function of position along a micropile as location, axial force, and transverse force triplets. This method allows completely general distributions of axial and transverse resistance to be incorporated into stability analyses and is the most straightforward method available. In contrast, Slide 5.0 allows the user to define the magnitude of resistance as a function of position along the member, but only allows a constant direction for the resisting force to be specified for a single member. This approach limits the user’s ability to correctly model the resistance provided by a micropile using a single model member. However, this deficiency can be “worked around” by using two identically located members in the stability model – one member to model the axial resistance components and another to model the transverse components. Some programs are more restrictive and do not have capabilities for modeling members with variable resistance. Such programs should be used with great care as they may lead to erroneous conclusions from stability analyses.

Resisting forces used by most commercial software are commonly a force per unit width of slope so resistance for individual micropiles must be divided by micropile spacing for input into most slope stability analysis software. However, a few programs incorporate options to specify the resistance for individual micropiles and micropile spacing to automatically determine the resisting force per unit width of slope. Users must therefore be aware of the method used by their specific software and input values appropriately.

Regardless of the specific software used, computation of factors of safety for slopes reinforced with micropiles using modern commercially available software is relatively straightforward once the values of micropile resistance are determined. The challenge to evaluating stability lies primarily in establishing appropriate values for micropile resistance for different locations of the sliding surface. Procedures for accomplishing this following the general approach described in Chapter 2 are provided in the following sections.

### Calculation of Micropile Resistance for Use in Slope Stability Analysis Software

The magnitudes of axial and transverse components of micropile resistance vary with the properties of the surrounding soil, the stiffness of the micropile, the out-of-plane spacing of micropiles, and the position of the sliding surface along the micropile (i.e. sliding depth). Because of the large number of variables, the non-linear response of the micropiles, and the need to compute resistance for varying sliding depths, computer methods are generally needed to establish appropriate resistance data. The following sections describe methods for computing transverse resistance using p-y methods implemented in the commercial computer program, L-Pile, and for computing axial resistance using t-z methods implemented in computer spreadsheet software. These techniques are again demonstrated
using the simple example problem presented in Chapter 2 for consistency of presentation and to provide a comparison of results for hand calculations as compared to computer solution for the factor of safety.

**General Approach**

The general approach adopted to establish mobilized axial and transverse forces as a function of global soil movements is to first resolve the global soil movement into components perpendicular and parallel to the micropile as illustrated in Figure 3.2. The respective axial and transverse components of the soil movement are computed as

\[
\delta_{\text{axial}} = \delta_{\text{soil}} \sin(\theta - \alpha) \\
\delta_{\text{lat}} = \delta_{\text{soil}} \cos(\theta - \alpha)
\]

where \(\delta_{\text{soil}}\) is the global soil movement, \(\delta_{\text{axial}}\) and \(\delta_{\text{lat}}\) are respectively the axial and lateral components of soil movement, \(\theta\) is the batter angle of the pile measured from the vertical (taken as positive measured into the direction of sliding), and \(\alpha\) is the inclination of the sliding surface measured from the horizontal (taken as positive counterclockwise from horizontal). In this form, the term \((\theta - \alpha)\) is the relative batter angle of the pile with respect to the direction of sliding.

![Illustration of calculation of axial and lateral components of soil movement for use in evaluating load transfer to micropiles from moving soil.](image)

For the example considered in Chapter 2, \(\alpha\) is 9.5 degrees and \(\theta\) is positive 45 degrees for the upslope pile and negative 45 degrees for the downslope pile. Thus, the respective components for the pile inclined upslope are:

\[
\delta_{\text{axial-up}} = \delta_{\text{soil}} \sin(45 - 9.5) = 0.58\delta_{\text{soil}} \\
\delta_{\text{lat-up}} = \delta_{\text{soil}} \cos(45 - 9.5) = 0.81\delta_{\text{soil}}
\]

whereas the respective components for the downslope pile are:

\[
\delta_{\text{axial-down}} = \delta_{\text{soil}} \sin(-45 - 9.5) = 0.81\delta_{\text{soil}} \\
\delta_{\text{lat-down}} = \delta_{\text{soil}} \cos(-45 - 9.5) = 0.58\delta_{\text{soil}}
\]

The axial and transverse components of soil movement computed in this way are used as inputs to independently assess the mobilized axial and lateral resistance for a specific overall soil movement as described in more detail in the following sections. These computations are then repeated for several values of soil movement to develop results showing how the axial loads, bending moments, and shear forces are developed with overall slope movement.
Calculation of Transverse Resistance as a Function of Soil Movement

Transverse resistance for a given value of soil movement is computed using the commercial software, L-Pile, to determine the magnitude of the shear force at a given sliding surface depth. These computations are then repeated for a series of global soil movements to produce a curve indicating the magnitude of the mobilized shear force (or bending moment, etc.) as a function of soil movement (either the overall slope movement or the lateral component of that movement). Recent versions of L-Pile (versions 4.0M and 5.0) provide capabilities for analyzing piles subjected to specified profiles of soil movement without requiring tedious separate analysis of the upper and lower portions of the pile as described in Sabatini et al. (2005). A conceptual model for these computations is illustrated in Figure 3.3. The computations are similar to conventional L-Pile calculations except that loading of the pile is induced by lateral soil movements that are input to the program rather than through boundary conditions applied at the top of the pile. The solution is obtained using an iterative procedure to solve for lateral loads and pile deflections considering both the stiffness of the pile (EI) and the non-linear stiffness of the surrounding soil (defined by p-y curves). The output from the software includes the mobilized bending moments, shear forces, pile deflections, and net lateral soil pressures along the length of the pile.

Figure 3.3 Conceptual illustration of L-Pile model used to compute lateral response of piles subjected to lateral soil movements.

For the example of Chapter 2, the upslope and downslope micropiles were modeled independently to more accurately reflect the geometry of the problem. The micropiles were modeled as vertical piles within L-Pile; however, the transverse soil deformations \( \delta_{lat} \) applied to the upslope and downslope micropiles were different because of the different orientations of the upslope and downslope micropiles with respect to the direction of soil movement as described in the previous section. Thus, for a global downslope soil movement of 1 inch, the applied transverse displacements were 0.81 inches for upslope micropiles and 0.58 inches for downslope piles.

Figures 3.4, 3.5, and 3.6 show the computed lateral response of the upslope piles for selected values of global slope movement and for several sliding depths. In general, pile deflections and the magnitude of the mobilized bending moments, shear forces, and net soil reactions increase with increasing soil movement. However, the specific response varies with the depth of the sliding surface – the responses shown being generally representative of shallow sliding well above the soil-rock interface, sliding near the soil-rock interface, and sliding below the soil-rock interface. The computed response for downslope piles is generally similar to that shown for upslope piles with two minor exceptions related to the problem geometry. First, the magnitude of the transverse component of soil movement relative to the global soil movement is lesser for the downslope piles \( (0.5N\delta_{soil} \text{ vs. } 0.81N\delta_{soil}) \) so that greater global soil movement is required to mobilize similar response for downslope piles. Secondly, the depths where the piles intersect the soil-rock interface is greater for the downslope piles so that the pile response observed for sliding near the soil-rock interface actually occurs for sliding at greater depth.
Figure 3.4  Upslope pile response to soil movement with sliding depth of 10-ft: (a) pile deformation, (b) bending moment, (c) shear force, and (d) soil reaction.

Figure 3.5  Upslope pile response to soil movement with sliding depth of 33-ft: (a) pile deformation, (b) bending moment, (c) shear force, and (d) soil reaction.

Figure 3.6  Upslope pile response to soil movement with sliding depth of 45-ft: (a) pile deformation, (b) bending moment, (c) shear force, and (d) soil reaction.
Loehr and Brown, 2007  
Prediction of Micropile Resistance for Slopes

Differences in pile response with sliding depth are further illustrated in Figures 3.7 and 3.8, which respectively show the mobilized bending moments and shear forces for both upslope and downslope piles as a function of global soil movement for three different sliding depths. The data plotted in Figure 3.7 is the maximum moment determined at any location along the pile because this is the value that is relevant for structural design of the micropiles. In contrast, the data plotted in Figure 3.8 is the value of shear force taken at the location of the sliding surface, which may or may not be the maximum value along the length of the pile, because this is the value that is the lateral resisting force that is used for slope stability analyses. The maximum value of the shear force along the length of the pile also has some relevance related to the structural capacity of the micropiles, but is not plotted here for conciseness. These figures are similar to Figure 2.8 except that the mobilized shear forces and bending moments are plotted versus global soil movement rather than the transverse component of this movement used in Figure 2.8. The depths shown in Figures 3.7 and 3.8 differ for upslope and downslope piles for direct comparison of response for sliding above, near to, and below the soil-rock interface rather than at consistent depths.

![Figure 3.7](image_url)

**Figure 3.7** Mobilized maximum bending moments in micropiles versus global soil movement for: (a) upslope micropiles and (b) downslope micropiles.

Figures 3.7 and 3.8 show that substantial soil movement is required to mobilize transverse resistance in the micropiles. This is especially true for sliding depths above the soil-rock interface where full mobilization of resistance is not achieved at movements as great as 8 inches. Even at movements of 8 inches, the data indicate that the maximum bending moment continues to increase with additional movement although the mobilized shear forces level off at large displacements. Bending moments and shear forces are mobilized at substantially smaller displacements for sliding surfaces passing beneath the
soil-rock interface. It is also noteworthy that the downslope micropiles require greater soil movements to mobilize the same lateral resistance as a result of the orientation of the micropiles.

![Graph showing mobilized shear forces at sliding depth in micropiles versus global soil movement for upslope and downslope micropiles.]

Figure 3.8 Mobilized shear forces at sliding depth in micropiles versus global soil movement for: (a) upslope micropiles and (b) downslope micropiles.

**Calculation of Axial Resistance as a Function of Soil Movement**

As illustrated conceptually in Figure 3.9, the approach used to predict mobilization of axial resistance is similar to that used for computing the lateral resistance. The approaches are similar in that loading is induced by relative pile soil movements and the response is computed using an iterative procedure that accounts for the axial stiffness of the pile (EA) and the relative stiffness of the pile-soil interface as represented by non-linear shear springs. No commercially available software with capabilities for performing such analyses could be identified so a spreadsheet program was developed based on procedures described by Isenhower (1999). The spreadsheet program is capable of modeling cases with variable soil properties and soil displacements along the length of the pile, and for modeling the effect of a capping beam using a non-linear axial spring. Because the problem is non-linear, the solution is an iterative one that requires the user to successively estimate movements of the pile tip until an acceptable solution is achieved. Acceptably accurate solutions are generally obtained in 5 to 10 iterations.

Primary inputs to the model include properties to define the axial stiffness of the pile, data to define the response of soil surrounding the pile in side shear (i.e. “t-z” curves), and a profile of soil movement defining the magnitude of the axial component of soil movement as a function of position along the pile. For all computations performed as part of this project, t-z curves were assumed to follow an
elastic-perfectly plastic type of behavior similar to that illustrated in Figure 3.10. Specific curves were establish by computing the ultimate side shear capacity per unit length of pile using classical techniques to establish the ultimate value, \( t_{ult} \), and assuming a value for relative soil movement at the pile-soil interface where the ultimate side shear would be mobilized, \( z_{ult} \). Axial springs representing contributions from end bearing and from the pile cap are also modeled using an elastic-perfectly plastic type of response with the ultimate values respectively taken as the bearing capacity at the pile tip and the ultimate pullout resistance of the micropile-cap connection.

