

Effect of Coupling on A-Walls for Slope Stabilization

Final Report
August 2018

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Midwest Transportation Center
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16. Abstract <p>A-walls are retaining structures composed of regularly spaced deep foundation elements battered in opposing directions and connected through a grade beam to mitigate movements of a slope or embankment. Analysis of A-walls for slope stabilization applications is challenging because of complex interactions between deep foundation elements and moving soils. A previous method was successful in modeling A-walls with consideration of both lateral and axial load transfer, but interaction between upslope and downslope A-wall elements through the capping beam is neglected in the "uncoupled" analysis. To evaluate the effect of coupling, the research team analyzed slopes stabilized with A-walls using finite element models with upslope and downslope piles connected at the pile heads. Results of the analyses were compared to those of uncoupled lateral and axial analyses utilizing the <i>p-y</i> and <i>t-z</i> methods. Load transfer parameters for the analyses were calibrated to field measurements of load transfer in A-walls to demonstrate viability of the revised methodology. Results of the coupled analyses were then compared to results from uncoupled analyses to evaluate the effect of interaction between upslope and downslope piles.</p> <p>Coupled analyses produced bending moment and axial force profiles in reasonable agreement with measured values. Calibration of <i>p-y</i> and <i>t-z</i> curves to achieve predictions consistent with field measurements required significant softening of ultimate lateral and axial resistance values, but the softening was less than that required for calibration of uncoupled analyses. Modeling of interaction through the capping beam resulted in more reasonable calibrated values of lateral and axial soil resistance, better agreement with measured axial force profiles, and better agreement with measured bending moments and axial forces at shallow depths. The results indicate that interaction facilitated by the capping beam has a significant effect on development of forces within the A-wall elements. For deep sliding, reasonable predictions of pile resistance can be achieved using uncoupled models. However, for shallower sliding, the capping beam is likely more consequential and a coupled analysis is prudent. Coupled analysis is also beneficial for structural design of the capping beam.</p> <p>Both calibration case histories involved relatively deep sliding and relatively small values of total soil movement. Additional analyses with new datasets from A-wall applications for shallower slides and greater movement are recommended.</p>			
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**Final Report
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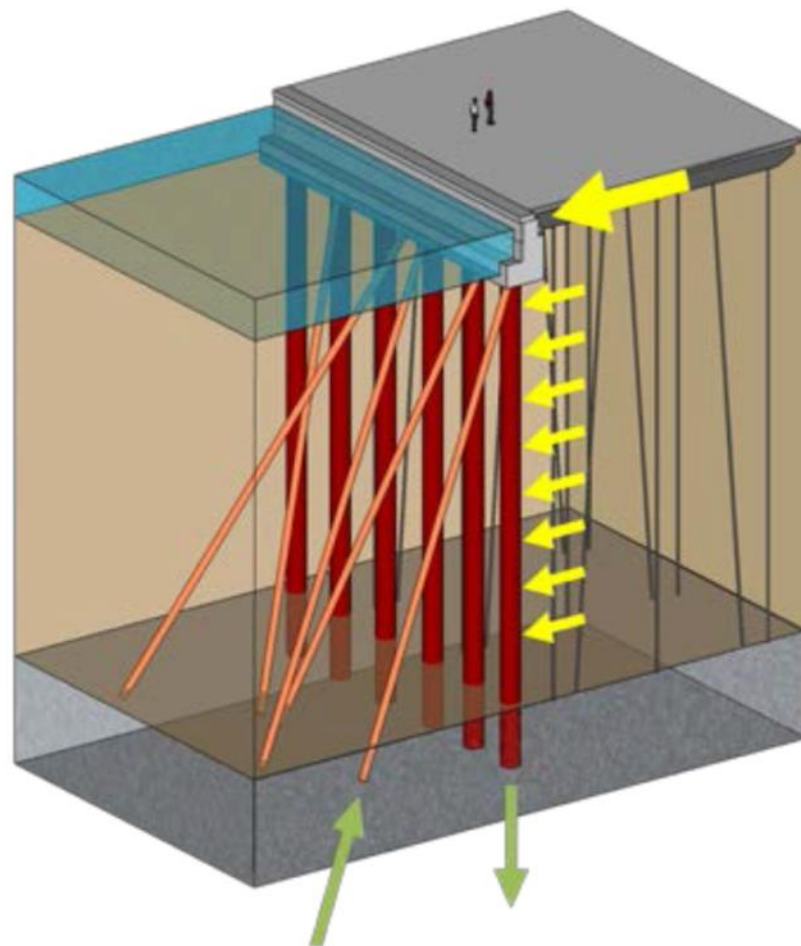
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INTRODUCTION

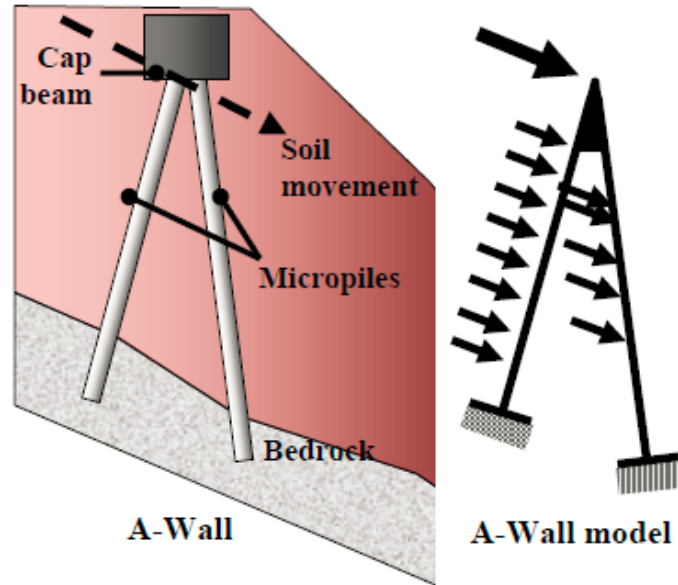
A-walls are retaining structures composed of at least two rows of regularly spaced deep foundation elements battered in opposing directions and connected through a grade beam to mitigate movements of a slope or embankment on soft soils. While A-walls are commonly constructed using micropiles, they can be constructed using any type of deep foundation element. For example, Gómez et al. (2013) described the use of a large A-wall for mitigation of lateral movements of the north plaza of the Jefferson Memorial in Washington, DC. The lateral movements accompanied settlement of the edge of the fill under the north plaza and had caused significant disturbance to the original seawall. The A-wall consisted of drilled shafts and driven piles extending to depths greater than 100 ft and connected through the new, replacement reinforced concrete seawall, as depicted in Figure 1.



Gómez et al. 2011

Figure 1. A-wall used for stabilization of lateral movements of the north plaza of the Jefferson Memorial in Washington, DC

A-walls have been successfully used for slope stabilization using schemes similar to that shown in Figure 2 (Gómez et al. 2013).



Gómez et al. 2013

Figure 2. Schematic representation of an A-wall for slope stabilization

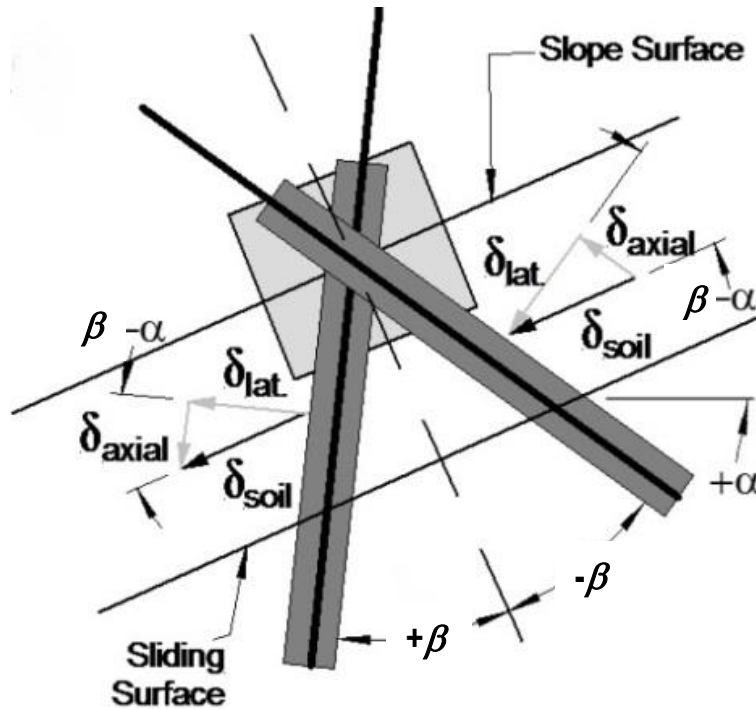
Loehr and Brown (2008) describe a method for predicting resisting forces in A-walls for slope stabilization based on measurements from full-scale field installations of A-walls and physical model tests involving scaled micropile elements. The method was a significant development because it appropriately accounts for the complex interaction between deep foundations and moving soils. Although the method satisfies displacement compatibility, it does so with uncoupled analyses involving separate lateral and axial analyses, without consideration of interaction between upslope and downslope piles (which are connected through a capping beam). This assumption may produce errors in predictions of reinforcement forces and could have a notable effect on the predicted performance of A-wall systems.

To evaluate the effect of coupling, the research team analyzed slopes stabilized with A-walls using a finite element model with upslope and downslope piles connected at the pile head. Results of the finite element analyses were compared to those of uncoupled lateral and axial analyses utilizing the p - y and t - z methods. Load transfer parameters for the analyses were calibrated to data from field installations of A-walls to demonstrate viability of the revised methodology. Results of the coupled analyses were then compared to results from Loehr and Brown (2008) to evaluate the effect of interaction between upslope and downslope piles. This report includes design implications resulting from the coupling effect and recommendations for further research.

BACKGROUND AND LITERATURE REVIEW

Overview of A-Walls for Slope Stabilization

Figure 3 schematically depicts a typical A-wall and the components of soil movement relative to the system of piles.



Boeckmann 2006

Figure 3. Components of retained soil movement in an A-wall

A-walls for slope stabilization consist of piles installed through a capping beam that runs along the length of the slope at the ground surface. Typically, inclination is alternated between successive piles, with half the piles inclined upslope (i.e., inclined into the direction of soil movement; $-\beta$ in Figure 3) and the adjacent piles inclined downslope (i.e., inclined away from the direction of soil movement; $+\beta$ in Figure 3). In some cases, ground anchors are also installed through the capping beam, although generally at greater spacing than the piles.

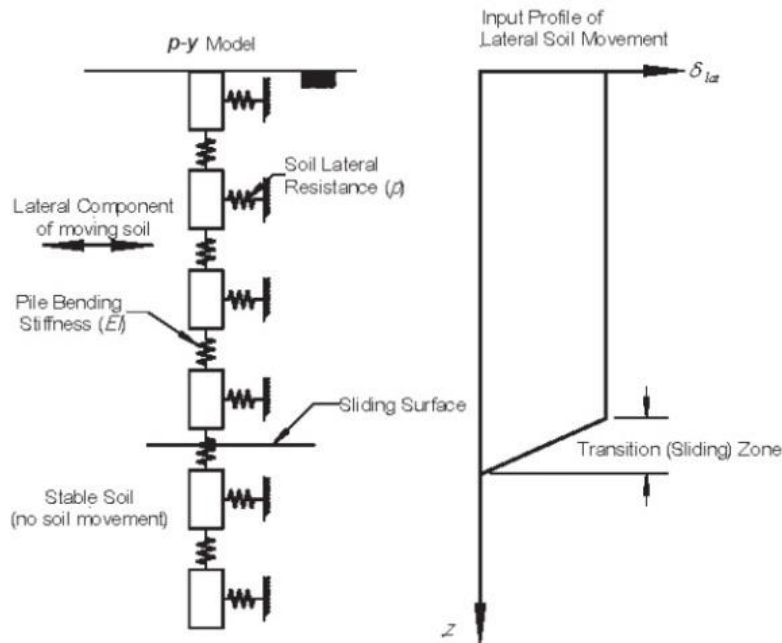
As the soil mass moves, load is transferred to the A-wall elements. The resulting forces within the piles provide resistance to movement of the ground mass. If the A-wall functions as intended, the forces will increase as the soil moves until the system reaches equilibrium and ground movement ceases. Design of an A-wall system is not trivial. Predicting pile forces is difficult and typically requires a considerable number of soil-structure interaction analyses for each potential sliding surface. Separate limit equilibrium slope stability analyses are completed using the predicted reinforcement forces. A-wall design must also include structural design of the

individual elements of the A-wall using the estimated axial forces, bending moments, and shear forces.

Loehr and Brown (2008) Method

In 2008, Loehr and Brown published a report documenting a procedure for analyzing micropiles used for slope stabilization to a joint committee of the Deep Foundations Institute (DFI) and the International Association of Foundation Drilling (ADSC). The procedure considers the uncoupled response of piles installed in an A-wall configuration and neglects the influence of the capping beam. The procedure involves first predicting load transfer to the piles via soil-structure interaction methods and then incorporating the reinforcement force into limit equilibrium slope stability analyses. The soil-structure interaction analyses consist of p - y analyses to determine lateral resistance and t - z analyses to predict axial resistance.

Loehr and Brown (2008) adopted a variation of the p - y method first presented by Isenhower (1999) in order to determine the lateral response of the pile to soil movement. A schematic of the approach is shown in Figure 4.

