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Design of axially loaded piles – Belgian practice

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PREAMBLE : This report has been put together by combining individual contributions provided by the authors listed above, reviewing assembled drafts in plenary meetings, and allowing the General Reporter to edit the report for general consistency. The sole intent of the report is to provide our foreign colleagues with a pragmatic representation of the current piling design practice in Belgium. It is therefore explicitly stated that this report should not be construed as superseding work products presently elaborated by several professional bodies such as Committees, Ministries, and Institutes. As no final decisions have been made regarding the Belgian National Application Document (NAD) for EC7 so far, references to the NAD contained in this report have to be considered as tentative.

1 REGIONAL GEOLOGY

The Belgian territory is rather flat with a continuous transition from a plain at the North Sea and the Dutch border to the highlands of the Ardennes, the highest point being situated at Botrange (694 m above sea level). The geology of the Tertiary and Quaternary formations in Belgium is characterised by a Southeast Northwest oriented epirogenetic axis [34], which follows the valleys of the rivers Haine, Sambre, Meuse and Vesdre (Figure 1), and which divides Belgium into approximately two equal parts.

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

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

2 PRACTICE FOR SOIL INVESTIGATION

Table 1 provides a summary of the Belgian practice for soil investigation, based on differences identified from the standpoint of usage, major applications, pile design objectives, and results used for pile design in common practice.

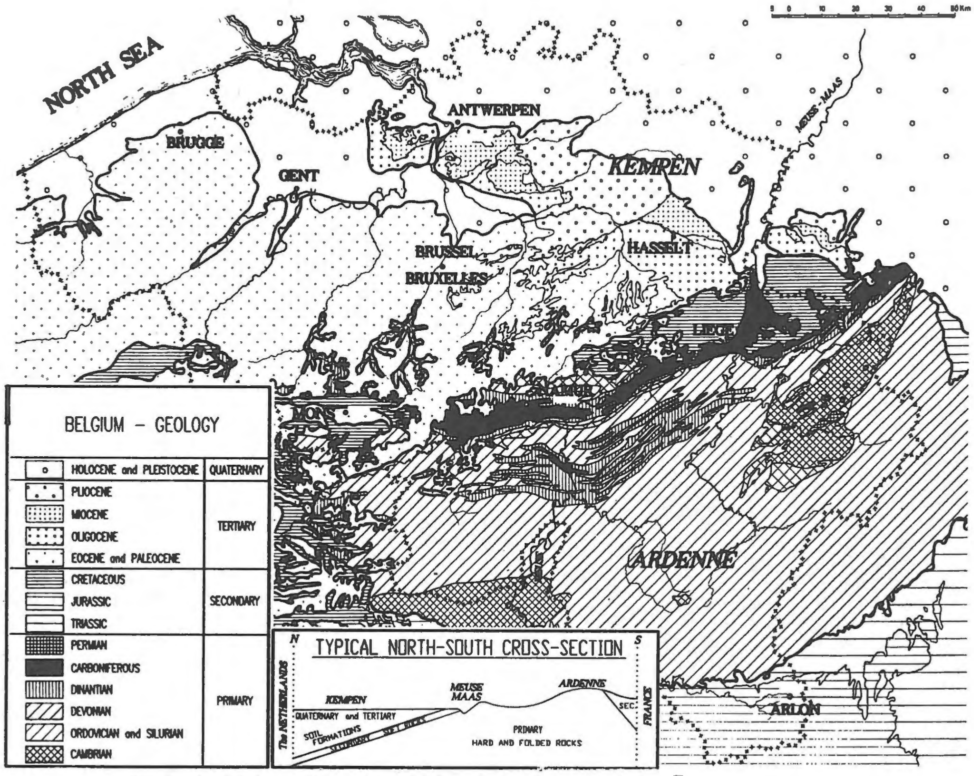


Figure 1. - Geological Map of Belgium (After de Béthune [21b])

The scope of the soil investigation performed for a particular project at a given site depends on a large number of project and site dependent factors. The project dependent factors such as the magnitude and distribution of loads, settlement sensitivity, and Owner requirements are mainly discussed in Section 5.1 relating to the general design philosophy supporting a piling project. This section 2 attempts to present the Belgian general practice for soil investigation as far as it relates to site dependent factors.

As a result of Belgium's high population density and intensive development, an extensive soil data base has been generated over the years. That data base consists not only in informal libraries of soil investigation files in private firms but also in formally accessible libraries or publications. Two main official sources of information are noteworthy: the library of the Belgian Geological Survey which contains logs of borings (copies of borelogs provided by boring companies) and the published Geotechnical Maps. The geotechnical maps currently cover parts of the most developed areas of the country (Cities of Antwerp, Brussels, Charleroi, Ghent, Mons, and Liège). Geological maps covering Belgian territory can also be consulted.

Based on this wealth of information and experience allowing for correlations, most soil investigations consist exclusively in CPT tests where feasible, i.e. where the CPT tests can be performed to a depth allowing the piling project to be designed. This is particularly the case for the very heterogeneous quaternary soil layers encountered in large areas in Belgium, where soil profiling is essential. For the reasons explained in Section 1, sites located in the northern part of Belgium, where piles are required, generally fulfil those conditions.

The use of the CPT was developed in Belgium by De Beer and Verdeyen from 1945 on. De Beer did pioneering work in developing his own interpretation and calculation methods based on the simple cone with closing nut (M4). In addition to interpretation methods providing a friction angle and the compression index, a particular method was published to derive the axial bearing capacity of a pile, based on the results of a CPT test (see Section 5.4.). The history, equipment and use of the CPT in Belgium are discussed in detail in Nuyens e.a. [31].

Table 1. Belgian Practice for Soil Investigation for Axially loaded Pile Design

Type	Sounding/ Testing	Usage	Major applications	Pile design objectives	Results used for pile design (see note below)
IN SITU (Belgian Practice First Choice)	CPT (Cone Penetration Test using 10 cm^2 cones M4, M1, M2 and E1)	Extensive	- Northern part of Belgium (M and E) - Southern part of Belgium (M exclusively)	Soil profiling, identification of potential end bearing layer, quantification of base and shaft resistance	q_c and f_s vs. depth q_c and Q_{st} vs. depth
	PMT (Pressure meter Test Menard type)	Rare to occasional, but growing	- Southern part of Belgium - Settlement sensitive structures	Quantification of base and shaft resistance, settlement	p_l, p_f, E_m
	Destructive Boring with parameters logging	Occasional	- Combined with CPT - Southern part of Belgium - Combined with non-destructive borings for calibration	Soil profiling and classification (to complete that inferred from CPT, where applicable), control of continuity and quality potential end bearing layer	Penetration and rotation velocity, crowding force, torque, vibration, pressure of flushing fluid
	Non-destructive Boring	Occasional	- Combined with CPT - Southern part of Belgium	Same as above + Quantification of base and shaft resistance	Results of LABORATORY Testing (see below) performed on collected samples
	DMT (Dilato-meter Test)	Experimen-tal	Research	Quantification of shaft resistance	c_u , K_D
	Vane	Rare	Clayey sites, friction piles	Quantification of base and shaft resistance	c_u
	Geophysical	Occasional, growing	South of Belgium (seismic and resistivity surveys)	Identification of end bearing layer	V_p, V_s
LABORATORY (in combination with IN SITU testing and when IN SITU testing is not feasible)	Identification	Occasional	General	General	Grain size, Atterberg limits, Organic content, CaCO3 content, w, γ_d, γ, γ_s
	Oedometer	Occasional	Compressible layer below pile toe	Pile group settlement analysis	C_c
	Unconfined compression	Rare	Rock	Quantification of shaft and base resistance	q_u
	Triaxial Compression	Exceptional	Soil and rock	Base and shaft resistance	c' , φ'
	Vane	Exceptional	Clayey layers	Base and shaft resistance	c_u

Note : ISSMFE Symbols used for soil properties, specialised literature symbols used for test results.

Investigation methods other than CPT testing are performed to complement CPT tests where warranted or where CPT testing is deemed unfeasible. The major components of those alternate piling investigations will still include in situ testing, leaving a minor role to laboratory testing. Information complementary to the normal CPT practice is warranted when investigation is performed far away from prior developments or when settlements must be specifically evaluated.

Table 2. Piling technologies used in Belgium

GROUP I	PILES EXECUTED WITH HIGH SOIL DISPLACEMENT
IMPACT DRIVEN PILES	
Type A:	Prefabricated (concrete and timber) piles
Type B:	Steel tube piles, close ended or open ended with plugging
Type C:	Cast-in-situ piles (with or without enlarged base)
SCREWED PILES	
Type A:	Cast-in-situ piles with screw shaped shaft
Type B:	Cast-in-situ piles with smooth shaft
Type C:	Prefabricated (concrete or steel tube) piles
JACKED PILES	
Type A:	Prefabricated concrete elements
Type B:	Steel elements
GROUP II	PILES EXECUTED WITH LIMITED SOIL DISPLACEMENT OR LOW SOIL RELAXATION
IMPACT DRIVEN OR VIBRATED PILES	
Type A:	Prefabricated concrete piles with enlarged base
Type B:	Steel beams with or without partial plugging
Type C:	Steel open ended tubes
Type D:	Steel beams with enlarged base
Type E:	Steel piles with grouting (low or high pressure)
DRILLED PILES	
Type A:	Cast-in-situ piles with special provisions to limit soil relaxation
GROUP III	PILES EXECUTED BY EXTENSIVE EXCAVATION OF THE SOIL
DRILLED PILES	
Type A:	With prefabricated concrete shaft
Type B:	With steel tube shaft
Type C:	Cast-in-situ piles (a) executed with temporary casing, (b) executed under thixotropic liquid, (c) continuous flight auger (CFA)
Type D:	With post grouting (low or high pressure)
GROUP IV	MICROPILES

More sophisticated soil mechanical parameters may be needed when pile design is enhanced by means of finite element programs. As these calculation methods are getting more popular in geotechnical practice, in situ and laboratory tests to define specific parameters such as the shear modulus may also become a larger part of the investigation practice.

