H Steel Piles in Dense Sand

Les Pieux Métalliques H dans les Sables Compacts

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SYNOPSIS The results of dynamic and static tests performed on H steel piles with and without lagging, and driven in dense sand are described. From the analysis of the results data concerning several factors influencing the ultimate bearing capacity are obtained. Based on these factors and the results of CPT tests, a method of prediction of the ultimate bearing capacity is presented and the results are compared with those obtained with the dynamic "Case" and "Capwap" methods.

INTRODUCTION

Rolled H steel beams used as bearing piles present interesting features from the point of view of handling and driving, as well as from the point of view of bearing capacity. In order to obtain detailed information about the behaviour of such piles in site conditions where they are not pure endbearing, a research project was undertaken with test piles in different types of soils. The E. C. S. C. sponsored project was carried out by ARBED with the cooperation of the two first authors and the Belgian Geotechnical Institute.

The overall aim of the research program was to investigate about driving, to estimate pile resistance by different methods based on dynamic measurements and preliminary soil exploration data, and to analyse static load transfer and bearing capacity. The influence both on driving and static load carrying, of different kinds of laggings and enlargements of the pile section were studied.

The first series of tests on which will be reported in this paper, were performed at Kallo, West of Antwerp, on piles driven into a dense sand layer. From 12 piles driven only 4 were loaded, these are the piles no. I, II, III, VI.

SOIL CHARACTERISTICS

The soil profile is as follows: 1 m fill; 4 m soft clay and peat (holocene); 3, 5 m loamy sand; 13 m dense to very dense sand (tertiary pliocene scaldisian); to great depth: Boom clay (tertiary oligocene).

In order to get data on soil resistances CPT tests M4 were performed at the vertical of each test pile.

The results of CPT test II are shown as an example on fig. 1. The full line gives the variation of the cone resistance q_c and the dashed line the variation of the total side friction $Q_{s,t}$. It can be observed that in the dense sand layer locally q_c values of more than 35 MN/m^2 are measured.

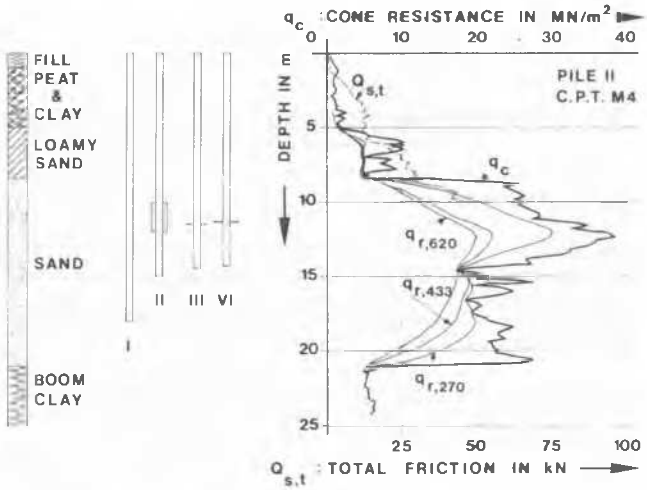

Fig. 1. Boring profile and CPT test M4

DESCRIPTION OF THE TEST PILES

Data concerning the 4 tested piles and their laggings are given on fig. 2.

The piles are hot rolled wide flange beams from the American W series (W 14x16x142). Pile I is an ordinary beam without lagging. Pile II is equipped with a closed box lagging. Piles III and VI are equipped with a plate lagging. Steel used for the test piles is grade ST 52-3 according to German DIN 17100, with a yield strength of 345 N/mm^2 and a tensile strength of 490 N/mm^2. In order to measure stresses at different levels during driving and static load testing, strain gauges were fitted on the web surface of the test piles. Locations are shown on fig. 2 and 4.