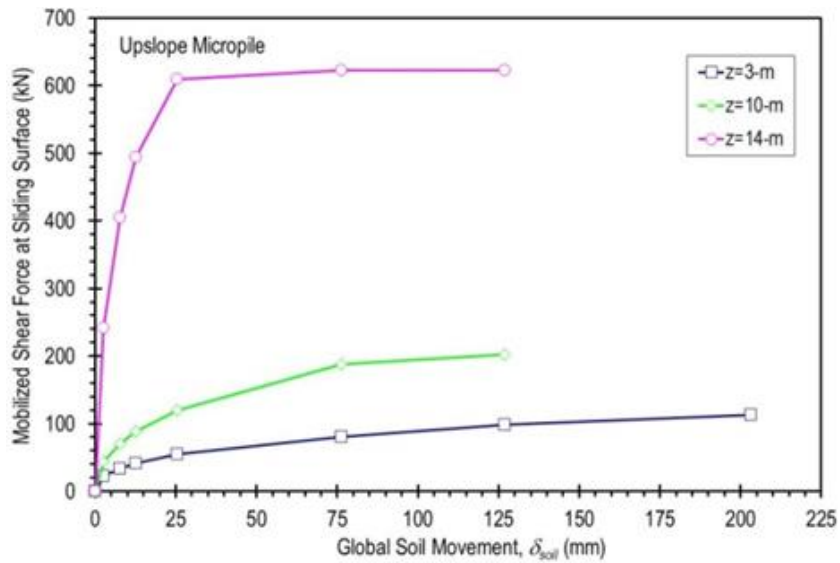


Loehr and Brown 2008

Figure 4. p - y model for deep foundations for slope stabilization

The lateral soil movement value (δ_{lat} in Figure 3) is applied to the pile at and above the depth of sliding. For simplicity, the magnitude of displacement is assumed uniform with depth except for a linear transition zone at the base of the sliding mass. Pertinent results from the p - y analysis include the shear force in the pile at the depth of sliding for subsequent slope stability analysis and the maximum shear force and maximum bending moment for subsequent structural design. The analysis is repeated for a range of soil movement magnitudes and different sliding depths to

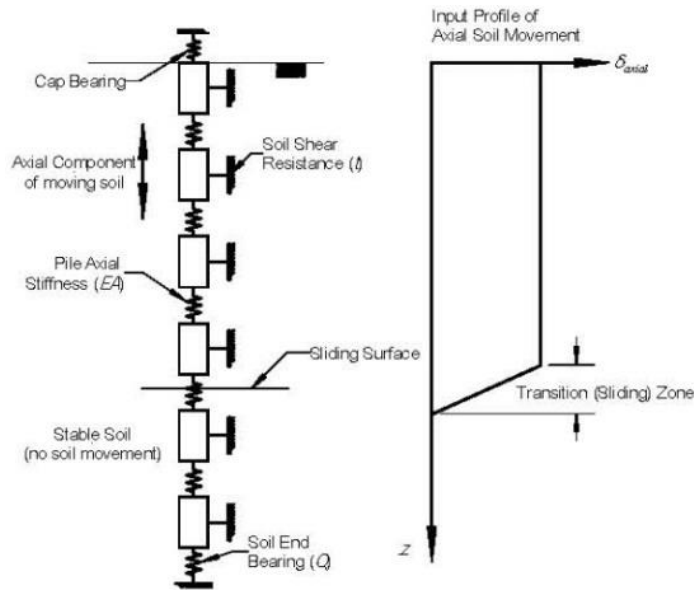
produce plots like the one shown in Figure 5.



Loehr and Brown 2008

Figure 5. Mobilized shear force at depth of sliding versus soil movement for three sliding depths

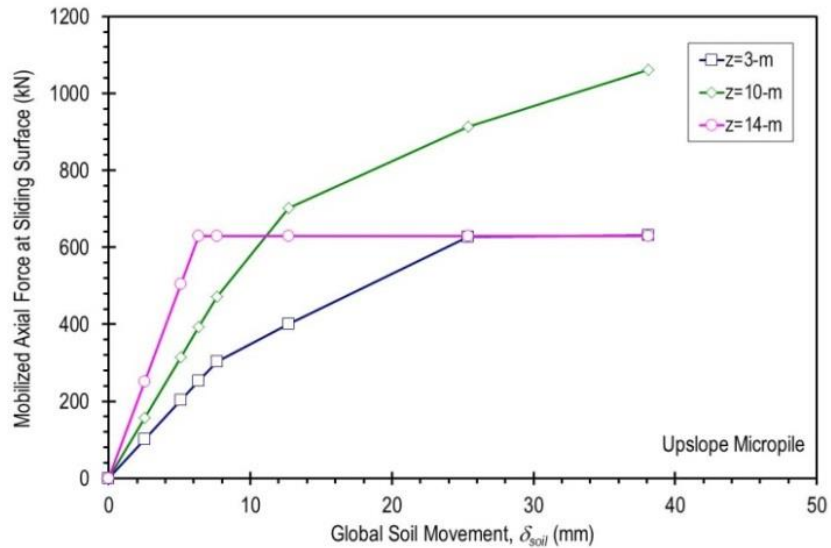
Loehr and Brown (2008) similarly adopted a variation of the t - z method in order to determine the axial response of the piles to soil movement. A schematic of the approach is shown in Figure 6.



Loehr and Brown 2008

Figure 6. t - z model for deep foundations for slope stabilization

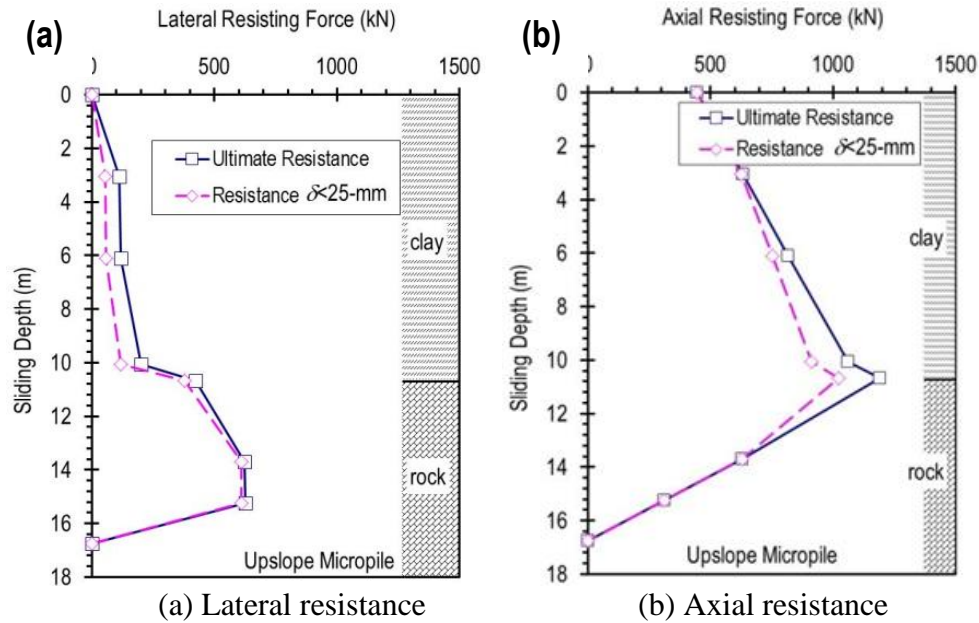
The axial soil movement value (δ_{axial} in Figure 3) is applied to the pile at and above the depth of sliding. Pertinent results from the t - z analysis include the axial force in the pile at the depth of sliding for subsequent slope stability analysis. The axial force at the depth of sliding is typically also the maximum axial force in the pile (for structural design). The analysis is repeated for a range of values of soil movement and for different sliding depths in order to produce plots like the one shown in Figure 7. Note that axial resistance is typically mobilized at significantly smaller displacement compared to lateral resistance.



Loehr and Brown 2008

Figure 7. Mobilized axial force at depth of sliding versus soil movement for three sliding depths

Results like those shown in Figures 5 and 7 are used to generate profiles of resisting forces with depth for use in limit equilibrium slope stability analysis. Example profiles are shown in Figure 8.



(a) Lateral resistance
 (b) Axial resistance
 Adapted from Loehr and Brown 2008
Figure 8. Example profiles of resisting forces with depth

Two sets of profiles should be generated: one for upslope piles and one for downslope piles. The reinforcement forces for each should be divided by the corresponding design pile spacing (i.e., the forces on upslope piles divided by their spacing) for input into the two-dimensional slope stability analysis. The deep foundation elements forming the A-wall must also be designed structurally to provide the required resistance.

Limitations of Current Methods

In the method developed by Loehr and Brown (2008), the upslope and downslope piles are uncoupled, i.e., the effect of the capping beam is neglected. This technical limitation may have significant effects for some applications of A-walls for slope stabilization. Use of the computer program GROUP rather than LPILE (both by Ensoft) would couple the piles and model the effect of the cap, but GROUP does not perform t - z analyses, and the soil movement analyses in GROUP require the same soil movement be applied to all piles. This limitation motivated development of the finite element models described subsequently.

Loehr and Brown (2008) also noted the approach described above can be tedious since it requires numerous p - y and t - z analyses for each sliding depth considered. The number of analyses can grow significantly if different sizes, depths, and inclinations of piles are to be considered. The number of analyses is a practical rather than technical limitation. Although making the procedure more efficient was not the explicit motivation for developing the finite element models, such modeling is more efficient since it combines p - y and t - z analyses. The finite element computations could be automated for further efficiency gains (e.g., to analyze multiple sliding depths or displacement values in one execution).

Effect of Pile Batter

Lateral soil resistance has consistently been observed to increase with the angle of pile inclination (Kubo 1965, Awoshika and Reese 1971). Iqbal (2015) back-calculated p -modifiers to account for batter (p_b) from physical model tests on micropiles in moving soil described by Bozok (2009). These multipliers are plotted versus batter angle (β), along with interpretations from tests on actively loaded piles described by Reese et al. (2006), in Figure 9.

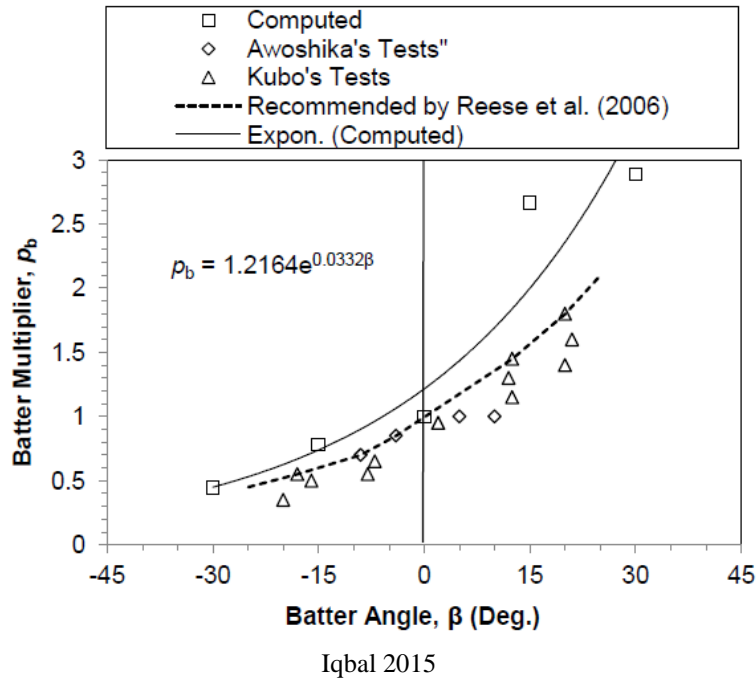


Figure 9. p -modifier, p_b , to account for effect of batter versus batter angle, β , where “computed” results were calculated from large-scale physical model tests

For piles in moving soil, the batter angle is negative for piles inclined upslope (i.e., “into” the moving soil) and positive for piles inclined downslope (i.e., “away” from the moving soil). For actively loaded piles, the batter angle is considered negative if the load is applied in the direction of batter and positive if the load is applied in the opposite direction. As shown in the figure, both Reese et al. (2006) and Iqbal (2015) observed p_b to increase exponentially as a function of batter angle.

NUMERICAL MODELING

The effect of coupling on A-walls for slope stabilization was evaluated using two-dimensional finite element models of A-walls including the capping beam. The models were created and analyzed using MATLAB Code and spreadsheet calculations. The finite element models were used to calibrate p - y and t - z models for instrumented case histories. Results of the calibrations were compared to results using the uncoupled method reported by Loehr and Brown (2008).

A-Wall Finite Element Model

The two-dimensional finite element models for analyzing A-walls consists of an upslope pile, a downslope pile, and a capping beam connecting the two piles, as shown in Figure 10.

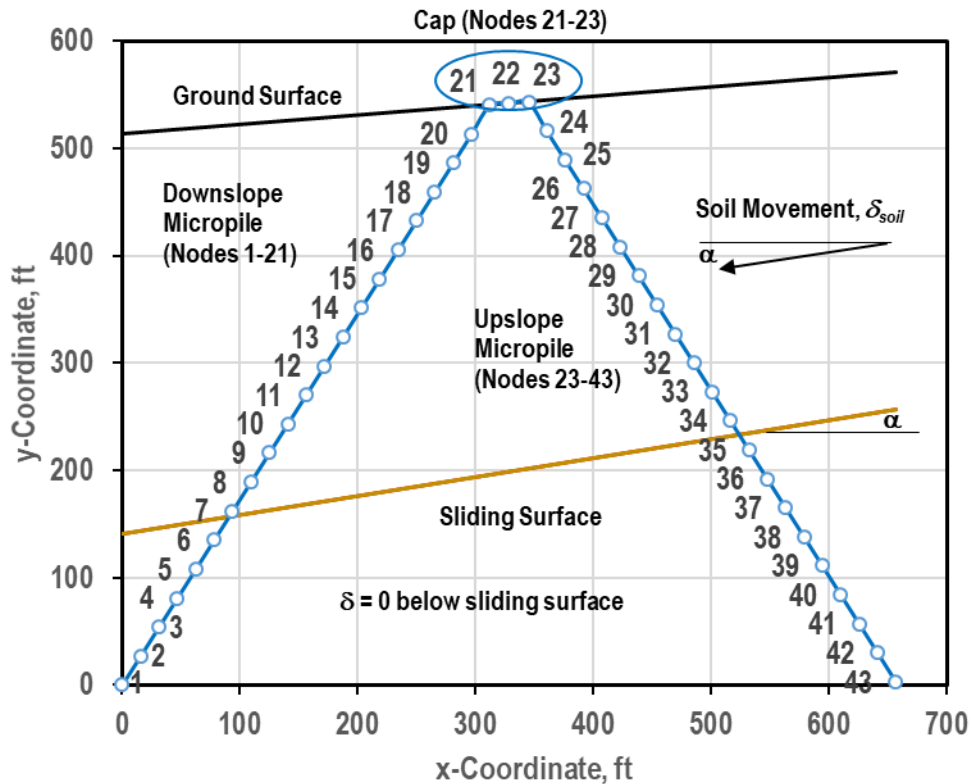


Figure 10. Schematic of A-wall finite element model

An equal number of elements were used to model the upslope and downslope piles. Two elements were used to model the cap.

