Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)
Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5
https://doi.org/10.53243/ICSMGE2026-1360

Load transfer mechanism on drilled shafts in Buenos Aires City

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ABSTRACT: Drilled shafts are the most common type of deep foundation in Argentina. In most cases, the design translates into defining the allowable stresses of the foundation using resistance reduction factors or a global safety factor, without much consideration of how these values relate to expected displacements, load transfer mechanisms, mobilized resistances, or the type and combination of loads considered. This set of design aspects means that the foundation, under service conditions, interacts with the ground differently than originally anticipated in the project hypotheses (Ultimate Limit State, ULS). Although there are different methods to estimate load-settlement behavior and load transfer in drilled shafts, conducting in situ static load tests on instrumented drilled shafts is the best option to analyze their behavior and adjust design hypotheses. This article presents the results of a static load test on a 500 mm diameter and 22 m length drilled shaft in soils of the Pampeano formation in Buenos Aires City. The drilled shaft was provided with a series of strain gauges installed at different depths to determine load transfer along the shaft until failure. Additionally, a production drilled shaft, 1500 mm diameter and 26 m depth, belonging to a railway viaduct located in the same geotechnical environment as the test pile, was instrumented. The instrumentation allowed determining the load transfer mechanism throughout the entire construction sequence of the viaduct pier and deck and even during the load tests of the bridge deck applying maximum live loads. This process allowed adjusting the unit shaft resistance values based on the in-situ testing performed during site investigation phase and verifying the drilled shaft's working conditions under service loads. The main conclusion of this article is that, for the viaduct's self-weight loads, there is no load transferred to the drilled shaft base.

KEYWORDS: Pampeano – Puelchense – Drilled shaft – Instrumentation – Pile load testing

1 INTRODUCTION

The typical soil profile in the city of Buenos Aires consists of well-differentiated formations whose behavior has been extensively studied since the mid-20th century (Bolognesi & Moretto, 1957; Bolognesi, 1972; Nunez, 1986; Codevilla & Sfriso, 2011). Figure 1 presents a schematic diagram of the main geotechnical units:

- Postpampeano Formation (PP): Fine, soft soils including low and high plasticity clays and low plasticity silts, normally consolidated to lightly overconsolidated due to ageing, with loose sandy lenses.
- Pampeano Formation (PA): Fine, stiff to hard silts and clays of low to medium plasticity, overconsolidated due to desiccation and erratically cemented by calcium carbonates and other precipitates. This formation is typically subdivided into three sub-units: Upper, Middle, and Lower Pampeano, which are mainly distinguished by their mechanical properties and degree of cementation. The Middle Pampeano exhibits greater strength and stiffness than the other two, (several times as a soft rock with UCS ~ 5 - 10 MPa).
- Puelchense Formation (PU): Fine to medium dense and very dense clean quarzitic uniform sands.

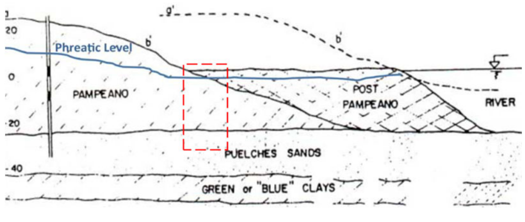

Figure 1: Typical ground profile in Buenos Aires city (not in scale).

Between the Pampeano and Puelchense geological formations, interbedding's of silty sands, sandy clays and sandy silts with dense to very dense consistency is commonly found, forming a transition layer between these formations. When it is found, this geotechnical unit is commonly referred to as Epipuelche. The thickness of each unit varies significantly depending on the distance to the coastline. Above the hill (elevation +25 masl), the Pampeano Formation is between 20 and 40 meters thick. The dashed red rectangle in Figure 1 schematically indicates the area of interest for the present study.

The conventional design approach historically employed for large diameter drilled shafts subjected to substantial loads has been to establish the shaft base within the Puelchense Formation, characterized by dense sands. Pampeano formation has been studied using various approaches for the design of excavations, anchors, tunnels, and shallow foundations. However, despite its competent geotechnical properties, when it comes to piles that must support large loads in urban areas (ex: large buildings), pile foundation in the Puelchense Formation is mandatory.

