Field tests with drilled shafts in tension in frictional soil

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Abstract : Three series of ten piles each have been installed. The lengths of the piles varied from 2 m to 6 m and the diameters were 14 cm, 25 cm and 60 cm. The piles were constructed above the ground water table using continuous flight augers and the concrete was placed by gravity free fall. All piles were tested to failure in axial uplift and the load-displacement relations were recorded and the results from the tests have been compared with theoretical values based on current design practice and the methods proposed by Fleming et al. and Reese & O'Neill seem to produce the best match with the test results.

1 INTRODUCTION

Piles in cohesionless soils subject to uplift forces carry their load by skin friction forces which develop on the sides of the pile. These side resistance forces are normally computed using an empirical formula of which a large number exist. The purpose of the present project is to compare the uplift capacities found by some of the most common codes and standards and also to see how well these values match with results from field tests. This project has been dealing with three different codes and standards and a striking thing is the fact that according to some codes the carrying capacity in uplift is almost independent of the strength of the soil provided the quality of the soil is above a certain lower limit whereas other methods put much more emphasis on the strength of the soil. This is quite remarkable as all methods are based on results from field tests.

This point has been investigated by comparing the results from the present project with results from tests carried out in loose sand with bored piles of similar size.

Another point of interest which has been dealt with is to verify the significance of the diameter of the pile on the unit side resistance.

The present paper deals with the results of three test series, with bored piles of different diameters in dense sand. Altogether 30 piles have been tested.

2 THE TESTING PROGRAMME

2.1 *The testing area and soil conditions*

The testing area is situated on the campus of Esbjerg Institute of Technology, Aalborg University, in Esbjerg, Denmark, and the piles were placed in a grid of 4x4 metres. Three series of piles were cast and each series consisted of ten piles of which the lengths varied from 2 metres to 6 metres at one metre intervals and two piles of each length were cast. The diameter of the piles in the three series were 14 cm, 25 cm and 60 cm.The borings, which were uncased, were carried out using a continuous flight auger at a depth of maximum 6 m and samples were taken at 1 m intervals. The ground water table was just only touched in the deepest borings and thus the water did not cause any practical problems.

The samples, which were all disturbed, have shown very homogeneous soil conditions and they all consisted of alluvial quartz sand from the ice age named Saale.

The grading of the sand has been determined on 3 different samples taken out in 3 different borings at depths of 2 metres, 4 metres and of 5 metres. The grading curves of the 3 samples were almost identical.

The mean grain size d_{50} is app. 0.22 mm, the coefficient of uniformity C is equal to 1.8 and the specific gravity of the quartz sand is 2.621.

Average values for void ratios of the 3 samples : $e_{max} = 0.764$, $e_{min} = 0.467$.

Because of the homogeneous soil conditions – just 1 SPT boring was carried out and the SPT blow count using the below equations (1) and (2) have resulted in the values for the relative densities and friction angles shown in table 1.

Table 1 Soil Characteristics

Depth in m	N	Id	φ_{tr}
0,0 - 1,0	14	0.48	40
1,0 - 2,0	17	0.58	40
2,0 - 3,0	17	0.57	39
3,0 - 4,0	26	0.66	40
4,0 - 5,0	54	0.95	44
5,0 - 6,0	57	0.97	44

On the basis of the SPT tests the sand is characterized as dense and very dense.

$$I_d = \sqrt{\frac{N}{60 + 25 \cdot \log(d_{50})}} \qquad (1)$$

In this equation - Kulhawy & Mayne (1990) - d_{50} is mean grain size in mm and N is the SPT blow count.

The peak triaxial angle of friction has been calculated using the equation suggested by Bolton (1986) :

$$\varphi_{tr} = \varphi_{cv} + 3 \cdot (I_d \cdot [10 - \ln(p')] - 1) \qquad (2)$$

In this equation p' is the mean principal effective stress at failure which is approximately equal to twice the value of the vertical effective stress $\sigma'z$, (Rollins et al. 2005) and φ_{cv} is the friction angle at constant volume which is taken to be 33°, Bolton, (1986).

3 CONSTRUCTION OF PILES

In the open holes left by the continuous flight augers one or more 20 mm reinforcing bars were put in position, and the concrete was placed by gravity free fall. The quality of the concrete was 25 MPa with a high degree of workability (flow value = 530 mm) and when the hole within a few minutes had been filled to the ground level, a high strength steel bolt of length 2 m was installed to a height of 1 m above the top of the concrete; that is the lap length with the reinforcing bars is 1 m. For the 14 cm piles a 16 mm bolt was used and for the 25 cm piles and the shorter of the 60 cm piles a 24 mm bolt was used. For the longer of the 60 cm piles a 32 mm 950/1050WR Dywidag bar was used over the entire length of the piles projecting 1 metre above the top of the piles. Two piles of each length were constructed. Construction of the 14 cm piles and the 25 cm piles took place in February 2006 and the 60 cm piles were cast in June 2006.