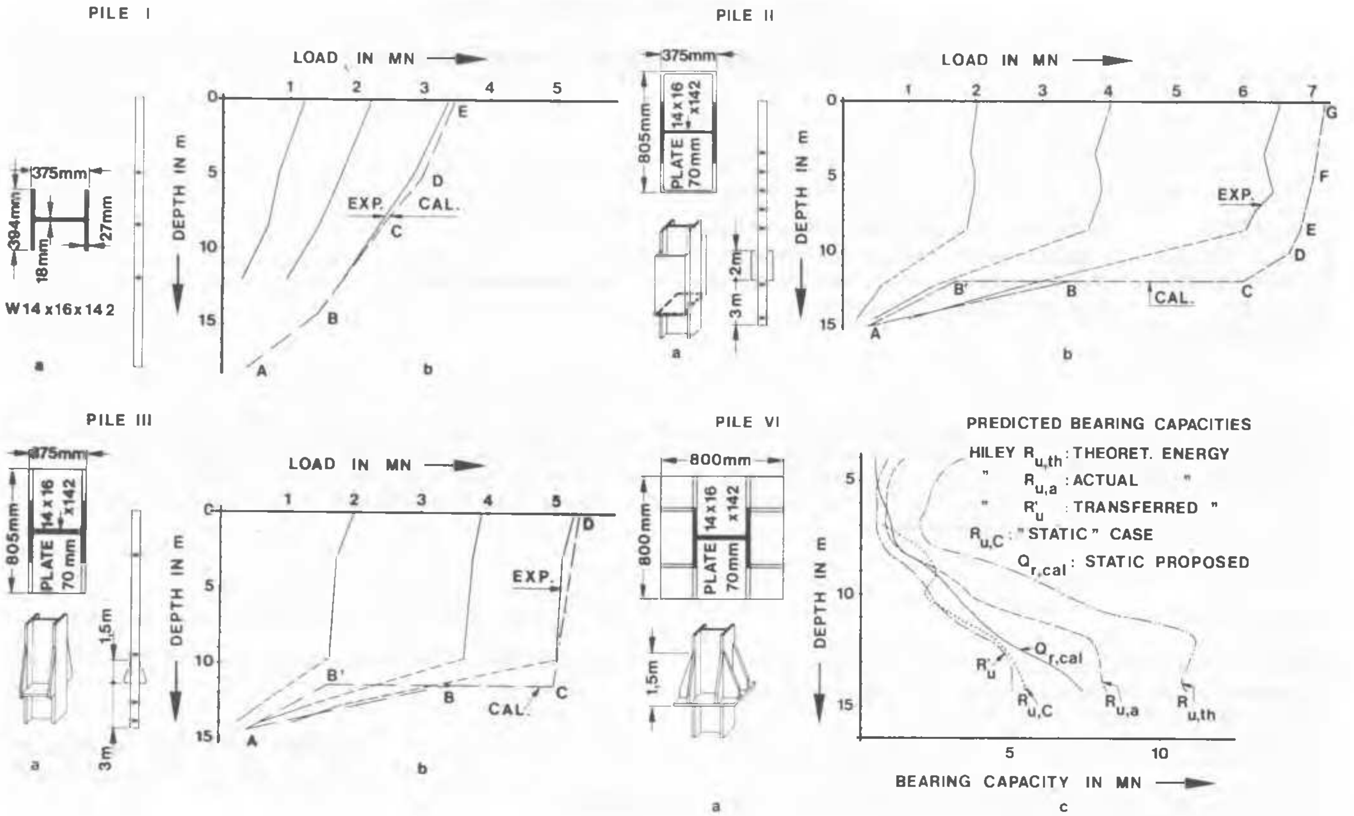

Fig. 2. a - Geometry of the piles; b - Measured and predicted load distribution curves
c - Predicted dynamic and static bearing capacities.

DRIVING METHOD

The piles were driven by an open end Diesel hammer developing a theoretical maximum energy of 115 kNm/ blow and having a drop weight of 36 kN. The pile cap made of welded steel had a high density polyethylene fill and weighed 3 kN.

