Practical experience with screw piles used for the high-speed railway in Belgium

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ABSTRACT: For the realization of the high-speed railway in Belgium screw piles are often used in different kinds of applications such as foundation piles, ground improvement piles and in retaining walls. The piles are installed in different kinds of soft soils and often take important charges. Different design methods lead to a wide range of theoretical bearing capacities. To verify the bearing capacity of the piles axial loading tests have been performed. Based on the results of those tests a design method has been elaborated.

1 INTRODUCTION

The high-speed railway passes through Belgium and makes the links between Paris-Brussels-Liège-Köln and Paris-Brussels-Antwerp-Amsterdam. These tracks are presently under construction. Therefore large structures as bridges, viaducts, retaining walls need to be constructed. Because the dimensions and the loads of these constructions are important, the bearing capacity of the piles must be rather high.

Since the geology of Belgium is very varied screw piles are installed in different kinds of soils and have different lengths and diameters.

The bearing capacity of the piles is often tested with static loading test. The results of these tests as well as the applied design methods are discussed.

2 GEOLOGY OF BELGIUM

The Belgian territory is mostly flat with a continuous transition from a plane at the North sea and the Dutch border to the Ardennes. Geologically Belgium can roughly be divided in a northern and a southern area separated by a line, which, from Southwest to Northeast, follows the valleys of the rivers Haine, Sambre, Meuse and Vesdre, known as the Sambre and Meuse axle.

In the North, the stratigraphy has been governed by fluctuations in the coastal line. Consequently the bedrock is covered by alternating Tertiary clay, sand and (occasionally) gravel sediments, with thickness up to hundreds of meters. The Quaternary Pleistocene and Holocene formations have been heavily influenced by the glacial periods, giving rise to the formation of marine, coastal, river, lake or wind deposits of sand, clay, peat and silt (loess). Holocene erosion and river sedimentation, as well as human activities, have further influenced the actual subsurface.

In the South of the Sambre and Meuse axle the bedrock is often found at rather shallow depths, overlain by colluvium layers consisting of weathered rock and river sediments.

As a result of the geological history, one can find a wide variety in stratigraphy in the North, with complicated and heterogeneous soil layer patterns. Therefore it is not surprising that the North of Belgium (like the Netherlands) has to face serious foundation problems, requiring particular foundations such as piling or ground improvement. In accordance with those geological conditions, depths for deep foundations generally range between 10 and 25 meters, and more typically between 13 and 18 meters.

3 THE USE OF SCREW PILES FOR THE HIGH-SPEED RAILWAY

Pile foundations are often necessary. On the first project between the French border and Brussels large diameter bored piles and driven piles with or without enlarged base were mostly used. On a few occasions screwed piles were used.

For the bored piles different quality assurance controls were performed. As we experienced a lot of problems relating to the execution of the bored piles, these piles are now only used when high bearing capacities and large diameter piles are absolutely necessary.

During the installation of the driven piles vibrations are generated. This caused a lot of complaints of the inhabitants so this type of pile is no longer used at construction sites situated in de vicinity of existing buildings.

For the other railway projects through Brussels, the project between Brussels and Leuven and the project between Antwerp and the Netherlands screw piles with high and partial soil displacement are now mostly used because of the possibility of execution of these piles without vibrations, their speed of execution and their cost-effectiveness.

Typical applications of screw piles are given in fig.1a and 1b. For retaining structures CFA-piles with casings are used to realize secans walls (see fig. 1c)

Figure 1a: Screw piles with high and partial soil displacement used as foundation pile

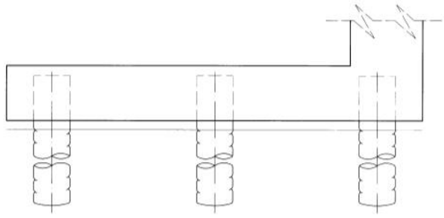


Figure 1b: Screw piles with high and partial soil displacement used as ground improvement

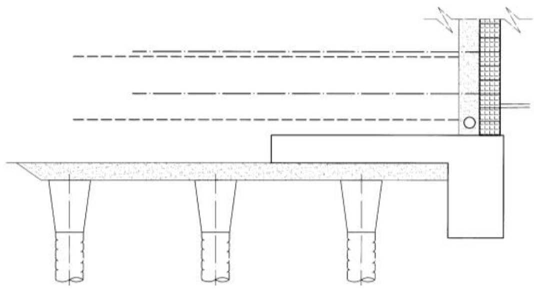
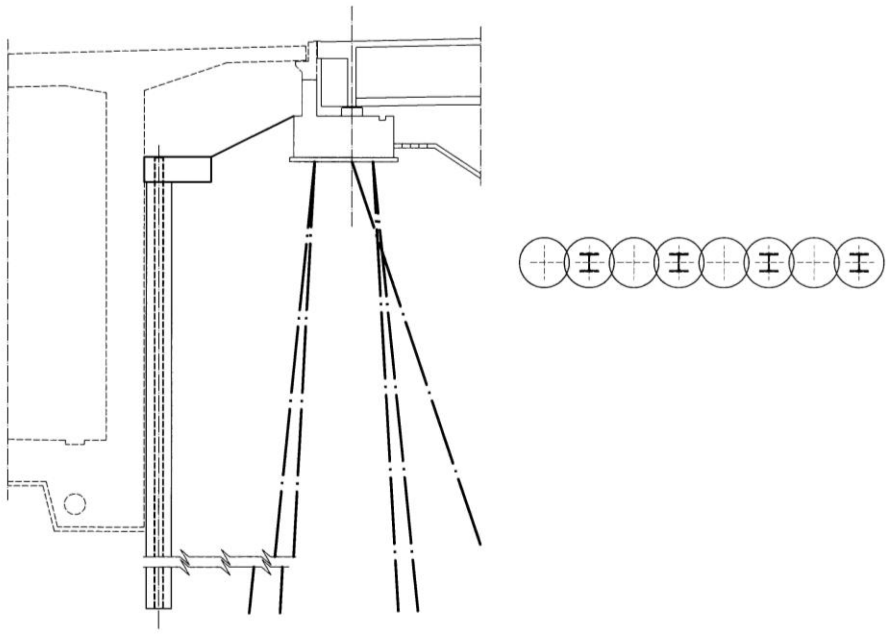


