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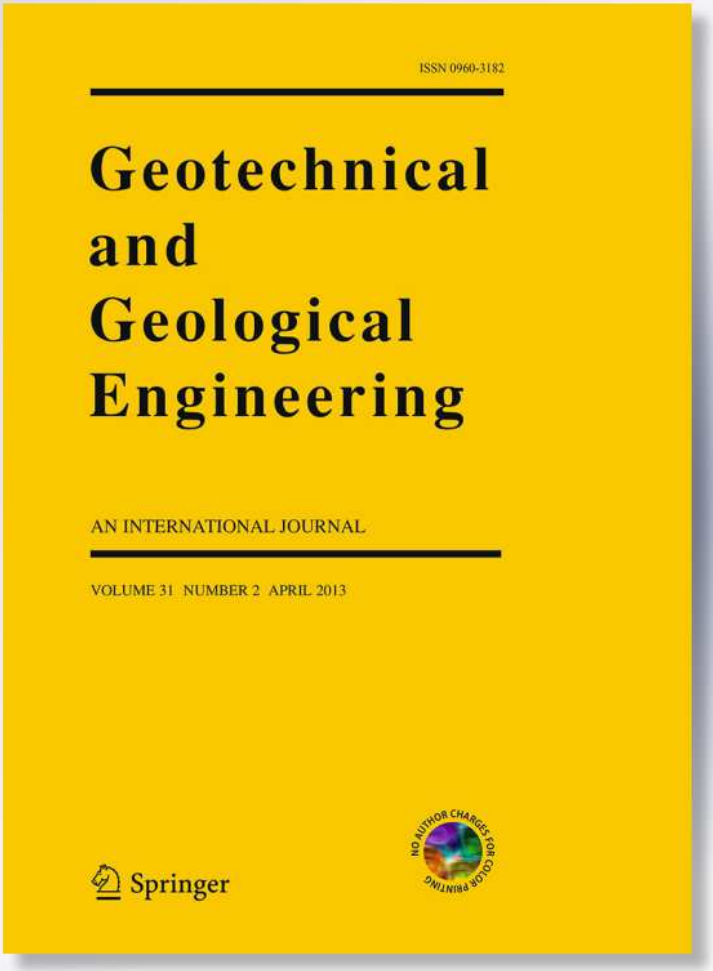

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Keywords Micropiles Á Polyurethane resins Á Mechanical properties Á Microstructure Á Interface

R. Valentino (&) Á E. Romeo
Department of Civil, Environmental, Territory Engineering and Architecture, University of Parma, Viale G.P. Usberti 181/A, 43100 Parma, Italy
e-mail: roberto.valentino@unipr.it

A. Misra
Civil, Environmental and Architectural Engineering Department, The University of Kansas, 1530 W. 15th Street, Lawrence, KS 66045-7609, USA
e-mail: amisra@ku.edu

1 Introduction

Different innovative technologies are employed 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 traditionally used in many applications of ground improvement to reduce settlements of shallow foundations of existing buildings.

For the time being, micropiles are the most commonly adopted, for example, especially for monuments and old buildings (Bruce 1989; Han and Ye 2006a, b; Fross 2006). Moreover, the micropile technique is particularly diffused in Italy, where it was introduced since the early 1950s by Lizzi (1980). Despite of a rather wide application, their reliability is not always sufficiently investigated. The design of

such structures is commonly based on empiricism and past experience, although a standard methodology of foundation reinforcement requires a study that provides a sound scientific base for a design procedure. On the other hand, a highly reliable technology and an in-depth knowledge of mechanical phenomena that can occur in consequence of an underpinning work are required.

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 effect of micropiles.

Recently, some authors were interested in investigating, under experimental and theoretical viewpoints, the field behavior of micropiles made with different technologies. They also provided analyses aimed to define analytical solutions 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).

Traditionally, various kinds of micropiles are used in underpinning of existing foundations on problematic soils. Recently, an innovative technique has been set up to construct micropiles made with expanded polyurethane resin (Novatek 2007). On the other hand, it is well known that polyurethane foams are widely used among foundation remediation techniques and, from a geotechnical perspective, they can be considered between underpinning and grouting (Buzzi et al. 2008, 2010). Nevertheless, a few data are available in the literature on the characteristics and use of polyurethane resins.

This paper shows the results of a research activity aimed at identifying the main mechanical characteristics of these innovative micropiles, thus determining their applicability.

The reinforced resin (RR) micropile behavior has been firstly analyzed through a series of field loading tests, and, on the basis of experimental results, a method for the calculation of the load-bearing capacity has been defined (Valentino and Stevanoni 2010).

The activity was then devoted to investigate the microstructure of the interface between soil and micropile and to determine the physical properties of its constitutive elements. These latter properties are evaluated with the scope of calibrating the parameters required for the analytical solution proposed by Misra and Chen (2004), in order to model the micropile–soil mechanical interaction. The characteristics of this type of micropile and of the applied model will be described later on in the paper.

2 Reinforced Resin Micropiles

Reinforced resin micropiles, which are recently patented products (Novatek 2007), have particular characteristics that make them well distinct in comparison with the more traditional use of expanding resins. Firstly, they are confined in a definite shape unlike expanding resins, which have been widely used by free injections in underpinning to limit structures' settlement. Similar to micropiles made through more traditional techniques, RR micropiles appear as structural elements that transfer external loads from the structure to the deeper layers of the soil, characterized by better mechanical properties.

