Loading test experience with bored piles in limestone/weak rock
Expérience de tests de charge avec des pieux forés dans le calcaire/rock faible

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ABSTRACT: For foundations in limestone rock, bored piles (rock sockets) are typically an economical solution. The paper describes the experience gained from installation, prediction of resistance and loading test results obtained from case histories in Denmark and abroad. These case histories are supporting evidence of an empirical methodology using a cautious estimate of the unconfined compressive strength of the rock for derivation of the resistance. In Denmark, this approach is restricted by a clause in the Danish Annex to the Eurocode stipulating, "the capacity of a bored pile may only be taken as 30% of the corresponding driven pile". The main aim of the paper is to repudiate this demand.

RÉSUMÉ : Dans le cas de fondations dans de la roche calcaire, les pieux forés dans la roche sont généralement une solution économique. Ce document décrit l'expérience acquise lors de leur installation, la prévision de leur résistance et les résultats des essais de chargements obtenus au Danemark et à l'étranger. Ces informations acquises montrent la compatibilité d'une méthodologie empirique en utilisant une estimation prudente de la résistance uniaxiale à la compression de la roche, en milieu confiné, pour déterminer sa résistan ce. Au Danemark, cette approche est limitée par la clause de l'annexe nationale danoise aux Eurocodes qui stipule que "la résistance du pieu foré doit être prise à 30% du pieu enfoncé correspondant." L'objectif principal de ce document est de répudier cette demande.

KEYWORDS: Bored piles, Limestone, Loading tests, Code of Practice

1 INTRODUCTION

In the Danish Annex to Eurocode 7 (DS/EN-1997-1 DK NA:2015) the resistance of bored piles is restricted by Clause (6) to Annex L (informative) *"For bored in situ cast piles, the resistance may be considerable less than for corresponding driven piles. The maximum allowable shaft resistance is limited to 30% of the shaft resistance of the corresponding driven pile and the toe resistance to 1000 kN/m² unless recognised documentation for higher resistance is provided"* (COWI translation).

The restriction is not qualified in terms of the soil or rock strata considered. In Denmark this would typically be in limestone. However, the qualification "30% of the shaft resistance of the corresponding driven pile" implicitly suggests that the restriction is intended for piles in fine grained soils where the resistance is mainly derived from shaft resistance.

Unfortunately the qualification/restriction has been used indiscriminately for bored piles per say and hence also for piles in rock. The fact that the qualification/restriction is in an informative Annex seems to be lost in the application.

Basically the qualification in the Danish Annex is not operational, as it does not state if the "corresponding driven pile" is a displacement pile or a non-displacement pile (typically a steel tube); and for the latter if the pile is plugged or unplugged.

For bored piles in rock, "an equivalent driven pile" does not rationally exist. A driven pile in rock will rely on toe resistance and normally be driven to refusal. In contrast, a bored pile in rock will typically act as a rock socket deriving the major part of the resistance as shaft resistance. The rationale being that a bored pile is typically used where limited pile displacement is allowed/desired, as a rock socket develops the maximum shaft resistance at very limited (3-15 mm) vertical displacement. Provided the bored pile is constructed correctly, it may well provide a very substantial toe resistance (particularly in rock) but development of this resistance typically requires significantly larger displacement than the displacement required to mobilise the shaft resistance (typically 3-10% of the pile diameter).

It is unclear how the resistance of "a comparable driven pile" shall be calculated in a rock deposit. Driving the pile will crush the rock and hence a very low shaft and uncertain resistance will result. The same applies to the toe resistance where continued crushing will take place and for a displacement pile the resistance will rapidly increase and even for very low unconfined compressive strength the toe resistance would exceed 1000 kN/m² (also part of the restriction). If the rock was thought of in terms of undrained shear strength, $c_u = \sigma_c/2$, the geostatic design toe bearing would be 1000 kN/m² for $c_u = 1000/9 \times 1.3 \times 1.5 = 217$ kPa. This would correspond to a competent soil but with far lower resistance than a rock stratum.

To add insult to injury the background for the limitation on resistance of bored piles is not indicated (It appears that the restriction was added to the previous Danish Code of Practice as a result of major problems with Franki piles which are very far from being bored piles).

