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GUIDELINES FOR THE APPLICATION OF THE EUROCODE 7 IN BELGIUM ACCORDING TO THE NBN EN 1997-1 ANB Part 1: the soil mechanical design in the ultimate limit state (ULS) of axially loaded foundation piles and micropiles based on static CPTs (revision of Report No. 19)

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REPORT NO. 20 GUIDELINES FOR THE APPLICATION OF THE EUROCODE 7 IN BELGIUM ACCORDING TO THE NBN EN 1997-1 ANB

Part 1: the soil mechanical design in the ultimate limit state (ULS) of axially loaded foundation piles and micropiles based on static CPTs (revision of Report No. 19)

The first edition of this document was prepared under the leadership of the BBRI working group 'Eurocode 7 – Poles'. This second revision was carried out by a select working group and approved by the standardisation committee NBN E25007 'Eurocode 7 – Geotechnical design', whose secretariat is provided by the BBRI and SECO.

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C. Basunga Ngelesi (FOD Economie), C. Bauduin (BESIX/KUL), M. Bottiau (ABEF), B. Buysse (SECO, secretaris van de commissie), R. Caspeele (UGent), J. Deceuster (CFE), H. de Chaunac (Fondytest), F. De Cock (Geo.be), F. De Meyer (SECO), M. De Vos (WTCB, secretaris van de com- missie), B. François (ULB), P. Gerard (ULB), K. Haelterman (Vlaamse overheid, MOW, Afdeling Geotechniek), A. Holeyman (UCL), N. Huybrechts (WTCB/KUL), S. Huyghe (CIVIEL), G. Jaspar (SPW), E. Leemans (ABEF), A. Madarasz (NBN), W. Maekelberg (Tuc Rail), J. Maertens (Jan Maertens bvba), L. Maertens (BESIX, voorzitter van de commissie), I. Mariën (Vlaamse overheid, MOW, Afdeling Expertise Beton en Staal), P. Meireman (Geo Design), P. Mengé (DEME Group), H. Peiffer (Alpha studiebureau), X. Raucroix (BESIX), O. Rens (ABEF), C. Treve (CT-Geoconsult), L. Vincke (Vlaamse overheid, MOW, Afdeling Geotechniek), P. Vandenbosch (Regie der Gebouwen), J. Verstraelen (ABEF), V. Whenham (BESIX), Thomas Wulleman (Franki Foundations)

The present guidelines are based on the most recent literature on this subject and also on the test data on the load-bearing capacity of foundation piles. In the preparation of this Report, the aim was to achieve the greatest possible usefulness. However, the members of the working group and the standardization committee cannot be held responsible for any inaccuracies in this document.

The guidelines included in this Report can be applied in Belgium from the moment of their publication for the soil mechanical design in the ultimate limit state (ULS) of axially loaded piles based on static CPTs.

If the design is carried out in accordance with the Belgian National Annex to the Eurocode 7 – Part 1 (NBN EN 1997-1 ANB [B11]), these directives must be applied to the soil mechanical design in the ultimate limit state (ULS) of axially loaded piles based on static CPTs and belonging to the geotechnical category 2 (see § 2.1, p. 9).

SCIENTIFIC AND TECHNICAL CENTRE FOR THE CONSTRUCTION INDUSTRY

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This is a publication of a scientific nature. The aim is to help disseminate the results of the building research from home and abroad.

Contents

FOREWORD

The first part of the Eurocode 7 entitled 'Geotechnical design – General rules' was published in 2005 as the Belgian standard NBN EN 1997-1 [B9]. In 2014, a revision of this standard, the NBN EN 1997-1/A1 [B10], was published.

In 2014, the Belgian National Annex to this standard was also published (the NBN EN 1997-1 ANB [B11]). This Annex lays down a number of choices and values at national level, but does not stipulate calculation methods. Parallel to the preparation of this Report, the necessary adjustments and references have also been integrated into the NBN EN 1997-1 ANB.

At the same time, guidelines have been developed that describe the application of the Euro Code 7 in Belgium in a detailed and pragmatic manner. These activities were started within the interprofessional BBRI working group 'Eurocode 7 – Piles' and continued within the standardisation committee NBN E25007 'Eurocode 7 – Geotechnical design'. As input for the committee's work, grateful use was made of the results of various pre-normative research projects, which were organized by the BBRI and co-financed by the FPS Economy, the NBN and the ABEF (Belgian Association of Foundation Works Contractors).

This document is a revision of the first part of these guidelines, which was published as Report No. 19 and in which the design of the soil mechanical design in the ultimate limit state (ULS) of axially loaded foundation piles by means of static CPTs (CPTs) was explained.

On the one hand, this revision relates to the integration of the design of micropiles into the Report. On the other hand, a number of adjustments have been made to the text in order to align the content of the Report with the quality framework for pile systems that has been set up in Belgium in the meantime.

With the publication of this practical document, a number of fundamental changes are made to the design of axially loaded foundation piles. For example, a change is made from a deterministic method to a semi-probabilistic approach and procedures and a framework are provided for the optimization of the design, the valorization of investments in a high-quality execution and the development of new systems.

1 INTRODUCTION

This Report describes the **soil mechanical design** in the **ultimate limit state** (ULS) of **axially loaded foundation piles** based on the results of **static** CPTs.

However, the following aspects are not covered in this document:
- the inspection of the foundation element itself (Eurocode 2 for concrete piles, Eurocode 3 for steel piles) poles...)
- the control of settlements (serviceability limit state – BGT – and ultimate limit state) – ULS – of the overlying structure that is subject to excessive differential settlements of the piles)
- Loads other than axial compressive and tensile loads (horizontal loads, dynamic loads) gen...)
- the load-bearing capacity of pile groups and of combined piled-raft foundations
- the load-bearing capacity of poles in or on rock
- the control of punching
- the soil mechanical design of foundation piles by means of pressiometer tests, pile loading, tests ...

If these aspects apply, they should be checked by the designer. In the following parts of this Report, guidelines will be formulated on this subject.

In certain cases (e.g. for pile groups set in or above highly compressible soil layers, for bored piles where the bearing capacity is mainly due to the point resistance or for structures that are highly susceptible to settlement), the serviceability limit state (SLS) may be decisive.

The calculation method described in this document can also be used to determine the load-bearing capacity of axially loaded diaphragm walls and pile walls.

A good design is based on a thorough soil investigation. Particular attention must be paid to the quality, scope and reporting of the soil investigation. In 2012, the BGGG (Belgian Group for Soil Mechanics and Geotechnics) established a number of standard procedures for geotechnical research [B1].

When this Report was drawn up, it was assumed that the piles will be carried out in accordance with current regulations, by qualified personnel and with appropriate material and equipment. It was also assumed that the implementation would be thoroughly checked and monitored (see the implementation standards NBN EN 12699 for displacement piles [B14], NBN EN 1536 for bored piles [B2] and NBN EN 14199 for micropiles [B15]).

The Eurocodes and the guidelines in this Report apply to current constructions and design conditions, but in no way replace a good technical assessment.

The values of the safety factors lead to a normal acceptable level of safety. However, in certain cases, it may be appropriate or permitted to increase or decrease the level of reliability. More information about the choice of the confidence level and the way in which this level can be achieved can be found in the NBN EN 1990 [B3] and the accompanying National Annex [B4].

The guidelines mentioned in this Report are valid for foundation piles whose length is at least five times greater than the pile diameter.

2 GEOTECHNICAL CATEGORIES, DEFINITIONS AND SYMBOLS

2.1 GEOTECHNICAL CATEGORIES

In order to establish the geotechnical design requirements, three geotechnical categories have been introduced in Eurocode 7. The general principles for determining the geotechnical categories (GCs) can be found in § 2.1 of the NBN EN 1997-1 [B9]. These are supplemented by the provisions of the NBN EN 1997-1 ANB [B11]. In this National Annex, an additional distinction is made between the geotechnical categories 2A and 2B (GC2A and GC2B).

Geotechnical category 1 (GC1) only covers small and relatively simple structures with a negligible risk. Geotechnical category 2 (GC2) includes conventional construction and foundation types without extraordinary risks and/or complex soil conditions and loads. The geotechnical category 3 (GC3) includes all geotechnical structures that do not belong to the geotechnical categories 1 or 2.

The guidelines in this Report apply to structures of the geotechnical category 2 (GC2). Common pile foundations belong to the geotechnical category 2B (GC2B).

A number of examples of pile foundations that belong to geotechnical category 3 (GC3) are:

- pile foundations of some wind turbines (depending on the size of the wind turbine, the soil conditions) and the consequences of failure)
- pile foundations of liquid tanks (due to the lack of hidden safeties)
- pile foundations of *offshore structures*.

For pile foundations of the geotechnical category 3 (GC3), the Eurocodes and the guidelines must be evaluated with particular attention to the project conditions and must be adapted and/or supplemented if necessary.

2.2 DEFINITIES

For the general definitions, we refer to the NBN EN 1990 [B3] and the NBN EN 1997-1 [B9].

2.2.1 PILE POINT LEVEL AND PILE BASE DIAMETER

The pile point level is defined as the lowest level at which the pile base has its maximum section. Note that for certain types of pile, the pile point level does not always correspond to the physical bottom of the pile, as illustrated in figure 1 (p. 10).

The pile base diameter D_b is equal to the maximum outer diameter of the pile base.

In the case of posts with a widened base plate, the base plate must be sufficiently rigid to withstand the forces to which it is subjected, both during the construction of the post and during subsequent use.

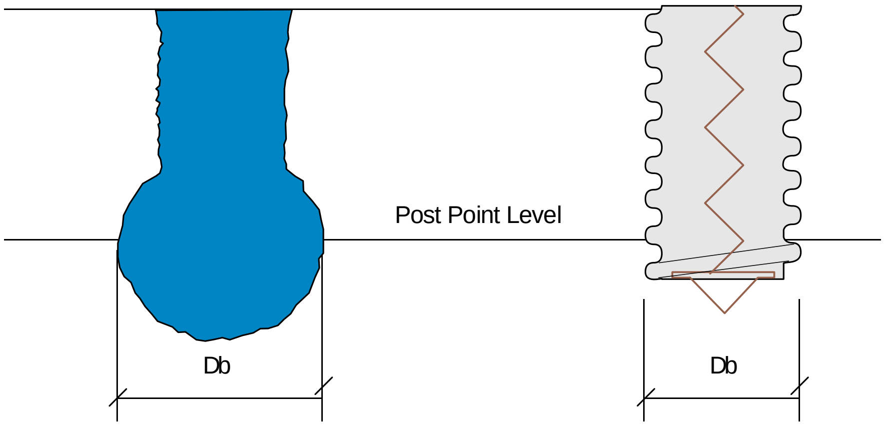
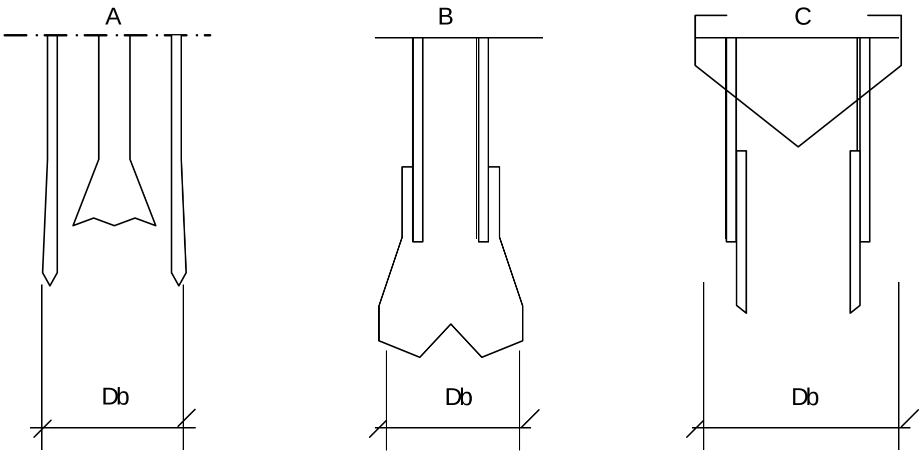


Fig. 1 Example of the definition of the pile point level and the pile base diameter D_b.



A. The pile base diameter D_b of a micropile, drilled with double rods, is equal to the maximum diameter of the bottom of the casing.

B. The pile base diameter D_b of a micropile, drilled with self-drilling hollow rods, is equal to the maximum diameter of the lost drill head.

C. The pile base diameter D_b of a micropile, drilled with a temporary casing (single rods) with a lost tip, is equal to the maximum diameter of the lost drill tip.

Fig. 2 Definition of the pile base diameter D_b for different types of drilled micropiles.

For drilled micropiles, the pile base diameter D_b is equal to the maximum diameter of the drilling equipment used at the level of the outflow of the grout. In the case of a bore with double rods or with single rods without a lost point, this concerns the maximum diameter at the level of the bottom of the lower casing element. For micropiles, equipped with single rods with a lost tip or with self-drilling hollow rods or tubes, this refers to the diameter of the lost drill tip or the drill head (see Figure 2).

