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SOUTH-EAST VIADUCT OF THE NEW TOULOUSE METRO LINE: CFA PILES BEHAVIOR IN THE TOULOUSE MOLASSE

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ABSTRACT: In the context of the construction of the viaducts of the new Toulouse metro line, it was decided to follow an experimental approach to verify the feasibility and to optimize the design of CFA piles in a geological and geotechnical context still not so well apprehended. Indeed, the Toulouse molasse, a very hardened soil whose behavior is not perfectly known and which is locally considered as a marl even if laboratory tests identify it as mainly silt, is a complex geological horizon, consisting mainly of alternating silty to sandy silt passages, and whose state can vary from very altered to sound, depending on the depth and location. CFA piles, on which only one load test was carried out in this context, are known to show many advantages but also have limitations, which could hinder the schedule of the project if this technique proved to be not adapted to the geological context. This publication states the problematic of the project, and summarizes the geological and geotechnical context of the test plot. Then, the instrumentation (using fiber optics and vibrating wire strain gauges) and the results obtained are presented. In particular, it was shown that the q_s achieved during the tests are much higher than the maximum value allowed by the French standard, but that the base resistance is on the contrary smaller than expected. The analysis of these results made it possible to propose in the conclusions optimized values to retain for the design of the future foundations of the project.

Keywords: CFA piles, Static loading tests, Design, Instrumentation

1. INTRODUCTION

The third metro line in the Toulouse conurbation will link Colomiers to Labège over a distance of 27 km, via 21 stations. At its southeast end, the project includes a viaduct some 5 km long. At the same time, a project is planned to connect line B to the third line at the future INP station, extending the current line from Ramonville over a distance of 2.7 km. In all, 7.7 km of viaduct will be built on 190 supports lying on more than 700 piles executed in compact, sandy silts locally called 'Toulouse molasse', with the heart of the city of Toulouse crossed by tunnel.

For reasons of cost and speed, continuous flight auger piles (CFA) were selected as the preferred solution, as it is well known that these piles allows for a faster drilling process, compared to more traditional piles, and also avoid the use of slurry or casing for temporary support [1]. These piles may also allow for higher unit shaft friction compared to traditional bored piles, in most types of soils [1-5]. However, some limitations occur, such as the need for a higher torque (and therefore bigger drilling machine) and the difficulties to go through thick layers of very stiff soils or when large boulders are encountered. Furthermore, data remain scarcely available in very stiff soils [6], as these are particularly difficult to investigate and pile loading tests necessitate significant ressources. For example, in the particular geotechnical context of this project, only one instrumented CFA pile was ever tested [7], making it difficult to generalize the results of this particular test. The unique pile test by [7] exhibited high unit shaft friction friction values, averaging almost 300 kPa, but at the same time exhibited a base resistance which was deemed fairly poor, compared to the expected value. Others piles of comparabable execution process were tested in other locations, such as Dublin [2-3]: very high unit shaft friction values were achieved in each case, with values on average ranging from 200 kPa to 300 kPa according to [2] and from 150 kPa to at least 300 kPa according to [3]. However, as these tests piles did not reach failure, there remains a high level of uncertainties, which, combined with the differences in terms of practice regarding the soil investigations methods between a vast majority of countries and France, render the design of CFA piles in stiff sandy clays, clays or silts for such projects certainly overconservative [8-9].

It was thus chosen to build an experimental plot comprising four piles, in order to verify the feasibility of large-diameter CFA piles in the geotechnical context (presented below), and to optimize the design of the structure's piles, thanks in particular to measurements of limit pressure q_b and limit unit shaft friction q_s that can be mobilized in the geological horizons encountered by the deep foundations of the viaduct's supports, coupled with a complete geotechnical investigations and a robust geotechnical model, as well as to refine the determination of the creep load of the piles, once again with a view to optimizing the design of the foundations.

This paper presents in details the geotechnical context of the project, as well as the implemented instrumentation (such as fiber optics and vibrating wire strain gauges) and followed testing protocols used during the tests. The results and discussion are then presented below, showing how impactful this well planned experimental study was for the project.

2. RESEARCH SIGNIFICANCE

This study will allow for the optimization of the behavior of piles in stiff clayey sandy soils, leading to the improvement of their design, in the context of the global process of optimizing civil engineering design in order to achieve substantial savings in terms of energy and natural resources.

