Design of piles – Belgian practice

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ABSTRACT

This national report gives an overview of the actual practice with regard to the design of piles in Belgium. Some of the content has been taken over from the report that was published by Holeyman et al. at the occasion of the 1st ETC3-symposium in 1997, but it contains a lot of new elements, as in the meantime Eurocode 7 was introduced in Belgium. Above that, quite a lot of new instrumented pile load tests have been carried out since 1997, adding supplementary and even new insights to the existing experimental pile load testing database in Belgium. In 2009 a first edition of the Belgian guidelines assessing pile design according to the principles of EC7, which had been elaborated under the auspices of the Belgian standardization committee of EC7, was published by WTCB-CSTC (2009). Meanwhile the content of this design guide has been revised and extended and the 2nd edition will be published in 2016 by WTCB-CSTC (2016). The design principles reported in this contribution are mainly based on the content of this new revised version of the Belgian pile design guide.

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1. REGIONAL GEOLOGY

The Belgian territory is rather flat with a continuous transition from a plain at the North Sea and the Dutch border to the highlands of the Ardennes, the highest point being situated at Botrange (694 m above sea level). The geology of the Tertiary and Quaternary formations in Belgium is characterized by an approximately SouthEast NorthWest oriented epirogenetic axis (Silence, 1992), which follows the valleys of the rivers Haine, Sambre, Meuse and Vesdre (Figure 1) and which divides Belgium into approximately two equal parts.

In the North part, the stratigraphy was 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 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 epirogenetic axis, 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 in the North a wide variety in stratigraphy, with complicated and heterogeneous soil layer patterns. It is not therefore 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.

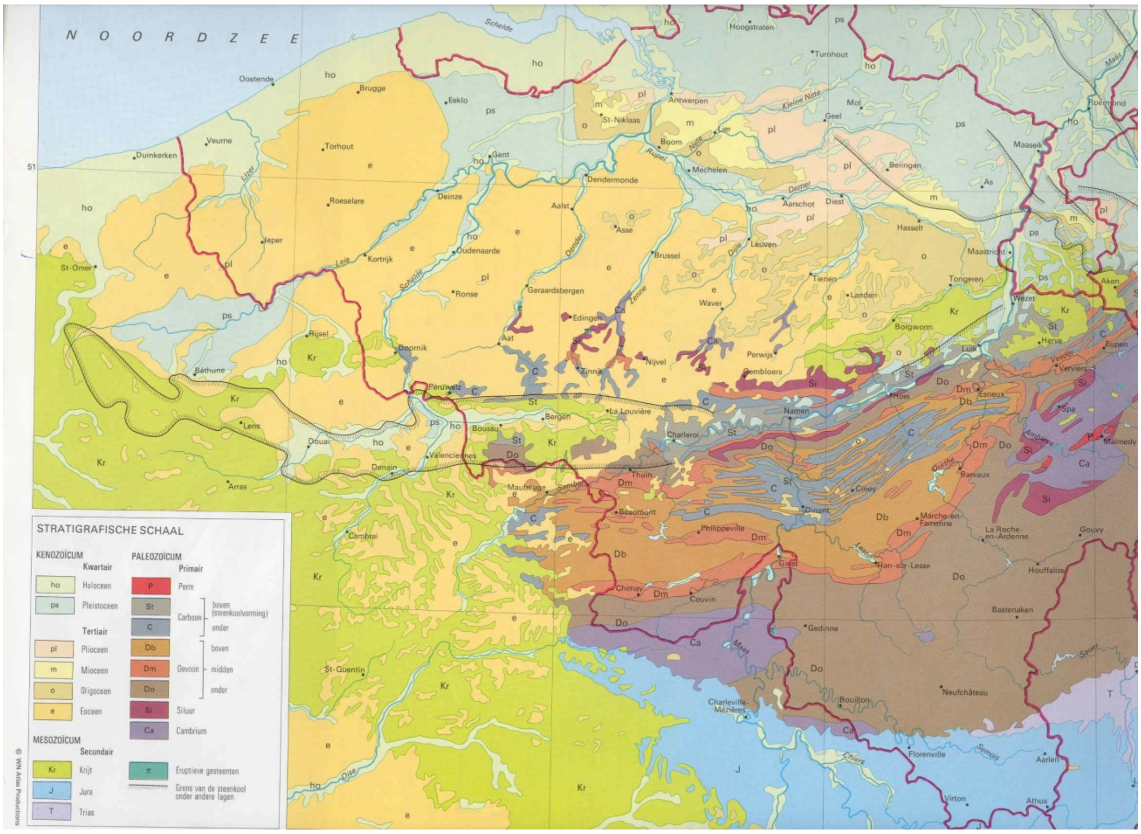


Figure 1: Geological map of Belgium

2. SOIL INVESTIGATION

The execution of soil investigation in Belgium has to be carried out according to the principles set out in the NBN EN 1997-2 (EC7 part 2). The scope, the extent and the type of soil investigation program for a given site depends of course on a large number of project and site dependent factors as well as the requirements of the owner. In general three phases can be distinguished: the preliminary investigation phase, the project-oriented soil investigations and the control-oriented soil investigations.

With regard to the *preliminary investigations*, public accessible libraries, publications or databases with geological and geotechnical data can be consulted. Official sources of information that are noteworthy are:

- the library of the Belgian Geological Survey which contains logs of borings (copies of bore logs legally required from boring companies)
- the published Geotechnical Maps. The geotechnical maps currently cover parts of the most developed areas of the country (Cities of Antwerp, Brussels, Charleroi, Ghent, Mons, and Liege)
- the geological maps covering the Belgian territory
- the historical Ferraris-maps (online available)

Since the last decade geological and geotechnical data can also be consulted online. Most important are the “Databank Ondergrond Vlaanderen” and “Geopunt” (for the Flemish Region and Brussels) and the “Portail environnement de Wallonie” (for the Walloon region).

Based on the wealth of information in the previous mentioned databases and the experience allowing for correlations, the *project-oriented soil investigations* performed for piling projects mostly consist exclusively in cone penetration tests (CPT) where feasible, i.e. where the CPT tests can be performed to a depth allowing the piling project to be designed. This is particularly the case for the very heterogeneous quaternary soil layers encountered in large areas in Belgium, where soil profiling is essential. For the reasons explained in Section 1, Regional Geology, sites located in the northern part of Belgium, where piles are often required, generally fulfil those conditions. Nuyens et al. (1995), give an overview of the history, the equipment and use of CPT in Belgium.

Although the CPT with mechanical cones (M4 and M1) have a strong historical background in Belgium and are still extensively used, the application of the CPT-E (with electrical cone) has increased considerably in the last decade, probably because it is considered as the reference cone in the national documents of Belgium (see §4), and because conversion factors between CPT-M and CPT-E have been integrated in these same national documents (see table 1). These conversion factors are based on a comparative analysis by Whenham et al. (2004).

Table 1: Reduction factor ω to be applied on the measured cone resistance q_c from CPT performed with a mechanical cone (M1, M2 or M4)

ω	**Tertiary clay**	**Other soil types**
M1	1.30	1.00
M2	1.30	1.00
M4	1.15	1.00

Investigations methods other than CPT testing are performed as a complement to CPT tests where warranted or where CPT testing is deemed unfeasible. The major components of those alternative investigation tests will still include in situ testing (mostly the pressure meter test), leaving a minor role to laboratory testing. Information complementary to the normal CPT practice is warranted when investigation is performed far away from prior developments or when settlements must be specifically evaluated.

Requirements with regard to the set up and execution of a soil investigation program in Belgium (number of CPT, the minimal depth of penetration, requirements for other in situ or laboratory soil investigation tests, …) have been specified in recent publications of the Task Force 2- Soil Investigation of the BGGG-GBMS (2012 and 2016). These documents are coherent with the NBN EN 1997-2 and the recent execution standards for CPT-testing.

An important change with regard to the past is that the number of CPT on a given job site might influence the design value of the calculated pile resistance. As further explained in §5 this is integrated in the design methodology by means of the correlation factors ξ_3 and ξ_4.

Finally, in some cases *control investigations* are performed after pile installation in order to evaluate the installation effect of the pile on the ground resistance near the piles shaft and pile base. Mostly these control investigations exist out of CPT executed at a limited distance from the pile shaft, sometimes dilatometer tests are used for this purpose.

3. PILING TECHNOLOGY & CLASSIFICATION

Table 2 provides the classification of the piling technologies in Belgium according to Belgian pile design guide (WTCB-CSTC, 2009/2016). This classification has been formalized by the Belgian standardization commission of EC7.

The evolution of the piling technologies used in Belgium has been originally mainly influenced by the historical development of the Franki-type rammed driven pile with dry concrete. The original system has evolved with the years while its Belgian and foreign competitors have developed alternative systems of impact driven piles with a shaft concreted with plastic concrete and with or without enlarged base. These systems are still widely used in Belgium, among the driven piles.

Cast-in-situ piles are the predominant type. Precast piles are used where the soil geotechnical conditions are homogeneous enough and usually for limited bearing capacities or special applications. Since major concerns started to arise more and more in the 1980’s with regard to the problems of noise and vibrations, vibration-free systems were extensively developed. One of the particularities of Belgium is the coexistence of different types of soil-displacement screwed piles which are well suited to our soil conditions. Thanks to many scientific load testing programs that were performed since the 1990’s, the confidence in this piles system increased, resulting in a significant market share in Belgium of about 30 to 40 % (Legrand & Poorteman, 2003).

Driven piles (Category I in the classification below, and Category II to some extent) and soil displacement screw piles (Category I) are thus preferred in many cases, especially in weak subsurface conditions where soil failure governs the design.

When a hard layer is encountered (intermediate or bearing layer), piles with partial or total soil excavation (Category II and III) are generally preferred, especially when pile embedment into the hard layer is required. However, the last decade soil displacement screw pile rigs integrating grout injection during installation that can deal to a certain extent with hard intermediate or bearing layers were developed, and are regularly used in these conditions as well.

Micropiles (Category IV) are applied in particular conditions, e.g. in zones that are not or in a difficult way accessible for large pile rigs. They are typically applied for the underpinning of existing foundations, as tension piles for basement slabs, for the extension of existing railway infrastructure,…

The other pile types that are listed in table 2 are used in a less regular way or for special purposes.

With regard to the most recent pile classification in Belgium (Table 2), it is important to remark that:

- in the category II a sub-category screw piles has been integrated, in order to deal with new non-proven so-called "displacement" screw pile systems that appear on the Belgian market. As long as for such new screw pile systems it has not been proved that they have soil displacement characteristics, they have to be considered as piles of category II. One can only deviate from that if it has formally been approved by the Belgian standardization committee of EC7.
- jet grout piles and soil mix piles are not covered by the classification in table 2, although they are regularly used in practice to develop vertical bearing capacity, often in combination with a retaining function. With regard to the vertical bearing capacity of soilmix-walls a methodology has been proposed in the recent handbook soil mix walls published by SBRCURnet & BBRI (2016).

4. NATIONAL DOCUMENTS

Historically, no Belgian standards were available to officially regulate the national piling practice. In the absence of truly relevant national documents, several owners and engineers did develop in the past their own specifications or recommendations (see Holeyman et al, 1997).

The situation changed with the publication of the Eurocode 7 as Belgian Standard in 2005, the NBN EN 1997-1: 2005, which created a formal framework in which a harmonised Belgian methodology could be elaborated.

This process has taken several years and it was in 2009 that a first guideline for the application of the EC7 for the ultimate limit state design of axially loaded compression piles based on CPT was published by WTCB-CSTC (2009). This pile design guideline is also referenced in the Belgian national annex of the Eurocode 7 (NBN EN 1997-1 ANB: 2014) as the Belgian reference method.

For the moment the 2009 guideline has been revised and further elaborated. It covers a.o. also axially loaded piles under tension, the effect of negative skin friction, etc. The content of this new guideline has formally been approved by the Belgian standardization committee of EC7 and will be published by WTCB-CSTC in 2016. In parallel the ANB will be adapted as well and the new version will probably be available in 2017. The design methods reported in §5 are based on this revised pile design guide.

With regard to the execution of piles, the standards that have been elaborated by CEN TC288 are in Belgium published as NBN EN 12699 (displacement piles), NBN EN 1536 (bored piles) and NBN EN 14199 (micropiles). In addition to these standards the establishment of a series of datasheets describing the characteristics of different pile systems is initiated by the Belgian standardization commission of EC7 and WTCB-CSTC. The aim of these data sheets is to link the installation characteristics of a pile system with the Belgian pile design guide described here before.

These data sheets give for example a detailed description of the installation characteristics of a pile system, the (sub)category to which its belongs, its nominal dimensions that need to be used in the pile design method as detailed further in §5, the minimal requirements with regard to the concrete quality, pile reinforcement and quality control, etc. For the moment data sheets of 5 soil displacement screw piles have been published by WTCB-CSTC (2014).