If there is a Technical Approval (ATG) with certification or equivalent for the pile type in question, the specific pile point level for the pile type in question can be found here.

2.2.2 PILE BASE AREA

The pile base surface A_b can v For the different base pile shapes, the following are determined:

- for a circular section: $A_b = \frac{\pi . D2}{4}$
- for a square or rectangular section: A_b = a * b, where a and b respectively the short and the long side of the rectangular section
- for an I-beam or sheet pile: A_b = the steel section
- For an open tubular pile, situation without plugging (1): Ab = the steel section
- for an open tubular pile, situation with plugging (1): $A_b = \frac{\Pi . D2}{4}$.

2.2.3 EQUIVALENTE PAALBASISDIAMETER

The equivalent pile base diameter $D_{b,eq}$ is used to determine the unit point resistance q_b, the factor for the influence of the crack of the soil on the point resistance ε_b and the reduction factor for piles with a widened base λ (see § 4.3.2, p. 24):

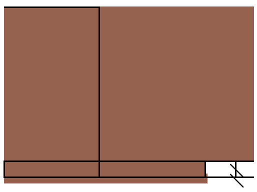
- for a circular section: $D_{b,eq}$ = Db
- for a square or rectangular section:

– $D_{b,eq} = \sqrt{\frac{4.a.b}{\Pi}}$ Indian B ≤ 1.5 A

– $D_{b,eq} = \sqrt{\frac{6.a2}{\Pi}}$ Indian B > 1.5 a,

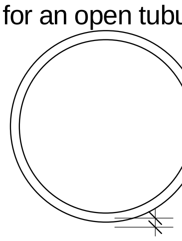
where a and b represent the short and the long sides of the rectangular section, respectively

- for an I-profile or sheet pile: $D_{b,eq} = \sqrt{\frac{6.e2}{\Pi}}$, where e represents the thickness of the flanges



and Represents

- for an open tubular pile, situation without plugging (1): $D_{b,eq} = \sqrt{\frac{6.e2}{\Pi}}$, where e is the thickness of the steel



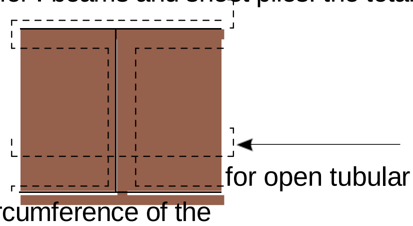
- for an open tubular pile, situation with plugging (1): $D_{b,eq}$ = Db
- for other sections, the equivalent pile base diameter $D_{b,eq}$ should be determined using the previous rules and a good technical-substantive assessment.

(¹) See footnote 6 to Table 5 (p. 27).

2.2.4 PAALSCHACHTOMTREK

The pile shaft circumference $\chi\sigma$ is determined for the different pile types as follows:

- for prefabricated piles: the circumference of the nominal section of the pile shaft
- for piles driven into the ground: the outer circumference of the feed tube
- for I-beams and sheet piles: the total circumference of the steel section



- for open tubular piles, situation without plugging (1): the sum of the inner and outer circumference of the tube
- for open tubular piles, situation with plugging (1): the outer circumference of the pipe
- steel posts closed at the bottom: the outer circumference of the pipe
- For screw piles with a shank in plastic concrete: the maximum outer circumference of the retracted system (tube or displacement drill). The width of the flanges to be taken into account is a maximum of 10 cm (e.g. 36/56)
- For screw piles with a lost feed tube: the outer circumference of the lost feed tube
- for screw piles with a lost feed tube and a grout injection: the circumference, determined on the basis of the average diameter of the feed tube and the pile base
- for screw piles with a temporary feed tube and a grout injection: the circumference, determined on the basis of the average of the maximum diameter of the retracted system (tube or displacement drill) and the pile base diameter
- for auger posts (CFA posts) without feed tube: the maximum outer circumference of the screw blade
- For cased auger posts or bored posts with a feed tube: the maximum outer circumference of the feed tube
- For bored piles without a feed tube: the maximum outer circumference of the drill bit
- For drilled micropiles, the circumference is determined by the maximum circumference Π . Db of the drilling equipment at the level of the outflow of the grout (see also Figure 2, p. 10).

2.3 SYMBOLEN

As far as the symbols used are concerned, we refer to the standards NBN EN 1990 [B3] and NBN EN 1997-1 [B9]. Table 1 (p. 13 to 16) gives an overview of these symbols, as well as a number of additional symbols specific to this Report.

2.4 ABBREVIATIONS

- SLS: Serviceability limit state
- CFA-paal: avegaarpaal (*Continuous Flight Auger Pile*)
- CPT: statische sondering (*Cone Penetration Test*)
- GC: geotechnical category (see § 2.1, p. 9)
- SLT: Static *Load Test*
- ULS: Extreme Limit State

(1) See footnote 6 to Table 5 (p. 27).

Table 1 Overview of the symbols from the NBN EN 1990 [B3] and NBN EN 1997-1 [B9] standards, as well as a number of symbols specific to this Report.

Symbol		Unit	Definition
Roman symbols	Ab	m²	pile base surface
	a	m	short side of a rectangular pole base
	b	m	long side of a rectangular pole base
	Db	m	paalbasisdiameter
	Db,eq	m	equivalente paalbasisdiameter
	Dc	m	diameter of the cone of a static CPT
	Ds	m	Paalschachtdiameter
	and	m	thickness of the flanges of a steel profile or thickness of the steel in the case of an open tubular pile
	F	kn	tax
	Fc	kn	axiale drukbelasting (*axial compression load*)
	Fc,d	kn	Design value of the axial compressive load
	Fc,rep	kn	Representative value of the axial compressive load
	Fd	kn	rekenwaarde van de belasting (*design value of an action*)
	Fk	kn	Characteristic value of the tax
	Fnk	kn	Load due to the negative adhesive
	Fnk,rep	kn	representative value of the tax due to the negative adhesion
	Frep	kn	Representative value of the tax
	Ft	kn	Axial Tensile Load
	Ft,d	kn	Design value of the axial tensile load
	Ft,rep	kn	Representative value of the axial tensile load
	G	kn	Permanent load
	Gd	kn	Approximate value of the permanent load
	Gdst	kn	Permanent destabilizing load
	Gstb	kn	Permanent stabilizing load
	Gstb,d	kn	design value of the permanent stabilizing load
	hi	m	Thickness of layer I
	Who,i	—	Neutral soil pressure coefficient of layer I
	qb	kPa	Unit Point Resistance
	qc	MPa	cone resistance
	qc,corr	MPa	corrected cone resistance, to be used in excavations after the static CPT has been carried out
	qc,mi,i	MPa	Average cone resistance over low I
	qs	kPa	Unit friction resistance
	qs,i	kPa	Unit friction resistance of layer I
	Q	kn	Variable load
	Qd	kn	Approximate value of the variable load
	Qdst	kn	Variable destabilizing load

(continued from the table on p. 14)

Table 1 Overview of the symbols from the NBN EN 1990 [B3] and NBN EN 1997-1 [B9] standards, as well as a number of symbols specific to this Report (continued).

	Symbol	Unit	Definition
Roman symbols	R	Kn	resistance
	R_b	Kn	Point resistance
	$R_{b,cal,i}$	Kn	Calibrated value of the point resistance, calculated from the static CPT I, where i is the identification of the static CPT
	$R_{b,k}$	Kn	Characteristic value of the point resistance
	R_c	Kn	compressive resistance (= load-bearing capacity of piles loaded on pressure) (*compressive resistance*)
	R_{cal}	kn	Calibrated resistance
	$R_{c,cal}$	kn	calibrated load-bearing capacity of compression-loaded piles
	$R_{c,cal,i}$	Kn	Calibrated load-bearing capacity of compression-loaded piles, calculated from the static CPT I, where i is the identification of the static CPT
	$R_{c,d}$	Kn	design value of the load-bearing capacity of piles loaded with pressure
	$R_{c,m}$	kn	measured value of the load-bearing capacity of piles loaded with pressure
	$R_{c,k}$	kn	Characteristic value of the load-bearing capacity of piles loaded under pressure
	R_d	kn	rekenwaarde van de weerstand (*design value of the pile resistance*)
	R_f	%	friction number
	R_e	kn	Resistance (load-bearing capacity), calculated on the basis of the static CPT i, where i is the identification of the static CPT
	R_k	kn	Characteristic value of the resistor
	R_m	kn	Measured value of the resistance
	R_s	kn	Frictional resistance of piles loaded under pressure
	$R_{s,cal,i}$	kn	Calibrated value of the frictional resistance of compression-loaded piles, calculated from the static CPT I, where i is the identification of the static CPT
	$R_{s,k}$	kn	Characteristic value of the frictional resistance of piles loaded under pressure
	R_t	kn	tensile resistance (= load-bearing capacity of tension-loaded piles)
	$R_{t,cal}$	kn	calibrated load-bearing capacity of tension-loaded piles
	$R_{t,d}$	kn	Design value of the load-bearing capacity of posts loaded on tension
	$R_{t,k}$	kn	characteristic value of the load-bearing capacity of piles loaded on tension
	in the	kPa	prevailing groundwater pressure
	$V_{dst,d}$	Kn	Design value of vertical destabilizing loads
	V_g	m³	Volume of the root ball
	S_w	m	water level
	$S_{W,M}$	m	Measured water level
	B&B,M,max	m	highest measured water level

(continued from the table on page 15)

Table 1 Overview of the symbols from the NBN EN 1990 [B3] and NBN EN 1997-1 [B9] standards, as well as a number of symbols specific to this Report (continued).

	Symbol	Unit	Definition
Greek symbols	α	°	angle that determines the shape of the root ball
	$A\beta$	—	installation factor for the point resistance
	τo	—	Magnification factor for the diameter of a pile
	$\alpha\sigma$	—	Installation factor for the frictional resistance to pressure
	$\alpha_{s,i}$	—	Installation factor for the frictional resistance to pressure of layer I
	$\alpha\tau$	—	Installation factor for the frictional resistance on tensile
	B	—	form factor for a non-circular or non-square pile base
	γ'	kN/m³	Effective volumetric weight of the soil
	γ'_g	kN/m³	Effective volumetric weight of the soil of the root ball
	$\gamma\beta$	—	partial factor for the point resistance, which takes into account the probability of unfavorable deviations from the actual point resistance from the characteristic value
	$\gamma\Phi$	—	factor for the loads, which takes into account the probability of deviations from the actual load in relation to the representative value. The factor takes into account not only uncertainties on the model used to determine the load, but also geometric variations
	γ_{Gdst}	—	partial factor for the permanent destabilising burden
	γ_{Gstb}	—	Partial factor for the permanent stabilising load
	γ_{Qdst}	—	Partial factor for the variable destabilizing load
	$\gamma P\delta$	—	modelfactor
	$\gamma\sigma$	—	Partial factor for the frictional resistance to pressure, which takes into account the probability of unfavorable deviations from the actual frictional resistance to the characteristic value
	γ_{st}	—	Partial factor for the frictional resistance on tensile, which takes into account the probability of unfavorable deviations from the actual frictional resistance from the characteristic value
	$\gamma\omega$	kN/m³	Volumical weight of water
	Δ_i	°	Friction angle in the contact patch between the pile shaft and layer I
	$\varepsilon\beta$	—	Factor for the influence of the cracking of the soil on the point resistance
	η^*_p	—	Factor that gives the soil type-dependent ratio between the cone resistance and the unit friction resistance (independent of the pile type)
	)'	°	Effective angle of internal friction
	)'i	°	Effective angle of internal friction of layer I
	κ	—	Factor that makes it possible to verify whether more favourable installation factors (AB and/or AΣ) than those in table 5 (p. 27) may be applied for a pile system
	λ	—	Reduction factor for piles with a widened base
	μ	—	factor that can be used to verify whether more favourable model factors ($\gamma P\delta 2$ or $\gamma P\delta 3$) may be applied for a pile system

(continued from the table on page 16)

Greek symbols

Table 1 Overview of the symbols from the NBN EN 1990 [B3] and NBN EN 1997-1 [B9] standards, as well as a number of symbols specific to this Report (continued).

Symbol	Unit	Definition
ν	–	factor that makes it possible to verify whether more favourable model factors (γPδ2 or γPδ3) and/or more favourable installation factors (αβ and/or ασ) than those in table 5 (p. 27) may be used for a pile system
Iν S'$_v$	kPa	Effective vertical tension
σ'$_{v,i}$	kPa	Effective vertical tension in layer I
ω	—	conversion factor to be taken into account in the case of static CPTs with a mechanical cone
ξ	—	correlation factor that takes into account, among other things, the variation of the soil characteristics across the terrain
Xσ	m	Paalschachtomtrek

3 SOIL INVESTIGATION

3.1 GENERAL

For more information on the method of carrying out the static CPTs, the reporting of the results, the guidelines on the minimum depth and the number of static CPTs, please refer to the following documents:

- Standard procedures for geotechnical surveys: CPTs. Part 1: planning, implementation and reporting Porting [B1]
- Eurocode 7: geotechnical design. Part 2: soil investigation and testing [B12] and the Belgian National Investigation Annex [B13].

