Analisi dei dati inerenti le prove dinamiche su pali condotte in svizzera, con una riflessione sui micropali

Data analysis on pile dynamic load tests in Switzerland, with a focus on micropiles

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ABSTRACT

Il test Pile Driving Analyzer® test è stato largamento usato in Svizzera negli ultimi 20 anni, senza che sia mai stato condotto uno studio globale dei risultati. Lo scopo di questa ricerca è di analizzare le circostanze per le quali il test è adatto ed efficace. Durante lo studio sono stati raccolti dati degli ultimi 20 anni a proposito di test effettuati su pali battuti, pali trivellati e micropali. L'analisi è stata condotta in tre fasi. Nella prima fase sono state considerate le informazioni riguardanti l'esecuzione della prova e i valori di progetto. Nella seconda fase, è stata studiata l'influenza dei parametri geometrici. Nella terza prova, è stata analizzata la relazione tra i risultati della prova e i valori calcolati dal programma di modellazione associato. Al termine di questa ricerca, è possibile affermare che questo test è particolarmente adatto ai pali battuti, mentre dà risultati soddisfacenti ma non sempre affidabili nel caso dei pali trivellati. Per quanto riguarda i micropali, saranno necessarie ulteriori ricerche allo scopo di definire un sistema di adattamento del test a questo tipo di fondazioni e di identificare diversi criteri che possano determinare la conformità della prova.

ABSTRACT

Pile Driving Analyzer® test has been widely used over the past 20 years in Switzerland, without ever having conducted a comprehensive study of results. The aim of this study is to analyse the circumstances in which this test is actually effective. This research analyses collected data of last 20 years about different tests on driven piles, drilled shaft piles and micropiles. The analysis has been made in three phases. In the first phase data about test operation and design have been considered. In the second phase geometrical data have been studied. In the third phase PDA test parameters have been analysed. At the end of this study it is possible to state that the test is particularly suited to the driven piles only and satisfactorily suited to the drilled ones. For micropiles it will be necessary further research to design a test adaptation and to identify different criteria to assess the compliance.

Keywords: *deep foundations, PDA test, micropiles*

SYMBOLS AND ABBREVIATIONS

P_{mob} mobilized load during the test
P_{serv} service load
P_{dim} design load
$\bar{d}_{blow}$ sinking of the blow chosen for Capwap® analysis
t sinking
RMX pile Case method calculated resistance
RMX_{cc} pile Case method calculated resistance of the blow chosen for Capwap analysis
EMX energy released after the blow

1. INTRODUCTION

Every year, hundreds of piles and micropiles, with different diameters and lengths, are driven or drilled into the Swiss region, as shown in *Figure 1.*

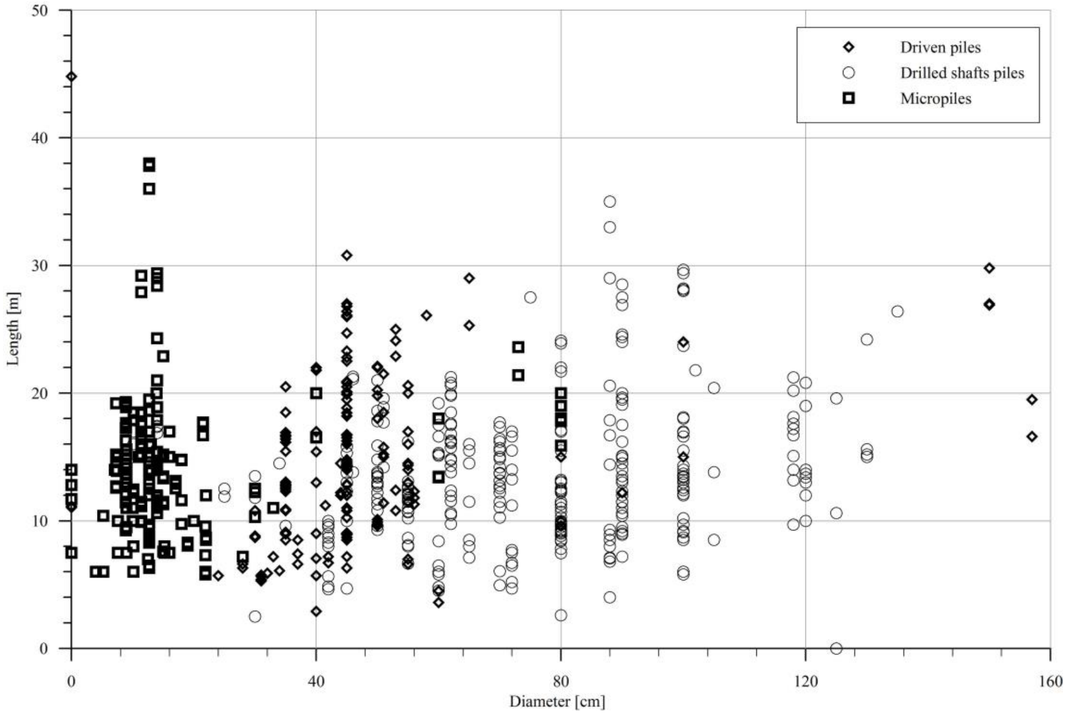

Figure 1 - Length diameter type of piles and micropiles used in Switzerland. Copyright Erika Prina Howald.

Companies require specialists to regularly test piles' integrity and bearing capacity. Dynamic tests, most commonly called PDA (Pile Driving Analyser), allow specialists to determine the pile-soil unit's bearing capacity at a lower cost than that is required by the static test. As a result, PDA has been widely used over the past 20 years, without ever having conducted a comprehensive study of results obtained in different circumstances.

The principle, on which the PDA tests are based, was conceived in the late 60s in the United States. The test consists of hitting the pile head with a hammer or a drop weight, the energy of which is well defined in terms of weight and height of fall. In the stressed pile, a compression wave propagates. Along the pile and stem, the wave undergoes reflections, which are dependent on the characteristics of soil and irregularity of the perimeter. Finally, the waves

rise to the surface at the head of the pile. They are measured by sensors and analysed by the unit of acquisition. An approximate calculation of bearing capacity is directly executed by the control unit, by following the Case-Goble method.
Subsequently, the measurements are analysed through a program that uses modelling, overlapping the theoretical signal and the measured one. The comparison follows an iterative process, gradually changing the different parameters that characterize the pile and the soil.
This research therefore welcomes the demand for different companies and engineering firms to better understand the PDA test and analyse the circumstances in which this type of test is actually effective, given the wide variability of stratigraphy present in Switzerland, as well as of geometry for piles and micropiles.

2. PILE DYINAMIC TEST

2.1 The test

Following extensive experience, in particular from Case Western Reserve University, on dynamic methods for non-destructive foundations' testing, the traditional static load test has begun to be supported by the dynamic methodology, called "Case Method". The method, known as a high strain dynamic test, replaces the use of the static load test in the United States. The software for the interpretation, Capwap, which models the pile-soil interaction, is standardized by Eurocode 7.
By controlling the weight and hammer's height of fall, the test hits the head of the pile with well-defined energy.
The hammer falls on the head of the pile from different heights, in ascending increments of 20 cm, until the achievement of energy necessary to mobilize all the resistances of the pile-soil system, or in the case of verification, until the achievement of an impact force equal to the load test.
The relief of the deformation and of the propagation velocity of the compressional wave is made by sensors. On the tested pile, a pair of accelerometers and at least a pair of strain transducers are attached at least two pile diameters below pile head, to ensure that the measured deformation is uniformly distributed on the measuring section. Strain gauges and accelerometers communicate with the acquisition unit. A first simplified processing, real-time, provides a variety of information to help manage the progress of the test, until the desired load is achieved.
Should one wants to cause experimentally the load at ultimate limit state, it is necessary that the pile present a permanent penetration after the impact. According to the experience of engineering studies, the extent of this penetration must be between 3 and 10 mm. The measure of this effect is performed using a laser incident on a target placed on the head of the pile, indicative of the sinking.
Cross sectional area, elastic modulus, specific weight and pile length are the four quantities that make up the pile profile. Two additional pile properties are the pile toe area and the overall wave speed, if it differs from the wave speed calculated from elastic modulus and specific weight.

2.2 Theoretical basis

The wave equation approach is an excellent predictive tool for analysis of impact pile driving, but it has some limitations. These are mainly due to uncertainties in quantifying some of the required inputs, such as actual hammer performance and soil parameters.

To overcome such uncertainties, the first Pile Driving Analyser (PDA) and his associated software, now known as Capwap, was produced in 1972, in the USA.

When a hammer or drop weight strikes the pile head, a compressive stress wave travels down the pile at a speed *c* (wave speed).

We assume the pile is one-dimensional. If we define a vertical coordinate *z* with origin at pile head and downward direction, we can define the quota at time *t* of the point at quota *z*, when $t=0$, $m(z,t)$. Assuming *E* Young's modulus, *A* section area, *ρ* density and R_f lateral friction of the pile, it is possible to write the equilibrium of an infinitesimal element, whose length is *dz*. We can then obtain the equilibrium differential equation along the pile:

$$\frac{E}{\rho}\frac{\partial^2 m(z,t)}{\partial z^2}-\frac{\partial^2 m(z,t)}{\partial t^2}=\frac{R_f}{\rho A}$$

This is a wave equation.

The solution's form is:

$$m(z,t)=f_1(z-ct)+f_2(z+ct)$$

where $c=\sqrt{\dfrac{E}{\rho}}$ is the wave speed.

If we define the mechanic impedance of the pile $Z=\dfrac{EA}{c}$, the wave velocity in the pile *v* and the force in the pile *F*, it is possible to write the wave up and the wave down equations:

$$F\uparrow=\tfrac{1}{2}(F-Zv)$$

$$F\downarrow=\tfrac{1}{2}(F+Zv)$$

When we have a one-directional down wave, we can say $F=Zv$, and for the up wave $F=-Zv$.