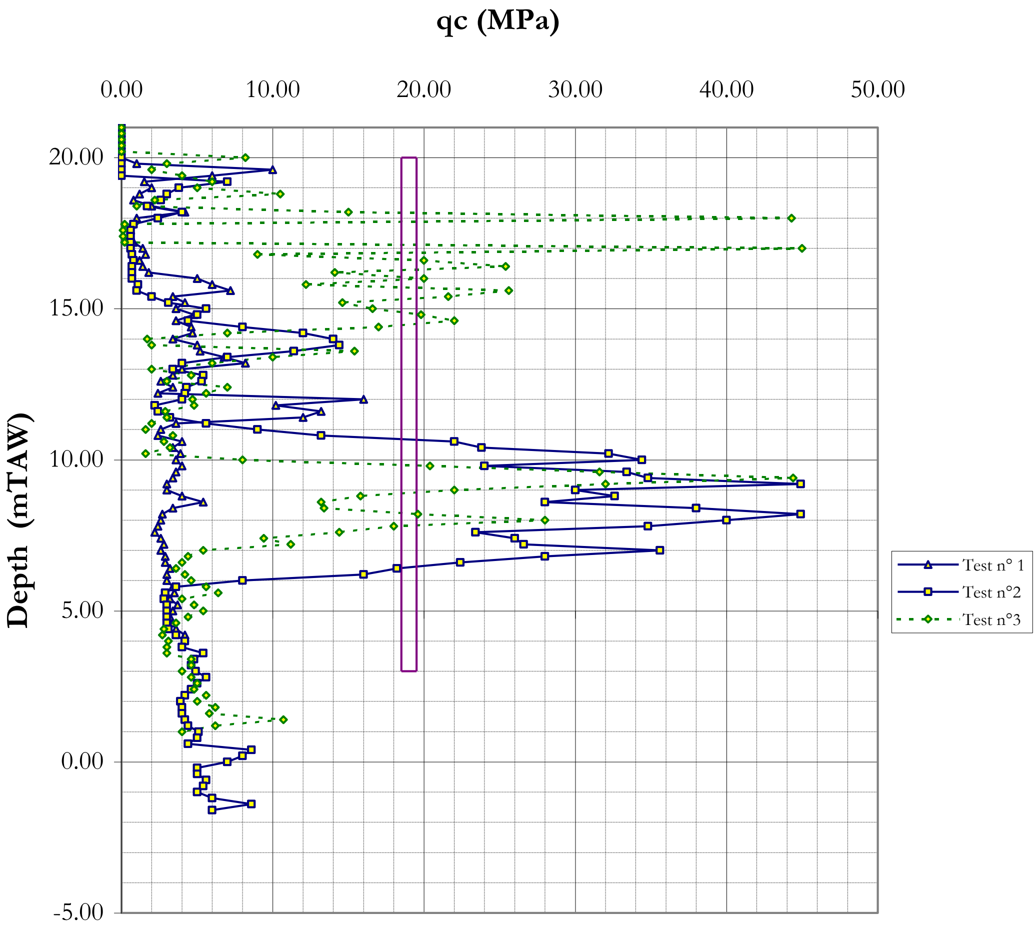
Figure 1c: CFA-piles with casing used as retaining wall



The screw piles with partial soil displacement mostly used at our construction sites are CFA piles executed with an auger with enlarged diameter of the central tube, more precisely PSC lambda piles (Franki). This pile type has two advantages compared with the screw pile with full soil displacement. The reinforcement cage of the pile can be put in place within the central tube of the auger so the reinforcement cage is always well positioned in the center of the pile. Furthermore this pile type can be drilled through resistant layers of dense sand and gravel and through sand with sandstone layers. Screw piles of that type have been used for the first time at the construction site of Brussels-South. Some typical results of CPT tests performed on that site are given in figure 2. They show that a resistant layer with a sufficient thickness and a constant level is not always present. Therefore the foundation piles had to be installed at the level +3,00.

Screw piles with partial soil displacement offer the advantage that they can be installed without any problem through the locally encountered dense sand layers. In this situation the use of screw piles with full soil displacement was not possible because they cannot be installed through the locally encountered very dense sand layers.

Figure 2: CPT's at a site in Brussels



4 TYPICAL CASES STUDIES

4.1 *Screw piles on the site Brussels-South*

4.1.1 *Design of the piles*

When designing the CFA piles with an auger with enlarged central tube, a comparative study has been performed of different design methods :

- French DTU 13.2 using cone penetration tests (CPT) (DTU 13.2 1992)
- French DTU 13.2 using cone penetration tests for the shaft resistance and pressiometertests for the base resistance (DTU 13.2 1992)
- Belgian practice according to ERTC3, Holeyman et al (1997)
- Belgian practice according to ERTC3, with $S_s = 2$ instead of 3

4.1.2 *DTU 13.2 using CPT*

The determination of the friction Qs is done as follows:

$$Q_s = \pi \cdot D \cdot \sum_i H_i \cdot \frac{q_{ci}}{\alpha_i}$$

With:

D	:	The diameter of the pile
H_i	:	The pile height in the bearing layer
q_{ci}	:	The cone resistance
α_i	:	A friction coefficient according to table IV of the DTU 13.2

The pile-type used is ‘pieu foré, fût béton’. When two values are mentioned in the table the one within brackets is used. This supposes that the piles are executed with the necessary care and with an execution method that results in minimal soil disturbance. Compared with bored piles we suppose that this is the case for screwed piles with partial soil displacement.

The determination of the base resistance Q_p is done as follows:

$$Q_p = K_c \cdot q_c \cdot \Omega$$

With:

K_c	:	The average bearing factor of bored and full soil displacement piles
Ω	:	The surface of the pile base

4.1.3 *DTU 13.2 with base resistance deduced from pressiometertest*

The determination of the base resistance Q_p is done as follows :

$$Q_p = k \cdot p_l \cdot \Omega$$

With:

k	:	The bearing factor dependant of the pile type and equivalent depth H_i
p_l	:	The limit pressure of the pressiometertest
Ω	:	The pile surface

As a CPT and a boring with pressiometer tests have been performed into the homogeneous clay layer in the vicinity of the project site, the values of limit pressure p_l could be extrapolated from the cone resistance values q_c. The soil category 2 is used and the bored pile type ‘pieu foré’, resulting in a bearing factor k = 1.6. The obtained base resistance is higher than that obtained using the CPT method.

Our experience has shown that, in homogeneous layers, when calculating the base resistance Q_p using the limit pressure p_l of the pressiometertest, significantly higher values of Q_p are obtained compared to Q_p calculated with CPT tests as described before. The shaft friction Q_s in general is about the same using both methods.

The reason for this has to be found in the fact that in France nearly always pressiometertests are used for the determination and design of foundations. This results in a vast experience and confidence in this design method. Cone penetration tests, on the contrary, are, till recently, not so often used and are not always executed using 20 ton penetration apparatus. Furthermore the geology in France does not always permit the use of the CPT.

As a result the bearing factor K_c contains a large amount of safety taking into account this lack of experience. It also automatically considers a small anchorage of the pile in the bearing layer, while in the pressuremeter method an equivalent anchorage depth H_e is used for the determination of the bearing factor k for the base resistance.

4.1.4 *Belgian practice according to ERTC3*

During the ERTC3 conference of 1997 an overview of the currently used Belgian design principles of pile foundations was given by Holeyman e.a. (1997). This is now generally used as a reference document. The described design method will hereafter be referred to as "Belgian Practice, ERTC 3 (1997)".

The determination of the friction R_{su} can be done in different ways:

- Using the total shaft friction ΔQ_{st} of the cone penetration test:

$$R_{su} = \frac{X_s}{\pi \cdot d} \cdot \sum_i \xi_{fi} \cdot \Delta Q_{sti}$$

- Using the cone resistance values q_c (same principle as DTU13.2):

$$R_{su} = X_s \cdot \sum_i H_i \cdot \xi_{fi} \cdot \eta^*_{pi} \cdot q_{ci} = X_s \cdot \sum_i H_i \cdot \xi_{fi} \cdot q_{si}$$

The base resistance R_{bu} can be calculated as :

$$R_{bu} = \alpha_b \cdot \varepsilon_b \cdot A_b \cdot q_b$$

with $\alpha_b = 0{,}80$; $\xi_{fi} = 0{,}80$ and $\varepsilon_b = 1{,}00$

The allowable load[1] $Rcal$ is calculated as:

$$R_{cal} = \frac{R_{bu}}{2} + \frac{R_{su}}{3}$$

4.1.5 *Belgian practice according to ERTC3 with $S_s = 2$*

The formulae from par. 4.1.4. are used. The allowable load R_{cal} is calculated as:

$$R_{cal} = \frac{R_{bu}}{2} + \frac{R_{su}}{2}$$

4.1.6 *Example*

As an example, the results of the calculation of the allowable load according to the different methods for a representative cone penetration test (figure 3) are shown in table 1. The diameter of the piles used here is 0,60m. The piles are founded on level +3.00.