Moreover, RR micropiles can be built by means of both less costly equipment than those required for other pile typologies, and small machines, characterized by a great flexibility of use.

In order to understand the peculiar characteristics of reinforced resin micropiles, it is necessary to describe their execution technique. The execution phases of reinforced resin micropiles are schematically reported in Fig. 1.

In the first phase, a hole with a diameter of 85 mm is made in correspondence of the foundation to be reinforced. The hole crosses the foundation and reaches the ground down to the desired depth. The borehole is made by using a hydraulic packer, 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, the hydraulic dilator device (packer) is used to expand the borehole to a diameter ranging between 100 and 120 mm. Consequently, the micropile reinforcement, which consists of an externally threaded steel tubular with a diameter of 60 mm and a thickness of 8 mm, 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), the resin is injected. Before strengthening, the resin spreads both inside the pile and in the space between the tubular and the surrounding soil (Fig. 2).

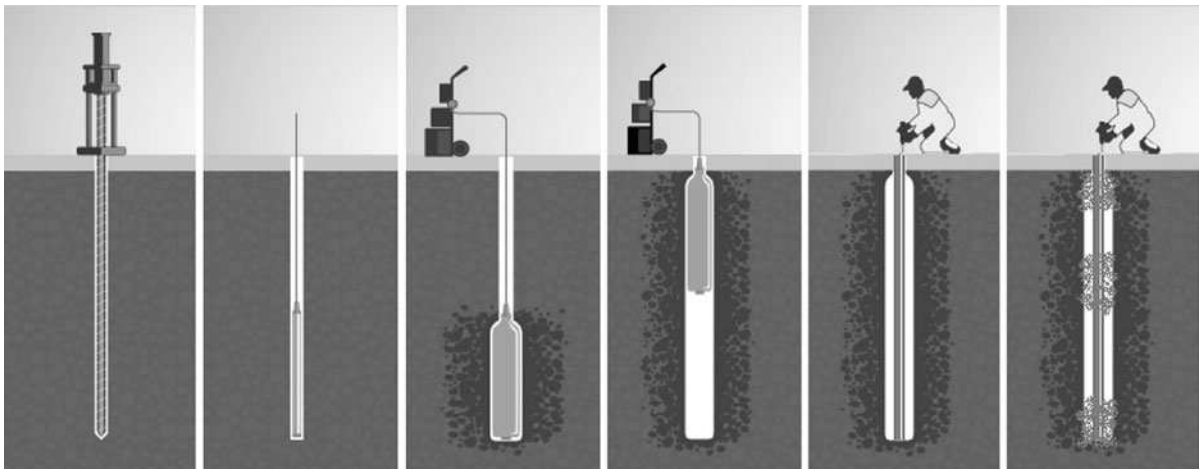

Fig. 1 Different phases in the execution technique for a reinforced resin micropile

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 reinforced resin micropile behavior.

The injected expanding resin, which is identified by the abbreviation HDR200 (high-density resin) for the analyzed micropiles, substantially constitutes a filling. 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) are combined, they give rise to a high-speed chemical reaction that quickly determines the expansion of the injected product, increasing up to 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 the solidification process. The different confinement conditions in the inner part of the steel tube and in the hollow space between the tube and the soil determine, in the same micropile, different expansion pressures. Final densities of the resin in the two zones are different as well. Further mechanical aspects regarding the same kind of resin can be found in Buzzi et al. (2008, 2010). The characteristics of the resin used for the injection of micropiles analyzed in this research have been reported in Table 1.

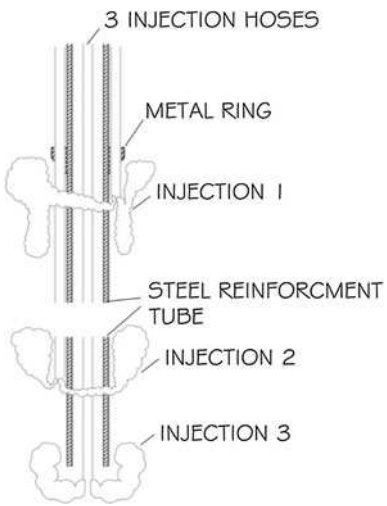

Fig. 2 Scheme of the execution technique for a reinforced resin micropile without shallow foundation

Table 1 Properties of the expanding polyurethane resin (Novatek 2007)

Property	Value
Mass per unit volume in free expansion (kg/m^3)	80–130
Temperature at mixing (°C)	30–50
Mixing pressure (bar)	40–60
Injecting pressure (bar)	5–10
Time of reaction (s)	6–10

Reinforced resin micropiles, built through the above-described process, provide resistance to the pile-soil system both along the shaft and by the tip.

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

1. the execution of the beaten borehole in a coarse soil above the ground water level provokes initial compacting of the soil, without material removal;
2. the action of the hydraulic packer radially dilates the borehole's wall, thus determining the ground tamping and a reduction in voids volume in coarse soils. This procedure allows to obtain a final diameter of about 120 mm, starting from an initial hole having a diameter of about 85 mm;
3. the expanded action of the resin causes an increase in 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, as it will be explained in the next section.