Table 2: Pile classification in Belgium according to WTCB-CSTC (2016)

CATEGORY I: PILES WITH HIGH SOIL DISPLACEMENT
DRIVEN & JACKED PILES
- Precast concrete piles without enlarged base - Cast in situ piles, without enlarged bottom plate, shaft in plastic concrete - Cast in situ piles, with enlarged bottom plate, shaft in plastic concrete - Cast in situ piles with enlarged base formed in the ground , shaft in rammed semi-dry concrete - Steel tube pile close-ended, without enlarged bottom plate - Steel tube pile close-ended, with enlarged bottom plate - Steel tube pile open-ended, situation with soil plugging
SCREW PILES CATEGORY I
- With temporary tube and shaft in plastic concrete - With lost tube - With lost or temporary tube and grout injection during installation

CATEGORY II: PILES WITH LOW SOIL DISPLACEMENT OR LOW SOIL RELAXATION
DRIVEN PILES
- Steel pile open ended, situation without soil plugging - Steel profiles and sheet piles
SCREW PILES CATEGORY II
- With temporary tube and shaft in plastic concrete - With lost or temporary tube and grout injection during installation
CFA PILES WITH PROVISIONS TO LIMIT SOIL RELAXATION
- With large diameter of the hollow stem and small flanges - With concrete overpressure - With temporary casing
CATEGORY III: PILES WITH SOIL EXCAVATION
CFA PILES WITHOUT PROVISIONS TO LIMIT SOIL RELAXATION
BORED PILES
- Executed with temporary casing - Executed under thixotropic fluid - Executed without casing or thixotropic fluid (dry boring)
CATEGORY IV: MICROPILES
- With grout placement under gravity (no pressure) - With mono-phase or stepwise grout placement under a global pressure higher than gravity pressure - With multi-phase selective and repetitive grout injection via TAM and double packer

5. DESIGN METHOD ACCORDING TO THE PRINCIPLES OF EUROCODE 7

5.1. General principles

5.1.1. Geotechnical Categories (GC)

Pile foundations are in general considered as constructions of Geotechnical Category 2 (GC2), which means that pile design must be based on quantitative geotechnical data and calculations. As mentioned before, Task Force 2 of the BGGG-GBMS (2016) specifies requirements with regard to the soil investigation that has to be performed, and this in function of the GC and the type of construction.
It is also important to notice that the design guideline for pile foundations in Belgium (WTCB-CSTC, 2016) is valid for constructions belonging to GC2.
For constructions of Geotechnical Category 3, the pile design guideline can be used but supplementary measures are necessary. Constructions of GC3 are for example constructions with abnormal risk or unusual or extreme difficult soil conditions and loads, constructions in areas susceptible to earthquakes or mass movements, …

5.1.2. Design approach (DA)

As specified in NBN EN 1997-1: ANB (2014), Design Approach 1 (DA1) must be applied in Belgium. This means that basically two combinations of partial factors have to be verified (see further), named DA1/1 and DA1/2.
For the design of axially loaded piles however, DA1/1 will in all cases be the determining combination. Consequently, the geomechanical and structural pile capacity need only to be verified for the combination DA1/1.

5.1.3. Methodology

In Belgium a semi-empirical design method to deduce the pile base resistance and the shaft friction from CPT measurements is mostly applied. The influence of the pile type and the installation effects on the pile bearing capacity is introduced in the design method by means of installation factors as explained further. These installation factors have been derived from many scientific pile load tests in the past, allowing to

calibrate/fit the semi-empirical relations. To deduce the "real" pile resistance from static pile load tests the conventional settlement criterion of 10 % D_b is applied.
In order to deduce a design value of the pile resistance, model factors, correlation factors and safety factors are introduced (see further).

5.2. Definitions and symbols

5.2.1. Definitions

<u>General definition:</u>
For the general definitions reference is made to NBN EN 1990 and NBN EN 1997 – 1.

<u>Pile base level and pile base diameter:</u>
The level of the pile base is defined as the lowest level where the pile base reaches its full section. This principle is illustrated in figure 2.
The pile base diameter D_b equals the maximum outer diameter of the pile base.

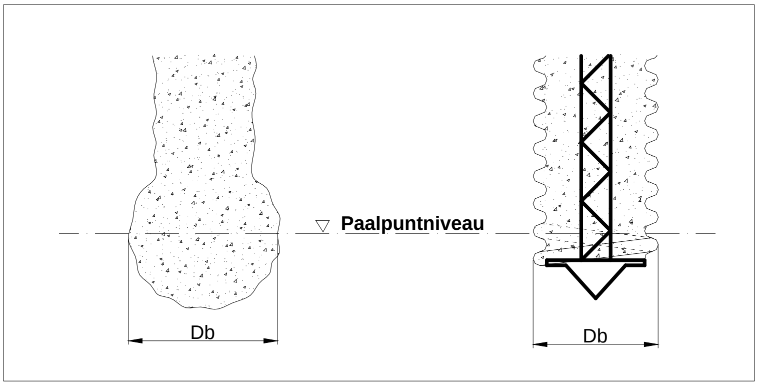


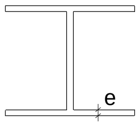
Figure 2: Example of the definitions of the pile base level and the pile base diameter

For piles with an enlarged bottom plate, the strength and stiffness of this bottom plate needs to be sufficient in order to resist the forces during the installation of the pile as well as the loads during the design life of the pile.

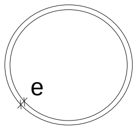
<u>Pile base section:</u>
The pile base section A_b is determined as follows:
- for a circular section: $A_b = \frac{\pi \cdot D_b^2}{4}$
- for a square or rectangular section: $A_b = a * b$ with a and b respectively the short side and the long side of the rectangular section
- for an I-beam or sheet pile: A_b = the steel section
- for an open-ended tubular pile, situation without plugging : A_b = the steel section
- for an open-ended tubular pile, situation with plugging: $A_b = \frac{\pi \cdot D_b^2}{4}$

<u>Equivalent pile base diameter:</u>
The equivalent pile base diameter $D_{b,eq}$, which is needed to determine q_b, ε_b and λ (see §5.3.2), is defined as follows:
- for a circular section $D_{b,eq} = D_b$
- for a square or rectangular section: $D_{b,eq} = \sqrt{\frac{4 \cdot a \cdot b}{\pi}}$ if $b \leq 1.5$ a
  $D_{b,eq} = \sqrt{\frac{6 \cdot a^2}{\pi}}$ if $b > 1.5$ a,
  with a and b respectively the short side and the long side of the rectangular section

- for an I-beam or sheet pile: $D_{b,eq} = \sqrt{\frac{6\,e^2}{\pi}}$, with e representing the thickness of the flanges



- for an open-ended tubular pile, situation without plugging: $D_{b,eq} = \sqrt{\frac{6\,e^2}{\pi}}$, with e representing the thickness of the steel



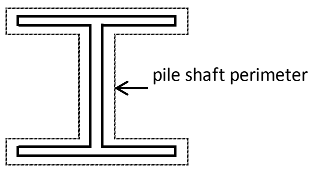
- for an open-ended tubular pile, situation with plugging: $D_{b,eq} = D_b$

For other sections $D_{b,eq}$ needs to be determined based on the rules explained above and on "*engineering judgement*"

<u>Pile perimeter:</u>

The perimeter of the pile χ_s is determined as follows:

- for precast concrete piles: the perimeter of the nominal section of the pile shaft
- for driven cast in situ piles: the outer diameter of the temporary tube
- for steel profiles and sheet piles: the total perimeter of the steel section



- for an open-ended steel tube piles, situation without plugging: the sum of the inner and outer perimeter of the tube
- for an open-ended steel tube piles, situation with plugging: the outer perimeter of the tube
- for close-ended steel tube piles: the outer perimeter of the tube
- for screw piles with temporary tube and shaft in plastic concrete: the maximum outer diameter of the system that is withdrawn (temporary tube or displacement auger). The maximum width of the screw flanges that may be taken into account equals 10 cm (e.g. 36/56)
- for screw piles with lost tube: the outer perimeter of the lost tube
- for screw piles with lost tube and with grout injection during installation: the perimeter is based on the average of the diameter of the lost tube and the diameter of the pile base
- for screw piles with temporary tube and with grout injection during installation: the perimeter is based on the average of the maximum outer diameter of the system that is withdrawn (temporary tube or displacement auger) and the diameter of the pile base
- for CFA piles without casing: the maximum outer diameter of the auger
- for CFA piles with temporary casing or bored piles with temporary casing; the maximum outer diameter of the temporary casing
- for bored piles without casing: the maximum outer diameter of the drilling tool

5.2.2. Symbols

With regard to general used symbols reference is also made to NBN EN 1990 and NBN EN 1997 – 1.

A_b (m²)	the pile base section
a (m)	the short side of a rectangular pile base
b (m)	the long side of a rectangular pile base
D_b (m)	the pile base diameter
$D_{b,eq}$ (m)	the equivalent pile base diameter
D_c (m)	the diameter of the cone of a CPT
D_s (m)	the diameter of the pile shaft
e (m)	the thickness of the flanges of steel profiles or the thickness of open-ended tubular piles
F (kN)	Load

F_c (kN) axial compression load
F_d (kN) design value of an action
F_k (kN) characteristic value of the load
F_{nk} (kN) load due to negative skin friction
F_{rep} (kN) representative value of the load
F_t (kN) axial tension load
G (kN) permanent load
G_d (kN) design value of the permanent load
G_{dst} (kN) destabilizing permanent load
G_{stb} (kN) stabilizing permanent load
h_i (m) thickness of soil layer i
q_b (kPa) unit pile base resistance
q_c (MPa) cone resistance
$q_{c,corr}$ (MPa) corrected cone resistance in the case that excavations have been carried out after execution of the CPT
q_s (kPa) unit pile shaft friction
Q (kN) variable load
Q_d (kN) design value of the variable load
Q_{dst} (kN) destabilizing variable load
R (kN) pile resistance
R_b (kN) pile base resistance
R_c (kN) compressive resistance of the pile
R_{cal} (kN) calibrated pile resistance
R_d (kN) design value of the pile resistance
R_i (kN) pile resistance calculated on base of CPT i, with i the identification of the CPT
R_k (kN) characteristic value of the pile resistance
R_m (kN) measured value of the pile resistance
R_s (kN) shaft friction
R_t (kN) resistance op a pile subjected to tension loads
$V_{dst,d}$ (kN) design value of the vertical destabilizing loads

α_b (-) installation factor fort the pile base resistance
α_s (-) installation factor for the shaft friction of a pile subjected to compressive loads
α_t (-) installation factor for the shaft friction of a pile subjected to tension loads
β (–) shape factor for non-circular nor square-shaped pile bases
γ' (kN/m³) effective volumetric weight of the soil
γ_b (-) partial factor for the pile base resistance
γ_F (-) partial factor for the loads
γ_{Gdst} (-) partial factor for permanent destabilizing loads
γ_{Gstb} (-) partial factor for permanent stabilizing loads
γ_{Qdst} (-) partial factor for variable destabilizing loads
γ_{Rd} (-) model factor
γ_s (-) partial factor for the shaft friction of piles subjected to compressive loads
γ_{st} (-) partial factor for the shaft friction of piles subjected to tension loads
ε_b (-) factor for the influence of the fissured state of a soil on the pile base resistance
η^*_p (-) factor giving the soil dependent ratio between the cone resistance q_c and the unit shaft friction q_s (independent from the pile type)
φ' (°) effective friction angle
λ (-) reduction factor for piles with an enlarged base
σ'_v (kPa) effective vertical stress
ω (-) conversion factor to be applied on q_c in the case of CPT with mechanical cone
ξ (-) correlation factor
χ_s (m²) perimeter of the pile shaft

5.3. ULS Design based on soil investigation test results

5.3.1. Introduction

In Belgium, the ULS design is in most cases based on the cone resistance diagram measured with in situ cone penetration tests. The design methodology to perform ULS design for axially loaded piles based on

CPT results is described in the Belgian pile design guide (WTCB-CSTC, 2009/2016), which is referenced in the Belgian national annex of the EC 7 as the reference method. This methodology is summarized in §5.3.2 and §5.3.3.
In some soil types however it is difficult to execute CPT (e.g. weak rock), and one have to apply alternative in situ test methods like the pressure meter test (PMT). For the moment no methodology to perform the ULS design on the base of PMT has been elaborated in Belgium, although some comparative exercises have recently been published by Allani et al. (2015). As long as no Belgian methodology has been integrated in the pile design guide, French reference documents are used for the moment. In practice the former French guidelines of the DTU 13.2 or the Fascicule 62 are still applied, although they have recently be replaced by an harmonised standard for the design of deep foundations, namely the NF P91-262 (AFNOR, 2012).
With regard to micropiles, a design methodology is for the moment under discussion in the Belgian standardization committee of EC7 and will probably be available by the end of 2016. In practice the same French reference documents as mentioned before or the methodology of Bustamante et al. (1985) are still regularly applied for the design of micropiles. Also the outcome of the anchor test campaign of BBRI (2008) is applied regularly for the design of micropiles.

