Buckling of Micropiles

A review of historic research and recent experiences

By:

Allen Cadden, P.E., and Jesús Gómez, Ph.D.

Schnabel Engineering Associates

West Chester, PA

Schnabel Engineering Associates, Inc.

MAY, 2002

Abstract

The use of micropiles has changed significantly since their inception in the 1950's. Early applications consisted of lightly loaded groups of elements intended to enclose and reinforce an unstable soil mass. Following recent developments in the United States, the micropile concept has evolved into high-capacity load bearing elements. Often, these elements are installed through relatively soft materials to be founded in hard rock, and routinely reach lengths over 50 feet. Presently, micropiles are designed for ultimate load carrying capacities that may exceed 400 tons for seven-inch diameter elements. Buckling of such slender elements is a common concern among designers and structural engineers. This paper contains a review of relevant historic research extrapolated to new micropile uses, as well as case histories of recent experiences obtained directly from high-capacity micropile applications. It is shown that although buckling of micropiles may not generally occur, it should be considered for design of micropiles installed through very soft soils or in karstic terrain.

Introduction

Micropile technology has evolved significantly since its inception in the 1950's, when Dr. Fernando Lizzi developed the concept of lightly loaded micropile groups for underpinning of historic buildings. Micropile technology developed slowly in Europe over the next 20 or 30 years, until publication of case histories of successful applications induced a rapid growth in the application of micropiles in the United States. This growth, however, evolved more towards the use of heavily reinforced micropiles with high axial load-carrying capacities.

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. This is the case for micropiles installed through very soft sedimentary deposits or karstic formations.

Buckling of piles has been a long-standing issue that has been addressed by several investigators (Mandel 1936, Cummings 1938, Glick 1948, Bjerrum 1957, Davisson 1963). Development of mathematical equations fully representing the problem of buckling of a pile in an elastic medium was one of the earliest developments. However, closed-form solutions for design were only available for constant-section piles in linear-elastic and homogeneous media. To address concerns regarding buckling under static loads 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 also been subject to the attention of several researchers (Mascardi 1970, 1982; 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. Recent experimental research carried out by CALTRANS on high capacity micropiles installed through a very thick (110 ft) deposit of San Francisco Bay Mud, and case histories of rock-socketed micropiles through karst, 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. In this paper, the authors attempt to present a summarized description of the procedures available to study the issue of buckling of micropiles, and recommendations to perform buckling analyses of micropiles in a systematic manner. Analyses are presented for hypothetical cases of micropiles installed through soft soils or in karstic terrain. In addition, some case histories are provided as reference.

Background

The question of buckling in columns most often refers to the determination of the allowable compression load for a given unsupported length. Initially, the question of buckling was limited to determining the maximum compression load at which a structural element will return to a straight shape after being subject to an initial deflection. The mathematician Leonhard Euler solved this question in the 18th century with a basic equation contained in most strength of materials and structural design manuals.

The buckling problem was later expanded to the determination of the deflection and stresses produced by a given load and eccentricity. Euler's solution is generally applied to very slender columns, in which buckling is typically not allowed at all. In shorter columns, buckling-induced deflection is generally allowed as long as an appropriate factor of safety against failure exists. It is important to note that, because the principle of superposition is not applicable to buckling, the allowable compression load of a column is defined as the load causing buckling divided by an appropriate factor of safety. It should not be calculated as the load that produces the maximum allowable stress in the column.

Bjerrum (1957) referred to the danger of buckling of piles in soft clay as a "ghost which now and then appears in technical discussions." This "ghost" has seen resurgence with the evolution of high-capacity, Case I micropiles. Case I micropiles are defined by the Federal Highway Administration (FHWA 2000) as load-carrying elements intended for direct load bearing, as opposed to Case II micropiles intended to create a composite network effect within the soils. The argument has been made mathematically and through experimentation that the danger of buckling is reduced by the lateral restraint provided by the surrounding soils and can be ignored for most soil conditions. Bjerrum also stated that the buckling problem is "not so simple that it can be dismissed by a reference". Where a load-carrying element penetrates very soft soils, voids, or materials such as peat, the potential for buckling should be examined further.

Bjerrum (1957) used the following theoretical equation for the determination of the critical load, P_{cr}, of a pile:

$$P_{cr} = \frac{\pi^2 EI}{l^2} + \frac{E_s l^2}{\pi^2} \qquad (1)$$

where:

E = modulus of elasticity of the pile material,

I = minimum moment of inertia of the pile,

l = "unsupported" length of the pile, and

E_s = modulus of lateral reaction of the soil.

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 (1) corresponds to Euler's equation for buckling in columns. The second term reflects the contribution of the lateral restraint provided by the soil.

Equation (1) assumes that the soil follows a linear load-displacement relationship and that the lateral reaction modulus is constant with depth, which is not true for most soft soils. It does not address cases in which the micropile is not initially straight, the micropile section is variable, the load is eccentric, the soil type varies with depth, etc.

The critical length, l_o, which produces the minimum value of P_{cr}, can be obtained by differentiating equation (1), and is defined by:

$$l_o = \pi \cdot \sqrt[4]{\frac{EI}{E_s}} \tag{2}$$

Equation (2) indicates that there is a length l_o for which the critical load reaches a minimum in an elastic medium. The minimum value of P_{cr} is obtained by substituting equation (2) into equation (1):

$$P_{cr} = 2 \cdot \sqrt{E_s \cdot E \cdot I} \tag{3}$$

According to Bjerrum, buckling should only be a concern for design of a pile if the compression load that produces yielding of the pile material exceeds the value of P_{cr}

$$P_{cr} \le \sigma_{\max} \cdot A \tag{4}$$

where:

σ_{max} = yield stress of the pile material, which is equal to f_y in the case of steel piles, and

A = cross-sectional area of the pile.