To limit the loss of energy in lateral deflection of the pile, it was held mid-way between hammer and soil by a guide which was attached to the driving lead.

MEASUREMENTS DURING DRIVING

The measurements during driving were of four types:

A) By the means of an U. V. oscillograph running at a speed of 2000 mm per second signals of 6 of the 9 strain gauges attached to the pile were recorded.

B) A strain transducer bolted on the top of the web of the H beam and two accelerometers gave the response signals of the pile head to a dynamic impulse.

The signals were treated in a pile driving analyzer, a kind of field microcomputer which integrates the acceleration to get the velocity. With this force and velocity data measured, the pile driving analyzer runs a wave equation program and gives per blow:
1. the energy transferred to the pile head
2. the maximum effort in the pile
3. the dynamic resistance
4. the "static resistance"

This is the so called "Case" method (Goble et al, 1975)

C) The data of the pile driving analyzer were retreated for piles I, II, III on a computer by "Goble and Associates, Inc." according to the "Capwap" method (Rausche, 77) in order to find the dynamic stress distribution and the distribution between friction and end bearing and to determine the damping factor J.

D) The blow count and the determination of the energy actually developed by the hammer was made by the mean of a saximeter. This stroke indicator detects the occurence of an impact, and determines the duration (Δt) between consecutive blows. This Δt is converted to the corresponding fall height or stroke of an open end Diesel ram.

RESULTS

On fig. 3 are given as an example for pile II, versus depth on a log scale, per meter the theoretical energy of the hammer, the actual energy of the hammer and the energy transferred to the pile.

The ratio ε between the actual developed energy and the theoretical energy varies between 0, 36 and 0, 75, the lower value being measured in the beginning, correlated to easy driving, and the latter at the end. It must be said, that the value of 0, 75 is not an absolute value for this type of hammer, but depends

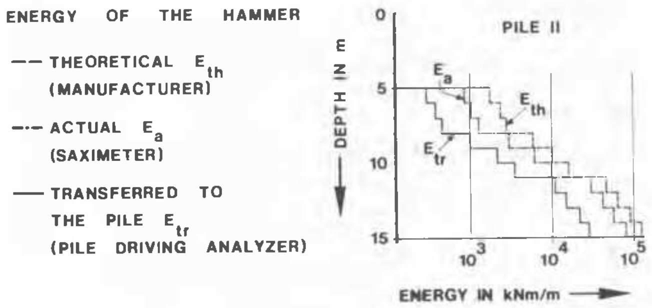

Fig. 3. Energy of the Diesel hammer

essentially on the maintenance of the hammer. Indeed at another jobsite, where a well maintained hammer was used, a value of 1 has been found.

The ratio between developed energy and energy transferred to the pile varied from blow to blow between 0, 30 and 0, 40 which seem to be usual values for this kind of cap and cap-fill.

Nevertheless these values show clearly the danger of using the theoretical values of energy in the dynamic formulas for the determination of the bearing capacity of the piles.

They do also show the considerable amount of energy lost in transmission, which is much larger than anticipated, as until yet the ratio of the transferred energy to the theoretical energy was estimated for the cap with polyethylene fill to be 0, 7 - 0, 8.

PREDICTION OF THE BEARING CAPACITY BY DYNAMIC METHODS

First the Hiley driving formula is used:

$$R_u = \frac{\varepsilon \cdot E_{th}}{s + 1/2\,(C_1+C_2+C_3)} \cdot \frac{W_r + n^2 W_p}{W_r + W_p} \qquad (1)$$

with:

R_u = ultimate driving resistance

ε = efficiency of the hammer (for Diesel hammer = 1 for hard driving)

E_{th} = theoretical energy of the hammer

W_r = weight of the ram

W_p = weight of the pile + cap

n = coefficient of restitution of the cap (= 0, 5 for polyethylene cap filling)

s = permanent set of pile

C_1 = elastic compression of the pile cap; the measurements show this compression to be negligible against C_2 and C_3

C_2 = elastic compression of the pile

C_3 = elastic compression of the soil

C_2 and C_3 are measured by the rebound of the pile head.

