Full-scale static compression pile load test in sand with post installation pre-stressing to engage toe resistance

K M Iversen¹ and B S Roesen¹
COWI A/S, Jens Chr. Skous Vej 9, 8000 Aarhus C, Denmark

E-mail: KMIV@cowi.com

Abstract. A full-scale static compression pile load test was conducted in connection with construction of a new road bridge in Silkeborg, Denmark. The new road bridge crosses a river between two existing bridges, one for pedestrians and one for an existing railway. An existing pedestrian bridge was moved five meters to the north to give space for the new road bridge. The western part of the existing bridge was therefore to be founded partly on the existing footing of the pedestrian bridge and partly on a new pile-based footing. Bored cast-in-place piles were chosen to minimize vibrations during construction. The pile load test was necessary to document the bearing capacity of the bored piles because the Danish National Annex to the Eurocode 7, part 1 (2015), Annex L (DS/EN 1997-1 DK:NA) obligates the designer only to regard 30% of the shaft resistance determined by geostatic calculation for the corresponding driven pile. Furthermore, it is a common design practise to disregard the toe resistance, as mobilization only occurs after deformations larger than normally acceptable for the superstructure. To overcome the latter, the test pile was constructed with a flat-jack device to allow post installation pre-stressing by grouting underneath the pile toe. Hence, it was possible to include toe resistance with only minimal displacements of the pile toe, as the grouting was conducted successfully, and post installation pre-stressing was obtained. The results of the pile load test show a bearing capacity up to 50% larger than estimated by geostatic calculation, even when including toe resistance and disregarding the 70% shaft reduction in the geostatic calculation for the corresponding driven pile. An estimate of the displacement of the new pile-based footing was made based on load-displacement curves for the test pile, and the subsequent production piles were installed using the same principles due to the flat-jack device’s effective post installation pre-stressing.

1. Introduction
Due to the increase of traffic in the city centre of Silkeborg, the city council had decided to construct a new road into the city centre. The purpose of the new road was to ease the traffic on the current roads as well as guide the traffic faster into and away from the city centre.

The road was constructed through the forest Nordskov in the alignment of the old railway track from Silkeborg to Langaa. The road had to cross Remstrup River where the old ferry boats sail, e.g. Hjejlen, which is the oldest coal-fired paddle steamer in the world still in operation and which places some practical bindings on the execution of the project. The road crosses the river on a new bridge between two old railway bridges which were constructed in the beginning of the 19th century. The northern bridge now serves as a pedestrian bridge, while the southern bridge still serves as a railway bridge. However, the space between the bridges did not allow for the new road bridge, and therefore the
pedestrian bridge was moved approximately 5 m to the north in the western end of the bridge, partly on the existing footing and partly on a new footing. The location of Silkeborg in Denmark is seen in Figure 1 and the city centre with the locations of Remstrup Å, Nordskov and the two existing bridges is shown in Figure 2.

The old bridges are founded on footings, and it is expected that they have only experienced small settlements during their lifetime. The new footing was designed as a pile foundation, as this gives a relatively settlement-free foundation. The existing bridge construction had to be preserved, and therefore large vibrations were inexpedient. Hence, bored cast-in-place piles were the obvious solution for the production piles for the new pile foundation.

2. Geotechnical conditions

Prior to the design of the pile foundation, a geotechnical investigation was carried out covering the entire road project. The investigation counted 40 boreholes and 18 CPT profiles. However, only a few of these investigation points were relevant for the production piles of the existing pedestrian bridge.

2.1. Soil conditions

In general, the geotechnical conditions at the location of the bridges are quite uniform. The upper layers consist of fill-deposits, under which a late glacial sand is deposited. Miocene deposits are found under the late glacial sand deposits. The Miocene deposits consist of both clay, sand and silt. The Miocene sand deposits was found to be very dense. The railway and the pedestrian path runs on an embankment with terrain level around +27.6 m. The terrain level below the embankment is around +21.6 m.

One borehole and corresponding CPT, namely B/CPT022, was carried out approximately 10-15 m from the pile foundation on top of the existing rail way embankment. This borehole was the basis for the preliminary design of the pile foundation. Further, one geotechnical borehole and corresponding CPT, namely B/CPT020, was carried out 3 m from the location of the static compression pile load test. This borehole was carried out beneath the existing rail way embankment and was the basis for the design of the static compression pile load test. A plan of the borehole locations together with the location of the test pile and the production piles for the existing pedestrian bridge is shown in Figure 3.
Figure 3. View of the two conducted drillings (blue crosses) together with the placement of the test pile (blue circle) and the production piles for the existing pedestrian bridge (red ellipse).

