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Atlas screw piles and tube screw piles in stiff tertiary clays – Assessment of pile performance and pile capacity on basis of instrumented loading tests

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ABSTRACT : In order to further develop the understanding of the static as well as the dynamic behaviour of screwed piles of the Atlas type in stiff overconsolidated clays, an extended research program has been recently executed in Belgium. Also a new type of tube screw piles, consisting of a sacrificial cylindrical steel tube provided at the bottom of a steel enlarged base, has been analysed. The paper deals with the results of the static loading tests performed on instrumented piles. The resulting mobilisation functions and ultimate values of shaft friction and end bearing are analysed here.

1 INTRODUCTION

On a test site in Koekelare/Belgium, located nearby the plant of Franki-Atlas Palen, an extended research program has been set up by Franki, with the scientific collaboration of the University of Ghent and Louvain-la-Neuve, the LCPC/France and Cementation Piling & Foundations Company/UK. The subsoil under the top layers consists over a great depth of medium stiff to stiff tertiary clay (Ypresian clay or Flanders clay). These eocene clays occur over most south western part of Belgium as well as north east of France. They have the same geological age and comparable geotechnical characteristics as the London clay.

The aims of the research were multiple and dealt essentially with the following items :

1. a comparative analysis of soil tests such as CPT (with different cone types) and DMT, executed before and after piling works;
2. the assessment of dynamic pile load testing and the dynamic pile behaviour in general;
3. a critical analysis of the advantages and reliability of the TIMESET/CEMSET approach for the execution and interpretation of static loading tests;
4. the interpretation of soil stress parameters during the installation process;
5. the analysis of the pile boring parameters, measured during the screwing-in and the extraction of the steel tube;
6. the quantification of the mobilisation functions for shaft friction and end bearing of the considered piles in the considered ground under static loading;
7. the deduction of design formula and design factors for the two considered pile types on basis of CPT and DMT.

Mainly items 6 and 7 are detailed in this paper. Items 1, 3 and 5 will only briefly be discussed. Finally, item 2 resp. 4 will be treated in later publications. Before assessing however under par. 5 the said analysis, the piles considered, the content of the research program concerned and the results of the tests performed are respectively described in par. 2, 3 and 4.

It is worth to be mentioned that the research must be considered as a prolongation of former research work in Belgium and abroad on Atlas piles, which has been described and discussed in several papers (Bustamante 1983) (Hollingsworth 1992) (Van Impe 1984, 1985, 1988) . Reference is also made to the special lecture of Bustamante in this seminar, proposing general design rules of concrete Atlas screw piles as well as tube screwed displacement piles in different soil types on basis of cone penetration tests CPT, Menard pressiometer tests PMT and standard penetration tests SPT, and to an extended research program performed in 1977 by Franki S.A. with the scientific co-operation of several Belgian research institutes, dealing with piling in the tertiary oligocene Boom clay (I.W.O.N.L. 1977).

2 PILE TYPES CONSIDERED

The research program considered deals with the two following pile types :

- cast-in-situ Atlas screw piles, also referred to as "single-auger piles with double lateral soil displacement";
- tube screw piles, which are also of the soil displacement type.

The former pile type and its applications have widely been described in literature (De Cock 1993) (Hollingsworth 1992) (Imbo 1984) (Van Impe 1988).

The latter is a new type, introduced beginning 1992 by Franki. It is realised by the screwing-in of a rigid bottom-closed cylindrical steel tube, comprising an enlarged helicoidal steel flange at the base. It combines the advantages of tube piles in general (application in agressive soils, underground cavities, important flow gradients, restricted working areas and construction heights), with a vibration free installation procedure and a good bearing performance.

3 GENERAL DESCRIPTION OF THE TEST PROGRAM

A general overview of the test site is given in Figure 1. In a whole, the following tests and installations have been performed.

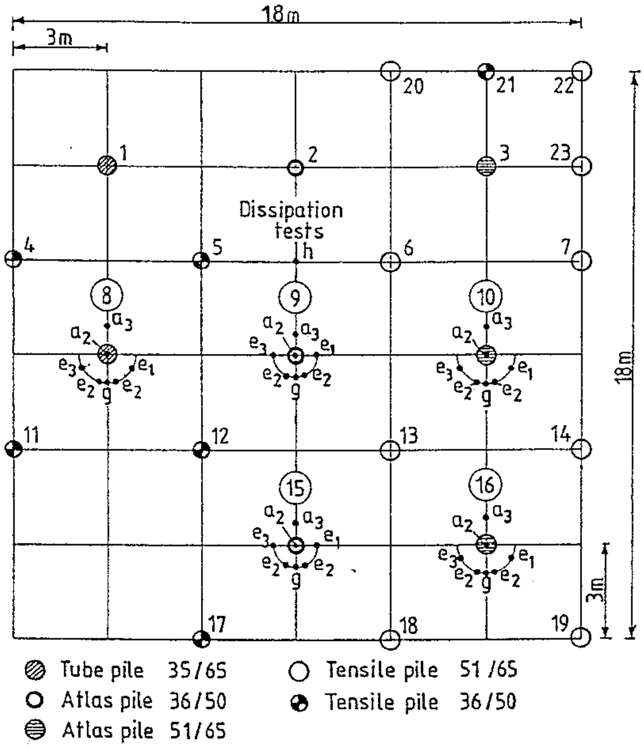


Fig. 1 General overview of test site

3.1 Soil tests

Soil tests have been executed before, after and during the execution of the piles, namely:

