Research Article

A Local Design Method for Pile Foundations

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Received 2 May 2018; Accepted 4 September 2018; Published 22 October 2018

Academic Editor: Luigi Di Sarno

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The work at hand attempts to propose a local pile design method based on pile load test results for a reference site. Such LPDM is simply based on the identification of three dimensionless quantities, such as the capacity ratio CR, the stiffness ratio SR, and the group settlement ratio Rs. To prove the LPDM reliability, experimental data collected during years in the Neapolitan area (Italy) have been used to obtain the abovementioned coefficients. Then, LPDM has been applied, as a preliminary design method, to three well-documented case histories applying capacity and settlement-based design (CBD and SBD) approaches. The satisfactory agreement between the geometry in the original design of piles and the one obtained by applying the LPDM proves that the proposed methodology may be very helpful for preliminary design, allowing for reasonable accuracy while requiring few hand calculations.

1. Introduction

The design of foundation systems is an engineering process which therefore involves a simplistic modelling of the more complex real world. With reference to pile foundations, pile design always involves calculation of the axial bearing capacity of the single pile. Among the main methods for estimating values of the unit base resistance, q_b, and the unit shaft resistance, q_s, there are those based on fundamental soil properties (theoretical methods), such as angle of friction, and those based on in situ test results (empirical methods), such as standard penetration tests (SPTs) or cone penetration tests (CPTs). The comprehension of the difference between model and reality, of the model limits and the feasibility of different methods, is crucial. Theoretical methods consist in evaluating the design values of q_s and q_b by the following expressions:

\[ q_s = \sigma_h' \tan \delta' = k_f \sigma_v' \tan \delta', \]  
\[ q_b = N_q \sigma_{vL}', \]  

where \( \sigma_h' \) is the effective horizontal stress at failure, its evaluation being one among the most challenging methods in geotechnical engineering, and \( \delta' \) is the soil-pile friction angle. The horizontal effective stress may be taken as some ratio \( k_f \) of the vertical effective stress \( \sigma_v' \), thus resulting in the second form of the expression in Equation (1).

In Equation (2), \( N_q \) is the bearing capacity factor, often assumed as function of the angle of internal friction of the soil in proximity of the pile tip, as suggested in Berezantzev et al. [1]; \( \sigma_{vL}' \) is the vertical effective stress acting at the depth of the pile tip.

Empirical methods based on CPT results consist in evaluating \( q_s \) and \( q_b \) by the following empirical relations:

\[ q_{si} = \alpha_s q_{cxi}, \]  
\[ q_{bi} = \alpha_b q_{cb}, \]  

where \( \alpha_s \) and \( \alpha_b \) are the empirical coefficients depending on both soil type and pile type, \( q_{cxi} \) is the value of the point resistance of CPT representative of the layer \( i \) along the pile shaft, and \( q_{cb} \) is the average value of \( q_s \) measured in a suitable interval of depth around the pile base.

To improve the reliability of Equations (3) and (4), data from load tests on experimental piles can be interpreted to get \( \alpha_s \) and \( \alpha_b \) values for the reference site, and for such specific site and only, the use of backcalculated values of the above coefficients makes pile design certainly more accurate.
Although in the last decades significant improvements have been made in comprehension of the processes governing the soil-pile system behaviour up to failure, recent papers [2, 3] demonstrate that our capability to evaluate pile response to loading is still far from being satisfactory for practical purposes on a specific project.

Orr [3] analyzed the predictions made by 15 geotechnical specialists with reference to driven, bored, screw, and CFA piles in different subsoil conditions. The predictions are fully theoretical, in the sense that 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 ultimate vertical bearing capacity, \( Q_{\text{lim}} \), comes out (Table 1) especially with reference to cast in situ piles (bored, screw, and CFA).

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, and 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 [2] 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.

A way to improve the reliability and accuracy of pile design at a local scale is to set up local pile design methods (LPDMs), which may be used either in a preliminary stage or in a final stage of design, depending on the data (quality and quantity) on which they have been developed.

The aim of this work is to (1) propose an LPDM based on the interpretation of pile load test results for a reference site, (2) describe some case histories located in the reference site and report the most relevant experimental evidence, and (3) apply the proposed LPDM to the selected case histories. It will be shown that LPDM may be very helpful for a preliminary design of the foundation, being rather accurate from an engineering viewpoint despite involving few hand calculations.

2. Local Pile Design Method

Since the prediction of pile response to loading is affected by several uncertainties, a pile load test programme should be considered as an essential part of the design and the construction process. Pile tests may fall into one of the two categories: tests to failure on trial piles, to prove the suitability of the piling system and to confirm the design parameters inferred from the site investigation, and tests carried out on production piles, to check the construction technique and workmanship and to confirm the performance of the pile as a foundation element [4].

Pile load tests are mainly used to determine the ultimate bearing capacity of piles, directly from the resulting load-settlement curve or by its extrapolation, and the stiffness of the pile-soil system at certain load. Load tests also provide a considerable amount of extra data which often remain unused. Nevertheless, such data may be better exploited, as demonstrated by the LPDM proposed in the following sections.

2.1. Capacity Ratio. Mandolini et al. [5] introduced the capacity ratio, CR, a dimensionless parameter defined as follows:

\[
CR = \frac{Q_{\text{lim}}}{W},
\]

where pile ultimate axial bearing capacity, \( Q_{\text{lim}} \), derived from pile load test results, is divided by the pile weight, \( W \).