As was the case for analysis of lateral loads, the distribution of axial load within the pile is computed for a given profile of soil movement (the axial component of the global soil movement). These computations are then repeated for different soil movements to establish the magnitude of the axial load at the sliding surface as a function of soil movement. Typical results from these computations are plotted in Figures 3.11 and 3.12, which show the mobilized axial load along the length of pile for different values of global soil movement for the upslope and downslope piles, respectively. At relatively small movements, the mobilized axial loads are relatively low, with the peak axial load occurring at the sliding depth (i.e. at the interface between the moving soil and stable soil) and reduced axial loads occurring at positions away from the sliding surface due to load being transferred to the surrounding soil through side shear. With additional movement, the axial loads in the pile continue to increase until the side shear capacity is reached along the entire portion of the pile either above or below the sliding depth or until the
axial load exceeds the pile’s structural capacity. For the results shown in Figures 3.11 and 3.12, the maximum mobilized axial load is controlled by the side shear capacity above the sliding surface (as indicated by the linear shape of the axial load curve above the sliding surface and the slightly curved shape below) for the sliding surfaces above and near the soil-rock interface (Figs. 3.11a, 3.11b, 3.12a, and 3.12b). Conversely, the maximum mobilized axial load is controlled by the side shear capacity below the sliding surface for the case shown with the sliding surface below the soil-rock interface (Figs. 3.11c and 3.12c). It should be noted that the piles inclined upslope mobilize positive or tensile axial loads as the relative soil movement is upward, whereas piles inclined downslope mobilize negative or compressive axial loads because the direction of soil movement is downward with respect to the pile.

Figure 3.11 Axial response of upslope micropiles to soil movement for sliding depths of: (a) 10-ft, (b) 33-ft, and (c) 45-ft.

Figure 3.12 Axial response of downslope micropiles to soil movement for sliding depths of: (a) 10-ft, (b) 33-ft, and (c) 45-ft. Note axial forces are compressive.

Figure 3.11 also shows the influence of the capping beam, which allows for positive (tensile) axial loads at the pile top due to the relative soil movement between the soil and cap for piles inclined upslope (where soil tends to move into the cap). Figures 3.11a and 3.11b both indicate mobilization of axial load due to the capping beam with increasing soil movement and eventual mobilization of the ultimate capacity of the capping beam (100 kips) at large soil movements for sliding above or near to the soil-rock interface. In contrast, little axial load is attributed to the cap when the sliding surface is relatively deep (Fig. 3.11c)
because the side shear capacity of the pile below the sliding surface is fully mobilized prior to mobilizing significant load from the cap. Similarly, no axial load is mobilized at the pile top for piles inclined downslope (Fig. 3.12) because in this case the soil is moving away from the cap. It is important to note that the analyses for the upslope and downslope piles are, at present, uncoupled. Thus, loading induced in the cap from the upslope pile is not transferred to the downslope piles as would occur in reality.

Similar observations can also be made regarding mobilization of end bearing for the upslope and downslope piles. No end bearing contributions are mobilized for the upslope piles because the piles are being pulled out of the soil below the sliding surface. End bearing is mobilized for the downslope piles as they are pushed into the soil below the sliding surface; however, the magnitude of this contribution is only significant for the case where the sliding surface is below the soil-rock interface (Fig. 3.12c) because this is the case where the side shear capacity of the micropile below the sliding surface is fully mobilized.

The axial resisting force provided by the micropile at a given magnitude of soil movement is the axial force in the pile at the depth of the sliding surface. Figure 3.13 shows the mobilization of this resisting force for piles inclined both upslope and downslope for three sliding depths. The results shown illustrate the dependence of the mobilized and ultimate axial loads to sliding depth as a result of having differing lengths of pile over which to mobilize side shear. The results also indicate that the ultimate axial resistance tends to be mobilized at relatively small soil movements as compared to the movements required to mobilize lateral resistance (Fig. 3.8). In all of the cases shown, the ultimate axial resistance was reach at global soil movements of less than 2 inch.

![Figure 3.13](image-url)  
Figure 3.13 Mobilized axial resistance at sliding surface depth versus global soil movement for: (a) upslope micropiles and (b) downslope micropiles.
Establishing Appropriate Design Values for Axial and Lateral Resistance

Results defining the mobilization of axial and lateral resistance (i.e. Fig. 3.8 for lateral loads and Fig. 3.13 for axial loads), supplemented with results from similar analyses performed for additional sliding depths, are used to establish design values of axial and lateral resistance based on tolerable deformation limits established for a particular case. Because values for axial and lateral resistance are both dependent on the depth of sliding, values must be established for the range of sliding depths being considered to produce resistance functions defining the axial and lateral resistance as a function of sliding depth. Figures 3.14 and 3.15 show two sets of possible resistance functions for the upslope and downslope micropiles considered in the example problem, respectively. One set corresponds to the ultimate resistance determined for each loading mechanism, regardless of the deformations required to mobilize that resistance. These functions would be used for cases where relatively large deformations (in this case, up to 8 inches) would be considered tolerable. The other set corresponds to values of resistance established at a limiting global soil movement of 1 inch. Resistance functions could also be established at different levels of soil movement depending on case specific constraints.

![Resistance functions established for upslope micropiles: (a) axial resistance and (b) lateral resistance.](image1)

![Resistance functions established for downslope micropiles: (a) axial resistance and (b) lateral resistance.](image2)
As shown in Figures 3.14 and 3.15, the difference between the ultimate values of resistance and values established at a soil movement of 1 inch is substantial for some sliding depths and negligible for others. In the most extreme cases for this example, the lateral resistance values for sliding surfaces passing above the soil-rock interface differ by a factor of two. The alternative axial resistance functions generally show smaller differences; no difference in axial resistance is observed for the downslope piles. It is also worth noting that the magnitudes of the mobilized axial forces are generally greater than the magnitudes of the mobilized lateral forces for most sliding depths.

**Calculation of Overall Factor of Safety Using Established Micropile Resistance**

Once the axial and lateral micropile resistance functions are established (i.e. Figure 3.14 and 3.15), the process of performing stability analyses to evaluate the factor of safety for the improved slope is relatively straightforward. Axial and lateral resistance values established for individual piles are first divided by the micropile spacing to produce resistance values per unit length of the slope (unless the software will perform this operation automatically). These resistance values are then input into the slope stability analysis software and stability computations performed to evaluate the factor of safety.

For comparison with the results of the "hand calculations" presented in Chapter 2, the same example problem was evaluated using commercial slope stability analysis software with the predicted micropile resistance functions presented in Figures 3.14 and 3.15. Figure 3.16 shows the stability analysis model used for the example problem along with the location of the critical sliding surface for the slope without micropiles. The critical sliding surface shown is similar to the one considered in Chapter 2 and has a factor of safety of 1.0. Figure 3.17 shows the model with the locations of the micropiles indicated and values of axial resistance plotted along the micropiles. The axial resistance values are plotted on opposite sides of the upslope and downslope micropiles as a result of the fact that the downslope micropiles mobilize compressive (negative) axial forces while the upslope micropiles mobilize tensile (positive) axial forces.
software will establish appropriate values of axial and lateral resisting forces by interpolating between points defined in the input data according to the location where the sliding surface crosses the respective micropiles. This step is illustrated in Figure 3.18, which shows the sliding surface from Figure 3.16 with the axial and lateral micropile resisting forces determined by interpolation. The forces shown have the correct orientation, with the axial force for the downslope pile acting upwards (compression) while the axial force for the upslope pile acts downwards (tension). The magnitudes shown for the respective forces also indicate that the axial forces are substantially greater than the lateral forces for both the upslope and downslope micropiles.

Figure 3.18  Axial and lateral resistance forces (in lbs.) determined by interpolation of the input data at locations where sliding surface intersects micropiles. Sliding surface shown is the critical sliding surface for the unreinforced condition.

Figure 3.19 shows the distribution of effective normal stress and mobilized shear stress along the sliding surface from Figure 3.18 to illustrate the effect of the micropile forces. At the location of the downslope micropile, both the effective normal stress and mobilized shear stress are observed to decrease in response to the compressive axial load being applied on the sliding surface by the micropile. The component of this compressive force acting normal to the sliding surface in fact acts to reduce stability along the sliding surface because it reduces the normal stress and, thus, the available shearing resistance of the soil at this location (unless $\phi = 0$). This reduction in stability is counteracted by the component of the axial force acting parallel to the sliding surface, which tends to increase stability (reducing the required shear resistance), and by the normal and tangential components of the lateral resistance from the downslope pile; the overall effect (whether stability is improved or reduced) depends on the orientation of the micropile and the magnitude of the axial and lateral resisting forces. In contrast, the effective normal stress and mobilized shear stress are seen to increase at the location of the upslope pile in response to the applied tensile load at this location (Fig. 3.19). The normal component of the upslope axial force increases the normal stress on the sliding surface at this location, which in turn increases the available shear resistance and thus promotes stability. The effect of the axial resisting force is counteracted by the component of lateral resistance acting normal to the sliding surface at this location, but the overall effect is as shown. Both the axial and lateral resisting forces for both piles provide some component of direct resistance to sliding, which also contributes to the overall stability.

Figure 3.20 shows factors of safety for several representative sliding surfaces. The critical sliding surface for this case was determined to have a factor of safety of 1.16. This surface passes entirely outside of the reinforced zone as a result of the additional resistance provided by the micropiles. As shown in Figure 3.16, the location of the critical sliding surface without micropiles extends further back up the slope and beyond the slope crest in a similar location to that used in Chapter 2. Comparison of factors of safety and critical sliding surfaces for the unreinforced and reinforced cases reveals the common observation that inclusion of micropiles often causes the critical sliding surface to move, but in doing so produces an increase in stability as sliding is forced to occur along a surface that is more stable than the original critical surface. In this case, the critical surface is actually moved beyond the zone of soil reinforced by the micropiles. However, if fewer micropiles, or micropiles with less resistance were used, the critical sliding surface could move to other locations, including locations that intersect the
micropiles (e.g. the surface shown to have a factor of safety of 1.17 in Figure 3.20 could easily become critical if the micropile resistance was reduced).

![Graph of effective normal stress and mobilized shear resistance along the sliding surface shown in Figure 3.18.](image)

**Figure 3.19**
Graph of effective normal stress and mobilized shear resistance along the sliding surface shown in Figure 3.18.

**Figure 3.20**
Factors of safety determined using the proposed procedure for several different sliding surfaces.

Given these observations, it is not surprising that the overall factor of safety computed using slope stability analysis software with a thorough search for the critical sliding surface is substantially lower than the one computed from hand calculations because a search for the critical sliding surface was not performed for the hand calculations and the respective surfaces are substantially different. However, comparison of factors of safety from the hand calculations and computer solution for similar sliding surfaces reveals that the factors of safety are similar (1.44 for the computer solution versus 1.38 for the hand calculation). While this comparison supports both the hand calculations and computer solution
(they should be similar!), the fact that the sliding surface considered is not the critical sliding surface highlights the importance of performing thorough searches for the critical sliding surface when evaluating the stability of slopes with micropiles.

**Kinematically-induced Axial Load**

The computations described previously in this chapter account for axial and lateral loading induced by the soil moving with respect to the micropile. However, the computations neglect the potential for additional axial load that may be induced by deflection of the micropile near the sliding surface as illustrated in Figure 3.21. This "kinematically-induced" axial load develops as a result of "kinking" of the micropile near the sliding surface, which will tend to extend the micropile over a short distance and, in turn, induce a tensile axial load in the pile at this location.

![Figure 3.21 Illustration of "kinematically-induced" axial load in micropiles due to pile deflections near to the sliding surface.](image)

The magnitude of the kinematically-induced axial load can be approximated by considering the geometry of the undeflected and deflected micropile near the sliding surface as illustrated in Figure 3.22. The “kink” in the pile is assumed to occur over some transfer length, \( t \), and the deflected shape of the pile is approximated by the piecewise linear shape shown in the figure. The average strain induced in the pile is then computed as the increase in length of the deformed section relative to the original length, \( t \). This strain is then multiplied by the axial pile stiffness \( (EA) \) to produce an approximate kinematically-induced force in the micropile at this location. The resulting equation for the kinematically-induced axial load is

\[
Q_{\text{kin}} = \frac{1}{t} \left[ \sqrt{t^2 + \left( \frac{t}{\tan(\theta - \alpha)} + \delta_{\text{soil}} \right)^2} - \frac{t}{\sin(\theta - \alpha)} \right] - \delta_{\text{soil}} \cos(\theta - \alpha) \right] (EA)
\]

where \( t \) is the assumed transfer length, which depends on the stiffness of the pile relative to the surrounding soil and the thickness of the sliding zone (Fig. 3.3), \( \delta_{\text{soil}} \) is the magnitude of the global soil movement, \( \theta \) is the pile batter angle, and \( \alpha \) is the inclination of the sliding surface as defined previously. Estimates of transfer length for a given problem can be deduced from results of lateral load analyses such as those presented in Figures 2.5b, 3.4a, 3.5a, and 3.6a. While the transfer length can vary substantially from problem to problem, it will generally vary from as short as a few feet for cases with relatively flexible piles in very stiff soil/rock to several dozen feet or more for cases with relatively stiff piles in softer soils.

