Micropiles are routinely used in the United States for foundation support and stabilization of slopes. In recent years, several retaining walls have been constructed using micropiles in an A-frame type configuration to support temporary and permanent excavations at sites with limited access and limited right-of-way. Typical A-frame micropile walls consist of a row of vertical micropiles at the face of the wall and a row of battered micropiles extending into the soil or rock to be retained. The two rows are then tied together with a pile cap such that the front pile acts primarily in bending and the back pile acts primarily in tension.

Because of tight right-of-way constraints at a residential site in Aspen, Colorado, it was not possible to construct the wall using conventional shoring systems or micropiles in the typical A-frame configuration. Rather, the wall was designed such that the row of battered piles was installed at the wall face and the row of vertical piles was installed adjacent to the property line behind the wall. As a result of the reverse configuration of the A-frame wall, FLAC, a two-dimensional, finite difference software program, was used to model the soil-structure interaction. The final wall design consisted of both temporary and permanent retaining wall sections with maximum heights of 26 feet. Inclinometers were installed in several piles to monitor the behavior of the retaining walls during and after excavation. This paper presents an overview of the design method, a discussion of construction techniques, and a comparison between the FLAC model and the measured behavior of the wall in the field.

INTRODUCTION

Project Description

Construction in Aspen, Colorado has flourished in recent years, most notably in the residential sector. The majority of residential construction projects within the city limits involve replacement or upgrade of existing structures by maximizing square footage in the form of basements and larger structure footprints. This maximization typically pushes the limits of rights-of-way and easements along the boundaries of the properties where subgrade walls and footings are required. Because of the proximity of most excavations to neighboring properties, excavation shoring is typically required to protect these adjacent properties from damage such as soil sloughing, structure settlement, or utility movement.

Excavation shoring has also become more complex as builders push the envelope on building footprints and layouts. In the project discussed herein, a twenty-two- to twenty-six-foot deep cut was desired within ten feet of the property line and within twenty-five feet of a residence on the neighboring property. Thus, in order to not undermine soils and the structure on the neighboring lot, shoring was required within the ten foot easement along the property boundary (Figure 1). Adding to the complexity of the project, a portion of the shoring was specified as permanent.

Several shoring options were discussed for the project; however, because of the limited horizontal space that was available, and the “cobbly” geological conditions, a micropile wall became the only viable option. To not encroach under the neighboring property, the wall would be an atypical A-frame micropile wall in that the battered pile would be at the face of the wall, and the vertical pile would be along the property line itself.

Project Geology

Published geologic maps (Bryant, 1972a; 1972b) of the area indicate that the site is located near the base of Smuggler Mountain and is part of the larger Sawatch Range. The site is mapped as Pleistocene-age (10,000 to 1.8 million years old) glacial moraine deposits, though localized deposits of colluvial, alluvial
fan, landslide, and solifluction deposits exist. Generally, these deposits consist of an unsorted mixture of loose to medium dense gravels, cobbles and boulders within a silty, sandy matrix.

Site-specific soil conditions were determined from subsurface explorations completed by others for another phase of the project (Hepworth-Pawlak, 2005). The boring indicated that subsurface conditions consisted of about 2.5 feet of granular fill overlying medium dense, silty sand and gravel to the bottom of the boring at 25 feet. Cobbles and boulders were encountered in the upper 13.5 feet of the sand and gravel layer.

During construction, soil conditions at the wall location were generally consistent with those encountered in the boring. The observed soil conditions consisted of a layer of topsoil 1 to 3 feet thick at the ground surface, underlain by an unsorted, unconsolidated mix of subrounded to rounded boulders, cobbles and gravel supported in a silty sand matrix. Drilling during micropile installation indicated that the boulders and cobbles likely persisted to at least 50 feet below the existing ground surface at this site.

**DESIGN**

**Micropile Design Methods**

The Federal Highway Administration (FHWA) Micropile Design and Construction Manual (2005) is generally recognized as the industry standard for micropile design. While the manual states that micropiles can be used for earth retention and outlines a method for design of A-frame micropiles for slope stabilization, it does not explicitly describe a method for design of micropiles for earth retention. The design method for slope stabilization assumes that the slope is unstable and a slip surface has formed prior to micropile installation. The micropiles are used to provide resisting forces to the slope. It was our opinion that this method was not directly applicable for a retaining wall where earth pressures develop against the wall.

