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Integrity tests on various types of piles

J.De Jaeger
Ecole Centrale des Arts et Métiers, Brussels, Belgium, and Catholic University of Louvain, Louvain-la-Neuve, Belgium

M.Van den Broeck
Laboratorium voor Grondmechanica, State University of Ghent, Belgium

A.Holeyman
Research and Geotechnics, s.a. Franki, Liège, Belgium, and University of Brussels, Belgium

Ch.Legrand
Division Structures, Belgian Building Research Institute (CSTC-WTCB), Brussels, Belgium

ABSTRACT : Full scale tests were performed in Ghent to improve the knowledge of the behaviour of screwed cast-in-place concrete piles. The Belgian Geotechnical Society took this opportunity to test the validity of dynamic and integrity pile tests. Five pile types were tested. It was interesting to examine how the roughness of the shaft influenced the propagation of the waves and their dissipation into the surrounding soil. Five professional and research organizations were invited to make integrity tests and to give predictions about the length and the form of the piles. The predictions and measurements are given. The values assumed for the wave velocity, the soil and pile influences are discussed.

1 INTRODUCTION

The use of pile dynamic testing, and particularly of integrity testing is getting more and more widespread because of its several advantages : these tests can become a cheap, rapid and reliable pile quality control.

If the principle of the test is well established, its application in pile engineering has led to some difficulties which have not been completely solved up to now. Integrity tests were originally designed for driven prefab piles. Their use has spread to other types of cast-in-situ piles. This extension requires to solve the problem of wave propagation in a pile affected by geometrical irregularities. It's also necessary to investigate the pile type influence and the damping of the wave as it dissipates into dense layers surrounding the pile.

Two methods are commonly used : the sonic echo test and the mechanical impedance test

The first method checks the integrity of the shaft and gives an information about the quality of the soil contact. It involves the impact of the pile head with a light hammer. The resulting vibration signal is measured. The test interpretation attempts to detect the sonic echo reflected from some irregularities down the pile. If the echo is clearly visible, then the length of the pile down to its first discontinuity (hopefully the pile length) can be ascertained. It requires an hypothesis on the velocity at which the waves can travel up and down the pile.

The second method (mechanical impedance) involves the frequency analysis of the mechanical admittance and leads to an additional information such as the contact quality of the pile with the soil.

2 RESEARCH PROGRAM

The Belgian Institute for Scientific Research in Industry and Agriculture sponsored an extensive research program to compare the bearing capacity and the behaviour of four pile types. The research was conducted by the Belgian Building Research Institute (CSTC-WTCB).

Four different pile types were investigated on a site in Ghent : De Waal precast concrete piles, Socofonda continuous auger piles, Fundex and Atlas screw piles.

This opportunity was seized by the Belgian Geotechnical Society to organise in Brussels a symposium on dynamic pile testing.

Several firms and institutions, from Belgium and from neighbouring countries, were invited to perform integrity and dynamic load tests, and to give a length and bearing capacity prediction before

any static load test and an eventual pile extraction were carried out.
Here, only the integrity test results are reported.

3 SITE INVESTIGATION

The test-site is in the neighbourhood of the railway station in Ghent. The choice resulted from a preliminary documentary investigation (geological and geotechnical maps, tests previously executed). An extract from the geotechnical map is given in Table 1. The investigation study was carried out with 26 CPT-tests, 1 boring, 1 SPT-test, 1 pressuremeter-test and 4 static plate loading tests (diam. 3.20m). The rather homogeneous shape of the subsoil was confirmed by CPT tests executed at the place of the future piles. One may consider the results as representative for the whole site.

Figure 1 gives a detailed plan of the site itself and indicates the location of the different piles, tests, borings, a.s.o.

Some CPT-test results obtained from the site investigation are given in figure 2, 3 and 4.

4 PILE TYPES AND CHARACTERISTICS

Various pile types were to be tested : replacement screw piles (Socofonda piles bored with a continuous hollow auger), displacement screw piles (Atlas and Fundex cast-in-situ piles), and De Waal driven precast concrete piles as reference.

4.1 Socofonda continuous auger piles

These piles are driven into the soil by screwing down a continuous hollow auger with a loose cap. Boring is performed by means of a torque combined with a static axial load. Once the desired depth is reached, concrete is pumped under high pressure trough the hollow shaft to the pile base. The concrete pressure draws the auger up. After full concreting a reinforcement cage can be introduced by means of an electric vibrator.

These piles (5, 9, 13, 17, 21; fig. 1) had a nominal diameter of 0.45 m and a nominal embedded length of 14.5 m. The pile 13 had a total length of 16.0 m (head included).