When the pile is floating and the tip resistance is negligible, the wave reflected at the tip will be a tension wave ($F_{tot}=F\uparrow+F\downarrow=0\rightarrow F\downarrow=-F\uparrow$). If instead the pile rests on a very stiff layer, the reflected wave will be a compression wave ($V=0\rightarrow F\uparrow-F\downarrow=0\rightarrow F\downarrow=F\uparrow$).

Considering a finite length pile, knowing material characteristics and formulating hypotheses about resistances, it's possible to evaluate the total resistance to pile penetration, as stated by (Pavan, 2010), by:

$$R(t^*)=\frac{1}{2}\left[F_m(t^*)+F_m\left(t^*+\frac{2L}{c}\right)\right]+\frac{Mc}{2L}\left[v_m(t^*)-v_m\left(t^*+\frac{2L}{c}\right)\right]$$

Where $F_m(t^*)$ is the vertical force, measured at time t^*, v_m is the experimentally measured particles velocity and *M* is the pile mass.

This formula is valid under these assumptions: constant and uniform section, perfectly elastic and homogeneous material, single axial pulse, without any other component in the other directions.
The total resistance is not a sufficiently representative model of reality, as it presents significant restrictions on the soil resistance, due to the friction side, and consequently on real bearing capacity.
So the total pile penetration into the ground resistance is divided into two components: a static component R_s and a dynamic component R_d, through the relationship:

$$R_t = R_s + R_d$$

Despite the dynamic resistance dependence on the value of static resistance and velocity, it is assumed that it is a linear function of the speed in the bottom of the pile v_t only:

$$R_d = J \cdot v_t$$

With a negligible difference in the results, in fact, this assumption simplifies the computational process.
J represents the viscous damping constant. This constant can be expressed as a function of Z and, through a damping constant dimensionless J_c, we obtain the relation:

$$J = \frac{J_c}{Z}$$

The exact determination of J_c is only possible by performing a static load test, which is why we rely generally on the values found in the literature (*Table 1*).

Soil type	**J_c range**	**J_c suggested value**
Sand	0.05-0.20	0.10
Silt sand	0.15-0.30	0.20
Silt	0.20-0.45	0.30
Silt clay	0.40-0.70	0.55
Clay	0.70-1.10	1.00

Table 1 - Literature suggested values for J_c (*Pavan, 2010*).

2.3 Results analysis

After having evaluated static quantities in the pile, it is possible to proceed to the comparison between measured and calculated waves, as to determine the value of the match.
The Capwap results are based on the best possible match between the computed pile top variable, say the pile top force, and its measured equivalent. Once this agreement has been optimized, the analysis is finished. When the match is unsatisfactory, the process of iteratively changing the soil resistance parameters (J_c) and computing the pile top variable, is repeated. To compute forces at the pile top during each trial analysis, pile and soil are modelled mathematically and algorithms are followed that allow for a step by step computation of all pile variables along pile length and over time.
Capwap's numerical analysis is based on the method of characteristics which solves the one-dimensional wave equation. Capwap performs the actual calculations, however, by dividing the pile into a series of segments which are individually of uniform properties.

The program calculates the wave down and the wave up values in each segment at times when downward (upward) waves arrive at the bottom (top) of the segments. At that point, reflections are calculated from the relative properties of neighbouring segments as well as the resistance forces. Since superposition of waves is possible according to the one-dimensional differential wave equation, total forces and velocities for each pile segment can then be directly calculated from sum of the downward and upward wave values.
In order to produce an objective measure of the quality of a match, it is necessary to calculate a match quality number which is independent of a personal assessment. The Capwap Match Quality Assessment number (Match) is a mismatch indicator: the higher its value, the worse the match.
Match is calculated by taking into account the absolute value of the difference between the measured wave and the calculated wave, at different time instants.
It is not possible to establish a maximum limit of Match below which the match is good, because of the difference in the signals due to the different testing conditions in site. However, experience shows that three is a generally acceptable threshold to determine the quality of the Match.

3. DATABASE

In order to assess whether the PDA test is suitable for the evaluation of different types of piles' bearing capacity through a statistical analysis of data collected in very different circumstances over a period of twenty years, it was decided to create a database.
The database was created using Microsoft Access®. Since the data were provided by construction companies, several types of information for each pile were collected.
Database structure consists of eight tables:

- Piles: it contains all the information regarding pile type, geometry, design loads, test data, presence or absence of other tests performed.
- Project: it contains information pertaining to the case design
- Engineering firm: it contains information about the engineering firm that provided data
- Blows: it contains specific information about each blow undergone by piles
- Enterprise: it contains information about enterprises that installed piles
- Hammer: it contains information about employed hammers
- Soil: it contains parameters of soil in which piles have been installed
- Stratigraphy: it contains information concerning stratigraphy in which piles have been installed
- RMX: it contains the *RMX* value chosen for Capwap analysis

In the database, all the data available for each test have been entered. The pile types inserted were: driven piles, drilled shaft piles, micropiles, vibrated stone columns and jettings. Since data for vibrated stone columns and jettings are rare, they will not be considered in the following discussion.
Data, however, are not always complete, so it is not possible to fill in all the fields for each entry. In particular, there are limited data on the stratigraphy, which made it impossible to correlate the behaviour of the pile to the soil in which it is immersed. This result implies that it is not possible to carry out an analysis about soil-structure interaction, if only through the coefficient *J*.

4. STATISTICAL PROCESSING OF DATABASE

The statistical processing of data has followed three main phases. During the first phase, data about operation and design were considered. Then, geometric data concerning the various tests were analysed. Finally, we evaluated the relationship between some parameters that emerged from the test and the results provided by the software Capwap.

4.1 Operation and design

The first classification, performed on data received, concerns the relationship between mobilized load and service load (or design load, if service load is not known). This ratio provides information on the success of the test, according to the following criteria, currently used in design in Switzerland:

Pile is compliant if: $P_{mob} \geq 1.8 P_{serv}$ or $P_{mob} \geq 1.4 P_{\dim}$

Pile is not compliant if: $P_{mob} < 1.8 P_{serv}$ and $\bar{d}_{blow} \geq 3mm$ or $P_{mob} < 1.4 P_{\dim}$ and $\bar{d}_{blow} \geq 3mm$

Pile is inconclusive if: $P_{mob} < 1.8 P_{serv}$ and $\bar{d}_{blow} < 3mm$ or $P_{mob} < 1.4 P_{\dim}$ and $\bar{d}_{blow} < 3mm$

In order to compare the data, the value of $F_s = P_{mob} / P_{\dim}$, calculated from the ultimate load, with those of F_s, calculated from the service load, the first value has been multiplied by a homogenization coefficient equal to 1.8/1.4.

The first interesting result concerns the percentages that we can find in each category, as shown in *Table 2.*

		Total	Compliant	Not compliant	Inconclusive
Driven piles	N°	390	170	151	69
	%	100	43.6	38.7	17.7
Drilled shaft piles	N°	396	221	91	84
	%	100	55.8	23	21.2
Micropiles	N°	278	109	49	120
	%	100	39.2	17.6	43.2

Table 2 - Percentages of compliant, not compliant and inconclusive piles.

It is possible to observe that the test is particularly suited to the driven piles only and satisfactorily suited to the drilled ones; the tests that are inconclusive, in fact, relate to 17.7% of the former and to 21.2% of the latter. It should be remembered that the pile-soil interaction in the two cases is different, accounting for the difference between the two percentages.

The highest percentage found for the drilled shaft piles, however, leads one to query about the role of stratigraphy. As can be seen in graphs of following paragraphs, in fact, for most cases, the PDA test for drilled shaft piles gives reliable results, however, it happens that some results

do not make sense. The stratigraphy could play an important role in disturbing PDA test in this case, and it is appropriate to discuss, when more complete data are available
Regarding micropiles, however, in 43.2% of cases, it is not possible to make statements on the conclusiveness of piles. This evidence suggests that the criterion chosen for the conclusiveness of driven and drilled shaft piles is not suitable for micropiles.

4.1.1 Relationship between safety factor and sinking

The graph relating driven and drilled shaft piles shows a decrease in the safety factor's dispersion correlating with the sinking's increase (*Figure 2*). Notably, the dispersion reaches a stable value, for sinking's values comprised between 5 and 10 mm.

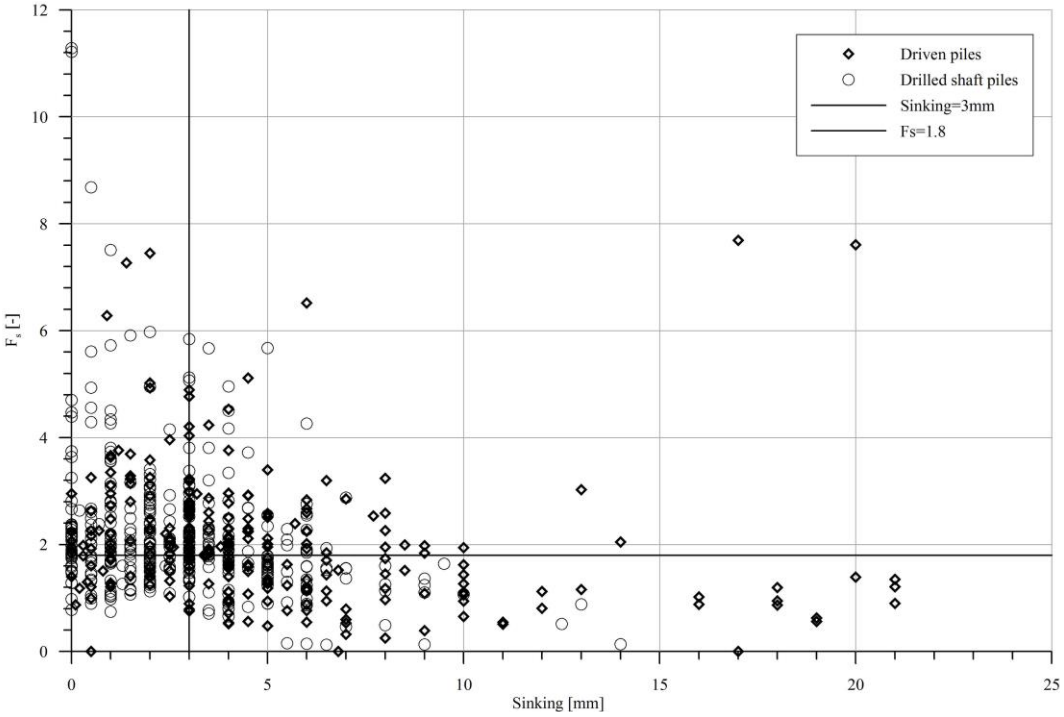

Figure 2 - Relationship between the safety factor and the sinking for driven and drilled shaft piles. Copyright Erika Prina Howald.

