

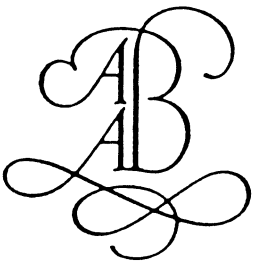
PROCEEDINGS OF THE 3RD INTERNATIONAL GEOTECHNICAL SEMINAR ON DEEP
FOUNDATIONS ON BORED AND AUGER PILES/GHENT/BELGIUM/19-21 OCTOBER 1998

Deep Foundations on Bored and Auger Piles BAP III

Edited by

W.F. VAN IMPE & W. HAEGEMAN

Laboratory of Soil Mechanics, Ghent University, Belgium



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Load-settlement behaviour versus distinctive Ω -pile execution parameters

W.F. Van Impe & P.O. Van Impe

Laboratory of Soil Mechanics, Ghent University, Belgium

C. Viggiani & G. Russo

University of Napoli 'Federico II', Italy

M. Bottiau

S.A. Socofonda, Brussels, Belgium

ABSTRACT: At the test site of Feluy in Belgium, four Omega screw piles were extensively tested up to failure. Changing for each of the piles some basic installation parameter the research aimed at determining the resulting overall installation parameter reflected in the pile capacity design procedure.

1. INTRODUCTION

The scientific and technological principles of the new Omega type pile were proposed early 1993. The Omega type pile (fig. 1) is an in situ casted concrete auger pile. For control of the new technology and in

order to check the overall behaviour of the pile due to some specific installation details, another group of field load tests was performed in 1997.

The shape of the auger was developed in such way so as to ensure that the increasing volume of the transported soil between the flanges of the screw can be stored at each level, at a given rotational speed (n) and vertical translation speed (v). This leads to a very efficient low energy screwing sequence of the auger head and consequently to a higher penetration rate and better soil displacement of the casted pile.

The auger is indeed screwed in with lateral soil displacement. An additional thrust can be activated. Once the required depth is reached, concrete is casted under pressure through the hollow stem of the auger while the auger is continuously retrieved while rotating in the same direction as for the downward screwing sequence. The tip underneath the auger displacement body is left behind. The reinforcement can be introduced either before or after the casting of concrete. Details on this pile type, pile dimensions and specific execution parameters are discussed in another paper (Bottiau et al.) to this conference.

2. DISTINCTIVE PILE INSTALLATION PROCEDURES

At the occasion of the new research program, four instrumented Omega piles were installed at this test site. The location of the piles and typical soil conditions is shown in fig. 2. Table 1 summarises the main characteristics of each pile as follows :

Omega pile 1 : casing diameter 0.41m, was not installed at the time of the pile test loading program

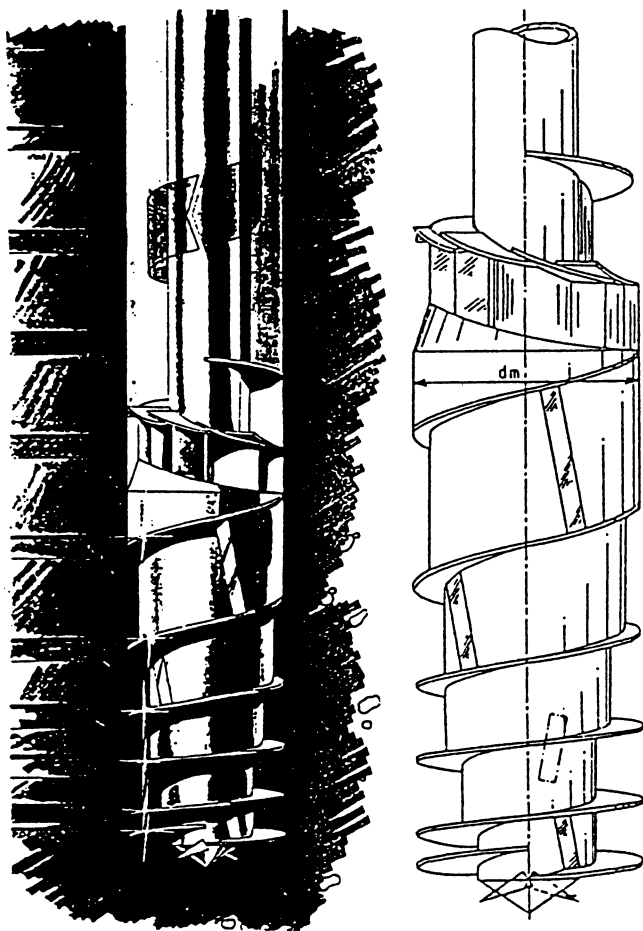


Fig. 1. The Omega pile - Van Impe et al., 1993

Table 1

| Nr. Ω | Casing diameter (cm) | full length (m) | concrete- type | tell-tale rods at level : (m) below GL) | Comments |
|-----------------|----------------------------|-----------------------|----------------------------------|---|--|
| 2 | 36 | 14.5 | slumb 20 | TTR1 : 5.5 | casted under gravity |
| 3 | 36 | 14.5 | slumb 20 | TTR2 : 10.75 TTR1 : 5.5 TTR2 : 9.5 | casted with 2 bar overpressure at pump |
| 4 | 36 | 11 | slumb 12 | TTR3 : 14.25 TTR1 : 5.5 TTR2 : 10.75 | expanded tip due to re-entering of auger head, high overpressure at casting (up to 10 bar) |
| 5 | 36 | 11 | slumb 12 (tip) /slumb 20 (shaft) | TTR1 : 5.5 TTR2 : 10.75 | expanded shaft and tip due to re-entering of auger head, high overpressure at casting (up to 10 bar) |

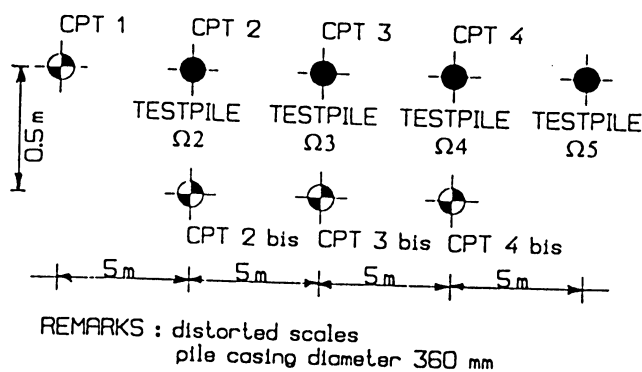


Fig. 2a. Location of the performed CPT's and test piles at the Fély test site

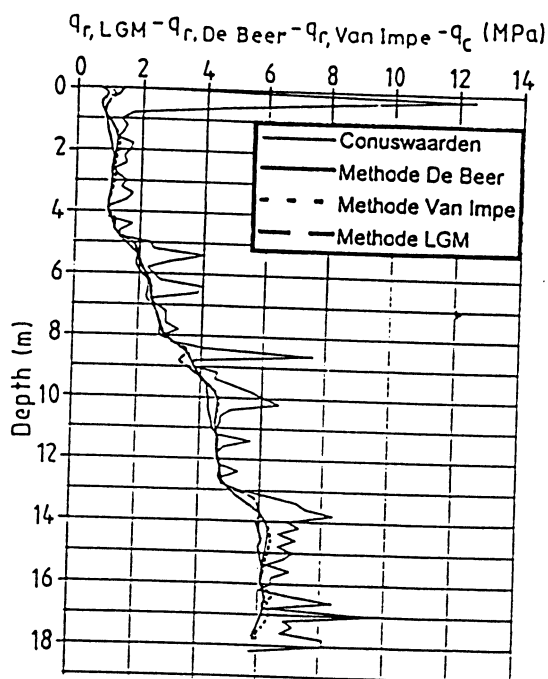


Fig. 2b. Calculation of the point bearing capacity

Omega pile 2 : casing diameter 0.36 m, length 14.5m, casted under gravity

Omega pile 3 : casing diameter 0.36 m, length 14.5m, casted with small (2 bar) overpressure at the pump