5.3.2. Axial compression of a single pile

General

In order to prove that the pile foundation is capable to sustain the load with sufficient safety against geomechanical failure, the following condition has to be fulfilled [NBN EN 1997-1 §7.6.2.1 (1)]. This is the “GEO”-verification according to Eurocode 7

$$F_{c,d} \leq R_{c,d}, \tag{1}$$

with:

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

Design value of the load $F_{c,d}$

As mentioned before, $F_{c,d}$ is obtained by multiplying the representative values of the loads $F_{c,rep}$ with the partial load factor γ_F:

$$F_{c,d} = \sum F_{c,rep,i} * \gamma_{Fi}. \tag{2}$$

The partial load factors are determined in NBN EN 1990 ANB. The values for permanent and variable design situations are given in table 3. For accidental design situations all load factors are set to 1.00.
The own weight of the pile is not taken into account, unless it has specifically been requested.

Table 3: Partial load factors γ_F

Load		γ_F (DA1/1)
Permanent	Unfavourable [(1)]	1.35
	Favourable [(2)]	1.00
Variable	Unfavourable [(1)]	1.50[(3)]
	Favourable [(2)]	0.00

(1) Destabilizing.
(2) Stabilizing.
(3) For bridges an adapted value is valid according to Annex A2 of NBN EN 1990: for road traffic γ_F = 1.35 and for railway traffic γ_F = 1.45.

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

Figure 3 gives a schematic overview of the different steps to calculate the design value of the compressive resistance of the pile $R_{c,d}$.

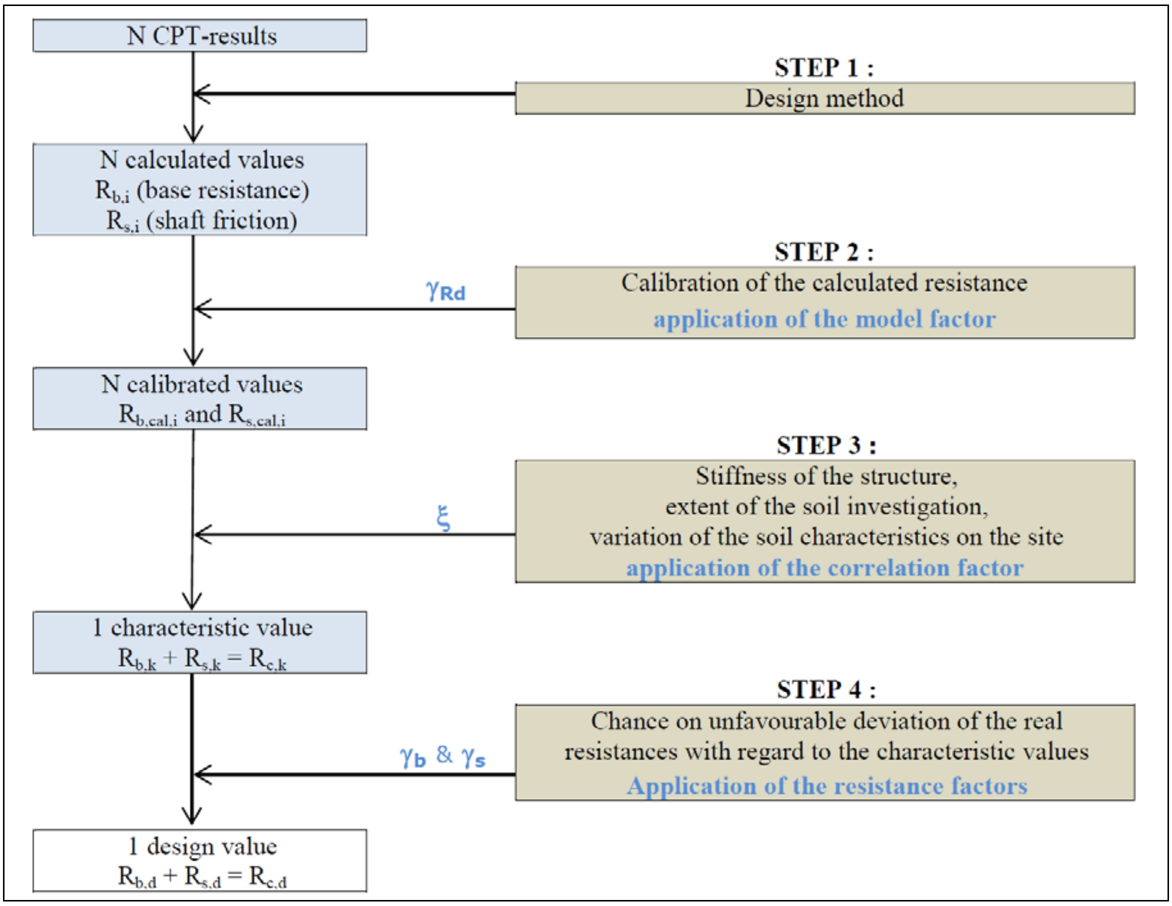


Figure 3: Schematic overview of the different steps to calculate the design value of the pile bearing capacity

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

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

$$R_b = \alpha_b \cdot \varepsilon_b \cdot \beta \cdot \lambda \cdot A_b \cdot q_b, \tag{3}$$

with:

q_b (kPa) the unit pile base resistance calculated with the De Beer Method out of the cone resistance (q_c) diagram of the CPT. In case that the CPT has been performed with a mechanical cone, q_c needs to be reduced with the values of table 1. When the soil is excavated after execution of the CPT, the q_c-values under the excavation level need in certain cases to be reduced (see WTCB-CSTC, 2016).
The pile base diameter that has to be introduced in the calculation model is $D_{b,eq}$ as defined in §5.2.1.
The basic principles of the De Beer method are explained further.

α_b (-) an empirical factor taking into account the installation method of the pile and the soil type. These values are summarized in table 5.

ε_b (-) a parameter referring to the scale dependent soil shear strength characteristics (e.g. in the case of stiff fissured clay):

$$\varepsilon_b = \max\left(1 - 0.01\left(\frac{D_{b,eq}}{D_c} - 1\right); 0.476\right)$$ in tertiary OC-clay

$\varepsilon_b = 1$ in all other soil types.

$D_{b,eq}$ represents the equivalent pile base diameter and D_c the diameter of the CPT-cone (in general D_c = 0.0357 m for a standard cone).

β (-) a shape factor, introduced for non-circular nor square-shaped bases:

$\beta = \frac{1 + 0.3\, a/b}{1.3}$ for a rectangular pile base, with a and b the dimensions of respectively the short and the long side of the pile base

β = 0.77 for walls

β = 1 for circular or square shaped pile bases.

A_b (m²) the section of the pile base as defined in §5.2.1.

λ (-) a reduction factor for enlarged pile bases that generate soil relaxation around the pile shaft during installation of the pile. The value of λ is determined as follows:

- for piles with an enlarged base that has been formed at depth, not causing soil relaxation around the pile shaft during installation: λ = 1.00
- for piles with a prefabricated enlarged base, with $D_{b,eq} < D_s + 0.05$ m: λ = 1.00
- for all other piles with a prefabricated base, the reduction can be deduced from figure 4.

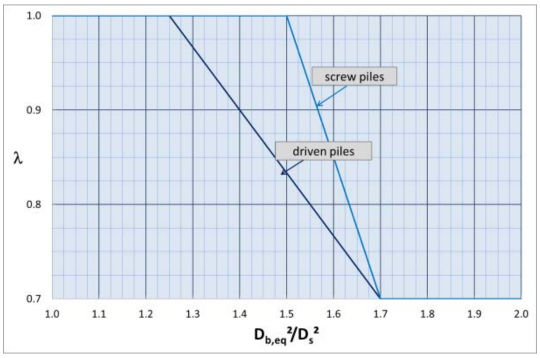


Figure 4: Reduction factor λ for piles with enlarged pile base that generates soil relaxation during installation

The De Beer method

One fundamental aspect of the Belgian pile design for axially loaded piles under compression is the introduction of the so-called "scale effect" for the pile base resistance. The scale effect aims to take into account that the base resistance of a pile is defined by the failure pattern, which extends over a certain height below and above the pile toe, this height being related to the pile base diameter. In other words, the approach aims at transforming the CPT diagram (generally obtained with a 3.6 cm diameter cone) into the CPT diagram that would be obtained with a sounding rod having a diameter equal to that of the pile base.

While in foreign countries this scale effect is calculated by rather simple mathematical approaches (smoothing and averaging the q_C-values over a certain range such as in France and The Netherlands), a more analytical method has been developed in Belgium in the 1970's by De Beer (1971) and then been widely introduced in the Belgian design practice. The De Beer method is based on a thorough application of the principles of the scale effect, when transitioning from a soft to a hard soil layer as shown in figure 5.

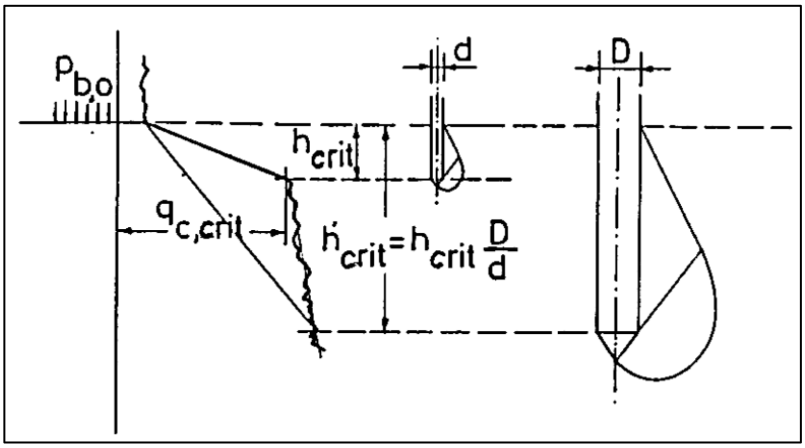


Figure 5: Scale effect principle

This application of the scale effect is done in 4 steps, designated by the terms (a) homogeneous values, (b) descending or downward values, (c) upward values and (d) mixed or blended values. These final mixed values q_b are the basis values for the further base resistance calculation of the pile.
To demonstrate the procedure, step by step results of a De Beer calculation are given in figure 6 for a simplified soil profile.

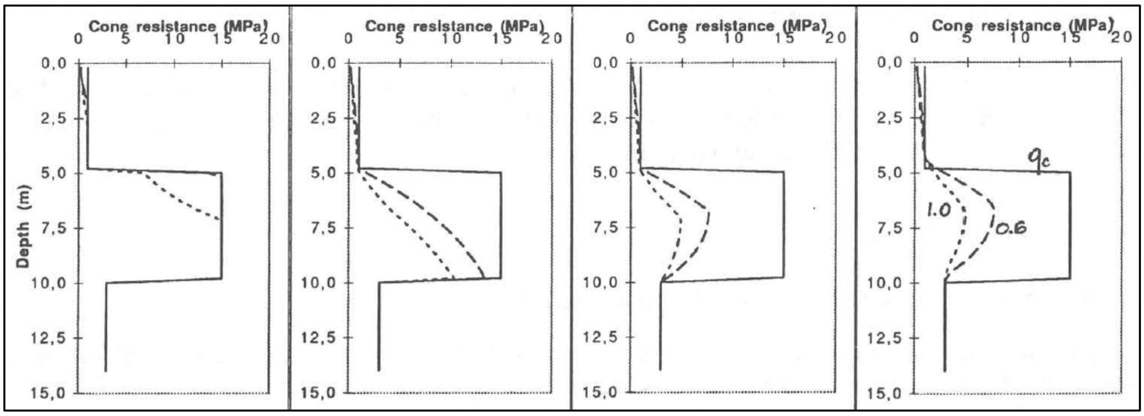


Figure 6: Step by step illustration of the De Beer procedure: (a) homogeneous values, (b) downward values, (c) upward values and (d) blended values; for 0.6 and 1.0 m diameter base, respectively

A practical calculation example to illustrate the scale effect for different pile base diameters is given in figure 7.

The method and later modifications of the De Beer method have also been reported in ECSMFE and ICSMFE (BGGG-GBMS, 1985) proceedings by De Beer and Van Impe among others.
In the Belgian design pile guide the original De Beer method has been retained.

