ABOUT THE INFLUENCE OF TIME ON THE BEARING CHARACTERISTICS OF PRECAST DRIVEN PILES

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ABSTRACT

The time-dependent increase of pile resistance is a known effect, which can be observed in cohesive and non-cohesive soils as well. In this contribution, approaches for the prediction of time-dependent increase of axial pile resistance are presented and compared with new field tests and project-related database results on precast piles in different soils. The aim of the evaluation is to develop a practical concept, for a reliable prediction of set up, which can be used as a tool for an a technically and economically optimized design of driven piles.

Keywords: driven piles, time-dependent increase, set-up, prediction

1. Motivation

Due to several recent investigations and publications, the increase of the axial pile resistance of precast driven piles after installation can be considered as well known. This time-dependent increase of resistance (known as 'set-up') can be significant and the magnitude greatly depends on soil type, but is influenced also by several further aspects like installation process and initial stress status in soil continuum. Reasons for this set-up are not yet fully understood especially in granular soils. In cohesive soils, set-up usually results from consolidation, i.e. degradation of installation-induced excess pore-water pressure, and creeping processes (e.g. Karlsrud et al. 2014). In non-cohesive soils set-up was initially considered to be caused by relaxation and changes to the inter-granular contact zone due to chemical processes. Recent research indicates that ageing effects of remolded soil might be the decisive process and that a complete understanding can only be achieved with appreciation of the micro-mechanical response of sand to the complex stress and strain paths induced during pile installation and subsequent equalization (Gavin et al. 2015). Bowman & Soga (2003) proposed a mechanism which allows the structure of a granular material to change over time as a result of creep strains and microstructural change. Due to Zhang & Wang (2014) the pile set-up in sand is mainly attributed to an increase in radial stress during pile loading as a result of soil ageing. Pile installation pushes the surrounding soil to the side, thereby imposing additional loading on the soil inside the influence zone. This loading action initiates an associated ageing (or creep) process during the set-up period; the ageing effects is considered to ultimately give rise to an increase in radial stress and pile shaft resistance. The measurements also reveal that the increase in ageing-induced soil stiffness is due to contact normal forces among soil particles gradually becoming more homogenized during the set-up period (Zhang & Wang 2014). Gavin et al. (2015) concluded that changes in stationary radial stress during set-up and enhanced dilation during loading appear to be the principal mechanisms controlling pile ageing in sand.

Apart from soil type, increase of axial pile resistance results only from increase of shaft friction. Pile toe resistance remains constant or decreases slightly.

Understanding and accurately predicting the time-depending increase of resistance is an important condition prior to an economic design of pile foundations. In order to predict the increase of resistance due to set-up, various approaches using empirical calculations are described in the literature. Most of them presuppose a direct relationship between time after end of driving and resulting increase of bearing resistance. The characteristics of soils are considered only indirectly, therefore, the aim is to derive an approach which allows to predict the time-dependent increase considering the soil type and soil properties. In this way an economic pile design could be realized.

2. Prediction of time-dependent increase of pile resistance

The set-up of driven piles in cohesive and granular soils has been decribed in several papers e.g. by Fellenius et al. (1989), York et al. (1994) Long et al. (1999) and Bullock et al. (2005). In recent years comprehensive studies and investigations on pile ageing effects mainly based on systematic field tests have been published, among others by Axelsson (2000a/b), Augustesen (2006), Gavin et al. (2013, 2015), Jardin et al. (2006) and Karlsrud (2012) resp. Karlsrud et al. (2014). These studies intend that the set-up factor $Q(t)/Q_0$ often lies in a range of 1.1 to 2.0 for a period of approx. 100 days after pile installation but might reach values of up to 3.0 within 1,000 days. Although there might be a distinct variability in the degree of set-up even within a construction site, long-term experience allows to derive empirical approaches for predicting the time-dependent increase of pile resistance.

Skov and Denver (1988) describe the relationship between time t after end of driving and increase of axial pile resistance $Q(t)/Q_0$ for sand, clay and chalk based on static and dynamic load tests on precast driven piles as a semi-logarithmic function [1].

$$Q(t)/Q_0 = 1 + \Delta_{10} \log_{10} (t/t_0) \qquad [1]$$

Q_0 is the axial pile resistance determined at reference time t_0 after end of driving and Δ_{10} is an empirical factor (set-up factor) which corresponds to the increase of axial pile resistance per logarithmic time step depending on the type of soil. The authors determine this factor for sand with $\Delta_{10} = 0.2$, for a clay with $\Delta_{10} = 0.6$ and for chalk with $\Delta_{10} = 5.0$. The corresponding reference time is $t_0 = 0.5$ days for sand, $t_0 = 1$ day for clay and $t_0 = 5.0$ days for chalk.