Indeed, piles are the most common type of foundations encountered as they typically allow for a better overall safety level and for limited settlements. However, they are costly, and most of the design codes are based on a few tests [10], meaning that there should be still room for optimization.

3. METHODOLOGY

In this section, the followed methodology in order to reach the purpose of the study is described in details.

3.1 Location and Geological/Geotechnical Context

The loading tests were carried out at the southern end of the future viaduct, on land (a parking lot) on the territory of the city of Labège and made available to the project by the local authority.

From a geological point of view, the site lies within the Aquitaine Basin. This vast triangular basin is bounded by the Bay of Biscay and the Atlantic Ocean to the west, and the Pyrenees to the south. It is made up of Tertiary sedimentary fill resulting from the erosion of the first Pyrenean reliefs, formed from the Eocene onwards, when the European and Iberian plates met, until the beginning of the Oligocene.

Towards the end of the Miocene, the Pyrenean landforms were flattened and only vertical subsidence movements occurred in the basin. A rapid uplift movement finally began in the Pliocene and continues to this day.

The deposits that characterize the Toulouse region and form the current general bedrock are the regional molasses of Stampian to Aquitanian age (terminal Oligocene to base of Miocene). This term designates the finite to post-orogenic formations that resulted from the formation of the Pyrenees and the erosion of these first reliefs.

This formation, mainly detrital in nature, is deposited at the foot of the reliefs and extends into vast foreland subsidence basins. Its thickness at Toulouse can reach up to 200 m. The Garonne and its various secondary rivers then cut into this bedrock and deposited sedimentary sequences, creating the context we know today.

The test site, like the entire length of the two viaducts of the Line B and Line C extensions, lies in the narrow valley of the Hers, a secondary river of the Garonne. This valley is bounded by the molassic slopes of Pech-David and Pechbusque to the west and Saint-Orens to the east.

The project cuts across several tiered alluvial terraces: modern alluvium of the low plains of the Hers (flood silts with a clayey tendency), alluvium of the low and medium terraces of the Hers (more sandy in nature and poor in gravel).

These alluvial deposits of the Hers are the result of erosion of the underlying molasse formations, and are not of Pyrenean origin, unlike the alluvial deposits of the Garonne.

To determine in details the geotechnical model, the test site was the subject of a geotechnical investigation campaign involving core drilling, pressuremeter tests and static penetration tests.

From the surface, the first soils encountered are alluvial overburden considered to be fine soils, with an average thickness of 1.85 m, underlain by the local molasse bedrock. This molasse is weathered down to a depth of 8 m, then sound below to a depth of at least 25 m.

Molasse is a detritic sedimentary formation of fluvial continental origin, as stated before, composed of compact, sandy silts with low clay content (Fig. 1) and varying degrees of calcareous contents (Fig. 2) as defined in [11], with sandy channels running through them, corresponding to the sedimentation process of river beds and arms filling.

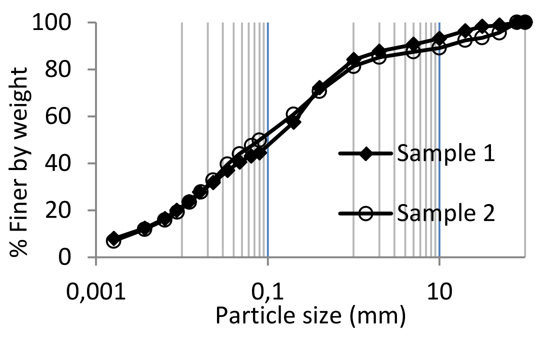


Fig. 1 Grain size distribution of the molasses

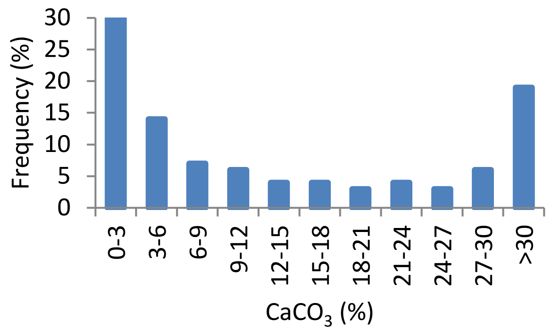


Fig. 2 $CaCO_3$ contents distribution among 379 specimens of molasses.

