National Report 10 - CPT in Belgium in 1995.

J. Nuyens
Free University of Brussels, Belgium

P.Menge
Ghent University, Belgium

F.Decock
Franki, Belgium

G.Van Alboom
Ministry of Flemish Community, Belgium

C.Legrand
C.S.T.C., Belgium

M.Van Den Broeck
Dredging International, Belgium

J.Maertens
TUC Rail, Belgium

P.Welter
Ministery of Wallon Community, Belgium

SYNOPSIS:
The CPT has been used extensively in Belgium since the first years that followed World war II, so the test procedures, the interpretation methods and the design methods are well established and are summarized in the present report.

1. GEOLOGY

The country 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 (640 m above sea level).

The geology of the Tertiary and Quaternary formations in Belgium is characterized by an approximately South-East North-West oriented epirogenetic axis (Silence, 1992), which follows the valleys of the rivers Haine, Sambre, Meuse and Vesdre (Figure 1), divides Belgium to approximately two equal parts.

In the North part, the stratigraphy was governed by fluctuations in the coastal line. Consequently the bedrock is covered by alternating Tertiary clay, sand and (occasionally) gravel sediments, with thicknesses 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 (loëss). 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. The soil layers interferring in the foundation design most frequently allow for the execution of CPT. In the South, soil investigation as well as foundation design may be based on CPT as well, although during the last decade other tests, e.g. pressuremeter tests, have known an increasing use.

So, in Belgian practice, calculations for foundation design are by far based on CPT. During the last decades, very extensive research work by geotechnical experts, universities, research institutes and contractors, with the support of the authorities and control organizations, has resulted in a wide experience and know-how on the use of the CPT to the design of piles and their behaviour, and also to some extent on most common soil improvement methods (such as soil compaction or stone columns).

