Proceedings of the XVIII ECSMGE 2024
GEOTECHNICAL ENGINEERING CHALLENGES
TO MEET CURRENT AND EMERGING NEEDS OF SOCIETY

ISBN 978-1-032-54816-6
DOI 10.1201/9781003431749-148

Behaviour of continuous flight auger piles in homometric and carbonated sands

Comportement des pieux à la tarière creuse dans les sables homométriques et coquilliers

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ABSTRACT: Continuous flight auger piles allow for a quick on-site installation, short execution time and a bearing capacity usually much higher than that of other bored piles. For these reasons, they are the most common piles in France, whether the encountered soils be fine or granular. However, homometric and carbonated sands are identified as 'problematic' for these kinds of piles, either in terms of execution or with regard to the bearing capacity, without the reasons being clearly stated in the scientific literature and technical documents. This communication presents the results of static loading tests carried out on three sites and on seven CFA piles in order to better understand the behaviour of these piles in this type of sands and to optimize their design. The geological and geotechnical context is first described in details. The implemented instrumentation and the obtained results are then presented. An analysis is then carried out, in the light of mechanical tests and further identification of the sandy soil, making it possible to understand the atypical behaviour of these piles in such soils, as well as to propose amended procedures for the design of continuous flight auger piles in these sands.

RÉSUMÉ: Les pieux réalisés à la tarière continue permettent une installation rapide sur site, un temps d'exécution court et offrent une capacité portante généralement bien supérieure à celle des autres pieux forés. Pour ces raisons, ce sont les pieux les plus couramment réalisés en France, que les sols rencontrés soient fins ou granulaires. Cependant, les sables homométriques et carbonatés sont identifiés comme 'problématiques' pour ce type de pieux, que ce soit en termes d'exécution ou de portance, sans que les raisons soient clairement énoncées dans la littérature scientifique et les documents techniques. Cette communication présente les essais de chargement réalisés sur trois sites et sur sept pieux CFA afin de mieux comprendre le comportement de ces pieux dans ce type de sables et d'optimiser leur conception. Le contexte géologique et géotechnique est d'abord décrit en détail. L'instrumentation mise en place et les résultats obtenus sont ensuite présentés. Une analyse est ensuite menée, à la lumière d'essais mécaniques et d'une identification plus poussée du sol sableux, permettant de comprendre le comportement atypique de ces pieux dans de tels sols, ainsi que de proposer de nouvelles valeurs à retenir pour le dimensionnement des pieux à tarière creuse dans ces sables.

Keywords: Deep foundations; carbonated sands; static load tests; design.

1 STUDY BACKGROUND

Continuous flight auger (CFA) and displacement piles are the most common types of piles used in Northern France. They offer a number of advantages: high production rates and expected axial resistance, and the equipment required for their realisation is limited compared with other techniques.

For research purposes, seven static loading tests were carried out on these types of piles at three sites between the cities of Calais and Dunkirk, on the French North See Coast. The piles are all founded in the quaternary sands of the Middle Flandrian, which are fine, homometric and shell-bearing.

The piles were all tested in compression, with the exception of one in tension. Loading tests were carried out until the piles reached geotechnical failure.

The strain instrumentation used on the piles varied from sites to sites, ranging from LCPC removable extensometers to fiber optics, vibrating string extensometers and strain gauges.

For all seven piles, the results obtained showed a systematically lower bearing capacity than that calculated according to current French rules.

This communication presents in detail these tests, the geotechnical context of the sites and the results obtained, and proposes an attempt to explain the low bearing capacities observed. New values of parameters k_p and $\alpha_{pieu\text{-}sol}$, defined as the pressuremeter bearing capacity and used to calculate the base resistance, and as a parameter depending on both the soil types and the pile technique and used to determine unit shaft friction, respectively, are proposed for CFA piles in these sands, which are known to be a potential source of problems both during the execution phase and with regard to bearing capacity.

2 TEST SITES AND TEST PILES SPECIFICATIONS

2.1 Location and geological/geotechnical context

The loading tests were carried out in three test plots carried out between 2011 and 2020.