The ultimate load of a pile is usually not well defined starting from the observation of the pile load-settlement curve. A simple criterion which can be used to overcome this problem is to conventionally define \( Q_{\text{lim}} \), as the load causing a displacement of the pile head equal to 10% of the pile base diameter (as, for instance, suggested in Eurocode 7). If the load test has been stopped before the pile head could experience such a displacement, \( Q_{\text{lim}} \) can be obtained extrapolating the load-settlement curve; for example, Chin’s empirical method [6], which assumes that the form of load-settlement curve is hyperbolic, may be applied. In order to obtain a reliable \( Q_{\text{lim}} \) value by extrapolation, during the load test, a pile head settlement of at least 5% of the pile base diameter must be measured.

The capacity ratio, CR, allows to compare data from different piles (type and geometry) belonging to the same area in terms of geological and geotechnical subsoil conditions. For a given installed pile volume, the capacity ratio, same as \( Q_{\text{lim}} \), depends on the pile type and the soil type. Since the subsoil conditions is fixed, CR is expected to be strongly affected by the pile specific installation technique. In a preliminary stage of design, \( (CR)_{av} \), the average value of capacity ratios obtained for the reference site, allows to predict the expected value of \( Q_{\text{lim}} \). Clearly, an adequate amount of CR values are needed to ensure that \( (CR)_{av} \) provides a reliable estimation of \( Q_{\text{lim}} \). Therefore, it is suggested to compute the coefficient of variation \( (CV) \) of the CR population to express the precision of \( (CR)_{av} \).

2.2. Stiffness Ratio. Mandolini et al. [5] introduced the stiffness ratio, SR, expressed as follows:

\[
SR = \frac{K_0}{K_0},
\]

where \( K_0 \) is the initial axial soil-pile stiffness (inclination of the initial tangent of the experimental load-settlement curve; in order to process the data in an objective and repeatable way, \( K_0 \) may be obtained as the initial tangent of a hyperbola fitted to the first three points on the experimental load-settlement curve). Its knowledge is essential to predict the value of the expected settlement of the single pile, \( w_s \), under working load, in a preliminary stage of design.

\( K_0 \) is the axial stiffness of a column having a length equal to the critical value, \( L_c \). It represents that length beyond which any increase of the pile length causes little or no
increase of the pile stiffness. Fleming et al. [4] defined \( L_c \) as follows:

\[
L_c = 1.5 \cdot d \cdot \sqrt{\frac{E_p}{G_t}}
\]

(7)

where \( E_p \) is Young’s modulus of the pile material; \( G_t \) is the value of the soil shear modulus at a depth \( L_c \) from the ground surface and it can be estimated, iteratively, using the results of seismic tests (down hole, cross hole, etc.) in terms of shear wave velocity \( V_S \).

The critical length \( L_c \), instead of the full pile length \( L \), has been introduced in SR definition because the pile response under working loads (hence far from failure) is affected by \( L_c \), whereas \( L \) is typically fixed by pile capacity requirements.

For a given pile geometry in the reference site, SR values are expected not to be as strongly affected by the pile-specific installation technique as CR because the specific pile area is expected not to be as strongly affected by the pile-specific pile technology and evaluating acceptable stress level on the pile head section, after choosing diameter \( d \), can be derived from Equation (6), considering, again, the following reduced value of \((\text{SR})_{av}\):

\[
(SR)_{av} = (SR)_{av} \cdot [1 - CV(SR)].
\]

(9)

The corresponding elastic component of displacement of a single pile, \( w_{s,el} \), under the average vertical load can be evaluated as \( Q_{av}/K_{pl} \). In wider terms, \( w_s \) is the sum of two contributions: \( w_{s,el} \) (elastic component) and \( w_{s,ni} \) (nonlinear component) = \( w_s - w_{s,el} \), as shown in Figure 1.

Nevertheless, if the pile load level is low enough, \( w_s \approx w_{s,el} \) may be assumed. The evaluation of the average settlement of the piled foundation is described in the following section.

### 3.2. Group Effects in terms of Settlement

The interaction between piles belonging to a group amplifies only the elastic component of the single-pile settlement (e.g., [5, 11–13]). Thus, the average settlement of piled foundations, \( w_{g,av} \), may be expressed as follows:

\[
w_{g,av} = R_s \cdot w_{s,el} + w_{s,ni},
\]

(10)

where \( R_s \) is the amplification factor named “group settlement ratio,” originally introduced by Skempton et al. [15] and measuring the effects of the interaction between piles.

Considering the assumption \( w_s \approx w_{s,el} \), \( w_{g,av} \) has the following expression:

\[
w_{g,av} = R_s \cdot w_{s,el}.
\]

(11)

Research works (e.g., [14, 15]) suggested that \( R_s \) can be expressed as a function of geometrical factors such as the number, \( n \), the spacing, \( s \), and the slenderness, \( L/d \), of piles.