This result may suggest a loss of resistance in the tip for punching over 5 mm of sinking. The loss of resistance allows to validate the value of 3 mm of sinking, chosen as a threshold, in order to mobilize the lateral resistance of the pile, without causing soil failure.
Regarding micropiles, it is possible to note that, for a sinking less than 3 mm, the number of micropiles with a not compliant safety factor is greater than that for which this factor is compliant (*Figure 3)*. This leads one to the conclusion that the threshold value of 3 mm for the resistance mobilization cannot be extended to the micropiles. It is not, however, possible to evaluate an alternative proposal to the threshold value, because it is necessary to run an ad hoc test campaign.

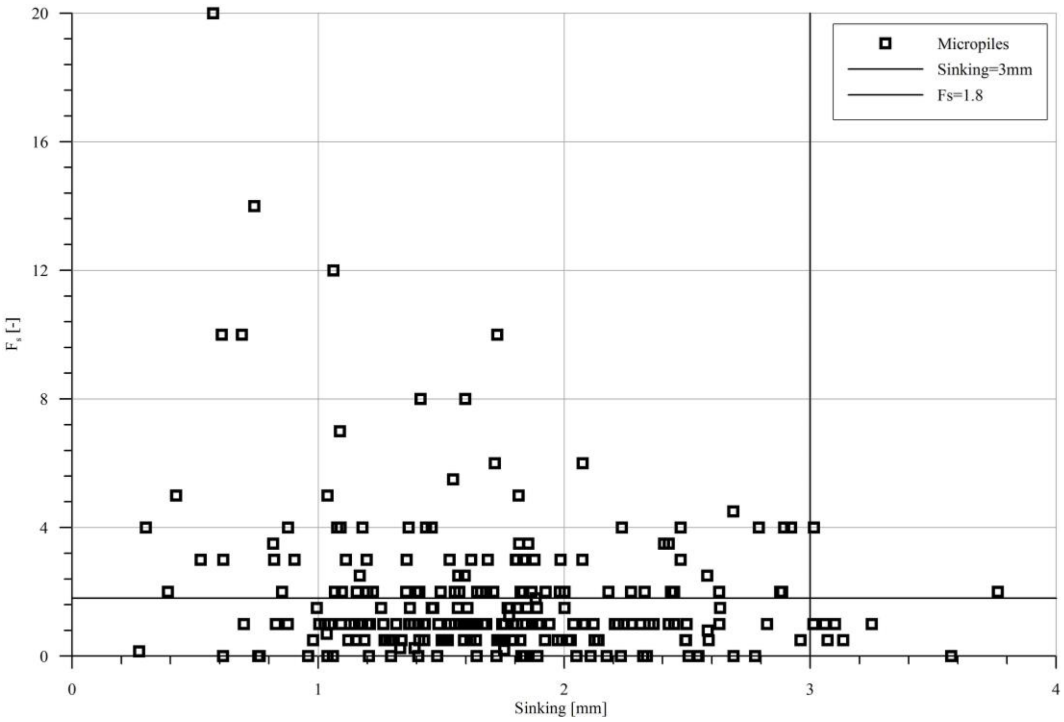

Figure 3 - Relationship between safety factor and sinking for micropiles. Copyright Erika Prina Howald.

4.1.2 Compliant piles' sinking

Since the condition of compliance does not provide a criterion about the sinking, it is interesting to note the sinking value of compliant piles.

		Total	**t<3 mm**	**t≥3 mm**
Driven piles	N°	170	63	107
	%	100	37.1	62.9
Drilled shaft piles	N°	221	140	81
	%	100	63.3	36.7
Micropiles	N°	109	86	23
	%	100	78.9	21.1

Table 3 - Percentages of compliant piles that have reached 3 mm of sinking and of piles that have not.

Table 3 **Errore. L'origine riferimento non è stata trovata.**shows that only the driven piles generally have mobilized all their strength, according to the criterion of 3 mm, while drilled shaft piles and micropiles have not. It may suggest that the last two categories of piles are oversized, since the value of the safety factor required has been achieved with a remaining part of resistance to be mobilized.

4.1.3 Trend of piles with sinking < 3 mm

Since it has been noted that it is possible to have compliant piles that have not mobilized all resistance, it is useful to assess what may be the trend of inconclusive piles if their mobilized load is increased by 25%. This trend could represent the scenario in which piles were able to reach a sinking of more than 3 mm. *Table 4* shows the overall results obtained.

		Total	**Compliant**	**Not compliant**	**Inconclusive**
Driven piles	N°	390	186	151	53
	%	100	47.7	38.7	13.6
Drilled shaft piles	N°	396	259	91	46
	%	100	65.4	23.0	11.6
Micropiles	N°	278	161	49	68
	%	100	57.9	17.6	24.5

Table 4 - Percentages of compliant, not compliant and inconclusive piles, if the mobilized load is increased by 25%.

The fictitious increase of mobilized load has almost no effect on driven piles, although it widely changes the situation of drilled shaft piles. This evidence means that many of inconclusive drilled shaft piles belong to a limit zone. It is therefore possible to consider that the inconclusive drilled shaft piles had not reached a sufficient sinking. The high percentage of compliant drilled shaft piles suggests that the extension of the PDA test is appropriate to them.

4.2 Geometrical analysis

This phase allowed to highlight some relationships between different geometrical parameters and mobilized load.

4.2.1 Relationship between mobilized load and pile's diameter

As known, the load that can be mobilized depends on the pile's diameter, in a linear manner with respect to the lateral surface friction, and in parabolic manner with regard to the tip load.
In the following figures, mobilized load, with respect to the diameter trends are shown: in *Figure 4* for driven and drilled shaft piles, in *Figure 5* for micropiles.

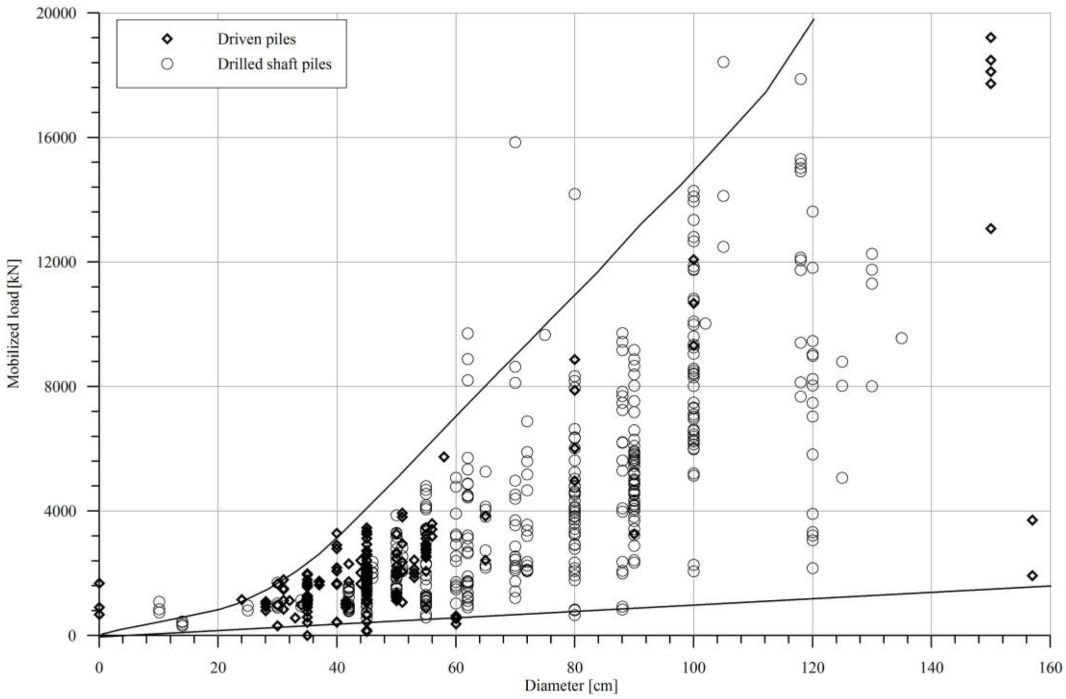

Figure 4 - Relationship between mobilized load and pile's diameter with parabolic upper limit and linear lower limit for driven and drilled shaft piles. Copyright Erika Prina Howald.