The pile 13 was extracted and measured (fig. 4 A) at the end of the research campaign. The actual length was 16 m (1.5 m head included) with 0.458 m as actual mean diameter; the minimum diameter was 0.44 m at a depth of 2.0 m. There was no significant defect to observe over the total pile length.

4.2 Fundex piles

The Fundex pile is driven into the soil by screwing a drill tube whose bottom has a stem auger with a diameter wider than the one of the tube. At the required depth, a reinforcing is placed and concrete is poured into the tube. The tube is extracted with an upward and downward movement. The pile shoe (drill tip) remains in the soil. One obtains a pile with a relati-

Table 1. General description of the soil in the research site at Ghent. Extract from the geotechnical map n.22.1.6 (Scholencampus, Voskenslaan, Ghent)

Description of the layer	Thickness (m)	Relative level (top) (m)	Absolute level (top) (m)
Backfill (sand)	1.25	0.00	+7.50
Sand and clay (Holocene) and sand (Pleistocene)	2.25	-1.25	+6.25
Sand and clay (Paniselian)	6.00	-3.50	+4.00
Clay (Paniselian) : the site is situated just north-west of the limit of a slip surface in the sandy-clayey Paniselian	0...1	-9.50	-2.00
Sand and clay (Ypresian) with glauconite and sandstones	16.00	-10.00	-2.50
Clay and silty clay (Ypresian)	-	-26.00	-18.50

vely regular lateral surface and a base diameter equal to the wider part of the screw, somewhat higher than the one of the shaft.

These piles (3, 8, 12, 16, 20; fig. 1) had 0.38 m as nominal diameter. They went to 13.0 m depth. Only the pile 12 had a total length of 14.5 m (1.5 m head included).

The pile 12 was extracted and measured (fig. 4 B) at the end of the research campaign. The actual length was 14.4 m (1.5 m head included) with 0.391 m as actual mean diameter; the minimum diameter was 0.36 m at 3.2 m and 9.8 m depth. There was no significant defect.

4.3 Atlas pile

The Atlas pile is formed by using a steel tube the bottom of which is made into a hollow stem auger which is screwed into the soil. At the required depth a steel reinforcing cage is introduced into the tube prior to concreting. The smaller steel part of the drill tip uncouples and remains in place while the tube is extracted by reverse screwing. The concrete under pressure flows out of the tube and fills the helix formed by the reverse rotation and by the upward moving part of the auger screw.

These piles (6, 10, 14, 18, 22; fig. 1) had a nominal diameter ranging between 0.36 and 0.46 m. The embedded length of the piles was 12.5 m. Only the pile 14 was 14.3 m long (head included).

The Atlas piles 14 and 18 could only be extracted partially. The actual diameter of those parts varied between 0.43 and 0.53 m.

4.4 De Waal precast concrete piles

These piles (7, 11, 15, 19, 23; fig. 1) had a 0.32 x 0.32 m square section. Their embedded length varied between 13.3 and 15.7 m and is given in Table 2 (Nom. depth). The piles 19 and 23 had a greater total length (head included).

4.5 Franki piles

Cast-in-situ driven Franki traction piles were used to obtain the reaction needed for the static loading tests. These piles were driven before execution of the screw piles. They were not included in the integrity test program.