The governing equation for the finite element model is as follows:

$$\{f\} = [K_{flexural} + K_{axial} + K_{py} + K_{tz}]\{u\} \quad (1)$$

where f is a vector of nodal forces, $K_{flexural}$ and K_{axial} are stiffness matrices representing the structural stiffness of the deep foundation elements, K_{py} and K_{tz} are stiffness matrices representing the lateral and axial response of the soil along the deep foundation elements (i.e., p - y and t - z springs, respectively), and u is a vector of nodal displacements. Flexural stiffness of the piles is calculated from EI , the product of Young's modulus and the pile moment of inertia. Axial stiffness of the piles is calculated from EA , the product of Young's modulus and the pile cross-sectional area. For the two capping beam elements, the value of Young's modulus was taken to be 1,000 times greater than that of the piles in order to model a stiff cap. This assumption is realistic because the cross-sectional dimensions of the capping beam are most often determined by geometrical considerations and typically result in a relatively stiff connection with a low value of demand-to-capacity ratio. In A-walls constructed with more than two rows of piles, or where the deep foundation elements are relatively stiff, this assumption may need to be revisited.

Forces on the A-wall, f , consist of externally applied forces, $f_{external}$, and forces imposed by the moving soil, $f_{soil\ movement}$:

$$f = f_{external} + f_{soil\ movement} \quad (2)$$

Forces imposed by moving soil are calculated from the p - y and t - z curves and a vector of relative soil movement values along the A-wall:

$$f_{soil\ movement} = [K_{py} + K_{tz}]\{u_{soil} - u_{pile}\} \quad (3)$$

where u_{soil} is a vector of soil movement values at each node and u_{pile} is a vector of nodal displacements for the A-wall. Calculations are performed by entering A-wall inputs into a spreadsheet that builds matrices with nodal inputs according to the definitions for equations (1) through (3). Inputs for the model are summarized in Table 1.

Table 1. Finite element model inputs

Type	Input	Notes
Geometry	Number of nodes	
	Upslope pile length	
	Upslope pile inclination	
	Downslope pile length	
	Downslope pile inclination	
	Ground surface inclination	Used to calculate depths, stresses
	Sliding surface depth	
	Sliding surface inclination	
	Depth of transition between layers 1 and 2	Often assumed equal to sliding surface depth
	Inclination of transition between layers 1 and 2	
Structural	Pile flexural stiffness, EI_{pile}	
	Pile axial stiffness, EA_{pile}	
	Capping beam flexural stiffness, EI_{cap}	Capping beam assumed to be 1,000 times stiffer than pile
	Capping beam axial stiffness, EA_{cap}	
	Diameter	Used for p - y , t - z curves
Soil	p - y parameters	Discussed in text
	t - z parameters	
	p -modifier	To adjust p -scale of p - y curve
	y -modifier	To adjust y -scale of p - y curve
	t -modifier	To adjust t -scale of t - z curve
Loading	z -modifier	To adjust z -scale of t - z curve
	Lateral load at cap	In direction of ground surface
	Axial load at cap	Perpendicular to ground surface
	Bending moment at cap	
	Soil movement (δ_{soil}) for upslope pile	Applied above sliding surface at inclination of sliding surface
	Soil movement (δ_{soil}) for downslope pile	

The input matrices are passed to MATLAB, which solves the system of equations for the finite element model. The finite element formulation used is a “small-strain” formulation that does not consider the “ p - δ ” effect.

Various forms of p - y and t - z curves can be used within the calculations. For the cases evaluated in this study, the “stiff clay without free water” p - y model commonly implemented with LPILE was used. The model is described in detail by Reese et al. (2006), as well as within LPILE’s technical documentation. For t - z curves, a scaled exponential model was implemented wherein the mobilized unit side resistance, t , is computed as follows:

$$t = t_{ult} \left(1 - \exp\left(-\frac{k_{tz}}{t_{ult}} * z\right) \right) \quad (4)$$

where t_{ult} is the ultimate unit side resistance (frequently denoted f_{s-ult}), k_{tz} is the initial tangent slope (stiffness) of the t - z curve, and z is the net axial displacement between the soil and pile (i.e., the axial component of $u_{soil} - u_{pile}$). The negative exponential form was implemented to avoid numerical instabilities associated with the elastic-perfectly plastic curve that is frequently applied for t - z curves, including by Loehr and Brown (2008) for uncoupled A-wall analyses. During the calibration procedure discussed below, the initial stiffness for the negative exponential curves was taken to be twice that implemented by Loehr and Brown (2008) to ensure both models were starting from similar t - z curves. This modification results in t - z curves that are reasonably similar for all displacements.

Solution of the finite element equations results in vectors of forces and displacements at each node of the A-wall. Output from the computations includes a graphical representation of the deflected shape of the A-wall and plots of bending moment, shear force, lateral displacement, lateral soil resistance, and axial force versus depth for both the upslope and downslope piles.

Calibration of Numerical Models

To evaluate the A-wall finite element models as well as the effect of coupling, two instrumented A-wall case histories were analyzed. The finite element models were calibrated by adjusting p - y and t - z curves until the computed A-wall responses reasonably matched measured bending moments and axial forces along the lengths of the piles. Both cases were analyzed by Loehr and Brown (2008) and contributed to the development of their uncoupled method for analysis of piles for slope stabilization. The cases are therefore a useful benchmark for evaluating the effect of coupling.

The calibration process was similar to that adopted by Loehr and Brown (2008). Calibration was achieved via p -modifiers and α values, which effectively scale the vertical (resistance) axis of the p - y and t - z curves, respectively. Note α is equivalent to the coefficient commonly used to define deep foundation side resistance in terms of undrained shear strength, where α equal to 1.0 represents “perfect” adhesion. However, in the context of the current calibrations, the p -modifiers and α values are simply convenient scaling factors used to back-calculate p - y and t - z models that produce mobilized axial forces and bending moments that are consistent with observations. The calibration results are non-unique; different combinations of p - y and t - z curves could likely be used to generate results that reasonably agree with the measured values. However, to identify meaningful results, the calibrations were constrained in several ways:

- Modifiers were not applied to the displacement axes (y and z), primarily because the magnitude of observed displacements for both cases was limited.
- The same p -modifiers and α values were applied for all values of soil movement.
- The same p -modifiers and α values were applied along the entire length of the piles.
- The same p -modifiers and α values were applied to both upslope and downslope piles. (In addition, the p - y curves were not modified to account for the effect of pile batter, a topic that is discussed in a subsequent section.)
- Models for both case histories were calibrated to the same set of p -modifiers and α values.

The constraints are a logical means to limit the number of possible solutions such that the resulting p -modifiers and α values are relevant for future applications. The use of one set of p -modifiers and α values is also an acknowledgment of limitations in available data for calibration.

Model parameters and results are presented for each case history under the headings below. For both cases, loading induced by the moving soil against the capping beam was neglected. This assumption is believed to have little effect on the computed results for these two cases because sliding is relatively deep. However, interaction between the capping beam and the moving soil may have more significant effects for shallower sliding and should be explicitly considered in such cases. A discussion of the calibration results, including comparison with results by Loehr and Brown (2008) for uncoupled analyses, is presented in the following section.

Case 1: Littleville, Alabama (Brown and Chancellor 1997)

The A-wall installed for the Littleville, Alabama, slide consisted of 6 in. nominal diameter micropiles reinforced with 4.5 in. outer diameter steel pipes with 0.3 in. wall thickness. Alternating micropiles were installed 30 degrees upslope and 30 degrees downslope from vertical, with micropiles in each row spaced at 33 in. center-to-center. The A-wall also included ground anchors, which were modeled for the coupled analyses by applying an external load to the capping beam that is consistent with the measured loads in the ground anchors. Stratigraphy and soil model parameters for the Littleville case history are summarized in Table 2.

Table 2. Model parameters for the Littleville, Alabama, case

Stratum	Thickness (ft)	Undrained Strength, s_u (lb/ft ²)	Load Transfer Parameters (prior to scaling p_{ult} and f_{s-ult})	
			p - y	t - z
Embankment Fill	30	2000	Stiff Clay Model: $\epsilon_{50} = 0.02$; $c = 14$ lb/in ²	$f_{s-ult} = 8.3$ kip/ft $z_{ult} = 0.03$ in. ⁽¹⁾
Native Weathered Shale		5250	Stiff Clay Model: $\epsilon_{50} = 0.02$; $c = 36.5$ lb/in ²	$f_{s-ult} = 8.3$ kip/ft $z_{ult} = 0.03$ in. ⁽¹⁾

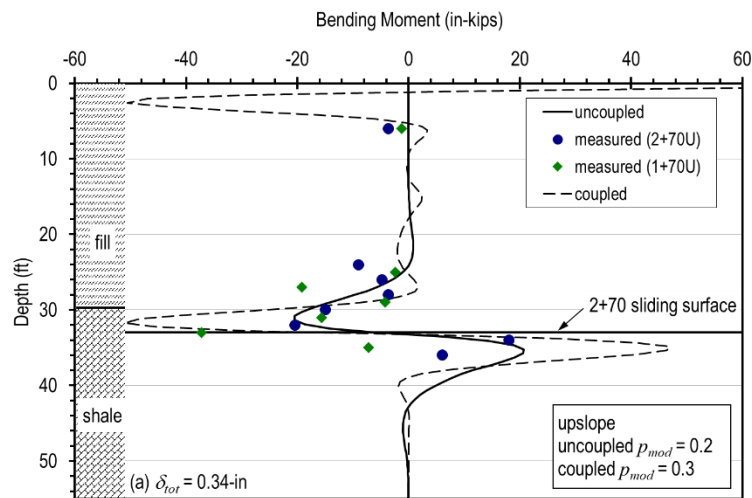
⁽¹⁾ Half the value assumed by Loehr and Brown (2008) to account for differences between their elastic-perfectly plastic t - z curves and the negative exponential curves used for the coupled models in this study.

Model structural inputs considered only the reinforcing pipe. The soil and structural model inputs are consistent with the uncoupled analyses by Loehr and Brown (2008), although loading from the ground anchors was not considered by Loehr and Brown (2008).

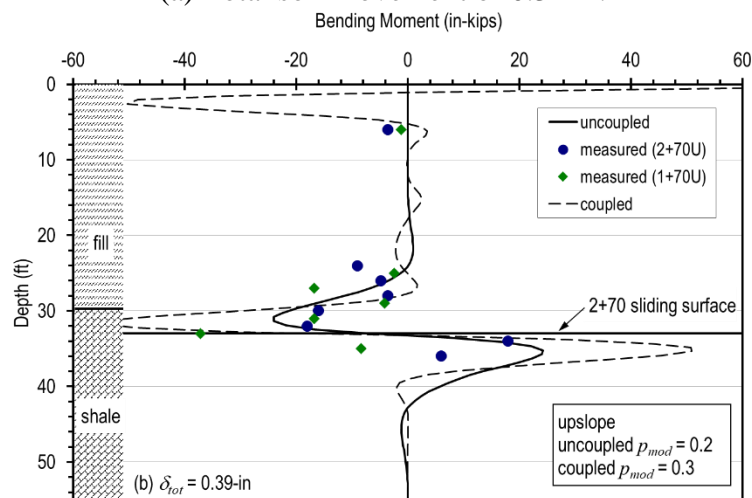
The Littleville case history by Brown and Chancellor (1997) included four instrumented micropiles, one upslope and one downslope at project stations 1+70 and 2+70, as well as two inclinometers at project station 2+70, one upslope and one downslope of the capping beam. The sliding surface for the Littleville case was near the top of the weathered shale, with the upslope inclinometer indicating the sliding surface at fill-shale contact and the downslope inclinometer

indicating the sliding surface just below the transition. The upslope inclinometer indicated total soil movement of 0.34 in. during an initial period of movement and an additional 0.05 in. of total soil movement (total 0.39 in.) during a subsequent period. For the same time periods, the downslope inclinometer indicated total soil movements of 0.24 in. and 0.31 in. Two calibration data sets were considered, one for each stage of movement. For the calibration, preference was given to the instrumented micropiles at project station 2+70 because this was the location of the inclinometers and because Brown and Chancellor (1997) questioned the results for the instrumented micropiles at project station 1+70.

Measured and calculated bending moment profiles are shown for upslope micropiles in Figure 11 and for downslope micropiles in Figure 12.