The two boreholes showed similar soil conditions. The soil conditions are shown in Figure 4.

Figure 4. Soil conditions for the two geotechnical boreholes. The locations of the boreholes are shown in Figure 3. Further, the terrain level and toe level of the test and production piles are shown.

2.2. Groundwater conditions

The production piles for the existing pedestrian bridge as well as the location of the static compression pile load test were placed very close to Remstrup River. The upper free water level generally corresponded to the water level in the river and was expected to vary due to season and rain together with the water level in the river. The water level was measured at level +20.7 ± 20.8 m in the geotechnical investigation.

In the Miocene sand an artesian water level was measured up to level +21.2 m in the geotechnical investigation.
3. Methods
The pile foundation was designed based on the Eurocode system, DS/EN 1997-1, including the Danish national annex, ref. [1] and [2]. According to the national annex the shaft resistance for drilled piles should be reduced to 30 % of the shaft resistance for a corresponding driven pile. Furthermore, it is common design practice to disregard the toe resistance, as this only occurs after deformations larger than normally acceptable for the superstructure.

3.1. Toe resistance and post installation pre-stressing
In the preliminary design the new foundation piles were installed from terrain level at the location of the production piles at +21.6 m. with pile toe level at +12.8 m in the Miocene sand. With the preliminary design and a pile toe in sand deposits, it was found necessary to engage toe resistance to achieve enough bearing capacity. The toe resistance was engaged by a flat-jack device, developed by the contractor MJ Erickson, that would allow for post installation pre-stressing. The pre-stressing was made by grouting under the entire toe of the pile with a controlled injection procedure. Thereby full contact between the pile toe and the underlaying deposits was made, and full base resistance could be considered.

3.2. Shaft resistance for bored piles
According to Danish National Annex to the Eurocode 7, part 1 (2015), Annex L, ref. [2], only 30 % of the shaft resistance of the corresponding driven pile can be considered. However, it is possible to disregard this paragraph if recognized documentation can be provided. To provide recognized documentation as well as to document the effect of the post installation pre-stressing, it was planned to carry out a static compression pile load test. The test was planned to be conducted on a full-scale pile with same dimensions as the production piles and drilled with the same equipment using the same methods as for the production piles.

3.3. Installation of test pile
In March 2019 the test pile was drilled with a Liebherr 155 with Kelly drilling equipment by contractor MJ Eriksson, see Figure 5. The test pile was drilled with an outer diameter of 880 mm. The drilling was conducted with casing on the full length of the pile. During drilling the water level in the bore hole was kept over the measured groundwater level at the location. This was to prevent base failure in the bottom of the drilled hole. If base failure occurred, it would not be possible to engage toe resistance for the pile. After the toe level of the pile was reached, the drill head was retracted, and the reinforcement cage was installed in the bore hole. The pile was cast from terrain by use of a tremie pipe lowered to the bottom of the pile.

3.4. Post installation pre-stressing
Seven days after installation and casting of the pile, the toe was pre-stressed to engage toe resistance. The pre-stressing was conducted under pressure from terrain level while the top of the pile was monitored. It was ensured that no noticeable movements (observation criterion was 2 mm) of the top were registered.

The result of the post installation pre-stressing was that 140 liters of grout was pumped under the toe at a pressure of 8 bars. The movement of the pile top was registered to be approximately 0.1 mm. The measuring of the injected volume does not singlehandedly assure the quality of the pre-stressing, however coupled with the small upwards deformation at the top of the pile the injection procedure was assumed to have been successful.
3.5. Static compression pile load test

In the end of May 2019, the static compression load test was conducted. The bearing capacity of the test pile was estimated by a geostatic calculation. Full toe resistance as well as full shaft resistance was considered. The geotechnical parameters were set to a realistic value, i.e. rather mean values than characteristic values. The bearing capacity was estimated to be approximately 2500 kN. The static compression load test was planned to a maximum load of 3750 kN. To counterbalance the load applied to the pile an anchoring arrangement was used which was anchored to the ground by four soil anchors. The capacity of the anchoring arrangement was designed for a load of 3750 kN, which was also the maximum capacity of the hydraulic jack.