3 PILING TECHNOLOGY

Table 2 provides a classification of piling technologies available in Belgium, according to installation process and its potential resulting influence on design.

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

Cast-in-situ piles are the predominant type. Precast piles are used where the soil geotechnical conditions are homogeneous enough and usually for limited bearing capacities or special applications. In recent years, major concerns have arisen around the problems of noise and vibrations, and vibration-free systems were extensively developed. One of the particularities of Belgium is the coexistence of different types of soil-displacement screwed piles which are well suited to our soil conditions.

Driven piles (Group I in Table 2, and Group II to some extent) are thus preferred in many cases, specially in weak subsurface conditions where soil failure governs the design. When a hard layer is encountered (intermediate or bearing layer), piles with partial or extensive soil excavation (Groups II and III) are generally preferred, specially when pile embedment into the hard layer is required. The classification provided in Table 2 also lists other types of piles which can be used more marginally or for special purposes.

4. NATIONAL RELEVANT DOCUMENTS

To this day, there is no single Belgian standard (norm or code) available to officially regulate the national piling practice. In the absence of truly relevant national documents, several owners and engineers have developed their own specifications or recommendations.

For the public construction market, the calculation and the choice of the pile system is always governed by the specifications of the different Administrations. This means that the relevant documents for the public market depend on the owner. These particular piling specifications from each Administration change from time to time, taking into account new technologies or/and relevant calculation criteria. For each tender on the public market one can ask for the general provisions of the piling specifications of the relevant Administration (e.g. Public Works, Federal Public Buildings Agency, National Railways, etc.) while special provisions for the project are normally part of the tender documents.

In the private construction market, every private consultant, small or large, uses his own piling specifications. Beside the lack of standards it is perhaps interesting to mention the so-called "Type Specifications 104", dated 1973, which are often used as a basis by administrations and consultants to draft their own specifications. On the other hand, the Federal Public Buildings Agency is drafting its own technical specifications "STS 21" on pile foundations [55], as an accompaniment to Type Specifications 104.

Belgium is also working on its National Application Document (NAD) for pile foundations, in order to implement the Eurocode 7 in Belgium. This document is expected to be available in the short-term future.

5 NATIONAL DESIGN METHODS

5.1 *General Philosophy*

5.1.1 Background

The general philosophy supporting the design of piles within a construction project stems from historical and structural factors influencing the organisation of the Belgian profession in general and of the project in particular. Those factors provide an imperfect framework that yet facilitates the achievement of the goal of pile design: "identify, within the physical (mainly geotechnical) and human environment of the site, the most adequate foundation system taking into account the loads and the deformability of the structure".

As a result of the paucity of national standards, the Belgian approach to pile design and construction is characterised by a truly integrated process combining the responsible contributions of key members of the piling project team: the Owner, the Civil Engineer, the General Contractor, the Piling Subcontractor, and, when adopted, the Technical Controller. It is common practice that the Engineer recommends the pile type(s) and specifies the so-called "nominal" load(s) Q_{SP} (more strictly called "specified" load) based on actions, pile layout, and identification of the bearing layer. It is also common practice that the Piling Contractor assumes the responsibility for the performance of the piles and for their design as far as toe level and soil bearing capacity are concerned. His assessment of the allowable value of the pile resistance (R_{ca} or R_{ta}) is however reviewed by the Engineer and the Technical Controller. The advantage of that division of responsibility is that the project benefits from the piling specialist's (subcontractor) know-how.

Because Belgian piling specialists remain competitively involved and rewarded in the design process, they have developed an engineering and problem solving expertise that is respected. Conflicts between "smart engineer" and "dumb contractor" are also thereby reduced. In the case a Technical Controller is assigned to the project, the project is covered by an umbrella insurance, which then further enhances the co-operative atmosphere of the design team, and permits more creative divisions of responsibility.

It should also be noted that civil Engineering has been and is being taught and practised in Belgium as an integrated field of engineering revolving around the construction process. There is no professional segmentation of the civil engineering practice, such as sanctioned abroad for example by titles distinguishing "Structural Engineers" from "Geotechnical Engineers". The structure of the civil engineering profession in Belgium also tends to facilitate communication between engineers employed by key members of the design team.

5.1.2 Basis of design .

In the vast majority of cases, piling solutions are designed on the basis of a geotechnical investigation, performed as discussed in Section 2, with the objectives to identify possible bearing levels and provide quantitative data to determine pile ultimate capacities. Methods to derive pile capacity from the geotechnical investigation data are often imposed in the special provisions of the project specifications. Although the methods specified are generally uniform, thereby confirming the existence of a body of generally accepted design principles (GADP), some differences can be noted between sets of specifications regarding the numerical value of coefficients.

The scope of the investigation programme depends on the size of the project and requirements from the Owner. However one has to acknowledge a strong tendency from the owners to limit the cost of the geotechnical investigation, fuelled by the lack of incentive attached to the present GADP.

Pile design methods generally accepted in Belgium are characterised by the semi-empirical, yet direct transformation of soil bearing parameters measured using in situ testing or sounding, as discussed in more detail in Section 5.4. Mostly used in Belgium are design methods based on the CPT test: the unit base resistance is obtained from a scaling procedure of the cone resistance diagram [15, 38, 53] while the pile shaft friction is obtained from the CPT total friction, local friction, and/or cone resistance diagrams. The De Beer method has been validated and further calibrated thanks to an extensive experimental basis spanning 30 years of full-scale co-operative research, as described in Section 8.

Because the basic CPT-based design method provides the ultimate base and shaft resistance of jacked or driven compression displacement piles, installation coefficients have to be introduced to account for the installation effects of each pile type with reference to the displacement pile types which were used to validate the basic design method. Tension piles are also designed based on CPT tests, but with a higher degree of conservatism, reflecting the limited tension pile test data base . When PMT is used in Belgium, the French design approach [52, 54] is followed.

As discussed in more detail in Section 5.3, static load testing of piles is rarely used as a design tool and generally confined to the control of designed piles. Safety and serviceability issues are discussed in Sections 5.7 and 5.8, respectively.

5.2 *Definitions and symbols*

s_b, s_h	Pile settlement; at the toe and at the head, respectively
$Q^m_{s_b}$, $Q^{s_h}_m$	Load corresponding to a pile settlement s_b (subsript) at the toe and s_h (superscript) at the head, respectively; m indicating the value measured from a static load test.
Q_{SP} or Q_n	Specified (or "nominal" per Belgian usage) pile "load". Used as reference in the piling contract and for control testing, the "nominal" load is specified generally by the engineer based on an envelope analysis of the reactions needed from isolated piles and piles in groups, when considering "normal" actions and the identification of an economical reaction module. "Normal" actions include all loads except for accidental and exceptional loads; exceptional wind but no snow is however considered if less favourable; negative skin friction if applicable should be accounted for. In all rigor, the "nominal" load Q_n is a reference loading level on the pile (i.e. a reference action) while the "specified" load Q_{SP} can be viewed as a reference value to establish the required performance of the pile (i.e. a reference value of the pile resistance, which could be labelled R_{SP}).
R^m_{ca}, R^c_{ca}	Allowable pile resistance in compression obtained from a pile load test and from a calculation, respectively. A satisfactory load test implies $R^m_{ca} \geq Q_{SP}$; a satisfactory design for an individual pile, as generally guaranteed by the Piling Contractor, implies $R^c_{ca} \geq Q_n$.
R_{bu}, R_{ba}	pile *base* resistance; ultimate and allowable value, respectively
R_{su}, R_{sa}	idem for pile *shaft* resistance
R_{cu}, R_{ca}	idem for total pile resistance in *compression*
R_{tu}, R_{ta}	idem for total pile resistance in *tension*
q_b, q_s	unit values for pile base resistance and shaft resistance, respectively
$q^{(m)}_{bu}$, $q^{(m)}_{su}$	ultimate unit pile base resistance and pile shaft resistance, respectively, directly derived from the q_c values in the natural ground conditions
q_c, f_s, Q_{st}	CPT values : cone resistance, local unit side friction, and total side friction, respectively
d	diameter of the CPT sounding tube and/or cone
α_b, ε_b	empirical factors for pile base resistance
α_s, β_s, ε_s, ξ_f, η_p	empirical factors for pile shaft resistance
A_b, X_b, D_b	respectively the cross section, perimeter, and diameter of the pile base
A_s, X_s, D_s	respectively the cross section, perimeter, and diameter of the pile shaft

5.3 *Static load tests*

A clear distinction between design load tests and control load tests is made in Belgium. Because of the proven reliability of in-situ tests based design methods and of the high cost and time consumption of static load tests, loading test piles with a view to design production piles is used only in extreme projects where the large quantity of piles is able to leverage out the benefit of an improved design. The Belgian practice of pile load testing is therefore primarily motivated by the need to control the conformity and intrinsic quality of the piles and should in principle, be discussed in Section 7.2. However because control static load tests also provide assurance of the design and may occasionally warrant its fine-tuning, procedures and interpretation of results are discussed in this section.

