Behaviour of Small Diameter Bored Piles under Axial Load

Comportement des Pieux Moulés de Petit Diamètre Axialement Chargés

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SYNOPSIS The results of a full scale investigation on two instrumented small diameter bored piles ("pali radice") are reported and analysed. Both piles are drilled in a volcanic subsoil, the first one floating in "pozzolana" and the second end-bearing to a tuff substratum; they have been tested under axial load, up to a maximum of 700 KN.
From the test results some interesting considerations on the behaviour of the micropiles, somewhat different from that of the medium or large diameter piles, are drawn. Particularly it is pointed out the occurrence of a side resistance much higher than that to be expected, according to the conventional bearing capacity theories, on the basis of the shearing characteristics of the sorrounding soil.
To clarify this particular behaviour further theoretical and experimental work should be necessary.

INTRODUCTION

The use of small diameter bored piles (micropiles) in foundation engineering dates to late thirties, when the firm FONDEDILE of Naples first patented his "palo radice" (Lizzi, 1964). Though they where originally developed for underpinning works, their field of application has progressively broadened, as their very high bearing capacity in comparison to the small diameter was recognized (Mascardi, 1968).
Such a bearing capacity must be ascribed mainly to side resistance, being the point resistance almost negligible; the mechanism of its development, however, is not yet completely understood. On the other hand, unlike medium and large diameter piles (Reese et al., 1973; Touma, Reese, 1974; Vesic, 1970; Whitaker, Cooke, 1966; Marchetti, D'Angelo, 1976), the difficulty of inserting instruments in a small section makes the available experimental evidence on the behaviour of such piles extremely scarce.
In the present paper the results of load tests on two instrumented micropiles, 127 mm in diameter, are reported and analysed as a first step toward a better understanding of their behaviour.

EXPERIMENTS AND RESULTS

The investigation has been carried out during the underpinning of an ancient building located in the historical centre of the town of Naples.
Subsoil conditions and soil properties are schematically represented in fig. 1, and are typical of the volcanic soils of Naples area (Pellegrino, 1967). Below about 2 m of paving, a layer of pozzolana with pumices 3 m thick is found, followed by two other layers of pozzolana separated by a thin intercalation of weathered pozzolana. At a depth of 17 m below ground surface a volcanic tuff substratum is found.

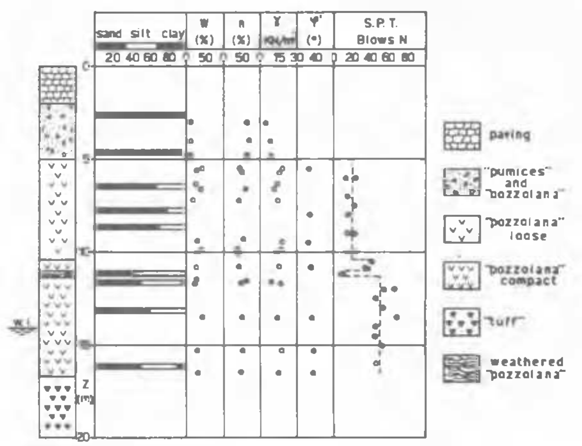

Fig. 1 Soil profile

It may be seen from fig. 1 that the two layers of pozzolana overlying the tuff are characterized by a markedly different penetration resistance, as expressed by the number of blows per foot N of the SPT.
The two test piles have been drilled with a temporary steel casing 127 mm in outer diameter, and employing the technique of "palo Radice" (Lizzi, 1970).