Equations (3) and (4) can be combined as:

$$I/A^2 \le \frac{\sigma^2{}_{\max}}{4E_s E}$$

$$(5)$$

Bjerrum evaluated pile materials typical at the time including round and square bars, railroad rail sections, H-sections and pipes. He used a yield stress of 52,000 psi and an elastic modulus of 3×10^7 psi for the steel, and assumed a low value of 75 psi for the modulus of lateral reaction of the soil. Using equation (3), Bjerrum calculated that there was no danger of buckling for most pile sections. Only in the case of small steel rods and rails was there actual danger of buckling.

Buckling of micropiles

Table 1 lists some of the sections and steel types (bars and casing) most frequently used for micropile work in the United States. These typical sections are used as reference for the buckling analyses presented in this paper. More recent advances in the development of improved structural elements have led to the advent of hollow bars, which can be used for micropile construction. These bars are typically drilled into the ground utilizing grout as a drilling fluid injected through the hollow core. Table 1 includes an example of the injection bore pile material for reference. The reader is warned that sections in use in other countries may produce results that differ appreciably from the general results presented herein.

To analyze buckling of steel-reinforced micropiles, it is convenient to express equation (5) as follows:

$$E_s \leq \frac{1}{\left[\left(4 \cdot \frac{I}{A^2}\right) \cdot \left(\frac{E}{f_y{}^2}\right)\right]} \qquad (6)$$

The first of the two terms inside the bracket 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* in this paper and is given in units of [stress^{-1}]. Pile factors for each section are listed in Table 1.

The value of E_s calculated using Equation (6) 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 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 (6) is represented graphically in Figure 1. The pile factor and modulus of lateral reaction of the soil are represented logarithmically in the coordinate and ordinate axis, respectively. 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. The line in Figure 1 can thus be used to determine whether or not checking for buckling of a given pile is necessary.

Also represented in the figure is the range of pile factor values for each micropile type in Table 1. 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. For bar-reinforced piles and injection bore micropiles, the critical E_s value is 700 psi and 150 to 300 psi, respectively. For casing-reinforced micropiles with yield stresses of 80 ksi, the critical E_s value ranges between 80 and 150 psi depending on the casing dimensions. For 36 ksi casing, the critical E_s ranges between 20 and 30 psi. It must be noted that the ratio of I/A is constant for a solid circular section; therefore, for all solid bar sizes, the critical E_s is constant. Figure 2 provides a bar chart for comparison of the typical materials contained in Table 1.

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

Table 1. Typical properties of micropile reinforcement in USA

It must be noted that the lower values of lateral reaction modulus that are necessary for buckling of 36-ksi cased micropiles do not imply that the load capacity of these piles is larger than the 80-ksi cased micropiles. To explain this, let us pick two casings out of Table 1: one is an 80-ksi, 5.5-inch diameter casing; the other is also 5.5-inch in diameter but has a yield stress of 36 ksi. Let us imagine that the casings are tested for buckling under identical conditions. It is found that the 80-ksi casing buckles under a load of 350 kips, which is lower than its yield load in compression. According to equation (3), the 36-ksi casing would buckle under an identical load as the 80-ksi casing; however, during testing, the 36-ksi casing fails under its compressive yield load of 286 kips, as would be expected. Therefore, failure occurs under compression loads prior to reaching the critical buckling load for 36-ksi casing.

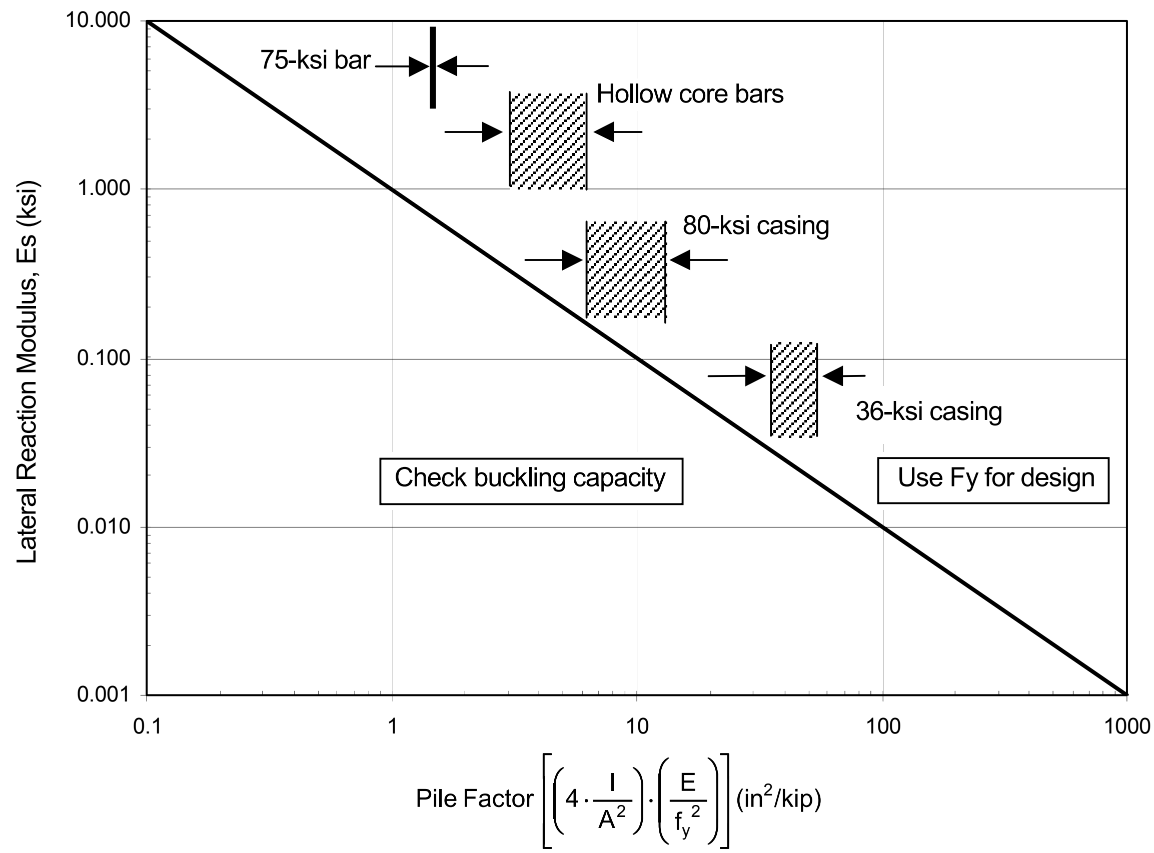

Figure 1. Chart for approximate buckling evaluation of micropiles subject to centered loads (see text for definition of terms)

Figure 1 may be used for an approximate determination of whether or not buckling may occur in a micropile. If, according to Figure 1, a particular combination of soil and micropile type may be susceptible to buckling, then the minimum critical load should be estimated using Equation (3). This procedure assumes that the pile has constant cross-sectional properties, and that 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.