4 TESTING OF PILES

The uplift tests of the 14 and 25 cm piles were carried out in May 2006 and the 60 cm piles were tested in September 2006. The load was applied to the piles by a hollow ram hydraulic jack resting on two steel beams which for the 14 cm and 25 cm piles were of the type IPE 240 of length 6 m supported at either end by 100x200 mm timber. For the 60 cm piles 2 beams of the type HE 240B were used. The distance between the supports for the 60 cm piles was 2,1 metres for the 2 and 3 metre piles, 1,6 metre for the 4 metre piles, 1,35 metre for the 5 metre piles and 1,1 metre for the 6 metre piles. The purpose of the two longitudinal beams was to transfer as little additional horizontal forces to the piles as possible. As described later in this paper account has been taken of the additional horizontal forces. During the test the load was recorded using a pressure transmitter of the type Danfoss MBS 33 and for the vertical displacements were used two displacement transducers of the type HBM W20TK fixed on a separate steel beam. Both pressure transmitter and displacement transducers were calibrated before the tests started. The displacements of the piles were taken as the average of the two transducer readings. The load was raised continuously and the rate of displacement was app. 3 mm pr. minute. All the test values were recorded by means of a datalogger of the type Spider 8 from HBM. The set up of the load test for the smaller piles is shown in figure 1.

Figure 1 Load test arrangement

5 TEST RESULTS

The results of the tests are shown in figure 2, 3 and 4.

