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Validating Large-Diameter Bored Pile Design in Danian Limestone via Testing

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Abstract

This paper presents the case history of preliminary piles tested in Danian limestone as part of the design validation for large-diameter bored piles supporting the Kronprinsesse Marys Bro bridge crossing the Roskilde Fjord in Denmark. The project involved extensive geotechnical investigations and static load tests on two preliminary piles to assess shaft and base resistance in weak, highly variable limestone. The piles, with diameters up to 1.19 m and rock sockets of 5–8 m, were tested using bi-directional O-cell systems, providing direct measurement of load transfer and resistance mobilisation. A statistical analysis of more than 300 UCS tests revealed a significant strength variability, highlighting the importance of site-specific characterisation. The findings demonstrate that bored piles socketed in Danian limestone can achieve higher shaft and base resistances than currently permitted by the Danish National Annex to Eurocode 7. The proposed methodology enables the derivation of representative design parameters, advocating the use of percentile-based UCS values and empirical correlations tailored to local rock conditions. This case history provides valuable insights for practitioners and regulators, supporting the revision of Annex L of the Danish National Annex to Eurocode 7 to incorporate robust methods for estimating shaft and base resistance in soft rock.

Keywords

Danian limestone, Bored piles, Weak rock, Pile load testing, Eurocode 7.

1 Introduction

In Denmark, the resistance of bored, cast-in-place piles is constrained by provisions in the Danish National Annex to Eurocode 7 (DS/EN 1997-1 DK NA), which limit the shaft resistance that may be

adopted relative to comparable driven piles unless project-specific documentation is provided. Although originally introduced to address uncertainties associated with bored pile construction (Knudsen et al., 2019), these provisions often require justification when bored piles are installed in rock formations and may result in conservative foundation designs (Steenfelt, 2017). The increasing use of large-diameter bored piles in Danish projects therefore highlight the need for well-documented case histories demonstrating their performance.

The Kronprinsesse Marys Brobridge is a 1.4 km crossing of Roskilde Fjord, northwest of Copenhagen (Fig. 1a). The offshore piers are founded on large-diameter (1.5-2m) bored piles, approximately 25–50 m long, socketed into Danian limestone (Demeersman and Bauduin, 2019; Petrella and Wudzka, 2021). Owing to the variable strength and induration of the limestone, preliminary pile testing was undertaken during the design phase to validate the correlations adopted for estimating shaft and base resistance. Two preliminary piles were tested using bi-directional static load tests with Osterberg (O-cell) system, allowing direct assessment of load transfer and resistance mobilisation within the rock socket. This paper evaluates UCS-based prediction methods for shaft and base resistance of bored piles socketed in Danian limestone using results from preliminary O-cell tests and a statistically derived rock-strength characterisation. A percentile-based approach is proposed to derive representative design parameters for heterogeneous soft rock.

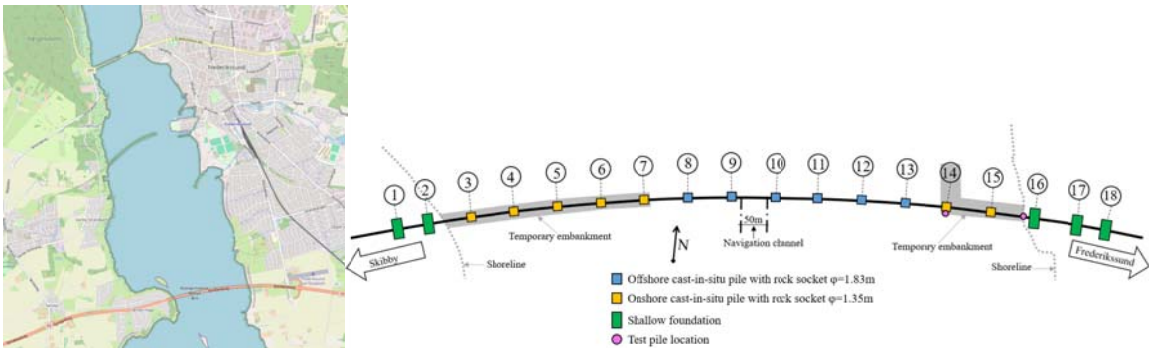


Fig. 1(a) Location of Kronprinsesse Marys Bro crossing Roskilde Fjord and (b) layout of bridge foundations and location of preliminary test piles TP1B and TP2 used for design validation.