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 during grout installation, 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 as discussed later in this paper.

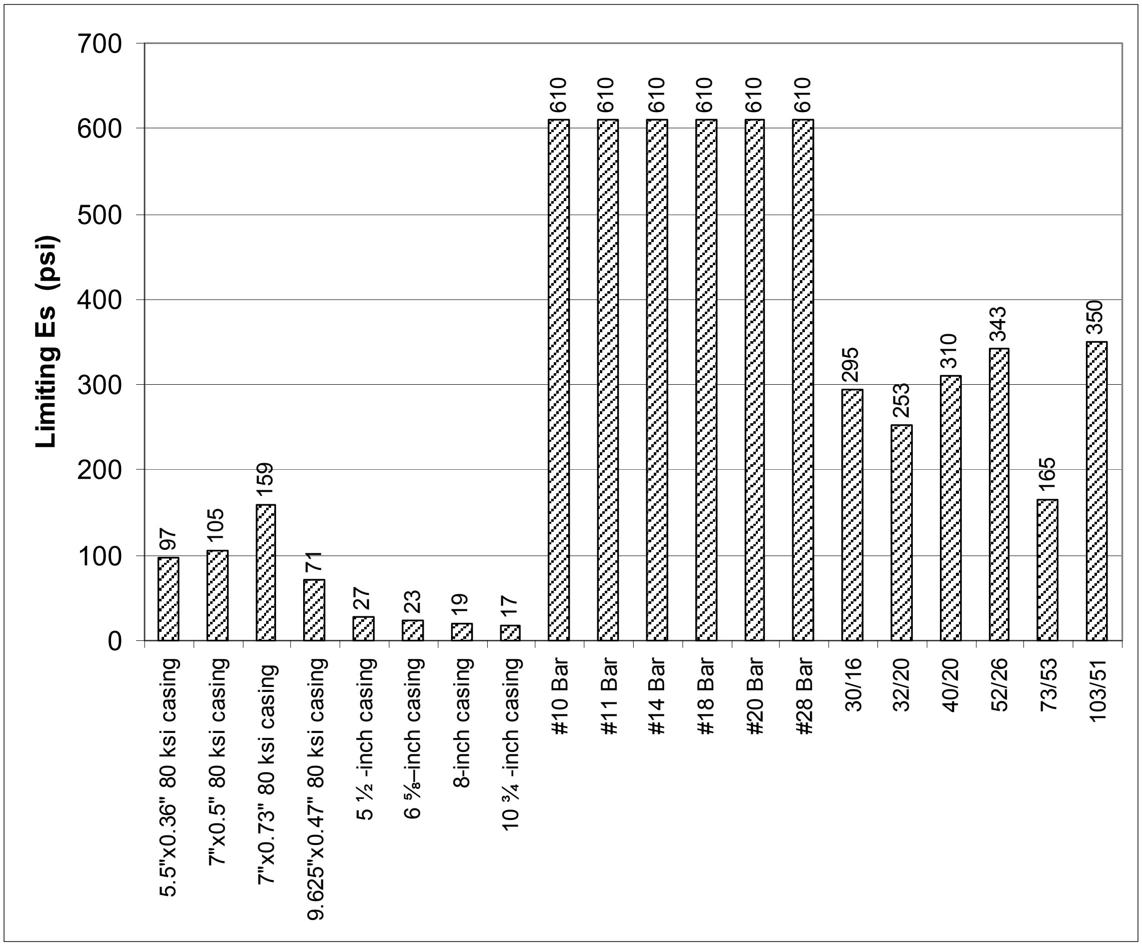

Figure 2. Chart for comparison of micropile materials from Table 1 (see text for definition of terms)

Observations regarding the lateral reaction modulus

In the procedure described in the previous section, it is assumed that the soil behaves linearly and does not fail. Because neither of these two assumptions is true, it is necessary that the lateral reaction modulus be selected according to the expected range of soil-pile displacement and based on the strength of the soil.

Figure 3, illustrates the type of response to lateral loading of the soil in contact with a pile. The coordinate axis represents the lateral displacement at a given point of the pile. The ordinate axis represents the corresponding lateral soil reaction given in units of [force/length]. The dashed line represents the secant reaction modulus, E_s, at any point during loading. For very small displacements of the pile, the soil may behave linearly. As the displacement increases, the lateral reaction modulus decreases until the soil yields. Such a curve is usually referred to as a p-y curve. A variety of procedures is available to predict the shape of p-y curves (Reese et al. 2000), which are based both on theoretical considerations and on experimental data.

