Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering

doi:10.3233/978-1-61499-656-9-2161

A case history on the investigation techniques for large diameter pile foundations

L'historique d'un cas sur les techniques de recherche pour des fondations sur pieux de grand diamètre

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ABSTRACT
The need of verifying the integrity, and thus the efficiency, of large diameter pile foundations has lead, in the last years, to the development of non-destructive investigation techniques, based on the theory of waves propagation through elastic media (Timoshenko and Goodier, 1934), in order to detect both the real length and the possible imperfections due to the construction process. In the following paper the Authors outline some suggestions to best interpret the standard ecometric field data, after an extensive site experimentation performed on a 344 pile-group.

RÉSUMÉ
Dans les dernières années, le besoin de vérifier l'intégrité, et aussi l'efficacité, des fondations sur pieux de grand diamètre a permis le développement des techniques non distructives de recherche, basées sur la théorie de propagation des ondes par les moyens élastiques (Timoshenko and Goodier, 1934), cette-ci qui détecte la vraie longueur et les possibles imperfections dues au procédé de construction. Dans l'article suivant les auteurs fournissent des conseilles pour interpréter dans le mieux possible les données dérivent par des essaies par reflexion (T.N.O.), à la suite d'une investigation en place par un groupe de 344 pieux.

1 INTRODUCTION

The site, object of the investigation performed, is located in the northwestern area of Genoa (Italy), close to the confluence between the torrents Secca and Polcevera; in this former industrial area, covering an area of about 77'000 m^2, the new City Marketplace is being built.

Due to the difficult soil conditions, the designed superstructure will lie on 344 large diameter pile foundations (600 ÷ 800 mm), having an average length of 14.8 m.

2 SITE DESCRIPTION

Examining the site situation, it can be argued that its geomorphologic equilibrium depends more on changes and evolution of the urban-planning than on natural factors. With regard to the natural hazards, being the area in the nearby of a two rivers' confluence, the potential dangers are represented by floodings.

Therefore, the most significative geologic aspects are connected to the stratigraphy and the geotechnical parameters of the subsoil.

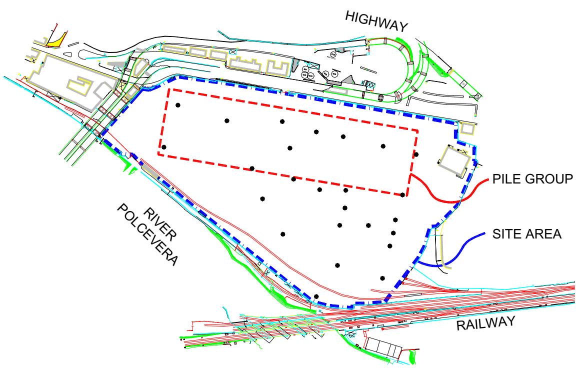


Figure 1. Site plan with the position of the 25 boreholes (dots).

In order to assess the stratigraphic, geotechnical and hydrogeologic conditions of the site, 25 boreholes (see fig. 1) and geophysical measurements have been carried out. Piezometers have been installed in some of the boreholes, in order to measure the phreatic level evolutions.

By the analysis of the soil sampling, the following stratigraphy has been defined; starting from the ground surface, one can find:

- backfills, embankments, various remolded materials, coming from the dismantled stuctures of the former industry;
- alluviums, starting as silt with clayey silt interbeds, deepening into gravelly sands and gravels;
- the bedrock formation (> 25 m) is represented by a sequence of shale and marly-limestone beddings.

Any borehole has shown a thick layer of backfill and/or remolded material; this is constituted mainly of rubble, concrete slabs, bricks and stones, used to level and rise the ground surface in order to protect the buildings from the river's floodings.

This first layer deepens for about 3 ÷ 4 meters from the ground level and rests on the alluvial soil, which has a thickness spanning from 20 to 25 meters.

All the in-situ tests performed have allowed to define the volumetric and spatial distribution of the main silty layers: a significative layer, between 1 and 6 meters thick, has been found at a bottom depth of 10 ÷ 12 meters. The bedrock was sounded only in some part of the area, at depths varying between 24 and 30 m from GL.

The phreatic level is assessed at 5 ÷ 6 meters from GL, and it is consistent in the overall area.

3 SOIL LAYERING

In order to correlate the test results with the stratigraphy, a three-dimensional model of the interfaces between the different soil layers has been carried on. Thus, it has been possible to evaluate the depth of the intersection between any pile and its surrounding soil layers.

By the analysis of the samples taken in the boreholes, two interfaces, one between backfills and sandy silt and one between sandy silt and gravel, have been defined.

