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Micropiles Made of Reinforced Polyurethane Resins: Load Tests and Evaluation of the Bearing Capacity

Roberto Valentino, PhD

Department of Civil, Environmental, Territory Engineering and Architecture

University of Parma; Viale G.P. Usberti 181/A, 43100 Parma, Italy

e-mail address: roberto.valentino@unipr.it

**Corresponding author, Tel: +39 0521 906112, Fax: +39 0521 905924*

Davide Stevanoni

Novatek s.r.l.

Via dell'Artigianato 11, 37021 Bosco Chiesanuova (VR), Italy

e-mail address: davide.stevanoni@novatek.it

ABSTRACT

Micropiles are traditionally used in many applications of ground improvement to increase the bearing capacity and reduce the settlement of existing shallow foundations. The mechanism for ground improvement is provided by frictional resistance between the soil and the lateral surface of the pile and by the associated group effects of micropiles. This paper deals with the field behavior of innovative micropile typology and the method for the evaluation of micropile bearing capacity. Micropiles made with reinforced polyurethane resin have been tested both in compression and tension field tests. Micropiles were installed in various sample sites generally characterized by the presence of silty-clayey soils. On the basis of the test results, it was possible to evaluate the bearing capacity of the reinforced resin micropiles by using the well known Bustamante and Doix method. The paper describes the construction method of this kind of micropile, the execution of field tests and provides a comparison between experimental results and theoretical analysis.

KEYWORDS: micropiles, polyurethane resins, load tests, load-displacement, bearing capacity, soil reinforcement.

INTRODUCTION

It is of essence to solve, in a short time and with minimal effort, the problem of settlements of shallow foundations of existing buildings. There are several systems, which, with the employment of different innovative technologies, allow to increase the load-bearing capacity of shallow foundations and limit their settlement both in short-term and long-term. Among different underpinning technologies, micropiles are the most commonly adopted (Bruce, 1989; Han and Ye, 2006a, 2006b; Fross, 2006), especially for monuments and old buildings. Moreover, the micropile technique is particularly diffused in Italy, where it was introduced since the early 1950s by Lizzi (1980).

To be able to deal with the problems associated with the soil-structure interaction, which cause structure damage of various kinds, a highly reliable technology and an in-depth knowledge of mechanical phenomena that can occur due to an intervention are required. Recently some Authors have provided experimental and theoretical analyses aimed to investigate an analytical solution for micropile design under tension and compression (Juran et al., 1999; Babu et al., 2004; Misra and Chen, 2004; Misra et al., 2004; Stuedlein et al., 2008).

The design of such structures is commonly based on empiricism and past experience, but a standard methodology of foundation reinforcement requires a study that provides a sound scientific base for a design procedure. Novatek s.r.l., an Italian company specializing in foundation reinforcement and ground improvements, in cooperation with the University of Parma, recently started an activity aimed to study the mechanical behavior of micropiles made with expanded polyurethane resin. Specifically, the activity was aimed at identifying and determining the applicability of an innovative kind of micropiles which employ recently patented technology (patent code: PCT/IT2007/000808).

The activity was mainly devoted to defining a method for the calculation of the bearing capacity of a micropile made with expanded reinforced resin (RR). The characteristics of this type of micropile will be described later on in the paper. The RR micropile behavior has been analyzed through a series of field loading tests. These tests allowed to obtain load-displacement curves and to determine the ultimate bearing capacity of micropiles realized in sample sites characterized by the presence of different soils. Through the analysis of experimental results, a method for the calculation of the load-bearing capacity of this new kind of micropiles has been defined.

BEARING CAPACITY OF GROUT INJECTED MICROPILES: THE BUSTAMANTE AND DOIX METHOD (1985)

Scientific literature contains several methods for the design of piles and micropiles. It is well known that in most cases the design procedure is strictly linked to the construction method, so that each kind of pile can be designed on the basis of an empirical or semi-empirical method.

In the present work, in order to define a design method for RR micropiles, the method proposed by Bustamante and Doix (1985) for the calculation of the bearing capacity of grout injected micropiles was taken into consideration. This method has been adopted by the France code CCTG (1993) and also by standard rules of other European countries (Juran et al., 1999; Viggiani, 1999).

The method proposed by Bustamante and Doix (1985) was applied to a series of experimental data collected through a number of field tests in sample sites distributed on French territory. A grout injected micropile consisted in a metallic reinforcement introduced in a drilled small diameter borehole and sealed to the ground by mortar or binding grouting mixture injections made under high pressure. Micropile injections described by Bustamante and Doix were carried out through a flexible grouting pipe, open in the lower part, or with a rigid, metallic tube that was fitted with a succession of no-return valves called “*tube à manchettes*".

The method referred to two different types of injected micropiles: IRS and IGU. The abbreviation IRS was applied to the micropiles characterized by an execution methodology that permitted the repetitive and selective grout injection at different depths. The abbreviation IGU referred to the systems that allowed grout injection of micropiles in a single procedure.

During the field tests carried out on RR micropiles, that will be described later, a substantial analogy of behavior with grout injected micopiles described by Bustamante and Doix was observed. For this reason, it seems essential to report here some steps for the design of grout injected micropiles as proposed by Bustamante and Doix. This method has been adopted, preliminary, to be compared with loading test results of RR micropiles.

For the definition of the ultimate load capacity, Bustamante and Doix refer to the well known expression:

$$Q_{lim} = P+S \tag{1}$$

where P is the total resistance in correspondence to the pile tip and S is the lateral total resistance. The lateral resistance S, in turn, is determined by:

$$S = \pi \cdot d_s \cdot L_s \cdot s \tag{2}$$

where d_s is the equivalent-diameter of the micropile, L_s is the length of the injected zone, and s is the unit tangential resistance at the interface between the injected zone and the surrounding soil. In the case where micropile is injected into soil layers with different characteristics, the relation for the determination of S is:

$$S_l = \sum_i (\pi \cdot d_{si} \cdot L_{si} \cdot s_i) \tag{3}$$

In Equations (2) and (3) it is assumed $d_s = \alpha \cdot d$, where d is the diameter of the perforation and α is an increasing coefficient, and its value can be determined by using Table 1 (Bustamante and Doix, 1985). As it can be seen from Table 1, Bustamante and Doix provided further indications about the minimum quantity of injected mixture in terms of V_s, which is the volume of the joint bulb.