Mandolini [13] postulated that \( R_s \) can be expressed as a function of the aspect ratio, \( R \), which was originally introduced by Randolph and Clancy [16] as \( R = (n \cdot s/L)^{0.5} \), but with the critical pile length, \( L_c \), instead of the total pile length, \( L \), as shown in the following expression:

\[
R = \sqrt{\frac{n \cdot s}{L_c}}
\]

(12)

To check the validity of this assumption, Mandolini [13] evaluated the ratio between the experimentally measured average settlement, \( w_{g,av} \), for six buildings in the eastern area of Naples and the settlement of the single pile, \( w_s \), under the average working load as measured during the load test on one or more production piles belonging to the same foundation. By interpolating all the experimental data, he suggested the following expression:

---

**Table 1: Results of the prediction exercises [3].**

<table>
<thead>
<tr>
<th>Pile type</th>
<th>Number of predictions</th>
<th>( Q_{lim} ) (kN) min. value</th>
<th>( Q_{lim} ) (kN) max. value</th>
<th>Ratio max/min</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driven</td>
<td>3</td>
<td>1748</td>
<td>2262</td>
<td>1.3</td>
</tr>
<tr>
<td>Bored</td>
<td>10</td>
<td>989</td>
<td>3026</td>
<td>3.1</td>
</tr>
<tr>
<td>Screw</td>
<td>8</td>
<td>351</td>
<td>1500</td>
<td>4.3</td>
</tr>
<tr>
<td>CFA</td>
<td>11</td>
<td>1290</td>
<td>5093</td>
<td>4.0</td>
</tr>
</tbody>
</table>

---
To preliminarily design a piled raft, the abovementioned method is slightly adjusted. First, the load sharing between the pile group and the raft must be predicted. After evaluating, through classical methods, the average settlement, \( \psi \), associated with the unpiled raft, the raft-soil stiffness can be readily obtained as \( K_g = Q/\psi \). Assuming an admissible value for the average settlement, \( \psi \), of the piled raft and neglecting the raft contribution to the overall stiffness of the combined foundation, the latter may be obtained as \( K_g = Q/\psi \). The fraction of load transmitted by piles to the soil, \( \alpha_{pr} \), can be expressed as follows [16]:

\[
\alpha_{pr} = \frac{1}{1 + \beta}
\]

\[
\beta = \frac{0.2}{1 - 0.8 \cdot (K_r/K_g)} \left( \frac{K_g}{K_r} \right).
\]

The load, \( Q_g \), to be assigned to the pile group is therefore \( \alpha_{pr} \cdot Q \). While in the capacity-based design approach, the pile length needed to guarantee the requested safety factor against a bearing capacity failure is derived; in a settlement-based design approach, pile length is derived from SR evaluation and is needed to guarantee an acceptable average settlement of the piled raft. In such circumstances, the influence exerted by nonlinearity on the average displacement cannot be neglected, due to the high load level, and therefore, Equation (10) should be utilized. If the load-settlement curve is interpolated by a hyperbola according to Chin [6], \( \psi_w \) can be expressed as follows:

\[
\psi_w = \psi_{w_{el}} - \psi_{w_{el}} \cdot \frac{\psi}{1 - \psi}.
\]

where \( \psi = Q_{pr}/Q_{lim} \) is the load level.

Upon combining Equations (10) and (17), the following expression for \( \psi_w \) is derived:

\[
\psi_w = \psi_{w_{el}} \cdot \left( \frac{R_s}{R_s + \psi} \right),
\]

which can be evaluated for any given combination of piles diameter and number. Substituting Equation (18) in \( K_g \) definition (\( K_g = Q / \psi_w \)), considering that \( \psi_w = Q_{pr}/Q_0 \) and \( n = Q/Q_{pr} \) and expressing \( K_0 \) as (SR)\( \alpha_{pr} \cdot C \), the following expression for \( K_g \) is derived:

\[
K_g = \frac{n \cdot (SR)_{\alpha_{pr}}}{R_s + (\psi/(1 - \psi)) \cdot C}.
\]

Setting a first attempt value of pile length, \( L \), the pile weight can be calculated. Thus, from Equation (5), adopting (CR)\( \alpha_{pr} \) (Equation (8)), axial bearing capacity of a single pile, \( Q_{lim} \), and therefore the load level, \( \psi \), may be calculated. Pile group-soil stiffness, \( K_g \), is then derived from Equation (19), and therefore, a new value of \( \psi_w \) is obtained as \( Q/K_g \). The procedure is repeated until the selected length, \( L \), guarantees the acceptable settlement, \( \psi_w \).

The entire procedure may be repeated for an admissible value of the maximum settlement, \( \psi_{w_{max}} \), of the piled raft,

### Figure 1: Schematic load-settlement curve.

$$\begin{align*}
R_s &= \frac{w_s}{w_g} = 0.23 \cdot n \cdot R^{-0.91}. \\
R_s &= \frac{w_s}{w_g} = 0.29 \cdot n \cdot R^{-1.35}.
\end{align*}$$

These findings seem to support the idea that the group effects in terms of settlement are ruled out mainly by geometrical factors (by means of the aspect ratio, \( R \)) and not by the specific pile types, whose influence enters the analysis via the value of \( w_g \), obtained by a load test.

Later on, the dataset needed to estimate \( R_s \) has been increasing, including experimental data related even to monitored piled foundations not located in the eastern area of Naples. In 2005, 63 well-documented case histories were available, including a wide range of pile types (driven, bored, and CFA) assembled in a variety of geometrical configurations (4 ≤ \( n \) ≤ 6500; 2 ≤ \( s/d \) ≤ 8; and 13 ≤ \( L/d \) ≤ 126) and regarding very different soils (clayey to sandy soils, stratified, saturated or not, etc.).

