Degradation of homometric sands mechanical properties during continuous flight auger piles installation

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ABSTRACT: Flight auger piles are a widely used technique for installing deep foundations, thanks to their numerous advantages, such as high execution rates and improved geotechnical resistance compared to other bored pile methods. However, these techniques are known to be sensitive to certain ground conditions, particularly in the case of homometric fine sands below the water table. Such conditions can lead to over-augering effects, but quantitative impacts on (i) ground mechanical properties and (ii) pile behaviour remain vague and are not fully addressed in current practices or reference documents. This paper is based on two documented construction sites in northern France, both located in fine homometric sands under the water table and affected by significant over-augering. It shows a detrimental reduction in mechanical resistance, as evidenced by CPT testing conducted before and after CFA piles installation, at various distances and under cumulative piles installation effects. The study provides insights into the magnitude of these effects and their attenuation with distance from the piles. The consequences for piles behaviour are illustrated, focusing on axial resistance and stiffness.

KEYWORDS: Sands, execution, CFA, over-augering, over-flighting.

1 INTRODUCTION

Pile foundations are widely employed in civil engineering to transfer loads from heavy structures through weakly compressible or water-saturated soil layers to more resistant strata. Within the field of deep foundations, various types and categories of piles exist, along with multiple implementation techniques. In France, the CFA (Continuous Flight Auger) piling method has gained prominence as an efficient solution, offering both fast installation and reduced costs compared to more conventional methods (Ait Ali, 2022).

However, despite its numerous advantages, this technique may induce disturbances in the surrounding soil, in some specific geological contexts (Van Weele, 1988, FHWA, 2007, Symkiewicz et *al.*, 2024). In northern France and in so-called Dunkirk Sands, during the installation of piles in saturated, homometric fine-grained sandy soils under the water table, significant degradation of the surrounding soil's mechanical properties was observed. These disturbances, documented on two separate construction sites, were characterized by a substantial decrease in soil strength and noticeable surface settlements.

This study aims to investigate the evolution of the soil's mechanical behaviour during the execution of CFA piles, to identify the underlying degradation mechanisms, and to propose a constitutive model capable of representing this phenomenon. The primary objective is to assess the impact of such degradation on the bearing capacity and settlement performance of piles—both isolated and grouped—to enhance the design and construction practices of deep foundation systems.

2 BACKGROUND

In order to investigate the degradation phenomenon, several site investigation campaigns involving cone penetration tests (CPT) were conducted at both project sites, prior to and following the installation of the piles. These piles were founded in marine white sands, belonging to the Upper Flandrian series, which are characterised by their fine grain size and homometric nature, under the water table.

The CPTs were performed at varying radial distances r from the pile axes using an electric cone. The measured cone resistance values q_c revealed a significant reduction in proximity to the pile shafts when compared to reference values obtained at distances greater than 15 metres. Moreover, this degradation occurred down to the base of the piles, after which the soil remained unaltered.

This article provides a detailed account of the testing procedures, the geotechnical conditions of the sites, and the results obtained. Furthermore, it presents an interpretation of the observed low bearing capacities, attributed to the phenomenon of over-augering.

Excessive rotation of the auger relative to its vertical advancement rate can result in over-augering (or over-flighting) by drawing soil upward through the flights, thereby reducing mechanical properties of the surrounding soils, compromising the bearing capacity of adjacent foundations, but also under some extreme conditions resulting in surface settlements.

The proper execution of CFA piles relies on achieving a balanced ratio between penetration speed and rotational torque (NF EN 1536+A1, CEN, 2015).

Although no quantitative approach is provided for this parameter in execution codes, for non-cohesive, homometric soils situated below the water table, the limiting rate of penetration (corresponding to the number of rotation by auger pitch during drilling, that can be obtained by Equation (1)) is generally considered to lie between 1 and 2 revolutions per pitch maximums (FHWA, 2007).

$$RP = \frac{p\,.RS}{AS} \tag{1}$$

Where p is the auger pitch length, RS the rotational rate (number of revolutions divided by time) and AS the auger downward penetration rate (drilled length divided by time) (Kenny et *al.*, 1997).

3 SITES AND PILES FEATURES

3.1 *Properties of sandy soils*

Geotechnical investigations were carried out at the two construction sites. The geotechnical cross-sections are provided in Table 1.

