Proceedings of the XVIII ECSMGE 2024
GEOTECHNICAL ENGINEERING CHALLENGES
TO MEET CURRENT AND EMERGING NEEDS OF SOCIETY

ISBN 978-1-032-54816-6
DOI 10.1201/9781003431749-131

ECSMGE 24
XVIII EUROPEAN CONFERENCE ON SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

Bi-directional testing of bored piles in chalk

Essais bidirectionnels de pieux forés dans la craie

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ABSTRACT: Large diameter bored piles (Ø1800 mm) will be used as foundation for a new railway bridge crossing Guldborgsund between the Danish islands of Falster and Lolland. The ground conditions at site are dominated by 10-15 m of glacial and late- postglacial deposits above relatively soft Cretaceous chalk. The experience with bored piles in chalk in Denmark is limited and a conservative design approach would have to be used due to lack of testing. Therefore, two full-scale, instrumented piles (26m and 31m long) were constructed and tested using bi-directional loading by means of O-cell technology. The piles were instrumented with tell-tales and strain gauges to allow for determination of the mobilized unit shaft resistance as function of depth and soil type. The piles were loaded to 22.5 MN and 30 MN, respectively, which significantly exceeded the capacity predicted by normal design methods. Based on calibration of an α-method for shaft resistance versus local CPT profiles, it was possible to establish a site-specific design model allowing for optimization of the production piles.

RÉSUMÉ: Les pieux forés de grand diamètre (Ø1800 mm) seront utilisés comme fondation pour un nouveau pont ferroviaire traversant le Guldborgsund entre les îles danoises de Falster et Lolland. Les conditions du sol sur le site sont dominées par 10 à 15 mètres de dépôts glaciaires et tardifs/post-glaciaires au-dessus de la craie crétacée relativement tendre. L'expérience avec les pieux forés dans la craie au Danemark est limitée, et une approche de conception conservatrice devra être utilisée en l'absence d'essais. Par conséquent, deux pieux à pleine échelle instrumentés (26 m et 31 m de long) ont été construits et testés en utilisant une charge bidirectionnelle grâce à la technologie O-cell. Les pieux étaient équipés de “tell-tales” et de jauges de contrainte pour permettre la détermination de la résistance à la traction unitaire mobilisée en fonction de la profondeur et du type de sol. Les pieux ont été chargés à 22,5 MN et 30 MN respectivement, dépassant significativement la capacité prédite par les méthodes de conception normales. En se basant sur l'étalonnage d'une méthode α pour la résistance à la traction du fût par rapport aux profils locaux CPT, il a donc été possible d'établir un modèle de conception spécifique au site permettant l'optimisation des pieux de production.

Keywords: Bored piles; Pile loading test; chalk, CPT based methods.

1 INTRODUCTION

In connection with the construction of the Fehmarnbelt tunnel connecting Denmark and Germany from 2029, the railway between Copenhagen and the tunnel are currently being upgraded to meet European interoperability standards. For parts of railway with only a single track, a second track is to be constructed. This also requires construction of a new bridge to carry the extra track across the strait of Guldborgsund between the islands of Falster and Lolland.

The bridge will be constructed next to the existing Kong Frederik IX's bridge (E3006 project) carrying a single railway track. The new bridge will be built as a bascule bridge similar to the existing bridge. The bridge will be founded on Ø1800 mm bored piles, two at each pier location. The bored piles will be installed into Cretaceous chalk formation and equipped with flat-jacks to ensure immediate activation of both toe and shaft resistance.

Danish experience with bored piles in un-indurated chalk is limited. The Danish Annex to Eurocode 7, Dansk Standard (2021), sets strict limits on the shaft and toe resistance without testing. This leads to significantly lower design resistance than found e.g. using the recommendations from CIRIA (1999). To optimize the pile design, it was therefore chosen to test two full scale piles by means of bi-directional loading tests to establish a site-specific design model that could be used to optimize the production piles.

2 TEST SETUP

2.1 Bi-directional loading test

The loading tests on bored piles at the E3006 project are carried out as bi-directional loading test using Osterberg cells (O-cells). The O-cell is a hydraulic jack that is installed inside the reinforcement case of the bored pile. During a loading test, the pile is loaded by expanding the O-cell inside the pile. The O-cell is cast into the pile and the pile itself acts as a reaction system, see Figure 1.

