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MICROPILE FOUNDATIONS IN KARST: STATIC AND DYNAMIC TESTING VARIABILITY

Jesús Gómez, Ph.D., P.E
Schnabel Engineering, Inc.
West Chester, PA-USA-19380

Allen Cadden, P.E.
Schnabel Engineering, Inc.
West Chester, PA-USA-19380

O. Christopher Webster, P.E.
Schnabel Engineering, Inc.
Charlottesville, VA-USA-22901

ABSTRACT

Development of a major industrial facility required support of large loads from machine foundations. The site was underlain by highly variable karstic limestone conditions, which resulted in irregular depths to rock, very soft residual soil layers, and potential for voids in the rock and soil matrix. Foundation mats on micropiles were selected for support of the machines. The benefits associated with the micropiles were the speed of installation, and relative cost and schedule savings.

Two load tests were performed before the start of micropile installation. One of these tests failed prematurely. A third test, performed during the initial stages of construction, also failed prematurely. Pile Driving Analyzer (PDA) testing of micropiles was used to investigate the capacity and variability of production piles that were already installed. The authors believe this may be the first application of the PDA technology to estimate the carrying capacity of micropiles bonded into rock. Because of the lack of previous experience in this application of PDA testing, suitable testing procedures needed to be developed in the field to reduce the potential for damage of the production micropiles, and to assess the accuracy of the tests. The results of the testing program showed that PDA testing may provide very accurate estimates of the capacity of micropiles bonded into rock. This paper discusses the techniques used for PDA testing of the micropiles, and compares the results of the PDA tests to the data from static load tests. The paper also contains a brief discussion on the site conditions, and the effect of the construction methods on the measured capacity of the micropiles and their variability.

INTRODUCTION

Micropile technology has evolved significantly since its inception in the 1950s. Early applications of micropiles in Europe consisted of lightly loaded groups of elements intended to enclose and reinforce an unstable soil mass for slope stabilization or underpinning of historic buildings (Lizzi, 1982). Micropile technology developed slowly in Europe over the next 20 or 30 years, until publication of successful case histories induced its rapid growth in the United States, where it evolved more towards the use of heavily reinforced micropiles with high axial load-carrying capacities.

In the United States, the most common application of micropiles has traditionally been underpinning of existing structures. For this particular application, micropiles are often a more economical alternative. In addition, they may install more quickly than other underpinning alternatives within confined spaces and low headroom conditions, and produce a limited amount of spoils.

More recently, foundation designs of new structures have used micropiles as an economical alternative to other foundation systems. The authors have designed foundations for several new structures using micropiles. One good example is in Manhattan, New York City, NY, where micropiles were used for foundations of a new building. The micropiles traversed the upper layers of old fill containing rubble and debris typical of the area, and the deeper hardpan to reach the underlying granite. The owner preferred micropiles over driven piles since the micropiles could be installed with less disturbance to adjacent old structures and could achieve higher working capacities.

Another recent example of the application of micropiles for new structures is a new electric power generation plant in the Piedmont of Virginia. The plant is located in a karst area, where depths of significant karst features varied from a few feet to over 100 ft. Large interconnected voids and layers of very soft clays and silts existed within the formation among harder limestone pinnacles and ledges. Micropiles provided a suitable foundation alternative since they could be installed through the upper karst features into competent rock. A single unit price per linear foot could be defined regardless of the type of the material traversed. This represented a significant advantage over drilled shafts, where rock drilling and concrete overages in the karst terrain could easily exceed the foundation construction budget.

In this project, the specialty foundation contractor tested three micropiles before or right after the start of production pile installation. Two of the three piles failed prematurely during load testing. This prompted an investigation by the geotechnical consultant into the causes of failure, and the capacity of the production piles that the contractor had already installed.

The geotechnical consultant concluded that the piles failed due to a combination of the installation procedures selected by the contractor, lack of adequate field observation during installation, and the variable karstic conditions existing at the site. It was also concluded that, to be able to estimate the available capacity of the production piles already installed, a significant portion of the piles had to be tested.