This paper focuses on analyzing the load transfer mechanism in drilled shafts installed within the Pampeano Formation. These analyses are based on measurements obtained from one load test carried out on a sacrificial drilled shaft instrumented with strain gauges at various levels, and on one production drilled shaft from the actual project (also instrumented with strain gauges at various levels). The latter was monitored throughout the entire construction sequence of the railway viaduct.

These drilled shafts are part of the San Martín Railway viaduct project. This elevated viaduct crosses the city of Buenos Aires over a length of 4.5 km, supported by approximately 800 drilled shafts with founding depths ranging from 20 to 25 meters and diameters between 1.0 and 1.5 meters. During the project, a series of six load tests were carried out on different testing drilled shafts, including two pull-out tests and four compression tests. Two of the compression tests were provided with deformation sensors (sister bars) to determine the load transfer mechanism along the shaft. One of those two tests was selected to compare its results with the load transfer measurements recorded on the production drilled shaft during the construction sequence of the viaduct. The selection criterion for the test drilled shaft was based on its proximity and similarity in geotechnical conditions to the production one.

The main goal of this article is to document the values of limit unit shaft resistance obtained from the tests for drilled shafts in the Pampeano formation. This aims to expand the available data on this type of soil, allowing readers to complement it with other recent studies addressing reliability in driven pile design (Díaz-Amar et al., 2022) and uncertainty in determining the physical and mechanical properties of the Pampeano Formation (Codevilla et al., 2024).

2 EXPERIMENTAL PROGRAM

2.1 *Site-specific ground conditions*

The section of the project under study is in the city of Buenos Aires, among the neighborhoods of Palermo, La Paternal, and Villa Crespo. Figure 2 presents an aerial view of the Buenos Aires city sector showing the location of both drilled shafts under analysis, indicating the distance between them.

Figure 2: Aerial view of the location of both drilled shafts under analysis.

Figure 3 displays the representative geotechnical profile of the sector of interest, along with the location and depth of the two drilled shafts analysed.

The profile matched well with the typical sequence described in Figure 1: an upper layer of poorly competent natural high plasticity soft fill (Postpampeano) approximately 2.0 m thick, followed by the Pampeano Formation with variable thicknesses in the project area ranging from 18 m to 22 m. Beneath the Pampeano Formation, a layer of sandy silts and dense to very dense silty sands were identified, with a variable thickness of between 2 m to 11 m, along with some isolated sand lenses. This geotechnical unit is associated with the Epipuelche.

Below the Epipuelche, another layer of overconsolidated silts was encountered, with a thickness ranging from 4 m to 10 m. Finally, at depths beyond 30 m to 35 m, the Puelchense Formation was detected. As shown in Figure 2, both drilled shafts under analysis resulted in their bases bearing on the Epipuelche Formation.

2.2 *Drilled shaft load testing*

In the context of the viaduct project, an instrumented axial compression load test was carried out on a 500 mm diameter 20.8 m length drilled shaft. The location of the test drilled shaft was selected in an area of the project with available space so as not to interfere with construction activities. The objective of this test was to determine the following:

1. Load (kN) – settlement (mm) curve,
2. Q_{ult} (kN): ultimate axial compressive drilled shaft strength,
3. load–transfer relationship,
4. Q_{sL} and $Q_{b,ult}$ (kN): ultimate shaft resistance and ultimate base resistance, respectively,
5. q_{sLi} (kPa): limit unit shaft resistance for each soil layer crossed,
6. $q_{b,ult}$ (kPa): ultimate unit base resistance.

