MICROPILES: RECENT ADVANCES AND FUTURE TRENDS

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ABSTRACT: Micropiles have been in use for more than 50 years. Originally, they were conceived as innovative solutions to aid in difficult post-war reconstruction efforts. Over the past 20 years, micropile technology has expanded significantly and has evolved from the concept of low capacity micropile networks to the use of single, high capacity elements. These small elements allow engineers to solve some difficult structural support problems involving high loads and restricted access. Engineers and researchers are now giving renewed attention to micropile networks as technically and economically viable solutions to problems of slope stabilization, lateral loading, and seismic retrofit. This paper explores these recent advances and looks beyond to the newest developments and future advances envisioned for micropiles.

BACKGROUND

Micropile technology has evolved continuously since its introduction by Fernando Lizzi in the 1950s. Over the past 20 years, advances in drilling equipment and techniques have extended the applicability of micropile techniques to infrastructure repair and seismic retrofit projects of increasing complexity. Additionally, research, product development, and a larger pool of experienced contractors have created a more cost-effective tool for civil engineering projects.

Significant equipment innovations have improved the ability to drill through difficult geological conditions in areas of very limited access. These improvements are a result of enhanced mechanical efficiency and drill tooling, including new

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families of duplex drilling systems. Another area of development has been the technological advance in micropile materials. Of recent note is the increased availability and use of hollow core bars (or self drilling bars) as micropiles. Although they carry a much higher material cost, their increased rate of installation has made

FIG. 1. Installation of Micropiles to Underpin Existing Footings

them attractive to contractors by giving them commercial advantages in many situations.

Growth of the micropile industry is made evident by the proliferation of technical publications, trade organizations, and design guideline publications. More importantly, growth is evident in the variety of projects and applications where micropiles have solved some difficult problems. Table 1 provides a summary of some of these accomplishments. There has been a steady increase in the number and scale of micropile projects, as well as in the design capacities and applications of micropiles, as evidenced when comparing the information presented in the table and the previous data provided by Xanthakos, Abramson and Bruce (1994) and the Federal Highway Administration (FHWA, 1997).

This paper reviews the recent advances in the micropile industry and provides a basic understanding of the state of the industry at this time. The paper also explores advances anticipated in the near future for micropile technologies, and points out a few issues that must be addressed to continue technical and commercial development.

TABLE 1. Case 1 Micropile Projects Reported in the Literature

Project and Location (1)	Year (2)	Pile Description (3)	Bond Zone Geology (4)	Design / Test Load (kN) (5)	Difficult Access (6)
Missouri River Bridge, Boonville, MO, Pearlman et al. (1997)	1995	245 mm casing to Rx, 178 mm casing full depth	Karstic limestone	1425 / 2850	No
Williamsburg Bridge, New York, NY, Pearlman et al. (1997)	1997	338 mm seismic casing and 178 mm casing, 2-57 mm and 1 No. 32 mm bars (#18 and #10 English)	Gneiss	535-1335 / 1079-4000	Yes
Mandalay Bay Resort, Las Vegas, NV, Vanderpool et al. (2002)	1998	178 mm – 12.7 mm wall casing, 61 m long	Soft to very stiff clayey sand and clay with caliche layers	2000 / 2670	Yes
Exton Mall, Exton, PA, Cadden et al. (2001)	1999	178 mm N80 casing, 2-#57 (#18 English) bars	Karstic limestone	1335 / 3740	Yes
Luge Track, Lake Placid, NY, Gruner et al. (2001)	1999	1300 piles – Full depth casing Bond length: 4.5 m in till, 1.5 m in rock	Glacial Till and Anorthosite rock	310 / 200 kN	Yes
First Union Commons, Charlotte, NC Traylor and Cadden (2001)	2000	178 mm N80 casing	Granidiorite	1425 / 4000	Yes
Verdun 528 Building, Beirut, Lebanon, Groh (2002)	2000	150 mm hole with 89/74 mm tube and 73/60 mm tube concentric Preloaded Bond length: 7.2 m	Silty sand	680 / 900	Yes
Industrial Building, Emeryville, CA, Weatherby et al. (2001)		168 mm casing and 57 mm (#18 English) Gr. 520 bar Regrouted – 2.76 MPa Bond length: 3 m	Medium stiff to stiff clay, dense sandy gravel	890 kN compression, 667 kN tension / 1780 kN compression	Yes
Caisson Underpinning, Leesburg, VA, Gómez et al. (2003)	2001	178 mm N80 Casing	Diabase	1425 / 2850	Yes
		#43 (#14 English), 1030 mPa (150 ksi) threadbar	Diabase	630 / 1260	Yes
Spallation Neutron Building, Oak Ridge, TN, Walkington et al. (2001)	2001	245 mm casing full depth with 299 mm casing top 6.1 m.	Karstic dolostone	1780 / 3560 175 kN lateral load test and 3560 kN tension test	No
2150 Shattuck Ave., Berkeley, CA, Seismic Retrofit, Aschenbroich (2002)	2001	TITAN 103/52		-- / 2000	Yes
1800 2nd Ave, New York, NY	2002	#89 (#28 English), 520 mPa (75 ksi) threadbar	Granite	210 / 420	No
Hardsfeldt Airport, Atlanta, GA, Wolosick (2003)	2003	300 mm outer casing and 178 mm inner casing	Granite	3560 / 8450	No
Richmond San Rafael Seismic Retrofit, Hadzariga (2002)	2003	476 piles – 219 mm - 22 mm wall casing	Franciscan Formation - greywacke sandstone, shale & limestone	290 to 1140 / 1212	Yes

RESEARCH ADVANCEMENTS

Research into the behavior and design of micropiles has been underway in earnest since about 1993 when the Government of France funded a major micropile research effort entitled Fondations Renforcées Verticalement (FOREVER). The program was partially funded by various parties comprising owners, consultants, contractors, suppliers, research organizations, and universities. The FHWA was also a significant partner, providing funding of almost one million dollars. The results of this effort were published in 2003 and are currently being translated into English.

As a result of the international efforts of FOREVER and the Micropile State of the Practice Survey, also sponsored by FHWA in 1997, an international organization of micropile specialists was established. This group, the International Workshop on Micropiles (IWM), supported the transfer of micropile technology to Japan shortly after the Hanshin earthquake in 1995. At that time, an organization of micropile contractors in Japan, the Japanese Association of High Capacity Micropiles (JAMP), began a research and development program.

The results of the work performed by JAMP were recently presented to the industry in the form of a design and construction guide entitled "Design and Execution Manual for Seismic Retrofitting of Existing Pile Foundations with High Capacity Micropiles" (JAMP, 2002). This document contains a wealth of new information regarding fundamental micropile behavior and design methodologies, particularly as they relate to seismic retrofit applications and group performance of battered piles.

The JAMP manual takes the approach of designing for two types of earthquakes: Level 1 and Level 2. Level 1 earthquakes are those with a high probability of occurrence during the service life of the structure. For these, the pile design must ensure that the stresses developed during the event are within the allowable code service limits.

For the Level 2 earthquake, which corresponds to a strong ground motion with low probability of occurrence during the life of the structure, yielding of the foundation is allowed. The JAMP manual states that the allowable ductility factor and the allowable displacement of the pile shall be set so that the damage to the foundation remains at a degree that "allows the bridge function to be easily restored." Thus, the design is predicated on the piles yielding and not carrying the total earthquake loads imparted.

A major, full-scale research project for military applications in the United States also explored the behavior of networks and groups of micropiles. This work, unpublished to date by Bruce and Weinstein (2002), demonstrated the relative benefit of groups of micropiles as well as the clear advantages of reticulation. A parameter was defined in this study to compare groups of vertical piles and battered piles based on the total length of pile in the group and the measured deflection under load. This parameter was named the Network Stiffness Ratio (NSR). Based on typical NSR values, Bruce and Weinstein (2002) concluded that a group of battered piles was as much as 7 times more effective in lateral loading than a similar group of vertical piles. They also found the performance of the network was highly dependent on the configuration and direction of the piles.