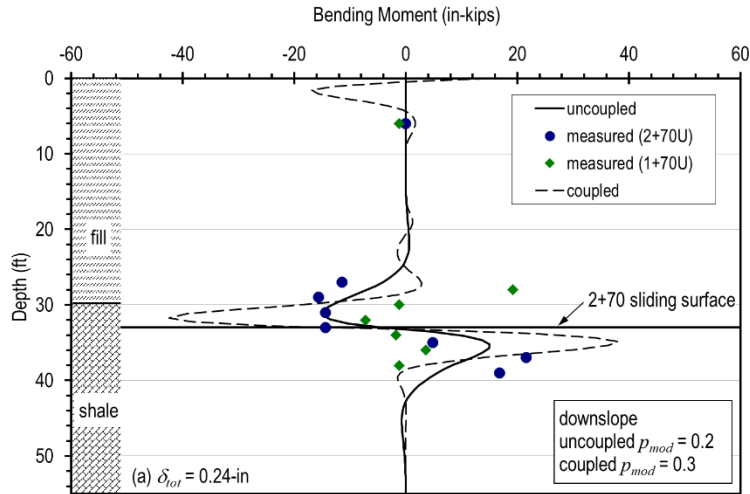
(a) Total soil movement of 0.34 in.



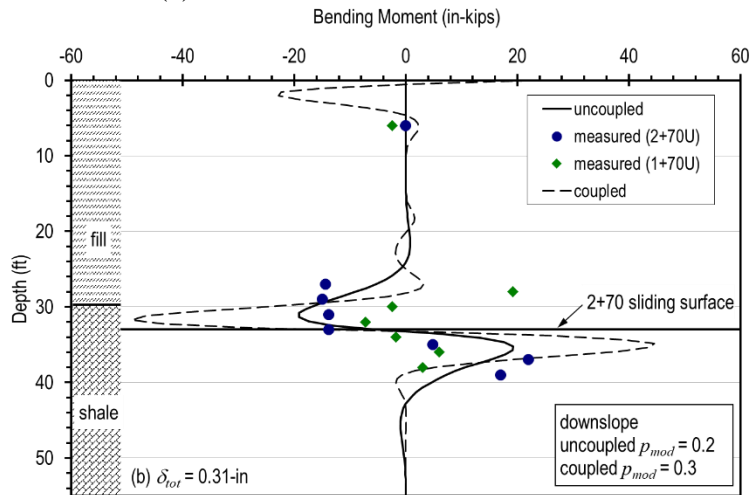
(b) Total soil movement of 0.39 in.

Updated from Loehr and Brown (2008)

Figure 11. Bending moment profiles for upslope micropiles in the Littleville, Alabama, case history



(a) Total soil movement of 0.24 in.



(b) Total soil movement of 0.31 in.

Updated from Loehr and Brown (2008)

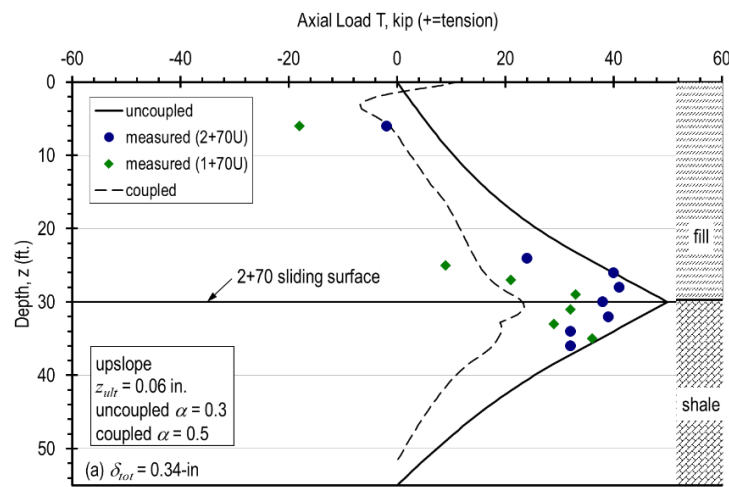
Figure 12. Bending moment profiles for downslope micropiles in the Littleville, Alabama, case history

The figures show calculated bending moments from the current study superimposed on figures originally presented in Loehr and Brown (2008). Each figure includes two plots, one for each stage of soil movement. Each plot includes the measured data points as well as two lines for calculated values, one from the uncoupled analysis by Loehr and Brown (2008) and one for the coupled analysis of this report.

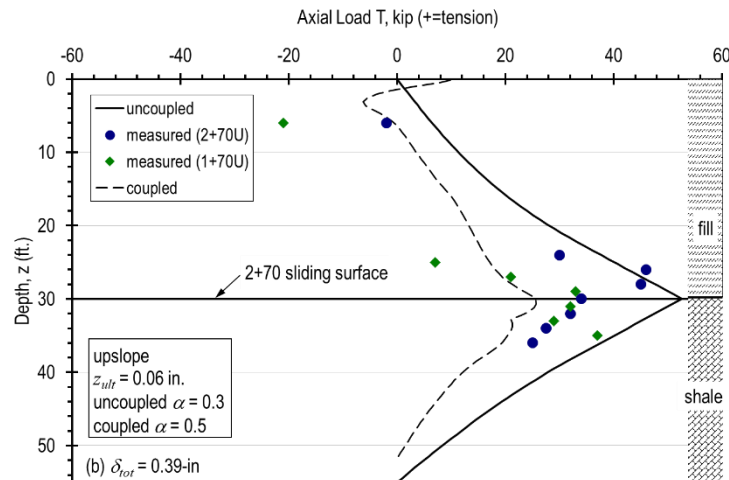
For both the upslope and downslope micropiles, the calculated bending moment profiles for both the uncoupled and coupled analyses indicate bending moments near zero along most of the micropile length but with sharp peaks to the maximum bending moment just above and just below the sliding surface. Bending moments near the sliding surface are greater for the coupled model compared to the uncoupled model because of the different p -modifiers used for the coupled and uncoupled analyses (recall that the p -modifiers used for the coupled models were

constrained to be the same for both case histories; thus, the p -modifier of 0.3 resulted in the closest fit to measurements from both case histories). Calculated bending moments for the coupled model also indicate substantial bending moments just below the capping beam as a result of the constraint provided by the capping beam and the applied anchor load. The uncoupled model produces zero moment at the ground surface because the capping beam and the anchor load were not considered. The magnitude of measured bending moments are reasonably consistent with calculated bending moments for both models.

Measured and calculated axial force profiles are shown for upslope micropiles in Figure 13 and for downslope micropiles in Figure 14. Both figures are also updates of figures originally presented in the Loehr and Brown (2008) report, and the presentation follows the convention used for bending moment profiles of Figures 11 and 12.



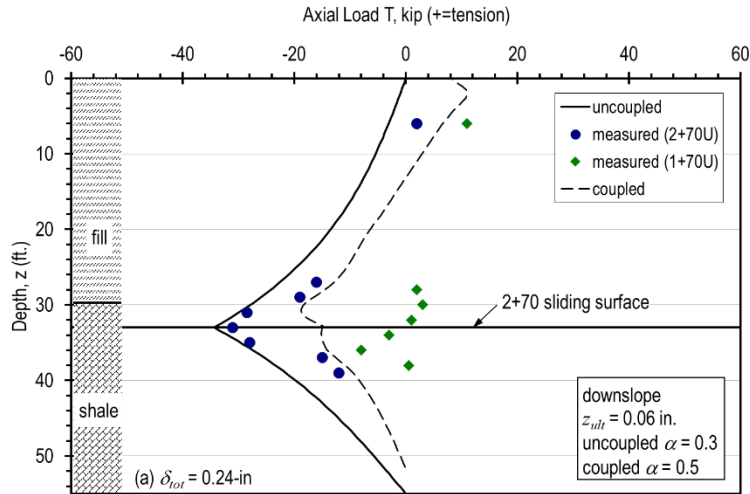
(a) Total soil movement of 0.34 in.



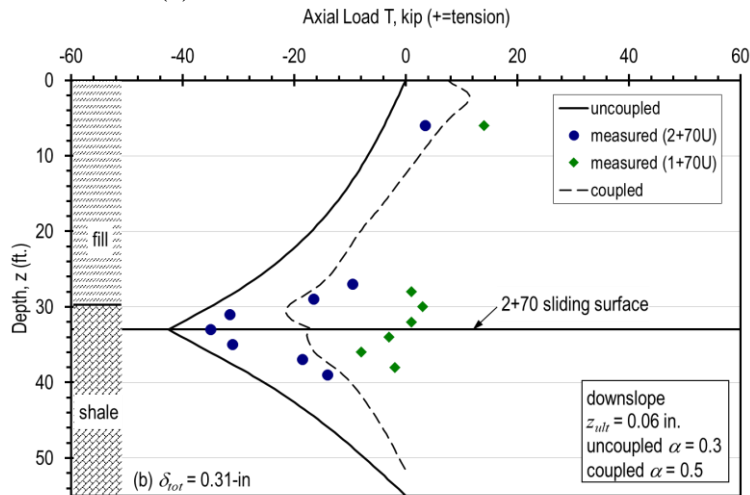
(b) Total soil movement of 0.39 in.

Updated from Loehr and Brown (2008)

Figure 13. Axial force profiles for upslope micropiles in the Littleville, Alabama, case history



(a) Total soil movement of 0.24 in.



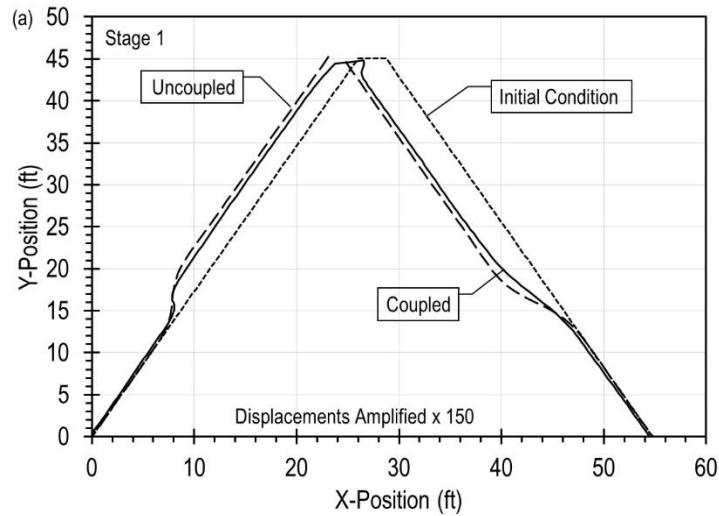
(b) Total soil movement of 0.31 in.

Updated from Loehr and Brown (2008)

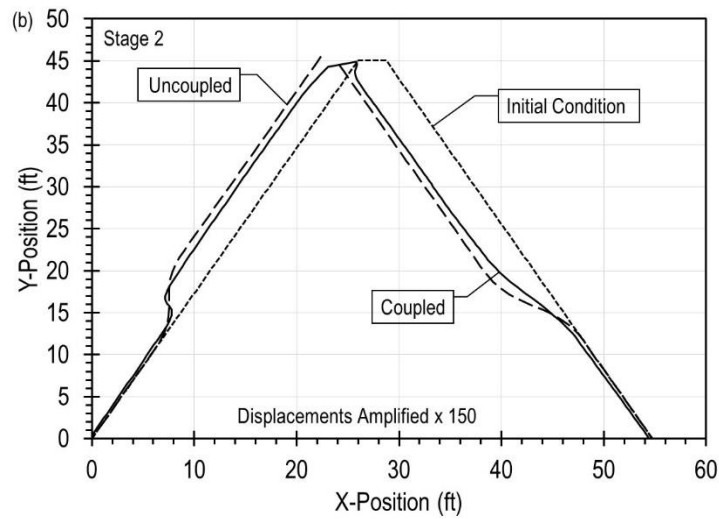
Figure 14. Axial force profiles for downslope micropiles at Littleville, Alabama, case history

Both the uncoupled and coupled models produce axial force profiles that increase in magnitude from either end of the micropile to a maximum value near the sliding surface. The coupled model produces compressive loads near the top of the upslope micropile and tensile load near the top of the downslope micropile, both of which are consistent with the measured axial load, consistent with the applied load due to the ground anchors, and consistent with the constraint provided by a stiff capping beam. In contrast, the uncoupled model enforces zero axial force at the micropile head and does not account for load due to the ground anchor. The measured axial forces near the sliding surface are generally bounded by the two sets of predictions, with the uncoupled model producing an upper bound of the measured axial forces and the coupled model producing a lower bound. Both models show reasonably good agreement with the measured data.

Figure 15 shows the calculated deformed shapes for the A-walls from coupled and uncoupled analyses.



(a) First movement stage



(b) Second movement stage

Figure 15. Deformed shapes of A-walls from coupled and uncoupled analyses for Littleville, Alabama, case

As shown in the figure, calculated deformations from the uncoupled analyses are slightly greater than those determined from coupled analyses. Additionally, deformations near the capping beam are substantially different for the coupled and uncoupled models as a result of the constraint provided by the capping beam for the coupled models.

Case 2: SUM 271 (Liang 2000)

The A-wall installed for the slide at the SUM 271 site in Summit County, Ohio, consisted of 8-in. nominal diameter micropiles reinforced with 5.5-in. outer diameter steel pipes with 0.3-in. wall thickness. Micropiles were spaced at 54 in. along each row, with alternating micropiles installed 30 degrees upslope and 30 degrees downslope from vertical. The A-wall also included ground anchors, which were modeled by applying an external load to the pile cap with a magnitude and direction consistent with the measured anchor load. Stratigraphy and soil model parameters for the SUM 271 case history are summarized in Table 3.