Omega pile 4 : casing diameter 0.36 m, length 11m ; increased pile tip diameter due to a combination of dry pile tip concrete and repushing of the casing and auger head into the freshly casted concrete

Omega pile 5 : casing diameter 0.36 m, length 11m ; increased pile tip and pile shaft diameter due to a combination of dry concrete and repushing of the casing and auger head into the freshly casted concrete ; concrete in the shaft is of normal fluidity

3. SUMMARY OF THE RESULTS OF THE PILE TEST LOADING

The static dead-weight loading process was planned in well controlled increasing steps. One single loading procedure was adopted (fig. 3) :

- 6 loading steps with an equal load increment of 150 kN, the time interval for each step is one hour
- followed by a number of loading steps with an equal load increment of 75 kN until the pile head settlement reaches a value of 15 % of the pile casing diameter (i.e. 15 % of 36 cm = 41.4 mm)
- 8 unloading steps with equal load decrement, the time interval for each step is 10 minutes

During each load-step, measurements were recorded at 0', 1', 2', 3', 4', 5', 10', 15', 20', 25', 30', 40', 45', 50' and 60'. During each unloading step, measurements were made at 0', 1', 5' and 10'.

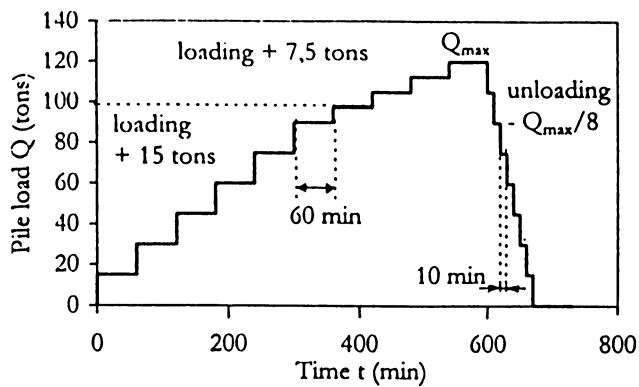


Fig. 3. Loading program for Omega pile 2

During last unloading step, (load is reduced to zero), additional measurements were made at 20', 30' and 60'.

The pile load test on Omega pile 2 was performed in October 6th 1997. The critical pile head displacement of 41.4 mm was reached at a load of 1200 kN. Fig. 4 shows the load/pile head settlement curve for this pile.

The pile load test on Omega pile 3 was performed on October 8th 1997. The critical pile head displacement of 41.4 mm was reached at a load of 1350 kN. Fig. 4 shows the corresponding load/pile head settlement curve. As can be seen in fig. 5, the tell-tale rod reading of the vertical displacement at a depth of 9.5 m indicates larger values than the one at 5.5 m. This suggests the malfunctioning of the tell-tales, since negative skin friction can be excluded here.

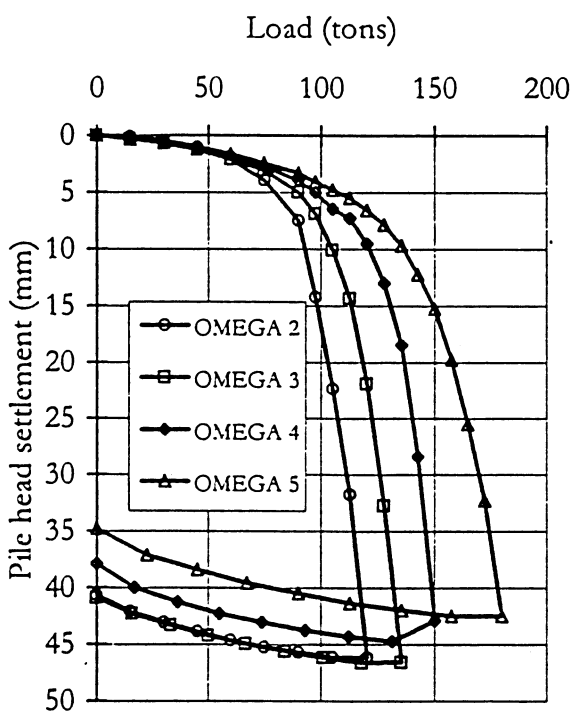


Fig. 4. Pile load test results

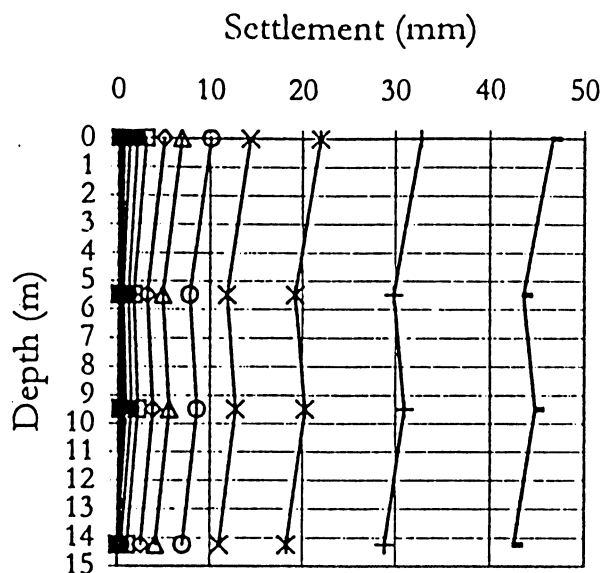


Fig. 5. Tell-tale rod displacement vs. load Omega-pile 3

The pile load test on Omega pile 4 was performed on October 10th 1997. The critical pile head displacement of 41.4 mm was again reached at a load of 1500 kN. Fig. 4 shows the corresponding load/pile head settlement curve.

The pile load test on Omega pile 5 was performed on October 14th 1997. The critical pile head displacement of 41.4 mm was reached at a load of 1800 kN. Fig. 4 also shows the load/pile head settlement curve for this pile.