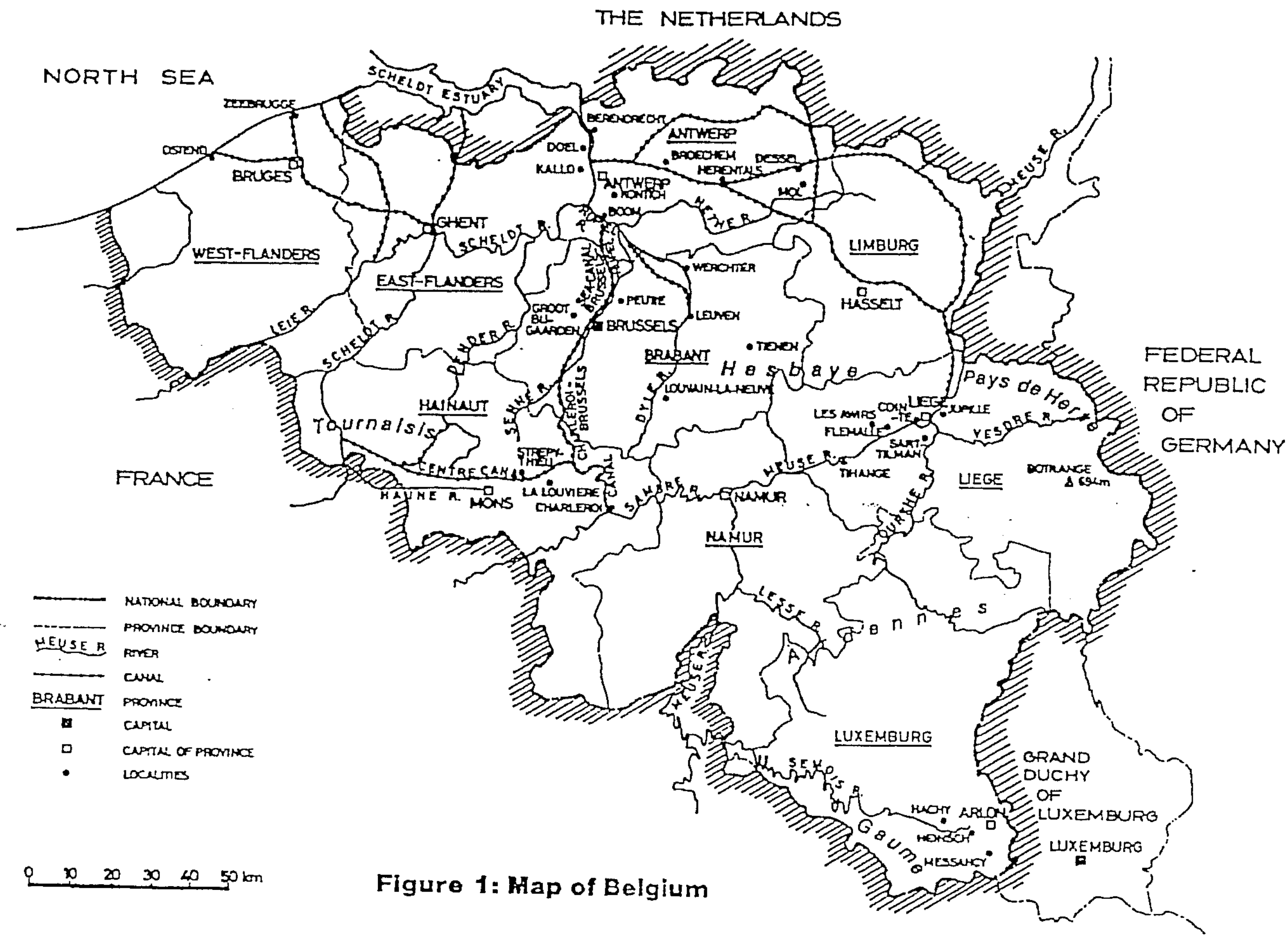


Figure 1: Map of Belgium

2. INVESTIGATION METHODS

The use of the CPT was developed in Belgium by De Beer (1945) with the elaboration of the mechanical simple cone with closing nut M4 and by Verdeyen (1945) with the use of the Dutch mantel cone M1 (Figure 2). The CPT remained the most commonly used technique for in situ geotechnical testing in Belgium. De Beer (1948) did pionering work in developing his own interpretation and calculation methods based on a bilinear intrinsic curve while Verdeyen (1948) promoted the rational interpretation according to the classical theory of plastic limit equilibrium. De Beer's research work played a major role in the introduction and general acceptance of the CPT in Belgium from the early days this equipment was developed

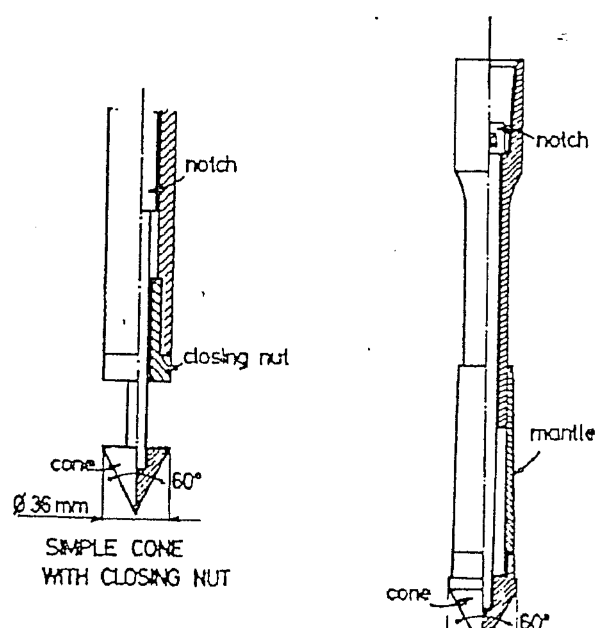


Figure 2: Mechanical cones M4 and M1

Dynamic penetration testing is also performed: the cone penetration testing is used mostly in the field of road construction; the SPT has been used a lot during the sixties for the design of industrial plants.

More recently, the growing need for rational stress-strain analysis is at the origin of the fast increasing use of pressuremeter techniques.

3. TYPE OF CPT - EQUIPMENT USED IN BELGIUM

3.1. NATIONAL CODES AND/OR STANDARDS

Recently, recommendations (available at CPT'95) were issued describing the CPT equipment, the way tests should be performed and how the data should be presented. Companies following these recommendations will be able to get a certification which is needed to work for the regional or federal government services. Also private companies will use these recommendations as a guideline for their site investigation tests. At present the possibility is considered to transform these recommendations to a standard for CPT testing in Belgium.

The above recommendations are mainly based upon the recommendations for CPT testing of the ISSMFE (Report of the ISSMFE TC16).

3.2. EQUIPMENT

For economical and technical reasons, the mechanical cone is still more often used than the electrical cone. This is especially the case in certain parts of Belgium where soil conditions are too severe (presence of rock, sandstones, ...) for the highly sensitive electrical cones. Because of this fact, both the mechanical cones and the electrical cones are accepted in the recommendations.

