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Recent advances in pile design, construction, monitoring and testing

Avancées récentes en matière de dimensionnement, d'exécution, de monitoring et d'essais sur pieux

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ABSTRACT: In this contribution an overview is given about the recent evolution in the harmonization of codes and standards that apply for the design of piles, in particular the Eurocode 7. Based on the results of some design exercises, it is shown that differences between the output of the different national approaches remain huge. Several discrepancies of pile design standards and codes are identified.
The authors then give an overview of some recent technical progresses with regard to piling equipment, testing, instrumentation and monitoring methods. They try to analyse how those advances can be taken into account in the codes and design practice. The concrete example of the recent evolution of the Belgian practice is explained, where an attempt is made to reward testing and proper monitoring.

RÉSUMÉ: Dans cette contribution, les auteurs donnent un aperçu des évolutions récentes au niveau de l'harmonisation des codes et normes qui s'appliquent pour le dimensionnement des pieux, en particulier l'Eurocode 7. Sur base d'exercices de dimensionnement, il est démontré que les différences, en termes de résultats obtenus selon les différentes approches nationales, restent énormes. Certaines divergences entre codes et normes sont identifiées.
Les auteurs donnent ensuite un aperçu de quelques progrès techniques récents au niveau des équipements d'installation, des essais, de l'instrumentation et des méthodes de monitoring. Ils essayent d'analyser comment ces avancées peuvent être prises en compte dans la pratique des codes et normes. L'exemple concret de l'évolution récente de la pratique belge, où l'on tente de récompenser les essais et un monitoring approprié, est expliqué.

Keywords: Piles, Design, Execution, Testing, Monitoring

1 INTRODUCTION

Many evolutions took place in Europe over the last decades with regard to pile foundations, in the different areas of design, execution and testing.

Design:

- The Eurocode 7 was implemented throughout Europe, giving a common framework where design methods should fit in. As we discussed during the recent ETC3 Conference in Leuven (2016), this didn't result in a real harmonisation, but at least it helps designers and practitioners in the different countries to better understand design practices in the other countries.

- As a result, more detailed guidelines and prescriptions have been drafted in some countries, whereas others just tried to translate the "old" design rules into the framework of the EC7.

- The second generation of Eurocodes are currently being established with a publication expected in 2022. The aim is a.o. to foster harmonisation in Europe and increase the ease of use. the steel bars.

<u>Execution:</u>

- The last decades have seen a huge evolution with regard to machines capacity resulting in significantly deeper and larger foundation elements than before.

- Innovations in equipment and larger dissemination of information have created new possibilities but time to market has shortened so that detailed knowledge of the systems sometimes comes after their use in the field.

- Piling rigs are now equipped with all kind of electronic measurement devices, allowing for better information over the installation parameters and even offering interactive ways to automate installation procedure. Interconnectivity with BIM systems will be on the agenda of the coming years.

- The industry constantly requests increasing loads and higher strength materials. In the same time, recent research on materials such as tremie concrete, support fluids or Soilmix have shown how adequate material selection, prescriptions and testing is part of a successful installation process.

<u>Testing:</u>

- Because of the standardization exercise required by the introduction of the EC7 and facilitated by the easier access to miniaturized and accurate testing equipment, instrumented load tests have been more widely adopted giving better understanding of pile behaviour.

- In the same time, as expressed earlier, advances in instrumentation equipment also give new insights with regard to pile performance although the use of the provided information is currently usually only limited to the guarantee of a better pile documentation.

Based on what precedes, one might expect that theoretical advances and progress in norms and codes have made it easier to predict the pile's performance in given circumstances. However, as we will demonstrate, the scatter between different theoretical approaches remains very important, and still, the major factors determining real pile performance are the correct knowledge of local soil conditions, and the understanding of the exact impact of the pile installation in these particular soil conditions. Contractor's proven skill, repeatable process and previous experience is key in the final result. As systems and equipment capacities evolve, it is important to link expected pile capacity to measured pile performance. Modern testing and monitoring equipment can help to make this effective in the field.

A first section will be dedicated to the review of the remaining discrepancies in codes. A second section will give a few examples of recent technological advances and though outstanding issues. In a third chapter, we will look at the progresses which were made in the field of testing. And finally we will try to look at the main question we have: how can we improve the fit between predicted capacity using the codes and the installed capacity in the field.

2 CODES AND EVOLUTION OF DESIGN PRACTICE

2.1 The purpose of codes

In recent years, under the pressure of a stricter regulation and contractual base of our profession, new or revised codes have been drafted. Eurocodes, in particular, have been forcing many

European countries to have a fresh look on their local regulations. They have also been serving as a reference for many other regions in the world.

Codes are obviously important, because they are supposed to (Peter Day, 2017):

- Establish the norms of the profession, and provide protection against legal action based on negligence,
- Represent a distillation of existing knowledge on which there is consensus,
- Should ensure fair competition.

On the other hand, many practitioners dislike them as they present the risk to refrain innovation and inhibit engineering judgement.

As we all like to say, “the practice of geotechnical engineering is a skill rather than a science. It involves perception and judgement, both of which are difficult to encapsulate in a code”, (Peter Day, 2017). It is largely based on experience, a correct understanding of the basics behind it, and local knowledge. One of the main dangers is that the codes give the illusion of clarity and standardization. The code user’s, however, can:

- miss practical knowledge of the adequate construction method in specific circumstances,
- base his interpretation on insufficient or inadequate data,
- have insufficient background for a correct interpretation of specific requirements or prescriptions.

2.2 The accuracy of codes

2.2.1

During the International Symposium organized by the European Technical Committee 3 Piles of the ISSMGE (ETC3) in Leuven “Design of Piles in Europe-How did Eurocode 7 change daily practice”, (De Vos et al., 2016), the members of the different countries were invited to submit National reports on the design of piles in their countries, since the introduction of the EC7. Three design examples were also distributed to the members, asking them to provide solutions for these “simple” and well documented examples involving the design of driven, bored, screw, and CFA piles in different ground conditions. Trevor Orr accepted to analyse and compare the different solutions received. We show hereunder the details of two cases, bored piles and screw piles related to Example 2, in clayey soil.

Example 2 concerned the design of twenty piles for a stiff building in Belgium as part of a Belgian Building Research Institute (BBRI) research project on a site 36 m x 18 m. Two types of piles were to be considered: 900 mm bored piles with bentonite suspension and a temporary casing over the first 3 m, and 410 mm displacement (no soil excavation) screw piles. The soil consists of 1.0 m of fill over a deep deposit of stiff over-consolidated Boom clay. No additional surface load was to be considered and hence no downdrag.

The ground investigation involved three CPT, two SPT and two pressuremeter (PMT) tests well distributed over the site. Boring with undisturbed soil sampling and laboratory tests were carried out centrally on the site. The CPT qc and SPT N values are plotted in Fig. 1. The results of the triaxial tests gave a c’ value of 22 kPa and a φ’ value of 28.2°.The groundwater was at a depth of 1.0 m.

The objective of this example was to predict the compressive resistance of the bored pile and the screw pile, stating what ground investigation information was used. Ten solutions were transmitted for the bored pile and eight for the screw piles. Seven of the ten solutions were based on the results of the three CPT tests. Figures 2a and 2b give the results for both examples.

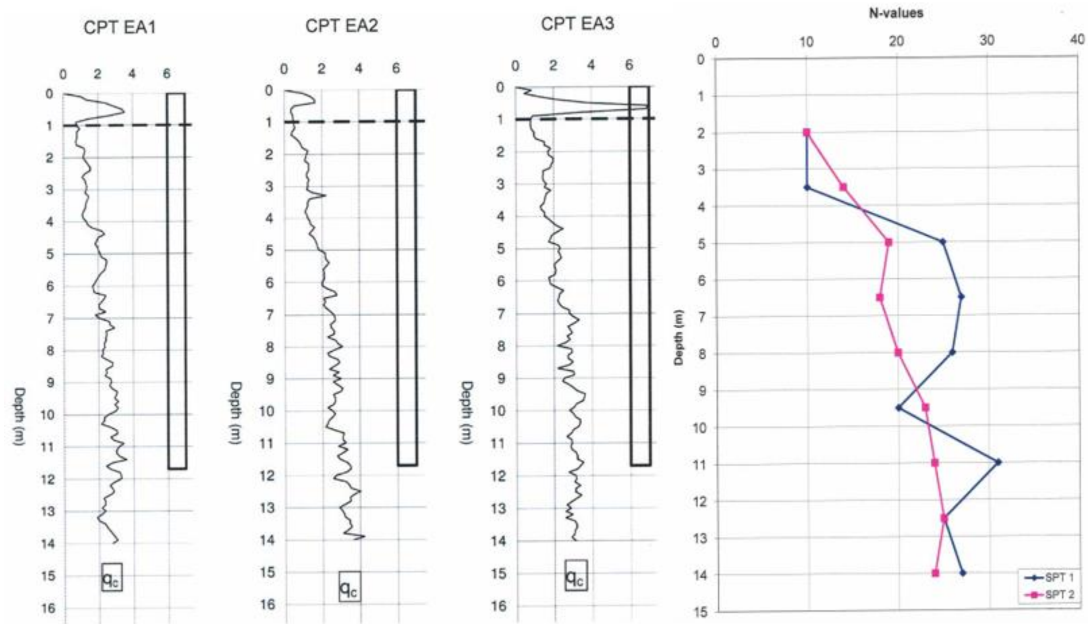


Figure 1. Example 2 (Bored and screw piles) – CPT and SPT plots

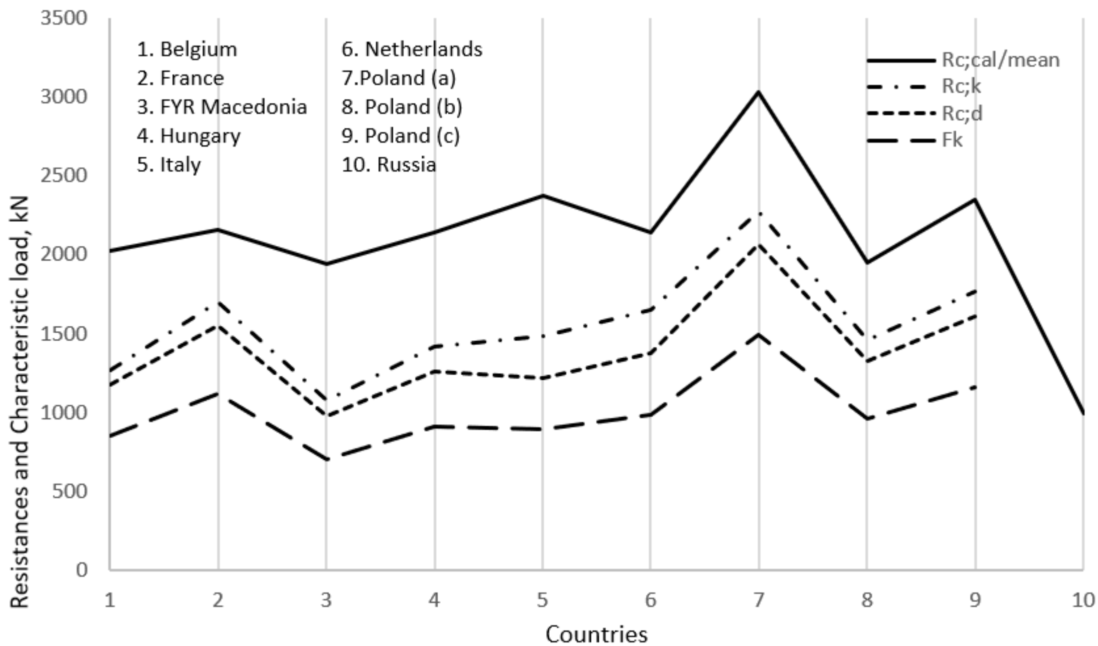


Figure 2a. Example 2 – Bored pile (T. Orr, 2016) – Resistances and characteristics loads

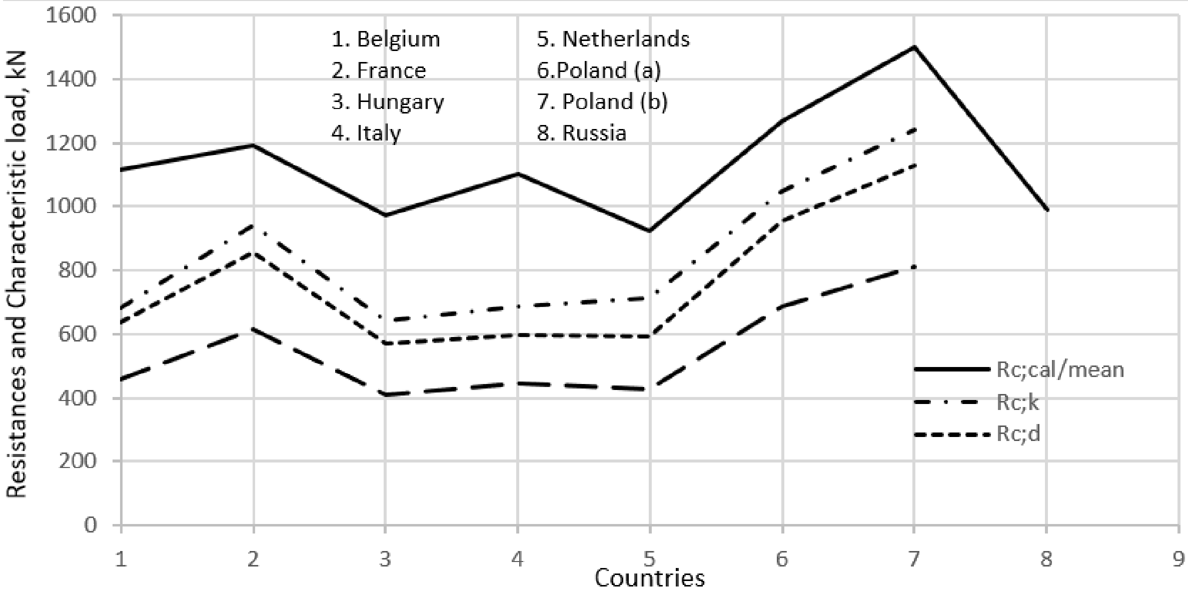


Figure 2b. Example 2 – Screw pile (T. Orr, 2016) – Resistances and characteristics loads

The graphs in Fig. 2a and 2b show that the calculated pile resistances and resulting characteristic loads are very variable, with coefficients of variation of 14% for the calculated resistance and a COV value of 22% for both the characteristic resistance and the characteristic load. The highest resistance was obtained by Poland using laboratory test data while the lowest value was obtained by Russia.

The OFS value (overall safety factor) is close to 1.5 in the case of all the solutions and theTSF values (total safety factors) range from 1.9 to 2.7.

The further analysis of all the design examples showed that (according to T.Orr, 2016):

- Harmonisation has occurred in the design method and consistent overall factors of safety are used. The outcome of the design, however, is far from being similar.

- Since Eurocode 7 does not provide any calculation models for pile design, different methods, mostly presented in national standards or guides, are used to calculate the pile resistances. Many of the national methods to calculate pile resistance involve additional factors or requirements to those in Eurocode 7, for example factors relating to pile type or pile shape.

- For more harmonisation to occur, there must be more agreement on the models to calculate pile resistance, but is that possible considering the different soil conditions, testing methods, pile types, installation methods and experiences that exist in Europe.

This last conclusion is not certain. In his conclusion, T.Orr also refers to a comparison exercise for retaining walls at an International Workshop on the Evaluation of the Eurocode 7 in Dublin (2005) by Simpson, showing again a considerable scatter in the results, even when calculated by authors using nominally the same method! This obviously brings us back to the fundamental comments raised under point 2.1.

2.2.2

Another interesting comparison exercise was organised in Bolivia by Bengt Fellenius and Mario Terceros as part of a prediction event related to a research study on construction and static and dynamic testing of four instrumented bored and screw piles (DFI Stockholm, 2014).

Geotechnical studies comprising SPT,CPT and tests with routine laboratory analysis on recovered soil samples were performed. CPTU soundings were performed at a later stage and were not available for the participants in the prediction event.

Figures 3a and 3b show diagrams of the SPT N-indices and the distribution of water content in the three boreholes, as well as the CPTU diagrams from CPT-1; measured cone stress, sleeve friction, and pore pressure on the cone shoulder (U2), and calculated friction ratio. As one can see from the SI data, the soil density is compact. The average water content is about 15 % plus-minus a few percent.

The test piles were bored and screw piles described as follows (see Fig. 4).

TP1: a nominally 400 mm diameter pile, 17.5 m long, bored under bentonite ("standard pile").

TP2: a nominally 360 mm diameter pile, 11.6 m long, built as a FDP (Full Displacement Pile) which is constructed without removing any soil (but for nearest the ground surface on starting the pile).

