Sense and sensitivity of pile load-deformation behaviour.

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**ABSTRACT:** The SLS design of piles is often treated in a stepmotherly way. This paper aims to sensitize and to encourage the practitioners to consider more often also the deformability of piled foundations. A summarising data-base of all well documented load tests available in Belgium on bored and auger piles is included in the paper for documentation. Second, the use of hyperbolic transfer functions as a powerful scientific tool for prediction, analysis, conversion, ... of the load-displacement behaviour of single piles is demonstrated by various practical applications. Third, the author is sharing a number of particular experiences with critical or non-critical, expected or not expected deformation behaviour of piled constructions.

1 INTRODUCTION

After 30 years of debate on the Eurocode 7 and the sense and nonsense of the semi-probabilistic design approach, one is actually working in the stage of infiltration of the Eurocodes design philosophy and its application in practice. In December 2004 Eurocode 7: Geotechnical design – Part 1: General rules (EC7-1) was unanimously ratified by the European Member States. On the national level, the involved national bodies and specialists are fully working on the elaboration of the National Annexes, required as link between the EC7-1 and the national standards. Around 2009 (?), after a 2-year calibration period and a further 3-year coexistence period, EC7-1 will become valid in all Member States and the relevant National standards will then have to be withdrawn. With regard to pile foundation EC7-1 pays quite a lot of attention to the ULS design of single piles and pile groups, in compression, in tension, for lateral loading, ... Conversely, the SLS design of piles (vertical displacements of axially loaded piles) is only handled in a rather short subchapter §7.6.4 of EC7-1. Beside some “self-evident” provisions, the most relevant specifications given in this concise subchapter are – in my opinion – the following (EN 1997-1:2004):

- “7.6.4.1(1) Vertical displacements under serviceability limit state conditions shall be assessed and checked against the requirements given in 2.4.8 and 2.4.9”
- “7.6.4.2 (for compression piles) NOTE When the pile toe is placed in a medium-dense or firm layer overlying rock or very hard soil, the partial safety factors for ultimate limit state conditions are normally sufficient to satisfy serviceability limit state conditions.”
- “7.6.4.2 (2) Assessment of settlements shall include both the settlement of individual piles and the settlement due to group action.”
- “7.6.4.2 (4) When no load results are available for an analysis of the interaction of the piles foundation with the superstructure, the load-settlement performance of individual piles should be assessed on empirically established safe assumptions.”
- “7.6.4.3 (for tension piles) NOTE Particular attention should be paid to the elongation of the pile material.”

This paper aims:

- to contribute to the understanding of the load-deformation behaviour of piles
- to demonstrate the relevance and importance of considering the SLS both for individual piles as for pile groups
- also to demonstrate the use of transfer functions – in particular of the hyperbolic type – as a practical tool to interpret as well as to predict the pile deformation behaviour.

The paper consists of the following chapters:
Chapter 2 contains a data-base of well documented pile load tests on bored and auger piles, available in Belgium.

Chapter 3 briefly describes the calculation methods considered in some National design standards (The Netherlands, Germany, France) to assess the load-deformation curve of single piles.

Chapter 4 deals with the use of hyperbolic transfer functions as a powerful tool for analysis, back-calculation, prediction and sensitivity analysis of the load-settlement curve of single piles of different types and in different soil conditions.

Chapter 5 shows us some illustrations where the deformation behaviour of piles merits to be considered and/or has strongly influenced the design or the behaviour of the structure.

2 PILE LOAD TESTS ON BORED AND AUGER PILES – BELGIAN DATA-BASE

A database of 27 pile loading tests on displacement screw pile in Western Europe – period 1970-2000 – has been set up by De Cock (2001) – see table 1.

<table>
<thead>
<tr>
<th>File No.</th>
<th>Test site and year</th>
<th>Pile Nr</th>
<th>Pile denomination</th>
<th>Mean dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Zwevegem 1984</td>
<td>P1</td>
<td>Atlas 36/46 – 13.05 m</td>
<td></td>
</tr>
<tr>
<td>A2</td>
<td></td>
<td>P2</td>
<td>Atlas 36/46 – 13.05 m</td>
<td></td>
</tr>
<tr>
<td>A3</td>
<td>Ghent I 1985</td>
<td>P1</td>
<td>Atlas 36/46 – 13.05 m</td>
<td></td>
</tr>
<tr>
<td>A4</td>
<td></td>
<td>P2</td>
<td>Atlas 36/46 – 13.50</td>
<td></td>
</tr>
<tr>
<td>A5</td>
<td>Oldenburg 1986</td>
<td>P1</td>
<td>Atlas 41/51 – 9.50 m</td>
<td></td>
</tr>
<tr>
<td>A6</td>
<td></td>
<td>P2</td>
<td>Atlas 41/51 – 7.00 m</td>
<td></td>
</tr>
<tr>
<td>A7</td>
<td>Ghent II 1987</td>
<td>P6</td>
<td>Atlas 46/60 – 12.50 m</td>
<td></td>
</tr>
<tr>
<td>A8</td>
<td></td>
<td>P9</td>
<td>Atlas 46/60 – 13.50 m</td>
<td></td>
</tr>
<tr>
<td>A9</td>
<td>Koekelare 1992</td>
<td>P10</td>
<td>Atlas 54/60 – 13.0 m</td>
<td></td>
</tr>
<tr>
<td>A10</td>
<td></td>
<td>P10</td>
<td>Atlas 54/60 – 13.0 m</td>
<td></td>
</tr>
<tr>
<td>A11</td>
<td></td>
<td>P15</td>
<td>Atlas 54/60 – 13.0 m</td>
<td></td>
</tr>
<tr>
<td>A12</td>
<td></td>
<td>P16</td>
<td>Atlas 54/60 – 13.2 m</td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td></td>
<td>P1</td>
<td>Fundex 45/56 – 11.0 m</td>
<td></td>
</tr>
<tr>
<td>B2</td>
<td></td>
<td>P2</td>
<td>Fundex 45/56 – 14.8 m</td>
<td></td>
</tr>
<tr>
<td>B3</td>
<td></td>
<td>P2</td>
<td>Fundex 45/56 – 12.5 m</td>
<td></td>
</tr>
<tr>
<td>B4</td>
<td></td>
<td>P2</td>
<td>Fundex 45/56 – 8.5 m</td>
<td></td>
</tr>
<tr>
<td>B5</td>
<td></td>
<td>P3</td>
<td>Fundex 38/45 – 13.0 m</td>
<td></td>
</tr>
<tr>
<td>B6</td>
<td></td>
<td>P8</td>
<td>Fundex 38/45 – 13.0 m</td>
<td></td>
</tr>
<tr>
<td>B7</td>
<td></td>
<td>P9</td>
<td>Fundex 45/56 – 11.0 m</td>
<td></td>
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<tr>
<td>B8</td>
<td></td>
<td>P5</td>
<td>Fundex 45/56 – 14.5 m</td>
<td></td>
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<tr>
<td>B9</td>
<td></td>
<td>P5</td>
<td>Fundex 45/56 – 14.5 m</td>
<td></td>
</tr>
<tr>
<td>B10</td>
<td></td>
<td>P5</td>
<td>Fundex 45/56/70 – 11.0 m shaft and base enlargement</td>
<td></td>
</tr>
<tr>
<td>B11</td>
<td></td>
<td>P5</td>
<td>Fundex 38/45 – 13.0 m</td>
<td></td>
</tr>
<tr>
<td>B12</td>
<td></td>
<td>P5</td>
<td>Fundex 38/45 – 13.0 m</td>
<td></td>
</tr>
</tbody>
</table>