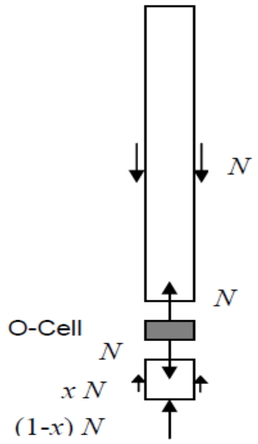

Figure 1. Principle of bi-directional test.

Using strain gauges and tell-tales at different levels of the pile, it is possible to derive the shaft resistance above the O-cell and the end-bearing and shaft resistance below the O-cell from the bi-directional loading test. Ideally, the O-cell should be placed at the location where the resistance above the O-cell is equal to the resistance below.

For this project two test piles were constructed; Test Pile 1 (TP1) and Test Pile 2 (TP2) with a length of 31 m and 26 m, respectively. The O-cells were placed 4 m above the toe of TP1 and 1 m above the toe of TP2. The position of the O-cell means that the piles are in principle loaded from the bottom and the chalk layers will therefore be activated before the glacial deposits. In this project focus was on evaluating the strength of the chalk.

2.2 Flat-jack preload system

The flat-jack system is installed at the bottom of the reinforcement case, as shown on Figure 2. The flat-jack system is used to preload the pile by introducing base grouting over a controlled area at the toe level of the pile after casting to establish full contact and prestressing at the pile toe.

Figure 2. Flat-jack system used for the E3006 project.

The installation of the flat-jack system allows for a degree of mobilization of the end-bearing resistance without any notable deformation and for prestressing of the shaft resistance. If the flat-jack is loaded to or above the SLS load, the settlement of the pile at SLS loads will be limited.

The activation of the flat-jack is in principle a primitive bi-directional loading test on a working pile giving a check of the pile installation and adding robustness to the design.

The flat-jacks are prestressed to 3 MN and 2 MN for TP1 and TP2, respectively. The diameter of the flat-jacks are 1650 mm with a maximum inflation height of 70 mm.

2.3 Site conditions

The soil strata at the test site (on land close to the bridge site) is evaluated based on one borehole and one CPT. Fill is found to 5 m below ground level, followed by glacial deposits down to 11 m below ground level. Below the glacial deposit, a glacially disturbed chalk layer (K2) is encountered to 21 m below ground level, after which competent chalk (K3) is found to large depth.

For both test piles, the piles were instrumented with six levels of strain gauges (SG) and 4 level of tell-tales. The instrumentation of TP1 is shown schematically in Figure 3.

