Design of Axially-loaded Piles: Experimental Evidence from 400 Field Tests

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ABSTRACT: This work is aimed at furnishing an experimental support to the design of axially-loaded piles, taking advantage of an extensive database of pile load tests carried out in different sites nearby Napoli, in South Italy. Experimental data consist of nearly 400 full-scale pile load tests, some of them reaching large values of settlement. Different construction methods, including Non-Displacement, CFA and Displacement piles, have been used. The main results of the work consist in furnishing experimentally-derived rules and indications for pile design. With regards to failure loads, mobilization curves relating properly normalized values of load and settlement are proposed as function of the installation technique; indications on the bearing capacity of piles as function of geometry and technology are also provided. Initial stiffness of piles is investigated, identifying a rule of thumb for a rapid assessment, function solely of pile diameter and valid regardless of length and specific properties of pile and soil material.

KEYWORDS: Piles, Load test, Full-scale experiment, Axial stiffness, Axial bearing capacity

1. INTRODUCTION

It is well known that the prediction of the behaviour of pile foundations up to failure is a quite challenging task, since along with the difficulties associated with the complex soil behaviour and the limited knowledge of the subsoil constitution, additional source of uncertainties is due to the modifications induced by the pile installation process. All the above is reflected in design, which necessarily involves a simplistic modelling of the more complex real world. With reference to axial bearing capacity of the single pile, the main methods for estimating values of the unit base and the unit shaft resistance may be broadly classified in those based on fundamental soil properties (theoretical methods), such as angle of shearing resistance, and those based on in-situ test results (empirical methods), such as Standard Penetration Tests (SPTs) or Cone Penetration Tests (CPTs).

Although in the last decades significant insight has been gained about the processes governing the soil-pile system behaviour and despite the assessment of empirical ingredients of the design methods is continuously taking advantage from calibration against newly available experimental data, recent papers (e.g. Orr, 2016; Fellenius, 2017) demonstrate that our capability to evaluate pile response to loading is still far from being satisfactory for practical purposes on a specific project.

Orr (2016) analysed the predictions made by 15 geotechnical specialists with reference to driven, bored, screw and CFA piles in different subsoil conditions. Each specialist received all the data needed to predict pile response, but no experimental data were available to compare predictions and performance. According to the author, a large scatter in terms of axial bearing capacity comes out (Table 1) especially with reference to cast in situ piles (bored, screw, CFA).

Table 1 Results of the prediction exercises (Orr, 2016)

Pile type	N° of predictions	Q_{lim} [kN] Min. value	Q_{lim} [kN] Max. value	Ratio Max/Min
Driven	3	1748	2262	1.3
Bored	10	989	3026	3.1
Screw	8	351	1500	4.3
CFA	11	1290	5093	4.0

Similar results have been obtained in the occasion of the International Prediction Event stimulated by ISSMGE TC212, whose results have been made known during the 3rd Bolivian International Conference on Deep Foundations held in Santa Cruz de la Sierra (Bolivia). In this case, 3 different piles (bored, screw, CFA) have been installed at B.E.S.T. (Bolivian Experimental Site for Testing) and then head-down loaded at failure. The scrutiny of the predictions (Fellenius, 2017) reveals that the ratio among the predicted maximum and minimum values (72 predictions carried out by 121 people) has been even larger than that reported in Table 1.

The reliability and accuracy of pile design may be improved at a local scale by setting up Local Pile Design Methods (LPDMs), where a significant amount of experimental data is used to calibrate design parameters. They possess the advantage of implicitly taking into account installation effects related to local geological conditions as well as skills and experience of local piling contractors; on the other hand, a limitation of these approaches is that their accuracy is not guaranteed beyond the specific site they are designed for.

In this framework, this paper is intended to furnish an experimental support to pile design by interpreting a large amount of load tests carried out nearby Naples, in South Italy. Easy-to-use rules of thumb, directly inferred from raw experimental data and applicable without using any model, are furnished for the assessment of initial stiffness, pile capacity and mobilization curves as function of installation technique.

2. LOAD TEST DATABASE

The database of load tests under consideration consists of 384 load tests on piles which are herein broadly categorized in Displacement (i.e. bored), CFA and Non-Displacement piles, the latter including both driven piles and FDP. The distribution of the different technologies is reported in Figure 1.