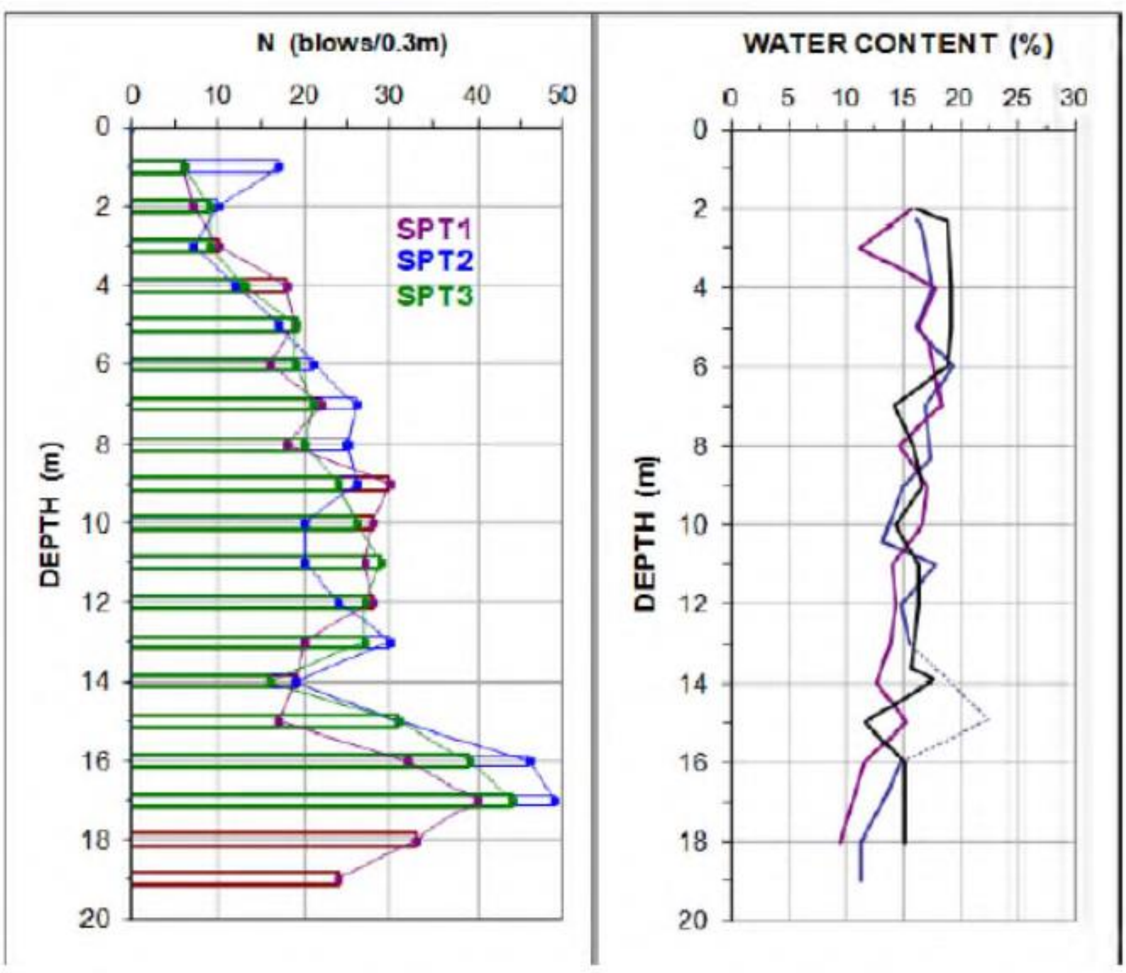


Figure 3a. Bolivian site (Fellenius et al, Stockholm, 2014) SPT N-indices and Water Contents

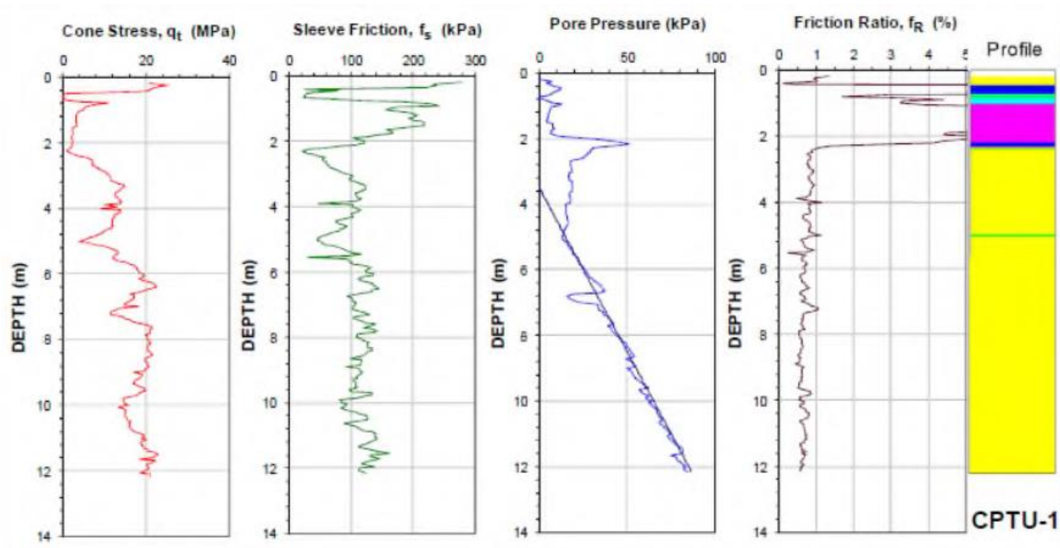


Figure 3b. Bolivian site (Fellenius et al, Stockholm, 2014) CPTU diagrams

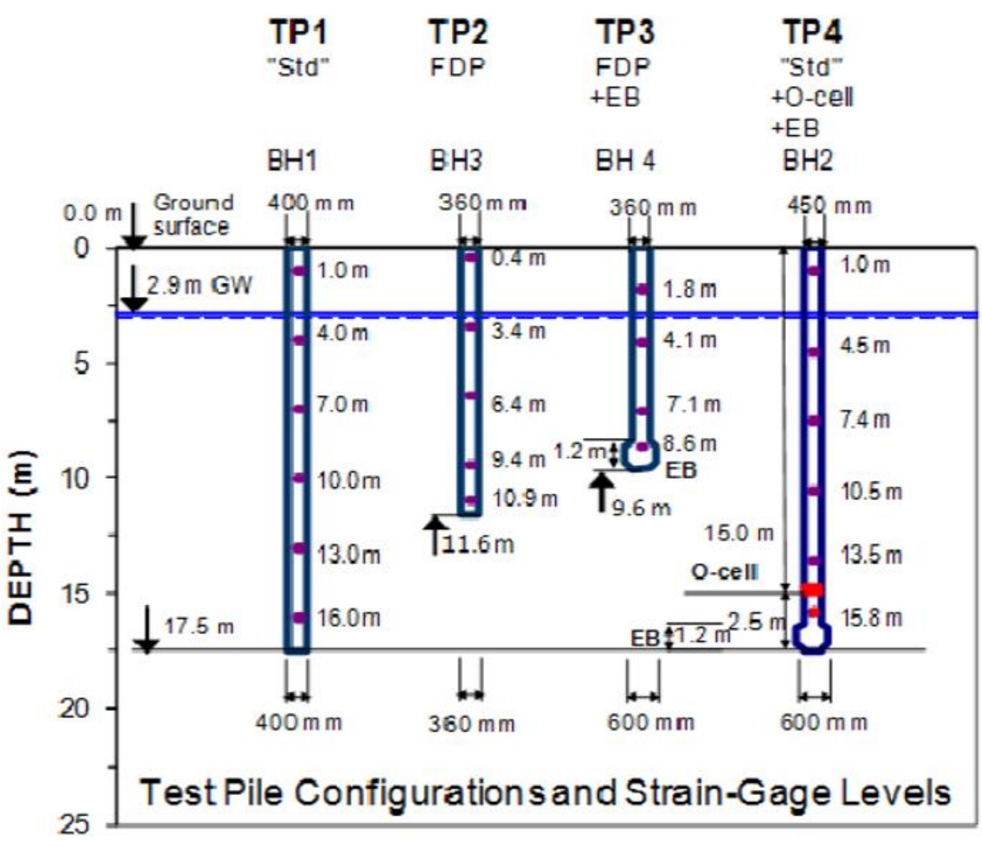


Figure 4. Bolivian site (Fellenius et al, Stockholm, 2014): test setup

TP3: a nominally 360 mm diameter pile, 9.6 m long, built as a FDP (Full Displacement Pile) and with a 600 mm diameter Expander Body, placed at the pile toe.

TP4: a nominally 450 mm diameter pile, 17.5 m long, bored under bentonite and with a 600 mm diameter Expander Body, EB, placed at the pile toe and with an Osterberg cell above the EB.

A total of 50 predictions were received from 63 individuals in 19 countries and all continents. Most predictions only addressed Pile TP1. Figure 5 shows a compilation of the submitted load-movement curves and the evaluated capacities.

Only one of the participants had experience from the Bolivian soil and piling conditions. According to Fellenius et al (2014), it is therefore no surprise that the upper and lower boundaries of the load-movement curves and the capacities are wide apart. They consider more surprising, that the range of pile head movements at the evaluated capacities is even wider.

Fellenius et al (2014): “Most geotechnical engineers would, we believe, accept an allowable load of half a capacity occurring at a movement of about 15 to 20 mm. But would they be equally willing to accept that same allowable load determined from a capacity that took 50+ mm movement to develop? Indeed, the main outcome of the prediction event is that the geotechnical community has very diffuse definitions of capacity as determined from the results of a static loading test. Few text books, guidelines, codes, and standards, if any, define how to determine the capacity that serves as reference to the proclaimed factors of safety or resistance factors, often with two-decimals precision. We consider this to be a definite weakness in the geotechnical practice.”

We could not have said it better.

2.2.3

When looking at these two examples, and many others published in the literature, we need to address the reasons of such a large variation:

- Codes sometimes miss a clear definition and approach to what is meant by ultimate/allowable/characteristic pile resistance. We believe this is a major achievement of EC7, that these concepts are usually better understood, at least by European practitioners.
- The distinction between the installation effect of a specific piling system and the reliability of the pile installation process is not always clear, and sometimes (large) global factors account for both.
- Calculation methods in codes are often based on empirical data (load-tests) which were performed decades ago. In many cases, insufficient data is (locally) available for adequate calibration of installation/model/safety factors resulting in a varying degree of sophistication/conservatism of the code and/or the calculation method.
- The correct understanding of the specific pile behaviour and the measured impact of the pile installation on the surrounding soil may be insufficient. Even more, the variation in pile equipment capacity and local construction practice can affect the (local) pile performance and result in irrelevant installation coefficients for other soils and/or other practices. This is why in-situ monitoring and testing is so important.

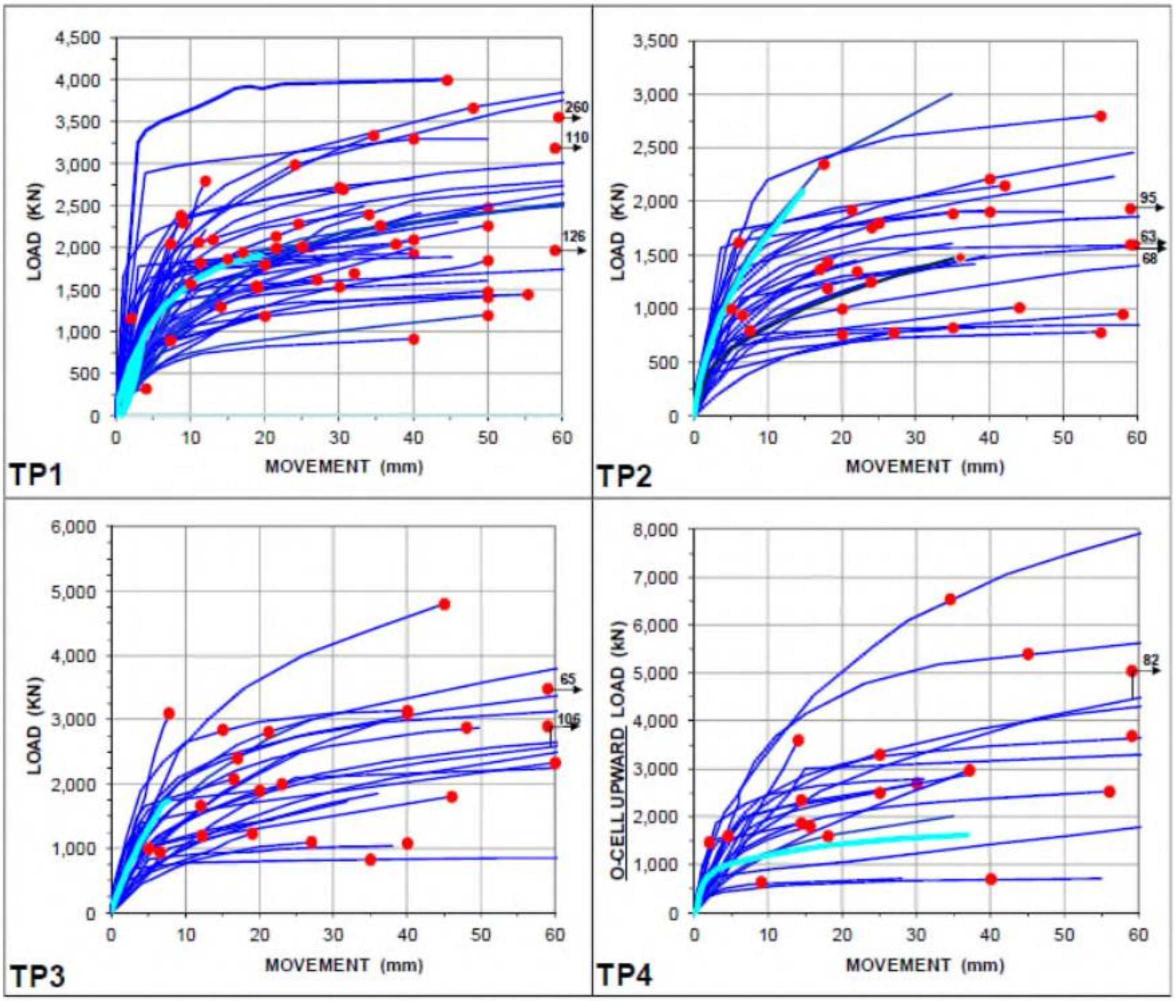


Figure 5. Bolivian site (Fellenius et al, Stockholm, 2014) - Piles TP1 - TP4 Capacities and Predicted Load-Movement Curves - Light Blue Curves are from the actual static loading tests

2.3 Codes and pile construction

We have shown how the simple definition of the theoretical pile capacity can differ in function of the code, the calculation method, the local experience or even the skill of the designer. The problem is: there is more, the installed pile capacity can largely vary as well! In the next chapters, we will try to show how system details and local circumstances or variable soil characteristics can alter the installed pile capacity.

In nearly all the models and codes, the effect of the pile execution on the stress state of the soil is accounted for by introducing a specific installation factor or a reduced fraction of the available shear and base resistance. Bored piles and CFA piles are often considered as less reliable than driven piles and affected by lower installation factors which can sometimes be very low.

It is the authors' opinion that the problem is more complex than usually addressed in the codes (Bottiau, 2014; Day, 2017):

- Influence of the pile installation and reliability of the pile installation should be treated separately, as all installation methods may prove to be inadequate in function of the soil conditions, and depending of the level of monitoring and testing of their installation.

- The different aspects governing the pile installation should all be considered: system details, equipment capacity, monitoring during and after pile installation.

- The correct interplay between pile installation and the surrounding soil is of predominant importance. In this respect, soil type exerts a major role. Codes usually limit soil categories to two or three main types of soils: sands, silts and clay, sometimes chalk, and/or weathered rock. In some cases, this classification is totally insufficient because the response of some types of soils to the solicitation of pile installation procedure, can be dramatically different than expected. In these cases, real scale load testing can prove to be the only adequate method to get the truth.

- Systems are in constant evolution, and small details are sometimes changed resulting in major differences. Too often, systems are classified into generic groups without paying enough attention for execution details. Moreover, modern alternative execution techniques (vibro-driving, jetting, grouting,…) are applied more regularly whereas their result in term of bearing capacity is not necessarily well understood.

- New knowledge or developments are only incorporated into codes and standards after they have been proven in practice and generally accepted. In this sense, codes and standards lag behind the introduction of new developments in the industry.

3 RECENT ADVANCES IN (THE UNDERSTANDING OF) DEEP FOUNDATIONS CONSTRUCTION

3.1 Evolution of the piling techniques

Based on a worldwide survey we update regularly (last update in 2016), the figure 6 shows the split between the different techniques used in Europe for pile foundations. It is obvious that new factors and challenges will drive the choice of one or another system, among which environmental considerations (vibrations, noise and/or production of spoil) will play a still larger role. Bored piles or assimilated represent a growing portion of piles installed: 59% in 2016. Equipment evolve dramatically, and quicker, and consequently, our profession pushes its limits further, sometimes reaching the limits of the existing design models. This evolution results in an increasing complexity of systems and execution details, but also in dimensions and ranges of depths or capacities which were never reached before. Moreover, the type of equipment and capacity available locally varies considerably, which can result in an extremely variable interaction with local soil type, and hence with totally different installed capacities. These are all areas which are difficult to account for in codes.

3.2 Examples and recent cases

3.2.1 Large diameter CFA (Augercast) piles

CFA are widely used worldwide. In Europe, they represent 24 % of the piles installed. The type of equipment used varies considerably from one end to the other of the spectrum, making it very difficult to predict the installed capacity. The system also gets acceptance to install larger diameters and longer piles. It becomes current practice to install piles down to 30 m, with diameters of 800 mm or 1000 mm. Soletanche-Bachy recently introduced a rig capable of installing Starsol piles of 50 m with a maximal diameter of 1500 mm (see Fig. 7).

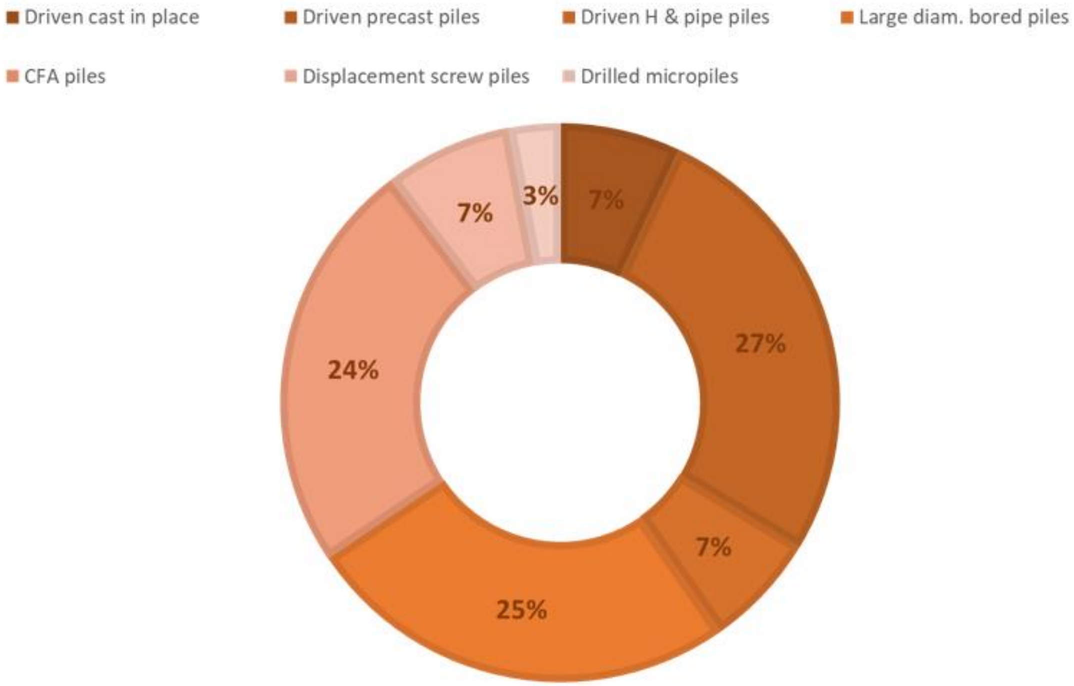


Figure 6. Distribution of piling construction methods across Europe (Bottiau, Leuven, 2016)

Figure 7. Soletanche-Bachy Starsol rig F5000

Even without reaching such records, the installation of CFA (augercast) piles remain a delicate matter, particularly in non-cohesive soils. As stated by many authors (Van Weele, 1988 and 1993; Viggiani, 1993; Van Impe, 1994) and recalled by Bustamante (2003), the complexity of the soil response to drilling is such that the ideal non disturbance conditions of the original soil conditions are far from being fulfilled in practice. The risk of over-augering and loosening of the soil is present, and is much dependent of different parameters, among which the installed power of the equipment plays a predominant role. This is particularly the case with large diameters CFA piles (> 600 mm).

Ideally, the vertical downward speed (potentially enhanced by an additional thrust) should be sufficient in order to avoid the scraping effect (over-augering). In other words, the importance of controlling the screwing ratio is governing the successful installation of augercast piles:

$$SR = \mathrm{n}.\rho/\mathrm{V} \tag{1}$$

where n is the revolution rate of the auger (rpm), ρ is the pitch of the auger (meter p round) and V is the rate of penetration of the auger (m/min).

Viggiani (1993) states that SR should ideally be equal to 1, what means that the auger is screwed in the soil without any soil removal. In real conditions, SR largely exceeds 1 and can even exceed 3, depending on soil and equipment conditions. There are not enough data available on SR values obtained in the field, and no consensus on a target value.

Based on recent advances in monitoring though, it is possible to get a real time value of SR based on the recorded n, p and V. On a recent job in Antwerp, we tried to analyse the SR ratio of large diameter CFA piles in very dense sands, and the correlation with the results of SLT on selected piles. CFA piles of diameter 1000 mm and 1200 mm were to be installed down to 18 m with a penetration of several meters of a sand layer with qc values in excess of 15 MPa (see Fig. 8). Four piles were subject to control Load Tests up to 1.5 x the working load. Control CPT tests were performed at chosen locations after pile installation in order to check the possible decompaction.