- 34 cone penetration tests (CPT) with different geometry (electric cone E1, Dutch mantle cone M1, Begemann cone M2 and Belgian simple cone M4), different test procedures (continuous and discontinuous penetration) and at different stages (before and after piling works);
- 10 flat dilatometer tests (DMT) before and after pile execution;
- stress measurements nearby the piles during execution, using the flat dilatometer blade and a piezocone;
- one boring with soil sampling and execution of laboratory tests on the samples.

3.2 Test piles

In a whole, 10 test piles have been executed, most of them being instrumented :

- 5 instrumented piles for static loading tests under vertical compression, namely piles P8 (tube pile 35/65), P9 and P15 (Atlas pile 36/50) and P10 and P16 (Atlas pile 51/65); the dimensions mentioned refer to the minimum diameter Ds of the shaft respectively to the outer diameter of the sacrificial or recovered helicoidal flange Df;
- 2 piles for static loading tests under vertical tension, namely piles P21 (Atlas pile 36/50) and P23 (Atlas pile 51/65);
- 3 instrumented piles for dynamic loading tests, namely piles P1 (tube pile 35/65), P2 (Atlas pile 36/50) and P3 (Atlas pile 51/65).

In addition, 14 Atlas piles have been installed as reaction piles for the static loading test or for additional stress measurements.

All executed piles have a length between 13 and 13.5 m. They all have been monitored during the execution process using a fully computerised process control unit ATKWAP, developed by Atlas Palen. The registrated parameters are :

- the vertical pull down or uplift force and the torque exerted on the piling tube during the entire penetration and extraction;
- the vertical and rotational penetration and extraction speed.

The graphs with the boring parameters obtained during the screwing-in of piles P8, P9 and P10 are given in Figure 2.

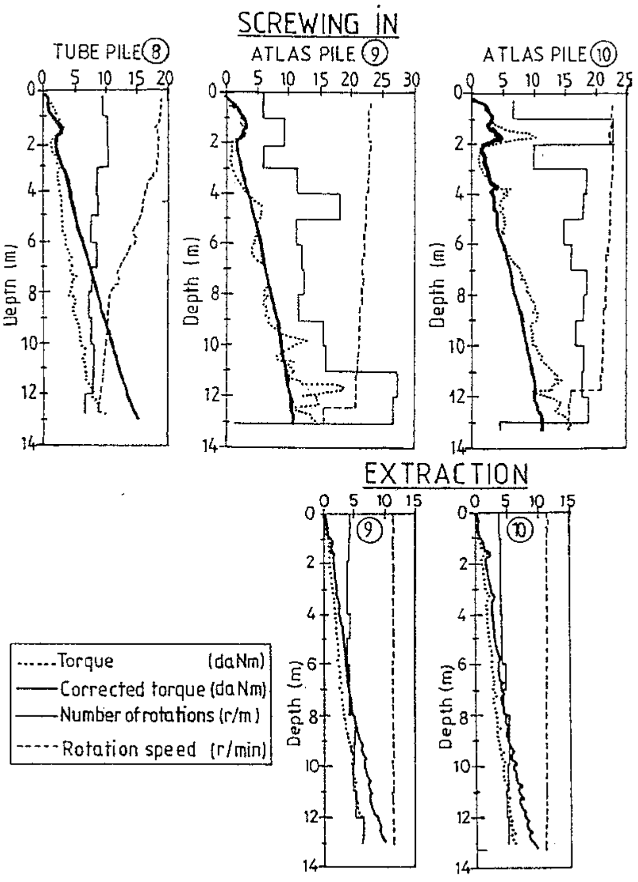


Fig. 2 Pile execution parameters

In what follows, only the static vertical bearing capacity is considered, and consequently all further analysis concerns the 5 first mentioned statically tested compression piles only.

3.3 Pile instrumentation

In the 5 piles considered, a central bottom-closed steel auscultation tube was installed over the entire pile length (Figure 3) This tube allowed the installation of a simple extensometer (tell-tale) in piles P15 and P16 at the bottom of the pile, or of the LCPC recoverable multiple extensometer (LCPC 1990) in piles P8 to P10. In the different piles, also a limited number of strain gauges of the inductif type and with a working length of 0.50 m locally have been installed (Figure 3 and 4).