We performed preliminary analyses of the retaining wall using traditional earth pressure theory and limit equilibrium methods. While it is our opinion that this method is applicable for a micropile wall, the traditional limit equilibrium method lacked two key components. First, this method does not provide estimated deflections of the piles or the soil mass behind the wall. Second, the method does not account for potentially complicated soil-structure interaction. For example, we were concerned about the front, battered pile having both lateral and axial
loads that would cause a buckling effect in the pile.

Because the published design methods and traditional analysis methods were not adequate, we sought to use a more robust method of analysis that incorporated soil-structure interaction and an estimate of displacements. An article published by Uebelacker (1996; 1997) in Foundation Drilling Magazine described analysis of an A-frame micropile wall using numerical modeling. Because the referenced project did not have similar site constraints to our project, the front pile was installed vertically and the back pile was battered. However, the article indicated that the design and installation was successfully completed. Because of our experience with numerical modeling methods and the successful completion of the project described in the Uebelacker article, we chose to use numerical modeling to design the A-frame micropile wall.

FLAC Analyses

Numerical analysis of the A-frame wall was accomplished using FLAC-2D (Itasca, 2005), a two-dimensional, plane-strain, finite difference program specifically developed to simulate soil-structure behavior and interaction. The FLAC program represents soil and rock as a connected mesh of elements, assembled to represent the geometry of the physical structure being modeled. Each element is assigned a constitutive equation that represents the stress-strain response and failure criteria of the material being modeled. Mass, elasticity, and strength properties representative of the material being modeled are also assigned to each element. Construction operations such as excavating, placing fill, and other changes in applied pressures are simulated by adding or removing elements or by applying forces at element nodes (corners). At each step of the modeling process, a system of linear equations is explicitly solved by evaluating changes in stresses and displacements at each of the nodes to obtain a condition of static equilibrium.

A two-dimensional approach was selected for this project based on the assumption that the soil is a uniform mass and that displacements resulting from wall movement can be approximated by a plane-strain model. Further, based on our numerical modeling of piles used to stabilize slopes (Fischer and Kershaw, 2004), it was our opinion that the micropiles would be spaced close enough such that squeezing between the piles did not need to be considered by a three-dimensional analysis.

For this project, a mesh (Figure 2) of approximately 9,000 elements was created to represent the soil mass. The mesh was tilted at an angle of approximately 10 degrees to model the natural slope of the ground at the site, and the mesh was modified in front of the wall to allow for horizontal excavation lifts.

Engineering properties of the soil were assigned based on borings completed by others for the project. We assumed that the soil was homogeneous and would act as a Mohr-Coulomb material. A friction angle of 34 degrees, cohesion of 50 pounds per square foot (psf), and a unit weight of 125 pounds per cubic foot (pcf) were assigned to the soil mass. Based on the recommended default values for a sandy gravel provided by the FLAC program and the lack of any site-specific data, we used a soil elastic modulus of 15x10^6 psf.

After the general mesh was created, the FLAC analysis was completed in several consecutive steps, attempting to accurately model site conditions and the construction sequence. The first step of the modeling procedure was to establish in situ stress conditions throughout the mesh. This was accomplished for each case by initializing vertical and horizontal stresses based on the unit weight of the soil under consideration, then running the model to static equilibrium subject to gravitational acceleration. After initialization of in situ stresses, accumulated displacements were set to zero before completing subsequent steps.

The second step of the procedure was to install the micropiles and the pile cap. The micropiles were modeled as elastic beam elements. Both the front and back piles consisted of a cased and uncased length, and the back, vertical pile also had a center reinforcing bar. For the cased lengths, we ignored the grout and assigned pile properties equivalent to the elastic modulus of steel (29 x 10^6 psi) and the area of the steel casing. For the uncased lengths, we ignored the steel reinforcing bar, where present, and assigned concrete properties to the pile using the area of the grout bulb and 4,000 psi concrete. The pile cap was modeled as an
elastic material with properties of 4,000 psi concrete. Because the pile cap tied the two piles together and was continuous along the wall, it was modeled such that it was not allowed to rotate. This rotation resistance was equivalent to modeling a fixed-head pile condition. The structural design of the pile cap included provisions to provide a fixed condition, as discussed later.

