Behaviour of bored and auger piles in normally consolidated soils

Comportement de divers types de pieux forés

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SUMMARY

Based on the results of pile tests and CPT's design formulas for the load settlement curves of bored piles could be established. These formulas seem to be also valid for the design of the load settlement curves of auger piles. The results of pile-load tests on bored and auger piles are shown in this paper.
Further investigation on the influence of several parameters and the validity of this design method for other pile types is still going on.

1. INTRODUCTION

Due to the soil conditions in the Netherlands, i.e. normally consolidated fine to medium fine pleistocene sand overlain by soft clay and peat layers, driven precast-concrete piles is by far the most economic and most applied piling system. The ultimate bearing capacity of this type of displacement pile, as well as the load settlement behaviour, is calculated from Dutch Cone Penetration Tests (DCPT's) since 30 years and has proved to be very reliable |lit. 1|. However, during the last decade non-displacement piles like large diameter bored piles and auger piles are also applied, in particular in case of high pile loadings and too heavy pile driving (noise, vibrations). As a standard site investigation programme always consists of DCPT's, it would be very much appreciated if for these types of non-displacement piles design rules could be developed based on DCPT's. Therefore - 3 bored piles with 0,6 m diameter and 1 bored pile with 1,2 m diameter constructed with a bucket under bentonite, without casing, have been testloaded stepwise; after every step 4 cycles of loading-unloading have been applied. During the tests the pile head and pile toe settlement have been measured, while built-in Telemac cells at several levels allowed for determination of skin friction and base resistance as a function of the pile displacements. As non-displacement piles might cause a reduction of the initial in-situ stresses in the soil, DCPT's before and after pile construction have been carried out. With respect to the 6 auger piles, constructed without bentonite, only the pile head settlement has been measured as a function of the applied load, according to the usual test procedure in the Netherlands.

2. BORED PILES

2.1. Base Resistance

The ultimate base resistance p_{ult} turned out to be much lower than that of an equivalent driven pile and also lower than according to the expressions of Caquot-Kerisel, Brinch Hansen, a.o., which obviously is a result of the way of installation. A reasonable agreement could be obtained if with respect to the Dutch design rule for driven piles, making use of DCPT-results, a reduction factor of a = 0,5 is applied. So, the ultimate base of a bored pile in normally consolidated sand is about 50% of the base resistance of an equivalent driven pile calculated according to the Dutch method. Also the relation pile settlement and base resistance is different from driven piles: the base resistance p_{base} of a bored pile in sand slowly developes as a function of the displacement. The expressions

$$s_{head} = \alpha \log \frac{a \cdot p_{ult}}{a \cdot p_{ult} - p_{base}} \text{ cm}$$

with α = 15 (cm) seems to be reasonable fitting of the load-settlement curve.
For N.C.-sands it is doubtful if $a \cdot p_{ult}$ exceeds a value of 5000 - 6000 kN/m².

2.2. Skin Friction

In sand the average ultimate skin friction $\bar{\tau}_{ult}$ is in between 0,6% and 0,7% of the mean measured cone resistance $\bar{q}_c$. It seems to be wise not to exceed a value $\bar{\tau}_{ult}$ = 90 - 100 kN/m². The development of the skin friction of the displacement is more rapidly than the base resistance. The expression

$$\tau = \tau_{ult}(\beta 1 + \beta 2 \log \frac{100\, s_{head}}{D}) \quad \text{for} \quad \frac{s_{head}}{D} > 0{,}2\%$$

with $\beta 1 = \beta 2 = 0{,}5$

is an attempt to describe the behaviour of the test piles. The influence of the diameter D and also the β-factors are subject to further research.
Compared with case-in-situ concrete displacement piles the ultimate skin friction is lower, what can be explained from the installation method, causing a reduction with 30 - 50% in measured local friction values after pile installation.
Figures 1A, 1B, 1C show the results evaluated from a test loading on a bored pile Ø 0,6 m.

2.3. Design Fromulas for Load-Settlement Curves

From the above-mentioned pile load tests the following design rules for bored piles have been derived:

base resistance: $$s_H = 15 \log \frac{0{,}5^* \, p_{ult}}{0{,}5^* \, p_{ult} - p_{base}}$$

skin friction : $$\tau/\tau_{ult} = 0{,}5(1 + \log \frac{100^* \, s_H}{D})$$

with : $$\tau_{ult} = 6\ ^{o}/oo * \bar{q}_c$$

It will be clear that with respect to the ultimate base resistance and skin friction different safety factors have to be applied.