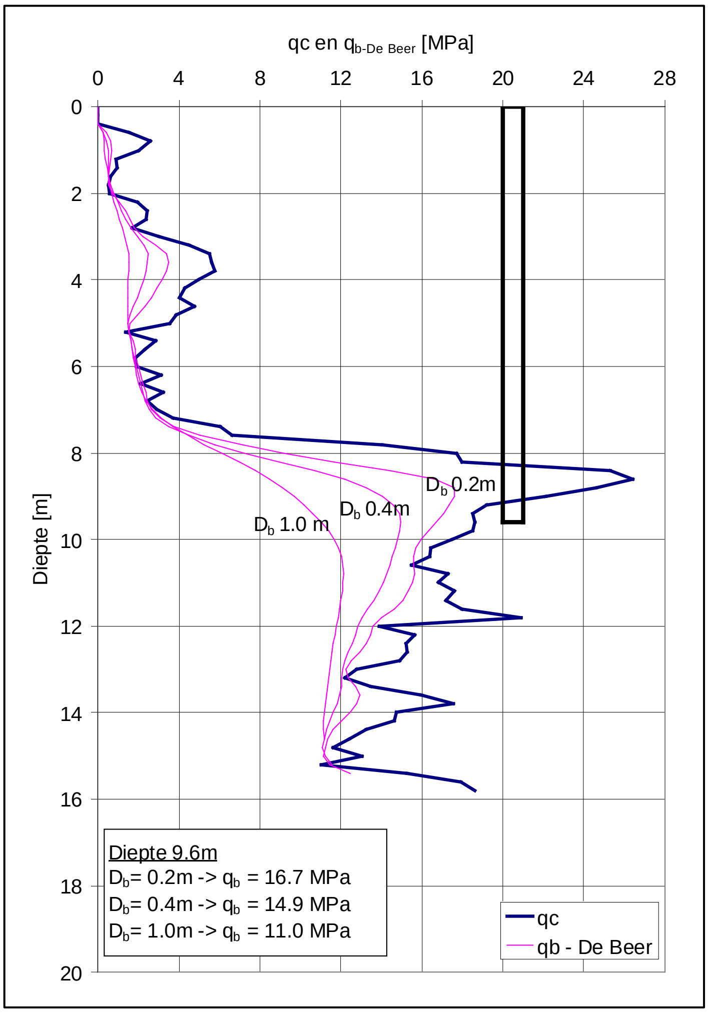


Figure 7: Example of the De Beer method to determine q_b for pile diameters 0.2 m; 0.4 m and 1.0 m

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

$$R_s = \chi_s \cdot \Sigma (\alpha_{s,i} \cdot h_i \cdot q_{s,i}), \qquad (4)$$

With:

$q_{s,i}$ (kPa): the unit shaft friction: $q_{s,i} = 1000 \cdot \eta^*_{p,i} \cdot q_{c,m,i}$

$\eta^*_{p,i}$ (-): an empirical factor, giving the ratio between the unit shaft friction $q_{s,i}$ and the cone resistance q_c for a given soil type. These values are summarized in table 4.

$q_{c,m,i}$ (MPa): the average cone resistance (q_c) for layer i.
In the case that the CPT has been performed with a mechanical cone, q_c needs to be reduced with the values of table 1.
Only relevant layers (in general with $q_c > 1$ MPa) may be considered.

χ_s (m): the perimeter of the pile shaft as defined in §5.2.1.

$\alpha_{s,i}$ (-): an empirical factor for layer i, taking into account the installation method of the pile and the roughness of the pile shaft in a given soil type. These values are summarized in table 5.
For piles that are subjected to an alternating load (which means that in SLS the pile is also subjected to a tension load), the effect of the factors affecting the shaft resistance (number of cycles, amplitude,...) need to be verified. If lack of data or proof of this effect, the values of $\alpha_{s,i}$ in table 5 need to be reduced with a factor 1.33.

h_i (m): the thickness of layer i.

Table 4: Values of the empirical facto η^*_P *and* q_s

Soil type	**q_c (MPa)**	**η^*_p (-) or q_s (kPa)**	**R_f (%) (*)**
Clay	1 – 4.5	η^*_p = 1/30	3–6 %
	> 4.5	q_s = 150	
Loam (silt)	1 - 6	η^*_p = 1/60	2–3 %
	> 6	q_s = 100	
Sandy clay / loam (silt)	1 – 10	η^*_p = 1/80	1-2 %
Clayey sand / loam (silt)	> 10	q_s = 125	
Sand	1 – 10	η^*_p = 1/90	< 1 %
	10 – 20	$q_s = 110 + 4 * (q_c – 10)$	
	> 20	q_s = 150	

(*) determined by means of CPT-E.

In this way a calculated value of the *total pile bearing capacity* $R_c = R_b + R_s$ is obtained for each individual CPT.

Table 5: Values of the empirical factors α_b *and* α_s

Pile type	**Base α_b**		**Shaft α_s[g]**	
	Tertiary Clay	**Other soil types**	**Tertiary Clay**	**Other soil types**
CATEGORY[a] I: PILES WITH HIGH SOIL DISPLACEMENT				
DRIVEN & JACKED PILES				
Precast concrete piles without enlarged base	1	1	0.9	1
Cast in situ piles, without enlarged bottom plate[b], shaft in plastic concrete	1	1	0.9	1
Cast in situ piles, with enlarged bottom plate[b], shaft in plastic concrete	1	1	- [e]	- [e]
Cast in situ piles with enlarged base formed in the ground , shaft in rammed semi-dry concrete	1	1	1.15	1.15
Steel tube pile close-ended, without enlarged bottom plate[b]	1	1	0.6	0.6
Steel tube pile close-ended, with enlarged bottom plate[b]	1	1	- [e]	- [e]
Steel tube pile open-ended, situation with soil plugging[c]	1	1	0.6	0.6
SCREW PILES CATEGORY I[d]				
With temporary tube and shaft in plastic concrete[i]	0.8	0.7	0.9	1.0
With lost tube	0.8	0.8	0.6	0.6
With lost or temporary tube and grout injection during installation[i]	0.8	0.7	0.9	1.0
CATEGORY[a] II: PILES WITH LOW SOIL DISPLACEMENT OR LOW SOIL RELAXATION				
DRIVEN PILES				
Steel pile open ended, situation without soil plugging[c]	1	1	0.6	0.6
Steel profiles and sheet piles	1	1	0.6	0.6
SCREW PILES CATEGORY II[d]				
With temporary tube and shaft in plastic concrete[i]	0.8	0.5	0.6	0.6
With lost or temporary tube and grout injection during installation[i]	0.8	0.5	0.6	0.6
CFA PILES WITH PROVISIONS TO LIMIT SOIL RELAXATION				
With large diameter of the hollow stem and small	0.8	0.5/0.6[j]	0.6	0.6/0.7[j]

Pile type	Base α_b		Shaft α_s [(g)]	
	Tertiary Clay	**Other soil types**	**Tertiary Clay**	**Other soil types**
flanges[(h)]				
With concrete overpressure	0.8	0.5	0.6	0.6
With temporary casing	0.8	0.5	0.3	0.5
CATEGORY[(a)] III: PILES WITH SOIL EXCAVATION				
CFA PILES WITHOUT PROVISIONS TO LIMIT SOIL RELAXATION				
	0.8	0.5	0.3	0.4
BORED PILES				
Executed with temporary casing	0.8	0.5	0.3	0.5
Executed under thixotropic fluid	0.8	0.5	0.5	0.5
Executed without casing or thixotropic fluid (dry boring)	0.8	-[(f)]	0.5	-[(f)]
CATEGORY IV: MICROPILES				
With grout placement under gravity (no pressure)	-[(k)]	-[(k)]	-[(k)]	-[(k)]
With mono-phase or stepwise grout placement under a global pressure higher than gravity pressure	-[(k)]	-[(k)]	-[(k)]	-[(k)]
With multi-phase selective and repetitive grout injection via TAM and double packer	-[(k)]	-[(k)]	-[(k)]	-[(k)]

(a) *The pile category is related to the effect of the pile installation method on the surrounding soil (displacement, relaxation,…)– not to be confounded with the geotechnical category (GC) of a construction.*

(b) *With regard to the effect on the shaft friction (reduced α_s factors) the pile base is considered as"enlarged" if it concerns a prefabricated enlarged base with $D_{b,eq} > D_s + 0.05$ m. The influence on the pile base resistance is assessed by the reduction factor λ.*

(c) *For open-ended tubular steel piles plugging at the lower end of the tube might occur during installation. For the design two situations have to be considered:*

- *situation without plugging: shaft friction on the inner and outer side of the tube, pile base resistance only over the steel section of the base*
- *situation with plugging: shaft friction only on the outer side of the tube, pile base resistance over the complete section of the pile base.*

The minimum value of both needs to be retained for the design.

(d) *Only for screw piles with flanges of maximum 0.10 m (e.g. 36/56)*

(e) *The shaft friction may not be taken into account, unless it is demonstrated by means of SLT on the job site.*

(f) *The pile base resistance and the shaft friction have to be determined by means of instrumented SLT on the job site*

(g) *Reduction in case of alternating load.*

(h) *The diameter of the central stem is at least 50 % from the pile base diameter*

(i) *to be formally decided by the Belgian standardization committee of EC7 if the pile system belongs to category I or II*

(j) *maximum value only for pile systems that have formally been approved by the Belgian standardization committee of EC7*

(k) *under discussion*

The empirical factors in table 5 have been deduced by fitting the results of scientific static pile load test to the semi-empirical calculation models explained above. As for some pile types no scientific load test are available, their installation factors have been assessed based on engineering judgement

In particular circumstances (specific soil type, new or adapted pile type or installation method,…) or in the case of important constructions, specific installation factors can be deduced by means of instrumented static pile load tests. Guidelines to deal with such an approach are given in the Belgian pile design guide. This guide provides also a procedure for pile systems that wish to obtain other installation factors as those published in table 5.

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

$$R_{c,cal} = R_c / \gamma_{Rd}, \qquad (5)$$

with:

$R_{c,cal}$ (kN): the calibrated bearing capacity of the pile
γ_{Rd} (-): the model factor (γ_{Rd1}, γ_{Rd2} or γ_{Rd3} – see table 6).

The values of the model factors are based on statistical analysis, and aim to obtain in 95 % of the cases a calculated value of the bearing capacity that is higher than the "real" (or measured) bearing capacity.
The values of the model factors have been determined per group of pile types and are summarized in table 6. For screw piles and CFA piles, γ_{Rd} depends on the fact if a pile system has (not) been subjected to static pile load testing in comparable geotechnical site conditions or on the job site itself. The reduced model factors (γ_{Rd1} or γ_{Rd2}), may only be applied if the static pile load tests satisfy the requirements of the pile design guide WTCB-CSTC (2009-2016) and have formally been approved by the Belgian standardization committee of EC7.

Table 6: Values of the model factor γ_{Rd}

Group of pile types	**Without SLT: γ_{Rd1} (-)**	**With SLT: γ_{Rd2} (-)**	**With SLT on the job site: γ_{Rd3} (-)**
Driven and jacked piles	1.00	1.00	1.00
Screw piles	1.30	1.10	1.00
CFA-piles	1.35	1.20	1.10
Bored piles	1.20	1.20	1.10

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

$$R_{c,k} = min\left\{\frac{(R_{c,cal})_{average}}{\xi_3};\frac{(R_{c,cal})_{min}}{\xi_4}\right\} \tag{6}$$

The correlation factors are applied in order to take the variation on the soil characteristics and the uncertainty on this variation into account. As this uncertainty depends on the amount of soil investigation, ξ_3 and ξ_4 depend on the CPT density on the job site, as illustrated in table 7 and table 8.
For pile foundations existing out of more than 3 piles the reduced ξ_3 and ξ_4 values of table 7 and 8 may only be applied for structures with sufficient stiffness and strength that allows for a redistribution of the load on a weak pile to the neighbouring piles. According to the Belgian pile design guide, a structure can be considered as stiff, if the removal of one pile leads to a calculated settlement of not more than 5 mm.

Table 7: Values of the correlation factor ξ_3

NUMBER of PILES	**CPT DENSITY**				
	1 CPT 10 m²	**1 CPT 50 m²**	**1 CPT 100 m²**	**1 CPT 300 m²**	**1 CPT 1000 m²**
1-3	1.25	1.29	1.32	1.36	1.40
4-10	1.15	1.19	1.21	1.25	1.29
> 10	1.14	1.17	1.20	1.24	1.27

Table 8: Values of the correlation factor ξ_4

NUMBER of PILES	**CPT DENSITY**				
	1 CPT 10 m²	**1 CPT 50 m²**	**1 CPT 100 m²**	**1 CPT 300 m²**	**1 CPT 1000 m²**
1-3	1.08	1.17	1.23	1.31	1.40
4-10	1.00	1.07	1.13	1.21	1.29
> 10	1.00	1.06	1.12	1.20	1.27

In **step 4**, the design value of the pile resistance $R_{c,d}$ is finally obtained by applying the partial safety factors γ_b ad γ_s from table 9 on the characteristic pile base and shaft resistances:

$$R_{c,d} = R_{b,k} / \gamma_b + R_{s,k} / \gamma_s. \tag{7}$$

The values of the partial factors depend on the guarantee that can be given on the quality of the pile installation. For the moment, the reduced factors can be applied when the piling contractor proves that the pile installation takes place according to a well-established quality plan, often inspired on common quality standards as e.g. ISO 9001. In the future it is however the aim of the Belgian standardization committee of EC7 to set-up an independent process certification system for pile systems that will be mandatory for allowing to apply the reduced partial safety factors.