In case of Omega pile 5, we have a larger variation in vertical displacement between the three pile head dial gauge readings, than for the other piles. This can be seen in fig. 6. Again it is clear,

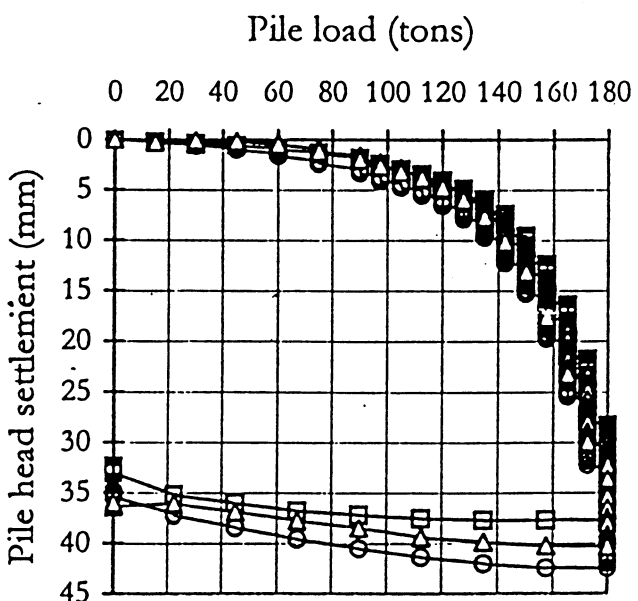


Fig. 6. Load pile head settlement curve Omega pile 5 (separate curve for each dial gauge)

when analysing fig. 7, that the displacement measures obtained from the tell-tale rods seem to be somewhat inaccurate.

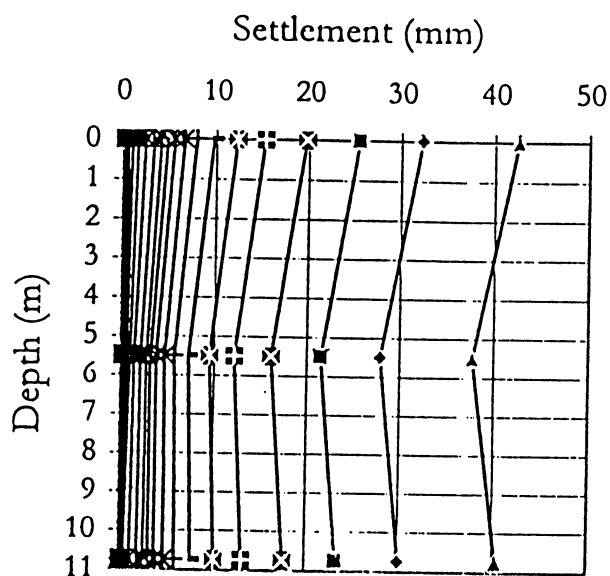


Fig. 7. Tell-tale rod displacement vs. load Omega Pile 5

4. METHODS FOR PILE TEST RESULTS ANALYSIS

4.1. The completed data of the tests hold readings on load measurements, pile head vertical displacement measurements, tell tale rod displacement measurements, horizontal displacement measurements of the pile head, control measurements of the pile diameter, and measurement of the pile material's Young modulus after pile installation.

Some of these data show quite a large scatter. First of all, already during the performance of the tests, it was clear that most of the tell tale rod displacement readings were questionable. Secondly, as the pile diameter and pile material modulus have been checked at only a few points along the full pile length, this also holds some danger for smaller inaccuracies.

The pile diameter evaluation was based on the in situ measured concrete consumption during pile concrete casting. In this respect, the real measured concrete consumption, adapted for the measuring pump device calibration, resulted in :

Concrete consumption Ω -pile 2 : 1.33 m^3 .
 Concrete consumption Ω -pile 3 : 1.48 m^3
 Concrete consumption Ω -pile 4 : 1.35 m^3
 Concrete consumption Ω -pile 5 : 3.09 m^3

Those consumption values are obtained taking into account the very specific and well distinguished execution methods, concrete compaction and percentage of spreading out (cfr. paper M. Bottiau et al. to this conference), and with each pile length carefully measured, one comes to the following pile diameter estimations :

Ω -pile 2 : base diameter = shaft diameter = 0.342 m (length = 14.5 m)

Ω -pile 3 : base diameter = shaft diameter = 0.36 m (length = 14.5 m)

Ω -pile 4 : pile shaft diameter (upper part, length 2 m) = 0.36 m , pile shaft diameter (remaining part, length 7 m) = 0.36 m ; pile tip diameter = 0.6 m (length 2 m)

Ω -pile 5 : pile shaft diameter (upper part, length 2 m) = 0.36 m , pile shaft diameter (over a length 7 m) = 0.6 m ; pile tip diameter = 0.7 m (over a length of 2 m)

4.2. The overall prediction factor κ is defined as the ratio of the overall pile capacity out of failure load tests over the predicted overall pile capacity. In our approach, this overall pile capacity prediction follows out of a CPT-based design analysis (De Beer 1972, Van Impe et al. 1986) for soil displacement screw piles. In the idealised case of a displacement screw pile with its well-known pile dimensions, responding exactly to the theoretically assumed failure pattern along pile shaft and pile tip (inherently implemented in this CPT-based prediction method), the value of the overall prediction factor κ should become exactly equal to unity : $\kappa = 1$.

In such case we would be able to say that for this pile type, with its specific dimensions and installation procedure, the capacity can be rationally predicted with the proposed design method for displacement piles.

In case κ deviates from unity, the pile tip or pile shaft interaction failure model differs clearly from the assumed one, expected from the CPT-based soil-displacement behaviour

The conclusion is that, going out from the real pile dimensions, implemented by the installation details of a specific type of screw pile, only an overall prediction parameter of $\kappa = 1$ (or slightly higher) proves that such screw pile behaviour indeed is corresponding to the rational CPT-based design method valid for soil-displacement pile types.

Besides of all considerations on κ stated above, one could also, for a given specific type of screw pile, auger head and casing dimensions, simply compare the outcome of various installation

procedures (type of concrete, way of casting, fresh concrete preloading etc...) based on the measured overall pile capacity as compared to the standard design prediction. In such comparison, the standard pile dimensions are the dimensions taken from the auger head and/or casing, equal for all piles, even with distinctive installation details.

The overall pile capacity Q_{tot}^m as the outcome of "failure" pile load tests could be compared to its predicted capacity $Q_{tot,ref}$ as a reference value. The ratios of $Q_{tot}/Q_{tot,ref}$ do reflect, for this specific pile type and the corresponding standard reference dimensions, the beneficial or the negative influence of the variable installation details. We called this parameter in the corresponding table hereafter the "practitioners' design installation coefficient" (PDI or κ^*).

4.3. As the load on the pile during the test is not maintained until the final full displacement has occurred, the basic load settlement curve, as given by the raw data, is not complete nor ready for full analysis. So, as a first step in the analysis, the 'ultimate' settlement (at $t = \infty$) is approximated. From such adapted curve, the ultimate load of the pile can be calculated, as well as the separate values of the ultimate shaft and base load. All this allows furthermore to derive the κ^* (or PDI) installation parameter.