Table 3. Model parameters for the SUM 271 case

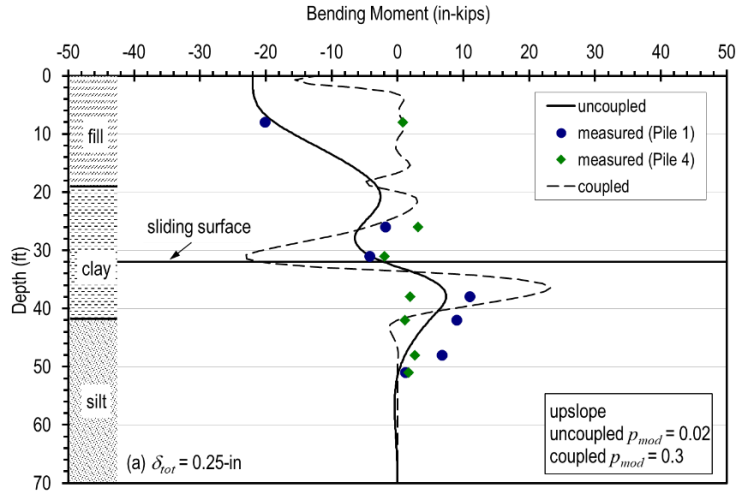
Stratum	Thickness (ft)	Undrained Strength, s_u (lb/ft ²)	Load Transfer Parameters (prior to scaling p_{ult} and f_{s-ult})	
			p - y	t - z
Embankment Fill	19	3000	Stiff Clay Model: $\epsilon_{50} = 0.02$; $c = 20$ lb/in ²	$f_{s-ult} = 6.3$ kip/ft $z_{ult} = 0.02$ in. ⁽¹⁾
Soft Silty Clay	23	1000	Stiff Clay Model: $\epsilon_{50} = 0.02$; $c = 6.9$ lb/in ²	$f_{s-ult} = 1.0$ kip/ft $z_{ult} = 0.02$ in. ⁽¹⁾
Dense Silt		5250	Stiff Clay Model: $\epsilon_{50} = 0.02$; $c = 31.3$ lb/in ²	$f_{s-ult} = 9.5$ kip/ft $z_{ult} = 0.02$ in. ⁽¹⁾

⁽¹⁾ Half the value assumed by Loehr and Brown (2008) to account for differences between their elastic-perfectly plastic t - z curves and the negative exponential curves used for the coupled models.

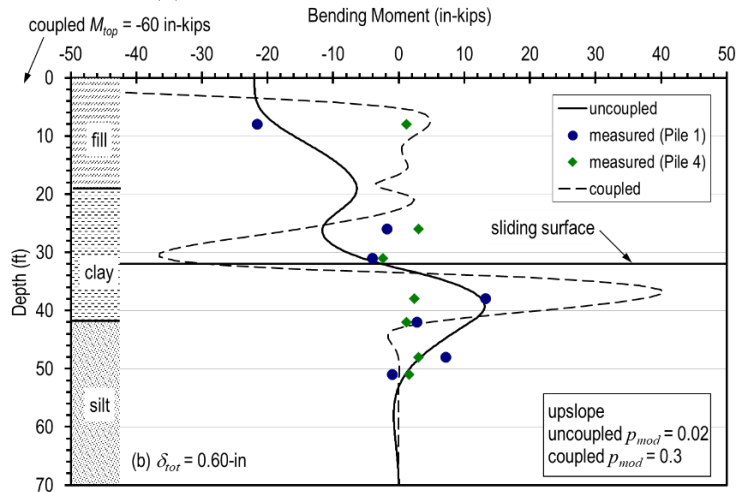
Model structural inputs considered only the reinforcing pipe. The soil and structural model inputs are consistent with the uncoupled analysis by Loehr and Brown (2008).

The SUM 271 case by Liang (2000) included four instrumented micropiles, two upslope and two downslope, as well as two inclinometers. Inclinometers were installed upslope of the A-wall, near the crest of the slope, which introduces uncertainty regarding the location of the sliding surface at the A-wall and regarding the total soil movement at each micropile. Inclinometers indicated the sliding surface was near the center of the soft silty clay layer. Inclinometers also indicated two stages of movement, with 0.25 in. of total soil movement during the first stage and an additional 0.35 in. (total 0.6 in.) during the second stage. Ground anchors were tensioned after the second stage of movement, so calibrations were performed for three stages: one for the first stage of movement, one for the second, and one for the second including the ground anchor force. For the calibration, evaluation of the upslope micropile response was primarily based on Pile 1 since measurements for Pile 4 were erratic.

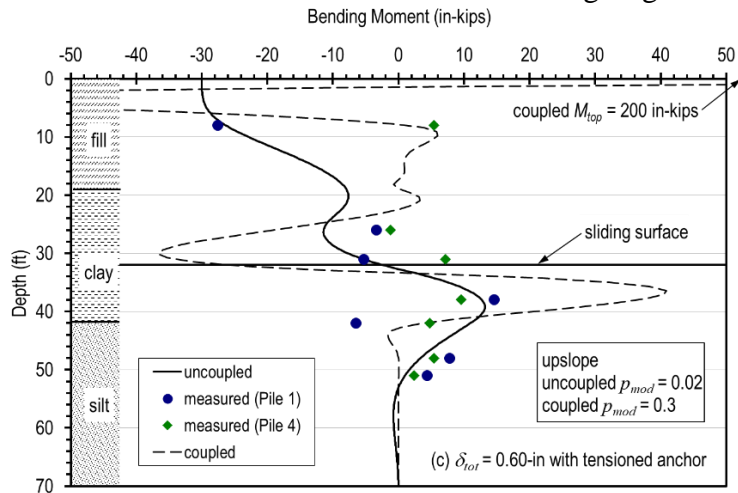
Measured and calculated bending moment profiles for the SUM 271 case are shown for upslope micropiles in Figure 16 and for downslope micropiles in Figure 17.



(a) Total soil movement of 0.25 in.



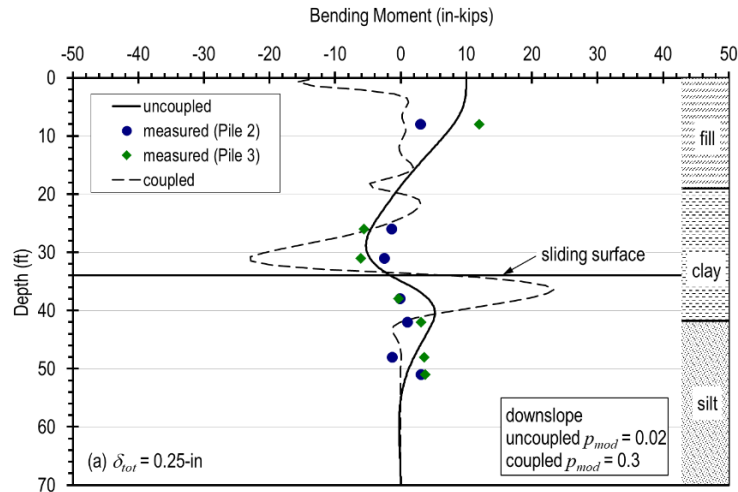
(b) Total soil movement of 0.60 in. before tensioning of ground anchor



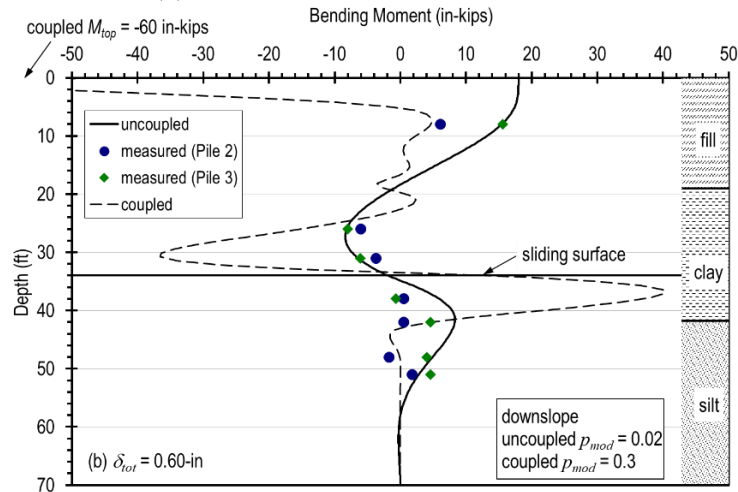
(c) Total soil movement of 0.60 in. after tensioning of ground anchor

Updated from Loehr and Brown (2008)

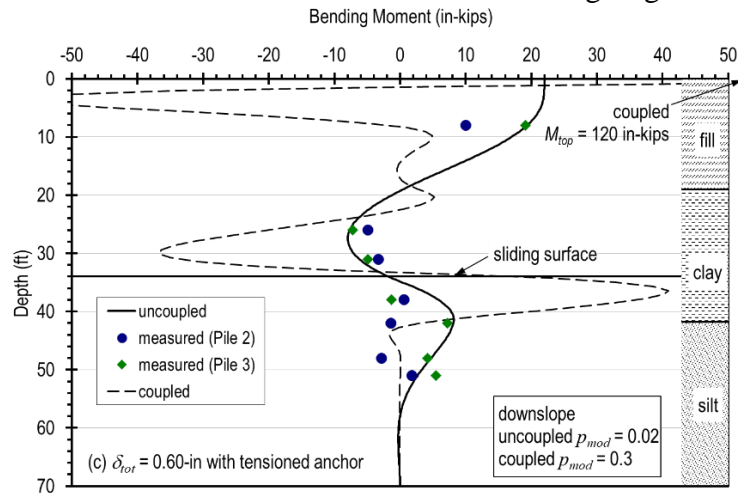
Figure 16. Bending moment profiles for upslope micropiles in SUM 271 case



(a) Total soil movement of 0.25 in.



(b) Total soil movement of 0.60 in. before tensioning of ground anchor



(c) Total soil movement of 0.60 in. after tensioning of ground anchor

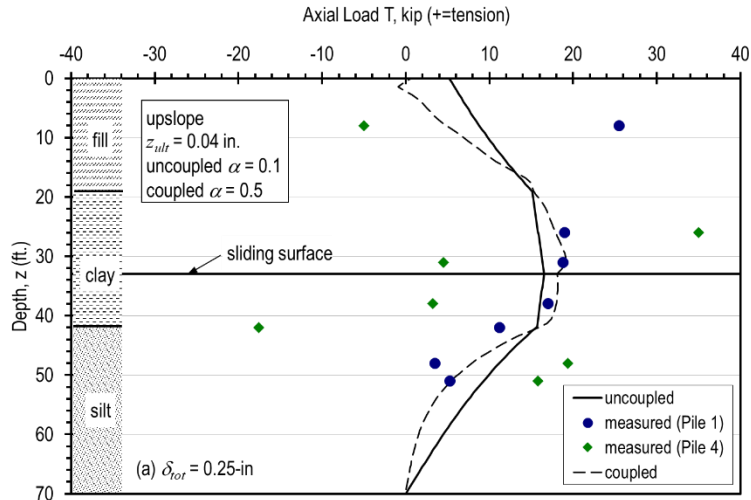
Updated from Loehr and Brown (2008)

Figure 17. Bending moment profiles for downslope micropiles at SUM 271 case history

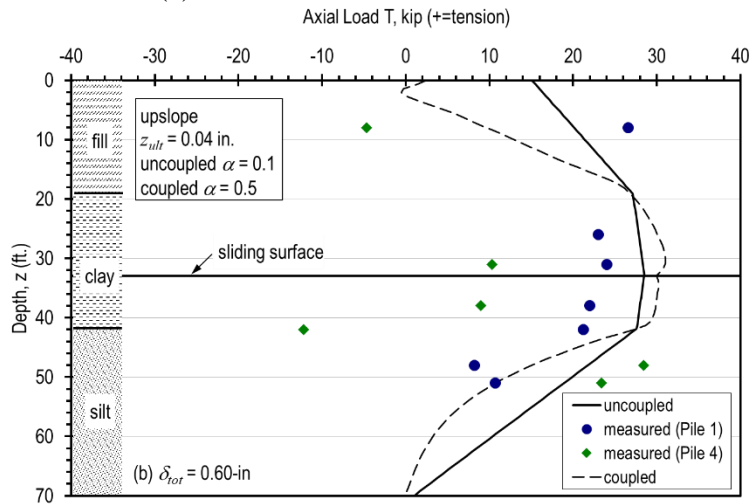
Both figures are adapted from figures originally presented in the Loehr and Brown (2008) report, and the presentation follows the convention used for the Littleville case history figures. The shape of the observed and calculated bending moment profiles are similar to those from the Littleville case, with sharp peaks just above and just below the sliding surface. However, for the SUM 271 case, large bending moments calculated near the capping beam are supported by observed moments in Pile 1. For the uncoupled model, a reasonable match between measured and calculated bending moments could not be obtained without assigning a bending moment boundary condition at the top of the micropile in order to match the observed moments. The moment boundary condition was imposed solely to match the observed data and could not reasonably be predicted in a normal design scenario. By comparison, the coupled model predicts the top-of-micropile moments “on its own,” by virtue of modeling the capping beam.

As observed for the Littleville case, measured bending moments are generally more supportive of the uncoupled model. However, it is important to note that the improved comparison between measured and calculated moments for the uncoupled model required modeling of an applied bending moment at the head of the pile, something that cannot be predicted at the design stage. For both the uncoupled and the coupled models, inclusion of the 110-kip anchor force at the capping beam had little effect on the overall shape of the bending moment profiles and on the magnitude of the bending moments at the sliding surface. This observation is consistent with measured bending moments, which did not change significantly near the sliding surface after the anchors were tensioned. For the coupled model, inclusion of the anchor force resulted in a large reversal of sign and increase in magnitude of the bending moment at the cap, a result that could be of significance for structural design of the capping beam.