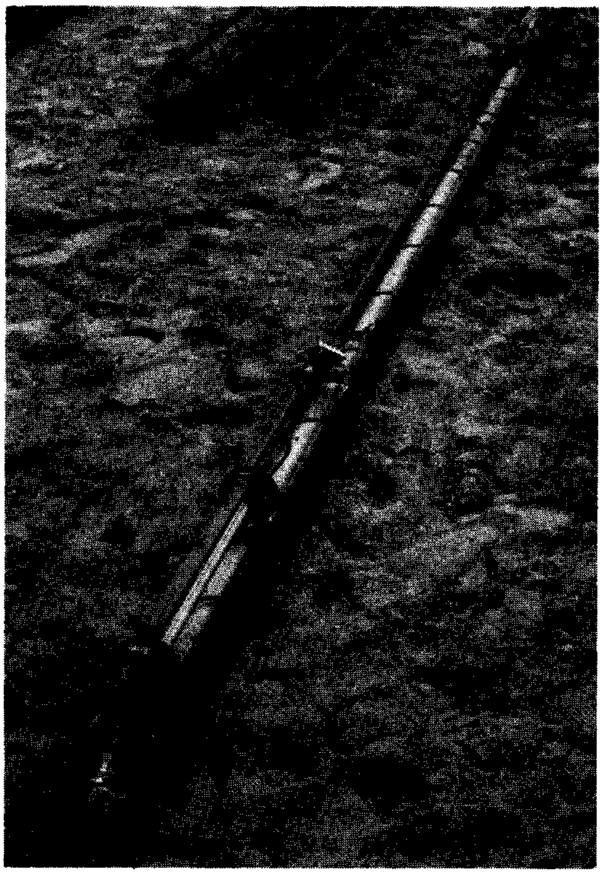


Fig. 3 Auscultation tube

3.4 Static loading tests

The static loading tests on the piles P8 to P9, P15 and P16 have been completely computer controlled with the collaboration of Cementation Piling & Foundations Ltd/UK. A programmable datalogger linked to a pneumatically steered load maintainer unit, an electronic load cell and electronic displacement transducers and a continuous and

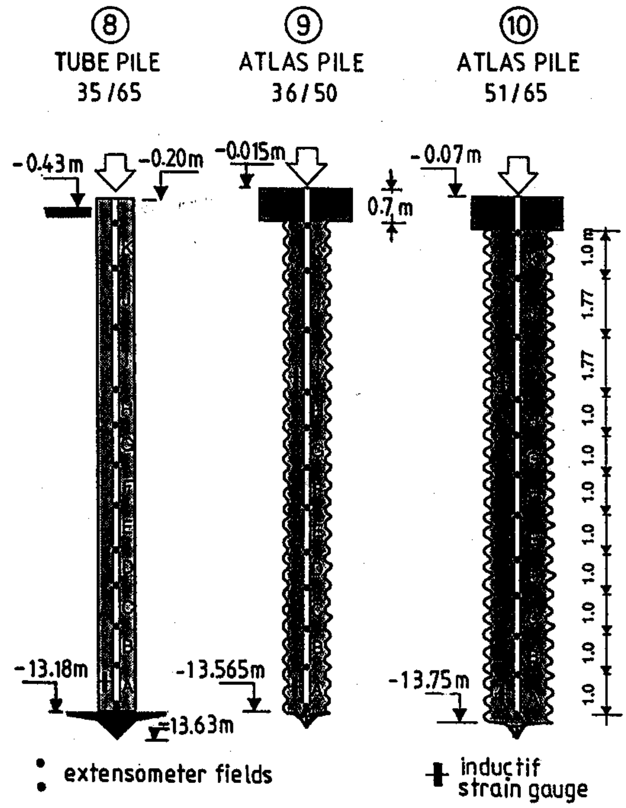


Fig. 4 Pile instrumentation

programmable data acquisition on PC make up the key points of the test equipment and guarantee a correct follow up of the loading schedule and a high accuracy of the measurements (England 1992). Particular computer codes allow for further analysis of the load test results, to extrapolate the observed settlements for each load step to the asymptotic values and to deduce soil parameters and mobilisation curves for pile shaft friction and end bearing separately (Fleming 1992). For the latter analysis, an hyperbolic transfert function is adopted for shaft friction and end bearing.

All load tests have followed a similar loading program :

- a first loading-unloading cycle was performed up to approximately 50 % of the presumed rupture load using 5 load/unload steps;
- the pile was then re-loaded following the same load increments until large displacements were achieved, and finally unloaded.

Particular care has been taken to maintain the different load steps during a sufficient time (in general increasing with increasing load), in order to allow for a reliable extrapolation to the asymptotic settlements and to avoid a ballistic pile behaviour by a too quick application of the next load.

4 TEST RESULTS

4.1 Soil tests

A selection of some representative soil tests (CPT with E1, M1 and M4 cone, DMT and boring log) is given in Figure 5. The results of a detailed comparative analysis of the different soil tests and execution procedures will be published in a later stage.