Piles have been installed in different sites around Naples. The subsoil of the whole area has been thoroughly investigated by a number of authors (a summary is given in Mandolini, 1994; Mandolini and Viggiani 1992, 1997) and it is well known in its general features.

Starting from the ground surface and moving downwards, the following soils are typically found: (a) made ground, (b) volcanic ashes and organic soils, (c) stratified sands, (e) pyroclastic soils (pozzolana, cohesionless or slightly indurated, and volcanic tuff). In some cases, a groundwater table in the proximity of soil surface is also found.

The load tests were carried out until a certain value of settlement. For some cases, the test was conducted up to a settlement, w, of the pile head large enough to consider that the pile has attained conventional failure conditions.

The entire distribution of reached settlements (normalized by pile diameter, d) as function of technology is reported in Figure 2.

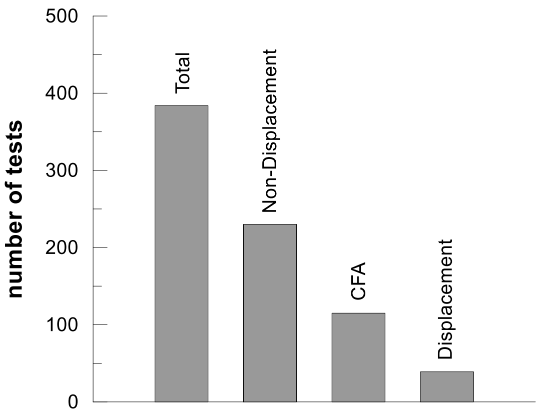

Figure 1 Number of pile tests as function of the installation technique

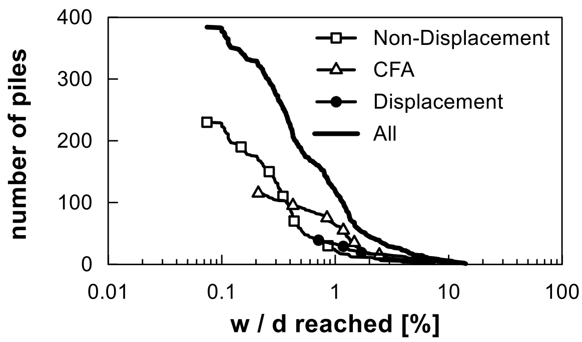

Figure 2 Displacement reached by load tests as function of installation technique

With the aim of providing an experimental support to the design of axially-loaded piles, results coming from all pile tests have been interpreted. This led to identify simple expressions for a rough assessment of initial stiffness, bearing capacity and load-settlement curve, as detailed below.

3. BEARING CAPACITY

3.1 Load-settlement curves from load tests

Figure 3 shows a typical load-settlement distribution derived from field tests carried out up to large values of settlement. When this data is reported in the plane w/Q:w (i.e. secant compliance on the vertical axis and settlement on the horizontal), it is evident that starting from a certain value of the settlement, the compliance varies linearly with the settlement itself, that is the load-settlement curve is an hyperbole. This does not hold for initial stage of an actual test, as points associated to low settlement values do not fall on the same line. With the aim of defining a failure load, we herein propose to interpolate by a straight line only points corresponding to values of settlement larger than 1% of pile diameter. The slope of the interpolating line in the w/Q:w plane is known to be the reciprocal of the asymptotic value of the load, defined here as Q_{lim} (Chin, 1970, 1971). The hyperbole derived from such interpolation procedure is shown in the figure along with the original data from the field test.

However, the above asymptotic value of load is reliable only when very high values of the settlement have been reached in the test (w > 5%d or so). A proof of this statement is that if only points corresponding to lower settlements were interpolated, a very different value of Q_{lim} would be obtained. In order to exploit a larger number of field test data available, we here refer to a conventional limit load corresponding to a settlement of 5% of pile diameter, $Q_{5\%d}$, since the assessment of the latter is evidently much more reliable, compared to Q_{lim}, when tests are conducted until settlements of the order of 2-3%d.

The interpolating hyperbole is expressed by the following equation (Chin, 1970):

$$Q = \frac{w}{\dfrac{1}{k_0} + \dfrac{w}{Q_{lim}}} \tag{1}$$

where k_0 is the reciprocal of the intercept of the interpolating line and has the meaning of initial stiffness. Note that this may be rather different from the measured one given that the hyperbole is built to fit higher values of loads. The values of Q_{lim} and k_0 are obtained by the method of least squares by means of a simple spreadsheet.