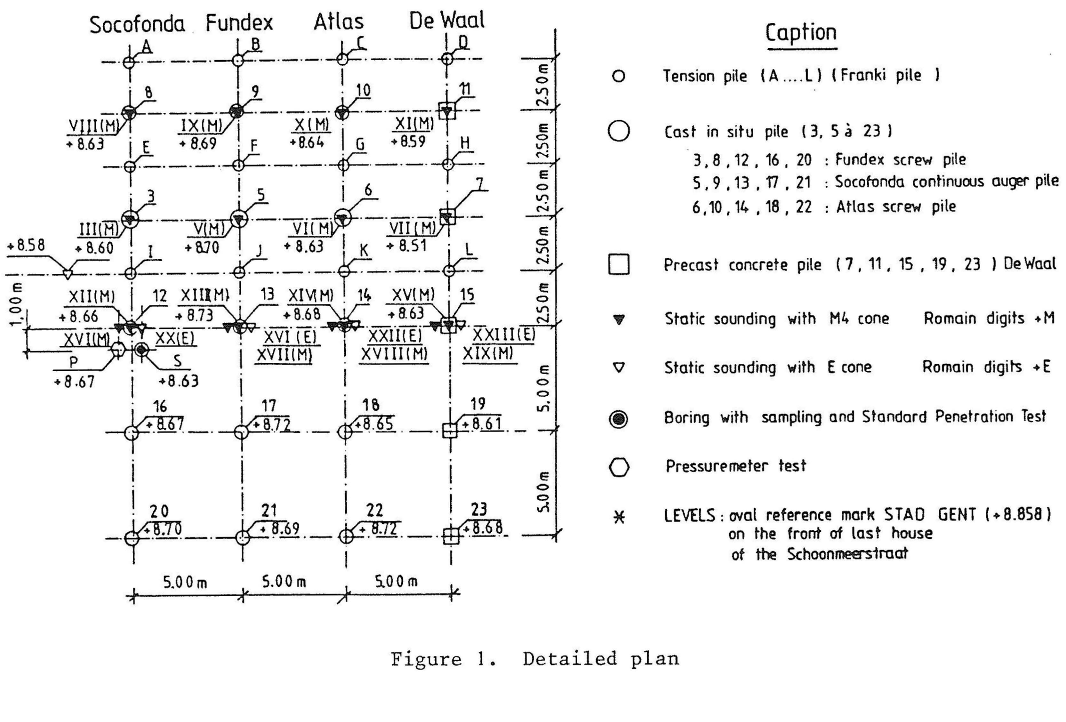

Figure 1. Detailed plan

5 TEST PROGRAM AND PARTICIPANTS

Five piles of each type were installed as shown on figure 1 and made available for integrity tests. The destination of the five piles for each type was, in principle :
- 1 start-up pile (20 to 23) which allowed the pile contractor to adjust its procedure to the local soil characteristics
- 1 integrity test pile (14 to 19)
- 1 dynamic load test pile (12 to 15)
- 2 static load test piles (3 to 9).

Integrity tests were performed on two piles of each type by the following institutions :
1. CEBTP (Centre Expérimental de Recherches et d'Etudes du Bâtiment et des Travaux Publics) from Paris, France
2. FRANKI from Liège, Belgium
3. FUGRO GEOTECHNIEK from Leidschendam, The Netherlands
4. Laboratorium MAGNEL - Rijksuniversiteit Ghent. Belgium
5. TNO (Nederlanse Organisatie voor Toegepast Natuurwetenschappelijk Onderzoek) from Delft, The Netherlands

CEBTP and TNO made some more tests than the two required determinations.
The participants used the well known echo method (see e.a. Stain, 1982; Seitz, 1986) except CEBTP which used the mechanical impedance method (Paquet, 1968; Davis and Guillermain, 1980).

6 TEST RULES

The organizing committee invited the participants to take the challenge of a class-A Prediction, and to submit their report before any answer element was made available. An independant evaluation committee was created to organize the tests and to screen the results. Game rules have been stated to the participants who agreed with them.

The committee has declined to give precise informations on the pile geometry and length in order to be able to evaluate objectively the reliability of the predictions. However it has given a classical geotechnical survey consisting of extracts from the Ghent geotechnical map and the results of three CPT tests performed in the vicinity of the test piles. Information publications given by the foundation contractors were made available.
The contributing institutions were informed that the piles had a nominal diameter between 0.3 and 0.45 m, and that their depth could vary from 11 to 16 m.

Participants had to test the piles on the same day (25th May 1987) being assigned 60 minutes each for integrity testing of at least two piles.

At the end of the day, contributors were asked to submit their measured signals and, if possible, a first data interpretation to the independant evaluation committee.
Complete reports were submitted afterwards, but before the performance of the static load tests.

Finally for each pile type, a pile was extracted by means of water injection for form and length controls. Only for the Atlas pile it was not possible to extract the total pile, due to a too low reinforcement ratio.

