# A full-scale test on large diameter bored piles for the construction of the HST-tunnel in Antwerp (Belgium)

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ABSTRACT: For the high-speed railway network in Belgium, a tunnel is being made under the city of Antwerp. The first part of this tunnel is made according to the cut and cover method below an embankment with monumental retaining walls and bridges. Temporary large diameter bored piles are installed to support the covering structure during the execution period of the tunnel. Full-scale loading tests were executed to determine the behaviour of the bored piles in the most critical phase of excavation.
In this article the instrumentation and execution of the test piles as well as the interpretation of the results of the loading tests are given. According to the test results the piles were redesigned.

## 1 INTRODUCTION

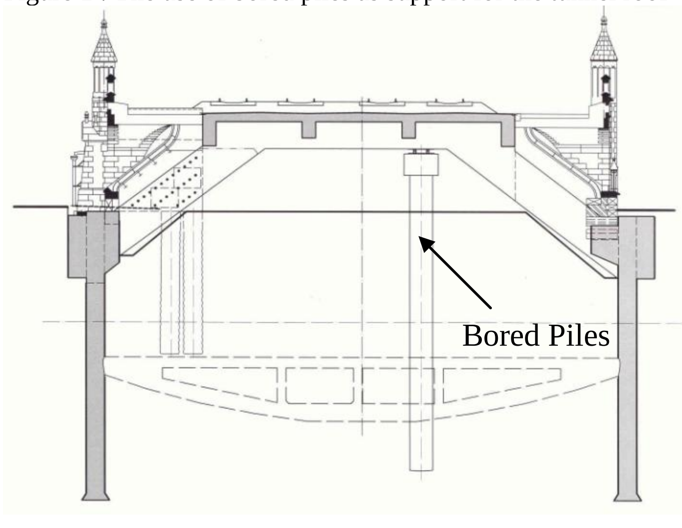
For the realisation of the High-Speed Railway link between Brussels and Amsterdam a tunnel underneath the city of Antwerp needs to be built.

Until now the central station of Antwerp was an end-station and was used at full capacity. As for the future much higher capacity needs are expected an expansion of the station was indispensable. Because of the urbanised environment of the station, a subterraneous expansion under the station was designed. Underneath the station two levels are being constructed and will be accessible by a tunnel.

In the southern part of the city the tunnel has to be constructed beneath a 100 years old railway embankment containing several bridges. This railway embankment is devoted national heritage and therefore the retaining walls and bridges need to be maintained.

In a first phase the walls of the embankment are underpinned and the roof of the tunnel is constructed between the two retaining walls. In the further phases of the construction of the tunnel the tunnel roof is used as access to the central station of Antwerp. Because of this and because the tunnel roof is constructed in 2 phases, temporary supports for the tunnel roof were necessary. Therefore large diameter bored piles are used. (See figure 1) As the bored piles had to be installed through rather thick fill layers the use of a casing was imposed in the tender documents over the full height of the piles.

Figure 1 : The use of bored piles as support for the tunnel roof




As important loads had to be taken by the piles, post-injections on some piles were to be performed to increase the bearing capacity.

In order to optimise the design of these piles it was decided to perform 2 static loading tests. Because the pile loads are very important, special arrangements needed to be taken to apply the test loads. In this contribution the installation procedure of the test piles and the test results are discussed.

## 2 GEOLOGY AND SITE INVESTIGATION

The geologic stratification at the site is resumed in table 1.