A current research project, Micropile Interstate Cooperative Research on Foundation and Slope Reinforcement (MICROFOR) is under development by the International Center for Ground Improvement (ICGI), Polytechnic University, New York, in conjunction with FHWA and the International Association of Foundation Drilling (ADSC). The main objective of this program is to establish a relevant experimental database of the performance of CASE 2 (FHWA, 1997) micropile network systems with respect to lateral, static, cyclic and seismic loading, and to develop reliable engineering procedures for their use.

A cooperative, private research effort is also presently underway to investigate the mechanism of load transfer at the connection between micropiles and existing footings. This research involves full-scale testing of micropiles installed through reinforced concrete elements, and will likely provide data on grout-to-existing concrete bond strength and mechanisms of connection failure that may be very useful for design of micropile underpinning solutions. Results of this research are expected by mid 2004.

BUILDING CODES AND DESIGN MANUALS

International building codes such as Eurocode, DIN, Nordic Committee on Building Regulation (NKB), and the JAMP manual have become the standards applied to micropile construction as they address the particular materials and construction techniques being used. Within the United States, there are several localities such as Massachusetts and Chicago where standard codes for micropiles are in place. There are also informal guide specifications from trade organizations such as the Deep Foundations Institute (DFI, 2002), and from the FHWA (FHWA, 1997 and 2000). However, the building codes adopted by most localities in the United States do not address micropiles specifically. When applying such codes, inferences are made from specifications for driven piles, cast-in-place concrete piles, and concrete-filled pipes to develop micropile designs. This lack of formal guidance has resulted in highly variable design methodologies. Table 2 compares several code interpretations that have been applied.

Application of these different codes to a typical 178 mm OD, 12.5 mm wall micropile installed into a 203 mm drill hole would result in allowable structural capacities ranging from about 800 to 2100 kN. The authors have witnessed this pile configuration loaded during testing to more than 4000 kN without failure.

TABLE 2. Comparison Between Different Codes Commonly Applied to Micropile Construction (after Richards, 2003)

CODE	Compression					Tension	
				ALLOWABLE STEEL STRESS (MPa)	ALLOWABLE GROUT STRESS (MPa)		
	CASING	BAR	GROUT	fy=550 Mpa	f'c=34.5 Mpa	CASING	BAR
ACI with LF = 1.55	0.45	0.45	0.38	249.1	13.2	0.58	0.58
FHWA Micropiles	0.47	0.47	0.40	259.2	13.8	0.55	0.55
AASHTO Caisson	0.35	0.35	0.30	193.1	10.3	0.35	0.35
AASHTO Driven Unfilled with increase for unlikely damage	0.33	0.33	0.40	113.8	13.8	0.33	0.33
AASHTO Driven Concrete Filled NO increase for unlikely damage	0.25	0.25	0.40	86.2	13.8	0.33	0.33
MASS BLDG CODE	0.40	0.40	0.33	220.6	11.4	0.60	0.60
IBC2000 & BOCA Drilled uncased piles	0.34	0.34	0.25	175.8	8.6	0.34	0.34
IBC2000 Concrete filled pipe piles > 8"	0.35	0.35	0.33	86.9	11.4	0.35	0.35
IBC2000 Concrete filled pipe piles with soils report and load test	0.50	0.50	0.33	124.1	11.4	0.50	0.50
IBC2000 Caisson Piles > 18"	0.35	0.50	0.33	224.1	11.4	0.35	0.50
BOCA Concrete filled pipe piles > 8"	0.35	0.50	0.33	275.8	11.4	0.35	0.50
NYC Building Code - Pipe Pile fy max 248 Mpa	0.35		0.25	86.9	8.6		
NYC Building Code - Caisson fy max 248 Mpa		0.50	0.25	124.1	8.6		
JAMP	0.59	0.59		324.5		0.59	0.59

GEOTECHNICAL CAPACITY

It is commonly assumed that micropiles derive their capacity entirely by friction or adhesion along the interface between the grout and the surrounding soil, and that the tip capacity is negligible. This assumption is justified in most cases because of the relatively small cross sectional area of the pile tip. Additionally, some of the common installation techniques used for micropiles may result in soft soil or debris accumulating at the bottom of the drill hole that may be difficult to remove completely. Therefore, this paper only discusses aspects related to the estimation of the side resistance, or bond strength along the micropile. It must be kept in mind, however, that the tip may contribute significantly to the overall capacity of micropiles with short embedment lengths in hard rock.

As with most other types of deep foundation elements, prediction of the side resistance of micropiles is not an exact science. The bond strength along the micropile depends on the characteristics of the geological media that surround it, the material characteristics of the micropile, and the micropile installation process. In addition, typical design procedures incorrectly assume a uniform distribution of bond stresses along the bonded zone. The bond stress distribution will vary relative to the stiffness of the pile and the geologic medium, as well as the stiffness of the pile-medium interface (Gómez, Cadden and Bruce, 2003).

Table 5.2 of the Micropile Design and Construction Guidelines (FHWA, 2000) provides guidance on the ultimate bond values that can be used for design. It was developed based on a review of published information on drilled shafts, soil nails, and tiebacks. Based on the experience of the authors, the bond values contained in the table must be weighed by the designer based on the geological conditions of the site, and local installation practice and experience. They may be used for preliminary estimates of micropile capacity that will need to be confirmed through load testing where appropriate.

The ultimate bond strength information presented by FHWA (2000) has been condensed into Table 3 only for the purposes of the discussion presented herein. It is observed that the values presented in the table reflect the influence of the type of grouting used for the micropile. For granular soils, the upper bound value of ultimate bond strength increases significantly from Type-A micropiles (gravity-grouted) to Types B through C (pressure grouted). For fine-graded soils, the values shown reflect an influence of pressure grouting that is less pronounced than in granular soils. The ultimate bond values for different rock types are significantly higher than for soils, and are often limited by the unconfined compressive strength of the grout.

TABLE 3. Summary of Nominal Bond Strength Values (Adapted from Table 5-2, FHWA, 2000)

Soil/Rock Description (1)	Range of Grout-to-Ground Bond Strength (kPa)			
	Type A (2)	Type B (3)	Type C (4)	Type D (5)
Silts and Clays	35-120	35-190	50-190	50-190
Sands and Gravels	70-265	70-360	95-360	95-385
Soft to Medium Rock	205-2,070	N/A	N/A	N/A
Hard Rock	1380-4,200 (maximum)	N/A	N/A	N/A

It can be noted, however, that the values listed may not accurately reflect the differences in the achievable bond strength values between different pressure-grouting procedures. For fine grain soils for example, it would be expected that Type-D micropiles (selective post grouting) might achieve significantly larger bond strength values than Type-B micropiles (pressure-grouting through casing only during installation). This is not apparent from the bond strength range given in the table.

The lack of detailed measurements of bond strength for Type-C (global postgrouting) and Type-D micropiles has resulted in the conservative values shown in Table 3. As a result, the application of higher bond values for these piles is not typically implemented in most designs. To resolve this inherent conservatism in the bond strength values, development of a more complete database on achieved bond stresses, or on measured bond strength in micropiles installed in different geological environments is necessary.

Jeon and Kulhawy (2001) compiled and interpreted a database of 21 load tests on pressure-grouted micropiles from ten different sites. Most of the compiled tests did not achieve ultimate geotechnical capacity; however, the average ultimate bond strength along each test micropile was estimated by extrapolating the measured load-displacement relationship using a hyperbolic formulation. This type of extrapolation has been used successfully in the past for piles that are loaded close to failure. For the same types of soils present at each test site, Jeon and Kulhawy (2001) also estimated the corresponding side resistance of drilled shafts using the Alpha Method and Beta Method for cohesive and cohesionless soils, respectively. They observed that, for bonded length-to-diameter (D/B) ratios lower than 100, the interpreted bond strength of the test micropiles was significantly larger than the estimated drilled shaft side resistance. They reported that the ratio of the interpreted micropile bond strength to the estimated drilled shaft side resistance varied between 1.5 to 2.5, and it reached values as high as 6 for cohesionless soils. The micropile bond strength values interpreted from the tests typically ranged between 50 and 130 kPa for cohesive soils, and between 50 and 500 kPa for cohesionless soils.