Tables 3 and 4 includes indicative values of the relative pile displacement at service load, presuming a factor of safety of 2.0 on the ultimate resistance. Both tables fit quite well with each other.

A summary of the normalised load-displacement curves included in this database, is given in figure 1. Since then, 2 extended research programs were conducted in Belgium on screw piles. Within the framework of the further elaboration of the EC7-NA guidelines, the BBRI has elaborated a more complete database of Belgian pile load tests. A selection of normalised load-displacement curves (Table 2), subdivided for end bearing and shaft bearing piles and separately for precast driven piles, screw piles and bored/CFA piles is given in figures 2a to 2g.

<table>
<thead>
<tr>
<th>No.</th>
<th>Soil</th>
<th>Site</th>
<th>Pile id</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Sand</td>
<td>LimII</td>
<td>B1</td>
<td>Precast 35 x 35</td>
</tr>
<tr>
<td>S2</td>
<td>Sand</td>
<td>LimII</td>
<td>B2</td>
<td>Precast 35 x 35</td>
</tr>
<tr>
<td>S3</td>
<td>Sand</td>
<td>LimI</td>
<td>P8</td>
<td>Precast 29 x 29</td>
</tr>
<tr>
<td>S4</td>
<td>Sand</td>
<td>Gent</td>
<td>Pre_b</td>
<td>Precast 32 x 32</td>
</tr>
<tr>
<td>S12</td>
<td>Sand</td>
<td>LimII</td>
<td>A1bis</td>
<td>Fundex 38/45</td>
</tr>
<tr>
<td>S13</td>
<td>Sand</td>
<td>LimII</td>
<td>A2</td>
<td>Olivier 36/51</td>
</tr>
<tr>
<td>S14</td>
<td>Sand</td>
<td>LimII</td>
<td>A3</td>
<td>Omega 41/41</td>
</tr>
<tr>
<td>S15</td>
<td>Sand</td>
<td>LimII</td>
<td>A4</td>
<td>De Waal 41/41</td>
</tr>
<tr>
<td>S16</td>
<td>Sand</td>
<td>LimII</td>
<td>B3</td>
<td>Atlas 36/51</td>
</tr>
<tr>
<td>S17</td>
<td>Sand</td>
<td>LimII</td>
<td>B4</td>
<td>Atlas 36/51</td>
</tr>
<tr>
<td>S18</td>
<td>Sand</td>
<td>LimII</td>
<td>C2</td>
<td>Olivier 36/51</td>
</tr>
<tr>
<td>S19</td>
<td>Sand</td>
<td>LimII</td>
<td>C3</td>
<td>Omega 41/41</td>
</tr>
<tr>
<td>S20</td>
<td>Sand</td>
<td>LimII</td>
<td>C4</td>
<td>De Waal 41/41</td>
</tr>
<tr>
<td>S26</td>
<td>Sand</td>
<td>Loenhout</td>
<td>S4</td>
<td>CFA with casing diam. 61</td>
</tr>
<tr>
<td>S27</td>
<td>Sand</td>
<td></td>
<td>A1</td>
<td></td>
</tr>
<tr>
<td>S28</td>
<td>Sand</td>
<td></td>
<td>C2</td>
<td></td>
</tr>
<tr>
<td>S29</td>
<td>Sand</td>
<td>Gent</td>
<td>CFA_a</td>
<td>CFA diam. 45</td>
</tr>
<tr>
<td>S30</td>
<td>Sand</td>
<td>Gent</td>
<td>CFA_b</td>
<td>CFA diam. 45</td>
</tr>
<tr>
<td>S31</td>
<td>Sand</td>
<td>KalloIII</td>
<td>C</td>
<td>Bored pile diam. 60, bento</td>
</tr>
<tr>
<td>S32</td>
<td>Sand</td>
<td>KalloIII</td>
<td>D</td>
<td>Bored pile diam. 60, bento</td>
</tr>
<tr>
<td>C1</td>
<td>Clay</td>
<td>SKW</td>
<td>A</td>
<td>Precast 35 x 35</td>
</tr>
<tr>
<td>C2</td>
<td>Clay</td>
<td>SKW</td>
<td>A4</td>
<td>Precast 35 x 35</td>
</tr>
<tr>
<td>C3</td>
<td>Clay</td>
<td>SKW</td>
<td>A2</td>
<td>Fundex 38/45</td>
</tr>
<tr>
<td>C4</td>
<td>Clay</td>
<td>SKW</td>
<td>A3</td>
<td>Fundex 38/45</td>
</tr>
<tr>
<td>C5</td>
<td>Clay</td>
<td>SKW</td>
<td>B1</td>
<td>De Waal 41/41</td>
</tr>
<tr>
<td>C6</td>
<td>Clay</td>
<td>SKW</td>
<td>B2</td>
<td>De Waal 41/41</td>
</tr>
<tr>
<td>C7</td>
<td>Clay</td>
<td>SKW</td>
<td>B3</td>
<td>Olivier 36/51</td>
</tr>
<tr>
<td>C8</td>
<td>Clay</td>
<td>SKW</td>
<td>B4</td>
<td>Olivier 36/51</td>
</tr>
<tr>
<td>C9</td>
<td>Clay</td>
<td>SKW</td>
<td>C1</td>
<td>Omega 41/41</td>
</tr>
<tr>
<td>C10</td>
<td>Clay</td>
<td>SKW</td>
<td>C2</td>
<td>Omega 41/41</td>
</tr>
<tr>
<td>C11</td>
<td>Clay</td>
<td>SKW</td>
<td>C4</td>
<td>Atlas 36/51</td>
</tr>
<tr>
<td>C12</td>
<td>Clay</td>
<td>SKW</td>
<td>C3</td>
<td>Atlas 36/51</td>
</tr>
<tr>
<td>C13</td>
<td>Clay</td>
<td>Kortrijk</td>
<td>A</td>
<td>Bored pile diam. 76, bento</td>
</tr>
<tr>
<td>C14</td>
<td>Clay</td>
<td>Kortrijk</td>
<td>B</td>
<td>Bored pile diam. 76, bento</td>
</tr>
<tr>
<td>C15</td>
<td>Clay</td>
<td>Kortrijk</td>
<td>D</td>
<td>Bored pile diam. 90 casing</td>
</tr>
<tr>
<td>C16</td>
<td>Clay</td>
<td>Kortrijk</td>
<td>Z</td>
<td>Bored pile diam. 76 casing</td>
</tr>
</tbody>
</table>
Table 3. Relative displacement at service load for screw piles (De Cock, 2001)