After careful consideration of the testing options available, the geotechnical consultant decided that the micropiles be dynamically tested using the Pile Driving Analyzer (PDA) (Smith, 1960; Goble Rausche Likins, 1996). This testing program presented several challenges, the most important of which was reducing the potential for damage of the production micropiles tested.

This paper focuses on the techniques used for PDA testing of the micropiles, and presents several interesting conclusions regarding the use of PDA as a Quality Control (QC) tool. The most important finding is that PDA may be a suitable procedure for verifying the capacity of micropiles bonded into rock, provided that certain precautions are taken during testing to prevent pile damage.

GEOLOGICAL SETTING

The site is located in the Piedmont Geographic Province in north central Virginia. The major site stratigraphy consists of karst terrain of the Everona Limestone and its overlying residual soils and disintegrated rock. The Everona Limestone Formation varies in depth from 20 to 1100 ft and is likely of Early Paleozoic Age.

Figure 1 illustrates the general stratigraphy at the site. Surface and near surface intervals consist of residual soils composed by medium stiff to hard silts and clays. A layer of soft to very soft silt underlies the stiff residual soils. This layer is commonly referred to as epi-karst. The epi-karst is not a continuous layer; instead, the epi-karst appears randomly within stiffer layers of soil and rock and in seams of variable thickness. The boundary between the epi-karst and the underlying limestone is not well defined. Based on the field observations, the geotechnical consultant estimated that the contractor would likely encounter suitable limestone for foundation purposes at depths ranging from 40 to 110 ft.

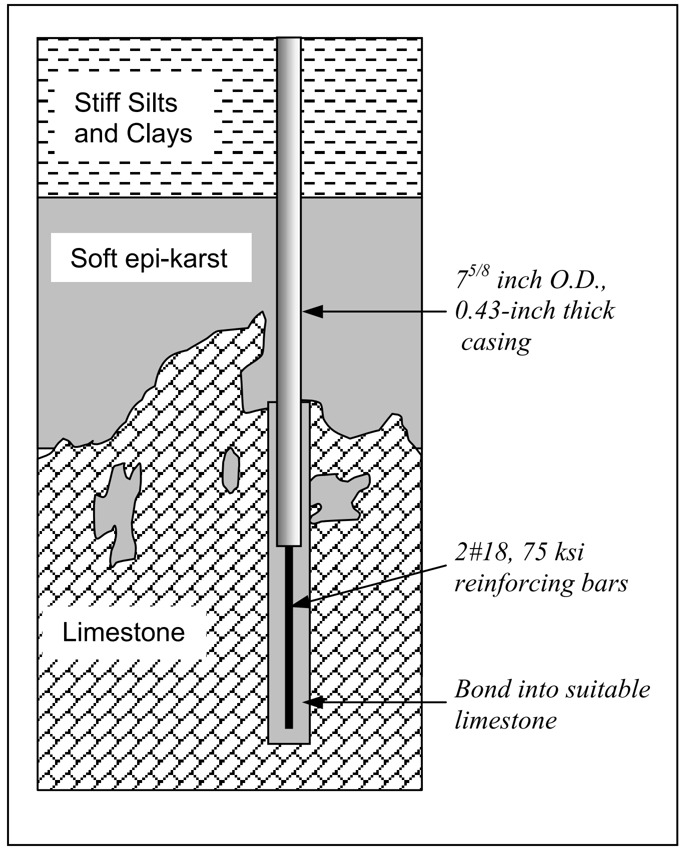

Fig. 1. Typical stratigraphy at the site.

MICROPILE INSTALLATION

As shown in Fig. 1, each micropile consisted of a 7e inch O.D., 0.43-inch thick casing along the unbonded zone. The bonded zone consisted of two central #18, 75-ksi All-Thread bars extending from 5 ft above the tip of the casing and into the bond zone. The foundation design established the design load of the micropiles at 150 kips, a relatively low value for micropiles adequately bonded into rock. The specialty foundation contractor installed the micropiles by pre-drilling with a Down-Hole Hammer (DHH). The final tip elevation of the micropiles was established using a 10-ft penetration criterion into the bearing material. Thus, determination of the length of the micropile was highly dependent on the operator and the field inspector, who should identify continuous rock based on the resistance to drilling with the DHH.