Upon defining a load level ψ as:

$$\Psi = \frac{Q}{Q_{lim}} \tag{2}$$

the interpolated load-settlement curve may be expressed in dimensionless terms as function of the load level as:

$$\frac{w}{d} = \frac{Q_{lim}}{k_0 d} \frac{\Psi}{1-\Psi} \tag{3}$$

If a conventional failure is set at a reference value w = w_{ref}, the corresponding load Q_{ref}, by making use of Eq. 3, is found to be related to Q_{lim} through the following equation:

$$\frac{Q_{ref}}{Q_{lim}} = 1 - \frac{Q_{ref}}{k_0 w_{ref}} = \frac{1}{1 + \dfrac{d}{w_{ref}} \dfrac{1}{\dfrac{k_0 d}{Q_{lim}}}} \tag{4}$$

It is therefore possible to define a conventional load level as:

$$\Psi_{ref} = \frac{Q}{Q_{ref}} \tag{5}$$

so that the interpolating hyperbole assumes the form:

$$\frac{w}{d} = \frac{Q_{ref}}{k_0 d} \frac{\Psi_{ref}}{1 - \Psi_{ref}\left[1 - \dfrac{Q_{ref}}{k_0 d}\left(\dfrac{w_{ref}}{d}\right)^{-1}\right]} \tag{6}$$

Given that in this work we refer to the load at w=5%d, $Q_{5\%d}$, the interpolating hyperbole is expressed by the equation:

$$\frac{w}{d} = \frac{Q_{5\%d}}{k_0 d} \frac{\Psi_{5\%d}}{1 - \Psi_{5\%d}\left[1 - \dfrac{100}{5} \dfrac{Q_{5\%d}}{k_0 d}\right]} \tag{7}$$

In the plane (w/d):$\Psi_{5\%d}$, the above equation is function only of the shape parameter $Q_{5\%d}/k_0 d$. Figure 4 helps clarifying the physical meaning of this parameter: it is the "elastic" settlement (if the elastic stiffness was indeed the initial slope of the hyperbole) at the conventional failure Q = $Q_{5\%d}$ and, for typical safety factors, is very close to the normalized settlement under working loads.

It will be shown that, for the sandy soils considered in this study, the parameter $Q_{5\%d}/k_0 d$ is essentially related to the construction method and will be therefore referred to as TF (Technology Factor) in the sequel.