Pile n. 1 is 14,4 m long, ending in the lower pozzolana layer; pile n. 2 is 19,1 m long, and penetrates about two meters into the tuff.
Both piles are reinforced with a steel tube, 33,2 mm in outher diameter and 3,6 mm in wall thickness, inserted before grouting the piles with a malta of sand (75%) and pozzolanic cement (25%). During the injection, under a pressure of $0,1 \div 0,2$ MP_a, the volume of the malta has been continuously monitored in order to evaluate the area of pile section at different depths.
Laboratory compression tests on specimens of the malta, cured in conditions similar to those occurring in situ, gave on average a compressive strenght of 33 MP_a, and an elastic modulus of 21,500 MP_a.
It is to point out that in the upper part of both piles the shaft diameter has been enlarged to 280 mm and the casing has been left around the pile. This strenghtened part (1,9 m long in pile n. 1 and 1,3 m in pile n. 2) is intended as a preventive measure against bending under load.
Each pile has been instrumented with six strain-gaged bars (Gatti et al., 1980), compensated to be insensitive to temperature variation and to flexure, and with a pressure cell at the tip, fixed at the end of the reinforcement tube. The instruments (figs. 2 and 3) are manufactured by SIS-Geotecnica of Segrate.
The strains due to the setting and hardening of the malta, as measured by the strain-gaged bars after the installation, are plotted in fig. 4. It may be seen that the setting process is practically completed 20 to 30 days after casting, that is in a time much shorter than that needed for the concrete of cast-in-situ piles of medium and large diameter (Picarelli, Sapio, 1979).