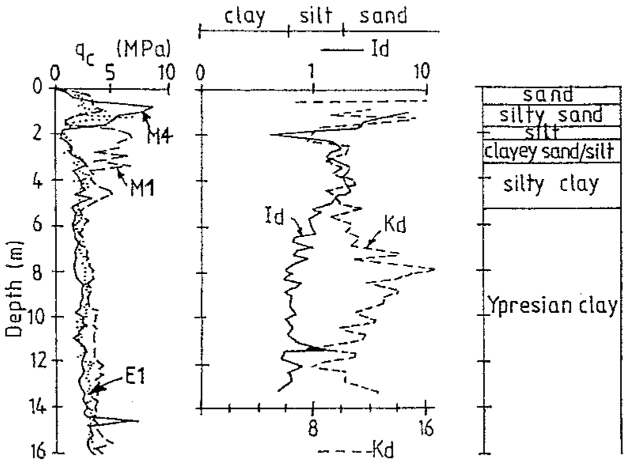


Fig. 5 Results of in-situ testing

The value of the horizontal stress index K_d of approximately 8 to 10 out of the DMT illustrates the strongly overconsolidated character of the stiff Ypresion clay. A first analysis allows for the following correlations of the cone resistances as measured with the different cone types :

- qc(M4) = 1.01 to 0.88 qc(E1);
- qc(M1) = 1.26 to 1.34 qc(E1),

and out of this :

- qc(M1) = 1.25 to 1.50 qc(M4).

Such differences are entirely due to the cone type on the one hand, and the execution method on the other hand. The first value resp. the second value of the above given ranges is valid at a depth of 5 m resp. 15 m under ground level. These magnitudes correspond quite well with the correlations found in Kontich for the overconsolidated Boom clay (I.W.O.N.L. 1977)

- qc(M4) ≈ 1.00 qc(E1);
- qc(M1) = 1.35 à 1.45 qc(M4).

Results of CPT and DMT before piling have also been compared with the results obtained in similar tests executed after piling at a distance of 1.5 X the pile diameter from the pile axis. It appears that the changes in the q_c-values after and before pile installation are of the order of -5% up to +10%; although these values are of the same order of magnitude as the dispersion that already may occur in the clay, they certainly proove the abscence of loss of soil strength due to the pile installation. On the other hand, the DMT before and after pile installation allow to demonstrate more clearly the beneficial result of the displacement effect of the Atlas pile, even in the considered overconsolidated clay. As an illustration, the results obtained nearby the Atlas screw pile P16 are given in Figure 6. Comparison of the different DMT nearby the 5 test piles have given the following mean increase in K_d values :

- tube pile P8 : 1.22;
- Atlas pile P9 and P15 : 0.98 resp. 1.04;
- Atlas pile P10 and P16 : 1.14 resp. 1.16.

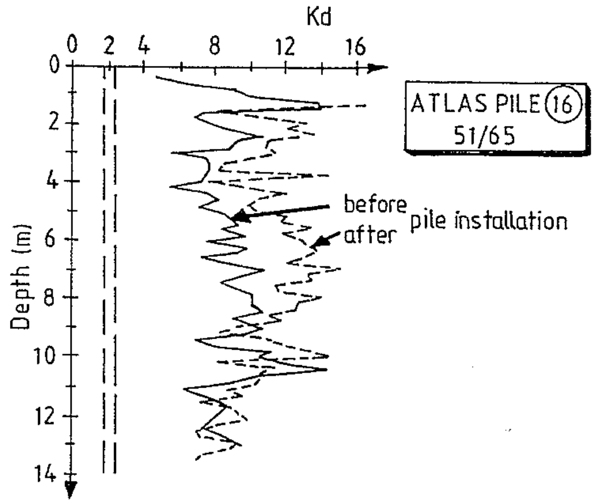


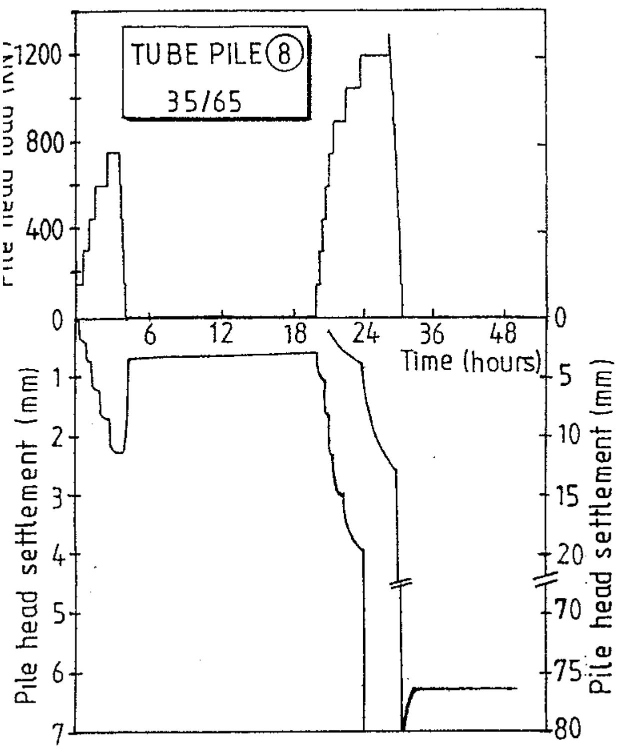
Fig. 6 DMT test before and after pile installation

The results of the comparative study as well as the results of the stress measurements in the soil nearby the piles during the execution of the piles, are discussed in detail in a contribution of Peiffer et al to this Seminar.

