TENSION TESTS ON BORED PILES IN SAND

ESSAIS DE TRACTION SUR PIEUX FORÉS DANS LE SABLE

Sven KRABBENHOFT[1], Johan Clausen[1] and Lars Damkilde[1]
[1] *Esbjerg Institute of Technology, Aalborg University Esbjerg, Niels Bohrs Vej 8 6700 Esbjerg, Denmark, Phone 45 79 127666, e-mail :skr@aaue.dk*

ABSTRACT - The lengths of the bored piles varied from 2 m to 6 m and all were of a diameter of 140 mm. The piles were tested to failure in tension and the load-displacement relations were recorded. The investigation has shown pronounced differences between the load bearing capacities in sand obtained by different design methods. The methods proposed by Fleming et al. and Reese & O'Neill seem to produce the best match with the test results.

RÉSUMÉ - in french

1. Introduction

Piling foundations are normally used in conditions, where the distance from the ground level to the firm layers able of carrying the actual load is more than three to four metres, and very often the choice is between driven piles usually made from reinforced concrete and bored concrete piles cast in situ. In terms of economy bored piles seem to be advantageous over driven piles at smaller depths, around 4 – 6 m, whereas driven piles are usually preferred at greater depths. The equipment used for driven piles is much heavier than the equivalent used for bored piles which - due to transportation costs - makes the latter more attractive, where only a smaller number of piles are to be installed. In urban areas there is always a risk of the driving of piles to cause damage to neighbouring buildings, and also in such circumstances bored piles may be preferable. Naturally a crucial factor when deciding the type of piles to be used at an actual project, must be the load bearing capacities, which can be attributed to the two types of piles.

The present investigation of the bearing capacity of piles is limited to dealing with the uplift capacity of bored piles in sand, as this seems to be an area of disagreement among the codes of practice of various countries. In order to examine this issue ten bored piles of lengths varying from 2 - 6 m all of a diameter of 140 mm have been tested, and furthermore the capacities according to The Danish code of practice – DS 415, The German code of Practice – DIN 4014 and British/American practice have been calculated.

2. The testing programme and results

2.1. The testing area and soil conditions

The testing area is situated in a gravel pit in Oksbol, Denmark and the layout of the ten piles is shown in figure 1. It should be noted, that the same designation has been used for the individual borings and the corresponding piles, that is boring P1a corresponds to pile P1a and so forth. 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 not reached in any of the borings.

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

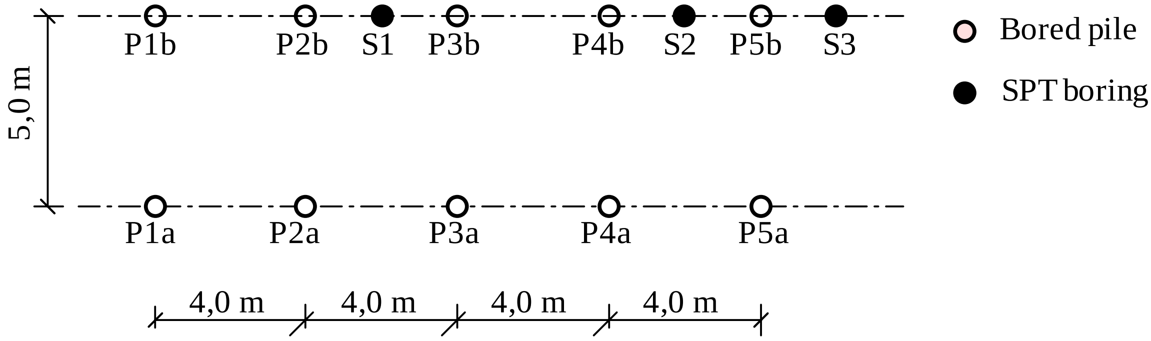

Figure 1. Pile layout and designations.

From the sample in P5a at a depth of 3 m the mean grain size d_{50} is app. 1.00 mm, and due to the homogeneous soil conditions this sample can be regarded as being a representative of the whole area. The specific gravity of the quartz sand is 2.63 and the void ratios e_{max} and e_{min} from samples in P3a and P5a have been determined producing the following results :

P3a : e_{max} = 0.732, e_{min} = 0.446, P5a : e_{max} = 0.729, e_{min} = 0.444.

