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Design of bored piles in Denmark – a historical perspective

Conception de pieux forés au Danemark - une perspective historique

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ABSTRACT: According to the Danish National Annex to Eurocode 7, part 1 (2015), Annex L the shaft resistance for a bored cast-in-place pile is limited to 30 per cent of the shaft resistance of the corresponding driven pile and the design toe resistance is limited to 1000 kPa unless recognised documentation allowing a larger bearing resistance is available. This principle has been enforced in Denmark (Code-requirement introduced in the Code of Practise for Foundations in 1977, with further adjustments regarding allowable toe resistance in 1984 and 1998) allegedly due to problems encountered in one or two un-documented case histories.

This paper presents the historical aspects of the code-requirement as well as a case study on the results of recent full-scale static compression pile load tests on two bored piles performed on clay till in Nyborg, Denmark. The results from the full-scale tests are compared with an analytical method based on the Danish National Annex to Eurocode 7, part 1 (2015) which represents a conservative estimate for the bored piles. Collectively the historical perspective and the test results show that the code- requirement, limiting the shaft resistance to 30 per cent of the corresponding driven pile, is too conservative.

RÉSUMÉ: Selon l'annexe nationale danoise à l'Eurocode 7, partie 1 (2015), annexe L, le frottement latéral pour un pieu foré coulé sur place est limitée à 30% d'un même pieu battu et la résistance de pointe est limitée à 1000 kPa, à moins que des documents justificatifs permettent de démontrer qu'une résistance supérieure soit disponible. Ce principe a été apparemment appliqué (une exigence des normes à partir de 1977, avec amendements au sujet de la résistance de pointe admissible en 1984 et 1998) au Danemark en raison de problèmes rencontrés dans un ou deux cas non documentés.

Cet article présente les exigences des normes en question sous un aspect historique ainsi qu'une étude de cas sur les résultats d'essais de chargement de pieux grandeur nature en compression statique. Ces essais ont été réalisés sur deux pieux forés dans du till argileux à Nyborg au Danemark. Les résultats de ces essais grandeur nature sont comparés à une méthode analytique, utilisée comme estimation prudente pour les pieux forés, basée sur l'annexe nationale danoise à l'Eurocode 7, partie 1 (2015). Dans l'ensemble, l'historique et les résultats des tests montrent que l'exigence des normes actuelles limitant la résistance du frottement latéral à 30% du pieu battu correspondant est trop prudente.

Keywords: Bored cast-in-place piles; Danish Code- requirement: Historical aspect; Case study

1 INTRODUCTION

Denmark has a long tradition of using driven precast concrete piles, with the first driven pile installed in Denmark in 1904 (Winkel, 1960). Bored cast-in-place piles however, are not traditionally used in Denmark, and even today, bored piles are still not used regularly.

According to the Danish National Annex to Eurocode 7, part 1 (2015), Annex L (DS/EN_1997-1_DK_NA, 2015) the shaft resistance for a bored pile is limited to 30 per cent of the shaft resistance of the corresponding driven pile and the design toe resistance is limited to 1000 kPa unless recognised documentation allowing a larger bearing resistance is available. These restrictions were introduced in the Danish Code of Practise for Foundations (DS415, 1977) in 1977 (with further adjustments regarding allowable toe resistance in 1984 and 1998). Allegedly due to problems encountered in one or two un-documented case histories.

However, it is widely recognised that the reduction in shaft resistance imposed by (DS/EN_1997-1_DK_NA, 2015) is overly conservative, if the bored pile is established correctly. Nevertheless, the reduction is still enforced due to limited understanding of the governing mechanisms, limited knowledge of the complex soil-pile interaction and a lack of documentation from full scale testing. Consequently, bored pile design in Denmark is today overly conservative and bored piles are considered economically unattractive.

The use of bored piles in Denmark is however increasing, due to construction in steadily more challenging areas, requirements to reduce noise and vibration in the cities, taller and more complex buildings with larger loads and increasing demands towards stiffness of the foundation.

The aim of this paper is to shed light over the historical background for the code-requirement from 1977, and furthermore, to show the consequences today a case study of two full-scale static compression pile load tests at Nyborg Slot (Nyborg castle) is presented.

2 HISTORICAL PERSPECTIVE

The authors have tried to map the historical background and likely motivations for the code-requirement from 1977 based on articles, personal interviews and archive reports, notes and minutes of historical events and discussions. However, the exact reasons cannot be ascertained as they have not been documented. A timeline of events from 1972 till 2000 is shown in Figure 1.