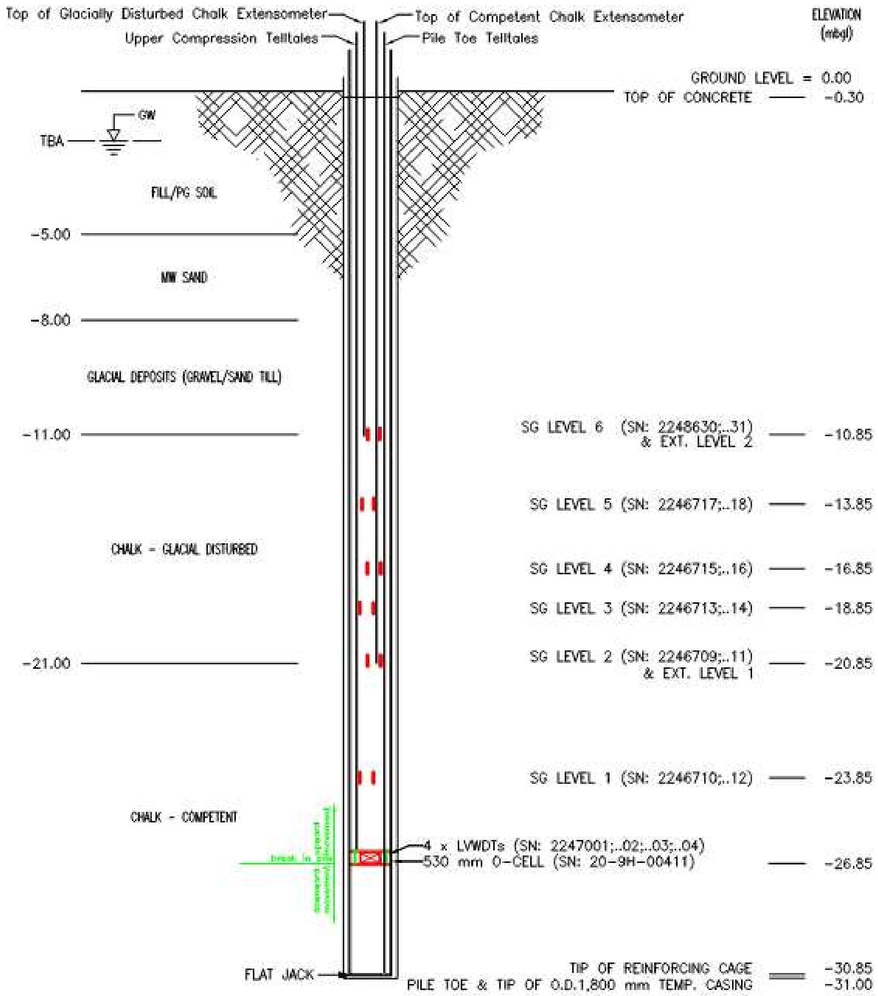

Figure 3. Instrumentation of TP1 together with approximate soil profile

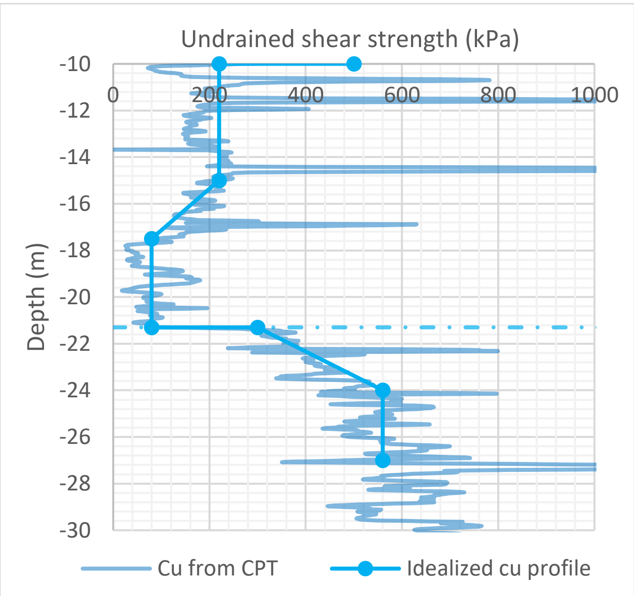

Figure 4. Undrained shear strength profile based on CPT for the test site (in the chalk layers).

From the CPT investigation a c_u – profile for the test site can be evaluated based on the corrected cone resistance (q_t) and the vertical stress (σ_{v0}). The c_u – profile is used for developing the design model for bored piles in chalk. The c_u parameter can be evaluated using the following method:

$$c_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{1}$$

A cone factor of $N_{kt} = 20$ has been derived for the chalk (both K2 and K3) based on laboratory tests. The derived c_u – profile is shown in Figure 4.

2.4 Loading schedule

The loading schedule consisted of two load cycles:

In the first load cycle, the O-cell was loaded to 6 MN in four (4) steps of 1.5 MN with a holding time in each step of 50 minutes or until the rate of creep (measured as expansion of the O-cell) was less than 2 mm/log cycle of time. The pile was then unloaded in three (3) steps to 4.5 MN, 3 MN and 0 MN.

The second load cycle aimed to load the pile to failure in maximum 12 steps (of 1.5 MN) with a holding time in each step of 60 minutes or until the rate of creep (measured as expansion of the O-cell) was less than 2 mm/log cycle of time. Finally, unloading was done in six (6) steps of 3MN.

In the tests, failure was assumed to be reached either if the measured pile head movement exceeds 20 mm (maximum shaft resistance reached) or if the rate of O-cell expansion exceeds 10 mm/lct after 50 min sustained loading.

3 TEST RESULTS

3.1 Load-displacement behaviour

The pile head and toe displacement for both test piles are shown in Figure 5.