The mechanical CPT can be performed with the mantle cone (M1), the friction sleeve cone (M2) and the simple cone (M4).

As electrical cones, both the standard electrical cone (CPTE) equipped for measuring cone resistance and local friction (and preferably inclination of the cone) and the piëzocone (CPTU) can be used.

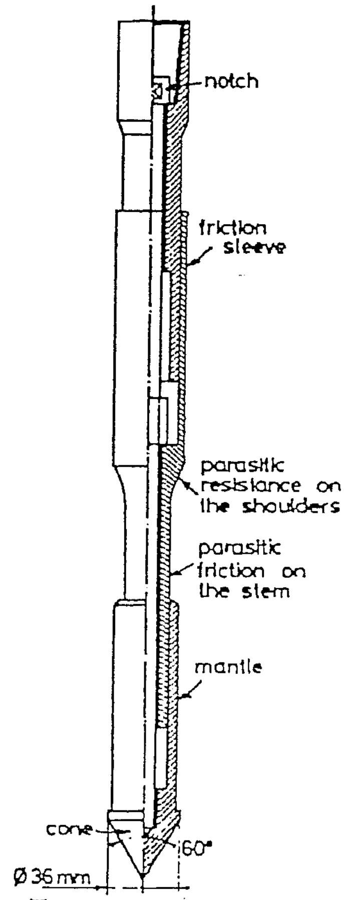


Figure3: Mechanical cone M2

3.3. TEST PROCEDURES

One condition which has to be fulfilled concerning the execution technique when performing the mechanical CPT test is that the different parts of the cone must have a significant displacement relative to each other.

For CPTU tests, no recommendations are given for performing dissipation tests, although for completeness it is mentioned that such a test can be performed.

The measuring equipment of both the mechanical cone and the electrical cone has to be calibrated by an officially accredited laboratory every 6 months. When extensively used, a more frequent calibration is advised: after 250 tests or after 3000 m of sampling.

Calibration of the electrical cone consists of calibrating the cone as used in the field with analog and digital equipment. A small apparatus for checking calibration of electrical cones with measuring equipment on the site is advised for regular use. A logbook, which should always be present in the CPT truck, serves as a possible means for control of this requirement. The logbook has to contain data and information about the use of the equipment and the calibrations.

3.4. CORRECTION AND PRESENTATION OF TEST RESULTS

For mechanical cone tests, a correction for the weight of the tubes and inner rods is recommended but not mandatory.

For electrical cone tests, a correction of the q_c value for the porewater pressure acting on the back side of the CPTU tip and a correction of the depth for the inclination of the tubes are advised but not mandatory.

For all CPT procedures, when corrections are included, they must clearly be stated in the report.

It is generally known that the different cone types can give different results. Because of the large influence of soil type and stress condition, no general correction formula is available for these differences. The importance of possible errors must be recognised and it is mentioned in the recommendations. In this respect, it is generally accepted that the CPTE-cone serves as a 'reference test'.

The presentation of the test results follows the instructions of the TC16 report.

4. INTERPRETATION OF TEST RESULTS

4.1. SOIL PARAMETERS AND OTHER DATA

Soil parameters derived from CPT-results are :

- friction angles $\varphi'_{De\,Beer}$ and $\varphi_{De\,Beer}$ or φ'_d;
- deformation modules;
  - stiffness index C as derived from oedometer tests, used for settlement calculations in Belgium and the Netherlands and related to the compression index C_c by

$$C = 2.3\ (1 + e_0)\ /C_c$$

  - E-modulus;
  - G-modulus;
- undrained shear strength c_u.

4.1.1. FRICTION ANGLE

De Beer's method for assessing the friction angles φ and φ' (commonly taken as 30°) of the bilinear intrinsic curve of his foundations design methods was used for a long time in Belgium and has been described by Sanglerat (1972).

For limit equilibrium and retaining wall calculations in granular soils, φ'_d-values are derived according to Meyerhof, resp. Caquot & Kerisel from equation $N'_q = q_c/\sigma'_{v0}$.

Besides these two methods, different correlations between φ'_d and q_c for granular soils are used for practical applications (Durgunoglu and Mitchell 1975) , (Robertson and Campanella 1983).

Tables presented in the Dutch standard NEN 6740 and in the German Standard DIN 1055 Teil 2 give for different q_c-levels and/or soil types representative values for different soil parameters, including φ' and c'-values.