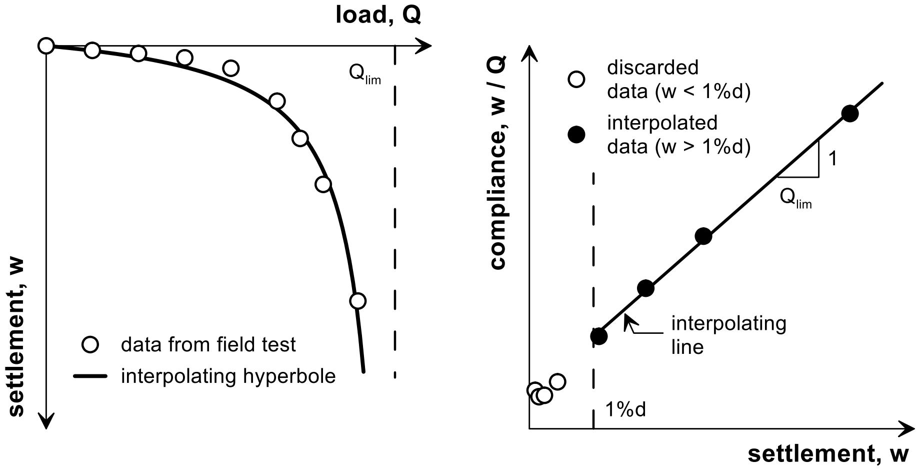

Figure 3 Interpolation procedure adopted in this work

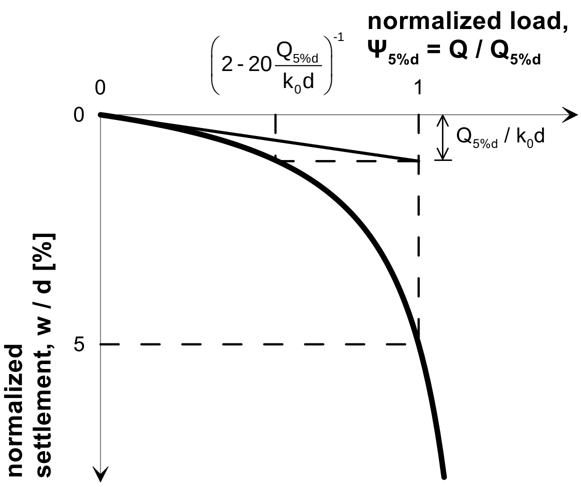

Figure 4 Interpolating hyperbole as function of normalized load and settlement parameters

3.2 Mobilization curves

The above procedure may be used to fit data from field tests conducted in sandy soils at large values of settlement (> 2%d) to derive design curves for different installation technologies.

Field data for Non-Displacement, CFA and Displacement piles are shown in Figures 5, 6 and 7.

From inspection of the graphs, the following aspects are noteworthy:

(a) The TF parameter is indeed strictly related to installation technique and does not depend in a significant manner on other parameters like pile geometry.
(b) TF increases from bored to driven piles. This is anticipated as the construction method is known to affect bearing capacity much more than pile stiffness;
(c) TF factors for bored, CFA, FDP and driven piles are nearly in the increasing sequence 0.7 – 1 – 1.7 %.

As a corollary of the above statements, under working loads a pile will settle from about 0.5 to 2% of its diameter going from Non-Displacement to Displacement piles. The fact the above displacement is larger for Displacement compared to Non-Displacement piles should not come as a surprise, as for the first category the working load is larger as well due to a larger capacity. As a side comment, care must be taken when considering for driven piles working settlements of about 1.5-2% d since at these values shaft capacity may be mobilized.

3.3 Capacity Ratio

With the aim of providing a gross preliminary estimation of pile bearing capacity as function of the installation technique, it is possible to define a Capacity Ratio as (Mandolini et al. 2005):

$$CR = \frac{Q_{5\%d}}{W_p} = \frac{Q_{5\%d}}{\gamma_p \cdot \frac{\pi d^2}{4} \cdot L} \tag{8}$$

with γ_p unit weight of the pile material, which expresses the ratio between the failure load (i.e. the load at the conventional settlement of 5%d) and the weight of the pile. Note that this definition was adopted in past works, with the exception that in previous studies the numerator corresponded to the asymptotic value of the load, and not to a specified level of settlement.

Notwithstanding the practical appeal of the above definition, one may think that the failure load of the pile is expected to depend on the state of effective stress in the soil, so that an alternative CR may be defined as:

$$CR = \frac{Q_{5\%d}}{\gamma_s \cdot \frac{\pi d^2}{4} \cdot L} \tag{9}$$

where γ_s is the dry or the buoyant unit weight of the soil, depending of the presence of groundwater table. Mean values, standard deviation and Coefficient of Variation of these indexes of failure are reported in Table 2 for different technologies, with reference to tests conducted until a final settlement not lower than 2% of pile diameter, in analogy with the above section.

It is noted that, as expected, CR values increase from Non-Displacement to CFA and Displacement piles. Values of CoV are larger, indicating larger dispersion of values, for the last category.

As a side comment, a quite low mean value of CR is found for bored piles. Since CR (Eq. 9) has to be larger than the bearing capacity factor N_q (CR = N_q if pile failure load is due only to base resistance), this seems to indicate that value of bearing capacity factors utilized in practice may largely overestimate base capacity (Berezantzev et al., 1961). However, discussing the reasons lies beyond the scope of this work.

4. INITIAL AXIAL PILE STIFFNESS

The entire database of nearly 400 load tests may be analyzed to derive indications on the initial axial stiffness of piles. should be inferred directly from the experimental data. To this end, many reasonable criteria may be identified.