Table 9: Values of the partial safety factors γ_b and γ_s

Group of pile types	**DA1/1**			
	Without quality assurance		**With quality assurance**	
	γ_b	γ_s	γ_b	γ_s
Driven and jacked piles	1.00	1.00	1.00	1.00
Screw piles	1.07	1.00	1.00	1.00
CFA-piles	1.10	1.00	1.00	1.00
Bored piles	1.20	1.00	1.00	1.00

5.3.3. Axial tension of a single pile

General

A pile can be subjected to a tension load due to external actions (e.g. wind, eccentric loading, pylons,…) and/or ground-water pressure acting on the structure. In that case two failure mechanisms or situations, a GEO and an UPL situation, need to be assessed (see §7.6.3.1 (3) of NBN EN 1997-1).

In a first situation the pile itself is pulled out of the ground, which requires a verification of the friction resistance along the shaft-soil interface. According to Eurocode 7 this is a GEO-verification and the following condition (8) needs to be satisfied [NBN EN 1997-1 §7.6.3.1 (2)]:

$$F_{t,d} \leq R_{t,d}, \tag{8}$$

with:

$F_{t,d}$ (kN): the design value of axial tension load on the pile

$R_{t,d}$ (kN): the design value of the resistance of the pile subjected to a tension load (=shaft friction under tension load)

A second situation that can occur is that the pile and a certain volume of soil sticking to the pile are pulled out of the ground. This is an uplift (UPL) situation according to Eurocode 7, and the following inequality (9) needs to be verified [NBN EN 1997-1 §2.4.7.4 (1)]:

$$V_{dst,d} \leq G_{stb,d} + R_d, \tag{9}$$

with:

$V_{dst,d}$ (kN): the design value of the vertical unfavourable (destabilizing) load

$G_{stb,d}$ (kN): the design value of the permanent favourable vertical load

R_d (kN): the design value of the resistance against uplift

Axial tension load

The axial tension load on a pile can result from external actions and/or ground-water pressure acting on the structure.

Pile loads resulting from ground-water pressures are to be considered as permanent loads. Guidelines for the determination of the ground-water level that can be taken into account to deduce upward pressures are summarized in table 10.

Permanent favourable (stabilizing) loads result from the weight of the structure and, possibly from the effective weight of the soil (e.g. on a tunnel or a reservoir). The latter may only be considered if no excavation will be carried out during the design life time.
The own weight of the pile may be taken into account as permanent favourable (stabilizing) load at the condition that it is mentioned explicitly in the design report.

Table 10: Values of the ground-water level Z_w to be taken into account to assess the upward ground-water pressure

Type of ground-water layer	**Available measurements of the ground-water level** [(a)]	**Z_w (m)** [(c)]
Free – no ground-water lowering	no	Ground surface
	1 measurement	$Z_{w,m}$ + 1.50 m
	Measurement period[(b)] ≥ 6 months	$Z_{w,m,max}$ + 1.00 m
	Measurement period [(b)] ≥ 1 year	$Z_{w,m,max}$ + 0.50 m
Artesian – no ground-water lowering		Out of hydrogeological study
With ground-water lowering		Out of the ground-water lowering design/study

(a) ***Measurements executed by means of a piezometric device installed on the job site.***
(b) ***Minimum 1 measurement a month.***
(c) ***Z_w is the ground-water level that needs to be taken into account; $Z_{w,m}$ is the measured ground-water level; $Z_{w,m,max}$ is the highest measured ground-water level***

Shaft resistance – GEO verification

As explained before the inequality (8) $F_{t,d} \leq R_{t,d}$, need to be verified. The calculation method to determine the pile resistance (shaft friction) under tension load $R_{t,d}$ is quite similar to the one for axially loaded piles under compression. The steps are summarized here below.

Step 1: determination of the pile resistance R_t under tension load according to the following equation:

$$R_t = \chi_s \cdot \Sigma (\alpha_{t,i} \cdot h_i \cdot q_{s,i}), \qquad (10)$$

With:

$q_{s,i}$, χ_s and h_i: see §5.3.2.

$\alpha_{t,i}$ (-): an empirical factor for layer i, taking into account the installation method of the pile and the roughness of the pile shaft in a given soil type. These values are summarized in table 11. For piles that are subjected to an alternating load (which means that in SLS the pile is also subjected to a compression load), the effect of the factors affecting the shaft resistance (number of cycles, amplitude,...) need to be verified. If lack of data or proof of this effect, the values of $\alpha_{t,i}$ need to be reduced with a factor 1.33.

Table 11: Values of α_t

Load	**α_t**
Only tension load	$\alpha_s \div 1.25$
Alternating load	$\alpha_s \div (1.25 * 1.33)$ $= \alpha_s \div 1.66$

Step 2: determination of a calibrated pile resistance under tension load $R_{t,cal}$ by introducing a model factor γ_{Rd} [NBN EN 1997-1 §2.4.1 (6), §2.4.1 (8), §2.4.7.1 (6), §7.6.2.3 (2)]:

$$R_{t,cal} = R_t / \gamma_{Rd}, \qquad (11)$$

with

$R_{t,cal}$ (kN): calibrated pile resistance under tension load
γ_{Rd} (-): the model factor (γ_{Rd1}, γ_{Rd2} of γ_{Rd3}).

The values of the model factor are given in table 6.

Step 3: determination of the characteristic value of the pile resistance by applying the correlation factors ξ_3 and ξ_4 on the average and the minimum value of the calibrated pile resistances respectively, and by retaining the smallest value of both:

$$R_{t,k} = \min\left\{\frac{(R_{t,cal})_{average}}{\xi_3};\frac{(R_{t,cal})_{\min}}{\xi_4}\right\} \quad (12)$$

ξ_3 and ξ_4 are given in table 7 and table 8.

Step 4: determination of the design value of the pile resistance under tension $R_{t;d}$ by introducing the partial factor $\gamma_{s,t}$. In Belgium, $\gamma_{s,t} = \gamma_s$ (see table 9).

The design value of the axial tension load acting on the pile $F_{t,d}$ (kN) is obtained by multiplying the representative value of the load with the partial factor γ_F from table 3:

$$F_{t,d} = F_{t,rep} * \gamma_F, \quad (13)$$

Uplift (UPL) verification for a single pile

The UPL verification implies that the inequality (9) $V_{dst,d} \leq G_{stb,d} + R_d$ is satisfied.
The weight of the soil volume sticking to the pile is considered as a resistance against uplift. The friction along the surface of the soil volume is generally not taken into account. The shape of the volume of soil that is considered in this verification can be deduced from figure 8. The value of the angle α is given in table 12.
In order to calculate the weight of this soil volume, the volumetric weight of the soil according to table 2.1 of the Belgian National Annex of NBN EN 1997-1 is used.

Table 12: Value of the angle α ([a] for the category of piles: see table 5)

q_c	Piles[a]	α (°)
< 1 MPa	Cat. I, II, III	0
≥ 1 MPa	Cat. I	2/3 φ’
	Cat. II, III	1/2 φ’

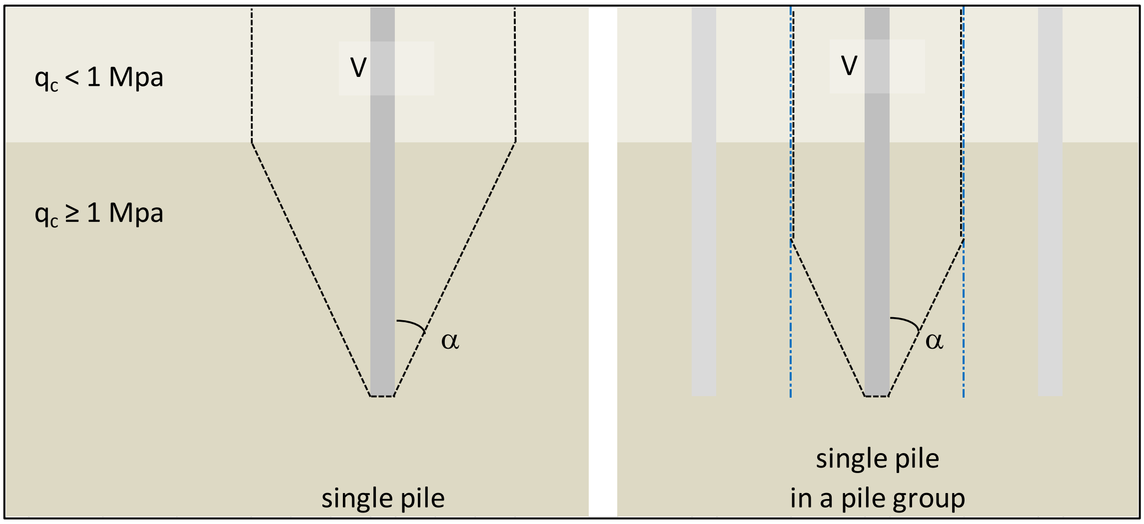


Figure 8: Shape of the volume of soil sticking to the pile in an uplift verification

Other uplift failure mechanisms than the one represented in figure 8 are possible as well, e.g. in the case of piles with an enlarged base, which will rather behave as a plate anchor (Holeyman et al., 1997).

The design values of the axial loads are determined by multiplying the representative values of the load with the partial load factors for the UPL-verification [NBN EN 1997-1 ANB A.4].

By neglecting the shear resistance along the surface of the soil volume and by introducing a model factor γ_{Rd} on the weight of the soil volume, inequality (9) becomes:

$$G_{dst} * \gamma_{Gdst} + Q_{dst} * \gamma_{Qdst} - G_{stb,d} * \gamma_{Gstb} \leq \gamma'_{kluit} * V_{kluit} / \gamma_{Rd} \qquad (14)$$

The partial load factors that need to be applied for the UPL-verification are given in table 13.

Table 13: Partial load factors for the UPL-verification

Load		Symbol	Factor
Permanent	Unfavourable (1)	γG_{dst}	1.0
	Favourable (2)	γG_{stb}	0.9
Variable	Unfavourable (1)	γQ_{dst}	1.1 (3)
(1) Destabilizing. (2) Stabilizing. (3) This value deviates from the informative value in table A.15 of the standard NBN EN 1997-1 [1].			

In the UPL-verification a model factor γ_{Rd} that equals 1,40 is introduced in Belgium. This model factor accounts for the uncertainty on the shape of the soil volume and the fact that the volumetric weight of the soil from table 2.1 the ANB of NBN EN 1997-1 is a high characteristic value.

5.3.4. Lateral loading of a single pile

Until now, no guidelines have been elaborated for the design of laterally loaded piles in Belgium. For such cases, the design is mostly applied with the help of design methods available in the international literature, e.g. the methods published and referenced in Tomlinson (2008) and Reese & Van Impe (2001), or design methods available in foreign standards, such as the French standard (NF P94-262) or the former Dutch standard NEN 6724.
Some numerical programs for retaining structures that simulate the soil as elasto-plastic springs (Winkler model) allow also for the design of laterally loaded piles (e.g. D-sheet) and are regularly applied as well in practice.
For important foundation structures that are subjected to important lateral loading, preliminary lateral load tests are sometimes carried out, as illustrated recently by Verstraelen et al. (2015).

5.3.5. Specific issues

<u>Negative skin friction (downdrag)</u>
The Belgian pile design guide contains a procedure to deal with negative skin friction. The methodology exists in quantifying the settlement of the soil around the pile due to the effect a surcharge, ground-water lowering, etc. The settlement of the soil (and the ground surface) around the pile is determined with characteristic values of the soil parameters (C, A, volumetric weight,…) and representative values of the load. Depending on the calculated settlement of the ground surface, (part of) the effect of negative skin is introduced as a supplementary load in the pile design:

- when the settlement of the soil surface is higher than 10 cm or in the case that no settlements are calculated, the calculated negative skin friction is to be considered as supplementary load acting on the pile
- when the settlement of the soil surface is between 4 and 10 cm, only half of the calculated negative skin friction is to be considered as supplementary load acting on the pile
- when the settlement of the soil surface amounts between 2 and 4 cm, negative skin friction does not need to be taken into account, but no positive shaft friction may be taken into account for the concerned soil layer

- when the settlement of the soil surface is lower than 2 cm, negative skin friction does not need to be taken into account and the designer may decide if positive shaft friction is taken into account or not in the pile design.