4.1.2. DEFORMATION MODULES

C-values are derived from CPT-results according to the theory developped by Buisman (1940)

$$C = \alpha\ q_c/s'_{v0}$$

With the Buisman value of α = 1.5 a conservative estimate of C is derived from CPT tests with the same order of magnitude as from consolidation tests. For tertiary overconsolidated soils the derived C-value is then multiplied by 3 for clay, and 10 for sands. In today's practice the α factor is differentiated for each soil type as proposed by Sanglerat (1972).

The constrained modulus M is obtained by a similar formula

$$M = E_0 = \alpha\, q_c$$

with the same values for the α factor. M is also calculated starting from tables from other authors like Mitchell and Gardner (1975) and Lunne and Christofferson (1983).
Relationships between the drained Young modulus (E_{25}, E_{50}) and q_c for NC-sands and between dynamic shear modulus G_{max} for NC sands and q_c are used, after Robertson and Campanella (1983). A table giving normalised E-values for different soil types and corresponding q_c-levels is found in the Dutch report on setpiles CUR 166 (1993).

4.1.3. UNDRAINED SHEAR STRENGTH c_U.

Derived from the classical theory bearing capacity of a pile, the in situ undrained shear strength c_u is provided by

$$c_u = \frac{q_c - \sigma'_{v0}}{15}$$

N'_c for $\varphi = 0$ is theoretically equal to 0 but one has to take into account the cone type and the soil plasticity. In most cases $N'_c = 15$ and the influence of soil plasticity is taken into account when design values are put forward (e.g. use of Bjerrum correction factor for slope stability calculations). Most often the equation is simplified to

$$c_u = \frac{q_c}{15}$$

For stiff overconsolidated clays c_u-values are derived according to the following expression, which is a function of the sensitivity S_t of the clay.

$$c_u = \frac{q_c}{10 \times S_t}$$

5. USE OF CPT IN GEOTECHNICAL DESIGN

5.1. BEARING CAPACITY OF SHALLOW FOUNDATIONS

In Belgium, the calculation of the bearing capacity of a centrally and verticaly loaded shallow foundation based on CPT results uses the classical formula

$$q_u(kN/m^2) = s_q \cdot d_q \cdot p_t \cdot N_q$$
$$+ s_c \cdot d_c \cdot c \cdot N_c + s_\gamma \cdot d_\gamma \cdot \gamma \cdot B \cdot N_\gamma$$

The three terms represent the contribution of surface loading p_t (subscript q), cohesion c (subscript c) and soil unit weight γB (subscript γ), respectively. They are multiplied by dimensionless bearing capacity factors (symbol N), shape factors (symbol s) and depth factors (symbol d). B and L are the dimensions of the footing in meters. The values of p_t and c are expressed in kN/m², γ is expressed in kN/m³. The factors N_φ, N_q and N_c are the same for the different methods:

$$N_\varphi = tg^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right)$$

$$N_q = N_\varphi \cdot e^{\pi \cdot tg\varphi}$$

$$N_c = \left(N_q - 1\right) \cdot \text{cotg}\, \varphi$$

The factor N_γ and the shape and depth factors depend on the method used. The most frequently used factors are those of Meyerhof (1951)

$$N_\gamma = \left(N_q - 1\right) \cdot tg(1.4 \cdot \varphi)$$

$$s_q = 1 + 0.1 \cdot \frac{B}{L} \cdot N_\varphi$$

$$s_c = 1 + 0.2 \cdot \frac{B}{L} \cdot N_\varphi$$

$$s_\gamma = s_q$$

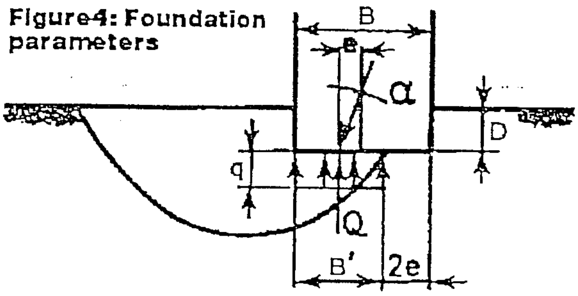
$$d_q = 1 + 0.1 \cdot \frac{D}{B} \cdot \sqrt{N_\varphi}$$

$$d_c = 1 + 0.2 \cdot \frac{D}{B} \cdot \sqrt{N_\varphi}$$

$$d_\gamma = d_q$$

but those of Vesic and of the DIN 4017 and NEN 6744 are also used. In case of an eccentric and/or inclined load the corrections of Meyerhof (1951) are used:

Figure4: Foundation parameters



$$B' = B - 2\, e_B$$

$$L' = L - 2\, e_L$$

$$i_q = \left(1 - \frac{\alpha}{90°}\right)^2$$

$$i_c = i_q$$

$$i_\gamma = \left(1 - \frac{\alpha}{\varphi}\right)^2$$

The calculated bearing capacity should be divided by a safety factor to obtain the design value. In case of normal loading conditions, a safety factor of 2.5 is used with De Beer's while other methods need a safety factor of 3 to 3.5.