Regarding grain size distribution, it is observed, among other characteristics, that the Flandrian sands (see Figure 1) are

i) fine, ii) highly uniform graded or homometric, and iii) exhibit a low fines content. The lower sands show a broader range of mean particle diameters, with D_{50} values reaching up to 0.3 mm, whereas in the upper sands this value does not exceed 0.2 mm.

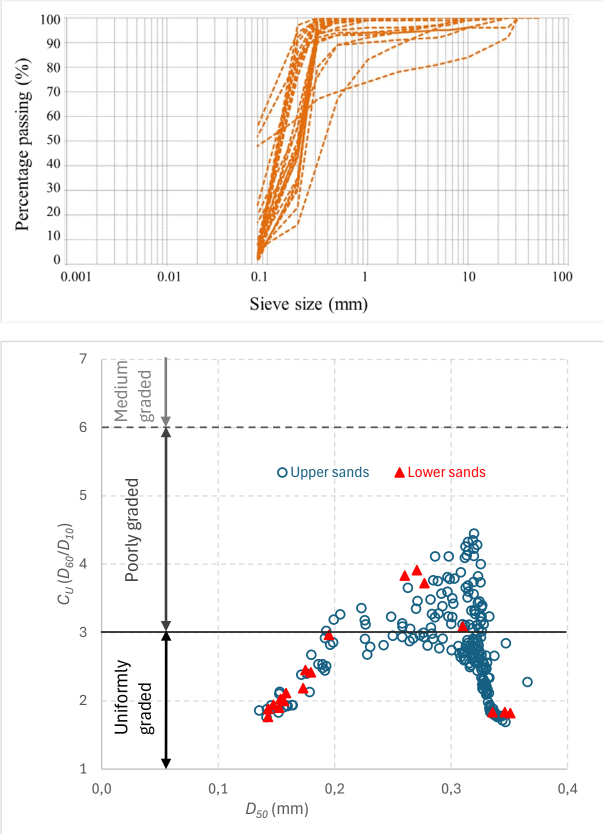

Figure 1. Typical grain size distribution curves of Flandrian sands – a) typical granular curves, b) D_{50} and C_U.

The fill materials are predominantly originated from in-situ Upper Sands.

Table 1. Geotechnical models at Dunkirk.

	Site 1	Site 2
Nature	**Top of the layer (m/ground level)**	
Sandy Fill	0	-
Upper Sand	14	3
Sandy Clay	-	11
Lower Sand (1, 2 and 3)	-	15
Flanders Clay	-	25

A total of 26 CPT tests has been carried out between 2017 and 2024. The initial tests, carried out prior to pile installation, revealed a soil profile with cone tip resistance values ranging between 25 and 40 MPa for the sand, as shown in Figure 2.

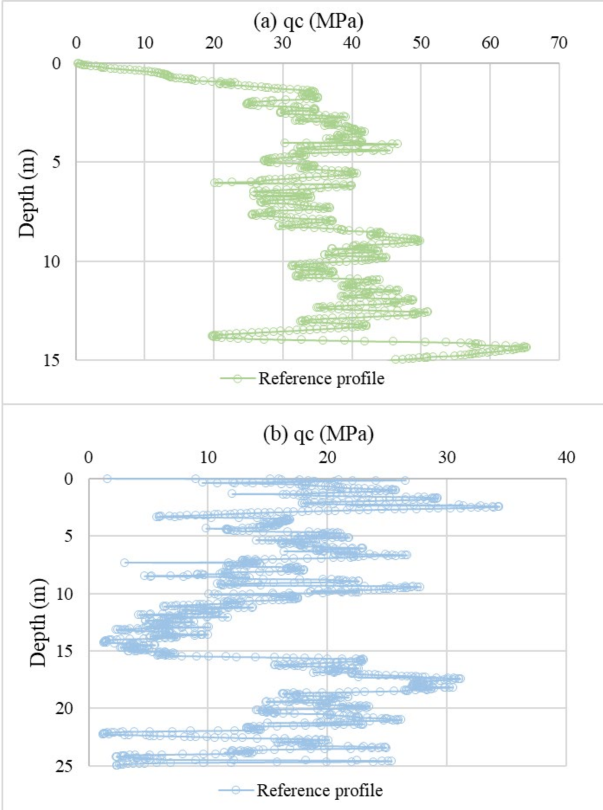

Figure 2. Evolution of the penetrometer cone resistance as a function of depth, before pile execution for (a) sandy fill and (b) in-place sands.