Karlsrud et al. (2014) found in large-scale studies a relatively good agreement with the approach of Skov and Denver (1988) for the determination of the increase of shaft resistance Q_S (t) / Q_{S0} for cohesive soils, but therefore a reference time $t_0 = 100$ days shall be applied. In non-cohesive soils, Karlsrud et al. (2014) suggest - based on the results of large-scale static load tests on steel tube piles - that the time-dependent increase in shaft resistance could be described by a hyperbolic tangent function [2]:

$$Q_s(t) = Q_{s,ref} + a \tanh [b \cdot (t - t_{ref})] \qquad [2]$$

Based on empirical studies Karlsrud et al. (2014) found that the factor Δ_{10} increases with decreasing degree of overconsolidation ratio OCR and decreasing index of plasticity I_P as mentioned in [3]:

$$\Delta_{10} = 0{,}05 + 1{,}3\ (1 - I_P/50)\ OCR^{-0{,}5} \qquad [3]$$

In other investigations Augustesen (2006) detected correlations between decrease of the factor Δ_{10} while undrained shear strength c_u decreases, so an application of undrained shear strengths of 10 kN/ m² $\leq c_u \leq$ 100 kN / m² results in values of $0.22 \leq \Delta_{10} \leq 0.29$.

$$\Delta_{10} = 1{,}24 - (c_u/60)^{0{,}03} \qquad [4]$$

3. Field tests

To quantify the time-dependent increase of axial pile resistance field tests were executed near the city Constance at Lake Constance and near Emden in Northern Germany.

In former investigations significant increases were observed, but the magnitude of these changes varied widely. In addition to natural scattering in soil characteristics, deviation between results of static and dynamic pile load tests, influences from repeated loading, variability caused by lack of limit resistance or the influence of different reference times could be adopted.

In order to limiting the mentioned variance, 6 piles with identical cross-sections, lengths and installation conditions were installed in a row with an internal distance of 2.0 m ($e = 5.2\ D_{eq}$). The pile resistance of each pile was determined by dynamic load tests. The pile resistance was evaluated using the CAPWAP-Method.

All 6 test piles were installed at identical point of time t_0. Only pile no. 1 was subjected to a dynamic load test at time t_0. At the next time $t_1 > t_0$, pile 1 was repeated tested and pile 2 subjected to a first sample load. Next, piles 1, 2 and 3 were loaded; piles 1 +2 were repeatedly loaded and pile 3 was loaded the first time. This procedure was carried on in this way.

3.1 Field tests no. 1

Field test no. 1 was carried out at a location near the city of Constance at Lake Constance, where the following regional soil conditions were detected (Figure 1).

Under fillings and sandy deposits, the regional lacustrine clay called lake clay appears as a sandy clay in a predominantly very soft state with a natural water content close to the liquid limit of between 30 and 60%. The undrained shear strength of the clay was determined to be $c_u \leq 15$ kN/m². At a depth of 10 m below surface, a ground moraine of low plasticity follows, which shows in the upper 10 m a soft to stiff state ($c_u = 20\text{-}50$ kN/m²) and lower predominantly a semi-solid to solid state.

The cone resistance q_c, which was determined by two cone penetration tests (CPT), was almost zero at about 15 m under surface and then increased to $q_c \geq 10$ MN / m² in the semi-solid ground moraine. The shaft friction measured at the cone penetration test at depths between 15 m and 20 m was on average at $f_s = 0.45$ MN/m^2 and friction ratio was $R_f = 4$ to 6% (figure 1).

Figure 2 illustrates the results of the dynamic load tests on a semi-logarithmic scale, separated by shaft friction and toe resistance. At first, figure 2 shows that the measured initial shaft resistance with values of 150 kN to 200 kN is significantly lower than toe resistance with values of 600 to 700 kN. After 7 days shaft resistance increased to values between 540 and 620 kN.

The toe resistance remained constant for a period of 14 days. The increase in the shaft resistance 76 days after installation was on average more than 3 times with respect to t = 1d.

For the initial drive as well as restrikes a 70 kN hammer with drop heights between 30 and 40 cm were used. The pile toe was mobilized in all restrikes.