Jeon and Kulhawy (2001) attributed the large bond strength values in micropiles with D/B lower than 100 to the effect of pressure grouting. This conclusion is supported by the larger bond strength interpreted in micropiles installed in cohesionless soils. However, the authors of this paper believe that this conclusion may also be applicable to micropiles with D/B values higher than 100. The ultimate load of the micropile tests was interpreted from load tests not carried to failure; therefore, it is reasonable to believe that the true ultimate bond stress was not attained throughout the test micropiles, especially when D/B was greater than 100. Consequently, the average bond strength interpreted from the test results may be lower than the ultimate bond strength. Additionally, extrapolation of load tests using the hyperbolic model often gives conservative results, especially for load tests that are not carried close to failure.

It should be noted that FHWA (2000) also promotes the use of the Alpha and Beta Methods for design of micropiles. Thus it is likely that these designs are conservative. Ultimately, the estimated capacity of micropiles must be verified through load testing. Appropriate procedures must be followed to obtain reliable load test data that can be used to verify or to enhance the micropile design.

LOAD TEST INTERPRETATION USING THE ELASTIC RATIO

Load testing in micropiles is typically performed following the requirements described in ASTM D1143 and D3689, for compression and tension load testing (FHWA, 2000). Compression tests are typically recommended for piles that are to be subject to compression loading. For piles that are to be subject to both tension and compression loading, both types of tests should be performed. Tension tests are sometimes performed for piles to be subject to compression loading only as a verification of the bond strength values used for design.

The authors recommend the practice of using a load testing sequence including a series of unload-reloading cycles to assess the load transfer along the micropile and estimate bond stresses (Bruce, Hall and Tripplett, 1995; Gómez, Cadden and Bruce, 2003). Figure 2 shows data from a typical micropile load test that includes several unload-reload cycles. The elastic length, L_e, of the test micropile can be calculated for each cycle using the following equation:

$$L_e = \frac{\partial_e \cdot \Sigma EA}{\Delta P} \tag{1}$$

where:

∂_e = elastic compression

ΣEA = composite elastic modulus of the micropile in compression

ΔP = magnitude of the load decrement

∂_e is usually assumed equal to the rebound measured during unloading at each cycle. The magnitude of the load decrement is determined by subtracting the maintained alignment load from the maximum load in the cycle. The Elastic Ratio (ER) (Bruce, Hall and Tripplett, 1995) is defined as the ratio between the elastic deflection of the pile and the applied load (expressed in thousandths of an inch per kip). The parameters L_e and ER are equivalent, and can be used interchangeably; however, the elastic ratio provides certain advantages especially when comparing behavior between several micropiles by normalizing the results based on the structural properties of the pile.

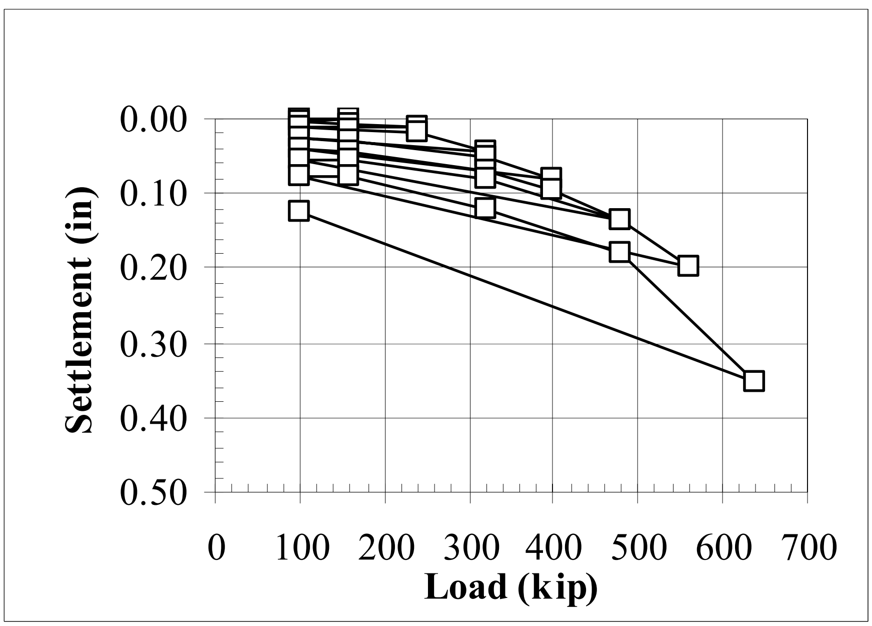

FIG. 2. Results of Micropile Load Test with Several Unload-reload Cycles
(1 in = 25.4 mm, 1 kip = 4.45 kN)

The composite EA value for the micropile is typically calculated as $E_sA_s+E_gA_g$ where E_s and E_g are the modulus of elasticity of the steel and grout, respectively, and

A_s and A_g are the areas of the steel and grout. The calculation of the steel areas and modulus is rather straightforward. However, due to the installation methods, the area of grout outside the steel elements is not clearly defined. Furthermore, E_g is typically overestimated in practice. JAMP (2002) specifies that E_g shall be taken as 2.0×10^4 N/mm^2 for the specified minimum compressive strength grout of 30 N/mm^2. The authors have measured similar and slightly lower values for grout stiffness. Finno (2002) reported values of E_g in the range of 8.9×10^4 N/mm^2.

When evaluating load test results using a composite section modulus, it is imperative that the engineer consider the changes in micropile cross section where appropriate. Changes occurring in areas of load transfer make the evaluation of the test results difficult, since the load transfer to the variable pile cross section is not easily interpreted.

Figure 3 physically illustrates the meaning of L_e. The value of L_e is related to the length of the micropile carrying axial load; therefore, it may be used to estimate average bond stresses acting along a test micropile. It is also used to assess whether an end bearing condition is developing. Given the small cross sectional area of a micropile, development of end bearing may suggest the onset of micropile failure in some cases (Bruce, 1993).

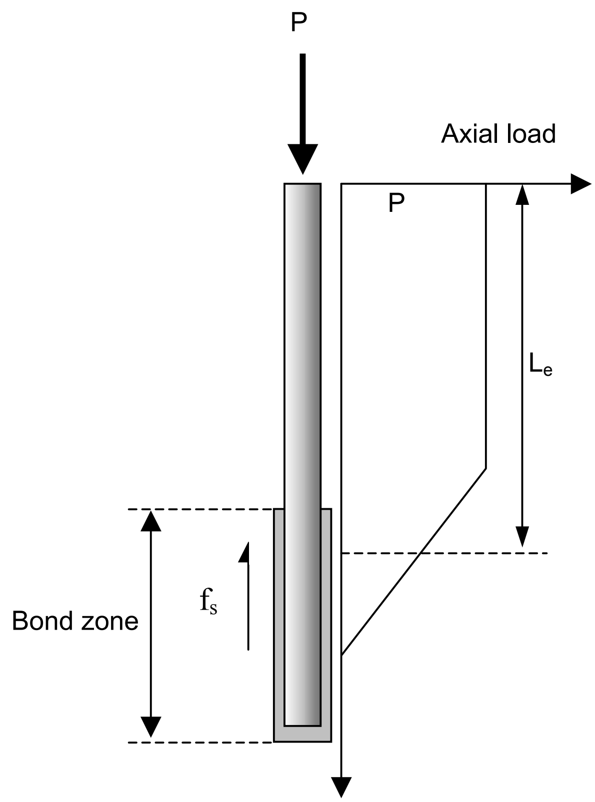

FIG. 3. Illustration of the Elastic Length, L_e, Concept

When interpreting load tests, care should be exercised in estimating the values of L_e or ER. Upon each unloading step during the load test, the micropile may retain significant locked-in elastic deformations. Residual elastic deformation of the pile is not accounted for when calculating the elastic length or elastic ratio, as it cannot be discerned from the deflection data measured at the head of the pile. Results of a load test on an instrumented micropile in rock (Gómez, Cadden and Bruce, 2003) indicated the locked-in elastic deformations were approximately 50 percent of the permanent deflection after unloading (see Figure 4). This means that in that case the actual elastic length was approximately 30 percent larger than estimated using Equation (1). Although the tip of the pile was not deflecting, full rebound was not being realized at the pile head.