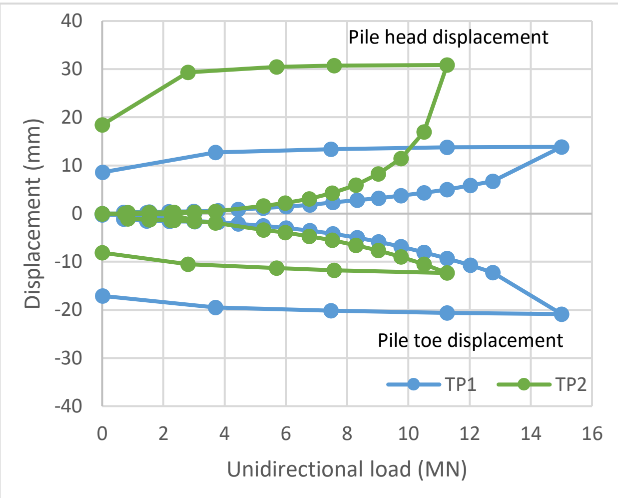

Figure 5. Pile head and toe displacement for all loading – unloading steps for both TP1 and TP2.

Figure 5 shows that the measured pile head displacement exceeded 20 mm for unidirectional load 11.3 MN for TP2, while the measured pile head movement is below 20 mm for unidirectional load of 15 MN for TP1. This means that TP2 is in failure. This is also confirmed by the creep rates of the O-cell expansion for the final loading step for both test piles, see Figure 6. For TP1, the creep rate at a unidirectional load of 15 MN was 2.8 mm/lct, which is below the failure criterion. For TP2, the creep rate at a unidirectional load of 11.3 MN was 15.2 mm/lct, which is above the defined failure criterion.

Note that even though the failure criterion is met for TP2, the pile is not in ultimate failure. While the shaft resistance is fully mobilized, the working curve for the tip resistance does not show any indication of imminent end bearing failure.

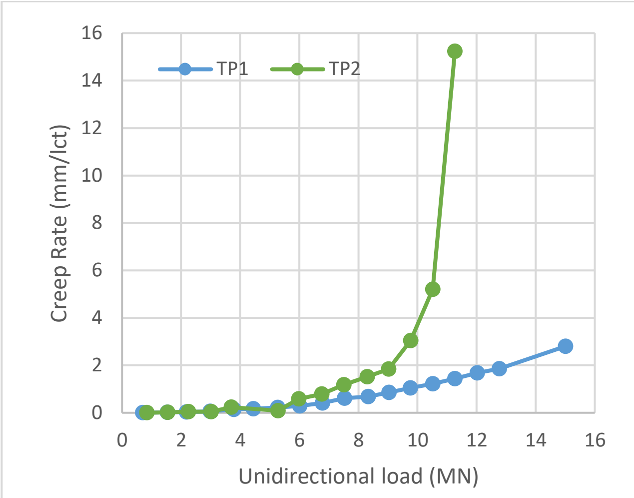

Figure 6. Development of creep rates for increasing unidirectional load

3.2 Flat-jacks

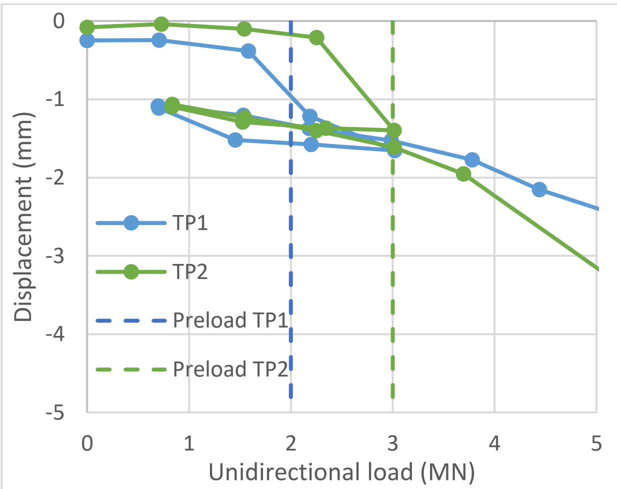

Figure 7. Pile toe displacement for prestress range of the flat-jack.

Flat-jacks are used to ensure that there is full contact between the toe of the bored pile and the chalk below. In the tests, the flat-jacks for TP1 and TP2 were preloaded to 2 MN and 3 MN, respectively. The preload not only secures a stiff initial response, but also preconsolidates the chalk – indicated by the shift of the pile toe displacement when the preload is exceeded as seen in Figure 7. Preloading flat-jacks to or above the maximum service load on the bored piles may therefore in practice eliminate future SLS pile movements.