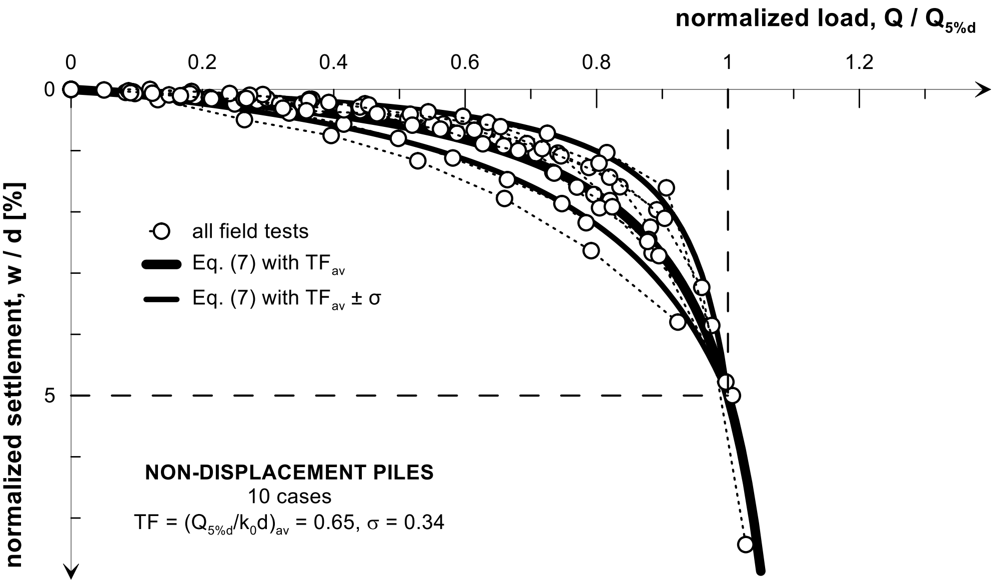

Figure 5 Load-settlement curves for Non-Displacement piles

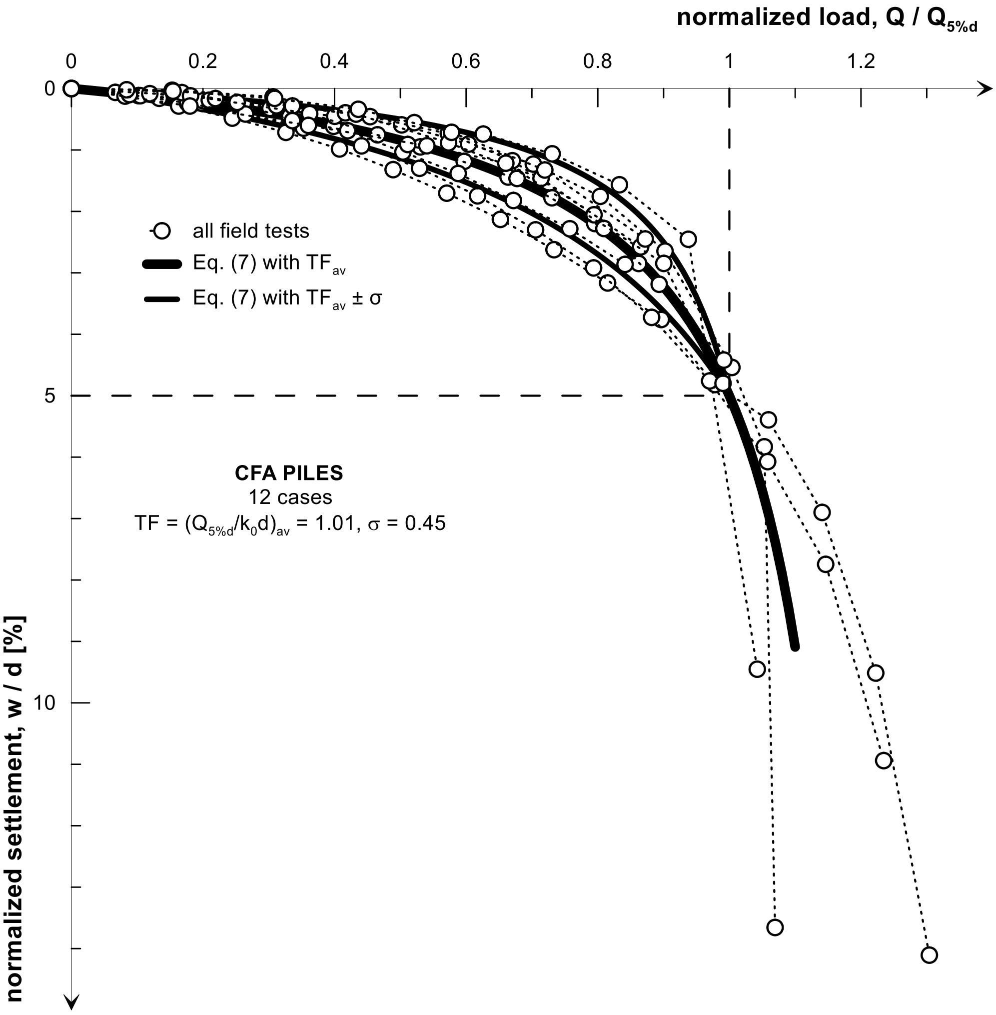

Figure 6 Load-settlement curves for CFA piles

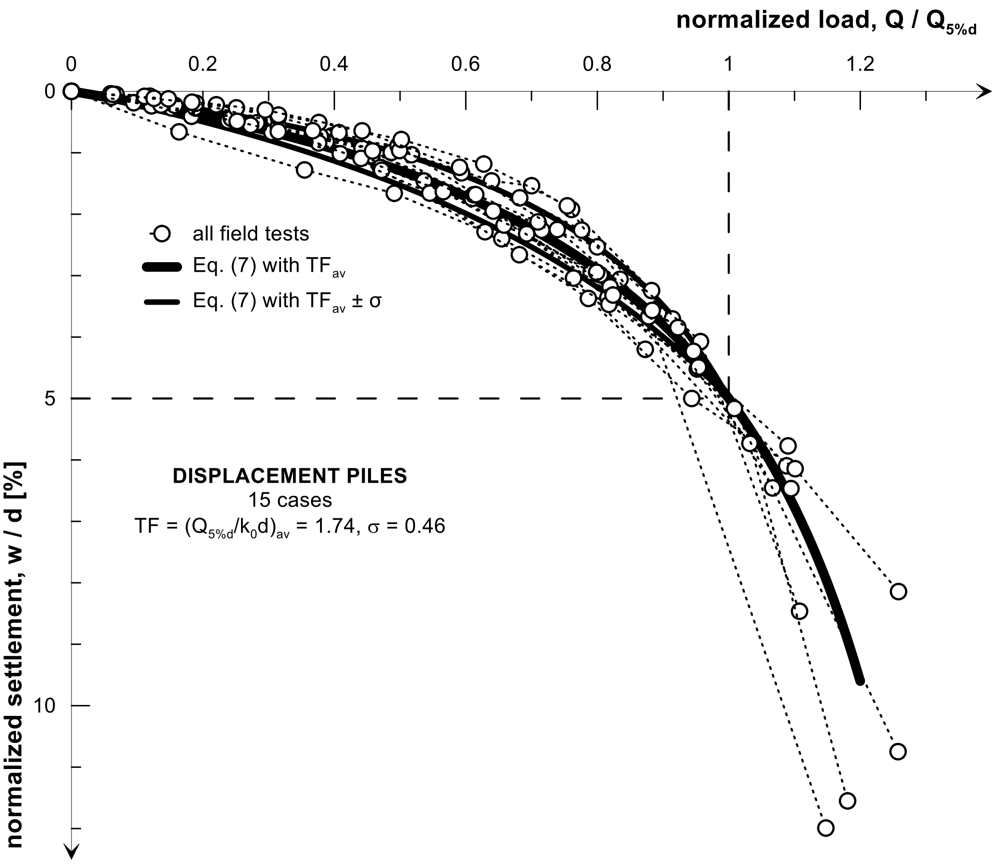

Figure 7 Load-settlement curves for Displacement piles

Table 2 Mean values, Standard Deviation and CoV of Capacity Ratio as function of pile installation technique

Installation technique	Mean μ	Standard Deviation σ	Coef. of Variation σ/μ
Non-Displacement	19.15	6.35	0.331
CFA	74.63	15.98	0.214
Displacement	85.36	48.86	0.572

The first step is to define an objective criterion to define initial axial stiffness of the pile-soil system. For example, one may simply consider the first load value divided by the associated settlement. Or, to make the procedure more stable, to consider the average stiffness, or the reciprocal of the average compliance, of the first three pairs load-settlement measured in the test. As an alternative, one may refer to a standard settlement value, say 0.1%d, to avoid the dependence of the result on the load step chosen for the specific test. Accordingly, the average stiffness of all the points with settlement below this threshold value may be considered as representative of the initial stiffness. However, this procedure still suffers of all the unavoidable "noise" associated to the experimental procedure at the very first load steps, for which a clear trend is not identifiable. For this reason, we here propose to define the initial stiffness as the load to settlement ratio at w = 0.1%d, the latter obtained simply interpolating linearly between the points immediately before and after this settlement value.