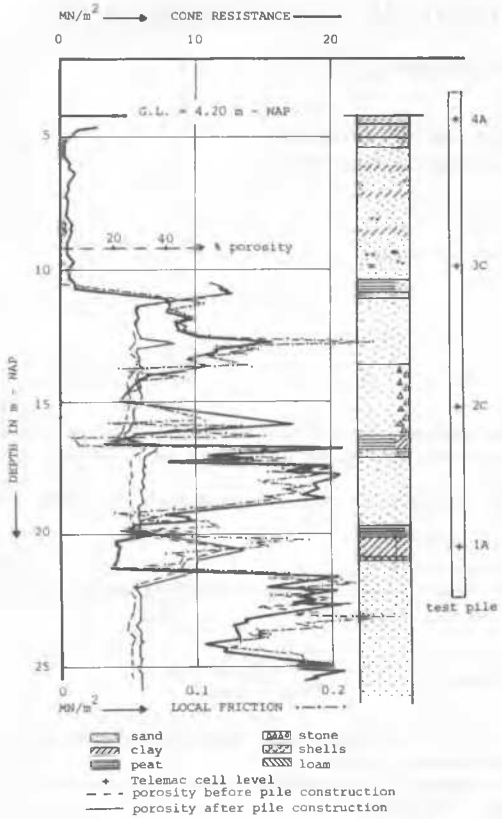

Fig. 1A Static Penetration Test 01

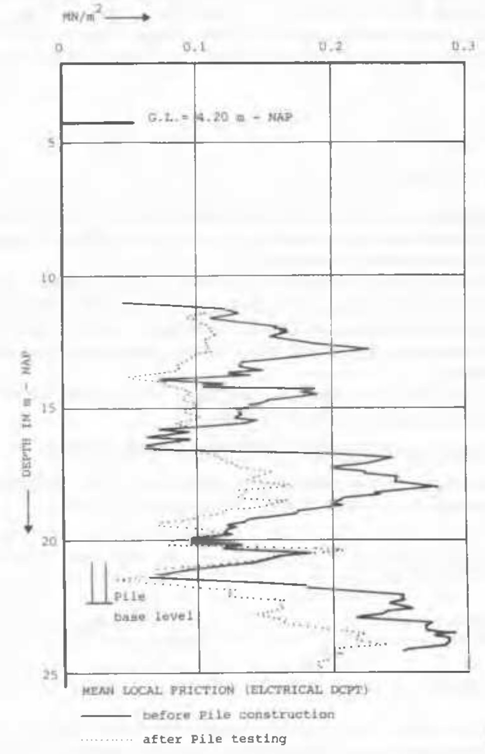

Fig. 1B Mean Local Friction (electric DCPT)

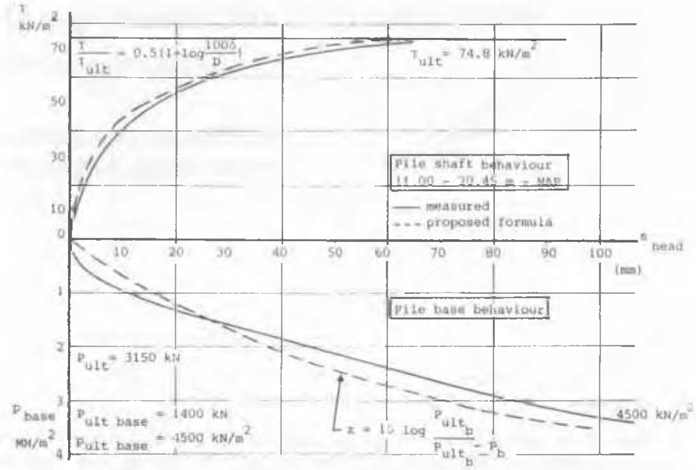

Fig. 1C Pile Test

The allowable pile load depends on the allowable deformations which depends on the type of structure.
Deformations of about 1,5 to 2% of the pile diameter can be quite acceptable which means with respect to the expression in § 2.2 that about 65% of the average ultimate skin friction has been mobilised and about 20% of the ultimate base resistance.

2.4. Case Studies

Based on the fore-mentioned equations some major projects have been designed. During and after construction load and displacement measurements have been carried out for some projects.
The results of two cases are presented here, where also the design load-settlement curves have been plotted.
It can be seen from these graphs, that the design curves are on the safe-side (fig. 2).