5.2. PILE BEARING CAPACITY

5.2.1. END BEARING CAPACITY

The "De Beer" method, is without any doubt, the most widely used method in Belgium for the calculation of the end bearing capacity of single piles. Its theoretical background has been elaborated and published some 25 years ago (De Beer, 1971). The method and later modifications have also been reported in ESOPT and ISOPT proceedings by Van Impe among others.

The De Beer method is based on a multi-step consideration of scale effects, when passing from a soft to a hard soil layer. This application of the scale effect is performed in 4 steps, designated by the terms homogeneous values, descending or downward values, upward valus and mixed or blended values. The final mixed values $q_{u,b}^{(m)}$ are the basis values for the further end bearing calculation of the pile. They have to be reduced, according to the way of pile installation and according to the soil type.

An example of calculation is shown in Figure 5, whereby the variation of $q_{u,b}^{(m)}$, as calculated according to the basic De Beer method ($q_{u,b}^{(m)}{}_{DB}$) is given, together with the measured unit cone resistance q_c.

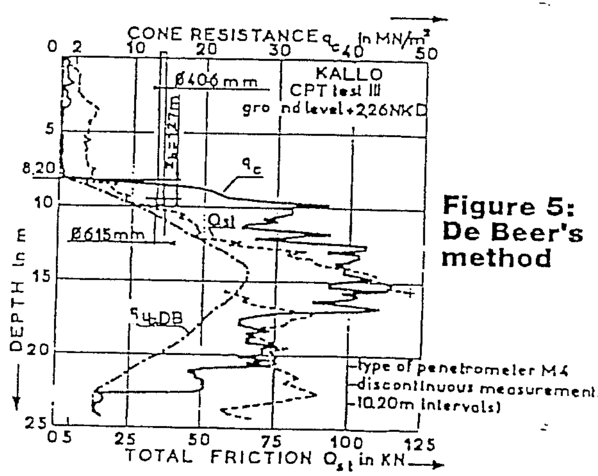


Figure 5: De Beer's method

The pile ultimate end bearing $Q_{u,b}$ is further defined as :

$$Q_{u,b} = q_{u,b} \times A_b = \alpha_b \times \varepsilon_b \times q_{u,b}^{(m)} \times A_b$$

with : α_b = a coefficient taking into account the way of installation of the pile;

ε_b = a parameter referring to the scale effect by the fissuring of the soil;

A_b = the nominal pile base section.

The ε_b *parameter* 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 sounding cone. From an extensive research program in the tertiary overconsolidated Boom clay (De Beer, e.a, 1977), it was found that the following is approximately true:

$$0.476 \le \varepsilon_b \approx 1 - 0.01(D_b/d - 1)$$

with : D_b resp. d = diameter of pile base resp. of sounding cone.

The α_b coefficient has empirically been deduced from static pile load tests in various research projects. An overview of values, commonly used in the Belgian design practice, is given in table 1.

It should be emphasized that the above mentionned empirical α_b coefficients have been deduced :

(1) on basis of correlations with CPT's executed with the mechanical cone of type M4 or the electrical E1 cone;

(2) refers to the conventional rupture load for displacement piles, corresponding to $s_b/D_b = 10\ \%$, and to the critical rupture load for bored piles, corresponding to $s_b/D_b = 5\ \%$.