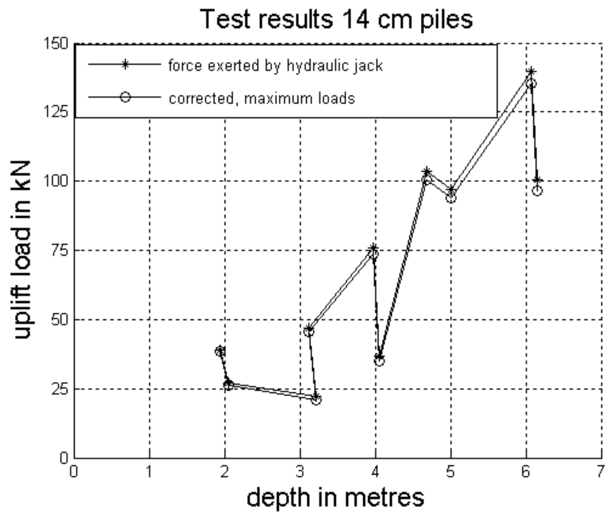

Figure 2. Test results of 14 cm piles

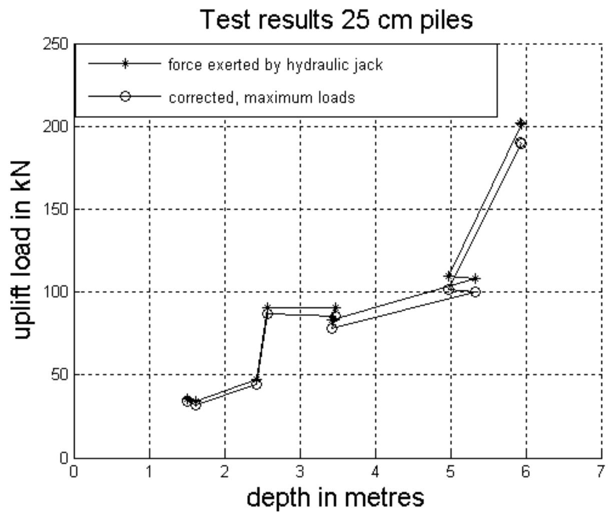

Figure 3. Test results of 25 cm piles

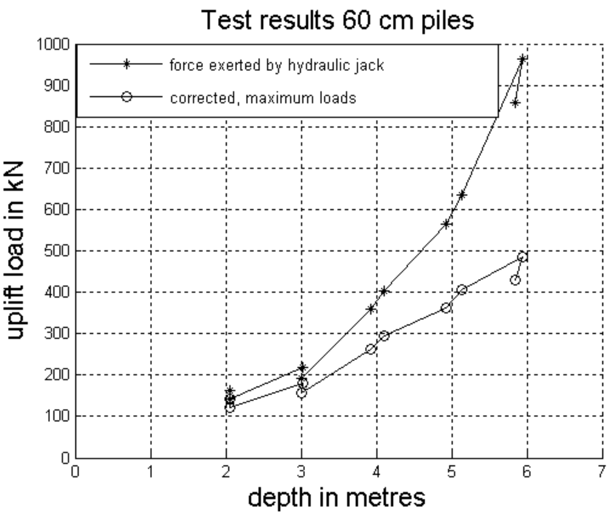

Figure 4. Test results of 60 cm piles

The vertical reaction forces transmitted from the steel beams to the ground will cause additional horizontal forces on the sides of the piles which again are giving rise to an apparent increase in the uplift capacity. This increase in the uplift capacity must together with the selfweight of the pile be deducted from the force applied by the hydraulic jack to give the true capacity of the pile. The additional horizontal forces caused by the vertical reactions from the supporting beams been have been calculated in the following way :

The additional, vertical stresses $\sigma_{v,z,b}$ at the depth z in the centreline of the pile due to the reactions from the supporting beams are calculated according to the theory of elasticity, (Aysen 2005) and the horizontal stresses $\sigma_{h,z,b}$ on the pile is found from the equation :

$$\sigma_{h,z,b} = K(z) \cdot \sigma_{v,z,b} \tag{3}$$

In this equation K(z) is the earth pressure coefficient which is assumed to vary with the depth below ground level.

The side resistance of a pile can be calculated from the general equation, Kulhawy (1991) :

$$Q_s = \int_0^L K(z) \cdot z \cdot \gamma' \cdot \tan\delta \cdot \pi \cdot d \cdot dz \tag{4}$$

where L is the length of pile, d the diameter of pile, γ' the effective soil unit weight, K(z) the coefficient of earth pressure ($K = \sigma'_h / \sigma'_v$) σ'_h being the effective horizontal and σ'_v the effective vertical pressure at depth z and δ = interface friction angle. For cast-in-place concrete a rough interface develops resulting in $\delta = \varphi$. The value of Q_s is found from equation (5) in which Q_s is equal to the expression on the left hand side of the equal sign.

$$\begin{aligned}&\int_0^L K(z) \cdot z \cdot \gamma' \cdot \tan\delta \cdot \pi \cdot d \cdot dz = \\ &Q_{jack} - Q_g - \int_0^L K(z) \cdot \sigma_{v,z,b} \cdot \tan\delta \cdot \pi \cdot d \cdot dz\end{aligned} \tag{5}$$

In this equation Q_{jack} is the force exerted by the hydraulic jack, Qg is the selfweight of the pile, $\sigma_{v,z,b}$ is the vertical stress caused by the force from the hydraulic jack - equation (3) - and K(z) is the earth pressure coefficient.

Form figure 2 and 3 it can be seen that the correction due to the vertical forces from the reactions of the beams is almost negligible for the 14 cm and 25 cm piles because the supporting beams here had a length of 6 metres whereas the supporting beams for the 60 cm piles had a much shorter span leading to a substantial correction which is shown in figure 4.

The earth pressure coefficient K is as mentioned before assumed to vary with depth and for the test piles it turned out to be appropriate to express this variation by the equation :

$$K(z) = K_u \cdot \exp(-p \cdot \sqrt{z}) \qquad (6)$$

where K_u is the coefficient at ground level and z the distance measured from the ground level.

Backcalculations of the K values for the test data have yielded the following values for K_u and p for z < 6 metres :

14 cm piles : K_u = 2,8892, p = 0.5917
25 cm piles : K_u = 4.3214, p = 0.9345
60 cm piles : K_u = 4.6247, p = 0.8801

In figure 5 a graphical representation of the variation in K with depth is shown and as can be seen there is no significant difference between the K values and thus the unit side resistance for the three different diameters.

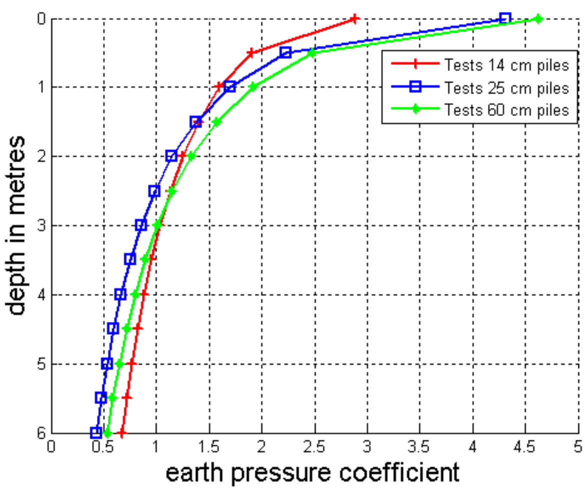

Figure 5. Variation of earth pressure coefficient K.