Table 2. Pile lengths and Predictions

No	Nom. dpth	Nom. lgth	Act. lgth	Predicted lengths FUGRO	FRANKI	MAGNEL	TNO	CEBTP
	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)
Socofonda continuous auger piles								
5	14.5	14.5		-	-	-	14.0	-
9	14.5	14.5		-	-	-	13.5	-
13	14.5	16.0	16.0	-	-	-	15.3	15.2
17	14.5	14.5		14.0	14.0	14.0	14.0	14.0
21	14.5	14.5		14.5	14.4	13.6	14.5	14.5
Fundex piles								
3	13.0	13.0		-	-	-	13.0	-
8	13.0	13.0		-	-	-	13.1	-
12	13.0	14.5	14.4	-	-	-	14.2	14.5
16	13.0	13.0		13.0	13.3	12.9	12.9	13.0
20	13.0	13.0		13.0	13.0	12.3	12.5	12.4
Atlas piles								
6	12.5	12.5		-	-	-	ND	-
10	12.5	12.5		-	-	-	ND	-
14	13.0	14.3		-	-	-	ND	14.5
18	12.5	12.5		15.0	13.3	14.6	ND	15.6
22	12.5	12.5		14.0	13.2	ND	ND	ND
De Waal prefab piles								
7	13.9	13.9		-	-	-	15.0	-
11	13.3	13.3		-	-	-	15.0	-
15	13.4	14.8		-	-	-	15.0	ND
19	14.7	15.0		18.0	16.8	17.9*	15.0	ND
23	15.7	17.0		14.0	16.7	15.6+	ND	ND

(ND = not determinable)
(*+ : after a second interpretation, Magnel gives resp. 15.3 m * and 15.2 m +)

7 PILE LENGTH PREDICTION

The participant predictions are summarized in Table 2 and compared with the nominal lengths. The term "depth" is used for the dimension of the pile embedded into the ground. The prefab piles 15, 19, 23 had a part above the ground level; the piles 12, 13, 14 had a concrete head necessary for the dynamic loading tests. The "length" term is used for the total pile dimension including the concrete part above the soil level.

For the precast De Waal piles, the relative errors vary form -18 % to +20 %, and the length is not determinable after CEBTP. The length prediction seems to be unaccurate for those driven precast piles. In theory the regular form and the well known section must ease the estimation. In practice difficulties are introduced by the high contact pressure between the soil and the shaft, and the high friction which hides the point echo.

For the Socofonda continuous auger piles and for the Fundex piles, the estimations are very good. The relative errors are limited to some percents.

The prediction of the Atlas pile length is more difficult because of the very irregular shape of the shaft due to the form of the metallic drilling tip. Three participants upon five have not given a determination at least for one pile. When the values were given, one finds relative errors ranging from +1 % to + 25 %; all overestimated.

8 WAVE VELOCITY

The pile length prediction requires an assumption on the wave propagation velocity in concrete. Thus the prediction accuracy is limited by the validity of the wave velocity assumption. After Vanneste (1984) this limitation is about 5 %.

The wave propagation velocity is depending upon the concrete quality, the pile execution method, a.s.o. Ellway (1987) gives an approximate linear relationship between the logarithm of the concrete cube strength and the wave velocity; for most practical purposes values in the range from 3500 to 4000 m/s are assumed.

For the length determinations CEBTP, Fugro and TNO assumed a mean wave velocity equal to 4000 m/s as commonly used in current practice.

The values adopted by Franki and Magnel are reported in Table 3. The Magnel choices resulted from ultrasonic measurements performed on the test piles. For Franki the assumed values are a function of the presumed concrete quality.

For the De Waal prefab piles, Franki measured the wave velocity on a removed part, 3.5 m long, coming from the cut back of the static load test pile 11. The measured velocity was 4400 m/s. However the velocities measured transversal on the integrity test piles 19 and 23 were approximately equal to 4100 m/s. It is the value adopted by Franki for the interpretation.

For the Atlas piles, Franki has reduced the assumed velocity value. For Franki the spiral around the Atlas pile shaft is the cause of a low wave velocity, as observed earlier by Stranger and Vanneste (1984). Also Vyncke and Van Nieuwenburg (1987) have given an other explanation of the low wave velocity in the Atlas piles: they showed by numerical simulation that the echo delay is longer with a shaft having numerous section variations than for a regular one. Therefore Franki supposes for the length calculations that the wave propagates at 3400 m/s.

After the pile extraction the wave velocity has been measured by the CSTC-WTCB on parts coming from De Waal precast pile 19. The determined values varied from 4100 to 4200 m/s on head parts ; a value of 3900 m/s was found on a base element.

Those assumptions and determinations are compared in Table 3 to the wave velocities calculated from the nominal pile lengths and the echo time values measured by the participants.

Table 3. Wave velocity values

Pile		Assumed Franki m/s	Assumed Magnel m/s	Deduced from actual length and echo time m/s
17	Socofonda	4000	4020	4140 ... 4160
21	Socofonda	4000	3855	4000 ... 4110
16	Fundex	4200	4000	4000 ... 4120
20	Fundex	4200	3960	4000 ... 4220
18	Atlas	3400	3885	3200 ... 3330
22	Atlas	3400	3460	3220 ... 3570
19	De Waal	4100	4120	3330 ... 4000
23	De Waal	4100	4075	4180 ... 4860

The values reported in Table 3 show that, for a given recorded echo time, the wave velocity assumption can be the source of errors greater than the 5 % previously announced. According to Seitz (1986) and Ellway (1987), the errors may be 10 %.