For a given pile geometry, the magnitude of the kinematically-induced axial load depends on the transfer length and the magnitude of global soil movement as shown in Figure 3.23. Kinematically-induced load is generally small for small values of soil movement, but increases dramatically with increasing soil movement with the rate of increase depending primarily on the assumed transfer length \( (t) \). As shown in the figure, the kinematically-induced load can become quite large at relatively large soil movements, especially if the transfer length is small.
Any kinematically-induced axial load produced by kinking of the pile will be in addition to load developed due to the soil moving with respect to the pile. The kinematically-induced load can only be tensile (positive) and acts to induce a point load applied at the location of the “kink” in the pile. This load is then distributed along the length of the pile above and below the sliding surface according to the appropriate load transfer relationships in addition to loading induced by the axial component of soil movement. Figure 3.24 illustrates the effect of the kinematically-induced axial load for the upslope and downslope piles from the example problem considered in Chapter 2 and this chapter at a global soil movement of 1 inch. As shown, the kinematically-induced axial load acts to increase the tension in the upslope micropile and to decrease the compression in the downslope member. At smaller values of soil movement, the differences between mobilized loads determined with and without consideration of
kinematic loads is much smaller and often negligible, while for greater soil movements the differences become much greater. Additional impacts of the kinematically-induced load include:

- “Faster” mobilization of tensile axial load for piles inclined upslope
- Mobilization of tensile load for piles oriented perpendicular to the direction of soil movement (i.e. when the axial component of soil movement is zero)
- Reduction and potentially even reversal of compressive loads induced by the soil movement in piles inclined downslope.

Whether these impacts are significant for a particular case will depend on the magnitude of soil movement, the transfer length, and the orientation of the pile relative to the direction of sliding. Generally speaking, the influence of kinematically-induced axial load is small for small deformations but can become quite large, even dominating the load in the piles if soil movements become large. Additional discussion regarding kinematically-induced axial load, including consideration of kinematically-induced axial load in light of field measurements, is provided in subsequent chapters.

![Figure 3.24](image.png)

**Figure 3.24** Comparison of predicted axial load for upslope and downslope micropiles with and without consideration for kinematically-induced load.

**Summary**

In this chapter, the method recommended for implementation of the proposed approach for predicting mobilization of micropile resistance in slope stabilization applications is presented. The approach generally involves use of the computer program L-Pile to predict the magnitude of mobilized shear forces for different magnitudes of total soil movement and similar use of a spreadsheet program developed as part of this work to predict the magnitude of mobilized axial forces for different soil movements. From these calculations, the magnitude of the axial and lateral resisting forces provided by micropiles can be established for a range of sliding depths with consideration for compatibility between the axial and lateral loads as well as considering the magnitude of tolerable movements that are acceptable for the specific case. Once the magnitude of the axial and lateral resisting forces are established for a range in depths, it is a relatively straightforward process to input these forces into one of a number of commercially available slope stability analysis programs to evaluate the overall factor of safety for the slope incorporating the effects of micropiles.
Chapter 4 – Evaluation of Proposed Method Based On Field Measurements

To evaluate the proposed method, measured values of axial forces and bending moments from two case histories where micropiles were installed in moving slopes were compared to values predicted using the proposed technique. These evaluations also serve as preliminary “calibrations” of p-y and t-z models that are appropriate for application to design in similar conditions. General descriptions of the case histories are provided in this chapter along with a description of the techniques used to establish the predictions and calibrated models. Comparisons between measured and predicted values are then presented. Finally, predictions of the complete mobilization of axial and transverse loads using the “calibrated” models established through this process are presented and discussed.

Case Histories

Data from two projects were used to evaluate the proposed method of predicting micropile resistance for slope stabilization. For both projects, selected micropiles installed with upslope and downslope batter were instrumented with strain gages to monitor axial and bending loads in the micropiles as the soil moved and mobilized resistance in the members. The sites were also instrumented with slope inclinometers to monitor ground movements as well as various additional instruments to monitor pore water pressures at select locations, tilt in the capping beam, and loads in the tieback anchors installed as part of the stabilization. This instrumentation provided measured values for both soil movement and mobilized axial loads and bending moments that can be used to evaluate the general capability of the proposed technique to predict observed load transfer and, furthermore, to establish preliminary “calibrated” p-y and t-z models for use with the technique.

The first project, referred to as the Littleville Alabama project, is located on US Route 43 south of Florence, Alabama. Project design, construction, and monitoring results are described by Brown and Chancellor (1997). The generalized soil profile at the site consists of sandstone and shale embankment fill overlying a native shale stratum that is weathered near the original ground surface. The observed sliding surface at this site occurs at the interface of the weathered shale where it contacts with the overlying fill materials. Measured values of slope displacement, axial loads, and bending moments were taken from Brown and Chancellor (1997) and compared to values predicted using the proposed method.

The second project, referred to here as the SUM 271 project, is located in Summit County, Ohio along Interstate 271, approximately one-half mile east of the intersection of Interstate 271 with Interstate 77. The project design, construction, and monitoring are described by Liang (2000). Measurements of soil displacement, axial load in the micropiles, and bending moments in the micropiles were taken from this reference and compared to predictions as described subsequently in more detail. The generalized soil profile for this site consists of stiff to hard silty clay fill and native silty clays overlying a layer of softer silt and silty clay, which in turn overlies dense to very dense silt. The observed sliding surface for this site passes through the overlying fill and through the soft silty clay zones beneath the fill.

Approach to Comparison of Predicted and Measured Response

In both case histories, soil deformations and loads in the micropiles were periodically monitored during construction and for some period of time following construction. The available data thus provide information to relate slope movements to mobilized loads. The approach used to evaluate the proposed method was to take measured soil deformations at various dates, resolve these overall slope deformations into axial and transverse components as described in Chapter 3, and then to develop predictions of axial and lateral load transfer for the observed movements using t-z and p-y analyses to respectively establish axial and lateral loads based on the observed movements. These predictions were then compared to the measured axial loads and bending moments in the instrumented micropiles on the same dates (i.e. at same soil movement). For both cases, dates for analyses were selected to correspond to time periods where significant movements were observed at the respective sites.

Simple application of this approach using conventional, empirically derived p-y and t-z curves established from load tests consistently produced predicted moments and axial loads that greatly exceeded measured values. This suggests that p-y and t-z responses that are conventionally used for “actively loaded” piles may not be appropriate for use when loading is derived from moving soil (possible reasons for this will be discussed subsequently). As such, the empirically derived p-y and t-z curves were
subsequently modified to produce p-y and t-z curves that predicted the measured values of moment and axial load with reasonable accuracy when considering the entire range of soil deformation observed for the respective cases. Aside from demonstrating the capability of the proposed technique to model the observed response from these two cases, the "calibrated" p-y and t-z curves established from this evaluation serve as initial estimates of p-y and t-z curves to be used for design of micropile stabilization systems at other sites based on the available data.

Existing p-y and t-z models can be modified in a variety of ways to produce reasonable matches of predicted and observed pile response. However, to maintain some consistency in the evaluations, all p-y curves were modified using so-called "p-modifiers" as illustrated in Figure 4.1 for two different p-y models used in the evaluations. P-modifiers serve to modify the empirically established, or "baseline", p-y curves for a particular set of conditions by simply multiplying the p-values for the empirically established curves by the p-modifier value to produce a modified p-y curve. Use of p-modifiers greater than one produces modified p-y curves that are stiffer and have a higher ultimate capacity than the baseline (empirically derived) p-y curves. Conversely, use of p-modifiers less than one produces p-y curves that are less stiff and have a lower ultimate capacity (limit soil pressure). It is also possible to apply "y-modifiers", which alter the stiffness of the p-y curves but not the ultimate capacity, in addition to or instead of p-modifiers. However, this was not done in the current study and y-values were not changed from those established by the empirically determined models.

Figure 4.1  Example p-y curves for (a) API Sand model and (b) Soft Clay model with different p-modification factors applied to produce differing p-y response.

A similar approach was used to modify t-z curves for the evaluations as illustrated in Figure 4.2. The "baseline" t-z curves for the respective cases were established based on a simple elastic-perfectly plastic model shown in the figure. The ultimate side shear load for the baseline curve was established
using estimated undrained shear strengths and assuming “perfect adhesion” between the micropile and the surrounding soil (i.e. using an adhesion factor, $\alpha$, of 1.0). This ultimate side shear was assumed to be mobilized at a relative pile-soil axial displacement of 1% of the micropile diameter (Reese et al., 2006) to produce the t-z response shown for $\alpha=1.0$ in Figure 4.2. Modified t-z curves were then established by varying the adhesion factor, $\alpha$, while keeping the relative displacement values ($z$) constant. Similar to the p-modifiers, values of $\alpha$ greater than one produce a stiffer response with a greater ultimate side shear capacity while values less than one produce less stiff response with lower ultimate side shear capacity.

![Figure 4.2 Example t-z curves with different $\alpha$ factors applied to produce differing t-z response.](image)

The process of matching predicted to observed behavior is a non-unique one, especially when measured behavior is limited to relatively small deformations as is the case for the two case histories presented here. Frequently multiple “matches” to observed behavior can be produced. To account for this fact, and the fact that several reasonable assumptions for type of loading can be supported, two or more matches were generally established for the respective cases. Alternative matches were established using empirically derived “clay models” as well as using empirically derived “sand” models within L-Pile to attempt to account for the potential to have drained or undrained loading of the micropiles as the soil moves. It is likely that neither of these models is entirely appropriate as loading induced by soil movement differs substantially from the loading used to establish the empirical models, but at least they provide for a range of potential responses.

All analyses were performed using the best available data for soil properties and sites conditions provided in the reference reports to constrain sources of error to the extent possible. To further constrain the problem of matching predicted to observed behavior for development of appropriate p-y and t-z models, the following constraints were employed:

- P-modifiers and t-modifiers used to match the response of the micropiles were constrained to be the same for both upslope and downslope micropiles at each site (i.e. different modifiers were not used for upslope and downslope piles to produce better matches);
- P-modifiers and t-modifiers were constrained to be the same for all values of soil movement considered for each site (i.e. different modifiers were not used at different values of soil movement to produce better match); and finally,
- The same modifiers were used for all strata for each case (i.e. different modifiers were not used for different strata to produce better matches).

While these constraints alone do not ensure accuracy of the resulting models, they do serve to logically limit the number of possible solutions to the matching problem in a way that is likely to produce p-y and t-z models that can reasonably be used at future sites. It is certainly true that better matches between the predicted and observed response could be obtained if these constraints were not employed. However, models established from matching predicted and observed behavior without these constraints would have little value for future applications. In this sense, the constraints are used to avoid development of models that might match observed behavior, but that would not be logical or consistent with current...
understanding of load transfer to micropiles. The models presented subsequently using these constraints thus serve as models that produce the best available matches of the available data in aggregate rather than the best possible matches that can be achieved.