The load procedure for the test consisted to two load cycles; one load cycle to model the estimated serviceability limit state with a net load of 850 kN, and one load cycle to achieve failure and assess the maximum capacity of the pile. The first load cycle consisted of four load steps and the second load cycle consisted of 15 load steps. Between the two load cycles the pile was fully unloaded, after which the second load cycle started. For each load step the load was held constant at a beforehand chosen time limit between 10 and 30 minutes. After the end of the second load cycle the pile was fully unloaded.

The displacement of the pile top, middle of the pile and pile toe was monitored with tell tales connected to analog dial gauges during the test and noted at specific times during the load step. The load procedure for the static compression test was designed by recommendations in DS/EN ISO 22447-1:2018, ref. [3].

The load was applied by a hydraulic jack through an anchoring arrangement. The upper part of the anchoring arrangement was constructed by a beam support system, which transferred the loads to the four vertical ground anchors. The bound length of the ground anchors was placed lower than the toe of the drilled pile. The part of the anchoring arrangement above ground can be seen in Figure 6.
4. Results
The results from the static load compression test consisted of corresponding data points of load, displacements and time from each load step.

4.1. Load-displacement curve
The displacements with the corresponding load steps and time made it possible to draw a load-displacement curve and calculate creep rate for each of the measured load steps, see Figure 7 for the load-displacements curves.

![Load-displacement curve from the static compression pile load test.](image)

Figure 7. Load-displacement curve from the static compression pile load test.

The figure shows approximately similar behaviour of the displacements at the top, middle and toe. Due to the compression of the concrete section it was expected that the top of the pile would undergo larger deformations compared to the middle, and it was expected that the middle would undergo deformations larger than the toe. This behaviour was not recognized in the load test. The difference between the expected and the measured results were believed to be caused by uncertainty in the measurements. The method of registration of the displacement meant that the error can be accumulated, as the displacement was registered for each load step and then added to a total displacement afterwards. Even though the measurements deviated slightly from the expected results with a larger top than toe...
displacement at the end of the test, the overall behaviour of the pile was still evaluated to be reliable for further interpretation and design.

4.2. Failure load
Failure of the pile was defined as a creep rate of the deformations larger than 8 mm/log(t), corresponding to deformations larger than 1.4 mm from the 10th to the 15th minute or from the 20th to the 30th minute in each load step. The creep rate was calculated for load steps above 2500 kN for the measured displacements in the middle and the toe of the pile. The creep rates are calculated at the pile toe, as this excludes the deformations from the elastic compression of the pile itself. The creep rate as a function of the load is plotted in Figure 8.

Figure 8. Plot of the creep rate for the latter load steps in the static compression load test.

Figure 9. Interpretation of creep rate for the last load step at the measured displacement at the bottom.

Figure 8 shows that the calculated creep rates were in good alignment in the middle and the toe of the pile, however with a minor difference at the maximum load. An example of the interpretation of the creep rate for the last load step of 3722 kN for the measured displacement at the toe is shown in Figure 9.

Further, Figure 8 shows relatively small creep rates indicating that the pile is not exhibiting failure at the end of the test. The test had to be stopped at a maximum load of 3722 kN as the anchoring arrangement was not designed for higher loads. The creep rates increase towards the end of the test, and together with the load-displacement curve it was assessed that the pile was going towards failure as the test was terminated.

5. Discussion
In the following sections the results from the pile load test are analyzed and discussed.

5.1. Ultimate limit state
The failure load is for further assessment of the static compression load test estimated to be 3700 kN for a single pile, corresponding to the applied/measured load in the last load step.

The design value of the pile resistance for a single pile was found as:

\[ R_{c,d} = \frac{R_{c,m}}{\xi \cdot \gamma_b} = \frac{3700 \text{ kN}}{1.5 \cdot 1.3} = 1897 \text{ kN} \quad (1) \]
However, the production piles were placed too close together to be considered as single piles, and the group effect must therefore be considered. The pile foundation for the movement of the pedestrian bridge were made of three piles placed in a row with only 0.95 m between the center of the piles. The dimension of the piles was 880 mm, which leaves only 70 mm free space between the piles. The total bearing capacity of the piles was reduced by adding the shaft bearing capacity from the circumference of the pile group and the toe resistance from the three single piles. This reduction gave a design value of the bearing capacity for the pile group of 5100 kN in total, corresponding to 1700 kN per pile in the group.