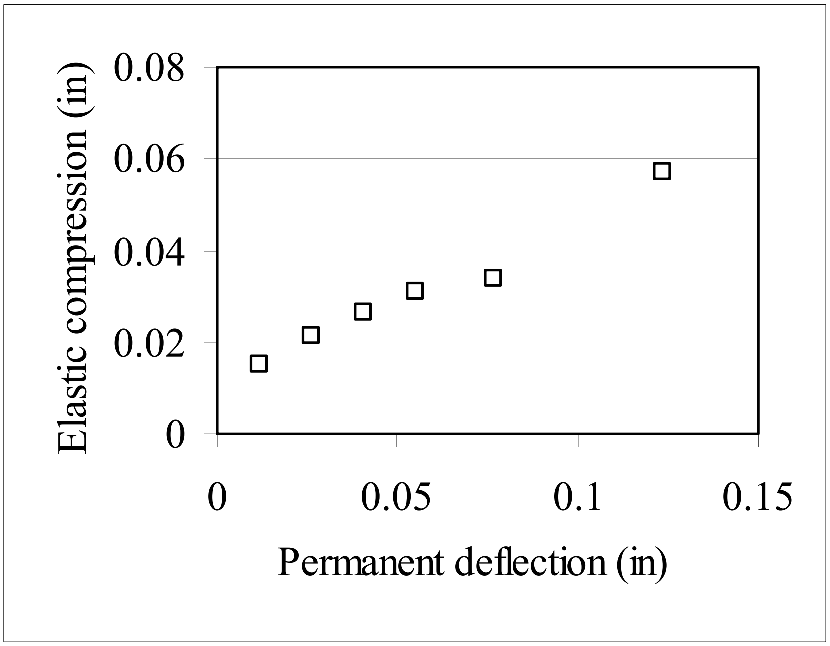

FIG. 4. Comparison Between Elastic Compression and Permanent Deflection at the Head of a Test Micropile
(after Gómez, Cadden and Bruce, 2003)
(1 in = 25.4 mm)

DEBONDING OF MICROPILES

The term "debonding" is frequently used indiscriminately to refer to a variety of phenomena that can take place during monotonic or cyclic loading of micropiles. In a traditional sense, "debonding" is used to imply physical separation of the two sides of an interface. This phenomenon would be accompanied by full loss of interface shear strength.

The utilization of this term for micropile-to-ground interfaces may be somewhat misleading. When a micropile is subject to axial loading, relative interface displacement in the direction of the load may take place along the reinforcement-to-grout, or along the grout-to-ground interfaces. This process is not necessarily accompanied by full loss of interface shear strength, as the two materials at each interface may remain in contact. However, the nonlinear response of the grout-ground interface to relative displacement, and possible post-peak reduction of bond strength, may induce a non-uniform bond stress distribution along the micropile that progressively shifts deeper along the pile under increasing or cyclic loads.

Finno (2002) reported “debonding” at the interface between the steel casing and surrounding grout during load tests on short micropiles embedded in dolomite rock. Based on strain gauge measurements, the interpreted bond strength values along the grout-casing interface ranged between 3,200 and 5,400 kPa in two of the test piles. Failure along the grout-casing interface was confirmed by examination of some exhumed micropiles, which revealed intact rock-grout interfaces.

Evidence by Gómez, Cadden, and Bruce (2003) suggests that post-peak reduction of grout-to-ground bond strength may take place in micropiles. Figure 5 summarizes the interpretation of one load test in terms of mobilized bond stress versus relative grout-ground interface displacement. From these results, they concluded that a significant post-peak reduction of bond strength occurred between the micropile and soil, and between the micropile and weathered diabase rock. However, this is based on the results of one load test, and should not be extrapolated to other cases. Post-peak reduction of bond strength may hypothetically induce brittle response of highly loaded micropiles with short embedment into rock, which should be accounted for when establishing acceptable factors of safety. Further research needs to be performed on this issue.

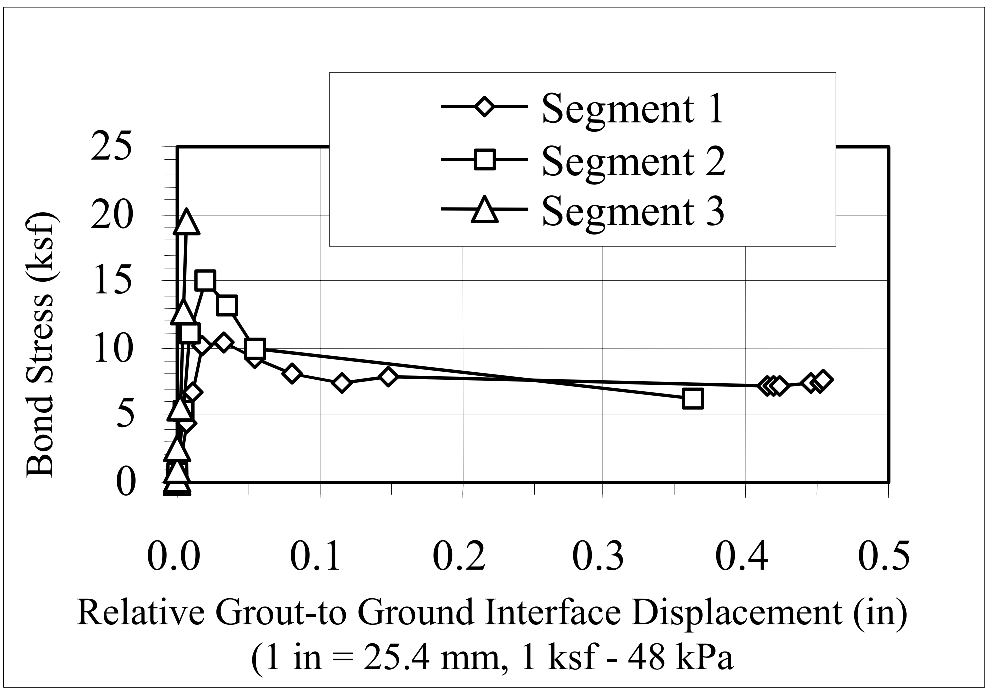

FIG. 5. Mobilized Bond Stresses Along a Micropile (after Gómez, Cadden, and Bruce 2003)

Several others have associated “debonding” of micropiles in rock due to repeated loading. This concept of "debonding" implies that, upon several load repetitions, there is a reduction of the bond strength along the micropile-rock interface. For micropiles with long embedment into rock, this would result in a progressively deeper transfer of the load along the micropile. In reality, this phenomenon may be associated with the post-peak reduction in bond strength discussed previously (Gómez, Cadden and Bruce, 2003). The increase in apparent elastic length due to repeated loading may be an important issue in micropile applications with strict deflection tolerances. Where cyclic loading is expected at a level that may result in post-peak reduction of bond strengths, the apparent elastic length may increase over the useful life of the foundation. Further investigation is also needed on this matter.

Finno (2002) also reported differing strains between the steel casing and the grout inside. He concluded that “debonding” between the grout and casing may have occurred even at relatively small test loads. It must be kept in mind, however, that Finno's load tests were performed on relatively short micropiles. End effects, caused by direct application of the load at the top of the pile against the casing and not the grout, may have induced differential strains between the grout and the casing at the pile top. It is possible that for longer micropiles, these strains would have become similar at larger depths. Data from instrumented load tests on micropiles in rock, having embedment lengths larger than those used by Finno (2002), would be needed to address this issue.

BUCKLING OF MICROPILES

Because micropiles are frequently installed to or into hard rock, their capacity is frequently dictated by the structural strength of the element, rather than by the geotechnical bond between the micropile grout and surrounding soils. Therefore, it is reasonable to believe that, where soft soils or voids overly the bearing strata, buckling may potentially control the load-carrying capacity of a micropile. To address concerns regarding buckling of steel piles driven to rock, Bjerrum (1957) published results of buckling tests and related them to the methods available at the time. He presented results of load tests performed on piles with a variety of sections, including bars, rails, and H sections. He concluded that even very soft soils could provide enough lateral restraint to prevent buckling of most pile sections.