The third step of the FLAC analysis was to excavate in front of the wall. Excavation was completed in six steps for the temporary wall and eight steps for the permanent wall. The lift heights for each excavation step ranged from 1 to 5 feet with the larger lifts occurring near the top of the wall and the smaller lifts occurring near the base of the excavation. The model was run to static equilibrium after each excavation step and the displacements and stresses were cumulative throughout the excavation process.

The fourth step of the process was only used in our analysis of the permanent micropile wall. As recommended in the FHWA Manual, we assumed that corrosion protection for the steel casing would consist of sacrificial steel. Because we did not have site-specific soil corrosion tests, we assumed that the corrosion potential was aggressive. For a service life of 100 years, the FHWA Manual recommends using a sacrificial steel thickness of 0.236 inches. To model the corrosion, we reduced the thickness of the outer casing by the recommended sacrificial steel thickness following excavation of the last lift and ran the model to static equilibrium.

The fifth step of our analysis was also only implemented for the permanent A-frame micropile wall. A pseudo-static horizontal ground acceleration of 0.06g was applied to the mesh and the model was run to equilibrium.

The sixth and final step of the FLAC analysis was to determine the factor of safety (FS) of the wall system. To determine the FS, the soil strengths are reduced by dividing the cohesion and the tangent of the friction angle by a factor and the last step of the analysis is re-run. This factor was increased incrementally until the model no longer came to static equilibrium. The largest factor that, when applied, would still allow the model to come to static equilibrium was termed the FS for the micropile wall system. The target FS was 1.3 for the temporary wall, 1.5 for the permanent wall, and 1.1 for the permanent wall with seismic loads applied.
Analysis Results

The output for each FLAC run included: micropile deflection, axial load, shear, and bending moment; soil deflection behind the wall; and FS. Based on comparisons of these results to the project requirements, modifications were made to the FLAC model and additional cases were run until the design satisfied the project requirements. In particular, because the micropile elements were modeled as elastic beam elements, we compared the axial load, shear, and bending moment output from FLAC to the micropile capacities calculated in accordance with the FHWA Manual. For most cases, combined axial and bending (buckling) of the front, battered pile controlled the design.

Using the iterative design procedure described above, final design of both a temporary and permanent A-frame micropile wall was completed. As previously described, both walls consisted of a row of battered piles and a staggered row of vertical piles tied together with a pile cap (Figure 3). The battered piles had a cased length of 37 feet and an uncased length of 10 feet, while the vertical piles had a cased length of 28 feet and an uncased length of 10 feet. For the temporary wall, the casing for both the battered piles and vertical piles consisted of a single 7.625-inch outer diameter (O.D.) casing with a wall thickness of 0.5 inch. In addition, a #10 central reinforcing bar was used in the vertical pile. For the permanent wall, the casing and central reinforcing bar for the vertical pile were identical to the temporary wall. However, because of the relatively large amount of sacrificial steel and the increased excavation depth for the permanent wall, additional steel was added to the battered pile to obtain the required resistance against buckling. Two additional sections of steel casing were inserted into the outer 7.625-inch O.D., 0.5-inch wall casing. The two inner casings consisted of 5.5-inch O.D. casing with a wall thickness of 0.361 inch and 4.0-inch O.D. pipe with a wall thickness of 0.32 inch.

Figure 3 – A-Frame Micropile Wall Cross-Section
CONSTRUCTION

Installation of the wall began in early November 2006. Initially, the site was leveled and pre-excavated three feet below existing grade to the bottom of the proposed micropile cap elevation. Micropile installation proceeded along the planned wall alignment and alternated between the installation of vertical and battered piles. A duplex drilling system was used to drill the casing as well as the deeper uncased length at the same time. The outer drill string consisted of the 7.625-inch O.D., N80 micropile casing while the inner drill string consisted of a five-inch down-the-hole hammer with a six-inch button bit. The casing was installed to its full design depth, and then the uncased bond zone was drilled with the inner drill string. A Klemm 806D double-rotary-head drill was utilized for the project and each drill string was rotated and advanced with its own rotary head.

Once the 7.625-inch casing was installed and the bond zone was drilled out, the inner casing (where required) was inserted and hung with centralizers in the cased pile. A high-strength threadbar was installed into the pile and the pile was tremie grouted from the bottom of the hole with neat cement grout at a water to cement ratio of 0.45 and minimum yield strength of 4,000 psi.