It can be concluded that:

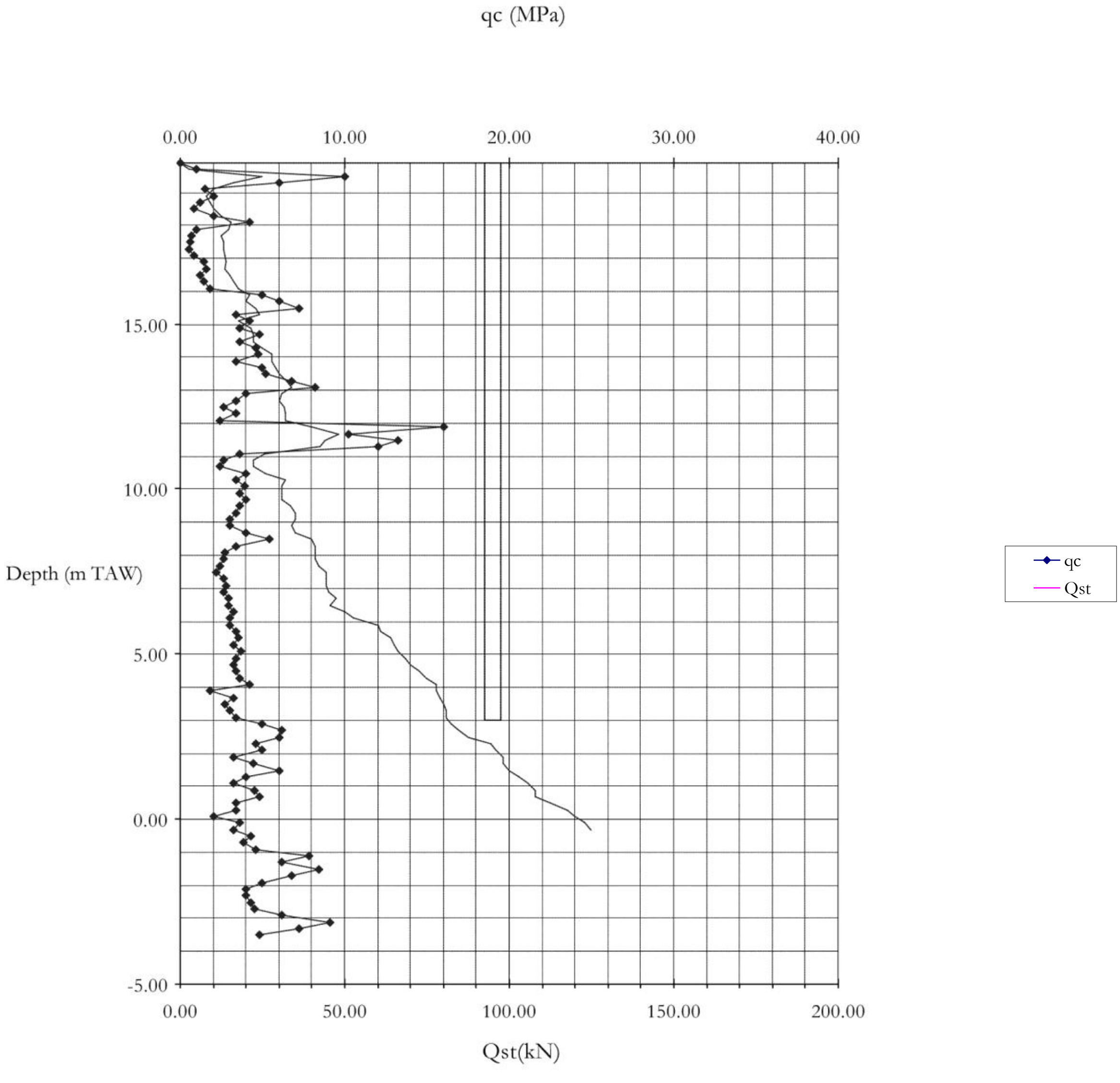
- the allowable load calculated according to ERTC3 using the qc-values is significantly higher than that calculated using the total shaft resistance ΔQ_{st};
- the allowable load calculated according to DTU 13.2 is significantly higher than that calculated according to ERTC3;
- the allowable load calculated according to DTU 13.2 with the base resistance from pressiometertest is equivalent to the calculated resistance according to ERTC3 with the safety factor S_s on the friction resistance R_{su} equal to 2.

[1] The allowable load is the maximum load a pile can withstand in the serviceability limit state.

Table 1: Allowable load of CFA piles (diameter 60 cm) with an auger with enlarged central tube (PSC Lambda pile) founded at level +3.00, calculated using CPT from figure 3

Design method	Allowable load
DTU 13.2	920 kN
DTU 13.2, base resistance from pressiometertests	1075 kN
Belgian practice, ERTC3	
- from total shaft resistance	687 kN
- from qc-values	841 kN
Belgian practice, ERTC3 with Ss = 2	
- from total shaft resistance	833 kN
- from qc-values	1063 kN

Figure 3 : CPT



It was decided to determine the allowable load of the piles as an average value of the allowable load determined by the DTU 13.2 methods and the ERTC3 with $S_s = 2$. To check this, pile loading test were to be performed before execution of the foundation piles.

4.1.7 *Pile loading tests*

Three pile loading tests have been performed according to the method described in Bustamante & Jezequel (1989).

The pile loading program consisted of 8 equal steps of 1 hour, followed by 4 unloading steps of 5 minutes. The maximum load equals 1.7 times the design load[2] of the pile. Only the load and the head displacement is measured. The tests were performed by the contractor and suppervised.

The test results and the CPT's near to each tested pile are assimilated in appendix A-C. The results of one test are shown in figure 4.

The allowable load is determined using the method as described in Bustamante & Jezequel (1989).

The allowable loads, determined from each test, along with the theoretical allowable loads calculated using the methods mentioned before are summarized in table 2.