In 1972 the pile supported Fiskebæk Bridge collapsed (Krak, 2001, Vejdirektoratet, 2018). This case is by many believed to be one of the undocumented case histories which triggered an investigation into the installation procedures and capacity of bored piles in the early 1970’s that ultimately lead to the aforementioned code-requirements for bored piles in 1977. The collapsed section of the Fiskebæk Bridge was supported by Frankipiles, with a length of 20 m and a diameter of 0.5 m (JL, 1972). Frankipiles are displacement piles, and the execution method is described by (Winkel, 1960).

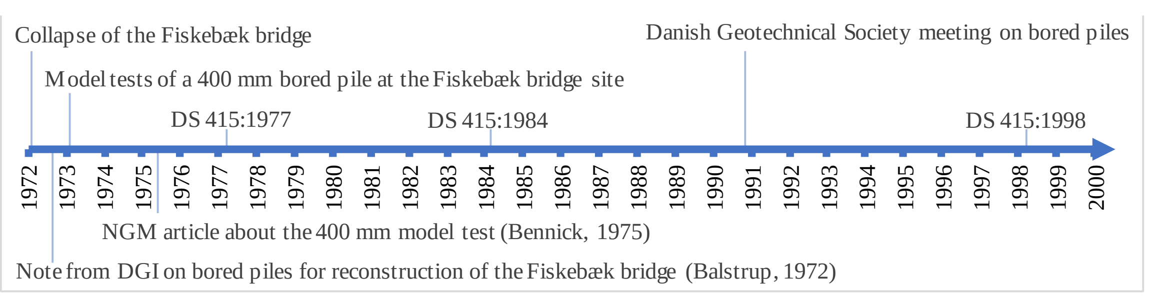

Figure 1. Timeline of events leading up to and in the years after the code-requirement from 1977.

The Investigation Committee concluded that critical executions errors on the Franki piles had caused the Fiskebæk Bridge to collapse. Franki piles are however not traditional bored piles but cast-in-situ displacement piles. Hence, the collapse of the Fiskebæk Bridge was not the direct reason for the code-requirement.

Nevertheless, following the bridge collapse model tests on 400 mm bored piles were carried out at the Fiskebæk Bridge site to further investigate the capacity of bored piles which were considered as an alternative to Franki piles.

2.1 Danish experience with bored piles in the 1970's

Danish experience with bored piles in the 1970's was limited, and the use of bored piles considered unusual. Bored piles were generally considered special piles, and no distinction was made between the different types of bored piles and installation techniques. The shaft resistance in fine grained soil was determined by the following equation:

$$\mathrm{R}_{s;cal} = \mathrm{k} \cdot \mathrm{r} \cdot \mathrm{c}_u \cdot \mathrm{A}_s = \alpha \cdot \mathrm{c}_u \cdot \mathrm{A}_s \qquad (1)$$

where $\mathrm{R}_{s;cal}$ (kN) is the calculated shaft resistance, k (-) is a reduction factor valid for bored piles (k = 1 for driven piles), r = 0.4 is the regeneration factor (strength reduction of clay due to remoulding) valid for $c_u \leq 500\ kPa$, c_u (kPa) is the undrained shear strength which is assumed equal to the field vane strength c_{fv} and A_s is the surface area of the pile (m^2). The multiplication of k and r is equal to the typical used reduction factor α.

Table 1 gives and overview of pile load tests performed in Denmark prior to the late 1970's. Only seven cases (all undated and unpublished) have been found, where six cases are in clay till and one case is in high plasticity, fissured Paleogene clays Septarieler and Søvind Marl. Four different pile installation methods were used in the listed cases. Benoto piles and the colcrete method are described by (Winkel, 1960). Scandrill is described by (Knudsen, Undated) and the derrick is described by (Bennick, 1975). Pile diameters range from 0.22 to 0.50 m.

The intact field vane strength for the test cases (A and B) with Benoto piles and Scandrill piles was too high to assume an regeneration factor, r = 0.4 which is valid for $c_u \leq 500\ kPa$. Scandrill piles were also stabilised by a bentonite slurry, which reduces the shaft resistance.

The piles installed using the Colcrete method (test cases C, D and E) were displacement piles and shares no resemblance with traditional bored piles, and the piles from test case F has no information regarding installation.