The geology at the site was particularly advantageous in that the bond zone beyond the cased length remained open after being drilled with the inner string. Had the soils in the bond zone shown potential for collapsing into the hole, casing would have been required to the full micropile depth. The bar would have been inserted and the pile grouted before withdrawing the casing to the planned elevation.

Installation of the micropiles was completed in mid-December and the micropile cap was poured shortly before excavation began. As excavation progressed, studs were welded onto the front of the casing, drain board was installed, and rebar and wire mesh were hung. The face of the wall was then shotcreted in lifts to prevent sloughing of the soils. The shoring portion of the project was completed in the third week of December. A photograph showing the partially completed wall is shown in Figure 4.

INSTRUMENTATION

Instrumentation was installed in selected micropiles to observe field behavior and to verify the results of our FLAC analyses. Because of the ease of installation and monitoring, as well as the type of field data obtained, we chose inclinometers as the preferred instrumentation for the micropile wall.
An inclinometer is a relatively flexible plastic pipe (casing) installed such that it moves with the soil mass in which it is installed. In this case, the inclinometers were installed directly into the micropile steel casings. Lateral deformation is determined by measuring the change in casing inclination from vertical over time with a gravity sensitive probe. The change in inclination angle can then be used to trigonometrically calculate deflection. The inclinometer casing used for this project consisted of 1.5-inch I.D. plastic pipe with four longitudinal guide grooves cut into the inside at 90-degree intervals. The internal machine-broached grooves allow the inclinometer probe to track within the casing for consistent precision monitoring. Typically, the casing is installed with one set of grooves parallel and one set of grooves perpendicular to the anticipated direction of movement.

For this project, inclinometers were installed in four selected micropiles. The type of micropile, name designation, and length of casing for the four inclinometers included: a temporary vertical micropile (TV) with 35 feet of inclinometer casing; a temporary battered micropile (TB) with 47 feet of inclinometer casing; a permanent vertical micropile (PV) with 33 feet of inclinometer casing; and a permanent battered micropile (PB) with 43 feet of inclinometer casing.

Locations of the micropiles with inclinometer casings are shown on Figure 1. The locations were chosen, in part, due to the progress of the wall construction at the time of installation. However, our additional criteria for inclinometer location selection was that each inclinometer set includes a vertical micropile and an adjacent battered micropile. In addition, we wanted to have one set of inclinometers in the permanent portion of the wall and one set in the temporary portion of the wall.

Installation

The inclinometers were installed in the micropiles on December 1 and 2, 2006. The inclinometer casings were strapped to the central reinforcing bar, where present. Centralizers were modified to fit around the inclinometer casing and/or rebar as they were lowered into the micropiles to keep it centered within the steel casing. The micropiles were then grouted and the pile cap was poured with concrete, cementing the inclinometer casings into place. Once the pile cap was poured, the top of the inclinometer casing was cut approximately 12 inches above the top surface of the pile cap, and a top cap was placed on the casing.

Data Acquisition

Initial inclinometer readings were acquired on December 12, 2006, prior to excavation in front of the wall. Subsequent inclinometer readings were taken to compare to the initial readings and develop plots of cumulative lateral displacement along the length of the casing. The subsequent readings included December 13, 2006, after excavation adjacent to the wall had been advanced to an approximate depth of 12 feet. Additional readings were taken on December 20, 2006, at which time the excavation had been advanced to the maximum depth of approximately 22 to 26 feet below the existing grade. Two sets of final readings were obtained on January 19, 2007, prior to construction of the basement wall in front of the micropile retaining wall.

For all readings, inclination measurements were obtained at 2-foot intervals beginning approximately one foot from the bottom of the inclinometer casing to within approximately one foot of the top of the pile cap.

Data Post-Processing

The data from the readings were compiled through Slope Indicator's DataMate Manager (DMM) application and plotted using Slope Indicator’s Digipro application. The plotted data was written to a text file and imported into a spreadsheet to compare with predicted displacements calculated using FLAC.

Plotted raw data from the vertical casings indicated up to 0.2 inch of horizontal displacement in the permanent section and up to 0.25 inch of horizontal displacement in the temporary section, both with maximum deflections at depths between 10 and 15 feet.