2 Rock strength characterisation

The preliminary test piles were installed adjacent to two boreholes considered representative of the geological and geotechnical conditions across the project area. The piles are socketed in Danian limestone, whose strength is influenced by the degree of induration. Limestone cores, recovered from the boreholes with triple barrel core drilling,were described according to the engineering geological classification presented in DGF Bulletin 1 (Larsen et al., 1995), which distinguishes induration categories ranging from unlithified chalk to strongly indurated limestone.

Unconfined compressive strength (UCS) denoted as σ_c,was determined from laboratory testing of limestonecore samples. In total, more than 300 UCS tests were undertaken on specimens selected across all induration categories. The results exhibit significant variability reflecting the heterogeneous nature of the limestone and the occurrence of stronger beds within otherwise weaker material. The distribution of tests across the different induration classes and the corresponding strength ranges are summarised in Table 1. It is noted that no UCS tests were performed on H1 material due to its unlithified nature.

Table 1. Distribution of UCS tests from the D&B Contractor'sground investigation, average adopted by the Designer for each induration and proposed 50th percentile used in this study.

Induration Category	ISRM Rock Category	Description	No. of UCS tests	Range of results, σ_c (MPa)	Average σ_c (MPa)	50th Percentile σ_c (MPa)
H1	R0	Unlithified	-	-	1.3	-
H2	R1	Slightly indurated	59	0.80 – 21.96	6.4	5.7
H3	R2	Indurated	98	0.47 – 44.10	19.5	15.3
H4	R3, R4	Strongly indurated	84	2.90 – 87.00	27.7	29.6
H5	R5, R6	Very strongly indurated	22	1.60 –> 100.00	27.7	35.0

Two approaches for selecting representative UCS values were considered. In the deterministic approach adopted in the original design, representative UCS values were taken as the average strength measured for each induration category. For each pile, these values were assigned to the corresponding induration layers identified in the borehole log, forming a representative UCS profile along the rock socket.

Given the large scatter in the UCS results, a statistical evaluation of the dataset was also performed. The UCS distribution was analysed using both log-normal and kernel density estimators considering the full dataset of 319 test results, as shown in Fig. 2. Unlithified material (H1) and very strongly indurated material (H5) were treated using reference values proposed by Kryger Hansen and Foged (2002), as these materials are rarely tested due to sampling difficulties or testing limitations. The kernel distribution was found to best represent the UCS dataset, and representative UCS values were derived from the corresponding percentiles, with the 50th percentile values adopted for each in duration class (Table 1). These values are broadly comparable to deterministic averages but are less influenced by extreme results and thus provide a more robust representation of rock strength.

For both approaches, the representative UCS profile along the pile was then used for the resistance calculations. A weighted average UCS was calculated over the full length of the rock socket for shaft resistance calculations and over a distance equal to one pile diameter below the toe pile toe for base resistance calculations. The resulting representative values are illustrated in Fig. 3.

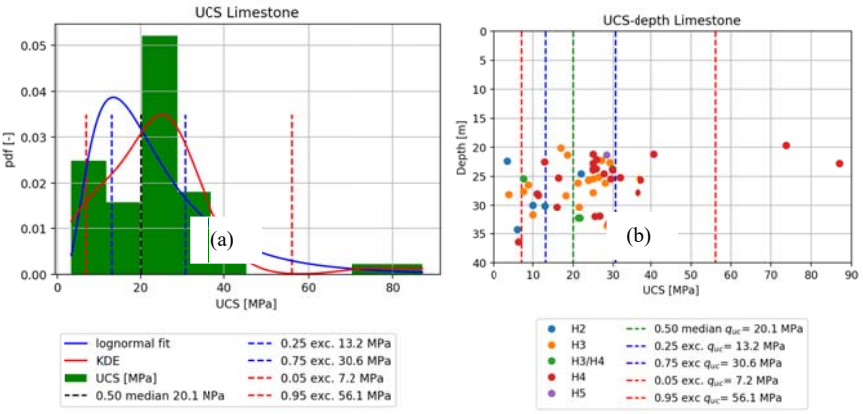


Fig. 2 Kernel/Lognormal density distributions of unconfined compressive strength (UCS, σ_c) Statistical description of the laboratory and borehole log data (a) histogram and (b) scatter data.