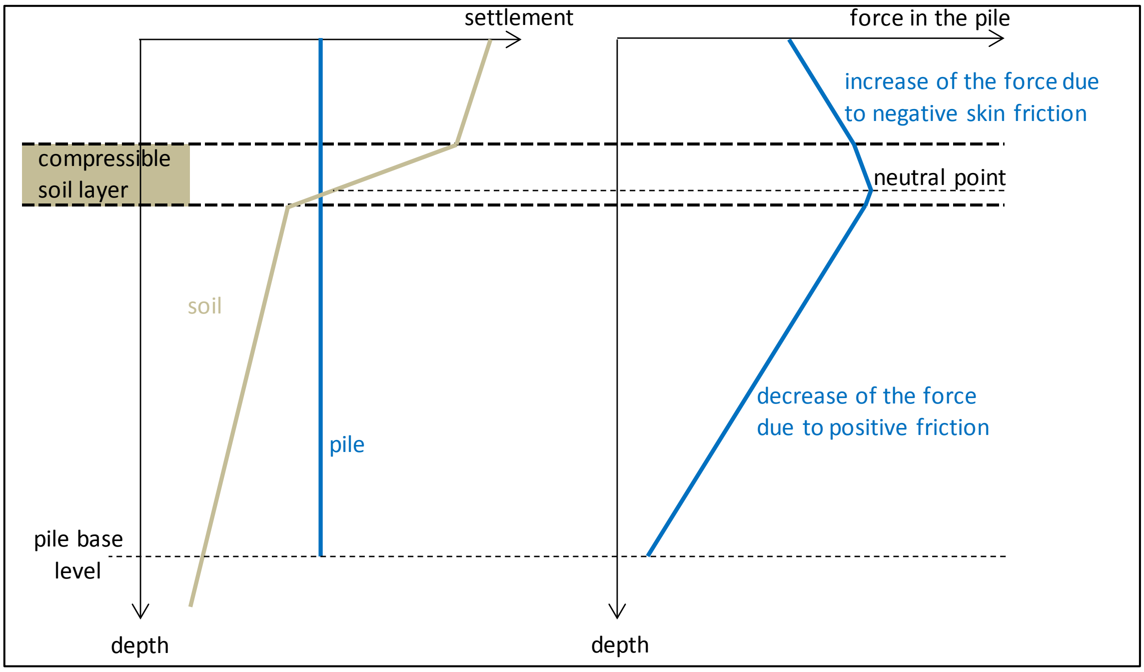


Figure 9: Negative skin friction over the zone along the pile shaft where a relative downward movement of the soil with regard to the pile occurs

The negative skin friction can be calculated with the slip method or with a method analogue to the method to determine the shaft resistance (or the positive skin friction) of a pile.
With the *slip method*, the representative value of the pile load due to negative skin friction is summated over the layers for which a relative downward movement of the soil with regard to the pile (see figure 9) occurs:

$$F_{nk,\,rep} = \chi_s \,.\, \Sigma \,(h_i \,.\, K_{o,i} \,.\, \tan \delta_i \,.\, \sigma'_{v,i}) \quad (15)$$

With:

$F_{nk,\,rep}$ (kN): the representative value of the pile load due to negative skin friction
χ_s (m): the perimeter of the pile as defined in§5.2.1
h_i (m): the thickness of layer i
$K_{o,i}$ (-) $= 1 - \sin \varphi'_i$
δ_i (°) $= \varphi'_i$ for cast in situ concrete piles
δ_i (°) $= 0.75\ \varphi'_i$ for precast concrete piles and steel piles
but $K_{o,i} \,.\, \tan \delta_i$ equals minimum 0.25
$\sigma'_{v,i}$ (kPa) = the average effective vertical stress in layer i
For all these parameters characteristic values are introduced in the formula

With the *method analogue* to the method to calculate the *positive skin friction*, $F_{nk,rep}$ can be evaluated as follows:

$$F_{nk,\,rep} = \chi_s \cdot \Sigma \,(\alpha_{s,i} \cdot h_i \cdot q_{s,i}) \quad (16)$$

with
χ_s (m): the perimeter of the pile as defined in§5.2.1
h_i (m): the thickness of layer i
$\alpha_{s,i}$ (-): an empirical installation factor for layer i as defined in §4.3.3
$q_{s,i}$ (kPa): the unit shaft friction as defined in §4.3.3

The negative skin friction can also be calculated with the method of Zeevaert -De Beer, published by De Beer (1966), De Beer et al. (1968) and Zeevaert (1969). This method includes the reduction of the negative skin friction by the interaction between pile and soil, as well as simplified rules to define the neutral point.

Finally, the design value of the negative skin friction is obtained by applying a partial factor of 1.0 to $F_{nk,rep}$. In the pile design, the negative skin friction does not need to be combined with transient loads. When the pile is subjected to transient loads, only the most disadvantageous of the following combinations needs to be assessed:
Permanent loads + variable loads (long term) + negative skin friction
Permanent loads+ variable loads (long term) + transient loads

Group effect

Until now, no guidelines have been elaborated to take the group effect into account in the ULS design of piles in Belgium. In practice it is in general assumed that for pile inter-distances higher than three times the pile diameter, the effect on the ULS design of the pile foundation can be neglected.
When beneath the base level of the pile foundations compressible layers occur in the zone of influence, the settlement of the pile group is commonly assessed (see §5.4).

Cyclic loading

Until now, no guidelines have been elaborated to take the effect of cyclic loading into account in the ULS design of piles in Belgium. In the case that cyclic loading becomes of importance, the methods that need to be adopted are often specified by the client. Otherwise, methods available in the literature are applied.
In the case that piles are subjected to alternating loads, which means that in the SLS verification the pile is subjected to tension loads and compression loads, the Belgian pile design guide introduces for the ULS verification of axially loaded piles, as highlighted in §5.3.2 and §5.3.3, a reduction factor of 1.33 on the installation factor for the shaft friction in compression α_s and in tension $\alpha_{s,t}$. One can only deviate from this reduction factor in the case that the effect of the factors affecting the shaft resistance (number of cycles, amplitude,...) is verified with proven methods.

Seismic design

Seismic design of foundations in Belgium is assessed by the Eurocode 8, in particular by NBN EN 1998-1 and NBN EN 1998-5 and its national annexes. Figure 10 illustrates the seismic zones and the corresponding reference peak ground acceleration a_{gR} that have been defined in Belgium. The combination of a_{gR}, the importance class of the construction and the ground type (stratigraphic profile) determines the seismic risk. For common buildings with no particular risk for public safety, the seismic risk will be very low (no specific measures) to low (simplified measures) in a large part of Belgium. In zones with a higher a_{gR} (the east and the west of the country) and/or in the case that the construction represents a considerable to high risk with regard to public safety, seismic design needs to be performed according to the principles of the Eurocode 8. With regard to pile foundations this signifies in particular a verification of transverse load resistance of the pile foundations under the action effects of the inertia forces from the superstructure and the kinematic forces arising from the deformation of the surrounding soil due to the passage of seismic waves.
In the case of pile foundations for common constructions, it can be stated that in Belgium seismic design of piles is not often assessed and only exceptionally considered, mostly for important or high-risk structures such as power plants, nuclear plants, high-risk chemical installations, high-rise structu:res,…

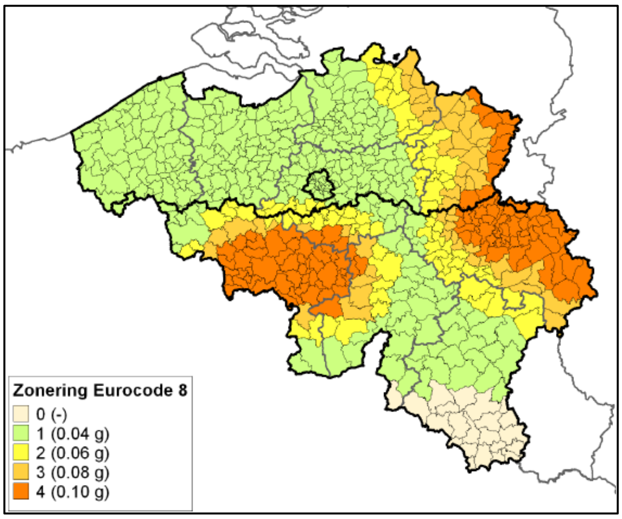


Figure 10: Seismic zones in Belgium and the reference peak ground accelerations a_{gR} according to NBN EN 1988-1: ANB

5.3.6. Problems not covered by National Annexes and future developments

As mentioned before the Belgian national annex of the Eurocode 7, refers to the pile design guide (WTCB-CSTC (2009/2016) as reference method. This pile design guide considers the ULS design of axially loaded piles based on CPT.
The main aspects that for the moment are not covered by the Belgian national annex and the pile design guide are the following:

- the design of micropiles (to be expected by the end of 2016)
- the design of piles based on pressure meter tests for soils where CPT are not applicable
- the design of piles in rock
- the design of piles subjected to lateral loads, dynamic loads, …
- the design of pile groups and piled raft foundations
- the verification of punching.

Another aspect with regard to the design of piles that is not covered by national documents or guidelines is the SLS verification (see §5.4).

5.4. SLS design

For common constructions with pile foundations, serviceability conditions are not usually explicitly analysed in Belgium, as experience has shown that the ULS-design covers often serviceability. One of the reasons is the fact that the semi-empirical calculation methods as explained in §5.3 are fitted to the measured pile resistances of SLT and that for this latter the conventional failure criterion, being the load corresponding with a settlement of the pile of 10 % D_b, is applied.
The Belgian pile design guide does not give particular guidelines with regard to SLS design, and specifies only, in the case that serviceability is an issue, to assess it, e.g. for pile groups located above potentially compressible layers, for end-bearing piles that mobilize their base resistance only sufficiently with increasing settlements (bored piles), for settlement sensitive structures, or for marginal and challenging subsurface conditions.
Serviceability analysis for single piles is then mostly conducted using mobilization curves for the shaft and the end bearing reactions derived from pile load tests performed under similar conditions, and later refined using the results of control load tests performed on the site.
A database of well-documented load-displacement curves in Belgium as well the use of hyperbolic transfer function to estimate the displacement of single piles in practical cases were published by De Cock (2001 & 2008). In De Cock (2008) a range of settlements is given of the relative displacement of different pile types that can be expected at service load (for single piles). This is illustrated in table 14 and table 15.

Table 14: Relative displacement at service load for screw piles according to (De Cock, 2001 & 2008)

Pile type	Shaft bearing	Shaft+end bearing	End bearing
Atlas	0.5 - 0.75 %	0.75 – 1.50 %	1.5 – 1.75 %
Fundex	No data	0.75 – 1.0 %	0.75 – 1.25 %
Omega	0.5 – 0.75 %	No data	2.0 – 2.5 % (*)

(*) *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 15: Relative displacement at service load based BBRI database according to (De Cock, 2008)

Pile type	Shaft bearing		End bearing
Driven Precast concrete	0.5 – 0.75 %		1.0 – 2.0 %
Screw piles	0.5 – 1.0 %	No data	0.75 – 1.5 %
Bored and CFA	0.5 % bentonite 1.0 – 1.5 % casing		0.5 – 1.5 % *

When relevant (e.g. in the case of compressible layers beneath the pile bases and depending on the length and the distance between the piles, the amplitude of the construction area and construction load), the settlement of a pile group can be estimated with the equivalent raft method or equivalent block method as described in Tomlinson (2008). In some cases advanced numerical models are applied.

5.5. Design based on load tests

Although, the Belgian National Annex of Eurocode 7 provides a methodology and correlation factors [NBN EN 1997-1 ANB:2014, Annex A table A.9ANB & table A10ANB] to deduce the design resistance from the measurements of static load tests or from dynamic load tests on the job site, this method is in general not applied in Belgium.
When design load tests are performed on the job site, the results of these tests are mostly linked to the semi-empirical design method based on CPT as explained in §5.3 (see table 6), in order to have the allowance to apply a reduced model factor γ_{Rd}.
Only in exceptional cases the results of preliminary static pile load tests are used to locally calibrate the semi-empirical design method (=determination of the installation factor) and/or to adapt the original pile design based on CPT.
When dynamic load tests are used, calibration with regard to static load testing on the same job site is required.

5.6. Design based on experience

In general not applied in Belgium for structures of GC2.

5.7. Structural safety

The structural resistance of piles needs to be verified according to the structural Eurocodes and their Belgian National annexes. The following standards apply in Belgium:
NBN EN 1992-1-1 and its national annex (ANB) for piles existing out of reinforced concrete,
NBN EN 1993-1-1 and NBN EN 1993-5 and their respective ANBs for steel piles and profiles,
NBN EN 1994-1-1 and its ANB for composite steel and concrete piles.