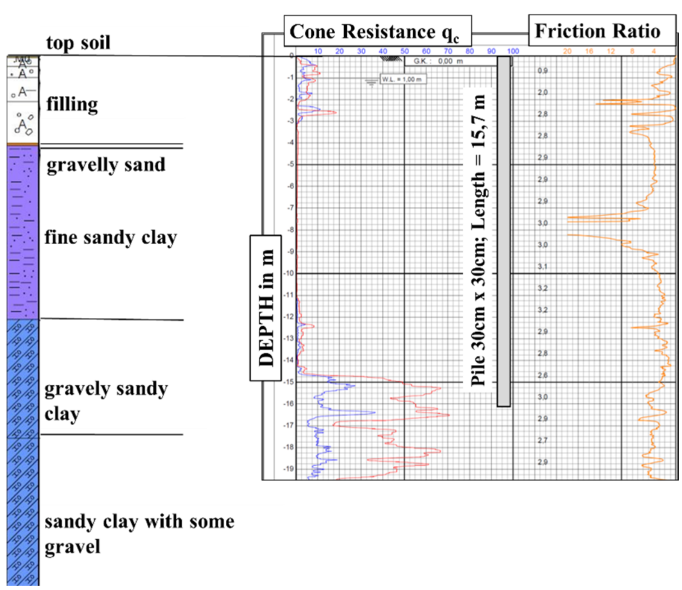

Figure 1: Soil profile at the investigation site no.1

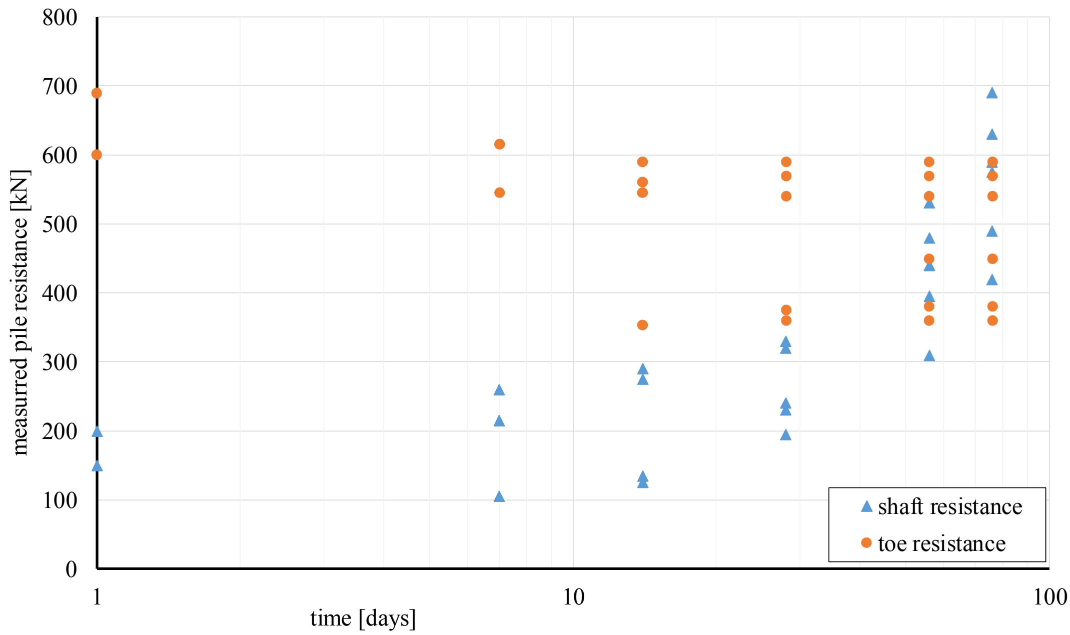

Figure 2: Results of dynamic load tests separated in shaft and toe resistance

3.2 Field test no. 2

At the site of the second field test near the city of Emden, organic soft layers (marine clay + peat) with thicknesses of up to 2.0 m are located fillings and sands underneath according to the ground investigation. Glacial loam and till are located with sand deposits of different thicknesses. The glacial till is a low plastic clay of a stiff consistency. The glacial till mixed with sandy deposits of different sizes were found (Figure 3). At depths from NN -20 m to NN -22 m embedded silt layers were found.