The representative values retained as a result of the surveys are presented in Table 2.

In addition, the soil layers presented exhibit the properties listed in Table 2. This table includes the average values of D_{50}, Uniformity coefficient C_U and of CPT tip resistance q_c measured prior to pile installation.

Table 2. Layer's properties at Dunkirk.

Nature	D_{50} [mm]	C_U (D_{60}/D_{10})	q_c [MPa]
Upper Sand	0.15-0.35	3 (2-5)	15 (5-30)
Lower Sand	0.15-0.35	3 (2-4)	20 (10-30)
Sandy Fill	0.15-0.35	3 (2-5)	35 (25-45)

3.2 *CFA piles dimensions and execution*

A lot of piles were constructed across both projects and were installed using the CFA technique. In this study, 25 piles were subjected to measurements taken in their immediate vicinity. Their dimensions vary depending on the project and are summarised in Table 3.

Table 3. Geometric characteristics of piles.

Parameter	Symbol	Project 1	Project 2
Length (m)	D	12.0	27.5
Diameter (m)	B	0.8	1.0

The piles in Project 1 form part of the foundation of a gantry crane. They are spaced 2.3 metres apart.

In Project 2, the structure is a portal-frame type supported on piles. In this case, the pile spacing is 3 metres.

Regarding the rate of penetration, this parameter was recorded for nine piles from Project 1 and two piles from Project 2. The corresponding values are shown in the Figure 3.

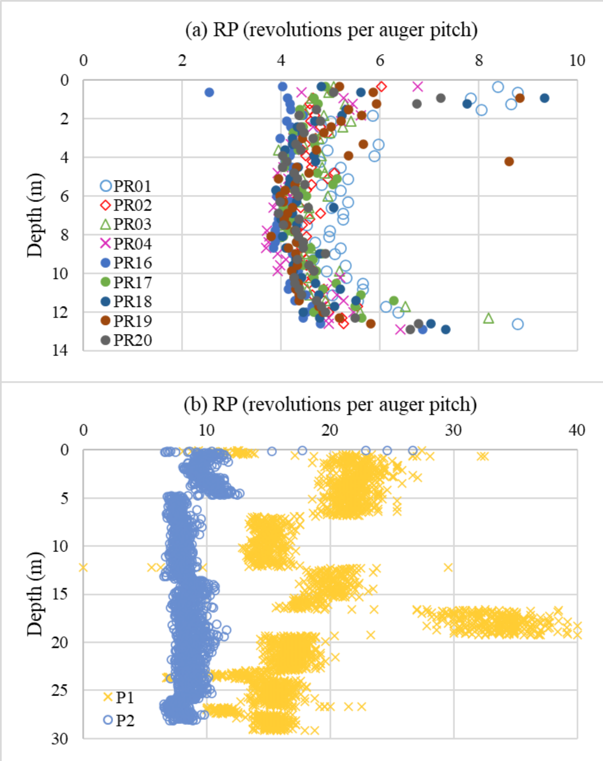

Figure 3. Rape of Penetration (RP) as a function of depth, (a) Project 1, (b) Project 2.

Considering that the recommended upper limit for *RP* is 2 revolutions per auger pitch, it is evident that this threshold was significantly exceeded in both cases—with values ranging from 4 to 6 revolutions in Project 1 and reaching 10 to 30 in Project 2.

4 EXPERIMENTAL RESULTS

4.1 *Raw experimental data*

CPT results have revealed that soil degradation is dependent on both the radial distance from the pile axis *r* and the depth.

Focusing first on the former, Figure 4 presents the results of some of these tests: it can be seen in both cases, (a) and (b), that the reference profile for each project is shown, along with a series of profiles at various distances (r = 1 m, 2.3 m, and 4 m for Project 1, and 1.2 m and 2.5 m for Project 2).