However, depending on the projects and on the engineers, this heterogeneous formation is classified as clay, marl or sometimes even rock.

Laboratory tests carried out on molasses as part of the 3rd line studies have shown that these soils are sandy silts with a low clay fraction (Fig. 1), and generally low carbonate contents, even if locally CaCo3 contents can be higher than 30% (Fig. 2). Indeed, on the tests site, the average $CaCO_3$ content was found to be 3.3%, well below the 30% threshold to allow for a marl classification.

These soils have a very hard consistency as defined in Table 4 of [12], and are considered very stiff according to [13].

Three pressumeter soundings were carried out on the tests site, with pressuremeter tests performed at each location every meter and down to a depth of 14 meters. Results achieved throughout these tests were in good agreement with the assumptions made above.

The geotechnical model finally adopted is shown in Fig. 3 and summarized in Table 1.

Table 1 Geotechnical assumptions

Depth (m)	Soil Nature	Classification (NF P 94-262)	Limit Pressure Pl* (MPa)
0-1.85	Overlying fill	Silt	0,8
1.85-8	Altered Molasse	Silt	4,7
8-15	Sound Molasse	Silt	6,7

3.2 Test Piles Specifications

The four test piles were continuous flight auger piles manufactured in accordance with [14] and corresponding, following [13], to the 6th category of piles.

The diameter B of these piles is 0.72 m, and their length D is 11.3 m. The reaction piles are also CFA piles, 1 m in diameter and 20 m long. Analysis of the drilling and concreting parameter records showed that there were no significant differences between the four piles in terms of execution.

Table 2 shows the values for axial unit frictions and base factors (pressuremeter method) for calculating the bearing capacity of the piles tested, in accordance with [13] and calculated considering the molasse as silt, as mentioned in the previous paragraph. The estimated bearing capacity of the tests piles according to this method and to the soil investigation is equal to 5.6 MN.

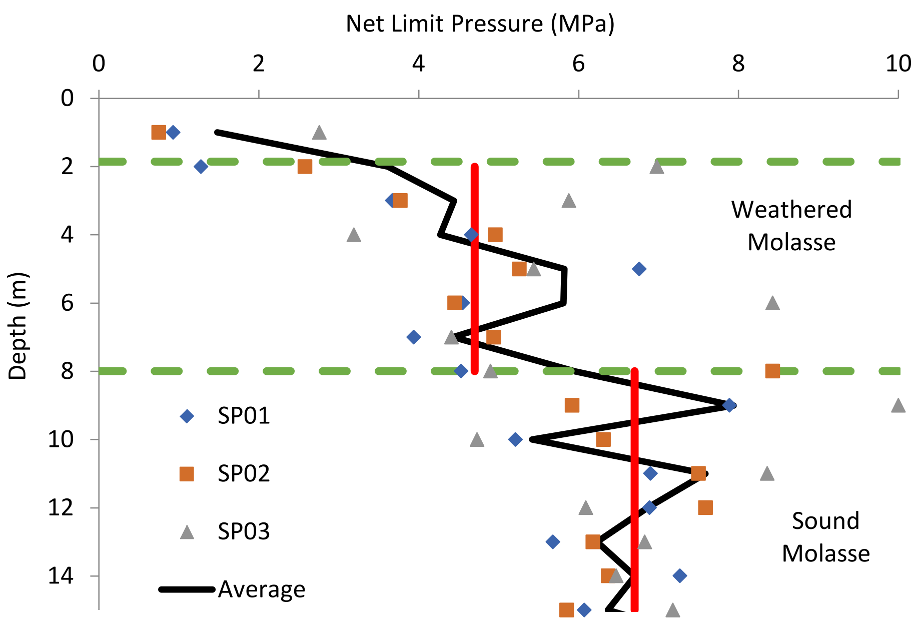


Fig. 3 Geotechnical model selected for the site

Table 2 Ultimate bearing capacity calculation assumptions for the four piles tested, in accordance with [3], and calculated strengths

Nature of soil	z (m)	Pl* (MPa)	q_{smax} (kPa)	R_s (kN) / layer	R_s (MN)	k_p	R_b (MN)	R_c (MN)	$R_{c;cr}$ (MN)
Overlying fill	0-1.85	0.8	60	251.08					
Altered Molasse	1.85-8	4.7	81	1126.79	2.05			5.60	3.21
Sound Molasse	8-11.3	6.7	90	671.80		1.30	3.55		

3.3 Load Application and Test Piles Instrumentation

The force is applied to the pile head by means of a jack with a capacity of 15 MN and a stroke of 25 cm. A spherical seating was placed at the top of the jack.