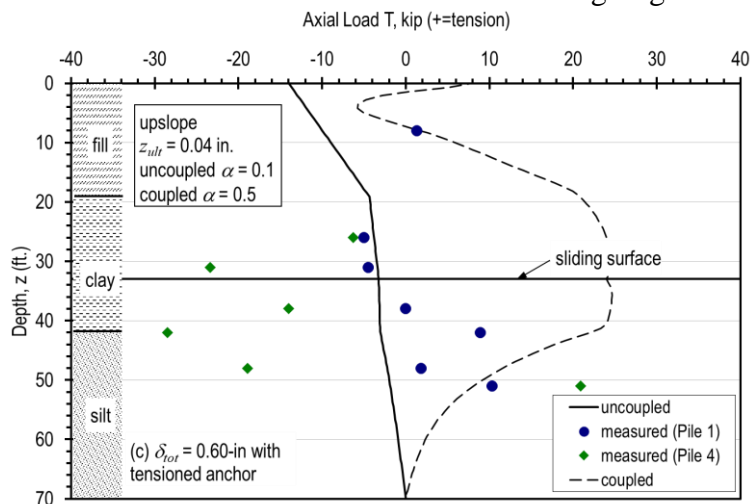
Measured and calculated axial profiles for the SUM 271 case are shown for upslope micropiles in Figure 18 and for downslope micropiles in Figure 19. The presentation follows the convention used for the figures presented previously.



(a) Total soil movement of 0.25 in.



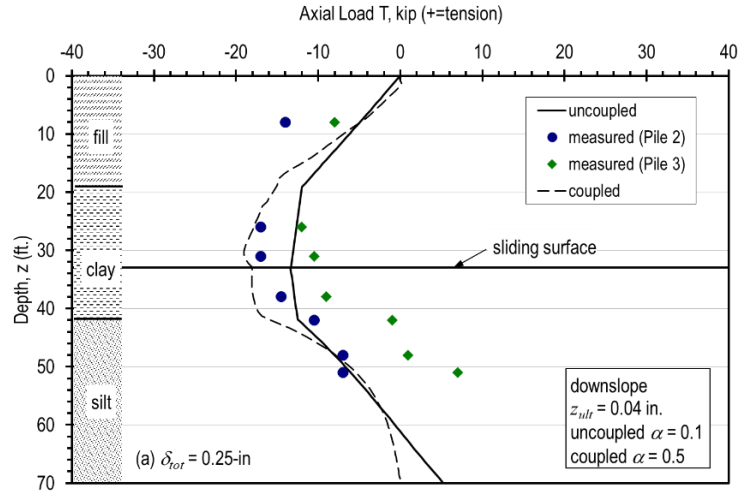
(b) Total soil movement of 0.60 in. before tensioning of ground anchor



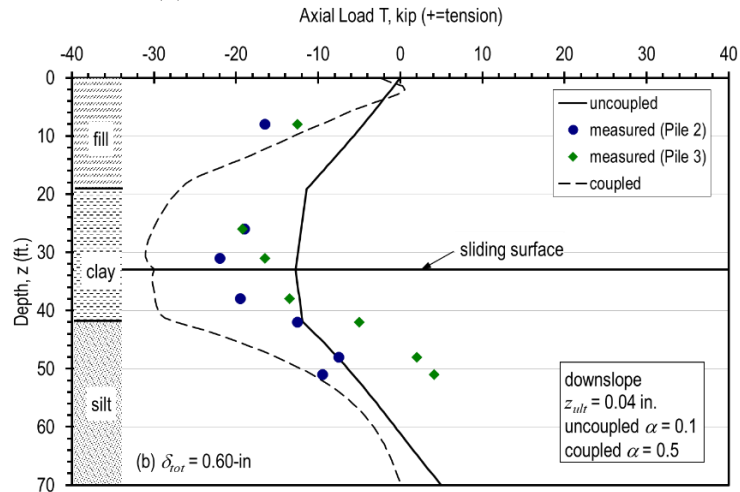
(c) Total soil movement of 0.60 in. after tensioning of ground anchor

Updated from Loehr and Brown (2008)

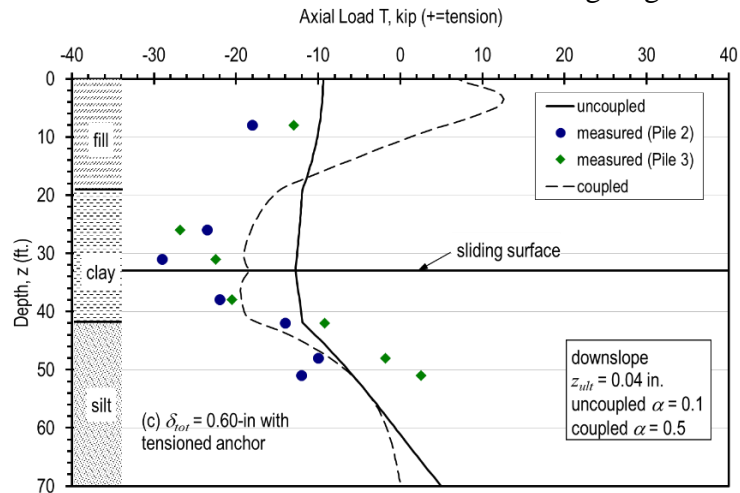
Figure 18. Axial force profiles for upslope micropiles at SUM 271 case history



(a) Total soil movement of 0.25 in.



(b) Total soil movement of 0.60 in. before tensioning of ground anchor



(c) Total soil movement of 0.60 in. after tensioning of ground anchor

Updated from Loehr and Brown (2008)

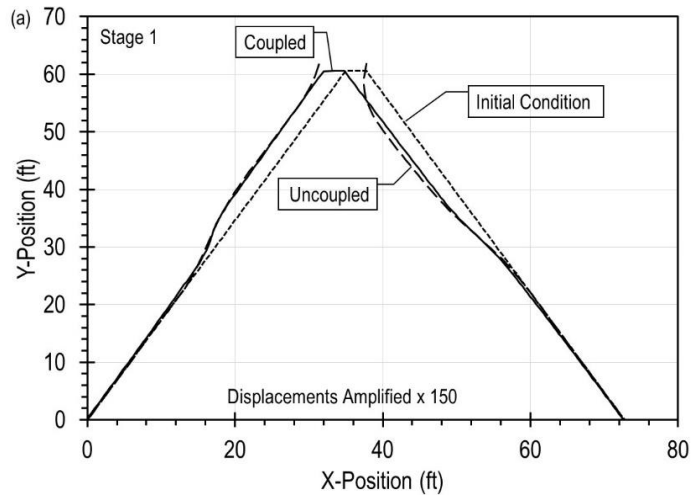
Figure 19. Axial force profiles for downslope micropiles for SUM 271 case

As with the Littleville case, calculated axial force profiles for both the uncoupled and coupled models indicate increasing axial load from either end of the micropile to a maximum value near the sliding surface. Tensile forces for the uncoupled model near the capping beam result from axial load boundary conditions applied to produce a reasonable match between calculated and measured axial loads; such boundary conditions would be difficult to predict in a design scenario. Interestingly, only small tensile forces are produced near the cap for the coupled models until the anchor load is applied, which calls into question the measured magnitudes of axial force near the cap. Also, as observed for the Littleville case, the predictions from both models are generally consistent with the measured data, particularly for Pile 1. Prior to including the anchor force, the two models predict axial force profiles that are quite similar, despite the coupled model's use of t - z curves with essentially five times greater resistance than those used for the uncoupled models.

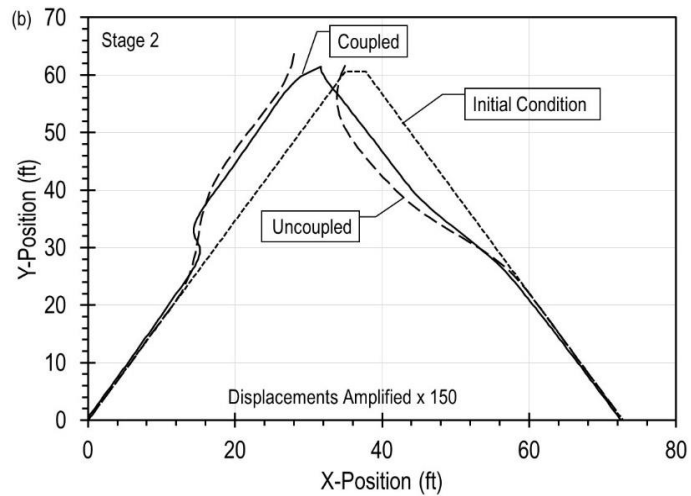
That the axial response is so similar for such different inputs suggests interaction facilitated by the capping beam is having a significant effect on development of forces within the micropiles. The t - z parameters employed for the coupled models are more consistent with those typically observed (and calculated) for actively loaded deep foundations, which suggests including the effect of the capping beam could result in predictions of axial load transfer that are more accurate than predictions that do not consider the effect of coupling.

For the uncoupled model, including the anchor force results in development of compressive forces in both upslope and downslope micropiles. The effect of including the anchor force on the coupled model is more modest, resulting in a slight shift toward compression in the upslope micropile and a slight shift toward tension in the downslope micropile. Measured axial forces near the capping beam in the upslope micropiles differ for the two instrumented piles but suggest development of significant tensile forces. Data observed for the downslope micropiles indicate a slight shift toward compression.

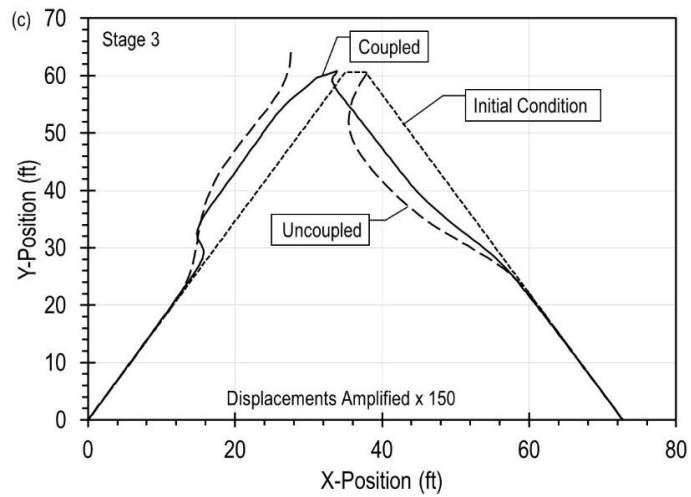
Figure 20 shows the calculated deformed shapes for the A-walls from coupled and uncoupled analyses.



(a) First movement stage



(b) Second movement stage



(c) Third movement stage

Figure 20. Deformed shapes of A-walls from coupled and uncoupled analyses for SUM 271 case

As observed for the Littleville, Alabama, case, calculated deformations from the uncoupled analyses are slightly greater than those determined from coupled analyses. Deformations near the capping beam are also substantially different for the coupled and uncoupled models as a result of the constraint provided by the capping beam for the coupled models. Pile deformations for the uncoupled models near the ground surface are also unusual and unrealistic because of the axial force and bending moment boundary conditions that were applied for the uncoupled models to produce a reasonable match between the measured and calculated axial force and bending moments.

Observations from the Calibrations

Calibration of the coupled finite element models resulted in several noteworthy observations regarding the performance of A-walls and the effect of coupling. Observations are presented under the general headings below.

Agreement with Measured Values

In general, the calculated A-wall response to soil movement for both case histories was consistent with the measured response. Calibration was achieved by applying a p -modifier of 0.3 and an α value of 0.5 for both cases, for the entire length of both upslope and downslope piles, and for all values of soil movement considered.

For both case histories, bending moments calculated by the coupled model were greater than measured values. However, the measured values are likely a lower bound for the true bending moments since the measurements are from strain gages installed at discrete points along the length of the pile and may not capture the true maximum bending moment. Small misalignment of the pile reinforcement during installation will also significantly reduce the interpreted moment since the actual distance between the gage and the neutral axis is less than the values assumed. Finally, in some cases the calculated moments exceed the yield moment of the micropile sections. Yielding of the micropiles would tend to limit the magnitude of the moment induced in the pile while also causing the moment to be “distributed” further above and below the sliding surface (something that is observed at both sites). The greater calculated moments may therefore be a result of both the coupled and uncoupled analyses considering only the elastic stiffness of the micropiles.

Axial force profiles calculated by the coupled model were generally within the range of measured values, with strong agreement overall. Axial force profiles are not subjected to the same degree of measurement difficulty as bending moments.

Comparison with Calibrations for Uncoupled Model

The agreement between calculated and measured values for the coupled models was generally comparable to that achieved by Loehr and Brown (2008) for their uncoupled analysis procedure. However, to achieve the level of agreement shown in the previous section, Loehr and Brown

(2008) applied different p -modifiers and different α values for the two case histories, whereas calibrations for the coupled models were achieved using one value for the p -modifier (0.3) and α (0.5) for both cases. The modifiers applied for the coupled model are more reasonable, particularly for the SUM 271 case. The uncoupled model required considerable reductions for the SUM 271 case: p -modifier of 0.02 and α of 0.1. The calibration parameters used for the coupled model are notably more realistic.

In addition, the coupled models had several other advantages compared with calibrations for uncoupled models. The coupled models generally resulted in better agreement for axial forces and better agreement near the capping beam, which suggests interaction facilitated by the capping beam has a significant effect on development of forces within the micropiles. For the SUM 271 case, Loehr and Brown's (2008) calibration with the uncoupled analysis required that bending moment and axial force boundary conditions be applied at the top of the piles in order to reasonably match the observed response above the sliding surface. The need for such boundary conditions would not be evident without prior knowledge of the measured data. The coupled analysis calculated bending moments similar to those observed above the sliding surface without requiring "artificial" manipulation of boundary conditions beyond simply modeling the cap.