Table 1. Pile load tests available in the 1970's (Knudsen, Undated), (Balstrup, 1972) and (Bennick, 1975)

Test case	Soil type	Pile length	Diameter	Field vane strength	Reduction factor		Installation method	Reference
	(-)	(m)	(m)	(kPa)	k (-)	α (-)	(-)	(-)
A	Clay till	9	0.50***	690 - 980	0.28	0.11	Benoto	DGI 5725*
B	Clay till	-	0.22	690 - 980	0.46	0.18	Scandrill	DGI 5725*
C	Clay till	-	0.40	70	0.85	0.34	Colcrete	CTW**
D	Clay till, sand	-	0.45	120 - 255	0.41	0.16	Colcrete	DGI 63510*
E	Septarieler / Søvind Marl	-	0.45	110 - 295	0.38	0.15	Colcrete	DGI 63510*
F	Clay till	-	-	255	0.58	0.23	NA	DGI
G	Clay till	20.9	0.40	165 - 590	0.50	0.20	Derrick	NGM 1975

* Local project no.; ** C. T. Winkel; *** Diameter of pile toe

The pile from test case G installed by the derrick was a test pile with an installation time of approximately 1 month before casting of the pile. The long installation time allows for the clay till to fully soften and reduce the shaft resistance.

From the seven cases the reduction factor, k was found to range from 0.28 to 0.85 in Clay till and 0.38 for high plasticity clays.

2.2 Shaft resistance of bored piles – experience from abroad

Experience from London clay was available in the 1970's, and this was particulary relevant because the Eocene London Clay like the Danish Palaoegene clays is characterised as a stiff and fissured marine sedimentary clay.

Pile load tests on ten different sites in the London area was investigated by (Skempton, 1959), and a reduction factor $0.3 \leq \alpha \leq 0.6$, with a recommended value of $\alpha = 0.45$ was determined for London clay with an average c_u of $215\ kPa$. The undrained shear strength used by (Skempton, 1959) was obtained from triaxial compression tests $c_{u,comp.}$ on 38 mm diameter specimens (Clayton and Milititsky, 1983). The ratio between the undrained shear strength from triaxial tests on 38 mm diameter specimens and penetration tests were determined cf. (Marsland, 1972) to approximately 0.46 in average (ranging between 0.37 and 0.55). This is comparable to Danish experience ($c_{fv} \sim 2$ á $3 \cdot c_{u,comp.}$).

If it is assumed, that the shear vane strength determined from penetration test measured in the laboratory (intact strength) and field vane test are equal, then Skempton's study will lead to a reduction factor $\alpha = 0.45 \cdot 0.46 = 0.2$ if used together with the intact field vane strength.

2.3 Danish recommendations regarding design of bored piles.

Based on the Danish experience with bored piles as well as experience from abroad (primarily London Clay), several suggestions for calculation of bored piles were made by (Knudsen, Undated), (Balstrup, 1972) and (Bennick, 1975). On this basis an apparent lower bound reduction factor k = 0.3 ($\alpha = 0.12$) was implemented in the Danish Code of Practice for foundations (DS415, 1977) for the first time in 1977. This is still enforced today in (DS/EN_1997-1_DK_NA, 2015) and it effectively means that the shaft resistance for a bored pile is limited to 30 per cent of the shaft resistance of the corresponding driven pile. The shaft resistance in fine grained soil can therefore be determined from Equation 1, which gives:

$$\mathrm{R}_{s;cal} = 0.3 \cdot 0.4 \cdot \mathrm{c}_u \cdot \mathrm{A}_s = 0.12 \cdot \mathrm{c}_u \cdot \mathrm{A}_s \quad (2)$$

3 CASE STUDY IN DENMARK

Nyborg Slot is a restored Medieval castle in Nyborg on Funen in Denmark and was built towards the end of the 12th century. The castle became a museum after an extensive renovation from 1917 to 1923. Today, the museum is temporarily closed for new extensive renovation and expansion.

Bored piles support a part of the new expanded castle, and two bored piles, P1 and P2 were installed and tested by static compression pile load test. Dimensions of the bored piles are shown in Table 2.

Table 2. Dimensions of test piles P1 and P2.

Pile no.	Diameter (mm)	Length (m)
P1	Ø620	13.0
P2	Ø620	12.8

3.1 Soil profile and methods

The test piles were installed using the Vor Der Wand (VDW) drilling method in a soil profile dominated by clay till. The strength profile and ground conditions for P1 and P2 are given in Figure 2 and Figure 3, respectively.