The paper aims to demonstrate the misguidance provided in the informative Annex L to the Danish National Annex to Eurocode DS/EN-1997-1 by case histories of bored piles in limestone/weak rock from Denmark and abroad serving as "recognised documentation".

2. DESIGN METHODOLOGY FOR ROCK SOCKETS

The design of the piles follows the international standard adopted for numerous projects with rock sockets carried out by COWI. The characteristic shaft, τ_{char}, and toe resistance, $q_{toe,char}$, are based on the measured unconfined compressive strength, σ_c, found by UCS testing. According to Fleming et al. (1992; 2009) the characteristic resistances for (soft) rock may be found as:

$$\frac{\tau_{char}}{p_a} = 1.3\sqrt{\frac{\sigma_c}{p_a}};\ 2\sqrt{\frac{\sigma_c}{2p_a}}; \qquad \frac{q_{toe,char}}{p_a} = 3\frac{\sigma_c}{p_a};\ 15\sqrt{\frac{\sigma_c}{p_a}} \quad (1)$$

Here, σ_c, represents the cautious mean value of the strength distribution of individual or total layers of the rock and p_a is atmospheric pressure. This methodology has been applied on numerous projects and have been found to be conservative for all the pile load tests interpreted by COWI.

3. CASE HISTORIES

3 .1 *BLOX (Bryghusgrunden), Copenhagen, Denmark*

A large number of bored piles in Copenhagen limestone were constructed for the project at Bryghusgrunden, BLOX, in Copenhagen (Figure 1). One pile loading test, pile B12, was carried out representative of the most heavily loaded type A piles (axial compressive design load of 19 MN) on the project. The nominal bore of the rock socket was Ø1060 mm but unfortunately, the temporary casing, outer diameter Ø1200 mm, was taken down coincident with the drilling process to the base of the test pile at level -22.5 m. This means that a "crushing zone" of 70 mm width is present throughout the limestone deposit. The pile was concreted to level+1 m with concurrent removal of the temporary casing.

Figure 1. (a) Project BLOX, Copenhagen harbour; (b) Lowering of reinforcement cage with O-cell into temporary casing.

The top of the limestone is at level -11.8 with an upper 1 m disturbed zone. The pile was tested using a bi-directional O-cell assembly 3 m above the toe at level -19.5 m (cf. Figure 1b) with a maximum sustained (bidirectional) load of 28.5 MN.

Based on strain gauge measurement on the lower 3 m pile (maximum vertical downward displacement of 15 mm) a maximum shear resistance of 2.1 MPa was recorded (still increasing to the maximum displacement). It is suspected that the relatively large displacement associated with development of the shear resistance is due to the presence of the crushed zone around the pile. The nominal diameter of the pile in the limestone corresponds roughly to the outer diameter, Ø1200 mm, of the temporary casing.

For the ~7.7 m pile part in limestone, above the O-cell, the shear resistance drops rapidly with the distance from the O-cell despite displacement of the order 7.8 mm at the O-cell and 5.8 mm at the pile top (the average shear resistance was 0.95 MPa). This is in contrast to the expected behaviour but is likely due to the crushed zone around the pile. The effect will be more pronounced for tension (upper pile part) than for compression (lower pile part). The contribution to the shaft resistance from the 2.9 m glacial sand, 3.8 m glacial clay till and 6.1 m mixed fill (above the limestone) was negligible.

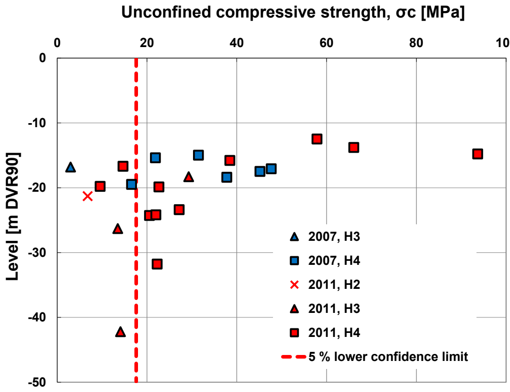

Figure 2. Distribution of unconfined compressive strength for 2007 and 2011 boring campaigns at Bryghusgrunden

For the project, a series of borings was carried out allowing estimation of the unconfined compressive strength in the limestone. 22 UCS tests were carried out for samples ranging from H2 to H4 induration (see Figure 3). The mean value was 30.1 MPa ± 21.6 MPa assuming normal distribution and 23.4 MPa ± 2.2 MPa for log normal distribution. The latter is more plausible and the corresponding lower 5% confidence limit is 17.6 MPa. There is no evidence of lower strength values at the top of the limestone deposit.