<table>
<thead>
<tr>
<th>Pile type</th>
<th>Shaft bearing</th>
<th>Shaft+end bearing</th>
<th>End bearing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atlas</td>
<td>0.5 – 0.75 %</td>
<td>0.75 – 1.50 %</td>
<td>1.5 – 1.75 %</td>
</tr>
<tr>
<td>Fundex</td>
<td>No data</td>
<td>0.75 – 1.0 %</td>
<td>0.75 – 1.25 %</td>
</tr>
<tr>
<td>Omega</td>
<td>0.5 – 0.75 %</td>
<td>No data</td>
<td>2.0 – 2.5 % *</td>
</tr>
</tbody>
</table>

(*) the relevance of the data - resulting from 1 short pile (O2) in heterogeneous soil and from 1 pile (O7) with only partial mobilization of the resistances – may be moderate.

Table 4. Relative displacement at service load (BBRI database)

<table>
<thead>
<tr>
<th>Pile type</th>
<th>Shaft bearing</th>
<th>End bearing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driven Precast concrete</td>
<td>0.5 – 0.75 %</td>
<td>1.0 – 2.0 %</td>
</tr>
<tr>
<td>Screw piles</td>
<td>0.5 – 1.0 %</td>
<td>No data</td>
</tr>
<tr>
<td>Bored and CFA</td>
<td>0.5 % bentonite</td>
<td>0.5 – 1.5 % *</td>
</tr>
<tr>
<td></td>
<td>1.0 – 1.5% casing</td>
<td></td>
</tr>
</tbody>
</table>
Figure 2a to h. Normalised load-displacement curves for different piles types (BBRI-database)
3 CALCUATION METHODS FOR LOAD-SETTLEMENT BEHAVIOR ON THE BASIS OF IN-SITU SOIL TESTS

A few methods to calculate the load-settlement behaviour are prescribed in European National Codes or Recommendations.

The Dutch piling code NEN 6743 (1993) provides a method to define the design value of the pile head displacement as a function of the mobilised base resistance and shaft resistance, as calculated on the basis of CPT. The method is semi-graphical and based on two charts, one for base resistance and one for shaft resistance. Each chart contains 3 normalised load-displacement curves for displacement piles (without making any distinction between e.g. driven piles or screwed piles), for CFA piles and for bored piles respectively.

The German bored piling code DIN 4014 comprises 4 tables, giving values of experience of mobilisation curves for bored piles in non-cohesive soils (based on CPT cone resistances) and in cohesive soils (based on c_u-values).

The French code Fascicule 62-V contains technical rules for the design of foundations of civil engineering structures. It also describes a method to determine the load-displacement curve of a single pile under axial loading, based on bilinear elasto-plastic mobilisation curves, whereby the stiffness factors k_b and k_s for respectively base resistance and shaft resistance result from the work of Frank and Zhao (1982) and are function of the PMT pressuremeter modulus E_M and the diameter D of the pile. The functions are different for non-cohesive and cohesive soils, but the pile type does not interfere.

In particular with regard to the load-settlement prediction of displacement auger piles, one also refers to the former work of Van Impe (1988) published in the first BAP-seminar.

4 HYPERBOLIC TRANSFER FUNCTIONS – GENERALITIES

4.1 Basic principles

Every geotechnical engineer is aware of the use of the single hyperbolic function for back-analysis of the pile load-settlement curve. The graphical inverse slope method, as suggested by Chin (1970), allows in many cases for a quite satisfying curve fitting.

The main purpose of this curve fitting was and still is to extrapolate the measured load-settlement curves and to allow for a mathematical estimate of the ultimate or asymptotic pile resistance. The method is semi-graphical and based on the conversion of the “measured Q-s” values into a s/Q versus s diagram.

In fact, the basis equation (1)

\[ Q = \frac{s}{a + bs} \]

(1)

can be transformed into:

\[ \frac{s}{Q} = a + bs \]

(2)

which corresponds to a straight line in the s/Q versus s plane. An example of the method is shown in the calculation sheet in figure 3.
The base flexibility factor $K_b$ (dimensions m/kN) corresponds to the tangent slope at the origin of the hyperbolic curve, as shown in figure 4. It also gives, multiplied with $R_{bu}$, the base displacement at 50% mobilisation of $R_{bu}$.

\[ R_b = \frac{S_b}{K_b + \frac{S_b}{R_{bu}}} \]  
\[ R_s = \frac{S_s}{K_s + \frac{S_s}{R_{su}}} \text{ or } q_s = \frac{S_s}{K_s + \frac{S_s}{q_{su}}} \]  

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The base flexibility factor $K_b$ (dimensions m/kN) corresponds to the tangent slope at the origin of the hyperbolic curve, as shown in figure 4. It also gives, multiplied with $R_{bu}$, the base displacement at 50% mobilisation of $R_{bu}$.

The shaft resistance flexibility factor $K_s$ (dimensions m/kN) also corresponds to the tangent slope at the origin of a $q_s$-$s$ diagram. With Fleming, one states that $K_s$ is proportional to the pile’s shaft diameter $D_s$ and inversely proportional to the ultimate shaft friction $R_{su}$, and so:

\[ K_s = \frac{M_s \cdot D_s}{R_{su}} \]  

with $M_s$ a dimensionless flexibility factor in the nature of an angular rotation. From this one deduces that 50% of the ultimate shaft friction is mobilised at a pile shaft displacement of:

\[ s_{50\%} = K_s \cdot R_{su} = M_s \cdot D_s \]  

4.2 Deduction of parameters

It appears from the above mentioned formulae (4) to (7) that only a few parameters are required to define the various hyperbolic functions:

- $R_{bu}$ or $q_{bu}$: ultimate pile base resistance, total or unit value
- $E_u$: secant modulus (at 25% of ultimate stress) of the soil beneath the pile base
- $R_{su}$ or $q_{su,i}$: ultimate pile shaft resistance, total or unit value in the different layers around the pile shaft
- $M_s$: shaft flexibility factor
- $E_c$ or $E_s$: pile material modulus (concrete, steel, grout, …)

Some remarks with regard to the parameter choice.