Sensitivity to p -Modifier and α

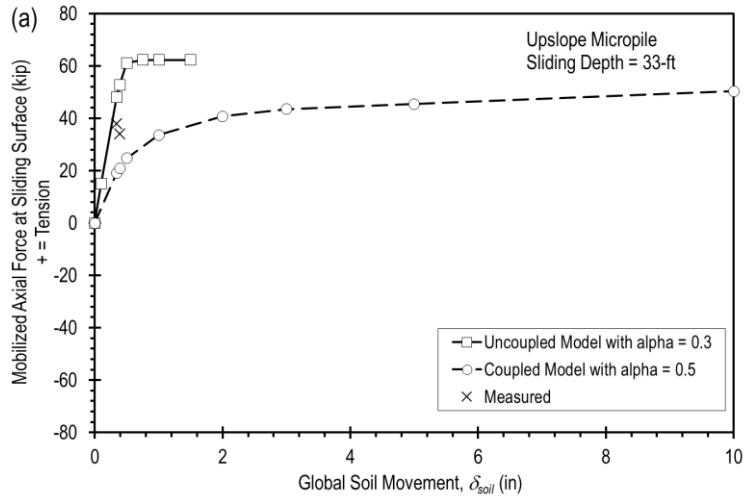
Relatively large changes in lateral resistance typically produced only modest changes in calculated bending moments. For example, for the Littleville downslope micropile at 0.4-in. movement, reducing the p -modifier from 0.3 to 0.2, a 33 percent decrease in ultimate lateral soil resistance, reduces the calculated maximum moment at the sliding surface from 45 to 39 kip-in., a 13 percent decrease. A similar, but less significant, trend was noted for changes in axial resistance. For the same case, reducing α from 0.5 to 0.4, a 20 percent decrease, reduced the calculated axial force at the sliding surface from 17 to 15 kips, a 12 percent decrease.

Changes in lateral resistance frequently produced changes in axial pile response as great as changes in the flexural pile response. For the same case discussed above, the reduction in p -modifier from 0.3 to 0.2 resulted in a reduction of the calculated axial force at the sliding surface from 17 to 15 kips. Although this is a modest decrease, it is equivalent to the effect produced by a similar reduction in axial resistance. This interaction between axial and lateral response cannot be predicted with uncoupled analyses. A similar, but less significant, trend was noted for the effect of changes in axial resistance on calculated bending moments.

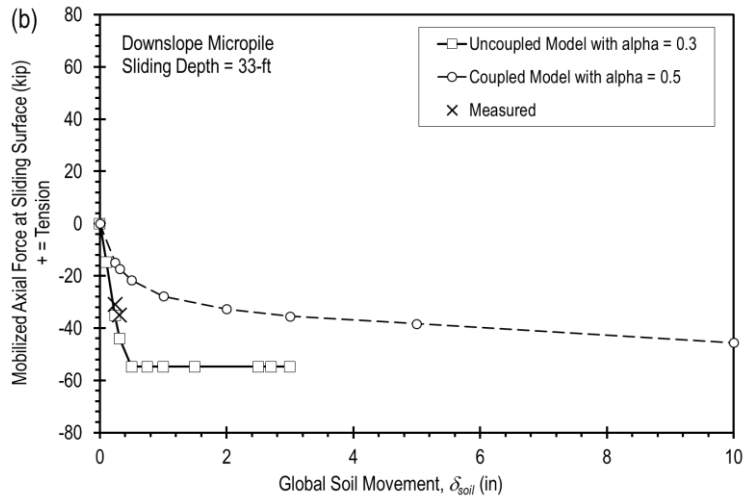
Predictions for Large Displacements

Observed slope movement values for both case histories were limited to less than 1 in. The finite element models were used to extrapolate the performance of the case history A-walls at greater displacement values in order to evaluate the effect of coupling at slope movement values up to and including the ultimate resistance. The results are presented alongside similar results for uncoupled models, which were originally presented by Loehr and Brown (2008).

Figures 21, 22, and 23 show the calculated response for the Littleville, Alabama, case.

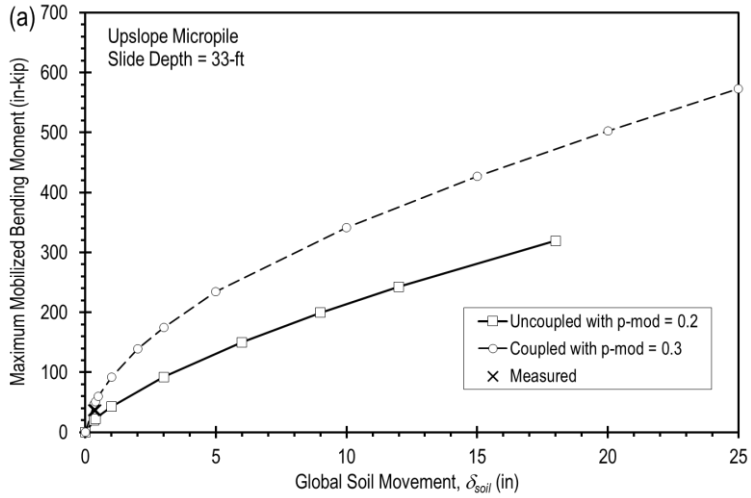


(a) Upslope micropile

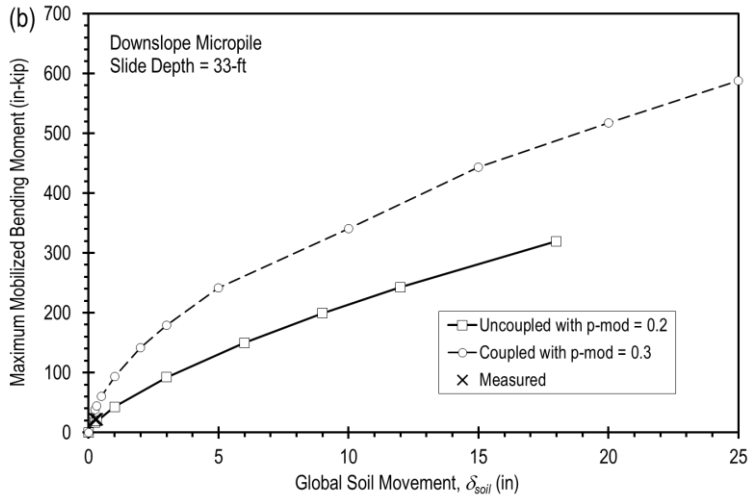


(b) Downslope micropile

Figure 21. Calculated mobilized axial force at the sliding surface versus global soil movement for Littleville, Alabama, case

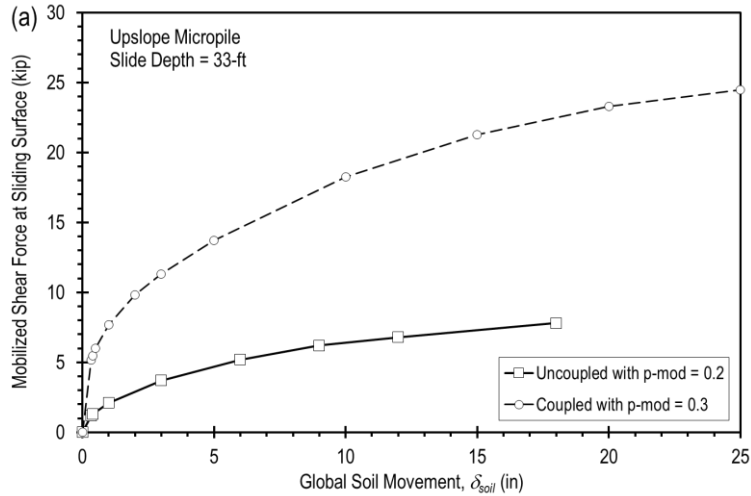


(a) Upslope micropile

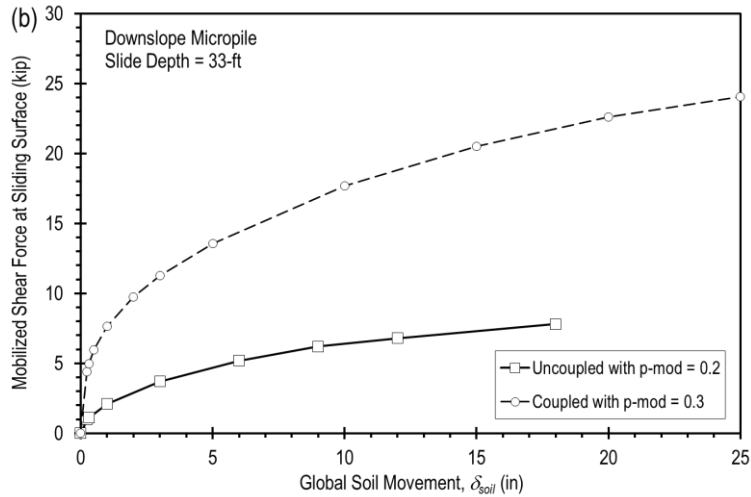


(b) Downslope micropile

Figure 22. Calculated maximum mobilized bending moment versus global soil movement for Littleville, Alabama, case



(a) Upslope micropile



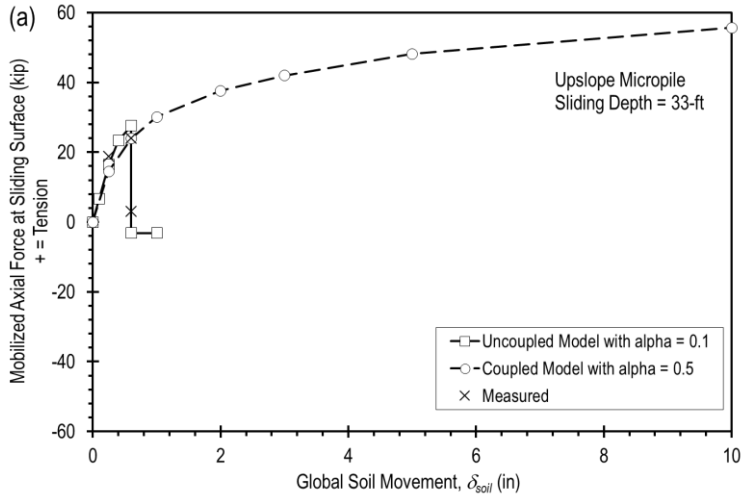
(b) Downslope micropile

Figure 23. Calculated mobilized shear force at the sliding surface versus global soil movement for Littleville, Alabama, case

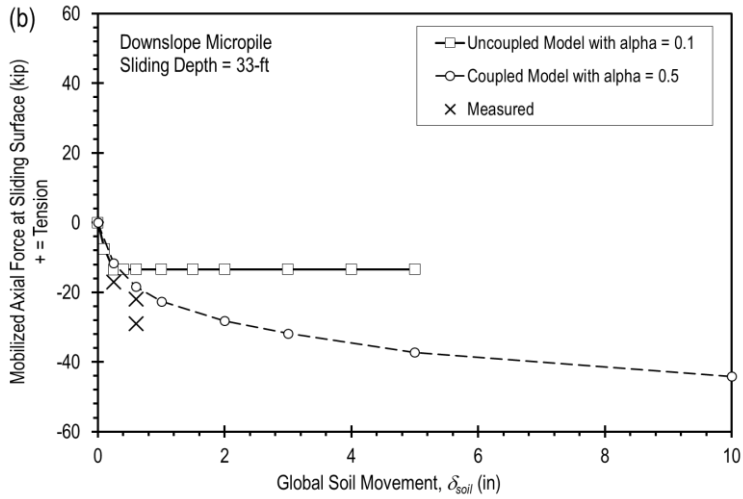
Mobilized axial forces at the sliding surface are shown in Figure 21. Calculations for both the uncoupled and coupled models suggest mobilization of the ultimate axial resistance at relatively small displacements, an observation consistent with previous studies (including Loehr and Brown [2008]). For both upslope and downslope piles, the coupled model predicts less overall axial resistance than the uncoupled model. This is a result of differences in the calibrations, rather than an effect of coupling; as discussed previously, the uncoupled model predicted axial forces near the upper bound of observed axial forces while the coupled model predicted axial forces near the lower bound. Whereas the uncoupled model predicts an abrupt realization of the maximum axial force, the coupled model predicts axial forces that increase slightly for large displacements. This effect is primarily the result of continued interaction between the upslope and downslope micropiles through the capping beam, whereby the axial and lateral load from one pile is partially transferred to the other, rather than an artifact of the smoothly curving $t-z$ curves used for the coupled models.

The maximum mobilized bending moment for the Littleville case is shown in Figure 22, and mobilized shear force at the sliding surface is shown in Figure 23. Both figures indicate significantly greater displacements are required to mobilize the maximum lateral resistance compared to axial resistance. Both the uncoupled and coupled models predict greater than 12 in. of global soil movement is necessary to fully mobilize the maximum shear force. The lateral resistance calculated by the coupled model is significantly greater than that calculated by the uncoupled model. The difference is primarily due to differences in calibration; as discussed previously, calculated moments for the coupled model were greater than observed values (and values calculated for the uncoupled model). However, it is interesting to note that the initial slope of the mobilized shear force versus soil displacement curve is substantially greater for the coupled model. Calibration differences likely contribute to the earlier mobilization of shear forces, but it is also likely the capping beam is providing a beneficial effect. Finally, it is worth noting that the coupled model predicts the moment capacity of the steel pipe is fully mobilized when the global soil movement exceeds 12 in.; for this case, serviceability constraints would tend to control design more than structural or geotechnical limit states.