According to Eq. (1) this corresponds to a characteristic shear resistance of τ_{char} = 1.7 MPa. This is lower than the measured lower bound value of shear resistance $\tau_{test,min}$ = 2.1 MPa in the loading test and hence, a conservative estimate.

3.2 *Kissing Bridge, Copenhagen Harbour*

For the Kissing Bridge (Bro over inderhavn, Figure 3) one pile loading test was carried out on an Ø1080 mm bored pile in Copenhagen limestone. The piles were constructed using a casing driven into the top of the limestone bedrock, drilling of an Ø1080 mm rock socket with associated cleaning of the base insertion of an Ø1000 mm steel pile with external shear keys and Tremie concreting to the top of limestone.

Figure 3. Kissing Bridge, Copenhagen Harbour; (a) overview; (b)/(c) sliding part closed/open

In boreholes 1, 3, 4, 6 and 7, UCS and Brazil tests were carried out on limestone samples. The limestone surface varies between level -8.5 m and -9.8 m in the area and is close to the seabed in the channel (the ground investigations included other, nearby channel bridges).

Two samples on H2 limestone (levels -13.7 m and -14.8 m) provided σ_c = 5.3 and 5.1 MPa and the remaining three H4 samples (levels -13, -14.5 and -11.5) σ_c = 46, 80 and 55 MPa, respectively.

Notwithstanding the high degrees of induration the Client's consultant concluded that due to the inherent fissuring the limestone should be treated as a soil with a characteristic undrained shear strength of $c_{u,char}$ = 200 kPa or plane strain characteristic drained parameters φ'_{char} = 40° and c' = 0 kPa. For the limestone denoted competent below level -13 m the drained parameters could be raised to φ'_{char} = 40° and c' = 50 kPa or φ'_{char} = 45° and c' = 0 kPa.

The consequence of applying Mohr-Coulomb parameters, as indicated above, was a rock socket length of 13 m even without the 30% reduction indicated by the Danish Annex. Thus, bored piles would be excessively long and costly.

As an alternative COWI suggested the use of the empirical approach based on unconfined compressive strength, σ_c, as indicated by EQ. (1). Using conservative estimates of σ_c = 1 MPa for the top 2 m of limestone and σ_c = 5 MPa for the underlying limestone, an Ø1000 mm rock socket length of 5.5 m was required. Due to the extreme divergence in results it was decided to carry out a load test on one of the production piles, an Ø1080 mm rock socket using O-cell testing.

The pile, TP1, was installed from level -7 m through 3.5 m sand/sediment and from -11.0 m to -16.7 m in Copenhagen limestone. The base of the O-cell was at level -16.7 m resulting in a length in limestone of 2 m in weaker limestone and 3.7 m in the stronger limestone.

As the pile was a production pile the load test was only taken to a bi-directional maximum load of 150% of the working load, 4.4 MN, which was very far from failure. The upward displacement of the O-cell was only 1.6 mm, i.e. comparable with the elastic shortening of the pile.

The average mobilized shear resistance for the upper 5.7 m pile was thus only 259 kPa. A simple extrapolation (without considering elastic deformation) to 10 mm displacement gives a shear resistance of 0.97 MPa.

The design assumption, using the UCS values, corresponds to an average shear resistance of 0.74 MPa. Based on the results and noting a very considerable toe resistance the pile design was approved based on the empirical UCS approach.

3.3 *Wali Al Ahed flyover in Bahrain*

This case story (Steenfelt, 2004) concerns Ø800 mm traditional bored piles rock socketed in limestone (see Figure 4). Data concerning the strength of the limestone were scarce as only one UCS test was available together with some 51 point load tests.