Pile type	Table 1 : α_b coefficient for	
	sand	stiff O.C. clay
Cast in situ driven displacement pile with low dry concrete (Franki expanded base)	1.15	1.0
Cast in situ driven displacement pile with bottom plate and plastic concrete	0.8-1.0 (1)	0.8
Driven precast concrete pile	1.0	0.85
Cast in situ screwed displacement pile with double soil displacement (Atlas type)	1.0	1.0
Bored piles (large diameter and CFA)	0.33-0.8(2)	0.8

(1) : function of diameter of the bottom plate relative to diameter of driving tube;

(2) : for bored piles in soft cohesive soil and in sand De Beer-Van Impe (1977) proposed to determine $\alpha_{b,z}$ out of CPT results by:

$$\alpha_{b,z} = 0.8 - (\alpha_{d,o} - \alpha_d) \times \left[\frac{q_{u,b(z)}^{(m)} - q_{u,b(\min)}^{(m)}}{q_{u,b(\max)}^{(m)} - q_{u,b(\min)}^{(m)}} \right]$$

5.2.2. ULTIMATE SHAFT FRICTION

Estimation of the ultimate shaft friction is based on one of the following CPT values :

1. the total skin friction $F_{s,z}$ from the CPT;
2. the cone resistance q_c;
3. the local skin friction $f_{s,z}$ from the CPT.

Ultimate shaft friction deduced from $F_{s,z}$

The easiest way for evaluating the total pile shaft resistance $Q_{u,s}$ remains related to the (cumulative) measurement of the total skin friction $F_{s,z}$ from the CPT :

$$Q_{u,s} = \xi_f \times F_{s,z} \times \frac{D_s}{d}$$

with : ξ_f = a parameter depending on soil and pile type;

D_s resp. d = diameter of the pile shaft resp. of the sounding rod.

The ξ_f coefficient should be seen as the product of mainly three factors, α_s, β_s and ε_s. The ξ_f parameter is to a large extend influenced by the method of installation of the pile (factor α_s) , which defines the densification or eventual loosening of the soil in the vicinity of the pile, as well as the stress state around the pile after installation. Also the nature of the lateral surface of the pile (steel, rough or flat concrete, ...) has a consistent influence on the shaft friction (factor β_s). Finally, scale effects related to the soil structure (such as the fissuring of the soil) may also be an influencing factor on the overall skin friction (factor ε_s).

Table 2 gives indications on the design values for the overall factor ξ_f, deduced from various research work.

Pile type	Table 2: ξ_f coefficient for	
	sand	stiff O.C. clay
Cast in situ driven displacement pile with rammed dry concrete (old Franki type)	1.75	1.15
Cast in situ driven displacement pile with vibrated plastic concrete	0.8-1.0	0.65
Driven prefabricated concrete pile	1.0	0.85
Cast in situ screwed displacement pile with double soil displacement (Atlas type)	1.25	1.25
Rammed or screwed steel pipe piles	-	0,45-0,55
Bored piles (large diameter and CFA)	-	-

Altough the F_S approach is very simple, it should not however be overlooked that the assumed linear correlation between total friction on the steel "jacked" CPT sounding rods and the total friction on the pile shaft should be corrected on some occasions. Such cases include, for example, the following :

* deviation of the sounding rods or the use. of unsmooth rods (by the presence of e.g. the ball-clamp used for the extraction of the rods) often leads to higher (unsafe) CPT friction values;
* it is often experienced in particular sand layers, even being medium dense to dense, that the increase of F_S is very moderate and even zero; this might be explained by some buckling of the sounding rods, leading to certain widening of the penetration hole, to some re-arrangement of the grain structure, or to very low horizontal contact stresses between the rods and the surrounding soil; these phenomena occur in particular in dry, somewhat cemented sands;
* rod buckling and relaxation also are reflected in the CPT values by a somewhat delayed drop in the F_S value after penetrating through a dense sand layer.

Ultimate shaft friction deduced from q_c

The pile shaft friction can also be evaluated on a semi-empirical correlation between the cone resistance values q_c and the ultimate unit shaft friction $q_{u,s}$:

$$Q^{+}_{u,s} = \pi\, D_s \times \sum H_i \times \eta_p \times q_c$$

$$q_{u,s} = \eta_p \times q_c$$

Values for η_p have been reported by Van Impe (1986, 1989) and De Cock (1993).

Ultimate shaft friction deduced from $f_{s,z}$

A third method relates the pile shaft friction to the unit skin friction on CPT tubes, by :

$$q_{u,s} = \alpha_s \times f_{s,CPT}$$

Again, α_S depends largely on the installation method of the pile and the consequent stress state around the pile, but also (Carpentier e.a., 1985) :

* on the nature of the soil layers, and for cohesive soils on the rigidity of the soil;
* on the nature of the skin of the shaft : steel or concrete;
* for concrete, on the way the shaft is fabricated: precast shaft, shaft constructed in situ with tamped dry concrete, shaft constructed in situ of vibrated wet concrete.