When the pile is set in resistive soil layers, including less resistant layers that could affect the bearing capacity of the pile, the static CPTs must in any case extend to these underlying soil layers.

3.2 CONE TYPE

For the purpose of the soil mechanical design based on the results of static CPTs, the statistical CPT with an electric cone is considered as the yardstick.

Static CPTs with a mechanical cone (M1, M2 or M4) are permitted, provided that the measured cone resistance is divided by a conversion factor ω, as indicated in Table 2. These factors were determined by means of comparative tests in different fields. However, if there are comparative tests in the field itself, a conversion factor specific to the site in question can be derived. The conversion factors ω are applied to the cone resistors qc. On the basis of these reduced cone resistors, the unit point resistance q_b (see § 4.3.2, p. 24) and the unit friction resistance of layer i $q_{s,i}$ (see § 4.3.3, p. 25) are calculated.

Table 2 Conversion factors to be applied ω for the reduction of the measured cone resistances in static CPTs with a mechanical cone.

Conustype	Soil type	
	Tertiary klei	Other soil types
M1	1,30	1,00
M2	1,30	1,00
M4	1,15	1,00

3.3 INFLUENCE OF EXCAVATIONS

If soil is still excavated after the static CPT has been carried out, this can have a significant impact on the cone resistance. This could deviate from the cone resistance that would be obtained if the static CPT was carried out after the excavation. That is why it is advisable to carry out the static CPTs, where possible, after the excavation.

If it is not possible to perform the CPT after excavation, the cone values measured before excavation should be reduced as follows:

- If one or more of the following conditions are met, no reduction in the factor are applied:
  - – in tertiaire klei
  - – for piles with displacement of soil displacement of category I from table 5 (p. 27)
  - – for piles of categories II, III or IV from Table 5 (p. 27), which were installed before the Caught
  - – in the excavation of a trench the width of which is not more than 50% of the depth
  - – if a thorough analysis shows that there is no significant impact on the
- In all other cases, the cone resistance must be reduced, as shown in the illustration. thing 3.

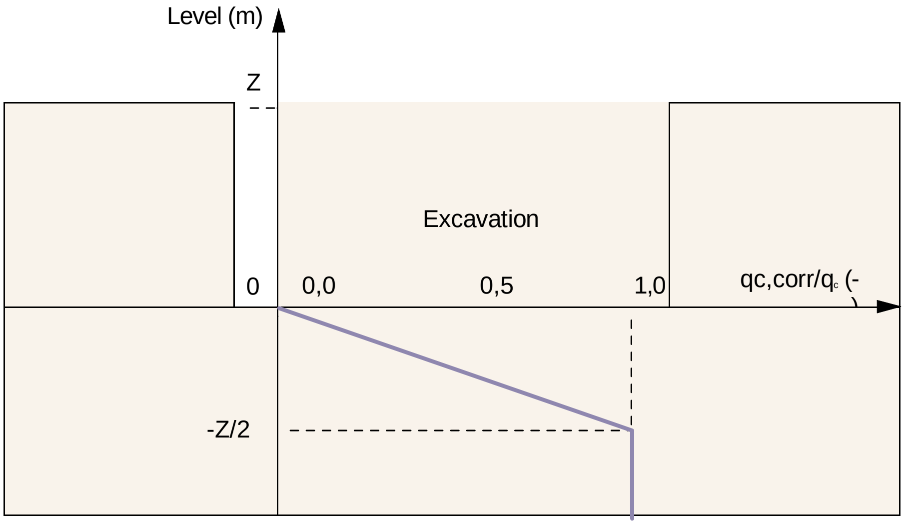


Fig. 3 Reduction of cone resistance q_c.

When calculating the unit point resistance using the De Beer method (see § 4.3.2, p. 24), both the cone resistance and the vertical terrain tension, originating from the dead weight of the soil, must in all cases be equated to zero in the zone above the excavation level.

4 AXIALLY LOADED ON PRESSURE ALEN

4.1 GENERAL

In order to demonstrate that a pile foundation is able to absorb the foreseen loads with sufficient safetyin relation to soil failure, the inequality [1] must be met (see § 7.6.2.1 (1) of NBN EN 1997-1 [B9]). This is a GEO check (collapse or very large deteriorationofthe subsoil, in which the strength of the rockor rock makes a significant contribution to the resistance) according to Eurocode 7 (see § 2.4.7.1 (1) of NBN EN 1997-1 [B9]):

$$F_{c,d} \leq R_{c,d} \qquad [1]$$

whereby:

- Fc,d = the design value of the axial compressive load on the pile [kN]
- Rc,d = the design value of the load-bearing capacity of the compression-loaded pile [kN].

The **design value of the axial compressive load** Fc,d is determined by multiplying the representative value of the load $F_{c,rep}$ by a partial factor for the loads γΦ (see § 4.2.1).

The **design value of the load-bearing capacity of the compression-** $_d$ is determined by the characteristic value of the bearing capacity $R_{c,k}$ by a resistance factor the frictional γβ (for the dot resistance) or ΓΣ (for resistance) (see § 4.3.6, p. 30).

For piles loaded with axially loads, the soil bearing capacity and structural control shall only be verified in accordance with design approach 1 (DA1/1) (2).

4.2 DESIGN VALUE OF THE TAX

4.2.1 GENERAL

As stated in § 4.1, the design value of the axial compressive load $F_{c,d}$ determined by the value of the tax $F_{c,rep}$ by a partial factor for the loads representative γ_F :

$$F_{c,d} = F_{c,rep} \cdot \gamma_F \qquad [2]$$

The values to be taken into account for the load factors γΦ are laid down in the standard NBN EN 1990 ANB [B4]. The values for the permanent and variable design states are given in Table 3 (p. 20). For the accidental design states, all load factors are set to 1.00.

The dead weight of the pole will not be charged, unless this is explicitly stated.

4.2.2 NEGATIVE ADHESIVE

4.2.2.1 General

When the soil around the pile shows a downward displacement relative to the pile shaft,

(2) In Belgium, Design Approach 1 (DA1) from Eurocode 7-1 (see § 2.4.7.3.4.2 of NBN EN 1997-1 [B9]) applies. This means that for the inequality [1], two combinations of partial factors (DA1/1 and DA1/2) should be examined. In view of the choice of the values of the load and resistance factors in both combinations, DA1/1 is always decisive for axially loaded piles.

Table 3 Load factors $\gamma\Phi$ for the permanent and variable design states (DA1/1).

Tax		$\gamma\Phi$ (DA1/1)
Lasting	Zero (1)	1,35
	Favorable (2)	1,00
Variable	Zero (1)	1,50 (3)
	Favorable (2)	0,00

(1) Destabilizing.
(2) Stabilizing.
(3) A different value applies to bridges according to Annex A2 of the NBN EN 1990 [B3] standard: for road traffic, the factor $\gamma\Phi$ is equal to 1.35 and for rail traffic to 1.45.

Then the ground will exert a downward friction on the pole. We call this phenomenon the negative stick.

The settlement of the soil in relation to the pile can be caused by various factors: upper load, drainage, a supplement to soil that is not fully consolidated, etc.

Since the pile subsidence is 10 % of the pile base diameter in the ultimate limit state (ULS) and the negative adhesive only occurs in the case of subsidence of the soil that is larger than this pile subsidence, in some cases there will be no negative adhesive in the ultimate limit state (ULS), but in the serviceability limit state (SLS).

4.2.2.2 Calculation of the settlement of the soil around the pile

The settlement (and therefore also the ground level) is determined on the basis of the characteristic values of the soil (compression constants C, relief or reload constants A, volumetric weight, etc.) and the representative values of the upper load.

4.2.2.3 Calculation of the negative adhesive as an additional load on the pile

It goes without saying that no positive friction may be taken into account for soil layers for which the negative adhesive is taken into account.

The negative adhesive must be considered as an additional load on the pile if no ground level subsidence is calculated or if the ground level subsidence after the pile installation is more than 10 cm.

In the case of ground level subsidence of up to 2 cm after the pile installation, the negative adhesive has hardly any effect and therefore does not have to be taken into account.

In the case of a ground level subsidence of 2 to 10 cm after the pile installation, the negative adhesive to be taken into account is determined by linear interpolation between the maximum value of the negative adhesive (100%) for a ground level subsidence of 10 cm and no negative adhesive (0%) for a ground level subsidence of 2 cm.

In the case of a ground level subsidence of less than 2 cm after the pile installation, it is the designer himself who determines whether or not a positive friction should be taken into account.

The zone subject to the negative adhesive is determined by the relative motion of the pile relative to the ground, as shown in Figure 4 (p. 21).

In the serviceability limit state (SLS), the area over which the negative adhesion occurs will usually extend to the bottom of the compressible layer when approached.

If a separate SLS verification is carried out to assess the effect of the negative adhesive on the settlement behaviour of the pile foundation, the effect of the negative adhesive should not be considered in the ULS verification of the bearing capacity of the pile. However, no positive friction above the neutral point may be included in this verification.

If no separate SLS verification is carried out to assess the effect of the negative adhesive on the settlement behaviour of the pile foundation, the negative adhesive must be considered in the ULS verification of the bearing capacity of the pile. In this case, one should also always assume that the negative adhesive occurs up to the bottom of the compressible layer.

The negative adhesive can be calculated using the slip method.

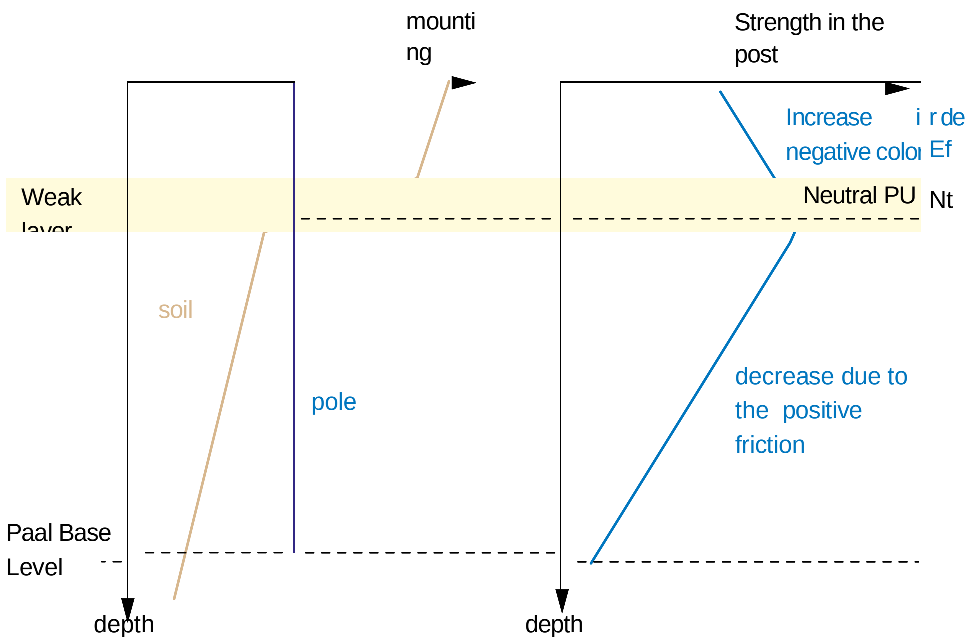


Fig. 4 Zone subject to the negative adhesion.

De slipmethode

The calculation of the negative adhesive based on the slip method is done using the following formula:

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

where it is summed up about all layers in which negative adhesion occurs and where:

- $F_{nk;rep}$ = the representative value of the load by the negative adhesive [kN]
- $\chi\sigma$ = the pile shaft circumference, as defined in § 2.2.4 (p. 12) [m]
- hi = the thickness of layer i [m]
- $K_{o,i}$ = the neutral soil pressure coefficient in layer i, $K_{o,i} = 1 - \sin \varphi'_i$
- φ'_i = the effective angle of internal friction of layer i [°]
- Δ_i = the angle of friction in the contact patch between the pile shaft and layer i. $\Delta_i = \varphi'_i$ for the ground with steel piles en prefabricated earth betonpalen en Shaped betonpalen [°], $\delta_i = 0{,}75 \,.\, \varphi'_i$ for schight [°]. However, $K_{o,i} \,.\, \tan \delta_i$ is at least equal to 0.25
- $\sigma'_{v,i}$ = the average effective vertical stress in layer i [kPa].

For all parameters, the characteristic values are entered.