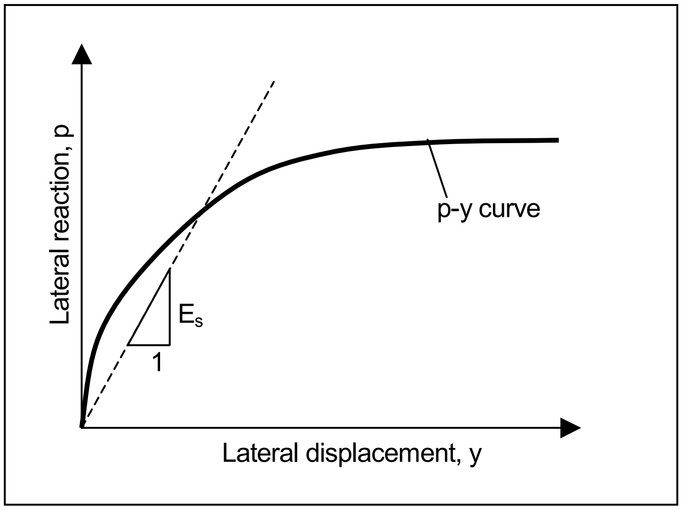

Figure 3. Hypothetical p-y curve

For simple analyses of the type described in the previous section, the secant value of E_s for soft clays can be approximated using the following equations:

$$p_{\max} = 9 \cdot c \cdot b \tag{7}$$

$$E_s = \frac{0.5 \cdot p_{\max}}{0.04 \cdot b} \tag{8}$$

where:

p_{max} = ultimate lateral load

c = shear strength of the soil, and

b = width of the pile.

Combining Equations (7) and (8), the following approximate expression is obtained:

$$E_s \approx 100 \cdot c \tag{9}$$

Equation (9) assumes that 50 percent of the ultimate capacity of the soil to resist lateral loading is mobilized after a strain of 2 percent. It also assumes that most of the soil deformation occurs within a zone extending a distance equal to $2b$ ahead of the leading edge of the pile. Therefore, Equation (9) should be used only if these assumptions are applicable to the particular problem under analysis, and may require modifications in certain cases.

As an example, for a 7-inch micropile installed in very soft clay with c=250 psf, the corresponding value of secant lateral reaction modulus, E_s, would be 25 ksf (175 psi). A 7-inch micropile with a casing thickness of 0.5 inch, has a pile factor of 9.5 in^2/kip. Using Figure 1, it is seen that buckling would not control the design of the 7-inch micropile under a centered vertical load. If the pile was subject to eccentric loading, moments, or horizontal loading, more sophisticated analyses should be performed.

On the other hand, for any micropile reinforced with one 75-ksi bar (pile factor = 1.64) and installed in the very soft clay described above, buckling may indeed control the design, and Equation (3) should be used to estimate the minimum critical load. It is found that for all bar sizes, the minimum critical load of a micropile installed in this soft clay is 80 percent of the compression yield load.

It must be noted that if lateral pile displacements are much smaller than 0.04 times the width, b, of the pile, the secant lateral reaction modulus may become much larger than that estimated in the above calculations. Similarly, if pile displacements are much larger than $0.04b$, the modulus may become much lower. It is thus necessary to use a value of lateral reaction modulus that is consistent with the lateral deformation of the pile. Equation (9) is not applicable for sensitive clays, stiff clays, granular soils, or rock. Complete details for development of p-y curves for soft soils are given by Reese et al. (2000), and can be adapted for use in buckling analyses for special cases.

The work documented by Bjerrum (1957) references clays with shear strength values varying from 2 to 2.8 psi, as determined from undisturbed specimens, and lateral reaction modulus values of 100 to 130 psi as interpreted from compression tests. Bjerrum does not indicate what procedures were followed to determine the strength and lateral reaction modulus. As can be seen by the above example, the values of E_s determined from Equation (9) are larger than those reported by Bjerrum for the same range of shear strength values. It must be kept in mind that the materials tested by Bjerrum were sensitive clays and, therefore, Equation (9) may not be applicable.

Final selection of the lateral reaction modulus is the responsibility of the design engineer based on the conditions present. In order to facilitate an even comparison of various micropile alternatives, the determination of the stiffness for use on the project should be presented by the designers in the geotechnical engineering report.

Contribution of grout to buckling capacity

Generally, the contribution of the grout, in and around a micropile, to its buckling resistance is not considered for design. In some cases, however, it may be desirable to include the contribution of the grout to the structural capacity of the micropile against buckling. A bar-reinforced micropile with significant grout cover may be such a case. It would be possible to evaluate the contribution of the grout to the flexural resistance by treating the micropile as a reinforced concrete element. Thus, the contribution of both the reinforcing steel and the grout would be included in the value of EI input into Equation (2).

In addition to its structural contribution, the grout surrounding a pile may also improve the surrounding soil and increase the contact area between the pile and the soil. Consequently, the horizontal stresses imparted on the soil may be increased, and the overall lateral capacity of the pile may be increased due to the presence of the grout surrounding the pile. This would be the case of micropile systems where the casing is inserted and grouted into an oversized hole, or where grout intrusion into the soil occurs due to pressure-grouting, compaction-grouting, permeation grouting, etc. The increased capacity of the pile to buckling can be assessed considering the increased E_s value within the improved zone around the pile. This increased E_s value should be an average between the values of E_s for improved and non-improved soils lying within the zone of influence of the pile. As discussed previously, the thickness of the zone of influence can be estimated as twice the pile diameter, b.

Soil-pile interaction analyses

Complete analyses of pile buckling require finding the solution to the general equation for a beam over an elastic foundation subject to axial loading:

$$E \cdot I \frac{d^4 y}{dx^4} + P \frac{d^2 y}{dx^2} + E_s \cdot y = 0 \qquad (10)$$

where:

P = axial load applied to the pile,

x = coordinate along the axis of the beam or pile, and

y = deflection perpendicular to the beam or pile axis.

The first term of the equation corresponds to the equation for beams subject to transverse loading. The second term represents the effect of the axial load. The third term represents the effect of the reaction from the soil. For soil properties varying with depth, it is convenient to solve this equation using numerical procedures such as the finite element or finite difference methods. Reese et al. (2000) outlines the process to solve Equation (10) using a finite difference approach. Several computer programs are commercially available that are applicable to piles subject to axial and lateral loads as well as moments. Such programs allow the introduction of soil and pile properties that vary with depth, and can be used advantageously for design of micropiles subject to centered or eccentric loads.