4 INTERPRETATION

For both piles, six levels of strain gauges have been installed above the O-cell. From the strain gauges, the mobilized shaft resistance of the part of the pile above the O-cell can be assessed.

The assessment of the mobilized shaft resistance and end-bearing (toe) resistance below the O-cell carries some uncertainty. The toe resistance can only be deduced by assuming a constant shaft resistance below the O-cell (as no strain gauge measurement are available). Here, the shaft resistance is assumed constant below the O-cell level and subtracted from the downward O-cell load to provide the toe resistance.

The load transfer along the piles from the O-cell to the top of chalk are shown in Figure 8. The load transfer is found based on interpretation of the strain gauge measurements.

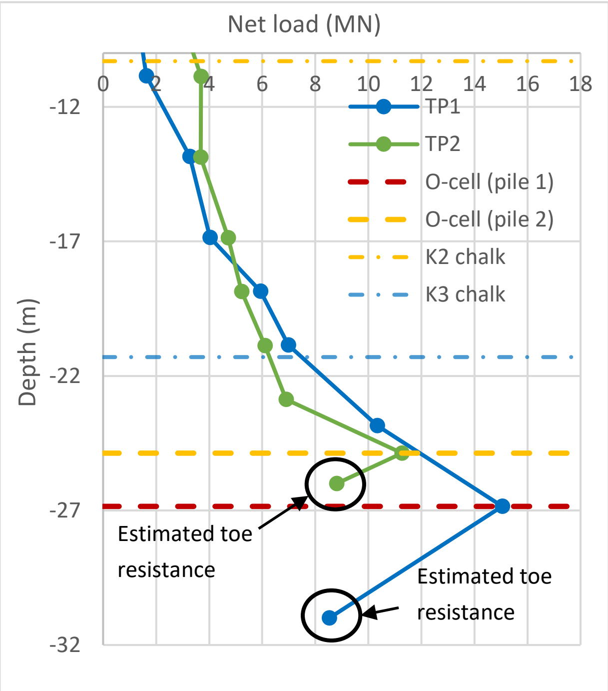

Figure 8. Load transfer head to toe based on strain gauge measurements and O-cell load for the last loading step.

The unit skin friction can then be derived along the pile from the load transfer results, see Figure 9. From the analysis of the strain gauge on TP2, it was found that the results of strain gauge level 6 were unreliable and therefore disregarded in the analysis of shaft resistance.

4.1 Shaft resistance

The shaft resistance of the piles was evaluated using the α-method. The α-values in the chalk were evaluated based on the load transfer along the pile shown on Figure 8. The α-values for both test piles are derived based on the average shaft resistance in K2 and K3 versus the average c_u value for each layer. The average c_u values are found from the idealized c_u-profile shown in Figure 4.

The derived α-values for TP1 and TP1 are given in Table 1 and Table 2, respectively.

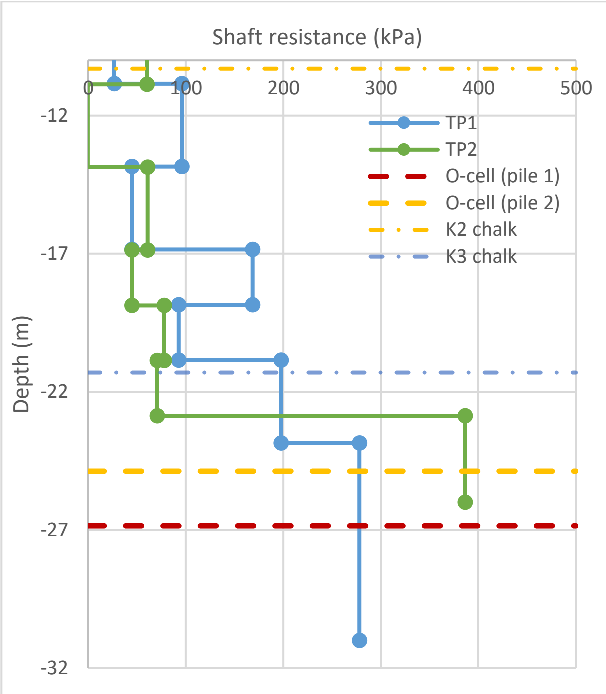

Figure 9. Unit shaft resistance along the shaft.

Table 1. α-values for test pile 1.