The soil, where the piles are cast in, is thus divided in three layers: the first unit being represented by backfills, the second sandy silts and the third gravels.

Knowing the 3D coordinates of every pile top and the interfaces' depth of the three soil units at the boreholes locations, by using a GIS spatial analysis software, the three coordinates of the two interfaces are collected in a continuous grid, then interpolated by a TIN (*Triangular Irregular Network*) technique in order to reckon two three-dimensional surfaces.

This operation allows to interpolate over the grid the depth of the soil layers' interfaces (see Fig. 2).

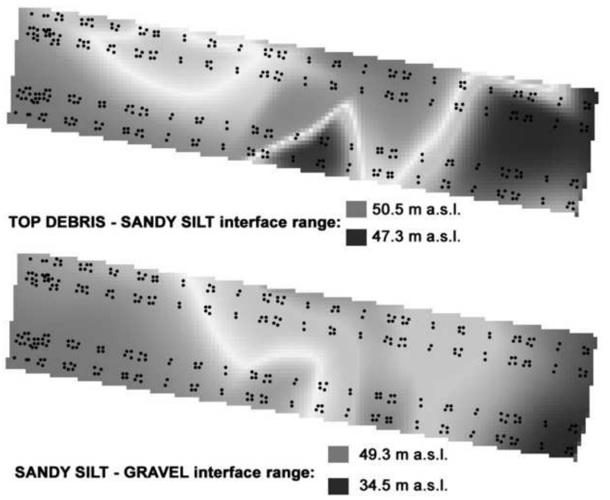


Figure 2. The interface interpolations for the three layers', within the pile group.

These surfaces have been used to set a three-dimensional model of the area, by which the depth of the intersection between every pile and the layers' interfaces can be evaluated.

This procedure allowed to extrapolate the "best fit" stratigraphy along everyone of the 344 pile axes.

4 PILE FOUNDATIONS

A first set of the cast-in-place pile foundations had been bucket-drilled with a temporary steel casing; all the other ones have been built by a special CFA system, named *Trelicon®*.

The former technique consists of the installation of a driven steel casing, the excavation of the soil by a short (1.3 m) rotating tool (bucket), the placement of the reinforcing steel bars and grouting with a bottom tremie.

In the latter system, the soil is drilled by a special hollow continuous-flight auger, closed at the bottom by a plug, to the required depth; the auger is then withdrawn, with or without rotation, while the grout is pumped inside the hole from the bottom of the auger. The reinforcing steel is immediately pushed down into the fluid cement; thus temporary casing is not needed.

This technique can also achieve the compaction of the surrounding soil during the excavation, improving its characteristics.

5 NON-DESTRUCTIVE TESTING

Dynamic integrity tests on pile foundations, conceived at the end of the sixties, are quite commonly used all over the world, due to the fact that they are non-destructive, quick and always more reliable tests (see e.g. Caputo, 1994).

The most important element that characterizes the different dynamic tests is the presence of inertial forces, whereas the better distinction in terms of their final aim, is based on the strain level induced in the surrounding soil.

Low-strain tests are aimed at checking the integrity of piles, whereas high-strain ones are aimed at the evaluation of bearing capacity and thus they are called dynamic load tests (also STATNAMIC tests).

In this paper, the results obtained from the application of a SIT (Sonic Integrity Testing) technique are reported; among these kind of tests, the most used one is the ecometric test, where the pile head is struck with a small hammer and its response is recorded by an accelerometer. This procedure should allow to assess the presence of anomalous response, due to sudden geometrical changes (e.g. necking or section increase), soil inclusions or weak concrete. Infact, such variations of physical, mechanical or geometrical conditions influence the pile impedance, with consequent partial reflection of the travelling compression waves.

The compression wave is partially reflected by the pile bottom and recorded by the accelerometer, after having been amplified; it is then sent to an acquiring unit that turns it into a signal in the time domain.

The compression waves velocity through concrete, usually within the range $3500 \div 4500$ m/s, depends on its quality.

To measure with a certain accuracy this value, the most important parameter for all the subsequent evaluations, cylindrical samples of the fresh concrete have been collected during the piles' casting. Measurements have been made by the use of an *Ultrasonic Pulse Velocity Tester* and, following the UNI 9524/1989 prescriptions; the waves velocity has been determined in the last column of the following Table 1.

Table 1. Summary of sonic tests on concrete samples.