Figure 4: Test results of loading test on pile 1

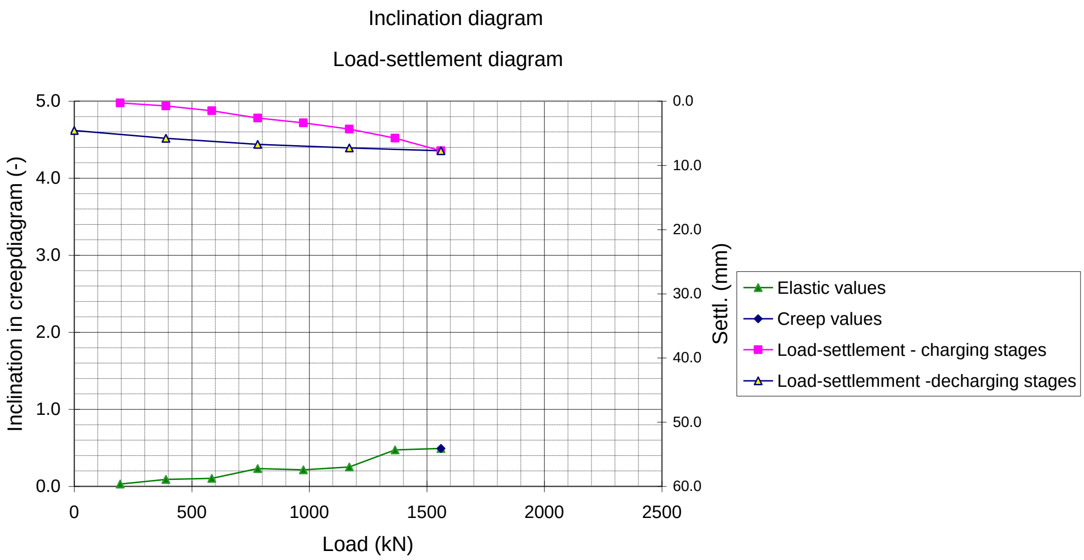


Table 2: Allowable load of screwed piles with partial soil displacement, diameter 60 cm, determined from pile loading tests and calculated using different methods

Design method	Allowable load		
	Pile 1	Pile 2	Pile 3
Determined from test results	>1248 kN	1075 kN	1337 kN
DTU 13.2	1110 kN	920 kN	1106 kN
DTU 13.2, base resistance from pressiometertests	1230 kN	1075 kN	1298 kN
ERTC3 ($\alpha_b = 0{,}80$; $\xi_{fi} = 0{,}80$ and $\varepsilon_b = 1{,}00$)			
- from total shaft resistance	743 kN	687 kN	721 kN
- from qc-values	922 kN	841 kN	911 kN
ERTC3 with Ss = 2			
- from total shaft resistance	934 kN	833 kN	878 kN
- from qc-values	1202 kN	1063 kN	1163 kN

[2] The design load is the maximum load on the pile due to the structure and the related loads.

4.2 *Screw piles on the site Leuven*

On the project site the subsoil consists of 6 m of very compressible clay-peat layers underlain by a resistant sand layer containing hard sandstone layers. Since the cone penetration tests executed during the preliminary site investigation showed this layer to be rather irregular in nature it was preferred to use piles with a fixed foundation level assuring an adequate bearing capacity even for the most negative cone penetration test. As the piles needed to be installed through intermittent resistant layers CFA piles with an auger with enlarged central tube (PSC Lambda piles) were used.

The CPT taken on the site is shown in figure 5. The piles used here have a 40cm diameter and are founded on level +19.00.

A pile loading test was performed similar to the ones described above. The allowable load, determined from the test, together with the theoretical allowable loads calculated using the before-mentioned methods are summarized in table 3.

Figure 5: CPT

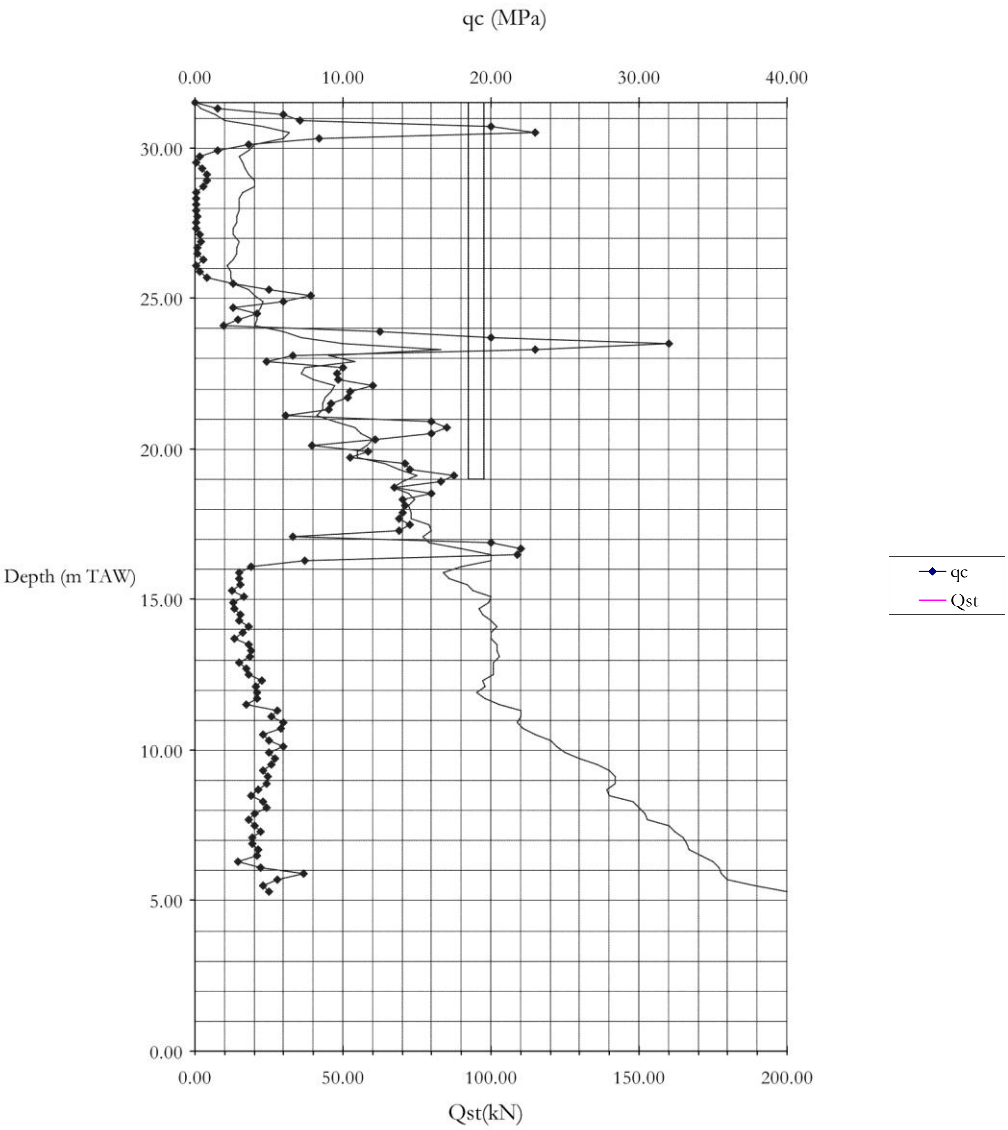


Table 3: Allowable load, determined from the test, together with the theoretical allowable loads calculated using the before-mentioned methods – pile diameter 40 cm

Design method	Allowable load
Determined from test results	> 816 kN
DTU 13.2	508 kN
DTU 13.2, base resistance from pressiometertests	--
ERTC3	
- from total shaft resistance	551 kN
- from qc-values	524 kN
ERTC3 with Ss = 2	
- from total shaft resistance	626 kN
- from qc-values	586 kN

It can be seen that:

- the allowable load calculated according to ERTC3 using the qc-values is similar to that calculated using the total shaft resistance ΔQ_{st};
- the allowable load calculated according to DTU 13.2 is similar to or lower than that calculated according to ERTC3;
- the allowable load determined from the pile loading test is significantly higher than the calculated capacities. This is often observed in this sand layer having intermittent harder layers and sandstone layers.