The results of dynamic load tests in Emden are shown in figure 4, separated in shaft resistance and toe resistance. In Constance, the proportion of shaft resistance with a value of 1.742 kN is higher than toe resistance with a value of 410 kN (t = 1d). After 7 days the measured pile shaft resistance was between the lowest value of 1.209 kN and the maximum value of 1.834 kN. The measured toe resistance was between 422 kN and 938 kN. In the further course of the testings, both shaft and toe resistance remained relatively constant, so that no relevant increase in the pile resistance could be detected.

In the investigations initial drive as well as restrikes were performed with a modern 70 kN hammer with drop heights between 50 and 60 cm. The sum of driving energy while initial drive was between 20.000 kNm/m and 30.000 kNm/m.

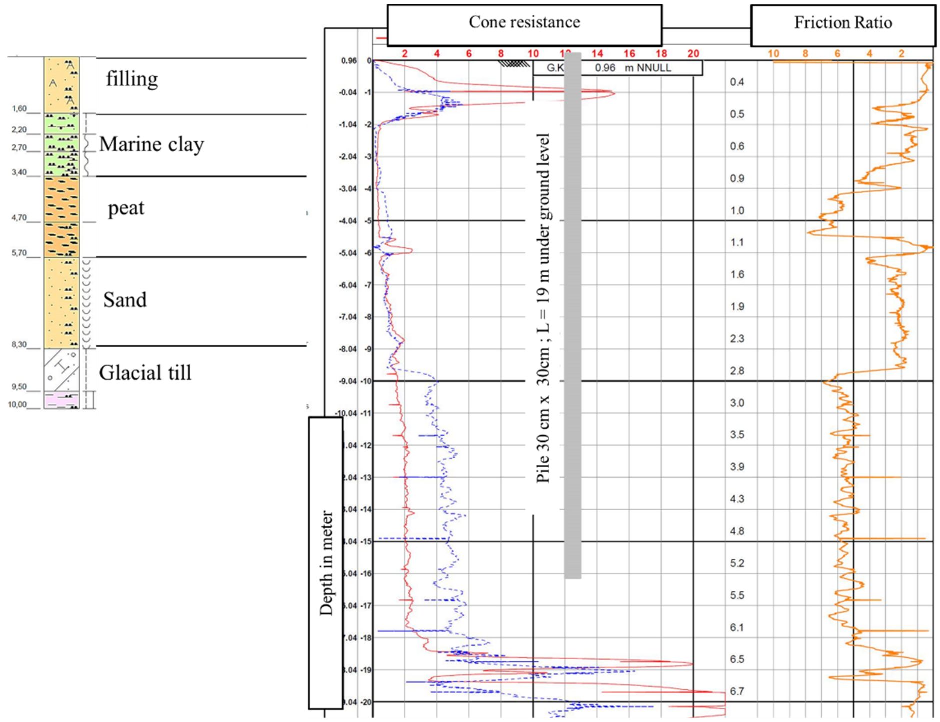

Figure 3: Soil profile at the investigation site no.2

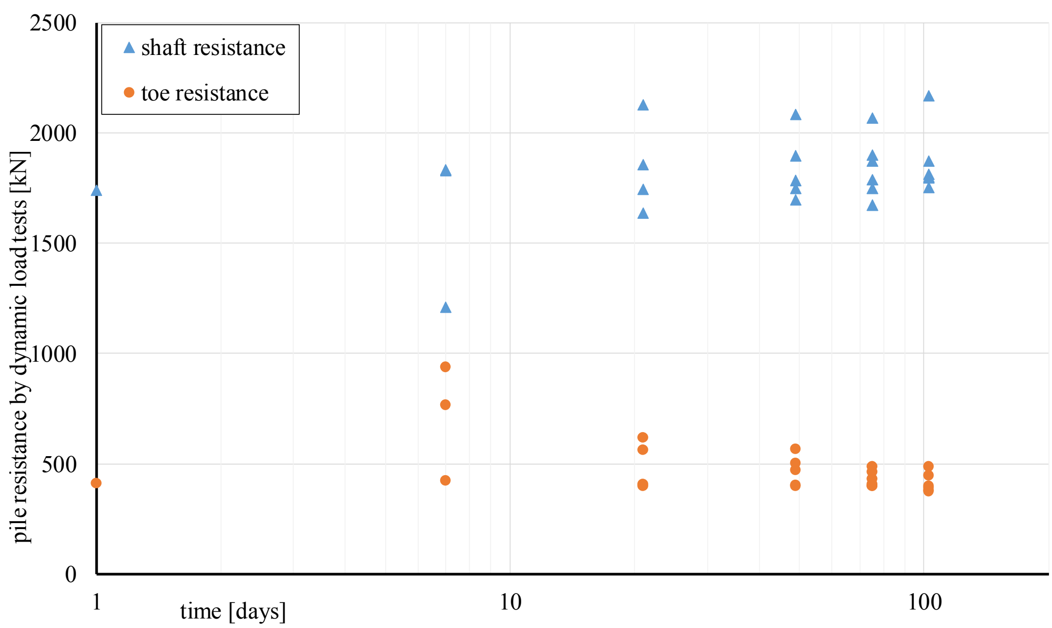

Figure 4: Results of dynamic load tests separated in shaft and toe resistance

4. Data based evaluation

In this chapter data of project based load tests on precast driven piles in cohesive and non-cohesive soils were compared with literature (Skov & Denver 1988, Jardine et al., 2006, Karlsrud et al., 2014 and Gavin et al. 2015).

Applied data of time-dependent increase in non-cohesive soils (Figure 5) were determined on precast driven piles, which were installed at various project sites in the area of the Elbe estuary between Hamburg and Cuxhaven. The detected subsoil was comparatively homogeneous with organic layers (peat and marine clay) and medium to dense sand underneath.