**Comparison for Littleville Alabama Project (Brown and Chancellor, 1997)**

For the Littleville Alabama case, two periods of notable movement were observed during monitoring. The first period occurred from August 4, 1994 through December 14, 1994 during construction of the micropiles and ground anchors. During this time, soil movements of 0.24-in were observed at the downslope micropile and movements of 0.34-in were observed at the upslope micropile at project station 2+70. The second period extended from the end of construction through March 18, 1996 during which an additional 0.05-in of movement was observed at the upslope micropile and 0.07-in of movement was observed at the downslope micropile. The micropiles utilized at this site were 6-in nominal diameter micropiles reinforced with 4.5-in O.D. pipe sections with 0.3-in wall thickness. The micropiles were installed in a conventional “A-frame” arrangement through a capping beam constructed at the ground surface, with adjacent micropiles inclined 30 degrees upslope or downslope. Center-to-center spacing of the upslope and downslope micropiles was 33 inches.

Measured values of bending moment and axial force were taken from instrumented micropiles at project stations 1+70 and 2+70. Measured soil movements were taken from inclinometers installed upslope and downslope of the capping beam at project station 2+70 near to the intersection of the piles with the sliding surface. Because measured soil movements were taken from station 2+70, and because the validity of measured bending moments and axial forces from station 1+70 was questioned by Brown and Chancellor (1997), strong preference was given to data from station 2+70 in the matching process. Measured data from station 1+70 is nevertheless included in subsequent graphics for completeness although it should be realized that the soil deformations at station 1+70 differ somewhat from the soil deformations from station 2+70 that were used in the analyses. Some of the differences between predicted bending moments and axial loads and measured values at station 1+70 can be attributed to these differences in movement.

Table 4.1 summarizes the generalized soil profile used for the evaluations and the relevant properties used for the respective strata. For all analyses, the properties of the micropile were taken to be those of the pipe alone, which had a diameter of 4.5-in, cross-sectional area of 3.96-in$^2$, moment of inertia of 8.77-in$^4$, and Young’s modulus of 29,000 kip/in$^2$.

| Table 4.1 Summary of parameters used in analyses for Littleville Alabama case. |
|----------------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Stratum                               | Nom. Thickness (ft) | Unit Weight, γ (lb/ft$^3$) | Shear Strength Parameters | Base t-z parameters | Base p-y model and parameters |
|----------------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Embankment Fill                        | 30              | 125             | 2000            | 0               | 25              | $f_{sult} = 8.3$ kip/ft |
|                                        |                 |                 | $c_{ult} = 0.06$-in |                 | $z_{ult} = 0.06$-in |
|                                        |                 |                 |                 |                 | $\phi = 25$ k = 10 lb/in$^3$ |
| Native Weathered Shale                 | --              | 125             | 5250            | 0               | 25              | $f_{sult} = 8.3$ kip/ft |
|                                        |                 |                 | $c_{ult} = 0.06$-in |                 |                 | $\phi = 25$ k = 10 lb/in$^3$ |

**Comparison of Mobilized Bending Moments**

Figures 4.3 and 4.4 show comparisons of measured bending moments with bending moments predicted using the “calibrated” p-y model for the instrumented piles battered upslope and downslope, respectively. In general, the predicted bending moments compare favorably in both shape and magnitude to measured values taken from station 2+70 over the range of deformations observed. The comparisons to data from station 1+70 are less favorable although similar trends are observed. It is important to emphasize that the soil movements used for the predictions shown are taken from station 2+70 so it is not surprising that better predictions are found for measured data from this station.
Figure 4.3 Comparison of measured and predicted mobilized bending moments for micropiles inclined upslope at Littleville Alabama project: (a) slope movement of 0.34-in and (b) slope movement of 0.39-in.

Achieving the comparisons shown required use of a $p$-modification factor of 0.2 regardless of whether the soft/stiff clay models or the API sand model were used. This suggests that current empirical $p$-$y$ models are “stiffer” than the data from this case support and that modified $p$-$y$ models may be more appropriate for this case and similar cases. Possible reasons for use of such a low $p$-modifier are presented subsequently.

**Comparison of Mobilized Axial Loads**

Figures 4.5 and 4.6 show comparisons of predicted and measured axial loads for the upslope and downslope micropiles, respectively, using t-z models calibrated to match the observed response. These figures indicate close comparison of measured and predicted results for these models with the greatest differences being observed at shallow depths near the capping beam. Differences in measured and predicted results near the capping beam are not surprising given that the proposed method does not
account for interaction between upslope and downslope piles due to the capping beam. Nevertheless, for this case the differences are relatively small even near the capping beam.

![Graph showing comparison of measured and predicted mobilized bending moments for micropiles inclined downslope at Littleville Alabama project](image)

**Figure 4.4** Comparison of measured and predicted mobilized bending moments for micropiles inclined downslope at Littleville Alabama project: (a) slope movement of 0.24-in and (b) slope movement of 0.31-in.

The required adhesion factor to produce the predictions shown in Figures 4.5 and 4.6 was 0.3. This value is lower than expected, but within a range of plausible values especially when considering that the lateral soil response (i.e. p-y response) required to produce a reasonable match also required significant reduction of empirically derived p-y curves. Combined these results could suggest that the soil properties used for the analyses may not be entirely appropriate; however, it is unlikely that any errors in shear strength are large enough to explain the required modifiers.

Overall, substantial reductions to conventional p-y and t-z curves were required to achieve reasonable matches between measured and predicted axial loads and bending moments for the Littleville Alabama case. Nevertheless, the favorable comparisons achieved in terms of both magnitude and
distribution of values suggest that the proposed technique is capable of accurately predicting mobilization of bending and axial loads in micropiles, at least over the range of deformations observed.

![Graph showing comparison of measured and predicted mobilized axial loads for micropiles inclined upslope at Littleville Alabama project: (a) slope movement of 0.34-in and (b) slope movement of 0.39-in.](image-url)
Comparison for SUM 271 Project (Liang, 2000)

Four micropiles, designated as Pile 1 through Pile 4, were instrumented for the SUM 271 project. All micropiles were 8-in diameter micropiles reinforced with 5.5-in O.D. steel pipe with 0.3-in. wall thickness. The micropiles were installed in a conventional “A-frame” arrangement through a capping beam constructed at the ground surface, with adjacent micropiles inclined 30 degrees upslope or downslope to create the A-frame arrangement. Upslope and downslope piles were spaced at 4.5-ft center-to-center. Piles 1 and 4 were both inclined upslope; piles 2 and 3 were inclined downslope. In general, data acquired from Pile 4 were erratic and inconsistent with data from the remaining piles. Data for Pile 4 were therefore not used for comparison of measured and predicted values of bending moment or axial load when establishing suitable p-y and t-z curves, but are included in subsequent graphics for completeness.
Unfortunately, no inclinometers were present near the location of the micropiles during or after construction at the SUM 271 site as inclinometers in these locations had to be abandoned for construction activities. However, two inclinometers located near the crest of the slope remained in service throughout the monitoring period. Based on observations from these inclinometers, two significant periods of soil movement were observed at this site prior to tensioning of the ground anchors. The two periods range from May 13, 2000 to May 26, 2000 during which nominally 0.25-in of soil movement occurred and May 26, 2000 to June 30, 2000 when an additional 0.35-in of soil movement was observed. These movements occurred during installation of the micropiles and ground anchors, but prior to tensioning of the ground anchors. Subsequent tensioning of the ground anchors also induced changes in the mobilized loads; these changes were also modeled and predictions compared to measured loads following tensioning of the ground anchors.

The generalized soil profile used for the evaluations and the relevant properties used for the respective strata at the SUM 271 site are summarized in Table 4.2. For all analyses, the properties of the micropile were taken to be those of the pipe alone, which had a diameter of 5.5-in, cross-sectional area of 4.96-in$^2$, moment of inertia of 16.80-in$^4$, and Young’s modulus of 29,000 kip/in$^2$.

### Table 4.2 Summary of parameters used in analyses for SUM271 case.

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Nom. Thickness (ft)</th>
<th>Unit Weight, $\gamma$ (lb/ft$^3$)</th>
<th>Shear Strength Parameters $s_u$, $c'$, $\phi'$</th>
<th>Basel t-z parameters</th>
<th>Base p-y parameters</th>
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</thead>
<tbody>
<tr>
<td>Embankment Fill</td>
<td>19</td>
<td>100</td>
<td>3000 0 25</td>
<td>$f_{u,ult} = 6.3$ kip/ft $z_{ult} = 0.08$-in</td>
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<td></td>
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<td>$\phi = 25$</td>
<td>API Sand Model:</td>
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<td>Soft Silty Clay</td>
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<td>1000 0 10</td>
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<td>$\phi = 10$</td>
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<tr>
<td>Dense Silt</td>
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<td>4500 0 40</td>
<td>$f_{u,ult} = 9.5$ kip/ft $z_{ult} = 0.08$-in</td>
<td>Stiff Clay Model: $\varepsilon_{50} = 0.02$ $c = 31.3$-psi $k = 10$ lb/in$^3$</td>
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<td>$\phi = 40$</td>
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**Comparison of Mobilized Bending Moments**

Comparisons of measured and predicted bending moments for the three events that produced changes in mobilized loads are presented in Figures 4.7 and 4.8 for micropiles inclined upslope and downslope, respectively. Unlike what was observed for the Littleville Alabama site where mobilized bending moments in the piles near the capping beam were limited, substantial bending moments were observed in the micropiles near the capping beam at the SUM 271 site even prior to tensioning of the ground anchors. These moments were modeled in the predictions by assigning bending moment boundary conditions at the pile top that were approximately equal to the measured values. Such boundary conditions generally had a limited effect on predicted bending moments near to the sliding depth so the predicted bending moments near to the sliding surface are similar to those that would be predicted with a zero moment boundary condition (as used for the Littleville case). However, it is important to note that these boundary moments were not independently predicted by the proposed method since the method is currently not capable of modeling the effect of the capping beam in coupling the response of the upslope and downslope piles as would be required to accurately model this development.
Figure 4.7  Comparison of measured and predicted mobilized bending moments for micropiles inclined upslope at SUM 271 project: (a) slope movement of 0.25-in, (b) slope movement of 0.60-in, and (c) slope movement of 0.60-in following anchor tensioning.
Figure 4.8 Comparison of measured and predicted mobilized bending moments for micropiles inclined downslope at SUM 271 project: (a) slope movement of 0.25-in, (b) slope movement of 0.60-in, and (c) slope movement of 0.60-in following anchor tensioning.
As shown in Figures 4.7 and 4.8, reasonable matches to both the trend and magnitude of observed bending moments were generally obtained for all three "loading events" (the two soil movement events and the tensioning of ground anchors event). Some tendency to overpredict moments above the sliding depth is observed but all in all the predictions are reasonably close to measured bending moments. However, as indicated in the figures, achieving these matches required use of a p-mobilization factor of 0.02. The reasons for requiring such a large reduction in p-y response from empirically derived curves are not evident, but one plausible factor is that the soil movements used to derive the predicted responses were measured near the crest of the slope and may be greater than movements that occurred closer to the micropiles. If soil movements adjacent to the micropiles were substantially smaller than those used for the calibrations, substantially smaller reductions (or greater p-modifiers) would produce a better match between measured and predicted response.

Comparison of Mobilized Axial Loads

Comparisons of measured and predicted mobilized axial loads for micropiles inclined upslope and downslope are shown in Figures 4.9 and 4.10, respectively. These figures indicate that the matches achieved do tend to predict reasonable trends in mobilization of axial load, even including the reversal of the axial loads due to tensioning of the ground anchors (Figures 4.9c and 4.10c). However, the magnitudes of the predictions and measurements for the SUM 271 case do not compare as favorably as do those for the Littleville case. Predicted axial loads at low soil movements (Fig. 4.9a) were less than measured values while predicted axial loads at higher soil movements (Fig. 4.9b) were greater than measured values, which suggests that the "calibrated" model is not quite correct. One possible reason for the difficulty encountered in developing matching predictions is that significant axial load was induced by interaction of the piles with the cap as evidenced by the large axial loads measured near the capping beam. Such loads are not explicitly incorporated into the proposed method at this time so it is perhaps not surprising that a poorer match was obtained.