Geotechnical investigations were carried out at the test sites, using core sampling and pressuremeter drilling.

The geotechnical cross-sections obtained from the various boreholes are presented in Table 1.

Table 1. Geotechnical models at Calais (a), Coudekerque (b) and Loon (c).

Depth of the layer (m)	**Nature**
0	Fills
1,4	Clays / Sands alternations
2,7	Coarse sand
3,7	Flandrian sand

(a)

Depth of the layer (m)	**Nature**
0	Silts
3	Flandrian fine sand

(b)

Depth of the layer (m)	**Nature**
0	Fills
0,6	Clays
2,2	Flandrian fine sand

(c)

The particle size analyses carried out on the samples recovered from the sands show them to be fine and very uniform with a coefficient of uniformity (D_{60}/D_{10}) less than 1.5.

The pressuremeter tests carried out at the three sites are shown in Figure 1. The p_{lM}* Ménard pressuremeter net limit pressure values are very comparable from one site to the next, with values increasing with depth in the homometric sand layers.

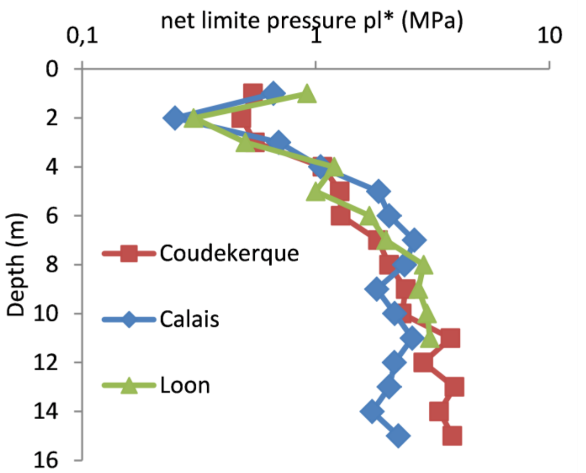

Figure 1. Evolution of net limit pressure as a function of depth at the three sites.

Moreover, the sands encountered are carbonated sands, being largely made up of pieces of shellfish shells, the presence of which was confirmed by the observation of a clear effervescence when drops of hydrochloric acid were applied, reflecting the presence of calcium carbonate. A more precise analysis allowed to quantify the carbonate content of these sands, with values up to 10%.

Uniform sands have been identified abroad as problematic during the execution phase of hollow auger or cast screw piles, but also with regard to bearing capacity (Brown et al., 2007 and Van Weele, 1988).

With regard to carbonated (shell) sands, Spagnoli et al. (2015) and Agemeer et al. (1973) pointed out that the constituent grains of these soils are fragile and often hollow, and can easily break during shearing, resulting in a reduction in volume and therefore limited friction (which can sometimes be as little as 10% of that expected for conventional sand). Noorany (1985) found that this limited friction is not due to a low pile-soil friction angle, but rather to the low horizontal stresses prevailing at the soil-pile interface.

However, current European and French rules for the execution and justification of deep foundations devote only one line to these soils, saying only that they "require special analysis due to their specific behaviour, which can result in very low resistances" (AFNOR, 2012 and AFNOR, 2015).

2.2 Test Piles

The seven piles tested on the three sites are hollow auger bored piles corresponding, according to standard NF P94-262, to category 6 piles (FTC).

Table 2 gives the details of the various piles at the Calais, Coudekerque and Loon sites. All the piles were instrumented, with the exception of the pile tested in tension at Loon. This instrumentation, irrespective of the measurement technology used, enables the shaft resistance R_s of the compression-tested piles to be dissociated from the base resistance R_b, and to estimate the axial unit shaft friction that can be mobilized in the various horizons crossed (Szymkiewicz et al., 2021).

Analysis of the drilling parameters for the seven piles also confirmed that the work was carried out without any particular problems.

Table 2. Geometric characteristics, types and strengths of piles tested in accordance with French design method.