Figure 4. Wali Al Ahed flyover; (a) bored piles; (b) completed structure

Based on “comparable experience” from the Copenhagen limestone database and the lowest value of $\sigma_c \sim 13\ I_{s(50)} \sim 21$ MPa a characteristic value of shaft resistance (using Eq. (1)) of 1.8 MPa was adopted together with a characteristic value of toe resistance of 18 MPa.

These values were very different from the values of the local/British designer using a characteristic shaft resistance of $\tau_{shaft,\ char}$ = 120 kPa and a toe resistance of $\sigma_{toe,\ char}$ = 10.8 MPa. However, these recommendations were based on data and experience from weathered Chalk in accordance with the CIRIA C574 (Lord et al., 2002).

After some discussion, it was decided to use the COWI recommendations with the proviso that the design was verified by pile loading tests. Originally, the plan was to test two trial piles to 13.5 MN and eight working piles to 5.2 MN.

One vertical 11 m long, Ø800 mm trial pile was tested in compression using the same size reaction piles. The socket length (in competent limestone) was 6.5 m with a cased length in fill and de-structured limestone of approximately 4 m.

The maximum load applied was 13.5 MN with a total pile head displacement of 5.2 mm. The 4.5 m “free part of the pile” within the casing to limestone contributed with 3.3 mm and hence the displacement in the limestone rock was only 1.9 mm. Thus, the contribution from the toe capacity was insignificant.

The conclusion from the pile loading test was that the ultimate shaft capacity was 31.4 MN corresponding to $\tau_{shaft,\ char}$ = 1.9 MPa. Due to the promising results, the second trial pile loading test was not carried out and the design rock socket length were reduced by a factor 1.5 (assuming the originally proposed shaft and toe resistances proposed by COWI).

The case history amply demonstrated the danger of confusing Chalk and Limestone when estimating the capacity of bored piles.

3.4 *Bridge foundations on bored piles in Marl/Limestone, Algeria*

The pier and pylon foundations of the Transhumel Viaduct in Constantine, Algeria (see Figure 5), are founded on Ø2000 mm bored piles in Marl and Limestone (Steenfelt and Schunk, 2013).

Figure 5. View of completed Transhumel Viaduct, Constantine, Algeria

Two working piles (one for each of the pylon foundations founded on 12 piles) were tested using O-cells close to the base to derive the shaft resistance. The piles above the O-cell are subjected to tension and will hence provide a lower bound value of the shaft resistance.

The unconfined compressive strength of the Marlstone, weathered limestone and non-weathered limestone was 13.5 MPa, 7 MPa and >18 MPa, respectively.

The two tests showed very high shear and toe resistances in excess of the design assumptions, using EQ. (1), see Figure 6. Extrapolated values of shaft and toe resistances at displacements of >5 mm and 200 mm, respectively, showed capacities at or above the structural capacity of the concrete piles.

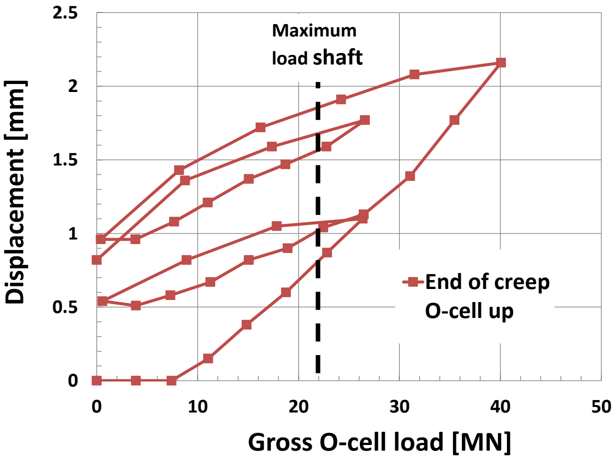

Figure 6. Measured shaft resistance from the upwards O-cell displacement, Pile P4/3

Thus, EQ.(1) appears to provide safe estimates for high strength deposits of marl and Limestone too.

3.5 *Ruwais Sulphur Expansion Phase III, Abu Dhabi*

The marine facilities (Quadrant beam, mooring and breasting dolphins) are founded on raking steel piles rock socketed into weak mud/siltstone (Steenfelt and Abild, 2011; see Figure 7).