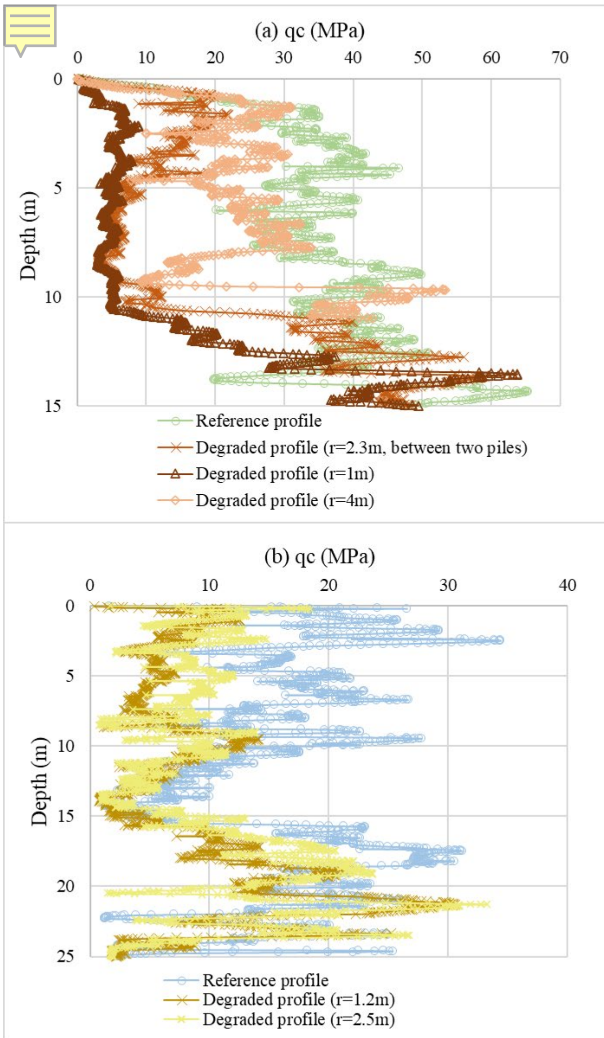

Figure 4. Evolution of the penetrometer cone resistance as a function of depth, with an undeteriorated and an altered profile for (a) Project 1 and (b) Project 2.

From the Figure 4, the drop in resistance is clearly observed at short distances from the pile and this reduction gradually diminishes as the radial distance *r* increases. Furthermore, it is apparent that the degradation extends down to the pile base, beyond which the soil appears to return to its initial, undisturbed state.

Furthermore, from Figure 4 (a), the CPT carried out at r = 2.3 m corresponds to a test performed between two piles. Therefore, the degradation is greater and approaches the q_c values of the CPT conducted at r = 1 m.

When analysing the second influencing factor — depth — it can be observed from the Figure 5 that the difference between the degraded cone resistance and the reference value $\Delta\ q_c$ increases linearly with depth.

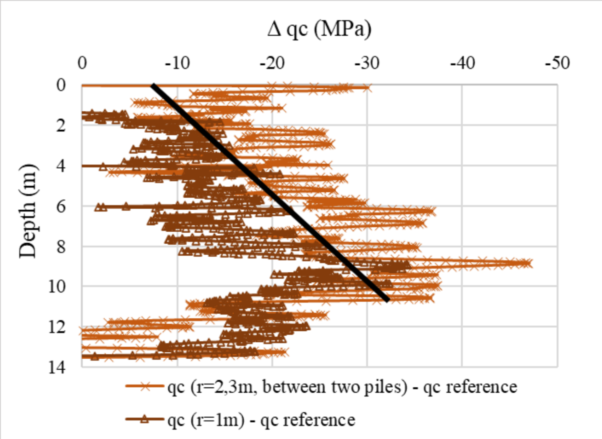

Figure 5. Evolution of the difference between degraded qc(r) and reference qc as a function of depth, for Project 1.

4.2 *Characterization of mechanical properties degradation*

Given that soil disturbance is therefore a function of both radial distance and depth, a representative degradation model has been proposed for each soil layer. The reduction factor F is defined as the ratio between the degraded cone tip resistance and the reference cone tip resistance.

$$F = \frac{q_{c,degraded}}{q_{c,reference}} \tag{2}$$

For each radial distance and soil layer, average values of F have been determined (see Figure 6). These were then subjected to various regression analyses, including linear and logarithmic models.

It's worth to mention that these data point correspond to varying revolutions per auger pitch ratios RP and to CPT were performed at varying times after the pile installation: it has however not possible to distinguish the effect of these parameters on the cone tip resistance degradation.