Mandolini et al. [5], fitting all the abovementioned data by the same power function as Equation (13), proposed the following expression to estimate \( R_s \):

\[
R_s = \frac{w_s}{w_g} = 0.23 \cdot n \cdot R^{-0.91}. \\
R_s = \frac{w_s}{w_g} = 0.29 \cdot n \cdot R^{-1.35}.
\]

The data collected in the abovementioned 63 case histories include the experimentally measured maximum settlement of piled foundations, allowing to obtain an expression to estimate \( R_s = \max \), defined as \( w_{g_{\max}}/w_s \):

\[
R_s = \max = \left( \frac{0.50}{R} + \frac{0.13}{R^2} \right) \cdot n.
\]

Substituting Equation (14b) in Equations (10) and (11), the maximum settlement of piled foundation, \( w_{g_{\max}} \), may be obtained.

### 3.3. Settlement-Based Design (SBD) of a Piled Foundation

A piled raft is a foundation system that combines both rafts and piles. Since in such a foundation system piles are used to reduce and/or regulate settlements and their distribution, no limitations are prescribed to the safety factor of piles against a bearing capacity failure, resulting in an optimized foundation cost.
adopting, in Equation (19), $R_{s,max}$ (Equation (14b)) instead of $R_s$ (Equation (14a)).

4. Experiences at the Eastern Area of Naples (Italy)

In 1995, in Naples, the construction of the “New Directional Center” (CDN) was completed. It is a large urban settlement located in the eastern area of the city, mainly devoted to business. It includes high-rise buildings up to 100 meters.

Piled foundations, designed with a capacity-based approach, were adopted for almost all the buildings. Due to the importance of the work and the usual uncertainties connected to the design of piled foundations, a broad experimental investigation took place before, during, and after the construction works. In particular, 20 head-down load tests to failure on different trial piles, 125 head-down load tests on different production piles, and careful monitoring of the performance of several buildings during and after their construction were carried out.

4.1. Geological and Geotechnical Contest. The subsoil of the whole area has thoroughly been investigated by a number of authors (a summary is given by Mandolini [13]).

The collection of geological and geotechnical information revealed the existence of rather uniform subsoil condition. Starting from the ground surface, located at an elevation ranging between 5 and 8 m above the mean sea level, and moving downwards, the following soils are found (Figure 2): (a) made ground; (b) volcanic ashes; (c) stratified sands with organic soils; (d) pozzolana, cohesionless or slightly cemented; (e) volcanic tuff; and (f) sea sands.

The groundwater table is at a shallow depth from the ground surface (ranging between +2 and +5 m above the sea level).

In Figure 2, the results of CPT in terms of cone resistance, $q_c$, and friction, $f_s$, as well as the measurements of shear wave velocity, $V_S$, are also reported. All the data refer to the vertical (marked at the head with full dots) where volcanic tuff has not been found.

As it can be seen, the values of $q_c$ are highly variable and very often lesser than 10 MPa in the upper 30 m. Once pozzolana is encountered, the values are still quite low but, even slightly, linearly increasing with depth up to 40 m where slightly cemented pozzolana is found as demonstrated by a sudden increase in $q_c$. Beyond the depth of 60 m (sea sand), the $q_c$ values turn to be highly variable.

Looking at $V_S$, independently of the soil type, the values tend to increase linearly with depth from about 150 m/s at shallow depths to more than 300 m/s at greater depths.

4.2. Data for LPDM Application in the Neapolitan Area (2005). In 2005, Mandolini et al. [5], processing data collected in the previous years, provided the information needed to apply the LPDM for the Neapolitan area. They are reported in Table 2.

Bored piles give the smallest $\langle CR \rangle_{av}$ value ($Q_{lim}$ on average 12 times greater than the weight of the pile) and the larger scatter ($CV(CR) = 26\%$), while driven piles give the largest $\langle CR \rangle_{av}$ value (73 times the weight of the pile) and the smallest scatter ($CV(CR) = 8\%$). CFA piles are intermediate, even if with a scatter similar to that of bored piles ($CV(CR) = 25\%$). These results confirm the expected strong influence of the pile installation technique on the pile axial bearing capacity. On the contrary, $\langle SR \rangle_{av}$ is not so affected by the specific pile installation. In fact, it ranges from 1.29 (screw and driven) to 1.46 (bored) for all the piles, with $28\% < CV(SR) < 42\%$. These findings seem to support what has been claimed by a number of authors since more than 20 years [11, 13, 17, 18]: the installation technique affects the axial stiffness of piles much less than their bearing capacity and depends primarily on the small-strain shear modulus of the soil.

4.3. Data for LPDM Application in the Neapolitan Area (2018). The data collection started during the CDN construction never stops. A large number of load tests on trial and production piles have been carried out during building works of a different structure in the province of Naples up to now. The dataset now includes the results derived from 384 load tests performed on piles realized in 15 comparable sites in terms of the geological and geotechnical context. The improvement of such a dataset allows the update of $\langle CR \rangle_{av}$ and $\langle SR \rangle_{av}$ values (and the corresponding coefficients of variation), as shown in Table 3.