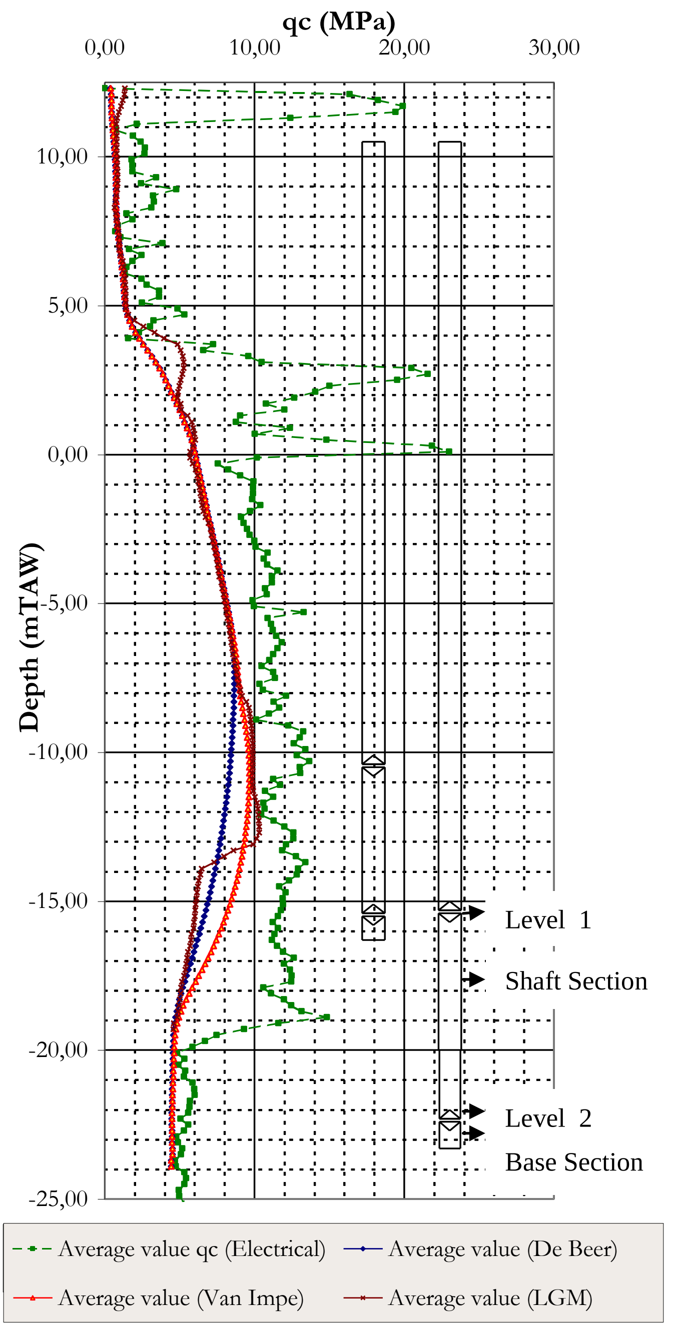
Table 1: Geologic stratification on the site

| Name | Soil type | Thickness |
|---|---|---|
| Top layer | Quaternary Loam | 3m |
| Formation of Berchem | Tertiary sand (40% Glauconite) | 23m |
| Boom Clay formation | Tertiary clay | >50m |

Before the execution of the test piles 2 electrical CPT's have been performed in the surroundings of the piles. The average of the measured cone resistance is given in figure 2.

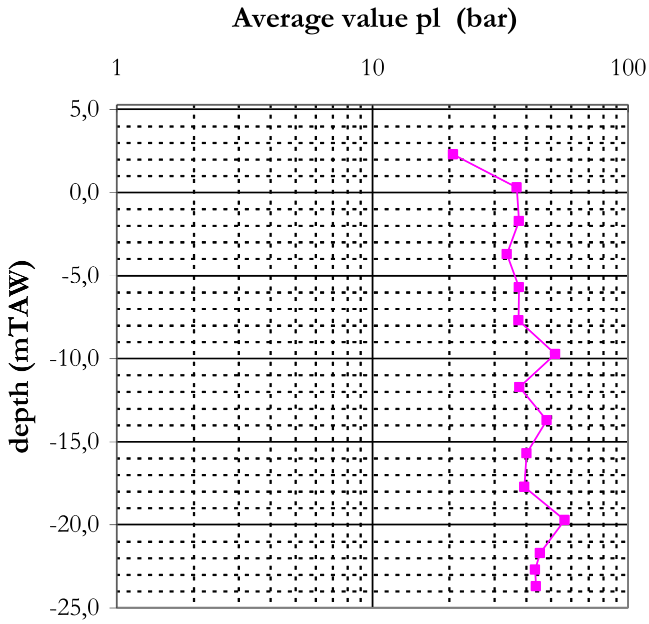
The calculated value of the ultimate unit base resistance according to the methods of De Beer, Van Impe and LGM are also given in figure 2.

Figure 2. CPT-test before execution of the test piles




In the area of the test site also results of 3 pressuremeter tests (Ménard type) are available. In figure 3 the average value of the measured limit pressure $p_l$ at different depths is given.