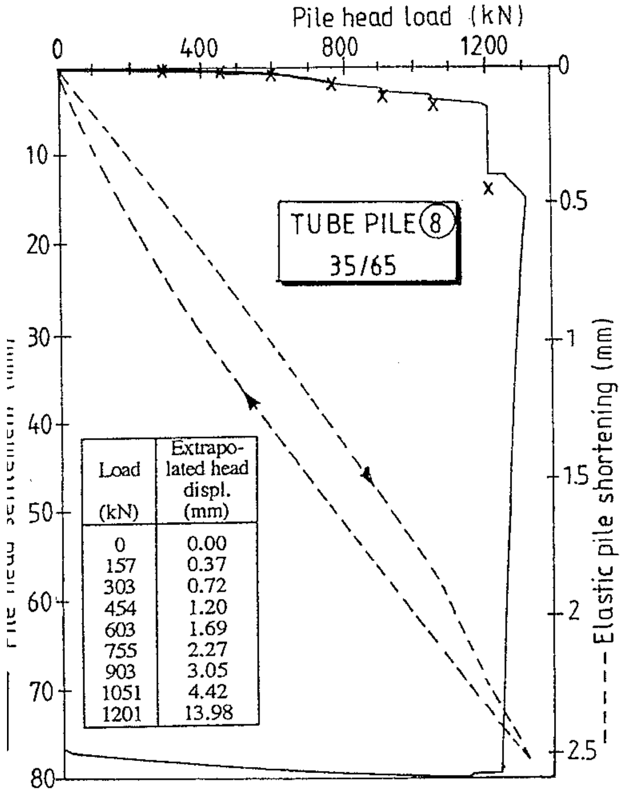
4.2 Static loading tests

The results of the static loading tests on piles P8, P9 and P10 are given in Figures 7, 8 and 9 respectively. The following data are presented :

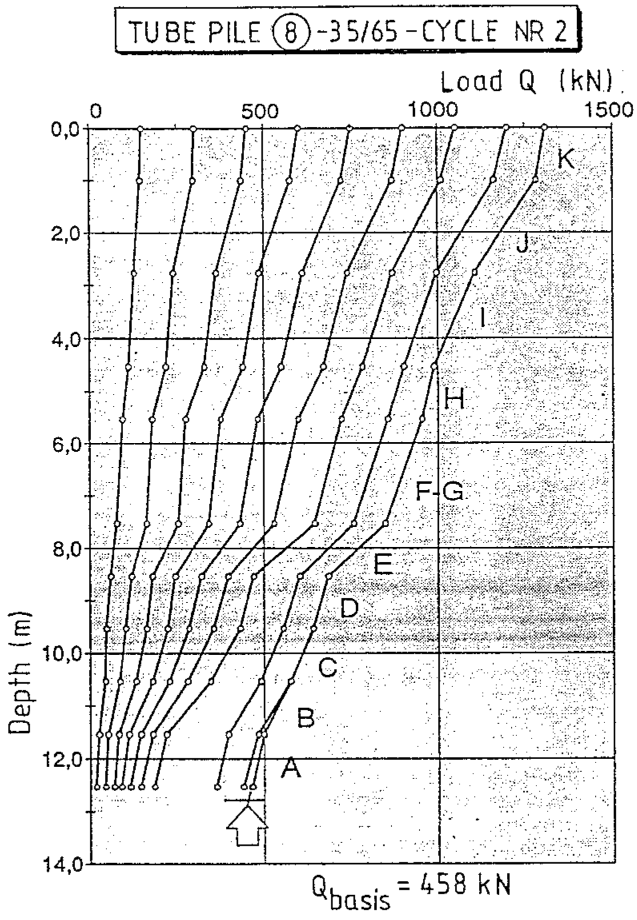
- in the graphs a : the evolution of the applied load Q and the pile head displacement s_t as a function of time;
- in the graphs b : the evolution of the measured pile head displacement s_t and the total elastic shortening s_e as a function of the applied load Q; the tabeled values as well as the cross marks correspond to the theoretical asymptotic displacements for the second loading cycle, as determined by a TIMESET analysis for each single load step, giving thus a theoretical approach of the unique pile behaviour;
- in the graphs c : the load distribution in the pile for the different load steps applied in the second loading cycle.



a)

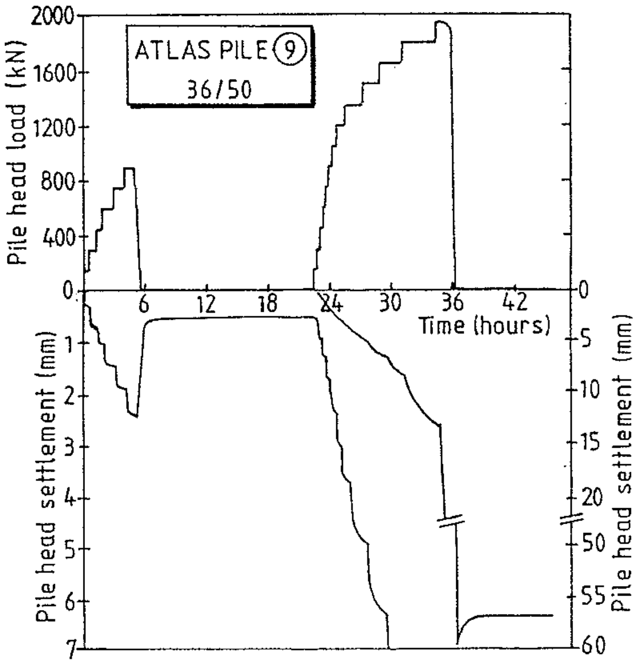


b)



c)