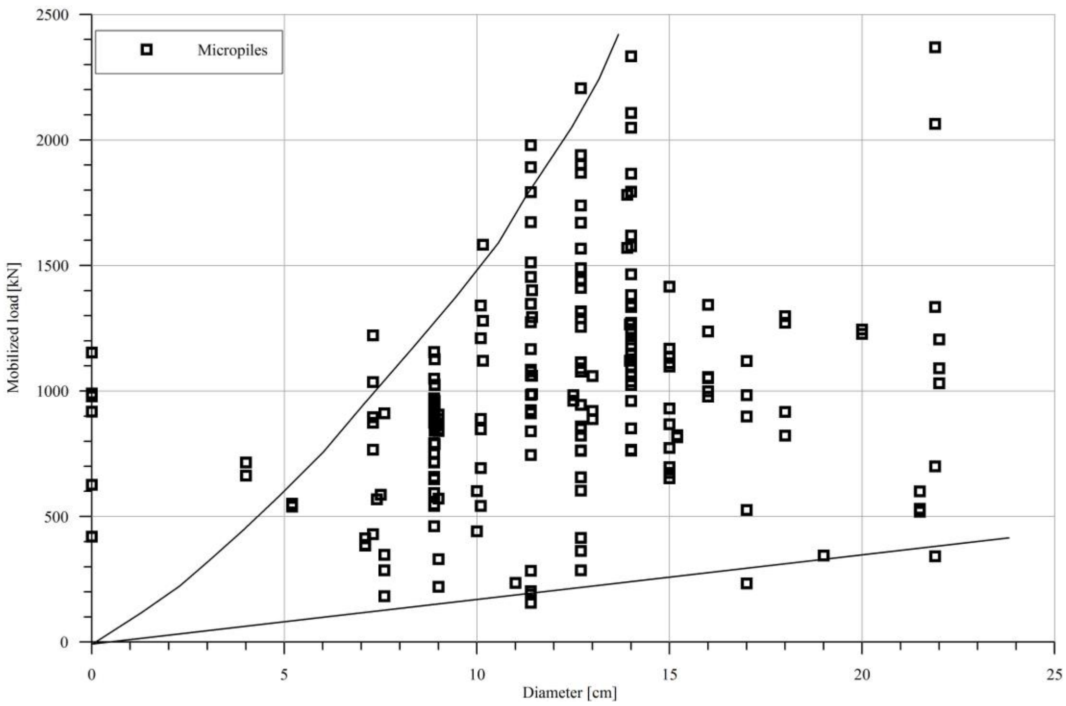

Figure 5 - Relationship between mobilized load and pile's diameter with parabolic upper limit and linear lower limit for micropiles. Copyright Erika Prina Howald.

For driven and drilled shaft piles a lower limit of linear type and an upper limit of parabolic type are clearly noticeable, reflecting the considerations concerning the increase of load.
A similar trend is evident in the micropiles, though less pronounced.

4.2.2 Relationship between mobilized load and pile's length

It is interesting to evaluate the trend of the mobilized load with regard to pile's length. In *Figure 6* all types of piles are shown.

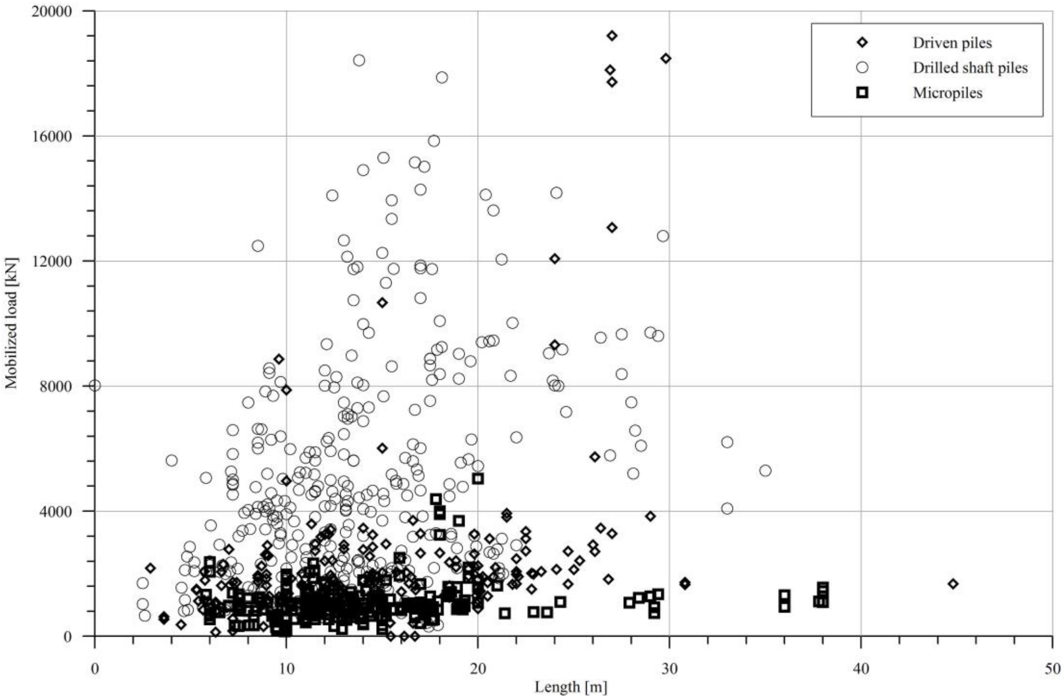

Figure 6 - Relationship between mobilised load and pile's length for driven and drilled shaft piles, and for micropiles. Copyright Erika Prina Howald.

It shows that the increase of mobilized load is linear with the length *L*, as expected, since it is dependent on the perimeter of the pile; up to a certain value of length, while increasing pile's length, mobilized load remains almost the same, according to the theory of piles dimensioning.

4.3 PDA test parameters analysis

4.3.1 Relationship between blowcount and sinking

The blowcount represents the theoretical number of blows, calculated by Capwap, necessary for the pile to sink one meter. This calculation is based on the signals of waves that rise and fall, as well as the measured sinking of blow chosen for analysis.

As the analysed tests are not part of a battery of tests and the tests were carried out in very different circumstances, transforming the blowcount into sinking calculated per hit was considered more useful for the purpose. So it is possible to compare it with the measured sinking and note the correlation.

In the following figures, these trends are shown separately for the three types of analysed piles: driven piles (*Figure 7*), drilled shaft piles (*Figure 8*) and micropiles (*Figure 9*), with the distinction between compliant, not compliant, and inconclusive piles.

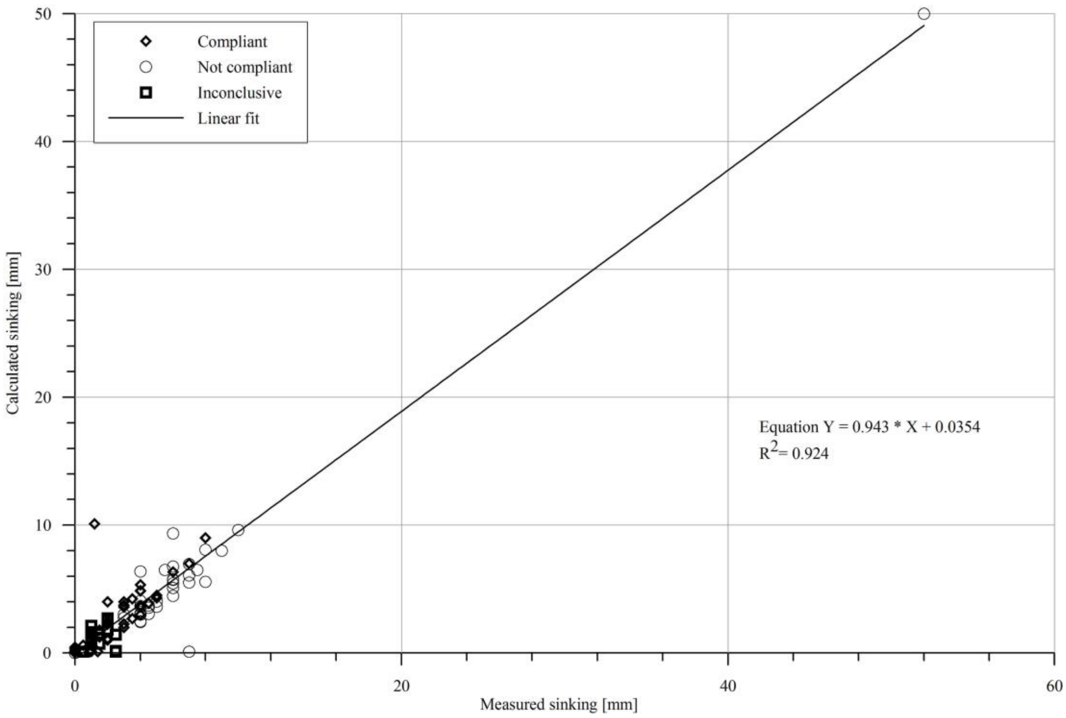

Figure 7 - Relationship between calculated by Capwap and measured sinking of driven piles. Copyright Erika Prina Howald.