	Coudekerque			Calais		Loon	
	P1	**P2**	**P3**	**P1**	**P2**	**P1**	**P2**
Length (m)	12	12	12	10	10	8	10,5
Diameter (m)	0.52	0.52	0.52	0.52	0.52	0.42	0.42
Type	FTC	FTC	FTC	FTC	FTC	FTC	FTC
Instrumented	yes	yes	yes	yes	yes	yes	no
Test			compression				traction
Rc (kN)	3297	3297	3297	2438	2438	1588	
Rs (kN)	2088	2088	2088	1685	1685	959	1685
Rb (kN)	1209	1209	1209	753	753	629	

3 LOADING TEST RESULTS AND ANALYSIS

The loading tests were carried out in compliance with the requirements of the test standards in force at the time.

3.1 Resistance of the piles

It is important to note first of all that the P2 and P3 piles at the Coudekerque site were subjected to heating-cooling cycles for 4 months and it has been shown that these cycles had an impact on the shaft resistance of these piles (Szymkiewicz et al., 2015). Therefore, for this study, only the base resistance was compared to the theoretical resistance.

The tests were all carried out at near failure, taking as the failure criterion a displacement of the pile head equal to B/10. At these displacement levels, the base and shaft resistances were almost completely mobilized (Figure 2). It should be noted that, for piles tested in compression, the normalized load-displacement curves are quite similar.

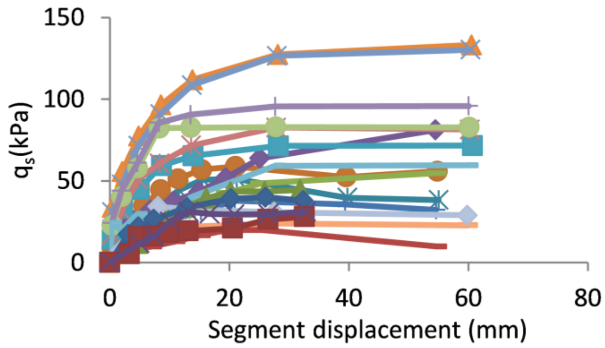

Figure 2. t-z curves in sands, for all piles.

In general, the measured tensile and compressive strengths ($R_{t/c\ ;\ m}$) are well below the expected theoretical strengths ($R_{t/c\ ;\ cal}$, estimated in accordance with NF P94-262). Measured ultimate strengths are at best 74% of theoretical ultimate strengths (Table 3), for the P1 pile tested at the Coudekerque site, while for the other three the difference is even greater, with an average ratio of 64%.

The shaft resistances of the instrumented piles tested in compression and of the one tested in tension are also well below the expected resistances. In the case of the Coudekerque P1 pile, measured resistances represent at best 84% of the theoretical resistance, and only 61% on average, with a minimum of 54% (Table 3).

The base resistances of the instrumented piles tested in compression are below the expected strengths, too. Measured resistances represent at best 92% of the theoretical resistance (extrapolating the peak resistance for a limit resistance of 1100 kN and considering the shaft friction fully saturated during the last maintained bearing) for the P1 pile in Loon, and only 70% on average, with a minimum of 58% (Table 3).

It is interesting to note that at all three sites, shaft resistance is on average the most overestimated, but at the Coudekerque site, the opposite is observed.

Table 3. Theoretical (cal) and measured (m) limit (c/t), shaft (s) and base (b) resistances, and ratios.

	$R_{s;\,cal}$	$R_{s;\,m}$	$R_{s;\,m}$ / $R_{s;\,cal}$	$R_{b;\,cal}$	$R_{b;\,m}$	$R_{b;\,m}$ / $R_{b;\,cal}$	$R_{c\,cal}$	$R_{c\,m}$	$R_{c;\,m}$ / $R_{c;\,cal}$
Calais P1	1685	898	0,53	753	577	0,77	2438	1475	0,61
Calais P2	1685	925	0,55	753	550	0,73	2438	1475	0,61
CDK P1	2088	1751	0,84	1209	699	0,58	3297	2450	0,74
CDK P2	-	-	-	1209	726	0,6	-	-	-
CDK P3	-	-	-	1209	752	0,62	-	-	-
Loon P1	959	521	0,54	629	579	0,92	1588	1100*	0,69
Loon P2	1334	820	0,61	-	-	-	-	-	-

* CDK = Coudekerque

3.2 Axial unit shaft friction mobilization friction mobilization and values

Analysis of the t-z transfer curves achieved via the instrumentation reveals that the curves all have virtually the same shape, with no softening (Figure 2). This shape is characteristic of a rather loose sand, by analogy with a shear test on sand.