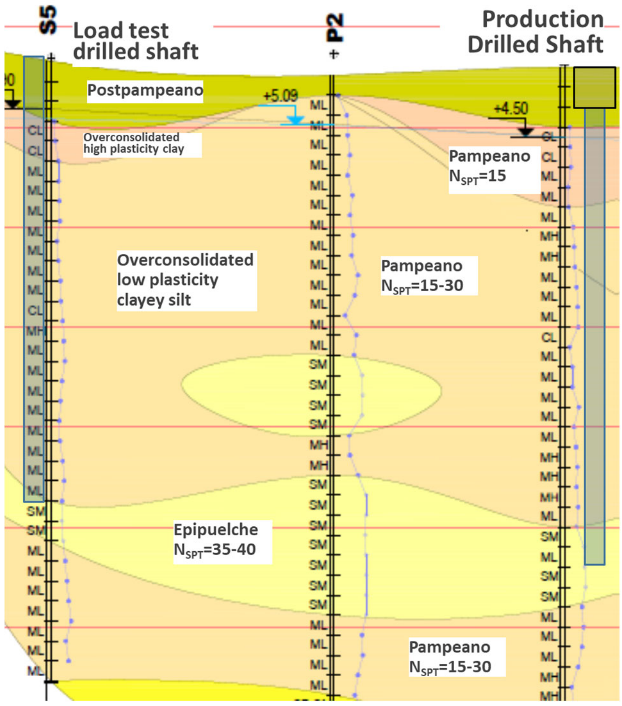

Figure 3: Site-specific ground profile corresponding to the project sector under analysis.

The test involved the installation of a main frame by adding four reaction drilled shafts arranged in a square configuration, with the test shaft positioned at the center. The test drilled shaft was provided with a square reinforced concrete cap to evenly distribute the load from the two hydraulic jacks, which rested on a 2-inch-thick steel plate to prevent stress concentrations on the cap. The distance between the reaction drilled shafts and the test pile was 2.50 m. This reaction drilled shafts supported a large and stiff steel beam, which enabled the application of load to the central shaft via a system of two 2000 kN hydraulic jacks placed between the beam and the shaft head. The loading procedure followed a test protocol based on the NFP 150-1 standard (1999), applying a continuous sequence of load steps and cycles to the central pile. A total of two full loading and unloading cycles were carried out, comprising 17 load steps in total. Each step lasted at least 1 hr, allowing settlement stabilization at each loading stage. Displacement readings of the test shaft were taken using 6 dial gauges: 4 measured vertical settlements on each side of the cap, while the remaining 2 were installed to measure in a horizontal plane along two orthogonal directions to monitor parasitic bending. In addition, a dial gauge was placed on each reaction drilled shaft to measure the potential load-uplift behaviour and monitor the safety of the test. All dial gauges were installed on a system of two reference beams, the ends of which were supported outside the influence zone of the five drilled shafts involved in the test. Additionally, these reference beams were tied to an external system through topographic measurements taken at the beginning and end of each load step. Figure 4 shows a picture of the load test configuration.

To determine the load–transfer relationship and, from this, calculate the ultimate shaft resistance and ultimate base resistance along with their respective limiting unit resistances, strain gauges were installed along the shaft. Six vibrating wire strain gauge sensors, known as sister bars (Geokon, Model 4911), were chosen due to their ease of installation on-site. These sensors consist of a 900 mm long reinforcement bar with a vibrating wire strain gauge installed in the center of the length. The installation of these sensors was carried out using nylon ties to secure the Siter bars to the longitudinal rebars of the drilled shaft. In the same manner, the sensors cables extending to the surface were also fastened. The sister bars were installed by pairs on opposite sides of the rebar cage at three different levels: -3.5 m, -9.0m and -17.9 m all measured from ground surface level. Figure 5 shows a pair of sister bars attached to the rebar cage before its installation within the borehole. The sister bars readings were carried out using a datalogger Geokon, Model LC-2x16-2 (USB) and stored in a laptop. In this case, the reinforcement steel cage was installed in 2 sections. Therefore, it was necessary to plan the installation of sensors for the lower section and prepare for the passage of cables in the overlapping length between the lower and upper sections of the reinforcement.

Figure 4: Execution of the drilled shaft axial load testing.

Figure 5: Strain gauges (sister bars) installed in the rebar cage.

Twenty-eight days later, the load test was carried out. Prior to commencement, the sensors were connected to the data logger, the stability and consistency of the readings were verified, and a zero-reading was taken before the application of load began. Deformation readings were recorded every 30 seconds throughout the duration of the test.

2.3 *Instrumentation of the production drilled shaft and monitoring program*

With the aim of verifying the performance in terms of the load-transfer, a production drilled shaft was instrumented. It forms part of a center pier for a two 45-metre-long spans crossing the Juan B. Justo and Cordoba avenues. This pier is founded unsymmetrically on two drilled shafts connected by a stiff cap. The shafts have a diameter of 1.5 m, with their bases founded at a depth of 26 m below natural ground level. The group cap has a height of 2.0 m, resulting in a total shaft length of 24 m.