Concrete sample	Length [cm]	Test [ms]	v [m/s]
1 (08/10/03)	24.6	58.80	4183.67
2 (26/09/03)	24.5	58.30	4202.40
3 (11/09/03)	24.5	59.40	4124.58
4 (17/09/03)	24.3	59.40	4090.91
5 (15/09/03)	24.5	61.50	3983.74
6 (13/10/03)	24.3	63.63	3818.75
7 (16/10/03)	24.7	60.47	4084.90
8 (29/09/03)	24.6	60.57	4061.64
9 (14/09/03)	24.7	59.00	4186.44
10 (22/09/03)	24.4	59.50	4100.84
11 (07/10/03)	24.4	61.23	3984.76
12 (30/09/03)	24.4	57.93	4211.74
13 (12/09/03)	24.4	56.77	4298.30
14 (14/10/03)	24.8	61.17	4054.50
15 (09/10/03)	24.6	61.20	4019.61
16 (06/10/03)	25.0	61.30	4078.30
17 (15/10/03)	24.5	58.37	4197.60

The average value, obtained from the 17 tests performed, is 4098.98 m/s. Thus, the value for the waves velocity in the ecometric tests has been set equal to 4100m/s.

6 TEST RESULTS

Usually, the suggested test procedure considers three blows for every pile; the average of the three signals and the resulting pile length is then calculated by a custom-tailored software.

To perform an easier interpretation the output signals are amplified by an exponential function, which has its main effect in the final portion of the pile, close to the toe. This choice avoids the amplification of the signal in the upper part of the piles, where more noise is concentrated, due to poor soil characteristics or possible imperfections, and allows to single out more accurately the peak and the associated pile length.

Former experiences highlighted the strong influence of boundary conditions during test execution, like the preparation of the pile head, bad instruments positionings and eventual accelerometer instabilities. Thus, in order to have a greater number of signals to be processed, it was chosen to apply at least thirty hammer blows per pile head.

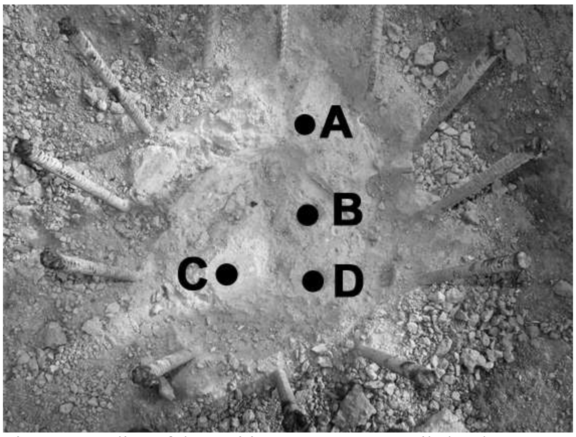


Figure 3. Outline of the positions A-B-C-D at a pile head.

Moreover, a greater number of blows has been hit on the piles which displayed erratic signals, according to a layout such as the one shown in Fig. 3. In fact, by varying the relative position of the hammer blow and that of the receiving accelerometer, it is possible to exclude the anomalous responses due, for instance, to superficial concrete discontinuities. Figures 4 and 5 report the results (as pile head velocity vs. depth) of the tests on pile nr. 53 (position BD and AD), where the grey curve is one of the many collected samples, whereas the black one reports the average of all the acquired signals.

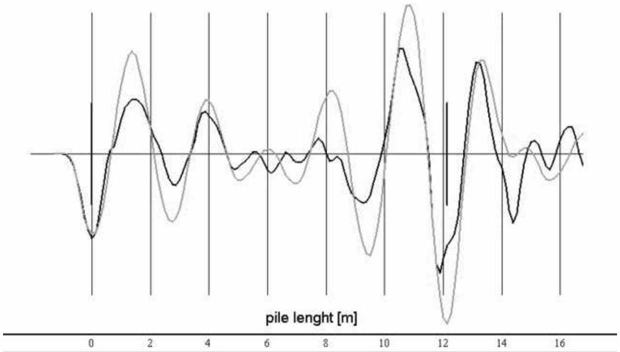


Figure 4. Pile 53 (BD): erratic output due to superficial concrete discontinuities; in grey one of the 30 blows, in black the averaged signal.

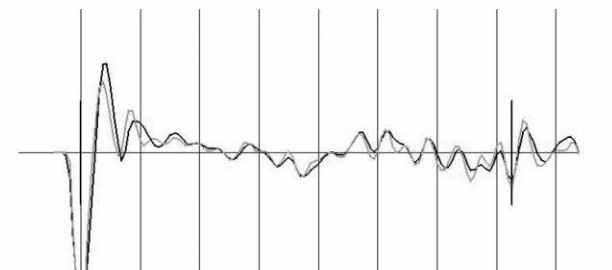


Figure 5. Pile 53 (AD): output signal through intact concrete.

