Tension tests on bored instrumented piles installed in marine Eocene clay

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Abstract. This paper presents parts of the results from two full-scale loading tests (tension) on bored cast-in-place piles installed in high plasticity marine Eocene clay at a test site in Hinge, central Jutland, Denmark. Testing was carried out to investigate the development and distribution of shaft resistance on the test piles. Each test pile was instrumented with distributed fiber-optic strain and temperature cables, in addition to traditional vibrating wire strain gauges and tell tales. Selected examples of retrieved data are presented. The measured shaft resistance is compared to an analytical method based on the Danish National Annex to Eurocode 7, part 1 which represents a conservative estimate for the shaft resistance of bored piles. The results from the two full-scale loading tests (tension) on bored cast-in-place piles in marine Eocene clay show that the Danish code-requirement, concerning shaft resistance, is too conservative.

1. Introduction
This paper presents the preliminary results from the first two full-scale loading (tension) tests from a test program consisting of 8 full-scale bored test piles installed in a high plasticity marine Eocene clay. The purpose of the test program is to determine the development and the distribution of the shaft resistance along the length of the pile as the pile is gradually being loaded to a failure condition. In addition, by testing the piles in pairs at different times after installation (1.5 months, 6 months and 12 months) the full test program will also investigate possible ageing effects on the shaft resistance (results are to be published at later time). To achieve detailed measurements of the distribution of shaft resistance each test pile was instrumented with distributed fibre-optic strain and temperature cables in addition to traditional vibrating wire strain gauges and tell tales. The results are compared to an analytical method based on the Danish National Annex to Eurocode 7, part 1 [1] The code-requirement limits the shaft resistance for a bored pile to 30 per cent of the shaft resistance of the corresponding driven pile unless recognised documentation to show otherwise is available.

1.1. Background
The development and magnitude of shaft resistance on bored piles in fine grained soils is controlled by various mechanisms and highly depend on the chosen execution methods. Stresses will gradually build up after installation of the pile and shaft resistance during static pile load test will depend on the degree of swelling and softening taking place in the clay during installation.
Major shear distortion from pile installation is confined to a relative thin zone around the pile shaft [2-3] making it practically impossible to measure the detailed effects of the governing mechanism in terms of normal and shear stresses, their distribution along the pile shaft as well as excess pore pressure in the adjacent clay. Hence, using the effective stress analysis, also known as the β-method, is not practically possible for determination of shaft resistance for bored piles.

Determination of shaft resistance for bored piles in clay is therefore conventionally based on an empirical approach, where a reduction factor is applied to the undisturbed undrained shear strength of the clay layers which the pile penetrates. This is also known as the α method:

\[ R_{s,cal} = \sum \alpha \cdot c_u \cdot A_s \]

where \( R_{s,cal} \) is the shaft resistance, \( \alpha \) is an empirical adhesion coefficient and \( A_s \) is the shaft surface area.

Experience from pile load tests in similar ground conditions is required to determine the \( \alpha \)-value. Uncertainties in the \( \alpha \)-value and the variation of the undrained shear strength of the soil adjacent to the pile means that the α-method only gives an estimate of the total shaft resistance. Hence, the distribution of shaft resistance along the pile shaft is unknown.

The distribution of shaft resistance along the pile length during load testing can be determined from the measured displacements and strains inside the piles at selected predetermined locations by using tell tales and strain gauges [4-5]. However, the conversion from strains to normal stresses in the pile and finding the actual stresses acting on the pile shaft is not a simple matter. This requires knowledge of the variation in pile stiffness along the length of the pile and the change in stiffness with increasing strains. In addition, the possible existence of locked-in residual forces acting on the pile after concrete curing and prior to pile testing can be hard to estimate [6].

1.2. Shaft resistance of bored piles based on the Danish National Annex to Eurocode 7, part 1

The results from the conducted pile load tests are compared to an analytical method based on the Danish National Annex to Eurocode 7, part 1 [1], where realistic strength parameters (best estimate) are applied. According to [1] the shaft resistance \( R_{s,cal} \) of a bored pile can be estimated from:

\[ R_{s,cal} = \sum \alpha \cdot c_u \cdot A_s = \sum (0.3 \cdot m \cdot r) \cdot c_u \cdot A_s \]

where \( m \) is a material factor, \( r \) is a regeneration factor, \( c_u \) is the undrained shear strength and \( A_s \) is the shaft surface area. The factor 0.3 is the 30 per cent reduction of the shaft resistance of the corresponding driven pile.