Once pre-drilling was complete, the casing was spun to the bottom of the predrilled hole. The bond zone was typically established by raising the casing 10 ft above the bottom of the hole. Simultaneous injection of pressurized air was used throughout the process. Contamination of the bond zone due to ingress of soft soil and mud was an important concern.

Before grouting, the hole was probed with a weighted tape and tremie-flushed with water to attempt to displace soils and mud often detected inside the bond zone during probing. The contractor used a tremie pipe inserted to the bottom of the hole to gravity fill grout, and continued until the grout reached the top of the casing.

During drilling, communication between predrilled holes was observed frequently (see Fig. 2). In a number of piles, the level of grout inside the casing decreased over time, which indicated the existence of open voids along or near the bond zone.

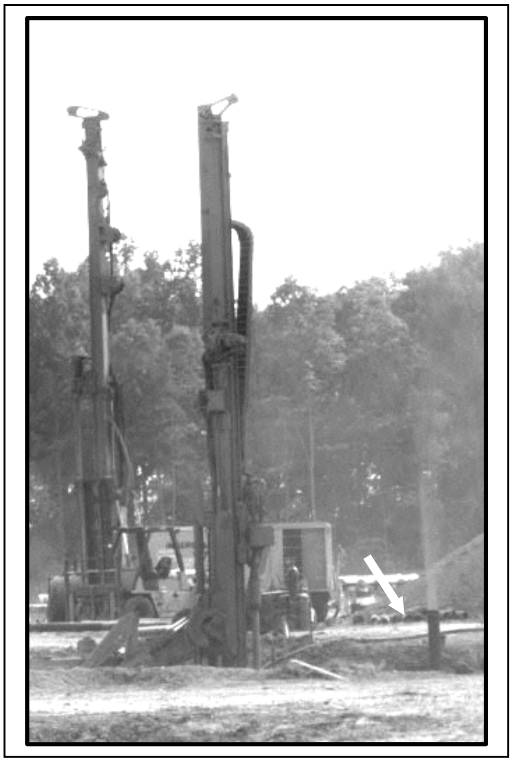

Fig. 2. Communication with adjacent holes was often evident during drilling.

STATIC LOAD TESTING

Initially, the contractor conducted two static load tests in sacrificial micropiles. To address the particular design characteristics of the project, the contractor conducted compression, tension, and lateral load testing on the micropiles. To allow for tension testing, the project requirements included reinforcing the upper 10 ft of the unbonded zone of each micropile with one #13, 150 ksi Williams All-Thread bar. The contractor installed both piles following the general procedures described previously.

Micropile TP-1 was drilled to a depth of 39 ft. The tip of the casing was left at a depth of approximately 29 ft. Micropile TP-2 was drilled to a depth of approximately 56 ft. The tip of the casing was left at a depth of 46 ft. During installation of this micropile, it was observed that some zones within the overburden contained very moist and soft soils, and multiple flushing of the casing was necessary to remove these sediments from the bond zone, and ensure that the casing was set at the proper depth.

The compressive axial load tests were conducted in general accordance with the procedure for quick load tests described in ASTM D1143, and recommendations by the Federal Highway Administration (FHWA) presented in the Micropile Design and Construction Guidelines Implementation Manual (FHWA-SA-97-070). The test piles were loaded to twice the design load. Figure 3 illustrates the results of the compression load tests.