Our proposal is physically justified considering that the conventional settlement at which the stiffness is evaluated is anyhow very low. The results coming from the above stiffness definitions will be summarized later on.

A further step towards a simple, experimentally based, criterion for a rapid assessment of the pile initial stiffness is to identify normalization parameters to allow grouping of all the different tests. We therefore propose to normalize the above pile stiffness, clearly representing the pile-soil system, by the stiffness of the pile intended as a column (i.e., the structural stiffness $k_c = E_pA/L$).

Table 3 reports mean values, standard deviation and coefficient of variation utilizing the stiffness definitions for the ratio k/k_c, with reference to bored, CFA and driven piles. Due to the lack of reliable data on the pile Young's modulus, a constant value of 30 GPa has been considered for all cases.

It is interesting to observe that stiffnesses defined on the basis of a specified number of points of the load tests lead to high dispersion of data. In some cases, the standard deviation is larger than the mean, so that no practical advantage can be gained from such results. On the contrary, the distribution gets more narrow if reference is made to a specified level of settlement. The narrowest distribution is found for the secant stiffness at w = 0.1%d, which will be therefore denoted as $k_{0.1\%d}$ in the ensuing. If some anomalous cases, associated with an unrealistic value of $k_{0.1\%d}/k_c < 1$, are excluded from the statistical evaluation, a lower coefficient of variation is obtained for all pile installation techniques.

The most important aspect emerging from the table is that mean values of $k_{0.1\%d}/k_c$ are independent of pile technology and coefficients of variation are not that high, considering all the involved uncertainties in comparison to the simplicity of the normalization parameter k_c.

A better parameter describing in a more accurate manner the physics of the phenomenon is definitely represented by the ratio of pile stiffness and the stiffness of the column with height equal to pile critical length (say k^*_c) as referred to SR (Stiffness Ratio) in previous works (Mandolini et al. 2005). Such a parameter reflects the fact that the portion of pile exceeding the critical length does not offer any significant contribution to increase pile stiffness, and may be expressed as:

Table 3 Mean values, Standard Deviation and CoV of Capacity Ratio as function of pile installation technique

	NON-DISPLACEMENT (230 cases)			**CFA (115 cases)**			**DISPLACEMENT (39 cases)**		
Stiffness definition	Mean μ	Standard Deviation σ	Coef. of Variation σ/μ	Mean μ	Standard Deviation σ	Coef. of Variation σ/μ	Mean μ	Standard Deviation σ	Coef. of Variation σ/μ
first test point	3.425	4.573	1.335	1.388	0.792	0.571	1.468	1.033	0.704
average stiffness of the first three points	2.588	3.690	1.426	1.199	0.687	0.573	1.213	0.757	0.624
reciprocal of average compliance of first three points	2.209	2.114	0.957	1.157	0.683	0.591	1.157	0.726	0.627
average stiffness of points with w < 0.1% d	2.351	1.690	0.719	1.530	0.769	0.503	2.110	1.461	0.692
stiffness at w = 0.1% d (proposed)	1.537	0.718	0.468	1.233	0.656	0.532	1.304	0.665	0.510

$$SR = \frac{k_{0.1\%d}}{k_c^*} = \frac{k_{0.1\%d}}{\dfrac{E_p \dfrac{\pi d^2}{4}}{L_c}} = \frac{k_{0.1\%d}}{\dfrac{E_p \dfrac{\pi d^2}{4}}{1.5d\sqrt{\dfrac{E_p}{G}}}} \tag{10}$$