Although the $q_{u,s}$ approach based on $f_{s,CPT}$ should be less subject to parasitic phenoma than the case of total $F_{s,CPT}$ values, only little experimental data are available in Belgium, so that actually, this method is not widely used.

The unit skin friction $f_{s,CPT}$ may be directly measured in the CPT using mechanical or electrical friction sleeves, or may be derived from the q_c values. In the latter case, the following rules are often used in Belgium.

For cohesionless quartz sands (Carpentier & al. 1985) :

$$f_{s,CPT} = \frac{q_c}{200} \quad for \quad q_c \geq 20\ MPa$$

$$f_{s,CPT} = \frac{q_c}{150} \quad for \quad q_c \leq 10\ MPa$$

For intermediate values of q_c, the value of $f_{s,CPT}$ is calculated by linear interpolation between the values q_c:200 and q_c:150.

For cohesive soils with a small rigidity index :

$$f_{s,CPT} = \frac{q_c}{15}$$

For stiff clays :

$$f_{s,CPT} = \frac{q_c}{36.6}$$

6. COMPARISON WITH OTHER METHODS

There seems to be no direct comparison possible with the pressuremeter methods.

7. MAJOR AREAS FOR RESEARCH ACTIVITIES

* Atlas piles in stiff clay : performance, stress measurements
* Steel piles
* PCS and Omega pile
* Static and cyclic model pile tests in sand (Kanai)
* Prediction of pile settlement (Verbrugge)
* Different behaviour of bored and driven piles

8. FUTURE TRENDS AND NEW DEVELOPMENT

In general, the mechanical CPT will more and more be replaced by the electrical cone when soil conditions or improved equipment allow the safe use of the CPTE. Related to the electrical measurement and digital storage of the measurements, automatic and faster presentation and analysis of tests results will become a rule. An important condition for digital data collection and exchange is the use of a uniform data format. In Belgium, a Geotechnical Data Interchange Format (GDIF) was presented already several years ago (Nuyens, 1989). This format will become of utmost importance for flexible data exchange between site investigation companies and design offices in the future. The GDIF format is suggested in the CPT recommendations mentioned above. Furthermore, the GDIF will allow a fast and reliable means to transfer data for research and mathematical analyses. Thus GDIF will play a key role in future research, performance evaluation and methodology improvement.

New types of cones such as the seismic cone, the resistivity cone, the environmental cone, ... are not (yet) commonly used in Belgium. Although some companies are considering buying such equipment, the opportunity of such an investment for the moment remains unclear. The use of this equipment up to now is mainly restricted to (university) research centres or the regional governmental geotechnical departments. With the growing importance of site investigation techniques for quick and reliable screening of possible polluted sites, some of these cones will certainly gain importance and will more generally be used in Belgium. Certainly more research will be needed to fully implement these techniques in Belgian geotechnical practice, in Belgium but also abroad.

With respect to the environmental site investigation, test methods to detect groundwater pollution are - although very limited - already in use. The CPT probes to withdraw water samples at different soil levels become more important. A possible area of research will be the reliability of the results considering the possibility of cross contamination which can occur because of the polluted water remaining in the filter element of the water suction device.

At the laboratory of Soil Mechanics of Ghent University, a research project is running on the use of the acoustic emission technique with penetration testing. In a first stage of the project, the parameters influencing the AE signal were investigated in the laboratory. Therefore two special test set-ups were designed and a AE needle apparatus and a prototype AE cone were built. It is planned to prepare a prototype AE cone for field testing. The AE signal generated in the soil during penetration of the cone is considered as additional information with the classical CPT data which will also be measured. As a result of the first stage of the project is was concluded the AE cone is a highly sensitive apparatus, measuring events being generated at grainsize level. The AE signal helps in identifying more clearly soil type, grain size, (micro)layering, density, shear resistance, ... The main advantage of this research equipment seems to be its possibilities in defining different soil structures and ageing effects in non cohesive material. More details about this research are given in the paper (Mengé and Van Impe, 1995) also presented at this conference.