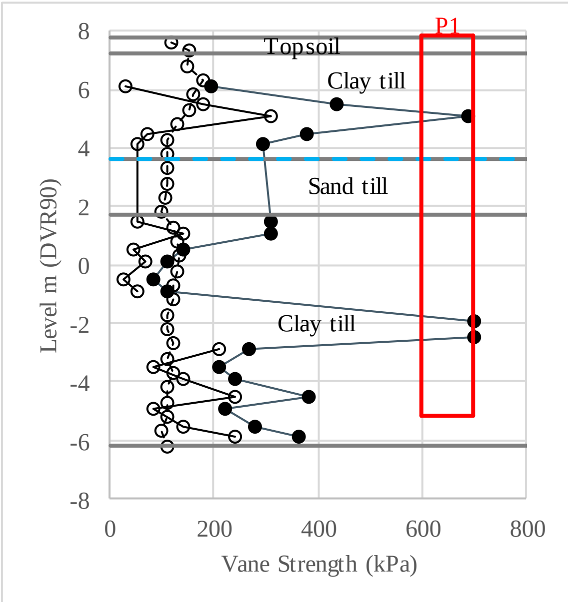

Figure 2. Ground conditions for P1.

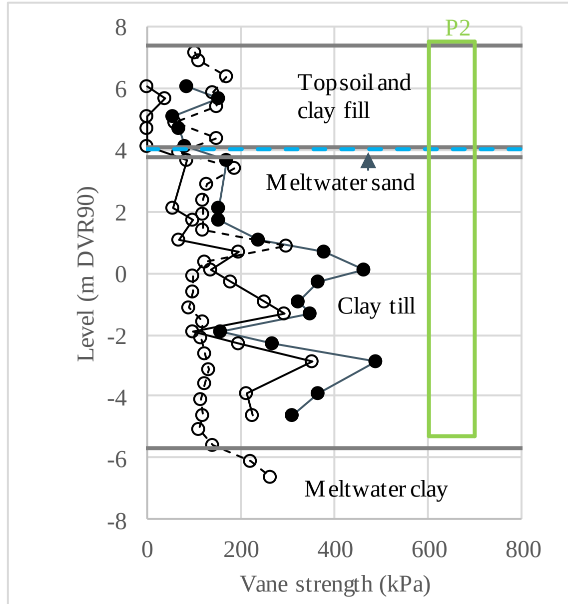

Figure 3. Ground conditions for P2.

VDW drilling is characterised by simultaneously drilling of the auger and casing, where the auger head and casing can be adjusted to approximately 300 mm of each other. Adjustment of the auger head relative to the casing depends on the ground conditions. The auger head is then drilled down to the required depth, and the spoil is ejected through openings on the upper end of the casing. The concrete is pumped through the swivel of the auger head, while retracting auger and casing simultaneously during the concreting. The reinforcement is then installed after concreting.

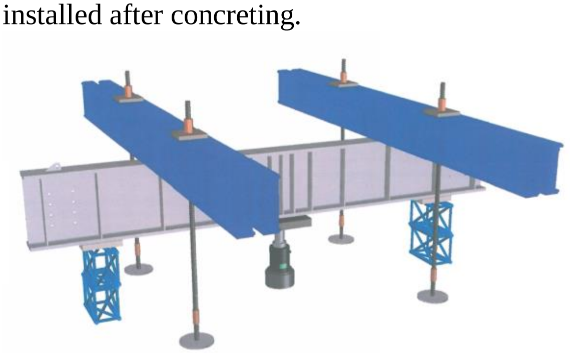

Figure 4. Test set-up for static compression pile load tests (Bartsch, 2018).

Test piles P1 and P2 were tested using static compression pile load tests by DMT Gründungstechnik Gmbh in accordance with the requirements given in EA-Pfähle (2012). Four reaction piles of the samt length and diameter as the respective test pile were established around each test pile to provide the reaction force. The force was transferred from the actual test pile to the corresponding reaction piles by means of a system of steel beams as shown in Figure 4.

A maximum load of 3 MN was applied by one hydraulic jack at the centre of the pile axis. The hydraulic jack had a capacity of 7.4 MN and a stroke of 250 mm. The load was measured by a 5 MN load transducer (HBM C6A 5 MN circular load cell). The accuracy was 1 kN and reading were carried out with 0.1 kN resolution.

The vertical displacement of the test piles was measured against an independent reference beam by means of three displacements transducers, and two additional transducers were used to moniter the horizontal movement (HBM WA100-T displacement transducers with 100 mm range). The accuracy was < 0.001 mm, and readings were carried out with 0.01 mm resolution.