where G represents the soil stiffness al low strain level. No sufficient information is available to calculate the values of this parameter for all the sites considered in this study. However, if for the sake of simplicity E_p and G is taken as constant for all piles and subsoils, SR is found to possess values of CoV much lower than the values in Table 3. Therefore, if SR is constant, Eq. 10 reveals a direct proportion between pile stiffness and pile diameter. The ratio $k_{0.1\%d}/d$ was found to be as an average 1.70 GPa (CoV = 0.312) for Non-Displacement piles, 1.06 (CoV = 0.340) for CFA and 0.95 (CoV = 0.456) for Displacement piles. It is reasonable to furnish the following rule of thumb valid for all technologies:

$$k_{0.1\%d} \simeq d \cdot 1.5GPa \tag{11}$$

Figure 8 shows the performance of the above rule. Note that 312 out of 379 cases (= 82%) fall within the range of Eq. 11 ± 50%. It is quite surprising that such a simple relation, considering pile stiffness proportional to pile diameter and only, allows for a quite reliable prediction. However, this result could indicate that pile settlement at very low loads stems from the mobilization of soil strength at pile interface until very shallow depths. In light of this interpretation, it is not puzzling that initial axial pile stiffness does not depend on pile length.

As a side comment emphasizing the potential of Local Pile Design Methods mentioned in the introduction, the scatter of the stiffness estimation may be largely reduced with reference to a unique site. Table 4 shows the values of $k_{0.1\%d}/k^*_c$ only for the subset of tests carried out at Centro Direzionale di Napoli (CDN), involving 105 pile load tests in well-characterized soil. Very low CoVs are found and, in agreement with data of the entire database, installation technique does not play a relevant role. Again, larger dispersion is observed for Displacement piles.

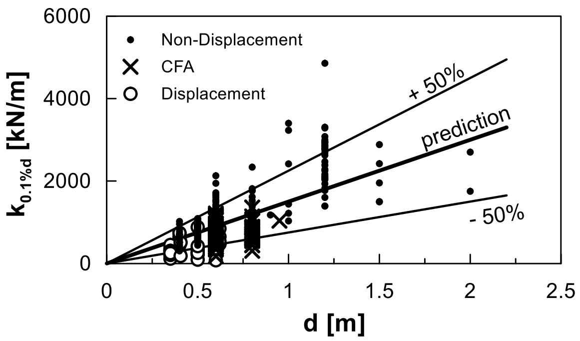

Figure 8 Initial stiffness as function of pile diameter

Table 4 Mean values, Standard Deviation and CoV of $k_{0.1\%d}/k^*_c$ for CDN site as function of pile installation technique.

Installation technique	**Mean μ**	**Standard Deviation σ**	**Coef. of Variation σ/μ**
Non-Displacement	1.278	0.114	0.089
CFA	1.300	0.112	0.086
Displacement	1.328	0.280	0.211

5. CONCLUSIONS

In this work results coming from nearly 400 full-scale load tests of axially-loaded piles in sandy soils around Napoli (Italy) area are summarized. Some of the tests were conducted until a large value of settlement and the results have been used to derive indications about failure load, with a focus on the role played by installation technique. The remaining tests have been anyway exploited to investigate the initial axial pile stiffness. The main results of the study may be summarized in the following points:

- Mobilization curves in the plane $Q/Q_{5\%d}$: w/d, fitting in a satisfactory manner all the available experimental data. These curves are function of a unique parameter related to construction method, referred to as Technology Factor (TF);

- A novel definition of Capacity Ratio (CR) is proposed, and the experimentally-derived values are reported. This allows a rough estimation of pile bearing capacity as function of pile geometry and installation technique;
- A settlement of 0.1% of pile diameter is detected as the value that, while being still sufficiently low to define an initial stiffness, is free of all the noise observed in the field test at very low loads. Such stiffness is found to be related to pile diameter and only, and the rule of thumb $k_{0.1\%d}$ = d*1.5 GPa fits well all the experimental data, regardless of pile length, installation technique, pile and soil material stiffness.

While the practical significance and usefulness of the above indications can be hardly overstated, it is worth highlighting that they cannot substitute a proper engineering modelling in the design. Instead, they can be used as a 'local' robust guidance for preliminary design purposes and to check the significance of the analytical and/or numerical results.

In addition, caution must be used when utilizing quantitative results of this work. While general trends may be of general validity, the specific values derived as function of installation technique may significantly vary for different areas worldwide.

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