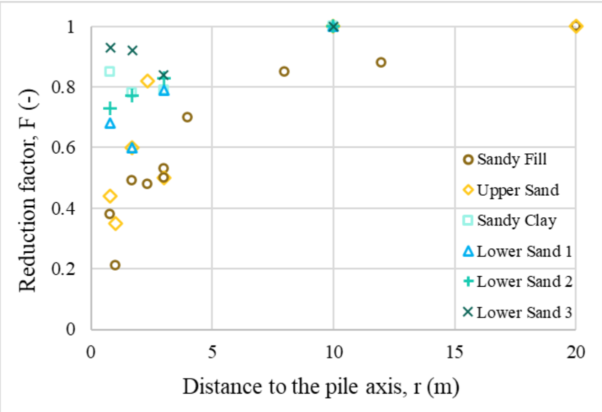

Figure 6. Evolution of the average reduction factors with distance r, for each soil layer, obtained in both projects.

Ultimately, the linear regression provided the best approximation, resulting in a degradation law dependent on distance on the pile axis r and diameter B, of the following form:

$$F(r, B) = a + b \cdot \frac{r}{B} \tag{3}$$

Furthermore, since the degradation extends only down to the pile toe and not below it, not all the soil layers presented in Table 1 are associated with a degradation law.

By imposing the condition that $F(r{=}15m) = 1$, thereby assuming a return to the initial soil properties beyond this distance, the following values of a and b were obtained, along with their respective coefficients of determination R^2.

Table 4. Soil degradation model for each soil layer

Nature	a	b	R²
Sandy Fill	0.344	0.044	0.808
Upper Sand	0.453	0.036	0.696
Flandrian Sand	0.746	0.017	0.620
Lower Sand 1	0.542	0.031	0.822
Lower Sand 2	0.732	0.018	0.890
Lower Sand 3	0.926	0.005	0.010

5 APPLICATION TO DESIGN

The degradation law for each soil layer was applied to evaluate the consequences of this degradation on the behaviour of piles: axial behaviour (bearing capacity and settlements), considering both the undisturbed and degraded soil profiles.

In French practice, piles design is based on in situ tests. Bearing capacity is assessed through correlations to the Ménard pressuremeter limit pressure p_{lM} or CPT tip resistance q_c. whereas. Axial and transversal behaviour are then modelled through load transfer approaches, simplifying the soil around the pile as equivalent independent elasto-plastic springs with t-z (axial loading, Seed and Reese, 1957) and p-y (transversal loading, Matlock & al, 1956) modelling. All these methods suppose that the ground is laterally homogeneous

In the present case, as the mechanical properties depend now on the distances to the piles, effect of pile installation has been addressed implementing finite element method (FEM).

Using the FEM, the soil–structure domain is discretized into small elements, and the field variables are approximated using polynomial shape functions. This transforms the governing differential equations into a solvable system of algebraic equations. Soil behaviour was modelled using the Mohr–Coulomb failure criterion, a linear elastic perfectly plastic model that describes the material response.

The pile model to be used has a diameter of 1 m and a length of 12 m (Project 1).

5.1 *Axial performance*

5.1.1 *Single pile*

To begin with the axial loading case, an isolated pile was analysed. The interface was modelled by assigning the cohesion value equal to the unit shaft friction resistance q_s.

As previously mentioned, since the q_c values vary as a function of the radial distance from the pile, parameters such as the internal friction angle φ and Young's modulus E, which can be derived from cone resistance, are shown to vary as a function of r. The former was calculated using the relationship proposed by the Eurocode (CEN, 2006), expressed as:

$$\varphi(r) = 13.5 \cdot \log(q_c(r)) + 23 \tag{4}$$

With q_c expressed in MPa. Young's modulus was estimated as follows (Robertson and Campanella, 1983):

$$E(r) = 2 \cdot q_c(r) \tag{5}$$

Knowing that the ultimate compressive resistance $R_{c,\,ULS}$ can be estimated as the load corresponding to a settlement equal to $B/10$, and that this resistance is the sum of shaft friction R_s and end bearing R_b, it was calculated for the undisturbed soil profile.

Table 5. Ultimate compressive resistance values before calibration of an isolated pile with intact soil for two methods: FE and t-z.