As the actual energy $E_a = \varepsilon \cdot E_{th}$ and the energy E_{tr} transferred to the pile have been directly measured, the formula (1) can be rewritten as:

$$R_{u,a} = \frac{E_a}{s + 1/2\,(C_2 + C_3)} \cdot \frac{W_r + n^2 W_p}{W_r + W_p} \qquad (2)$$

or

$$R'_u = \frac{E_{tr}}{s + 1/2\,(C_2 + C_3)} \qquad (3)$$

For each pile a graph has been drawn giving versus depth the dynamic resistance calculated by introducing in the Hiley formula the theoretical, actual and the transferred energy. As an example the graph of pile VI is shown on fig. 2c. Also the "static resistance" given by the pile driving analyzer with the "Case" method (damping factor J used = 0, 1) is drawn.

The curves of fig. 2c show that, when the energy really transferred to the pile is introduced in the Hiley formula, values are obtained which do not differ very much with the "Case" method. At the contrary when using the theoretical and even the actual energy, the Hiley formula gives much too high resistances. When the driving formula is used in the usual way of introducing E_{th}, the difference between the predicted resistances and the "static" resistance of the "Case" method is not only extraordinary large, but furthermore there is not a constant ratio of these two values, making it very difficult to define an adequate factor of safety.

DRIVING STRESSES

The stresses in the beams during driving have been obtained from the analysis of the readouts of the strain gauges placed along the pile. However the direct data obtained in this way have been locally completed by data taken from the "Capwap" analysis (Rausche, 77).

As an example fig. 4 gives for pile I the variation of the axial force versus depth with the time elapsed after the instant of maximum impact at the level of the highest strain gauge M_9. The time parameter is given as a multiple of the ratio L_9/C. In this ratio L_9 is the distance between the strain gauge M_9 and the tip of the pile, and C is the wave velocity. The ratio L_9/C is the time needed for the wave to travel the distance L_9.

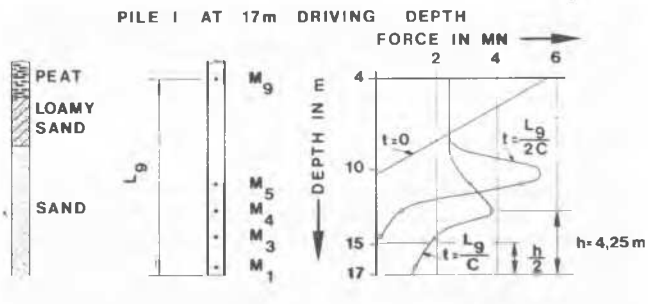

Fig. 4. Measured force waves at different instants.

From stress waves theory considerations, at time t = L_9/C the maximum of the force wave should have reached the tip of the pile. As however the measured curve shows that the maximum is already located at a height h = 4, 25 m, it can be deduced that a local resistance is located at a height of approximately h/2 = 2, 12 m, causing an upwards running reflexion. From this is confirmed that above the tip of the pile exists a resisting plug of soil with a height f 2-3 m.

On fig. 5 is drawn for each of the piles the curve giving versus depth the maximum wave force registered by the strain gauges of course at different instants. It must be remarked that at maximum driving depth some of the strain gauges were damaged and therefore the curves are given for the maximum depth at which the strain gauges were working.

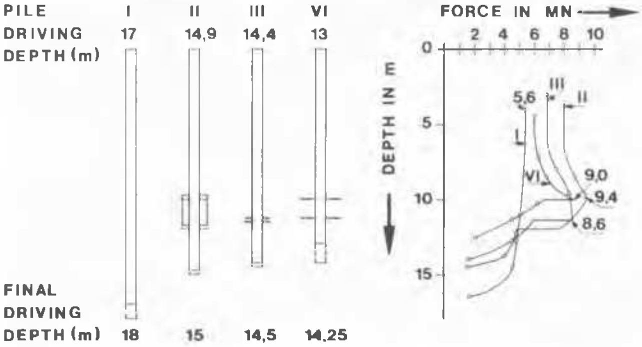

Fig. 5. Maximum dynamic force envelope.