If the negative adhesive can occur with piles placed close to each other, the effect of the negative adhesive can also be calculated using the Zeevaert — De Beer method [D1, D3].

4.2.2.4 Design value of the negative adhesive

The design value of the negative adhesive is determined by multiplying the representative value of the load by multiplying the negative adhesive $F_{nk,rep}$ by a partial factor equal to 1.0 for both ULS and SLS verification.

Remark
For the verification of the structural resistance (STR) of the pile foundation in the ULS, the design value of the negative adhesive is obtained by multiplying F $_{nk,rep}$ by a partial factor equal to 1.35.

The negative adhesive should not be combined with temporary loads. Temporary loads are understood to mean loads that are relevant for a period of time that is much shorter than the life of the structure (e.g. only during construction or repair). When temporary loads are applied to the pile, the most disadvantageous of the combinations listed below should be taken into account:
- permanent loads + long-term variable loads + negative adhesive
- permanent loads + long-term variable loads + temporary loads.

4.2.3 ALTERNATING LOAD

If the pile is subjected to alternating loads (i.e. also subjected to tensile loads in the serviceability limit state), the influence of this on the frictional resistance must be checked. Several factors play a role in this (e.g. number of cycles, amplitude, etc.). In the absence of data (tests, literature, etc.) on the influence of the alternating load on the frictional resistance, a reduction must be applied, as indicated in § 4.3.3 (p. 25).

4.3 DESIGN VALUE OF THE LOAD-BEARING CAPACITY ON PRESSURE

4.3.1 OVERVIEW

The determination of the design value of the bearing capacity of a pile foundation $R_{c,d}$ is schematically shown in Figure 5 (p. 23). The following four steps must be followed:
- **STEP 1**: the calculation of the load-bearing capacity (point and friction resistance) on the basis of each individual
  Static CPT (CPT), in which **installation factors** are applied to take account of the differences between the types of piles (influence of the degree of soil displacement or possible soil relaxation at the base and along the shaft, influence of the friction between the material of the shaft and the soil). The aim here is to estimate the actual load-bearing capacity as accurately as possible if the pile were to be constructed in the axis of the static CPT
- **STEP 2**: the calibration of the calculated carrying capacity by introducing a **model factor**. This model factor takes into account systematic deviations between the calculated values and the actual values, as well as the uncertainty on the results (see § 2.4.1 (9) of the NBN EN 1997-1 [B9]). The model factor is determined, where possible, on the basis of analyses of static pile load tests (SLTs) in such a way that the calculated load-bearing capacity does not exceed the actual load-bearing capacity in about 95% of cases. The conventional bearing capacity is determined as the mobilized bearing capacity at a pile base setting equal to 10 % of the pile base diameter. In the absence of sufficient useful information, a safe estimate must be made of the carrying capacity. The model factors are determined for each group of pile types (driven piles, pressed piles, screwed piles, CFA piles, drilled piles and micropiles), the value depending on the average value and on the spread on the ratio between the calculated and measured bearing capacities
- **STEP 3**: on the basis of steps 1 and 2, the load-bearing capacity of a pile that is carried out in the axis of a static CPT is determined. In order to take into account the variation of the soil characteristics over the site and the degree of uncertainty about this (the latter depends, among other things, on the extent of the soil investigation), a reduction is applied by the

introduction of correlation factors. The correlation factors are applied to both the minimum value and the average value of the calculated carrying capacity. The smaller of the two values thus reduced is decisive. In addition, it is also taken into account that the forces that act on a pile with an insufficient load-bearing capacity can be transferred to the adjacent piles, insofar as the structure shows sufficient rigidity. In this case, the reduction to be applied will be smaller

- **STEP 4**: from step 3, the characteristic value of the bearing capacity of the pile foundation can then be derived. If implemented correctly, this value will not be exceeded (theoretically) in 95% of cases. In order to limit the probability of failure to generally accepted values, **safety factors are** applied **to the point resistance and to the frictional resistance**. The values of these factors depend on the extent to which the implementation is monitored.

The concrete details of each step are described in more detail below.

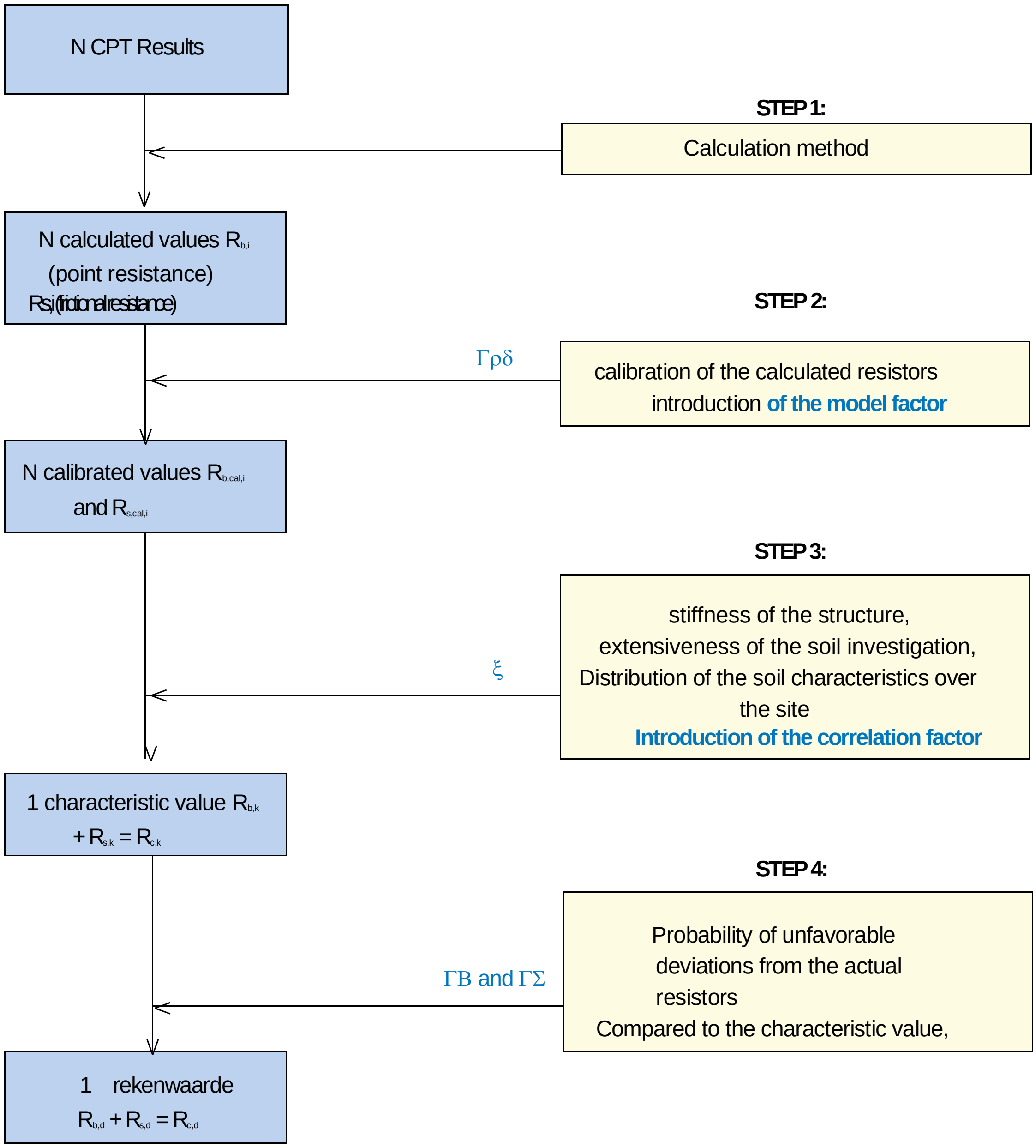


Fig. 5 Flowchart illustrating the various steps in the calculation of the design value of the load-bearing capacity of a pile foundation.

4.3.2 POINT RESISTANCE

The point resistance R_b is determined using the following formula:

$$R_b = \alpha_\beta \, . \, \varepsilon_\beta \, . \, \beta \, . \, \lambda \, . \, A_b \, . \, q_b \qquad [3]$$

whereby:

- q_b = the unit point resistance calculated using the De Beer [D2] [kPa] method.
  If the calculation is based on the results of a static CPT with a mechanical cone, then a conversion factor ψ must be applied to the cone resistance q_c beforehand, as described in § 3.2 (p. 17).
  The diameter to be entered in the calculation of the unit point resistance q_b is the equivalent pile base diameter $D_{b,eq}$ as defined in § 2.2.3 (p. 11).
  If the static CPT was carried out with an electric cone, an average of the measured cone resistances over the zone ranging from 10 cm above to 10 cm below the level at which the load capacity is calculated, must be made.
  If the value of the equivalent pile base diameter $D_{b,eq}$ is not a multiple of 20 cm, then two calculations must be performed with the nearest multiples of 20 cm, between which linear interpolation is then performed. Some calculation programs do this automatically.
  If the equivalent pile base diameter $D_{b,eq}$ is less than 20 cm, the calculation must be carried out with a diameter equal to 20 cm.
  On the website of the BGGG (www.bggg-gbms.be) you will find a description of the De Beer method as well as a number ofcalculation examples
- α_β = an empirical installation factor that takes into account the influence of the installation method of the pile in a certain type of soil. For piles of categories **I**, **II** and **III**, the values of this installation factor are given in table 5 (p. 27) depending on the type of pile. For piles of categories IVa, IVb and IVc, the values of α_β are included in table 6 (p. 28)
- ε_β = a parameter that takes into account the scale effect on the shear resistance due to the cracking of the soil:

$$\varepsilon_b = \max\left[1 - 0{,}01 \cdot \left(\frac{D_{b,eq}}{D_c} - 1\right); 0{,}476\right] \quad \text{in tertiaire klei}$$

– ε_β = 1 in l other soil types.

$D_{b,eq}$ represents the equivalent pile base diameter, while D_c represents the cone diameter of the static CPT (for a standard cone D_c = 0.0357 m)

- β = a form factor that takes into account the influence of a non-circular or non-square pile base

  Brings:
  - $\beta = \frac{1 + 0{,}3 \cdot a/b}{1{,}3}$ for a rectangular pole base, where a and b respectively have the dimensions of the short and long side of the post base
  - β = 0.77 for walls
  - β = 1 for circular and square pile bases
- Ab = the pile base area, as defined in § 2.2.2 (p. 11) [m²]
- λ = a reduction factor for piles with a widened base, where the widened pile base provides a relaxation of the ground around the pile shaft during the installation of the pile. The value of λ is determined as follows:
  - for piles whose widened base is formed in depth and therefore does not result in a relaxation of the soil along the shaft: λ = 1.00
  - for posts with a prefabricated widened base, where $D_{b,eq} < D_s + 0.05$ m: λ = 1,00 ($D_{b,eq}$ represents the equivalent pile base diameter, while D_s represents the pile shaft diameter)
  - for micropiles: λ = 1
  - For all other piles with a prefabricated widened base, the reduction can be derived from the from figure 6 (p. 25).

If there is a Technical Approval (ATG) with certification or equivalent for the type of pile in question, you can find the applicable values for $D_{b,eq}$, Ds, αβ, ®, Ab and λ . These values take precedence over the values specified in this Report.

Fig. 6 Reduction factor for piles with a widened pile base, where the widened base provides relaxation of the ground during installation.

4.3.3 FRICTIONAL RESISTANCE

The frictional resistance R_s is determined using the following formula:

$$R_s = \chi \sigma . \sum (\alpha_{s,i} . \alpha_{D,i} . h_i . q_{s,i}) \quad [4]$$

whereby
:

- $q_{s,i}$ = the unit friction resistance [kPa]
  $q_{s,i} = 1000 . \tau\eta\varepsilon^*_{p,i} . q_{c,m,i}$
- $\eta^*_{p,i}$ = an empirical factor indicating the ratio of the unit friction resistance of laa g i $q_{s,i}$ and the cone Resistance q_c according to the type of soil. These values are specified in Table 4 (p. 26)
- $q_{c,m,i}$ = the average cone resistor q_c over layer i [MPa].
  If the calculation is based on the results of a static CPT with a mechanical cone, a conversion factor must be applied to the cone in advance.
  resistance q_c, as described in § 3.2 (p. 17).
  Only the layers that are relevant to the friction between the pile and the ground are taken into account.
  If there are layers with a local cone resistance q_c < 1 MPa, then it is necessary to assess for each individual situation whether or not the frictional resistance of the overlying layers is should not be taken into account.
  If possible, try to assess the type of soil derived from the CPT results.
  local geology data (e.g. from the results of local boreholes, soilmechanical or geologicalmaps or the Database website).
  Underground Flanders (http://dov.vlaanderen.be)). In static probes with a mechanical cone, mand usually does not have the valueof the friction number R_f and one is still more dependent on knowledge of local geology
  =
- χ_s = the pile shaft circumference as defined in § 2.2.4 (p. 12) [m]
- $\tau o_{s,i}$ an empirical installation factor for layer i, which, in addition to the influence of the installation mode of the
  in a certain type of soil also takes into account the roughness of the pile shaft. These values are given in Table 5 for posts of categories I, II and III (p. 27) and in Table 6 for micropiles of category IV (p. 28).
  If the pile is subjected to an alternating load (i.e. also subjected to tensile loads in the serviceability limit state), then the influence of the alternating load on the frictional resistance R_s be checked. Several factors play a role in this (e.g. number of cycles, amplitude, etc.). In the absence of data (tests, literature, etc.) on the influence of alternating load on frictional resistance, the value of the $\alpha_{s,i}$ reduced by a factor of 1.33
- $\alpha_{D,i}$ = an increase factor for the pile shaft diameter for layer I, which takes into account the effect of the

y grout injections at micropiles (category IV). For poles of the category

Rows I, II, III, IVa and IVb, $\alpha_{D,i}$ is always equal to 1. For micropiles of category IVc, $\alpha_{D,i}$ values of more than 1 may be used. The value of $\alpha_{D,i}$ depends on various factors (type of injection material, type of cuffs, post-injection parameters and post-injection method, drill diameter, soil type, etc.) and cannot be determined unambiguously. However, for micropiles of category IVc, where the post-injection was carried out in a selective manner and in several phases using sleeve tubes and a double *packer*, reference can be made to the indicative values from the informative Appendix D (p. 45).