The values of the tangential resistance per unit area (s) in correspondence to the interface between injected pile and soil depend both on the nature and the characteristics of the soil and on the technology used, and can be easily evaluated through appropriate diagrams reported by Bustamante and Doix (1985). According to this method, the determination of the interface unit resistance is strictly correlated to the results of Standard Penetration Tests (SPT).

For IGU and IRS micropiles the total tip resistance (P) is equal to 15% of the total skin resistance (S) (Bustamante and Doix, 1985; Viggiani, 1999).

Indeed, Bustamante and Doix have proposed an alternative expression for the calculation of P, based on the limit pressure of the soil, which can be determined through the Mènard pressiometer (Bustamante and Doix, 1985; Viggiani, 1999).

Table 1: (Bustamante and Doix, 1985)

SOIL	α		Suggested quantity of mixture
	IRS	IGU	
Gravel	1.8	1.3-1.4	1.5Vs
Sandy gravel	1.6-1.8	1.2-1.4	1.5Vs
Gravelly sand	1.5-1.6	1.2-1.3	1.5Vs
Coarse sand	1.4-1.5	1.1-1.2	1.5Vs
Medium sand	1.4-1.5	1.1-1.2	1.5Vs
Fine sand	1.4-1.5	1.1-1.2	1.5Vs
Silty sand	1.4-1.5	1.1-1.2	IRS: (1.5-2.0)Vs; IGU: 1.5Vs
Silt	1.4-1.6	1.1-1.2	IRS: 2.0Vs; IGU: 1.5Vs
Clay	1.8-2.0	1.2	IRS: (2.5-3.0)Vs; IGU: (1.5-2.0)Vs
Marl	1.8	1.1-1.2	(1.5-2.0)Vs for compacted layers

REINFORCED EXPANDED RESIN MICROPILES

Traditionally various kinds of micropiles are used in underpinning of existing foundations on problematic soils. However, some recent foundation remediation techniques use polyurethane foams, which, from a geotechnical perspective, can be considered between underpinning and grouting (Buzzi et al., 2008; Buzzi et al., 2010).

The use of reinforced resin micropiles provides a solution to several problems in the more traditional use of expanding resins. Firstly, it solves the necessity to confer a definite shape to expanding resins, which have been widely used by free injections in underpinning to limit structures' settlement, and secondly, to create structural elements that allow to transfer external loads from the structure to the deeper layers of the soil, characterized by better mechanical properties.

Moreover, the tested innovative technology allows to use less costly equipment than those required for other pile typologies, with machines of reduced dimension and great flexibility of use. In order to understand the peculiar characteristics of reinforced resin micropiles, it is necessary to refer to the execution technique. In correspondence of the foundation to be reinforced, a hole with a diameter of 85mm that crosses the foundation and reaches the ground below, up to the desired depth (approximately about 3-4m under the foundation for most of the considered cases) is made. The borehole is made using a special tool that permits ground undermining, without soil removal. If the foundation base is not wider than the rising wall, the borehole is made at a suitable distance from the wall, with an adequate slope to allow alignment between the pile's head and the wall axis.

In the second phase, a hydraulic dilator device (*packer*) is used to expand the borehole in the soil below the foundation to a diameter ranging between 100 and 120mm. Consequently, micropile reinforcement, which consists of an externally threaded steel tubular with a diameter of 60mm and a thickness of 8mm, is inserted into the empty borehole. In correspondence to two or three different points along the length of the steel reinforcement (depending on the micropile length and on the presence of the shallow foundation), resin, that spreads both inside the pile and in the space between the tubular and the surrounding soil, is injected (Figures 1 and 2).

As pointed out by Juran et al. (1999) with regard to grout injected micropiles, high capacity steel elements are used as the principal load-bearing element, with the surrounding grout serving only to transfer, by friction, the applied load between the soil and the steel. The same principle is at the basis of the RR micropile behavior.

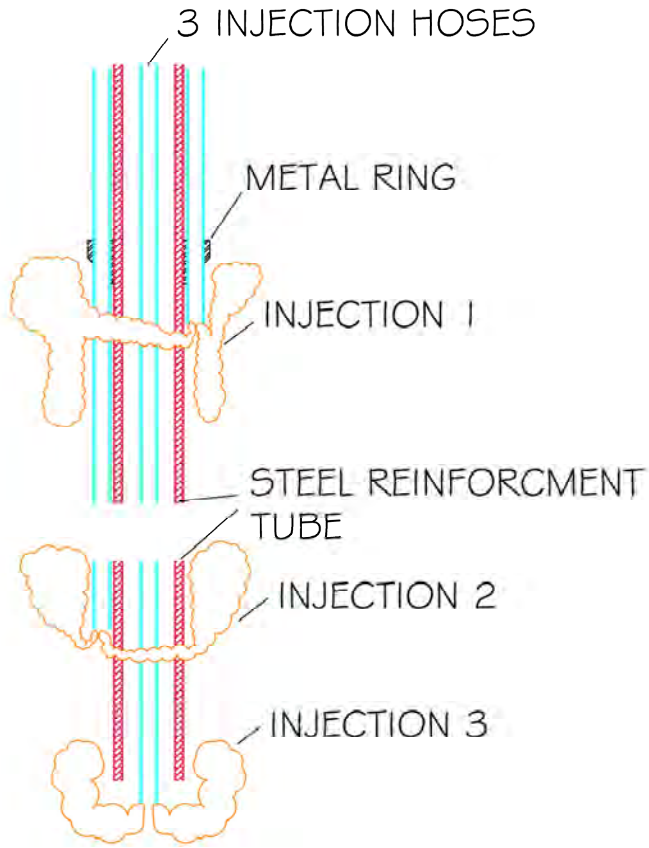

Figure 1: Scheme of the execution technique for a reinforced resin micropile without shallow foundation.