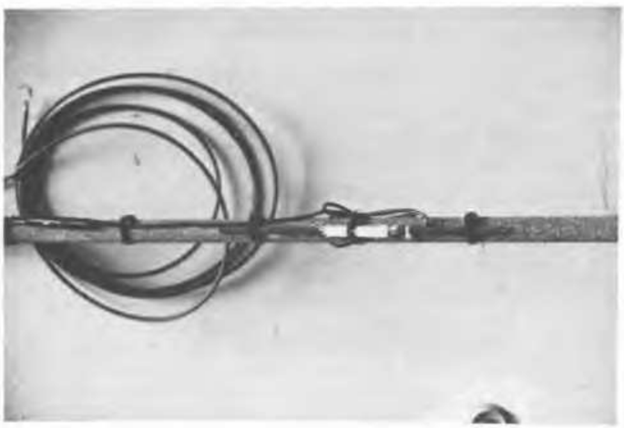

Fig. 2 Strain-gaged bar mounted on the steel reinforcement tube

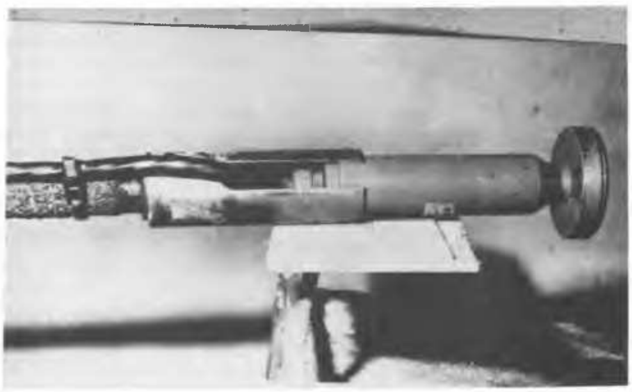

Fig. 3 Pressure cell at the end of the reinforcement tube

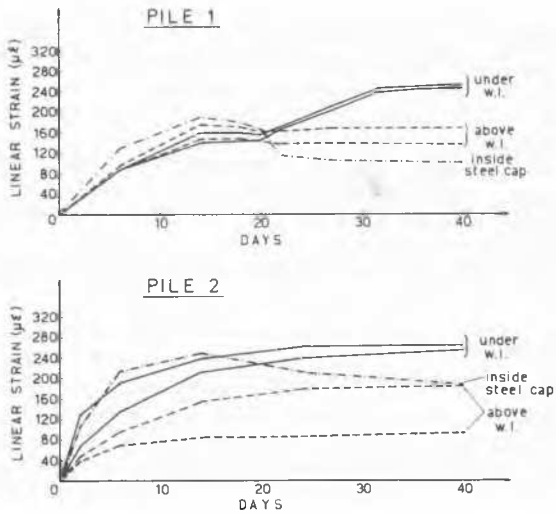

Fig. 4 Strain-time diagram during the setting and hardening of the malta

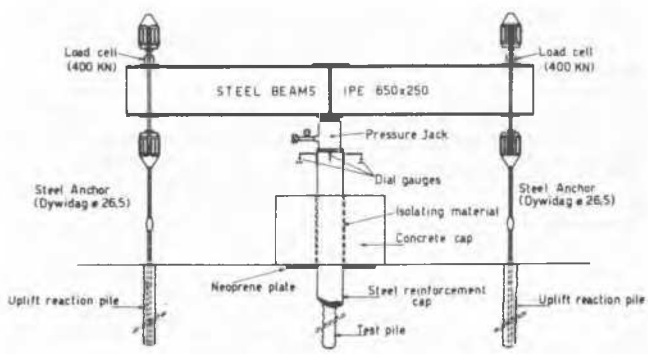

Fig. 5 Load test set-up

40 days after construction, the piles have been subjected to load test. The test set-up is represented in figs. 5 and 6; the reaction to the jack is provided by tension piles and the applied load is measured by means of two load cells with a capacity of 400 KN each.
Both tests have been carried to a maximum load of 700 KN without attaining the ultimate bearing capacity of the piles; a failure in bending occurred, at the maximum test load, near the top of pile n. 1.

Fig. 6 View of the load test set-up

The vertical displacements, measured at the pile head, are plotted in figs. 7 and 8 against the applied load.

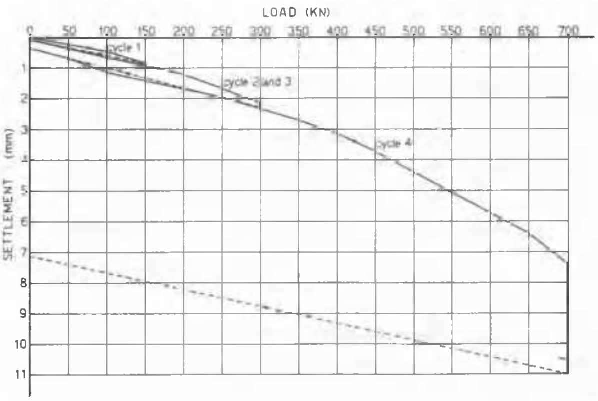

Fig. 7 Pile n. 1 - Load-settlement curve

The distributions of axial load along the pile shaft, as measured by the strain-gaged bars and the pressure cell, are reported in figs. 9 and 10. In the same figures, for each cycle, the residual values of the axial load after unloading are also reported (dash-dotted lines).
For loads above 300 KN a significant bending has been found to occur in the upper part of the piles, and the resulting flexure exceeds the compensation range of the instruments. Accordingly, in the 4th load cycle the measurements at the strain-gaged bars in the upper part of the piles have been disregarded, and the load distribution at that level has been obtained by interpolation.