The three main documents available in Belgium covering the execution of a static pile load test place the emphasis on the Control type tests. These documents are the Recommendations of the former National Committee on Pile Foundations [49], Draft provisions of the Federal Public Buildings Agency (STS 21) [55], and Specifications of the Flemish Community [50]. A waiting period of 1 to 12 weeks must be respected between the installation of the pile and the execution of a static load test, depending on soil type and pile material. The loading procedure belongs to the maintained load type (ML) and uses the specified load as a reference. The maximum load of 1.5 or 2.0 Q_{SP} is achieved after 6 to 10 load increments, followed by complete unloading achieved after 3 or 4 steps. The loads are maintained for at least half an hour and as long as the pile head settles more than 0.05 mm per half-hour.

The load that is allowed on a pile depends on several criteria relating not only to the amplitude of settlement at given load levels but also to the shape of its load-settlement curve. Alternatively, the load test data can be interpreted to evidence a creep load [54]. The Belgian settlement criteria to assess the reference value of the pile resistance R^m_{ref} on the basis of one measured load-settlement curve are as follows:

$$R^m_{ref} = \min\left(Q^m_{0.010D_b}, \frac{Q^m_{0.017D_b}}{1.35}, \frac{Q^m_{0.025D_b}}{1.70}\right), \quad \text{according to [49],}$$

$$= \min\left(Q_m^{0.0075D_b}, \frac{Q_m^{0.015D_b}}{1.5}\right), \quad \text{according to [55], and}$$

$$= \min\left(Q^m_{3mm}, \frac{Q^m_{6mm}}{1.5}\right), \quad \text{according to [50]}$$

Settlement criteria specified for the pile toe are converted to pile head settlement criteria, allowing for the shortening of the pile shaft (either determined experimentally or calculated from the elastic compression assuming a given transfer of the load to the pile toe). In addition, a pile is not acceptable according to [49] and [55] if its load-settlement curve does not fulfil the following shape criterion: the measured settlement at any load step may not exceed by more than 3 mm the value obtained for the same load step by linear interpolation between any couple of other measured points.

The ultimate load R^m_{cu} is usually assessed based on a 10% settlement criterion for the pile toe. In case that condition cannot not be experimentally evidenced, it is inferred from an extrapolation of the load settlement curve using procedures suggested by Van der Veen [34b] or Chin. The value of R^m_{ca} is deduced either from R^m_{ref} or from R^m_{cu} after taking into account effects not included in the testing situation such as downdrag. In case the testing condition is deemed representative of the service conditions, one usually adopts $R^m_{ca} = R^m_{ref}$.

5.4 Design by calculation on basis of soil ground test results

5.4.1 CPT-related direct method for the ultimate resistance design of compression piles

At least 90 % of all pile design in Belgium is based on semi-empirical formulae, directly assessing both base and shaft resistance of compression piles from CPT data in the natural ground conditions (i.e. before pile installation). The formulae include pile and soil depending empirical factors, which are calibrated based on various research programs (see Section 8). The method itself is conceptually the same for most common pile types (displacement as well as bored piles) and for both non-cohesive and cohesive soils.

It should be noted that in most research work undertaken to date, the simple mechanical cone (M4) or the electrical cone (E1) have been used for the soil investigation, and thus also for calibrating the calculation of the ultimate pile resistance with the measured pile resistance. All empirical factors given hereunder are in principle applicable to M4 or E1 tests only. In practice however, many engineers are unaware of this limitation with the result that in many cases, correction factors for other CPT methods are not considered in the design.

In order to improve that situation, the Belgian NAD is anticipated to establish the electrical cone E1 as the reference cone and to require that data from other cones be transformed to E1-values. Studies on the influence of the CPT method on test results suggest that conversion factors should depend not only on cone type and penetration mode, but also on soil type. $q_{c,M}$ values have been noted to be approximately 35 % higher than $q_{c,E}$ values measured in OC Clay, but have been observed to be approximately 10 % lower than $q_{c,E}$ values measured in sand. Conversion factors for intermediate soils are expected to belong to the range covered by those two soil types. Total side friction (Q_{st}) does not appear to significantly depend on the CPT method.

Basic formulae

The pile ultimate base resistance R_{bu} is deduced from the CPT data by :

$$R_{bu} = \beta \times q_{bu} \times A_b = \beta \times \alpha_b \times \varepsilon_b \times q_{bu}^{(m)} \times A_b \tag{1}$$

with :

β = a shape factor introduced for non circular nor square-shaped bases; (e.g. for barrettes); $\beta = \dfrac{1+0.3B/L}{1.3}$ with B = width and L = length of rectangular base

α_b = an empirical factor taking into account the method of installation of the pile and soil type;

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

$q_{bu}^{(m)}$ = ultimate unit pile base resistance derived from the CPT results in the natural ground conditions

A_b = the nominal pile base cross-sectional area.

Estimation of the ultimate shaft friction R_{su} is based on one of the following CPT values : the total side friction Q_{st} (easiest and most common method); the cone resistance q_c; and/or the local unit side friction f_s.

The total pile shaft resistance R_{su} can be directly evaluated by proportioning the pile shaft resistance to the CPT total side friction increment ΔQ_{st} in the relevant shaft bearing layer(s) :

$$R_{su} = \frac{X_s}{\pi d} \times \xi_f \times \Delta Q_{st} \quad \text{or} \quad = \frac{X_s}{\pi d} \times \Sigma\, \xi_{fi} \times \Delta Q_{sti} \tag{2}$$

with :

ξ_f = an overall empirical factor ($=\alpha_s \,.\, \beta_s \,.\, \varepsilon_s$) introducing the effects of pile installation method (α_s), of the nature of the shaft's material and roughness (β_s) and soil structure scale effects (ε_s);

X_s resp. πd = the perimeter of the pile shaft and of the sounding rod, respectively.

The pile shaft friction can also be evaluated from a semi-empirical correlation between the ultimate unit shaft friction q_{su} and the cone resistance values q_c:

$$q_{su} = \eta_p \times q_c \quad \text{or further detailed as :} \quad q_{su} = \xi_f \times \eta_p^* \times q_c \tag{3a)\&(3b}$$

and thus :

$$R_{su} = X_s \times \Sigma\, H_i \times \eta_{pi} \times q_{ci} = X_s \times \Sigma H_i \times \xi_{fi} \times \eta_{pi}^* \times q_{ci} \tag{4}$$

wherein η_p = an overall empirical factor depending on both soil *and* pile type. For clarity, the correlation $\eta_p = q_{su} / q_c$ can be split into (1) a pure soil parameter η_p^* equal to the ratio of q_c and the average unit side friction $q_{su}^{(m)}$, and (2) a pile/soil dependant empirical factor ξ_f as already defined in equation (2). Values for $q_{su}^{(m)}$ and η_p^* have been suggested for Belgian practice in [48], and are summarised in Table 3 for ease of reference.

A third method relates the pile shaft friction to the directly measured local unit side friction f_s by

$$q_{su} = \alpha_{fs} \times f_s \tag{5}$$

Table 3. $q_{su}^{(m)}$ and η_p^* values (in Eq. 3b and 4) commonly used in Belgian Practice, after [48]

CLAY	q_c [Mpa]	0.075	0.2	0.5	1.0	1.5	2.0	2.5	3.0	≥ 3.0
	$q_{su}^{(m)}$ [kPa]	5	10	18	31	44	58	70	82	q_c[kPa]/36.6
SAND	q_c	≤ 10 MPa		10 < q_c < 20 MPa						> 20 MPa
	$q_{su}^{(m)}$	q_c / 150		Linear interpolation between q_c / 200 and q_c / 150						q_c / 200

Again, one can expect that a_{fs} depends on pile and soil type, and should hence be defined by calibration with static load tests. This method is not widely used for direct pile design (except in the Begemann concept for cyclic loaded piles; See section in §5.4.6.2), because little calibration data or available in Belgium and because experience has revealed the high sensibility of the f_s values from cone type and cone wear.

Calculation of ultimate unit base resistance $q_{bu}^{(m)}$

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

While in foreign countries this scale effect is calculated by rather simple mathematical approaches (smoothing and averaging the q_C-values over a certain range such as in France and The Netherlands, a more analytical method has been developed in Belgium in the 70's by De Beer [15] and then been widely introduced in the Belgian design practice. The method and later modifications have also been reported in ECSMFE and ICSMFE [48] proceedings by De Beer and Van Impe among others. It has been observed that the method aims to predict the limit load ($Q_{0.025Db}$) near the upper and lower boundaries of the bearing layer but provides the conventional rupture load ($Q_{0.10Db}$) at large depths in that layer.