For the evaluation, values at reference time $t_0 = 0$ were determined by linear extrapolation. As a result of this calculation, some computational values are $Q / Q_0 < 1$.

Figure 5 shows data of time dependent pile resistance with the previously discussed large variance of measurement results. It has to be mentioned that test results are differentiated in single loaded and repeatedly loaded pile tests. However, results of dynamic load tests on single loaded piles show larger values of $Q / Q_0 \leq 1.84$ than test results, which were repeatedly loaded piles ($Q / Q_0 \leq 1.64$). The drawn minimum level indicates a moderate increase in the first 10 days that increases in half logarithmic scale exponentially (see Figure 5).

According to Skov & Denver (1988) the increase of axial pile resistance measured by static and dynamic load tests on single loaded piles with values of $2.0 \leq Q / Q_0 \leq 2.2$ forms the upper limit. The static tensile tests on steel tube piles in Dunkirk, France (Jardine & Standing, 2000) show an increase of $Q / Q_0 = 2.37$ while static tensile tests in Blessington (Garvin et al. 2015), within a period of 210 days show an increase of axial pile resistance about $Q / Q_0 = 2.1$.

For comparison,the results from the database with the prognosis according to equation eq. 1 are shown in figure 5 as well. It can be seen that up to approx. 80 days after installation the prediction is above the lower limit. From a time > 80 days, values of eq. 1 are below the lower limit of the measured values and thus eq. 2 leads to a prediction on the safe side in a sense of engineering.