Generally speaking, we note that the unit shaft friction q_s measured are well below that expected according to standard NF P 94-262 (Figure 3).

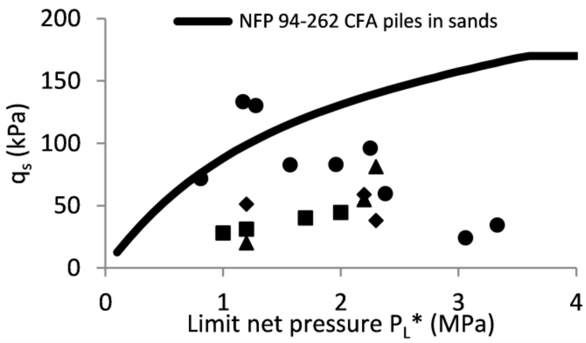

Figure 3. Measured q_s against net limit pressure, and comparison with the French theoretical values.

It is also interesting to note that the friction measured in the top layers is on average higher than expected and therefore 'compensates' in part for the low friction in the sands.

4 RESULTS EXPLANATIONS

The results achieved in these seven tests assess the fact that the bearing capacity of CFA piles in this type of soil is much lower than expected.

As stated previously, there is already some tentative explanations for these low values. However, another alternative explanation could be related to the homometric properties of the sands.

Indeed, homometric granular soils are known to exhibit a dilatant behavior during the shearing process. This shearing process occurs naturally during the pile loading, at the interface soil-pile, but also already during the pile execution process, when the auger penetrates the soil. Therefore, it may be interesting to look at the residual effective friction angle and to compare it to the peak friction angle.

To evaluate the possible decrease of the friction angle, it was decided to perform shearbox tests as well as alternate shearbox tests.

Peak friction angle was determined to be equal to 41°, while the residual friction angle was derived as 34°. This induces a decrease of the friction of about 22 %, which could explain the low values of resistance achieved during these tests.

However, it is difficult to conclude, as the shape of the t-z curves, with no softening, is more clearly linked to a contractant behavior.

Furthermore, the sands of this study show a fairly high voids ratio, consistently above 0.6. Using the e_{max} and e_{min} values available in the literature, it is possible to estimate the soil relative density, in order to better classify these sands. They are fairly loose to very loose, except at the Loon site, where the sand is moderately dense.

This contradicts our proposed explanation, and implies that the study of this topic should be continued.

5 NEW DESIGN PARAMETERS FOR CFA PILES IN HOMOMETRIC CARBONATED FINE SANDS

Even if the reasons for the low values are still not clear, it is possible to propose new values for design parameters, in order to increase the safety when planning for CFA in these sands.

First, regarding the determination base resistance, determined following the French method according to Eq. (1), the pressuremeter bearing capacity k_p factor shall be decreased from 1.65 as stated in the French standard to 1.1:

$$R_b = A \times k_p \times p_{le}^* \quad (1)$$

where A is the base area, and p_{le}^* is the equivalent net limit pressure around the base of the pile.