Alternatively, a finite difference solution can be developed using any commercial spreadsheet software. The advantage of spreadsheet solutions over commercial software is that they can be prepared and customized for special cases. They also allow easy tracking of the numerical procedure used. However, preparation of spreadsheets requires an experienced user, and calculations are much slower than those performed using lower level programming languages. The authors have developed a spreadsheet (PILE-BUCKLE) that includes a finite difference algorithm, and can be used to analyze a variety of unconventional micropile arrangements.

Special cases for micropiles

Installation through voids in karstic terrain

Figure 4 illustrates the case of a micropile installed through karstic terrain. The micropile is subjected to a centered load P applied at the top, and penetrates a relatively consistent medium throughout its entire length, except at the void location.

The structural capacity of such a micropile should be checked using procedures for design of steel columns. Depending on the procedure followed for micropile installation, the portion of the pile through the void may be analyzed as a doubly pinned, pinned-fixed, or fixed-fixed column. The model in the figure assumes that the pile has been grouted only along the bond length, and that an annular gap around the casing exists throughout its unbonded length. Under these conditions, the micropile can be conservatively assumed to be pinned on both ends.

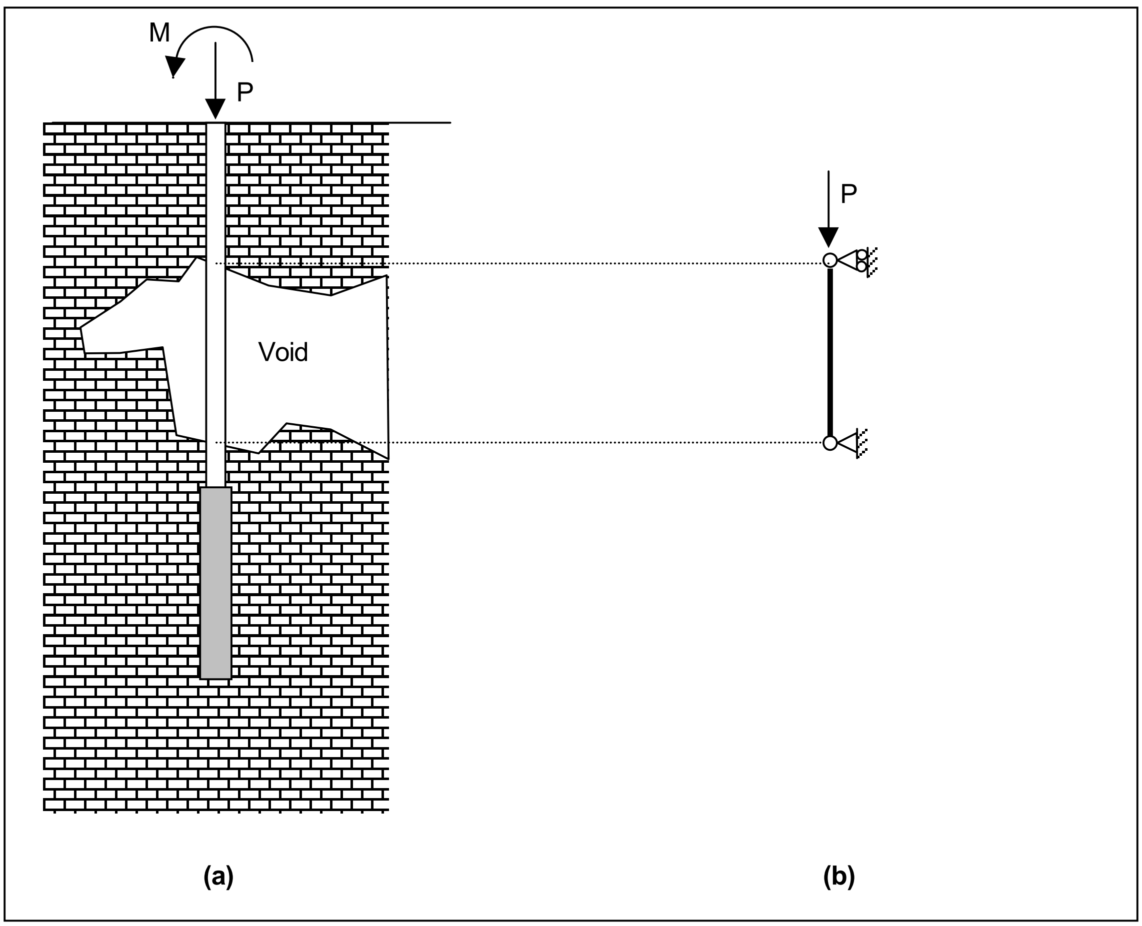

Figure 4. Micropile installed through voids in karstic terrain; (a) actual configuration, (b) model used for estimation of structural capacity