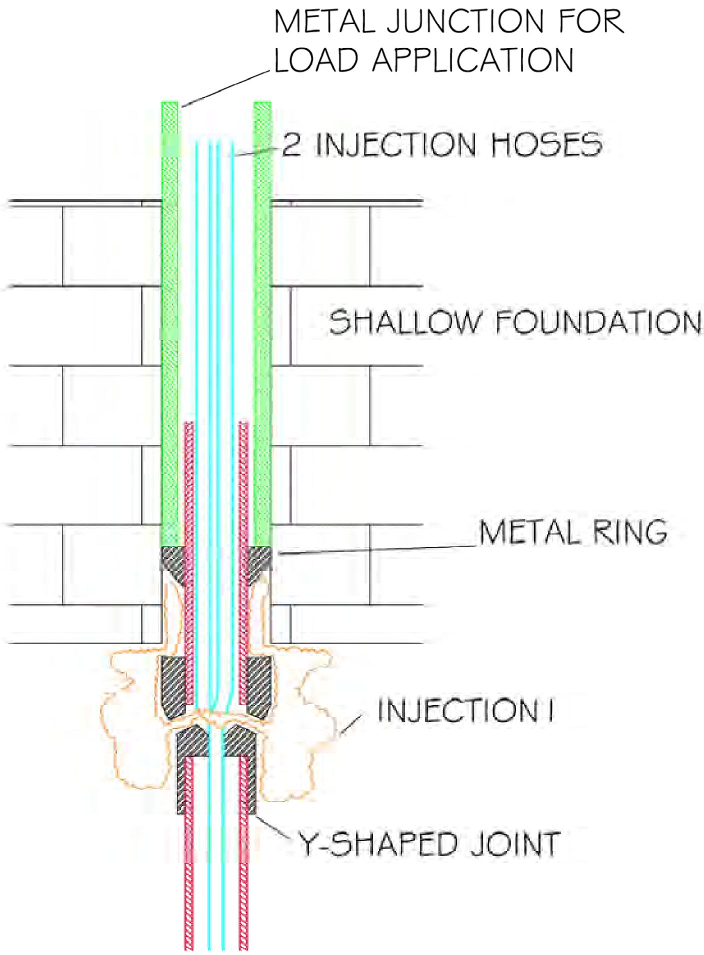

Figure 2: Scheme of the execution technique for a reinforced resin micropile under an existing shallow foundation.

The injected expanding resin, which substantially constitutes a filling, is identified by the abbreviation HDR200® (High Density Resin). It is a two-component resin and is created through a mixture of polyol and diphenylmethane diisocyanate in specified quantities. When the two components of the mixture (originally in liquid state) combine with each other, they give rise to a chemical high speed reaction that quickly determines the expansion of the injected product, till it increases 15-20 times its original volume.

Thanks to the water dosage in the mixture, it is possible to determine the final properties of the resin. Indeed, water acts as a catalyst in the chemical reaction, speeding up the expansion and solidification processes.

The characteristics of the resin used for the injection have been reported in Table 2.

Execution phases of reinforced resin micropiles have been schematically reported in Figure 3.

Table 2:

Name	Density (kg/m^3)	Temperature at reaction (°C)	Pressure at reaction (bar)	Pressure for injection (bar)	Time employed for reaction (s)
HDR200	80-130	30-30	40-60	5-10	6-10

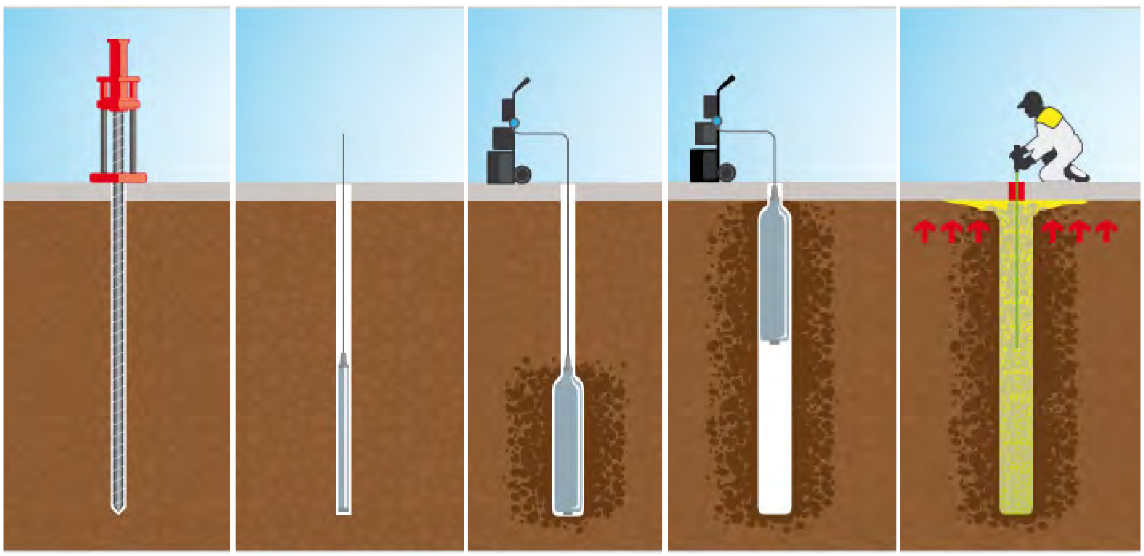

Figure 3: Different phases in the execution technique for a reinforced resin micropile

RR micropiles, built through the above described process, provide resistance to the pile-soil system both along the shaft and in correspondence to the tip.

The following characteristics of micropile execution mode contribute to the resistance provided by the soil surrounding the micropile:

1) the execution of the beaten borehole provokes initial compacting of the soil, without material removal;

2) the action of the packer radially dilates the borehole's walls, determining the ground tamping and allowing to obtain a final diameter of about 100mm, starting with an initial hole with the diameter of about 85mm;

3) the expanded action of the resin causes an increase of radial stress on the hole's internal wall and permits a perfect adhesion of the pile to the ground;

4) the capacity of the resin to infiltrate into the soil pores allows to obtain a good degree of roughness at the interface between the micropile and the surrounding soil.

FIELD LOAD TESTS

In a preliminary phase, after a geotechnical characterization of the site, a field test has been carried out in which four resin micropiles, reinforced with steel bars, have been used for the execution of load tests (Stevanoni, 2009). In this case, in the absence of superficial foundation on the top, for each micropile tested a contrast structure appropriately anchored to the ground with two other reaction micropiles, for the execution of the compression test, has been used. Two contrast beams placed at a distance of 30cm in respect to the ground level have been used for the application of the load.

The uniform distribution of the load was obtained thanks to the assemblage of a “castle” placed over the test pile axis and thanks to the use of an hydraulic jack, thus laterally transferring the forces to the contrast beams (Figure 4). The results of these preliminary tests, carried out in the municipal area of Bosco Chiesanuova (Verona, Italy), are reported in Table 3 and in Figure 6.

Figure 4: Apparatus with reaction beams for the execution of commpression tests at Bosco Chiesanuova (VR).