5 APPLIED RULES TO DESIGN THE SCREW PILES AND OTHER PILES

The experience with different static loading tests and the comparison of different application documents as discussed in the previous paragraph led to generalization of the design method to all types of piles. The pile foundations for the high-speed railway structures of the last 4 years were designed with this method. Because lower safety factors are taken into account static loading tests on the installed piles are always planned.

5.1 *Calculation of the ultimate resistance of compression piles*

For the calculation of the ultimate resistance of compression piles two methods, depending on the available ground investigation tests are used.

When pressiometer tests are available the design method according to DTU 13.2 is used or the method according to the Fascicule 62 (this method is generally similar to the DTU 13.2 and is in fact to be preferred for the design of foundations for infrastructure works).

When only CPT's are available, a method similar to the Belgian practice is used. In this paragraph the installation factors for the different piles types and the safety factors are set.

5.1.1 *Ultimate base resistance R_{bu}*

The following basic formula is used:

$$R_{bu} = \alpha_b \cdot \varepsilon_b \cdot A_b \cdot q_b$$

With:

q_b : Ultimate unit pile base resistance derived from the CPT results in the natural ground conditions

α_b : An empirical factor taking into account the method of installation of the pile and soil type

ε_b : A parameter referring to the scale dependant soil shear strength characteristics (e.g. in case of fissured clay)

= $1 - 0{,}01\left[\dfrac{D_b}{D_c} - 1\right]$ for fissured clay but always > 0,476

= 1 in other cases

with D_b = diameter of pile base and D_c = Diameter of cone (CPT = 35,7mm)

β : A shape factor introduced for non circular or square-shaped bases

$$= \frac{1+0{,}3\cdot B/L}{1{,}3}$$ met B = width and L =Length of rectangular base

β =1 for circular piles

A_b : The nominal pile base cross-sectional area

In table 4 the values of α_b used for the different types of piles are summarized.

Table 4: the values of α_b used for the different types of piles

Pile type	α_b
Driven piles	1,00
Bored piles in sand and loam	0,50
Screw piles with full soil displacement	0,80
Screw piles with partial soil displacement	0,67
Bored and screw piles in over-consolidated and fissured clay	0,80

5.1.2 *Ultimate shaft resistance R_{su}*

The ultimate shaft resistance can be calculated with two formulae:

- Using the total side fricton ΔQ_{sti} from CPT

$$R_{su} = \frac{\mathrm{X}_s}{\pi \cdot d} \cdot \sum_i \xi_{fi} \cdot \Delta Q_{sti}$$

With:

ξ_{fi} : An overall empirical factor introducing the effects of the pile installation method, the nature of the shaft's material and roughness and soil structure scale effects

X_s and $\pi \cdot d$: The perimeter of the pile shaft and the perimeter of the sounding rod, respectively

ΔQ_{sti} : The CPT total side friction increment in the shaft bearing layer(s)

In table 5 the values of ξ_{fi} used for the different types of piles are given.

Table 5: the values of ξ_{fi} used for the different types of piles

Pile type	ξ_{fi}
Driven piles	0,67
Bored piles	0,50
Screw piles with full soil displacement	0,67
Screw piles with soil displacement and enlarged central tube	0,60

- Using cone resistance q_c :

$$R_{su} = \mathrm{X}_s \cdot \sum_i H_i \cdot \xi_{fi} \cdot \eta^*_{pi} \cdot q_{ci} = \mathrm{X}_s \cdot \sum_i H_i \cdot \xi_{fi} \cdot q_{si}$$

With:

$\sum_i H_i$: The height of the layer

ξ_{fi} : An overall empirical factor introducing the effects of the pile installation method, the nature of the shaft's material and roughness and soil structure scale effects

q_{si} : The ultimate unit shaft friction calculated as :

$\eta^*_p \cdot q_c$

with :

η^*_p = see Belgian Practice, ERTC 3 (1997)

q_c = The cone resistance (MPa)

In table 6 the values of ξ_{fi} used for this method are given.

Table 6: the values of factors ξ_{fi} used for the different types of piles

Pile type	ξ_{fi}
Driven piles	1,00
Bored piles	0,67
Screw piles with full soil displacement	1,00
Screw piles with partial soil displacement and enlarged central tube	0,80

5.2 *Determination of the allowable load*

The total ultimate resistance in compression of the pile is calculated as:

$$R_{cu} = R_{bu} + R_{su}$$

The creep load of the pile is set as:

$$R_{creep} = \frac{R_{cu}}{1,4}$$

The allowable load of the pile is then calculated as:

$$R_{cal} = \frac{R_{creep}}{1,4} = \frac{R_{cu}}{1,96}$$

5.3 *Static loading test*

As the design method is not totally in accordance with the common Belgian practice, static loading tests on foundation piles are always provided in the specifications. When a large number of piles is to be executed or a certain doubt on the execution is present, these loading tests are always performed. This is done by the contractor and supervised.

Normally the loading tests are done to control the bearing capacity of an executed pile within the foundation. Therefore the maximum load to be applied is limited till 1,7 times the design load of the pile. Instead of testing a pile within the foundation, in some cases test piles are executed on a test site near the foundation. These piles are tested till rupture of the pile. This rupture is theoretically set at a displacement of 10% of the pile diameter.

5.4 *Comparison design method with test results*

In table 7 the calculated values calculated using the design method described before are compared with the test results from the static loading test of paragraph 4.

Table 7: Comparison calculated allowable loads with allowable loads deduced from test results

Allowable load	Pile 1	Pile 2	Pile 3	Pile Leuven
Calculated value (R_{su} from Q_{st})	747 kN	673 kN	709 kN	515 kN
Calculated value (R_{su} from q_c)	1166 kN	1019 kN	1119 kN	531 kN
Value deduced from test results	>1248 kN	1075 kN	1337 kN	>831 kN

As indicated in table 7, the allowable load sdeduced from the test results are systematical lyhigher than the ones calculated with the design method described before.

6 PILE LOAD TEST AT LOENHOUT

6.1 *Geology and situation on the site*

Loenhout is a village situated between Antwerp and Breda. The subsoil at Loenhout consists of quaternary sand covered by an alluvial clay layer with alternating sandy layers in the upper layers (formation of Campine). The stratification at the site is presented in table 8.

Table 8: Stratification at the site

Soil	Depth	Geological
Alluvial clay with alternating sandy layers	0,00 – 3,50	Quaternary clay-sand
Alluvial sand with clay	3,50 – 5,00	Quaternary Sand
Alluvial clay	5,00 – 9,00	Quaternary clay
Alluvial sand	9,00 –……	Quaternary sand

The upper soil layer is very weak and compressible so that large settlements are to be expected. To avoid this, a large amount of screw piles with full soil displacement are installed at the site as ground improvement piles. The basic idea is show in figure 1b.

Locally certain ducts for the passage of water or for gas pipelines are necessary. To protect these structures passing under the high-speed railway, technical blocks in stabilised material are executed. As the screw pile with full soil displacement can't be drilled through such stabilized materials, CFA-piles with casing have to be executed.