The piles were installed using IHC rigs F2800 (piles diam. 1000 mm) and F3500 (piles diam. 1200 mm) with the following characteristics:

- Maximum torque : limited to 300 kN.m
- Maximum rotation speed 20 T/m
- Pull-down capacity : 15 T.

Figure 9 gives SR recorded during installation of test pile TP1 (diam. 1000 mm) and test pile TP2 (diam. 1200 mm). SR ratios of 5 to 7.5 (occasionally 9) were reached. It must be mentioned that very variable SR ratios between 4 and 10 in the dense sand layers were observed throughout the jobsite with no direct relation with local soil condition. Additional CPT's were performed after pile installation showing a decrease in qc-value. Again, this decrease was variable and no direct correlation could be found between local soil conditions or equipment-related information.

During execution, several attempts were made to influence the SR positively, by modifying one or another execution parameter (torque vs rotation or vice-versa; increased pull-down, decreased pull-down).

The results of the PLT are given in Fig. 10, as well as the matching of the αs value (installation factor for the assessment of the friction) used in the Belgian code using hyperbolic functions. The recommended αs value for CFA piles with specific provisions in order to limit soil relaxation is 0.6 in sands.

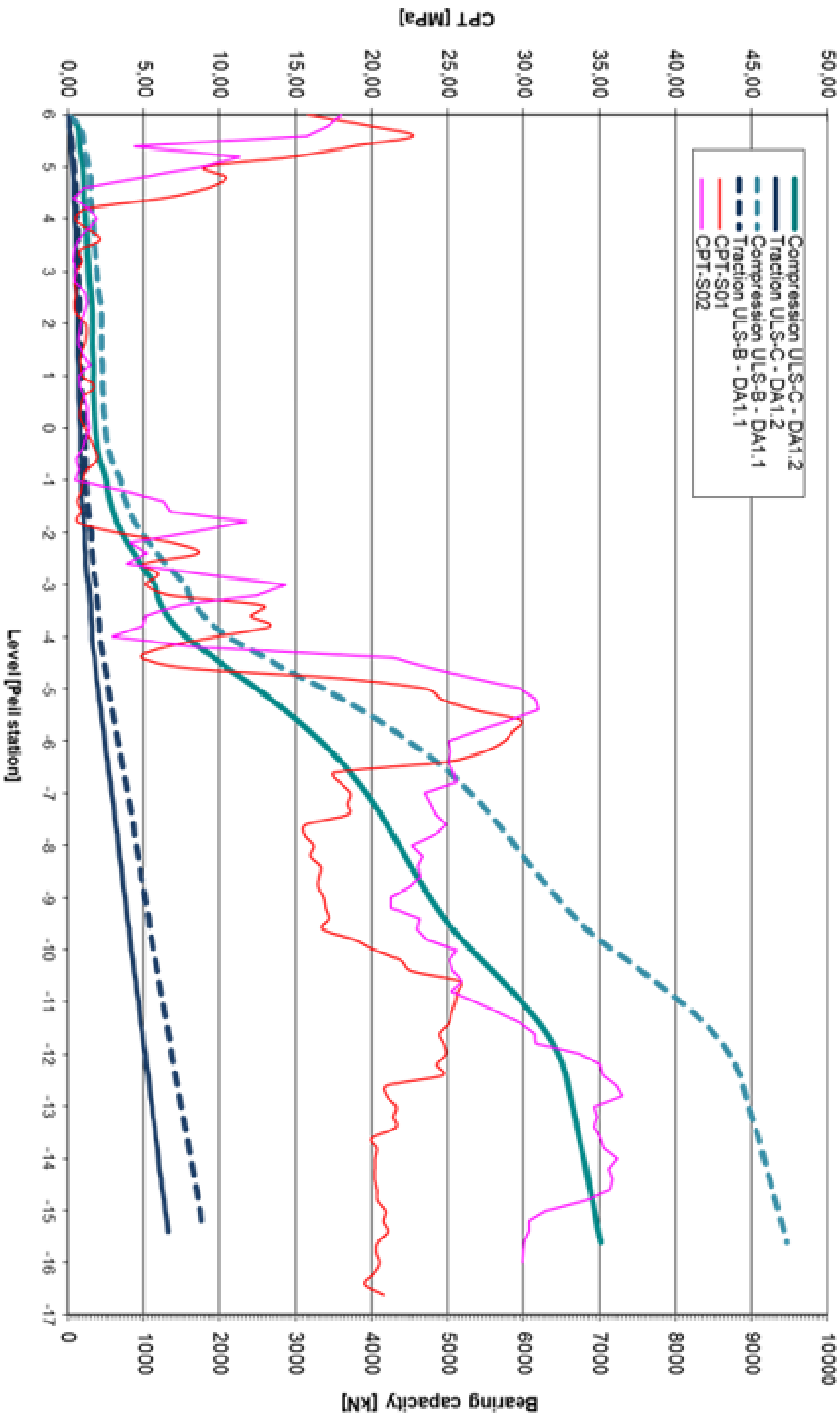


Figure 8. Antwerp jobsite – CPT tests

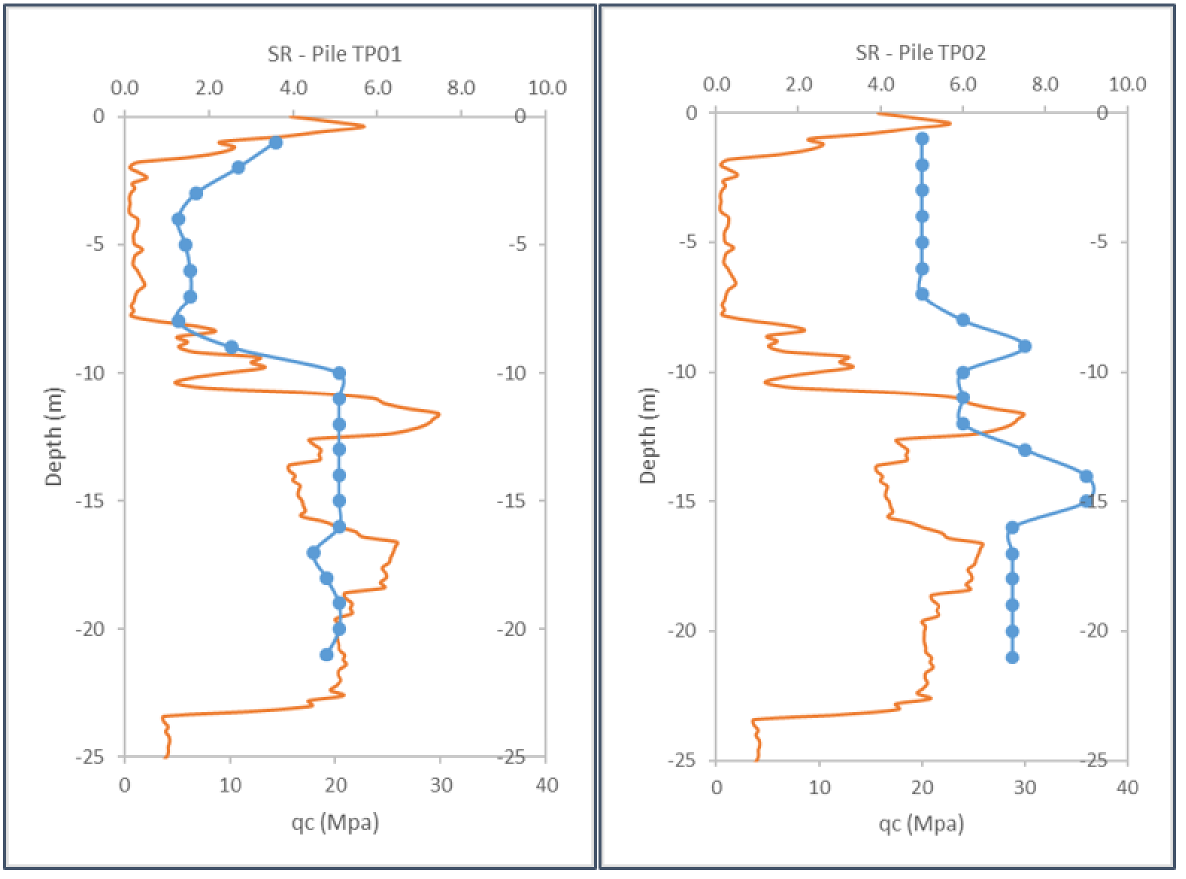
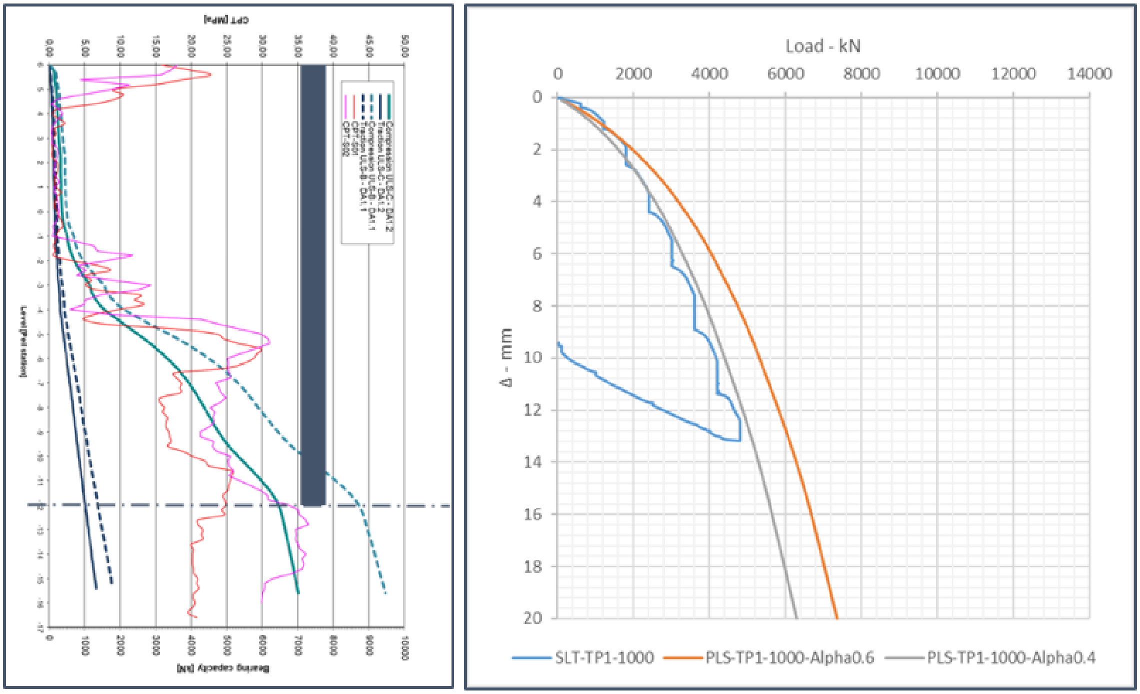
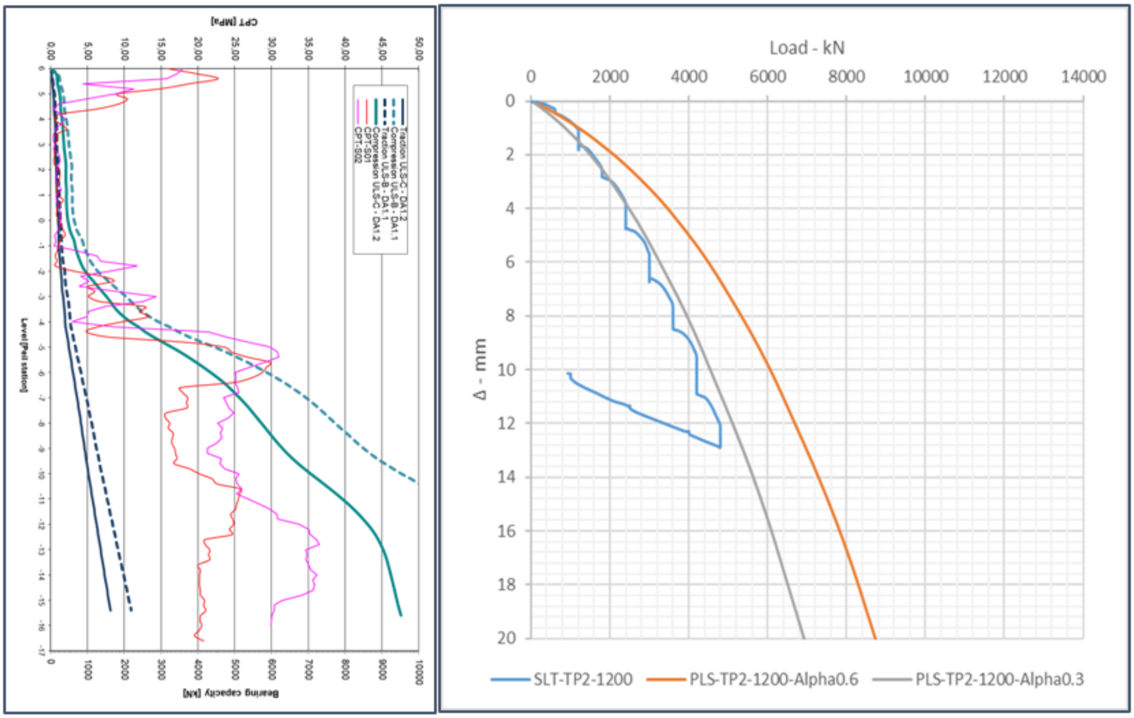


Figure 9. Antwerp jobsite – SR measured for TP1 and TP2



a)



b)

Figure 10. Antwerp jobsite: a) Pile Load Test results for TP1 – diam. 1000 mm; b) Pile Load Test results for TP2 – diam. 1200 mm

The friction values were found as matching the values recommended for bored piles (and not the ones recommended for CFA piles), with a α_s close to 0.4 (TP1-1000 mm) and 0.3 (TP2-1200 mm) which we believe is not surprising considering the diameter of the piles, and the deep penetration into dense sands.

Further research is needed in order to better understand:

- If it is possible to establish a relationship between SR ratio and installation factor in a given soil
- How the diameter of the pile should be taken into account in the assessment of the pile capacity.

3.2.2 Displacement auger (screw) piles

The initial cast in place industrial application of Displacement Auger Piles appeared in the 1960's in Belgium and The Netherlands. The typical alluvial soils in the "low lands" are ideal for this type of pile and to a large extent explain their quick success in the area. To date the capacity and range of piling equipment (available power, torques of the rotary drives, pull downs…) has been increased along with differing auger configurations to allow the application of Displacement Auger Piles in more soil conditions.

Because of the technological evolution, a range of systems, types and labels have emerged (Van Impe (1988, 1989, 1994), Bustamante (1988, 1993), Huybrechts (2001), Basu et al(2010)). The term "Screw Pile" is used at times because the finished cast in place pile takes the shape of a conventional screw at others because the pile is "screwed" into the ground and at others because the pile resembles a large screw. An understanding of systems, types and labels is required.

Today, it is recognized that screw pile performance will depend on the key parameters defining the system:

- Shape of the auger – movement of the spoil
- Shape of the auger – effect on end bearing,
- Shape of the auger – shape of the final pile,
- Power of the piling rig to Rotate,
- Power of the piling rig to Push – force auger penetration,
- Casting method (pressure) of the concrete placement,
- Control of auger extraction – effect on the shape of the final pile.

A combination of the above governs the reliability and repeatability of the pile construction.

Screw piles (Displacement auger piles) can benefit of very interesting installation factors. Extensive research has been conducted in Belgium in the early years 2000, on the main systems used at that time. This research can be viewed as pioneering as screw piles are extensively used in Belgium, which gathers probably the largest experiences in this field. One can in this respect refer to the proceedings of the two Symposia on Screw piles, Brussels, (Holeyman, 2001) and Maertens and Huybrechts (2003). These campaigns and previous research illustrated some particularities associated with each system, and varying installation factors ranging between 0.65 and more than 1.00. The major differences can be observed for the end-bearing resistance. Finally, a global installation factor was prudently adopted in the Belgian code.

In recent years, though, new systems or variations on existing systems have been introduced, sometimes with important differences as shown in Fig. 12, showing a screw auger with an oversized portion of the extraction auger. In the same time, the detailed analyses of recent load tests have been providing ground for a more specific approach, with specific installation factors validated for each system. We will come back on this recent evolution in the last section.

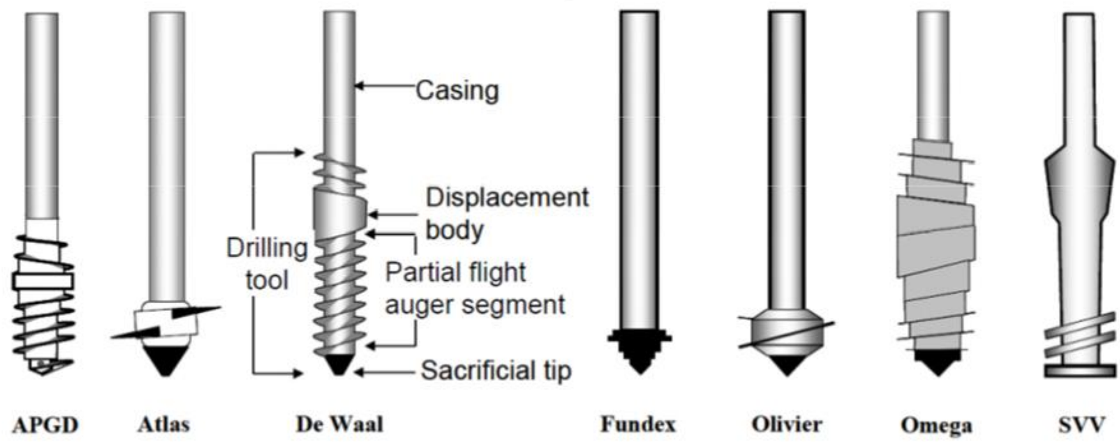


Figure 11. Drilled Displacement Piles – Current Practice and Design (Prasenjit Basu, Monica Prezzi and Dipanjan Basu, 2010)

Figure 12. Screw pile with oversized portion of excavation auger at the bottom (Bottiau, 2014)

In parallel, the use of injection in order to facilitate the penetration of displacement piles into dense sands or alternate loose and dense sands, as is encountered often in The Netherlands is getting more and more popular.

In this system, a tube corresponding to the pile concrete shaft and closed at its bottom by a prefabricated lost steel pile tip, with a diameter of the base diameter to be realized, is screwed in. During screwing-in phase simultaneous grout injection occurs at the bottom of the screw tip, in order to facilitate the penetration. The grout injection is mixed by the screw flanges with the in-situ soil and finally results in a grout shell around the concrete shaft. This shell can reach the outer diameter of the pile equal to the base diameter in sandy soils, although in clayey soils, the final diameter can be less. It is usual, in absence of other information, to take into account an intermediate diameter between the temporary tube and the pile tip.

This piling system proves to be very effective in difficult soil conditions. Recent applications in The Netherlands led to the application of larger dimensions reaching a combined diameter of more than 800 mm.

The exact effect of the grout injection, though, is not fully evaluated. For his reason, on major projects, SLTs are often required in order to control both the applicability and the pile capacity. This was the case on the jobsite in Brussels for the Railway Authorities, where displacement piles of diameter 540/660 had to be installed into relatively dense sands at a depth of 16.40 m.

Figure 13 illustrates the installation of the test piles and the results of a CPTE that was carried out in the axis of the test pile.

One fully instrumented SLT load test was performed by the BBRI. Instrumentation was installed after pile installation by integrating fibre optic extensometer sensors in reservation tubes that had been attached to the reinforcement cages (see Fig. 14).