Subsequently, a series of other loading tests on micropiles realized as real underpinning work for existing buildings, in the presence of a shallow foundation on the top, have been carried out, thus testing the micropiles in conditions similar to the current structural conditions. The results obtained after the campaign of load tests and of data collection refer to 14 micropiles in different sample sites on the Italian territory (Table 3).

To perform the load test, a suitable "castle" has been made to be directly anchored to the foundation through two threaded bars (Figure 5). After the castle has been fixed to the bars and it has been made integral with the foundation, a jack and a load cell have been placed coaxially in respect to the micropile. Moreover, three digital displacement comparators have been placed in correspondence to the micropile’s top. They have been anchored on rigid plates and fixed to the ground level, to measure micropile settlements.

Table 3:

Sample site	Micropile ID	Length (m)	Hole diameter (mm)	Depth of pile head from ground level (m)	Loading type	Ultimate load (kN)
Bosco Chiesanuova (VR)	VR-D	4	120	0.1	Compression	120
Bosco Chiesanuova (VR)	VR-H	5.1	120	0.1	Compression	120
Bosco Chiesanuova (VR)	VR-N	3.4	120	0.1	Compression	120
Bosco Chiesanuova (VR)	VR-R	3.45	120	0.1	Compression	200
S. Marino (site 1)	SM1-RR1	3	85	1.0	Compression	100
S. Marino (site 1)	SM1-RR2	3	85	1.2	Compression	90
S. Marino (site 1)	SM1-RR3	3	85	1.0	Compression	80
S. Marino (site 2)	SM2-RR2	3	85	1.43	Compression	80
S. Marino (site 2)	SM2-RR3	3	85	1.07	Compression	110
Cesena (FC)	FC-RR2	3	85	1.05	Compression	90
Cesena (FC)	FC-RR3	3	85	0.92	Compression	80
Lucignano (AR)	AR-RR1	3	85	1.34	Compression	60
Lucignano (AR)	AR-RR2	3	85	1.34	Compression	40
Lucignano (AR)	AR-RR3	3	85	1.76	Compression	120
S. Leo (PU)	PU-RR1	3	85	1.20	Compression	100
S. Leo (PU)	PU-RR2	3	85	1.15	Compression	120
S. Leo (PU)	PU-RR3	3	85	0.9	Compression	80
Teramo (TE)	TE-RR1	3	85	0.3	Tension	40
Teramo (TE)	TE-RR2	4.7	85	0.3	Tension	80
Teramo (TE)	TE-RR3	3.05	85	1.4	Compression	65

Figure 5: “Castle” apparatus for the execution of commpression tests on RR micropiles under an existing foundation.

Thanks to this structure, the micropile has been loaded at the head, according to the methodology proposed by Mandolini (1995) and in compliance with the recommendations of AGI (Italian Geotechnical Association) (1984). In particular, a load increase of 10kN has been applied to each step, while the values of settlements on the comparator devices have been acquired at fixed-time intervals: after 1, 5, 10, 15 and 20 minutes, starting from the when a new load is introduced.

The experimental load-displacement curves obtained for all the sample sites are reported in Figures 6-12 together with the results of some Standard Penetration Test we had at our disposal for the geotechnical characterization of each site.

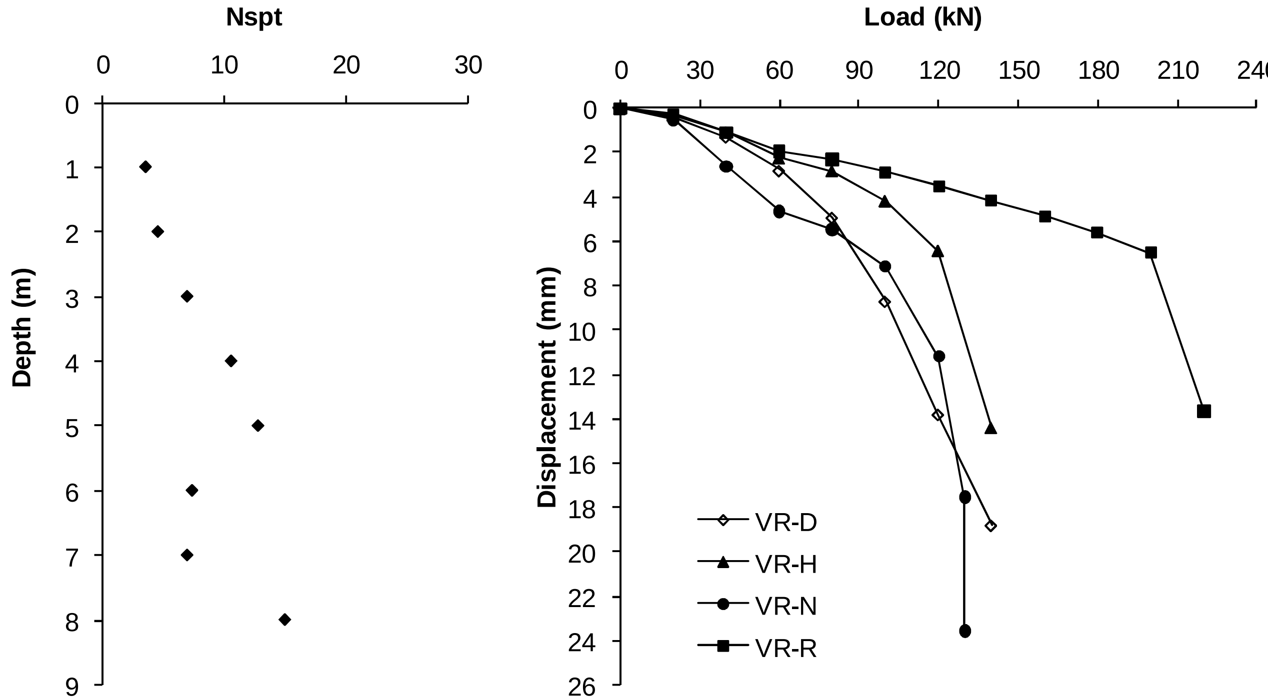

Figure 6: Sample site of Bosco Chiesanuova (VR): (a) Nspt data and (b) experimental results of the compression tests.