Method	R_s [MN]	R_b [MN]	$R_{c,ULS}$ [MN]
FE	6	7	13
t-z	6	3	9

As shown in Table 5, the resistance values — particularly in terms of end bearing — differ significantly between the two methods. In response, calibration was carried out for the FEM, to increase the tip resistance and the shaft stiffness, to better match the results obtained with the t–z method. The final FE model adopted is based on the following relationships:

$$\varphi(r) = 13.5\,.\log\big(q_c(r)\big) + 30 \qquad (6)$$

With q_c expressed in MPa.

$$E(r) = 5\,.\,q_c(r) \qquad (7)$$

In addition, the cohesion value was increased to ensure model convergence. It was set at 5 kPa.

The final resistance values, considering the adjustments mentioned above, are presented in the Figure 7. It shows the ultimate compressive resistance for both models and soil profiles—undisturbed and degraded.

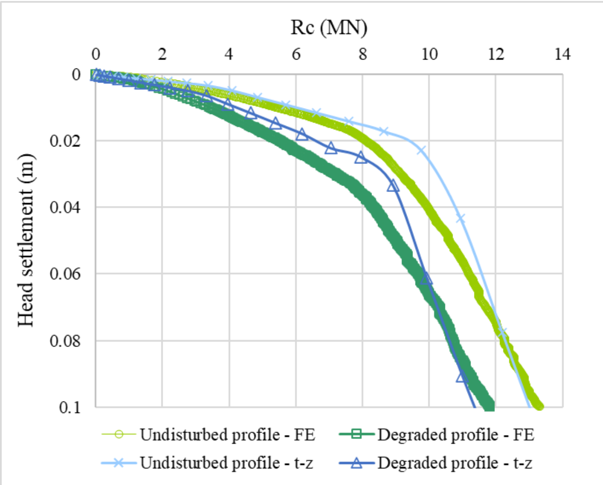

Figure 7. Ultimate compressive resistance of an isolated pile in intact and altered soil profiles for two methods: FE and t-z.

To model the conditions of the altered soil profile with the FEM, different vertical layers were defined, each assigned average q_c values based on the corresponding soil layer and radial distance. In contrast, the t–z method does not permit consideration of horizontal heterogeneity within the soil model; therefore, the soil parameters were calculated using q_c $(r = B/2)$.

Furthermore, by applying a service load calculated using Equation (8), the corresponding settlement values can be obtained.

$$R_{c,SLS} = \frac{R_{c,ULS}}{3} \qquad (8)$$

The Table 6 summarises the results for both resistance and settlement, for each method and soil condition.

It can be observed that, simplified approach for both bearing capacity and settlements assessment appear relevant. For both models, the loss in ultimate resistance is approximately 11%, while settlement nearly doubles in the case of the degraded profile: the higher effect on settlements can be explained by the cumulative effect both unit shaft friction and ground modulus decrease.

Table 6. Ultimate compressive resistance values, strength loss and settlement, after calibration of an isolated pile with intact and altered soil for two methods: FE and t-z.

Method	Soil Profile	$R_{c,ULS}$ [MN]	Strength loss [%]	Settlement under 4.5 MN [mm]
FE	Intact	13	11%	7
	Altered	12		12
t-z	Intact	13	11%	6
	Altered	12		12

5.1.2 *Pile group*

Subsequently, three different pile group configurations were analysed (comprising two, three, and twenty piles) using FEM.

Knowing the reduction factor *F* for an isolated pile in each soil layer, when considering a pile group, the overall value of *F* at a given point was computed as the product of the individual reduction factors associated with each pile, depending on the radial distance between that point and each pile. This approach enabled the determination of *F* isovalues for the various group configurations, and thus the geometry and degraded properties of each corresponding soil section.

The Figure 8 presents the *F* isovalues for an isolated pile and for a group of two and three piles. It is evident that, at an equal distance, the *F* values are lower for the two- and three-pile configurations than for the single isolated pile.