6 BEARING CAPACITIES ACCORDING TO STANDARDS

6.1 *British/American methods*

Fleming et al. (1992) suggest the unit side friction f_s to be calculated from the equation :

$$f_s = \sigma'_r \cdot \tan\delta = K \cdot \sigma'_v \cdot \tan\delta \qquad (7)$$

where K = 0.90 for all sands and 0.6 in silt; σ'_v is the vertical effective stress and δ the angle of friction in the interface between the soil and the pile and can be taken to be in the interval φ_{cv} and φ_{peak} No distinction is made between the values in tension and compression and in this study a value of δ equal to the average φ_{peak} = 41°. has been used.

On the basis of tests with 41 piles Reese & O'Neill (1988) have suggested the unit side friction to be calculated from the equation:

$$f_s = \beta \cdot \sigma'_z \qquad (8)$$

in which $\beta = 1.5 - 0.245 \cdot z^{0.5}$ and z is the depth below ground level and σ'_z is the vertical effective stress. It is assumed that 0.25 < β <1.20 and f_s < 200 kPa. For SPT values lower than 15 O'Neill has later recommended to scale down the side resistance by the factor N/15 (O'Neill, 1994).

The values according to the above mentioned methods together with the fitted values of the test results using the calculated values of K_u and p are summarized in figure 6, 7 and 8. In figure 9 are shown the fitted results from a test series of ten bored piles in loose sand with a diameter of 14 cm carried out by the authors in 2006, (Krabbenhoft et al. 2006). Also in figure 9 are shown the capacities computed according to the above mentioned methods.

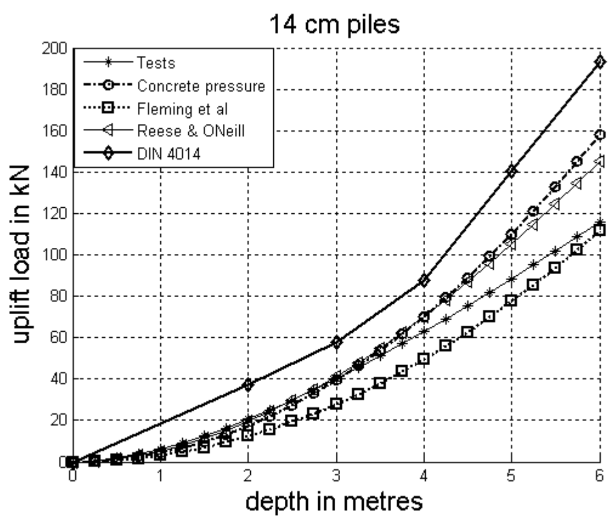

Figure 6. 14 cm piles in dense sand.

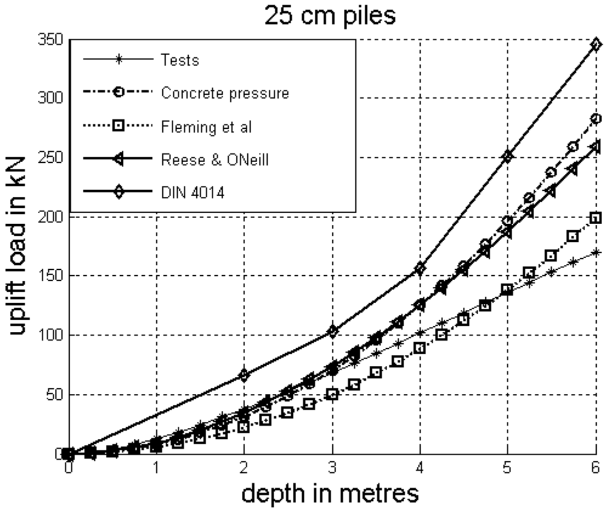

Figure 7. 25 cm piles in dense sand.