The instrumentation used is identical for each pile:

- four displacement transducers positioned at 90 degrees at the pile head level and resting on fixed supports, to monitor the displacement of the pile head during testing,
- 27 vibrating wire strain gauges distributed in triplets over nine levels, with the three transducers on each level arranged at 120°,
- two fiber optic loops installed on the reinforcing cages,
- a 15 MN load cell used to monitor the load applied to the head throughout the test.

4. RESULTS AND DISCUSSIONS

Loading tests were carried out in compliance with the requirements of current test standard [15].

4.1 Piles Bearing Capacity

The pile head load - displacement curves are fairly similar for the 'elastic' part (Fig. 4). On the other hand, the behavior of pile E1 differs greatly from the others for applied loads equal to or greater than 4000 kN.

The bearing capacity of piles E1, E2 and E4 are fairly similar, ranging from 8 to 9 MN, assuming a pile head displacement equal to B/10 as the failure criterion. Pile E3 has a much higher bearing capacity than the other three, but this could not be explained, despite an additional pressuremeter borehole and core hole having been performed after the tests.

4.2 Piles Creep Load

Analysis of creep speeds (calculated between 5 and 60 minutes, then extrapolated over an hour), creep coefficient values α and the difference in settlement between 60 and 10 minutes (criterion used by [7] in a test on a similar pile in the Toulouse Molasse) enables us to estimate the creep load of each of the piles, with the exception of pile E1, for which the evolution of the various parameters and coefficients used to determine it is too chaotic (Fig. 5).

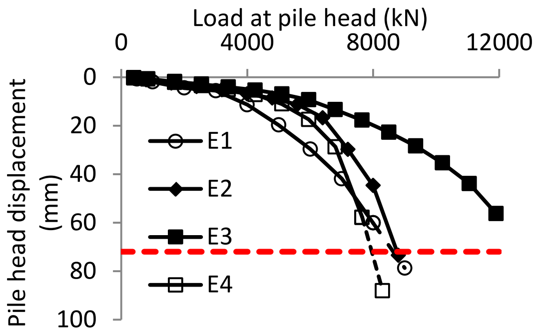


Fig. 4 Pile head load - displacement curves for the four piles

For piles E2, E3 and E4, however, it is possible to estimate a creep load of around 5.5 MN for piles E2 and E4, and in excess of 6.5 MN for pile E3.

At these load levels, the mobilized shaft resistance (see below) is equal to around 70% to 80% of the maximum shaft resistance measured, which is in line with French design practice and with observations made during numerous tests for this type of pile [16].

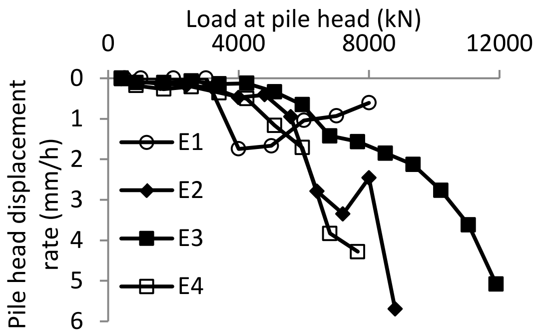


Fig. 5 Pile head displacement rate for the four piles

4.3 Base and Shaft Resistances and Unit Shaft Frictions

Analysis of the strain measurements was carried out in two stages: firstly, the measurements from the vibrating wire strain gauges were compared with those obtained with the fiber optic. The deformation measurements were comparable, as logically expected [17, 18]: the rest of the interpretation was therefore carried out with the vibrating wire strain gauges.

The analysis of measured strains was carried out with a modulus that varied according to depth, loading stage and therefore strain rate [19]. Fig. 6a shows that the base resistances measured are fairly comparable for piles E1, E2 and E4. Furthermore, it is clear that these resistances are almost completely mobilized during the last load step for all four piles.

