Non-linear analysis of the settlement of axially loaded piles

Analyse non-linéaire des tassements de pieux soumis à des charges axiales

Claudio Mancuso, Francesco Silvestri & Carlo Viggiani – *Dipartimento di Ingegneria Geotecnica, Università di Napoli Federico II, Italy*
Alessandro Mandolini – *Dipartimento di Ingegneria Civile, Seconda Università di Napoli, Italy*

SYNOPSIS: The paper analyses the load-settlement response of two piles, from the initial loading stages up to failure. The piles are steel tubes of 35/40 cm diameter, driven through 50 m of soft to medium plastic clay. The investigation included routine laboratory tests, CPT and DMT, field shear wave velocity measurements (CH and SASW) and laboratory cyclic/dynamic torsional shear tests. The analysis was carried out by means of a load transfer approach; the relevant soil parameters were obtained by the results of the above investigations. The results of the analysis are compared with experimental findings, discussing their sensitivity to the assumptions and values of parameters. Possible developments are finally outlined.

RESUMÉ: Cet article analyse la reponse charge-tassements de deux pieux d'essai jusqu'è la rupture. Les pieux sont des tuyaux d'acier du diamétre de 35/40 cm, battus à travers 50 m d'argiles molles. Les investigations ont inclus des essais de routine au laboratoire, des essais CPT et DMT, des mesures en situ de la vitesse des ondes de cisaillement de type CH et SASW et des essais de cisaillement dynamique et de torsion cyclique au laboratoire. L'analyse a èté effectuèe avec une approche a la transfert des charges; à travers ces investigations on a obtenu les paramétres plus importants du comportament du sol. Les resultats de l'analyse sont comparès avec ceux expérimentaux; leur sensitivitè aux hypothèses et aux valeurs des paramètres sont aussi dèscutès. Finalement, on souligne de possibles developpements de la recherche.

1. INTRODUCTION

For a safe and economic design of piled foundations an accurate prediction of the load-settlement behaviour of the pile is required.

Recent research on pre-failure behaviour of natural soils has considerably improved the capability of predicting the behaviour of geotechnical structures under working loads (see e.g.: Burland, 1989). In the field of deep foundations the available evidence is more scanty, apart from some significant exceptions (e.g. Randolph, 1994; Mandolini & Viggiani, 1996).

A chance of exploring the problem has been recently offered by the construction of a new bridge over the river Garigliano, 60 km north of Napoli, on the Thyrrenian coast in Southern Italy. The subsoil profile at the site is characterised by the occurrence of a thick layer of soft to medium plastic clays, whose properties were thoroughly studied by site and laboratory investigations (Mandolini & Viggiani, 1992). The bridge piers and abutments were founded on steel tubular piles with a diameter of 35/40 cm and a length of 48 m, driven by a mandrel and then filled by concrete. Some loading tests to failure on full scale instrumented test piles were also carried out (Bustamante *et al.*, 1994).

The paper reports the subsoil properties, as determined by updated site and laboratory investigations, and the results of two load tests on instrumented piles. The load-settlement behaviour of the piles is simulated by a load transfer approach, and the results are compared to the experimental data, discussing the influence of the various steps in the analysis and the choice of the relevant parameters. The present work is a part of a long-term research project aimed to the development of suitable practical procedures for the prediction of the behaviour of piles, either isolated or in a group, under working conditions and near to failure.

2. SUBSOIL PROPERTIES

The basic investigations at the Garigliano site included boreholes, CPTs, and routine laboratory tests (Mandolini & Viggiani, 1992).

The subsoil is rather homogeneous over the southern part of the 1.2 km long bridge. Accordingly, the data that will be presented refer not only to the exact location of the two test piles, in the vicinity of piers no. 18 and no. 20 (Bustamante *et al.*, 1994), but also to the surrounding area.

Figure 1 reports a profile of some physical and mechanical properties the Atterberg limits and natural water content versus depth, and the point resistance q_c of a number of CPTs. On the whole, the subsoil consists of rather homogeneous clays of medium plasticity and low to medium consistency. In the shallow desiccated crust ($0 \le z \le 5$ m) plasticity, consistency and strength are considerably higher.