- hi = the thickness of layer I [m].

If there is a Technical Approval (ATG) with certification or equivalent for the pile type in question, it can be found the applicable values for the pile shaft diameter D_s, the pile shaft circumference χσ, the magnification factor $\alpha_{D,i}$ and the installation factor for the frictional resistance on pressure σηαφτ . These values take precedence over the values in this Report.

Important note
The empirical factors from Tables 5 (p. 27) and 6 (p. 28) were determined on the basis of the results of a limited number of instrumented static pile load tests (SLTs). For some pile types, there are no or insufficient test results available and these factors must be determined on the basis of their similarities and differences with other pile types and their estimated influence on the load-bearing capacity. In certain circumstances (specific soil type or specific pile type) or for important structures, at least two instrumented static pile loading tests can always be carried out on site to determine the value of these factors. The SLTs must, where appropriate, meet the requirements of Appendix B (p. 41).

Table 4 Values for the unit friction resistance (q_s) and the factor (η^*_p) according to the soil type.

Soil type	qc [MPa]	η^*_p of qs [kPa]	Rf[%](1)
Clay	1 - 4,5	η^*_p = 1/30	3 - 6 %
	> 4,5	qs = 150	
Loam	1 - 6	η^*_p = 1/60	2 - 3 %
	>6	qs = 100	
Sand-containing clay/sand-bearing	1 - 10	η^*_p = 1/80	1 - 2 %
Clayey sands/clayey loam	> 10	qs = 125	
Sand	1 - 10	η^*_p = 1/90	< 1%
	10 - 20	QS = 110 + 4 (QC - 10)	
	> 20	q_s = 150	

(1) The friction number (R_f) is determined by means of a static CPT with an electric cone. R_f is guiding for the determination of the type of soil. However, additional data (e.g. sampling boreholes, recognition tests, etc.) may also be used to determine the type of soil.

Table 5 (1) Values of the installation factors for the point resistance αβ and for the frictional resistance to the pressure σηαφτ as a function of the pile type (categories I, II and III).

Pole type	Base φροµ		Shaft ασ (2)	
	Clay	Other types of soil (10)	Clay	Other types of soil (10)
CATEGORY I (3): PILES WITH SOIL DISPLACEMENT				
DRIVEN AND PRESSED PILES				
Prefabricated concrete post without widened base	1	1	0,9	1
Ground shaped post without widened base (4), plastic concrete shaft	1	1	0,9	1
Ground shaped post with widened base (4), plastic concrete shaft	1	1	—(5)	—(5)
Ground-formed post with ground-formed widened base, dry-concrete shaft	1	1	1,15	1,15
Steel post closed at the bottom, without widened base (4)	1	1	0,6	0,6
Steel post closed at the bottom, with widened base (4)	1	1	—(5)	—(5)
Open steel tubular pile, situation with clumping (6)	1	1	0,6	0,6
SCREW PILES OF CATEGORY I (7)				
Shaft in plastic concrete	0,8	0,5	0,6	0,6
With a lost feeding tube	0,8	0,5	0,6	0,6
With a lost or temporary feed tube and grout injection (8)	0,8	0,5	0,6	0,6
CATEGORY II (3): PILES WITH LITTLE SOIL DISPLACEMENT OR RELAXATION				
GEHEIDE PALEN				
Open steel tubular pile, situation without plugging (6)	1	1	0,6	0,6
I-beams and sheet piles	1	1	0,6	0,6
CATEGORY III (3): POSTS WITH SOIL REMOVAL				
CFA PILES				
	0,8	0,5	0,3	0,4
BORE PILES				
Equipped with a temporary feed tube	0,8	0,5	0,3	0,5
Executed under support fluid	0,8	0,5	0,5	0,5
Equipped without temporary feeding tube or support fluid	0,8	—(9)	0,5	—(9)

(1) For types of piles that have a Technical Approval (ATG) with certification or equivalent, there may be a conditions other AB and Aσ values if they apply, as stated in Table 5. The procedure fordrawing up a Technical Approval (ATG) with certification can be requested from the UBAtc (www.butgb.be, info@butgb.be).
(2) Reduction under alternating load: see above.
(3) Categories I, II and III referred to here refer to the degree of soil displacement or removal during the pole installation. Not to be confused with the geotechnical categories (GCs) 1, 2 and 3, which are mentioned in § 2.1 (p. 9).
(4) The pile base is considered to be widened with respect to friction (lower σηαφτ values) if it is a prefabricated Widened base goes with $D_{b,eq}$ > Ds+5cm. The influence of a widened base on the point resistance is contained in the reduction factor λ. (5) Or the friction is demonstrated by at least two instrumented static pile load tests on site (see Appendix B, p. 41), or no friction is charged.
(6) For steel tubular piles that are not closed at the bottom, a plug may be formed at the bottom of the pipe when the insertion of the pole. For the design, two situations must be examined:
– situation without clumping at the bottom: the friction is applied both on the outside and on the inside of the pipe Charged. Point resistance is considered across the steel section of the point
– situation with clot formation at the bottom: the friction is only taken into account on the outside of the pipe. The Point resistance is considered over the entire section of the point.
It is the minimum value of both situations that determines the design.
(7) Only for screw piles with flanges of up to 10 cm (e.g. 36/56).
(8) The diameter of the central pipe is at least equal to half the pile base diameter.
(9) Point and friction resistance shall be demonstrated by instrumented static pile load tests (SLTs) Spot.
(10) By other types of soil we mean common types of soil such as loam, sand, sandy clay or loam, clayey sand or clayey loam.

Table 6 (1) (2) Values of the installation factors for the frictional resistance to pressure σηαφτ and the pile base resistance αβ as a function of the type of pile and the type of soil for micropiles (categories Iva, IVb and IVc).

Soil type	CAT. IVa (3)		CAT. IVb (4) (6)		CAT. IVc (5) (6) (7)	
	ασ (9)	οφφ	ασ	οφφ	ασ	οφφ
Clay	1,0	0,5	1,0	0,5	1,5	0,5
Loam	1,5	0,5	2,0	0,5	2,5	0,5
Sandy clay/ sandy loam Clayey sand/ clayey loam	1,5	0,5	2,5 (8)	0,5	2,5 (8)	0,5
Sand	1,5	0,5	2,5 (8)	0,5	2,5 (8)	0,5

(1) For micropile types that have a Technical Approval (ATG) with certification or equivalent, other αβ and ασ values such as those set out in Table 6 may apply under certain conditions. The procedure to a Technical Approval (ATG) with certification can be requested from the UBAtc (www.butgb.be, info@butgb.be).

(2) The values of the factors in this table are valid for micropiles with a diameter D_b < 180 mm.

(3) Category IVa: micropiles in which the borehole around the reinforcement member is gravitationally filled with cement grout. Unless otherwise indicated, this category includes micropiles that are carried out with a temporary casing (single rods) that has a lost point or a widening at the bottom, as well as micropiles that are constructed with self-drilling hollow reinforcement rods or tubes.

(4) Category IVb: micropiles in which the borehole around the reinforcement element is injected under a certain global injection pressure higher than the gravitational pressure (typically 2 to 12 bar). This category includes micropiles that are carried out with double rods (tubular flush bore), whereby the *casing* is gradually retracted and where the *casing* grout is injected under a global pressure after each stepwise retraction. Micropiles that are constructed with a temporary casing that has a lost tip or a widening at the bottom, and where the necessary measures are taken to restore or improve the relaxation of the site (e.g. by means of a secondary grout injection with injection hoses equipped with one or more cuffs), also belong to this category, unless otherwise indicated.

(5) Category IVc: micropiles in which the grout is injected in a selective manner (d.w.z. dat one knows/chooses where to inject) and in several phases (over time) via sleeve tubes (TAMs) and with a double *packer*. In contrast to the In the previous categories, a significant diameter increase can usually be achieved with this type of micropiles compared to the dimensions of the drilling equipment.

(6) It should be borne in mind that the beneficial effect of a pressurised grout injection at depths < 4 m below the level of the site is rather limited. To that extent, the factors of category IVa should therefore be applied for all micropile types.

(7) For micropiles of category IVc, αΔι values > 1 may be used. The αΔι value depends on different factors (type of injection material, type of cuffs, post-injection parameters and post-injection method, drill diameter, soil type, etc.) and cannot be determined unambiguously. However, for micropiles of category IVc where the post-injection was carried out in a selective manner and in several phases using sleeve tubes and a double *packer*, reference can be made to the indicative values from the informative Appendix D (p. 45).

(8) In view of the fact that the number of experience data to date is too limited, the value of αση at q_c values should be < 8 MPa limit to the values of category IVa for all types of micropiles. (9) Reduction under alternating load: see above.

4.3.4 TOTAL CARRYING CAPACITY

The load-bearing capacity of compression-loaded piles R_c is equal to the sum of the point resistance R_b and the frictional resistance R_s:

$$Rc = R_b + Rs. \qquad [5]$$

In order to ensure that the calculated carrying capacity would be sufficiently safe (see § 4.3.1, p. 22), a gRd model factor is applied to the calculationof the $_{scaled}$ carrying capacity Rχ,χαλ (see § 2.4.1 (6), § 2.4.1 (8), § 2.4.7.1 (6) and § 7.6.2.3 (2) of NBN EN 1997-1 [B9]):

$$Rc;cal = R_c \,/\, \Gamma\rho\delta, \qquad [6]$$

whereby:

- Rc;cal = the calibrated load-bearing capacity of piles loaded under pressure [kN]
- γPδ = the model factor (γPδ1, γPδ2 or γPδ3, see Table 7, p. 29, and Appendix A, p. 39).

For each group of pile types, a model factor is determined.

For pile types for which no results of instrumented pile load tests (SLTs) are available, the model factor γPδ1 from Table 7 should be applied in formula [6].

Table 7 Values for the gRd model factor as a function of the pile type.

Group of pile types	Without SLT: γPδ1	With SLT: γPδ2 (2)	With SLT in the field: γPδ3 (1)
Piles driven and pressed in	1,00	N/a.	1,00
Screw piles	1,30	– (2)	1,00
CFA piles	1,35	– (2)	1,10
Bored piles	1,20	N/a.	1,10
Micropiles	1,55	– (2)	1,15

(1) The conditions for the application of a reduced model factor γPδ3 are set out in Appendix A (p. 39).
(2) Where applicable, the values and conditions for the application of γPδ2 σηαλλ be set out in a Technical Approval (ATG) with certification or equivalent.

If at least two instrumented static pile load tests are carried out on site, the model factor γPδ3 from Table 7 may be used in formula [6], provided that the results of the pile load tests meet the conditions of Appendix A (p. 39). In that case, the instrumented SLTs must meet the requirements described in Appendix B (p. 41).

For screw piles, CFA piles and micropiles that have a Technical Approval (ATG) with certification or equivalent, the ATG states which model factor (γPδ1 or possibly a reduced γPδ2 value) may be used in which circumstances. These values take precedence over the values in this Report. The procedure for having a Technical Approval(ATG) with certification drawn up can be requested from the UBAtc (www.butgb.be, info@butgb.be).

Table 7 gives the values for the model factor.