Analysing the theoretical load settlement curve, one can go out from several methods. Chin assumes the following hyperbolic function between time and settlement :

$$\frac{t}{s} = a \cdot t + b$$

which means that time settlement is a linear function of time (b is a constant). As t goes to infinite, the settlement reaches the value $1/a$. This value can be easily calculated, but is strongly depending on the number of accurately measured settlement data.

As it appears, at large values for time t , some deviation appears between the measured values and the single hyperbolic law according to Chin. Therefore, Fleming et al-proposed to use a double hyperbolic function, in this way separating the time settlement behaviour caused by the tip load and shaft load.

$$s = \frac{W_s \cdot t}{T_s + t} + \frac{W_b \cdot t}{T_b + t}$$

in which

W_s = asymptotic value for shaft related deformation

W_b = asymptotic value for base related deformation

T_s = mobilisation time for half of the shaft related deformation

T_b = mobilisation time for half of the base related deformation

t = is time elapsed from application of the pile head load

s = relative pile head settlement at time t

As the real values of the parameters W_s , W_b , T_s and T_b are not known, they are estimated using a least square approximation using the data of the measured time settlement curve. The final settlement is found adding W_s and W_b .

This seems to be working well when looking at time settlement behaviour at large loads. At small loads however, we do find a too small correlation. The solution according to the Chin method is strongly depending on the number of data used. The TIME-SET method on the other hand has a very low correlation at low loads, which does not allow to find a solution for W_s and W_b . This forces us to stick here with the CHIN method.

4.4. On the problem of the analysis of the overall ultimate load estimation, one could also apply several procedures :

VAN IMPE method (1986 ; 1994)

This criterion for ultimate load does not depend on the shape of the load settlement curve. The 'conventional' ultimate load is defined as the load which causes a settlement at the pile base of 10 % of the pile diameter in case of displacement screw pile and 25 % to 30 % in case of other auger and bored piles. A problem with this criterion is, in some cases, the uncertain estimation of the pile base settlement and pile base diameter.

CHIN method

This method was based on results of tests with (very stiff as compared to reality) model piles. The following relation between load P and settlement s was proposed :

$$\frac{s}{P} = M \cdot s + B$$

in which B is a constant and $1/M$ is the ultimate value for the load P . This relation is visualised by a line in a $s/P - s$ diagram. As in the Chin method for determining the final load settlement curve, the

number of data used to calculate the value $1/M$ strongly effects the final result. According to Fellenius (1980), in this criterion only measurements can be used showing a settlement larger than the so called Davisson critical settlement. Literature indicates that the Chin-value overestimates the ultimate load by approximately 10%, and therefore in the calculations, the ultimate load is taken 90% of the Chin-value.

BRINCH-HANSEN method

According to Brinch Hansen, the ultimate load is reached when the value of the pile head settlement at this load is twice the value of the settlement at 90 % of this load. The method is actually also based on a hyperbolic load settlement curve.

A second adapted method which is often used, is based on the so called 80 % criterion. Assuming in such case a parabolic curve at larger loads, this criterion can be easily applied as follows. All data are put in a \sqrt{s}/p -s diagram. At increasing loads, one should get a linear curve. The combination P_u and s_u describes the exact ultimate situation only if the point $(0.80 P_u ; 0.25 s_u)$ is a part the curve as well. This principle can be described as follows :

the linear curve : $\frac{\sqrt{s}}{P} = c_1 \cdot s + c_2$

$$P_u = \frac{1}{2 \cdot \sqrt{c_1 \cdot c_2}} \quad \text{and} \quad s_u = \frac{c_2}{c_1}$$

Theoretically, the value of the ultimate load according the 80 % criterion is about 91.67 % of the value according to the Chin method.

DAVISSON method

The ultimate load has been reached when the settlement of the pile head exceeds the critical settlement, defined as :

$$s_p = \Delta e + \Delta s + \Delta s_p$$

In this equation we have :

Δe = the elastic compression of the free standing pile when the total load is taken by the pile base
(= $P1/EA$)

Δs = the elastic compression of the soil underneath the pile base = $1/12$ pile base diameter

Δs_p = limiting plastic soil deformation = 3.81 mm

The value of the pile diameter strongly effects the results of such an approach.

4.5. Finally, regarding the analysis of ultimate base and shaft load estimation, one could go out from :

LEONARDS and LOVELL method

Research on this method indicated the extreme sensitivity to a variation in the parameters. A variation of 1% on the pile diameter leads to a variation of 17 % on the value of the pile shaft capacity $Q_{s,u}$ (Thooft - Van Impe - Van den Broeck, 1988).

For this reason, we have preferred not to implement this method.

CEMENTATION method (CEMSET)

The method is based on the assumption of a hyperbolic load settlement curve.

The CEMSET equations for an *infinitely stiff pile* can be written as :

$$p = \frac{a \cdot \dot{s}}{c + s} + \frac{b \cdot s}{d + e \cdot s}$$

$$a = Q_{s,u}$$

$$b = D_b \cdot E_s \cdot Q_{b,u}$$

$$c = M_s \cdot D_s$$

$$d = 0.6 Q_{b,u}$$

$$e = D_b \cdot E_{sb}$$

with :

P = pile head load

s = pile head settlement at pile head load P

$Q_{s,u}$ = ultimate shaft capacity of the pile

$Q_{b,u}$ = ultimate base capacity of the pile

D_s = pile shaft diameter

D_b = pile base diameter

E_{sb} = secant modulus of the soil below the pile tip

M_s = shaft/soil stiffness factor.

The elastic shortening of a *non infinitely stiff pile* can be found by adding the settlement resulting from the former equation to the one derived from one of the following equations :

$P \leq Q_{s,u}$ (ultimate shaft load capacity not yet fully activated)

$$s = \frac{4}{\pi} \cdot \frac{P \cdot (L_0 + K_E \cdot L_F)}{D_s^2 \cdot E_c}$$

$P > Q_{s,u}$ (ultimate shaft load capacity, fully mobilised):

$$s = \frac{4}{\pi} \cdot \frac{P \cdot (L_0 + L_F) - L_F \cdot Q_{s,u} \cdot (1 - K_E)}{D_s^2 \cdot E_c}$$

with :

L_o = length of the pile with no shaft friction

L_F = frictional length of the pile

K_E = equivalent column length/ L_F

E_c = Young modulus of the pile material.

Combining the equations allows to get a complete picture of the load/settlement curve. The aim is to vary the parameters $Q_{s,u}$ and $Q_{b,u}$ until the simulated curve and the measured load settlement curve fully match.

The parameters K_E and M_s can not be estimated very reliably but they anyhow show very little influence. The value of the Young modulus of the soil underneath the pile tip E_s , on the other hand, influences the results considerably. To eliminate this problem, this parameter E_s can be solved from the above equations as an unknown.

As the CEMSET method defines the 'ultimate' load as the load at which 'infinite' displacement of the piles occurs (an asymptotic value), this method will always result in larger values for the ultimate load, as compared to other methods, going out from smaller strain levels.