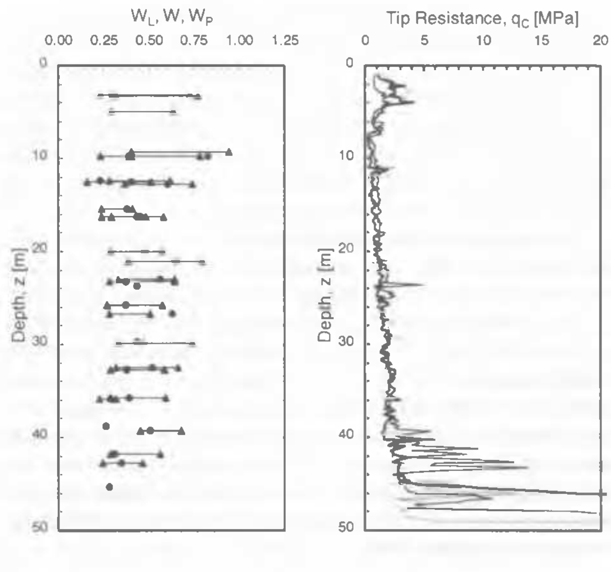


Figure 1. Atterberg limits, natural water content and q_c.

More recently, the site investigations have been supplemented by flat dilatometer (DMT) profiles and shear wave velocity measurements by multi-receivers Cross-Hole (CH) and Spectral Analysis of Surface Waves (SASW).

Details about the experimental procedures adopted for CH and SASW and a description of the measurements techniques are given by Mancuso (1992) and Mancuso & Vinale (1995). In this latter paper, it has been shown that CH measurements are significantly affected by the soil disturbance connected to the holes installation; accordingly, in the following only the CH results free from this effect and the SASW results will be considered.

In the laboratory, the small/medium strain behaviour of the soil has been investigated by cyclic and dynamic torsional tests on undisturbed samples, carried out by the Resonant Column-Torsional Shear (RC-TS) device described by Silvestri (1991).

Figure 2 reports the profile of the small strain shear modulus G_0, as obtained by the different field and laboratory techniques. The TS tests were driven at different frequencies, yielding results varying within a lower and an upper bound, which are indicated in the figure as LSR (low strain rate) and HSR (high strain rate).

The laboratory measurements have been plotted in Figure 2 by assuming $p' = \sigma'_{AV}$ where p' is the effective consolidation pressure in the laboratory tests and $\sigma'_{AV} = \sigma'_V (1+2k_0)/3$ is the mean effective stress in situ. In the evaluation of the latter, the values of k_0 back-figured from DMT results (Marchetti, 1980) have been used. The values of G_0 obtained by different techniques are in substantial agreement, with CH values slightly higher than SASW and RC-TS.

The stiffness profile is characterised by initially high values of G_0 in the desiccated crust and a subsequent sudden drop, after which there is a gradual increase with depth. Below the crust, the variation of G_0 is nearly linear and characterized by two branches (5÷24 m and 24÷50 m) with increasing gradients.

3. DEFINITION OF THE LOAD TRANSFER CURVES

The soil-pile interaction has been analysed by modelling the pile as a linear elastic body connected to a discrete number of nonlinear springs simulating the soil-pile interface. The load-displacement relationships of the springs are known as load transfer (or t-z) curves. Although this approach neglects the continuity of the strain field in the soil around the pile, it has been demonstrated that it is very effective in reproducing the behaviour of a single pile under axial load (Kraft *et al.*, 1981).

The RATZ computer code (Randolph, 1986) was used for the analyses. It assumes that the transfer curves have the general shape shown in Figure 3. The relevant parameters of the soil-pile interface are: the initial stiffness $[(\partial\tau/\partial w)_{w=0}]$, the peak ($\tau_y$) and the residual ($\tau_{res}=\chi_{res}\,\tau_y$) strength and the elastic limit ($\tau_e=\chi_e\tau_y$). The curve connecting τ_e to τ_y is assumed to be a parabola, while the curve linking τ_y to τ_{res} is an exponential function controlled by a displacement Δw_{res} and by a shape parameter η.

The assessment of these parameters was carried out in two different ways. In a first way (hypothesis 1), the limiting values of the lateral resistance were evaluated by the so-called "α-method" ($\tau_y=\alpha\, c_u$). The values of c_u were obtained by the CPT profiles $[c_u=(q_c-\sigma_v)/N_c]$, using the available laboratory values of undrained strength to select a value of N_c. The coefficient α was assumed equal to 0.5 in the overconsolidated crust and to 1.0 in the normally consolidated soils. The transfer curves were supposed to be non-linear from the origin (χ_e=0). All the other parameters were fixed following the literature; they are listed in Table I. In particular, χ_{res} was evaluated as suggested by Francescon (1982) and Chandler & Martins (1982).