The issue of buckling of micropiles has been the subject of attention of several researchers: Mascardi (1970, 1982); and Gouvenot (1975). Their results seem to support Bjerrum’s conclusion that buckling is likely to occur only in soils with very poor mechanical properties such as peat and soft clay. Experimental research carried out by CALTRANS (Brittsan and Speer, 1993) on high capacity micropiles installed through a very thick (33 m) deposit of San Francisco Bay Mud, and case histories of rock-socketed micropiles in karst (Cadden, Bruce and Ciampitti, 2001; Gómez, Cadden and Webster, 2004) have further shown that micropiles can be successfully applied in a variety of subsurface environments.

It cannot be inferred, however, that buckling in micropiles will never occur. Buckling of piles is a complex soil-pile interaction problem that involves the pile section and elastic properties, soil strength and stiffness, and the eccentricity of the applied load.

Equation 2 can be used to estimate the critical load, P_{cr}, of a pile (Bjerrum, 1957):

$$P_{cr} = \frac{\pi^2 EI}{l^2} + \frac{E_s l^2}{\pi^2} \qquad (2)$$

where:

E = modulus of elasticity of the pile material [Force/Area]
I = minimum moment of inertia of the pile [Length^4]
l = "unsupported" length of the pile [Length]
E_s = modulus of lateral reaction of the soil [Force/Area], i.e., slope of p-y diagram (not to be confused with modulus of subgrade reaction)

The term "unsupported" refers to the portion of the pile that is only subject to the lateral restraint provided by the soil. The first term of Equation (2) corresponds to Euler's equation for buckling in columns. The second term reflects the contribution of the lateral restraint provided by the soil. Theoretically, buckling should only be a concern for design of a micropile if the compression load that produces yielding of the pile material exceeds the value of P_{cr}.

Cadden and Gómez (2002) re-arranged Equation (2) as follows:

$$E_s \leq \frac{1}{\left[\left(4 \cdot \frac{I}{A^2}\right) \cdot \left(\frac{E}{f_y{}^2}\right)\right]} \tag{3}$$

where:

A = cross-sectional area of the pile [Length^2]
f_y = yield stress of the pile material [Force/Area]

The first of the two terms inside the brackets represents the geometric properties of the pile, while the second term represents its material properties. The combination of these two terms is referred to as the *pile factor* and is given in units of [stress^{-1}]. Table 4 lists some of the sections and steel types (solid and hollow core bars and casing) often used for micropile work in the United States. Pile factors are also listed for each section.

The value of E_s calculated using Equation (3) can be defined as the critical or limiting lateral reaction modulus. If the critical E_s value is less than the actual soil E_s, then the geotechnical and structural axial strength of the pile will control the pile capacity. If the critical E_s is greater than the actual soil stiffness, buckling should be evaluated further.

Equation (3) is represented graphically in Figure 6. Any given combination of micropile and soil can be represented by a point in the diagram. An undamaged pile represented by a point located to the right of the line will fail under compression before it buckles. A pile represented by a point to the left of the line may buckle before it fails in compression. Figure 6 thus becomes a tool for checking whether buckling of a given pile section should be explored further for a given site.

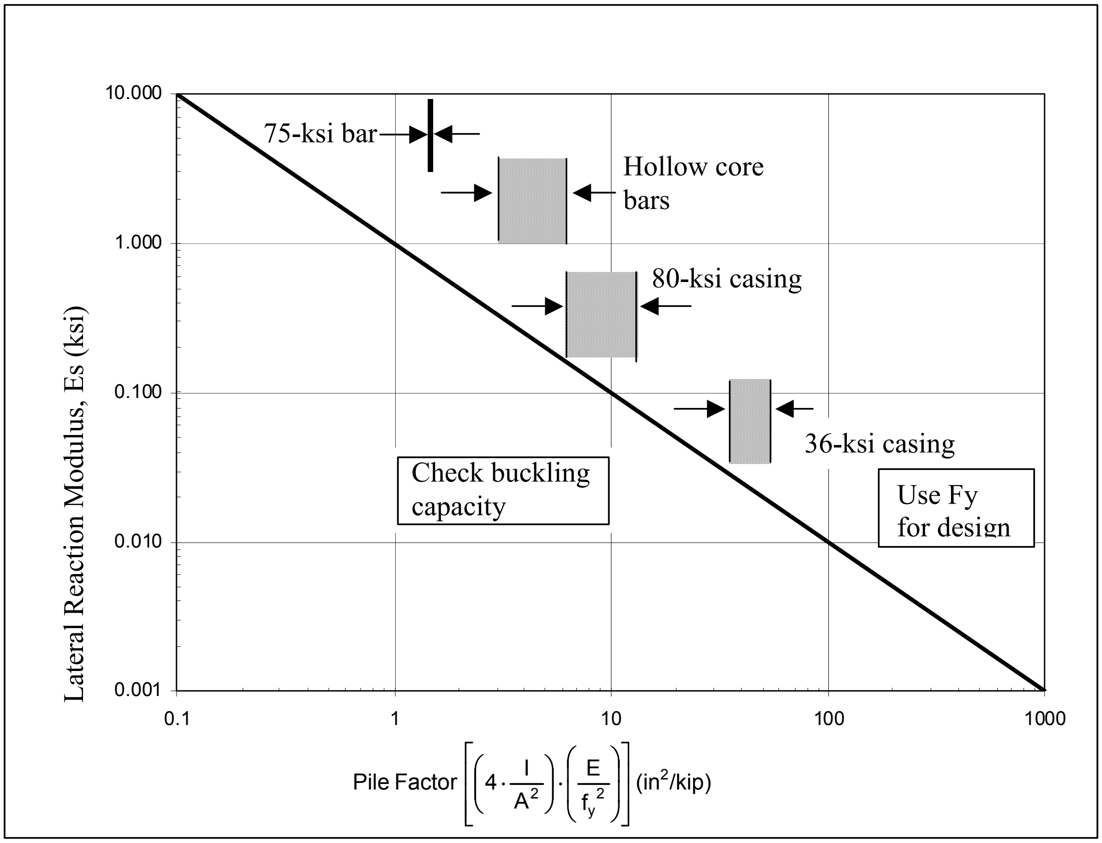

FIG. 6. Chart for Approximate Buckling Evaluation of Micropiles Subject to Centered Loads (Cadden and Gómez, 2002)

Also represented in the figure is the range of pile factor values for each micropile type in Table 4. It can be seen that, according to the theoretical background described previously, buckling does not control the design of micropiles except for very soft soils.

Figure 6 may be used for an approximate determination of whether or not buckling may occur in a micropile. If, according to Figure 6, a particular combination of soil and micropile type may be susceptible to buckling, then the minimum critical load can be estimated using numerical procedures. This chart assumes that the pile has constant cross-sectional properties, and there are no horizontal loads or moments applied to the top of the pile. In addition, the soil is assumed to have a constant value of lateral reaction modulus throughout the length of the pile, behaving as a non-yielding, linear elastic material. Finally, it should be noted that this procedure does not take into account the contribution of the grout in the micropile element.

TABLE 4. Typical Properties and Pile Factors of Micropile Reinforcement in the USA (Cadden and Gómez, 2002)

Casing F_y= 80 ksi				
	5½ -inch casing	7-inch casing		9⅝-inch casing
Casing OD, in	5.5	7	7	9.625
Wall thickness, in	0.36	0.5	0.73	0.47
Area (A), in^2	5.83	10.17	14.38	13.58
Moment of Inertia (I), in^4	19.3	54.1	71.6	142.6
I/A^2	0.57	0.52	0.35	0.77
Pile factor (PF), in^2/kip	10.3	9.5	6.3	14
Yield strength, kip	466	814	1150	1086

Casing F_y= 36 ksi				
	5½ -inch casing	6⅝–inch casing	8-inch casing	10¾ -inch casing
Casing OD, in	5.56	6.625	8.00	10.75
Wall thickness, in	0.5	0.5	0.5	0.63
Area (A), in^2	7.95	9.62	11.82	19.91
Moment of Inertia (I), in^4	25.7	45.4	83.4	256.2
I/A^2	0.41	0.49	0.6	0.65
Pile factor (PF), in^2/kip	36.4	43.9	53.5	57.8
Yield strength, kip	286	346	425	717