The test load had been defined at 4800 kN, corresponding to 1.2 times the geotechnical bearing capacity of 4000 kN, (pile base settlement = 10% pile base diameter), which was estimated with the principles of Belgian design methodology according to EC7.

Figures 15 to 17 show the test results.

Results were very satisfactory showing a very stiff behaviour of the pile, certainly up to the predicted pile capacity. The test proved that, in the given soil conditions, the installation factors of DA piles could be safely used for the design of DA-piles with grout injection.

While, as mentioned before, a lot of experience exists in the low lands with displacement auger piles in their typical alluvial soils, the experience in particular soil as chalk e.g. is limited.

For a recent job site in the Mons area in Belgium, the pile foundation of a building needed to be installed at a relative large depth in order to obtain sufficient resistance for the high building loads. Several piles of the foundation had therefore to penetrate a chalk layer that is encountered on the site at a quite constant absolute level of ± 11.8 m TAW, which corresponds with a depth of about 18.5 m below soil surface level. A typical CPTE, illustrating the soil profile at the test location, is given in Fig. 18. The chalk layer shows in general cone resistance values between 3 and 10 MPa, with occasionally some higher peak values. The chalk layer is overlaid with sand, soft clay and peat layers with a variable thickness at different locations on the site.

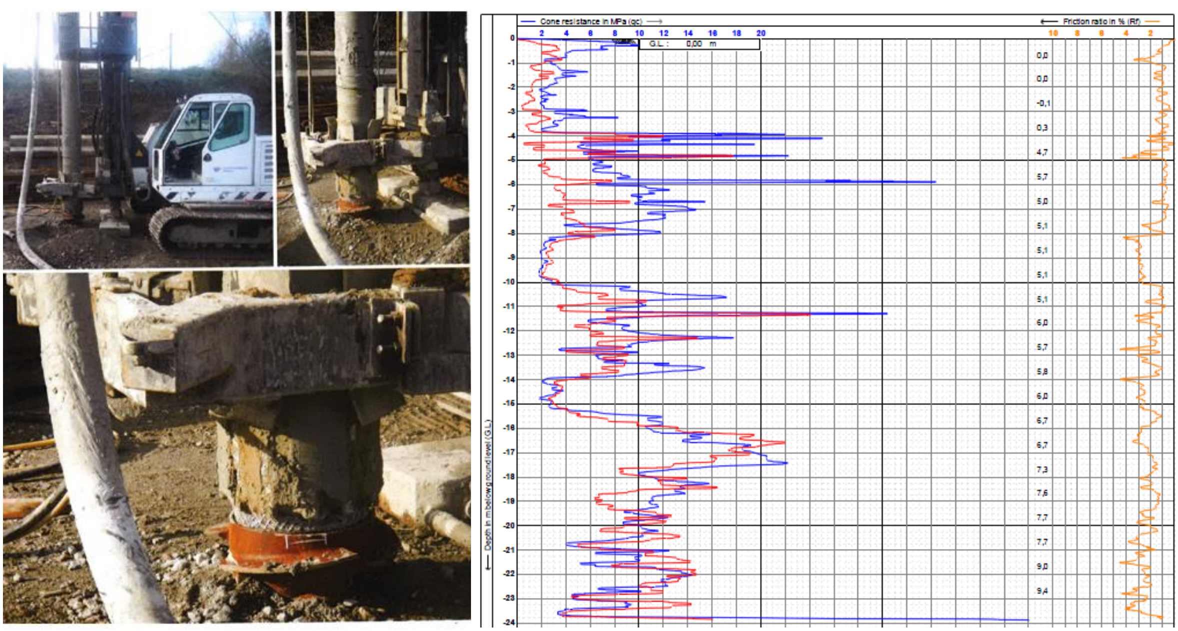


Figure 13. Haren test site - Illustration of the execution of a DA pile with grout injection (left) and results of the CPTE performed in the axis of the test pile

Figure 14. Haren test site - Illustration of the reinforcement cage provide with reservation tubes for the extensometer system

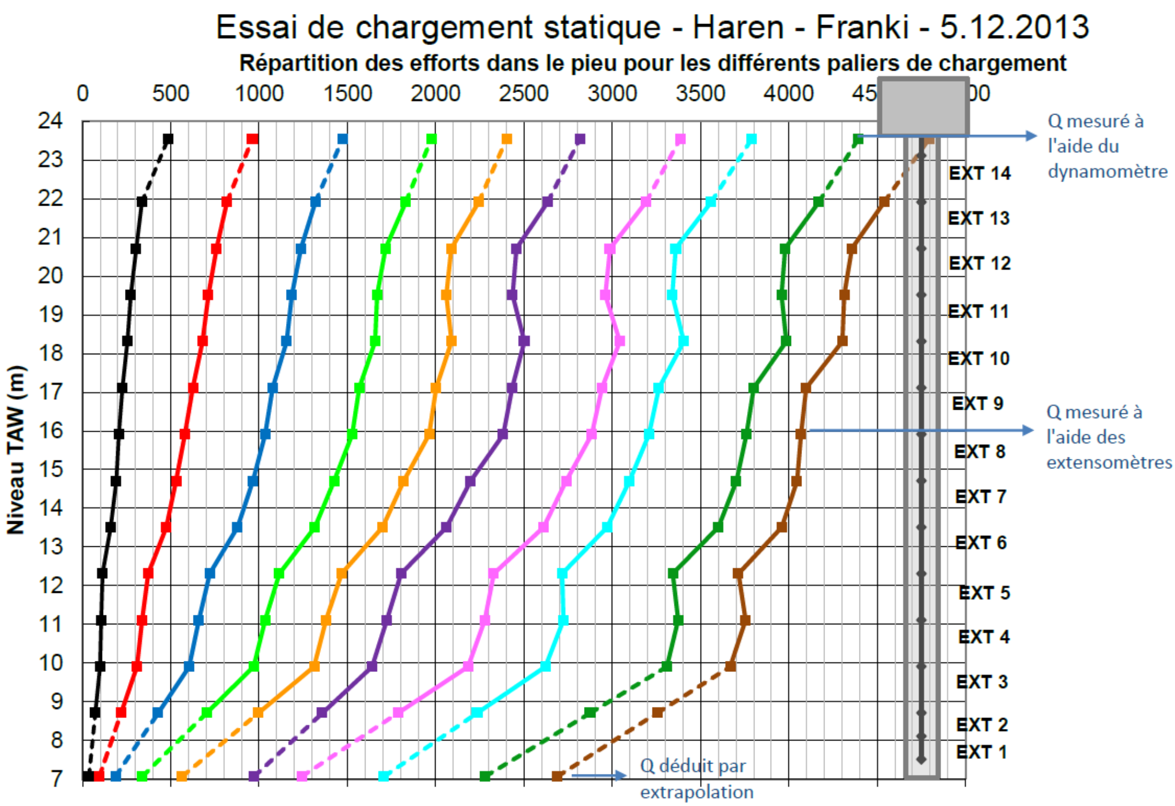


Figure 15. Haren test site - Normal load distribution in the test pile

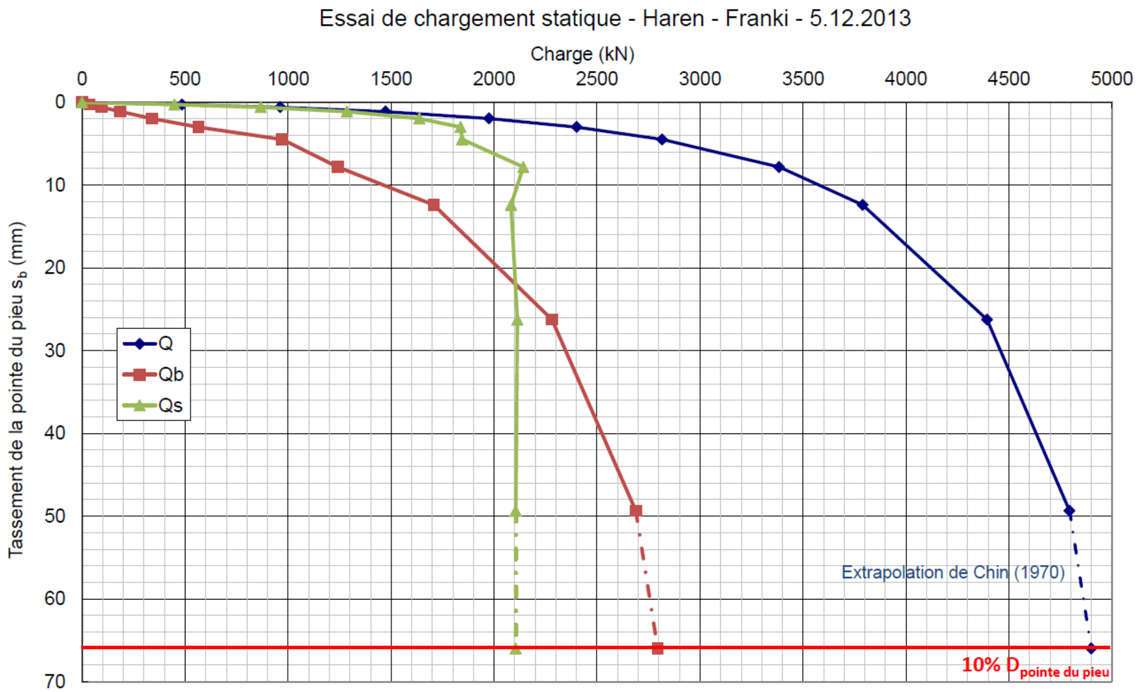


Figure 16. Haren test site - Load-settlement split up in pile base and shaft resistance

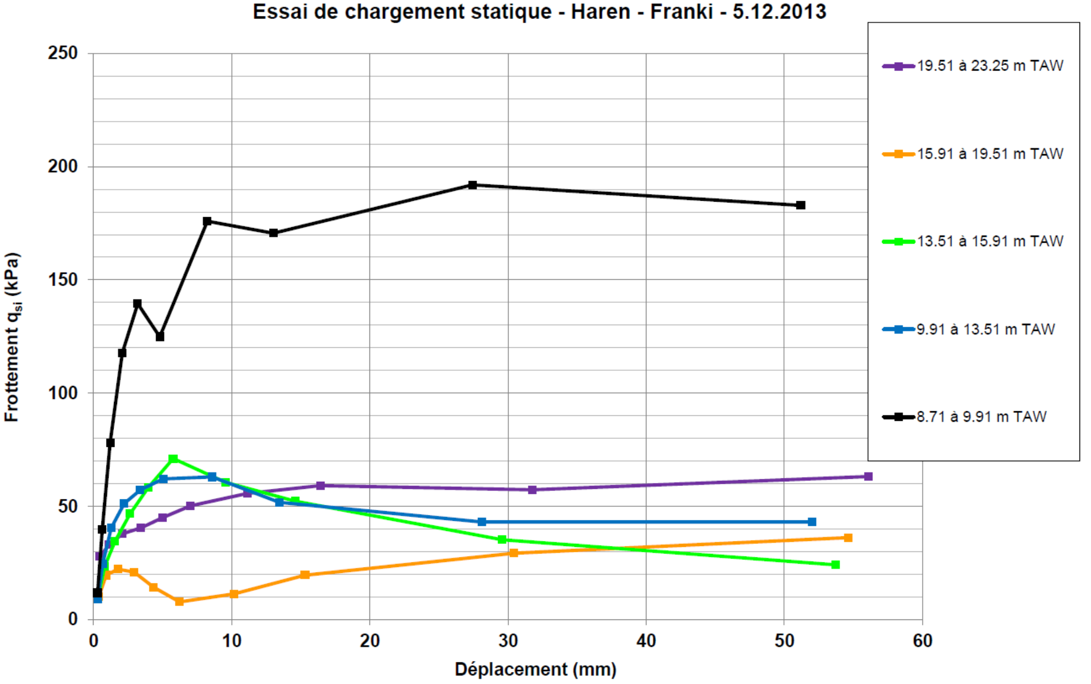


Figure 17. Haren test site - Mobilisation curves of the unit shaft friction in the relevant soil layers

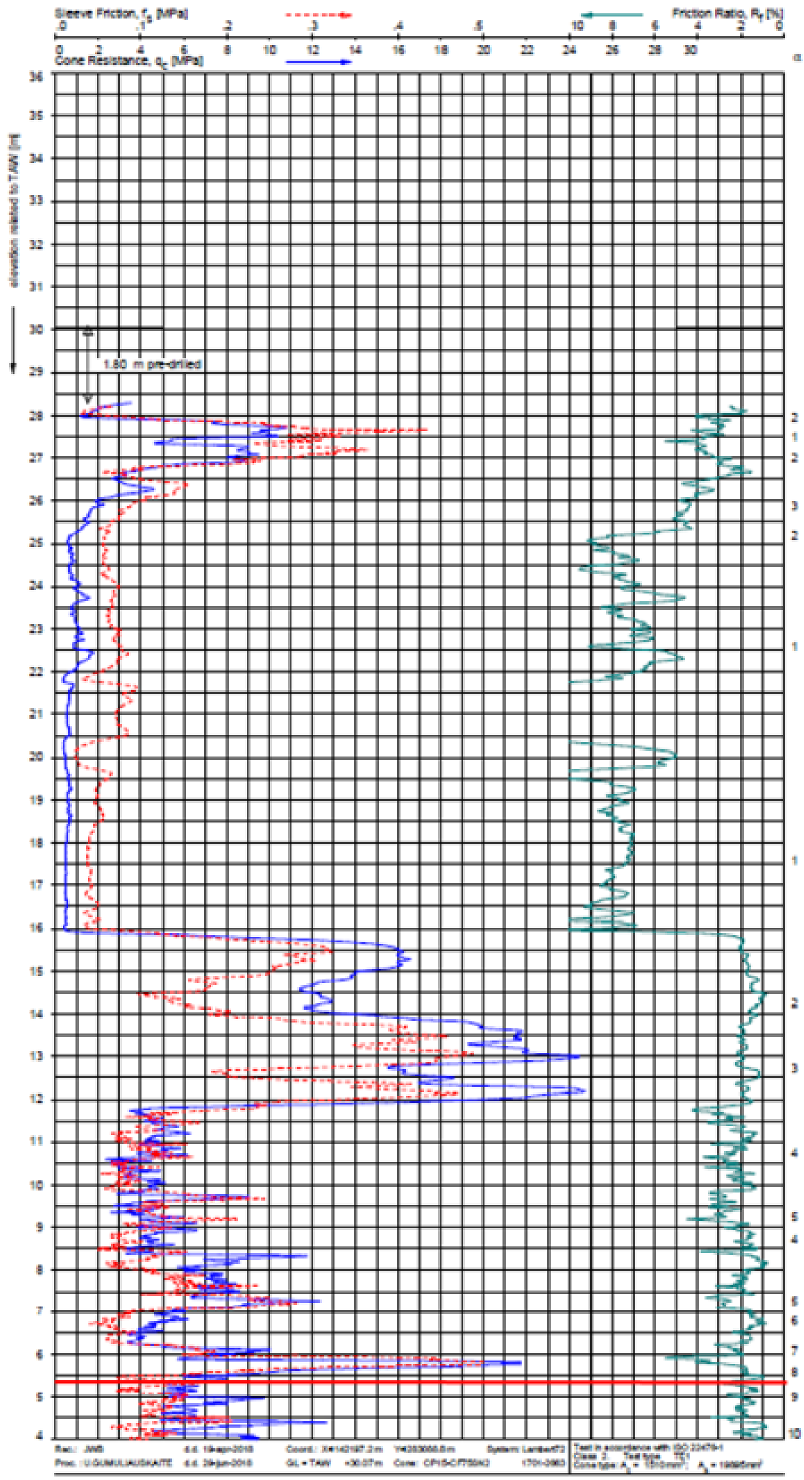


Figure 18. typical CPTE at the job site in the Mons area

A load test program on 8 fully instrumented test piles was carried out by the BBRI on the job site in order to obtain insight in the bearing capacity characteristic in the different soil layers and to verify the design according to the principles of the Belgian design methodology according to EC7 that provides up to now no factors for chalk.

DA piles where installed in 4 pairs at depths of 13.6 m and 17 m below soil surface level in the sand layer (corresponding with absolute levels of +16.7 m TAW and +13.3 m TAW respectively) and at depths of 23.2 m and 24.8 m in the chalk layer (which corresponds with absolute levels of +7 m TAW and +5.4m TAW respectively).

Moreover, for each pair a different DA-tool was used. For one pile of each pair a classical Ø 0.610 m Omega auger, with a relative short auger length of 0.95 m beneath the displacement body was used; for the second pile of a pair an alternative Ø 0.610 m auger with a larger auger length of 1.8 m beneath the displacement body was used(see also the previous Fig. 12).

Figure 19 shows the results of the load distribution for a 24.8 m long pile installed in the direct neighbourhood of the CPT shown in Fig. 18. The load distribution was deduced from the fibre optic sensors (type Fibre Bragg Grating, FBG) that had been integrated in the test pile.

A comparison of the load-settlement behaviour of all the test piles is given in Fig. 20. The full lines in Fig. 20 correspond with the results of the piles executed with the classical DA auger (short auger part), the dashed lines correspond with the results of the piles executed with the alternative DA auger (longer auger part). The reduction of bearing capacity observed between both augers varies between 5 and 20% (pair installed at a depth of 13.6 m). Taking into account the obvious differences and execution particularities, the reduction is considered as rather insignificant.

Anyway, the total pile capacity obtained from testing satisfied largely the predicted values, which were based on the Belgian design methodology according to EC7 and where a safe estimate for the pile resistance in the chalk layers was applied.

These kind of test campaigns on fully instrumented piles are very useful to increase the insight in the behaviour of different types of pile foundations and to improve systematically the design codes. As stated earlier, it isthe author’s opinion that design codes should incorporate significant incentives to favour instrumented pile load testing on the job site. We will come back in section 7 on how the Belgian code tries to implement this in practice.