It has to be remarked (Fig. 4) that the output signal BD, where B is the position of hammer blow and D the one of accelerometer, can be considered as an indication of the global poor quality of the pile.

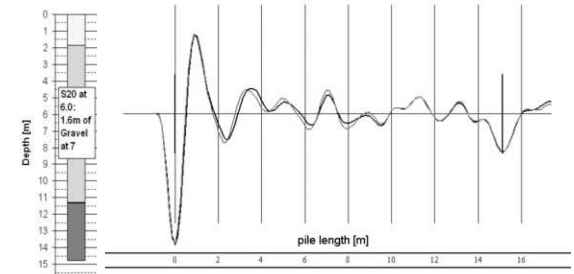
Nevertheless, analysing the signals AD (hammer blow applied in A, accelerometer settled in D) it can be pointed out the regularity of the response, with absence of necking or section increase, and the right pile length, as previously designed (14.8 m).

All the investigations and analyses carried out highlighted the dependency of the output response on the site stratigraphy. Infact, processing the signals without considering the different layers, the 13% of the piles would appear with necking, or section increases, and could not be considered completely intact.

Many of the bad records were recognized as due to clear and sharp changes in the soil surrounding the shafts: the different layers densities and their confining actions along the piles' axes have a real influence on the low strain pulse velocities. These assessments have been possible by keeping into account the piles' soil profiles, obtained as described in Paragraph 3.

The change from a soft stratum (silts - sandy silts) down to a stiffer one (sandy gravels - gravels) marks an output signal which could be erroneously interpreted as a section increase; on the contrary, a stiff to soft interface registers a wave reflection which, at first glimpse, appears as a definite shaft necking.

The following figures 6 and 7 show the mean signals (b) along with the soil profiles (a) for both the piles nr. 254 and 261. Two upward spikes are well evident at 7 meters in depth, even within of the typical sinusoidal trace layout, and they would suggest a section increase; these piles are really close to the nearby borehole nr. 20, where a consistent 1.60 meters thick gravel pocket was found within the sandy silt layer, at the very depth of 7 meters from GL.

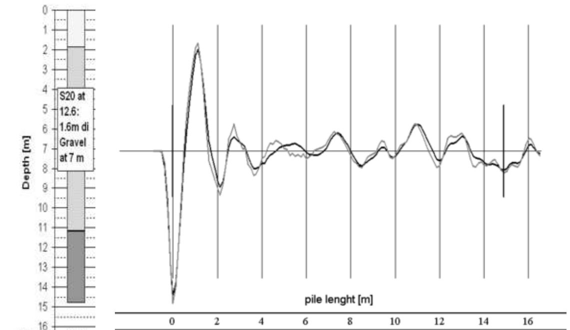


Pile	254
Layer "C"	3.49
Layer "B"	9.47
Layer "A"	1.84

PALO 254				
Wave Velocity[m/s]	Measured Length[m]	Expand Factor	Testing Date	Judgement
4100	15.1	20	16/12/2003	A

(a) *(b)*

Figure 6. Revised result for pile nr. 254, keeping into account of the anomaly represented by a 1.6 m thick gravel pocket (inset in a) found at 7 meters from the GL.



Pile	261
Layer "C"	3.63
Layer "B"	9.31
Layer "A"	1.86

PALO 261				
Wave Velocity[m/s]	Measured Length[m]	Expand Factor	Testing Date	Judgement
4100	14.9	20	10/12/03	B

(a) *(b)*

Figure 7. Revised result for pile nr. 261, due to the same gravel lens as said in Fig. 6, found in the nearby borehole nr. 20 at 7 meters from the GL.

Unfortunately, by a spatial analysis, it will never be possible to keep into account of local anomalies, such as pockets or lenses but, thanks to the borehole distribution on the site, in this instance it was possible to investigate and recognize the cause of some erratic signals, as said above.

The pile analysis, carried out with the abovementioned non-destructive technique, has allowed to check the real length and the kind of imperfection along the axes.

Keeping into account the two different techniques, the bucket system and the special CFA system *Trelicon*®, the results obtained are globally rather good. Nevertheless, the 3% of the piles showed anomalies not directly connected to soil changes, imperfect pile head preparation and the like.