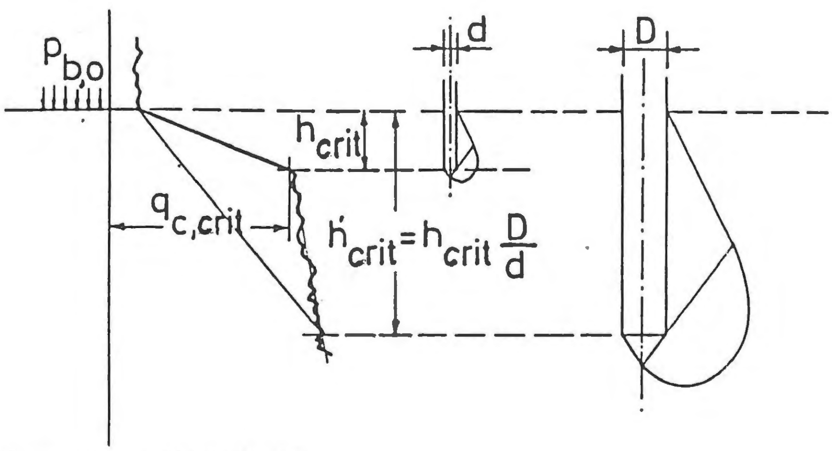


Figure 2. Scale effect principle

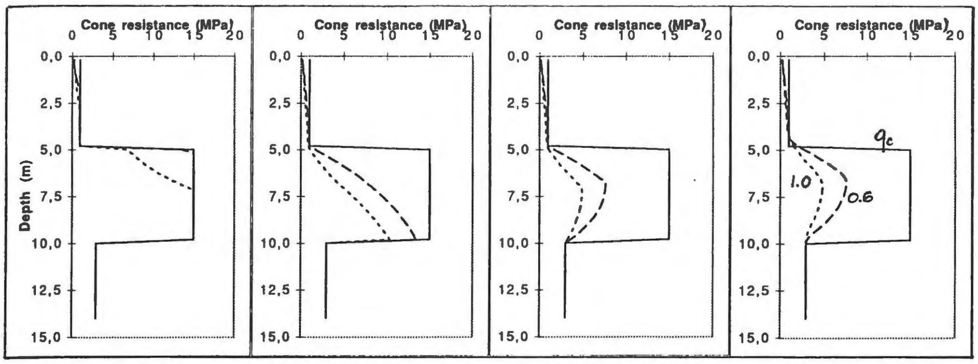


Figure 3. Step by step illustration of De Beer procedure : (a) homogeneous values, (b) downward values, (c) upward values and (d) blended values; for 0.6 and 1.0 m diameter base, respectively.

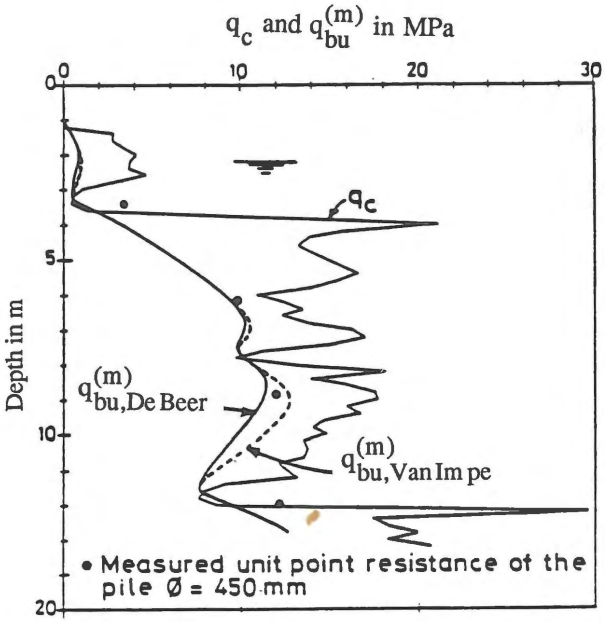


Figure 4. Comparison of calculated $q_{bu}^{(m)}$ with experimental data

The De Beer method is based on a thorough application of the principles of the scale effect, when transitioning from a soft to a hard soil layer as shown in Figure 2.

This application of the scale effect is done in 4 steps, designated by the terms (a) homogeneous values, (b) descending or downward values, (c) upward values and (d) mixed or blended values. These final mixed values $q_{bu}^{(m)}$ are the basis values for the further base resistance calculation of the pile. To demonstrate the procedure, step by step results of a De Beer calculation are given in figure 3 for a simplified soil profile. A practical calculation example is provided in figure 4, showing $q_{bu}^{(m)}$ for a given displacement pile, as calculated according to the basic De Beer method as well as the slightly modified method as proposed by Van Impe and as measured at several depths. The example design provided in section 6 provides an other illustration of the De Beer method [15]

Installation factors in design formulae

Installation empirical factors for a given pile in a given soil type should be deduced from calibration with static pile load tests. It is most often assumed in Belgian specifications, that for traditional piles of the displacement type, all empirical factors = 1.0, so that :

$$R_{cu} = R_{bu} + R_{su} = q_{bu}^{(m)} \times A_b + \Delta Q_{st} \times \frac{X_s}{\pi d} \tag{6}$$

More refined factors are however discussed hereafter.

The ε_b factor in equation (1) has been introduced to take into account the scale effect of the size of the failure mechanism of a pile base relative to the failure mechanism of the CPT cone, as recognised in the stiff fissured OC Boom clay [16]. Although size dependent shear resistance might also exist in other soil types, that phenomenon has been explicitly introduced only in stiff OC clay, where, one applies in Belgium :

$$0.476 \le \varepsilon_b \approx 1 - 0.01\,(D_b / d - 1) \tag{7}$$

Table 4. Commonly used α_b factors (in equation (1)) for various pile types

Pile type	α_b factor for sand	α_b factor for stiff OC clay
Group I - High soil displacement		
Impact driven piles	0.8-1.15$^{(1)}$	0.8-1.0$^{(1)}$
Screwed piles	0.8 -1.0$^{(2)}$	1.0
Jacked piles (smooth)	1.0	1.0
Group II - Low soil displacement or low relaxation		
Impact driven	see [20,21&46]	see [20,21 & 46]
Drilled with special provisions	0.6-0.8	0.8
Group III - Soil excavation		
Cast-in-situ bored piles (large diameter and CFA)	0.33-0.67	0.8

$^{(1)}$: highest value for expanded base with semi-dry concrete only; for cast-in-situ with plastic concrete, function of the diameter of the bottom plate relative to diameter of driving tube;
$^{(2)}$: depending on the allowance or not for vertical soil displacement near the pile base

Table 5. Commonly used ξ_f factors (in Eq. (2) and (4)) for various pile types and shaft material

Pile type	ξ_f factor for sand	ξ_f factor for stiff OC clay
Group I - High soil displacement		
Shaft in compacted semi-dry concrete	1.6	1.15
Shaft in plastic concrete or prefabricated concrete	0.8-1.0$^{(1)}$	0.65-1.0
Screwed piles - plastic concrete	0.8-1.25$^{(2)}$	0.8-1.25$^{(2)}$
Steel shaft	0.6	0.45-0.65
Group II - Low soil displacement or low relaxation		
Impact driven - steel shaft	see [20,21 & 46]	see [20,21 & 46]
Drilled with special provisions - wet concrete	0.6-0.8	0.65-0.85
Group III - Soil excavation		
Cast-in-situ bored piles (large diameter and CFA)	0.4 - 0.6	0.5

$^{(1)}$: for cast-in-situ with plastic concrete, function of diameter of the bottom plate relative to diameter of driving tube;
$^{(2)}$: highest values for screw shaped shaft.

The α_b, ξ_f and η_p factors in equations (1), (2), and (4) have been deduced from static pile load tests in various research projects (see Section 8). Tables 4 and 5 are giving an overview of values commonly used in the Belgian design practice for sand and stiff OC Clay, Intermediate values between those listed are adopted for intermediate soil types. For certain pile types, installation factors that have not yet been calibrated, based on an objectively conducted full scale load test program, require the input of some judgement and are therefore the subject of some debate. For the η_p factors, Van Impe [42] has summarised the values resulting from Belgian research work. In some cases, the factors prescribed in e.g. the Dutch code (NEN 6743) or the French regulations [52, 54] are applied.

Extended research work, on the other hand, has been performed on the bearing resistance of steel H-beam piles, with or without base or shaft enlargements [21,21a & 46]. That research indicates that basic formulae similar to formula (1) for the base resistance and to formula (2) or (4) for the shaft resistance can be used. However, the design is based on the most conservative of two possible rupture models : (1) the H-beam penetrates into the soil like a knife, without any plug formation; and (2) a partial plug (in granular soils) or a full plug (in cohesive soils) is formed in the space between the flanges of the H-section.

5.4.2 Other methods for ultimate resistance design of piles on basis of in situ ground tests

Beside the widely used direct method on basis of CPT results, as detailed above, other methods are occasionally used in Belgium. CPT-based design is sometimes performed according to codes or recommendations from neighbouring countries, (e.g. Dutch codes NEN6740 and NEN6743 and French DTU 13.2 or Fascicule 62). PMT-based design most likely follows the French methods (DTU 13.2 or Fascicule 62) as well. When dynamic penetration tests (DPT) or standard penetration tests (SPT) have been used, the geotechnical engineer generally transforms the test data into more familiar CPT values to apply the methods detailed in Section 5.4.1, but may occasionally refer to Bustamante (1993).

Micropiles are most commonly designed according to the LCPC method, published by Bustamante e.a. [3]. As the design charts are elaborated in terms of PMT values, CPT q_c data are converted into limit pressure p_l values. The conversion is often based on the comparative research work by Van Wambeke [44], and may be simplified into a 3-6-9 rule :

$q_c / p_\ell \approx 3.0$ for clays, ≈ 6.0 for silts, and ≈ 9.0 for sands.

Although currently being used in Belgium for e.g. tension piles, very little experimental data is available for *post-grouted* piles. Design most likely is performed using the French recommendations [52,54].

5.4.3 Design methods based on laboratory tests

Calculations of pile bearing resistance on basis of shear parameters (ϕ, c or c_u) using static formulae are usually not used. Their exceptional use is limited to the calculation or verification of the shaft resistance and in that particular case, for instance :

- to define a lower limit of the shaft resistance for bored piles in granular soils;
- to calculate the shaft friction in cases where the in situ performed soil tests are insufficient, or doubtful, or where they might not be representative of the soil conditions (e.g. in case of deep excavations after soil testing);
- to calculate the shaft friction of tension piles (see below).