The material factor considers the ratio of adhesion to undrained shear strength. For concrete piles a material factor of 1.0 is generally used. The regeneration factor considers the strength reduction of the clay due to remoulding during pile driving and also considers the subsequent strength gain over time due to dissipation of excess pore water pressure [7]. It is usual practice in Denmark to assume a regeneration factor of 0.4 (i.e. 60 % reduction of the intact undrained shear strength) for driven piles 28 days after pile installation if \( c_u < 500 \text{ kPa} \). In Denmark the empirical \( \alpha \)-factor used is applied to the intact undisturbed undrained shear strength determined from in-situ tests (e.g. from field vane tests, where the field vane strength, \( c_v \), can be assumed equal to \( c_u \)) [1].

The empirical \( \alpha \)-factors used for driven and bored concrete piles in Denmark are therefore equal to \( \alpha = 0.4 \) and \( \alpha = 0.12 \) respectively. The \( \alpha \)-factor used for bored piles is based on the results of a low number (7 documented cases) of in-situ pile loading tests carried out in Denmark in the 1970’s, consisting of different pile types and execution methods, primarily established in clay till cf. [8]. In addition, it is likely that experience was drawn upon from abroad, where especially pile testing in stiff over consolidated London Clay was considered relevant as it bears resemblance to the stiff over consolidated Danish Palaeogene clays. In this respect, it should be noted that in the UK the \( \alpha \)-factor is traditionally derived from a comparison of pile test results with the results of undrained triaxial compression tests on undisturbed specimens. This implies that the \( \alpha \)-factor will be highly affected by the presence of fissures, and the values reported in the literature for stiff fissured clays like London clay
are therefore often not directly comparable without careful consideration and knowledge of the fissure strength in relation to the intact strength. Hence, for stiff fissures clays it would generally seem better to derive the $\alpha$-factor based on the intact undisturbed undrained shear strength determined from in-situ testing.

Today’s pile types, execution methods and installation time has changed and together with more challenging ground conditions, this has created a need for a revision of the empirical $\alpha$-factor in Denmark.

2. Ground conditions and test methods

2.1. Ground conditions

Testing was carried out at a dedicated test site located in Hinge, central Jutland, Denmark. The location provides opportunity to work undisturbed for the duration of the entire full-scale test program and provides uniform ground conditions (very high plasticity clay) only approximately 3 m below ground.

Three geotechnical boreholes (B1-A, B2-A and B3-A) with field vane tests and four Cone Penetration Tests (CPT1-CPT4) were performed. Intact samples were collected in A-tubes (Shelby type) from six adjacent boreholes (B1-B&C, B2- B&C and B3- B&C). The locations of boreholes and CPT tests in relation to the test piles are illustrated in figure 1a.

![Figure 1a](image1a.png)

![Figure 1b](image1b.png)

**Figure 1.** (a) Overview of geotechnical investigations and bored cast-in-place piles; (b) Representative soil profiles (B1-A and B2-A) around the test piles.

The top of the geotechnical boreholes B1-A and B2-A indicate 0.8 to 1.8 m clay fill. This is underlain by 0.4 to 0.5 m late glacial deposits of solifluction clay and 1.2 to 1.5 m glacial deposit of clay till down to the top of the glacially disturbed marine Eocene clay 2.8 to 3.4 m below ground, see figure 1(b). The ground level is approximately at +58.0 m above mean sea level (Danish Vertical Reference 1990).

Prior to installation of the bored cast-in-place piles the uppermost 0.4 m of soil was replaced by base course and steel mats to create a stable working platform for the piling rig, and subsequently replaced by gravel around the test setup to minimize settlement in the upper layers during load testing.

At each pile location the top 3.4 m of soil was predrilled (Ø1000 mm diameter) and replaced by sand (poorly graded medium sand). This was done to minimize contributions from the upper soil layers to the shaft resistance, thereby making it easier to deduce the contributions from the underlying layers of marine Eocene clays, see figure 1(b).