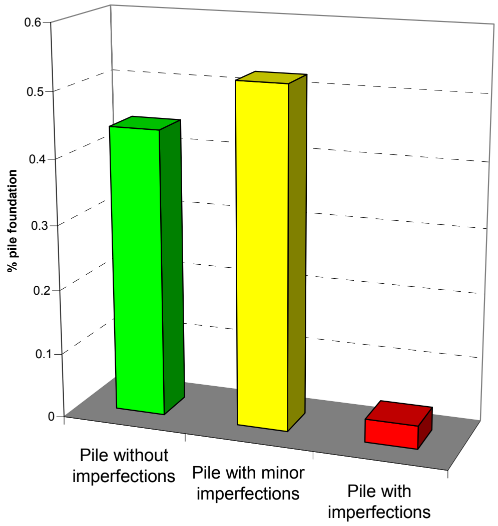


Figure 8. Judgement categories and percentage.

A further distinction should be done within the group of "piles with imperfections": it is more likely to find significative defects in the piles drilled by bucket than in the ones by the CFA system. In the former case, being both the drilling and the grouting/casing uplift operations discontinuos, some soil may easily slide into the shaft and cause neckings or discontinuities.

Infact, a higher percentage (about 9%) of piles having bad quality is concentrated in the bucket-drilled group, whereas only the 2% of the CFA-drilled piles show defects.

In order to check directly the effectiveness of the tests, a borehole has been drilled within pile nr. 270 and the concrete core was sampled continuously.

The ecometric tests (Fig. 9) highlighted not only various anomalies along this pile's axis but, above all, the real length was found to be about 11 m: quite different from the designed one, which had to reach 14.8 m.

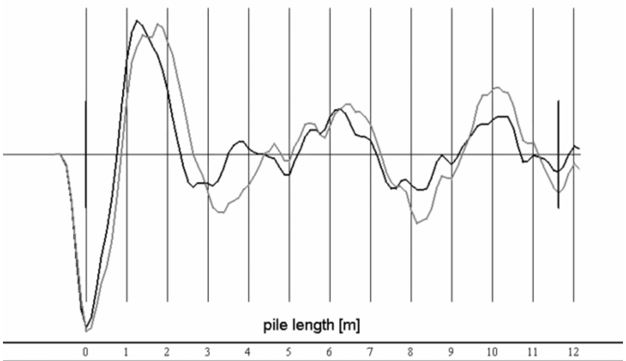


Figure 9. Output signal of pile 270.

The concrete core shows a good correspondence between the test results and the real pile layout. Particularly, it is evident (Fig. 10) that the pile length is 10.4 m, with a sand inclusion (about 23 cm long) at 1.9 m from the head and a clear necking in the last 188 cm.

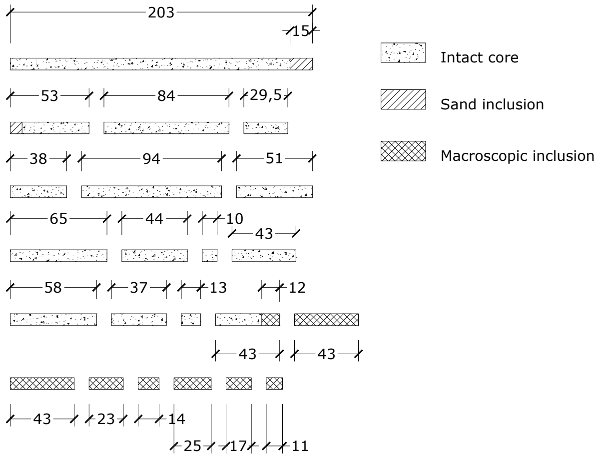


Figure 10. The axial profile of the concrete, bored through pile nr. 270, bears evidence of a total length of 10.41 m and of the definite presence of rubble along the last 1.88 meters.

7 FINAL REMARKS

Non-destructive tests are a simple, quick and rather cheap tool to verify the integrity and the quality of pile foundations, even at the early stages of construction.

Nevertheless, it has been shown how a simple analysis of the output results, without a deeper care about the boundary conditions, can lead to wrong estimations about their quality.

The person in charge to express the integrity judgements must have a profound knowledge into the fields of waves theory and soil dynamics; he himself should perform the on site acquisitions and, moreover, needs the best awareness of the site stratigraphy.

Even if this kind of non-destructive test is widely used and clearly settled (ASTM, 2000), some difficulties often arise to assess a correct interpretation.

In order to acquire a better knowledge upon some of the causes of erratic signals, the Authors have just started a laboratory investigation to identify the low strain response from medium-scale pile models with known defects, and/or different soil densities along the pile's shaft.

ACKNOWLEDGEMENTS

The authors wish to thank Contractors *Sparaco Spartaco* S.p.A., *Isedo Immobiliare* S.r.L. (Rome), and Prof. Edoardo Bozzo (Sidercad S.p.A. – Genoa) for their permission to use the site data in this paper. Thanks are also due to the students M. Poggini and P. Romaggi, who helped while developing part of their Master Degrees in Civil Engineering.

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