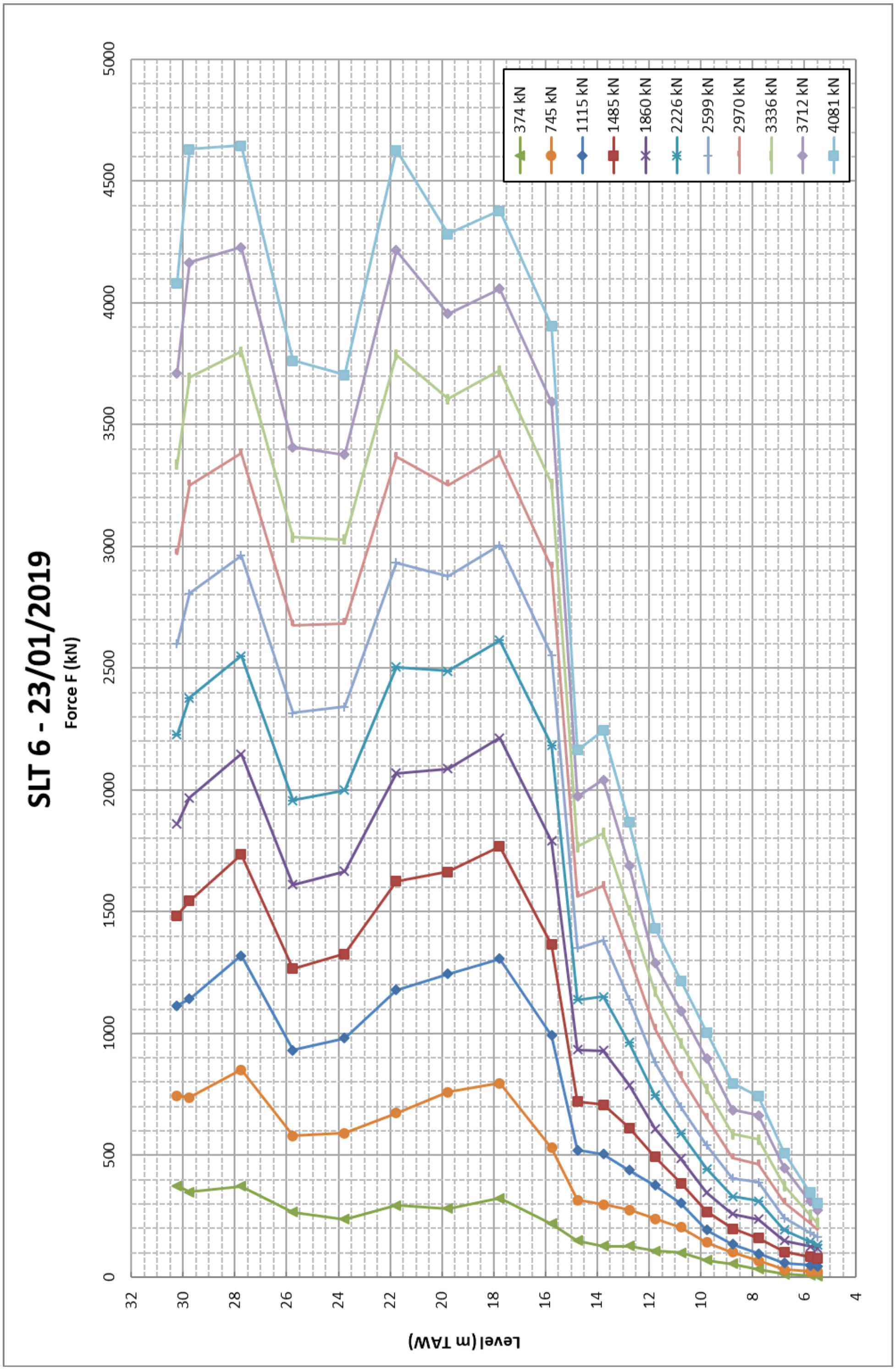


Figure 19. Load distribution of a 24.8 m long test pile deduced from fibre optic sensors (type FBG)

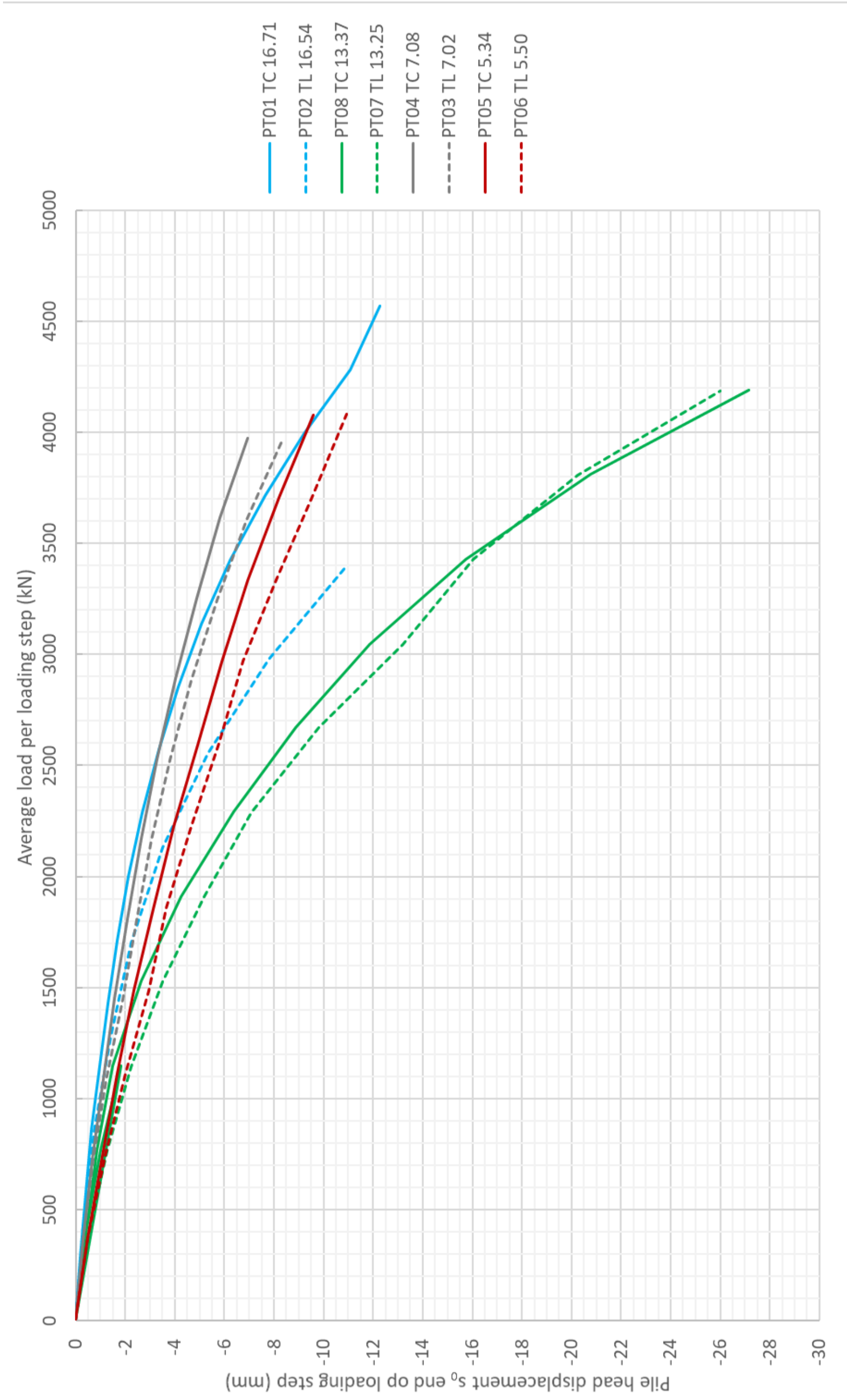


Figure 20. load settlement of the 8 test piles; each colour corresponds with a pair of piles; the full line represent the results of the piles with a classical DA auger, the dashed line correspond with the results of the piles with an alternative auger

3.2.3 Micropiles

The use of micropiles has grown significantly in recent years, together with the development of design and construction guidelines. Many application exist, from the logical underpinning and restricted access projects to even more complex situations where the ability to drill through a large panel of soils, including very variable and hard materials (boulders, karsts, debris,..) where large diameter piling would prove to be unsuccessful.

A large variation of equipment and tools both for drilling and for grouting are used, so that site specific load-testing is usually required to determine the final unit bond strength, certainly for serious structures if one wants to avoid over-conservative design. Here again, the limited dimensions of the foundation elements result in limited loads, and hence easier test setups.

In order to demonstrate the large domain of applicability and the flexibility of micropile systems it is worthwhile to make reference to a recent test campaign on 67 m deep micropiles in the Netherlands. These micropiles will make part of the foundation of a new highway bridge that crosses a high speed railway line.

In order to avoid too large settlements of the high speed railway infrastructure, a foundation system with minimal disturbance of the execution to the surrounding soil had to be applied. Moreover, as compressible clay layers are present at depths between 35 to 50 m, the pile load had to be transferred mainly below a depth of 50 m.

A typical CPTE on the site is given in Fig. 21.

To mitigate this problem the contractor proposed a Ø175-200 mm micropile system with self-drilling rods and with a total pile length of about 67 m, for which he would provide a system to avoid load transfer in the upper 50 m, without compromising the risk on buckling of the micropile system in the upper weak soil layers.

A preliminary compressive load test campaign on instrumented micropiles was performed to prove this concept. Several concepts to realise a “free length” of 50 m were applied by the contractor.

The instrumentation and tests were performed by BBRI. For the instrumentation a prototype with fibre optic sensors that could be installed immediately after the installation of the micropiles was developed (see Fig. 22).

In total 9 test piles were installed of which 6 piles were finally submitted to load tests : 3 compressive load tests on 67m long micropiles and 3 tension load tests on 26 m long micropiles.

Figure 23 illustrates the setup of the compressive load test on a micropile.

Figure 24 illustrates the results of the measured deformations with depth during the compressive load tests on 2 different micropile systems up to a maximum load of 725 kN. The differences between both micropiles exists mainly in the method and means that have been applied to create a “free length” over the upper 50 m.

This figure also shows the results of two different optic sensor technologies that were applied: a multipoint measurement system (Fibre Bragg Grating - FBG) and a distributed measurement system (Brillouin Optical Frequency Domain Analysis – BOFDA).

It can be seen that in the “free length” the measurements are somewhat influenced by variations in the pile section (coupling sleeves each 2 m) and by bending effects on the sensors, which is not surprisingly due to its installation method of the sensors in the fresh grout immediately after pile installation and due to the high length of the pile.

Nevertheless, the measurements allow to deduce a reliable normal load distribution with depth and show very clearly the effect of the load transfer in the upper 50 m. For the pile system on the left side the load transfer seems to be very limited and thus acceptable for the foundation concept. For the pile system on the right side, the applied load at the pile head is completely transferred in the upper 35 m and is thus not acceptable for the envisaged foundation concept.

The results of these test campaign are for the moment still under analysis and will be used to finalize the definite foundation concept of the bridge.

This case shows again the importance and added value of instrumented load test in the realisation and design of innovative but reliable foundation concepts.

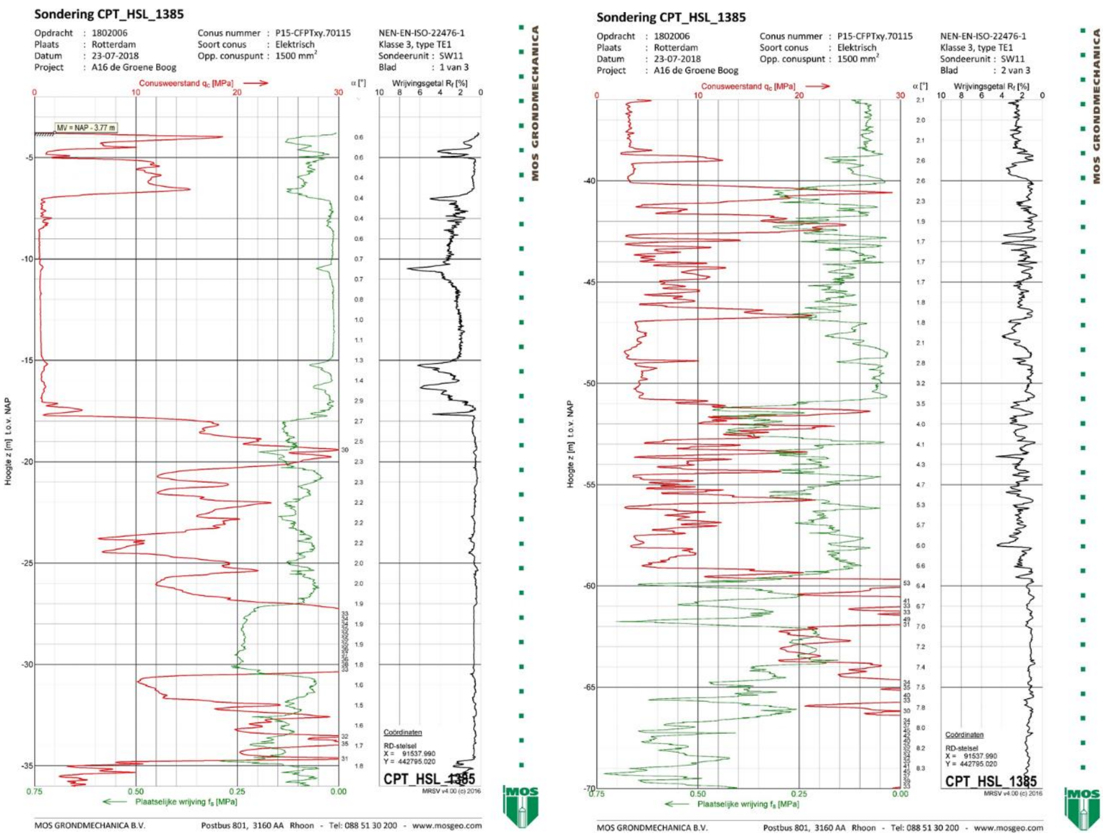


Figure 21. typical CPTE results on the site

Figure 22. Installation of the instrumentation in a 67m deep micropile

Figure 23. Set up of the compressive load test on a micropile

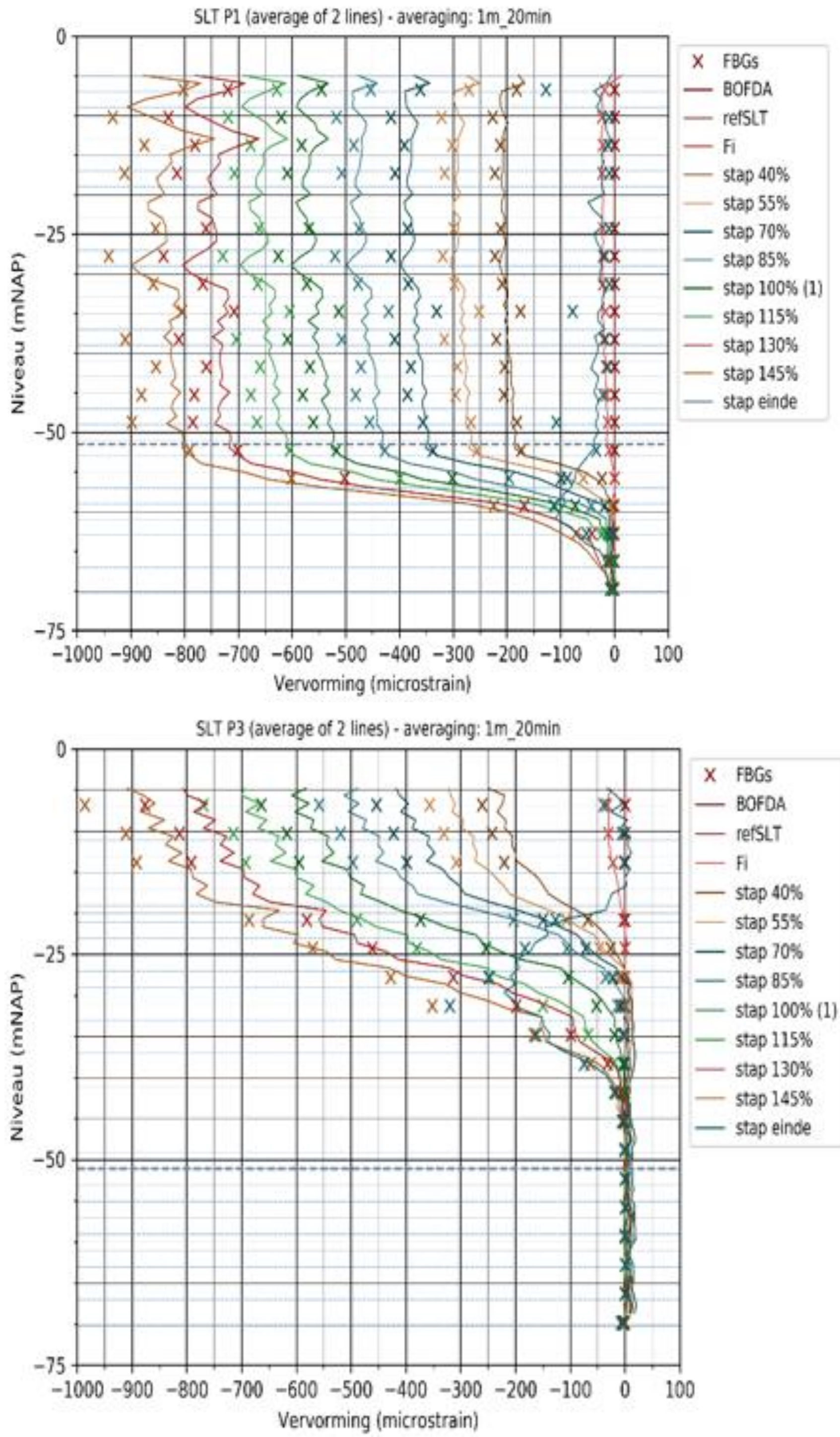


Figure 24. Results of the deformation measurements in 2 different micropile systems during a compressive load test

3.2.4 Bored piles

Tremendous improvements have been made to equipment capabilities (including the use of oscillators, rotators, multi-hammers) enabling to install larger piles at longer depth in harder soils. The use of oscillators or rotators enable to install full length segmental casings which is certainly favourable in loose sands or caving soils. In more extreme rock conditions or challenging working conditions, like limited headroom, Reverse Circulation Drilling is increasingly applied, with full face cutting at the base and removal of the spoil via air-lifting of the drilling fluid.

Because they are usually designed to carry high loads, the reliability of each bored pile is very important, and limited structural weaknesses can have major influence. As we discussed in other papers, many aspects of the pile execution, process may influence the pile capacity : drilling process and drilling tools, cleanliness of the pile bottom, quality of the casting procedure and interaction with the support fluid (bentonite or polymer).

Major advances have been made in the understanding of the critical role of these two fundamental components of the execution process of bored piles:

- The particular aspects related to tremie concrete.
- Support fluids

In 2014, the EFFC and DFI started carrying out a joint review of problems in bored piles and diaphragm walls cast using tremie methods. A Task Group was established and the 1st Edition of the "EFFC/DFI Guide to Tremie Concrete for Deep Foundations" was published in 2016. The 2nd Edition was published in 2018. The 1st Edition included some recommendations on support fluid properties, but it rapidly became clear that the preparation, characteristics and testing of support fluids required a dedicated approach. A Support Fluid Task Group was therefore established in 2017 resulting in a Guide presenting good practice in the use of support fluids for the construction of deep foundations and setting out the latest understanding of support fluids.

These two excellent documents have thus recently been made available and provide invaluable information on the topics. It is obviously impossible to summarize these two extensive documents here, but we want to give a few key hints we learned.

EFFC/DFI Guide to Tremie Concrete for Deep Foundations (Beckhaus & Harnan, 2018)

Modern mix designs clearly deviate from those of “classic concrete” (water, aggregates and cement), and consist of comparatively low water contents and low water-cement ratios, often below 0.45, using admixtures such as fly ash or other additions but also chemical admixtures (often as a cocktail of superplasticizers and retarders, sometimes even topped by stabilizers). On the other hand, recent evolution in the design of structures reflects the possibilities of making economic use of building materials, resulting in high strength requests and dramatically dense reinforcement, and increasing durability requirements. Finally, execution processes, as explained before, have resulted in larger and deeper foundation elements, and hence, longer installation and casting periods, requesting highly flowable and set-retarded concrete mixes for several hours if needed.

It is important that the concrete industry and the Deep Foundations specialists realize that this evolution can become dramatic if not taken seriously. A first advance was made when the requirements for Deep Foundation concrete was shifted from the Execution Norms to the Concrete standard EN 206, under the pressure of the Technical Working group of the EFFC. EN 206 in its recent Edition EN 206: 2013 comprises general regulations for the composition and quality of fresh concrete, and, in its <u>normative Annex D, particular rules for deep foundation</u>

concrete used in bored piles and diaphragm walls, taken from the execution standards EN 1536: 2010 and EN 1538: 2010.

Specific restrictions for deep foundations for composition and consistence, according to Annex D, depend on the placement procedure (see Table 1 for bored piles). It is understood that maximum consistency has been standardized on the basis of experiences at the time, to reduce the risk of segregation and associated negative effects. The upper consistency limits appear to disregard the frequent need for higher values corresponding to a sufficient flowability e.g. in order to flow freely around and through the reinforcement.