Figure 3: Pressiometer test




## 3 INSTALLATION OF THE TEST PILES

During the installation of the bored piles the soil is excavated under the protection of a steel casing. The excavation is done by means of a bucket. In the tender documents it was specified that the base of the casing had to be maintained at least 0,50m beneath the bottom of the excavation.

During execution this could not always be obtained and most of the times the base of the casing was almost at the same level as the bottom of the excavation. When drilling underneath the water table, an excess water level of at least 1m was maintained within the casing.

For the piles installed till level –23,00 the casing is fixed in the Boom clay. Further drilling is done without casing. In this way the pile diameter of the lower part corresponds to the diameter of the bucket (= 1,38m).

### 3.1 *Characteristics of the test piles*

To have a representative view on the bearing capacity of the different pile types to be installed, two test piles founded at different levels have been executed. As some piles had to be executed with a post-injection on the pile shaft one of the test piles was post-injected. The characteristics of the test piles are given in table 2.

Table 2: Characteristics of the test piles

| Pile | Foundation level | Diameter in sand | Diameter in clay | Post-injection of the shaft |
|---|---|---|---|---|
| Pile 1 | -16,3 mTAW | 1,50 m | - | - |
| Pile 2 | -23,3 mTAW | 1,50 m | 1,38 m | In sand |

The post-injection on pile 2 was only performed in the sand between level –14,4 and –20,0 mTAW. A total amount of 676 l grout (W/C = 0,5; with additives) was injected.

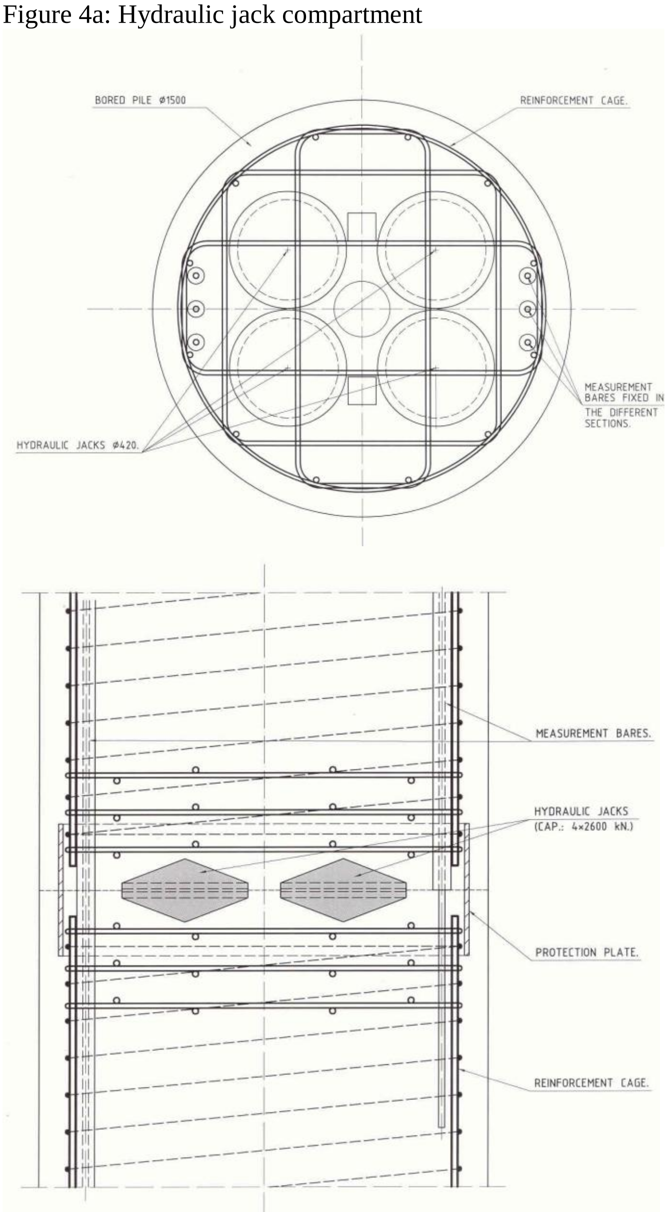
Due to the very high shaft friction in the upper part of the pile, it was not possible to perform the loading tests by applying a load on the top of the pile. Therefore hydraulic jacks with a total capacity of 1040 ton have been installed at two levels in the test piles. The levels of the hydraulic jacks are given in table 3 and figure 2.