Around P5a undisturbed samples have been taken for the determination of the void ratios and relative densities. The results are indicated in table I.

Table I. P5a. In situ void ratios and relative densities

Depth m	0.50	0.50	0.70	1.40	1.40
Void ratio e	0.58	0.59	0.64	0.65	0.71
Relative density I_d	0.523	0.488	0.312	0.277	0.067

Originally it had been planned to dig all the piles free, and in connection with this to take out undisturbed samples at intervals of 1 m to determine the void ratios, as this must be regarded as one of the best ways to determine the angle of friction of the sand. However this turned out to be practically impossible and instead three SPT tests indicated by S1, S2 and S3 in figure 1 were carried out. The results of these tests are shown in table II. The values N given in the table are average values of blows per 30 cm over an interval of 1 m.

On the basis of the results from the SPT tests the relative densities I_d have been determined from the formula by Kulhawy and Mayne (1990):

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

For the mean grain size 1.00 mm has been used. The triaxial angle of friction has been calculated using the equation developed by Bolton (1986)

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

In this equation p' is the mean principal effective stress at failure ≈ twice the value of the vertical effective stress σ'_z, (Rollins et. al., 2005) and φ_{cv} is the friction angle at constant volume which is taken to be 33°. (Bolton, 1986). From table II it can be seen, that the sand must be characterised as being loose or very loose, and it is also clear, that there is very little variation in the soil conditions.

Table II. Results of SPT tests

Depth in m	S1			S2			S3		
	N	I_d	φ_{tr} °	N	I_d	φ_{tr} °	N	I_d	φ_{tr} °
0 – 1.0	5	0.29	36	4	0.26	36	5	0.29	36
1.0 – 2.0	3	0.22	34	1	0.13	34	2	0.18	33
2.0 – 3.0	1	0.13	32	1	0.13	32	1	0.13	32
3.0 – 4.0	1	0.13	32	2	0.18	33	1	0.13	32
4.0 – 5.0	5	0.29	34	10	0.41	36	4	0.26	34
5.0 – 6.0	14	0.48	37	12	0.45	36	9	0.39	35

2.2. Construction of piles

In the open holes left by the continuous flight augers a single reinforcing rod of a diameter of 20 mm was put in position, and the concrete was placed by gravity free fall. The quality of the concrete was 25 MPa and when the hole had been filled to the ground level, a 16 mm high strength steel bolt of length 2 m was installed to a height of 1 m above the top of the concrete. No centralizers to keep the reinforcing bar in position were used, and as a result of this the bar in pile P2b was not properly installed. This was discovered during the extraction of the pile after testing. In pile P5a the reinforcing bar stopped 3 m above the tip of the pile. Two piles of each length were constructed. Construction of the piles took place in May 2005.

2.3. Testing of piles

The uplift tests were carried out in September 2005. The load was applied to the piles by a hollow ram hydraulic jack resting on two steel beams of type IPE 240 of length 6 m supported at either end by 100x200 mm timber. The purpose of the two longitudinal beams was to avoid any transfer of additional horizontal forces to the piles. 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 is shown in figure 2.

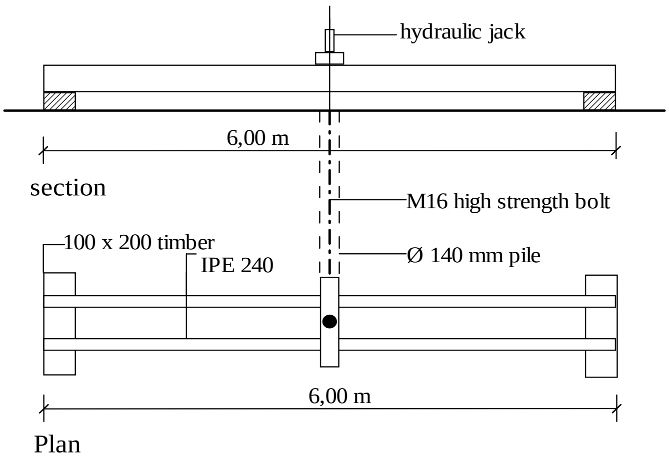

Figure 2. Load test arrangement.