For the structural design of axially loaded piles, DA1/1 will in all cases be the determining combination. Consequently, the structural pile capacity needs only to be verified for the combination DA1/1.

6. QUALITY CONTROL, MONITORING AND TESTING PRACTICE

6.1. Introduction

As already mentioned in §5.3, the values of the partial factors γ_b and γ_s that need to be introduced in the semi-empirical method based on CPT to deduce the design resistance of the pile capacity, depend on the guarantee that can be given on the quality of the pile installation. For the moment, the reduced factors can be applied when the piling contractor proves that the pile installation takes place according to a well-established quality plan, often inspired on common quality standards as e.g. ISO 9001. In the future it is however the aim of the Belgian standardization committee of EC7 to set-up an independent process certification system for pile systems that will be mandatory in order to apply the reduced partial safety factors.

6.2. Monitoring of pile installation

In addition to controlling the quality of the used materials such as concrete and reinforcement, one can also monitor the different parameters during the installation of the pile. To ensure the reliability of the monitoring, some basic data must always be recorded, such as: date and time of installation, coordinates of the pile location, etc. The execution standards of pile foundations elaborated by CEN TC288 and mentioned in §4 and in addition the series of data sheets of piling systems that has been initiated in Belgium (WTCB-CSTC, 2014), specify minimum requirements.
The following paragraphs address specific aspects of the monitoring of impact driven piles on one hand and drilled and screwed piles on the other hand.
To prevent the piles (precast piles, steel tubes, steel beams,…) from being damaged during driving, one should guard carefully, among other things, against excessive tensile stress waves. So, it is judged important to obtain, during the driving process, information on driving stresses, energy, as well as dynamic skin friction and toe resistance. The blow count against the penetration of the pile, which is normally recorded by the piling foreman, can also be registered by a monitoring device. Monitoring devices (such as Pile Driving Analyser (PDA) or blow count recorders versus depth) are not always specified, but are sometimes used by the piling contractors. The end of driving monitoring (final blow counts) can be used to check the adequacy of and fine-tune the penetration required by design as follows. During driving the 1st pile at the very location or at least in the close vicinity of a CPT test, the set is measured at the proposed level. This set is then imposed within a narrow margin (typically 30 %) when driving the neighbouring piles. Driving is thus to be continued until each pile is placed in the same layer as the test pile and at such a depth that the same set is obtained. For calculating the set (or the penetration per blow), the mean value over the last 10 or 25 cm or the mean value of 5 consecutive observations of 10 blows is taken.
Drilled and screwed piles are monitored with regard to the drilling or screwing process as well as the concreting process. Depending on the piling system and monitoring system, several of the following parameters are generally recorded: speed of penetration, speed of rotation, depth, and rotational torque (usually inferred from the hydraulic oil pressure of the drill table). For screwed pile types of category II and CFA piles, the scraping effect is an important parameter, especially in non-cohesive soils. The scraping factor is the number of rotations needed to penetrate the screw/auger over a depth equal to the pitch of the screw blades, and might be a measure to evaluate the relaxation of the soil around the pile shaft.
The concreting which is most often performed using a pump must be controlled by a monitoring device measuring the volume of the used concrete, the pressure applied to the concrete and the pull-out speed. For some pile types used in Belgium, a computer-based monitoring system has already been implemented. When demonstrated to be reliable, that type of monitoring is requested more and more often by the quality control department of the owners and consultants.
Soil relaxation resulting from the installation process can be evaluated on the basis of soundings performed alongside the pile and comparison with the soundings before pile installation. An example published by Bottiau (2014) is given in figure 11.

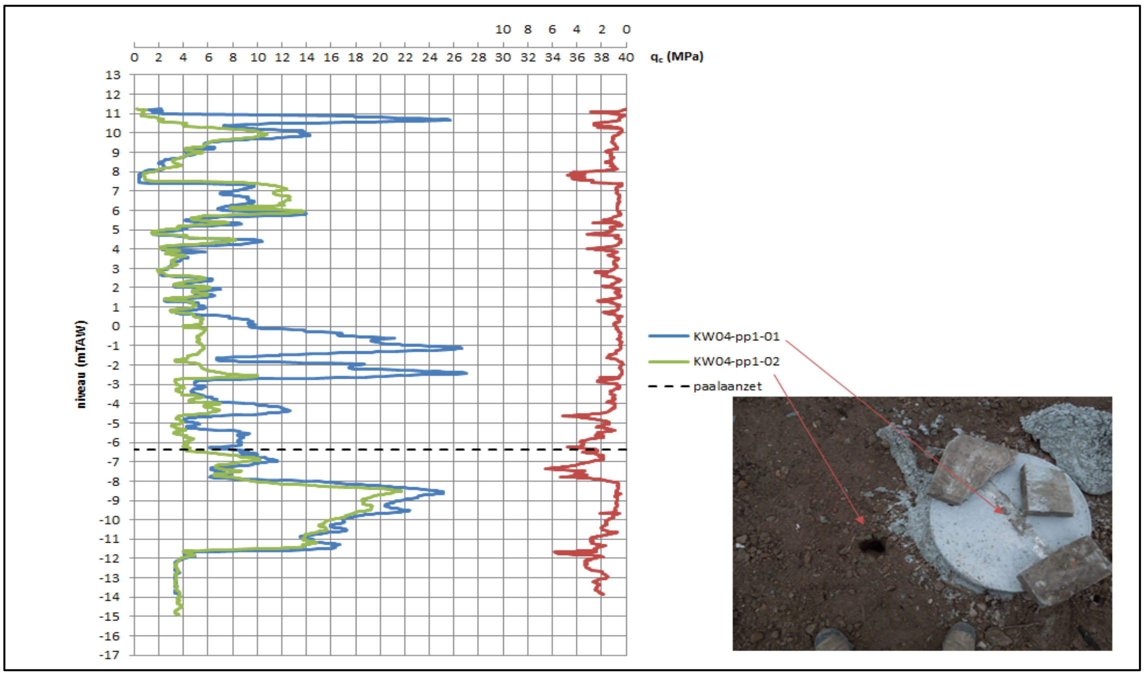


Figure 11: Comparison of the CPT before (blue) and after (green) pile installation – CFA with large hollow stem (Bottiau, 2014)

6.3. Visual inspection, non-destructive testing and core sampling

Although *visual inspection* gives limited information on the top surface and the small portion of the shaft which may be exposed, it is always performed.

Information on pile integrity is obtained using cross-hole sonic logging or gamma-gamma logging, the sonic echo method, and the mechanical admittance method.
Cross-hole sonic logging and *gamma-gamma logging* is usually performed on large diameter bored piles using access tubes mounted on the reinforced cage to evaluate the quality of concrete between emitter and receiver.
Depending on the extent and the success of the testing program, see e.g. De Jaeger, et.al (1988), Huybrechts (2001) and Huybrechts et al. (2003), the evaluations expected from the *sonic echo method* and the *mechanical impedance method* are the length of the pile, its cross-section, the extent to which these dimensions vary, the density of the concrete, the propagation velocity of stress waves in the pile and the soil, the pile toe condition in the bearing layer, etc…
Belgian experience of the sonic echo method has evidenced several limitations in the case of cast-in-situ concrete piles (driven, screwed, vibrated, injected or bored) which often have a very irregular lateral surface. A limitation has been found when one encounters several discontinuities in a particular pile: the number of echoes which may be partially superimposed is thereby increased and can make the interpretation of the graphs more difficult. Another limitation has been identified when heavy damping of the signal due to the corrugated texture of the shaft prohibits in some cases the interpretation of the test. It has also been observed that the wave speed travelling in piles with a screw shaped shaft is lower than the concrete bar wave speed.
The mechanical admittance method is used when quantification of the pile cross-sectional area and of the pile-soil interaction parameters is needed, in addition to information regarding the integrity of the pile.
Although most pile types that are installed on the Belgian market nowadays show some of the above mentioned inconveniences with regard to the interpretation of integrity testing, one notices an increased application of the sonic echo method in practice.

Especially for large diameter bored piles, *vertical core sampling* is sometimes carried out. The sampling provides a continuous control concerning the quality of concrete in the pile shaft. Continuing the sampling through and beyond the toe of the pile allows one to examine the contact between the base of the pile and the bearing soil layer.
The static loading test (see §6.3) is still in Belgium the most widely accepted method to test the integrity and to verify the bearing capacity.

6.4. Static load testing

6.4.1. Control tests

For large projects and/or in case of doubt or discussion about the installation or the performance of the piles, control tests can be required. Piles submitted to control tests are mostly loaded up to 1.5 times the design load in serviceability limit state. In many cases the acceptance criteria based on the "Technical Specification TB/CT 104 - Index 21/A" of the Federal Public Building Agency are referenced in practice. These specifications require that the pile base settlement is smaller than 3 mm or 6 mm for a load of respectively 100 % and 150 % of the design load in serviceability limit state. As these acceptance criteria have been derived from load test on mainly driven cast in situ piles with enlarged pile bases in the past, they seem however too severe for many pile types that are installed nowadays, and in particular for pile systems that mobilize their bearing capacity with somewhat larger pile settlements, e.g. like screw piles of Category I and Category II, as well as CFA piles. It seems more reasonable to adopt for these pile types acceptance criteria based on the values in tables 14 and 15 and/or on the settlements that are acceptable for the superstructure.

6.4.2. Design tests

As explained in §5.5 design as such based on the results of static load tests is not the common practice in Belgium.
Design tests, whereby specific (preliminary) test piles are loaded up to at least 200 % of the design load in serviceability limit state, are sometimes realized on specific job sites in order to calibrate locally the semi-empirical relations, especially in the case of new/adapted pile systems and/or specific soil conditions.
Since the publication of the Belgian pile design guide in 2009, the number of design tests that are executed is however increasing, as it allows to apply a reduced model factor γ_{Rd} in the semi-empirical design method based on CPT(see table 6).
Especially the Belgian Railways have performed many design tests on instrumented piles for that purpose in the last decade.

6.4.3. Test procedures

In Belgium the maintained load test procedure is applied. Details of the test procedure on axially loaded piles that has been applied for most design tests in the last 2 decades is extensively described in Maertens et.al (2001 and 2003). This test method procedure, which can be considered as the Belgian practice, is based on the procedures of ISSMFE (1985); NF P94-150-1, NF P94-150-2 and De Cock et al (2003).
For laterally loaded piles the NF P94-151 is applied.

6.5. Dynamic load tests

Dynamic load tests with measurement of the strain and velocity of the pile head are used sometimes to evaluate the behaviour of the pile.

The application of dynamic load tests, with measurement of the strain and velocity of the pile head, has been studied extensively in Belgium in the framework of several research programs, where the output and analysis of dynamic load tests on mainly displacement piles (driven and screwed) has been compared with the results of static load testing. One can refer to Holeyman (1984 & 1987), Holeyman et al. (1988), Holeyman et al. (2001) and Holeyman et al. (2003). With regard to the 2 last references kinetic load testing has been included as well in the test programs.
Deductions from dynamic load testing are made using available methods based on the wave equation, including the Case and Capwap type approaches. Studies of the Case method in Belgium (Holeyman, 1984) tend to show that the result depends strongly on the shape of the impacting force diagram (role of helmet) and on the level of energy.
For the Capwap-type procedure, Belgian experience has found a reasonable degree of reliability for the prediction of the ultimate skin friction and of the loading curve at the base, up to the mobilized load (Holeyman, 1984). The ultimate failure load, if required, is then a matter of extrapolation as in the case of a loading test not carried out to failure. The output of prediction events in Holeyman et al. (2001 & 2003) show however that extrapolation can lead to significant differences with the static pile capacity measured in static load tests.
For that reason dynamic load testing is not allowed in Belgium as design test, unless it is locally calibrated with static load testing. This methodology is rather exceptionally applied (e.g. for big piling projects).

Dynamic load testing is however sometimes accepted for control tests.

7. PARTICULAR NATIONAL EXPERIENCES AND DATABASES

At the occasion of the first ETC3 symposium that took place in 1997, an extended overview of pile load test campaigns that were performed in Belgium in the period 1968 – 1995 was given in the Belgian report. For more details of these individual test campaigns reference is made to Holeyman et al. 1997.
Since 1995, many other pile load test campaigns have been carried out on different pile types, most of them instrumented. A general overview and the references are given in Table 16.
Particular reference is made to the scientific research programs in Sint-Katelijne Waver (1999-2001) and Limelette (2001-2003) were about 48 piles, 5 different types of displacement screw piles and driven precast piles, have been load tested (static, dynamic and kinetic tests). The detailed results and analysis of these tests have been published by Holeyman (2001) and Maertens & Huybrechts (2003).
In combination with the former experimental database presented in Holeyman et al. (1997), the results of these scientific test campaigns have been important to elaborate the first version of the Belgian pile design guide in 2009 (WTCB-CSTC, 2009).
All the other tests mentioned in table 16 were design tests executed at real job sites, many of them of the Belgian Railways, on (preliminary) test piles. These design tests were executed in order to calibrate the semi-empirical design methods for pile types or soil type were no load tests were available and/or in order to have the allowance to apply a reduced model factor γ_{Rd} in the semi-empirical design method. The results of these design tests have been considered in the 2016 revision of the Belgian pile design guide, but require undoubtedly further analysis in order to refine the semi-empirical design method based on CPT in the future. It is to be expected that many of these tests, for which exists for the moment only internal reports, will be published in the coming years.