1. The ultimate pile base resistances and pile shaft resistances are in most cases obtained by calculation. An overview of the wide panoply of methods used in Europe resulted from the ERTC3 work in the period 1994-2004. (De Cock 1998, De Cock et al, 2003). It should be mentioned that the required value should be the asymptotic ultimate value at large displacements. However, one can also use the hyperbolic law on the basis of another ultimate value, e.g. at 10% of the pile base, and by deducing $R_{bu}$ from the next correlation at $s = 10\%$ of the pile base diameter $D_b$:

\[ R_{b,10\%} = \frac{0.1 D_b}{K_b + \frac{0.1 D_b}{R_{bu}}} \]  

2. For the secant modulus in non-cohesive soils, Caputo (2003) found a correlation factor of 10 between $E_u$ and the average CPT-cone resistance $q_c$ in the proximity of the pile base. According to the author, the correlation should also depend on the soil stress history (e.g. geological overconsolidation) and should also be “pile type” related. There is in fact enough evidence that the soil stiffness may be influenced by the execution method of the pile: e.g. a bored pile may result in some soil reluctance at the pile base, while a driven pile leads to a densification and prestressing of the pile base layer; the latter results in a much higher deformation modulus. Further literature survey and back-analysis are needed to define suitable correlations, but the following correlations appear to be quite promising in non-cohesive soils:

$E_u = 4 \text{ to } 6 \times q_c$ for bored piles in NC-sands
$E_u = 6 \text{ to } 8 \times q_c$ for bored piles in OC-sands
$E_u = 8 \text{ to } 12 \times q_c$ for screw piles
$E_u = 15 \text{ to } 20 \times q_c$ for driven piles

For cohesive soils (stiff OC-clays) it was found from back-calculation of screw piles in clay (see...
below in 5.2.1 and 5.2.2) that – at least for the pile load test – the undrained modulus should be used for the pile base stiffness, and so approximately:

\[ E_u = 50 \text{ to } 80 \, q_c, \text{ or } E_u \approx 750 \text{ to } 1.000 \times c_u \]  

(11)

3 For the shaft flexibility factor, Caputo’s statistical analysis (Caputo, 2003) confirmed the findings from Fleming that this factor generally is in the order of 0.001-0.002. This is also confirmed in many of our analysis, with only rare exceptions (see further).

4 The material moduli of elasticity for concrete and steel are supposed to be well known. For concrete several empirical formula relate the elasticity modulus to the compressive strength, as for example:

\[ E_{tg} = \sqrt{5600 \times (0.96) \times R_{wj,28,150}} \]  

(10)

On the other hand, the non-linearity of the material modulus at high concrete or steel stresses should be considered. In particular in the case of tension piles, the question rises whether and when the fissuring of the concrete under tension degrades during the tension test.

5 APPLICATIONS OF HYPERBOLIC TRANSFER FUNCTIONS

5.1 A multipurpose tool

When considering base resistance, shaft resistance and pile elasticity separately and by dividing the pile in e.g. 10 or 20 discrete elements, the hyperbolic transfer functions appear to be a fascinating engineering tool. Some of the possible uses are illustrated in the further paragraphs of chapter 5, namely: 5.2 For (class-A) prediction of the LS-curve 5.3 For back-analysis of pile load tests to deduce the adequate design parameters and/or to define calculation methods 5.4 To verify the SLS of a pile foundation (prediction of single pile displacement) 5.5 To convert a pile load test to other pile geometries, e.g. to a larger diameter 5.6 To invert a pile load test from compression to tension, or from bi-directional pile testing to top loaded testing. 5.7 To evaluate the impact of pile type, material, shape, execution method, … on the pile stiffness 5.8 To evaluate the influence of boundary conditions on the pile behaviour, e.g. downdrag, excavation of top layer, …

5.2 Class A prediction of pile load-deformation

5.2.1 Soil testing data

We used the discrete hyperbolic functions at the occasion of a class A-prediction event that was organised in relation to an extended research program on auger piles in clay, conducted in Belgium (Holeyman, 2001). The results of our predictions were published at the Int. Conference in Istanbul in 2001 (De Cock, 2001).

All in all six different types of ground displacement piles were installed: one prefab concrete pile and five cast-in-place screw types including Fundex, De Wall, Olivier, Omega and Atlas. From each pile type, a series of “short” piles, length approx. 7.5 m, and a “long” pile, length of about 11.5 m, was installed.

Typical CPT data are given in figure 5. CPT-EB2 with electrical cone and CPT-MB12 and CPT-MB23 with mechanical M1 (Dutch cone) are located in the same area, close to pile B2 (long De Waal pile).

5.2.2 Applied parameters in the prediction

The ultimate base resistance \( R_{bu} \) and the ultimate shaft resistance \( R_{su} \), both considered as the conventional values at a relative pile displacement \( s=10\%D_b \) (\( D_b=\)pile base diameter) have been deduced from the CPT data. According to the Belgian methodology (Holeyman et al., 1997) one defines:

\[ R_{bu} = \alpha_b \cdot \beta_b \cdot q_{bu,mean} \cdot A_b \quad \text{or} \quad R_{su} = \alpha_{su,mean} \cdot \beta_b \cdot q_{su,mean} \cdot A_b \]  

(13a-b)

with \( \alpha_b \) an empirical (installation) factor taking into account the method of installation of the pile and the soil type;

\[ R_{su} = \frac{X_s}{n_d} \cdot \alpha_s \cdot \Delta Q_{st} \quad \text{or} \quad R_{su} = \frac{X_s}{n_d} \sum \alpha_{st} \cdot \Delta Q_{sti} \]  

(14a-b)
with $\alpha_b$ an overall empirical factor introducing the effects of pile installation method, the nature and roughness of the pile shaft material and soil structure scale effects.

$$R_{su} = X_s \sum H_i \eta_{pi} \alpha_{pi} q_{ci} = X_s \sum H_i \alpha_{si} \eta_{pi} \alpha_{pi} q_{ci}$$  \hspace{1cm} (15)$$

with $\eta_p$ = an overall empirical factor depending on both soil and pile type. The correlation $\eta_p = q_c/q_{su}$ can be split into a pure soil parameter $\eta_p$ equal to the ratio of $q_c$ and the average unit side friction $q_{su}$ and a pile/soil dependent empirical factor $\alpha_p$ (as defined above).