5.2. Serviceability limit state

To describe the serviceability limit state for single piles the load-displacement curves could simply be used. However, as described in section 5.1 the pile foundation did not act as three single piles. The serviceability limit state for the pile group was assessed by applying a method suggested by Tomlinson, ref. [4]. The suggestion is, that settlements of a pile group can be calculated as a deep footing. A pressure distribution can be considered depending on whether the piles are shaft bearing, toe bearing or a combination of these two. This is illustrated in Figure 10. In general, the pressure distribution is started from the top of the layer, where the pile starts to engage shaft resistance.

Figure 10. Calculation of settlements of a group of piles, ref. [4].

In the current case for the pile group for the movement of the pedestrian bridge, the piles had contributions to the bearing capacity from both the shaft and the toe. Therefore, model (b) from Figure 10 was applied. However, there were no "soft layers" and the pressure distribution was therefore started from the bottom of the foundation. As suggested by Tomlinson, ref. [4], a pressure distribution of 1:4 was applied in all directions. To assess the suggested method a set of settlement calculations were carried out for an 880 mm pile in the soil conditions corresponding to location of the test pile, which is repeated in table-form in Table 1.

Table 1. Properties of the soil for settlement calculation.

| Level, top of layer (m) | Unit weight (kN/m³) | Oedometer modulus, E_oed (MPa) |
|-------------------------|---------------------|-------------------------------|
| Clay, Mi                | +16.5               | 18/10                         | 40                          |
| Sand, Mi                | +13.8               | 18/10                         | 50                          |
| Silt, Mi                | +10.8               | 18/10                         | 40                          |
| Sand, Mi                | +7.3                | 18/10                         | 50                          |
The pile foundation is equalized with a circular deep footing in level +15.5 m. The situation is shown in Figure 11.

![Figure 11. Simplification of settlement calculation.](image)

![Figure 12. Measured and calculated load-displacement curves for single piles.](image)

The calculated settlements were then compared to the actual measured settlements from the load displacement curve. The results are shown in Figure 12.

In general, Figure 12 shows a fine alignment of the measured and calculated settlements. The curves seem to have the same shape within the area of realistic serviceability loads. The calculated settlements are a bit higher compared to the measured. This difference could be explained by the uncertainty in estimating the modulus of elasticity for each soil layer. Since the settlement model from Tomlinson, ref. [1], is developed for driven piles with full contact at the pile toe, the agreement between the model and the test pile indicates that the grouting at the pile toe has worked as planned.

The results were found acceptable for further use, and the method was applied directly for the group of piles under the foundation for the movement of the pedestrian bridge. For calculation of the settlements of the pile group the foundation was compared with a deeply placed footing. The area of the pile group before pressure distribution is simplified to a rectangular foundation with the dimensions 2.7 x 0.88 m. The area after taking the pressure distribution into account was 5.43 x 3.61 m.

The calculated settlements as a function of the total load on the foundation is shown in Figure 13.

![Figure 13. Calculated load-displacement curve for the pile group.](image)

6. Conclusions
A static compression load test was performed on a Ø880 mm test pile. The pile was 9.3 m long and installed with the pile toe situated in a dense Miocene sand deposit. The pile was installed with a flat-jack device at pile toe to allow for post installation pre-stressing by grouting. The pile was loaded from
the top by using a hydraulic jack connected to a large anchoring arrangement anchored to the ground by four soil anchors. A total test load of 3750 kN was achieved without any indications of imminent failure of the pile. The maximum creep rate achieved during loading was approximately 2.3 mm/log(t).

The static load compression test measured bearing capacities much larger than could be shown by using geostatic calculation according to DS/EN 1997-1 DK NA Annex L, ref. [2]. Further, the test showed that the post installation pre-stressing was a success and allowed the designer to take base resistance into account for the production piles. In the performed static pile compression test it is not possible to separate surface and toe resistance. However, the measured loads are very high, which indicates that the reduction of the surface resistance to 30 % according to DS/EN 1997-1 DK NA Annex L is not realistic. The test indicates that a much higher surface resistance can be taken into account.

The measured load-displacement curve was used to find a calculation model to estimate settlements of the pile. The load-displacement curve was compared to settlement calculations of a single pile, which gave similar deformations. The calculation method was thereafter used on the pile group to estimate the settlements of the pile foundations.

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