The clay till varies from low plasticity in the top of the layer to high plasticity in the bottom of the layer due to the underlying marine Eocene clay. The marine Eocene clay is a very high plasticity clay with an intact undrained shear strength (assumed equal to the field vane strength) between 100 and 350
kPa with an average value of 210 kPa (over the depth of the piles). The natural water content ranges between 41-68% and the unit weight ranges between 16.0-17.2 kN/m³. See figure 2.

Atterberg limits and calcite content for the marine Eocene clay at test site Hinge are given in Table 1 cf. [9].

Table 1. Atterberg limits and calcite content for marine Eocene clay at test site Hinge.

| Water content (%) | Liquid limit (%) | Plastic limit (%) | Plasticity index (%) | Calcite (%) |
|-------------------|------------------|-------------------|----------------------|-------------|
| 44 - 62           | 160 - 233        | 11 - 44           | 86 - 202             | 13 - 36     |

The groundwater level at the site is varying between 0.5 and 1.5 m below ground level with a hydrostatic pressure distribution. Seasonal variations can be expected in the upper layers.

2.2. Test piles and pile instrumentation
The full-scale test program consists of eight instrumented full-scale bored cast-in-place piles. The test piles are 0.62 m in diameter and have a length of 8.5 m + 0.5 m (0.5 m above ground level). The reinforcement cage consists of eight Y32 bars with Y14/190 shear reinforcement in each pile. The concrete used was A35 MPa, and a pair of Ø63.5 mm GEWI PLUS bars were installed 6 m into the test pile to transfer the force from the hydraulic jacks to the test pile during pile load testing.

The reinforcement cage of each test pile was instrumented with 2 distributed fibre-optic strain cables, 1 distributed fibre-optic temperature cable, 10 vibrating wire strain gauges (one pair at 5 different levels) and 4 tell tales. All monitoring equipment was installed in pairs within the reinforcement cage on diametrically opposite sides to separate bending moments from axial loads, see figure 3.

Selected examples of the distribution of shaft resistance from vibrating wire strain gauges are presented in this paper, while the detailed measurement results will be presented and analysed in a future paper.

Installation of the test piles commenced three days after predrilling and replacement of the upper 3.4 m with sand, and the test piles were drilled as cased Kelly piles. No water was used in the borehole during drilling and cleaning. The casing was always kept 1 m above ground to ensure against contamination by debris, and the reinforcement cage was lowered into the borehole before concreting.

The piles were concreted from the bottom up with a tremie pipe, and the tip was kept below the surface of the concrete to avoid mixture and separation. The GEWI bars for the static loading test (tension) were installed manually after concreting of the piles.
2.3. Pile load tests and test setup

The first two static pile loading tests (tension), which are dealt with in this paper, were completed 48 days (test pile 7) and 51 days (test pile 1) after pile installation, respectively. Figure 4 shows the steel traverse system that was used to provide the reaction force for the pile load tests. The steel traverse system consists of two steel mats located with a clearance of approximately 2.5 m on both sides of the test piles. A pair of 4 m long HE300B load distribution beams were placed on top of the steel mats with two HE800B main beams on top. Finally, two pairs of HE400B beams with a length of 4 m were located crosswise above the centre of the test piles to distribute the load on both main beams and to stabilize the load application system. The test load was applied by four hydraulic jacks located central to the pile axis. Each hydraulic jack had a capacity of 300 t and a stroke of 250 mm. The load was measured by 3 MN load cells.

![Steel traverse system for static pile load test (tension).](image)

The reference beams (yellow beams at ground level) were 6 m long and mounted on wooden beams at the ends to ensure that the measurements of the pile displacement were independent from the test setup and any movements thereof. The reference beams were measured with surveyor’s level at two points to check for possible movements.

The test piles were loaded to failure in two load cycles. In the 1st load cycle the load was applied in 5 steps of 100 kN, which corresponds to a total of 500 kN or 60 % of the expected failure load. The
holding time at each load step was 30 min. The pile was then unloaded in steps of 100 kN with a holding time of 5 minutes at each load step, and a holding time of 30 minutes before the 2nd load cycle commenced. In the 2nd load cycle the load was applied in steps of 125 kN and then reduced to steps of 50 kN when the test pile showed sign of imminent failure. The holding time at each load step was 30 min. The actual time-load curves for test pile 1 and test pile 7 is presented in figure 5.