The resin can be injected in different ways in order to obtain different densities and, accordingly, different mechanical characteristics. The present study aimed to reach the following main goals:

a. qualitatively compare the characteristics of micropiles made of resins with different densities according to the results of field load tests;
b. investigate the main mechanical characteristics of the type of micropile that shows the best field behavior;
c. quantitatively determine the most important mechanical parameters of the investigated micropile;
d. calibrate the parameters required to model the micropile–soil mechanical interaction (Misra and Chen 2004).

3 Field Behavior Under Compression

A sample site in the municipal area of Bosco Chiesanuova (Verona, Northern Italy) was chosen to carry out the load field tests. In order to obtain a geotechnical characterization of the soil, 1 N_{SPT} and 2 CPT tests were carried out in the sample site, down to 9 m of depth from ground level. The results obtained are shown in Fig. 3. The geotechnical characterization of the site revealed the presence of a clayey silt with small sand lenses down to 5 m from ground level.

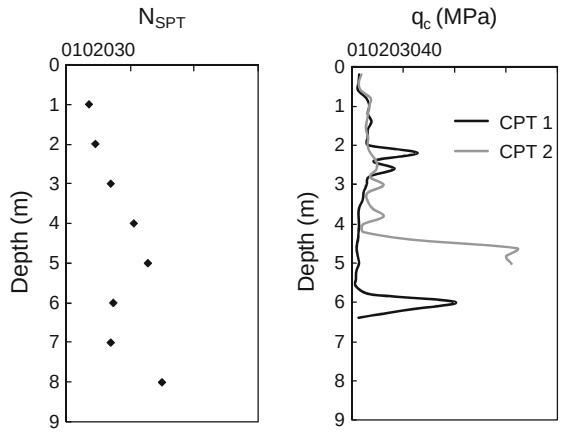

Fig. 3 N_{SPT} and CPT results in the test site of Bosco Chiesanuova (VR, Italy)

Three types of resin micropiles have been used for the load tests (Stevanoni 2009) without the presence of a shallow foundation.

3.1 Different Kinds of Micropiles

Three types of micropiles, identified through the capital letters F, S and M, respectively, have been considered in this study, and their characteristics will be described in this section. An horizontal cross section of each kind of these micropiles is shown in Fig. 4.

Micropiles F and S have been injected with a polyurethane resin having a relatively high density both inside the steel tube and in the space between the tube and the soil (Table 2). Micropile F reached a depth of 3.65 m, while micropile S reached a depth of 2.7 m from ground level. Notwithstanding the execution technique was the same for the two micropiles, the evaluation of the density of the resin in the inner part of the steel tube revealed that it was substantially different: 1,190 kg/m^3 for micropile F and 698 kg/m^3 for micropile S. How is possible to observe in Fig. 4, the "nucleus" of micropile F appears more homogeneous than that of micropile S (Fig. 4b); moreover, the latter is evidently composed by two parts

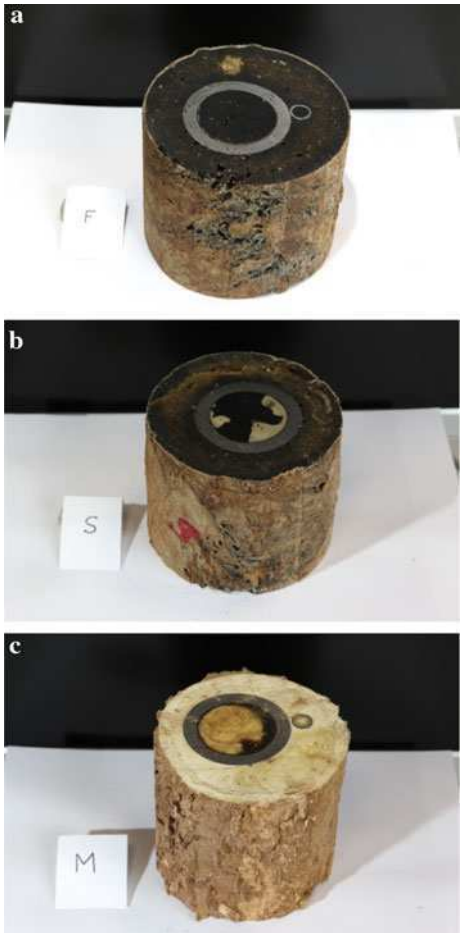

Fig. 4 Horizontal cross section of each type of analyzed micropile a F-type; b S-type; c M-type

characterized by a different color (black and brown, respectively)to which corresponds a very different density. The brown part is considerably less dense than the black one, and the two parts are not seeped through each other.A possible explanation is the difference between the total number of injections for each micropile and, then, the number of injection per unit length along the shaft.In fact, whereas three injections,corresponding to a mean of one injection each 1.2 m, were carried out for micropile F, only two injections,corresponding to a mean of one injection each 1.35 m, were carried out for micropile S. These differences between micropiles F and S reveal how a very important role is played by the execution technique,which can give different results in consequence of uncontrolled conditions. Moreover, the resin expanding inside the steel tube of micropile F reached a very differentdensity with respectto the resin that expanded along the hollow space between the tube and the surrounding soil.In particular, the outside resin density was almost one-third of the inside resin. This difference appeared less marked for micropile S, where the inside resin density was almost 42 % higher than the outside resin. It is worth noticing that the higher the resin density, the lower its expanding power.