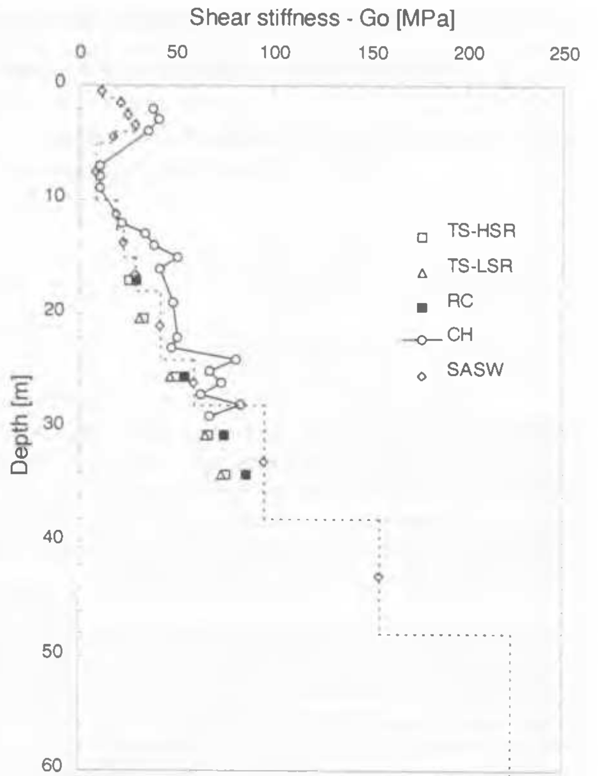


Figure 2. Variation of G_0 with depth from in situ and laboratory measurements.

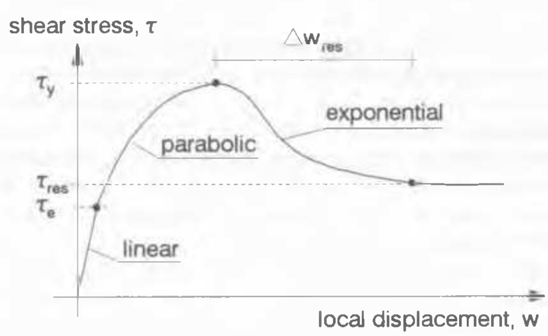


Figure 3. Load transfer curves assumed by RATZ code.

Table I - Selected values for the analysis.

		χ_{res}		η [(1)]	Δw_{res} (mm) [(1)]
Layer	z (m)	Hyp. 1	Hyp. 2	Hyp. 1	Hyp. 1
1	0÷5	0.50	1.0	0.5	50
2	5÷10	0.80	1.0	2.0	50
3	10÷24	0.93	1.0	2.0	50
4	24÷50	1.00	1.0	2.0	50
5	>50	1.00	1.0	2.0	50

(1) After Randolph (1986); not needed in Hyp. 2

To evaluate the parabola between the origin and τ_y, it is to remember that, according to Randolph & Wroth (1978), the strain γ in the soil adjacent to the pile-soil interface in a given point is proportional to the settlement w in the same point. This implies that a parabolic curve τ-w corresponds to a linear relationship between G/G_0 and shear strain γ.

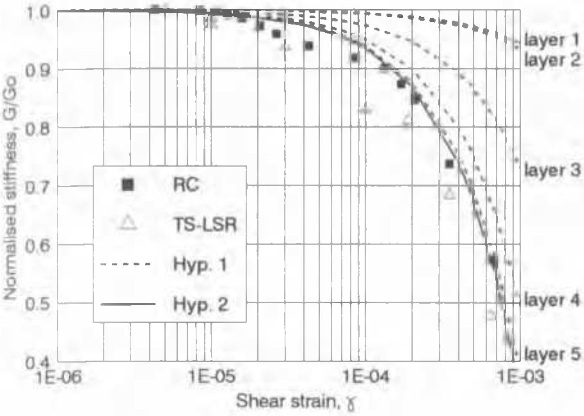


Figure 4. Decay of the shear stiffness with shear strain γ from RC-TS tests and hypotheses assumed for RATZ simulations.