All data were acquired from the transducers a HBM QuantumX MX840B amplifier using Catman AP software. Data were collected with a frequency of 1 Hz.

During the static compression the load was increased or decreased over a time interval of 5 min. The load was maintained for 10 min or until the displacement rate of the pile was less than 0.1 mm per 5 min.

For the maximum load in load cycles 1 and 2 the load steps were maintained for at least 15 min and 60 min, respectively cf. Figure 5.

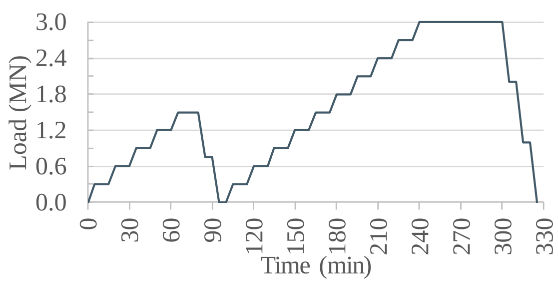

Figure 5. Load steps, digitized cf. (Bartsch, 2018).

3.2 Results and discussion

3.2.1 Load displacement curves and mobilised pile capacities

The results from tests on P1 and P2 are summarized in Table 3 and Table 4. The period of rest from installation to load testing was 47 and 49 days, respectively.

Table 3. Compression pile load test P1, 16.01.2018

Load cycle	**V_{max} (kN)**	**Δ_{max} (mm)**	**Δ_0 (mm)**
1st	1500	4.57	3.06
2nd	3000	10.58	7.52

Table 4. Compression pile load test P2, 18.01.2018

Load cycle	**V_{max} (kN)**	**Δ_{max} (mm)**	**Δ_0 (mm)**
1st	1500	3.54	1.59
2nd	2700	12.70	-
	3000	25.43	20.75*

* Displacement rate of < 0.1 mm/5 min. was not achieved for Vmax = 3000 kN and load step was stopped after 90 min.

V_{max} (kN) is the maximum load, Δ_{max} (mm) is the displacement at maximum load (before unloading) and Δ_0 (mm) is the permanent displacement after complete unloading.

The load-displacement curves for the static compression pile load tests on test piles P1 and P2 are shown in Figure 6.

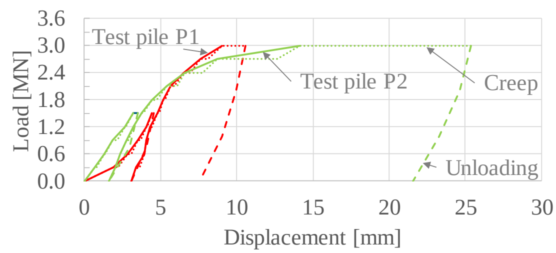

Figure 6. Load-displacement curves for static compression pile load tests on P1 and P2, digitized cf. (Bartsch, 2018).

3.2.2 Calculated shaft resistance of corresponding driven piles

An analytical method was used to determine the expected shaft resistance of the corresponding driven piles, where realistic strength parameters (best estimate) in the clay deposits were applied. The analytical method was based on equations according to (DS/EN_1997-1_DK_NA, 2015):

$$R_{c,cal} = R_{s,cal} + R_{b,cal} \tag{3}$$

where $R_{c,cal}$ (kN) is the expected bearing resistance for compression piles, $R_{s,cal}$ (kN) is the shaft resistance, and $R_{b,cal}$ (kN) is the toe resistance.

Shaft resistance for fine grained soils:

$$R_{s,cal} = \sum m \cdot r \cdot c_u \cdot A_s \tag{4}$$

Shaft resistance for coarse grained soils:

$$R_{s,cal} = \sum N_m \cdot q'_m \cdot A_s \tag{5}$$

Toe resistance for fine grained soils:

$$R_{b,cal} = 9 \cdot c_u \cdot A_b \tag{6}$$

where m (-) is a material factor, r (-) is a regeneration factor, c_u (kPa) is the undrained shear strength, N_m (-) is a bearing capacity factor, q'_m (kPa) is the effective overburden pressure, A_s (m²) is the shaft surface area, and A_b (m²) is the base area.

3.2.3 Estimated shaft resistance of bored piles

The total resistance from the static compression pile load tests were determined to be > 3000 kN for P1 and P2. However, P2 showed evidence of imminent failure, see Figure 7.