Micropile M was made with a polyurethane resin having a relatively low density and a high expansion power.Also in this case,the evaluation of the resin density revealed a substantial difference in the inner part of the steel tube (1,024 kg/m^3) with respect to that between the steel tube and the soil (130 kg/m^3). This marked difference is prevalently due to the steel tube, which blocks any radial strain to the resin expanding inside. For this, the radial confinement is much higher inside the tube than in the hollow space between the tube and the soil.

3.2 Field Load Tests

For the execution of the compression teston each micropile, a contrast structure appropriately anchored to the ground with two other reaction micropiles has been used. Two contrast beams placed at a distance of 30 cm with 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 a hydraulic jack, thus laterally transferring the forces to the contrast beams (Fig.5).

After the castle has been fixed to the bars, a jack and a load cell have been placed coaxially with 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.

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 ofAGI (Italian Geotechnical Association) (1984) and of ASTM D1143 M-07.In particular, a load increase of 10 kN has been applied to each step, while the values of settlementson the

Table 2 Properties of the tested micropiles

Micropile ID	Length (m)	Diameter (mm)	Depth of pile head from ground level (m)	Mean density (kg/m^3) (hollow space)	Mean density (kg/m^3) (inner space)	P_u (kN)
RR-F	3.65	120	0.1	400	1,190	100
RR-S	2.7	120	0.1	400	698	80
RR-M	5.1	120	0.1	130	1,024	140

Fig. 5 Apparatus with contrast beams for the execution of compression tests at Bosco Chiesanuova (VR)

comparator devices have been acquired at fixed time intervals: after 1, 5, 10, 15 and 20 min, starting from when a new load is introduced.

The experimental load–displacement curves obtained for the three sample micropiles are reported in Fig. 6.

The results of load tests have been analyzed in order to identify the ultimate load value (P_u) for each test (Table 2). The AGI recommendations suggest two different methods to determine the value of P_u. The first method defines as limit load as that 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 2d, where d is the settlement under the load of 0.9P_u. 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 compression load tests allowed to both directly determinate the bearing capacity of each micropile and use the collected data to elaborate a simplified design method (Valentino and Stevanoni 2010).

The diagrams reported in Fig. 6a reveal, in particular, that the ultimate load reached by the RR-F, the RR-S and the RR-M micropiles is of 100, 80 and 140 kN, respectively. In Fig. 6b, the same load–displacement curves have been normalized with respect to the length of each micropile. Notwithstanding the different length of the three tested micropiles, it seems that the different behavior is mainly due to their different characteristics. In particular, the stiffness of RR-M micropile appears much greater than that of the others. The different behavior seems due to the soil–micropile shear-boundary layer. After the tested micropiles had been removed from the ground, the analysis of the surface of their shaft revealed a quite different roughness for the three types of micropiles. In particular, the shaft of RR-F and RR-S micropiles was rather smooth (Fig. 3a, b), due to the relatively reduced expansion effect undergone by the dense resin. Figure 7 shows a particular of the cross section

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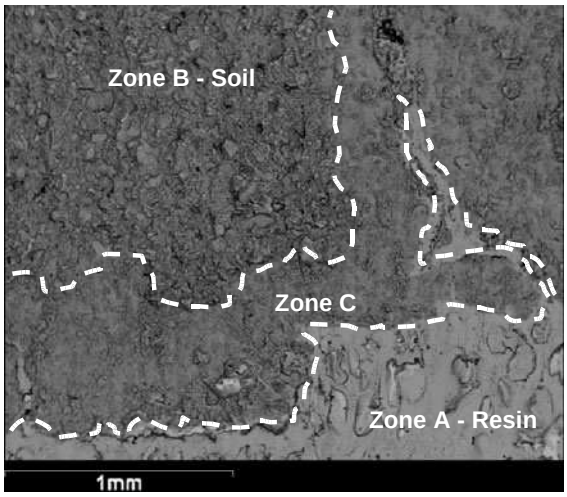