9 PILE INFLUENCE

9.1 Socofonda continuous auger piles

For all the participants, the toe reflection is clearly visible. This is due to a very low skin friction. According to CEBTP, this friction is lesser than for the Atlas piles.

The very good prediction is to note for every participant. The measured reflection time interval is quasi identic for everybody. A little dispersion is due to the wave velocity assumptions (Magnel).

9.2 Fundex piles

The Fundex piles have a moderate skin friction, less than the driven precast De Waal piles. According to CEBTP the impedance curves are similar to those of the Atlas piles, but the skin friction is lower.

The participants found a toe reflection echo clearly visible. Franki detects the second echo and shows that the time interval for the first toe-reflection is shorter than the second one. This fact was signalized by Stranger and Vanneste (1984). Franki calculates the pile length with the first interval.

9.3 Atlas piles

After all the participants the signals show a very high damping. CEBTP and Magnel explain that a very good contact exists between soil and pile. (For CEBTP this is proved by a high characteristic impedance). For Franki, Fugro and TNO, the very large damping is caused by the helicoidal outer surface.

For everybody the toe reflection is difficult to detect, perhaps not determinable. TNO explains that "the signal components which are present at the place where the toe reflection is to be expected show no consistancy". TNO doesn't give a length prediction, but affirms that there are no significant discontinuities. TNO and Magnel said that in a normal case they would test more of these piles.

The measurements justify the Franki assumption about a reduced wave propagation velocity in the Atlas pile. This fact confirms the observations and explanations of the helicoïdal shaft influence (Stranger & Vanneste 1984; Vyncke & Van Nieuwenburg 1987).

For the Atlas piles, a conclusion may be the difficulty of signal interpretation caused by the heavy damping and the helicoidal shape of the shaft. Note that the uncertainty concerning the wave velocity should be taken into account for the calculations.

9.4 De Waal precast driven piles

For all the participants, the De Waal piles show heavy clamping. The driving caused a very high skin friction along the whole length of the shaft. Toe reflection is not always visible.

10 SOIL VARIATION, PILE FORM AND DEFECT INFLUENCE

10.1 Introduction

The aim is to examine the possibility of making a distinction (out of the monitored signal) between some influence on the one hand due to differences or variations in soil properties and on the other hand due to irregularities in pile dimensions.

This will be done by means of a comparison and detailed analysis of all recorded signals (on a same time scale) out of the shock wave integrity tests, and only for the piles knowing the exact dimensions together with the available data of the insitu soil properties (especially the CPT results).

On this basis the piles which could be taken into account are the driven De Waal precast concrete piles to examine the soil influence and the excavated Fundex pile 12 and Socofonda pile 13 to examine the combined effect of variations in the pile dimension and in the soil characteristics.

More information concerning the total and embedded pile length, diameter and predictions are given in a previous point and in Table 2.

For the piles 7, 11, 15 (De Waal), and 12 (Fundex), and 13 (Socofonda), only TNO echo test measurements were available. For the piles 19 and 23 (De Waal) results of the four firms (Franki, Fugro, Magnel, TNO) could be taken into account.

10.2 De Waal piles 7, 11, 15

The first stage starts with the evaluation and the analysis of the prefabricated concrete De Waal piles. For these piles the exact cross-sections and lengths are very well known. It is obvious that in this case any signal irregularity is due to the variation of the surrounding soil properties (excluding some influence of possible cracks due to the installation process).

On figure 2 the De Waal piles 11, 7, 15 are represented together with the respective cone penetration test results and the registrated (TNO) reflection signal. This reflected signal is pictured on a time scale as it was recorded. The link between this time scale and the depth scale is given by means of the wave velocity. Here the standard value of 4000 m/s is used as by TNO.

Irregularities in the velocity-time scale compared with the variations of the $[q_c]$ and the $[Q_{s,t}]$ depth values ($[Q_{s,t}]$ cumulated lateral skin friction on the sounding tubes), together with the friction number $[f_s]=[\Delta Q_{s,t}]/[q_c]$, have to be considered at the same time with some theoretical considerations and the knowledge concerning the influence of the skin friction on the reflections; a general idea is: the higher the skin friction is, the higher there will occur a "negative" velocity reflection wave and the more the damping and the dissipation (into the surrounding soil) of the initial wave will be.