The G/G_0:γ curves resulting from hyp.1 are plotted with thin lines in Figure 4 (in a semi-log scale) for the five different layers into which the soil profile was subdivided. In the same figure, the decay of G/G_0 with strain, as obtained by the torsional laboratory tests, is also plotted with single points.

It can be seen that the actual decay of the shear modulus with increasing strain, as obtained by laboratory tests, is definitely faster than that corresponding to hyp. 1. For this reason, it should be expected that the assumption of a parabolic load-displacement curve will result in a response of the system definitely stiffer than that predictable on the basis of the laboratory results.

To correct this discrepancy, an alternative approach (hypothesis 2) was devised. The parabolic τ-w function implemented in RATZ was fitted to the shape of the curve G/G_0:γ observed in the RC tests, yielding the thick line in Figure 4. As a consequence, values of τ_y definitely lower than in the previous case were obtained, especially for the stiff dry crust. Being these values too low for a realistic prediction of the ultimate load of the pile, no post-peak softening of the τ-w curves was considered.

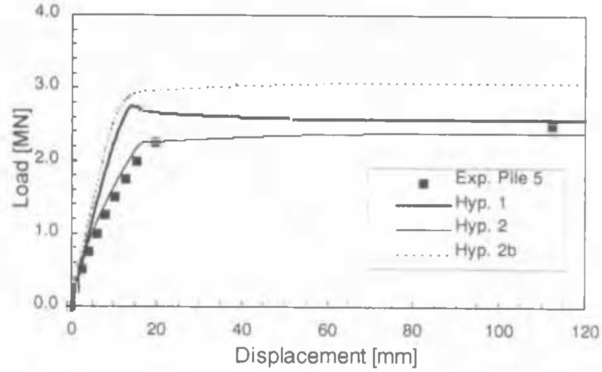
Because of the above assumptions, it should be expected that hyp. 1 is suited to predict the load-settlement response of the pile in the vicinity of the ultimate condition, while hyp. 2 can be thought as more effective at the working conditions.

4. NUMERICAL ANALYSIS

Theoretical and experimental research (e.g. Poulos, 1987, and Maiorano, 1996) has shown that the distribution of residual stress, generated in the pile-soil system during the installation of a driven pile, significantly affects the shape of the load-settlement curve. This load system, having zero resultant, can be predicted either by a dynamic analysis of the driving process or simulating it by a static load cycle to failure and back to zero load. Maiorano *et al.* (1996) shows that the two approaches lead to practically identical residual stress distributions.

In the present paper the static approach has been selected; the analysis of the load-settlement behaviour has been carried out starting from the stress state resulting from a previous load cycle to failu-re. To simulate the driving process, the pile has been modelled as a simple steel tube in the load-unload cycle, while during the subsequent load test the concrete-filled section has been considered. The RATZ code was used to model the load-settlement response for both the load-unload cycle simulating the installation and the analysis of subsequent load history.

The above procedure has been used to interpret the load tests on two instrumented piles (no. 5 and no. 6) close to pier 18 at the Garigliano site. Details of the instrumentation and load tests results are given by Bustamante *et al.* (1994). The piles were installed by driving with a mandrel two steel tube sections, each 24 m long; the lower one had an outer diameter of 0.356 m, the upper one of 0.406 m, both having a thickness of 6.3 mm. The two sections were welded in place through a connecting piece. After driving, a reinforcing cage was placed inside and the tubes were finally filled with concrete. From the data collected during the load tests, values of the equivalent Young's modulus of 26500 MPa for the pile no. 5 and 38000 MPa for the pile no. 6 were evaluated. The tip of pile no. 6 reached a dense sand layer, giving a point resistance much higher than that of pile no. 5; as a consequence, the measured ultimate loads were equal to 2.5 MN and 3.0 MN for the pile no. 5 and 6, respectively.