Bar F_y=75 ksi						
	#10 Bar	#11 Bar	#14 Bar	#18 Bar	#20 Bar	#28 Bar
Bar diameter, in	1.25	1.375	1.75	2.25	2.5	3.5
Area (A), in^2	1.27	1.56	2.25	4	4.91	9.61
Moment of Inertia (I), in^4	0.13	0.19	0.40	1.27	1.92	7.35
I/A^2	0.08	0.08	0.08	0.08	0.08	0.08
Pile factor (PF), in^2/kip	1.64	1.64	1.64	1.64	1.64	1.64
Yield strength, kip	92	133	180	236	368	722

Injection Bore Piles F_y=75 ksi (CON-TECH Systems)						
Bar diameter, O.D./I.D. (mm)	30/16	32/20	40/20	52/26	73/53	103/51
Bar diameter, O.D./I.D. (in)	1.18/0.63	1.26/0.79	1.57/0.79	2.04/1.02	2.87/2.09	4.06/2.00
Area (A), in^2	0.59	0.69	1.00	2.08	2.53	8.53
Moment of Inertia (I), in^4	0.06	0.09	0.16	0.61	1.88	10.08
I/A^2	0.17	0.19	0.16	0.14	0.29	0.14
Pile factor (PF), in^2/kip	3.39	3.96	3.23	2.92	6.07	2.86
Yield strength, kip	40.5	47.2	96.7	160.8	218.1	612.8

The presence of grout in a micropile has several effects on the potential for buckling. The grout, whether located within the casing or included as the bond material around the perimeter of the steel, will add to the structural stiffness of the micropile. Furthermore, when the grout is used as a drilling fluid, or where it is pressurized as part of the pile installation process, it may significantly increase the stiffness and strength of the surrounding soils. The contribution of the grout to the buckling capacity may be particularly significant for bar and injection bore micropiles where the effect of the increased gross area may be significant when compared to the limited structural section.

Of particular concern in the evaluation of lateral and buckling capacities of micropiles is the location of threaded connections relative to the shear and moment distribution in the pile. Bending test results reported for 178 mm threaded connections by L.B. Foster and Malcolm Drilling Company indicated failure loads of about 1180 kN-m and 1360 kN-m (Stress Engineering Services, 1995, 2003).

Further discussion of the formulation of this analysis, common questions such as the effect of voids penetrated by micropiles, effect of not achieving complete grout return to fill an annulus space, as well as several example calculations and case histories, can be found in the complete White Paper developed by the ADSC Micropile Committee and available through the ADSC Technical Library (Cadden and Gómez, 2002).

MICROPILE-SUPERSTRUCTURE CONNECTIONS

Detailing of the connection between the micropile and the structure can be a complex task. Micropiles for new construction are embedded into pile caps, which can be designed based on standard concrete construction codes. However, the comparatively large capacity of micropiles given their small cross-section may induce stress levels that are not typical in design of pile caps for other applications. For existing construction, micropiles may be installed through existing footings or pile caps, thus creating unusual concentrated loads in the footing that need to be checked.

In some occasions, a new micropile cap is attached to the existing footing. In this case, connection of the new to the old concrete needs to be achieved through dowels or other means (see Figures 7 and 8). Doweled connections need to be carefully designed and checked against shear through the old-to-new concrete contact. The amount and size of dowels may be estimated using the procedures contained in Section 11.7 for Shear Friction of the American Concrete Institute (ACI) Building Code Requirements for Reinforced Concrete, ACI 318-89 and subsequent revisions. This design methodology allows for the contribution of the old-to-new concrete bond plus the contribution of the dowels to the overall shear capacity at the connection. It must be noted that the procedures under ACI 318 correspond to load-factored design; therefore, all loads used must be factored appropriately. Once the design following ACI 318 is complete, it must be checked against the recommendations of the manufacturer of the dowels or resin. Manufacturers typically provide tables of shear and tensile capacity of the dowel bars based on dowel embedment depths, and also provide correction factors for spacing, distance to edge, etc., that must be checked.

For certain applications, it is possible to provide a suitable connection using pre-manufactured brackets between the micropile and the bottom of the footing (see Figure 9). These brackets may not be developed specifically for grouted micropiles, but can be adapted for this purpose. Although this type of connection is relatively inexpensive, it may be limited by eccentric loading on the micropile and by the capacity of the bracket. Sometimes it is not possible to provide a permanent bracket connection, and some other connection types must be used.

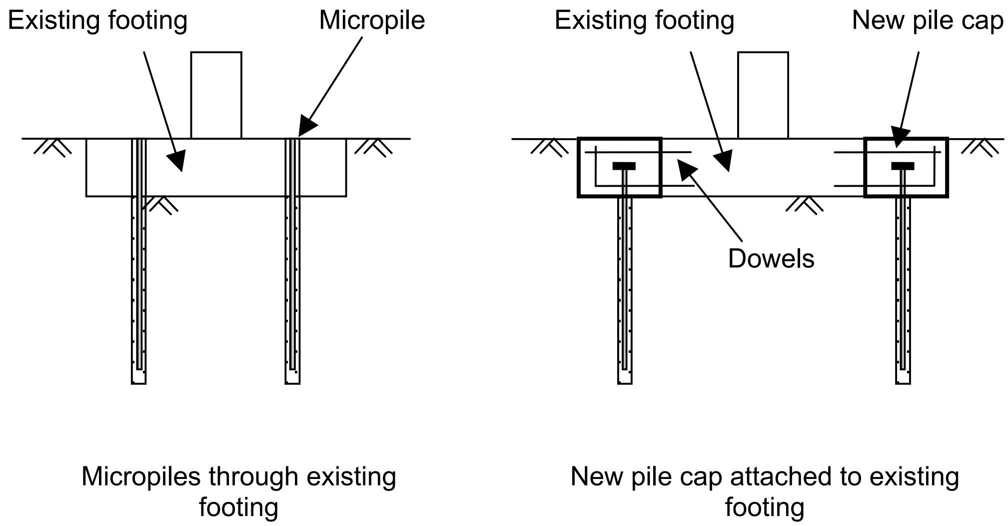

FIG. 7. Two Common Options for Connecting Micropiles to an Existing Structure

One of the main issues when installing micropiles through an existing footing is the grout-to-concrete strength available at the connection. Designers tend to use allowable bond values ranging between 700 to 1400 kPa for plain grout-to-concrete interfaces, and higher values, maybe up to 2100 kPa, when shear rings or grooving of the existing concrete is included in the connection. The authors are unaware of a definitive study on the proper design values or the benefits of these enhanced roughening or grouting methods. Several contractors throughout the industry take extra measures such as roughening of the side walls with gouging tools, use of non shrink grouts for the top connection fill, and bonding agents on the concrete.

An important difficulty with retrofitting a structure is the limited knowledge that is often available about the existing structure, including strength of the concrete, continuity of the structure with the foundation, and amount and location of the reinforcing steel. Although non-destructive testing techniques, such as Ground Penetrating Radar (GPR) and x-rays, may be used to assess the reinforcement of the structure, they are not frequently used given the schedule constraints typical of underpinning work.