Fig. 16 SEM image of the Zone 5 of the specimen in Fig. 11

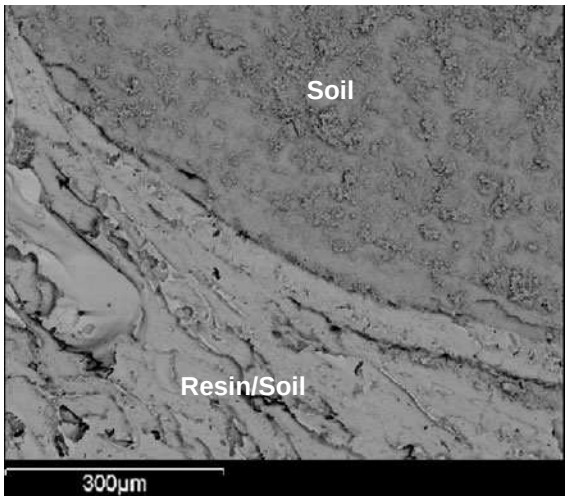

Fig. 18 SEM image of the Zone 1 of the specimen in Fig. 17

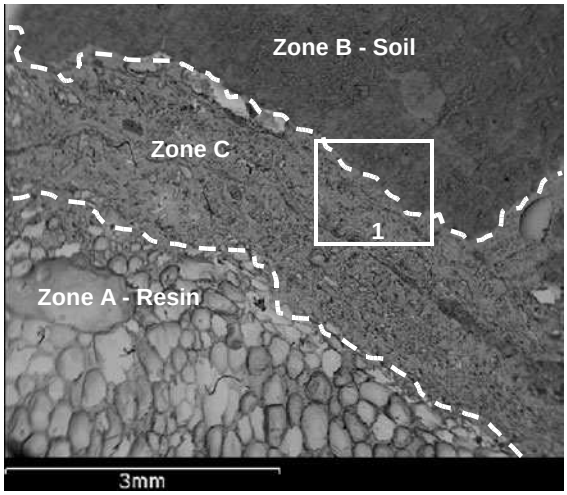

Fig. 17 SEM image of the Zone 5 of the specimen in Fig. 11

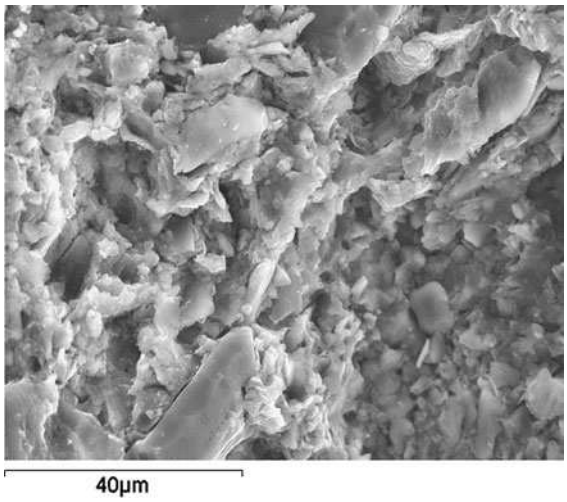

Fig. 19 The less dense structure assumed by the resin

and the soil (zone B). Figure 18 is a zoom image corresponding to the highlighted Zone 1 of Fig. 17. Even in a small section, such as that shown in Fig. 18, the resin appearscharacterized by a wide lack of homogeneity affecting both the density and the structure.

Figures 19,20 and 21 show,at the same enlargement level,three different structures assumed by the resin in different points of the micropile section.In particular, Fig.19, shows the less dense configuration, Fig. 20 shows the more dense structure of the resin, characterized by a vitreous consistency, and Fig. 21 shows the intermediate state. For the reinforced resin micropile of M-type, it has been observed thathe more dense structure of the resin corresponds to the zone that is close to both the steel tubular and the soil. In correspondence of these two zones, in fact, during the expansion phase,the resin meets a constraint, which provokes the closure of the ‘‘bubbles’’ and the occlusion of voids, thus assuming a more compacted structure.

It is possible to affirm that at the interface between the soil and the resin two main kinds of interactions are present:one of mechanicaltype and the other of chemical nature.These observations lead to suppose that the most part of the micropile strength under

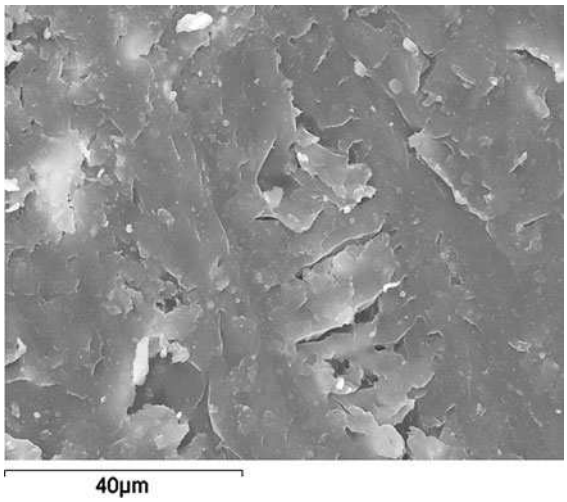

Fig. 20 The denseststructure assumed by the resin (vitreous consistency)

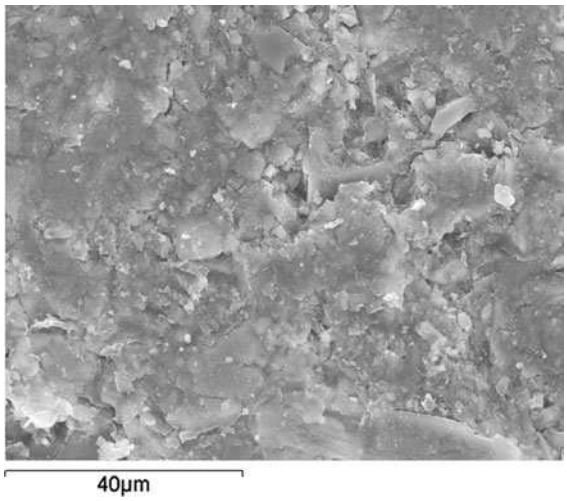

Fig. 21 Intermediate structure assumed by the resin

compression, which has been previously explained, is due to the peculiar characteristicsof the contact between the resin and the surrounding soil.

5 Mechanical Behavior of the Micropile's Nucleus

The mechanicalbehavior of the micropile's nucleus was investigated by performing laboratory unconfined compression strength tests (UCS) on cylindrical specimens,coming from the nucleus of the RR micropiles. The nine tested specimens (three

Fig. 22 Specimen of the micropile nucleus tested in the MTS

Table 3 HBM-Y series strain gauge specifications

Maximum elongation (lm/m)	50,000 (5 %)
Fatigue life	[10^7
Operating temperature range (°C)	-70… ? 200
Mechanical hysteresis (lm/m)	1

specimens foreach micropile type) had a diameter of 45 mm and a mean height of 100 mmThe mean density of the specimens is reported in Table 2The UCS is an axial compressiontest in which the specimen is provided with no lateralpressure while undergoing verticalcompression.The test was performed using an MTS closed-loop servo-hydraulic loading system.The load transmission occurs with a displacement control system, where the top plate drops with a 1.3 mm/min speed (ASTM D 695-08). Two HBM-Y series strain gauges,arranged in a quarter Wheatstone bridge, with a length of 8 mm are mounted on the centralsection of the specimen to measure radial deformations during testing (Fig. 22). Strain gauge specifications are listed in Table 3.