Predictions for the SUM 271 case at larger displacements are shown in Figure 24 (axial force), Figure 25 (bending moment), and Figure 26 (shear force).

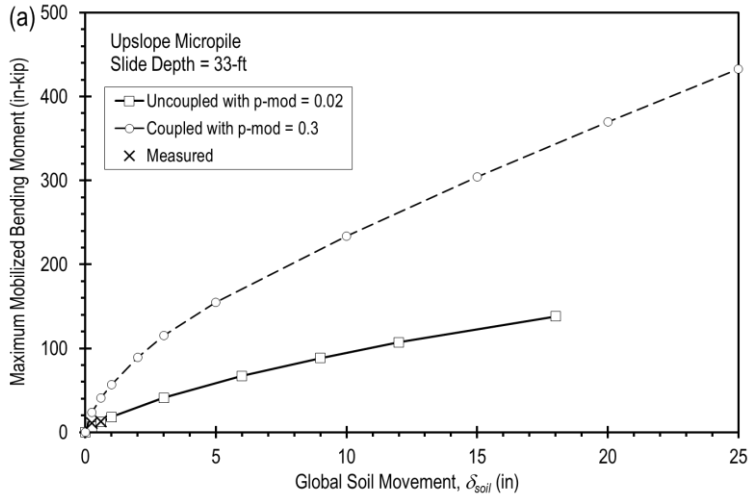


(a) Upslope micropile

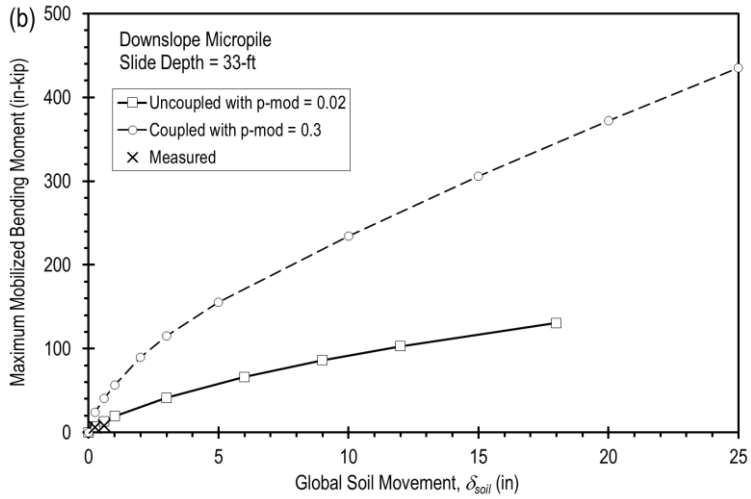


(b) Downslope micropile

Figure 24. Calculated mobilized axial force at the sliding surface versus global soil movement for SUM 271 case

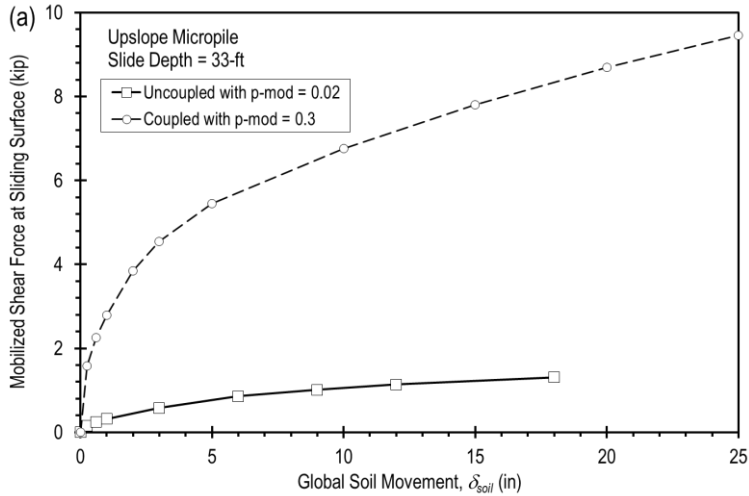


(a) Upslope micropile

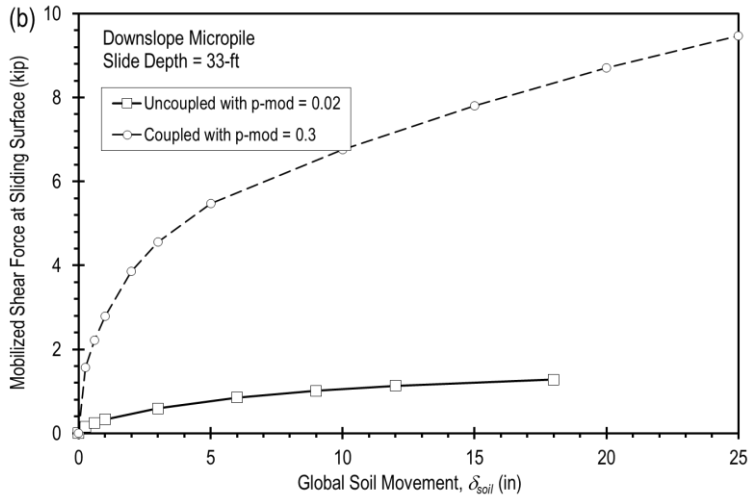


(b) Downslope micropile

Figure 25. Calculated maximum mobilized bending moment versus global soil movement for SUM 271 case



(a) Upslope micropile



(b) Downslope micropile

Figure 26. Calculated mobilized shear force at the sliding surface versus global soil movement for SUM 271 case

Observations from the SUM 271 analyses at larger displacements are generally similar to those noted for the Littleville case. As was observed for the Littleville case, the results of Figure 24 indicate gradual increases in axial force for the coupled model at displacements greater than the peak observed for the uncoupled model. Figures 25 and 26 indicate the coupled model predicts development of greater bending moments and greater shear forces than those calculated by the uncoupled model, but as with the Littleville case, these differences are likely mostly a product of differences in calibration parameters. The initial slope of the mobilized shear force versus displacement curve for the coupled model is significantly greater than that of the uncoupled model (Figure 25).

Effect of Pile Batter

Initial attempts to calibrate the coupled finite element models included p -modifiers to account for the effect of pile batter according to the relationship plotted by Reese et al. (2006) and discussed previously in this report. (The pile batter p -modifiers were multiplied by the calibration p -modifiers.) Reasonable agreement could not be achieved when applying the batter p -modifiers, which consistently resulted in predictions of axial forces that were too great for downslope piles and too small for upslope piles. The data in Figure 9 are all from battered piles that are either actively loaded (in which case load transfer is necessarily near the ground surface) or loaded by soil movement from shallow slides in pilot-scale physical models. The calibration cases considered in this report both involve sliding surfaces that extend to a depth of 30 ft. These observations suggest that batter effects on p - y models diminish with depth and should not be applied for sliding surfaces that are relatively deep.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary

Previous work by Loehr and Brown (2008) established a procedure for analyzing piles in slope stabilization applications. Loehr and Brown (2008) demonstrated the procedure could be used to predict pile bending moments and axial forces measured in field applications of A-walls, although the predictions were made using lateral and axial resistances significantly lower than would be predicted from soil strength values using most conventional techniques. The procedure developed by Loehr and Brown (2008) consider upslope and downslope piles separately (an “uncoupled” analysis), neglecting any effect from installing the piles through the A-wall capping beam. Although it can be used to model pile groups subjected to soil movement that is primarily horizontal, the widely-used software GROUP is inadequate for A-wall analysis since it does not incorporate t - z analysis for axial load transfer and since it applies the same value of lateral soil movement to all piles in the group.

Research described in this report involved analyzing A-walls for slope stabilization applications using finite element models that include a capping beam. Finite element models were used to analyze the same two test cases considered by Loehr and Brown (2008). Calibration of the test case models was achieved using p -modifiers and α values to scale the lateral and axial resistances, respectively. The calibration was constrained by applying one common p -modifier and one common α value for both test cases, for both upslope and downslope piles, and for all values of slope displacement considered. The calibration with a p -modifier of 0.3 and α of 0.5 resulted in calculated bending moments and axial forces that reasonably agreed with measured values.

The agreement between calculated and measured values for the coupled models was generally comparable to that achieved by Loehr and Brown (2008) for their uncoupled analysis procedure. This should be considered an improvement over the uncoupled models since Loehr and Brown (2008) applied different p -modifiers and α values for the two different case histories, including considerable reductions for the SUM 271 case: p -modifier of 0.02 and α of 0.1. For both case histories, bending moments calculated by the coupled model were notably greater than measured values. However, measured bending moments likely underestimate the maximum bending moment since strain gages may not be located precisely at the location of the maximum bending moment and since minor misalignment of instrumented micropile pipes can significantly reduce the interpreted moments. Axial forces calculated by the coupled model were generally within the range of measured values, whereas the uncoupled model tended to overpredict axial forces.

The coupled model also produced better agreement near the capping beam. For the SUM 271 case, Loehr and Brown’s (2008) calibration with uncoupled analyses required that axial force and bending moment boundary conditions be applied at the top of the piles in order to predict axial forces and bending moments that reasonably matched measurements. The need for such boundary conditions would not be evident without prior knowledge of the measured data and, thus, presents challenges for normal design scenarios. The coupled analyses produced bending

moments and axial forces that were practically similar to those observed without requiring any manipulation of boundary conditions beyond simply modeling the cap.

Additional coupled analyses were performed to evaluate the effect of coupling at displacements greater than those observed for the two cases. Results were mostly similar for both the uncoupled and coupled models. There was some evidence that interaction between upslope and downslope piles in the coupled model results in earlier mobilization of shear forces at the sliding surface, although the predicted improvement may be a product of calibration differences. Finally, the effect of pile batter was evaluated by applying an additional p -modifier with values consistent with batter effects documented by Reese et al. (2006). Including the batter p -modifier had a detrimental effect on agreement with the measured data. One logical explanation is that batter effects that consistently have been observed in previous studies diminish with depth.

Conclusions

Several significant conclusions are evident from results of coupled finite element models of slope stabilization A-walls:

- Coupled finite element models are a useful method for analyzing A-walls for slope stability applications. Models created as part of this work produced bending moment and axial force profiles in reasonable agreement with measured values.
- Calibration of p - y and t - z curves to achieve predictions consistent with measured values required significant softening of ultimate lateral and axial resistance values: p -modifier of 0.3 and α of 0.5. This finding is largely consistent with results for uncoupled analysis by Loehr and Brown (2008), which required even greater reductions in resistance.
- For the two case histories evaluated, consideration of the capping beam via coupled models improved predictions of pile bending moments and axial forces compared to results from the uncoupled model. Modeling interaction via the cap resulted in more reasonable calibrated values of lateral and axial soil resistance, better agreement with measured axial force profiles, and better agreement with measured bending moments and axial forces at shallow depths. These observations suggest interaction facilitated by the capping beam is having a significant effect on development of forces within the micropiles.
- Although the coupled model improved predictions above the sliding surface, it is important to note both the uncoupled and coupled models produced reasonable agreement at the sliding surface, where predictions are most consequential. However, sliding surfaces for the two cases considered were about 30 ft deep. It is logical to assume the effect of coupling would be more significant for shallower sliding surfaces. Additionally, for shallower sliding, the capping beam itself often serves to directly resist soil movement and transfer lateral load to the micropiles.

- Improvement in the calculated response below the capping beam suggests the coupled model can be used to determine bending moments and axial forces in the capping beam. Such knowledge could be used to improve structural design of A-walls, especially the capping beam.
- For displacements beyond those observed in the case histories, the uncoupled and coupled models predict generally similar performance. However, there was some evidence that interaction between the upslope and downslope piles facilitated by the capping beam results in greater mobilization of axial forces at large displacements and perhaps mobilization of shear forces at the sliding surface at displacements less than those required for the uncoupled model.
- Interaction effects were also noted during the calibration of the coupled models; for example, changes in lateral soil resistance often produced significant changes in calculated axial forces.
- For the relatively deep sliding (approximately 30 ft) considered for both cases, calibration attempts that incorporated published batter effects resulted in poorer agreement with axial force and bending moment measurements than when the batter effect was neglected. This suggests pile batter effects should only be applied for actively loaded foundations and perhaps some shallow sliding surfaces.
- Calibration generally resulted in more accurate predictions of axial forces than bending moments. The difference is likely in part due to difficulties measuring bending moments using strain gages as described in the summary.

Recommendations

Several important design recommendations result from observations regarding the effect of coupling on A-walls for slope stabilization. For deep sliding surfaces (surfaces greater than about 30 ft deep), reasonable predictions of mobilized axial and shear forces can be achieved using the uncoupled procedure recommended by Loehr and Brown (2008). For shallower sliding surfaces, the effect of the capping beam is likely more consequential and coupled analysis is likely prudent. Coupled analysis is also necessary when knowledge of the forces and bending moments in the capping beam or in the piles near the capping beam is required.

The results of this study also lead to recommendations for future study of A-walls for slope stabilization. Foremost among the future study needs is collection of field data from new project applications of slope stabilization A-walls. The research in this study and in previous work by Loehr and Brown (2008) is based on results from only two case studies. Additional data are necessary to make more meaningful advancements in the understanding and design of A-walls for slope stabilization. Loehr and Brown (2008) made specific and detailed recommendations regarding the types of data that should be collected from future A-wall applications. Future data collection should include measurement of inclination within the pile reinforcement (e.g., with

inclinometers) to provide a more continuous profile of bending moments along the length of the pile.

Verification of the range of p and α values identified in this report and by Loehr and Brown (2008) should be a priority for any additional field studies. Another recommended research topic that follows from this study is the effect of pile batter. Researchers should investigate the hypothesis that the pile batter effect diminishes with depth. Identification of a value of depth at which the pile batter effect can justifiably be neglected for design purposes would be useful.

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