As many screw piles with full soil displacement are to be installed and the CFA-pile with casing has never been tested for use as a foundation pile, both pile types have been tested by means of a static load test. These tests have been performed on a small test site nearby and both piles have been loaded till rupture. By this means a good comparison on the behavior of the two piles is possible.

Both piles were equipped and tested by the BBRI.

6.2 *Installation details on the test piles*

The screw pile with full soil displacement is installed according to the description given in Huybrechts (2001) for the Omega Pile (41/41). In contradiction to this installation procedure, the auger was not rotating while concreting the first 5m of the pile. This pile is installed by Socofonda. In figure 6 the installation of the omega pile is shown.

The CFA pile with casing is installed as follows:

- Positioning of the lost bottom point and auger ;
- Screwing in of the auger and the casing at the same time, rotating in opposite directions. In this phase a torque and a low vertical force is set on the casing and auger. Both are drilled by independent drilling tables, situated at the top of the casing. The tube and auger are drilled in continuously. Depending on the resistance of the soil the distance between the base of the auger and the bottom of the casing can be changed within certain limit. Normally the base of the casing is deeper than the base of the auger so the soil is removed within the casing. For resistant soils and soils containing stones the base of the auger and the bottom of the casing are at the same level so the stones can be crushed between the casing and the auger.
- Once the casing and the auger are at the pre-set level, concrete is pumped through the central tube of the auger and the auger is withdrawn. Depending on the system the auger and the casing can be pulled together or independently.

  In this case the auger is pulled up over 5m independently while the casing stays at the bottom of the pile. The casing is pulled up afterwards. When the casing is pulled up, it always stays under the upper level of the concrete to avoid ground inclusion in the pile shaft. Casing and auger are pulled up with a counter clockwise rotation.

- As it concerns a closed concreting system under pressure, the reinforcement cage has to be brought into the fresh installed concrete afterwards.

This pile (CFA-pile with casing) has a nominal diameter for shaft and base of 610mm which is the external diameter of the casing. The pile was installed by Franki Geotechnics Belgium. Figure 7 shows the installation of the CFA-pile with casing.

Figure 6: Installation of a omega pile

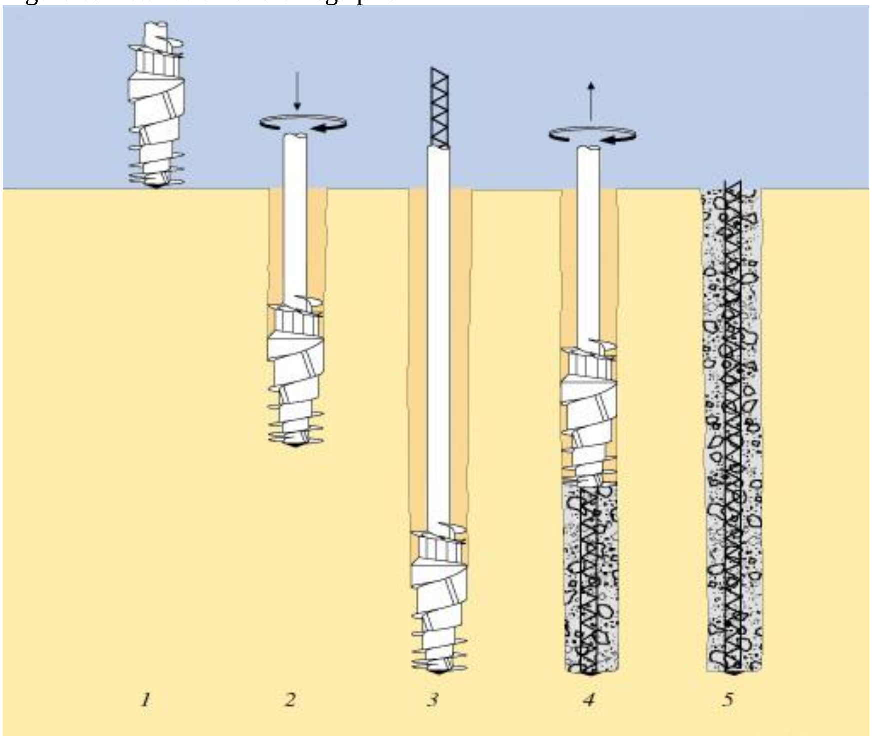
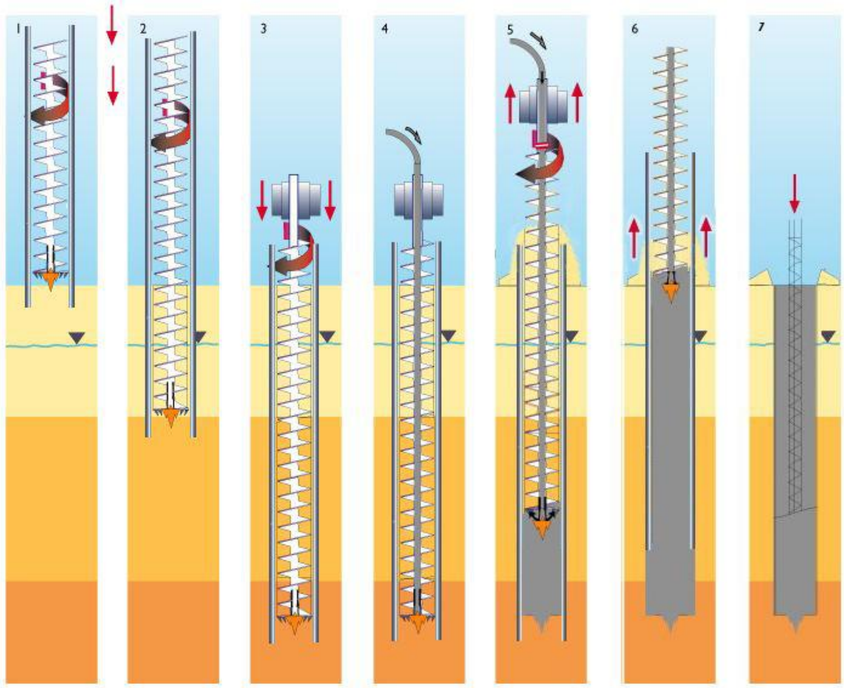