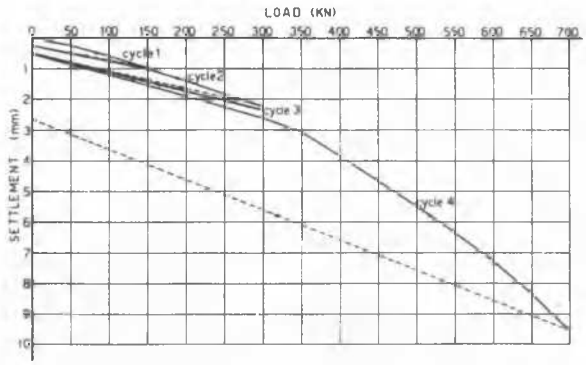

Fig. 8 Pile n. 2 - Load-settlement curve

ANALYSIS OF RESULTS

A simple look at the load test results, as reported in figs. 7 to 10, suggests some interesting comments.
First of all, the supposedly "floating" pile n. 1 and the supposedly "end bearing" pile n.2 exhibit exactly the same behaviour with respect both to load-settlement curves (figs. 7 and 8) and to load distribution along the shaft and at the base (figs. 9 and 10).
Furthermore, it is confirmed that the side resistance largely exceeds the point resistance, the latter being practically not yet mobilized at the maximum test load.
Finally, it is to point out that the ultimate bearing capacity has not yet been attained under a load as high as 700 KN.
Going now to a more detailed analysis of test results, in fig. 11 the vertical displacements measured at the pile head are compared to the elastic shortening of the pile. The latter has been obtained directly by strain measurements in the pile shaft; for the lengths of pile shaft near to the head and to the tip, where strain measurements where not anailable, the shortening has been computed referring to the measured values of axial laod, area of the pile section and elastic moduli of the malta (E=20.000 MP_a for the pile n. 1 and E=23.000 MP_a for the