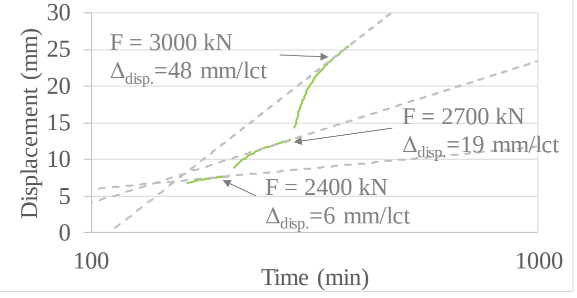

Figure 7. Time-displacement curves for the three last load step of static compression pile load test P2, digitized cf. (Bartsch, 2018).

The shaft resistance was determined by the bearing resistance from the static compression pile load tests, where the mobilised toe resistance, F_{mob} (kN) was deducted. The mobilised toe resistance from the static compression load tests was determined assuming the following relationship:

$$\frac{\Delta_{mob}}{\Delta_{failure}} = \left(\frac{F_{mob}}{F_{failure}}\right)^2 \quad (7)$$

where $\Delta_{failure}$ (mm) is the deformation corresponding to complete failure, Δ_{mob} (mm) is the actual deformation and $F_{failure}$ (kN) is the analytically calculated toe resistance cf. equation (6). Failure of toe resistance is assumed at a deformation corresponding to approximately 10 % of the pile diameter (62 mm) cf. (Hansen, 1965).

3.2.4 Bored piles vs. driven piles

The shaft resistance for the bored test piles determined by the static compression pile load tests are in the following sections compared with the shaft resistance of the corresponding driven piles.

Based on the static compression pile load tests P1 and P2 performed in connection with renovation and expansion of Nyborg Slot, the shaft resistance for the bored piles was higher than 30 % of the shaft resistance for the corresponding driven pile as illustrated in Figure 8.

The derived value of shaft resistance for test pile 1 was 2.72 MN, which is 232 % higher than 30 % of the shaft resistance for the corresponding driven pile. This corresponds to a value of $\alpha = 0.4$. For test pile 2, the derived value of shaft resistance was 2.57 MN, which is 274 % higher than 30 % of the shaft resistance for the corresponding driven pile. This corresponds to a value of $\alpha = 0.45$.

The values of α derived from the static compression pile load tests exceeds the values obtained from the pile load test available in the 1970's (cf. Table 1) with 18 % to 309 %.

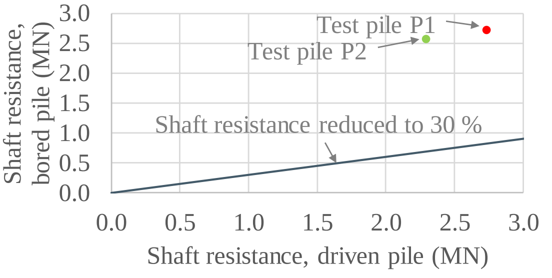

Figure 8. Shaft resistance for bored vs. driven piles.

It should be noted that all calculations were based on measured values, which means that the use of correlation factor ξ values and partial factor γ_s according to (DS/EN_1997-1_DK_NA, 2015) is omitted.

4 CONCLUSIONS

Experience with bored piles was limited in Denmark back in 1977, where the restrictions in the allowable bearing capacity in the Danish Code of Practice for Foundations (DS415, 1977) was first introduced. At that time the use of bored piles was considered unusual.

Bored piles were generally considered special piles, and no distinction was made between the different types of bored piles and installation techniques. Hence, the code-requirement for bored piles was based on load test on displacement piles, traditionel bored piles, piles with casing, piles without casing and some stabilised by bentonite. Furthermore, the time spent performing the bored piles were significantly longer then than today.

However, after more than 40 years of experience, bigger and better drill rigs, better testing methods, and a better understanding of the bored piles sensitivity towards execution, it might be time to reconsider the rationality and validity of the restrictions in the allowable bearing capacity of bored piles imposed still today by (DS/EN_1997-1_DK_NA, 2015). A clear definition of a bored pile should be stated, and the reduction factor should be determined for various ground conditions with today's execution methods.

For the case study on Nyborg Slot, the results clearly show that the actual shaft resistance demonstrated by the two static compression pile load tests is up to 274 % higher than indicated by the representative calculation carried out for a similar driven pile and reduced to 30 % based on the analytical method given by (DS/EN_1997-1_DK_NA, 2015).

5 ACKNOWLEDGEMENTS

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