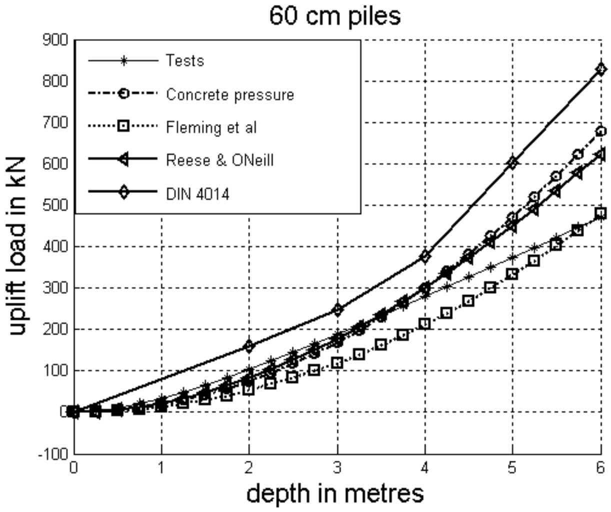

Figure 8. 60 cm piles in dense sand.

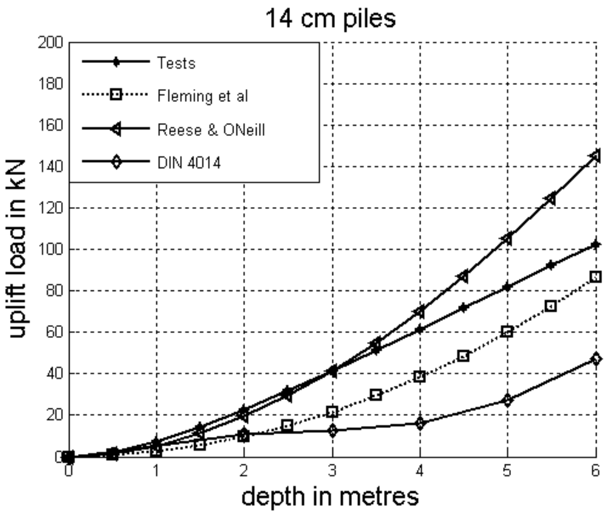

Figure 9. 14 cm piles in loose sand.

6.2 *The German Code of Practice – DIN 4014*

This code is based on a large number of tests for both cased and uncased borings and the unit side friction on the shaft may be related to the results obtained from a SPT test using the following equation :

$$f_s = a \cdot N \tag{9}$$

in which f_s is the unit side resistance in kN/m2 and N is the SPT blow count. The factor a takes on a value 4.14 for coarse sand and 2.73 for fine sand. The value of f_s should not be taken greater than 120 kN/m2. For the piles in this test a = 2.73 has been used and the calculated values to DIN 4014 code are indicated in figure 6, 7 and 8. For the DIN values in figure 9 a value of a = 4,14 has been used. The low values in figure 9 predicted by the German Code is due to very low SPT values.

Also in the figures 6, 7, 8 is shown the uplift capacity of the piles assuming the lateral pressure on the pile being equal to the pressure from the concrete during casting. It is interesting to see how close these values lie to the Reese & O'Neill values.

7 COMPARISON OF MEASURED AND COMPUTED VALUES

From the graphs in the figures 6-8 it can be seen that the German Code DIN 4014 is yielding the highest predicted ultimate values and the second largest predicted results are reached by the Reese and O'Neill method. The test results are in general in the interval between the capacities recommended by Reese and O'Neill and Fleming et al. and closer to the former for the shorter and closer to the latter for the longer piles.

The relatively high SPT blow counts reflect at the strongest on the DIN 4014 values whereas the Reese and O'Neill values are independent of the SPT for N > 15 blows.

By comparing the fitted test values for the 14 cm piles in dense sand (figure 6) with the fitted test values for the 14 cm piles in loose sand (figure 9) it can be seen that the values in dense sand on the average are only slightly above the values in loose sand indicating that the relative density of the sand only plays a minor role in estimating the uplift capacity. Also from figure 9 it can be seen that the test results in loose sand in general are in the interval between the values predicted by the Reese and O'Neill method and the Fleming Method.

8 CONCLUSIONS

Load tests on thirty piles with diameters 14 cm, 25 cm and 60 cm in dense sand have shown that the diameter of the pile has no significant effect on the unit side resistance. Also by comparison with tests carried out in loose sand it can be concluded that the uplift capacity of a bored pile is only to a smaller degree dependent upon the strength of the soil and this in line with both the method proposed by Fleming et al. and the Reese & O'Neill method.

The method proposed by Reese & O'Neill produces the best match with the test results for the shorter piles and for the longer piles the Fleming method gives values closer to the test values.

The German Code of practice DIN 4014 produces results which for a loose sand strongly understimates and for a dense sand overestimates the capacity of a bored pile.

9 ACKNOWLEDGEMENTS

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