5. DISCUSSION ON THE IMPLEMENTED METHODS

5.1. The Chin method has been used to determine the settlement at 'infinite loading time'. Starting from the measured pile displacements as a function of time, we derive the relative displacement which has occurred since the load was applied. The immediate pile head settlement at load application is not taken into account. The relation time versus relative settlement in function of time can be examined. We compute the parameters of the linear curve at a best fit for all points in the previous mentioned relation.

The figures 8 to 11 illustrate the results of this type of analysis.

5.2. The ultimate load from pile tests has been derived using several methods : the Van Impe-method, the Chin method, the Davisson method, the Brinch Hansen method, These methods were all applied on a load settlement curve as determined here by the Chin method (described in the section above).

As the Van Impe method goes out from the pile base settlement, the Chin time-method had to be used on the pile base settlement data as well in order to find the 'final' base settlement. In order to use the

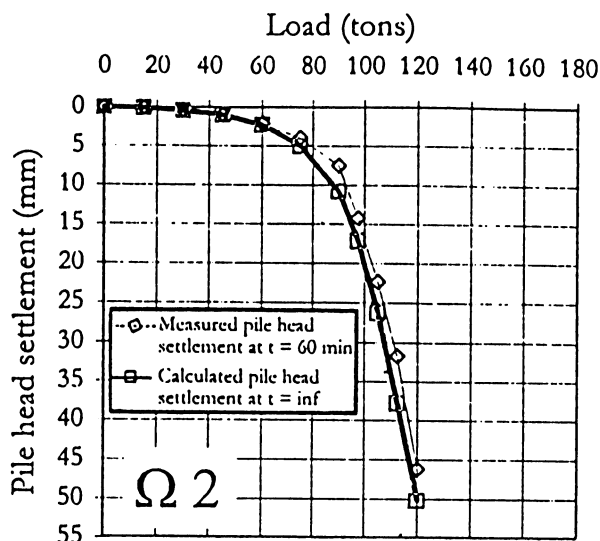


Fig. 8. Calculated versus measured pile head settlement Omega 2

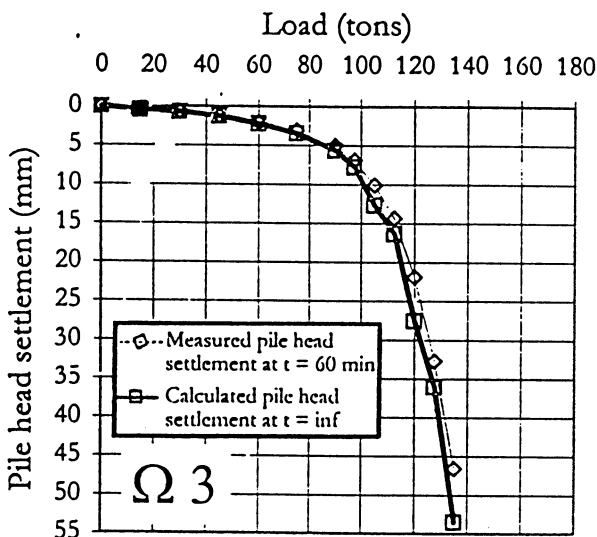


Fig. 9. Calculated versus measured pile head settlement Omega 3

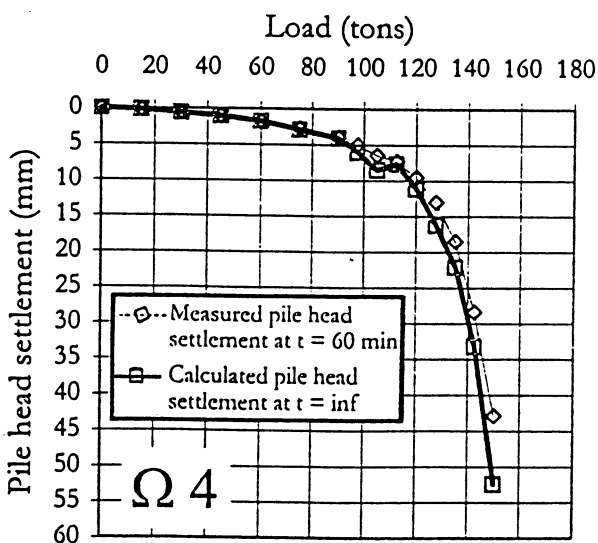


Fig. 10. Calculated versus measured pile head settlement Omega 4

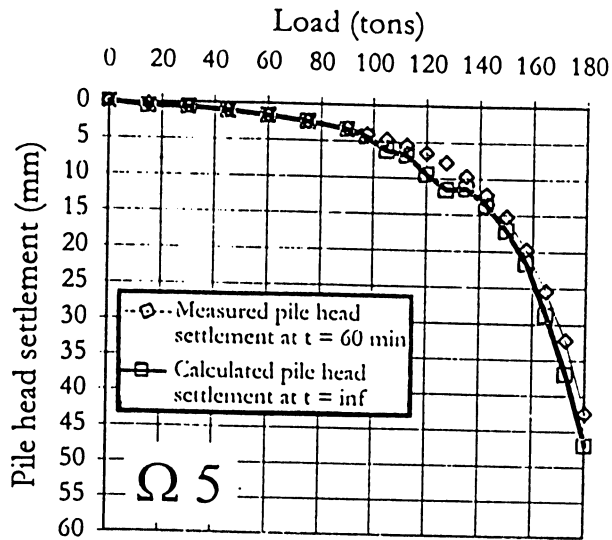


Fig. 11. Calculated versus measured pile head settlement Omega 5

Davisson method, a well corresponding Young modulus of the pile material of 30000 MPa was implemented. The results of the various methods are listed in table 2.

Table 2

| | OMEGA 2 | OMEGA 3 | OMEGA 4 | OMEGA 5 |
|---|---------|---------|---------|---------|
| Prediction based on CPT (estimated diameters) | | | | |
| Van Impe - min | 1053 | 1027.6 | 1381 | 1888.4 |
| De Beer max | 1070 | 1036.4 | 1389.7 | 1904.8 |
| mean | 1061.5 | 1031.97 | 1385.35 | 1896.56 |
| Prediction based on load-settlement curve | | | | |
| Van Impe | 1130.89 | 1287.7 | 1543.7 | 2030.3 |
| 90 % Chin min | 1105.8 | 1262.3 | 1406.3 | 1736.8 |
| max | 1244.3 | 1423.0 | 1494.2 | 1934.2 |
| mean | 1171.9 | 1317.5 | 1443.7 | 1790.16 |
| Davisson | 1159.6 | 1314.6 | 1480.5 | 1901.2 |
| Brinch min | 1201.6 | 1346.7 | 1489.6 | 1803.2 |
| Hansen max | 1230.8 | 1398.9 | 1513.2 | 1891.0 |
| mean | 1218.0 | 1371.6 | 1499.8 | 1841.5 |
| Mean value (kN) | 1195.0 | 1344.6 | 1471.8 | 1815.8 |
| (only 90 % Chin & Brinch-Hansen values) | | | | |

The 90% Chin-value corresponds very well to the values from the Brinch-Hansen method. For OMEGA-pile 2 and 3, also the Van Impe method and the Davisson method lead to very similar results, which are slightly smaller than the Brinch Hansen en Chin results. For the other OMEGA-piles on the other hand, the Van Impe and Davisson methods lead to much larger values, suggesting a probable overestimation of the assumed pile-shaft and pile-base diameters of the OMEGA-piles 4 and 5. To eliminate the influence of an uncertain pile diameter, only the results from the Chin and Brinch-Hansen methods were retained, because of their insensitivity to some dimension variability.