As shown in the figure, it was necessary to reduce the adhesion factor, $\alpha$, to a value of 0.1 to achieve the most favorable match with the observed data. This value is substantially lower than expected, and is probably below a plausible range of values. One possible explanation for this is that the estimated undrained shear strengths for the respective strata may be too high, which would then require use of lower adhesion factors to produce a reasonable match of predicted and observed axial response. The issue with slope movements being measured at some distance from the micropiles discussed previously also likely contributed to the required adhesion factor being very low.

Overall, reasonable comparisons of predicted and measured bending moments were obtained for the SUM 271 case. Comparisons of predicted and measured axial loads were less favorable, but the predicted responses generally produced trends in mobilized axial loads that were similar to those observed. However, the values of the p-modifier and the adhesion factor required to produce these matches were much lower than anticipated and were substantially lower than those required for the Littleville Alabama case (which were also lower than expected). A series of possible explanations for these observations are provided in the following section.

Factors Affecting Predictions and Measured Response

Comparisons presented in the previous section illustrate the general effectiveness of the proposed method for predicting mobilization of axial loads and bending moments in micropiles with reasonable accuracy. The fact that the approach is capable of such predictions is not surprising given its general nature and flexibility. More surprising, however, is that substantial modifications to common empirically established p-y and t-z curves are required to produce reasonable matches with field measurements for the two cases considered. It is certainly possible (perhaps even likely) that loading from moving soil simply requires different p-y and t-z models than are conventionally used for more typical foundation loading (i.e. loading applied at top of pile instead of applied over some length of pile from moving soil). However, there are also other factors that could be contributing to the apparent need to use modified p-y and t-z models. The following is at least a partial list of such factors:

1) Loading observed in the case histories could be fully or partially drained whereas loading for typical load tests (from which the empirical models were established) is generally undrained, at least for clayey soils;
Figure 4.9  Comparison of measured and predicted mobilized axial load for micropiles inclined upslope at SUM 271 project: (a) slope movement of 0.25-in, (b) slope movement of 0.60-in, and (c) slope movement of 0.60-in with tensioned anchor.
Figure 4.10 Comparison of measured and predicted mobilized axial load for micropiles inclined downslope at SUM 271 project: (a) slope movement of 0.25-in, (b) slope movement of 0.60-in, and (c) slope movement of 0.60-in following anchor tensioning.
2) “Group effects” produced by having closely spaced members could be impacting the observed response whereas empirically established p-y and t-z curves are based on individual pile response;

3) Batter effects produced by having the piles oriented at an angle to the direction of sliding;

4) Scale effects for small diameter micropiles as compared to larger diameter members used to produce the empirically established curves;

5) Possible grout-steel interface issues that could lead to unusual response for micropiles;

6) Possible softening of the pile-soil interface (e.g. from drilling fluid or other means);

7) Errors in measurement of soil shear strength or shear strength parameters;

8) Errors in estimated soil movements for soil surrounding the respective instrumented piles;

9) Inaccurate estimation of the soil deformation profile, and especially the thickness of the “transition zone” (Figs. 3.3 and 3.9), could lead to errors in predicted bending moments and axial loads and therefore errors in the deduced modification factors;

Of these factors, the first five would be “real” behavior that would justify use of modified p-y and t-z curves determined from the analyses presented above, as long as similar techniques were used in similar conditions. The one possible exception to this argument is item 5, which involves potential softening of the pile-soil interface during pile installation. If this factor were significant, it is possible that some “hardening” or “stiffening” of the p-y response could be observed with additional deformation, in essence producing a p-y response with a concave-upward shape for some range in deformations before full mobilization was achieved (not unlike what is often observed in pressuremeter tests). Alternatively, it is also possible that some “setup” or “freeze” could be experienced as the soil near the pile aged, which would tend to produce stiffer/stronger response with time. For both cases considered here, the deformations used to infer load transfer response occurred during or soon after construction (some movements occurred before all piles were installed), and movements were relatively small, so neither case provides information from which to judge the reasonableness of these ideas.

Group and batter effects seem unlikely to be significant contributors to the apparent need for modification of current empirical models for slope applications. Micropile center-to-center spacings for the two cases are on the order of 3 to 4 times the diameter of the micropiles, which is at a level where group effects are generally believed to be small. Batter effects, on the other hand, are known to be significant for piles that are battered at angles similar to that used for these cases (Reese et al., 2006). However, if batter effects were significant, one would expect that substantially different modifiers would be required for both the upslope and downslope micropiles to achieve a reasonable match of the observed response. Reasonable matches were obtained using identical modifiers for the upslope and downslope piles for both cases, which suggests that batter effects are likely small.

In contrast, the last three factors listed are really potential sources of error that would cause the results of analyses presented above to be misleading and inaccurate. Considering these potential sources of error, it is possible, and perhaps likely, that some errors in the estimation of soil strength may be present. However, such errors would generally lead to underestimation of soil strength (from sample disturbance, sample selection, conservative strength interpretation, etc.) rather than overestimation of soil strength, which would tend to produce larger modification factors rather than smaller ones. Furthermore, even if the soil strengths were overestimated, it doesn’t seem plausible that the soil strengths would be in error to the degree necessary to produce such great differences between the “calibrated” and empirically derived curves. So while errors in soil strength may contribute to the differences observed, it seems unlikely that they are producing such significant differences alone. In contrast, errors in the measured soil movements (either due to measurement errors or due to the measurements not being representative of movements near to the micropiles) have greater potential to affect the calibrated responses because the movements are small in magnitude. However, estimated soil movements for the Littleville Alabama case were established from inclinometers located very close to the respective instrumented piles (for station 2+70) and thus the potential for having significant errors in estimated movements seems unlikely for this case. Errors in estimated soil movements for the SUM 271 case are more plausible though as the inclinometers used to estimate total soil movements were a substantial distance from the instrumented piles. Similarly, inaccurate estimation of the soil deformation profiles (which can dramatically affect computed moments and axial loads) seems implausible for the Littleville case since the soil deformation profiles used were taken directly from measurements from nearby inclinometers whereas such errors are more plausible for the SUM 271 case since the inclinometers were further away and may not be
representative of movements in the soil surrounding the instrumented piles. These observations suggest that more weight be given to results from the Littleville site, but that it seems likely that empirically derived p-y and t-z response curves must be substantially reduced to produce predictions that are consistent with field measurements.

Predicted Mobilization of Resistance Using Calibrated Models

Results presented in previous sections demonstrate that the proposed approach is capable of predicting mobilized axial loads and bending moments that are consistent with field measurements provided that modified (reduced) p-y and t-z models are used. The resulting calibrated models are based on soil movements that are relatively small (less than one inch). Strictly speaking, these models are therefore only appropriate for soil deformations of similar magnitude or lesser magnitude. However, the fundamental issue for assessing the improvement provided by micropile stabilization schemes is to estimate how much additional resistance can be mobilized with greater soil movements (at least up to some reasonable tolerable deformation limit). The calibrated models for the Littleville and SUM 271 cases were therefore used to predict the mobilization of axial and transverse resistance in the micropiles with additional soil movement to provide some indication of the resistance that can be mobilized based on the proposed method and available data. The results of these analyses are presented in this section. Readers are cautioned that the predicted resistance forces presented for movements greater than those used for the calibrations are based on extrapolation of the models beyond the conditions for which they were calibrated and therefore should be viewed with some caution. Points of calibration are indicated on the respective graphics to emphasize this fact.

For each case, results are presented for several calibrated models (recall the calibration in non-unique) to give some sense of the potential range of results that can be obtained. Alternative calibrated t-z models generally differ in the ultimate unit side shear (t_{ult}) used and the magnitude of relative pile-soil deformation (z_{ult}) required to mobilize this resistance. The alternative t-z models are indicated simply as Model A, Model B, etc. Predictions are also shown for calculations performed both considering and ignoring kinematically-induced axial load contributions to further illustrate the potential influence of these forces. Results provided that include consideration of kinematic loads are indicated with an asterisk, e.g. Model A*, while results computed without consideration of kinematic loads have no such indication, e.g. Model A. For results of lateral load analyses, different calibrated models are indicated by the respective “base” p-y model used for the calibrations. Thus models established by modifying the common “stiff clay” model (via p-modifiers) are indicated as “modified stiff clay” and models established by modifying the common API sand model are indicated as “modified API sand”.

Predicted Mobilization of Resistance from Littleville Alabama Project

Figures 4.11 and 4.12 show the predicted mobilization of axial load for the Littleville case using several models calibrated to fit the observed field performance. Figure 4.11 shows comparisons of the predicted axial loads as a function of total slope movement for analyses that include and ignore the effect of kinematically-induced axial load as described in Chapter 3. For the micropiles inclined upslope, inclusion of the kinematically induce load has little effect and the axial load is fully mobilized (reaching a pullout limit) at soil movements of approximately 0.5-in. In contrast, inclusion of kinematically-induced load has a dramatic effect for the micropiles inclined downslope. For downslope piles, predictions of axial load without consideration of kinematically-induced load indicate increasingly negative (compressive) load to deformations on the order of 0.5-in after which the load remains steady (due a pullout or “push through” limit). This response is similar to the observed response of the upslope micropiles except that the loading direction is reversed. Predictions of axial load for the downslope piles including kinematically-induced contributions are quite different. In this case the pile experiences increasingly negative (compressive) axial loads with increasing soil movements up to movements of approximately 0.5-in. However, with continued movement, the kinematically-induced contribution (which is always tensile) begins to counteract the compressive load and eventually induces overall tensile load in the micropile. The overall tensile load continues to increase until it reaches the ultimate condition at just over 2.5-in (again corresponding to a pullout limit state). As discussed subsequently, the available data from the two case histories is limited to soil deformations below where kinematic contributions become dominant. As such, some caution is warranted regarding whether kinematically-induced load should be counted on without additional evidence to confirm that they in fact are mobilized with additional deformation.
Figure 4.11 Predicted mobilization of axial load for micropiles inclined (a) upslope and (b) downslope for the Littleville Alabama case using calibrated t-z model showing comparison of mobilization with and without kinematically induced axial load.

Figure 4.12 shows predictions from several alternative models calibrated to fit the available data from the Littleville case to give some idea of the potential range of predictions beyond the calibration points, which are indicated with open circles in the figure. As shown in the figure, the three different calibrated models predict similar responses at soil deformation less than those used for the calibration, but somewhat different responses at greater deformations. For the upslope micropiles, the ultimate axial loads for the different models vary from approximately 60 to 100 kips. The difference is substantially smaller for the calibrated models for downslope piles. It is also notable that the predicted ultimate axial loads for both the upslope and downslope micropiles are substantially less than the structural capacity of the micropiles used (at least 200 kips), which suggests that some cost savings could potentially be realized by using lighter pipe sections for the micropiles.
Figure 4.12 Predicted mobilization of axial load for micropiles inclined (a) upslope and (b) downslope for the Littleville Alabama case using several alternative calibrated t-z models.

Figures 4.13 and 4.14 respectively show the predicted mobilization of bending moment and shear resistance for several calibrated models established for the Littleville case. These figures show that mobilization of bending moments and shear resistance requires substantially greater soil movement than is required to mobilize the axial loads, generally more than a foot of soil deformation to reach an ultimate condition. The mobilized bending moments and shear resistance are also observed to vary substantially depending on the specific calibrated model used, an observation at least partially attributable to the fact that the soil movements for the calibration points are much smaller than those required to mobilize substantial shear resistance. The ultimate bending capacity of the micropiles used for this case is approximately 400 in-kip. Figure 4.13 indicates that reaching this limit is not reached for predictions obtained using the modified stiff clay models but is reached for predictions obtained using the modified API sand model. However, even when the modified API model is used, the ultimate bending moment is not predicted until approximately 12 inches of soil movement has occurred. Since such extreme
movements are unlikely to be tolerable, some efficiency could potentially be gained by using a lighter pipe section without compromising the stability of the stabilized slope.

Figure 4.13 Predicted mobilization of maximum bending moment for micropiles inclined (a) upslope and (b) downslope at Littleville Alabama project for several alternative calibrated p-y models.