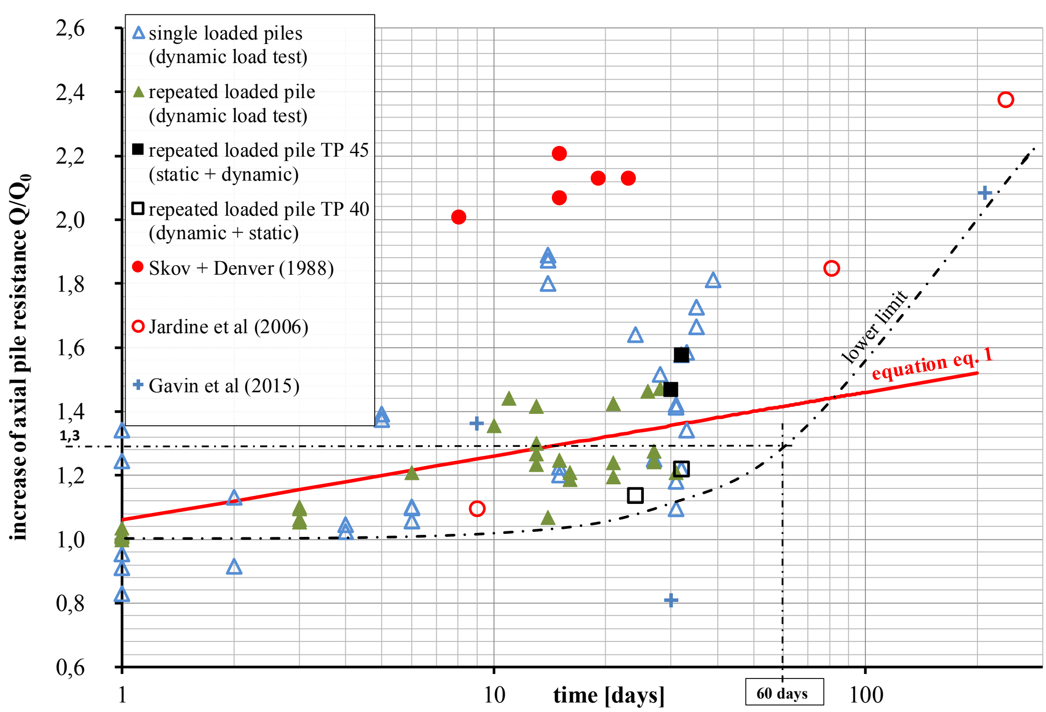

Figure 5: Data of project based load tests on precast driven piles in sands

Figure 6 shows the results of dynamic and static load tests in cohesive soils. One of these soils are over-consolidated tertiary clays with a rigid to semi-solid consistency and a classification of medium to high plasticity clay. The dynamic load tests, described in figure 7 as lake clay, were carried out on piles, which were driven into normally consolidated, very soft to soft clays. The increase of pile resistances is only moderate in that kind of soil, which is symbolized as lake clay + ground moraine.

Results of tertiary clay show within 50 days an increase of $Q / Q_0 = 1.08$ to 1.46. The results of dynamic load tests in clayey silt show significantly higher gains of up to $Q / Q_0 = 2.36$. An additional result of a static load test in clayey silt is also shown in figure 6. Prior to the static testing, a cyclic load was applied in 50 cycles. The increase in the load bearing capacity compared to the initial load capacity is smaller with $Q / Q_0 = 1.25$.

The observed increase in the pile resistance in lake clay after 8 days amounts to approximately $Q / Q_0 = 1.7$ and after 80 days to $Q / Q_0 = 2.2$.

The method described by Karlsrud et al. (2014) shows set-up factors depending on the ground conditions. The load tests carried out in low plastic, normally consolidated clays in Stöndal ($w_n = 0.3$; $I_P = 0.14$) show an increase of $Q / Q_0 = 1.9$ (after 65-175 days) to 2.7 (after 345-375 days). For the location Onsöy ($w_n = 0.6$, $I_P = 0.33$), values of $1.1 \leq Q / Q_0 \leq 1.3$ were determined in a normally consolidated, high plastic clay after up to 345 days. The results at the Cowden site ($w_n = 0.16$, $I_P = 0.18$), obtained in a low plastic over-consolidated ground moraine are of a similar order of magnitude as in Onsöy.

In Figure 6, the results of the data collection were compared with the prognosis showing a good conformity in normal consolidated soils. The measured values in over-consolidated soils are widely overestimated by the prognosis.