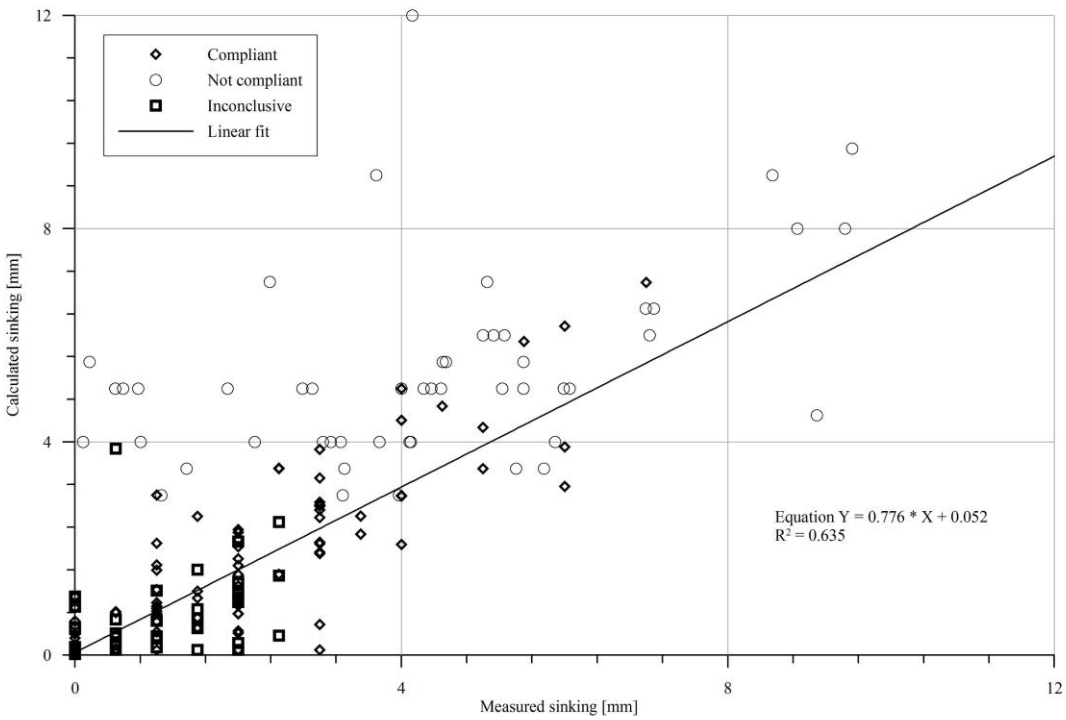

Figure 8 - Relationship between calculated by Capwap and measured sinking of drilled shaft piles. Copyright Erika Prina Howald.

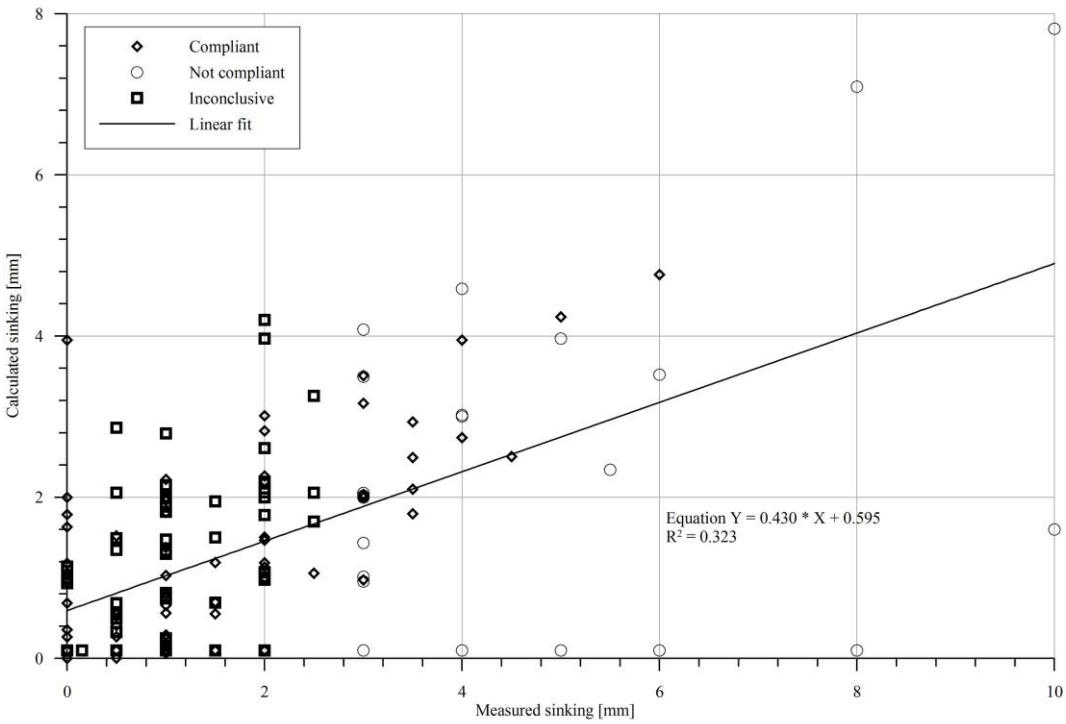

Figure 9 - Relationship between calculated by Capwap and measured sinking of micropiles. Copyright Erika Prina Howald.

Driven piles show a good correlation, with no distinction between compliant and not compliant piles. Drilled shaft piles show a wider correlation, but points also stand along a straight line. Micropiles, instead, show a particularity, since the points corresponding to the not compliant piles are very distant from the interpolation line. This result means that the calculated sinking does not match the measured one, in the case of tests where the entire load mobilization is not reached. The majority of the points, moreover, presents a measured sinking outside the admissible range of 3-10 mm. This leads to the conclusion that, on one hand, micropiles could be damaged and, consequently, the signal read by the control unit acquisition has been disturbed; on the other hand, it is possible that the forces exerted by the hammer were so weak as not to create signals properly usable by Capwap.

4.3.2 Relationship between RMX and mobilized load

It is interesting to compare the resistance values calculated from Capwap (*RMX*) with the mobilized load, for different pile types that show some difference. In the following figures, the ordinate axis shows the value of *RMX* made dimensionless by dividing it by mobilized load.
Driven piles (*Figure 10*), for which mobilized load rarely exceeds 4000 kN, present a wide dispersion around the value 1.0. This trend is also found for drilled shaft piles (*Figure 11*), which are, however, able to achieve much higher mobilized load values. Passed the threshold of 4000 kN, the dispersion decreases, accordingly as does the average value of the data going from 1 to 0.5. For micropiles (*Figure 12*), finally, the dispersion is very large and it is around an average value of 0.9. These average values below 1.0 show that the mobilized load tends to be greater than two times of *RMX* for drilled shaft piles and 1.11 for micropiles. It is noted, however, in some cases, that the mobilized load is less than *RMX*.

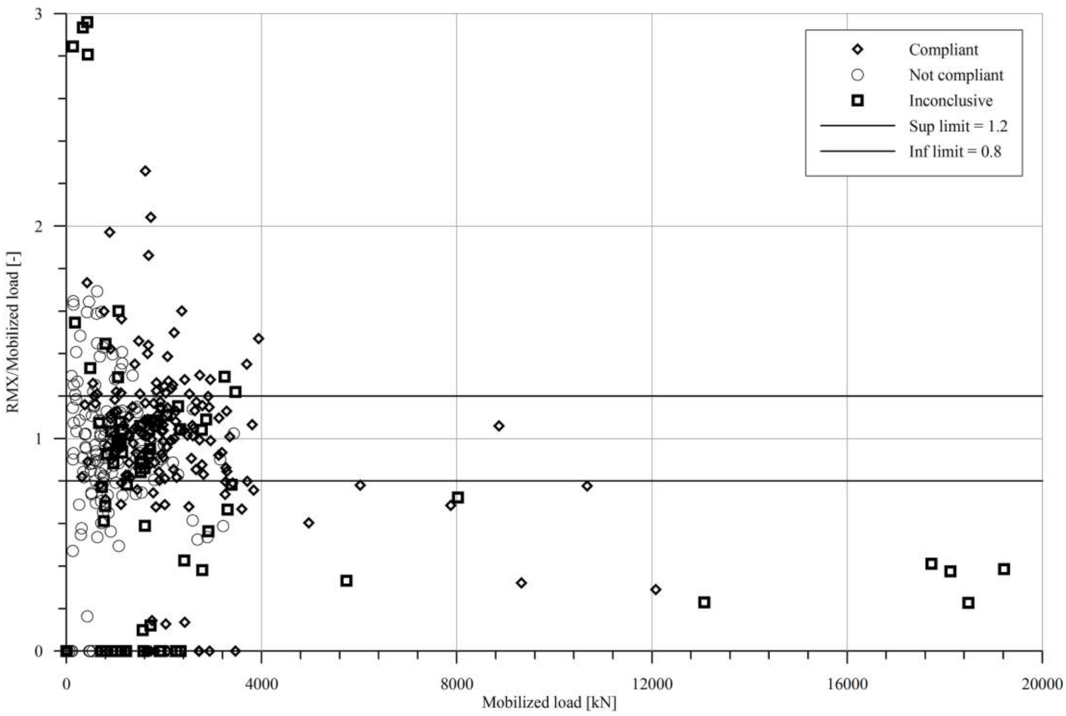

Figure 10 - Relationship between RMX, made dimensionless by dividing it by mobilized load, and mobilized load for driven piles. Copyright Erika Prina Howald.

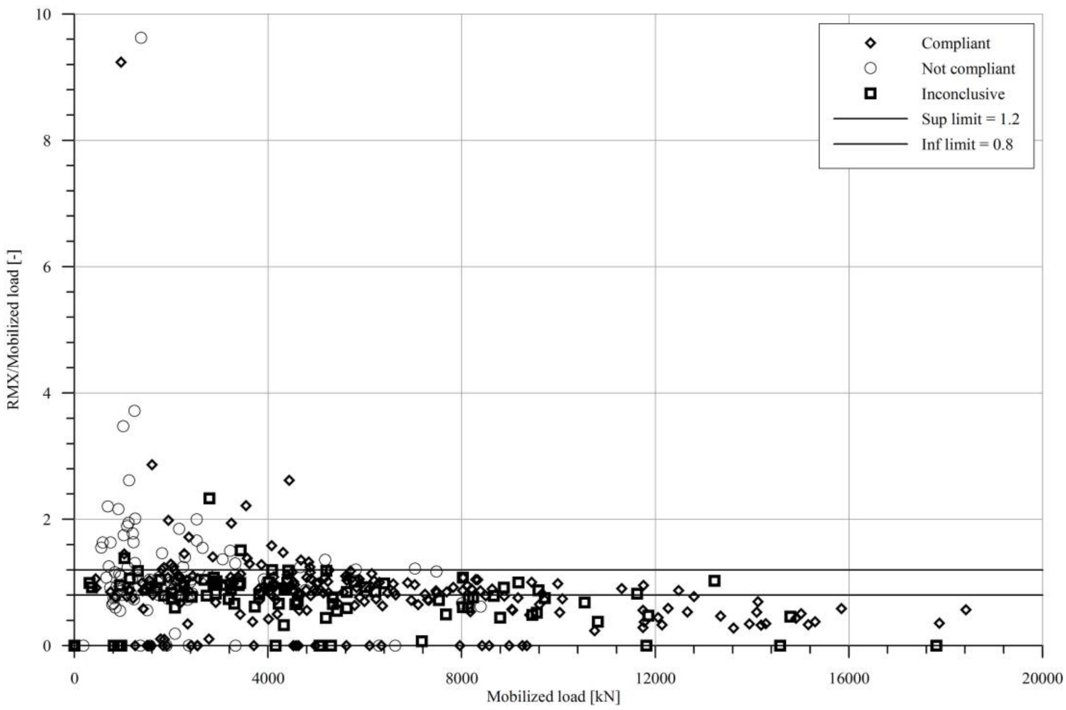

Figure 11 - Relationship between RMX, made dimensionless by dividing it by mobilized load, and mobilized load for drilled shaft piles. Copyright Erika Prina Howald.