One week after the uplift tests some of the piles were extracted with a view to visual inspection and control of dimensions. All these piles showed a very rough surface with soil adhered to the shaft. The diameter of the piles were measured in intervals of 1 m and there were only minor discrepancies between the designed (140 mm) and the constructed values (average 142 mm) No account has been taken in the calculations of this small difference.

2.4. Results of tests

The results of the tests are shown in table III and in figure 3 and 4.

Table III Dimension of piles and maximum loads. * : pile was not extracted

Pile no.	Designed length [m]	Constructed Length [m]	Maximum load [kN]	Displacement at max. load [mm]	Length/ Diameter
P1a	2.00	1.75	26.0	10	14
P1b	2.00	2.15	16.8	8	
P2a	3.00	2.96	39.1	12	21
P2b	3.00	3.05	43.6	11	
P3a	4.00	3.98	66.9	10	29
P3b	4.00	4.00 *	68.7	12	
P4a	5.00	4.85	85.9	15	36
P4b	5.00	5.00 *	56.8	12	
P5a	6.00	3.40	84.0	10	43
P5b	6.00	6.00 *	110.2	22	

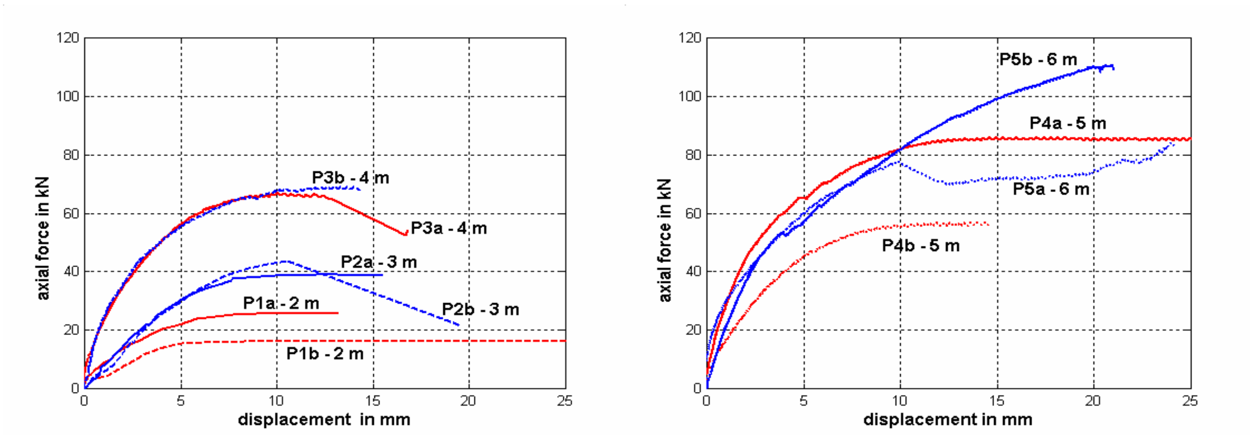

Figure 3 & 4. Load – displacement graphs

3. Bearing capacities according to standards

3.1. The Danish Code of Practice – DS 415

According to this code the axial load that a bored pile is able of sustaining can be found from the equation :

$$Q_s = 0.3 \cdot N_m \cdot q_m \cdot A_s \tag{3}$$

where N_m = 0.2 for piles in tension and N_m = 0.6 for piles in compression, q_m is the mean vertical effective stress over the length of the pile and A_s is the area of the surface of the pile. Current opinion is that there is no systematic difference in the value of skin friction in piles subject to tension and compression. (Fleming et al.1992) The calculated values to DS 415 code are indicated in table IV.

3.2. The German Code of Practice – DIN 4014

In contrast to the Danish Code, this code does not distinguish between the surface resistance in tension and compression and the rules are based on a large number of tests for both cased and uncased borings. The unit side friction on the shaft may be related to the results obtained from a SPT test using the following equation :

$$f_s = 2.86 \cdot N \tag{4}$$

in which f_s is the unit side resistance in kN/m^2 and N is the SPT value.

3.3. British/American methods

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

$$\tau_s = \sigma'_r \cdot \tan\delta = K \cdot \sigma'_v \cdot \tan\delta \tag{5}$$

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 pile and the soil and can be taken to be in the interval φ_{peak} and φ_{cv} . No distinction is made between the values in tension and compression.