Contrary to the traditional Belgian methodology, as used by the BBRI, the unit end bearing resistance in the natural ground conditions has not been calculated using the De Beer method (Van Impe, 1988) which gives $q_{bu}(m)$, but as the mean value of $q_c$ over a depth of $2D_b$ below the pile base (indicated in equation (11b) by $q_{b,mean}$). Comparison of both values for the various CPT performed in the test pile axis, is given in Figure 6. The De Beer method, which smoothes the $q_c$-diagram, gives on average 15 % lower values than this mean $q_c$.

$$F_{su} = \alpha_{su} \eta_{su} q_{su}$$

The $\alpha_{su}$ factor refers to the scale dependent soil shear strength of the fissured clay. It has been deduced from previous research in the considered clay layer to be related to the ratio of the pile diameter $D_b$ to the CPT cone diameter $d$ by:

$$0.476 \leq \alpha_{su} = 1 - 0.01(D_b/d - 1)$$  \hspace{1cm} (16)$$

In the initial prediction, the pile base area $A_b$ has been calculated from $1.0 \times \text{the external diameter of the auger for Fundex, De Waal and Omega}$, and from $0.9 \times \text{the maximum auger flange diameter for Olivier and Atlas}$. For clarity, however, in this paper all factors are put $= 1.0$, incorporating the section reduction factor of $0.92=0.81$ for Olivier and Atlas in the $\alpha_b$ factor. For $X_s = \pi D_b$, the maximum shaft diameters have been considered for all piles.

$$\frac{q_{bu}}{q_{bu,mean}} = \frac{q_{bu}}{q_{bu,mean}} \cdot \frac{q_{bu,mean}}{q_{su}}$$

The $\alpha_{b,mean}$ factor had been taken $= 1.0$ for all pile types (but so has now been replaced by 0.81 for Olivier and Atlas).

For $\alpha_b$, the following values have been used in the prediction: precast piles 0.85, Fundex piles 0.80, De Waal and Omega 1.0, Olivier and Atlas 1.25.

The correlation factor $\eta_{pi}$ has been defined in 2 steps:

1. The measured local friction values $f_s$ from the electrical cone CPT have been reduced by a factor of 1.35 in order to obtain a good correlation between this reduced integrated local friction $f_{s,red}$ and the total rod friction $Q_{st}$ with the mechanical cones; (see Figure 5);

2. The relation between $f_{s,red}$ and $q_c$ has statistically been analysed. Figures 7a and 7b give the collection of the data points, the deduced trend line, the relations earlier suggested by De Beer (1985) and Van Impe (1988), as well as the relation prescribed in the recent Belgian guidelines for the design of compression piles (BBRI, 2008).

![Figure 6. Unit end bearing (without installation factors) from CPT (St-Katelijne-Waver)](image)

![Figure 7a. Shaft friction ratio $\eta_p^*$ as a function of $q_c$](image)

![Figure 7b. Unit shaft friction $f_{s,red}$ as a function of $q_c$](image)

From this, one deduces the following trend lines:

$$\eta_p^* = 0.9 \cdot q_c^{-0.41} \text{ or } f_{s,red} = 0.9 \cdot q_c^{0.59}$$  \hspace{1cm} (17a)(17b)$$

For the calculation of $K_n$ and $K_u$, the following values have been adopted:

$$E_u = 175 \text{ to } 225 \text{ MPa} \quad (\equiv 1000 \text{ } c_u \text{ or } \equiv 70 \text{ } q_c)$$
- $M_r = 0.0015$ for $z<5$ m and $= 0.001$ for $z>5$ m.

The E-modulus of the pile’s concrete has been taken equal to 30000 MPa, which has appeared later on to be smaller than the values of 35 to 40000 MPa deduced from compression tests on concrete samples.

5.2.3 Comparison of predictions with SLT-curves

Figures 8a-8d gives for a selection of 4 piles the comparative charts of the SLT curve ($Q_{meas}$) as well as the predicted curves for mobilised base, shaft and total resistance ($Q_{b,pred}$, $Q_{s,pred}$ and $Q_{t,pred}$ respectively), all expressed as a function of the pile head displacement $s_h$.

With regard to the measured load-curves, the attention is drawn to the following:
- SLT were performed in 10 to 12 load steps of 60’
- After reaching a peak resistance, piles were further displaced at constant rate of 0.6 to 0.8 mm/min; in all cases the load therefore required decreased more or less significantly to what often is pretended to be the residual resistance.

Comparison of predicted and measured data leads to the following conclusions and remarks:
- All in all, the shape of the predicted curve corresponds fairly well with the observed load-displacement behaviour, considering that the measured displacements at high load levels as well as the peak resistance are very much influenced by the short duration of the load steps and therefore should be handled with caution.
- For prefab and Fundex piles, the prediction of the ultimate total pile resistance as well as of the total stiffness factor (see also the zoomed chart in Figure 8a) may be called perfect.
- For all other piles, the ultimate pile resistance is overestimated by about 15-25 %, with a maximum of 50 % for the short Atlas pile C4.

![Figures 8a to 8d. Predicted curves of Qb, Qs and Qt and measured total load-displacement curve (St.-Kathelijne-Waver) Screw piles in clay (St.-Kathelijne Waver) (De Cock, 2001)
5.3 Back-analysis of pile load tests

As already explained earlier, only a few parameters are needed to define the required transfer functions for base and shaft resistance. Therefore, curve fitting with the measured load-displacement curves (Qt, Qs, and Qb) allows quite easily to obtain the required parameter values to calibrate the chosen design method (e.g. a semi-empirical direct design method based on in situ soil tests such as CPT or PMT).

Some illustrations of such a curve fitting are given in the figures 9a to 9b. For clarity and simplicity, the back-analyses have been performed on the same piles as those reported above for the class A-predictions. However, one should keep in mind that there is a slight difference in the definition of the installation factors $\alpha_b$ and $\alpha_s$ used in the back-analysis, which fit into the recently published design guidelines within the EC7-NA (National Application document). Indeed:

- $\alpha_b$ is to be combined, according to equation (11a) with the unit pile base resistance $q_{b(u)}^m$ as calculated according to the De Beer's method and not with an averaged value cone resistance value $q_c{\text{mean}}$
- $\alpha_s$ is to be combined, according to equation (13), with the $\eta^*_p$-values as defined in EC7-NA; for the considered case of clay, $\eta^*_p = 1/30 q_c$ as indicated in figure 7b.

The best estimate of the resulting parameters is indicated in the different figures 9a to 9b. They are also summarised in table 5.