Mobilized shear forces are also seen to be smaller than the mobilized axial loads, even at large deformations. Predicted ultimate shear forces range from approximately 4 to 14 kips as compared to ultimate axial loads of 60 to 100 kips. The axial loads are also mobilized at much smaller soil deformations. This suggests that contributions from axial loads are likely to be more significant than contributions from transverse loads, especially at small deformations. However, this observation must be tempered by the fact that the axial loads often primarily contribute to stability by increasing (or decreasing in the case of compressive loads) the normal stress on the sliding surface, which in turn increases (or decreases) the soil shear resistance according to the soil friction angle. As extreme examples, a soil with a friction angle of zero would experience no contribution to stability from axial loads other than the
component of axial load acting parallel to the sliding surface while a soil with a friction angle of 45 degrees would realize the full benefit of the axial load.

Predicted Mobilization of Resistance from SUM 271 Project

Predictions of the mobilization of resistance for the SUM 271 project are presented in Figures 4.15 through 4.19. In general, these figures lead to observations similar to those provided in the previous section for the Littleville case with a few exceptions. The most noticeable difference is that the predicted ultimate axial loads, bending moments, and shear loads for the SUM 271 case are substantially lower than those predicted for the Littleville Alabama case despite the fact that the micropiles for the SUM 271 case were larger in diameter and more heavily reinforced than for the Littleville case. This observation is attributed to the fact that the soils present at the SUM 271 site were substantially weaker than those present at the Littleville site, especially near the sliding surface. Mobilized axial loads for the SUM 271
case range from 25 to 35 kips in tension for the micropiles inclined upslope to less that 15 kips for the micropiles inclined downslope. Mobilized shear forces are even smaller ranging from 0.7 to 1.4 kips for the micropiles inclined upslope and from 0.3 to 1.3 for the micropiles inclined downslope. It is noteworthy that the predicted ultimate axial loads and bending moments are substantially smaller than the structural capacity of the micropiles utilized, which suggests that more economical micropiles could have been used in this case without detrimental effect on stability (although member stiffness does play a secondary role in mobilization of axial and lateral resistance).

![Graph](image)

Figure 4.15 Predicted mobilization of axial load for micropiles inclined (a) upslope and (b) downslope for the SUM 271 case using calibrated t-z model showing comparison of mobilization with and without kinematically induced axial load.

Another noticeable difference is illustrated in Figure 4.17, which shows predicted axial loads for the upslope micropiles considering the effect of axial loading induced by tensioning of the ground anchors. As shown in the figure, tensile loads are mobilized in the member prior to tensioning of the ground anchor. Subsequent tensioning of the ground anchor then induces compression in the upslope
member which counteracts the tension induce by the soil movement. Additional soil movement, which tends to induce relative deformation promoting tension, does not induce additional tension however because the full frictional resistance of the pile-soil interface has already been mobilized prior to additional soil movement by the anchor tensioning and previous soil movement (both of which produce relative pile-soil movement in the same direction). Thus, anchor tensioning actually consumes some of the axial force that could be mobilized in tension and results in overall compression in the micropile. So while tensioning of the anchor promotes stability of the slope by inducing tension in the anchor itself, it actually decreases the effectiveness of the upslope micropiles by consuming some or all of the potential axial load that could be mobilized. The effect is not observed for the micropiles inclined downslope because the anchor acts perpendicular to the downslope anchor, which produces no significant axial loading on the downslope pile (but more significant lateral loading).

![Graph showing predicted mobilization of axial load for micropiles inclined upslope and downslope for the SUM 271 case using two alternative calibrated t-z models.](image-url)

Figure 4.16   Predicted mobilization of axial load for micropiles inclined (a) upslope and (b) downslope for the SUM 271 case using two alternative calibrated t-z models.
Figure 4.17  Predicted mobilization of axial load for micropiles inclined (a) upslope and (b) downslope for the SUM 271 case using two alternative calibrated t-z models and accounting for loading induced by anchor tensioning.
Figure 4.18 Predicted mobilization of maximum bending moment for micropiles inclined (a) upslope and (b) downslope at SUM 271 project for several alternative calibrated p-y models.
Summary
To evaluate the proposed method described in Chapters 2 and 3 for predicting mobilization of axial and transverse resistance for micropiles, the method was used to predict micropile resistance for two well-documented case histories. For each case, measured soil movements from the respective cases were used as input for the proposed method and the resulting predictions were compared to measured axial loads and bending moments from instrumented micropiles on the same dates. Predictions obtained using common empirically derived p-y and t-z models produced axial loads and bending moments that greatly exceeded measured values. As such, modified p-y and t-z models were subsequently used to produce reasonable matches of predicted and measured behavior for the two cases. Results of these evaluations indicate that the proposed method is capable of predicting the measured response, in terms of both magnitude and distribution of axial and bending loads, over the
range of deformations experienced for the two cases. However, achieving such predictions required use of p-y and t-z models that were substantially different from those in common use today.

In addition to the “calibrations”, results of additional analyses performed to evaluate the mobilization of resistance for the micropiles using several calibrated models at deformations that exceed those measured at the respective cases were described. Such analyses give some indication of the expected mobilization of resistance at greater deformations, albeit with some extrapolation beyond the calibrations. These results should be viewed with some caution because of the required extrapolation. Results presented in this chapter suggest that axial loads are mobilized at substantially smaller soil movements than are lateral loads. In addition, the results indicate that the predicted ultimate axial loads were substantially larger than predicted ultimate shear loads.
Chapter 5 – Practical Implications and Remaining Issues

Results presented in previous chapters have a number of practical implications for design and construction of micropile stabilization schemes. The results also raise or highlight several important issues that remain to be addressed if micropile stabilization schemes are to be used to their full advantage. In this chapter, a series of practical implications of the results presented in this report are presented and discussed along with several key issues that remain to be addressed.

Practical Implications

A number of practical implications can be drawn from the results presented in previous chapters. These implications can be grouped into three different categories:

1) Implications of incompatibility between axial and lateral load transfer
2) Implications of mobilization of axial loads, and
3) Implications of mobilization of lateral loads.

Implications within each of these categories are discussed in the following sections.

Implications of Incompatibility between Axial and Lateral Load Transfer

Results presented in this report clearly indicate that axial and lateral components of micropile resistance are often mobilized at greatly different rates. The ultimate axial resistance is frequently mobilized at slope deformations of an inch or less. Even in the worst cases evaluated, ultimate axial loads are mobilized within two to three inches of deformation. In contrast, substantially greater slope deformations are often required to mobilize significant portions of the ultimate lateral capacity and the ultimate lateral capacity may not be fully mobilized until slope movements exceed a foot or more. Factors such as micropile stiffness and orientation, site stratigraphy, and soil stiffness and strength affect the relative mobilization of axial and lateral loads. Nevertheless, the results suggest that consideration of compatibility between mobilized axial and lateral loads is necessary when predicting values of resistance to be included in slope stability computations. The method described in Chapters 2 and 3 for predicting mobilization of micropile resistance provides an appropriate means to perform such analyses, at least for cases where no capping beam is used or where sliding is deep enough to limit the influence of the capping beam on micropile resistance as in the cases used to evaluate and calibrate the technique (Chapter 4).

The fact that axial loads are mobilized at smaller soil movements than lateral loads suggests that axial loads may often be the primary contributor to stability, at least at “working deformations”. In a sense, the lateral resistance may be considered as providing important margins of safety against sliding, but in most cases the axial loads will tend to be the predominant load transfer mechanism under normal working conditions. This is analogous to the contributions of side shear (which frequently dominates load transfer at working loads) and end bearing (which frequently is mobilized at much greater settlements) to the overall stability of deep foundations. The fact that axial loads tend to be dominant at working deformations also leads to difficulties in evaluating appropriate p-y responses for micropiles from field measurements as it is problematic to acquire full-scale field measurements at deformations great enough to allow resolution of p-y response at larger deformations (as evidenced by the case histories considered in Chapter 4).

The fact that the ultimate axial resistance is generally mobilized at relatively small deformations also suggests that, in many cases, axial resistance can be reasonably estimated without performing t-z analyses unless strict control of deformations is needed (e.g. when the micropiles also support a structure of some form that necessitates control of deformations). This point of view generally seems warranted based on the evaluations presented in this report, although additional evaluations for differing site conditions and different micropile configurations should also be performed to ensure that similar behavior is observed for other conditions. For such estimates, data presented in Chapter 4 suggest that pile-soil “adhesion factors” on the order of 0.3 may be appropriate for slope stabilization applications, although additional data is needed to verify this value. It should also be emphasized that capping beam interactions between piles in a group can affect axial load transfer (especially when sliding is shallow) so improvements to the proposed method to account for the influence of the capping beam may lead to the conclusion that t-z analyses are necessary when capping beam interaction plays a significant role in load transfer.
The same cannot be said for estimation of mobilized shear resistance for micropiles. The results presented in previous chapters clearly indicate that the ultimate shear resistance for micropiles may not be mobilized at deformations that would be considered tolerable for the vast majority of applications. As such, it is apparent that mobilization of shear resistance in the micropiles should be predicted using techniques similar to what is described in Chapters 2 and 3 and then judgment used to establish an acceptable deformation limit from which appropriate and compatible values for shear resistance can be estimated.

**Implications of Mobilization of Axial Loads**

The importance of axial loads (or at least the realization of this fact) to some extent conflicts with assumptions inherent in many current design procedures, which are largely based on consideration of lateral resistance. Incorporation of appropriate axial resistance into many commercially available slope stability programs is generally straightforward, but attention must be given to the estimates of axial resistance used and in how they are incorporated into stability analyses. One specific practice that warrants careful consideration is that of including only the component of axial load that acts parallel to the sliding surface and ignoring the effect of the component acting perpendicular to the sliding surface. Because the perpendicular component changes the normal stress on the sliding surface and can have either a beneficial or detrimental effect on stability (depending on whether it is compressive or tensile), this practice seems unwarranted as it could lead to unconservative results in some cases or over conservative results in others depending on the specific problem.

The importance of axial loads also places increased importance on accurate estimation of shear strength parameters on the sliding surface(s) since errors in soil shear strength can lead to over- or under-estimation of the benefits of the micropiles. For example, a micropile with a 10-kip tensile load acting normal to the sliding surface would produce 1.7 kips of beneficial resistance [10-kips * tan (10°)] along the sliding surface if the friction angle is taken as 10 degrees but 2.7 kips of beneficial resistance [10-kips * tan (15°)] if the friction angle is taken as 15 degrees. This is a roughly 35% difference in the beneficial influence of the micropile for a 5 degree difference in friction angle. In the case of tensile micropile loads, the lower friction angle would be conservative. However, if the micropile had a 10-kip compressive load normal to the sliding surface, the normal load would actually reduce the resistance along the sliding surface in which case the higher friction angle would be conservative as it would produce a greater reduction in stability. This issue impacts the applicability of back-calculations as well. It is widely recognized that back-calculations are inherently non-unique (i.e. multiple combinations of shear strength parameters can produce a factor of safety of 1.0). However, many designers share the common belief that errors resulting from non-uniqueness will tend to be compensating if the same values are used for remedial design. This will not be true when evaluating the benefits of micropiles with substantial axial resistance as strength parameter combinations with greater cohesion intercepts and lesser friction angles will tend to diminish the (positive or negative) effects of the micropiles whereas combinations with lesser cohesion intercepts and greater friction angles will tend to enhance the influence of the micropiles. In general, estimates from back-calculations are greatly improved if as many other variables as possible (e.g. pore pressures, stratigraphy, slope geometry, etc.) can be constrained so designers should avoid complete reliance on back-calculations as much as possible.

The positive effects of tensile loads and the negative effects of compressive loads in micropiles suggest that use of micropiles inclined downslope should be avoided where possible. Similarly, use of ground anchors that may reduce tensile loads or even induce compressive loads in the micropiles should be carefully considered as it can greatly diminish the potential benefits of the micropiles. There are instances where the beneficial effects of anchors outweigh compromising the beneficial effect of some of the micropiles, such as when micropiles are necessary to create a reaction for the anchors or where sufficient stabilization cannot be achieved with micropiles alone. In such cases the micropiles will still provide some beneficial shear resistance but steps should nevertheless be taken to limit the impact of the anchors on the micropiles to the extent possible. t-z analyses similar to those presented in previous chapters are a good approach for estimating load transfer from anchor loads.