Strain gauge signals are acquired by a National Instrument SCXI Chassis which scans input channels at rates up to 333 kS/s.

A commercial software developed in a LabView environment was used to calculate stain gauge

parametersand acquire output signals. Failure is defined as the point on the stress–strain curve where the load reaches its maximum. The load to failure was recorded,and the unconfined compressive strength was computed as follows:

$$r_v ¼ P=A_h \quad ð1Þ$$

where r_v, unconfined compressive stress; P, load of the specimen; A_h, cross sectional area.

The test was performed on three replicates for each material. From the test results, it was possible to evaluate the behavior of each nucleus in terms of load displacement (vertical) response, stress–strain (radial) behavior and dissipated energy. Figure 23 shows the results in terms of load–displacement response: each curve is the mean behavior of the three tested specimens for each micropile type. It is clearly evident that the F nucleus exhibits a more brittle behavior compared to both M and S ones,which maintain a more ductile response, especially after the peak, where a residual resistance is asymptotically maintained. Moreover, even if M and S nuclei show a similar ductile trend, the S one results weaker than the M one.

Figure 24 shows the stress–strain response of the three materials. The S specimen exhibits a more sudden strength reduction, while M and F maintain a significantly higher residual resistanceover time, resulting in a higher failure strength for F specimen and a better response in terms of energy dissipation for both M and F specimens.

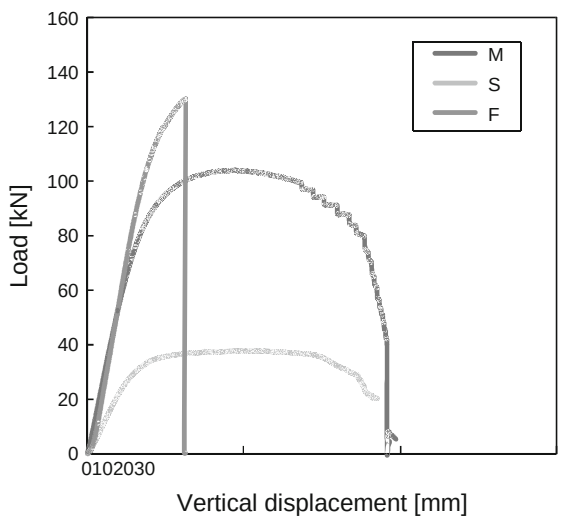

Fig. 23 Load–displacement response of the nucleus specimens

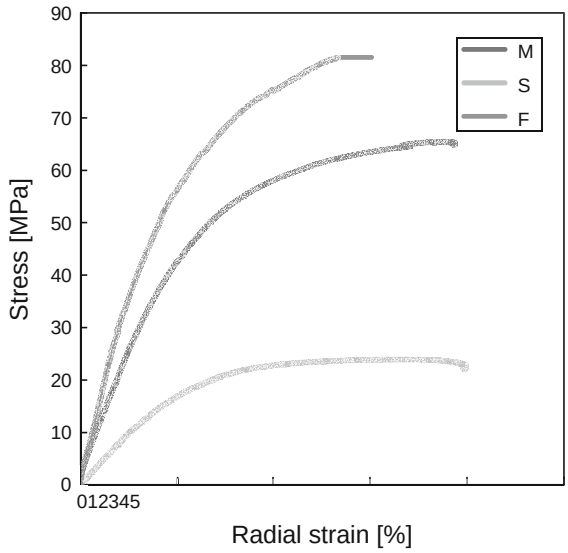

Fig. 24 Stress–strain behavior of the nucleus specimens

A further fundamentalproperty, named Fracture Energy, was evaluated from the test results. The Fracture Energy corresponds to the energy required to fracture the specimen, and it is easily determined as the area under the stress–strain curve. Table 4 compares the fundamental properties obtained from the UCS tests. Overall, the best-performing nucleus results the M one. Indeed, even if it exhibits a tensile strength 20 % lower than the F one, it fails at a critical strain significantly higher,resulting in a better resistance to damage, which is confirmed also by the higher Fracture Energy value. This means that the M nucleus has the ability to deform much more before the damage becomes great enough for causing the failure.

Finally, the results obtained from the unconfined compression strength tests were used to determine the stiffness of the three nuclei with the scope of calibrating the parameters required for the analytical solution proposed by Misra and Chen (2004). Young moduli of the three materials are listed in Table 4.