4.3.5 CHARACTERISTIC VALUE OF THE CARRYING CAPACITY

The characteristic value of the load-bearing capacity of compression-loaded piles $R_{c,k}$ is determined on the basis of the various calculated and calibrated values of the load-bearing capacity $R_{c,cal,i}$. This is done by the correlation factors ξ 3 and ξ 4 respectively to the mean value and the minimum value:

$$= \mu\iota\nu \left(\frac{(R_{c,cal})_{\Gamma\epsilon\mu\iota\delta}}{} ; \frac{(R_{c,cal})_{mi}}{} \right)$$

$$R_{c,k} \quad \Big| \frac{\quad}{\xi_3} \quad \frac{\quad n}{\xi_4} \Big| \; . \qquad [7]$$

India $\frac{(R_{c;cal})_{min}}{\xi_4} < \frac{(R_{c,cal})_{average}}{\xi_3}$, then the correlation factor is ξ_4 applied to the point resistance and

the frictional resistance corresponding to the static CPT that yielded the lowest total load-bearing capacity. In other words, the correlation factor is not applied to the minimum point resistance and the minimum frictional resistance, each of which can come from a different static CPT.

The values of ξ_3 and ξ_4 are given in tables 8 and 9 (p. 30) as a function of the number of static CPTs per square metre and the number of piles per group. It is assumed that the statistical CPTs are spread out in such a way that they are representative of the entire site. For intermediate CPT density values, an interpolation can be applied.

The values for pile groups with more than three piles are only valid for structures that are sufficiently rigid and strong to be able to transfer the forces acting on a pile with an insufficient load-bearing capacity (e.g. a weaker zone) to the neighbouring piles. A construction can be considered sufficiently stiff and strong if it shows a maximum settlement of 5 mm at that location when removing one pile in the calculation. If this is not the case, the correlation factors from the first row of Tables 8 and 9 (1 post) should be applied. It also follows from the above that the same correlation factors must be applied to a construction on two or three piles as to a construction on one pile.

Table 8 $\xi3$ values to be applied to the mean value.

Number of poles	CPT density				
	1 CPT/10 m²	1 CPT/50 m²	1 CPT/100 m²	1 CPT/300 m²	1 CPT/1,000 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 9 $\xi4$ values to be applied to the minimum value.

Number of poles	CPT density				
	1 CPT/10 m²	1 CPT/50 m²	1 CPT/100 m²	1 CPT/300 m²	1 CPT/1,000 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

For the characteristic value of the load-bearing capacity, derived from a static CPT carried out in the axis of the pile or at a distance of no more than 3 times the pile base diameter D_b, the correlation factors $\xi3$ and $\xi4$ may be equal to 1.08.

The x-values method is a simplified method to take account of the heterogeneity of the site, which means that the values for a very homogeneous site can be quite conservative. Subject to justification, other x-values may be applied. For example, values can be used that correspond to a higher CPT density (1 or 2 columns more to the left in tables 8 or 9). This justification can consist of a statistical analysis from which the characteristic value can be derived directly or of a study of the autocorrelation length.

4.3.6 DESIGN VALUE OF THE ABILITY TO PAY

The design value of the load-bearing capacity of compression-loaded piles $R_{c,d}$ is obtained by dividing the characteristic value of the point resistance $R_{b,k}$ and of the frictional resistance $R_{s,k}$ by the partial factors $\gamma\beta$ and gs:

$$Rcd = R_{b,k}/\gamma\beta + R_{s,k}/\gamma\sigma. \qquad [8]$$

The value of these partial factors is given in Table 10.

Table 10 Values for the partial factor for the point resistance $\gamma\beta$ and for the friction resistance $\gamma\sigma$.

Group of pile types	$\gamma\beta$	$\gamma\sigma$
Piles driven and pressed in	1,00	1,00
Screw piles	1,07	1,00
CFA piles	1,10	1,00
Bored piles	1,20	1,00
Micropiles	1,10	1,10

If, in addition to the minimum required quality checks on the execution – as specified in the implementing standards NBN EN 12699 [B14], NBN EN 1536 [B2] and NBN EN 14199 [B15] – additional quality checks on the execution are carried out, reduced safety factors may be added if necessary.

be tried on. The additional rules relating to the monitoring of the pile installation in order to increase confidence in the quality guarantee of the execution and the consequent use of reduced values for $\gamma\beta$ and $\eta\sigma$ are then mentioned in a Technical Approval (ATG) or equivalent.

5 AXIALLY LOADED ON TENSILE ALEN

5.1 GENERAL

If a structure is subjected to an upward force due to an external structural force and/or water pressure, it is necessary to check whether the weight of the structure is variable in order to ensure vertical equilibrium. If this is not the case and therefore tension piles are provided, the length of the tension piles must be determined in such a way that this imbalance is compensated.

A pile can be loaded on tension by an external structural force (e.g. in the case of abutments or masts, eccentric loads, cantilevers, etc.) and/or by the presence of upward water pressure (tunnels, construction pits, empty water reservoirs, etc.).

These guidelines only relate to the load-bearing capacity of an individual pile loaded on tension. For a group of piles, which must take into account, among other things, the group effect and a possible uneven load on the piles, additional checks are required that fall outside the scope of this Report. The load-bearing capacity of anchors is also not discussed in this Report. A separate document will be drawn up for this.

For a pole loaded on tension, the own weight of the pole may be charged, provided that this is explicitly stated. The dead weight of the pile is then considered as a stabilizing permanent load G_{stb}.

Two situations need to be examined:

- the pile is pulled out of the ground: checking the frictional resistance on pull along the pile shaft (see § 5.4, p. 34).
  By analogy with the dimensionering of piles loaded under pressure, the following disparity [9] must be met (see § 7.6.3.1 (2) of NBN EN 1997-1 [B9]). This is a GEO situation (collapse or very large deformation of theE undersoil, where the strength of the soil or rock makes a significant contribution to the resistance) according to Eurocode 7:

$$F_{t,d} \leq R_{t,d}, \qquad [9]$$

  whereby:
  – $F_{t,d}$ = the design value of the axial tensile load on the pile [kN]
  – $R_{t,d}$ = the design value of the load-bearing capacity of the post-load bearing post [kN]
- the root ball criterion, in which the pile is pulled out of the surrounding soil together with a volume of soil (see § 5.5, p. 36). This is a UPL situation (loss of balance of the structure or the subsoil due to buoyancy due to water pressure – buoyancy – or other verticalhttp://2.4.7.4/loads) according to Eurocode 7. In this case, inequality [10] must be met (see § 2.4.7.4 (1) of the NBN EN 1997-1 [B9]):

$$V_{dst,d} \leq G_{stb,d} + rd, \qquad [10]$$

  whereby:
  – $G_{stb,d}$ = the design value of the vertical permanent favourable (stabilizing) loads [kN]
  – $V_{dst,d}$ = the design value of the vertical unfavourable (destabilizing) loads [kN]
  – Rd = the design value of the resistance to pushing [kN].

For more background information on the previous verifications, please refer to Appendix E (p. 46).

5.2 AXIAL TENSILE LOAD

The axial tensile load originates from an external structural force action and/or upward water pressure.

Guidelines for determining the water level and the representative value of the corresponding tensile load can be found in Appendix C (p. 43). Tensile load caused by water pressure is considered a permanent load.

Permanent stabilizing loads come from the dead weight of the structure and possibly from the effective weight of the soil on top of the tunnel or reservoir. The actual weight of the soil may only be taken into account if no excavations will take place during the period that the piles are subject to tension.

5.3 ALTERNATING LOAD

If the pile is subjected to alternating loads (i.e. also under pressure in the serviceability limit state), the influence of this on the frictional resistance must be checked. Several factors play a role in this (number of cycles, amplitude, etc.). In the absence of data (e.g. tests, literature, etc.) on the influence of the alternating load on the frictional resistance, a reduction must be applied, as indicated in § 5.4.1.

5.4 FRICTIONAL RESISTANCE ON PULL ALONG THE PILE SHAFT: GEO-CHECK

By analogy with the dimensioning of compression-loaded piles and as mentioned above, the friction resistance on tension along the pile shaft must be checked by complying with the unevenness [9]: $F_{t,d} \leq R_{t,d}$. This is a GEO check (collapse or very large deformation of the subsurface, where the strength of the soil or rock makes a significant contribution to the resistance) according to the Eurocode 7. The following explains how $F_{t,d}$ and $R_{t,d}$ can be determined.

5.4.1 CHARACTERISTIC VALUE OF THE FRICTIONAL RESISTANCE ON TENSILE

The frictional resistance on tensile R_t is determined using the following formula:

$$Rt = \chi\sigma . S \ (\alpha_{t,i}\alpha_{D,i} . \ Hi. \ qs,i), \quad [11]$$

whereby:

- qs,i, $\chi\sigma$, $\alpha_{D,i}$ en h_i = zie § 4.3.3 (p. 25)
- $\alpha_{t,i}$ = an empirical installation factor for layer i, which, in addition to the influence of the installation method of the

  in a certain type of soil also takes into account the roughness of the pile shaft. These values are given in table 11 (p. 35).

In order to ensure that the calculated load-bearing capacity of tensile loaded piles would be sufficiently safe, a gRd model factor is used to calculate the calibrated load-bearing capacity $R\tau,\chi\alpha\lambda$ (see § 2.4.1 (6), § 2.4.1 (8), § 2.4.7.1 (6) and § 7.6.2.3 (2) of NBN EN 1997-1 [B9]):

$$Rt,cal = Rt / \ \gamma P\delta, \quad [12]$$

waarbij:

- Rt,cal = the calibrated load-bearing capacity of tensile-loaded piles
- $\gamma P\delta$ = the model factor ($\gamma P\delta 1$, $\gamma P\delta 2$ or $\gamma P\delta 3$, see Table 7, p. 29, and Appendix A, p. 39).

Table 7 (p. 29) gives the values for the model factor.

If there is a Technical Approval (ATG) with certification or equivalent for the type of pile in question, the applicable value for the gRd model factor can be found here. If no Technical Approval (ATG) with certification or equivalent is available, $\gamma P\delta 1$ αλωαψσ αππλιεσ. The procedure for having aTechnical Approval(ATG) drawn up with certification can be requested from the UBAtc (www.butgb.be, info@butgb.be).

Table 11 Values of the installation factor for the frictional resistance on tensile ατ.

Tax	Installation factor for the frictional resistance on pull ατ (²)
Axial Tensile Load (¹)	Posts of categories I, II and III: αξισ ÷ 1.25 Posts of category IV: αξισ ÷ 1.00
Alternating tensile load (¹)	Posts of categories I, II and III: αξισ ÷ (1.25 * 1.33)= αξισ ÷ 1.66 Posts of category IV: αξισ ÷ (1.00 * 1.33) = αξισ ÷ 1.33

(¹) See § 5.3 (p. 34).
(²) For posts of category IV (micropiles), the values of the factors in this table are valid only for micropiles with a diameter D_b < 180 mm and with a length in the resistive layer(s) of less than 18 m. For micropiles with a diameter D_b greater than 180 mm and/or if the length of the micropile in the resistive layer(s) is greater than 18 m, at least two static pile load tests shall be carried out to verify the load-bearing capacity (see Annex B, p. 41).

If a contractor has had instrumented static pile load tests carried out, he may – under certain conditions – not only apply a reduced model factor, but may also use installation factors (αβ and/or αξλε) for the point resistance and/or the rubbing resistance other than those mentioned in Table 5 (p. 27) for the pile type in question.

The more favourable installation factors may only be applied if they are included in a Technical Approval (ATG) with certification or equivalent. The procedure for having a Technical Approval (ATG) drawn up with certification can be requested from the UBAtc (www.butgb.be, info@butgb.be).

The characteristic value of the load-bearing capacity of tension-loaded piles $R_{t,k}$ is determined on the basis of the various calculated and calibrated values of the load-bearing capacity $R_{t,cal,i}$. This is done by the correlation factors ξ_3 and ξ_4 respectively to the mean value and the minimum value:

$$= \mu\iota\nu \left| \frac{(R_{t,cal})_{average}}{\xi_3} ; \frac{(R_{t,cal})_{min}}{\xi_4} \right|$$

$$R_{t,k} \qquad 3 \qquad 4 . \qquad [13]$$

In tables 8 and 9 (p. 30) the values of ξ_3 and ξ_4 can be found as a function of the number of statistical CPTs per m² and the number of piles per group. It is assumed that the static probes are spread out in such a way that they are representative of the entire site. For intermediate CPT density values, an interpolation can be applied.

The values for pile groups with more than three piles are only valid for structures that are sufficiently rigid and strong to be able to transfer the forces acting on a pile with an insufficient load-bearing capacity (e.g. a weaker zone) to the neighbouring piles. A construction can be considered sufficiently rigid if it shows a maximum rise of 5 mm at that location when removing one pile in the calculation. If this is not the case, the correlation factors from the first row of tables 8 and 9 (p. 30) (1 post) should be applied. It also follows from the above that the same correlation factors must be applied to a construction on two or three piles as to a construction on one pile.

For the characteristic value of the load-bearing capacity, derived from a static CPT carried out in the axis of the pile or at a distance of no more than 3 times the pile base diameter D_b, the correlation factors ξ_3 and ξ_4 may be equal to 1.08.