Table 3: Levels of the hydraulic jacks

| Pile | Level 1 | Level 2 | Max. Ext./level | Cap./level |
|---|---|---|---|---|
| Pile 1 | -10,4 mTAW | -15,4 mTAW | 2x4,50 cm | 4x2600kN |
| Pile 2 | -15,3 mTAW | -22,3 mTAW | 2x4,50 cm | 4x2600kN |

At each level a compartment of 8 hydraulic jacks is placed as shown in figure 4a.

Figure 4a: Hydraulic jack compartment




In this way the pile is divided in three parts. The lower part is used to test the bearing capacity of the pile base, the middle part is used to test the bearing capacity of the pile shaft situated below the future excavation level, and the upper part is used to supply a reaction force during the execution of the tests. During the tests each part can move independently from each other and the displacement of each part is measured.

During installation of the reinforcement cage all tubes, bares and wires necessary to supply the hydraulic jacks and to measure the displacements are lengthened till the pile head. Figure 4b shows a view on the installation of the reinforcement cage

Figure 4b: Installation of the reinforcement cage

As the hydraulic jacks take a great part of the section of the pile the concreting of the pile had to be done with special care. To avoid the segregation of the concrete at the start of the concreting "ping-pong"-balls have been used to create a piston for the concrete in the tube. The first phase of the concreting is done very fast till the concrete reaches the level of the first hydraulic jacks. Until the concrete level passes the level of the upper jacks, the concreting is done with great prudence.

### 3.2 *Special arrangements*

As described before the test piles are equipped with hydraulic jacks at two levels. Because each of the pile segments needs to move independently no link between the segments can be allowed. Therefore the reinforcement cage needed to be interrupted.

Two steel rods have been anchored in each pile segment to measure the displacements. These rods have been lengthened till the pile head and embedded in special tubes to guarantee their free displacement in the concrete.

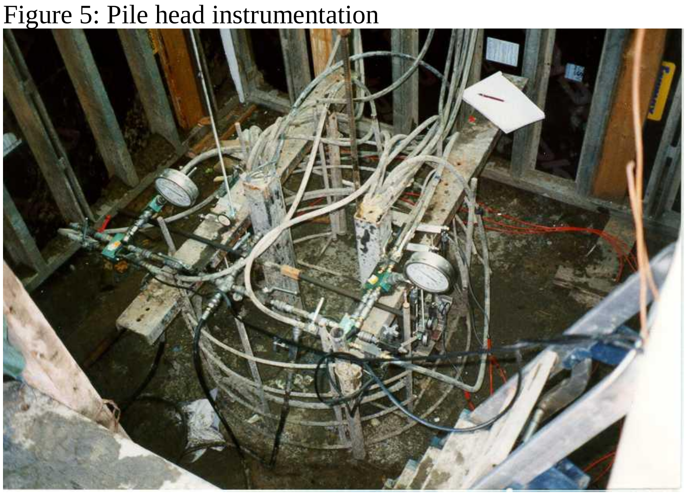
In order to measure the concrete pressure during concreting and to check the setting of the concrete, total pressure cells and piezometers have been fixed to the reinforcement cage 0,50 m below and above the level of the hydraulic jacks.

After installation of the piles a fissure is generated at the level of the hydraulic jacks by pressuring the jacks and generating a limited displacement of 2mm. To set the exact timing to perform this operation the temperature of the concrete during hardening was measured and multiplied with the time, starting from the beginning of the concreting. At a total of 1400°CH this operation is done. At this point the concrete normally has 50% of the maximum concrete resistance and a relatively small tensile strength.

Special arrangements are made to ensure that no soil can enter in the hydraulic jacks compartment. Therefore steel plates with a limited height were placed around the hydraulic jack compartments.

All displacement, pressure and temperature measurements are done with digital gauges and an automatic data acquisition system. A backup with analogue gauges has been taken. A view on the instrumentation of the pile head is given in figure 5.

Figure 5: Pile head instrumentation



## 4 PREDICTION OF THE ULTIMATE BEARING CAPACITY

The bearing capacity of the different segments of the piles has been estimated according to different methods.