Soil unit	Average shaft resistance (kPa)	Average c_u (kPa)	a-value (-)
K2	99.01	151.36	0.65
K3	241.11	508.47	0.47

Table 2. α-values for test pile 2.

Soil unit	Average shaft resistance (kPa)	Average c_u (kPa)	a-value (-)
K2*	61.59	124.85	0.49
K3	247.64	461.68	0.54

*Strain gauge level 6 gives unreliable results and not included in the analysis

Figure 10 shows the load distribution along both TP1 and TP2 together with the estimated load distribution based on calculated shaft resistance using the α-values for K2 and K3 chalk given in Table 1 and Table 2. In Figure 10, an estimated load distribution using a constant α-value of 0.55 for both K2 and K3 chalk is also shown. The calculated load distributions are found using the load at O-cell level, distributing the load upwards along the pile.

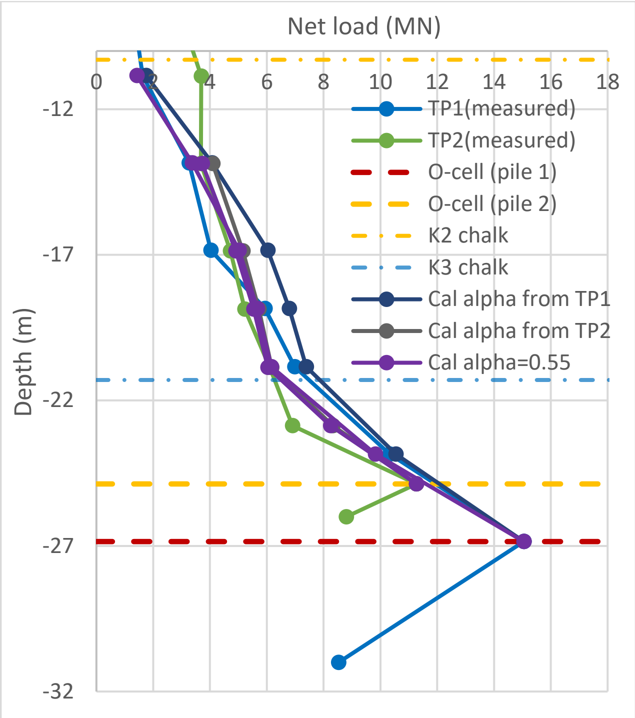

Figure 10. Model prediction against measured net load distribution from pile loading test on both test piles.

The back-calculated load distribution using an α-value of 0.55 for both K2 and K3 chalk gives a conservative solution compared to the measured load distribution. An α-value of 0.55 is adopted for the design of the bored piles in chalk.

4.2 Toe resistance

The toe resistance of 8.5 – 8.8 MN corresponds to a mobilized contact pressure of approximately 3340 – 3458 kPa. Based on the CPT profile, it is assessed that $c_u \approx 560$ kPa around the toe of the piles. Back-calculating from the estimated toe resistance, the end-bearing factor (N_{cu}) is found to vary between 6.7 and 6.9 for the two test piles. This end-bearing factor is below the commonly used value of 9 for fine-grained materials, indicating – as also observed from the load-displacement curves for the pile toe – that the toe resistance is not fully mobilized. In the design model, it is chosen to use a maximum bearing pressure of $q_{bk} = N_{cu}c_{uk} = 6\,c_{uk} \leq 2670$ kPa (model factor $\xi = 1.25$), where the cap is cautiously chosen as the maximum applied bearing pressure in the tests.

5 CONCLUSIONS

The load test performed on two bored piles at the E3006 project showed that the overall shaft resistance in the chalk is considerably higher than estimated based on common Danish Practice (as low as 25 kPa).

For the design of bored piles in chalk on the E3006 project an α-value of 0.55 is found to provide a reasonable fit for both K2 and K3 soil compared to the test piles.

Based on the two available pile loading test, the design methodology for the E3006 project was to take the full shaft resistance and a characteristic toe resistance of 2670 kPa into account, in contrast to 30% shaft resistance of an equivalent driven pile and maximum toe resistance of 1000 kPa specified in the Danish Annex to Eurocode 7, Dansk Standard (2021).

ACKNOWLEDGEMENTS

The authors are grateful to MT Højgaard for performing the bi-directional loading test and for granting permission to use the data from the pile loading tests.

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