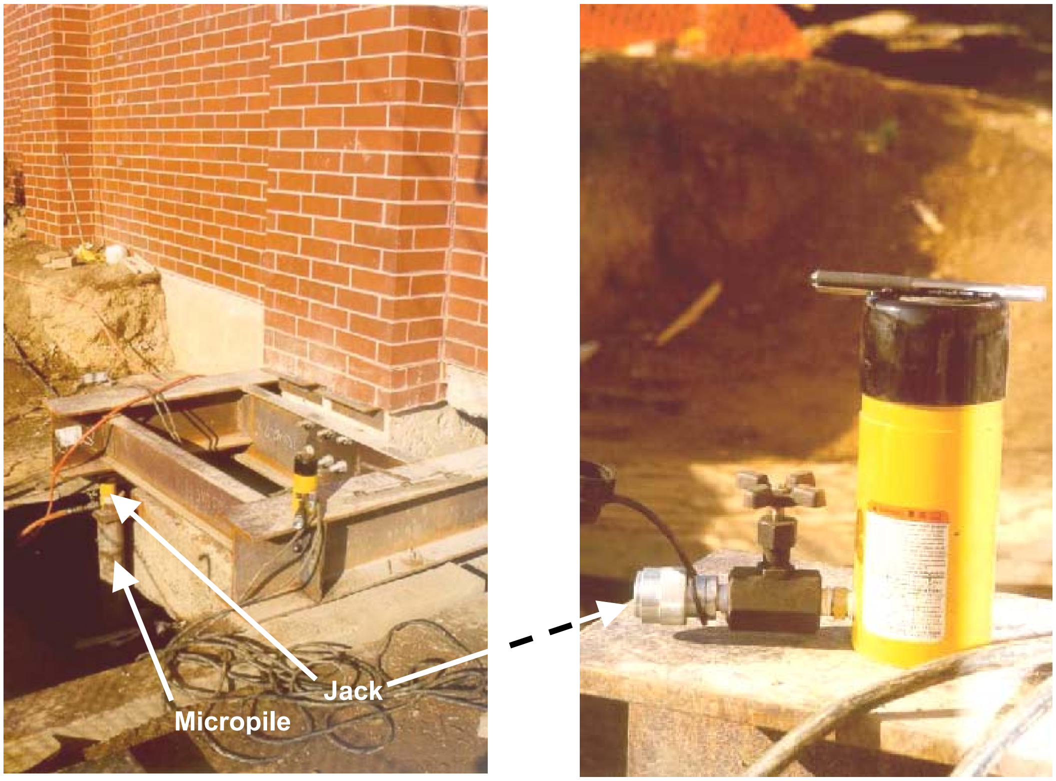

FIG. 8. Underpinning Using Preloaded Micropiles
(Note micropile-connection through steel beams bolted into the existing column pedestal.
Once piles are preloaded, the load is locked off and jacks removed.
The entire connection is cast in concrete.)

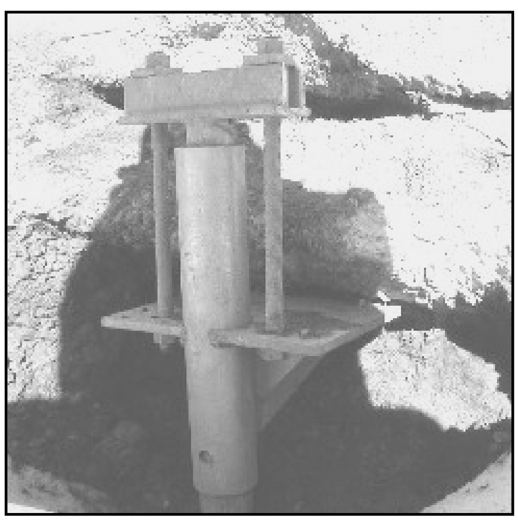

FIG. 9. Pre-manufactured Bracket that can
be used for Micropile Underpinning Applications

It is very common to observe that, for an existing structure, micropiles cannot be installed concentrically with the center of load application due to space restrictions. It is important to account for the eccentricity-induced moments in the design, when applicable. Figure 10 illustrates two cases of eccentric micropiles. In Case A, the eccentricity of the load with respect to the micropiles needs to be accounted for in design, as it may induce loads in the near-column micropiles that may be significantly larger than the average micropile load. In the situation shown in Case A, it might be possible to count on the stiffness of the overall structure to absorb the eccentric moment. If this was done, significant reductions in underpinning costs might be realized in certain projects. However, the decision of whether to count on the existing structure to absorb eccentric moments must be made by the structural engineer who, in many cases, would not be willing to do so.

Case B in Figure 10 corresponds to the underpinning of an existing basement wall. Here, it is unlikely that eccentricity of the micropiles will result in flexure of the micropiles. The wall is restrained from rotating by the lateral earth pressures and interior floor slabs. Even if some initial bending of the pile top occurs, it cannot result in uncontrolled rotation at the wall base. Again, there may be some additional loading imposed on the structural elements connected to the wall, and the structural engineer may be hesitant to accept this. The authors have experience in projects

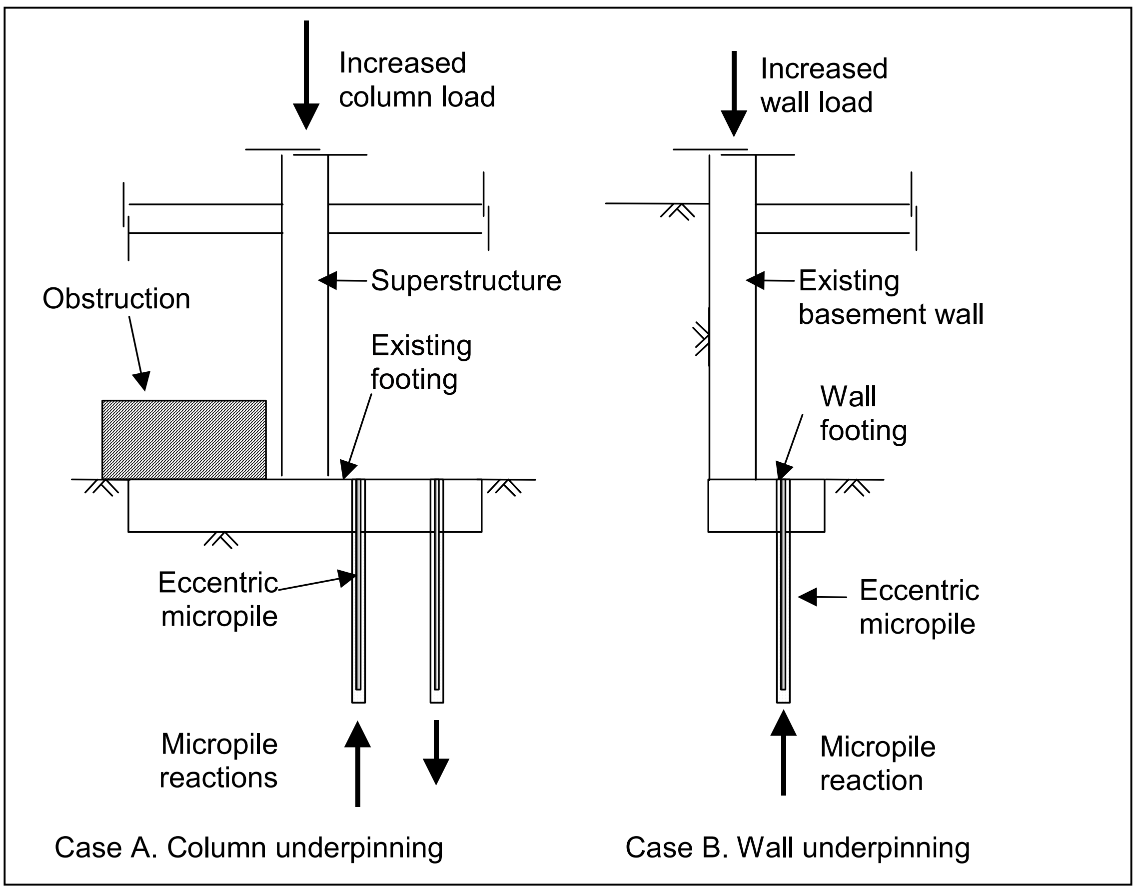

FIG. 10. Effect of Eccentricity on Micropile Reactions

where concerns about micropile eccentricity have been excessively magnified, thus impacting significantly the cost of the underpinning.

Another frequent occurrence is the use of micropiles to underpin existing caissons. Ideally, the micropile group should be concentric with the caisson. However, this is often impossible in existing construction and the micropiles are installed eccentrically as shown in Figure 11. If careful soil-structure interaction analyses are made of this situation, it could be concluded that the eccentricity of the micropiles may have no impact on the performance of the underpinning. The caisson would have little opportunity to rotate within the geological medium and, therefore, it is unlikely that any bending of the micropiles will occur. The eccentric moment will be absorbed by a redistribution of the lateral pressure on the caisson. The relative importance of the eccentricity will increase for decreasing caisson length.