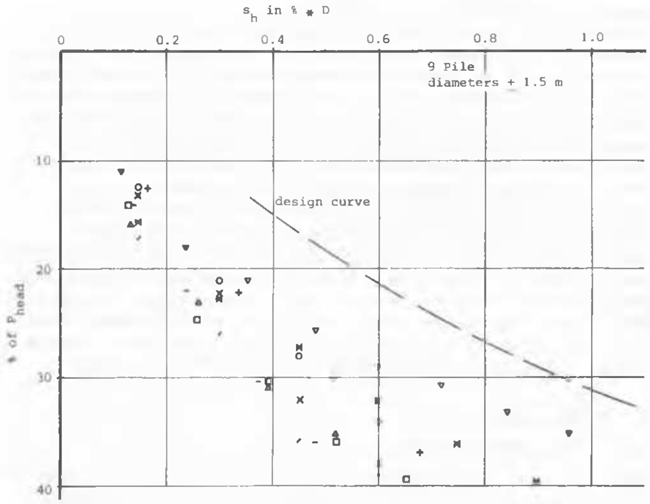

Fig. 2 - Case I: Design Curve and Measured Pile Settlements

Near the village Heerde, in the north of the Netherlands, the new highway No. 50 crosses a local road at a 23° angle, resulting into a viaduct consisting of 3 supports (besides the abutments). Each support consists of one bored pile of 3,26 x 2,20 m² with a working load of about 20000 kN (case 2). After constructing the viaduct on the initial

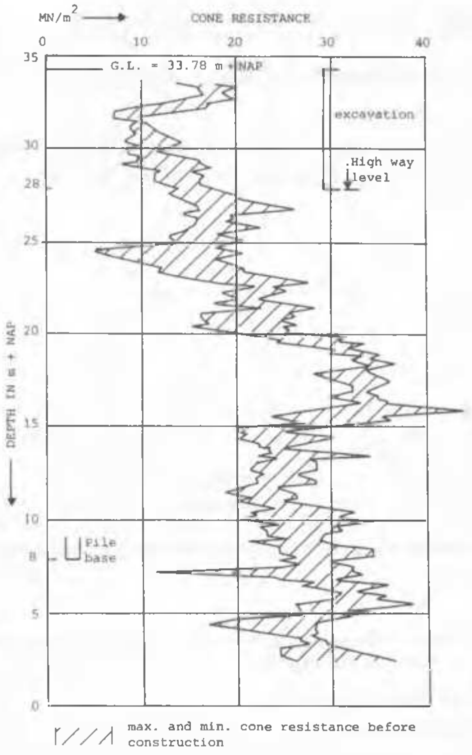

Fig. 3A

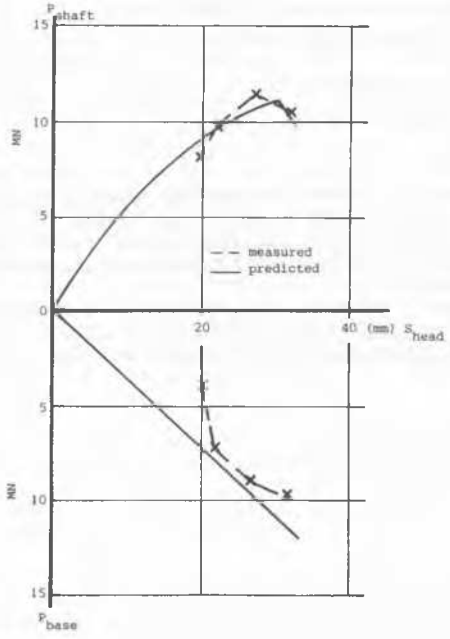

Fig. 3B

groundlevel the highway crossing was made by excavating the soil beneath the concrete structure. It was decided by the Ministry of Public Works to provide the middle bored pile with measuring device at several levels to monitor the strains during the construction stages of the viaduct. Prior to construction the following design rules were applied: skin friction $\tau = 0,5\ \tau_{ult}(1+\log\frac{100\ s_H}{D})$ with $\tau_{ult} = 90\ kN/m^2$;

base resistance $p_{base} = K \cdot s_H$ with the coefficient of pile subgrade reaction $K = 50\ MN/m^3$ for $p_{base} < 1,5\ MN/m^2$.

This is more or less equal to the first part of the expression mentioned in § 2.1.
The predicted and derived values from the measurements for skin friction and base resistance are shown in fig. 3b, together with the cone resistance of the sandy subsoil (fig. 3a). It can be concluded that only in the beginning more settlement has occurred, but that the other values are reasonably well predicted.

3. AUGER PILES

These piles are cast-in-situ by screwing in an auger provided with an injection pipe in the centre of the auger. When the required depth has been reached, the auger is pulled, simultaneously injecting the grout. The soil at least equal to the pile volume is removed during the installation of the pile. The effects of scraping and pulling result into some reduction of the surrounding initial stresses. Therefore, this pile is more or less similar to the bored pile. The bearing capacity of the auger pile consists of base resistance and friction between the pile shaft and the sand.

3.1. Pile Tests

Six pile tests have been analysed and the load settlement curves have been investigated in terms of the formulas derived for bored piles, only with adapted coefficients.

base resistance: $s_{head} = \alpha \log \frac{a \cdot p_{ult}}{a \cdot p_{ult} - p_{base}}$

skin friction : $\tau/\tau_{ult} = \beta\,(1 + \log \frac{100\ s_H}{D})$

In fig. 4 a review of the soil conditions (based on DCPT's and borings) at the test locations is given. Of all test piles the load-pilehead-settlement curves are available.