For pile I the maximum effort registered at the level of the highest strain gauge is 5, 6 MN. This maximum effort shows only a very small decrease until 2, 50 m from tip. It can be concluded that during driving the lateral friction is rather low for most of the length of the pile, and that the decrease is due to the influence of the tension wave travelling back.

The same plot for pile II shows a peak effort at the level of the top of the lagging, with a value of 9, 4 MN. The lagging is submitted to very high friction and a jump in the diagram shows a big point resistance at the bottom of the lagging. "Capwap" indicates a peak effort at the same level of 7, 7 MN. The maximum value of the jump force is rather low.

The diagram for pile III shows about the same features with however the difference that the peak effort of 8, 6 MN is situated at the level of the lagging, and that the friction shown above the reinforcement plate is rather small. "Capwap" gives a peak effort of 5, 7 MN at the reinforcement plate.

Maximum stress envelope for pile VI shows a big peak resistance at the level of the reinforcing plate with an effort of 9, 0 MN and a rather small jump at the re-inforcement.

LOAD TESTS

Loading and settlement measurements were carried out in a classical way with a testing device designed for a maximum compression load of 8 MN, using eight H anchor piles.

The time elapsed between driving and testing has been two months. Load was applied in pressure increments of 20 bars (corresponding to 0, 28 MN) for the first steps and 10 bars (corresponding to 0, 14 MN) for the following increments. Settlement readings were made every ten minutes. For each step, load was maintained until the settlement rate did not exceed 0, 05 mm/20 minutes but no longer than 200 minutes. Load was increased until soil rupture was presumed.

RESULTS

The load settlement diagrams of the 4 tested piles are given on fig. 6 on a linear scale and on fig. 7 on a double log scale.

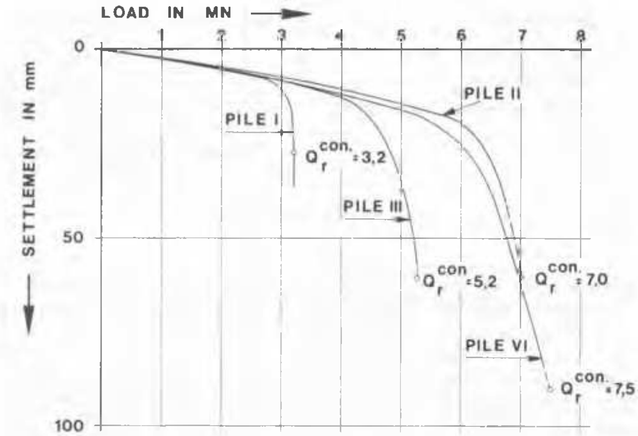

Fig. 6. Load-settlement (linear scale)

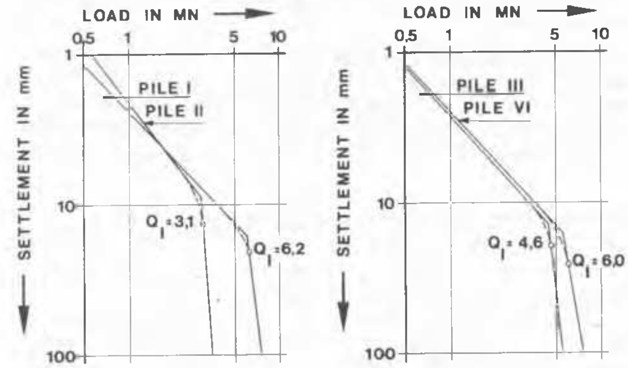

Fig. 7. Load-settlement (log-log scale)

From these diagrams it can be seen that pile I (simple beam) behaves essentially as a friction pile. From the point of tangency from the first part of the log load-log settlement curve, to the straight line corresponding to large deformations, the so called "Limit Load Q_l" can be deduced. From the straight lines also the so called "conventional rupture load Q_r^{conv}" corresponding to a relative settlement $s_r/D=0,1$ for the piles with a bearing plate have been deduced. The symbol D represents the diameter of the circle with the same area as the plate. The values of Q_l and Q_r^{conv} are given in table I.