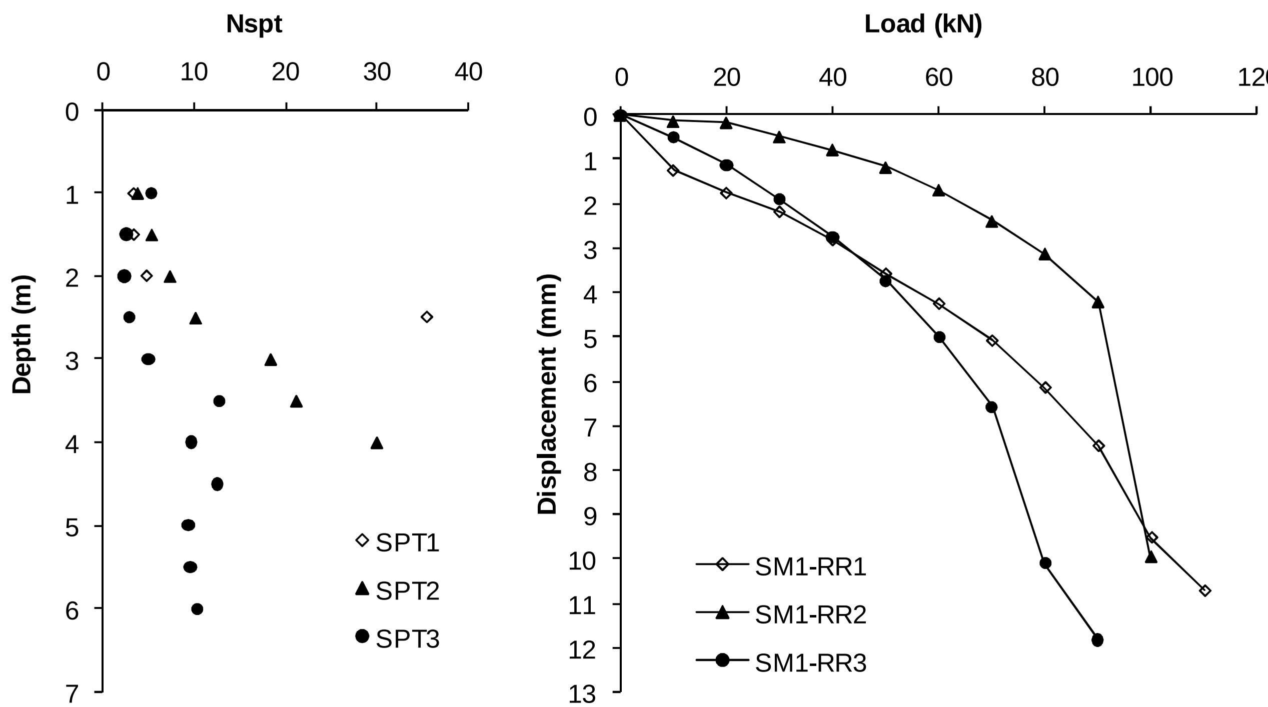

Figure 7: Sample site of S. Marino (site 1): (a) Nspt data and (b) experimental results of the compression tests.

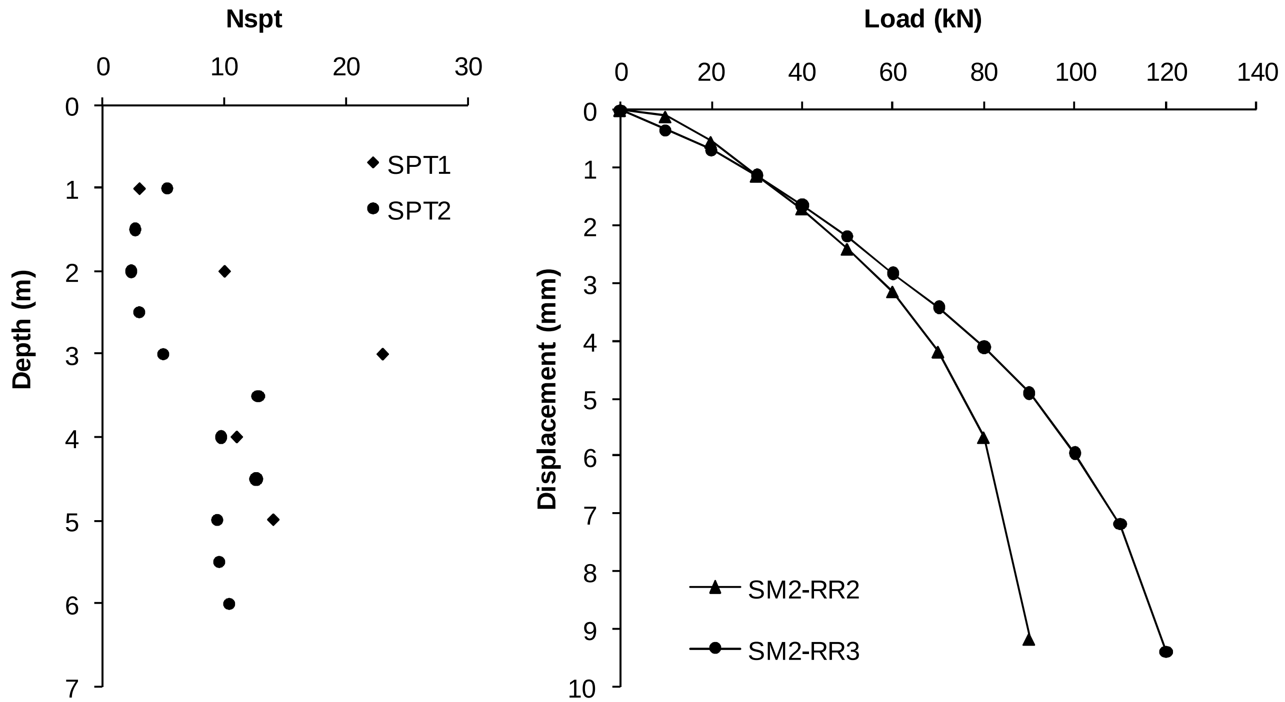

Figure 8: Sample site of S. Marino (site 2): (a) Nspt data and (b) experimental results of the compression tests.