### 4.1 *The ultimate resistance of the shaft section*

A first estimation of the ultimate shaft friction was done in accordance to Fasicule 62 (French Practice).

An average limit pressure in the sand and the clay of 3,50 MPa was taken into account (see figure 3).

The different curves of Fasicule 62 used to deduce the value of $q_s$ in the tertiary sand and the Boom clay as well as the value of $q_s$ are given in table 4.

Table 4: Curves used to deduce qs-values

| Class | Curve | $q_s$ |
|---|---|---|
| Argile, limons C (No post-injection) | $Q_1$ | 40 kPa |
| Sable, graves C (No post-injection) | $Q_3$ | 120 kPa |
| Sable, graves C (post-injection) | $Q_6$ | 250 kPa |

A second estimation was done according to the method described in Holeyman e.a. (1997) (Belgian Practice) using $q_c$ to calculate the unit ultimate shaft friction $q_{su}$. The average values of $q_c$ (see figure 2) and an installation factor $\alpha_s$ of 0,67 are used.

A third estimation was done according to the NEN (Standard of the Netherlands). Just like in the method according to the Belgian Practice the value of $q_{su}$ is calculated out of the cone resistance as $q_s = 0{,}006 \cdot q_c$.

As both these methods do not give any values for post-injection bored piles, the calculations have been done with the values of bored piles.

In table 5 the results of these calculations are summarised.

Table 5: Calculation of the ultimate shaft friction of the shaft section

| Pile | $Q_{su}$ (French Practice) | $R_{su}$ (Belgian Practice) | $P_{su}$ (NEN) |
|---|---|---|---|
| Pile 1 | 2827 kN | 1195 kN | 1690 kN |
| Pile 2 | 5936 kN | *2240 kN* | *1827 kN* |

Since even for pile 1 (without post-injection) the calculated values using Fasicule 62 ‘s method are much higher than those calculated according to Belgian Practice and NEN, the latter will not be taken into account.

### 4.2 *The ultimate resistance of the base section*

A first estimation of the ultimate pile base resistance was done in accordance with Fasicule 62 (French Practice).

As mentioned before the average limit pressure in the sand and the clay is taken as 3,50 MPa (see figure 3). In the Boom clay a value of k = 1,3 is taken into account using classification “Argiles Limons C” of Fasicule 62. In the sand a value k= 1,2 is used (classification “Sable, Graves C”)

A second estimation of the ultimate pile base resistance was done using the CPT’s (see figure 2) and De Beer’s method (Belgian Practice). The installation factor is taken at 0,50 in the sand layer and 0,80 in the Boom clay. The $\varepsilon_b$ factor normally applied for stiff fissured clays has not been taken into account.

A third estimation was done according to NEN (Standard of the Netherlands). With the LGM method the ultimate unit base resistance was calculated and an installation factor of $\alpha_p = 0{,}50$ was used.

The ultimate shaft friction on the pile base section of 1m height has been calculated according to the methods described above and has been taken into account.

In table 6 the results of these calculations are summarised.

Table 6: Calculation of the ultimate resistance of the base section

| Pile | $Q_{pu}+Q_{su(1m)}$ (French practice) | $R_{bu}+R_{su(1m)}$ (Belgian practice) | $P_{pu}+P_{su(1m)}$ (NEN) |
|---|---|---|---|
| Pile 1 | 7931 kN | 5712 kN | 5407 kN |
| Pile 2 | 6979 kN | 5814 kN | 4137 kN |

## 5 TEST PROGRAM AND TEST RESULTS

### 5.1 *Test program*

On each pile 2 tests were performed. Because the capacity of the hydraulic jack was limited, the first test is always done on the base section. In this way the displacement of the base section made it possible to move the shaft section independently during the second test. During both tests the displacements of each section were measured.

The static loading tests were all performed according to the same principles.

The estimated test loads (see table 7) were divided in 10 equal load steps.

Table 7: Estimated test loads

| Pile | Test load – Level 1 | Test load Level 2 |
|---|---|---|
| Pile 1 | 2500 kN | 6000 kN |
| Pile 2 | 6000 kN | 4000 kN |

During each stage the load was kept constant for 1 hour. The displacements were measured continuously. Because in the first stages the displacements are rather small and the pile behaves more or less elastically, the stages at 10% and 30% of the estimated test load were skipped.