3.2. Analysis of the Pile Tests

The analysis is based on the assumption that the development of the skin friction as a function of the pile settlement is equal to the bored pile. This assumption has been verified for pile nr. 1. For some reasons, dealing with the execution of these piles, there can be assumed that the bearing capacity of pile nr. 1 mainly consists of skin friction.

$$\tau/\tau_{ult} = 0,5\ (1 + \log \frac{100\ s_{head}}{D})$$

$$\tau_{ult} = 6\ ^o/oo * \bar{q}_c \text{ (fine medium sands)}$$

It can be concluded that these assumptioms are valid for test pile 1 (fig. 5).

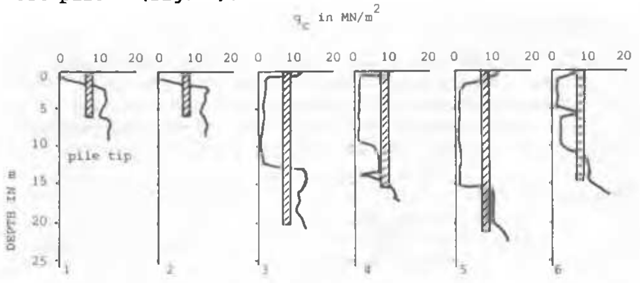

Fig. 4 Soil Conditions at Test Locations

The bearing capacities of the other test piles consist partly of base resistance. For these piles there has been assumed that the development of the friction is also according the above-mentioned bored-pile formula.
At all test locations the pile head loads and settlements have been measured. The bearing capacity of the piles has been separated in pile base and skin resistance.
The ultimate friction can be determined more accurately by measuring the local friction (q_f) by establishing the ratio q_f/q_c for each type of sand.
In table 3.2 the a and α factors are summarized:

$$s_h = \alpha \log \frac{a \cdot p_{ult}}{a \cdot p_{ult} - p_{base}}$$

pile nr.	a	α
2	0,7	5
3	0,67	6,7
4	0,61	3,5
5	0,60	-
6	0,54	6-9

3.3. Design Formulas for Load-Settlement Curves

Based on the analysis of the six pile tests the following design formulas are proposed:

pile base : $s_{head} = \beta \log \frac{0,67 * p_{ult}}{0,67\ p_{ult} - p_{base}}$ |cm|

skin friction: $\tau/\tau_{ult} = 0,5(1 + \log \frac{100\ s_{head}}{D})$

for: $\tau_{ult} = 6\ ^o/oo * \bar{q}_c$

The test piles have been recalculated with these formulas. The results are shown in fig. 6.

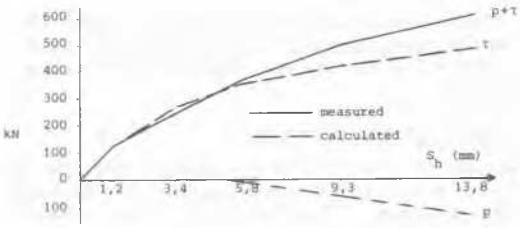

Fig. 5 Test Pile I

The allowable pile load depends on the allowable deformation of the pile head. Deformations of about 1,5% of the pile diameter mean that about 65% of the average ultimate skin friction has been mobilized and about 25% of the base resistance.

4. CONCLUSIONS

Based on the results of pile tests on bored and auger piles, load-settlement formulas have been derived.
The influence of the pile diameter and the validity of the presented formulas for other pile types (including displacement piles) are subject of further research.

SYMBOLS:

$s_h = \alpha \log \frac{a \cdot p_{ult}}{a \cdot p_{ult} - p_{base}}$

s_h = settlement of the pile head

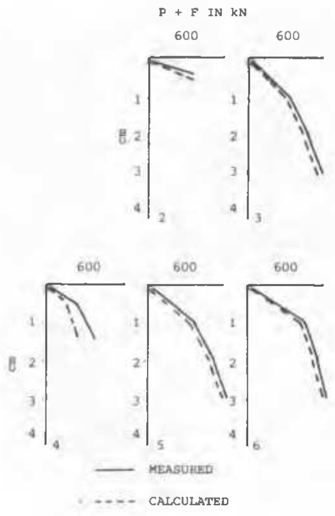

Fig. 6 Measured and Calculated Values of the Test Piles

p_{ult} = ultimate bearing capacity of the base according to the Dutch method

p_{base} = working base load

a, α = coefficients

$\tau = \tau_{ult}\ \beta(1 + \log \frac{100\ s_h}{D})$

τ = skin friction at working load

τ_{ult} = ultimate skin friction

D = pile diameter

β = coefficient

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