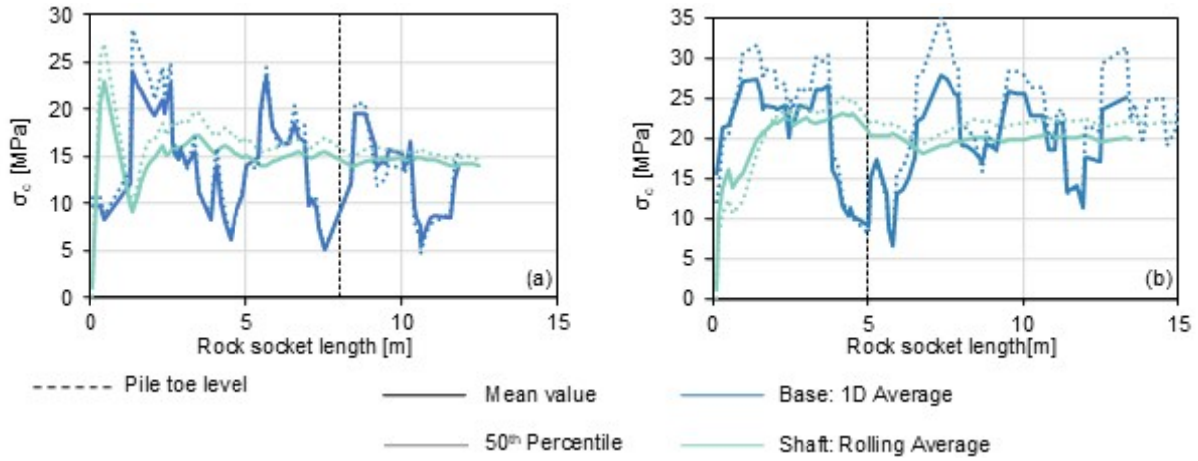


Fig. 3 Average of UCS over entire length (for shaft) and over 1D (for base) (a) TP1B and (b) TP2.

3 Prediction of rock socket resistance

The shaft and base resistance of the preliminary piles were estimated using empirical correlations commonly applied for bored piles socketed in rock. The unit shaft resistance was calculated using the formulation proposed by Zhang (2004), which relates the mobilised side shear resistance to the UCS of the intact rock, as described in Eq. 1.

$$\frac{R_{s;cal}}{A_{si}} = \eta_c \cdot \alpha \cdot {\sigma_c}^{\beta} \quad (1)$$

Where $R_{s;cal}$ Calculated shaft resistance
A_{si} Surfacearea of the pile within the considered layer
η_c Empirical factor, $\eta_c = 0.9$
α Empirical constant,$\alpha = 0.2$
σ_c Unconfined compressive strength of the rock
β Constant in Zhang equation, $\beta = 0.5$

The empirical parameters adopted correspond to values recommended for sockets in weak rock. The base resistance was estimated using the approach proposed by Serrano and Olalla (1994; 2002; 2014), summarized in Eq. 2, which relates the toe resistance to the unconfined compressive strength and rock mass parameters derived from the Hoek–Brown failure criterion.

$$\frac{R_{b;cal}}{A_b} = \frac{m \cdot q_u}{8} \cdot (N_\beta - \zeta) \cdot s \quad (2)$$

Where $R_{b;cal}$ Calculated base resistance
A_b Base surfacearea of the pile
m Parameters of the Hoek and Brown (1980) criterion, $m = 1.3$
N_β Load factor, $N_\beta = 6.7$
ζ Rock mass toughness coefficient, $\zeta = 0.006$
s shape factor, s = 1.5

4 Preliminary pile tests and results

The preliminary piles had diameters of approximately 1.19 m and rock socket lengths of 8.0 m for TP1B and 5.2 m for TP2. While smaller than the contract piles, the use of preliminary piles with different dimensions is permitted under EC7. The geometry of the test piles is summarised in Table 2. The piles were constructed using a rotary drilling rig with temporary casing installed through the soil deposits and soft limestone to the top of the competent limestone (Fig. 4a). The pile bore was filled with water during drilling. The pile base was cleaned using a cleaning bucket and sediments were removed by pumping prior to concreting. The reinforcement cage, incorporating the O-cell assembly and instrumentation, was installed immediately before casting using tremie concrete. Both piles were tested using bi-directional static load tests with O-cell systems installed approximately 1 m above the pile toe (Fig. 4b). The O-cell applies load in two directions, mobilising shaft resistance above the cell and base resistance together with shaft resistance below the cell. The applied loads and corresponding displacements were monitored using displacement transducers and vibrating wire strain gauges installed along the pile (Fig. 4c).