The tests were performed till one of the following criteria were met:

- Achievement of the maximum load capacity of the hydraulic jacks.
- Achievement of the maximum displacement capacity of the hydraulic jacks (9cm).

After this the hydraulic jacks were unloaded in 4 equal stages. The load in each of these stages was kept constant for 5 min.. After the last stage of unloading the measurements were continued for at least 12 hours.

During the tests special arrangements have been taken to minimise the influence of the weather conditions on the test results.

### 5.2 *Results of static loading test*

In figure 6 and 7 the results of the different tests are summarised in a load-settlement diagram.

Figure 6: Results of static load test on pile 1

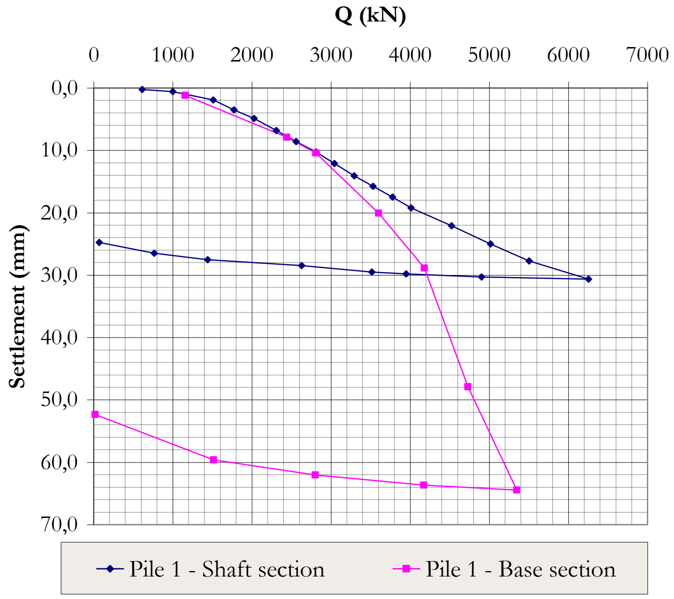
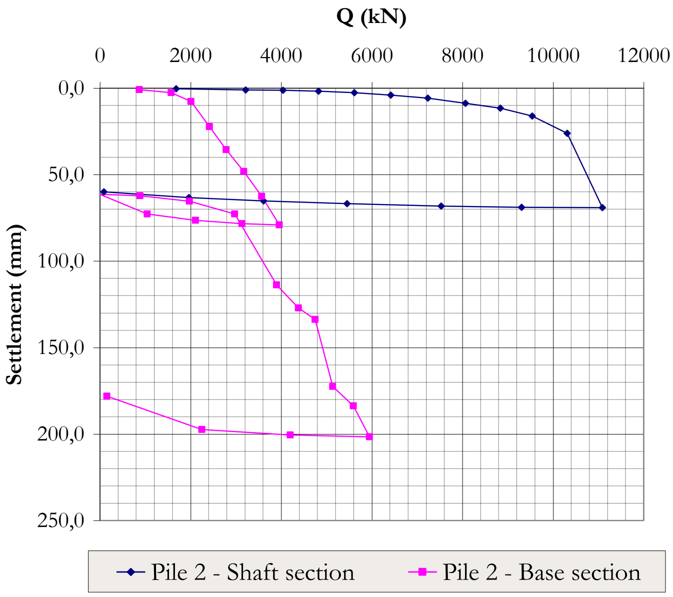



Figure 7: Results of static load test on the base sections




The test on the shaft section of pile 2 was stopped at the maximum load capacity of the hydraulic jacks. All other tests were stopped at the maximum displacement of the hydraulic jacks

A re-loading test on the base section of pile 2 was also done. The results of this test are also given in figure 7.

During each test no displacements of the other sections were measured except during the loading test on the shaft sections of pile 1 and 2. In the last stages of these tests a displacement of the base section was measured.

In the interpretation of the test results of these tests the different values of the loads are corrected with the load taken by the base section. The value of the load taken by the base section was deduced from the results of the loading test on the base section.

### 5.3 *Interpretation of the static load tests*

The test results can be interpreted in different ways.