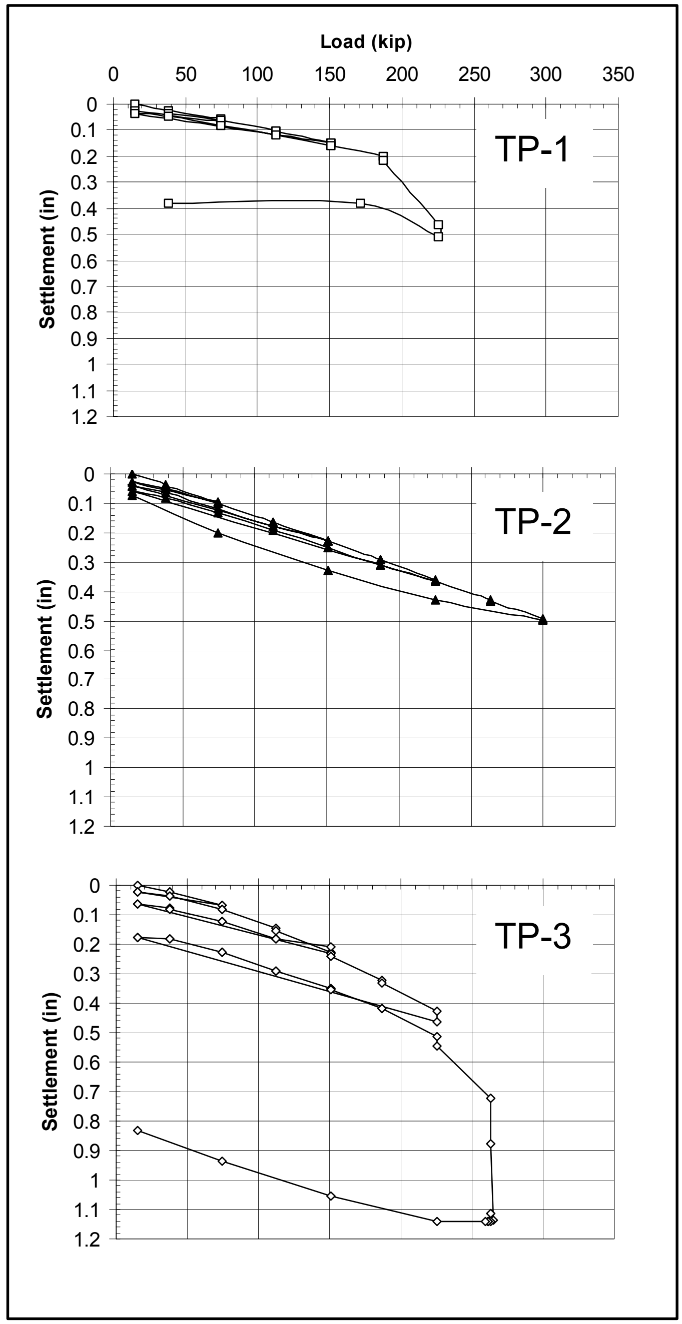

Fig. 3. Static load test results.

As can be seen in the figure, Micropile TP-1 failed during compression testing under a load of approximately 190 kips (125 percent of design load). Micropile TP-2 tested successfully. The gross settlement at the design load of 150 kips was approximately 0.23 inch. The gross settlement under the maximum test load of 300 kips was approximately 0.5 inch. Creep during load application was negligible. Interpretation of the elastic rebound data (Gómez et al., 2003) during each of the unloading cycles suggested that failure of Micropile TP-1 was due to poor bond along the grout-rock interface, and that no structural failure of the micropile took place.

Due to the failure of Micropile TP-1, the contractor performed a third load test on a production pile. This pile also failed prematurely under a load of approximately 260 kips. This was cause for concern requiring the geotechnical consultant to investigate the causes for the failure and evaluate the available capacity of the production micropiles that the contractor had already installed.

The geotechnical consultant concluded that the causes for the failure of the two micropiles were contamination of the contact between the grout and the rock due to the particular subsurface conditions, and the difficulties encountered to clean the bond zone by flushing with water.

It was decided that it was necessary to evaluate the capacity of the production piles that had already been installed. Two alternatives were available from a practical point of view. Statnamic Testing (Middendorp and Van Foeken, 2000) could be used to perform several load tests per day on production micropiles. Furthermore, this technique had been successfully applied to driven piles and drilled shafts. PDA testing was another practical alternative. Although PDA testing had been used in some cases to predict capacity of drilled shafts in soils, the geotechnical consultant could find no records on the use of PDA testing for micropiles bonded into rock. There was also a concern about potential structural damage of the micropiles during each impact. Finally, it was also believed that degradation of the micropile bond to the rock might take place during PDA testing.