Table 16: Overview of Test sites with pile load tests in the period 1997 – 2015 in Belgium

TEST SITES		STRATIGRAPHY	PROGRAM INFORMATION		
Site name & period	**References**		**Number**	**Type of piles tested**	**Objectives - results**
Feluy – 1997	Bottiau et al. 1998 Van Impe et al. 1998 Peiffer et al. 1998	(sandy) silt and tertiary Ypresian sandy clay	4	displ. screw piles	4 SLT
Antwerp I – 1999	Maertens et al. 2003c	Silt-sand and tertiary Boom clay	2	2 drilled piles with temporary steel casing	2 SLT
Antwerp II - 1999	Watt et al. 2000	sand and tertiary Boom clay	1	1 diaphragm wall element	1 SLT
St.-Katelijne-Waver - 1999 – 2001	Holeyman 2001 Huybrechts et al. 2008	tertiary Boom clay	24	20 displ. screw piles 4 precast driven	12 Instrumented SLT: α_b & α_s 6 DLT & 6 STN
Limelette II - 2001- 2003	Maertens et al. 2003b Huybrechts et al. 2008	silt (loam) and sand	24	20 displ. Screw piles 4 precast driven	12 Instrumented SLT: α_b & α_s 6 DLT & 6 STN
Loenhout – 2002	Theys et al. 2003	silt/clay and sand	2	1 displ. screw pile 1 CFA with temporary casing	2 instrumented SLT: α_b & α_s
Ekeren – 2003	Internal report – not published	dense sand	2	Tubular screw piles with grout injection,	2 Instrumented SLT: α_b & α_s
Kortrijk – 2007	Internal report – not published	tertiary ypresian clay	4	Drilled piles, 2 with temporary steel casing, 2 under support fluid	4 Instrumented SLT: α_b & α_s
Gent Kantienberg – 2007	Internal report – not published	sand and clayey sand	2	Soil mix piles	2 Instrumented SLT: α_b & α_s Structural resistance
Cerfontaine – 2009	Internal report – not published	slay and rock	2	Bored piles	1 instrumented SLT: α_b & α_s 1 instrumented SLT: α_{st}
Limelette III – 2009	Internal report – not published	silt (loam) and sand	4	Soil mix piles	4 Instrumented SLT: α_b & α_s Structural resistance
Haren I – 2009	Internal report – not published	weak clay and clayey sand	3	CFA with large hollow stem	3 SLT (2 instrumented): α_b & α_s
Anderlecht – 2011	Internal report – not published	Weak clay and clayey sand	2	Tubular screw piles with grout injection,	2 Instrumented SLT: α_b & α_s
Anderlecht - 2012	Internal report – not published	Fill-weak clay-sand and silty clay	2	CFA with large hollow stem	2 Instrumented SLT: α_b & α_s

TEST SITES		STRATIGRAPHY	PROGRAM INFORMATION		
Site name & period	**References**		**Number**	**Type of piles tested**	**Objectives - results**
Merksem I – 2012	Internal report – not published	sand-clay-silt and sand	2	Jet grout piles	2 Instrumented SLT: α_b & α_s
Merksem II – 2012	Internal report – not published	sand-silt and sand	4	2 screw piles with grout injection 2 tubular screw piles with grout injection	4 instrumented SLT: α_b & α_s
Oostende – 2012	Van Impe P.O. et al 2013 Van Impe P.O. et al. 2015	fill and sand	1	Displ. screw pile	1 instrumented SLT: α_b & α_s Load-settlement behaviour pile group
Anderlecht – 2012	Internal report – not published	Clay-sand and clay	1	CFA with large hollow stem	1 Instrumented SLT: α_b & α_s
Anderlecht - 2012	Internal report – not published	Sand-silt and sand	2	tubular screw piles with grout injection	2 instrumented SLT: α_b & α_s
Merksem III – 2013	Internal report – not published	sand-silt and sand	6	screw piles with grout injection	4 instrumented SLT: α_b & α_s 2 lateral load tests: p-y curves
Haren II – 2013	Internal report – not published	clayey sand	1	screw piles with grout injection	1 instrumented alternated SLT: α_b & α_s Load-settlement behaviour pile group
Mechelen – 2013	Verstraelen 2015	Silt/clay and sand	6	4 cast in situ driven 2 CFA with large hollow stem	4 instrumented SLT: α_b & α_s 2 lateral load tests (on c.i.s. driven piles: p-y curves
Mechelen - 2014	Internal report – not published	Silt/sand and sand	2	tubular screw piles with grout injection	2 instrumented SLT: α_b & α_s
Melle - 2015	Internal report – not published	Sand-Clay and sand	2	Displ. screw piles	4 instrumented SLT: α_b & α_s

8. DESIGN EXAMPLE

In this paragraph an example is given of the semi-empirical calculation method as specified in the Belgian pile design guide WTCB-CSTC (2016). Only the geomechanical design in ULS of a single pile is assessed. No group effect, nor the SLS is assessed in the example.

Project data:

- Type of building: residence (apartments)
- Building surface: 15 x 40 m²
- Representative load: 46 kN/m² (80 % permanent and 20 % variable)
- Stiff construction (redistribution of loads is possible)
- Type of piles: precast driven – square section 35 x 35 cm²
- Soil investigation: 3 CPT with electrical cone (Figure 12), distributed over the building surface
- Excavation after soil investigation 1 m
- The pile base level is supposed at a depth of 11 m with regard to the original soil surface

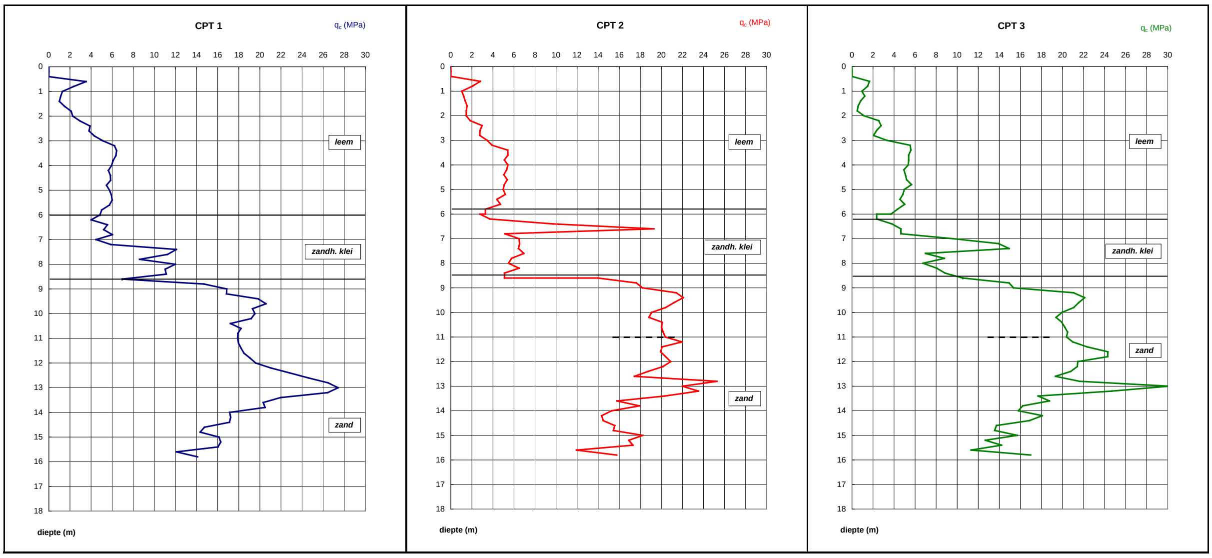


Figure 12: Results of the 3 CPT-E: cone resistance with depth and identification of soil layers

Design value of the load

Formula (2): $F_{c,d} = F_{c,rep} * \gamma_F$,

$F_{c,rep}$ = 46 kN/m² * 15 m * 40 m = 27 600 kN

$\gamma_F = 0.8 * 1.35 + 0.2 * 1.5 = 1.38$ (table 3, DA1/1)

$\Rightarrow F_{c,d}$ = 27 600 * 1.38 = 38 088 kN

Pile base resistance of a single pile

Formula (3): $R_b = \alpha_b \cdot \varepsilon_b \cdot \beta \cdot \lambda \cdot A_b \cdot q_b$,

A_b = 0.35 m * 0.35 m = 0.1225 m²

λ = 1.00 (no enlarged base)

β = 1.00 (square section)

ε_b = 1.00 (no tertiary clay)

α_b = 1.00 (see table 5, Category I – Driven piles - precast driven without enlarged pile base)

q_b calculated with the De Beer method; pile base diameter to be introduced in the method is $D_{b,eq}$ = 0.395 m (see §5.2.1)

Pile base resistance, calculated with the De Beer method:

	CPT 1	CPT 2	CPT 3
q_b	16.17 MPa	16.90 MPa	17.46 MPa
$\Rightarrow R_b$	**1.98 MN**	**2.07 MN**	**2.14 MN**

Shaft resistance of a single pile

Formula (4): $R_s = \chi_s \cdot \Sigma\, (\alpha_{s,i} \cdot h_i \cdot q_{s,i})$

Determination of q_{si}:
- $q_{s,i} = \eta^*_{p,i} * q_{c,i}$
- based on average $q_{c,i}$ over the layer (or individual values)
- only layers with q_c > 1 MPa are considered
- only relevant layers are considered, in this case 3 layers: silt (loam) – clayey sand – sand: see table 4

h_i = 0.2 m
α_{si} = 1.00
χ_s = 4 * 0.35 m = 1.40 m

		CPT 1	CPT 2	CPT 3
$q_{s,i} * h_i * \alpha_{s,i}$	Loam layer	0.371 MN/m	0.294 MN/m	0.307 MN/m
	Clayey sand layer	0.239 MN/m	0.202 MN/m	0.215 MN/m
	Sand layer	0.336 MN/m	0.368 MN/m	0.353 MN/m
$\Sigma\, (\alpha_{s,i} * h_i * q_{s,i})$		0.945 MN/m	0.864 MN/m	0.875 MN/m
⇒ R_s		**1.32 MN**	**1.21 MN**	**1.22 MN**

Total compressive resistance of a single pile
$R_c = R_b + R_s$
Calibration of the calculation method $R_{c,cal} = R_c * \gamma_{R,d}$ (formula 6), with $\gamma_{R,d}$ = 1.00 (see table 6)

	CPT 1	CPT 2	CPT 3
R_b	1.98 MN	2.07 MN	2.14 MN
R_s	1.32 MN	1.21 MN	1.22 MN
R_c	3.30 MN	3.28 MN	3.36 MN
⇒ $R_{c,cal}$	**3.30 MN**	**3.28 MN**	**3.36 MN**

Characteristic value of the pile resistance
Formula (6): $R_{c,k} = min\left\{\frac{(R_{c,cal})_{average}}{\xi_3}; \frac{(R_{c,cal})_{min}}{\xi_4}\right\}$

CPT density: 3 CPT's/600 m² = 1 CPT/200 m²
⇒ determination of ξ_3 (Table 7) and ξ_4 (Table 8) by interpolation between values for 1 CPT/100 m² and 1 CPT/300 m².
ξ_3 = 1.22 (stiff structure and number of piles estimated as > 10 piles)
ξ_4 = 1.16 (stiff structure and number of piles estimated as > 10 piles)
From the previous table:
$R_{c,cal,average}$ = 3.32 MN ⇒ $R_{c,cal,average}$ / 1.22 = 2.72 MN
$R_{c,cal,\ min}$ = 3.28 MN ⇒ $R_{c,cal,\ min}$ / 1.16 = 2.83 MN
⇒ $R_{c,k}$ = 2.72 MN (the average pile resistance is determining)

Design value of the pile compressive resistance
Formula (7): $R_{c,d} = R_{b,k} / \gamma_b + R_{s,k} / \gamma_s$.
$R_{b,k} = R_{b,average} / \xi_3$ = 1.69 MN
$R_{s,k} = R_{s,average} / \xi_3$ = 1.03 MN
γ_b = 1.00 and γ_s = 1.00 (table 9, DA1/1)
⇒$R_{c,d}$ = 2.72 MN

Determination of the number of piles
$F_{c,d}$ = 38 088 kN
$R_{c,d}$ = 2720 kN

$R_{c,d} \geq F_{c,d}$ ⇒ 15 Piles

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