On fig. 2, indicating a rough soil layer classification concerning the $[Q_{s,t}]$ values, one can observe at a depth of about 4 to 5 m (the top of the sandy-clayey paniselian soil layer with rather high friction values) any negative influence on the velocity signal, especially for the piles 7 and 15. The same phenomenon appears at a depth of 7 to 8 m at the top of a slightly sandier and higher friction soil layer. Also for the pile 15, due to the pile head of 1.4 m (longer than the minimum length of 0.50 m for detecting possible "defects" (after TNO)), it is rather easy to observe the influence of the medium dense packed sandy backfill top layer.

Once beneath a depth of 10 m a fine clayey sand to fine sand (12 m) (Ypresian soil layer) causes high damping of the signal due to some kind of clamping and wave dissipation into the surrounding soil. This phenomenon is almost obscuring the toe reflection.

As a conclusion one may say that together with the known soil variation data (here out of CPT) and the pile dimensions one can make an evaluation of the influence of those variations on the monitored signal. By means of simulation programs it is possible to improve the quality of this interpretation and to make reliable and objective judgements of the reflection wave.

10.3 De Waal piles 19 and 23

An analogous interpretation can be made for the precast De Waal pile 19 and 23, having the results of all participants. Those measurements are illustrated respectively on figure 3 A and 3 B, all on a same time scale (as they were gross registrated), linked with the length of the piles and the depth scale of a reference CPT (+ an extrapolation of the CPT's given on figure 2) by means of a wave velocity of 4000 m/s.

It is obvious that for the analysis no direct use is made of the (absolute) magnitude of the velocity signal given on the figure 3, due to different amplification methods in function of the time (linear, exponential,...) by the different firms; only the relative variation for each signal itselfs is taken into account.

Concerning the influence of the soil layer variation on the measurements one can

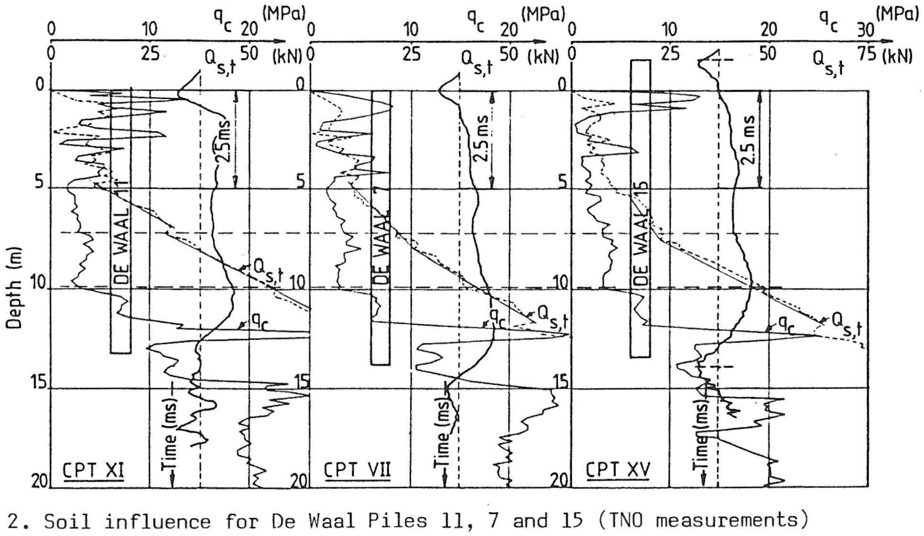

Fig. 2. Soil influence for De Waal Piles 11, 7 and 15 (TNO measurements)