PDA TESTING RESULTS

PDA testing of production micropiles was performed at different locations on micropiles of varying length. Once 22 production micropiles were tested, the General Contractor decided to discontinue further PDA testing.

PDA testing was performed using two accelerometers and two strain gauges (Fig. 4) attached to the casing and close to the head of each tested micropile. The head of each test pile was impacted using a Vulcan 01 air hammer (see Fig. 5). The hammer had an energy rating of approximately 15,000 ft-lb. This selected hammer was the only low energy hammer that was readily available at the time. Although this hammer performed well for this application, the authors' own experience and additional input from GRL suggest that hydraulic hammers may be more convenient for this application since they may allow better control on the energy imparted to the piles.

Fig. 4. View of the accelerometer and strain gauge used for PDA testing.

Fig. 5. PDA testing on a production micropile using a Vulcan 01 air hammer.

The top of each production pile was fitted with reinforcing bars for connection with the pile cap. Consequently, a custom follower device was fabricated and placed between the hammer and the pile head to permit testing without damage to the reinforcement. Cushioning was provided between the head of the pile and the follower device, and consisted of approximately 5 to 7 inches of plywood, as illustrated in Fig. 6. The amount of cushioning needed to prevent damage to the piles was determined during calibration testing of Micropiles TP-1, TP-2, and TP-3.

Fig. 6. View of the follower device and cushioning. Note the reinforcing steel of the micropiles.

Testing of each production pile was performed by striking the pile initially with a low energy blow. Once the first blow was applied and the PDA results were examined onscreen, the pile was subjected to two or more additional full hammer strokes. Striking was discontinued once a PDA capacity of 300 kips or more was measured, or if damage to the micropile was imminent based on the estimated stresses along the pile. Table 1 shows a summary of the results of PDA testing.

In addition to PDA testing, Case Wave Pile Analysis Program (CAPWAP) analyses were performed on selected piles to obtain additional data on pile response to loading and to confirm the capacity obtained from PDA testing. The results of CAPWAP analyses are also included in Table 1.

As seen in the table, the PDA results on Micropiles TP-1, TP-2, and TP-3 were consistent with the ultimate capacity values determined from the static load tests. In Micropiles TP-1 and TP-3, the PDA capacity was approximately 20 kips higher than the static load capacity. The CAPWAP and PDA results for Micropile TP-2 indicated a capacity larger than 300 kips, which was consistent with the static load test results. It is noted that the ultimate capacity given by PDA and CAPWAP on Micropile TP-2 is consistent with the authors' experience on micropiles bonded into limestone (Cadden et al., 2001).

Table 1. Summary of Results from PDA Testing, CAPWAP Analyses, and Static Load Testing

Test	Length (ft)	Capacity (Kip)		
		PDA	CAPWAP	Static
TP-1	40.00	243	250	225
TP-2	57.30	402	495	>300
TP-3	46.40	281	263	260
1	50.00	318		
2	44.60	307		
3	46.00	308		
4	64.80	312	399	
5	53.30	198		
6	96.50	272	288	
7	55.50	285		
8	46.00	272	330	
9	81.50	290		
10	45.70	201		
11	41.30	226		
12	74.00	251	290	
13	37.20	319		
14	45.00	314		
15	40.00	312		
16	41.00	199	258	
17	41.90	75	81	
18	41.30	319		
19	37.00	288		
20	57.00	293		
21	41.10	272		
22	69.20	282		

Based on the comparison between PDA and static load test results, it was concluded that the dynamic testing, as performed, could provide reasonably accurate estimates of capacity for the rest of the production micropiles tested.