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

$$\tau_s = \beta \cdot \sigma_z^{'} \tag{6}$$

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

Kraft and Lyons (1974) also suggest using equation (6) with the values β = F tan(φ' -5°) and F = 0.5 for piles in tension and F = 0.7 for piles in compression. In this study δ = φ' = 34° as an average has been used and the values according to the above mentioned methods together with the test results are summarized in Table IV.

Table IV Failure loads in kNewton for piles in tension. * : only P5b

Pile length [m]	Test result	DS 415	DIN 4014	Fleming et al.	Reese & O'Neill	Kraft & Lyons
2	21.5	1.0	8.3	9.6	18.8	4.4
3	41.3	2.1	9.5	21.6	40.8	9.9
4	67.9	3.8	11.1	38.5	69.7	17.6
5	71.4	5.9	19.0	60.1	104.6	27.4
6	110.2 *	8.6	34.0	86.5	144.9	39.5

4. Comparison of Measured and Computed Capacities

From Table IV it can be seen that the Reese and O'Neill method is yielding the highest predicted ultimate values and the Danish Code DS 415 the lowest values. The second largest predicted results are obtained by the method proposed by Fleming et al. and the German Code DIN 4014 and the Kraft and Lyons method are producing results that although being based on quite different assumptions on average only differ app. 17%. 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 except for the longer piles.

Considering the unusual low relative densities of the sand in the test field the measured capacities must be regarded as being surprisingly high although the free-fall placement of the concrete must be expected to increase the lateral pressure between the side of the pile and the borehole (O'Neill and Hassan, 1994). On the other hand the Reese and O'Neill method is not taking the relative density of the sand into account and the Fleming method is based on a fixed value of K (K = 0.9).

Unlike these two methods the capacities obtained by the German Code are strongly dependant upon the relative density and this explains the comparatively low values calculated in this study.

5. The earth pressure coefficient K

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

$$Q_s = \int_0^L K(z) \cdot z \cdot \gamma \cdot \tan\delta \cdot \pi \cdot d \cdot dz \qquad (7)$$

where L = length of pile, d = diameter of pile, γ = effective soil unit weight, K(z) = coefficient of earth pressure (K = σ'_h / σ'_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 δ = φ. The most difficult term to evaluate is K and values of K from analysis of tests range from about 0.1 to over 5; Kulhawy (1991). Several researchers, e. g. Reese and O'Neill (1988) have found that K varies with the depth z and for the test piles this variation has been expressed by the equation :

$$K(z) = K_u - z^p \qquad (8)$$

where K_u is the coefficient at ground level and z the distance measured from the ground level. Backcalculated K values for the test data have yielded a value of *K_u = 3.06 and of p = 0.487.*

For soil at rest the coefficient K is normally taken ≈ 0.5 and the reason for the increased values has by researchers been attributed to the increase in effective lateral pressure due to dilation. (e.g. Lehane and White 2005; Fioravante 2002). The angle of dilation is the greater the smaller the effective stress this being in accordance with the variation in K with depth. The effect of dilation has amongst others been investigated by Houlsby (1991), who has computed the increase in soil pressure caused by that effect on a pile of diameter 0.60 m based on the equation:

$$\sigma'_{rd} = 2 \cdot G \frac{\delta R}{R} \qquad (9)$$

where σ'_{rd} is the increase in horizontal pressure due to dilation, G is the shear modulus of the soil, R the radius of the pile and δR the expansion in horizontal direction of the zone of soil lying close to the side of the pile undergoing plastic deformations, as the load on the pile is increased. The soil lying outside of this zone is regarded as being elastic, but pointed out by Houlsby, this is a simple model and perhaps the most difficult property to predict, is the thickness of the plastic zone, which by some researchers (e. g. Atkinson, 1993) are believed to be of the order of 10 times the mean grain size. A consequence of this would be, that the failure load is proportional to the mean grain size for the same angle of friction, and Rollins et al. (2005) have studied the results of several tests and found, that the coarser the material the greater the shear stresses transferred between the soil and the pile. Using equation (6) they found for gravel content between 25 og 50% : $\beta = 2 - 0.15 \cdot z^{0.75}$ and for gravel content greater than 50% : $\beta = 3.4 \cdot e^{-0.137 \cdot z}$. The SPT values were greater than 25. Tests with model piles having a diameter = 70 mm and length = 700 mm carried by the authors have not been able to confirm the dependence of shear stress of the mean grain size.