Fig. 7 Static loading test pile 8



a)

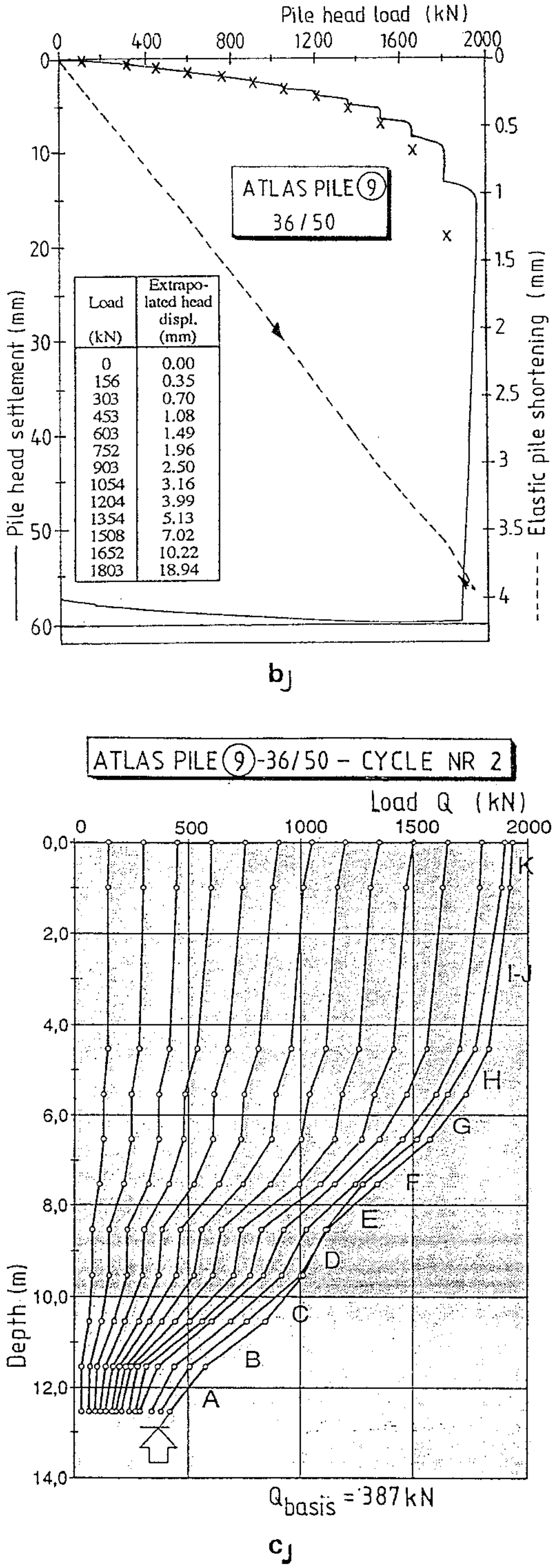


Fig. 8 Static loading test pile 9

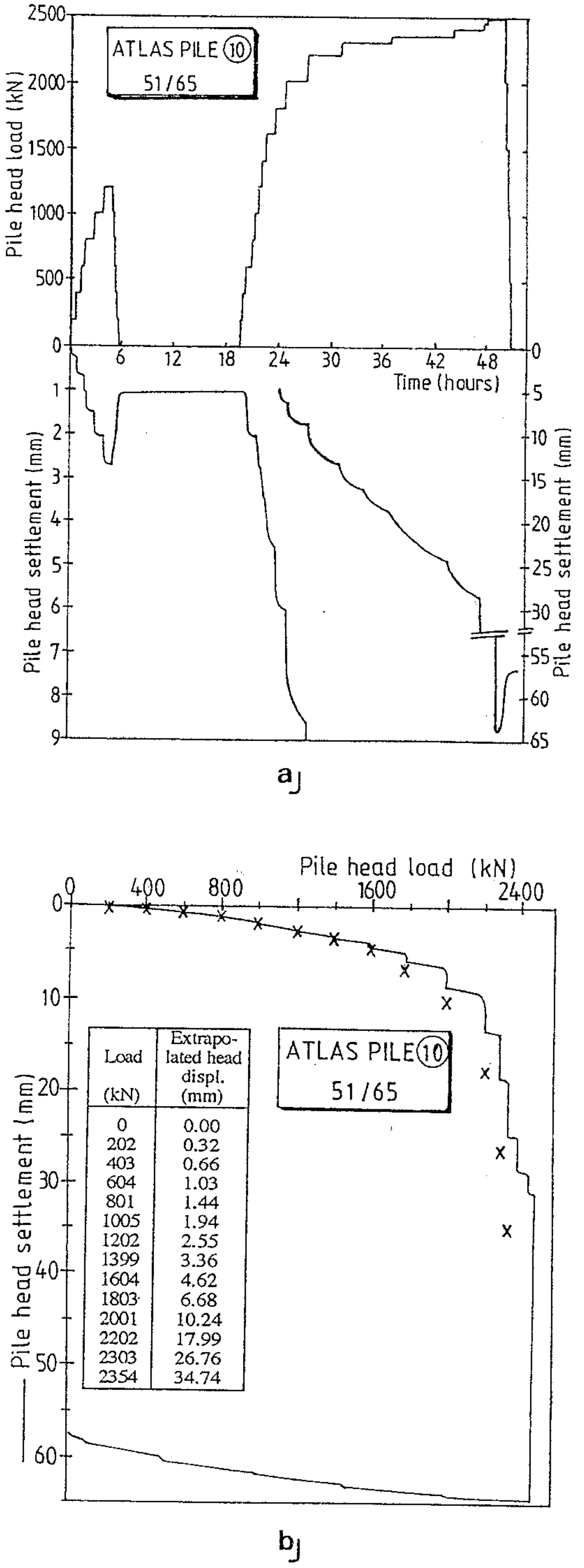


Fig. 9 Static loading test pile 10

5 ANALYSIS OF LOADING TEST RESULTS WITH RESPECT TO PILE PERFORMANCE

Preliminary remark

In what follows, the next nominal shaft diameter $D_{s,n}$ and base diameter $D_{b,n}$ have been assumed :

- for the Atlas piles : $D_{s,n} = D_f$

  $D_{b,n} = 0.9 \times D_f$
- for the tube piles : $D_{s,n} = D_s$

  $D_{b,n} = D_f$

The ultimate end bearing capacity used for prediction of the total end bearing $Q_{r,b}$ of the pile is obtained from : (Van Impe 1986 and 1988)

$$q_{r,b} = \alpha_b \cdot \varepsilon_b \cdot q_{r,b}^*$$

in which :

$q_{r,b}^*$: the ultimate end bearing capacity obtained from the CPT-test result

α_b : a coefficient taking into account the installation procedure of the pile

ε_b : a parameter referring to the scale effect from the fissuring of the soil. For the Boom clay in Belgium one proposes :

$$\varepsilon_b = 1 - 0.01\left(\frac{D}{d} - 1\right)$$

For the Atlas pile the experimental test results fit well with the theoretical prediction for α_b, namely $\alpha_b = 1.0$.