5.4.4 Design methods for tension piles on basis of ground test results

For straight-sided tension piles, the uplift capacity (or tensile resistance) R_{tu} is mostly calculated

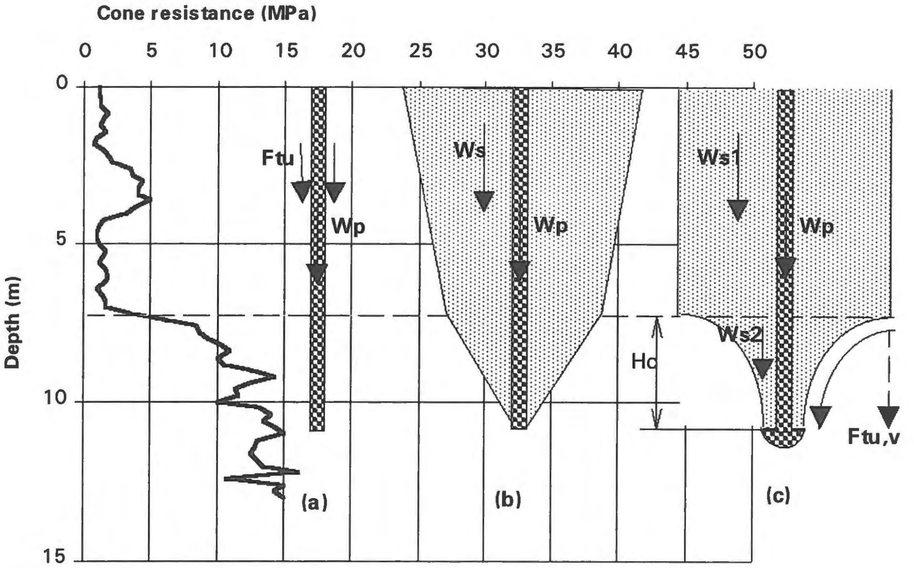


Figure 5. Pile uplift failure mechanisms

by assuming a slip failure along the shaft-soil interface (fig. 5a). The shear resistance along this interface is generally calculated by direct methods on basis of the CPT data, applying the same formulae and factors as those detailed above for the friction on compression piles. An occasional alternative consists in applying a capped Mohr-Coulomb failure criterion assuming drained and/or undrained soil conditions. In case of small D_S/L_p ratios, the uplift of the block of ground co-operating with the pile, has also to be checked (see figure 5b).

For piles which are strongly embedded at their lower end, the governing failure mechanism will be rather similar to that of a plate anchor. In this case, an internal slip line will occur, at least over the depth of embedment into the bearing layer. Conditions for such a slip line are generally fulfilled for piles with cast in situ expanded base (e.g. Franki type) and for piles installed with lateral displacement of the soil (e.g. helicoidal screw piles), both conditions being combined with a rough shaft-soil interface.
For granular soils, the tension resistance of such a piles is often calculated on basis of the analytical methods, elaborated by Lousberg e.a. [29]. The assumed slip line is given in figure 5c. The height of the trumpet shaped slip line along which the shear resistance is taken into account is limited to the minimum 4 D_b and the embedment height H_c into the dense layer.

5.4.5 Particular load cases

Downdrag

The downdrag F_n generally is calculated following the method of Zeevaert [47], further detailed by De Beer [11]. For a given critical height h_c over which the downdrag is estimated to occur, the downdrag caused by a surcharge p_o on a pile with perimeter X_s and an area of influence A amounts to : $F_n = F_{n,o} + F_{n,\gamma} = A_o p_o \left(1 - e^{-m_o h_c}\right) + A_\gamma \gamma_k h_c \left(1 - \frac{1 - e^{-m_\gamma h_c}}{m_\gamma h_c}\right)$

with : $m_o = \frac{K_o \tan\phi X_s}{A_o}$ and $m_\gamma = \frac{K_o \tan\phi X_s}{A_\gamma}$

The influence areas A_O and A_γ are calculated following the hypothesis of influence cones that extend over a top area of $A_O = \pi h_c^2 / 4$ and $A_\gamma = \pi h_c^2 / 16$ respectively.

Cyclic loading

Particularly for piles supporting pylons for high voltage lines, specifications often require a verification of the side friction on the piles under cyclic loading by the method suggested by Begemann (1969). The side friction resistance is then calculated on basis of the local friction as directly measured in the CPT, following formula (5) : $q_{su} = \alpha_{fs} \times f_s$. On the other hand, it is proposed by Begemann [2] for alternating loading (compression/tension), to reduce the friction over the middle half of the embedded length of the shaft by a factor of 3.0.

5.5 *Driving Formulae*

In Belgium, piles are not designed based on driving formulae. However the final blow counts can be used to check and fine-tune the penetration required by design as follows.

During driving the first pile at the very location or at least in the close vicinity of a CPT test, the set is measured at the proposed level. This set is then imposed within a narrow margin (typically 20 %) when driving the neighbouring piles. Driving is thus to be continued until each pile is placed in the same layer as the test pile and at such a depth that the set criterion is fulfilled. For calculating the set (or penetration per blow), the mean value over the last 10 or 25 cm or the mean value of 5 consecutive observations of 10 blows is taken.

For Franki-type piles, the set recorded at base level governs the volume of dry concreted required to form the expanded base.

5.6 *Wave equation analysis*

The deduction of the static bearing capacity from dynamic measurements is still considered by several Belgian engineers as controversial because of two considerations: (1) the dynamic loading behaviour is not necessarily representative of the static one, and (2) the displacement induced during driving or dynamic testing is much smaller than what is recognised to yield significant data about the ultimate bearing capacity of a pile. Development of pile dynamic testing in Belgium is however further discussed in Sections 7 and 8.

5.7 *Factors of Safety*

5.7.1 General Concept

The factor of safety that establishes the ratio between the calculated ultimate bearing capacity and the allowable load is meant to cover a sufficient margin of safety with regards to failure but also encompasses uncertainties attached to e.g. subsurface conditions, calculation methods, quality of the piles, and actual working load. In the Belgian practice, factors of safety are applied within a deterministic framework, i.e. they are globally applied to components of the bearing capacity.

When bearing capacity is evaluated from CPT tests, Belgian engineers generally use a factor of safety of 2 on the end bearing term and a factor of safety of 3 on the shaft resistance. In the case of a PMT based evaluation, French prescriptions are followed. There exists a current trend to assign identical factors of safety to all pile types after allowing installation coefficients to account for differences in the ultimate bearing capacity between different pile types. The working load of piles is generally assessed based on the most conservative CPT test within a given zone of the site, thereby neglecting the favourable effects due to load redistribution amongst piles and discounting the design advantage offered by a potentially more extensive geotechnical investigation.

5.7.2 Factors of safety with respect to ultimate bearing resistance

In the deterministic method, global factors of safety are used :

$R^c_{ca} = R_{bu} / S_b + R_{su} / S_s$ with : $S_b = 2.0$ and $S_s = 3.0$

One verifies that : R^c_{ca} (for all CPT's individually) $\geq Q_n$

In the case a downdrag $F_n \leq R_{bu}$ is expected, one generally uses :

$$R^c_{ca} = \frac{R_{bu} - F_n}{S_b} + \frac{R^+_{su}}{S_s} \text{ with}$$

R^+_{su} : pile shaft resistance accrued below the neutral point.

$R^c_{ta} = W'_p / S_w + F_{su} / S_s$ with : $S_W = 1.0$ to 1.3

and $S_S = 3.0$ to 5.0, depending on project type

One verifies that : R^c_{ta} (for all CPT's individually) $\geq Q_n\uparrow$

Alternatively, De Beer e.a. [19] suggested for displacement piles, using the results of several CPT's within a design zone:

$$R_{ca,1} = \frac{1}{\gamma_t}\left[\frac{R_{bu,\max}}{S'_b} + \frac{R_{su,\max}}{S'_s}\right] \quad \text{with : } \gamma_t = 1.4,\ S'_b = 1.5 \text{ and } S'_s = 1.3 \text{ and } R^c_{ca} = R_{ca,1} \leq R_{ca,2}$$

$$R_{ca,2} = \frac{1}{\gamma_t}\left[R_{bu,\min} + R_{su,\min}\right]$$

That approach has been extended by Van Impe (1986) who suggested $\gamma_t = 1.7$ for bored piles and a statistical determination of S'_b and S'_s .

Partial factoring is still to be decided in the Belgian NAD.

5.7.3 Structural Safety

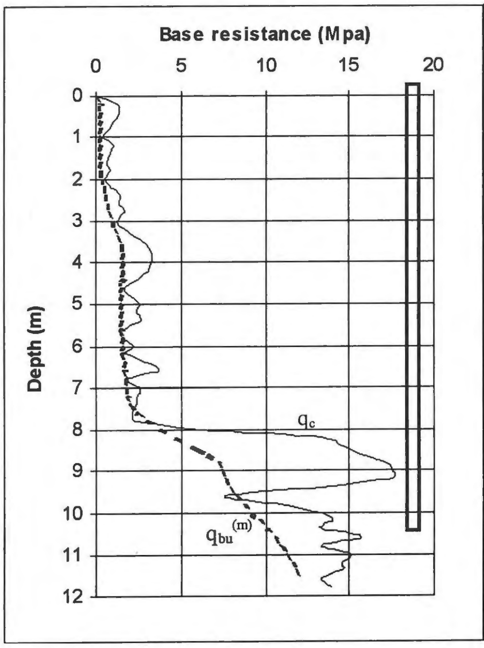
Belgian norms available for the design of structural elements made of reinforced concrete are not applied to design pile shafts. It is the practice to conduct that calculation on the basis of an equivalent composite cross section equal to the concrete section plus 14 times the steel cross sectional area and characterised by an allowable compressive stress. That allowable compressive stress varies little between current specifications (typically 5 to 7 MPa), but is always lower than that allowed to design reinforced concrete columns, on the grounds that the quality of concrete can not be enforced and exposed as easily.