5.3. Starting from the values obtained in the previous paragraph the prediction κ -value and the installation coefficient PDI can be computed :

$$\kappa = \frac{Q_{u,tot,loadtest}}{Q_{u,tot,CPTpredicted,(realdiameters)}}$$

$$\kappa^* = PDI = \frac{Q_{u,tot,loadtest}}{Q_{u,tot,CPTpredicted,(0.36mdiameter)}}$$

The results are summarised in table 3.

Table 3

| | OMEGA 2 | OMEGA 3 | OMEGA 4 | OMEGA 5 |
|--------------|----------|----------|----------|----------|
| κ (-) | 1.134825 | 1.308458 | 1.065727 | 0.953286 |
| PDI (-) | 1.050986 | 1.308458 | 1.897099 | 2.340575 |

As seen, the κ -factor reaches a value close to unity for the OMEGA-piles 2, 4 and 5, indicating that these piles indeed behave according to the failure model of the prediction method. In case of OMEGA-pile 3, on the other hand, a quite large value of κ is estimated. In this case, other factors dominate which apparently cannot be included in the failure system, inherently linked to the De Beer - Van Impe design method.

The PDI coefficient on the other hand is a real installation coefficient, gradually increasing from OMEGA-piles 2 to 5. The larger value in case of the OMEGA-pile 3, as compared to the value for OMEGA-pile 2, indicates that a small change in execution parameters can induce a far better displacement behaviour of the pile.

5.4. In order to obtain the ultimate base and shaft load capacity of each pile, we finally have preferred to use the CEMSET method mentioned earlier. The equations are solved for 3 unknown parameters : the total ultimate load $Q_{u,tot}$, the ultimate shaft load $Q_{u,s}$, the secant stiffness modulus of the soil E_b below the pile tip ; and fixing the other parameters.

The ultimate base load is solved automatically from $Q_{u,tot}$ and $Q_{u,s}$. The equivalent column length factor K_e and the shaft/soil flexibility factor M , have been fixed at a constant value of 0.5 resp. 0.00125 for each pile.

5.5. In order to estimate the partial installation coefficients α_b and ξ_f , we used the ultimate shaft and ultimate base capacity out of the CEMSET method. In figure 12, Q_s^* and Q_b^* represent respectively the ultimate shaft load and the ultimate base load

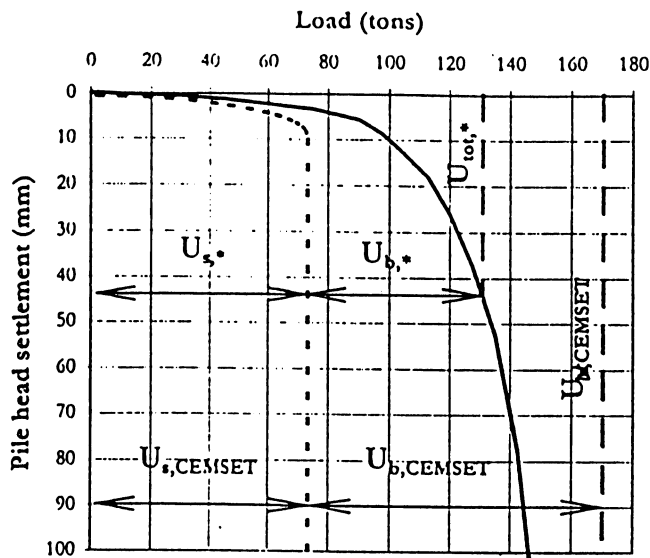


Fig. 12. Shaft load -base load vs. settlement

corresponding with the total ultimate load Q_{tot}^* as the mean value of the ultimate load from the 90% Chin & Brinch-Hansen criteria.

One consequently obtains :

$$\xi_{f,max} = \frac{Q_{s,CPTpredicted,max}}{Q_s^*} \text{ and}$$

$$\xi_{f,min} = \frac{Q_{s,CPTpredicted,min}}{Q_s^*} \text{ and}$$

$$\alpha_b = \frac{Q_{b,CPTpredicted}}{Q_b^*}$$

For all piles, the resulting shaft coefficients ξ_f at this test site do show very high values, reaching up to nearly a value of 3. On the other hand, the resulting base coefficients α_b tend to have unrealistic low values. This both ends discrepancy indicates clearly that, in these OMEGA-pile cases, the CEMSET method is not correctly dividing the total ultimate load into ultimate shaft and ultimate base capacity. Therefore a more appropriate analysis was proposed as described in the next paragraph.

6. DETERMINATION OF THE SHAFT AND BASE INSTALLATION COEFFICIENTS.

6.1. The analysis of the load-settlement behaviour has been carried out by means of the code SINGHYP, developed at DIG - Naples, Italy.

The pile is modelled as an elastic body and the soil as a horizontally stratified elastic half space. The connection between the surrounding soil and the

shaft of the pile is rigid plastic, with an ultimate resistance to sliding (either cohesive or frictional). The Interaction between the soil and the pile base is modelled via a hyperbolic spring, with an asymptote as ultimate base resistance of the pile and with the initial stiffness defined either (i) by the elastic properties of the soil, or (ii) by imposing that 90% of the ultimate base resistance is attained at a specified relative settlement (10% or 20% of the base diameter).

In the former case, the required parameters are the same as for the prediction of the ultimate pile capacity, as well as the elastic properties of the soil. SINGHYP represents a development of SINGPALO (Mandolini & Viggiani, 1997), with the addition of the base spring.

6.2. If a load test on the pile is available, the parameters are, by preference, obtained by fitting the results, as suggested by Mandolini & Viggiani (1997). The fitting procedure is simplified adding the base spring. As a rule, the above option (ii) is more convenient, selecting the base settlement as a function of the pile installation method.

6.3. As a first attempt, SINGHYP has been used for a class A prediction of the piles behaviour based on the CPT-results. The ultimate lateral and base resistances have been obtained by the Van Impe method (1986,1988), neglecting the shaft capacity over one metre of pile shaft at the top and above the pile tip. The undrained elastic moduli of the soil have been correlated to the undrained shear strength in the following way :

$$c_u = \frac{(q_c - \sigma_v)}{15} \text{ and}$$

$$\frac{E_u}{c_u} = 500 \text{ (slightly OC silty clay of low plasticity)}$$

Values of $E_u \approx 33 q_c$ have been suggested. The predicted and observed load-settlement curves are reported in fig 13a through 13d.