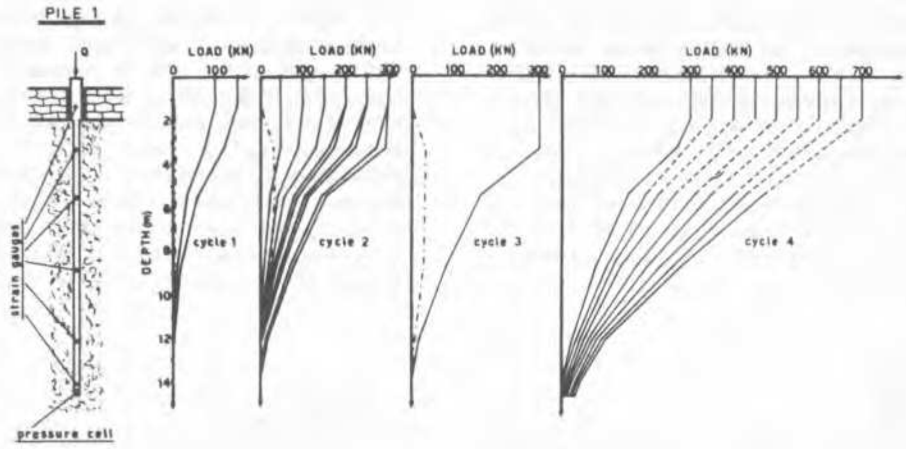

Fig. 9 Pile n. 1 - Axial load distribution along the pile shaft

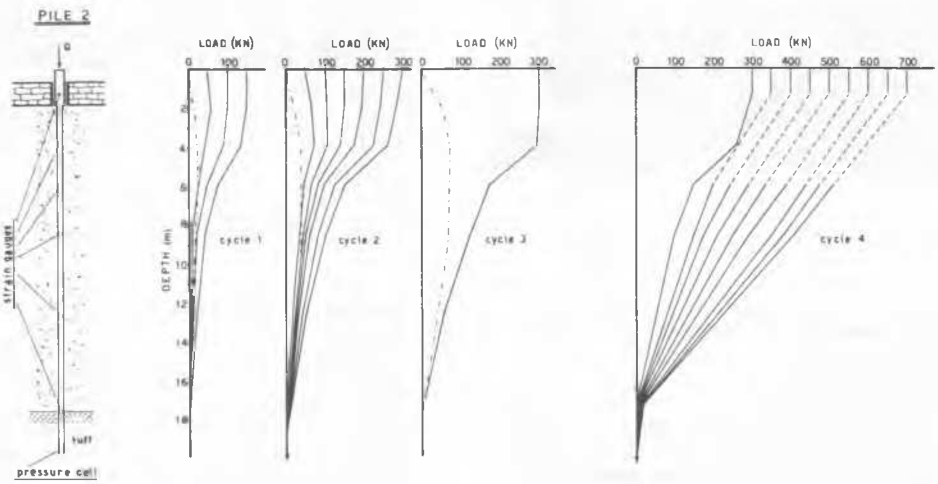

Fig. 10 Pile n. 2 - Axial load distribution along the pile shaft

pile n. 2).
Fig. 11 clearly shows that the settlement of the pile head is practically coincident with the elastic shortening. This means that the pile tips have not settled, even in the supposedly floating pile n. 1, in agreement with the negligible values of the point load.
Knowing the distribution of axial loads in the pile shaft, the average shear stress τ at the pile-soil interface for each pile element between two successive instrumented sections may be computed, and related to the corresponding vertical displacement s of the mid point of the element.
The curves τ -s (transfer curves) under increasing load for all pile elements between two successive instrumented sections have been fitted by means of a hyperbola(1) (figs. 12 and 13). It is then possible to extrapolate the transfer curves to evaluate the ultimate failure value τ_f of τ , that had not been attained in the tests.

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(1) As it is well known, the equation of hyperbola is :

$$\tau = \frac{s}{a+bs}$$

Plotting the experimental points in the plane (s, s/τ), they can be fitted with a straight line :

$$\frac{s}{\tau} = a+bs$$

thus allowing the determination of the parameters a and b of the hyperbola as the intercept and slope of the straight line.
The asintotic value of τ is given by

$$\tau_f = \lim_{s \to \infty} = \frac{1}{b}$$

The values of τ_f are listed in table I, together with the maximum values s_{max} and τ_{max} of the displacement and the shear stress attained in the tests. It may be seen that in the upper part of

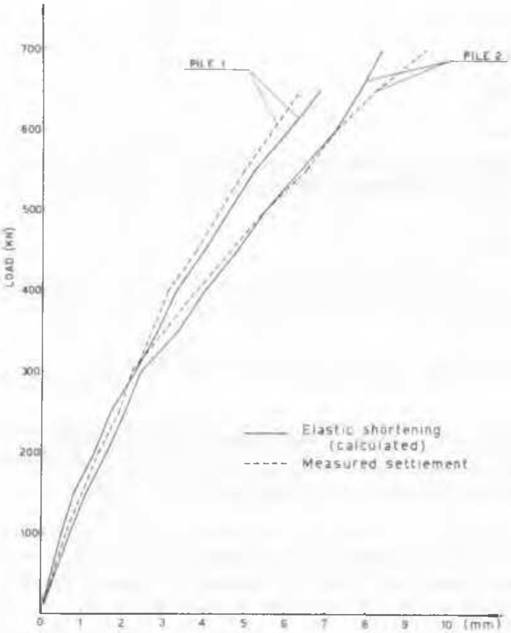

Fig. 11 Comparison between calculated and measured vertical displacement of the pile heads

the piles τ_{max} is very near to τ_f with s_{max} of 4÷6 mm; in the lower part, on the contrary, the mobilized τ_{max} is much lower than τ_f.
In the same table I the values of the overburden pressure σ_v at the depths considered, and the ratio τ_f/σ_v are also reported.

ry small values of the displacements, the ratio τ_f/σ_v ranges from 1,6 to 2,7.

TABLE I

Pile	Section from m	to m	τ_{max} (K Pa)	s_{max} (mm)	τ_f (K Pa)	σ_v (K Pa)	$\frac{\tau_f}{\sigma_v}$
1	0,00	3,40	119	4,28	125	47	2,66
	3,40	6,70	100	2,50	205	96	2,14
	6,70	10,00	96	0,90	314	144	2,18
	10,00	12,00	52	0,21	132	183	0,72
2	0,00	4,60	72	5,95	70	44	1,59
	4,60	7,20	72	3,88	177	96	1,84
	7,20	15,80	80	0,74	812	179	4,54

In the conventional bearing capacity theories for piles, the ultimate side resistance in cohesionless soils is expressed as :

$$\tau_f = K \mu \sigma_v$$

where μ is a coefficient of pile-soil friction and K is the ratio of the horizontal normal stress acting on the pile surface to the overburden stress.
For drilled cast-in-situ piles in cohesionless soils, customary handbook values are $\mu = tg\varphi$ and $K = Ko = 1-sen\varphi$ (Jaki, 1948). Being the friction angle of pozzolanas in the range 35°÷ 40°, this suggestion brings to a value :

$$\frac{\tau_f}{\sigma_v} = (1-sen\varphi)\ tg\varphi \cong 0,3$$

that is much lower than the experimental values listed in table I.
It must be concluded that conventional bearing capacity theories fail completely in predicting micropiles behaviour, and that further theoretical and experimental work is needed to develop a reliable design procedure for such piles.
On the other hand, it is to remember that the