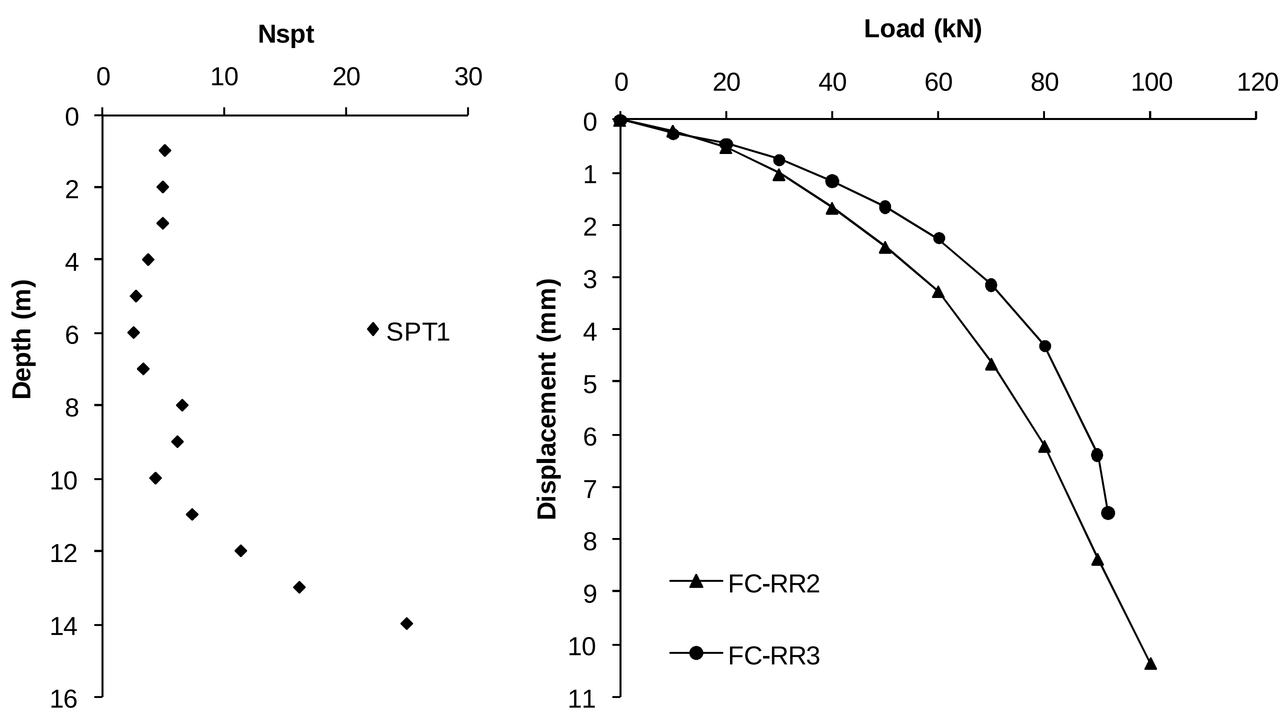

Figure 9: Sample site of Cesena (FC): (a) Nspt data and (b) experimental results of the compression tests.

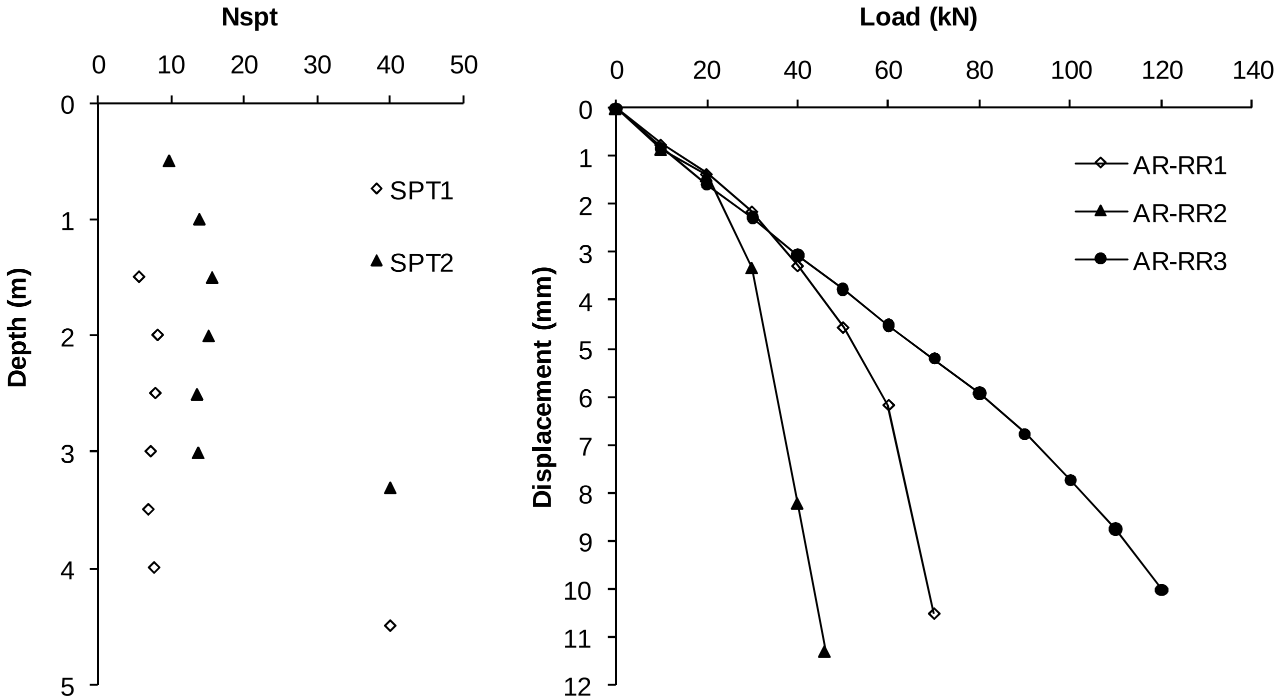

Figure 10: Sample site of Lucignano (AR): (a) Nspt data and (b) experimental results of the compression tests.