Figure 7: Installation of CFA-pile with casing

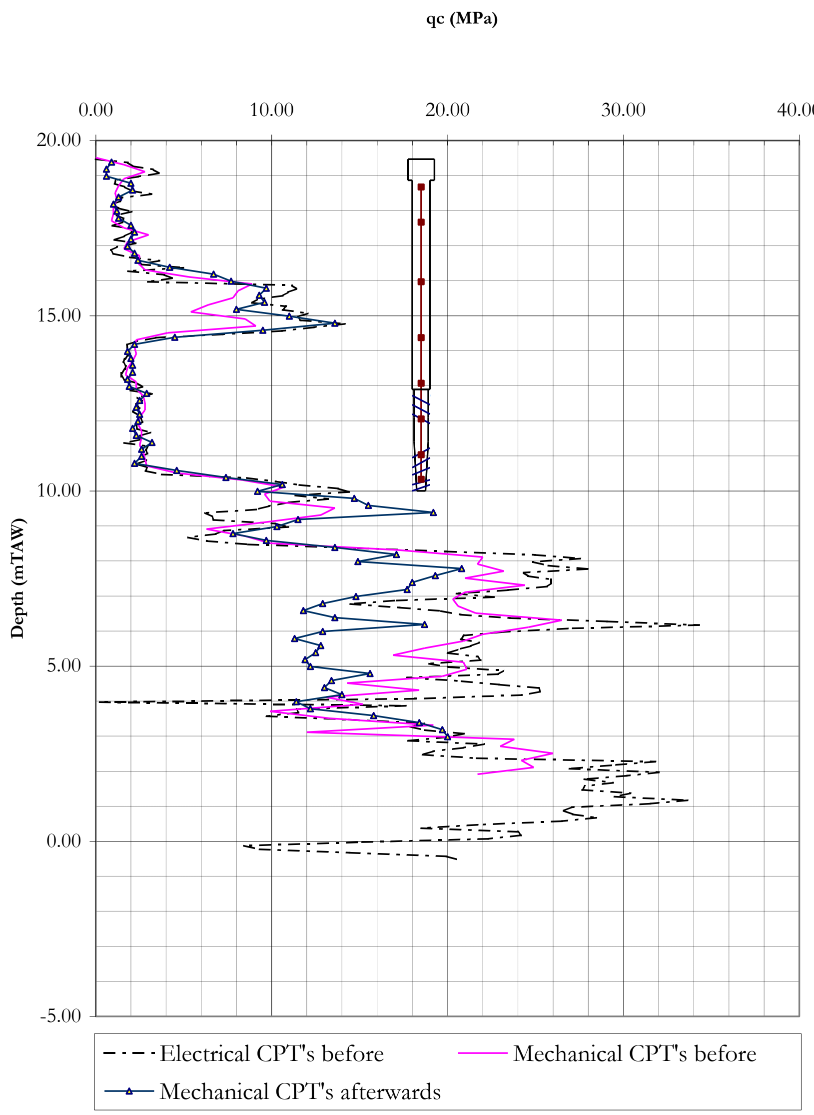


6.3 *Site investigation*

At the test site 2 electrical CPT's and 3 mechanical discontinuous CPT's are performed. The electrical CPT's are performed in the centre of the two test piles.

After execution of the static load tests, two CPT's are done aside each pile shaft, at 0,70m from the center of the pile. The average value of the measured cone resistance before and after installation of the pile is shown in figure 8 a en 8b.

Figure 8a: CPT's before and after installation of the omega pile



6.4 *Static loading tests*

The test piles are installed according to the installation procedures described above. The installation parameters are given in table 9.

Table 9: Installation parameters of the test piles

Pile	Installation date	Test date	Age	Pile head	Pile Length	Diameter
Omega-Pile	9/10/2002	28/11/2003	50 days	+19,47	9,47 m	0,410 m
CFA pile with casing	21/10/2002	5/12/2003	45 days	+19,51	10,40 m	0,610 m

After installation of the piles the pile heads are prepared as described in Huybrechts (2001).

Each test pile is equipped with a central tube containing a retrievable extensometer system at 7 different levels. The different levels for each pile are shown in figure 8.

The tests are performed according to the method described by Bustamante & Jezequel (1989) and as described in Maertens & Huybrechts (2001).

The total allowable load of the piles is calculated according to the calculating method described before. The result of these calculations and the estimated rupture load of each pile are given in table 10.

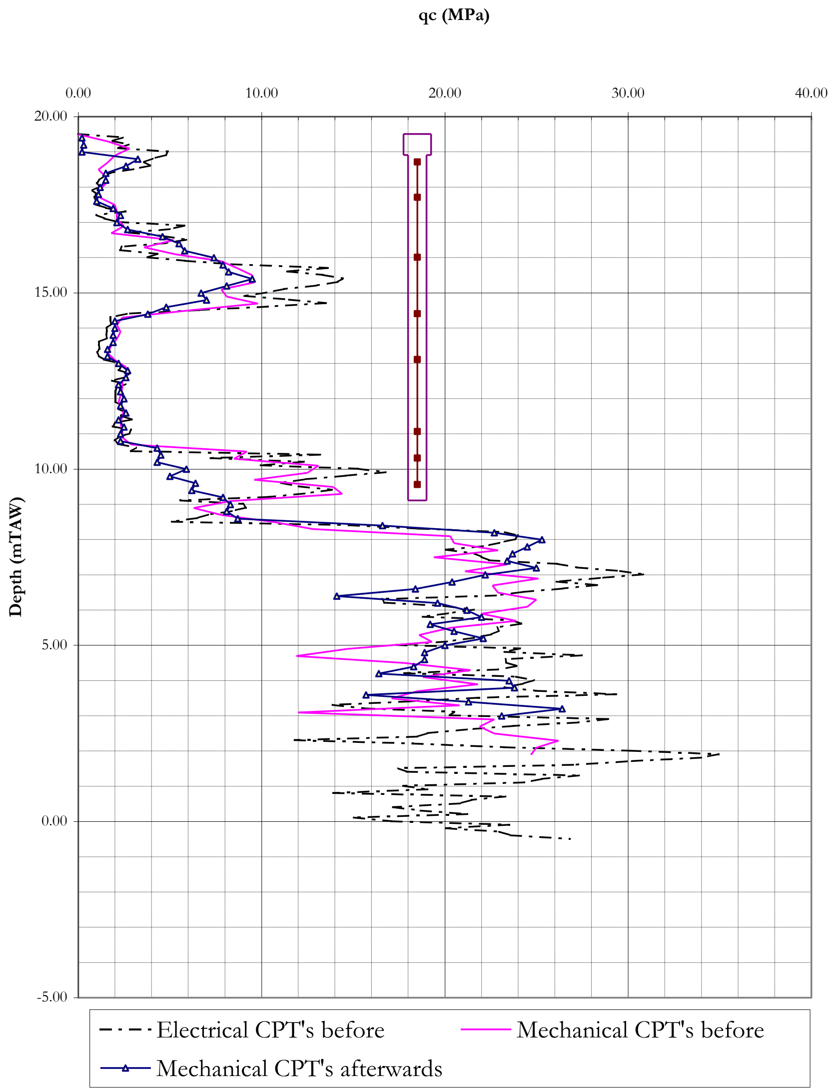
Table 10: The calculated ultimate bearing resistance and allowable load and the estimated rupture load of the test piles

Pile	R_{bu}	R_{su}	R_{cu}	R_{cal}	Estimated rupture load
Omega-pile	423 kN	420 kN	843 kN	430 kN	1300 kN
CFA-pile with casing	920 kN	962 kN	1882 kN	960 kN	2000 kN

The test load was adapted considering the high cone resistance at the base of the piles.

The static load is put on the pile by a hydraulic jack and a reaction massif consisted of containers filled with water. The whole system is carried by supports, placed according to the prescription of the ISSMFE (1985). A view on the dead load system is given in figure 9.

Figure 8b: CPT's before and after installation of the CFA pile with casing



6.5 *Interpretation of the test results*

The test results are interpreted as described in Bustamante & Jezequel (1989) and Maertens & Huybrechts (2001). The conventional rupture load is set at a settlement of 10 % of the pile diameter. The allowable load $R_{cal,test} = 0{,}80 \cdot Q_c$.

Figure 9: The dead load system

The results of the static load test are shown in appendix E-F. In table 11 the conventional rupture load, the creep load and the allowable load with according settlement of the test piles are resumed.