According to the above equation (9) the failure load is inversely proportional to the diameter of the pile, but this does not correspond to results produced by Turner and Kulhawy (1994) who found that the reduction in shear stress increasing the diameter from 150 mm to 300 is about 20% and that further increase of the diameter results in a reduction of only 5 – 10%.

6. Conclusion

On the basis of the field tests and the subsequent studies the following conclusions can be drawn :

- The Danish Code underestimates the ultimate failure load by a factor of the order of 10.
- The soil pressure coefficient varies and decreases with depth.
- The horizontal soil pressure on piles in loose sand seems to take on values that are not very. different from the values in firmer sand. This may be due to an increase in the lateral stresses caused by free-fall placement of the concrete.
- There seems to be no difference in side friction for piles in tension and compression.
- The friction angle δ for uncased borings is equal to the friction angle φ of the soil.
- There seems to be considerable disagreement among the various design methods as to the. importance of the friction angle of the soil in relation to the uplift capacity.

7. Acknowkedgements

The authors wish to thank The Lida and Oskar Nielsen Foundation for their financial support and also the field work carried out by the company Jysk Geoteknik is appreciated.

8. References

Atkinson, J. (1993) *An Introduction to The Mechanics of Soils and Foundations,* McGraw-Hill Book Company.

Bolton, M.D. (1986) The Strength and Dilatancy of Sands. *Geotechnique, Vol. 36, No. 1, 65-78*

DIN 4014, *Bohrpfähle. March 1990.*

DS 415, *Code of Practice for Foundation Engineering (1998)* Danish Standards Association.

Fioravante, V. (2002) On the shaft friction modelling of non-displacement piles in sand*. Soils and Foundations, Vol. 42 Issue. 2, 23-33.*

Fleming W.G.K., Weltman A.J., Randolph M.F., Elson W.K. (1992) *Piling Engeering.* Taylor & Francis Group, London and New York

Houlsby, G.T. (1991) How the Dilatancy of Soils Affects Their Behaviour*. Proceedings of the Tenth European Conference on Soil Mechanics and Foundation Engineering, Florence, May 27-30, Vol. 4, 1189-1202*

Kraft, L.M., Lyons, C.G. (1974). State of the art – ultimate axial capacity of grouted piles. *Proceedings of the 6th Annual OTC, Houston, Texas, pp. 485-504.*

Kulhawy, F.H., Mayne, P.W. (1990). *Manual on estimating soil properties for foundation design. Rep. No. EPRI EL-6800*, Electric Power Research Institute, Palo Alto, Calif. 2-25.

Kulhawy, F.D. (1991). Drilled Shaft Foundations. *Foundation Engineering Handbook, 2nd Ed.*, H. Y. Fang, ed., Van Nostrand-Reinhold, New York.

Lehane, B.M., White, D.J. (2005) Lateral stress changes and shaft friction for model displacement piles in sand. *Canadian Geotechnical Journal 42 (4): 1039-1052 August 2005*

O'Neill, M.W., Hassan, K.M. (1994). Drilled Shafts : Effects of construction on performance and design criteria. *Proc., Int. Conf. Des. Constr. Deep Founds., Orlando, FHWA, 1, 137-187.*

O'Neill, M.W. (1994). Drilled Shafts*. Proc., International Conf. on Design and Construction of Deep Foundations, Fed. Highway Admin., Washington, D.C., Vol. 1, 185-206.*

Reese, L.C., O'Neill, M.W. (1988). Drilled Shafts : Construction procedures and design methods. *Pub. No. FHWA-HI-88-042, U.S. Dept. of Transportation, Washington, D.C., 564-564.*

Rollins, K.M., Clayton, R.J., Mikesell, R.C., Blaise, B.C. (2005), Drilled Shaft Side Friction in Gravelly Soils. *Journal of Geotechnical and Geoenvironmental Engineering, Vol. 131, No. 8, August 1, 2005.*

Turner J.P., Kulhawy F.H (1994) Physical Modeling of Drilled Shaft Side Resistance in Sand. *Geotechnical Testing Journal, GTJODJ, Vol. 17, No. 3, September 1994, pp. 282-290.*