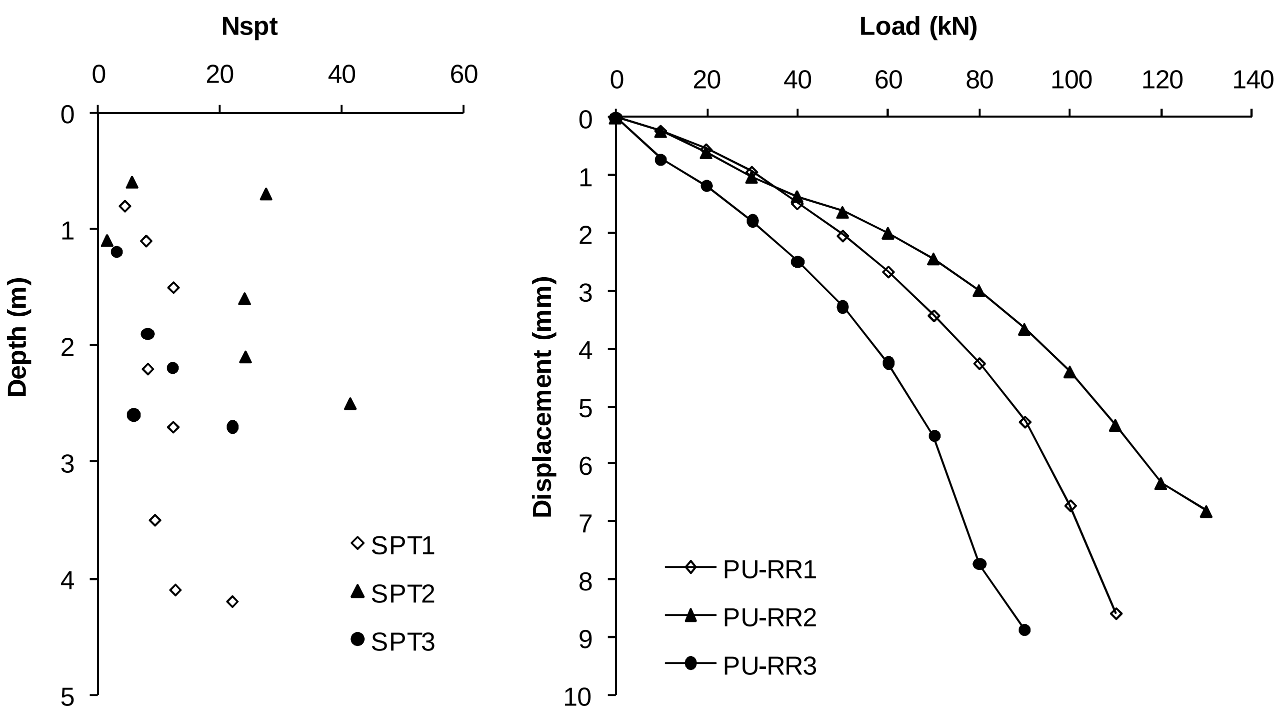

Figure 11: Sample site of S. Leo (PU): (a) Nspt data and (b) experimental results of the compression tests.

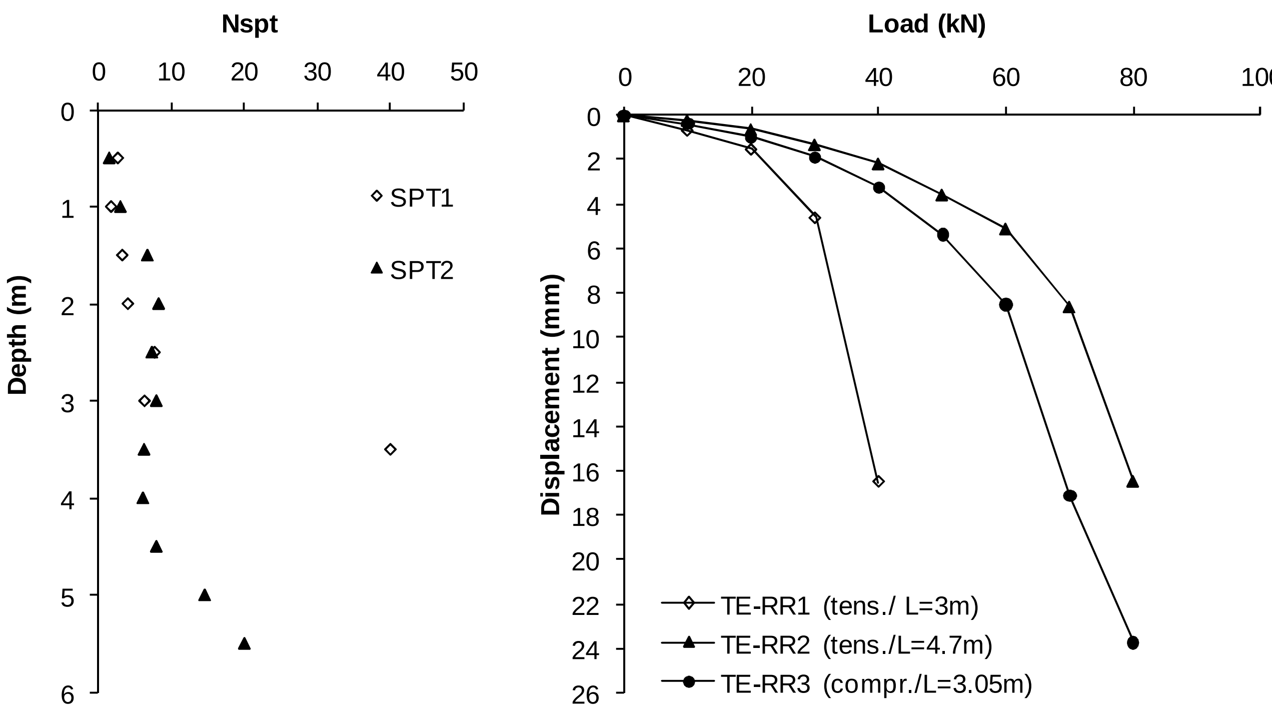

Figure 12: Sample site of Teramo (TE): (a) Nspt data and (b) experimental results of the load tests in tension (RR1 and RR2) and in compression (RR3).

The results of load tests have been analyzed in order to identify the ultimate load value (Qlim) for each test (Table 3). The AGI recommendations suggest two different methods to determine the value of Qlim. The first method defines as limit load the load in correspondence of which a settlement of the order of 0.1d is measured, where d is the pile's diameter. The second method assumes the limit load as that in correspondence of which the settlement of the pile top reaches a quantity equal to 2 , where is the settlement under the load of 0.9·Qlim. In this context, the limit compression load has been evaluated through the application of both the described methods, making an average of the obtained results and, by comparison, also through the method of hyperbolic interpolation, proposed by Mandolini (1995). The data referring to the 18 micropiles tested under compression, in different sample sites, are reported in Table 3. The compression load tests allowed both to directly determinate the bearing capacity of each micropile and to use the collected data to elaborate a simplified design method.