The shaft resistances of piles E1, E2 and E4 are also very similar (Fig. 6b). It can also be seen that the shaft resistances are almost completely mobilized during the last load step for piles E2 and E4, while for piles E1 and E3, R_s does not appear to be completely mobilized

Fig. 7 shows the t-z curves obtained by interpreting the strain and displacement measurements made, for test piles E1 and E4. Results of piles E2 and E3 are quite similar. As shown in Fig. 5, almost all unit shaft frictions are saturated or decreasing at the end of the tests. The values measured are very high in molasse, well above 200 kPa on average, whereas [13] limits $q_{s,max}$ to 90 kPa in these soils.

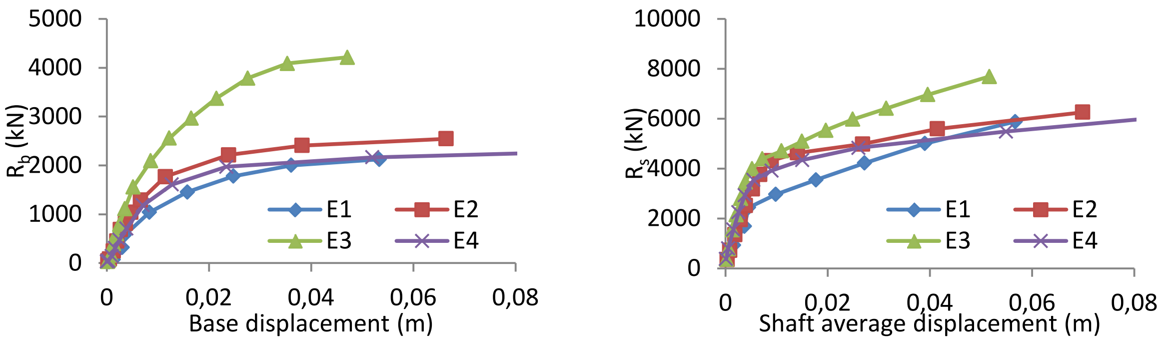
4.4 Discussions

All four piles were loaded to failure or near failure, both in terms of the 'displacement to failure criterion' and in terms of geotechnical resistance, as shown in Figs. 4, 6a and 6b. The four tests carried out showed that the bearing capacity of CFA piles in this molasse is well above that expected on the basis of [13] (Fig. 4 and Table 2).

Fig. 8 shows that for piles E1, E2 and E4, the distribution of forces between the base and the shaft is almost identical from one pile to another, throughout the tests.

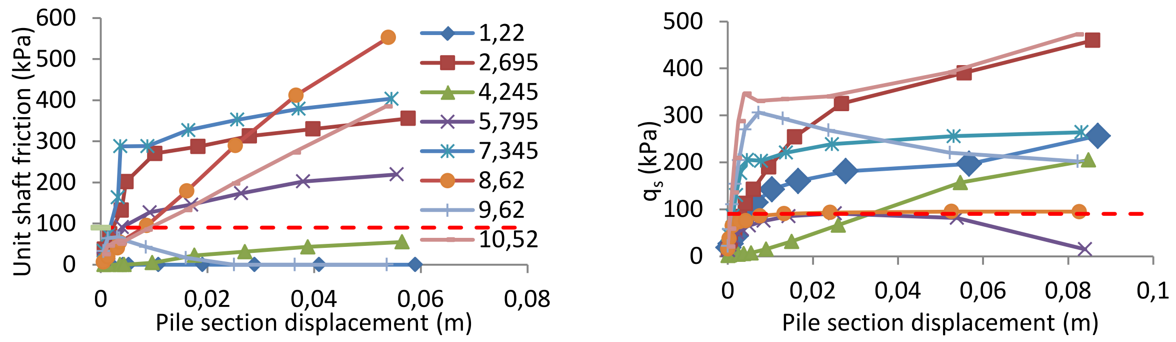
At failure, and for these three piles, load distribution was as follow: an average of 27% of the load was supported by the base while the shaft took up the remaining 73 %. For the E3 pile, the proportion of load taken up by the base is slightly higher, equal to 35%.

By way of comparison, a CFA pile tested in 1986 by [7] in the same molasse showed almost identical results to the E3 pile at the end of the test: 35% of the load was supported by the base and 65% by the shaft. The measured values of the pressuremeter bearing factor k_p, for piles E1, E2 and E4, are 27 % lower on average than those expected if we refer to [13], i.e. $k_p = 1.3$. The measured values are shown in Table 3.