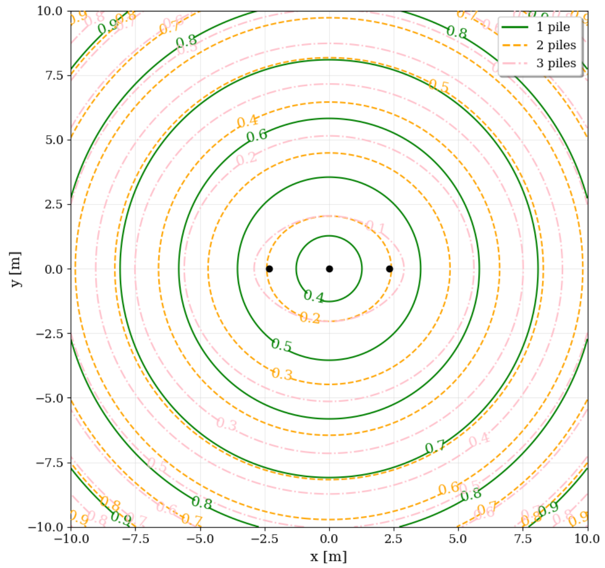

Figure 8. *F* isovalues for one, two and three piles configurations, for the Sandy Fill layer and considering B = 1 m.

Table 7. Ultimate compressive resistance values, strength loss and settlement, after calibration of a group of piles with intact and altered soil using FEM.

Number of piles	Soil Profile	Rc, ULS [MN]	Strength loss [%]	Settlement under 4.5 MN [mm]
Two	Intact	30	16%	11
	Altered	25		21
Three	Intact	41	26%	12
	Altered	30		28
Twenty	Intact	233	27%	16
	Altered	170		37

As shown in Table 7 and in Figure 9, the bearing capacity exhibits a sharp decrease between the two- and three-pile configurations. However, between three and twenty piles, the additional reduction in resistance is limited to only 1%. This latter observation is directly related to the method used to calculate the reduction factor *F* and the linear degradation law.

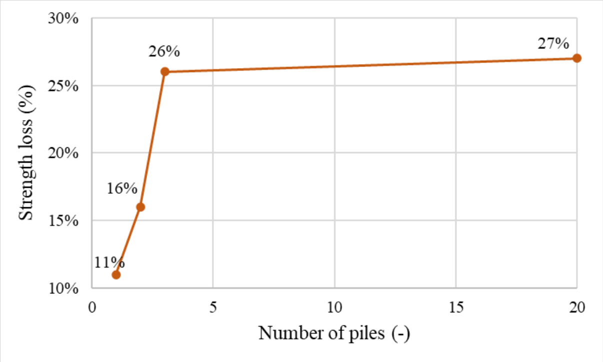

Figure 9. Compressive resistance loss as a function of the number of piles

In all cases, the loss in resistance is non-negligible and must be considered when designing the piles and verifying ultimate limit state conditions.

Regarding serviceability, the settlements observed are also significant and further highlight a response that falls outside acceptable performance limits.

6 CONCLUSIONS

Various cone penetration tests, performed at varying distances from CFA piles both before and after their installation, (with varying times after the installation), revealed a significant degradation of the surrounding soil attributable to the pile installation process.

A likely explanation for this degradation is the high rate of penetration (number of revolutions by auger pitch during penetration) observed during the execution of the test piles, although it has not been possible to identify an . This finding highlights the importance of monitoring this parameter during the installation of CFA foundations. Although possible issues associated to CFA piles in homometric sands are mentioned in existing execution codes, it also underlines a current gap in execution and design, as clear guidance is not provided to avoid the phenomenon: moreover, additional examples not presented here (Szymkiewicz et al., 2024) show that even the given criteria on the penetration rate provided in state of the art are not sufficient to prevent significant degradation of mechanical properties under some soil conditions.

The analysis of key indicators contributing to soil disturbance enabled the development of a degradation behaviour law. This model is essential for assessing pile performance in terms of bearing capacity, vertical settlement, and lateral displacement.

Application of this degradation law through various numerical models revealed a reduction in compressive bearing capacity of up to 27% when compared to an undisturbed soil profile. This substantial loss of resistance demonstrates the need to revise traditional calculation methods, which typically assume horizontal homogeneity in the soil—a simplification that is not representative of actual field conditions and is not addressed in design codes, such as the French NF P94-262 (AFNOR, 2013).

Furthermore, serviceability performance is also negatively impacted, with settlements exceeding acceptable thresholds. A similar phenomenon was observed under lateral loading, where failure is governed by bending.

This study therefore demonstrates the necessity to improve the reliability of the criteria to prevent or at least ensure limited soil degradation during CFA piles execution. In some cases, execution can explain some significant disorders still encountered in these sensible soils. From a general point of view, the resistances originally assumed—based on an idealised undisturbed profile—do not reflect the actual ground conditions following execution.

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