The results of the strain gauge readings during load tests are given on fig. 2b for the piles I, II, III for different loading steps. The readings of pile VI were not usable and are not reproduced.

Pile	D_e mm	Q_l kN	s_l mm	$\frac{s_l}{D_e}$ %	Q_r^{conv} kN	s_r mm	Q_r^{cal} kN	Q_r^{cal} / Q_r^{conv}
I	-	3100	13,5	-	3200	27	3446	1,08
II	620	6200	20,9	3,4	7000	62	7223	1,03
III	620	4600	19,6	3,2	5200	62	5338	1,03
VI	903	6000	26,2	2,9	7500	90,3	7444	0,99

Table I

Ultimate bearing capacity - Comparison between experimental and predicted values

PREDICTION OF THE BEARING CAPACITY FROM THE RESULTS OF THE CPT TESTS

1. Piles without lagging

From the analysis of the force wave diagrams it appears that a partial plug of about 2-3 m length has been formed after a certain penetration in the dense sand layer. A reasonable assumption is that the extension of this plug is a function of the ratio e/b (Fig. 8)

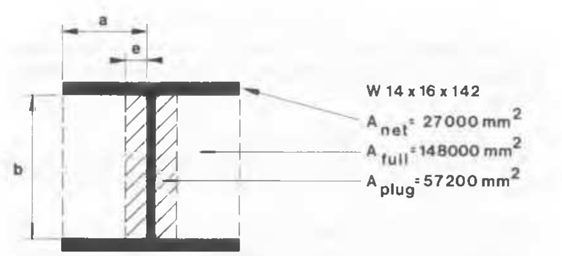

Fig. 8. Scheme of the plug

For the width e of the plug, it can be put if:

$$a < b < 2a \quad : \quad e = \frac{a}{4} \qquad (4)$$

$$b \geq 2a \qquad e = \frac{1}{4} \cdot \frac{a^2}{b} \qquad (5)$$

For the actual beam: e = 47,1 mm.

A_{plug} = 27000 + 2·47,1·321 = 57.200 mm^2 and the equivalent diameter $D_e = \sqrt{\frac{4}{\pi} \cdot 57.200}$ = 270 mm. For this equivalent diameter, with a known method (De Beer, 1971), from the value of q_c, the value of q_r, valid for a circular area, can be deduced.

As the exact shape of the plug is unknown, for the safety it is assumed that it is similar to an elongated rectangle, and consequently the value q_r is divided by the shape factor s=1,3.

The unit friction in the upper soft layers and in the loamy sand layer is obtained in a usual way from the total friction diagram ($F_{s,t}$) of the CPT test (De Beer, 1971).

This unit friction is assumed to act on the developed perimeter of the section. In the dense sand layers, because of the too high resistance, it was impossible to measure the total lateral friction. Therefore in the sand layer the empirical formula $f_s = q_c/200$ is used (Te Kamp, 1977; Van den Elzen, 1980)

As the plug has been formed while driving through the dense sand layer, above the plug a decompression occurs between the flanges. Because of this decompression over the height of the web the unit friction drops to 1/2 of its value in absence of decompression. When applying the figures on the circumscribed perimeter and the full friction on the developed perimeter of the plug one gets the calculated values as shown on table II and by line ABCDE on fig. 2b.