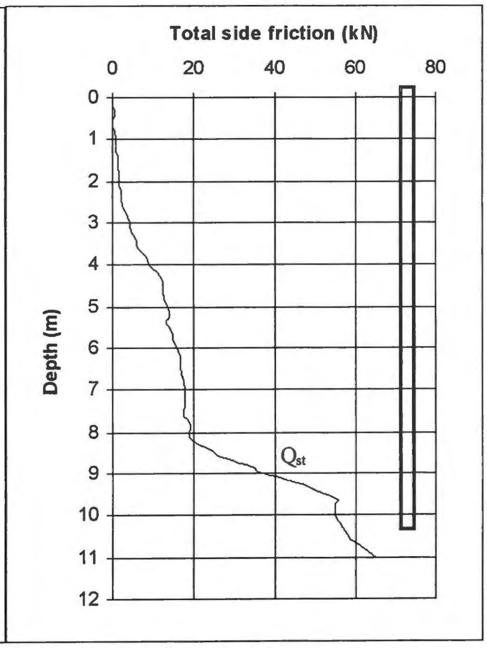
5.8 Serviceability

In regular present Belgian practice, serviceability conditions are not usually explicitly analysed. Experience has indeed shown that for piles designed under usual conditions and for regular structures, the factors of safety indicated in Section 5.7 are conservative enough to satisfy the service states. A settlement analysis is explicitly performed however for pile groups located above potentially compressible layers, for settlement sensitive structures, or for marginal and challenging subsurface conditions. That analysis is conducted first for a single pile using mobilisation curves for the shaft and the end bearing reactions derived from pile load tests performed under similar conditions, and later refined using the results of control load tests performed on the site. Alternatively, mobilisation curves can be derived from the results of CPT tests, as suggested by Verbrugge [56]. Settlement of the pile group is then evaluated using stress distribution theories (e.g. Buisman) and a linear or logarithmic stress-strain relationship for the soil.

6 PARTICULAR EXAMPLE

The following example illustrates the widely used procedure described in Section 5.4.1.
Pile : Cast-in-situ pile (group I type C), wet concrete shaft, D = 520 mm
Pile base depth : 10.5 m
Soil : Loam (0 m - 8 m) and sand (8 m - 12 m)
CPT : M4





Pile base resistance	Pile shaft resistance
$q_{bu}^{(m)}$ = 10.3 MPa (De Beer method)	ΔQ_{st} = 55 kN
$\alpha_b = 1.0$ $\varepsilon_b = 1.0$	$\xi_f = 1.0$
$R_{bu} = 1.0 \times 1.0 \times \frac{\pi \times 0.52^2}{4} \times 10300 = 2184$ kN	$R_{su} = 1.0 \times 55 \times \frac{0.520}{0.036} = 801$ kN
$S_b = 2$	$S_s = 3$
$\mathbf{R_{ca} = R_{bu}/S_b + R_{su}/S_s = 1.09 + 0.27 = 1.36}$ **MN**	

7. QUALITY CONTROL AND MONITORING

7.1 Monitoring of pile installation

In addition to controlling the quality of the used materials such as concrete and reinforcement, one can also monitor the different execution parameters during the installation of the pile. To ensure the reliability of the monitoring, some basic data is always recorded, such as date and time of installation, co-ordinates of the pile location, etc. The following paragraphs address specific aspects of the monitoring of impact driven piles on one hand and drilled and screwed piles on the other hand.

The blow count which is normally recorded against the penetration of the pile by the piling foreman can also be registered by a monitoring device. The monitoring of the end of driving is discussed in section 5.5. Monitoring devices (such as Pile Dynamic Analyser (PDA) or blow count recorders versus depth) are not currently specified, although some piling contractors are equipped with those systems.

Drilled and screwed piles are monitored with regards to the drilling or screwing process as well as the concreting process. Depending on the pile system and monitoring system, several of the following parameters are generally recorded : speed of penetration, speed of rotation, depth, rotational torque (usually inferred from the hydraulic oil pressure of the drill table). The concreting which is most often performed using a pump must be controlled by a monitoring device measuring the volume of the used concrete, the pressure applied to the concrete and the pull-out speed. For some pile types used in Belgium, a computer-based monitoring system has already been implemented. When demonstrated to be reliable, that type of monitoring is asked more and more often by the quality control department of owners and consultants.

Soil relaxation resulting from the installation process can be evaluated on the basis of soundings performed alongside the pile.

7.2 Visual inspection, Static Loading Testing, and Core Sampling

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

In spite of its rare use, the static loading test is still in the Belgium the least disputed method to test the integrity and to verify the bearing capacity, as discussed in more detail in section 5.3. In the case of a control test, a pile is deemed satisfactory when $R_{ref}^{m} \geq Q_{SP}$. In the event that condition is not fulfilled, further analysis by the engineer and negotiations between the contracting parties ensue, based on the value of R_{ca}^{m} or R_{cu}^{m} evidenced form the static load test.

Especially for bored piles, vertical core sampling is sometimes carried out. The sampling provides a continuous control of the quality of the concrete in the pile shaft. Continuing the sampling through and beyond the toe of the pile allows one to examine the tightness of the contact between the base of the pile and the bearing soil layer.

7.3 Non destructive tests

Information on pile integrity is obtained using sonic or gamma-gamma logging, the echo method, and the mechanical admittance method.

Sonic and gamma-gamma logging is usually performed on bored piles using access tubes mounted on the reinforced cage to evaluate the quality of concrete between emitter and receiver.

Depending on the extent and success of the testing program [23], the evaluations expected from the sonic echo and the mechanical impedance methods may include the length of the pile, its cross-section, the extent to which these dimensions vary, the density of the concrete, the

Table 6. Belgian Pile Testing Experience - Part 1: Programs

Test Sites				Summarized soil configuration				Program information		
Abrev.	Name	Reference	Period of tests	Depth (m)	Nature of upper stratum	Depth (m)	Nature of lower stratum(s)	Num -ber	and Type of piles tested	Objectives - Results
ZEL	Zelzate	[12]	1968	0 - 21	loose loamy sand	21 - 26	Boom-clay	1	Tension bored	Skin friction resistance under tension
ANT I	Antwerp	[12, 48]	1968	0 - 11	loose sand	> 11	Boom-clay	5	Tension driven cast-in-situ	Skin friction resistance and bulb effect under tension
ANT II	Antwerp (North)	[48]	?	0 - 8	soft clay and peat, and loose sand	> 8	dense sand	1	Tension driven cast-in-situ	Skin friction resistance and bulb effect under tension
OST	Ostend	[48]	?	0 - 13	soft clay and peat, locally sandy	> 13	dense sand	1	Tension driven cast-in-situ	Skin friction resistance and bulb effect under tension
ZWI	Zwijnaarde (Ghent)	[14, 45, 48]	1969/70	0 - 13	loose sand to sandy silt	> 13	Medium dense sand	4	Driven cast-in-situ	Scale effect for base resistance (medium dense sand)
KO I	Kontich I	[8, 16, 48]	1975/76	0 - 3	sandy loam (silt)	> 3	tertiary Boom-clay	12	Driven cast-in-situ, and prefabricated	Scale effect for base resistance (fissured clay), shaft friction
KA I	Kallo I	[17, 18, 48]	1977/78	0 - 8	soft clay and peat	> 8	Dense sand	7	Driven cast-in-situ	Scale effect for base resistance (dense sand)
KA II	Kallo II	[20, 21, 48]	1977/81	0 - 8	soft clay and peat	> 8	Dense sand	12	Driven H steel pile	Plugging, influence of pile length, enlargement in sand, prediction by stress wave analysis.
KO II	Kontich II	[20, 21, 48]	1977/81	0 - 3	sandy loam (silt)	> 3	tertiary Boom-clay	12	Driven H steel pile	Plugging, influence of pile length, enlargement in clay, prediction by stress wave analysis.
KA III	Kallo III	[38, 48]	1982	0 - 6	soft clay and peat	> 6	Dense sand	4	Bored and driven steel tube	Comparison bored and driven piles in sand (all same geometry)
GRB	Groot-Bijgaarden	[48]	1983/85	0 - 8	sandy loam and clay	> 8	Fine sand/ dense sand	6	Driven cast-in-situ	Influence of base plate enlargement on base resistance
ZWE	Zwevegem (Kortrijk)	[29]	1984	0 - 15	tertiary Ieper-clay	> 15	compact sand	2	Screwed (auger) cast-in-situ	Design of cast-in-situ displacement screwed piles in clay
GH I	Ghent I	[40]	1985	0 - 14	silt & silty clay	> 14	Dense clayey sand	2	Screwed (auger) cast-in-situ	Design of cast-in-situ displacement screwed piles in sand
KA IV	Kallo IV	[21a]	1986	0 - 5	soft clay	>5	Dense sand	3	Driven H steel pile	Influence of geometry and position of lagging, prediction by stress wave analysis.
GH II	Ghent II	[23, 26, 30, 36, 37, 38, 40, 41]	1987	0-10	silt & silty clay	> 10	Dense clayey sand	12	Screwed (auger) cast-in-situ, CFA & driven precast concrete	Comparison different screwed and CFA piles in sand. Prediction by dynamic loading tests.
GEE	Geel	[32]	1988	0 - 8	loose sand, silty sand	> 8	sand	3	CFA	Design of CFA piles- Use of DMT
NSC	North Sea Coast	[24]	1990	0 - 10	soft clay	10 - 16	clayey sand	2	Screwed (auger) cast-in-situ	Design of cast-in-situ displacement screwed piles in sand. Prediction by dynamic loading test
KOE	Koekelare	[4, 22]	1992	0 - 5	silty/clayey sand	> 5	tertiary Ypresian clay	10	Screwed (auger) cast-in-situ and screwed steel tube	Design displacement screwed piles in clay. Prediction by dynamic loading test. Use of DMT
LIM	Limelette	[28]	1995-96	0 - 9	loam	> 9	dense sand	4	Driven (cast-in-situ, steel tube, precast concrete)	Comparison of driving techniques in sand; induced vibrations, prediction by dynamic loading test.
VIL	Vilvoorde	[7]	1995	0-7.5	sandy silt	> 7.5	sand	3	Screwed cast-in-situ Omega and driven precast concrete	Design of Omega-piles, comparison with precast