The predictions of the four OMEGA-pile tests differ from each other only due to the slightly different CPT-profiles at the locations of the different piles, and the geometrical differences (diameter of the shaft and of the base, pile length) among the four piles.

The agreement between the predicted and observed behaviour is very acceptable as far as the initial load-settlement behaviour is concerned ; on the contrary, the pile capacity in some cases seems severely underestimated.

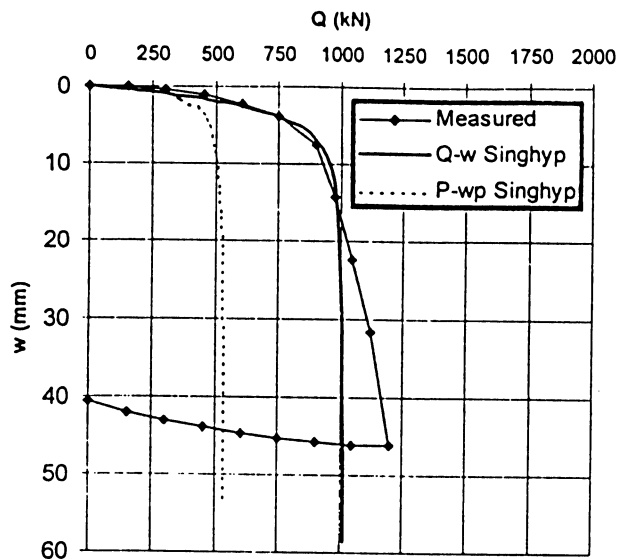


Fig. 13a. Load versus settlement - Omega 2

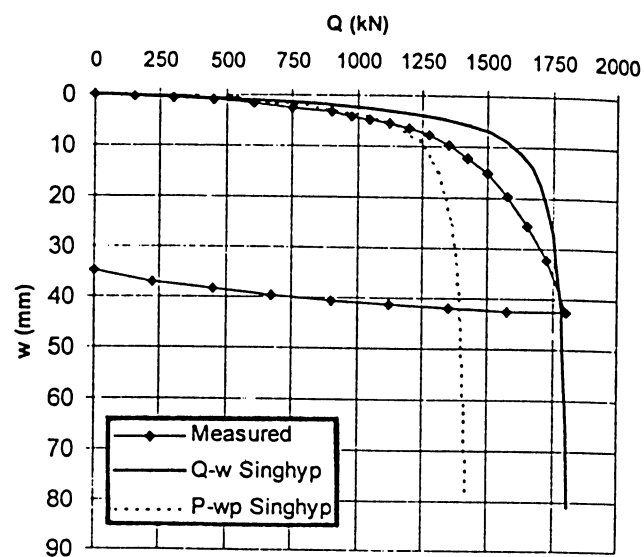


Fig. 13d. Load versus settlement - Omega 5

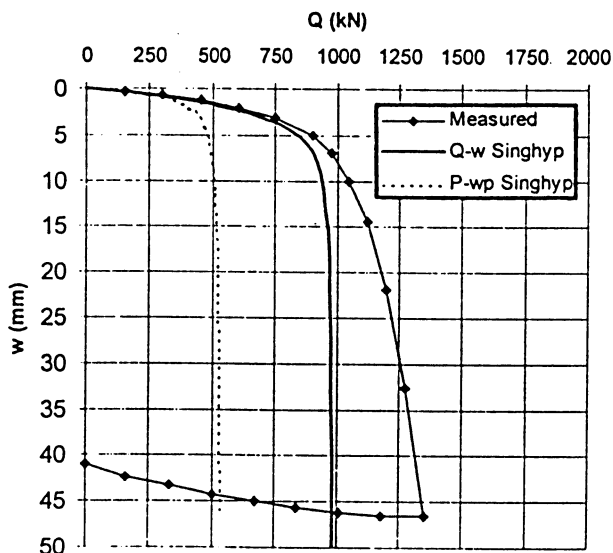


Fig. 13b. Load versus settlement - Omega 3

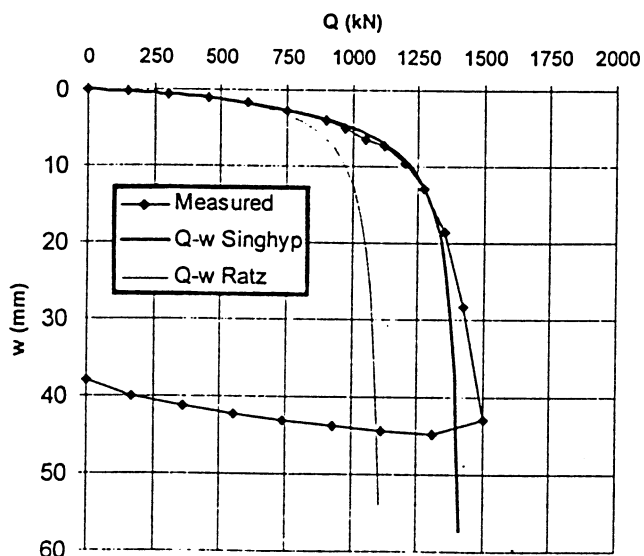


Fig. 13c. Load versus settlement - Omega 4

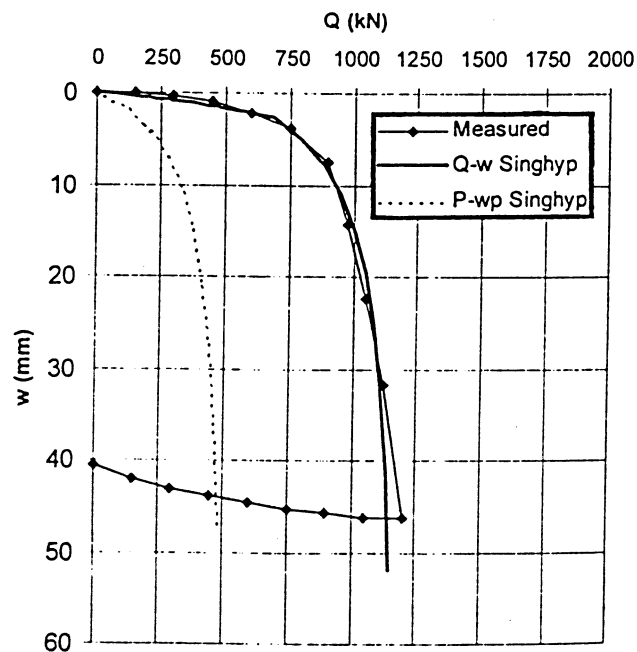


Fig. 14a. Load versus settlement - Omega 2

In a second attempt, an exercise of best fitting of experimental results has been carried out, varying the values of ultimate shaft and base capacity and of the initial stiffness. The values of the capacity have been increased by multiplying them by an installation factor, while the initial stiffness has been obtained by fitting the initial part of the experimental load-settlement curve. Values of $E_u \approx 45 q_c$ have been obtained in this respect. The characteristic settlement of the hyperbolic spring at the base has been selected on the basis of the installation procedures.

The results of such an exercise are reported in fig. 14a through 14d ; the values of the relevant parameters giving the best fit are listed in table 4.