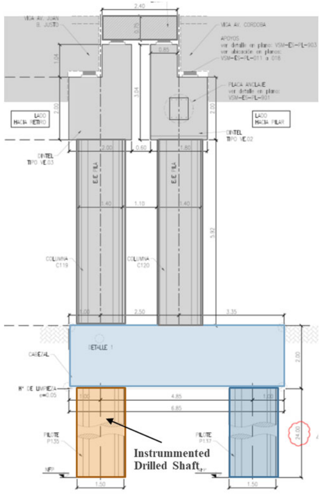

Figure 6: Diagram of the viaduct pier with the instrumented drilled shaft.

The same model of sister bars was selected for the instrumentation. 9 sister bars were installed at 3 levels. At each level, three sister bars were positioned at 120-degree intervals. The depths of these sections were -7.0 m, -10.0 m, and -22.0 m, measured from ground level. The sister bars were attached to the reinforcement cage using nylon ties, along with the transmitter cables. The installation process was carried out for the load test shaft. Figure 7 shows a photograph of the installation process. At the end of the installation, the cables were stored in a protective box. The correct functioning of the sensors was monitored during the first 14 days, and a zero-reading was taken after reading stabilization once temperature effects and shrinkage no longer had any influence on the strain readings.

Figure 7: Strain gauges (sister bars) installed in the rebar cage of the production drilled shaft.

Subsequently, deformation measurements were recorded throughout the pier construction process, with readings taken at the following milestones: (1) completion of the group cap, (2) completion of the pier column, (3) completion of the lintel beam, (4) installation on of bearing slabs, (5) installation of the viaduct decks, (6) completion of viaduct construction (including ballast and railway track), and (7) measurements during load tests using two 1100 kN locomotives positioned both on the viaduct spans and directly above the pier. All measurements were taken continuously using the data logger until deformation readings were stabilized. Figure 8 and Figure 9 show a picture of the strain's measurements during viaduct deck load testing.

Figure 8: Strain measurements during viaduct deck load testing.

Figure 9: Strain measurements of the production drilled shaft.

3 RESULTS AND ANALYSES

3.1 *Load testing results*

Figure 10 shows the load-settlement curve at drilled shat head. Chin (1970) suggested a method to interpret the load test results if the load test is extended to sufficiently large deflections. This method assumes that the load-settlement relation is hyperbolic:

$$\frac{\delta}{Q} = A\delta + B \qquad (1)$$

where Q is the axial load applied to the drilled shaft head, δ is the settlement at the drilled shaft head corresponding to the axial load Q, A and B are the slope and intercept of the axial load–settlement curve in the 1/Q against δ space. The limit axial load capacity (theoretical failure load) of the drilled shaft resulted equal to 1/A and resulted in 3670 kN for the tested drilled shaft. By adopting as the failure load the load corresponding to 10% of the settlement-to-diameter ratio (δ/D=10%) of the drilled shaft, a value of 3130 kN was obtained. The ratio between these two values was 0.85.

Figure 11 shows the load-transfer curves obtained for only the loading steps applied (unloading steps are not presented). From these load-transfer curves, the following was established: (1) by extending the slopes of each curve down to the level of the base of the drilled shaft, the load transferred to the base can be estimated, (2) the difference between the load applied at the head and the load transferred to the base corresponds to the load carried by the shaft; (3) From a load of 2100 kN onwards, the slopes of the curves begin to remain constant; (4) it was determined that from this load value, the shaft load remained constant (no showed in this paper); (5) the mobilized settlement for this load corresponds to δ/D = 2.7%.

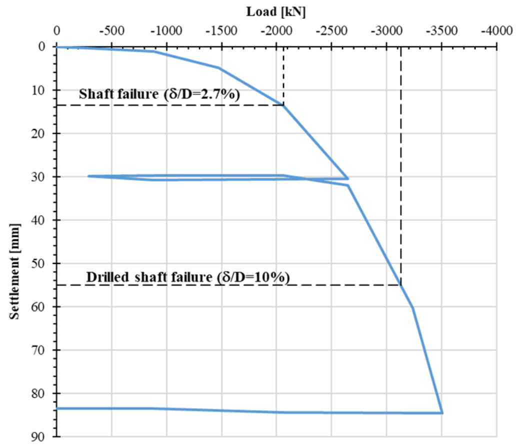

Figure 10: Load-settlement curve at drilled shaft head.