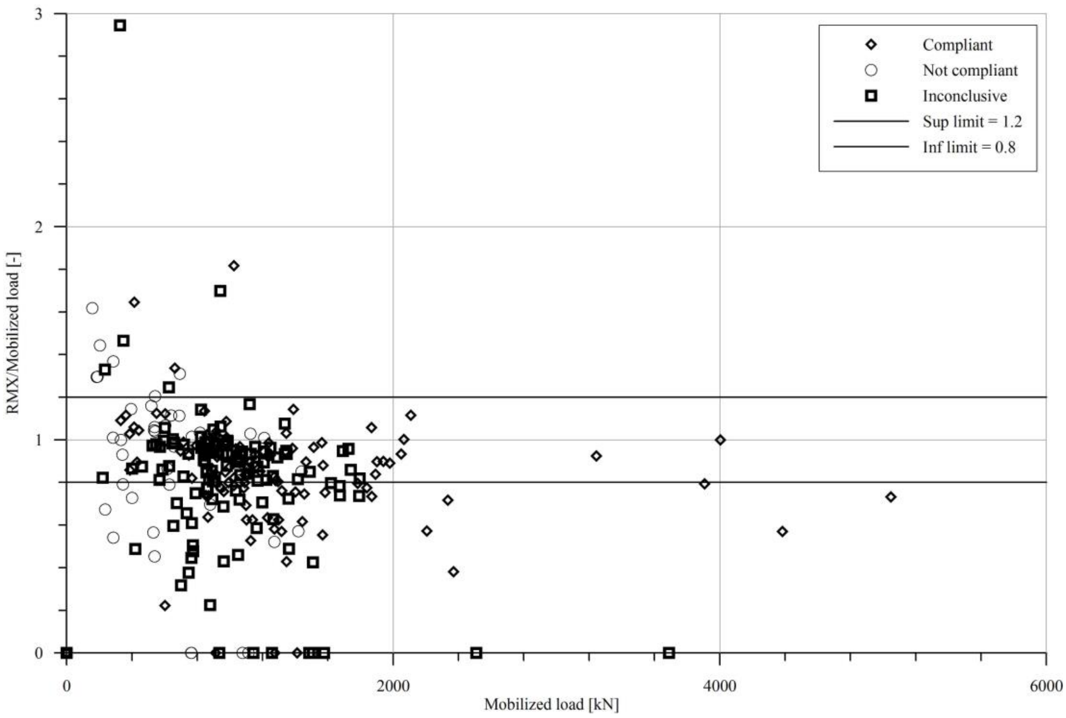

Figure 12 - Relationship between RMX, made dimensionless by dividing it by mobilized load, and mobilized load for micropiles. Copyright Erika Prina Howald.

4.3.3 Relationship between match and mobilized load

The following figure (*Figure 13*) shows the relationship between Match and mobilized load by entering the threshold value Match=5 for discerning the quality of the overlap between measured and calculated waves.

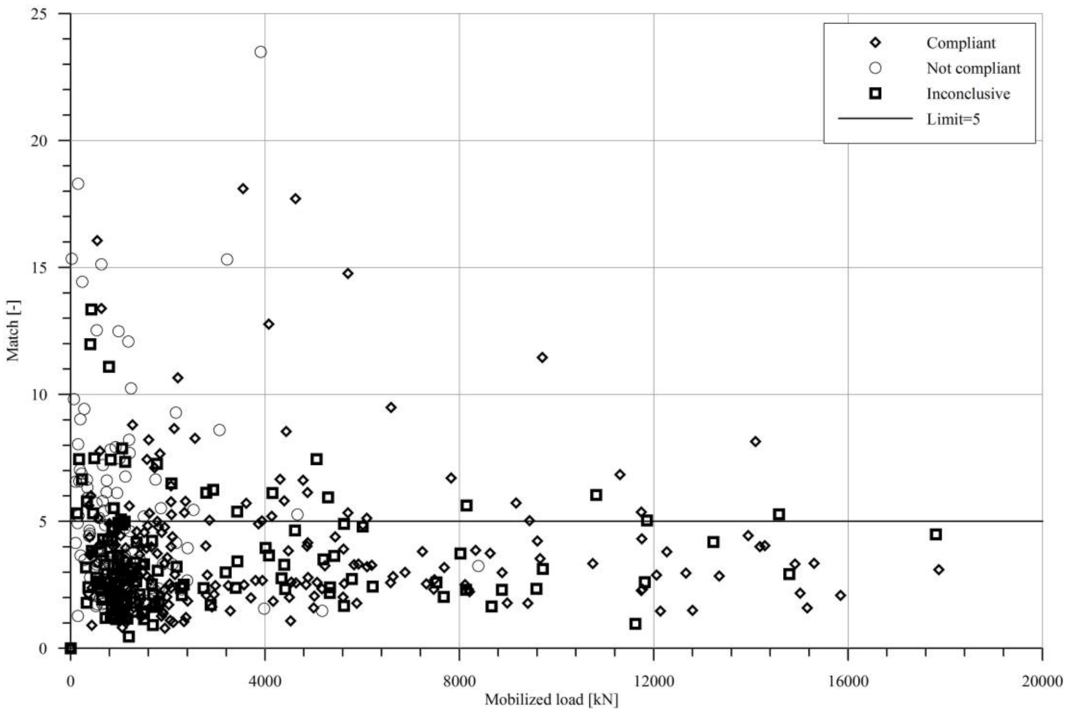

Figure 13 - Relationship between match and mobilized load for all types of piles. Copyright Erika Prina Howald.

The diagram allows three observations: the success of the match cannot be correlated to the compliance of the pile; most of the tested piles have obtained a good match between the measured values and calculated values; the value of Match dispersion decreases with the mobilized load increase. In the latter case, piles of greater diameter are considered. Along the stem, a better signal propagates, allowing a more precise calculation through the Capwap.

4.3.4 Relationship between RMX and sinking

In the following diagrams, all the blows exerted on piles were considered, with regard to the value of *RMX*. In order to compare piles with very different resistance, the value of *RMX* was made dimensionless by dividing it by RMX_{cc} value. Accordingly, the value of RMX_{cc} is worth one. In general, the highest value of *RMX* defines the blow chosen for the Capwap analysis. However, this is not the case for all the piles, which is why there are values higher than one for this ratio on the ordinate axis. *Figure 14* represents the trend for driven piles, *Figure 15* that for drilled shaft piles, and *Figure 16* that for the micropiles.

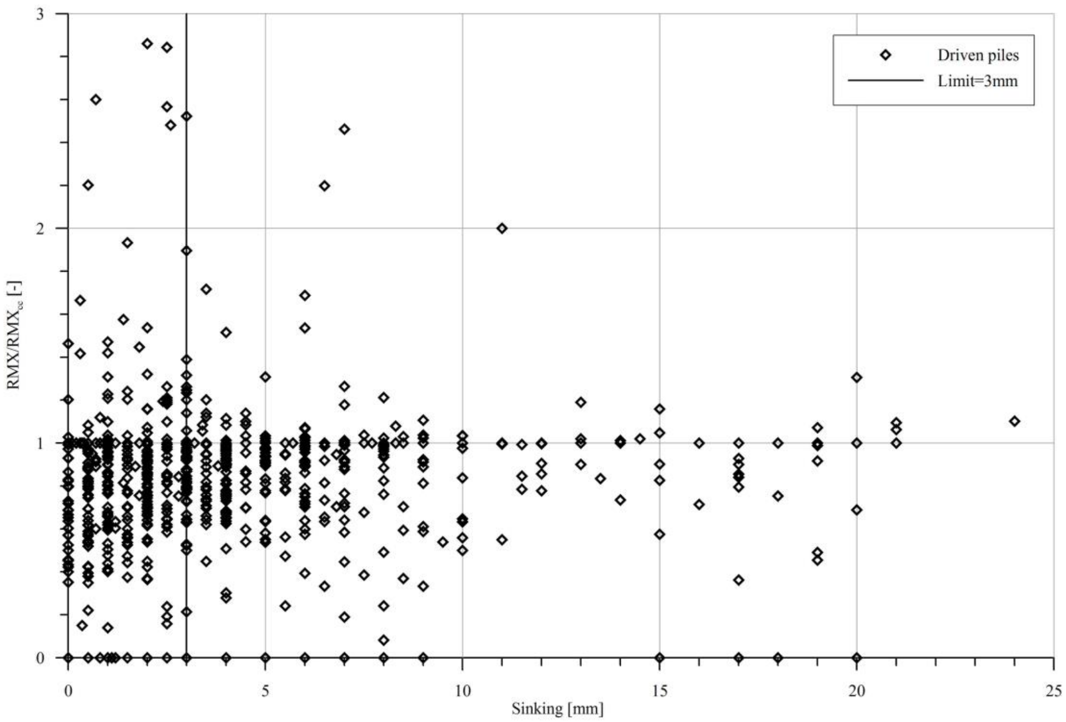

Figure 14 - Relationship between RMX, made dimensionless by dividing it by RMXcc, and sinking for driven piles. Copyright Erika Prina Howald.