It must be noted that PDA testing of each micropile was discontinued if a capacity of 300 kips or more was measured. Therefore, it must be kept in mind that the capacity of the piles that exceed 300 kips in Table 1 may actually be significantly greater. In these piles, it is possible that the impact energy applied was not sufficient to mobilize the bond strength along the bond zone. It is believed that capacity values of less than 250 kips are reasonably accurate, as they were typically obtained after several blows with increasing energy and after some measurable permanent displacement of the micropile.

The results presented in Table 1 are shown graphically in Fig. 7. It may be noted that there was a number of piles that did not reach a capacity of 250 kips. Pile 17 only showed a PDA capacity of 75 kips. During testing of this pile, large displacements at the head of the pile were noticeable.

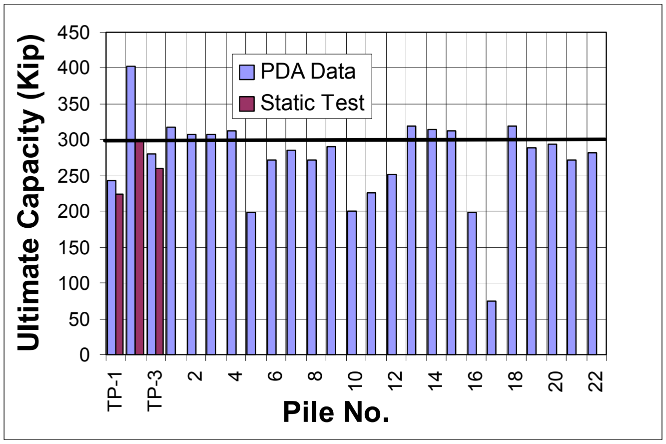

Fig. 7. Variability of micropile capacity as determined from PDA testing.

It is interesting to note the variation of capacity from one micropile to another. Relatively large variations were common in micropiles that were installed in close proximity and with similar lengths. This variability in micropile capacity may be attributed to the installation methods used.

Based on the results of the PDA tests, recommendations were developed to establish a reduced bearing capacity for the micropiles that could be used to adjust the original foundation design.

CONCLUSIONS

It is believed that the low capacity piles as identified by the PDA testing and static load tests were the result of inadequate development or inadequate cleaning of the bond zone.

A review of the overall PDA data indicates that of the 25 piles tested, ten micropiles (40 percent) had PDA capacities equal or greater than 300 kips, which corresponds to a factor of safety of 2 or more for a design working load of 150 kips. Eight micropiles (32 percent) had PDA capacities between 250 and 300 kips. As mentioned earlier, in some of these piles, PDA testing was stopped if there was potential for pile damage; therefore, their actual capacity may have been larger. Seven micropiles (28 percent) had ultimate capacities of less than 250 kips, which corresponds to a factor of safety of 1.7 or lower.

It can also be noted that 24 of the piles had ultimate capacities in excess of the design working load of 150 kips. Only one pile tested lower than this value, with an ultimate capacity measured of about 80 kips. Considering the testing completed to date, this could indicate that about five percent of the piles would not develop the intended design working capacity before geotechnical failure. Also, 30 to 50 percent of the piles did not meet the intended factor of safety of at least two over the ultimate capacity.

The authors believe that the variability in micropile capacity was due to a combination of the difficult karstic conditions at the site and the methods of installation of micropiles. Adequate installation of micropiles requires identification of suitable rock for load transfer during drilling, and adequate cleaning and thorough grouting of the piles, which was difficult to achieve given the presence of the epi-karst.

It is noted that the drilling method selected for the micropiles was essentially “open-hole.” Duplex drilling methods are believed to be much better suited for installation of micropiles in karst.

PDA testing of micropiles bonded into rock can be performed successfully. It is recommended that calibration tests be performed that allow comparison of measured static capacity values to PDA results. The calibration tests would also aid in establishing testing procedures that reduce the potential for damage to production piles. PDA testing should not be used as a substitute for static load testing.

Hydraulic hammers may be better suited for PDA testing of micropiles. However, air hammers can also be used successfully provided that the operator is experienced and able to control the drop height with some accuracy. It is noted that testing of cased micropiles with a diameter of less than seven inches may require additional precautions to prevent structural damage during testing.

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