The tube pile in this test program behaves differently. There we found a total factor ($\alpha_b \times \varepsilon_b$) = 0.61 at 10 % normalised pile base displacement, out of which $\alpha_b = 0.74$.

In Figure 10 the normalised unit end bearing ($\frac{q_b}{q_{r,b}^*}$) is plotted against the normalised pile base displacement $\frac{s}{D_b}$ (q_b : measured tip load, $q_{r,b}^*$: unit bearing capacity calculated from the electrical CPT test).

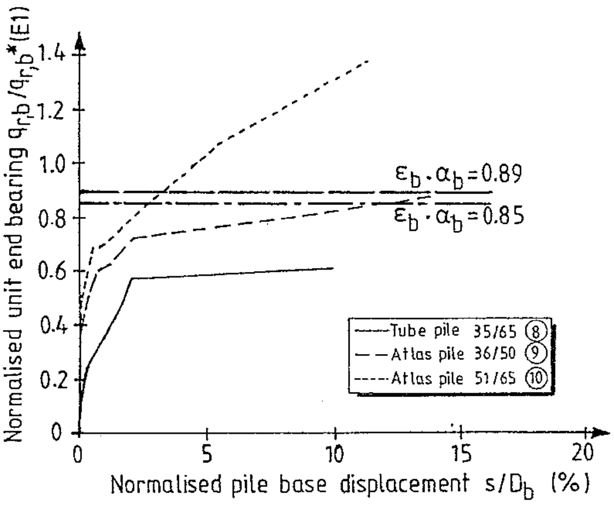


Fig. 10 End bearing capacity

For a displacement type of pile α_b equals at least $\alpha_b = 1$.

Several methods have been proposed for determination of the ultimate skin friction $q_{u,s}$. In the following chapters, only the behaviour in the Ypresian clay is discussed in this respect.

A first method is based on the use of direct $q_{u,s} \longleftrightarrow q_c$ relationship, proposed for some type of piles by Van Impe (1986). For the single Atlas pile, the ratio η_p of ultimate unit skin friction $q_{u,s}$ to cone resistance q_c in this test program amounts to about 0.044.The tube pile in the test set up on the contrary leaded to a lower ratio η_p of about 0.025 (Fig. 11).