Accordingly, in terms of pile or wall integrity and quality, it is also important to consider requirements for concrete cover and clearance between reinforcement bars. In this context, the maximum coarse aggregate size is correctly regulated in the European standards, for deep foundations to a maximum of ¼ of the clear space between bars, in order to ease the flow through and around reinforcement.

However, these requirements are insufficient. With modern, five-component concrete the options to individually design the concrete for specific properties are multi-dimensional, i.e. the characterisation of fresh concrete has become more complex and needs at least two rheological parameters to be sufficiently described. One is viscosity and the other is yield stress. The EFFC-DFI Guide helps understanding the concrete's rheology, which is necessary to properly appreciate the main placement characteristics of workability and stability, where:

- workability is simply defined as that "property of freshly mixed concrete which determines the ease with which it can be mixed, poured, compacted, and finished" , and
- stability is simply defined as the "resistance of a concrete to segregation, bleeding and filtration" .

The R&D program has shown that, for tremie concrete, additional requirements for the concrete should be specified in terms of single target values, test methods and acceptance criteria. The Guide usefully comes up with advanced recommendations for testing fresh concrete including the specific, aspects of workability and stability (see Table 2).

Workability:

It is shown that "the fundamental properties characterising concrete workability are yield stress and viscosity. As there are currently no practical field tests to measure these properties directly, indirect measurements are required".

The slump flow test (in accordance with EN 12350-8 and ASTM C1611) was found suitable to reliably measure the yield value of tremie concrete (see Fig. 27).

The viscosity is considered a secondary rheological property of fresh tremie concrete. As the viscosity is usually hard to adjust independently from the yield stress, no discrete range is recommended in the Tremie Guide. Nevertheless, a medium viscosity as shown in Fig. 28 is still considered preferable for tremie concrete. Instead of a direct measurement of viscosity by an outflow test, which can also be found in the Tremie Guide, a rough measure can be derived from the slump flow test by simply recording the time the concrete needs to its final spread, by dividing the average travel distance of the concrete ((slump flow in mm – 200 mm)/2) to the time required (in seconds).

Stability:

The stability of concrete is harder to assess. One of the simpler tests for stability is the Visual Stability Index (VSI) test which can be conducted as part of the slump flow test. It allows the concrete to be visually checked. After slumping, a visual check is made to look for segregation or bleeding i.e. no wet sheen on the surface, no mortar halo or aggregate pile in the centre of the mass (see Fig. 29).

Finally, the Guide gives some interesting research using numerical methods. For discrete site conditions a full scale trial with dyed concrete could prove such flow patterns but more generally numerical models may be used to explain the relevant dependencies on concrete flow. To cater for this and to prepare for the future opportunities given by modern computational tools a new Section 9 Numerical Modelling of Concrete Flow has been introduced in the new Tremie Guide. Figure 30 indicates how numerical studies can help to understand the flow path of freshly poured concrete and its general consumption over height and time.

Figure 25. Reverse Circulation Drilling – limited headroom and full face drilling tool (Brown , 2012)

Figure 26. Modern five-component substitutes the old three-component concrete technology

Table 1: Requirements for Bored Piles, in accordance with Annex D of EN 206:2013

Placement condition	Cement content [kg/m³]	Water-cement ratio [-]	Slump [mm]	Flow diameter [mm]
Dry	≥ 325	≤ 0.60 and in compliance with provisions valid for specified exposure classes	150 ± 30	500 ± 30
submerged under water,	≥ 375		180 ± 30	560 ± 30
under a stabilizing fluid			200 ± 30	600 ± 30

Table 2: Recommendations for testing tremie concrete

No	TEST	Recommended RANGE for TARGET VALUES	TOLERANCE on specified Target Value	RELEVANCE For SUITABILITY & CONFORMITY	FREQUENCY* * of specified ACCEPTANCE testing
A1.1	Slump Flow	400 – 550 mm	± 50 mm	M	Each load
A1.2	Slump Flow Velocity	10 – 50 mm/s	± 5 mm/s	M	At least 1/week
A1.3	VSI	0	-	M	Each load
A4	Modified Cone Outflow	3 – 6 s	± 1 s	R	As required
A6	Workability Retention	to be specified	- 50mm	R/M*	As required
A7	Static Segregation	≤ 10%	+ 2%	R/M*	As required
A9	Bleeding	≤0.1ml/min	+ 0.02 ml/min	R/M*	As required
A10	Bauer Filtration	≤ 22 ml***	+ 3 ml	R/M*	As required

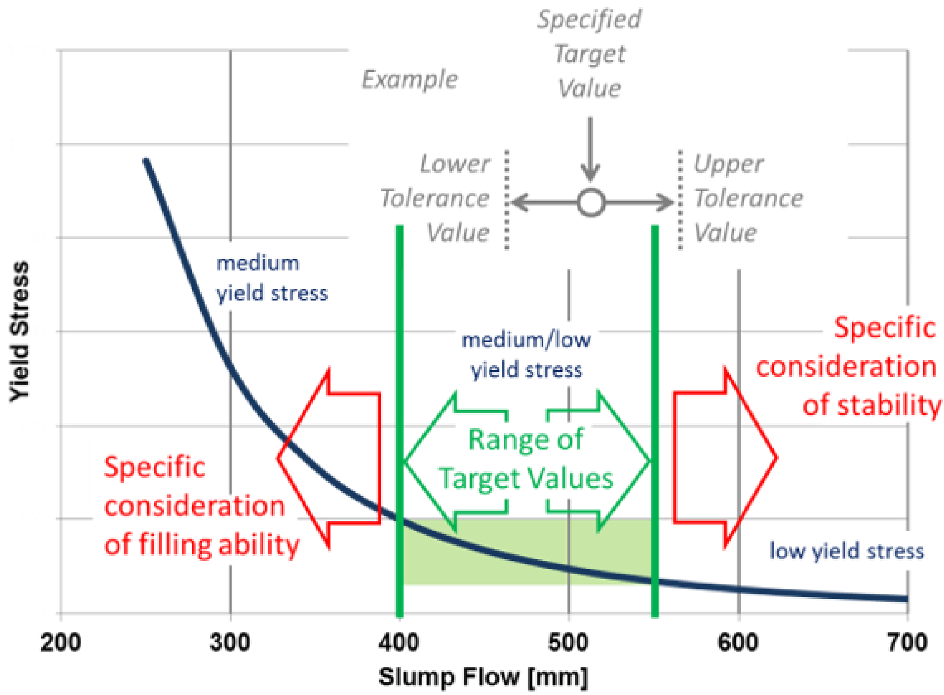


Figure 27. Slump Flow Curve related to yield stress and recommended range for tremie concrete

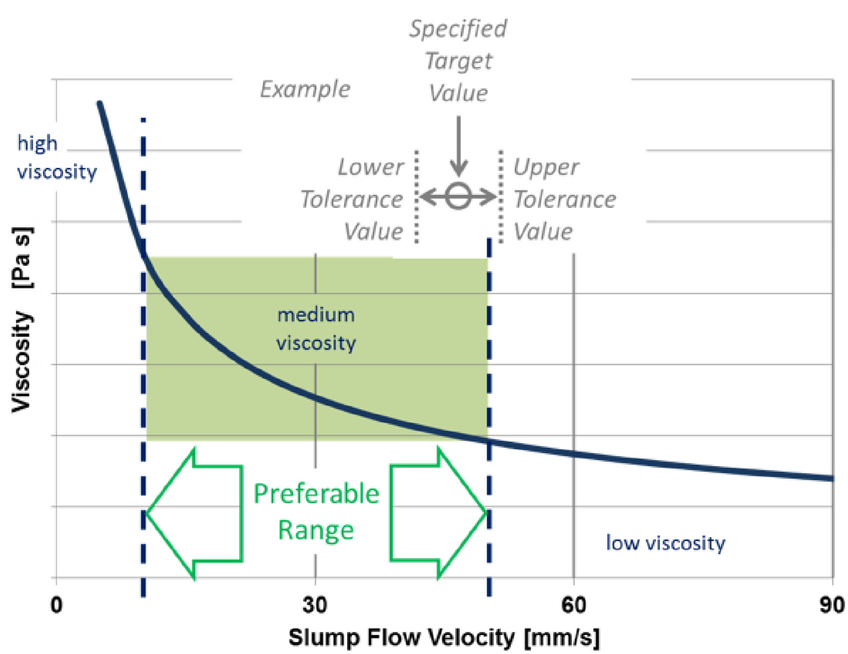


Figure 28. Slump Flow Velocity Curve related to viscosity showing the recommended range of medium viscosity for tremie concrete

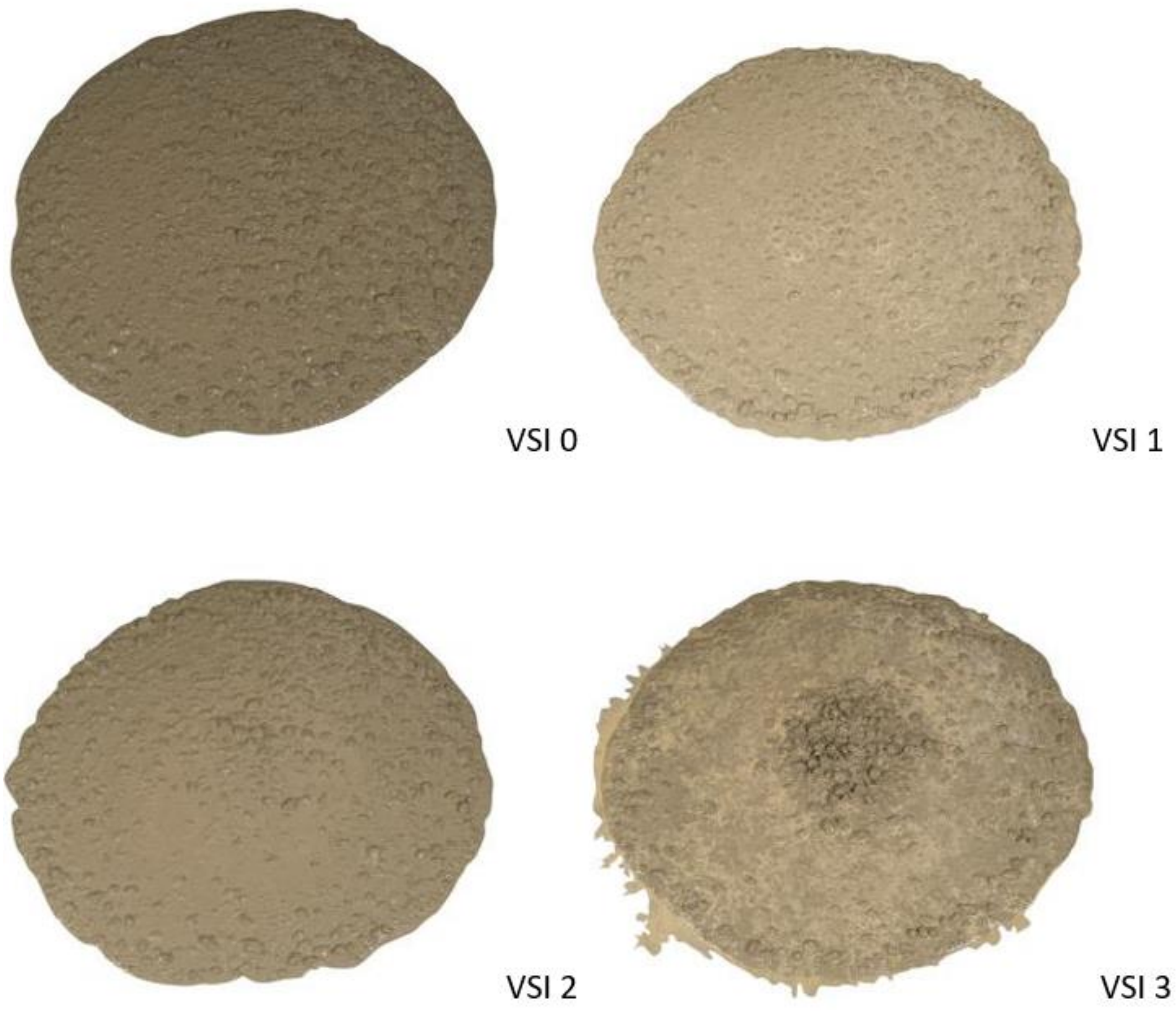


Figure 29. Examples of Visual Stability Index Classes [1], photo courtesy of BASF Corporation

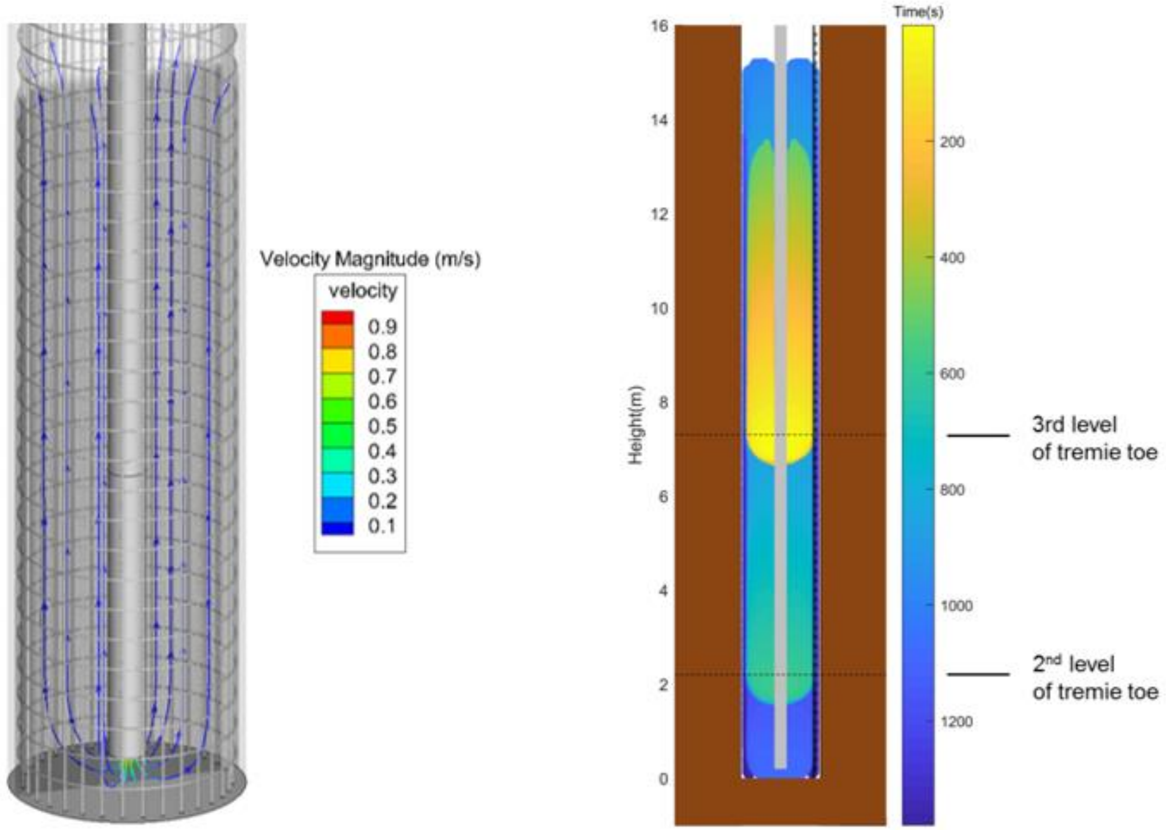


Figure 30. Simulations presenting bulging flow of bulk concrete by velocity streamlines (left), and by dyed concrete following a staged lifting of the tremie pipe [1, 4]

EFFC/DFI Guide to Tremie Concrete for Support Fluids(2019)

Bentonite slurries have been used for over sixty years for the temporary support of excavations such as bored piles and diaphragm walls. Technological developments of drilling methods and enhanced equipment capacities have greatly increased the impact of the soil chemistry and soil fineness on the behaviour of the support fluid.

There have also been advances in support fluid technology. Bentonite properties have evolved and other support fluids (e.g. polymers) have been used in place of bentonite. When used correctly, polymer fluids can offer advantages over their bentonite counterparts, including improved foundation performance, lower environmental impact, smaller site footprint (site area) and also simpler preparation, mixing and final disposal as they are used at much lower concentrations.

The purpose of the Guide is to present current understanding on bentonite, other clays, polymers and blended systems, including the advantages and limitations, in order to allow informed selection of the optimum technical solution(s) for the conditions on each individual worksite.

Three different classes of support fluid are :

- mineral (e.g. bentonite),
- natural (e.g. gum), and modified natural polymer (e.g. CMC and PAC) used alone or blended with bentonite,
- synthetic polymer (e.g. PHPA) – usually used alone.

Bentonite has the beneficial property of forming a filter cake at the excavation sidewall which acts to restrict fluid loss into the surrounding soil and allow a positive hydrostatic head to be maintained within the excavation. The filter cake forms as the bentonite particles are filtered out of the fluid as the hydrostatic head within the excavation drives the fluid into the surrounding permeable soil, as illustrated in Fig. 31.

Polymers perform in a different way to bentonite slurries. **High molecular weight synthetic polymers** are long chain-like hydrocarbon molecules (typically partially hydrolysed polyacrylamides, or PHPA) which interact with each other, with the soil, and with the water to effectively increase the viscosity of the fluid. Although there may be some indication of a polymer membrane at the soil interface, there is no formation of a filter cake as with bentonite. The long polymer chains can be damaged by pumping. Natural modified polymers such as modified celluloses (e.g. polyanionic celluslose, PAC) have been successfully used in the reverse circulation process (e.g. hydromill). They are considered as bio-degradable and produce very thin mud cakes and good fluid loss control.

The flow behaviour of support fluids can be investigated by plotting shear stress as a function of shear rate. Figure 32 shows shear stress-shear rate plots for some idealised flow types with examples of fluids that may show these rheologies.

The three classes call for a separate analysis of their properties and subsequent range of values to be attached to them. The Guide provides target acceptance values and testing methods for the three categories.

The Guide also provides guidance for the correct selection of the adequate support fluid. There is no universal fluid for all projects and selection of the right product(s) has to be made after considering all the parameters. When choosing the preferred support fluid, the following should, as a minimum, be considered:

- project and site dimensions: diameter, width, length and depth of foundation elements to be constructed,
- equipment (excavation, pumping, treatment etc.) and the length of casing (if used),
- excavation method (static drilling or reverse circulation drilling),
- soil conditions: geotechnical profile (e.g. type of soils, permeability, cohesion and chemistry),
- groundwater level and chemistry,
- make-up water quality,

- fluid requirements: ease of use and proven effectiveness in the soil conditions,
- environmental issues (known contaminants and obstacles),
- disposal requirements/restrictions,
- supply chain,
- economics.

Table 3a and 3b present indications of appropriate drilling fluids related to the method of construction and the soil type.

For any given project, the appropriate support fluid will be selected based first on fluid rheology requirements, then availability of resources and previous local experience.

Here again, an excellent way of obtaining important information on the construction aspects of any deep foundation element and thereby ensuring success of the works is to install one or more full-scale trial elements, e.g.:

- Trial pile load tests to assess shaft friction / filter cake performance and base performance,
- Excavations to expose completed panels and stop-end – to assess filter cake thickness and concrete imperfections, see Tremie Guide - Appendix D.
- Pile / panel verticality can be assessed and this is important for circular shafts acting in hoop compression. Various testing methods can be compared.
- Trials can also be undertaken to assess the base cleaning, the initiation of tremie concreting and the development of the interface layer by recording density profiles. When the concreting is undertaken to the ground surface the interface layer can be sampled at ground level (see Appendix A of the guide).