![Figure 9a to 9d. Back-analyses of pile load tests. Screw piles in clay (St-Kathelijne Waver)](image-url)
Table 5. Parameters resulting from back-analysis of pile load tests. Screw piles in clay (St.-Kathelijne-Waver)

<table>
<thead>
<tr>
<th>Pile</th>
<th>E_c</th>
<th>E_b</th>
<th>M_s</th>
<th>α_b</th>
<th>α_s</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1 Precast</td>
<td>38.000</td>
<td>130 qc</td>
<td>0.002</td>
<td>0.90</td>
<td>1.80</td>
</tr>
<tr>
<td>A3 Fundex</td>
<td>35.000</td>
<td>50 qc</td>
<td>0.002</td>
<td>0.85</td>
<td>0.96</td>
</tr>
<tr>
<td>B4 Olivier</td>
<td>25.000</td>
<td>65 qc</td>
<td>0.0045</td>
<td>0.95</td>
<td>1.25</td>
</tr>
<tr>
<td>C1 Omega</td>
<td>43.000</td>
<td>50 qc</td>
<td>0.002</td>
<td>0.79</td>
<td>0.98</td>
</tr>
</tbody>
</table>

* pile base resistance is likely to be underestimated and shaft friction overestimated

5.4 SLS design of a single pile

For the time being, the Belgian EC7-NA document only provides calculation rules for the ULS design of single axially loaded piles in compression. It is the aim of the piling committee to develop in due time also guidelines for the SLS design. The methodology still has to be elaborated and discussed. In order to get – at least – a feeling of the impact of the ULS design rules on the load-settlement behaviour, for the same 4 piles as considered before, one has now estimated the load-settlement curve using similar hyperbolic functions, with the same stiffness parameters E_b and M_s (and thus also K_b and K_s) as deduced from the curve fitting, but with the α_b and α_s factors as prescribed in the EC7-NA document. For the concrete modulus E_c a “conservative” value of 25.000 MPa has been adopted for all cases. The results are given in figures 10a to 10b.

**Figure 10a-10d.** Calculated curves of Q_b, Q_s and Q_t with factors from EC7-NA and measured total load-displacement curve. Screw piles in clay (St-Kathelijne Waver)
5.5 Converting pile load test result to larger diameter – bored piles

Eurocode 7 (7.6.2.2-(4)) allows for performing design load tests on smaller diameter trial piles, providing a.o. that the trial pile is instrumented in such a manner that the base and shaft resistance can be derived separately from the measurements. The hyperbolic transfer functions are an easy instrument to convert the measured pile-displacement curve for larger (or smaller) diameters. An example is given in figure 11. It is based on the scientific research programme on bored and driven piles in dense sands, conducted at Kallo (De Beer, 1988).

5.6 Comparison of compression and tension behaviour – shaft grouted auger tube piles

In 2006-2007 two extended load-testing programmes were performed on so-called CSG-piles (Continuous Shaft Grouted) at the demand of Franki Grondtechnieken BV/The Netherlands. These programmes aimed to define the adequate design parameters for this pile type in accordance to the Dutch piling code NEN 6743 for compression piles and the CUR report 2001 for tension piles. The piles consist of a central steel tube (diameter 140 mm, thickness 10 mm), provided at the base with 1 or several screw blades or with an enlarged drill bit (figure 12). The tube is installed by screwing, while injecting cement-grout through the tube which is mixed with the surrounding soil by the action of the screw blades. After reaching the required depth, additional pressure grouting is performed.

Figure 12. View on the steel tubes for the CSG pile (Franki Grondtechnieken B.V.)

The first test programme, located in Pijnacker, comprised 8 test piles. Two piles – each with a screw blade of 450 mm - were load-tested in both compression and tension. The other 6 piles – either with a screw blade of 350 mm diameter or with a drill bit of 180 mm – were load tested in tension only. All piles were instrumented with 3 tell-tales. A typical CPT and the configuration of the 2 test piles that will shortly be described below, together with the length of their respective tell-tales, are given in figure 13.

Figure 11. Conversion of pile load test to larger diameter pile.
Several disturbances on the tell-tales, the uncertainty about the axial stiffness $EA$ of the piles and the large share of the elastic deformations in the total pile head displacements appeared initially to hinder the interpretation of the test results. Finally, the back-analysis by using the hyperbolic function approach allowed obtaining an acceptable and justified interpretation of the test results. By way of illustration, two analyses are given below.

The test pile TP1 was initially used as a reaction pile for the tension test on the neighbour pile and consequently was compressed in 6 loading-unloading steps up to a maximum test load of 1363 kN; the maximum pile head displacement was 20.4 mm. The “asymptotic” ultimate pile resistance, which is estimated by a simple Chin analysis or a Vanderveen analysis lies in the order of 3250 kN. As this value is far higher than the maximum test load, the accuracy of the extrapolation may be questioned. Moreover, it appears from the tell-tale measurements that the maximum total head-displacement of 20.4 mm was essentially due to the elastic deformation of 18.2 mm, while the displacement of the pile base was only about 2.2 mm. Nevertheless, the curve fitting of the total load-displacement curve as well as of the tell-tale measurements (figure 14), allowed to obtain a fair estimation of the pile stiffness $EA$ (with a quite complex combination of steel tube, steel tell-tales, internal grout and external soil-cement mix), of the soil stiffness factors $E_b$ and $M_s$ and of the design parameters $\alpha_b$ and $\alpha_s$ (in accordance to NEN 4735). Once these parameters were obtained, it was a small challenge to convert the compression test into a tension test, and to compare this tension curve from conversion with the measured curve during the tension test on the same pile. In this conversion, all parameters were kept as deduced from the compression curve fitting. The only parameter that was changed was the pile stiffness factor, for which only the steel section and not the grout sections was considered for the “tension-stiffness”. Both the “converted” tension curve and the measured tension curve are shown in figure 15. The correspondence is fairly well.
The results of a second tension test on pile RP2 (diameter tube 140 mm and drill bit 180 mm) as well as the curve fitting are given in figure 16a for the fitting of the tell-tale measurements, and figure 16b for the pile head displacements.

The maximum applied load is 1814 kN, which is about 75% of the estimated “asymptotic” ultimate resistance of 2,400 kN. The pile head displacement at maximum load was 55.0 mm, from which 43.2 mm comes from the elastic elongation and 11.8 mm corresponds to the pile base lifting. These quite high displacements, which are typical for many types of steel tension piles (and anchors), may question the criteria that have to be applied in the ULS as well as the SLS design. One also notices at maximum load of 1814 kN a deviation between the calculated and measured pile elongation s-el. This is mainly due to yielding of the steel at the considered working stresses of well above 450 N/mm².

5.7 Impact of changes in pile type, material, geometry, execution method, ...

The hyperbolic functions also allow the SLS analysis of changed pile characteristics, such as:
- Changes in pile stiffness $EA$, e.g. by using a different steel section for steel piles
- Different pile type, e.g. impact driven pile versus auger pile
- Application of post-grouting.