In addition to the data processed in 2005, another pile type has been introduced, which is full displacement pile. It is worth noticing that the coefficients of variation are decreased for both $\langle CR \rangle_{av}$ and $\langle SR \rangle_{av}$ for each type of pile; thus, the provided $\langle CR \rangle_{av}$ and $\langle SR \rangle_{av}$ values are more reliable because of the extension of the dataset.

5. Application of LPDM for Three Well-Documented Case Histories

In order to illustrate the application of LPDM, the reference is made to the following three well-documented case histories:

(i) Case history #1, related to the construction of the New Law Court Building; data are reported in great detail by Mandolini [13], but the reader can find an exhaustive summary in Mandolini and Viggiani [17].

(ii) Case history #2, related to the construction of two towers; again, data are reported in great detail by Mandolini [13], but the reader can find an exhaustive summary in Mandolini and Viggiani [19].

(iii) Case history #3, related to the construction of a cluster of circular steel tanks; data are reported in detail by Russo et al. [20].

It is worth highlighting that the application of LPDM has been checked against other well-documented case histories in the eastern area of Naples, here not reported; its reliability for a preliminary design has been systematically confirmed.
5.1. Case History #1

5.1.1. Description. The New Law Court Building consists of three towers with heights ranging between 67 m and 110 m from the ground surface (Figure 3). Each tower has a steel frame structure with reinforced concrete stiffening cores to resist wind and seismic actions.

The total applied vertical load, $Q$, is approximately 1450 MN, and the whole foundation area is about 7000 m$^2$. The resulting average contact pressure ($\approx$ 200 kPa) would have produced an average settlement of the order of some tens of centimeters, larger than the admissible value. Therefore, the piled foundation sketched in Figure 4 was considered by the designer.

It consists of 241 bored piles with a preload cell at the base. All the piles are 42 m long, with diameters ranging between 1.5 m and 2.2 m (23 piles with $d = 1.5$ m, 62 piles with $d = 1.6$ m, 79 piles with $d = 1.8$ m, 57 piles with $d = 2.0$ m, and 20 piles with $d = 2.2$ m). The spacing among piles is, on average, $s = 6.1$ m.

Each pile is subjected to an average load $Q_{av} = Q/n = 6.0$ MN. Due to the load concentration under the reinforced concrete stiffening cores, the maximum expected load is $Q_{max} = 8.9$ MN.

Before the construction, four head-down load tests on trial piles (A, B, C, and D), all with a length $L = 42$ m, have been tested to failure [21].

Piles A (without preload cell at the base) and C (with a preload cell at the base) have a diameter $d = 1.5$ m, while piles B (without preload cell at the base) and D (with a preload cell at the base) have a diameter $d = 2.0$ m. All the piles are instrumented over the entire length in order to measure the contributions given by the shaft and the base.

Since the final decision was that of adopting piles equipped with a preload cell at the base, in Figure 5, only the load test results for piles C and D are shown.

As it can be seen, while the load-settlement curve for pile C ($d = 1.5$ m) clearly exhibits a brittle failure condition at $Q = 19.1$ MN, the same does not apply to pile D ($d = 2.0$ m). In this case, due to the problem with the reaction system, the load test has been stopped at $Q_{max} = 27.5$ MN.

Based on the interpretation of the internal strain measurements, Mandolini [13] estimated the following values for the average skin friction and the unit base resistance: $q_s = 63$ kPa and $q_b = 2.4$ MPa. From Figure 5, it is also possible to note that, under the average load $Q_{av} = 6.0$ MN, the measured settlement is ranging between 3.5 mm (pile C) and 2.3 mm (pile D).
The construction of the three towers took about seven years (1982–1989). During the whole construction period (Figure 6), a detailed record was kept of the applied load; contemporarily, the settlement of 41 points distributed over the whole foundation area was measured by a high-precision levelling survey.

As it can be seen, most of the load (95%) was applied until the end of 1987; at that time, the measured average settlements for the three towers range between 26 mm (Tower C) and 35 mm (Tower B), with an average value $\bar{w} = 31$ mm.

In the final part of the construction period (1987–1989) and for some years after the end of construction (1989–1995), the rate of settlement remained nearly unchanged ($\sim 5$ mm/year), in spite of the very small increase of the applied load and due to the occurrence of creep deformations in pyroclastic soils.

5.1.2. Summary of the Main Experimental Results. The piled foundation adopted for the New Law Court Building in the eastern area of Naples consists of 241 large-diameter bored piles, with different diameters ($d = 1.5/2.2$ m) but same length ($L = 42$ m), on average spaced by $s = 6.1$ m. In order to refer to one single value, by weighting each pile diameter by the number of corresponding piles, the following average diameter is found: $d_{av} = 1.8$ m.

Since there are no experimental data referred to this pile diameter, it is possible to reasonably estimate the ultimate vertical bearing capacity, $Q_{lim}$, by using the experimental values obtained from the load tests for $q_{sav} (= 63$ kPa) and $q_b (= 2.4$ MPa). The integration over the shaft area and base area of a pile with a diameter $d_{av} = 1.8$ m leads to $Q_{lim} = 21.1$ MN.

In terms of single-pile settlement under working load, $Q_{av} = 6.0$ MN, the same problem occurs. It is, however, reasonable to assume that the settlement of a pile
with \(d_{av} = 1.8\) m is within the measured values for the smaller (pile C, \(w_s = 3.5\) mm) and the larger (pile D, \(w_s = 2.3\) mm) diameter. For instance, by a simple linear interpolation, it is possible to estimate \(w_s = 2.8\) mm.