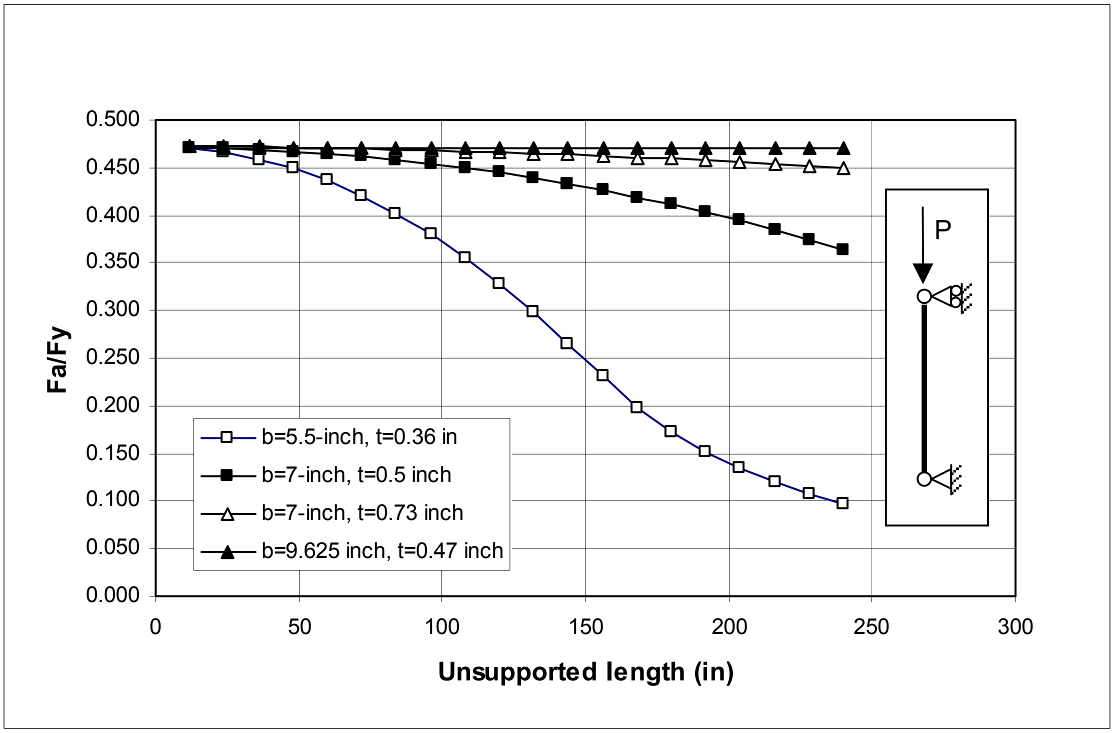

Figure 5. Chart for estimation of the reduction in allowable stress in vertically loaded, cased micropiles through voids in karstic terrain (the unsupported length is assumed pinned on both ends)

The allowable load for such micropiles can be calculated following the procedure described in Micropile Design and Construction Guidelines- Implementation Manual (FHWA 2000). The buckling load for 80-ksi cased micropiles (see Table 1) has been estimated for a range of unsupported lengths, as shown in Figure 5. It is seen that for 5.5-inch cased micropiles, the allowable stress may become significantly lower than the commonly accepted value of 0.47Fy. For an unsupported length of 120 inches (10 feet), the allowable stress of a 5½-inch casing is approximately 0.33Fy. It should be noted that the capacity of the 9⅝ inch diameter casing is not significantly reduced for unsupported lengths of less than 20 feet.

In cases where the pile extends through hard rock above the void, consideration should be given to the effect this may have on the ability of lateral loads or moments to transfer to the portion of the pile within the void. Eliminating such loads within the void zone would have a significant influence on the calculated allowable load.

As discussed in a subsequent section, micropiles may be susceptible to buckling-induced failure at the casing joints. Therefore, micropiles installed through karst must be designed for buckling, considering the presence of the casing joints, or should include installation of continuous internal reinforcement along portions of micropile traversing voids.

Effect of annular space around micropile

When cased micropiles are installed through relatively dense or cohesive terrain, an annular gap may be created around the casing during drilling, and remain open for a significant period. This gap may be typically ½ inch wide, and its effect on the buckling capacity of the micropile is not readily apparent.

Based on previous experience and simplified analyses, the authors believe that, for a micropile subject to a centered vertical load, the presence of the annular space may not reduce its allowable structural capacity. Conventional structural analyses used for column design would almost certainly limit the allowable compressive stresses in the micropile section below those commonly used for micropile design. However, it must be considered that, during buckling, the pile would eventually enter into contact with the surrounding terrain. Because annular gaps of any significant length around micropiles typically develop only in relatively stiff soils or rock, further buckling of the micropile against the terrain would be unlikely. The question then remains whether the structural capacity of the micropile should be limited due to the lateral displacement of the pile (½ inch).