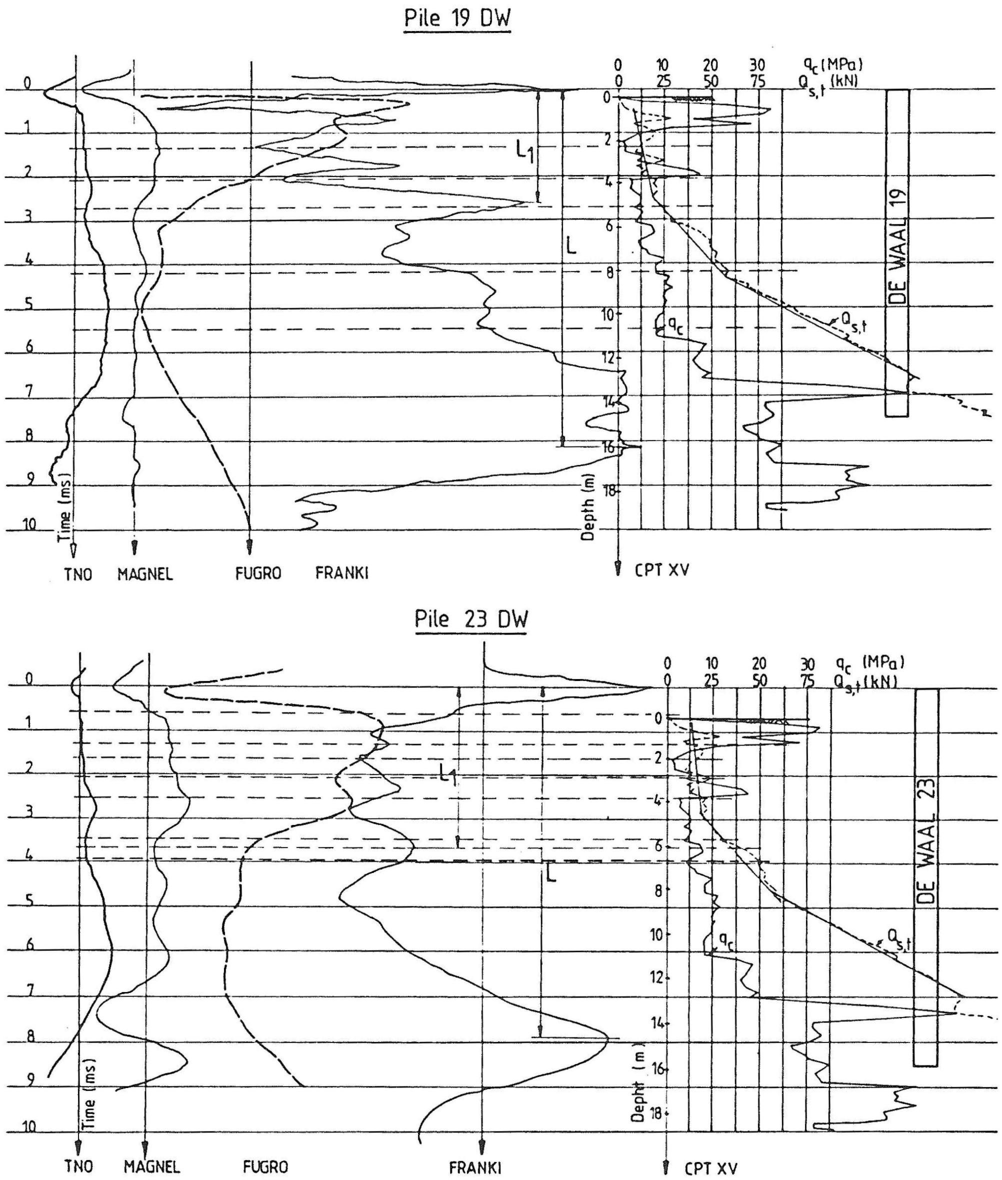

Fig. 3. Soil influence on the time scaled echo measurements for De Waal Piles 19 and 23

make an analogue discussion as before (piles 11, 7, 15) for the TNO results. Some kind of representation is made on the figures itselfs.

At the same time it is obvious that the different recorded signals are also easy to correlate (on a same time scale basis). As an exemple of such a good correlation there exists the interpretation made by the firms themselves : each firm (excepted Fugro) found for the pile 23 at the depth of 5.8 m to 6 m beneath the top of the pile an intermediar reflex of a compression wave, indicating a rather high clamping of the pile.

As a conclusion, one can say that having the pile and the soil strata data (here CPT's) it is possible to evaluate for most of the signals of the different firms the influence of the variation of the soil stratification.

10.4 Socofonda pile 13 and Fundex pile 12

Finally one can also try to make such an evaluation and interpretation for auger and screw piles. For this purpose we will use the Socofonda pile 13 and the Fundex pile 12. These two piles, as already told in a previous point, were excavated after finishing the different tests.

The dimensions are discussed earlier and are fully represented on figures 4 A and 4 B, together with the proper CPT result. On those diagrams, one can observe some minor pile enlargements in weak soil strata and some minor necking in dense packed soil strata.

Taken into account those pile diameter variations together with the conclusions made for the De Waal prefab concrete piles for the case of soil influence on the reflection signal, one can derive with some "goodwill" any influence of the pile diameter variations. The difficulty is even with the aid of sofisticated simulation programs that it asks an interpretation of a superposition of two different influences which are note easy to split up and which have some mutual physical dependances.