Table 6 (continued). Belgian Piles Testing Experience - Part 2: Piles data (1)

Test site	Test pile	Description	C, T (1)	Ø Shaft (m)	Ø base (m)	Base depth	Q_{max} (MN)	S_{max} (mm)	Q_{dyn} (MN)
ZEL	1	Bored with bucket with casing & under bentonite	T	0.80	0.80	26.4	2.90	40.66	nr
ANT I	B	Franki (normal expanded base, rammed shaft)	T	0.45	> 0.45	8.3	0.96	3.98	nr
ANT I	C	Franki (normal expanded base, rammed shaft)	T	0.36	> 0.36	8.3	0.90	6.24	nr
ANT I	D	Franki (normal expanded base, rammed shaft)	T	0.45	0.45	8.3	0.90	3.94	nr
ANT I	E	Franki (normal expanded base, rammed shaft)	T	0.36	0.45	8.3	0.90	6.28	nr
ANT I	G	Steel tube pile	T	0.32	0.32	8.3	0.90	17.63	nr
ANT II	1	Franki (normal expanded base, rammed shaft)	T	0.52	0.72	10.1	2.48	57.00	nr
OST	1	Franki (normal expanded base, rammed shaft)	T	0.52	0.72	10.1	3.04	35.00	nr
GRB	1	Vibrex pile + very enlarged steel plate	C	0.52	0.72	20.0	2.44	11.49	nr
GRB	2	Vibrex pile + normal steel plate	C	0.52	0.52	10.0	2.44	64.95	nr
GRB	3	Vibrex pile + normal enlarged steel plate	C	0.52	0.62	10.0	2.20	66.09	nr
GRB	4	Super-Vibrex pile + steel plate + cast in-situ enlarged base	C	0.52	0.52	10.0	2.44	30.14	nr
GRB	5	Vibrex pile + very enlarged steel plate	C	0.52	0.72	10.0	2.20	72.47	nr
GRB	6	Closed-end steel tube pile	C	0.51	0.52	10.0	1.46	55.15	nr
ZWE	1	Atlas screw (auger) cast-in-situ pile	C	0.36/0.46	0.46	12.7	1.58	12.00	nr
ZWE	2	Atlas screw (auger) cast-in-situ pile	C	0.36/0.46	0.46	12.7	1.76	36.00	nr
GH I	1	Atlas screw (auger) cast-in-situ pile	C	0.36/0.46	0.46	13.5	2.76	75.00	nr
GH I	2	Atlas screw (auger) cast-in-situ pile	C	0.36/0.46	0.46	13.5	3.00	75.00	nr
KA IV	1	H Steel without lagging	C	0.36x0.41	0.36x0.41	14	3.50	112.00	2.78
KA IV	2	H Steel with lagging near the toe	C	0.36x0.41	1.08x0.41	14	4.35	110.00	3.51
KA IV	3	H Steel with lagging and plate near the toe	C	0.36x0.41	1.08x0.41	14	5.50	167.00	4.97
GH II	3	Fundex screw (auger) cast-in-situ pile	C	0.38	0.45	13.0	2.04	14.00	nr
GH II	5	Continuous flight auger cast-in-situ pile	C	0.45	0.45	14.5	1.72	15.00	nr
GH II	6	Atlas screw (auger) cast-in-situ pile	C	0.36/0.46	0.46	12.5	2.01	13.00	nr
GH II	8	Fundex screw (auger) cast-in-situ pile	C	0.38	0.45	13.0	1.86	14.00	nr
GH II	9	Continuous flight auger cast-in-situ pile	C	0.45	0.45	14.5	1.62	15.00	nr
GH II	10	Atlas screw (auger) cast-in-situ pile	C	0.36/0.46	0.46	12.5	2.02	13.00	nr
GH II	11	Driven precast concrete pile	C	0.32x0.32	0.32x0.32	13.3	2.80	16.00	nr
GH II	12	Fundex screw (auger) cast-in-situ pile	C	0.38	0.45	13.0	nr	nr	1.81..2.60
GH II	13	Continuous flight auger cast-in-situ pile	C	0.45	0.45	14.5	nr	nr	0.87..1.66
GH II	14	Atlas screw (auger) cast-in-situ pile	C	0.36/0.46	0.46	13.0	nr	nr	1.24..2.60
GH II	15	Driven precast concrete pile	C	0.32x0.32	0.32x0.32	13.4	nr	nr	1.64..2.80
NSC	1	Atlas screw (auger) cast-in-situ pile	C	0.36/0.46	0.46	12.0	0.68	3.30	1.25
GEE	33	Continuous flight auger cast-in-situ pile	C	0.35	0.35	10.5	1.78	2.67	nr
GEE	55	Continuous flight auger cast-in-situ pile	C	0.35	0.35	10.5	1.55	1.97	nr
GEE	99	Continuous flight auger cast-in-situ pile	C	0.35	0.35	10.5	1.39	6.27	nr
KOE	1	Steel tube screw (auger) pile	C	0.35	0.65	13.0	nr	nr	nr
KOE	2	Atlas screw (auger) cast-in-situ pile	C	0.36/0.50	0.50	13.0	nr	nr	nr
KOE	3	Atlas screw (auger) cast-in-situ pile	C	0.51/0.65	0.65	13.0	nr	nr	nr
KOE	8	Steel tube screw (auger) pile	C	0.35	0.65	13.0	1.30	80.03	nr
KOE	9	Atlas screw (auger) cast-in-situ pile	C	0.36/0.50	0.50	13.0	1.95	60.04	nr
KOE	10	Atlas screw (auger) cast-in-situ pile	C	0.51/0.65	0.65	13.0	2.50	64.32	nr
KOE	15	Atlas screw (auger) cast-in-situ pile	C	0.36/0.50	0.50	13.0	2.05	26.04	nr
KOE	16	Atlas screw (auger) cast-in-situ pile	C	0.51/0.65	0.65	13.0	2.35	29.63	nr
KOE	21	Atlas screw (auger) cast-in-situ pile	T	0.36/0.50	0.50	13.0	nr	nr	?
KOE	23	Atlas screw (auger) cast-in-situ pile	T	0.51/0.65	0.65	13.0	nr	nr	?
LIM	5	Driven steel tube	C	0.30	0.30	9.5	1.22	40.64	nr
LIM	8	Driven precast concrete pile	C	0.29x0.29	0.29x0.29	9.5	1.74	34.42	nr
LIM	12	Driven cast-in-situ with steel plate	C	0.40	0.40	9.5	2.82	52.39	nr
LIM	16	Driven cast-in-situ with enlarged base (Franki)	C	0.40	0.52	9.5	3.07	174.83	nr
VIL	3	Screwed cast-in-situ Omega	C	0.41	0.41	14.0	2.08	18.66	nr
VIL	4	Driven precast concrete pile	C	0.35	0.35	7.2	1.1	76.6	nr
VIL	5	Screwed cast-in-situ Omega	C	0.41	0.41	14	1.9	55.93	nr