In the sample site of Teramo, two RR micropiles of different length (TE-RR1 and TE-RR2) were tested under tension in order to evaluate the total shaft resistance. The tension tests were performed using the load device shown in Figure 13 and the results are reported in Table 3. By comparing the results of tests on micropiles TE-RR1 (tension) and TE-RR3 (compression), which had the same length (Table 3), it is possible to affirm that the total resistance in compression is about 15÷20% higher than the total resistance in tension. The higher shaft resistance measured under tension for the micropile TE-RR2 is due to its wider length. It is reasonable to consider the total resistance measured during a tension test equal to the total skin resistance in compression. Under this assumption, the test results support the hypothesis that the tip resistance is equal to 15% of the total skin resistance, as affirmed also by Bustamante and Doix (1985) for IGU and IRS micropiles.

Figure 13: "Castle" apparatus for the execution of tension tests on RR micropiles in the sample site of Teramo, Italy.

DESIGN METHOD FOR REINFORCED RESIN MICROPILES

After the analysis of load tests results, the behavior of reinforced resin micropiles appears intermediate between the behavior of grout injected micropiles of IRS typology and that of IGU typology, described by Bustamante and Doix (1985). Once determined the experimental value of Qlim for each micropile under compression, by taking into account its geometry and by considering P=0.15·S and consequently Qlim=1.15·S, the mean unit shear resistance (s) has been determined.

Figure 14 shows the experimental results of field tests in terms of unit shear strength at interface (s) in respect to the mean value of Nspt determined for each sample-site. The values of s coming from field measures have been compared with the reference curves proposed by Bustamante and Doix (1985) for IRS and IGU micropiles realized in silty-clayey soils (Figure 14).

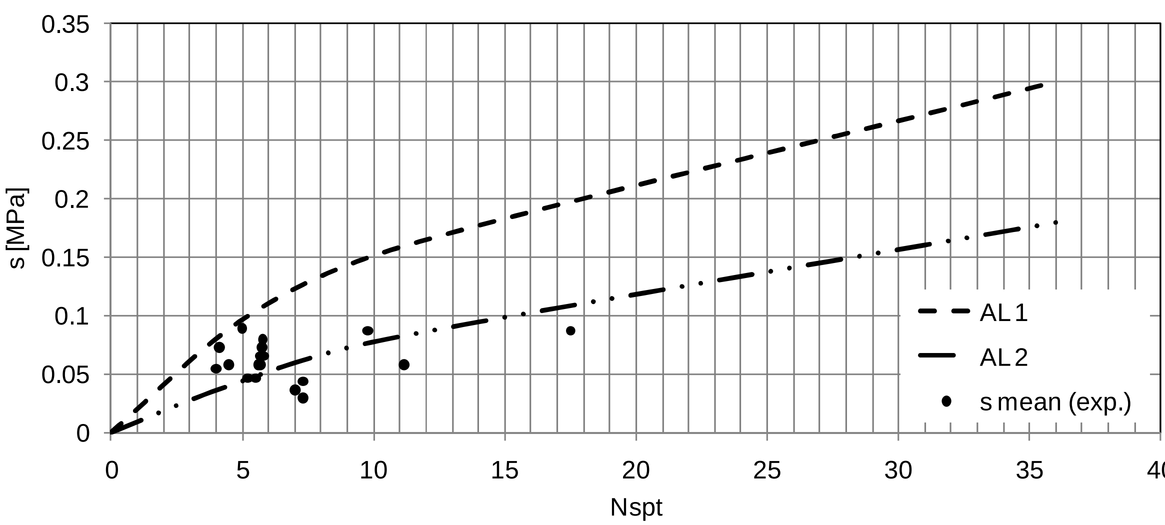

Figure 14: Comparison between experimental results of load tests (dots) and curves proposed by Bustamante and Doix (1985) for silty-clayey soils (AL1 refers to IRS typology; AL2 refers to IGU typology).

Therefore, in analogy with Bustamante and Doix's method, for reinforced resin micropiles it is possible to retain valid the following calculation methodology for the ultimate load capacity, at least for micropiles in silty-clayey soils:

$$Q_{lim} = P + S \tag{4}$$

where

$$S = \pi \cdot d_s \cdot L_s \cdot s \tag{5}$$

In Equation (5) ds is the equivalent-diameter of the micropile, Ls is the length of the injected pile, s is the unit shear resistance at soil-micropile interface.

It is assumed ds= ·d, where d in the diameter of the borehole, which is equal to 85mm for reinforced resin micropiles, and =1.5. This value of has been chosen on the basis of acquired experimental results and allows to obtain bearing capacity values that can be considered rather conservative as with regards to the real behavior of pile-soil system when it reaches its ultimate strength.

The tip resistance (P) for micropiles that do not reach layers with good mechanical characteristics is considered equal to 15% of lateral total resistance S (Bustamante and Doix, 1985; Viaggiani, 1999). This hypothesis has been confirmed by the tension tests, the results of which are reported in Table 3. So it is possible to write:

$$P = 0.15 \cdot S \tag{6}$$

The total bearing capacity of a RR micropile can be reasonably determined by the following expression:

$$Q_{lim} = 1.15 \cdot s \cdot L \cdot \pi \cdot \alpha \cdot d \tag{7}$$

where s is the unit shear resistance at soil-micropile interface, *L* is the length of the injected pile, d is the diameter of the borehole.

It is possible to evaluate s using the curve proposed by Bustamante and Doix (1985) relative to silty-clayey soils (Figure 14). In particular, with regard to the geotechnical characterization of the soil, the method refers to a direct correlation with the results of a Standard Penetration Test (SPT), which is widely used. But it is also possible to obtain similar results using Cone Penetration Test (CPT).

It is worth noting that such kind of micropiles are mainly set up in superficial soils, which are known for their high spatial variability. For this reason, it appears appropriate to refer to soil investigations and field tests that have been carried out very near to the location of the micropiles, in order to obtain a more suitable evaluation of their bearing capacity.

CONCLUSIONS

The calculation method proposed by Bustamante and Doix (1985), which derives from a data set obtained by tests in real scale on tie-rod and grout injected micropiles, has shown to be the natural reference for the determination of bearing capacity of reinforced resin (RR) micropiles.

A number of compression tests carried out on RR micropiles in silty-clayey soils have demonstrated how for this kind of micropiles a similar methodology for the calculation of their bearing capacity can be adopted.

With regards to the RR micropiles, the proposed method allows to obtain values of total bearing capacity that result conservative as to the behavior of micropile-soil system in ultimate conditions.

In perspective, more results from experimental tests in different sample sites are needed, for the evaluation of bearing capacity of RR micropiles in soils of different typology.

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