In a first interpretation of the test results the method as described in Bustamante&Jezequel (1989) was used. A creep load $Q_c$ is determined out of increase of the measured displacements during each step. As the two sections were tested independently no global creep value could be determined. Therefore the creep loads were determined at compatible displacements taking into account the lowest value of the displacement at creep load of the 2 sections of the pile. Out of the value of $Q_c$ the allowable load is fixed at $R_{cal,1} = 0{,}80 \cdot Q_c$. In table 8 the different values for each test are given.

Table 8: Creep value and allowable load for the different sections

| Pile | Section | Creep load $Q_c$ | Allowable load $R_{cal,1}$ |
|---|---|---|---|
| Pile 1 | Shaft | 1900 kN | 1520 kN |
| | Base | 3073 kN | 2458 kN |
| Pile 2 | Shaft | 6232 kN | 4986 kN |
| | Base | 1920 kN | 1536 kN |

In a second interpretation the allowable load was deduced out of the rupture load. According to Belgian practice this is normally fixed at 10% of the nominal pile diameter. As the displacement capacity of the hydraulic jacks was limited this displacement was never reached.

As however in reality the displacements of the pile head need to be limited, rupture load was determined as the load corresponding to a settlement of 25mm. This value of the displacement at rupture load is also assumed in NEN 6473.

On the rupture load a total safety factor of 1,96 was taken as proposed in Theys e.a. (2003).

In table 9 the different rupture loads and the deduced allowable loads are summarised.

Table 9: Rupture load and allowable load for the different sections

| Pile | Section | Rupture load | Allowable load $R_{cal,2}$ |
|---|---|---|---|
| Pile 1 | Shaft | 2754 kN | 1405 kN |
| | Base | 3924 kN | 2002 kN |
| Pile 2 | Shaft | 8184 kN | 4175 kN |
| | Base | 2492 kN | 1272 kN |
| Reloading | Base | 3830 kN | 1954 kN |

The last method has been used to redesign the piles.

## 6 FURTHER INTERPRETATION AND DISCUSSION

Out of the test results the following observations for the shaft resistance of the piles can be made :

- The rupture load deduced from the test on pile 1 (without post-injection)(see table 9) is comparable to the one calculated (see table 5 - French practice).
- Compared to the calculated value of the shaft resistance of the post-injected pile 2 the rupture load deduced for the test is 38% higher.

Out of the test results the following observations for the base resistance of the piles can be made :

- Compared to the calculated values of the base resistance (see table 6) the rupture loads from the tests are much lower (see table 9).
- When reloading the base section of pile 2 an increase of the rupture load of 53% was found. The bearing capacity at high displacements is still increasing.

In a attempt to explain this phenomenon and because a sand deposit was found during the quality control at the base of executed working piles a core drilling through the whole length of pile 2 was done.

The quality control on the working piles was done through tubes fixed at the reinforcement cage. The base of each tube stops at 1m above the base of the reinforcement cage and are placed at 0,60m from the centre of the pile. Normally a concrete core with a length of 1 m to 0,85 m should be redrawn.

In total 19 working piles were controlled and the average value of the concrete core length was only 0,53m with a minimum of 0,15m. It was obvious that a sand deposit was present.

The core drilling over the whole length of pile 2 was done at 0,60m from the centre of the pile. Under the lowest level of the hydraulic jacks, at 1m above the theoretical pile base, a core of 0,48 cm could be redrawn. So at the base of the pile the same sand deposit was found.

Taking this into consideration the low base resistance deduced from the tests can be explained, but as the sand deposit was found on a great number of working piles questions on the installation coefficient for the pile base or on the curing of pile base before concreting of the pile need to be posed.

The quality of the concrete of pile 2 in the shaft and around the hydraulic jacks was good and no inclusions of air or soil were found.

## 7 CONCLUSION

For the determination of the bearing capacity of large diameter bored piles different static loading tests are performed by means of hydraulic jacks installed at 2 different levels.

The test results show that the rupture loads of the shaft resistance calculated with French, Belgian and Netherlands methods are under-estimated. However for the pile executed without post-injection the rupture load of the shaft resistance calculated with the French method is comparable to the one derived from the test.

The rupture loads of the base resistance calculated with the same methods are over-estimated. This is mainly due to the deposit of sand at the pile base. Because this deposit was also found on a large number of working piles, questions on the installation coefficient for the pile base or on the curing of the pile base before concreting of the pile need to be posed.

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