![Figure 5. Loading procedure for test pile 1 and test pile 7.](image)

For each new load step, the load was increased or decreased continuously over a time period of approximately 3 minutes to avoid shocks and vibrations. The load was held as constant as possible during each load step. For test pile 1 the power was lost twice during the static pile load test, which resulted in minor data losses between load step 4 (400 kN) and load step 5 (500 kN), and at the final load step (825 kN). Furthermore, the hydraulic system for test pile 1 had a leak, which was discovered at load step 2 (200 kN) and repaired between the 1st and 2nd load cycle. Therefore, the holding time was extended to approximately one hour for load step 2 and three hours for load step 10 (between 1st and 2nd load cycle).

3. Results

3.1. Load-displacement curves and mobilized pile capacities

The load-displacement curves for the static pile load tests (tension) on test piles 1 and 7 are shown in figure 6. The results are based on data from the displacement transducers on pile top, and no correction were made relative to the surveyor’s level.

The overall behaviour of the shaft resistance response is seen from figure 6 to be strain hardening. The magnitude of creep displacement is seen to increase with increasing load level. This highlight the rate dependent behaviour of the load-displacement response and the mobilised shaft resistance. Test pile 1 is seen to experience creep failure at a sustained load of 825 kN, while test pile 7 fails at 750 kN. The initial almost vertical straight-line portion of the load-displacement curves during the first and second unloading highlight that the elastic lengthening and rebound of the pile is insignificant – this is furthermore confirmed by the measured very small internal strains shown in figure 8. The following sections will focus on the results from the tension load test on test pile 7.
3.2. Rate of creep and shaft resistance
The shaft resistance of the test piles equals the total bearing capacity from the static pile load test, as the performed static pile load tests were tension tests. However, the test piles were loaded incrementally to failure and therefore the log time-displacement/load curves and the corresponding rate of creep were investigated for the last two load steps to document in more detail the influence of creep rate on the shaft resistance of the test piles.

Figure 7(a) and figure 7(b) show the displacement per log time and creep displacement rate against time respectively for the last two load steps (625 kN and 750 kN) for test pile 7. The rate of creep per log cycle of time (trendline) is determined to be 0.18 mm/1c and 67.91 mm/1c at the end of each load step. In the last load step at 750 kN the creep displacement rate is initially seen to reduce and then after reaching a rate of approximately 0.005 mm/sec the creep displacement rate is seen to accelerate and the pile experiences creep failure. In the previous load step at 625 kN the creep displacement rate is found to reduce approximately linearly with time and there are no signs of eminent failure.

Test pile 1 shows similar behaviour in the last to load steps (775 kN and 825 kN) to what is seen for test pile 7. However, considering the magnitude of rate effects highlighted from the load-displacement curves in figure 6 and the small difference in load of only 50 kN between the two last load steps, it is likely that if enough creep time had been allowed, then the pile would eventually also experience creep failure at 775 kN.

3.3. Internal strain profile of the test pile
The results from the vibrating wire strain gauges have been used to determine the distribution of shaft resistance. In the following section examples from the vibrating wire strain gauges level 1 through 5 is presented for test pile 7. For vibrating wire strain gauges the notation SG-1 through SG-5 has been used.
The tangent stiffness therefore cannot be applied for the lower strain gauge levels despite full mobilisation of the shaft resistance above. If the applied load on the test pile is transferred along the pile shaft by gradual mobilisation of the shaft resistance down the pile, then it would be expected that the load-micro strain curves shown in figure 8(a) for the lower strain gauge levels would first rise and then become parallel with the upper strain gauge lines. This is however not seen to be the case, as can also be seen from the variation in tangent stiffness with increasing micro strain in figure 8(b). The tangent stiffness for SG-3 to SG-5 is on the other hand seen to initially reduce and then increase with increasing load level and micro strain. This is an indication, that the pile stiffness is significantly larger than the soil stiffness, and therefore the shaft resistance is activated simultaneously along the length of the pile. The tangent method therefore cannot be applied for the lower strain gauge levels despite full mobilisation of the shaft resistance above. Generally, the tangent method can only be expected to be applicable in compression testing where mobilisation of the base resistance is much slower than the mobilisation of shaft resistance.

Nonetheless, the pile tangent stiffness can be estimated based on SG-1 and SG-2. The E-modulus is often not a constant and can be expected to reduce linearly with increasing strain. From figure 8(b) it can however be seen that the tangent stiffness EₐA of the pile can be considered constant within the limited strain range (up to around 50 με) with a value of around 14 GN, hence the tangent stiffness EₐA and secant stiffness Eₐ can be considered equal.