9. REFERENCES

Buisman, A.S. Keverling. (1940). Grondmechanica. *Waltman. Delft.*

Carpentier, R., De Beer, E.E., De Jonghe, A., Holeyman, A., Legrand, C., Lousberg, E., Maertens, J., Raedschelders, H., Van Wambeke, A., Weber, L. and Wallays, M. (1985). Pile foundation problems: recent developments. *Belgian Geotechnical Volume published for the 1985 Golden Jubilee of the ISSMFE, Brussels,.*

CUR report No 166. (1993) Damwandconstructies

De Beer, E.E. (1945). Etude des fondations sur pilotis et des fondations directes. L'appareil de pénétration en profondeur. *Annales des Travaux Publics de Belgique. April, June and August.*

De Beer, E.E. (1949). Grondmechanica - Deel II. *Standaard Boekhandel. Antwerp.*

De Beer, E.E. (1971). Methodes de déduction de la capacité portante d'un pieu à partir des résultats des essais de pénétration. *Tijdschrift der Openbare Werken van België nr. 4, 5, 6,.*

De Beer, E.E. (1974). Decrease in penetration resistance due to excavation in an overconsolidated glauconitic sand. Proc. European Symposium on Penetration Testing, *Stockholm, p. 103.*

De Beer, E.E., Lousberg, E., De Jonghe, A., Carpentier, R. and Wallays, M. (1979). Analysis of the Results of Loading Tests Performed on Displacement Piles of Different Types and Sizes Penetrating at a Relatively Small Depth into a very Dense Sand Layer. *Proceeding Conference on Recent Developments in the Design and Construction of Piles, ICE, London.*

De Beer, E.E., Lousberg, E., De Jonghe, A., Wallays, M. and Carpentier, R. (1979) Prediction of the Bearing Capacity of Displacement Piles Penetrating into a very Dense Sand Layer from the Results of CPT Tests. *Proceedings 7th European CSMFE, Volume 3, Brighton.*

De Beer, E.E., Lousberg, E., Wallays, M., Carpentier, R., De Jaeger, J. and Paquay, J. (1977). Bearing Capacity of Displacement Piles in Stiff Fissured Clays. *Comptes Rendus de Recherches I.R.S.I.A., N° 39, Brussels.*

De Beer, E.E., Van Impe, W. (1977). Afleiding van het grensdraagvermogen aan de basis van geboorde palen, uit diepsonderingen. *Wetenschappelijke nota voor de inwendige dienst van het R.I.G.,.*

DIN 1055 Teil 2

Durgunoglu, Mitchell, (1975). Static penetration resistance of soils. *Proc. Conf. on In situ measurement of soil properties. ASCE, June, Vol. 1.*

Lunne, Christofferson, (1983). *Interpretation of cone penetrometer data for off shore sands. Norwegian Geotechnical Institute Report No 52108-15*

Mengé, P., Van Impe W.F. (1995). The application of acoustic emission testing with penetration testing. *International Symposium on Cone Penetration Testing CPT'95, Linköping, Sweden, October 4-5.*

Mitchell and Gardner (1975)

NEN 6740 (1990). Dutch standard - *Geotechnics TGB.*

Nuyens, J. (1989). Geotechnical Data Interchange Format GDIF, Version 1.0. *Belgian Society of the ISSMFE.*

Robertson, Campanella,. (1983). Interpretation of cone penetration tests - Part 1: sand. *Canadian Geotechnical Journal No 20*

Sanglerat, G. (1972). The penetrometer and soil exploration. *Elsevier.*

TC 16. Report of the ISSMFE Technical Committee on Penetration testing of Soils - with Reference Test Procedures: CPT - SPT - DP - WST. Swedish Geotechnical Institute. *SGI Information 7.*

Van Impe, W.F. (1985). The bearing capacity of screwed piles in cohesive layers. Proceedings *11th Internation Conference on Soil Mechanics and Foundation Engineering, San Francisco. Volume 3, p. 1493-1497.*

Van Impe, W.F. (1986) Evaluation of deformation and bearing capacity parameters of foundations from static CPT-results. *Proceedings 4th NTI International Geotechnical Seminar, Singapore, p. 51-70.*

Van Impe, W.F. (1988). Considerations on the auger pile design. *Proceedings of the First International Seminar on Deep Foundations on Bored and Auger Piles, Ghent, p. 193-218.*

Van Impe, W.F. (1991). Developments in pile design. *4th International Conference on Piling and Deep Foundations, General Report, Stresa.*

Van Impe, W.F., De Beer, E.E. and Lousberg, E. (1988). Prediction of the single pile bearing capacity in granular soils out of CPT results. *ISOPT I. First International Symposium on Penetration Testing, Orlando, , Specialty Session, 34 pages.*

Verdeyen, J. (1945). Etude des fondations sur pieux au moyen de l'appareil de pénétration en profondeur. *CEBTP. Paris.*

Verdeyen, J. (1948). Mécanique du sol et Fondations. *Desoer. Liège.*