6 Analytical Evaluation of the Micropiles' Field Behavior

Although an extensive quantitative analysisof the micropile–soil mechanicalbehavior is beyond the scope of this paper, it is possible to perform an analytical evaluation of the micropiles' behavior

Table 4 Fundamental properties of the three micropile nucleus

Micropile nucleus	Max load (kN)	Displ. cr (mm)	Max stress (MPa)	Strain cr (%)	Fracture Energy (MJ/m)	Stiffness (MPa)
RR-M	104	9.4	65	3.85	19.2	0.7
RR-S	35	11	22	3.96	7.5	0.5
RR-F	130	6	81	3.00	17.6	0.2

observed during field load tests. A simple and reliable model of the micropile–soil mechanical interaction has been developed by Misra and Chen (2004). This model can be considered suitable for RR micropiles as well. As described in Misra and Chen (2004), the following assumptions are made: (a) a soil–resin shear-boundary layer is formed as the micropile is subjected to loading; (b) the soil–resin interface behaves as an ideal elasto-plastic material, both in drained and undrained conditions. Moreover, on the basis of results reported in the previous section, the RR micropile itself can be assumed to behave elastically throughout considering that the load required to reach the soil–resin shear-boundary layer yield strength is much smaller than that required to yield the inner nucleus of the micropile reinforcement.

A detailed derivation of the model is presented elsewhere in Misra and Chen (2004); however, the necessary relations required for completeness are provided here. From the force balance of the soil–micropile interaction of an incremental slice, Dz, in the elastic and the yielded bonded (interaction) zones, respectively, the following equations are obtained:

$$\frac{d^2u}{dn^2} - k^2 u(n) = 0 \quad \text{for } 0 \le n \le n_o \qquad (2)$$

and

$$\frac{d^2u}{dn^2} - k^2 u_o = 0 \quad \text{for } n_o \le n \le 1 \qquad (3)$$

where a non-dimensional length, $n = z/L_b$, is used. In Eqs. 2 and 3, u(n) is the micropile deformation, $u_o = q_o/K$ is the micropile–soil interface displacement at yield, n_o is the location of yield zone along the micropile, and k is given by:

$$k^2 = \frac{KL_b^2}{K_m} \qquad (4)$$

where L_b is the micropile bond length, K_m is the composite micropile axial stiffness, and K is the shear modulus of micropile–soil interface sub-grade reaction. To ensure continuity of displacement and equilibrium at the transition point n_o, the following boundary conditions are satisfied:

$$u(n_o)^E = u(n_o)^P \quad \text{and} \quad \left.\frac{du(n)}{dn}\right|_{n_o}^E = \left.\frac{du(n)}{dn}\right|_{n_o}^P \qquad (5)$$

where superscript E refers to the elastic zone and superscript P refers to the yielded zone along the shaft. For convenience, the following factors are also defined:

$$a = \frac{P_u L_b}{K_m}; \quad \text{and} \quad b = \frac{K_t}{KL_b} \qquad (6)$$

where P_u is the micropile ultimate pullout capacity (or side resistance) given by:

$$P_u = \pi D s_u L_b \qquad (7)$$

where D is the micropile diameter in the bond zone, and K_t is the tip soil stiffness. Equations 2 and 3 are solved under the boundary condition of applied load P at n = 1, and tip load $P_t = K_t u(0)$ at n = 0. The resulting solution for micropile displacement, u(n), can be used to find the displacement of pile head, u(1), as a function of the applied load, P. A non-dimensional form of the micropile head displacement is written as follows:

$$\bar{u} = \frac{u(1)}{a} = \frac{1}{2}\left[1 - n_o^2 - 1 - \frac{P}{P_u}(1 - n_o)\frac{1}{k^2} + \frac{P}{P_u}\frac{L_d}{L_b}\frac{K_m}{K_c}\right] \qquad (8)$$

where the last term in Eq. 8 gives the deformation of the top non-interacting zone (of stiffness K_c) often assumed for all types of deep foundations, and the location of the yield zone, n_o, is obtained by solving the following identity

Table 5 Micropile load–displacement parameters for use in Misra–Chen model

Micropile	d (mm)	L_b (m)	L_d (m)	K_m (MN)	K_c (MN)	K (MPa)	s_u (MPa)	P_u (kN)
RR-M	120	5	0.1	300	300	2.0	74.2	140

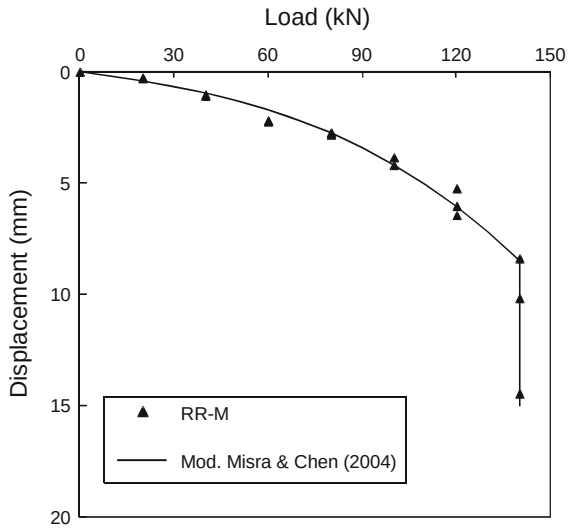

Fig. 25 Measured and predicted RR-M micropile compression behavior

$$\left(\frac{\tanh k\eta_0}{k} + \frac{P}{P_u}\right) - \frac{1}{\cosh k\eta_0}\left(\frac{b}{\cosh k\eta_0 + bk\sinh k\eta_0}\right) = 0 \quad (9)$$