Table 11: The conventional rupture load, the creep load and the allowable load with corresponding settlement of the test piles

Pile	$Q_{10\%D_b}$	Q_c	$0{,}80 \cdot Q_c$	$\dfrac{Q_{10\%D_b}}{0{,}80 \cdot Q_c}$	s_{0l} (For $0{,}80 \cdot Q_c$)
Omega pile	1170 kN	922 kN	738 kN	1.59	7,44 mm
CFA-pile with casing	1314 kN	806 kN	645 kN	2,04	3,85 mm

As the piles are equipped with retrievable extensometers the mobilized friction on the pile shaft could be measured. The unit frictions mobilized at the conventional rupture load are given in tables 12 and 13.

Table 12: Unit friction mobilized at conventional rupture load – Omega pile

Extensometer level	Level	Soil	$q_{c,average}$	$q_{su,10\%D_b}$	$\xi_f.\eta_p^*$
Ext. 7	+18,67 – +17,67	Clayey sand	1,66 MPa	25 kPa	1/66
Ext. 6	+17,67 – +15,97	Clayey sand	2,27 MPa	64 kPa	1/35
Ext. 5	+15,97 – +14,37	Sand	9,98 MPa	147 kPa	1/68
Ext. 4	+14,37 – +13,07	Clay	1,90 MPa	42 kPa	1/45
Ext. 3	+13,07 – +12,05	Clayey sand	2,52 MPa	58 kPa	1/43
Ext. 2	+12,05 – +11,03				
Ext. 1	+11,03 – +10,33	Clayey sand - sand	3,60 MPa	78 kPa	1/46

Table 13: Unit friction mobilized at conventional rupture load – CFA-pile with casing

Extensometer level	Level	Soil	$q_{c,average}$	$q_{su,10\%D_b}$	$\xi_f.\eta_p^*$
Ext. 7	+18,71 – +17,71	Clayey sand	1,79 MPa	19 kPa	1/94
Ext. 6	+17,71 – +16,01	Clayey sand	2,95 MPa	46 kPa	1/64
Ext. 5	+16,01 – +14,41	Sand	10,12 MPa	95 kPa	1/107
Ext. 4	+14,41 – +13,11	Clay	1,51 MPa	64 kPa	1/24
Ext. 3	+13,11 – +11,06	Clayey sand	2,23 MPa	76 kPa	1/29
Ext. 2	+11,06 – +10,31	Clayey sand - sand	4,45 MPa	90 kPa	1/49
Ext. 1	+10,31 – +9,56	Sand	12,39 MPa	18 kPa	1/688

Table 14 gives the ratio between the load taken by the pile base and shaft and the conventional rupture load of the pile

Table14: Ratio between the load taken at the pile base and shaft and the conventional rupture load of the pile

Pile	$Q_{10\%D_b}$	$R_{bu,test}$	$R_{su,test}$	$\frac{R_{bu,test}}{Q_{10\%D_b}}$	$\frac{R_{su,test}}{Q_{10\%D_b}}$
Omega pile	1170 kN	372 kN	798 kN	31,8 %	68,2 %
CFA-pile with casing	1314 kN	205 kN	1109 kN	15,6 %	84,4 %

The load settlement curves for the mobilized resistance at the base and the shaft of the pile are given in figure E4 and figure F4 of appendix E and F.

6.6 *Further interpretation and discussion*

Omega pile :

From the results presented in previous paragraphs one can notice that:

- The CPT's, done before and after execution of the pile, are more or less the same over the whole length of the pile. Beneath the pile a reduction in cone resistance is noticed over 6m beneath the pile base. No reasonable explanation can be put forward for this phenomenon. (see figure 8a)
- The disturbance due to execution of the pile is higher in the more sandy layers than in the clayey layers. (see figure 8a)
- The conventional rupture load at the base $R_{bu,test}$ is slightly lower than the calculated R_{bu} (-12 %) (see table 10 & 14)
- The conventional rupture load on the shaft $R_{su,test}$ is much higher than the calculated R_{su} (+90 %) (see table 10 & 14)
- The global safety factor calculated from the test results (S = 1,59) (see table 11) is lower than the proposed safety factor in paragraph 5.2 (S = 1,96). As the piles are used as groundimprovement piles the safety factor 1,59 could be accepted. For foundation piles the safety factor 1,96 should be taken.
- The scaled average of the factor $\xi_f.\eta_p^*$ for the alluvial layer is 1/47. With $\xi_{f.} = 0{,}80$ as proposed in Maertens & Huybrechts (2001) & (2003) the $\eta_p^* = 1/38$.

CFA-pile with casing :

From the results presented in previous paragraphs one can notice that:

- The CPT's, done before and after execution of the pile, are more or less the same except for 1,50m above the pile base. A reduction with a factor 2 is noticed. (see figure 8b)
- The conventional rupture load at the base $R_{bu,test}$ is much lower than the calculated R_{bu} (-78 %) (see table 10 & 14)
  This indicates that during the installation of the pile, the soil around the pile base has been loosened. This may have been caused by the fact that water could drain out of the fresh concrete within the casing at the bottom of the pile base. In this way the concrete could stick on the casing when the casing was lifted up.

As the “sealed” casing is pulled up a vacuum around the base occurs which provokes a suction effect till the concrete releases the tube. By this means the soil around the pile base is completely decompressed which results in a low bearing capacity at the pile base.
- From the results of CPT tests performed after the execution of the static load test it can be clearly seen that the soil around the pile base is loosened.
- The conventional rupture load on the shaft $R_{su,test}$ is slightly higher than the calculated R_{su} (+15 %) (see table 10 & 14)
- The global safety factor calculated from the test results (S = 2,03) (see table 11) is similar to the proposed safety factor in paragraph 5.2 (S = 1,96). This is mainly due to the low bearing capacity at the pile base.
- The scaled average of the factor $\xi_f.\eta_p^*$ for the alluvial layer is 1/44. With $\eta_p^* = 1/38$ as calculated for the omega pile, the installation factor for the shaft friction $\xi_{f.} = 0{,}86$.

The piles will be excavated to control the pile’s diameters and the stiffness of the pile material.

7 CONCLUSION

Comparison between calculated allowable loads of screwed piles with partial soil displacement using the traditional Belgian practice and the DTU 13.2 lead to different allowable loads.

The traditional Belgian practice, using a safety factor S_s = 3, seems to lead to a conservative design. Using a safety factor S_s = 2 instead of 3 a bearing capacity similar to the one calculated with the DTU 13.2 is obtained.

The results of 4 pile loading tests on screw piles with partial soil displacement (PSC Lambda piles) in Brussels and Leuven confirm that the traditional Belgian practice is conservative. The results of these tests, together with those of other tests done on different pile types, have been used to elaborate an alternative design method with other empirical factors and a safety factor S_s = 2.

For the last 4 years the pile foundations of the HST-project have been designed with this method. Since it is not according to the existing Belgian practice, pile loading tests are regularly performed on different pile types.

Two of those pile loading tests have been performed till rupture. They confirm the proposed factors ξ_f and η_p* for the shaft friction of the 2 pile types and the factor α_b for the base resistance of the screwed pile with full soil displacement. The factor α_b for the base resistance of the CFA pile with casing is similar to that of a bored pile. Additional static load tests are necessary to determine if the low value of the base resistance is due to the specific installation with independent uplift of the auger and the casing or if this low value is typical for all CFA-piles with casing.

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