### 3.4. Load distribution in test pile and mobilisation of unit shaft resistance

The load distribution for all load steps applied to test pile 7 in the 2nd load cycle (reloading up to 500 kN) is shown in figure 9a. The load distribution is converted from the measured strains using the previous determined secant stiffness and assuming that it is valid for the entire length of the pile (the
reduction in pile stiffness of around 1.3 GN at the bottom 2.5 m of the pile due to the lack of the GEWI bars is not considered, as it has only little influence).

The load distribution lines between SG-2 and SG-5 are seen to be approximately linear and rotate gradually with increasing load. This indicates that the mobilised unit shaft resistance is approximately constant with depth and is mobilised simultaneously along the pile below SG-2, as was also indicated from the tangent stiffness in figure 8(b).

![Figure 9. Test pile 7 - Back-calculated load distributions (a); Mobilisation of unit shaft resistance in-between gauge levels (b)](image)

The mobilised unit shaft resistance in-between each gauge level is plotted in figure 9(b) against the pile top movement which can be expected to be similar the relative movement between the pile and the soil. From figure 9(b) the unit shaft resistance below SG-2 is seen to increase gradually as the pile is moving upwards (relative to the soil) reaching ultimate values of around 41 kPa to 57 kPa in the marine Eocene clay (SG-3 to SG-5). Strain hardening behaviour is seen within the plotted range of movement. Surprisingly high mobilised unit shaft resistance is also indicated in the lower sand layer (between SG-2 and SG-3). If the average ultimate mobilised unit shaft resistance in the marine Eocene clay (49 kPa) is compared to an estimated average intact undrained strength of around 160 kPa from boring B2, an \( \alpha \)-value of around 0.3 is achieved.

In figure 10 the measured total shaft resistances from the two full-scale loading tests (tension) on test pile 1 and test pile 7 are compared to the analytically calculated shaft resistance of the corresponding driven pile (equivalent diameter) using the intact undrained shear strength profile from borings B1 and B2 respectively and applying an \( \alpha \)-value of 0.4 in clay as given by the equations according to [1]. The contribution from the sand represent less than five per cent of the analytically calculated shaft resistance of the corresponding driven pile and is thus omitted in the following. In the figure the assumed relationship between the shaft resistances for bored (\( \alpha = 0.12 \)) and driven piles (\( \alpha = 0.4 \)) in clays as given by [1] is also shown.

The derived value of shaft resistance for test pile 1 was 825 kN for a displacement rate of 0.005 mm/sec. This corresponds to an empirical adhesion coefficient \( \alpha = 0.37 \) which is close to the values used for driven piles and around 208 \% greater than given by [1] for at bored pile. For test pile 7, the derived value of shaft resistance was 750 kN for a displacement rate of 0.005 mm/sec. This corresponds to an empirical adhesion coefficient \( \alpha = 0.42 \) which again is close to the values used for driven piles and around 250 \% greater than given by [1] for at bored pile. The deviation between the adhesion coefficient \( \alpha = 0.3 \) for the marine Eocene clay estimated from the unit shaft resistance and the analytical back calculated empirical adhesion coefficient \( \alpha = 0.42 \) based on the total shaft resistance for test pile 7 is likely to be due to the observed mobilised shaft resistance in the lower sand layer.
Figure 10. Comparison of measured total shaft resistance and analytically calculated shaft resistance for an equivalent driven pile.

4. Conclusions
The results from the tension load tests directly gives a measure of the total mobilised shaft resistance of the test piles. Distribution of load in the test piles and hence mobilised unit shaft resistance along the length of the pile has been derived assuming a constant pile stiffness derived from the two uppermost gauges levels in the pile. The results from the load tests indicate that the mobilised unit shaft resistance is approximately constant with depth and is mobilised simultaneously along the pile in the marine Eocene clay. \(\alpha\)-values ranging from around 0.3 to 0.4 are back-calculated based on the two load tests. Hence, the results from the two full-scale loading tests (tension) clearly show that the Danish code-requisition, limiting the shaft resistance to 30 per cent of the corresponding driven pile (i.e. \(\alpha = 0.12\)), is too conservative.

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