During the execution, properties characterizing a support fluid such as rheology or chemistry are influenced by :

- the ground conditions and environmental considerations,
- the type of foundation system being constructed,
- the proposed construction method,
- the foundation construction cycle.

Those properties, determined by the standard tests described must be conformed to acceptable values in order to ensure the final quality and integrity of the structure.

The life line of the support fluid during the construction process is shown in Fig. 33 and Table 4 with a specific set of tests corresponding to each construction step. The specified properties must be checked and maintained at each step using the standard tests described in Appendix B of the guide to ensure the quality and integrity of the completed works.

During foundation construction it is essential that the contractor complies with the relevant standards for quality assurance and control. The Guide gives the different test methods for each stage for bentonite and polymer fluids with recommended frequencies (Tables 5a and 5b).

The Guide provides a summary of acceptance values for these different methods used in some existing standards.

This first edition presents acceptance values for drilling fluids as given in commonly used Standards. Current acceptance values, though, originate from oil well activities in the middle of the last century. Whilst the values have slowly evolved, there appears to be a lack of technical evidence as to why a certain value is specified. It is clear that the current standards do not adequately cover all the types of fluids available for use in deep foundations or the total tests required. With industry support organized with the help of both EFFC and DFI, a detailed data acquisition study is currently ongoing with visits to sites in both the US and Europe. Based on the findings of this study, it is hoped to give improved recommendations on acceptance values for bentonite, polymer and blended fluids and these will be contained in the second edition which is scheduled for publication in 2020/2021.

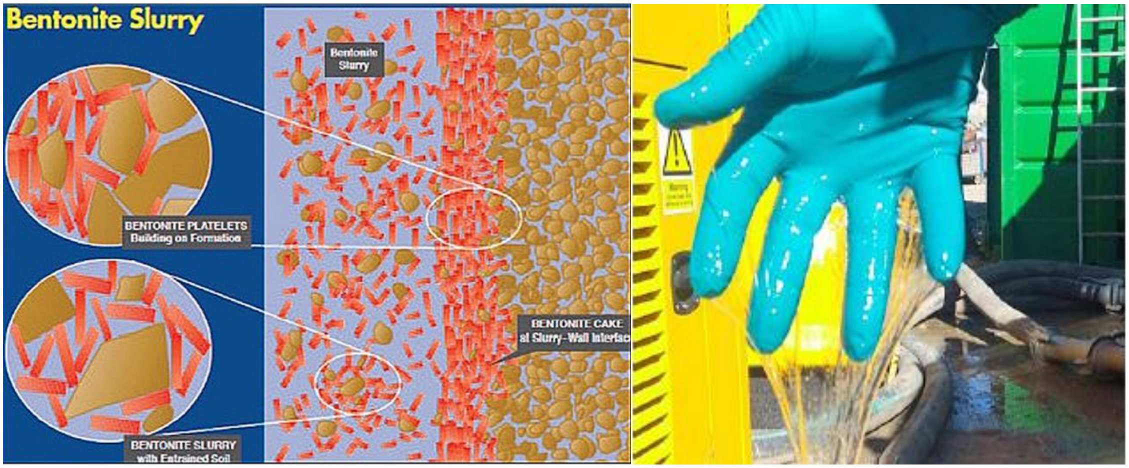


Figure 31. Comparative behaviours of bentonite and polymers

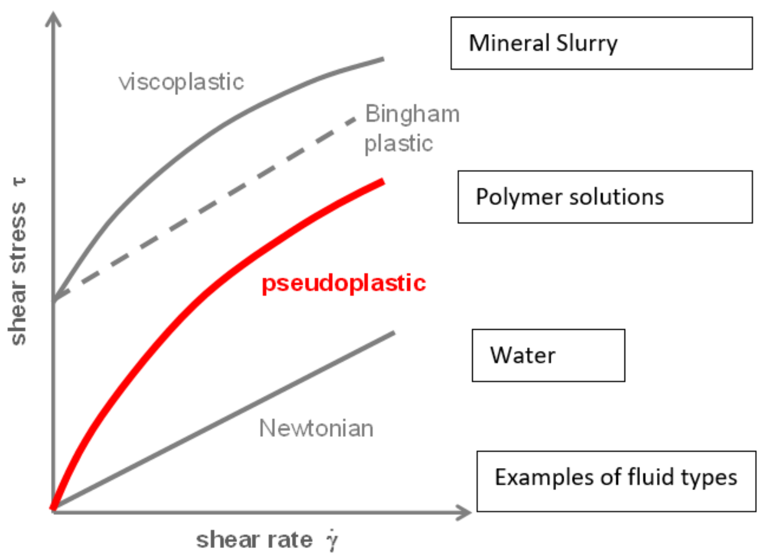


Figure 32. Fluid rheologies

Table 3a: Support Fluid choice related to soil conditions

Fluid Type	Common Examples	Method of Excavation				
		Grab	Hydromill	Auger / Bucket (Kelly)	Reverse or Direct Circulation	Trench Drains
Water		X	X	X	✔✔✔	X
Mineral	Natural Sodium Bentonite Activated Sodium Bentonite	✔✔✔	✔✔✔	✔✔	✔	X
Natural Polymer	Gum Polysaccharide	X	X	X	X	✔✔✔
Modified Natural Polymer	CMC PAC	✔	✔✔	✔	✔	✔✔
Synthetic Polymer	PHPA Vinylic	✔✔✔	-	✔✔✔	✔	✔

✔✔✔ *ideal* ✔✔ *acceptable* ✔ *possible* **X** *not recommended*

Table 3b : Support Fluid choice related to the method of construction

Soil Type	Bentonite		CMC/PAC		PHPA		Comments
	Static	Reverse Circulation	Static	Reverse Circulation	Static	Reverse Circulation	
Rock	✔	✔	✔	✔	X	X	**no stability issues**
Boulders/Cobbles	?	✔	X	X	X	X	**fluid loss / head**
Gravels	✔	✔	X	?	X	X	**fluid loss / head**
Coarse Sand/Gravel	✔	✔	?	?	?	X	**fluid loss / head**
Medium/Fine Sand	✔	✔	✔	✔	?	X	**head / low cohesion**
Silty/Clayey Sand	✔	✔	✔	✔	✔	X	**head / low cohesion**
Clay	✔	?	✔	✔	✔	X	**head / low cohesion**
Hard Clay/Limestone	?	?	✔	✔	✔	X	**no stability issues**
Swelling Clay	?	?	✔	✔	✔	X	**head / soil hydration (sloughing)**

✔ *applicable* **X** *nonapplicable* **?** *to be evaluated*

Note: *Using additives is a way to enhance each type of fluid listed to extend its application in the different soil types (especially those marked with a '?')*

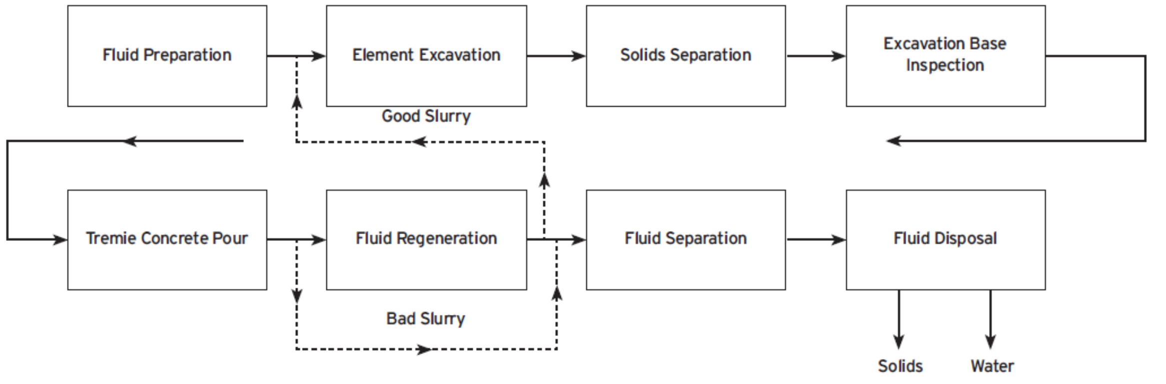


Figure 33. Support Fluid Lifeline During the Construction Process

Table 4 : Construction stages and testing

Stage	Activity	Description	Support Fluid Test
1	Fluid preparation	Mixing of fresh fluid	pH, viscosity, density, filter loss
2	Element excavation	Stabilize excavation	Density, filter loss
3	Solids separation	Mechanical, chemical or gravity treatment of fluid	pH, viscosity, density, sand content, silt content (gel strength)
4	Excavation base inspection	Post rebar placement inspection	Sand & silt content, filter cake
5	Tremie concrete pour	Fluid return from concrete pour	Check for cement contamination
6	Fluid regeneration	Mechanical, chemical or gravity treatment of fluid	pH, viscosity, density, sand content, silt content (gel strength)
7	Fluid separation	Waste fluid separation	Sand & silt content, viscosity
8	Fluid disposal	Waste fluid & solid disposal	pH, sand & silt content, oxygen demand

Table 5a: Applicable Tests and Frequency for Bentonite Support Fluids

Test	Test Method	S1 Fresh Fluid	S2 Excavation Fluid	S3 Before Concreting	S5 & S6 Fluid for Reuse
Frequency		daily	once per element	before pour	daily
Viscosity	Marsh Funnel	M	R	M	M
Density	Mud balance	M	R	M	M
Sand content	Sand content kit	N/A	R	M	R
pH	pH-paper	M	R	M	R
Filter loss	API filter press	M	R	M	M
Filter cake thickness	API filter press	M	R	M	M
Silt content	Calculation	N/A	N/A	O	O
Gel strength	Fann Viscometer	O	N/A	O	O

M : mandatory, R : recommended, O : optional, N/A : not applicable

Table 5b: Applicable Tests and Frequency for Polymer Mud (except for cutter)

Test	Test Method	S1 Fresh Fluid	S2 Excavation Fluid	S3 Before Concreting	S5 & S6 Fluid for Reuse
Frequency		daily	once per element	before pour	daily
Viscosity	Marsh Funnel	M	R	M	M
Density	Mud balance	M	R	M	R
Sand content	Sand content kit	N/A	R	M	R
pH	pH-paper	M	R	R	M
Filter loss	API filter press	N/A	N/A	N/A	N/A
Filter cake thickness	API filter press	N/A	N/A	N/A	N/A
Silt content	Calculation	N/A	N/A	O	O

M : mandatory, R : recommended, O : optional, N/A : not applicable

4 RECENT ADVANCES IN TESTING OF GEOTECHNICAL STRUCTURES

Realizing accurate measurements in deep foundations during testing is not evident, due to the fact that the installation conditions are in most cases very harsh. Often there is also a lack of space to integrate many sensors and their cabling into the foundation elements or it is even not possible to fix/integrate instrumentation at all. These arguments are in many cases valid for classical measurement devices, e.g. strain gauge or vibrating wire sensors. Several reference documents with regard to the more classical type of measurement devices that can be used in geotechnical engineering and piling in particular can be found in the literature, amongst others in Dunicliff (1993) and Hertlein, B. et al. (2006).

The major advances that took place in the ITC sector have found the last decade also their way to the geotechnical sector, leading to significant ameliorations of the classical measurement devices. Miniaturization, integrated electronics, wireless data transmission, real time and online visualization, etc. have been introduced and applied on regular base, - see e.g. (Soga, 2010) and (Van Alboom, 2012) -, but the most important advancement in the last decade originate, according to the authors' opinion, from the fibre-optics technology.

Optical fibres are very small and fragile materials and the use of them seems on first sight not coherent with the generally required robustness of sensors applied in geotechnical engineering. However, they show some important advantages:

- the optical fibre itself is the sensor and the information carrier for the light waves; generally spoken a fibre optic sensor works by modulating one or more properties of the propagating/reflected light wave (intensity, phase, polarization, frequency, …) in response to the parameter (e.g. deformation, temperature) that is being measured;
- the core of the optical fibres consists of a thin strand of glass (about 10 μm diameter) in which light is transmitted; glass is an inert material so very suitable in harsh and aggressive environment;
- all the sensing and transmitting components of a fibre optic sensor are non-electrical, so disturbance of the measurements due to electro-magnetic interference, stray currents, corrosion or short circuits due to e.g. water infiltration are not an issue; they can also be applied in an explosive environment;
- the measured reflections of the light waves are stable on the long-term (no zero drift);
- in the case of multiplexed or distributed optical sensors a multitude of sensor points are available on one single optical fibre. Above that, the dimensions of optical fibres are so small that they can easily be integrated in all kinds of geotechnical bearing elements. With some basic knowledge of the technology it is possible to find and prototype a measurement solution for almost all types of deep foundation elements at a reasonable cost.

This last point is according to the authors' opinion one of the main advantages when fibre optic sensors are applied in geotechnical engineering.

Figure 34 illustrates the evolution of the use of sensors for pile/anchor load test at BBRI during the last 2.5 decades. Since a few years BBRI applies systematically fibre optic sensors for geotechnical testing of all kind of geotechnical structures: piles, micropiles, anchors, retaining walls (diaphragm walls, secant pile walls, sheet piles …), ground improvement elements (inclusions, soil mix, reinforced earth,…),… In order to validate the optical fibre sensors that are used in geotechnical testing, many comparative tests with classical measurement devices have been carried out in the laboratory as well as in situ.

Some examples where optical fibre sensors have been applied to determine load distribution were already shown in the previous paragraph on displacement auger piles.

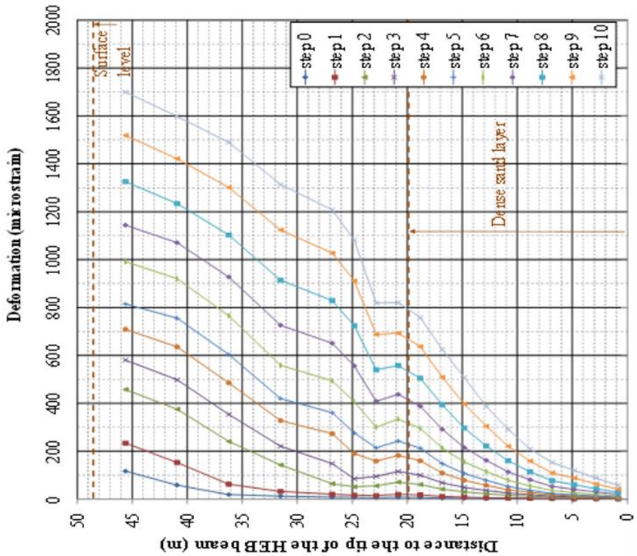
In Fig. 35 to 37 some more recent applications are illustrated.

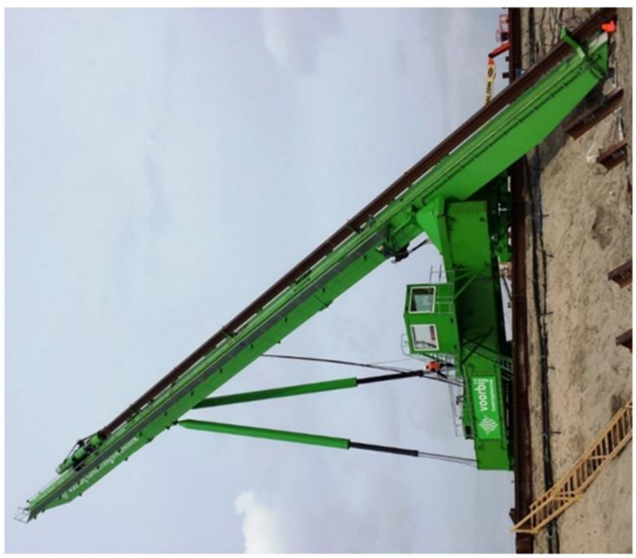
Figure 35 show the examples of a tension load tests on inclined MV-piles. These MV-piles exist out of +50 m long HEB profiles that are driven into the soil under a grout injection from the pile base, in this case in very dense sand layers in the port of Rotterdam, where they will serve as anchorage of a deep quay wall. The steel profiles were instrumented with optical fibre sensors before installation of the piles. Although the driving work was very hard and took a long time, all the sensors survived the driving process, which shows the advantage of optical measurement devices for this case, namely their small dimensions and their low weight resulting in low inertia forces on the measurement devices.

The detailed load distribution obtained out of these instrumented tests were used to validate the design of this case and to optimize the design of other similar applications in the port of Rotterdam.

More details of these test are published by Putteman et al (2017 & 2019).

Figure 34. Evolution at BBRI with regard to the use of instrumentation techniques in deep foundation testing





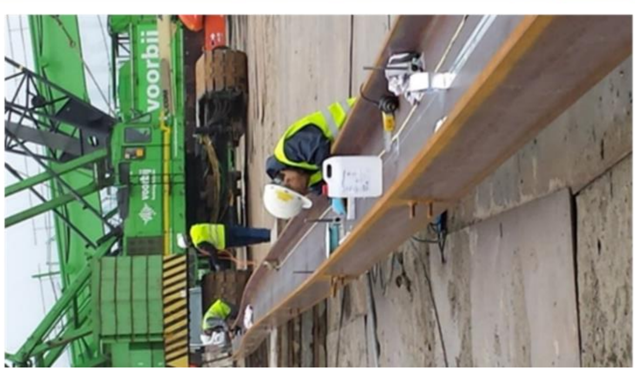


Figure 35. Instrumentation and tension load tests on MV-piles with L>50m (port of Rotterdam)

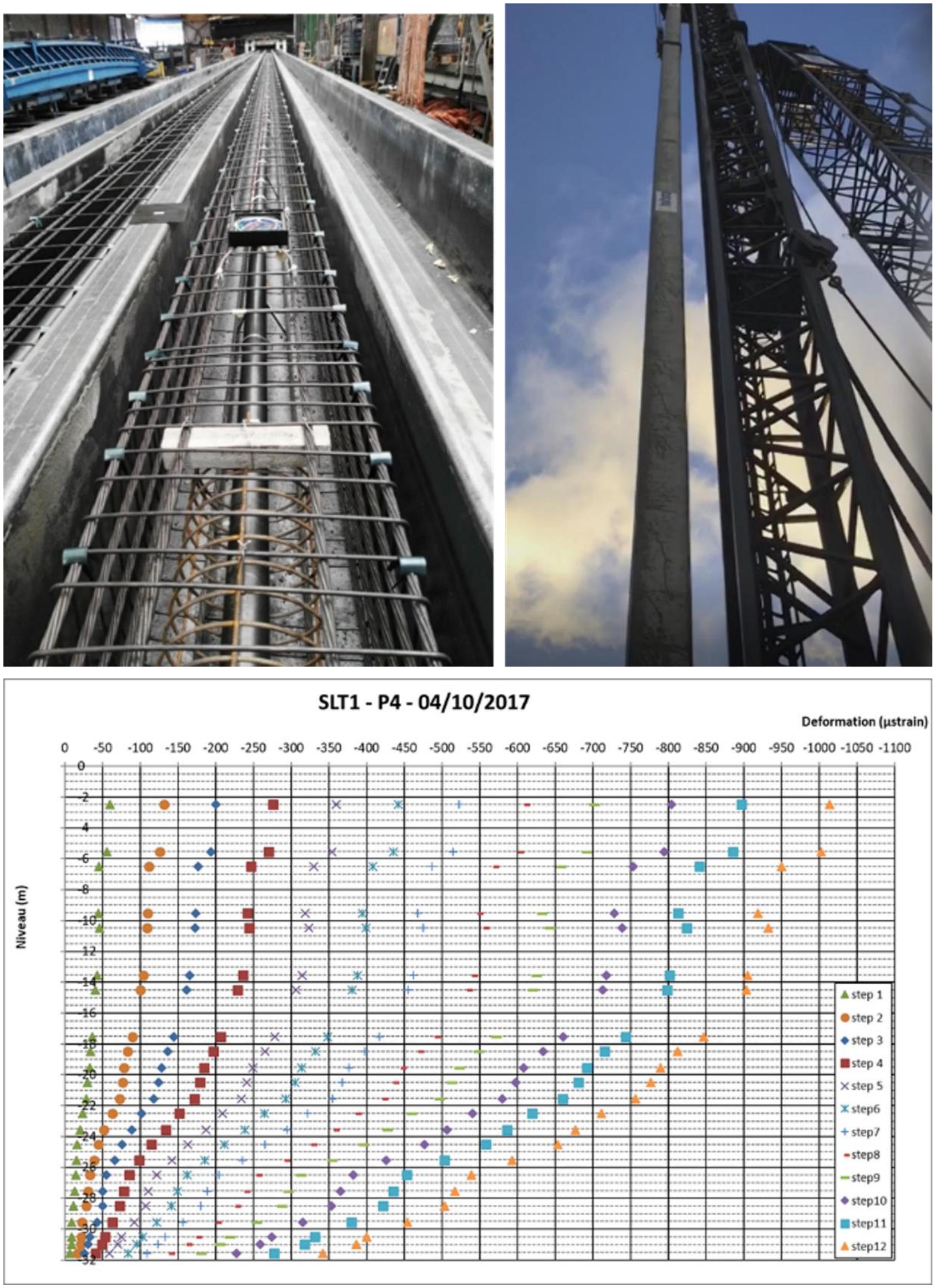


Figure 36. Instrumentation and compressive load tests on precast driven piles (port of Rotterdam)