Looking at the group effects, the measured settlement for the three towers yields an average settlement of the entire pile group \(w_s = 31\) mm; the resulting group settlement ratio is \(R_s = w_s/w_s \sim 11\).

5.1.3. Application of LPDM to New Law Court Building.

The structural analysis revealed high load concentration, with a maximum estimated value \(Q_{max} = 8.9\) MN. According to the Italian Code at that time (minimum factor of safety \(FS = 2.5\) for the heavily loaded pile), \(Q_{lim} = 22.25\) MN.

The pile diameter is assumed as \(d = 1.8\) m, corresponding to the pile cross section \(A = 2.54\) m².

From Table 3, for bored piles \((CR)_{av} = 11.7\) and \(CV(CR) = 0.27\), it follows that \((CR)_{av,t} = 8.51\).

Since \(Q_{lim} = 22.25\) MN, it follows that \(W = 2.61\) MN. Assuming \(Y_p = 24\) kN/m², such value for \(W\) leads to a pile length \(L = 42.8\) m (only 0.8 m, which means 2% longer than that selected in the final design stage). Assuming \(E_p = 25000\) MPa, on the basis of the \(V_S\) profile in Figure 2, after some iterations, a value of \(L_c = 33.4\) m is found. It corresponds to \(K_c = 1905\) MN/m. From Table 3, for bored piles
SR = 1.56 and CV (SR) = 0.09, it follows that (SR)_{av,r} = 1.42 and $K_0 = 2701 \text{ MN/m}$. The corresponding single-pile head displacement (elastic component) under the average vertical load is expected to be $w_{s,el} = 2.2 \text{ mm}$. If the nonlinear part of the single-pile settlement is considered, $w_s$ will be equal to 3.7 mm. A range for $w_s$ has been identified, practically coincident with the range of values measured during load tests (2.3 mm and 3.5 mm).

In terms of group effects, the resulting aspect ratio is $R = 6.6$ and the group settlement ratio is $R_g = 9.9$, which is only 10% smaller than the experimental value. The maximum group settlement ratio is $R_g = 18.9$.

It follows that the estimated average and maximum settlements of the piled foundation are, respectively, $\omega_g = 22.1 \text{ mm}$ and $\omega_{g,max} = 42.0 \text{ mm}$. It follows that the measured average settlement ($\omega_g = 31 \text{ mm}$) falls in the range of the estimated values.

Note that the nonlinear part of the settlement, $\omega_{s,nl} = 1.5 \text{ mm}$, represents about 6% of the total average settlement of the piled foundation and about 3% of the total maximum settlement of the piled foundation; it is, therefore, negligible.

5.2. Case History #2

5.2.1. Description. The two towers have the same height (86.5 m) from the ground surface (Figure 7). Each tower (U for office and A for hotel) has a steel frame structure with reinforced concrete stiffening cores to resist wind and seismic actions.

The total applied vertical load, $Q$, coming from the two towers (the small three-floor building is excluded) is approximately 410 MN, and the whole foundation area is about 2800 m². The resulting average contact pressure ($=145 \text{ kPa}$) would have produced an average settlement larger than the admissible value. Therefore, a total of 637 CFA piles (613 under the two main towers and 24 under the small building) were installed, with length $L = 20 \text{ m}$ and diameter $d = 0.60 \text{ m}$. The spacing among piles is, on average, $s = 2.4 \text{ m}$.

Each pile is subjected to an average load $Q_{av} = 0.67 \text{ MN}$. Due to the load concentration under the reinforced concrete stiffening cores, the maximum expected load is $Q_{max} = 1.37 \text{ MN}$.

Before the construction, two head-down load tests on trial piles have been tested to failure (Figure 8). The piles were instrumented over the entire length in order to measure the contributions given by the shaft and the base.

As it can be seen, pile 2 behaved better than pile 1: the maximum load reached at the end of the test was 4.8 MN and 4.2 MN, respectively, corresponding to a pile head settlement $\omega_s = 85 \text{ mm}$ and $\omega_s = 65 \text{ mm}$, respectively.

Based on the interpretation of the internal strain measurements, it is possible to estimate the following values for the average skin friction and the unit base resistance: $q_{s,av} = 90 \text{ kPa}$ and $q_b = 3.5 \text{ kPa}$. As expected, these values are slightly larger than the corresponding values for bored piles due to the positive effects induced on the surrounding soil during the screw penetration. From Figure 8, it is also possible to notice that, under the average load $Q_{av}$, the measured settlement is ranging between 1.7 mm (pile 1) and 2 mm (pile 2).

The construction of the two towers took about two years. During the whole construction period (Figure 9), a detailed record was kept of the applied load; contemporarily, the settlement of 39 points distributed over the whole foundation area of the main towers was measured by a high-precision levelling survey.

As it can be seen, at the end of construction, the measured average settlements for the two towers were different (29.2 mm for Tower A and 20.9 mm for Tower U).

It is important to add that the measurements for Tower A started before the concreting of the raft, whose corresponding average settlement measured was 2.6 mm. Since the two foundations are very similar, Mandolini [13] suggested to increase measured average settlement for Tower U by the same quantity, thus resulting in a total settlement $\omega_g = 20.9 + 2.6 = 23.5 \text{ mm}$. On the overall, at the end of construction, the two towers exhibited an average settlement $\omega_g = 26.4 \text{ mm}$. As for the previous case history, after the end of construction, an increase of the settlement has been registered, due to the occurrence of creep deformations in pyroclastic soils.