For the latter, we refer to section 6.5, where the application of post-grouted large diameter bored pile is described.

5.8 Influence of changing boundary conditions

Also the impact of changing boundary conditions on the pile settlements and safety can be verified on the basis of established transfer functions, e.g.:
- Loss of positive shaft friction due to excavation of the top layers
– Inversion of positive shaft friction into negative skin friction due to settlements of the surrounding soil
One refers to section 6.4 for a practical demonstration.

6 CASE HISTORIES TO ILLUSTRATE SENSE AND SENSITIVITY OF PILE DISPLACEMENTS

6.1 Introduction
The sensitivity of the structure to displacements of the pile foundations should be verified from the next 3 points of view:
1. the ULS of the pile foundation: what is the real load on the different piles?
2. the ULS of the structure: which stresses are developing in the superstructure due to the displacements of the piles?
3. the SLS of the structure: are the settlements (or heave) of the piles and the pile groups admissible for the structure and its functioning?

In particular with regard to the first and second item, the interaction between structure and pile foundation should be considered. This interaction mainly depends on the stiffness of the superstructure, the stiffness of the piles (individually or in group) and the stiffness of the intermittent structure (pile cap, beams, raft ….). The very different answers that one can get as a function of the relative stiffness are given in the example of figures 17 and 18. This sensitivity exercise was made in collaboration with the structural design office for a 12-storey apartment building. Six situations are considered for a hypothetic beam of 40 m in length, with 6 column loads of 1,000 kN each and 7 supports (say: 7 piles or pile groups) – see table 5. The stiffness of the beam (= structure) are considered to be either infinite or finite. The supports (piles) have been supposed to be rigid, or to act as (Winkler) springs of equal or different value; the latter case wants to simulate the higher stiffness of peripheral and/or single piles compared to internal piles or pile groups.

Table 5

<table>
<thead>
<tr>
<th>Combination</th>
<th>Stiffness of structure</th>
<th>Stiffness of piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Infinite</td>
<td>Infinite</td>
</tr>
<tr>
<td>2</td>
<td>Finite</td>
<td>Infinite</td>
</tr>
<tr>
<td>3</td>
<td>Infinite</td>
<td>Finite – 100 MN/m</td>
</tr>
<tr>
<td>4</td>
<td>Finite</td>
<td>Finite – 100 MN/m</td>
</tr>
<tr>
<td>5</td>
<td>Infinite</td>
<td>Finite – 200 &amp; 100 MN/m</td>
</tr>
<tr>
<td>6</td>
<td>Finite</td>
<td>Finite – 200 &amp; 100 MN/m</td>
</tr>
</tbody>
</table>

A summarise of the pile loads for the various combinations is given in figure 17. The comparisons, although for a somewhat fictive situation, demonstrate that the load distribution on the different piles is fundamentally dependent on both structure and pile stiffness. And so are also the bending moments and stresses in the superstructure, on the one hand, and the deformations (settlements) of the structure, on the other hand, as can be seen from the 3 deformation distributions given in figure 18.
6.2 Heterogeneous soil stratigraphy – Antwerp LB

In the ‘70s the strong residential needs nearby the city of Antwerp (located on the right bank of the river Scheldt) gave rise to the development of extensive housing estates in the former polder on the left bank opposite the ancient city centre (figure 19).

By hydraulic or mechanical landfill with sand, the site level was raised from the former polder level of about +1/+2 up to the level +6 to +7. Multiple housing projects were realised in short time and it were golden times for piling contractors, with an extended use of precast concrete piles which were driven into the medium dense or very dense sand layer underneath the polder clay and peat. A typical CPT for the region is given in figure 20.
However, some 30 years later, the new owner of one of the houses had serious doubts about the stability of the house because of various cracking in the subbase as well as in the superstructure. Levelling works revealed an overall tilting of the construction from SW to NE with differential settlements of 92 mm!! And this in spite of the fact—according to the information from the construction drawings—that the house—with a surface of 4.0 x 15.0 m²—was built on 14 piles with a capacity of 600 kN. But the reason was quite obvious. The nearby lake, called “Galgenweel” let assume that the subsoil had been disturbed centuries ago, by the occurrence of this “weel = pool, swirl”, which has caused deep erosion of the polder layer and of the underlying sand and later on sedimentation of loose sand-clay deposits. CPT tests at the North side and the South side of the house have confirmed the suspicions and revealed very different soil conditions from the “normal” stratigraphy in the region, with an increasing thickness of the soft layers towards the lake (figure 21). Very likely, the piles have been executed too short and the foundations have behaved more like piled raft on a non-consolidated subsoil.

![Figure 21. CPT tests and schematic presentation of building subsidence.](image)

**Figure 21.** CPT tests and schematic presentation of building subsidence.

### 6.3 Pile load and settlement distribution under uniform stiff high-rise building/Netherlands

An interesting case history has been reported by Joustra et al. (1977). It concerns a very homogeneous and stiff high-rise building with a surface of 25x47 m², which was founded via a 1.5 m thick concrete slab on 222 prefabricated concrete piles. The piles are 17 m long and have a shaft section of 45x45 cm² and an enlarged base of 72x72 cm². The total building weight is about 290.000 kN, and so the average pile load is of about 1.300 kN/pile (Figure 22).

![Figure 22. High-rise building – configuration, soil data and pile data (Joustra et al, 1977)](image)
Because of the existence of deep compressible layers underneath the pile-bearing sand stratum, some monitoring of the pile settlements and the pile load distribution was performed. The construction scheme is given in figure 23, showing also the location of:

- The piles EL1 to EL5 and ME1 to ME5, which were instrumented with a tell-tale, on the bases of which the head load was calculated
- The measuring points 1 to 10 for observation of the settlements.

Table 6 summarises the results of the tell-tale measurements and the pile loads that have been computed on the basis of the measured elastic shortening.

<table>
<thead>
<tr>
<th>Pile No</th>
<th>Shortening (mm)</th>
<th>Calculated pile load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EL1</td>
<td>2.72</td>
<td>1.780</td>
</tr>
<tr>
<td>EL2</td>
<td>1.14</td>
<td>750</td>
</tr>
<tr>
<td>EL3</td>
<td>1.47</td>
<td>960</td>
</tr>
<tr>
<td>EL4</td>
<td>2.62</td>
<td>1.710</td>
</tr>
<tr>
<td>EL5</td>
<td>defective</td>
<td></td>
</tr>
<tr>
<td>EM1</td>
<td>1.20</td>
<td>780</td>
</tr>
<tr>
<td>EM2</td>
<td>1.41</td>
<td>920</td>
</tr>
<tr>
<td>EM3</td>
<td>1.87</td>
<td>1.210</td>
</tr>
<tr>
<td>EM4</td>
<td>1.10</td>
<td>720</td>
</tr>
<tr>
<td>EM5</td>
<td>1.91</td>
<td>1.170</td>
</tr>
</tbody>
</table>

Figure 22. Construction schema and monitoring points.