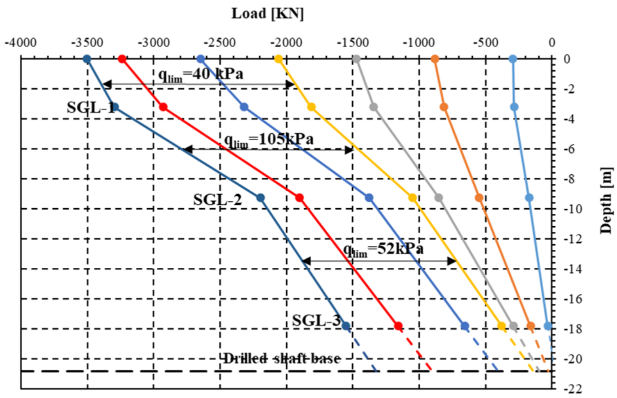

Figure 11: Load-transfer curves each loading steps.

Loads at each strain gauge level (SGL-i, in Figure 11) were determined for each loading step based on the stabilized strain gauges readings and the equivalent axial stiffness of the drilled shaft section. The equivalent axial stiffness was determined based on the concrete compressive strength, the Young's modulus of steel (200 GPa) and the respective areas of concrete and steel. In addition, 3 values of average limit unit shaft resistance for each soil layer crossed were after the shaft failure, 40 kPa, 105 kPa and 52 kPa. These values correlate well when adopting the upper bound values proposed by Codevilla and Sfriso (2011) for the mentioned soils, using the following methods:

- modified β-Burland method (adopting the $\beta = K_0 \cdot tan\phi'$ and the drained cohesion, where K_0 is the coefficient at rest pressure and ϕ' is the effective friction angle);
- α-Tomlinson method, adopting the upper bound values of the undrained shear strength, s_u, for each soil respectively.
- Decourt (1996) method, considering N_{SPT} = 15 for the Upper and Lower Pampeano, and N_{SPT} = 30 for the Middle Pampeano. In all cases, a reduction coefficient of 0.9 was adopted for clayey soils excavated with bentonite.

The ultimate unit base resistance determined for the failure load was 4.6 MPa. The empirical method of Decourt and

Cuaresma (1978), for a N_{SPT} value of 30 adopting the empirical coefficient C = 300 kPa (silty sands) and construction method and soil type factor a = 0.5 (drilled shafts in sandy-like soils), was the one that best matched that value of ultimate unit base resistance.

3.2 *Production of drilled shaft*

Strains were recorded at 3 different drilled shaft levels throughout the entire construction sequence of the viaduct pier and decks, as well as during proof load testing. The static proof tests involved positioning one or two 1100 kN locomotives at critical locations to assess structural integrity prior to commissioning the bridge for rail traffic. With the strain data and the axial stiffness of the drilled shaft, load-transfer curves were obtained for every single stage of construction. Figure 12 shows the load-transfer curves. These curves are the result of averaging multiple strain measurements taken during each construction stage. Three different sets of load-transfer curves can be clearly observed. The first set, comprising the curves furthest to the left on the graph, corresponds to the construction sequence of the pile cap, pier column, lintel beam, support slabs, and complementary structural elements of the pier. The second set, consisting of the blue and brown curves located in the center of the graph, corresponds to the intermediate construction stages of the viaduct decks (blue curve) and the completion of the viaduct decks (brown curve, marked as "Viaduct only"). This "Viaduct only" curve represents the load-transfer at the end of viaduct construction. Two main aspects emerge from the analysis of this curve:

1. The mobilized unit shaft resistance was in the order of 42 kPa, which is very close to the ultimate shaft resistance value determined during the load test for the Upper Pampeano geotechnical unit.
2. By extending the slope of this curve down to the level of the base of the drilled shaft, the load transferred to the base can be inferred as nil. This means that the self-weight of the viaduct is being resisted by the mobilized shaft resistance.