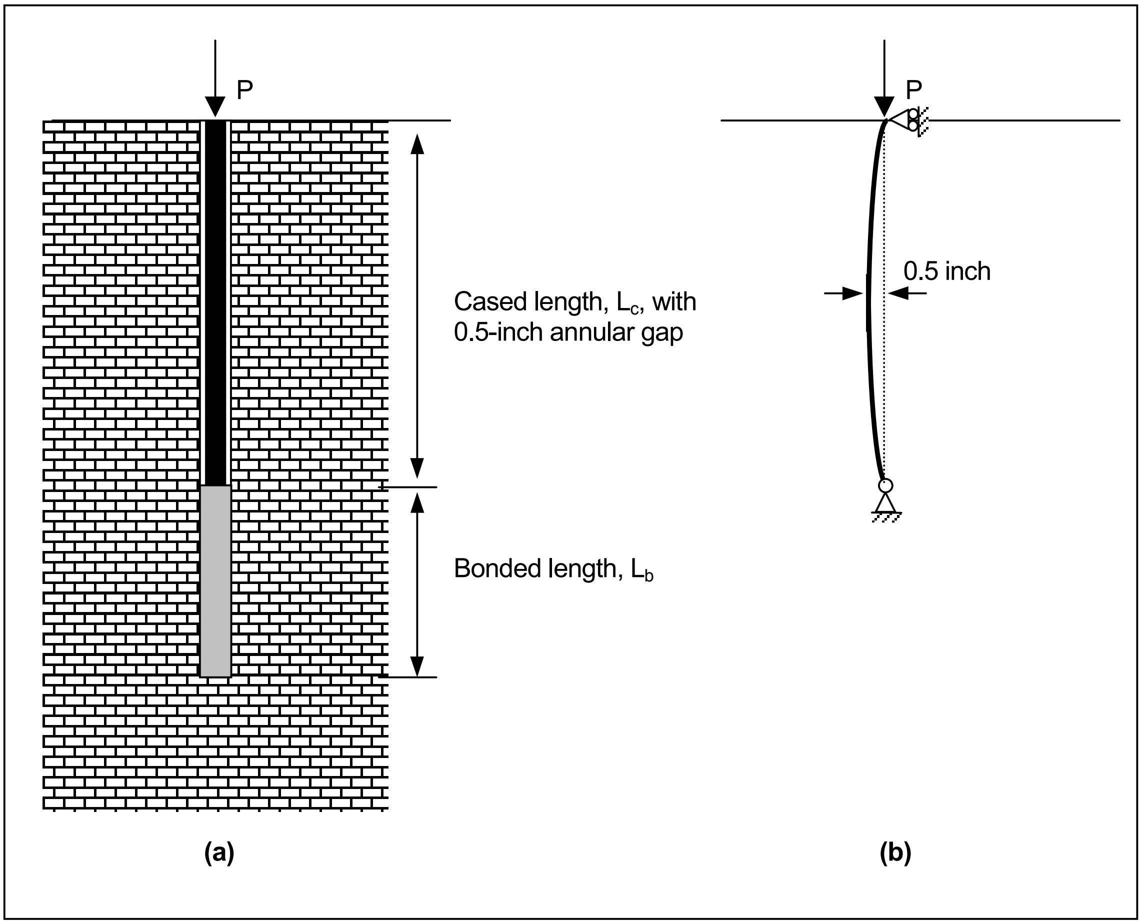

Figure 6. Buckling due to annular gap around micropile; (a) actual configuration, (b) simplified model

Figure 6 illustrates the problem of buckling of a micropile induced by the presence of the annular gap. The micropile is assumed to undergo a lateral displacement of 0.5 inch, equivalent to the gap width. To provide a simple and conservative estimate of the bending moment induced by buckling, other reactions are neglected. It is also assumed that the pile is pinned on both ends, which is a further conservative assumption as well. Under these assumptions, the maximum bending moment, M_{max}, occurs at the midpoint of the pile, and is calculated as:

$$M_{\max} = P \cdot 0.5\,inch \tag{11}$$

Let us assume that the micropile is reinforced with a 5.5-inch diameter, 80-ksi casing (see Table 1). The allowable load of such a micropile (not including the contribution by the grout) is typically calculated as 0.47Fy (FHWA 2000), or 220 kips. For P equal to 220 kips, the maximum bending moment is 110 in-kips, which produces a maximum combined stress in the micropile section of 53 ksi, or 0.66Fy. This magnitude of stress is generally considered allowable for structural design of steel beams subject to bending.

It must be mentioned, however, that the authors do not intend to disregard altogether the effect of an annular gap on the structural capacity of micropiles. Each case must be analyzed by the designer considering the installation process, and the material of the micropile. Solutions to this problem can be completed with a spreadsheet solution that would allow the introduction of an annular gap of a given size and length into the analyses of micropile buckling.

Discontinuities in micropiles

In all previous discussions presented in this paper, it has been assumed that the micropiles are continuous elements of constant cross sectional properties. Cased micropiles are typically installed in 5- or 10-ft sections connected through flush threaded joints. The cross sectional properties at the joints of several typical micropile casings are given in Table 2. These were estimated assuming that the thickness of each threaded joint is exactly one half that of the casing.

	Casing F_y= 80 ksi			
	5½ -inch casing	7-inch casing		9⅝-inch casing
Casing OD, in	5.5	7	7	9.625
I/A^2 at threaded joint	1.09	1.00	0.65	1.50
Pile factor (PF) at joint, in^2/kip	4.6	4.2	2.6	6.5
	Casing F_y= 36 ksi			
	5½ -inch casing	6⅝–inch casing	8-inch casing	10¾ -inch casing
I/A^2 at threaded joint	0.77	0.94	1.15	1.25

Pile factor (PF) at joint, in^2/kip	15.5	19.3	24.1	26.2

Table 2. Estimated values of Pile Factor at casing joints

The pile factor, *PF*, at the joints is calculated using the following equation:

$$PF = \left(4 \cdot \frac{I_{\text{joint}}}{A_{\text{casing}}{}^2}\right) \cdot \left(\frac{E}{f_y{}^2}\right) \tag{12}$$

where:

I_{joint} = moment of inertia of the reduced joint section (considering only one of the threaded ends in the joint, not the two), and

A_{casing} = area of the casing section (not at the joints).

For an approximate evaluation of the potential for buckling due to the presence of the threaded casing joints, the pile factor values of Table 2 can be used together with Figure 1. It is seen that buckling of an 80-ksi casing should be a concern if the lateral soil modulus is lower than 150 to 400 psi. Therefore, it is concluded that the potential for buckling of cased micropiles in soft soils may be significantly increased by the presence of the threaded joints.

It is obvious that casing joints would be critical in micropiles installed through voids. However, this method considers that the stiffness of the pile is equal to the joint stiffness throughout the void length, which is very conservative. If necessary, more detailed structural analysis can be performed for critical cases, where the micropile can be modeled as an element composed of discrete sections of different stiffness. This would allow a more accurate assessment of the effect of the joints on the response of the micropile. However, it is always prudent that cased micropiles installed through voids in karstic terrain be provided with the necessary internal reinforcement to prevent reduction in the allowable load capacity from buckling-induced failure at the joints.

Case histories

Countless Case I micropiles have been installed around the world in a variety of situations, including micropiles installed through very soft overburden conditions and through moderate soils to hard rock. An extensive research program was carried out by CALTRANS at a Deep Bay Mud Site in San Francisco, California, to evaluate the seismic performance of several pile systems (Mason 1993, and Brittsan and Speer 1993). Included in these were two configurations of Case I micropiles.

The CALTRANS load tests were being completed to evaluate the load carrying capacities of the New Bay Mud Formation. This material generally classifies as very soft to soft gray clay and silty clay (CH) with some organic matter (OH). The index testing and site history indicate that this material is normally consolidated above EL -50, and slightly over consolidated below. Ground surface level is at about EL 9.5 in the test site area.

Nicholson Construction installed two pile types as a part of this research. Both used 7-inch diameter N80 casing with 0.725-inch wall thickness throughout the length of the soft Bay Mud. Both pile systems included a #18, Grade 60 bar within the casing. Grouting methods within the bond zones differed somewhat. Both test piles penetrated about 90 ft of the New Bay Mud which extended to about EL –100. These piles were load tested to 385 and 400 kips with about 1.4 and 0.9 inch of total deflection each. Both piles had less than 0.2 inch of permanent movement when the load was removed. Neither of these piles reached ultimate geotechnical or structural failure. Similarly, neither of these piles showed any indication of buckling failure.