Evaluation of the raw data resulted in the determination that correction factors were necessary to remove systematic errors generated during the measurement of the battered pile inclinometer casings. The plots shown in Figures 5 and 6 show the corrected...
inclinometer data. There were several reasons for applying correction factors, as discussed below. First, the battered piles were installed at an inclination of about 20 degrees out of vertical. The manufacturer indicates that the inclinometer instrument has a measurement range of up to 35 degrees out of vertical, but when the casing is beyond 3 degrees from vertical, the accuracy and repeatability of the readings is diminished.

Second, each pair of installations (vertical and battered) was relatively close to each other and was horizontally restrained at the top by the concrete cap beam. As such, the horizontal displacement measured at the top of the battered casings should be approximately the same as the top displacement of the vertical casings.

Third, based on our observations at the site and data from the vertical inclinometer, it appeared that the bottom 10 feet of the casings were embedded in stable, relatively non-displacing ground, and that negligible horizontal displacements would occur in this depth interval.

COMPARISON OF FLAC RESULTS AND FIELD BEHAVIOR

Following construction, we compared the pile deflection estimated from our FLAC analyses to the actual deflection measured in the field using the installed inclinometers. Plots showing the field data superimposed on the FLAC results are shown in Figures 5 and 6 for the permanent wall and temporary wall, respectively. The plots for the permanent wall compare the field data to the FLAC results prior to reducing the cross-section for corrosion and application of the seismic load.

As shown on the plots, the field behavior matched our deflection estimates relatively well. The shapes of the deflection curves are similar and the magnitudes are also close. In general, our FLAC analyses overestimated the maximum micropile deflection that occurred at about mid-height of the cut. The inclinometer data indicated that the head of the pile was restrained and that the magnitude of the restraint was greater than predicted by FLAC.

![Figure 5 – FLAC vs. Inclinometer Comparison – Permanent Micropile Wall](image-url)
However, it appears that the fixity of the top of the pile did not extend as far into the pile cap as modeled in the FLAC analysis. In most cases, our FLAC model slightly underestimated the displacement at and just below the maximum excavation depth.

CONCLUSIONS

Based on the general agreement between deflection predictions using FLAC and behavior measured in the field with the inclinometers, it is our opinion that FLAC finite difference modeling is an acceptable method for designing A-frame micropile walls that provides relatively accurate modeling of field behavior.

Because there are not established design methods for A-frame micropile walls and because of the atypical configuration of the micropile wall (battered pile in front) for this project, we evaluated many different cases and learned several important lessons, as described below.

First, buckling of the front, battered pile controls the design. Micropiles typically do not have large bending or buckling resistance because of the small diameter and relatively small wall thickness of the steel casing. Therefore, to obtain additional bending/buckling resistance, additional steel was added. Placing smaller diameter casing within the outer micropile casing is not a particularly efficient method of obtaining additional bending/buckling resistance because of the relatively small moment of inertia of the small diameter casing. However, to stay within the limited wall envelope, we could not get more steel into the micropile by increasing the diameter of the outer casing.

Second, the use of sacrificial steel for corrosion protection of permanent micropiles can result in a significant amount of additional steel in the micropile. For this project, the highest level of soil corrosion potential recommended by the FHWA Manual was assumed because site-specific corrosivity tests were not conducted. Ideally, corrosion tests would be performed for each project prior to the micropile design phase. However, because the shoring system is sometimes not determined that early in the project, the geotechnical engineer may not have the foresight to perform these tests. If corrosion tests have not been performed, it may be beneficial to obtain additional representative samples prior to micropile design. A relatively inexpensive soil test may save money on sacrificial steel.
RECOMMENDATIONS FOR FURTHER STUDY

As discussed above, our analyses indicate that FLAC is a good design method for this type of micropile wall. However, the results presented herein cannot necessarily be extrapolated to other soil types and other situations. Therefore, we recommend that additional A-frame micropile walls be designed using FLAC and instrumented to observe field behavior. Because of time constraints on this project, we were only able to install inclinometers. To better evaluate the behavior of the steel casing, we recommend that strain gages also be included as instrumentation on subsequent projects. Design and instrumentation at additional sites with differing site conditions will allow designers to make more generalized statements regarding design and behavior of A-frame micropile walls. The results from additional projects may also aid us in determining simplified design methods for these walls.

REFERENCE LIST