The x-values method is a simplified method to take account of the heterogeneity of the site, which means that the values for a very homogeneous site can be quite conservative. Subject to justification, other x-values may be applied. For example, values can be used that correspond to a higher CPT density (1 or 2 columns more to the left in Tables 8 and 9, p. 30). This justification can consist of a statistical analysis from which the characteristic value can be derived directly or of a study of the autocorrelation length.

5.4.2 CALCULATION VALUES AND CONTROL

The design value of the load-bearing capacity of poles $R_{loaded\ on\ tension,d}$ is obtained by the characteristic value of the frictional resistance on tensile $R_{t,k}$ by dividing the partial factor $\gamma_{s,t}$:

$$R_{t,d} = R_{t,k} / \gamma_{s,t}, \quad [14]$$

whereby:

$\gamma_{s,t} = \gamma\sigma$ from table 10 (p. 30).

The design value of the axial tensile load $F_{t,d}$ is determined by multiplying the representative value of the axial tensile load $F_{t,rep}$ by a partial factor for loads gF:

$$F_{t,d} = F_{t,rep} * \gamma\Phi, \quad [15]$$

whereby:

$\gamma\Phi$ is equal to the value from table 3 (p. 20).

As mentioned earlier in § 4.1 (p. 19), the audit should only be verified according to draft approach 1 (DA1/1).

The inequality [9] from § 5.1 (p. 33) must be met: $F_{t,d} \leq R_{t,d}$.

5.5 ROOT BALL CRITERION: UPL CONTROL

The clod criterion, in which the pile is pulled out of the surrounding soil together with a volume of soil, must be checked by satisfying the equation [10] from § 5.1 (p. 33): $V_{dst,d} \leq G_{stb,d} + Rd$.

This is an *uplift* situation (UPL: loss of balance of the structure or the subsoil as a result of buoyancy due to water pressure – buoyancy – or other vertical loads) according to the Euro code 7. The weight of the root ball is considered a resistance. The frictional resistance along the surface of the root ball is usually not taken into account. In addition, a model factor is applied (see § 5.5.2, p. 37).

As mentioned earlier (see § 5.1, p. 33), only the root ball criterion of an individual pile is taken into account here.

5.5.1 WEIGHT OF THE ROOT BALL

The shape of the root ball is determined as shown in figure 7 (p. 37). The magnitude of the angle α depends on the effective angle of internal friction of the soil φ', the cone resistance q_c of the soil and the category of the pile (see table 5, p. 27) and is shown in table 12.

Table 12 Size of the angle α.

Cone resistance qc	Poles (·)	α [°]
< 1 MPa	Categories I, II, III	0
≥ 1 MPa	Category I and IV	2/3 . φ'
	Categories II and III	1/2 . φ'

(·) For the different categories of piles, we refer to table 5 (p. 27).

Comments

Piles with a widened base are more likely to behave like a plate anchor [D4].

In certain cases, the normative situation may consist of the combination of the weight of a root ball that engages at a level higher than the pile tip and the frictional resistance along the part of the pile located below this point of engagement. If necessary, this situation should be verified by means of formula [10].

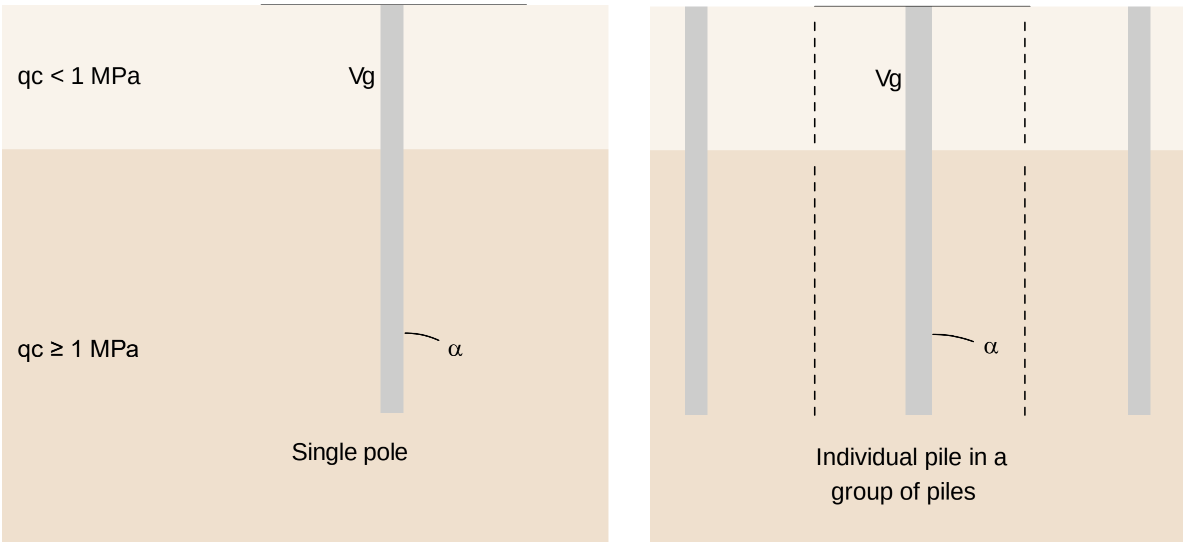


Fig. 7 Shape of the root ball.

The volumetric weight of the soil of the root ball is determined on the basis of table 2.1 of the National Annex to the Eurocode 7, the NBN EN 1997-1 ANB [B11].

5.5.2 DESIGN VALUES AND VERIFICATION OF THE ROOT BALL CRITERION

The inequality [10] from § 5.1 (p. 33) must be met:

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

The design values of the axial tensile loads are determined by multiplying the representative values of the loads by the partial load factors for a UPL check (see A.4 of the NBN EN 1997-1 ANB [B11]).

If the frictional resistance along the surface of the root ball is neglected and a gRd model factor is applied, inequality [10] changes into:

$$G_{Dst} . X_{Gdst} + Q_{Dst} . X_{Qdst} - G_{etc} . X_{Gstb} \, \delta \, \chi_{Gstb} . X_{g} . \frac{'V_{g}}{X_{Rd}}, \quad [16]$$

whereby:

- Gdst = the permanent destabilizing load [kN]
- γ_{Gdst} = the partial factor for the permanent destabilizing burden
- Qdst = the variable destabilizing load [kN]
- γ_{Qdst} = the partial factor for the variable destabilizing load
- Gstb = the lasting favorable (stabilizing) load [kN]
- γ_{Gstb} = the partial factor for the permanent stabilizing load
- γ'_{g} = the effective volumetric weight of the soil of the root ball [kN/m³]
- Vg = the volume of the root ball [m³]
- $\gamma P\delta$ = de modelfactor.

The values to be taken into account for the load factors γ_{Gdst}, γ_{Gstb} and γ_{Qdst} in a UPL check are given in Table 13 (p. 38).
The value to be taken into account for the gRd model factor in a UPL check of the root ball criterion is 1.25. This value takes into account, among other things, the uncertainty about the shape of the root ball and the fact that the calculated volumetric weight of the soil of the root ball is a high characteristic value (see table from the NBN EN 1997-1 ANB [B11]) and this, while for the root ball criterion a low characteristic value is decisive.

Table 13 Load factors γ_{Gdst}, γ_{Gstb} and γ_{Qdst} for a UPL check.

Tax		Symbol	Factor
Lasting	Zero (1)	γ_{Gdst}	1,0
	Favorable (2)	γ_{Gstb}	0,9
Variable	Zero (1)	γ_{Qdst}	1.1 (3)

(1) Destabilizing.
(2) Stabilizing.
(3) This value differs from the informative value in Table A.15 of the standard NBN EN 1997-1 [B9].

ANNEX A

Conditions for the application of a reduced model factor $\gamma P\delta 3$ (SLT on site)

If at least two instrumented static pile load tests (SLTs) are carried out on site, which meet at least the requirements mentioned in Appendix B (p. 41), then the model factor $\gamma P\delta 3$ may be used, provided that the following equations [17] and [18] are met:

$$\left(\frac{R_{c,m}}{R_c}\right)_{average,performer} \varepsilon \left(\frac{R_{c,m}}{R_c}\right)_{average,group}$$
$$\left(\frac{R_{t,m}}{R_t}\right)_{average,performer} \varepsilon \left(\frac{R_{t,m}}{R_t}\right)_{average,group} \qquad [17]$$

IN:

$$\frac{\left(\frac{P_{c,m}}{R_c}\right)_{min,executor}}{\left(\frac{P_{c,m}}{R_c}\right)_{max,executor}} \varepsilon \nu$$

$$\frac{\left(\frac{R_{t,m}}{R_t}\right)_{min,executor}}{\left(\frac{P_{t,m}}{R_t}\right)_{max,executor}} \varepsilon \nu \qquad [18]$$

whereby:

- $R_{c;m}$ = the measured value of the bearing capacity [kN]. In Belgium, this value is derived from the subsidence diagram of a pile load test on pressure, if the load corresponding to a basic pile subsidence of 10 % D_b
- $R_{t;m}$ = the measured value of the load-bearing capacity [kN], derived from a tensile load test (see Appendix B, p. 41). For piles of categories I, II and III, it can be derived from the load-rising diagram of a pile load test, as the load corresponding to a pile head rise of 10 % D_b. Alternatively, this value can also be derived from the creep curve, for example as an asymptote, or conventionally at a load that corresponds to a specific creep speed. This alternative method is usually used for category IV piles (micropiles)
- R_c = the calculated value of the load-bearing capacity of a pile loaded under pressure (with $\alpha\beta$ and $\alpha\xi\lambda\varepsilon$ from the table len 5 of 6, pp. 27 in 28) [kN]
- R_t = the calculated value of the load-bearing capacity of a pile loaded on tension (with α_t from table 11, p. 35)
- μ, ν = see table A.1 (p. 40).

When all individual values of $\left(\frac{R_{c,m}}{R_c}\right)_{performer}$ be greater than $\left(\frac{R_{c,m}}{R_c}\right)_{average,group}$, the comparison [18].

Table A.1 Values for the factors μ and Ψ.

Group of pile types	μ	Ψ		
		Number of static pile load tests		
		2	3	4
Screw piles	0,95	0,90	0,85	0,80
CFA piles	0,90	0,85	0,80	0,75
Micropiles	0,90	0,85	0,80	0,75

ANNEX B

Minimum requirements for pile load tests

In order for the pile load tests to be taken into account for the determination of the installation factors and/or the applicable model factor(s) and/or the boundary conditions, the preparation and execution of the tests must correspond to one of the following documents:

- **for pressure pile load tests**: the NBN EN ISO 22477-1 standard [B16]. This test standard allows for different methods. In Belgium, the *maintained load test procedure* is opted for , which is carried out in one cycle (i.e. without intermediate relief) and in which the load steps are kept constant for a period of one hour
- **for tensile pile load tests**: pending the publication of the standard EN ISO 22477-2 [E2], the procedures of NF P 94-150-2 [A1], ISSMGE 1985 [I1] or NBN EN ISO 22477-5 [B17] may be followed. The latter test standard is mainly applied to micropiles.

The design of the test campaign, the execution of the soil investigation, the installation of the test piles and the execution and analysis of the tests must be monitored by an independent organization.

The test piles shall be instrumented in such a way that the point resistance, the total friction and the mobilization curves of the unit friction in the various relevant soil layers can be derived from the test results.

The following information must be made available as a minimum:

1. an overview of the test site(s), geology and soil investigation, consisting of at least one electric static CPT, preferably in the axis of the pole or at a maximum distance of 5 metres from the tested pole. The soil investigation must be carried out sufficiently deep, namely at least 5 meters below the pile tip. The depth level of the soil investigation is given in metres of TAW (Second General Levelling, official height reference system in Belgium) or in relation to a local fixed reference
2. An overview of the pole installation:
   – the dimensions and power of the material and equipment used
   – the execution procedure followed, the execution parameters (in the case of screw pile types, this includes, for example, the drilling torque, the *pull down*, the penetration speed, the rotational speed, the pull-up speed, the concrete depositing pressure, etc.), the concrete composition, the concreting method and concreting times, the dimensions of the reinforcement cage ...
   – the levels (ground level, drilling depth, etc.) are given in relation to the same reference as the Soil investigation
3. An overview of the test set-up:
   – the type and structure of the reaction system
   – the distances between the supports of the reaction system and the test pile
   – the different types of measuring devices and the calibration data
   – the distances between the test set-up and the reference beacons of the measuring system
   – ...
4. The test results shall contain at least the following information:
   – the applied pile load, pile head subsidence and pile base subsidence as a function of time
   – the pile head subsidence per load step on a logarithmic time scale
   – the creep curve
   – the normal force progression over the pile with the depth for each load step
   – the load-subsidence diagram of the pile head, broken down into the point resistance and the friction

– the load-subsidence diagram of the pile base, broken down into the point resistance and the friction
– the mobilization curves of the unit friction in the relevant soil layers.