6.4 Interaction of foundations of adjacent buildings

The following case history illustrates the sensitivity even of a pile foundation in interaction with adjacent former or future foundation works, on the one hand, and the possibilities to analyse and to interpret this interaction by using the hyperbolic transfer functions, on the other hand.

The case history – which was the subject of a juridical expertise – concerns 3 adjacent houses:

- House No.1 was erected in the 60’s and is founded on 21 driven Franki-piles, 500 kN capacity and a length of about 13.5-14.0 m below ground level
- House No. 2 was built in 1989, it includes a subbase and is founded on 15 Atlas screw piles diameter 36/46 cm, 350 kN capacity and a length of 10 m below ground level
- House No. 3 dates from 1995. The foundation exists of 17 piers (“faux puits”), diameter 1.2 m, 240 kN capacity and a length of 6.9 m.

A schematic cross section parallel to the street and the diagram of the CPT at the location of house No. 2 is given in figure 24.
Quite important damages occurred in house No. 2 due to a tilting of the house towards house NO.3. From the various observations and measurements, the expert deduced what follows:

- a small tilting of house No. 2 resulted in the period 1990-1995 in some small fissuring at the interface with house No. 1
- in the first week after excavation and concreting of the pier foundation of house No. 3, important new damage and tilting of house No. 2 occurred.
- during and after the erection of house No. 3, a further increase of the damage and tilting of house No. 2 was observed.

In 2000, one measures:

- a differential settlement of 5 mm between house 2 and the common wall with house 1
- a tilting of 1/20 of the joint wall of houses 2 and 3, from which the settlement of this common wall is estimated to be in the order of 35-40 mm.

The observed deformations may be explained as follows:

1. the first slight fissuring was caused by the combined effect of:
   - a partial transfer of the load of house 2 to the adjacent piles of house 1 – due to some constructive connection between the 2 houses and/or by additional friction on the nearby Franki piles;
   - additional settlements of the pile group by the presence of compressible layer underneath the pile bearing layer
2. possible decompression (stress reduction), decrease of the shear parameters and or subsidence of the soil by the excavation of the pier; this may lead to:
   - loss of the positive shaft friction on the adjacent Atlas piles over the pier length and even inversion to negative skin friction on these piles
   - negative skin friction on the adjacent subbase wall what leads to an additional head load on the pile
3. additional settlements of the Atlas piles due to the weight of house 3, which causes again negative skin friction on the Atlas piles (particularly beyond the foundation level of the piers) and settlements in the layers underneath the Atlas base level.

The adverse effect of the different interactions on the Atlas piles at the interface of building 2 and 3 may be analysed on the basis of the estimated load-displacement curves for the Atlas piles by using hyperbolic transfer functions. The results of this exercise are given in figure 25.
6.5 Differential pile behaviour under an hyperstatic architec tonic museum building

The last case history concerns a new prestigious museum building – still under construction – in the city of Antwerp (the so-called MAS – Museum aan de Stroom) – Architects Neutelings-Riedijk/Rotterdam, Consulting Engineers ABT Belgium/Antwerp. The building has a square footprint of 40x40 m² and contains 10 floor levels with a height of 6 m each (Figure 26). The museum floors – always containing a gallery and a museum room – are stacked in such a way that the MAS becomes a spiral tower. The periphery – with 6-metre high outside glass façades – is conceived as a walking boulevard that remains accessible for everyone, day and night, with panoramic views of the city and the river.

The architectural conception and needs:
- free panoramic view from the peripheral promenades
- museum rooms free of obstacles
- concrete look of walls and ceilings
- high level heights
were a big challenge for the structural concept and resulted in a type of Christmas tree structure consisting of (figure 27 and 28):
- a centre square concrete core of about 12x12 m² with a double function: stability and transfer of building loads to the foundations
- absence of load-bearing façades

Figure 26. Maquette view of the MAS –museum.

Figure 27. Structural schema.

Figure 28. Cross section

- a minimum of load bearing elements in the peripheral galleries and promenade area’s
- large projection of the periphery by means of framework or concrete wall-beams.
The overall vertical load on the foundations is maximum about 300,000 kN, from which 205,000 kN is permanent. Almost 85% of the loads is transmitted to the central core, while to other 15% is distributed over a number of “second order” columns and walls. Several options have been considered with regard to the foundation concept. Finally there was chosen for the use of bored shaft grouted bored piles with temporary casing under the central core and for CFA piles for the peripheral loads. Useful data about the behaviour of post-grouted bore piles in the considered tertiary sands was gained from the extended bi-directional load testing performed for the HST-tunnel in Antwerp (Maertens et al, 2003) and other literature data. The pile lay-out is given in figure 29.

The main heavy load bearing bored piles have been kept concentrated under the central core. This has the advantage of a direct load transfer without the need for a heavy intermittent repartition slab. The use of shaft grouted piles aimed to increase the individual pile stiffness as well as the pile group stiffness and consequently to reduce the settlements of the central core:

− by the beneficial effect of the shaft grouting, the piles are essentially “friction” piles whereby the individual pile settlement under service load remains small (estimated at about 9 mm)
− the shaft grouting allowed to use relatively short piles, having their base level well away from the deep Boom clay of medium compressibility.

The long term settlements are calculated at about 67 mm for the central pile group and about 16 mm nearby the façades. From these values, about 18 mm respectively 8 mm occurs in the deep clay. In view of these absolute and differential settlements some structural measurements have been taken:

− there are no continuous stiff concrete walls going from central core to façade
− floor slabs between central core an façades are isostatic (hinge supports)
− during the construction period, some particular floor elements are delayed to allow a differential movement of façade and façade.

The settlement of the building are monitored and hopefully some results may be presented during the seminar.
ACKNOWLEDGEMENTS

Our sincere thanks go to ir. Monika De Vos and Noël Huybrechts (BBRI), ir. Ben Notenboom (ABT Belgium) and ir. Bart De Ridder (Studieburo Mouton) for their practical help for supplying practical graphical help for this paper. The author wants also to extend his gratitude to the Flemish government; their financial support and encouragement through IWT (the Institute for the Promotion of Innovation by Science and Technology in Flanders) have stimulated multiple actions to improve the regional geological knowledge and its dissemination across the borders.

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