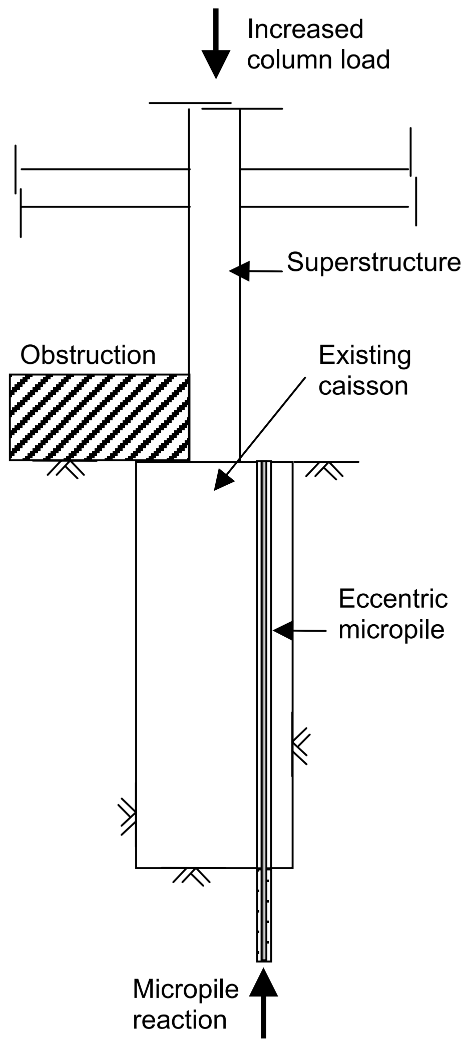

FIG. 11. Underpinning of Caisson with Eccentric Micropiles

ADVANCES IN CONSTRUCTION PRACTICE

It becomes increasingly apparent at conferences and seminars dealing with micropiles that equipment manufacturers and material suppliers are true and invaluable "technology partners," sharing equal credit for technological advances with the specialty contractors and the consultants.

This has not always been the case, but it is now fair to say that the increasingly onerous demands of application and understanding have required all parties to create and share knowledge for the benefit of the industry and for their respective clients.

Drill rig manufacturers continue to design new machines that combine maneuverability and flexibility of use with surprising power and user-friendliness. Many of these rigs are easily equipped with automated MWD (Measurement While Drilling) packages, which can provide detailed records of the drilling progress of each hole, and so, by interpretation, frequent and accurate representations of the surface conditions across the entire site. It may also be noted that every rig is a candidate for manual MWD: an experienced and knowledgeable driller or field technician can easily enhance substantially the value of a routine driller's log by taking periodic readings of the drilling performance parameters such as rate of penetration, down pressure, air pressure, rotation speed and torque.

New and/or modified overburden drilling systems continue to evolve - the goals being to provide a cost effective, reliable penetration in any subsurface condition, from a variety of "carriers," and by the understandable variety in operator skills. To date the considerable technical, economic and environmental benefits of rotary-sonic drilling (Bruce, 2003) have not been exploited in the micropile industry. As this technology becomes more readily available, the authors believe that, in appropriate conditions, rotary-sonic will be applied to micropile construction.

Specialists in the use of cement additives and admixtures are also coming into favor. Their expertise is being called upon to help design grouts of superior fluid and set properties. Care should be taken, however, to ensure that certain of the rheological properties, which can be imparted by such components, are not detrimental to the in situ performance of the grout. For example, an extremely thixotropic grout may not be able to penetrate into fissures or pore spaces or may not have good "adhesive" properties to the steel reinforcement. All such "innovative" mix designs must be fully evaluated prior to routine use.

Manufacturers of threaded casing are also increasingly valuable sources of technical information, especially regarding the behavior of their threaded sections when subjected to tensile or bending stress. Likewise the suppliers of "self drilling bars" have conducted extensive in-house research to show prospective customers that fears of corrosion, for example, can be allayed in practice, given the special conditions of the installation process and the configuration of the bar deformations.

QUALITY CONTROL

Quality control of micropile components is usually very simple and reliable. The reinforcement materials can be obtained with quality certification from most providers. The grout, if prepared consistently using colloidal mixers to the right proportions as verified by Baroid Mud Balance tests, will almost always yield the desired strength. However, there may be questions regarding the suitability of the installation and quality control procedures used for a given project.

The installation of micropiles is most similar to auger cast piles or continuous flight auger piles where limited data are retrieved during installation. Due to the equipment being used, the discernment of changes in geologic conditions is difficult except when transitioning between strata with clearly contrasting characteristics, such as from soft soils to dense sand and gravels, or from soil to rock. Installation records for most micropiles do not allow a numerical characterization of the materials encountered (such as blow count logs for driven piles). Furthermore, unlike drilled shafts, there is also limited ability to confirm the conditions of the sidewalls prior to grout placement. Lacking suitable logs for quality control means that the responsibility resides on experienced inspection of the installation process. This must include observation of the drilling, preparation of the hole, installation of the reinforcing, and placement and pressurization of the grout.

In view of the above, post-installation verification is a convenient tool for quality control in micropiles. Statnamic testing of micropiles has been performed on micropiles in sands (Ichimura et al., 1998), who obtained reasonable results when estimating the soil spring stiffness from the initial gradient of the interpreted static resistance-displacement curve using the unloading point method. However, Ichimura observed that difficulties may exist in the interpretation of this type of testing on micropiles in view of length/diameter ratio and the time difference between maximum load and maximum displacement. He also reported that, regardless of this difficulty, it was possible to estimate the first limit load.

Pile Driving Analyzer (PDA) testing has also been performed on micropiles founded in rock (Gómez, Cadden and Webster, 2004). Comparison between results of PDA tests and three static load tests showed excellent correlation. In the case analyzed, there was a significant variability in the geotechnical capacity of production micropiles, which was likely induced by poor quality control during installation of the micropiles in a difficult karstic environment. PDA equipment allowed testing of a significant number of micropiles in a short period, and provided valuable information for adjustment of the design.

Significant variation in micropile capacity may be expected under many conditions (Vanderpool et al., 2002; Hirany and Kulhawy, 2002). This further emphasizes the need for efficient means of testing production elements and for suitable specifications that require periodic testing of foundation elements as construction proceeds.

Further research is needed in the area of post-installation verification of micropiles. The methods discussed previously seem to provide fast and relatively accurate estimations of micropile capacity. Other methods may exist or may be developed that can also be useful for this application.

FUTURE ADVANCEMENTS UNDERWAY/ENVISIONED

The commercial advancement of micropiles has been an impetus to the material suppliers developing new pile materials, drilling methods and monitoring equipment. Advancements such as improved duplex drilling tools, rotary-sonic drilling, and hollow core bars, appear to be commercially viable. We are no longer seeing micropiles as having limited application to restricted access sites. We are now seeing very high capacity micropiles competing successfully with more traditional driven pile and drilled shaft systems. When comparisons are made based on cost per kN, micropiles are a very attractive solution for even new construction on open sites.

New contractors are obtaining the tooling to construct micropiles. As such, we will continue to see this commercial acceptance expand. We must also respond to this with educated designers and owners capable of evaluating proposals and providing adequate oversight to ensure successful projects. This will include an expanded role of QA/QC testing to verify production pile capacities.

Research will also continue to find definable ways to design reticulated micropile groups which will not only enhance the net benefit of the group of piles, but will also be economically viable. This will result in an explosion in the use of micropiles for resistance of lateral loading and retrofitting existing structures to resist seismic forces.

CONCLUSIONS

Micropile technology is gaining rapid acceptance. Although significant advances continue to come as a product of contractor innovation, significant efforts have been expended and are underway by different entities to accommodate the ever changing applications of micropiles. Advances have been realized in the form of increased capacities, and introduction of new materials and design methods for integrating micropiles into existing projects.

As demonstrated by FOREVER, JAMP, and numerous other commercial projects completed to date, micropiles provide viable solutions to difficult problems. Although sometimes designers wish to accommodate every conceivable loading condition, it is usually more appropriate to follow design procedures that balance safety and economic concerns.

Further research is necessary on several aspects of micropile behavior. The success of these efforts will be fundamental for future development of micropile technology and to create new areas of application.

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We would also like to extend our thanks to our respective employers who have supported our efforts to expand our knowledge and develop this paper. These include: Schnabel Engineering, Inc., West Chester, PA; Geosystems, LLP, Venetia, PA; and DBM Contractors, Federal Way, WA. Particular thanks go to Darrell Wilder, Carlos Englert and Helen Foreman who provided review and valuable comments throughout development of this paper.

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