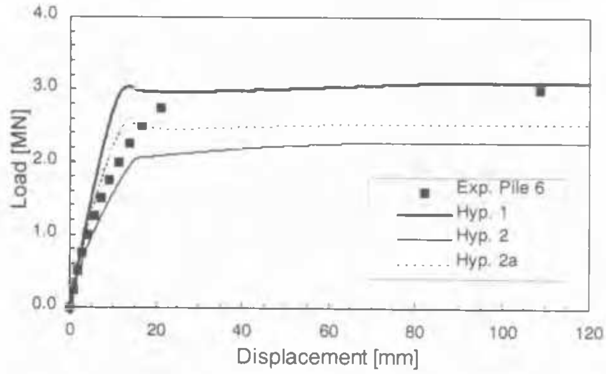
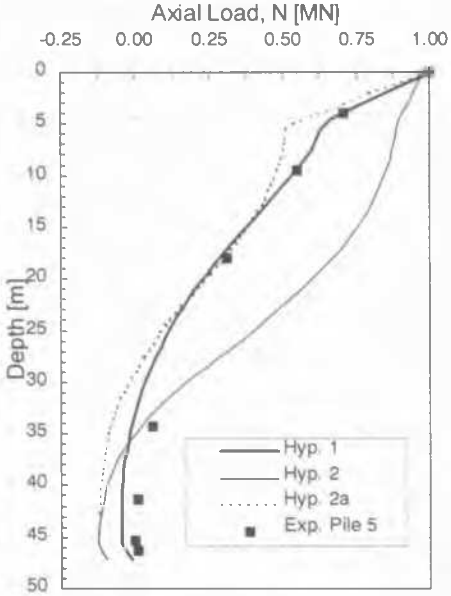


Figure 5. Load-settlement curves of pile no. 5 and no. 6.

The results of the two load tests, in terms of load-settlement response, are compared to those of the analyses in Figure 5. As expected, the solutions obtained under hyp. 1 satisfactorily predict the ultimate loads, but underestimate by around 40% the settlement at working load (~1 MN). On the other hand, the solutions obtained under hyp. 2 are slightly more satisfactory at working load, giving an overestimate of settlement of about 20%, but underestimate the ultimate loads.

In terms of load distribution along the pile shaft at working load, the comparison between experimental findings and results of the analyses is reported in Figure 6. It may be seen that the analyses carried out under hyp. 1 agree rather well with measurements, while hyp. 2 fails because it largely underestimates the contribution of the overconsolidated crust to the shaft resistance.

To improve the predictions, a modified version of hyp. 2 has been also tested; the results obtained are labelled 2a in Figures 5 and 6. In these analyses, the value of τ_y have been kept equal to that assumed in the hyp. 2, except for the dry crust where $\tau_y=\alpha c_u$ has been assumed as in the previous hyp. 1. It may be seen that the overall agreement between observations and analyses is



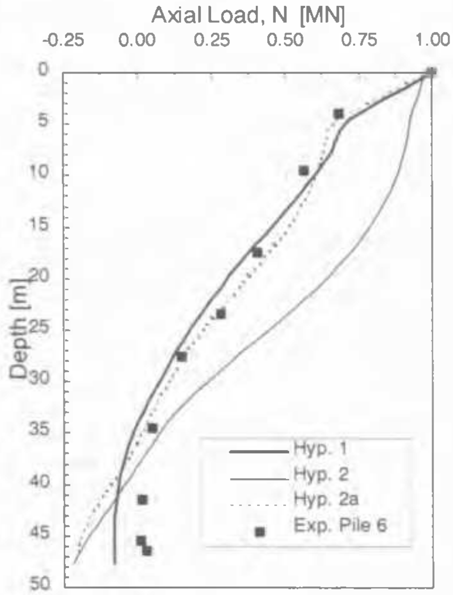


Figure 6. Load distribution along the piles at the working load.

slightly improved. A still better agreement might easily be obtained, in further class C predictions as the present one, by progressively tuning the values of the parameters.

5. CONCLUDING REMARKS

The application of a load transfer approach, combined with a procedure for the assessment of the relevant soil parameters, allows a satisfactory modelling of the load-settlement response of axially loaded piles from the beginning up to the ultimate load.

The assessment of the parameters of the model requires the determination of the variable stiffness of the soil in the small/medium strain range and the ultimate shaft resistance. It has been found that the results are sensitive to the values of parameters and to the shape of the non-linear transfer curves. From this latter point of view, the fit between the results of the analysis and the experimental data could be further improved by using functions other than the parabola implemented in RATZ. For instance, an interpolation of the laboratory results by spline or polynomial functions could be attempted.

The evidence presented refers only to driven piles in soft to medium clays. Before reaching any general conclusion, the outlined procedure has to be tested in different soils and with different installation techniques, to evaluate the influence of factors such as the roughness of the soil-pile interface, the residual stress system and the disturbance to the soil surrounding the pile.

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