Maximum bi-directional loads of 44.75 MN and 32.7 MN were applied during the tests on TP1B and TP2, respectively. The O-Cell load with respect to displacement curves are shown in Fig. 5. These values are comparable to the predicted resistances of 43.8 MN and 26.8 MN obtained from the design calculations. The load test results were interpreted independently by the Designer and by the testing contractor (Fugro 2017a, 2017b). The interpretations differ slightly due to assumptions regarding pile geometry and stiffness used in the evaluation of the strain gauge measurements. The resulting shaft and base resistances derived from the load tests are compared with the predicted values in the following section.

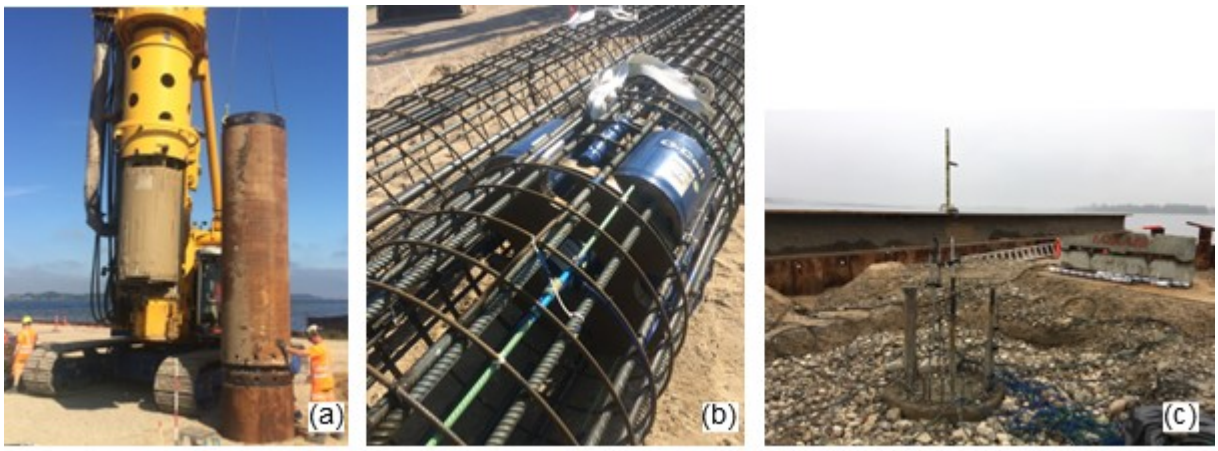


Fig. 4 (a) Drilling bucket used for construction, (b)reinforcement cage with installed O-cells and monitoring instrumentation, and (c) monitoring instrumentation at the top of the pile.

Table 2. Preliminary test piles geometry based on as-built data.

Preliminary pile ID	Platform level [mDVR90]	Pile top level [mDVR90]	Top of limestone [mDVR90]	Pile tip level [mDVR90]	Rock socket length [m]	O-cell base level [mDVR90]	Pile diam. in soil and rock [m]
TP1B	+1.0	+0.5	-19.7	-27.7	8.0	-26.7	1.19
TP2	+1.0	+0.5	-17.7	-22.9	5.2	-21.9	1.19