considered layer	A_{plug} mm2	D_e mm	q_r MN/m^2	q'_r MN/m^2	$Q_{b,r}$ MN
sand S_2 (lower)	57200	270	16,9	13	0,74
considered layer	mantle friction f_s MN/m^2	peri-meter m	height of layer m	F_s MN	$Q_{b,r} + \Sigma F_s$
sand S_2 (lower)	88,6	1,996	3,8	0,672	1,41
sand S_1 (upper) flanges	13,38	0,896	5,8	0,695	2,11
sand S_1 (upper) web	6,69	0,642	5,8	0,249	2,35
loamy sand	85,2	2,185	3,2	0,596	2,95
soft layer	43,3	2,185	5,2	0,492	3,44

Table II
Predicted load distribution, Pile I

2. Piles with lagging

With the lagged piles several cases have to be considered

2.a.: elongated rectangular lagging plate continued by a sufficient long casing.

2.b.: lagging plate with an elongated rectangular shape, and not provided with a casing.

2.c.: lagging plate with a large square shape, and not provided with a casing.

Pile II

The plates are located 3 meters above the pile tip. From the analysis of the results of the measurements, it appears that the presence of the lagging plate favours the formation of the plug, whose section is now about equal to the overall section of the beam.

Therefore from the values of q_c, the values of q_r, corresponding to a section A_{full} = 148000 mm^2 and D_e= 443 mm can be calculated. The friction on the plug is calculated in a similar way as for pile I.

For the friction on the casing a value $f_s = q_c/200$ is introduced. Above the casing, on the flanges, the unit friction in the sand is obtained by $f_s = q_c/200$, and in the loamy sand and the soft layers deduced in the usual way from the total friction $F_{s,t}$, measured in the CPT test.

In the decompressed material between the flanges over the height of the web, the unit friction has been taken equal to 20 kN/m^2. The unit resistance under the lagging plate is obtained by calculating from the q_c values the q_r values for a circular plate (De Beer, 1971) and by dividing the obtained values by the shape factor

$$s = \frac{1,3}{1 + 0,3\frac{B}{L}}$$

with B = width of the lagging plate
L = length of the lagging plate

Piles III and VI

Underneath the lagging plate, the calculations can be performed in the same way as for pile II.

Because no casing above the lagging plate is present, above the level of the plate the friction is reduced on every surface and in all layers to 20 kN/m^2.

In order that the calculation should give the same results as the conventional rupture load deduced from the measurements, it appears that the unit resistance q_r, as deduced from the CPT test for a plate with a sufficiently high casing should be divided by 1, 6 for a plate with a surface A_{plate} = 0, 32 m^2 and a ratio A_{plate}/A_{full} = 2, and also by 1, 6 for a plate with A_{plate} = 0, 64 m^2 and a ratio A_{plate}/A_{full} = 4, 32

The values obtained by this way are shown by the lines ABCDE.

The figures given for the decrease of the unit bearing capacity caused by the cantilevering plates are very high, and to avoid false conclusions may ask for some comments.

Indeed the presence of the lagging on the tested piles increases the bearing capacity in a two-fold way: indirectly by an increase of the plug formation and directly by a supplementary capacity under the lagging plate. It is only when considering the sum of these two influences, that one gets in fact the total positive influence of the lagging plate. Therefore in order to emphasize this influence calculations can be made with the hypothesis that the plug is the same than for the pile without lagging, all other assumptions remaining the same as for the calculations described before. Then for a lagging plate with a ratio A_{plate}/A_{full} of 2:1 the reduction factor for the unit bearing capacity is 1:1, 14 and for a lagging plate with the higher ratio of 4, 32:1 the reduction factor is 1, 42. The force distributions with these assumptions are given by the lines AB'CDE.

CONCLUSIONS

The very extensive tests performed on H steel-beam piles driven into very dense sand layers yield following results:

1) Very valuable predictions of the "static capacity" can be made from the energy really transferred to the pile and by "Case" and Capwap" wave equation analysis.

2) From these tests reduction factors have been deduced to be introduced in the usual method of prediction of the bearing capacity from CPT test, in order to take into account the special geometry of the piles.

3) The bearing capacity of H piles can be doubled in dense sands by a cheap reinforcement.

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