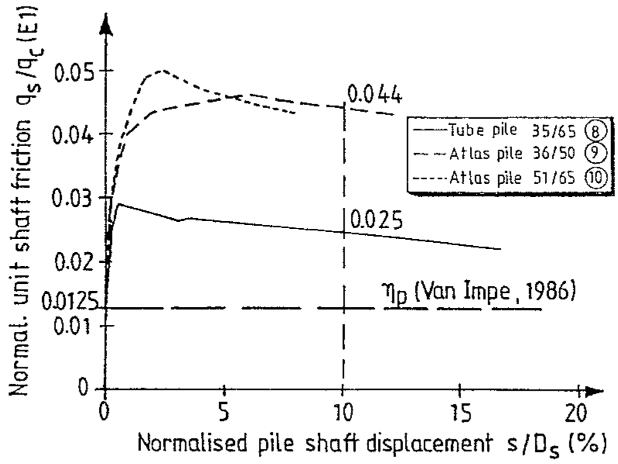


Fig. 11 Shaft bearing capacity based on q_c-measurement

Also interesting to see is the fact that the shaft bearing capacity fully is mobilized at about 1 % relative displacement for the tube pile, and about 2 % for the Atlas pile. This suggests a more pronounced soil slip behaviour along the smooth tube pile shaft. The displacement required to fully mobilize the skin friction can only depend on the pile diameter in cases of saturated cohesive reconsolidated soils and real displacement piles. The relative soil-pile shaft movement although is not linked to any variable thickness of the interacting soil element in case of slip phenomena.

The shaft bearing capacity also can be calculated out of the total shaft friction on the rods during a CPT test.

$$Q_{r,s} = F_{tot\ CPT} \cdot \frac{f_{pile}}{f_{cone}} \cdot \xi_f$$

For Atlas piles Van Impe (1988) proposed $\xi_f = 1.25$ as a safe estimation. In Figure 12 it becomes clear that the results are in good agreement with the measurements done in Zwevegem. For the tube pile the back-calculated value for the ratio measured shaft friction on the pile to total friction on the CPT-rods corrected by the diameter ratio was 0.75.This difference suggests a partial neutralization of the displacement effect immediately near the pile shaft after passing by of the auger.

Although no many references are available in literature, it is also possible to calculate the ultimate unit skin friction directly out of the local skin friction measered in the CPT. The correction factor, at 10 % pile shaft displacement is about 0.63 for the tube pile and 1.12 for the Atlas pile. This factor is 12 to 20 % less than the ξ_f parameter, what could be expected.

In the following table the results of the pile load tests and the different calculation approaches are gathered together.

For pile P15 (Atlas 36/50) and P16 (Atlas 51/65) the observed behaviour was identical to that of pile P9, resp. pile P10.

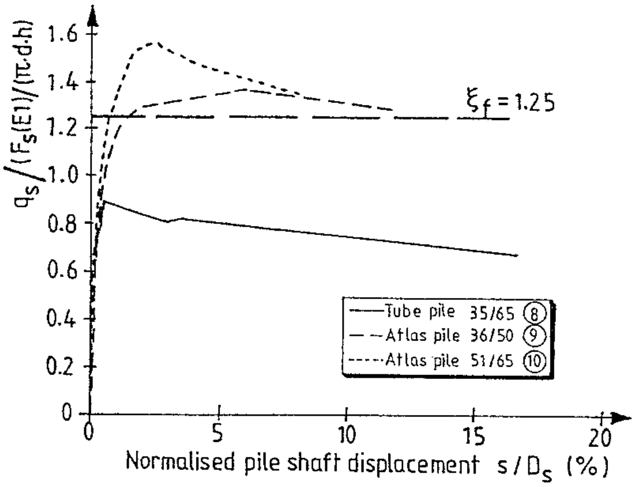


Fig. 12 Shaft bearing capacity based on total friction measurment

TABLE

Pile nr.	Pile type	Measured part of the bearing capacity in the clay layer			$\alpha_b \cdot \varepsilon_b$ for $\frac{s}{\phi_b} = 10\ \%$	$\frac{q_{u,s}}{q_c}$ for $\frac{s}{\phi_s} = 10\ \%$	ξ_f for $\frac{s}{\phi_s} = 10\ \%$	Settlement s_r for $\frac{s}{\phi_b} = 10\ \%$ (pile head)
		$Q_{r,b}$ (kN)	$Q_{r,s}$ (kN)	Q_r (kN)				
8	tube ϕ36/65	458	451	909	0.61	0.025	0.75	61
9	Atlas ϕ 36/50	387	1385	1771	0.83	0.044	1.31	58
10	Atlas ϕ 51/65	860	1753	2613	1.31	0.043 (*)	1.35 (*)	55

(*) $\frac{s}{\phi_s} = 8\ \%$

Note : all values are based on the results of the electrical cone CPT test.

6 CONCLUSIONS

In the framework of this extensive research project, many geotechnical topics have been included. Besides a comparative analysis of different in situ soil tests, the mean topic dealt with the behaviour of the Àtlas screw pile and the tube screw pile in stiff overconsolidated clay. The outcome was showing the displacement pile character of the screwed Atlas pile, resulting in an increased shaft bearing capacity, In the tested piles rupture loads in the range of 2.000 to 2.500 kN have been reached. For the tube screw pile, the smooth shaft surface and the remoulding of the surrounding soil by the passing of the enlarged flange somewhat neutralize the beneficial effect of soil displacement, resulting in a reduced pile bearing capacity. The test rupture load was of the order of 1.200 kN.

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