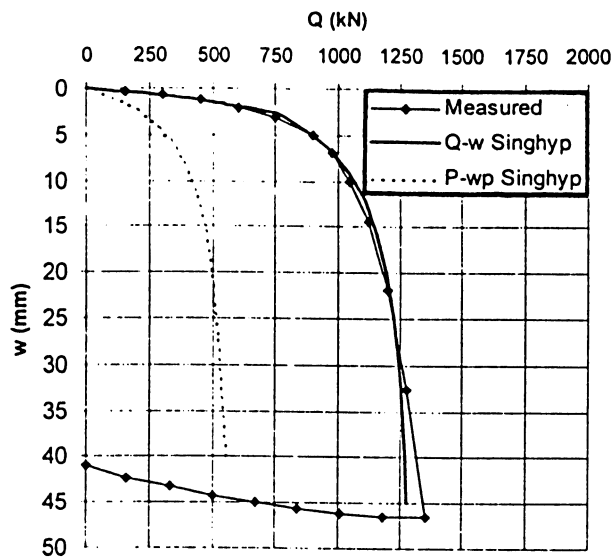


Fig. 14b. Load versus settlement - Omega 3

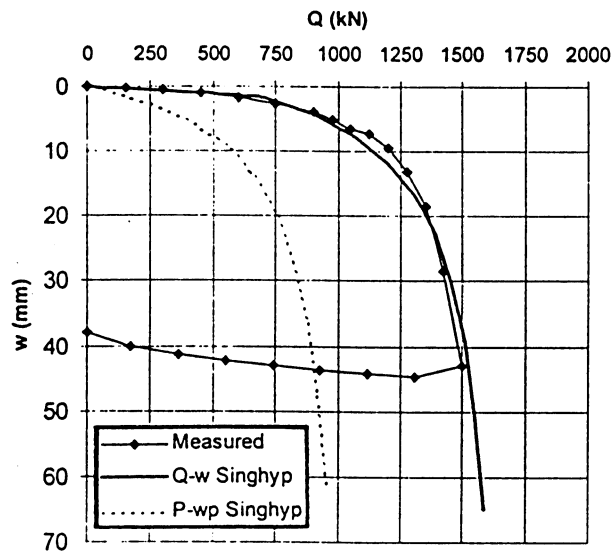


Fig. 14c. Load versus settlement - Omega 4

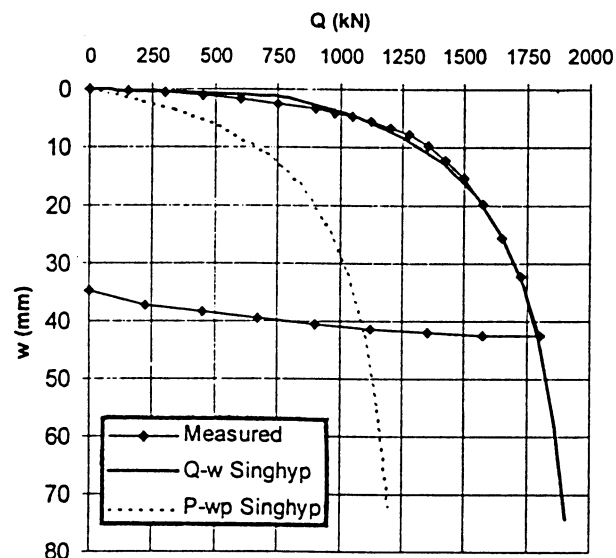


Fig. 14d. Load versus settlement - Omega 5

Table 4

| | Installation factor (linked to PDI) | | Settlement at 90 % base load (% ϕ) |
|---------|--|-----------------|--|
| | Shaft ξ_f | Base α_b | |
| OMEGA 2 | 1.4 | 1 | 15 |
| OMEGA 3 | 1.6 | 1.15 | 10 |
| OMEGA 4 | 2.1 | 0.95 | 10 |
| OMEGA 5 | 1.8 | 0.95 | 10 |

6.4. The agreement obtained between experimental results and back analysis is rather good.

The installation procedure seems not to affect significantly the base installation factor α_b while the stiffness of the base increases approximately in the same way for the OMEGA-pile 3 to 5 as compared to the pile OMEGA 2 - installed with the standard procedure. The shaft installation factor ξ_f increases from pile 2 to pile 4, with a maximum for pile 4 instead of the expected maximum at pile 5. This can be explained by the fact that we expect the shaft concrete fluidity, much higher in OMEGA-pile 5 than in OMEGA-pile 4, to be the predominant factor governing the shaft-soil interaction, irrespective of the other installation details during pile shaft execution.

7. SENSITIVITY OF INSTALLATION COEFFICIENTS TO THE STARTING ASSUMPTION

Several parameters were varied to study the sensitivity of the installation coefficients for the basic assumptions. We checked the effect of a variation of:

- the pile material's modulus of elasticity E_c (initial value 30 GPa for all cases)
- the pile shaft and pile base diameter

The CEMSET method appears to be rather insensitive to a change in the material modulus of the pile. The general effect of an increase of that parameter is a small increase in the ultimate base load $Q_{b,u}$ and pile soil modulus E_b a small decrease in ultimate shaft load $Q_{s,u}$. The total ultimate load is not affected. In particular, a variation of 10 % on E_c results in a change of less than 3 % on the ultimate shaft load $Q_{s,u}$ and the ultimate base load $Q_{b,u}$. The modulus of the soil beneath the pile tip E_b may change up to 10 %. All this results in a minimal effect on the installation coefficients : a maximum change of 3 % on the values of all coefficients.

The most influencing parameter is the pile diameter (both pile tip and pile base diameter). Changing the value of the diameters D_b and D_s , has very little effect on the value of the ultimate load capacity as resulting from the 90 % Chin & Brinch-

Hansen methods ; the same is valid for the values of the total ultimate capacity, ultimate shaft and ultimate base capacity resulting from the CEMSET method. However, varying slightly the pile dimensions has a very remarkable effect on the values of the predicted ultimate total, base and shaft load when predicted from CPT results.

8. CONCLUSIONS

Pile load tests were performed on four Omega piles, each of them with a different concrete casting procedure. Several methods were used to analyse the results of these tests.

The pile load tests at Félu allowed for example to at least qualitatively indicate the consequences of the concrete casting procedure. It is clear that applying even a small additional concrete pump pressure, as for Omega pile 3, a better overall behaviour of the pile is noticed. The installation coefficients indicate a far better behaviour of the piles with expanded tip (Omega pile 4) and for both expanded tip and shaft (Omega pile 5). The base installation coefficient α_b - values, however increasing as shown in the piles 4 and 5, illustrate the difficulty of screw piles to actually prestress significantly the soil below the pile tip.

Such a single pile tip/soil contact prestressing is on the other hand anyhow not influencing the pile group behaviour. Indeed, in case of a real pile group, an overall installation PDI coefficient (and its shaft installation coefficient ξ_r), are the only reliable tools to evaluate about the displacement character of the active deep foundation.

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