5.2.2. Summary of the Main Experimental Results. The piled foundation adopted for Towers A and U in the eastern area of Naples consists of 613 CFA piles, with the same length ($L = 20 \text{ m}$) and diameter ($d = 0.60 \text{ m}$), on average spaced by $s = 2.4 \text{ m}$.

In terms of single-pile settlement under working average load $Q_{av}$, the settlement measured during the pile load tests on trial piles is, on average, $\omega_s = 1.85 \text{ mm}$.

Looking at the group effects, the measured settlement for the two towers yields an average settlement of the entire pile group $\omega_g = 26.4 \text{ mm}$, corresponding to a group settlement ratio $R_g \sim 14.3$.

5.2.3. Application of LPDM to Towers A and U. The structural analysis revealed a maximum estimated value $Q_{max} = 1.37 \text{ MN}$. According to the Italian Code at that time (minimum factor of safety $FS = 2.5$ for the heavily loaded pile), $Q_{lim} = 3.43 \text{ MN}$. The pile diameter is assumed as $d = 0.60 \text{ m}$, corresponding to the pile cross section $A = 0.28 \text{ m}^2$.

From Table 3, for CFA piles (CR)$_{av} = 37.5$ and CV (CR) = 0.25, it follows that (CR)$_{av,r} = 28.18$.

Since $Q_{lim} = 3.43 \text{ MN}$, it follows that $W = 0.12 \text{ MN}$. Assuming $V_p = 24 \text{ kN/m}^2$, such value for $W$ leads to a pile length $L = 18 \text{ m}$ (only 2 m, which means 10% shorter than that selected in the final design stage $L = 20 \text{ m}$). Utilizing the $V_s$ profile reported in Figure 2 and assuming $E_p = 25000 \text{ MPa}$, after some iterations, a value of $L_c = 15.5 \text{ m}$ is found. It corresponds to $K_c = 456 \text{ MN/m}$.

From Table 3, for CFA piles (SR)$_{av} = 1.46$ and CV (SR) = 0.08, it follows that (SR)$_{av,r} = 1.34$ and $K = 613 \text{ MN/m}$.

The corresponding single-pile head displacement (elastic component) under the maximum vertical load is expected to be $w_{s,el} = 1.1 \text{ mm}$. If the nonlinear part of the single-pile settlement is considered, $w_s$ will be equal to 1.82 mm,
practically coincident with the average measured one (1.85 mm).

In terms of group effects, the resulting aspect ratio is $R = 9.7$ and the group settlement ratio is $R_s = 17.8$, which is about 20% larger than the experimental value. The maximum group settlement ratio is $R_{s, \text{max}} = 32.3$.

It follows that the estimated average and maximum settlements of the piled foundation are, respectively, $w_g = 19.4$ mm and $w_{g, \text{max}} = 35.3$ mm. It follows that the measured average settlement ($w_g = 26.4$ mm) falls in the range of the estimated values.

Note that the nonlinear part of the settlement, $w_{g, \text{nl}} = 0.73$ mm, represents about 4% of the total average settlement of the piled foundation and 2% of the total maximum settlement of the piled foundation; it is, therefore, negligible.
5.3. Case History #3

5.3.1. Description. Four steel tanks for the storage of sodium hydroxide, a toxic liquid with a unit weight of 15.1 kN/m³, have to be added to an already existing cluster in the area of the Port of Naples (Figure 10). The new tanks are characterized by diameters ranging from 10.5 to 12.5 m and height of 15 m. The total applied vertical load \( Q \) coming from each tank ranges from 18 to 25.5 MN. The resulting average contact pressure \( \approx 187 \text{kPa} \) would have produced an average settlement ranging from 90 to 105 mm, under static loads. It is larger than the value compatible with the safe operations of the tanks. Since the safety factor under the design load was satisfying (between 8 and 9), a piled raft foundation was considered (Figure 11).

A total of 52 CFA piles (13 piles below each tank) were installed, with length \( L = 11.3 \text{m} \) and diameter \( d = 0.60 \text{m} \). In the design stage, a trial pile was tested to about 2100 kN. From the resulting load-settlement curve (Figure 12), it may be noticed that, at a load of 1500 kN (average load level of the piles below the tanks), the secant stiffness of the test pile is 214 kN/mm. The corresponding single-pile head settlement is \( w_s = 7 \text{mm} \), which is the sum of \( w_s = 3 \text{mm} \) and \( w_s = 4 \text{mm} \).

The settlement of a number of points on the foundation rafts of the new tanks was monitored by precision levelling.
5.3.2. Summary of the Main Experimental Results. The piled raft foundation adopted for tank no. 12 in the Port of Naples consists of 13 CFA piles, with the same length \( L = 11.3\) m and diameter \( d = 0.60\) m, on average spaced by \( s = 3.5\) m.

Under the working average pile load \( Q_{av} = 1.5\) MN, the settlement measured during the pile load test on a trial pile is \( w_s = 7\) mm.