When the micropile–soil interface yields completely (i.e., $\eta_o = 0$), the tip carries any additional applied load. In this case, Eq. 2 is no longer applicable, and only Eq. 3 governs the micropile deformation. The resultant non-dimensional micropile head displacement is given by:

$$\bar{u} = \frac{u(1)}{a} = \frac{1}{2}\left(\frac{P}{P_u} - 1\right) + \left(\frac{P}{P_u} - 1\right)\left(\frac{1}{k^2 b} + \frac{P}{P_u}\frac{L_d}{L_b}\frac{K_m}{K_c}\right) \quad (10)$$

To apply the above analytical solutions, the laboratory tests were performed to determine the micropile axial stiffness K_m. Parameter K_m may be obtained as the sum of the axial stiffness of the micropile reinforcement and the axial stiffness of the inner resin nucleus, considering that the axial stiffness of the external resin is relatively negligible. It is assumed that $K_m = A_s \cdot E_s + A_n \cdot E_n$, where A_s and A_n are the cross-sectional area, and E_s and E_n are the Young's moduli of the micropile steel reinforcement and nucleus, respectively (Misra and Chen 2004). The stiffness of the external resin grout, which comes from previous laboratory tests (Buzzetti et al. 2008), can be estimated as 0.26 MPa and, in comparison with the steel stiffness, can be reasonably neglected in the computation of K_m.

Figure 25 gives the comparison between the measured load–displacement curve for the RR-M micropile and the calculated curve obtained by the Misra and Chen model. The measured load–displacement relationship is depicted by symbols, while the line shows the calculated relationship. The input parameters of the model are reported in Table 5.

Further mechanical laboratory tests are needed to investigate the mechanical behavior and to determine the shear modulus of the micropile–soil interface.

7 Conclusions

Among others techniques, micropiles are traditionally used in many applications of ground improvement to increase the bearing capacity and reduce the settlement of existing shallow foundations. In this paper, the field behavior of an innovative micropile typology, made with reinforced polyurethane resin, has been considered. The construction method of this kind of micropile has been described. Three types of micropiles, characterized by different densities of the resin, have been considered in this study. They were installed in a sample site, characterized by the presence of silty-clayey soil, and subjected to the execution of field load tests. The micropile made with a polyurethane resin having a relatively low density and a high expansion power presented the best field behavior under compression. Only for this kind of micropile, the interface characteristics between micropile shaft and surrounding soil have been deeply investigated under a microstructural viewpoint. It has been shown that

such structures is commonly based on empiricism and past experience, although a standard methodology of foundation reinforcement requires a study that provides a sound scientific base for a design procedure. On the other hand, a highly reliable technology and an in-depth knowledge of mechanical phenomena that can occur in consequence of an underpinning work are required.

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 effect of micropiles.

Recently, some authors were interested in investigating, under experimental and theoretical viewpoints, the field behavior of micropiles made with different technologies. They also provided analyses aimed to define analytical solutions 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).

Traditionally, various kinds of micropiles are used in underpinning of existing foundations on problematic soils. Recently, an innovative technique has been set up to construct micropiles made with expanded polyurethane resin (Novatek 2007). On the other hand, it is well known that polyurethane foams are widely used among foundation remediation techniques and, from a geotechnical perspective, they can be considered between underpinning and grouting (Buzzi et al. 2008, 2010). Nevertheless, a few data are available in the literature on the characteristics and use of polyurethane resins.

This paper shows the results of a research activity aimed at identifying the main mechanical characteristics of these innovative micropiles, thus determining their applicability.

The reinforced resin (RR) micropile behavior has been firstly analyzed through a series of field loading tests, and, on the basis of experimental results, a method for the calculation of the load-bearing capacity has been defined (Valentino and Stevanoni 2010).

The activity was then devoted to investigate the microstructure of the interface between soil and micropile and to determine the physical properties of its constitutive elements. These latter properties are evaluated with the scope of calibrating the parameters required for the analytical solution proposed by Misra and Chen (2004), in order to model the micropile–soil mechanical interaction. The characteristics of this type of micropile and of the applied model will be described later on in the paper.

2 Reinforced Resin Micropiles

Reinforced resin micropiles, which are recently patented products (Novatek 2007), have particular characteristics that make them well distinct in comparison with the more traditional use of expanding resins. Firstly, they are confined in a definite shape unlike expanding resins, which have been widely used by free injections in underpinning to limit structures' settlement. Similar to micropiles made through more traditional techniques, RR micropiles appear as structural elements that transfer external loads from the structure to the deeper layers of the soil, characterized by better mechanical properties.

Moreover, RR micropiles can be built by means of both less costly equipment than those required for other pile typologies, and small machines, characterized by a great flexibility of use.

In order to understand the peculiar characteristics of reinforced resin micropiles, it is necessary to describe their execution technique. The execution phases of reinforced resin micropiles are schematically reported in Fig. 1.

In the first phase, a hole with a diameter of 85 mm is made in correspondence of the foundation to be reinforced. The hole crosses the foundation and reaches the ground down to the desired depth. The borehole is made by using a hydraulic packer, 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, the hydraulic dilator device (packer) is used to expand the borehole to a diameter ranging between 100 and 120 mm. Consequently, the micropile reinforcement, which consists of an externally threaded steel tubular with a diameter of 60 mm and a thickness of 8 mm, 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), the resin is injected. Before strengthening, the resin spreads both inside the pile and in the space between the tubular and the surrounding soil (Fig. 2).

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