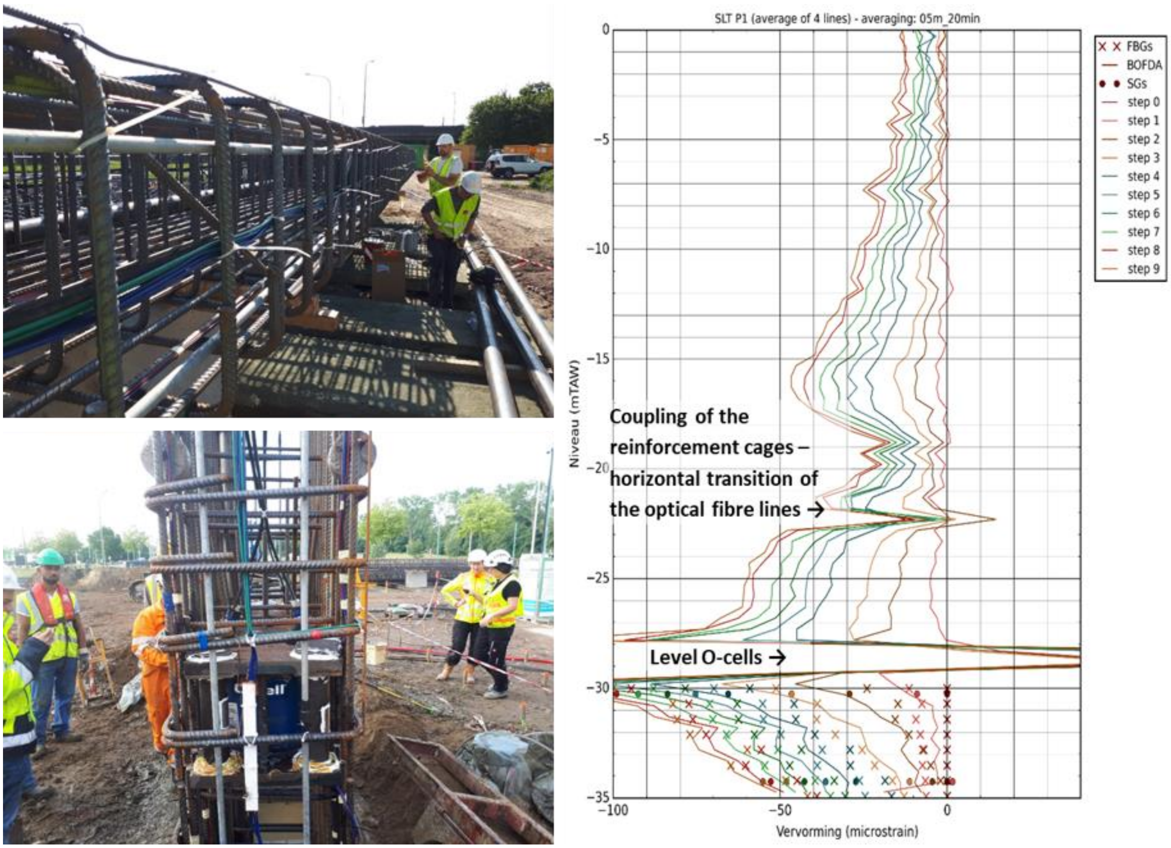


Figure 37. Instrumentation and O-cell load tests on diaphragm walls in Antwerp (Lantis)

Another interesting example is that illustrated in the Fig. 37. It concerns the instrumentation of four 35 m long precast driven piles existing of prestressed concrete in the manufactory. The optical fibre sensors were integrated in the reinforcement cage. Consecutively, the concrete pile was transported to the job site where they were driven into the soil by hydraulic hammering and submitted to load testing.

Also in this case the fibre optical sensors survived the hard driving work and allowed to determine a detailed load distribution with depth during load testing as well as the effect of the residual stresses due to the driving process. The results will be used to optimize installation factors for these kind of piles for future design in the port of Rotterdam.

More details of these test are published by Matic et al (2019) published at this conference.

Figure 37 shows a case were fibre optical sensors have been integrated in the reinforcement cage of four 40 m deep diaphragm walls in the Antwerp area. The diaphragm walls will support the load of a stapled tunnel construction that is planned in the near future in Antwerp (Oosterweel link) and in order to verify the foundation design, a load test program with integrated Osterberg cells was set up. As most interest was going out to the bottom part of the diaphragm wall situated in the stiff over consolidated Tertiary Boom clay, the panels were intensively instrumented with, a.o., optical fibre sensor lines.

The reinforcement cage existed out of three parts and it was a challenge to fix the fibre optical sensor lines to the reinforcement cage(s) during the installation. For all four diaphragm panels the optical fibre sensors survived the installation. It allowed to determine a detailed view of the load distribution in the diaphragm wall upon testing. An example of the deformation measurements between the level -25 m TAW and -35 m TAW is given in Fig. 37 (at the right).

The results of these tests are under analysis for the moment and are not yet public available.

For more details with regard to the optical fibre technology and more examples of applications, reference is made to Huybrechts et al (2016 and 2017).

Based on the evolution of the measurement techniques and as proven with the previous examples, it is clear that actually a lot more information of the behaviour of foundation elements can be obtained during testing than in earlier days.

This is interesting in the context of design codes, which are still often based on non- or poor instrumented load tests in past.

Anyway, the evolution of the measurement techniques allows to gain more and better insight in the real behaviour of a foundation under test loading and also to link the performance to certain observations with the execution or the soil conditions.

As a conclusion one can state that:

- more load testing is needed to calibrate installation factors,
- economic incentives that favour QC monitoring and testing should be introduced in codes and standards,
- standards and codes should leave the door open and even stimulate innovative techniques and processes,
- in the context of design it is important to have a relation between the risk class of a geotechnical structure and the safety factors.

5 HOW CAN CODES AND TECHNOLOGICAL EVOLUTION BE LINKED WITH EACH OTHER

As we tried to explain in the above sections, the link between installation details and installed pile capacity is tight (see also Bottiau 2014), but is difficult to quantify and to capture this in installation coefficients, which would be valid for all cases:

- Execution systems are in constant evolution, and small details can vary resulting in major differences. The codes are usually not enough up to date to take these new evolutions into account.
- It should be emphasized that all installation methods may prove to be inadequate in function of the local soil conditions, or the installed equipment capacity.
- The response of some types of soils to the solicitation of pile installation procedure, can be dramatically different than expected. This is specifically the case in intermediate soil types or in rapidly changing soil conditions.

For this reason, installation coefficients need to be related to the “real” set of parameters of the specific jobsite. All aspects governing the pile installation should be considered and documented: system details, materials, equipment capacity, monitoring during and after pile installation. In this respect, analysis of modern monitoring data are introducing a completely new paradigm in the way we look at pile performance and documentation.

- Real-time access to all site and equipment related data
- Detailed report with selection of specific data for further analysis and quality documentation
- Focused follow-up by in-house expert team.

For all these reasons, the authors believe that codes should adapt. Installation factors should be aligned on demonstrated performance and model and safety factors should account for proven reliability and reward testing. In this section, we want to show how the Belgian NA of the EC7 is trying in the recent years to introduce this in the daily practice.

5.1 Belgian procedure for ULS design (simplified)

In Belgium, the ULS design is in most cases based on the cone resistance diagram measured with in situ cone penetration tests. The design methodology to perform ULS design for axially loaded piles based on CPT results is described in the Belgian pile design guide (WTCB-CSTC, 2009/2016), which is referenced in the Belgian national annex of the EC 7 as the reference method.

This is the "GEO"-verification according to Eurocode 7:

$$F_{c,d} \leq R_{c,d} \tag{2}$$

where $F_{c,d}$ (kN) is the design value of the axial compression load on the pile and $R_{c,d}$ (kN) is the design value of the compressive resistance of the axially loaded pile.

Design value of the pile resistance $R_{c,d}$

Figure 38 gives a schematic overview of the different steps to calculate the design value of the compressive resistance of the pile $R_{c,d}$.

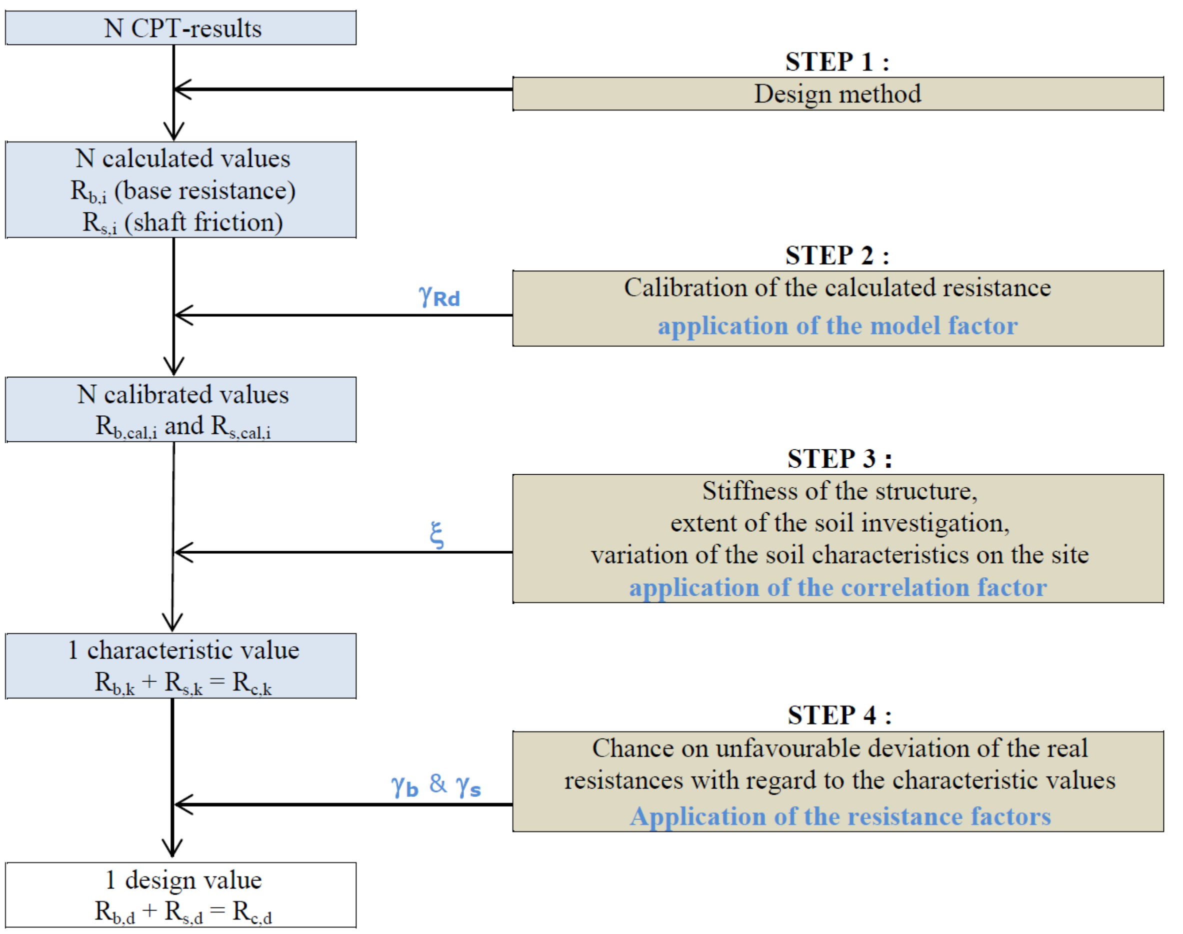


Figure 38. Schematic overview of the different steps to calculate the design value of the pile bearing capacity

In step 1, the compressive resistance of the pile R_c, existing out of the pile base resistance R_b and the shaft friction R_s, is calculated starting from the results of each individual CPT that has been carried out on the job site with the help of the semi-empirical methods, including the installation factors.

The pile base resistance R_b is determined according to the formula:

$$R_b = \alpha_b . \varepsilon_b . \beta . \lambda . A_b . q_b \quad (3)$$

where q_b (kPa) is the unit pile base resistance calculated with the De Beer Method out of the cone resistance (q_c) diagram of the CPT, α_b (-) is an empirical factor taking into account the installation method of the pile and the soil type, ε_b (-) is a parameter referring to the scale dependent soil shear strength characteristics (e.g. in the case of stiff fissured clay), β (-) is a shape factor, introduced for non-circular nor square-shaped bases, A_b (m²) is the section of the pile base and λ (-) is a reduction factor for enlarged pile bases that generate soil relaxation around the pile shaft during installation of the pile.

The shaft resistance R_s is determined according to the following formula:

$$R_s = \chi_s . \sum(\alpha_{s,i}\, h_i . q_{s,i}) \quad (4)$$

where $q_{s,i}$ (kPa) is the unit shaft friction:

$$q_{s,i} = 1000 . \eta^*_{p,i} . q_{c,m,i} \quad (5)$$

with $\eta^*_{p,i}$ (-) an empirical factor, giving the ratio between the unit shaft friction $q_{s,i}$ and the cone resistance q_c for a given soil type, $q_{c,m,i}$ (MPa) the average cone resistance (q_c) for layer i, χ_s (m) the perimeter of the pile shaft, $\alpha_{s,i}$ (-) an empirical factor for layer i, taking into account the installation method of the pile and the roughness of the pile shaft in a given soil type and h_i (m) the thickness of layer i.

In a second step the calculated values of the compressive resistance of the pile are divided with the model factor γ_{Rd} [NBN EN 1997-1 §2.4.1 (6), §2.4.1 (8), §2.4.7.1 (6), §7.6.2.3 (2)]. In this way a calibrated value of the pile resistance $R_{c,cal}$ is obtained for each individual CPT:

$$R_{c,cal} = \frac{R_c}{\gamma_{Rd}} \quad (6)$$

where $R_{c,cal}$ (kN) is the calibrated bearing capacity of the pile and γ_{Rd} (-) is the model factor.

In step 3, one characteristic value of the pile resistance $R_{c,k}$ is deduced by applying the correlation factors ξ_3 and ξ_4 on the average and the minimum value of the calibrated pile resistances respectively, and by retaining the smallest value of both:

$$R_{c,k} = min\left\{\frac{(R_{c,cal})_{average}}{\xi_3}; \frac{(R_{c,cal})_{min}}{\xi_4}\right\} \quad (7)$$

The correlation factors are applied in order to take the variation on the soil characteristics and the uncertainty on this variation into account.

In step 4, the design value of the pile resistance $R_{c,d}$ is finally obtained by applying the partial safety factors γ_b and γ_s on the characteristic pile base and shaft resistances:

$$R_{c,d} = \frac{R_{b,k}}{\gamma_b} + \frac{R_{s,k}}{\gamma_s} \quad (8)$$

The values of the partial factors depend on the guarantee that can be given on the quality of the pile installation.

5.2 Current evolution

As mentioned before, it is the authors' opinion that codes and standards should provide incentives in order to encourage Testing, Quality Control and monitoring, with as objective to level up the profession and increase the reliability. For that reason, the following principles have been provided in the Belgian design methodology as described in WTCB-CSTC, (2009/2016):

- Installation factors α_b and α_s, are function of pile (sub) category. These generic installation factors are rather conservative, but the document provides a methodology (instrumented pile testing program in different soil types) and acceptance criteria to get better installation factors for individual pile systems.
- Model factors γ_{rdi}, are function of the availability of instrumented SLT's.
- Correlation factors $\xi_{3,4}$ are function of the intensity of the soil investigation tests.
- Safety factors $\gamma_{b,s}$ are function of the quality of the QC provided for the production piles.

In order to enforce/facilitate this in practice, a system of Certified Technical Approvals is actually launched in Belgium by BUtgb-UBAtc, which is Belgium's only authority offering technical approval of construction materials, products, systems and installers. In this process, the contractor has to introduce for a particular piling system a dossier based on a formalized installation procedure and a documented PLT campaign in different soil categories. After analysis by an independent expert team of BUtgb-UBAtc,, the piling system and the contractor is granted specific installation factors, as well as model factors and safety factors that account for a period of five years. A renewal procedure is defined, requesting the performance of at least two SLT's on each system in the period.

This process has been discussed and approved by the contractors (ABEF – National Federation of Piling Contractors) and is currently being organized. The impact will be an increased demand for SLT's and will generate a stimulus for contractors to innovate and differentiate from their competitors, as the investment in innovative techniques can be valorised quite fast by means of the Certified Technical Approval procedure, which is in fact a complement to the design standard.

6 CONCLUSION

In this contribution an overview has been given about the situation concerning the harmonization of codes and standards that apply for the design of piles, in particular the Eurocode 7. The great advantage of EC7 is that, at least, the used design terminology is the same throughout Europe. However, some simple design exercise have shown that differences between the output of the different national approaches remain huge. This is not that surprising, as local experience (soil, piling techniques and testing) remain very important in the estimation of pile capacity.

Based on the analysis of the authors, current pile design standards and codes contain several discrepancies, a.o. because they are often based on load tests that were performed several decades ago, they don't account correctly for the real impact of pile installation on its performance. Moreover, they are often not adapted for recent technical evolutions with regard to the piling equipment (new, larger, deeper, faster, with automated registrations, …), the testing and instrumentation possibilities and the advances in numerical modelling. Several of those technical advances have been illustrated in this contribution.

It is the authors' opinion that:

- Standards and codes should reward more efficiently the correct understanding and monitoring of pile execution and the related increase in reliability. They should also integrate methodologies that anticipate on new technical advances, and provide economic incentives for testing, QC and monitoring. The Belgian example, where, complementary to the NA of the Eurocode 7, a system of Certified Technical Approval for pile systems is launched goes in that direction.
- Research is still necessary to better understand some critical issues with regard to the pile installation influence.

- The use of recent monitoring and testing technologies and numerical modelling can help a lot to improve insights in pile behaviour.

Taking into account the previous suggestions could lead on the long term to a significantly better fit between predicted capacity using the codes and the installed capacity in the field.

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