(a) (b)

Fig. 6 Base resistances (a) and shaft resistance (b) mobilization curves for the four piles



(a) (b)

Fig. 7 Unit shaft friction mobilization curves t-z for the piles E1 and E4

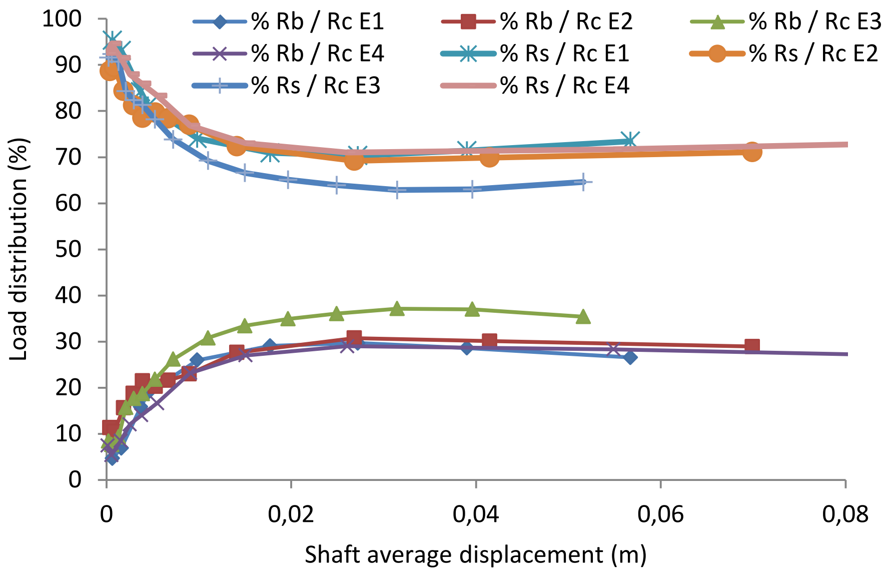


Fig. 8 Distribution of base and shaft resistances

Table 3 Measured resistances and bearing capacity factor, and calculated resistance based on test results and local practice

Measured or calculated resistance and bearing capacity factor	E1	E2	E3	E4	Marl	q_s = 265 kPa and k_p = 0,9
R_c (kN)	8500	8800	> 11900	8000	8612	8410
R_s (kN)	5877	6257	> 7687	6053	4247	5955
R_b(kN)	2123	2543	> 4213	2247	4365	2455
k_p	0.78	0.93	> 1.48	0.82	1,6	0,9
$R_{c;cr;\,meas}$ (kN)	?	5500	> 6500	5500	5155	5396
$R_{c;\,cr;\,cal}$(kN)	5175	5651	>7487	5360		

The test carried out by [7] measured a k_p of 1.2, almost 10% below the standard. It is worth noting that the k_p measured at the end of the test on pile E3 was significantly higher than these values, being equal to 1.47.

Even if pile E3 has a behavior and a bearing capacity different to the others, it is worth looking at the behavior of this pile under a load equal to 8 MN.

Under this load, the behaviors of the four piles are similar, with a distribution of the load along the shaft quite comparable, in particular at the base level (Fig. 9).

Regarding the creep loads measured, they are also much higher than those expected on the basis of [13] (Table 2 and Table 3). It is also interesting to recalculate a 'conventional' creep load from the results obtained in these tests, and compare it with the measured load. This is done in Table 3. It can be seen that the method proposed by the standard for calculating creep load from ultimate strength gives results quite similar to the test results.

The maximum unit shaft friction measured in these tests is very high, as previously mentioned Only three values out of a total of 28 are below the theoretical values of [13]. The average value calculated from these 28 values is 300 kPa.

The measured values are also comparably dispersed from pile to pile (Fig. 10), and are very similar to those measured by [7], as well as the one achieved by [2-3] in a somewhat similar soil.

It is also interesting to compare the results obtained with local practice, which treats molasse as marl when designing deep foundations, with the associated unit shaft friction and bearing capacity factors. In the geotechnical context of our test plot, such a pile would have a theoretical bearing capacity of 8612 kN (Table 3), very close to the 8433 kN obtained on average on the test piles (excluding E3). On the face of it, therefore, local practice is relevant. However, it is clear that this practice greatly overestimates the base resistance (k_p = 1.6 in marls according to [13]). In other geological configurations, the gain in friction may not be sufficient to compensate for this overestimation of base resistance.