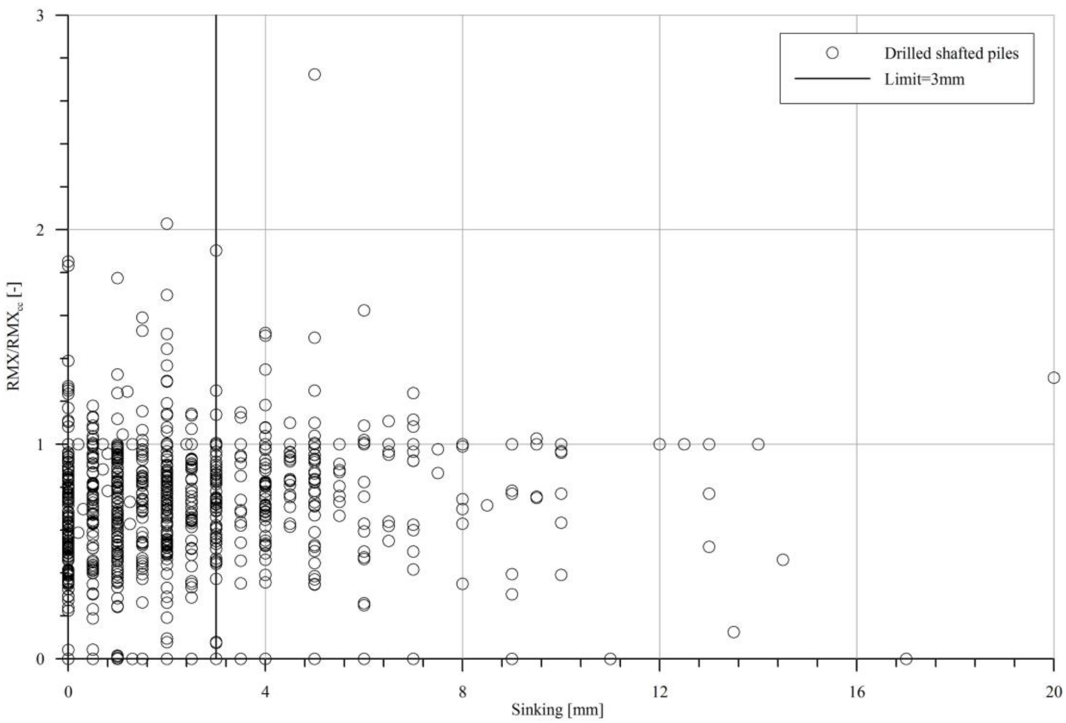

Figure 15 - Relationship between RMX, made dimensionless by dividing it by RMXcc, and sinking for drilled shaft piles. Copyright Erika Prina Howald.

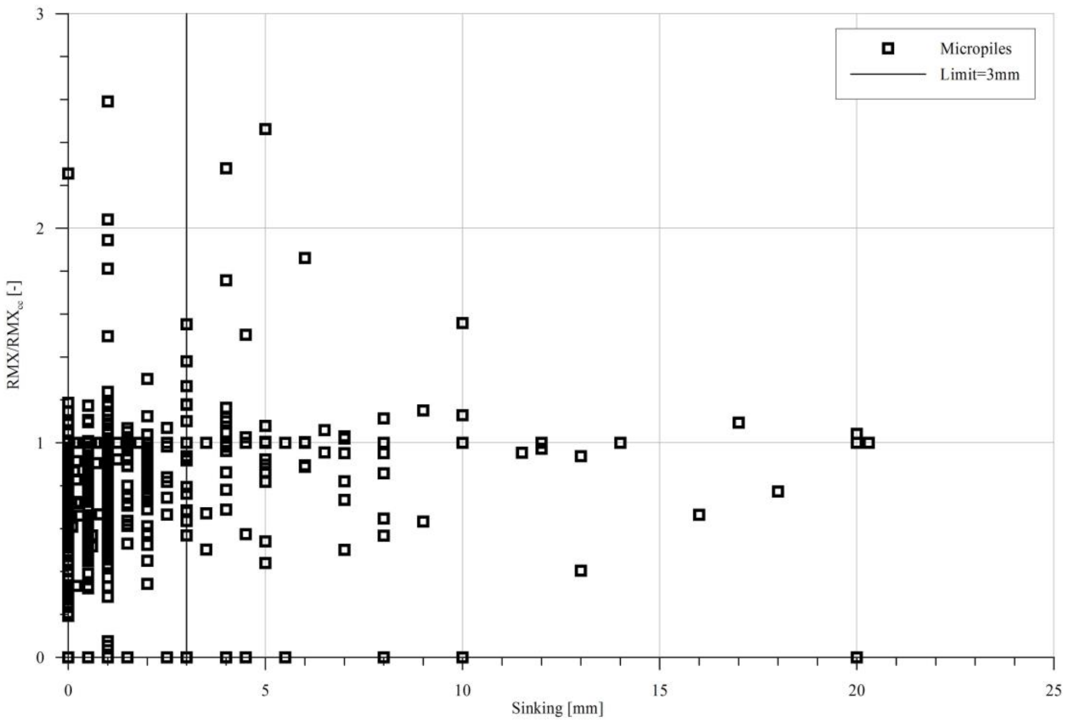

Figure 16 - Relationship between RMX, made dimensionless by dividing it by RMXcc, and sinking for micropiles. Copyright Erika Prina Howald.

The trend is fairly uniform in all three cases, showing a decrease in the dispersion of the points as the sinking increases. In particular, we note that starting from a value of sinking next to 3 mm, the gain on the calculated resistance value becomes minimum. While for the driven and drilled shaft piles this trend is gradual, for micropiles it is possible to note a sharp decrease of the dispersion before the sinking value of 3 mm. This result leads one to consider that a lower sinking is sufficient to mobilize the load of this type of piles.

4.3.5 Relationship between RMX and EMX

The following diagrams show the evolution of calculated resistance (*RMX*) as a function of the energy transmitted by the blow of the hammer (*EMX*), for the three types of piles considered: driven piles (*Figure 17*), drilled shaft piles (*Figure 18*) and micropiles (*Figure 19*). The *RMX* is made dimensionless by dividing it by the resistance value of blow chosen for the analysis.

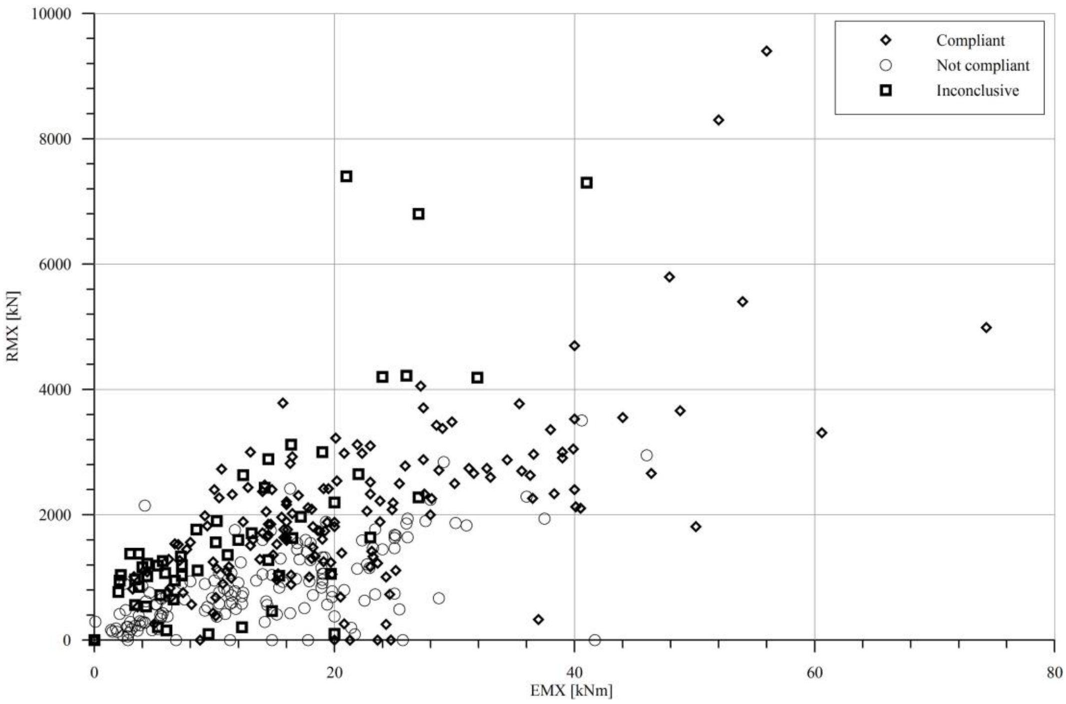

Figure 17 - Relationship between RMX and EMX for driven piles. Copyright Erika Prina Howald.