Finally, the importance of axial loads, and the dependence of axial loads on the side shear response of the pile-soil (and perhaps even the grout-steel) interface, suggests that axial pullout tests on micropiles may be more valuable than previously perceived for slope stabilization applications. Some attention must be paid to effects such as rate of loading and such tests should preferably be instrumented.
with strain gages along the length of the pile to allow for interpretation of t-z response in different strata, but even simple pullout tests would allow designers to more confidently assess the ultimate axial loads for micropiles, which could lead to substantial cost savings in some cases.

**Implications of Mobilization of Lateral Loads**

Results presented in Chapter 4 suggest that use of current empirically derived p-y models developed for conventional foundation loading is not necessarily justified for use where piles are loaded by moving soil. While based on a limited data set, results presented in previous chapters suggest that current p-y models should be modified by using a p-modification factor on the order of 0.2, and perhaps even less if the relevant soils are particularly soft.

Predicted ultimate bending moments from calibrated models for the two cases considered suggest that shear resistance values computed based on the ultimate structural capacity of the micropiles will tend to overestimate the resistance that can be mobilized within tolerable deformation limits. Estimates based on structural capacity may even exceed the ultimate shear load that can be mobilized at larger deformations since the evaluations presented indicate that the ultimate shear load tends to be controlled by factors other than structural capacity (at least for the sections and conditions considered). These observations also suggest that these piles may have “excess capacity” that cannot be realized due to other failure modes. As such, some cost savings could potentially be realized by reducing the quantity of steel in the micropiles. However, this has to be done with consideration given to the fact that reducing the quantity or form of reinforcing steel will affect the axial and bending stiffness of the members, which also plays a role in mobilization of resistance.

Results presented in Chapter 4 also show that mobilized shear loads can be substantially smaller than mobilized axial loads, particularly when the micropiles pass through substantial thicknesses of soft soil near the sliding surface as was the case for the SUM 271 site. In cases where mobilized shear loads are very small, or where excessive slope movements are required to mobilize significant shear resistance, some consideration could be given to reinforcing micropiles using deformed bars rather than pipe sections since deformed bars may promote better axial load mobilization, which could potentially improve overall stability. Use of deformed bars would also decrease the micropile bending stiffness, which in turn could increase contributions from kinetically induced axial loads (although issues discussed subsequently should be considered in this respect).

Finally, the fact that mobilization of lateral resistance requires substantial soil deformation is detrimental in slope stabilization applications because it means that the ultimate lateral resistance cannot commonly be counted on for stabilization. However, this fact is actually beneficial in cases where the primary issue is maintaining the structural integrity of piles in moving soil (e.g. in lateral spreading applications for bridge foundations, etc.) rather than controlling soil deformation as is the case for slope stabilization applications. Results of calibrations presented in Chapter 4 would suggest that design loads on such piles could perhaps be substantially reduced, although additional investigations for different site conditions and involving greater soil deformations should be performed to confirm this observation.

**Remaining Issues**

Results of evaluations presented in previous chapters lead to a number of significant observations and conclusions regarding load transfer in micropiles used for slope stabilization applications. However, the results also raise several issues regarding this problem and highlight other issues that were previously recognized, but perhaps underestimated. Several of the most important issues raised are described in the following paragraphs.

Perhaps the most notable issue arises from the fact that the observations and conclusions drawn from the work presented are based largely on evaluations for two well-documented case histories. While the primary observations and conclusions are generally consistent for the two cases and the cases considered are similar in many respects to conditions where micropiles are frequently utilized for slope stabilization, the fact remains that the observations and conclusions are drawn from a limited data set. Furthermore, the observations and conclusions are drawn based on loads mobilized at relatively small deformations and it is certainly possible that the behavior at larger deformations could be different from that predicted using the models calibrated using these two cases. Additional data is needed to provide verification of the observations and conclusions, especially for larger deformations and alternative site conditions. Several recommendations for acquiring such data are provided in Chapter 6.
The issue of kinematically mobilized axial loads also remains unconfirmed. Results provided in Chapters 3 and 4 suggest that kinematically induced loads, if they are mobilized, lead to more rapid mobilization of axial loads in micropiles inclined upslope, mobilization of significant axial loads for piles inclined perpendicular to the direction of sliding, and eventual reversal of compressive loads mobilized in micropiles inclined downslope at larger deformations. All of these results are positive and may contribute substantially to the effectiveness of micropile systems. However, at present there is little data to confirm that such loads are actually mobilized and, if they are mobilized, to what extent. The analyses performed also indicate that kinematically induced loads are sensitive to the “thickness” of the sliding surface (with thinner sliding zones producing greater kinematically induced load), which is difficult to predict in many cases. As such, caution should be used when considering the effects of kinematically induced loads until it can be more definitively demonstrated that they are in fact mobilized and unless evidence is available (e.g. from slope inclinometer measurements) to provide confidence in the thickness of the shearing zone.

Another issue that remains unresolved is to develop a method for predicting load transfer that appropriately accounts for the effect of capping beams. The “uncoupled” method proposed and evaluated in this work is a good place to start, but it is clear that this method does not explicitly account for interactions between upslope and downslope micropiles that are connected by a capping beam in its current form. What is needed to better address this issue is essentially a computer program similar to the “Group” program provided by Ensoft, Inc., but with capabilities added for modeling loading from soil movement in a manner similar to the capabilities provided in recent versions of L-Pile. Efforts to develop such a computer program could draw heavily upon techniques currently implemented in Group and L-Pile, as well as the spreadsheet programs developed for this project, but a concerted effort is required to integrate these different capabilities.

Finally, another issue affecting the ability to perform computations of the type described in this report is the fact that the computations are currently very tedious and time consuming. This issue poses considerable challenges to designers faced with limited time and budgets and limits the number of alternative scenarios that can be evaluated for a given project. While the computations do currently take advantage of computer tools such as L-Pile and commercial spreadsheet software, they still remain quite user intensive involving numerous repetitive calculations that can lead to errors in computations. Fortunately, the repetitive and mundane nature of the calculations makes them highly conducive to automation with computer software, but achieving this will also require a concerted effort.
Summary, Conclusions, and Recommendations

Effective use of micropiles for slope stabilization has been demonstrated via successful field performance for a large number of cases. However, accurate and reliable prediction of micropile resistance for design purposes remains challenging because of the complexity of load transfer mechanisms when micropiles are loaded by moving soil. The primary objective of this project was to identify a method for predicting micropile resistance in slope stabilization applications that is consistent with field observations from instrumented case histories. In the process of evaluating load transfer from the field performance, it became apparent that an alternative to existing methods was needed to more accurately predict the resistance provided by micropiles in earth slopes, and the overall stability of slopes reinforced with micropiles. This report describes a method for predicting micropile resistance in slope stabilization applications and subsequent analyses performed to evaluate the method. In this chapter, a summary of the content presented in previous chapters is provided. General observations and conclusions drawn from the work are then presented along with a series of recommendations both for implementing the proposed method in the short-term and for improving the method in the longer term.

Summary

The proposed approach for predicting micropile resistance for slope stabilization applications is illustrated through “hand calculations” for a hypothetical example problem in Chapter 2. The key feature of the approach is that it accounts for compatibility between mobilization of axial and transverse (shear) components of micropile resistance, which are observed to mobilize at substantially different rates. The analyses are performed by independently considering the axial and transverse (shear) load transfer in an uncoupled manner and then combining these components to produce an overall resisting force that can be included in stability analyses. While several approximations are necessarily invoked in the “hand calculations”, the solution demonstrates the necessity for considering both axial and lateral load transfer and for considering the compatibility of these two components of resistance. Furthermore, despite the fact that such calculations are likely to be insufficient for many practical cases, similar hand calculations may be useful for simple applications or as independent checks of more rigorous computer analyses.

More general implementation of the approach for predicting micropile resistance using computer analysis tools is described in Chapter 3. The more rigorous analyses permit the micropile response to be determined without restrictions or simplifying assumptions regarding stratigraphy, slope geometry, or micropile characteristics, and including consideration of the non-linear nature of the micropile response. The more general implementation follows the same approach as described for hand calculations, but makes use of more general “t-z” analyses implemented in computer spreadsheet software to evaluate axial load transfer and independent “p-y” analyses implemented in the commercial computer program, L-Pile, as suggested by Isenhower (1999). These techniques respectively permit prediction of mobilized axial and lateral resistance components as a function of overall soil movement, which can then be used to predict the overall micropile resistance at compatible soil movements. Once the overall resistance is established for an assumed micropile stabilization scheme, the overall stability of the slope can be evaluated using a number of commercially available slope stability analysis programs.

Analyses performed to evaluate the proposed technique for predicting micropile resistance are presented in Chapter 4. These evaluations consisted of comparing measured values of axial load and bending moment from two case histories with values predicted using the proposed approach for the observed soil movements at various stages of the monitoring periods. Preliminary analyses performed using common empirically derived t-z and p-y models produced predicted forces and moments that greatly exceeded measured values. This suggests that the conventional models commonly used for traditional foundation applications (i.e. where the pile is loaded with a point load at the top of the pile) are not suitable for analysis of micropiles for slope stabilization (where distributed loading along some length of the pile is induced by the moving soil, which in turn also contributes to the response of the pile). Subsequent analyses were therefore performed using modified forms of common p-y and t-z models to both evaluate whether the proposed approach was capable of predicting the mobilized resistance observed in the case histories as well as to establish preliminary “calibrated” p-y and t-z models that could be used for prediction of micropile resistance in similar conditions. These analyses were general successful in producing reasonable matches between measured and predicted mobilized axial loads and...
bending moments for the two case histories considered, albeit using p-y and t-z models that were substantially different from commonly used ones.

The results of analyses presented have a number of practical implications for prediction of resistance that can be mobilized for micropiles in slope stabilization applications. Several of these practical implications are presented and discussed in Chapter 5. In addition, several limitations of the work presented in this report are discussed and several issues that remain to be addressed are presented.

Finally, this chapter provides a general summary of the report and presents several of the most important observations and conclusions derived from the work. A series of recommendations for implementing the proposed technique are also presented along with several recommendations for addressing the limitations of this work and remaining issues.

**Significant Observations and Conclusions**

A number of observations and conclusions can be drawn from the work described in this report. The most significant of these include:

- The proposed method is generally suitable for accurately predicting the mobilization of axial loads and bending moments (or shear resistance) in micropiles used for slope stabilization and is general enough to permit accurate modeling of cases with complex slope geometries and varying micropile characteristics (e.g. capacity, inclination, stiffness, etc.) without undue approximations and simplifications.

- The uncoupled axial and lateral analyses utilized in the method appear to work well in cases where sliding is relatively deep, which limits the influence of the capping beam and ground anchors that are frequently used in combination with micropiles; in cases where sliding is more shallow, the current uncoupled analyses may not accurately predict the mobilization of micropile resistance if a capping beam is utilized, especially if used in conjunction with ground anchors placed through the capping beam.

- While the proposed method is capable of accurately predicting micropile resistance in slope stabilization applications, results of evaluations presented in this report indicate that doing so requires substantial modifications to current empirically established p-y and t-z models. Specifically, current p-y and t-z models must be modified to produce a “softer” response in order to match observations from field performance. This, in turn, suggests that use of current empirically derived p-y and t-z models developed for traditional foundation loading are not appropriate for predictions of micropile response in moving soil and that use of these conventional models will tend to overpredict the resistance that can be mobilized at a given soil movement.