Figure 7. Ruwais Sulphur Expansion project. Quadrant beam, mooring and breasting dolphins.

Socket diameters of 100 mm, 1200 mm and 1400 mm were applied with steel piles (with external shear keys) of Ø762 mm, Ø914 mm and Ø1067 mm, respectively.

The piles were designed based on the empirical approach of EQ. (1) based on the 5% fractile of 46 non-Gypsum tests, σ_c = 2.25 MPa. In the discrete Gypsum layers (19 tests) unconfined compressive strength up to 16 MPa was measured (Figure 8).

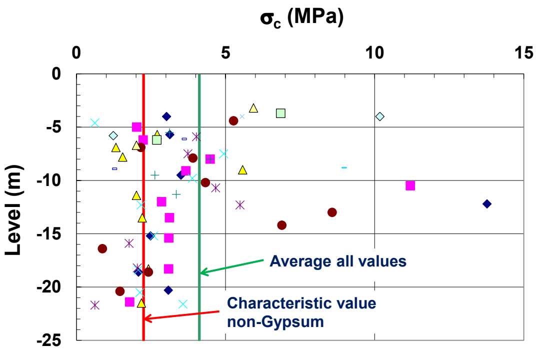

Figure 8. UCS values from all available offshore boreholes.

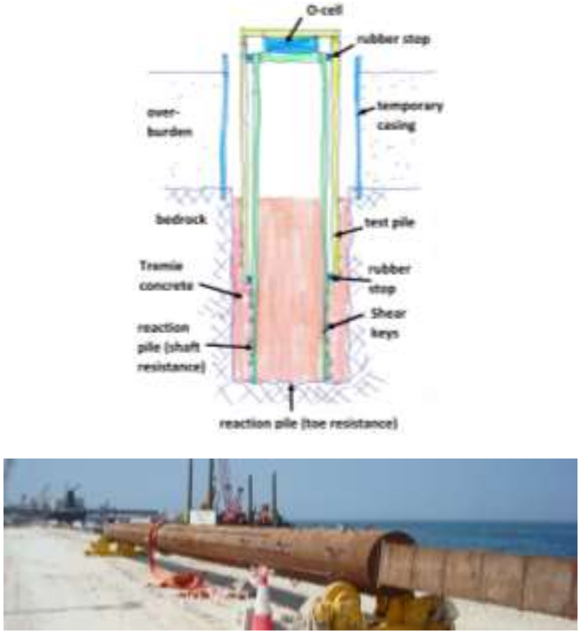

Figure 9. (a) Principle of O-cell testing; (b) test pile with reaction pile protruding (note shear keys on both piles).

The characteristic shaft resistance according to EQ. (1) was 0.615 MPa. In the initial design, this was applied together with the pile diameter, rather than the rock socket diameter.

Three piles (one of each diameter) were tested in tension using an elaborate O-cell arrangement (Figure 9) allowing true tension testing of the entire test pile length in rock.

The tests showed much higher resistances than the design. Although the loads were in excess of three times the working loads (SLS), all three test were far from failure. At 3.2 mm displacement a shear resistance of at least 0.832 MPa developed (even) based on the rock socket diameter rather than the steel pile diameter. The extrapolated failure loads (for shaft resistance in tension) were a factor 5.3, 4.8 and 6.4 higher than the working loads.

Thus, the empirical approach proved to be valid also for a weak mud/siltstone rock deposit.

Attention to cleaning at the toe of the pile or post grouting may allow for further, significant, increase of resistance by mobilising toe and shaft resistance concurrently.

4 CONCLUSION

Case histories, from Denmark and abroad, support the empirical methodology where the capacity of bored piles (rock sockets) is based on cautious estimates of the unconfined compressive strength of the rock.

There is no evidence to support the requirement in the Danish National Annex to Eurocode 7 (DS/EN1997-1 DK NA:2015, 2015) that the capacity of bored piles may only be taken as 30% of the "corresponding driven pile". As demonstrated by one of the case histories there is in fact evidence that the pile driving through limestone rock may severely reduce the shaft capacity, illustrating the high degree of irrationality of the requirement.

Removal of this clause from the Code and Annex is long overdue in order to remove the discrimination of bored piles.

5 ACKNOWLEDGEMENTS

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