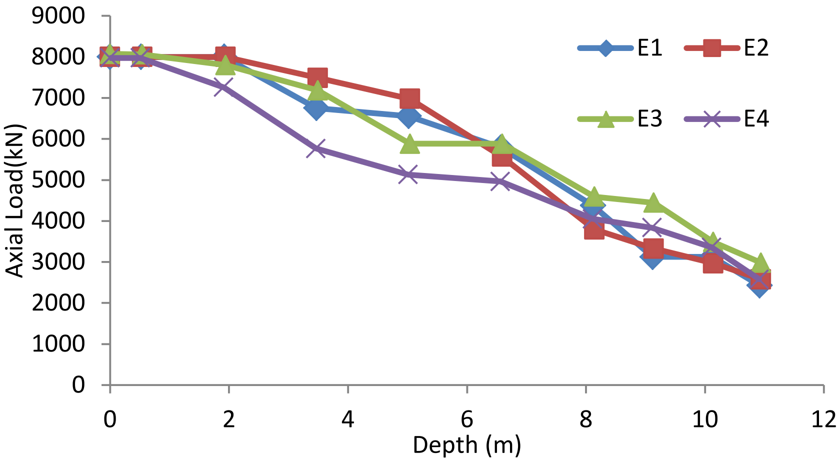


Fig. 9 Axial load distributions along the four piles under a load at pile head of 8000 kN

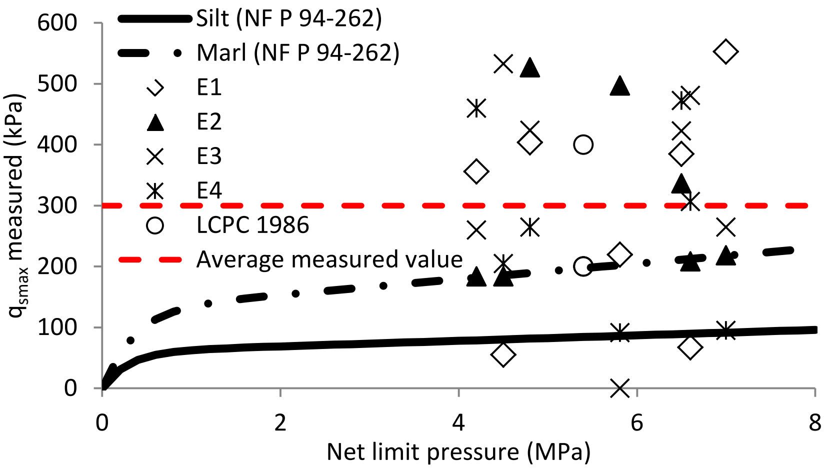


Fig. 10 Maximum unit shaft frictions measured as a function of the corresponding net limit pressure

5. CONCLUSIONS

The completion of this test plot confirmed the choice of using CFA piles: indeed, it was demonstrated that it was possible to realize large-diameter, long-length piles in these very steep molasses, thus making the work schedule and budget more reliable.

The tests also enabled to establish the reliability of the unit shaft friction value to be retained, without the need to differentiate between weathered and sound molasse, as well as that of the pressuremeter bearing capacity factor. The q_s value selected is 265 kPa, well above the 90 kPa specified in the standard. The selected k_p value (0.9) is well below the expected 1.3. These values correspond to 90% of the average value measured on this plot. Combined together, for this project, they enable considerable optimization of the foundations, thus saving time and natural resources.

For general improvement of the French design methods, further tests still need to be carried out in these molasses, as only CFA piles were covered in this study. Furthermore, as molasses are encountered in other regions of France, it is important to reproduce these tests in such regions, as the term molasses is often used as a generic term encompassing many types of soils.

Finally, the high values of unit shaft friction repeatedly measured during these tests, as well as the one achieved during previous tests, will allow for an increase of the maximum allowed value by [13] when this document will be revised. In the same way, these results will also allow for the improvement of the maximum values of unit shaft friction authorized in other countries, making it possible to optimize the design of CFA piles in soils of comparable nature and condition, provided that a thorough soil investigation is carried out.

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