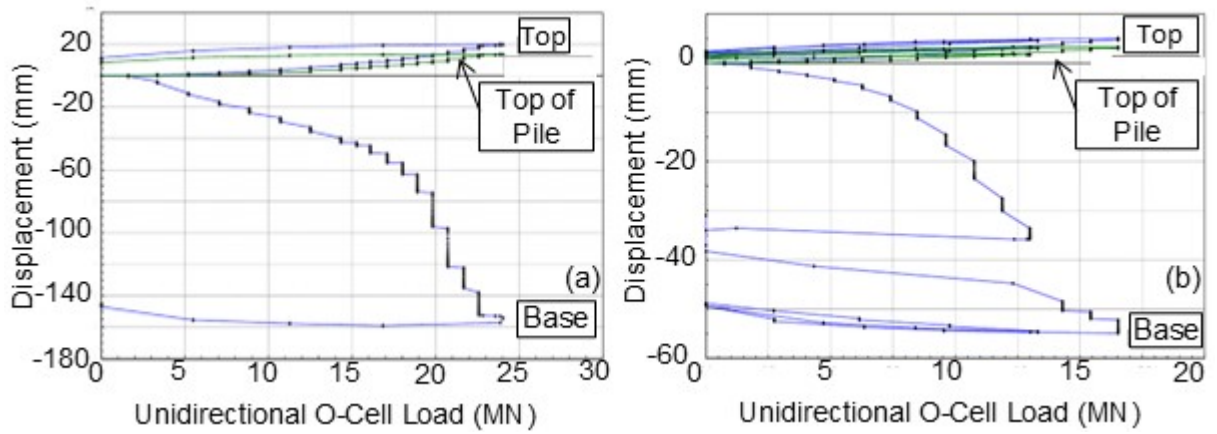


Fig. 5 O-cell load vs. displacement curves at the top and base level of the O-cell and recorded displacement at the top of pile for (a) TP1B and (b) TP2.

5 Comparison between predicted and measured resistances

The Designer evaluated the shaft and base resistances using the maximum upward and downward forces measured during the O-cell tests, assuming equal shaft resistance above and below the O-cell within the limestone. The upward load measured during the test was reduced by the shaft resistance mobilised in the overlying soil layers (5.0 MN for TP1B and 3.6 MN for TP2), which are not considered in the present study. The pile load test results were also interpreted independently by the testing contractor. Differences between the interpretations arise primarily from the assumed pile geometry and stiffness in the analysis. The Designer assumed a constant pile diameter of 1.19 m, whereas the testing contractor considered different diameters within the rock socket and the temporarily cased section in soil and explicitly accounted for pile stiffness in the interpretation.

Error! Reference source not found. compares the predicted unit shaft and base resistances with those derived from the load test interpretations. The predicted base resistance shows reasonable agreement with the values derived from the tests, whereas the measured shaft resistance is significantly higher than that predicted using the empirical correlation with the deterministic representative UCS values.

When representative UCS values derived from the statistical analysis are adopted, the predicted shaft resistance increases slightly. Furthermore, adopting empirical parameters within the range reported by Zhang (2004) for weak rock sockets results in predicted unit shaft resistances of approximately 1.4–1.5 MPa, which are consistent with the values interpreted from the load tests.The comparison indicates that the empirical prediction methods provide a reasonable estimate of the pile capacity when representative rock strength parameters and appropriate empirical factors are used.

For reference, the Danish National Annex L limits the design unit toe resistance of bored piles to 1 MPa, corresponding to a maximum design base resistance of approximately 1.1 MN (or 1.7 MN at characteristic level) for the tested pile diameters, which is lower than both the predicted and measured values.

Table 3. Comparison of UCS-based predictions with measured shaft and base resistances for test piles TP1B and TP2.

	TP1B				TP2			
Resistance	**Mean UCS**	**P50 UCS**	**Contractor**	**Designer**	**Mean UCS**	**P50 UCS**	**Contractor**	**Designer**
Shaft (MPa)	0.70	0.82	1.48	0.66	0.79	0.85	1.45	0.80
Base (MPa)	16.1	13.4	21.9	18.1	7.2	7.8	8.5	12.2

6 Conclusions

This study demonstrates the importance of statistically informed rock strength characterisation when designing bored piles socketed in heterogeneous weak rock. Analysis of a large UCS dataset showed significant variability in Danian limestone, for which percentile-based representative values provide a more robust basis for design than simple deterministic averages. Bi-directional O-cell testing confirmed that both shaft and base resistances mobilised in the limestone exceed conservative design assumptions, and that empirical prediction methods can provide reasonable estimates when appropriately parameterised using representative UCS values. The results highlight that the limitations of the current version of Annex L of the Danish National Annex to Eurocode 7 do not reflect the observed pile behaviour in Danian limestone and that documented testing combined with transparent parameter selection offers a defensible pathway for refining design resistance assumptions for bored piles in soft rock. Future works should include additional testing across a wider range of rock type.

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