Case I piles were installed by Nicholson Construction and Structural Preservation Systems into karstic limestone rock in Exton, Pennsylvania, for a retail mall expansion (Cadden et al. 2001). These piles penetrated overburden soils and broken rock, and were bonded to the hard fresh limestone. These piles varied in length from about 20 ft to over 120 ft, and consisted of 7-inch diameter, 0.5-inch wall N80 steel casing installed in 10-ft long sections. Because the materials surrounding these piles were generally firm to stiff, the drilling method created a minor annular space around the casing during installation. No special procedures were followed to ensure filling of this annular space before load testing. Load testing of these piles was performed to loads in excess of 800 kips. No sign of structural failure of the pile or failure of the rock bond was detected.

Conclusions

Micropiles in the form of high capacity, Case I elements have been successfully utilized for new construction and retrofit applications. With the completion of the FHWA Implementation Manual and the constantly increasing use of micropiles, the technology is shifting from being a field dominated exclusively by specialty contractors toward becoming more popular among traditional design engineers. This increased usage requires the clarification of issues such as buckling potential. In this paper, an attempt has been made to address this issue using theoretical concepts and practical experience, and in a way that can be easily implemented in the engineering practice.

It has been shown that, in most cases, the structural capacity of a micropile is dictated by the allowable compressive stresses before a critical buckling load is reached. However, there are special cases where the soil conditions would dictate evaluation of buckling on a case-specific basis. These cases may include micropiles installed through very soft soils, or in karstic environments where voids exist within the material above the bond zone of the pile. The pile factor evaluation depicted in Figure 1 is a good starting point for the determination of whether a detailed buckling evaluation is required on a project.

Where the critical load is dictated by buckling, more detailed analysis utilizing spreadsheet solutions or commercially available programs for laterally loaded piles can be employed by the engineer. In these analyses, a proper evaluation of the end restraint and the load transferred to the critical portion of the pile will be important in determining a realistic answer without undue conservatism. It is also critical that the variations in the pile construction, such as changes in pile cross section and influence of grout or threaded joints, be considered in the analyses.

Acknowledgements

This work is the result of a request from the ADSC Micropile Committee to address questions that were frequently asked by potential users. The Committee provided invaluable support, consultation and review in the development of this paper. Special thanks go to the review committee including Tom Armour, Horst Ashenbroich, Donald Bruce, Tom Richards, and John Wolosick. We would also like to thank the members of the International Workshop on Micropiles (IWM) for providing information related to international efforts to address this issue in the past. We would also like to thank Mr. Miguel Pando for his assistance in obtaining several of the references cited in this report.

Support from Schnabel Engineering Associates, West Chester, Pennsylvania, was provided in the form of administrative services and engineering from several members of the company without whom this paper would not have achieved its goals.

References

Bjerrum, L. (1957). “Norwegian experiences with steel piles to rock,” Geotechnique, Vol. 7, pp. 73-96.

Brittsan, D., and Speer, D. (1993). “Pile load test results for highway 280 pile uplift test site,” CALTRANS pile load test results at a deep Bay Mud site using various pile types, Deep Foundations Institute and CALTRANS Specialty Seminar, September 9 and 10, 1993.

Cadden, A.W, Bruce, D.A., and Ciampitti, L.M. (2001). “Micropiles in karst: a case history of difficulties and success,“ Foundations and Ground Improvement, Proceedings of a Specialty Conference, American Society of Civil Engineers, Blacksburg, VA, June 9-13, Geotechnical Special Publication No. 113, pp. 204-215.

Cummings, A.E. (1938). “The stability of foundation piles against buckling under axial load,” Proceedings Highway Research Board, 18(2), pp. 112-119.

Davisson, M.T. (1963). “Estimating buckling loads for piles,” Proceedings 2nd Pan-American Conference on Soil Mechanics and Foundation Engineering, Brazil, Vol. I, pp. 351-371.

FHWA (2000). “Micropile design and construction guidelines. Implementation manual,” U.S. Department of Transportation, Federal Highway Administration, Priority Technologies Program, FHWA-SA-97-070.

Glick, G.W. (1948). “Influence of soft ground on the design of long piles,” Proceedings 2nd International Conference on Soil Mechanics, Rotterdam, Vol. 4, pp. 84-88.

Gouvenot, D. (1975). “Essais de chargement et de flambement de pieux aiguilles,” Annales de l’Institute Technique du Bâtiment et des Travaux Publics, Comité Français de la Mécanique des Sols et des Fondations, No. 334, December.

Mandel, J. (1936). “Flambement au sein d’un milieu inelastique,” Annales des Ponts et Chaussées, Deuxieme Seminaire, pp. 295-335.

Mascardi, C.A. (1970). “Il comportamento dei micropali sottoposti a sforzo assiale. Momento flettante e taglio,” Verlag Leeman. Zurich.

Mascardi, C.A. (1982). “Design criteria and performance of micropiles,” Symposium on Soil and Rock Improvement Techniques including Geotextiles. Reinforced Earth and Modern Piling Methods, Bangkok, paper D-3.

Mason, J.A. (1993). “I-280/San Francisco pile load test. Results and seismic design guidelines,” CALTRANS pile load test results at a deep Bay Mud site using various pile types, Deep Foundations Institute and CALTRANS Specialty Seminar, September 9 and 10, 1993.

Matlock, H. (1970), "Correlations for design of laterally loaded piles in soft clay," Proceedings of Second Offshore Technology Conference, Houston, TX, pp. 577-594.

Reese, L.C., Wang, S.T., Isenhower, W.M., Arellaga, J.A. (2000). “Computer program LPILE Plus. Version 4.0. Technical manual,” Ensoft, Inc., Austin, Texas.