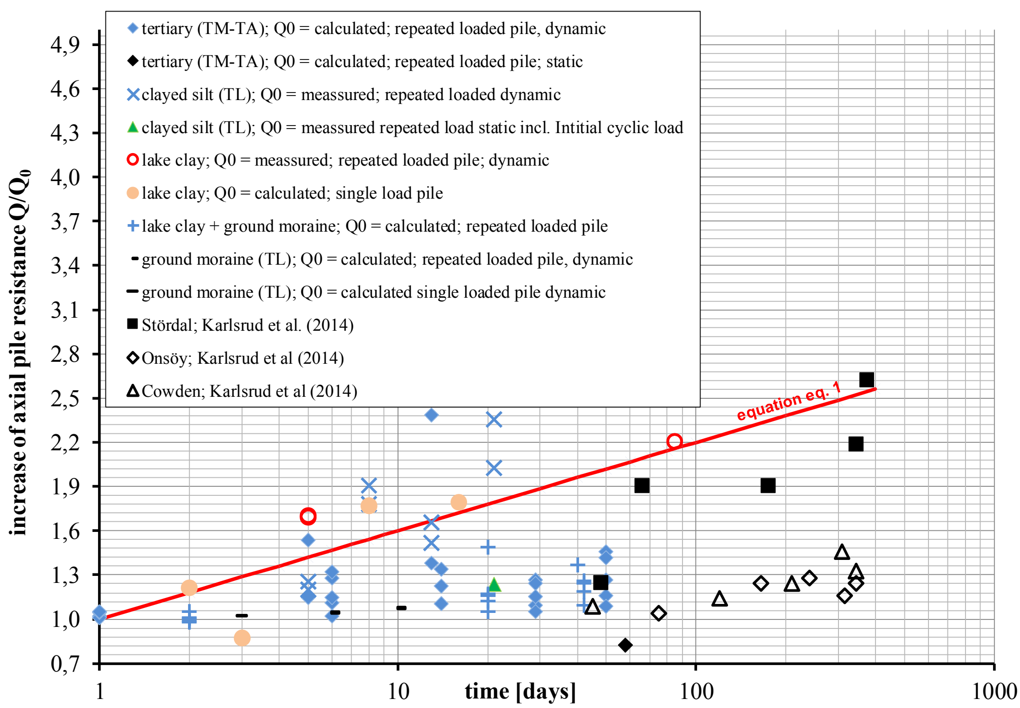

Figure 6: Data of project based load tests on precast driven piles in cohesive soil

5. Evaluation of prediction pattern

In the previous sections, results of increasing axial pile resistance Q / Q_0 on precast driven piles were presented. These results show that the increase in pile resistance can be attributed entirely to shaft friction. In cohesive soils in the first days after the pile installation, even a decrease of the toe resistance could be observed.

According to these evaluations, an increase of axial pile resistance $Q / Q_0 = 1.3$ can be assumed in sands, after ≥ 60 days. On average, growth was even at $Q / Q_0 = 1.5$ after 60 days. The application of equation eq. 2 yields an increase of axial pile resistance $Q / Q_0 = 1.4$ after 60 days with a gain factor of $\Delta_{10} = 0.2$

and a reference time of t_0 = 0.5 days. Thus the approach of an Q / Q_0 = 1.3 is confirmed after 60 days (see Figure 5).

The evaluation of the test results for cohesive soils confirms the approach according to Karlsrud et al. (2014) and shows that the set-up factor and the time increase with decreasing over-consolidation ratio OCR and decreasing plasticity I_P. The context according to Karlsrud et al. (2014) is shown in Figure 7, including set-up factors Δ_{10} for the field tests in Constance and Emden as well as clayey silt and ground moraine.

When evaluating set-up factors in Figure 8, however, it should be kept in mind that, according to eq. 4 the reference time is t_0 = 100 days. Since the duration of the project-related results was <100 days, the reference time t_0 = 1 day was calculated here based on Skov & Denver (1988).

The results of field test 1 (Constance) gave a set-up factor of Δ_{10} = 1.09 for the reference time of t_0 = 1 day and an initial shaft resistance of Q_{0s} = 141 kN. The set-up factor for the field results in Emden is also equal to t_0 = 1 day and an initial shaft resistance of Q_{0s} = 1.734 kN to Δ_{10} = 0.6. For clayey silt and ground moraine, set-up factor is Δ_{10} = 0.6 and Δ_{10} = 0.47 with initial shaft resistances of Q_{0s} = 1.597 kN and Q_{0s} = 244 kN respectively.

In total, the values for Δ_{10} calculated from the measurements are higher than determined by eq. 3.

There were no correlations found between the undrained shear strength and the increase Q / Q_0 (see Figure 8). Although the set-up factor decreases overproportionately with increasing shear strength, the overall growth factor here is in a different order of magnitude than by eq. 4.