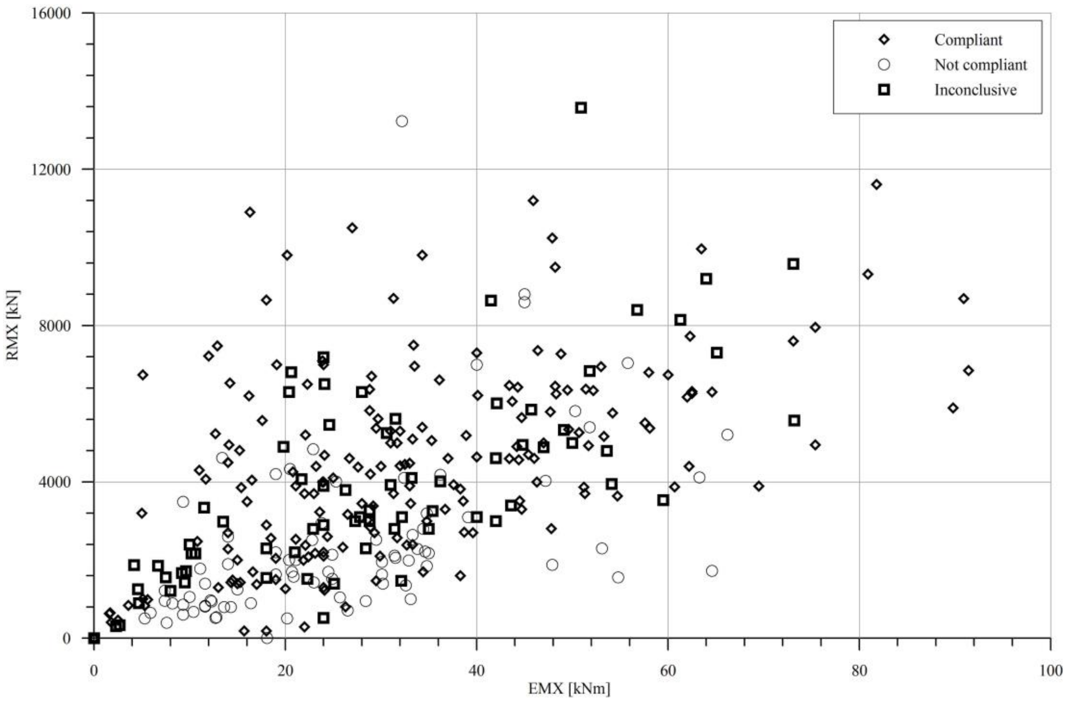

Figure 18 - Relationship between RMX and EMX for drilled shaft piles. Copyright Erika Prina Howald.

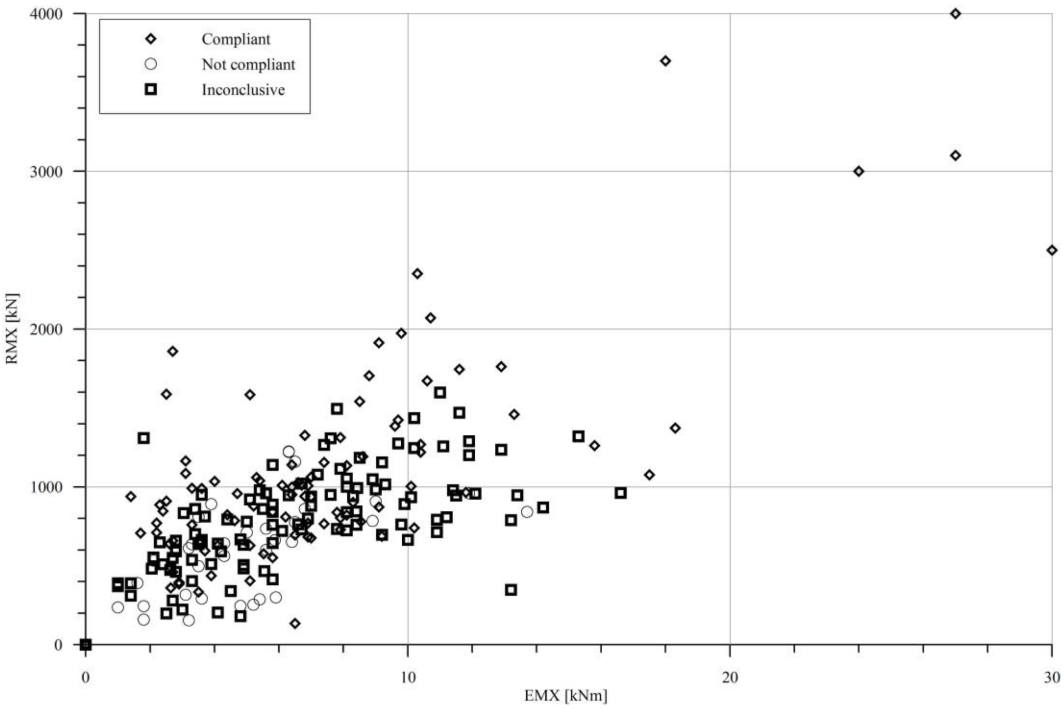

Figure 19 - Relationship between RMX and EMX for micropiles. Copyright Erika Prina Howald.

While for the driven piles it is difficult to recognize a particular tendency, in the case of drilled shaft piles and especially of micropiles, it is evident that, beyond a certain value of energy, there is not a gain in pile resistance at energy increase. This leads one to consider the possibility of placing an upper limit on the energy used during the tests, a value to be studied with a test campaign.

4.3.6 Comparison between static and dynamic test

Although there are not many values of bearing capacity obtained with static tests in the database, the results available are given below (*Figure 20*).

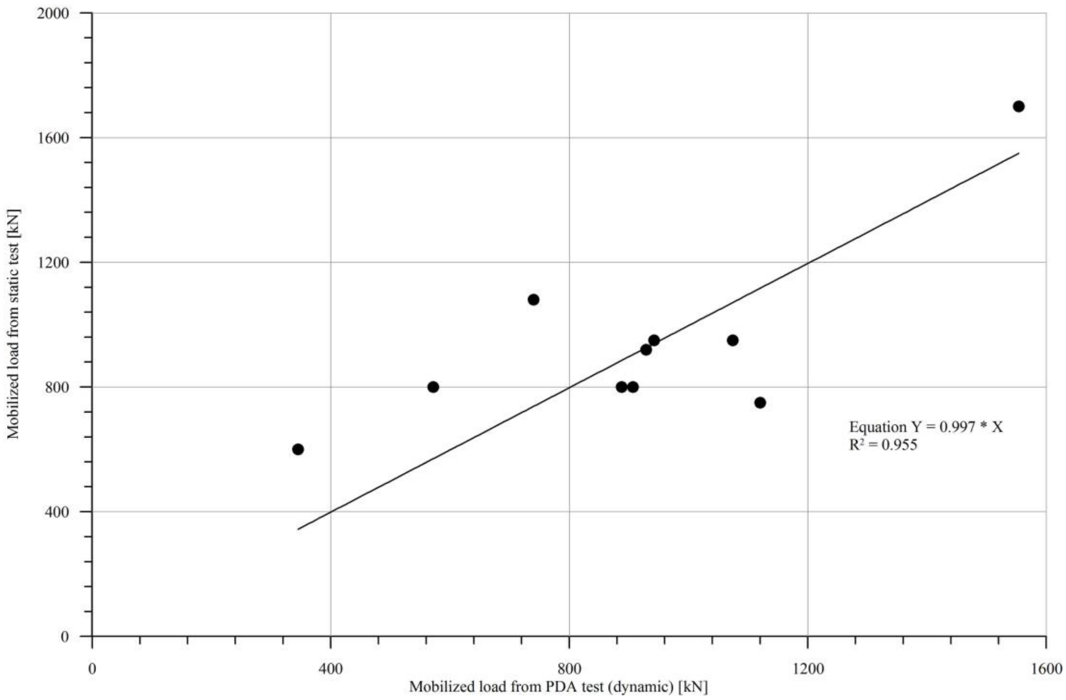

Figure 20 - Comparison between mobilized load from static and dynamic test. Copyright Erika Prina Howald.

Firstly, it is interesting to note that the regression line has a slope close to one, which implies there is good agreement between the values derived from the two tests. There is a certain dispersion of data, however, more than half of the piles presents a deviation from the regression value less than 15%, while 70% of the tests has a deviation within 30%. It is, however, not possible to establish a trend, since the points are evenly distributed above and below the regression line, id est they have values of bearing capacity deduced from the PDA tests sometimes higher and sometimes lower than those raised with static tests.

5. MICROPILES

From the classification of piles in the categories compliant, not compliant, inconclusive, we note that the percentage of micropiles test, from which it is not possible to make certain claims, it is far more important than for other types of piles. This evidence means that the micropiles' resistance is not fully mobilized due to the insufficient sinking during the test. However, it is also possible that micropiles have been damaged during the installation or during the test execution.

In most occasions, observed trends in different diagrams show a more or less marked deviation of micropiles results from the other two pile's types. It is, therefore, evident that, in the case of micropiles, further research is necessary to adapt the PDA test to this type of foundations.

One possibility to improve the data obtained from the test would be to draft hammers adapted to micropiles and to their structural characteristics. In these cases, hammer acts on the metal armature that emerges from the ground only and it must exert energy stress on it. It is clear that, unlike the other pile's types, in this case, it is much more difficult to ensure a good contact area for the energy transmission. It is therefore necessary to anchor the hammer along the

stem. As different types of armature for micropiles are available, it will be necessary to provide adapters for each of them.
Since it is much more likely to damage a micropile at the time of the hit and since the signal is very prone to noise, another important precaution is to create a cushioning system that reduces rebounds after hitting.

6. CONCLUSIONS

In this research, data of different piles implemented in very different circumstances over past 20 years in Switzerland were analysed. A database has been created to collect all information available, but it has been impossible to fill in each field for each pile.
The strong point of PDA method is, besides the speed and low cost of execution, the ability to know the trend of the resistances along the pile stem and at the tip, as well as the bearing capacity at the ultimate limit state. Thanks to the Capwap program, it is also possible to simulate the static load test.
As evidenced by the results, the test provides reliable data in the case of driven piles, for which it was designed.
In the case of drilled shaft piles, in order to optimise the test, some aspects should be studied in greater depth.
Regarding micropiles, the test shows good potential. However, it will be necessary further research and to provide a specific adaptation of the test.
Another important aspect for the perpetuation of this type of research is the correlation of data regarding deep foundations with the characteristics of soil in which piles are located. This aspect was not taken into account in this study, since data provided by the company were almost entirely devoid of soil stratigraphic characterization.

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