(1) C = compression, T = tension

Table 6 (continued). Belgian Piles Testing Experience - Part 2: Piles data (2)									
Test site	**Test pile**	**Description**	**C, T (1)**	**Ø Shaft (m)**	**Ø base (m)**	**Base depth (m)**	**Q_{max} (MN)**	**S_{max} (mm)**	**Q_{dyn} (MN)**
ZWI	3	Franki (overexpanded base, shaft friction eliminated)	C	nr	1.43	12.3	3.60	13.40	nr
ZWI	4	Franki (overexpanded base, rammed shaft)	C	0.52	1.47	12.3	3.60	9.10	nr
ZWI	5	Franki (overexpanded base, no shaft friction)	C	nr	1.58	7.3	2.38	51.00	nr
ZWI	6	Franki (overexpanded base, rammed shaft)	C	0.52	1.33	7.3	3.58	46.00	nr
KO I	1	Franki (overexpanded base, shaft friction eliminated)	C	nr	1.40	11.4	2.16	133.99	nr
KO I	2	Franki (overexpanded base, vibrated shaft)	C	0.41	1.45	11.4	3.12	211.85	nr
KO I	3	Franki (overexpanded base, rammed shaft)	C	0.41	1.45	11.4	3.44	160.06	nr
KO I	4	Franki (overexpanded base, vibrated shaft)	C	0.41	1.45	11.4	3.28	173.40	nr
KO I	5	Franki (overexpanded base, shaft friction eliminated)	C	nr	1.40	11.4	2.32	94.85	nr
KO I	6	Franki (normal expanded base, shaft friction eliminated)	C	nr	0.62	10.6	0.62	105.64	nr
KO I	7	Franki (normal expanded base, vibrated shaft)	C	0.41	0.64	10.6	1.52	40.60	nr
KO I	8	Franki (normal expanded base, rammed shaft)	C	0.41	0.64	10.6	2.16	192.37	nr
KO I	9	Vibrated shaft, steel plate	C	0.41	0.41	10.0	0.88	184.37	nr
KO I	10	Franki (no expanded base, rammed shaft)	C	0.41	0.41	10.0	1.20	198.69	nr
KO I	11	Driven prefabricated concrete pile	C	0.41	0.41	10.0	1.04	173.13	nr
KO I	12	Driven steel tube pile (closed end)	C	0.41	0.41	10.0	0.72	189.40	nr
KA I	1	Franki (normal expanded base, vibrated shaft)	C	0.52	0.90	9.7	6.18	96.39	nr
KA I	2	Franki (normal expanded base, shaft friction eliminated)	C	nr	0.54	9.7	2.87	103.33	nr
KA I	3	Franki (normal expanded base, vibrated shaft)	C	0.41	0.62	9.8	3.35	110.61	nr
KA I	4	Franki (normal expanded base, shaft friction eliminated)	C	nr	0.82	9.8	5.21	84.30	nr
KA I	5	Driven steel tube pile (closed end)	C	0.41	0.41	9.3	1.80	68.02	nr
KA I	6	Driven steel tube pile (closed end), enlargement 1.6m above basis, 1.6m length	C	0.41	0.41	11.4	4.74	81.35	nr
KA I	7	Vibrated shaft, enlarged steel plate at the basis	C	0.41	0.54	9.4	2.87	68.07	nr
KA III	A	Driven steel tube pile (closed end)	C	0.60	0.60	11.0	5.45	263.00	nr
KA III	B	Driven steel tube pile (closed end)	C	0.60	0.60	11.0	6.00	300.00	nr
KA III	C	Bored with bucket under bentonite	C	0.60	0.60	11.0	5.10	320.00	nr
KA III	D	Bored with bucket under bentonite	C	0.60	0.60	11.0	4.30	290.00	nr
KO II	1	H steel pile without lagging	C	0.39x 0.38	0.39x 0.38	18.0	2.28	48.00	2.11
KO II	2	H steel pile with local constant enlargement near the toe	C	0.39x 0.38	0.81x0.38	14.5	1.52	97.00	1.85
KO II	3	H steel pile with local variable enlargement near the toe	C	0.39x 0.38	0.81x0.38	15.5	nr	nr	1.62
KO II	4	H steel pile with local variable enlargement near the toe	C	0.39x 0.38	0.81x0.38	19.0	nr	nr	1.17
KO II	5	H steel pile with enlargement (plate) near the toe	C	0.39x 0.38	0.55x0.55	19.3	nr	nr	1.51
KO II	6	H steel pile with local variable enlargement near the toe	C	0.39x 0.38	0.80x0.80	14.5	nr	nr	1.50
KO II	7	H steel pile with local variable enlargement near the toe	C	0.39x 0.38	0.39x 0.38	19.0	nr	nr	0.50
KO II	8	H steel pile without lagging	C	0.39x 0.38	0.39x 0.38	50.0	6.51	119.00	6.85
KO II	9	H steel pile with steel plate at the bottom	C	0.39x 0.38	0.39x 0.38	18.5	nr	nr	0.40
KO II	10	H steel pile with steel plate near the bottom	C	0.39x 0.38	0.39x 0.38	18.5	nr	nr	0.31
KO II	11	H steel pile with local constant enlargement near the toe	C	0.39x 0.38	1.13x0.39	18.0	2.75	94.00	3.86
KO II	12	H steel pile with local constant enlargement near the toe	C	0.39x 0.38	1.13x0.39	18.5	nr	nr	3.20
KA II	1	H steel pile without lagging	C	0.39x 0.38	0.39x 0.38	18.0	3.10	13.50	3.09
KA II	2	H steel pile with local constant enlargement near the toe	C	0.39x 0.38	0.39x 0.38	15.0	6.50	41.00	6.54
KA II	3	H steel pile with local variable enlargement near the toe	C	0.39x 0.38	0.81x0.38	14.5	5.30	57.00	4.56
KA II	4	H steel pile with local variable enlargement near the toe	C	0.39x 0.38	0.81x0.38	18.0	nr	nr	3.49
KA II	5	H steel pile with enlargement (plate) near the toe	C	0.39x 0.38	0.81x0.38	18.5	nr	nr	3.20
KA II	6	H steel pile with local variable enlargement near the toe	C	0.39x 0.38	0.55x0.55	14.2	7.40	85.00	5.54
KA II	7	H steel pile with local variable enlargement near the toe	C	0.39x 0.38	0.80x0.80	18.7	nr	nr	4.53
KA II	8	H steel pile with local variable enlargement near the toe	C	0.39x 0.38	0.39x 0.38	18.0	nr	nr	4.53
KA II	9	H steel pile with steel plate at the bottom	C	0.39x 0.38	0.39x 0.38	19.0	nr	nr	-
KA II	10	H steel pile with steel plate near the bottom	C	0.39x 0.38	0.39x 0.38	19.0	nr	nr	4.06
KA II	11	H steel pile with local constant enlargement near the toe	C	0.39x 0.38	0.39x 0.38	19.0	nr	nr	5.15
KA II	12	H steel pile with local constant enlargement near the toe	C	0.39x 0.38	1.13x0.39	18.5	nr	nr	4.58

(1) C = compression, T = tension

propagation velocity of stress waves in the pile and the soil, and the pile toe condition in the bearing layer.

Belgian experience of the sonic echo method has evidenced however several limitations in the case of cast-in-situ concrete piles (driven, screwed, vibrated, injected or bored) which often have a very irregular lateral surface. A limitation has been found when one encounters several discontinuities in a particular pile : the number of echoes which may be partially superimposed is thereby increased and makes the interpretation of the graphs more difficult. Another limitation has been identified when heavy damping of the signal due to the corrugated texture of the shaft prohibits in some cases the interpretation of the test. It has also been observed that the wave speed travelling in screwed piles shafts is lower than the concrete bar wave speed.

The mechanical admittance method is used when quantification of the pile cross-sectional area and of the pile-soil interaction parameters is needed, in addition to information regarding the integrity of the pile.

7.4 Dynamic load tests

Dynamic load tests with measurement of the strain and velocity of the pile head are increasingly used to evaluate the behaviour of piles [25b, 26b]. Few dynamic tests have been performed on non-displacement (bored) piles however, and some judge that there is not enough experimental data to confirm the feasibility of the method in such cases.

In spite of these arguments, deductions are made using available methods based on the wave equation, including the "Case" and "Capwap" type approaches. Studies of the "Case" method in Belgium [25] tend to show that the result depends strongly on the shape of the impacting force diagram (role of helmet) and on the level of energy.

For the "Capwap-type" procedure, Belgian experience has found a reasonable degree of reliability for the prediction of the ultimate skin friction and of the loading curve at the base, up to the mobilised load [25]. The ultimate failure load, if required, is then a matter of extrapolation as in the case of a loading test not carried out to failure. Tests conducted in Belgium indicate that the maximum transient displacement at the base can exceed that corresponding to the limit load.

8 PARTICULAR NATIONAL EXPERIENCES

De Beer's method [13] finds its roots in theoretical and laboratory experimental research work [7,8] dealing with the interpretation of the CPT test. The method was further enhanced using experimental full scale research conducted on displacement piles at different sites in Belgium (Zwijnaarde, Kontich, Kallo I and III). Table 6 provides information on those test programs as well as on other research programs undertaken for other pile types. Three test sites were used to study the behaviour of H steel piles : two in Kallo II and IV (sand) and another in Kontich II (clay).

Other comparative tests were undertaken in Groot-Bijgaarden with Vibrex-type piles having different lengths and base types. More recently, a test site in Limelette with different driven piles has been undertaken in order to compare driving techniques, performance and induced vibration.

Tensile piles have not been frequently tested (test sites of Ostend, Antwerp I and II, Zelzate focused on bored piles and cast-in-situ driven piles with an enlarged base). The question of the prediction of their ultimate tensile load remains open in many cases.

For more than fifteen years, the market of the continuous flight auger cast-in-situ pile and the screwed (auger) cast-in-situ pile has been rapidly increasing in Belgium, and significant research work has been undertaken on such piles in different soil conditions (test sites of Zwevegem, Ghent I and II, North Sea Coast, Geel, Koekelare and Vilvoorde).

During the last decade, research work has been undertaken in order to improve prediction of pile behaviour by means of stress wave analysis (Kontich II and Kallo II), dynamic loading test (Ghent II, North Sea Coast, Koekelare) and the use of the dilatometer test, DMT (Geel and Koekelare).

In total, more than 100 piles have been scientifically tested during the last 30 years in Belgium in order to develop and improve design methods for axially loaded piles (compression or tension). All this research work has been accomplished thanks to the financial aid of the National Institute for Scientific Research in Industry and Agriculture, the services for funding industrial research in the three Belgian Regions, the Ministry of Public Works, the Belgian Geotechnical Institute, the Civil Engineering Department of the Université Catholique de Louvain, the Laboratory of Soils Mechanics of the University of Ghent, the Belgian Building Research Institute, and Belgian piling contractors.

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