Regarding the shaft resistance, in order to limit the design values of q_s, determined following the French method according to Eq. (2), it is recommended to decrease the value of the parameter $\alpha_{pieu\text{-}sol}$, that depends on both the soil types and on the pile categories, and to leave unaffected the f_{sol} function that is defined by parameters a, b and c dependant only on the soil types (Eq. (3)). The proposed value is 0.7, compared to the 1.8 given by the French standard:

$$q_s = a_{pieu-sol} \times f_{sol} \quad (2)$$

with

$$f_{sol} = (a \times p_{lM} + b) \times (1 - e^{-c \times p_{lM}^*}) \quad (3)$$

It also necessary to limit the maximum value of the unit shaft friction q_{smax} to a much lower value of 70 kPa, compared to the 170 kPa allowed at the moment. The new curve of mobilizable limit unit shaft friction in these sands is shown in Figure 4.

It is now possible to compare the measured data of these tests with the new theoritical values: this is what is done in Table 6.

It can be seen that, on average, the measured friction resistance is now 25% higher than the friction resistance with this new $\alpha_{pile\text{-}soil}$ value. The case of P1 de Coudekerque remains problematic, as the friction resistance is largely underestimated.

The base resistance measured remains on average 5% higher than that measured with this k_p value. However, this value is biased by the results obtained on the P1 pile in Loon. If this pile is discarded, the ratio $R_{b;\,m} / R_{b;\,cal}$ drops to 0.99.

Table 6. Ratios between measured and calculated resistances, with the new proposed values of $\alpha_{pile\text{-}soil}$ and k_p.

	$R_{s;\,m} / R_{s;\,cal}$	$R_{b;\,m} / R_{b;\,cal}$	$R_{c/t;\,m} / R_{c/t;\,cal}$	$R_{c/t;\,cr;\,m} / R_{c/t;\,cr;\,cal}$
Calais P1	1,10	1,15	1,12	1,20
Calais P2	1,14	1,10	1,12	1,12
Coudekerque P1	1,79	0,87	1,37	1,88
Coudekerque P2	-	0,90		-
Coudekerque P3	-	0,93		-
Loon P1	1,02	1,38	1,18	1,27
Loon P2	1,22	-	1,22	1,51

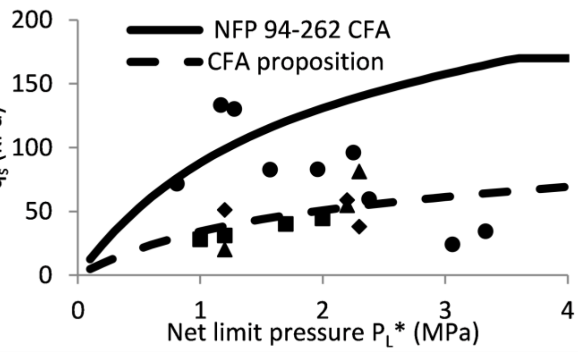

Figure 4. New proposition for a q_s – P_l curve for CFA piles in homometric carbonated sands.*

On the other hand, if we look at the limit resistances, we see that the ratio $R_{c/t;\,m} / R_{c/t;\,cal}$ averages 1.20. It is therefore possible to adjust the proposed values while maintaining a certain degree of safety.

6 CONCLUSIONS

Various Tests carried out on CFA piles have shown that the limit resistance of these piles are systematically overestimated in homometric carbonated sands.

A possible explanation for the low value of shaft friction and base resistance could be the dilatant behaviour of the soil associated to the pile realisation process. However, further tests are needed to assess the validity of this hypothesis.

Nevertheless, the results achieved allowed for the proposition of new design values for the $\alpha_{pile\text{-}soil}$ and k_p parameters, ensuring a safer design of CFA in these very special sands.

It is also necessary to propose lower values for the $\alpha_{pile\text{-}soil}$ and k_c parameters, as well as the q_{smax}, allowed for the CPT method, insofar as testing is possible in these soils.

Further loading tests in these soils are still necessary, not only to gain a better understanding of the behaviour of these piles, but also to analyze these data according to the principles used to draw up standard NF P94-262, so as to confirm the proposals presented in this report. A similar study on other types of piles should also be carried out, as all categories of piles are impacted by these very particular soils: two cast screw piles tested on the Calais site gave similarly poor results.

Finally, a further laboratory study focusing solely on the impact of sand particle friability could be carried out, with additional tests on homometric quartz sands.

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