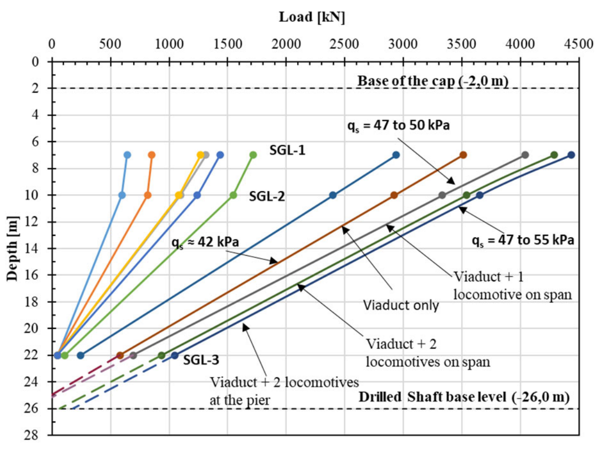

Figure 12: Load-transfer curves for the viaduct construction sequence and deck proof load testing.

The third set of load-transfer curves (located further to the right on the graph) corresponds to the curves obtained for the stages of load testing of the viaduct deck. From the analysis of this set of curves, the following can be highlighted:

1. The curve "*Viaduct + 1 locomotive on span*" shows a mobilized unit shaft resistance in the range of 47 kPa to 50 kPa, which slightly exceeds the ultimate unit resistance value for the Upper Pampeano obtained during the load test, but remains below the ultimate unit resistance of the Middle and Lower Pampeano. On the other hand, it is evident that the load produced by a locomotive positioned at the center of the viaduct span does not generate load transfer towards the base of the drilled shaft.
2. The curves "*Viaduct + 2 locomotives on span*" and "*Viaduct + 2 locomotives at the pier*" indicate that part of the load by the two locomotives has been transferred to the tip of the pile, although the amount is relatively low compared to the drilled shaft strength (estimated to be less than 200 kN). The unit shaft resistance increased slightly to values between 47 kPa and 55 kPa, which remain slightly above the ultimate unit resistance of the Upper Pampeano, are similar to the ultimate unit resistance of the Lower Pampeano and remain lower than that of the Middle Pampeano.
3. No evidence of settlements was found at the ground surface during the proof load test stage.

4 CONCLUSIONS

The study presented herein contributes to the understanding of load-transfer mechanisms in drilled shafts founded within the Pampeano Formation in Buenos Aires. Through the combined analysis of a static load test on an instrumented sacrificial shaft and instrumentation of a production shaft throughout the viaduct construction sequence, several key findings have emerged:

- The limit unit on shaft average resistance in the Pampeano Formation determined through instrumented load testing align well with established semi-empirical methods, including the modified β-Burland, α-Tomlinson methods, and empirical SPT-based Decourt (1996) approach by considering the upper bound values proposed by Codevilla and Sfriso (2011) for the Paampeano Formation subunits.
- Ultimate base resistance was determined to be approximately 4.6 MPa, consistent with predictions from the Decourt and Quaresma (1978) empirical SPT-based method for dense silty sands.
- The mobilised unit shaft resistance under service loads, particularly the self-weight of the viaduct, was found to be approximately 42 kPa. This value closely matches the ultimate shaft resistance determined for the Upper Pampeano unit, confirming the reliability of in-situ testing for design calibration.
- No load transfer to the base of the drilled shaft was observed under the viaduct's self-weight, indicating that the shaft resistance alone is sufficient to support permanent structural loads in this geotechnical and design contexts.
- Load testing with locomotives revealed that static live loads can mobilise slightly higher shaft resistances (up to 55 kPa), and in some cases, minor load transfer to the shaft tip was detected. However, these values remain within safe limits and below the ultimate resistance of the Middle Pampeano unit.
- The results validate the use of drilled shafts founded in the Pampeano Formation for substantial structural loads, challenging the conventional preference for founding in the deeper Puelchense sands.
- This study reinforces the importance of instrumented load testing and strain monitoring throughout construction to refine design assumptions and ensure geotechnical reliability.

- Field measurements during viaduct construction and live load testing revealed that base resistance is only marginally mobilised under extreme loading scenarios. These findings validate the use of shaft resistance as the primary load-bearing mechanism under service conditions (service limit states).

Overall, this work expands the body of knowledge on deep foundation performance in overconsolidated clayey soils and supports a more nuanced approach to pile design in urban geotechnical environments such as Buenos Aires.

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