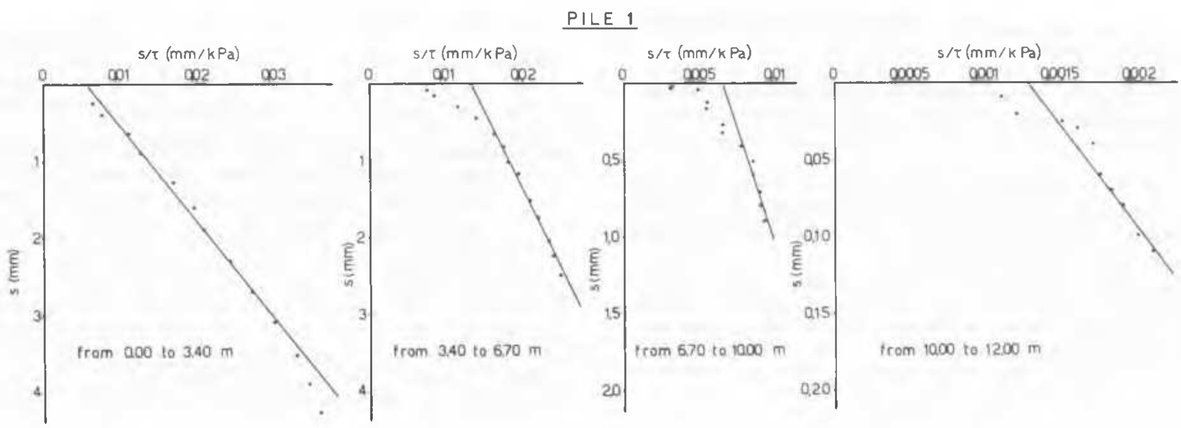

Fig. 12 Pile n. 1 - Extrapolation of the transfer curves by fitting with a hyperbola

If one disregards the values calculated for the elements near to the piles tip, where the extrapolation is higly unreliable because of the ve-

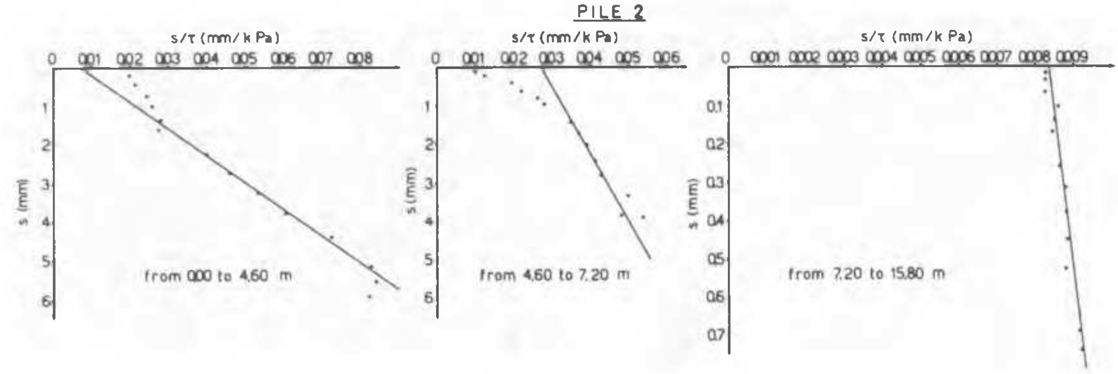

Fig. 13 Pile n. 2 - Extrapolation of the transfer curves by fitting with a hyperbola

ultimate load of these piles is often controlled by the strength on the pile section, rather than by the bearing capacity of the pile-soil connexion.

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