10.5 Conclusion on soil influence

Generally spoken, one can say, having adequate and correct soil strata data and reflections signals measured with a high sampling frequency, it seems to be possible to predict and specify small pile diameter variations and if or where discontinuities (together with the type of the discontinuity) in the pile are located.

Nevertheless to make such conclusions until now it necessitates together with the mentioned data, a high degree of experience and qualification of the persons involved within.

11 CONCLUSION

The acoustic control is a simple, cheap and rapid technique for concrete pile integrity testing. The low cost allows a systematic control and a selection for a more precise examination. However an apparent difficulty remains in a correct measurement interpretation.

The echo method is well adapted to the integrity control and the detection of major failures. The mechanical impedance method seems to be more adapted to test the integration of the pile into the soil.

These methods must be able to determine the distance from the pile top to the first sensible irregularity; for an intact pile one can measure the pile length, considering the tip as a major irregularity.

Initially acoustic control was reserved to short piles with a well known nominal section. For precast driven piles, the length determination could be eased by their section regularity. In fact the wave dissipation in the soil makes it rather dif-

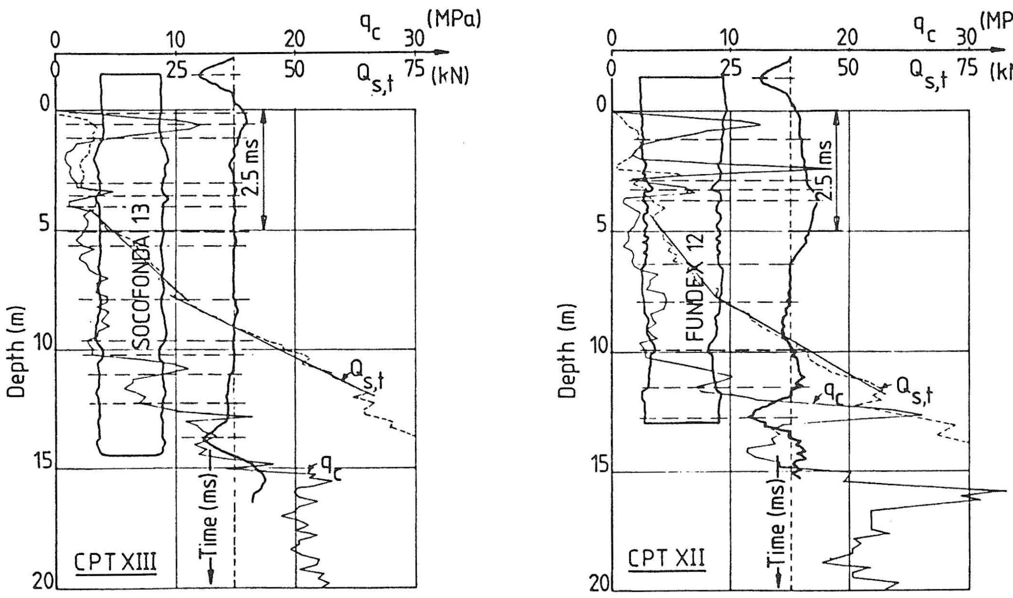

Fig. 4. Soil and pile combined influences (TNO result on Socofonda 13 and Fundex 12)

ficult because of the excellent contact between the pile and the soil.

For the cast-in-place screwed piles, the participants have given length predictions with a very good quality. Small imprecisions could be explained by the assumed values taken for the wave velocity. They can be minimized by better calibrations. The fact that the values are especially difficult to choose for the Atlas helicoidal piles is to be stressed. This choice is then critical for the predictions and interpretations.

The participants have proved that the detection of majors breaks of continuity was reliable. However, the pieces of advice have been very inconsistent concerning the shape variations and minor defects. It is true that signal superpositions coming from close irregularities can mask the searched phenomena (pile diameter and/or soil irregularities).

The echo must be interpreted in terms of soil nature variations and soil/pile interaction. Therefore one can detect acoustic anomalies, but strictly one cannot make a distinction between pile and soil variations.

Correct interpretation requires the understanding of the wave propagation in the pile and the knowledge of the influencing factors : concrete quality, pile shape, execution methods and side contact between the pile and the soil (e.a. soil layer profile and mechanical soil characteristics). The integrity test method has to be used in conjunction with sufficient informations concerning each of these factors. The interpretation must finally be aided by a good numerical model able to simulate the pile behaviour and to compare the results with the measurements.

The reliability of integrity tests requires a good theoritical model, a solid understanting and valuable site informations : three tools to use for a correct interpretation of acoustic control.

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