Looking at the group effects, the measured average and maximum settlements for the tank, under the working load \( Q = 23\) MN, are, respectively, \( w_g = 19.7\) mm and \( w_{g,\text{max}} = 35\) mm, corresponding to group settlement ratios \( R_s = 2.8\) and \( R_{s,\text{max}} = 5.0\).

5.3.3. Application of LPDM to Tank No. 12. The total applied vertical load is \( Q = 23\) MN. For the unpiled raft, a settlement \( w_r = 105\) mm has been estimated; thus, the soil-unpiled raft stiffness is \( K_r = 219\) MN/m. Assuming an admissible average settlement value, \( w_{sa}\), for the raft, equal to 20 mm, the...
corresponding pile group-soil stiffness is $K_g = 1150$ MN/m. From Equation (15), $a_{q_g} = 0.96$, and thus, $Q_g = 22$ MN is the load to be transmitted to the piles. 13 piles having diameter equal to 0.6 m are considered (pile cross section $A = 0.28$ m$^2$), resulting in an average pile spacing of 3.5 m.

After applying the proposed method, the pile length necessary to obtain $w_g = 20$ mm is $L = 9.8$ m, which is 15% shorter than that selected in the final design stage. For the sake of completeness, it is worth mentioning that the calculations result in a pile weight $W = 0.07$ MN; a single-pile axial bearing capacity $Q_{lim} = 1.9$ MN; a load level $\psi = 0.90$; an aspect ratio $R = 2.1$, a group settlement ratio $R_g = 1.5$; a column stiffness $K_c = 722$ MN/m; and a pile group-soil stiffness $K_g = 1150$ MN/m.

The procedure above described may be repeated assuming an admissible maximum settlement value, $w_{a,\text{max}}$, for the raft, equal to 35 mm. The corresponding pile group-soil stiffness is $K_g = 657$ MN/m. From Equation (15), $a_{q_g} = 0.92$, and thus, $Q_g = 21.1$ MN is the load to be transmitted to the piles. 13 piles having diameter equal to 0.6 m are considered, resulting in an average pile spacing of 3.5 m.

After applying the proposed method, the pile length necessary to obtain $w_g = 35$ mm is $L = 9.0$ m, which is 25% shorter than that selected in the final design stage. For the sake of completeness, it is worth mentioning that the calculations result in a pile weight $W = 0.06$ MN; a single-pile axial bearing capacity $Q_{lim} = 1.7$ MN; a load level $\psi = 0.95$; an aspect ratio $R = 2.2$; a maximum group settlement ratio $R_{a,\text{max}} = 3.2$; a column stiffness $K_c = 789$ MN/m; and a pile group-soil stiffness $K_g = 657$ MN/m.

### 6. Summary

Table 4 reports the main results obtained by the LPDM application, the main experimental results, and the main final design choices, for each case history analyzed.

As it can be seen, the agreement is rather satisfactory. For CBD case histories, the differences among the pile lengths adopted from the detailed design stages and those simply derived from LPDM are within the range −20% and +2%; the measured average settlements are within the range obtained by LPDM.

For SBD case history, that is, for a case where the settlements were imposed equal to those measured (average and maximum), the pile length from LPDM was slightly shorter (−2 than that adopted in the detailed design stage).

### 7. Conclusions

Design of a foundation system is structured in a succession of stages aimed at choosing the type of system which satisfies our needs, in the most economical way, with an adequate safety factor against a bearing capacity failure and a safe response under working loads, according to the reference regulations. An essential part of the design and construction process of a foundation is the site investigation and the pile testing. The latter must be carried out to prove the suitability of the piling system, to confirm the design parameters inferred from the site investigation, to check the construction technique and workmanship, and to confirm the performance of the pile as a foundation element. Analytical, empirical, semiempirical, and theoretical methods, in piled foundation design, have advanced very rapidly over the last decades. Nevertheless, their reliability generally depends on a wise choice of the parameters to be introduced. Although there are advances in our comprehension of geotechnical problems, it has been demonstrated [2, 3] that prediction of piles performance is often far from the actual one.

To improve our capability to evaluate pile response to loading for practical purposes on a specific project, the authors recommend setting up a local pile design method, as that illustrated in the present work. It is simply based on the identification of the following three dimensionless quantities: the capacity ratio CR and the stiffness ratio SR [5] and the group settlement ratio $R_g$ [14]. The abovementioned coefficients have been obtained by the authors for the Neapolitan area, where needed experimental data were available, but the procedure described is certainly repeatable everywhere.

LPDM has been successfully applied, as a preliminary design method, to three well-documented case histories regarding capacity- and settlement-based design of piled foundations. In the latter case, piles are designed as average settlement reducers; therefore, important considerations about the load sharing between the pile group and the raft and the stiffness of the soil-pile system have been introduced.

The agreement between the choices made by the designer for the final design of piles geometry, the experimental observations in terms of average foundation settlement, and the results of the application of LPDM is very satisfactory.

In addition, LPDM reliability has been confirmed by its application to other well-documented case histories in the eastern area of Naples, here not reported.

<table>
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<th>Case history</th>
<th>Design approach</th>
<th>$L_{\text{w}}$ vs. $L_{\text{LPDM}}$ (m)</th>
<th>$w_{g,\text{meas}}$ (mm)</th>
<th>$(w_g$ and $w_{g,\text{meas}})_{\text{LPDM}}$ (mm)</th>
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Data Availability
The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest
The authors declare that they have no conflicts of interest.

References