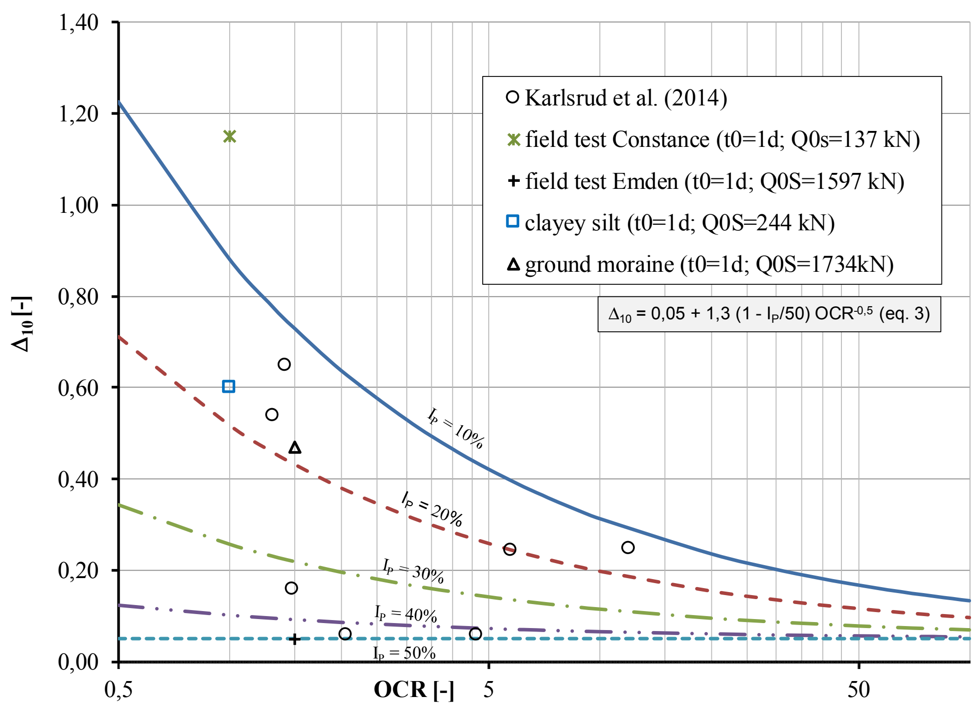

Figure 7: Connection of set-up, over consolidation ratio and plasticity index

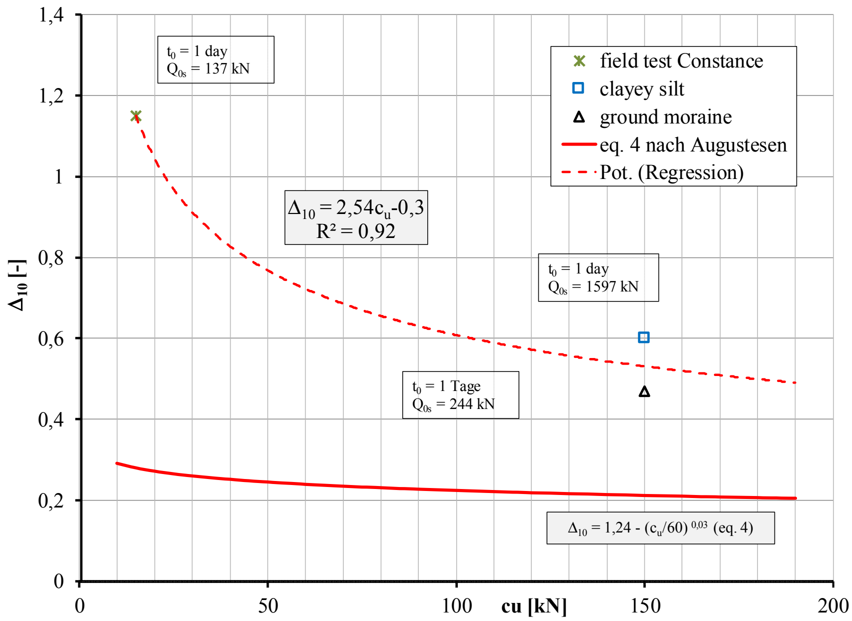

Figure 8: Connection of set-up and undrained shear strength

6. Conclusions

The presented pattern for prediction of time-dependent increase of pile resistance does not take the soil type or soil properties into account. Thus, equation eq. 1 differentiates between sand, clay and chalk and in comparison to the database equation eq. 1 shows a good conformity. However, for the prediction of time-dependent increase of pile resistance in cohesive soils, additional parameters should be noticed to consider the variability of cohesive soils.

If the consolidation ratio is taken into consideration , as predicted by Karlsrud et al. (2014) seems to be successful in comparison with results of the database whereas the approach based on undrained shear strength was not confirmed.

For a conceptual approach to the prediction of time-dependent axial pile resistance, the relevant period of observation must be defined first. In most construction projects, the full pile load usually occurs after the completion of the structural work, depending on the construction process, after at least 60 days. During this period, the short-term effects are usually completed depending on the type and characteristics of the soil, which is why a prediction for the period ≥ 60 days is important.

The following aspects can be considered a practical concept for pile design according to current knowledge:

• a prediction <60 days does not appear to be useful.

• in non-cohesive soils, an increase of 30% can be applied for time of ≥ 60 days or a prediction according to equation eq. 1.

• in cohesive soils, a distinction must be considered as follows:

- in case of normal consolidated soils, an increase of $Q / Q_0 = 2$ can be assumed after 60 days. equation eq. 1 has a good orientation for normal cohesive soils.

- In case of over-consolidated soils, an increase of $Q / Q_0 = 1.3$ can be expected after 60 days.

Generally, repeated load tests are recommended to prove a time-dependent increase in the resistance of precast driven piles or for further verification of prognosis, especially in cohesive soils.

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