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Shear resistance during and after installation of driven piles in soft clay

La résistance au cisaillement des pieux dans l'argile molle pendant et après le battement

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ABSTRACT: Within Gothenburg in western Sweden, the subsoil mainly consists of soft clay, often with a thickness exceeding 100 m. Buildings are therefore usually founded on piles, and the by far most common pile type is precast concrete piles. However, for the reason of mass displacements and high loads, these are sometimes replaced by bored piles or driven steel tubes. The risk of loosing the casing for bored piles, and for plugging of steel tubes, then brings that the shear resistance during and immediately after installation has become a critical factor. Some recent experiences, as well as some older research, is therefore compiled here. It is found that the shear resistance during installation is lower than usually assumed, but also that the shear resistance increases quickly.

RÉSUMÉ: À Göteborg, dans l'ouest de la Suède, le sous-sol est formé principalement d'argile molle, d'une épaisseur souvent supérieure à 100 m. Les bâtiments sont donc souvent fondés sur des pieux, et le type de pieux de loin le plus courant est en béton préfabriqué. Cependant, étant donné le refoulement latéral du terrain et les charges élevées, ceux-ci sont parfois remplacés par des pieux forés ou des tubes d'acier battus. Le risque de perte du tubage de soutien pour les pieux forés et celui de bouchons dans les tubes d'acier font que la résistance au cisaillement pendant et immédiatement après l'installation est devenue un facteur critique. On examine des essais récents ainsi que des résultats de recherche plus anciens. On constate que la résistance au cisaillement pendant l'installation est inférieure à ce qui est généralement supposé, mais aussi que la résistance au cisaillement augmente rapidement.

Keywords: Piles, soft clay, shear resistance, pile installation, reconsolidation.

1 INTRODUCTION

Within the coming decade, the city of Gothenburg will undergo a tremendous densification of the city center. Within these parts of Gothenburg there are considerable layers of soft clay, sometimes with more than 100 m thickness.

This brings that thousands of rather long piles will be installed within quite limited areas, see figure 1. Traditionally precast driven concrete piles are used in Sweden, due to its low cost per "carried ton of load".

Furthermore, these are often used as friction piles and not as end bearing piles, partly due to the deep soil strata, partly since ongoing settlements in these areas would lead to huge negative skin friction if used as end-bearing, (the piles would not even "carry themselves"!).

Such a massive piling within limited areas also causes considerable mass displacements, which may harm surrounding buildings, pipes etc.

Therefore, other pile types are often considered, such as driven steel tube piles and bored piles. However, whether a driven steel tube gives less displacements than a precast concrete pile, depends on whether plugging at the tip will occur or not.

Bored piles are avoided for friction piles, due to the disturbance of the clay during installation. And if using very long bored piles, the risk of loosing the casing due to too much skin friction, must be considered. This has lead to an increasing need of knowledge concerning skin friction during, and immediately after, installation of piles in soft clay, i.e. the process described in figure 2.

The research within this area is very limited, but here some older studies, as well as some resent practical experiences are presented.

Figure 1 Comprehensive piling is taking place in central Gothenburg

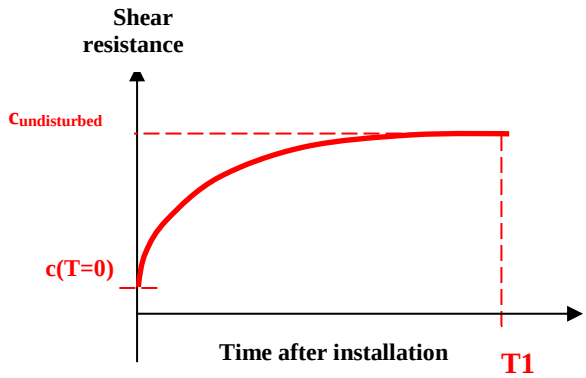

Figure 2 Schematic diagram showing shear resistance during and after pile driving. After the time T1, the skin friction is assumed to correspond to the original shear strength of the undisturbed clay.

2 SHEAR RESISTANCE DURING INSTALLATION

The bearing capacity of friction piles in Gothenburg is traditionally determined by means of the so-called "α-method", where α is an empirical relation between the maximum shear resistance along the pile shaft and the undrained shear strength of the clay, c_u, (in Sweden often interpreted from field vane tests).

Some time after installation, the α-value can be considered to be very close to 1.0 in Gothenburg clay, (c_u is typically 10-20 kPa in the upper part, and then increasing with 1,4 kPa/m. The water content is most often 60-80 %).

However, how large the skin friction is during driving is more or less unknown. A first guess might be that the sensitivity of the clay, St, determined in the laboratory, may be used. For Gothenburg clay, St is typically between 10 and 20, which then would bring that one could expect an α-value of 0,05-0,10.