- Results of analyses for both real and hypothetical cases indicate that full mobilization of axial resistance occurs at substantially smaller soil movements than is required to fully mobilize the lateral resistance. The ultimate axial resistance is generally mobilized at total soil movements of a few inches or less while the ultimate lateral (shear) resistance is generally not mobilized without substantially greater soil deformations, often requiring as much as a foot or more of deformation. This, in turn, suggests that the axial resistance of the micropiles can often be reasonably estimated using limit state approaches in most cases while estimation of the lateral resistance requires consideration of soil-structure interaction, compatibility, and tolerable deformations for the slope.

- Use of ground anchors tensioned through the capping beam can substantially reduce the potential resistance provided by micropiles inclined upslope if a portion of the anchor load is transferred to the depth of sliding. This effect is particularly pronounced if the soils above the depth of sliding are soft.

- Ultimate axial loads for the cases considered were also substantially greater than the shear loads that can be mobilized within tolerable deformation limits; however, it is important to realize that the axial loads primarily contribute indirectly to stability (via the frictional resistance of the soil on the sliding surface) by altering the normal stress on the sliding surface. In contrast, the lateral (shear) resistance often contributes more directly to stability
so the overall effect of the relative components cannot be appropriately evaluated by simple comparison of the magnitudes.

- Finally, the predicted ultimate axial, bending, and shear loads for the two cases evaluated were often substantially less than the structural capacity of the micropiles that were actually used, which suggests that the micropiles have substantial excess structural capacity that cannot be mobilized. Realization of this issue could lead to substantial cost savings for micropiles in future applications.

**Recommendations for Implementation**

Given the generally successful evaluation of the proposed technique based on available case history data, it is recommended that the method be used for prediction of resistance in micropiles for slope stabilization. Specific recommendations for use of the method in its present form include:

- In cases where the relative inclination of the micropiles and sliding surface are similar to those commonly used to date (e.g. micropiles inclined at least 20 degrees from the normal to the sliding surface), it seems justified to predict the axial micropile resistance using limit state techniques given the relatively rapid mobilization of axial resistance for micropiles with such inclinations. In other cases where micropiles are inclined roughly normal to the sliding surface, where micropiles have characteristics that significantly differ from those commonly employed (e.g. substantially stiffer or more flexible micropiles), or where there is potential for anchor loads to be transferred to the depth of sliding, t-z analyses should be performed to evaluate the magnitude of soil movement required to mobilize axial resistance and to account for the influence of the anchor loads.

- In all cases, the lateral or shear resistance of the micropiles should be estimated using p-y analyses to predict mobilization of shear resistance as a function of total soil movement with due consideration given to the magnitude of deformations that can be tolerated for a particular case. For these analyses, p-modifiers of approximately 0.2 should be applied to current empirical p-y models. The practice of computing the shear resistance for micropiles based on the ultimate bending (moment) capacity of the micropile sections should be avoided because soil movements required to mobilize such resistance is generally larger than can be practically tolerated in many cases and other failure modes may control the maximum shear resistance.

- The practice of considering only the component of micropile resistance parallel to the sliding surface should also be avoided because the component acting normal to the sliding surface can also have a substantial effect on stability; this effect can be either positive or negative depending on the specific conditions present.

- Given the importance of axial loads for stability and the fact that contributions from axial loads may be largely derived from changes in stress on the sliding surface, particular care should be paid to evaluation of shear strength parameters along the anticipated sliding surface or surfaces. Errors in estimates for the angle of internal friction can lead to over- or under-estimation of stability depending on the orientation of the piles.

- Particular attention should also be given to estimates for the ultimate side shear capacity of the micropile-soil (and potentially the grout-steel) interfaces for micropiles. This may be accomplished by acquiring quality measurements of shear strength parameters along the length of the piles, through field load testing, or some combination of both methods for establishing realistic side shear capacities for the micropiles. If field load tests are performed, consideration must be given to the drainage conditions in the soil since common field load tests are performed under conditions that are predominantly undrained, while actual field loading in slopes is more likely to be partially or fully drained.

- Use of ground anchors that are anchored in the capping beam should be done with caution since they may compromise the beneficial effects of the micropiles on stability. Ground anchors may still be justified on the basis of overall stability, but it is important that possible detrimental effects on upslope micropiles be properly considered in evaluating overall stability.
• Because of the uncertainty regarding mobilization of the kinematically induced component of axial loads and because these values are highly dependent on the thickness of the sliding zone, it is prudent, and generally conservative, to ignore this component of resistance until further evidence can be obtained to verify that they will be mobilized as analyses indicate.

Recommendations for Future Work

In this section, recommendations are provided to guide future work to enhance and more rigorously evaluate the proposed method of analysis. These recommendations include several analytical enhancements needed to make the proposed procedure more generally suitable for analysis of a broad range of applications and more efficient to allow designers to more effectively evaluate alternative stabilization schemes. Additional recommendations are provided to address the pressing need for more data to confirm results and conclusions presented in this report and to serve as the basis for improving the technique. Specific recommendations within each of these categories are provided in the following sections.

Recommendations for Enhancing Proposed Method

The analyses required to predict micropile resistance using the method recommended in this report are tedious and time consuming, even for relatively simple cases. Fortunately, the analyses are also amenable to computer automation because of their repetitive nature. Future efforts should be made to better automate the analyses to make them more efficient and effective for design purposes. In particular, the computer tools used to perform p-y and t-z analyses should be modified to provide capabilities for automatically producing results for a range of soil movements and for a range of depths of sliding. Present capabilities currently require separate analyses for each sliding depth and each value of soil movement, which is tedious and conducive to error. While such improvements will require concerted effort, they are relatively straightforward programming exercises that can be achieved with reasonable effort.

A more challenging, but perhaps more important need is to enhance the method to account for interaction of upslope and downslope micropiles through the capping beam, and the capability to account for loads induced by ground anchors founded through the capping beam. Addressing this need will require “coupled” analysis of axial and lateral load transfer and upslope and downslope piles in a single analysis as compared to the “uncoupled” analyses presented in this report. Fortunately, the general knowledge is present to implement such tools (in essence, what is needed is a “Group” program with capabilities for modeling soil movement), but additional effort is needed to produce and verify a reliable tool.

Finally, the issue of micropile resistance varying with the relative orientation of the sliding surface needs to be addressed. This issue is particularly important for performing thorough searches for the critical sliding surface since the inclination of the sliding surface relative to the micropiles will change as alternative sliding surfaces are evaluated. Present capabilities of commercially available slope stability analysis programs only allow for a single value of resistance to be input for a particular location. More rigorous evaluations require that the resistance be varied depending on the relative orientation of the sliding surface and micropile. Enhancements to existing slope stability analysis programs are thus needed to allow input of micropile resistance as a function of sliding surface inclination to account for the fact that micropile resistance depends on the sliding direction. As a stopgap measure, one can currently evaluate resistance for a range of sliding surface inclinations and select the least of the computed resistances as a conservative estimate; however this approach may lead to excessive conservatism in many cases and the former approach is strongly preferred if appropriate modifications to slope stability analysis programs are implemented.

Recommendations for Acquiring Additional Data

Acquisition of additional data to confirm or refute the conclusions and observations drawn from this project and to allow improvements to the method to be made (e.g. improved p-y and t-z models) poses a number of practical challenges. The primary challenge is the need to evaluate performance for a broad range of site conditions (e.g. soil type, slope geometry, cause of instability, etc.) and micropile characteristics (e.g. micropile size and stiffness, micropile inclination, capping beam and anchor characteristics, etc.). The challenge is enhanced by the need to acquire data to demonstrate
performance of micropile systems when subjected to relatively large soil movements to allow improvements to t-z and p-y models at larger deformations (of particular importance for confirming the mobilization of kinematically induced axial loads). Addressing these challenges requires comprehensive evaluations involving full-scale field instrumentation, dedicated full-scale axial and lateral load tests, and physical model tests to include large-scale 1-g physical models and centrifuge models. Each of these approaches has advantages and disadvantages in terms of applicability, schedule, and costs and combined use of the different techniques is likely to be most effective. Recommendations for each are provided here.

Instrumentation of full-scale field cases generally provides the most appropriate data for evaluating and enhancing design methods. Issues such as scale effects and loading methods are avoided and the behavior is that of actual field performance. However, full-scale field instrumentation also has limitations and challenges that include:

- Installation of field instrumentation and subsequent monitoring can be relatively expensive and time consuming, which tends to limit the number of cases that can be instrumented and therefore limit the number of alternative conditions that can be evaluated;
- Loading is generally uncontrolled, which poses problems for ensuring that instrumentation will remain functional for a sufficient period of time to acquire meaningful data and to ensure that instrumentation data is acquired at appropriate times;
- Field cases are generally designed to avoid failure so data at large deformations are generally only possible if unforeseen problems occur.

Because of these issues, full-scale field data alone will generally be insufficient to provide all of the data necessary to fully evaluate and enhance the design procedure. Nevertheless, all opportunities to acquire data on full-scale field performance should be taken advantage of to provide additional data for which to evaluate the proposed method. Cases where it may be possible to acquire data at large deformations, while unique, should be vigorously pursued.

In cases where full-scale field cases can be instrumented, the primary data that is needed to evaluate and improve the method are the total soil movement of the matrix soil and the mobilized axial loads and bending moments in the micropiles. However, supplementary data that enhances the ability to understand the load transfer and to develop improvements includes accurate estimates of stratigraphy, soil strength, construction sequence, and piezometric conditions throughout the monitoring period.

General recommendations for future instrumentation include:

- Soil deformations, micropile loads, and piezometric conditions should be acquired simultaneously (or continuously) to the extent possible so that conditions at various stages of monitoring can be consistently established;
- It is preferable to log the micropile holes themselves to establish stratigraphy at the micropiles instead of having to infer the stratigraphy from the closest site investigation borings;
- Inclinometers for measuring total soil movement should be placed as close to the location where instrumented micropiles intersect the anticipated sliding surface as possible;
- Both drained and undrained shear strength parameters should be determined for all strata intersected by the instrumented micropiles, preferably from specimens taken from locations near to the micropiles;
- Piezometric conditions need to be well understood throughout the monitoring period because of the importance of piezometric conditions to understanding the mobilization of resistance; piezometers should be installed near to the locations of instrumented piles and should be monitored frequently or continuously to ensure that fluctuations in piezometric conditions are accurately recorded.

Physical model testing consisting of large 1-g physical models and/or centrifuge models overcome several of the challenges of full-scale field instrumentation. Loading conditions can be controlled more precisely, which means that data can be acquired in a more timely fashion and challenges of maintaining instrumentation in the field for long periods are avoided. Scaled model testing is thus generally less expensive than full-scale field instrumentation. Scaled model testing does have
issues associated with scale effects (e.g. for stresses in 1-g models and representativeness in centrifuge models). However, the ability to perform relatively large numbers of tests cost effectively in relatively short time makes them highly useful for evaluating the large number of alternative conditions needed to develop a better understanding of load transfer in micropiles. Such testing should therefore be pursued with particular focus on evaluating load transfer under different soil conditions and different micropile configurations. Results of such tests will also be useful to help establish appropriate full-scale field tests.

Finally, full-scale axial and lateral load tests can also provide important data for improving t-z and p-y models and should be pursued whenever possible. Such tests also have the proper scale and are generally cost effective when compared to the cost of instrumenting and monitoring full-scale sites. However, it is difficult to reproduce appropriate loading conditions in such tests so considerable interpretation is generally required. Axial load tests are likely to be particularly useful because of the importance of axial load transfer on overall stability. However, it is critical that such tests be performed to mimic loading from moving soil as closely as possible. The issue of loading rate and the need to acquire data under fully drained loading conditions is crucial for such tests. Field axial load tests should also be instrumented along their length if possible to allow the distribution of axial load to be accurately established under various loads, which can dramatically improve the ability to establish appropriate t-z models for axial loading. Lateral load tests may also be somewhat useful for improving p-y models. However, it is commonly difficult to transfer lateral loads down to the depth of sliding in conventional lateral load tests unless sliding happens to be particularly shallow. If lateral load tests are performed, consideration must also be given to rate and method of loading in order to mimic drained loading from soil movement as closely as possible (something that is very difficult to do with conventional load tests).

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