Important note
Carrying out load tests on pressure on category IV piles (micropiles) is not always easy. It is difficult to introduce the load centrally into the micropile. Limited deviations can quickly translate into critical bending moments in the upper micropile part. For this reason, and because the load-bearing capacity of micropiles comes mainly from the shaft friction, it is allowed to carry out tensile load tests on micropiles and to equate ash to $\alpha\tau$.

ANNEX C

Guide values for the water level and loads due to water pressure

Guide values for the water level

The water level Swmust be determined for the period that the piles are subject to tension, i.e. the life of the structure or the duration of a construction phase (e.g. until the load on the structure has become so large that there is no longer a tensile load in the piles). Failing drainage, extreme precipitation, etc. must be taken into account. The guide values for the water level to be taken into account areshown in Table C.1. Water pressures are considered a permanent load.

Table C.1 Water level to be taken into account for determining the buoyant water pressures.

Regime of the water table	Available water level measurements (1)	Waterstand Z_w [m] (3)
Free – without drainage	no	ground level
	1 measurement	SW,m + 1,50 m
	Measurement series (2) ≥ 6 months	Sw,m,max + 1,00 m
	Measurement series (2) ≥ 1 year	SW,m,max + 0,50 m
Under pressure – without drainage	(–)	based on a hydrogeological study
With drainage	(–)	based on a dewatering study

(1) The measurement must be carried out in a monitoring well on the site.
(2) Minimum 1 measurement per month.
(3) S_w is the water level to be charged; $Z_{w,m}$ is the measured water level; $Z_{w,m,max}$ is the highest measured water level.

Physically unrealistic water levels (e.g. above the ground surface) should not be taken into account. In flood areas, however, the possibility of flooding must be taken into account.

When measuring the water level in advance, care should be taken to ensure that the measured values are not influenced by active dewatering in the area.

Loads due to water pressure

The representative value of the destabilizing load that acts on a pile as a result of the upward water pressure can be determined as follows:

- for open entrances and construction pits (see Figure C.1, p. 44), it can be derived from the water pressure on the underside of the floor slab A: u * A
- for closed volumes such as tunnels and closed reservoirs (see Figure C.2, p. 44), it can be derived from from the Archimedean force acting on the submerged volume V: $\gamma\omega$ * V.

Remark
The above provisions apply insofar as the water pressure over the length of the pile is hydrostatic with depth. If the pile passes through an impermeable layer and there is a higher water pressure underneath, then the root ball criterion at that level must also be determined for the UPL.

The representative value of the stabilizing load comes from the dead weight of the structure and, where appropriate, from the effective weight of the soil on top of the tunnel or reservoir. This effective weight may only be taken into account if no excavation will take place during the period that the piles are subject to tension.

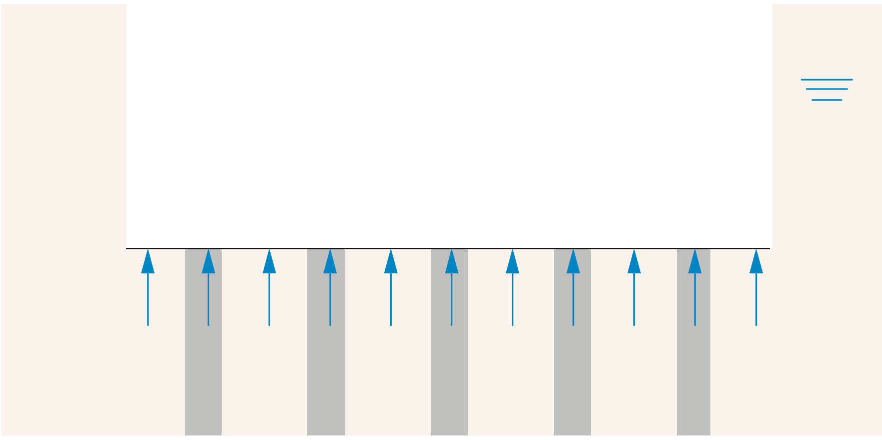


Fig.. C.1 Loads on a tension pile as a result of water pressure at an open access road or construction pit.

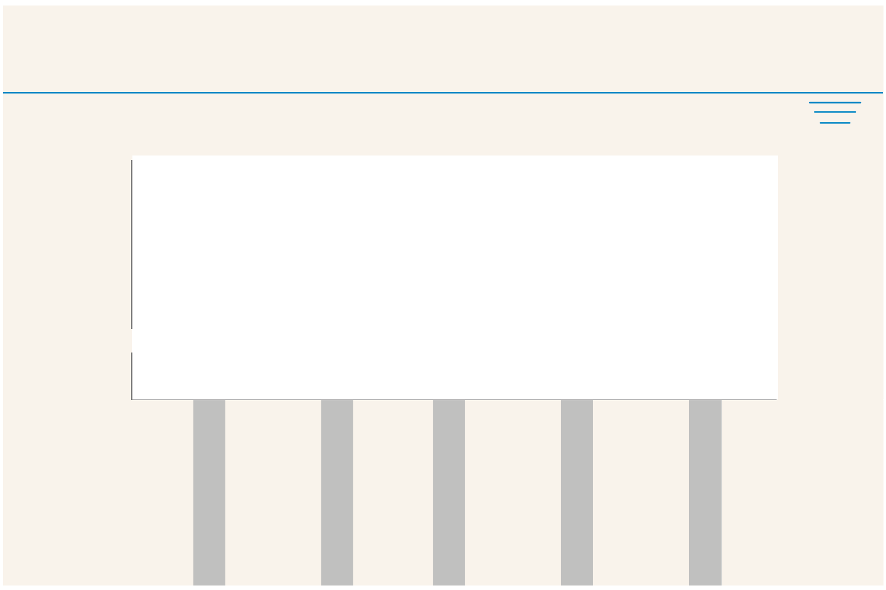


Fig.. C.2 Loads on a tension pile as a result of water pressure in a tunnel or a closed reservoir.

ANNEX D (INFORMATIONAL)

INDICATIVE VALUES FOR αΔ

For category IVc micropiles, where the secondary grout is injected selectively (i.e. knowing/choosing where to inject) and in several stages (over time) via sleeve tubes (TAMs) and with a double *packer*, the magnification factor for the pile shaft diameter αΔ can be based on the values used for IRS (*injection répétitive et sélective*) are included in the T.A. 95 [C1] and the NF P 94-282 [A2] (see Table D.1).

These are indicative values based on a secondary grout injection in which the pressure is greater than or equal to the limit pressure p_l (obtained from the pressiometer test) of the site and is (usually) less than 4 MPa (40 bar).

The infusion volumes given in Table D.1 are indicative only.

The αΔ values in Table D.1 are not necessarily applicable to other post-injection methods.

The correlation between the limit pressure p_l from the pressiometer test and the cone resistance q deer test can be based on the following correlations: $_c$ From the son-

- zand/grind: $q_c = 9 \cdot pl$
- loam, sandy clay/sandy loam, clayey sand/clayey loam: Q_c = 4 . Pl
- clay: $q_c = 2{,}65 \cdot Pl$.

Table D.1 Guide values for the magnification factor for the diameter aD for micropiles of category IVc carried out according to a classic IRS system (with TAMs and a double *packer*) according to the T.A. 95 [C1] and the NF P 94-82 [A2] (1).

Soil type	αδ	Indicative conditions of application	
	IRS (in prince $p_l \geq pl$)	Normal grout injection volume V_i	C/W dosing of the grout
Gravel	1,8	1.5 US	1.7 to 2.4
Sandy gravel	1.6 to 1.8	1.5 US	
Grindhoudend zand	1.5 to 1.6	1.5 US	
Coarse sand	1.4 to 1.5	1.5 US	
Middelgrof zand	1.4 to 1.5	1.5 US	
Fine sand	1.4 to 1.5	1.5 US	
Loamy sand	1.4 to 1.5	1.5 to 2 V_s	
Loam	1.4 to 1.6	2 US	1.7 to 2.4
Clay	1.8 to 2	2.5 to 3 V_s	
Marl	1,8	1.5 to 2 V_s for a compact layer	1.7 to 2.4
Succession of limestone and marl layers	1,8	2 to 6 Vs or more if fragmented	
Weathered or fragmented chalk	1,8	1.1 to 1.5 V_s for a finely torn layer	
Weathered or fragmented rock	1,2	2 Vs or more for a broken layer	1.7 to 2.4

(1) IRS = *repetitive and selective injection.*
The injection volume V_s is related to the circumference αδ . γ and is purely

ANNEX E

BACKGROUND TO THE VERIFICATIONS OF PILES LOADED ON AXIALLY ON TENSION

For axially loaded piles the following ULS verifications must be carried out according to the Eurocode 7:

- **GEO DA1/1**: verification of the frictional resistance on tension along the pile shaft according to design Ring 1, combination 1
- **GEO DA1/2**: verification of the frictional resistance on tension along the pile shaft according to design Ring 1, combination 2
- **UPL Friction**: Verification of the frictional resistance on pull along the pile shaft with the UPL ponderation coefficients (*uplift*)
- **UPL root ball**: verification of the root ball criterion whereby the pile together with a volume of soil from the surrounding soil.

We assume a situation in which a pile is loaded axially on tension as a result of water pressure and an external load on tension. Hereby:

- $You_{dst,rep}$ / $You_{dst,d}$ = the representative value / design value of the destabilizing upward force with regard to consequence Ev an waterdruk
- $G_{dst,rep}$ / $G_{dst,d}$ = the representative value / design value of any other permanent destabilizing This is directed load (pull) that is not caused by water pressure
- Q_{dst} / $Q_{dst,d}$ = the representative value / design value of any other variable destabilizing directed load (pull) that is not caused by water pressure
- GS Peter / $G_{stb,d}$ = the representative value / design value of the stabilizing downward cargo (pressure)
- $R_{t,k}$ / $R_{t,d}$ = the characteristic value / design value of the load-bearing capacity of the postage loaded on tension (afcom/shaft friction)
- R_k / R_d = the characteristic value / design value of the resistance to uplift.

The general balance that must be met in the ULS – GEO is described as follows:

$$F_{T,D} \leq T, _{D+G}, D_{+Q}, D, _{D-G, D-G, D,} \leq, T_D$$

With the GEO DA1/1 and GEO DA1/2 ponderation coefficients, this gives:

- **GEO DA 1/1**: $1.35\ U_{dst,rep} + 1.35\ G_{dst,rep} + 1.50\ Q_{dst,rep} - 1.00\ G_{stb,rep} \leq R_{t,k} / 1.00$ [E.1]
- **GEO DA 1/2**: $1.00\ U_{dst,rep} + 1{,}00\ G_{dtr,rep} + 1{,}10\ Q_{dst,rep} - 1{,}00\ G_{stb,rep} \leq R_{t,k} / 1.35.$ [E.2]

The general balance to be met in the ULS – UPL is described as follows:

$$V_{DST,D} \leq G_{STB,D} + R_D \text{ of } U_{DST,D} + G_{DST,D} + Q_{DST,D} - G_{STB,D} \leq R_D.$$

With the UPL ponderation coefficients (see Table 13, p. 38, for the load factors and the NBN EN 1997-1:ANB [B11] for the partial safety on the resistance) and assuming that the pile is pulled out of the ground (i.e. R_k = Rt, resistance from the shaft friction), this gives:

- **UPL friction**: $1{,}00\ U_{dst,rep} + 1.00\ g_{dst,rep} + 1.10\ Q_{dst,rep} - 0.90\ g_{stb,rep} \leq R_{t,k} / 1{,}40.$ [E.3]

With the UPL ponderation coefficients (see Table 13, p. 38) and assuming that:

– together with the pile, a volume of soil V_g (root ball) is pulled out of the surrounding soil, whereby the weight of this root ball is considered to be a stabilizing downward load

– and a model factor $\gamma_{P\delta}$ = 1.25 is applied to this root ball weight

– and the friction along the root ball is not taken into account (R_k = 0)

gives this:

• **UPL root ball**: $1.00\ U_{ff,\ rep} + 1.00\ G_{ff,\ rep} + 1.10\ Q_{ff,\ rep} - 0.90\ G_{ff,\ rep} \leq 0.90\ \gamma\gamma'.\ Vg / \gamma P\Delta \approx \gamma\gamma'\ .\ Vg / 1.40$. [E.4]

Because of the fact:

- that the verification [E.1] is always decisive in relation to the verification [E.2]
- that the verification [E.3] is very similar to the verification [E.2],

it was decided to use only the verifications GEO DA1/1 **[E.1] and** UPL root ball **[E.4]** for piles loaded on axial tension.

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For more information about the local geology and for results of drilling and probing from the area, we referto the website of the Flanders Subsurface Database: http://dov.vlaanderen.be.

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1000 BRUSSELS

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