Field tests carried out by Torstensson, (1973)

Torstensson carried out field tests in a model scale, ($\phi_{pile} \approx 75$ mm. $L_{pile} = 1{,}5$ m), where a pile was load tested immediately after installation. The pile was then driven further down and load tested again. In this way the pile was load tested at different depths down to 17 m. He found that the bearing capacity was roughly proportional to the shear strength of the remoulded clay, but about 2,5 times higher than this. Further he found that the shear strength of the remoulded clay was constant with depth, despite that the shear strength of the undisturbed clay varied from 17 to 35 kPa, (i.e. the sensitivity varied).

Except for these tests, there are no reported tests carried out during, or immediately after, pile driving. Therefore, information has been gathered from other projects where the shear resistance during driving has been determined indirectly.

In connection with the design of the foundation of the new Hising bridge in Gothenburg, long steel tubes, (ϕ = 0,406 m, t = 8 mm), were installed down to 66,5 m by drilling. After 19 days, the pile was redriven and then pulled up. The tension force was not measured, but considering the maximum capacity of the crane, the α-value must have been less than 0,02, (corresponding to an average shear resistance of 1,5 kPa).

α interpreted from plugging of tubular piles

Another way to find out the shaft resistance during driving is to see whether a steel tube pile will "plug" or not during driving. Plugging will occur when the internal shear resistance exceeds the point bearing capacity, see figure 3.

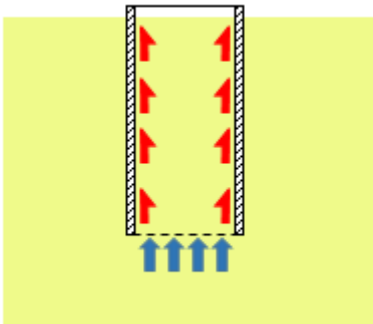

Figure 3 Plugging of a tubular pile will occur when the shaft resistance along the inside of the pile, (red arrows), will exceed the point resistance of the pile, (blue arrows)

This will be the case when:

$$9 \cdot \pi \cdot R_{pile}^2 \cdot c_{u,\,pile\,tip} < \pi \cdot D_{pile} \cdot L_{pile} \cdot \alpha \cdot c_{u,\,ave}$$

where

R_{pile} , D_{pile}	is the inner radius and diameter of the pile,
L_{pile}	is the pile length
$c_{u,\,pile\,tip}$, $c_{u,\,ave}$	is the undrained shear strength at pile tip, and a mean value along the pile, (undisturbed clay).

At Marieholm in eastern Gothenburg, 35-39 m long steel tubes, (ϕ 1,42/1,58 m, t = 16/17 mm), were installed by a vibratory hammer, as a retaining wall. No plugging occurred, which means that the α-value was less than 0,14.

Within the quarter Platinan in central Gothenburg, 78 m long steel tube piles, (ϕ 0,323, t = 12,5 mm) were installed by driving, and here 20-25 m long plugs were obtained.

In figure 4, the shaft resistance for different assumptions concerning the α-value, as well as the point resistance, for these piles are shown. The observed plugging then corresponds to an α-value of approximately 0,025.

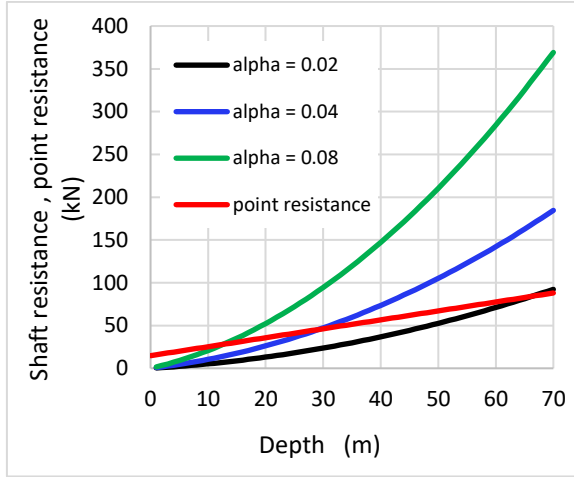

Figure 4 Bearing capacity for pile shaft at different α-values, and point resistance ϕ 0,323 m, t = 12,5 mm, Gothenburg clay, (c_u = 15+1,4 z)

One of the major ongoing projects in central Gothenburg is the Hising bridge.The foundation work for the Hising bridge started in 2017, and will end in 2019. Driven steel tube piles of different diameters, 0,323, 0,406 and 0,610 m are used.

Very close to Platinan, the Hising bridge project installed an 80 m long test pile, (ϕ 0,323, t = 12,5 mm). This pile was installed by a vibratory hammer, and resulted in an appr. 11 m long plug, corresponding to an α-value of appr. 0,02.

This is despite the fact that the jointing of the vibrated piles, (threaded joints), took much longer time than for the driven piles in the neighboring site Platinan, (conical joint), and that this might have led to some reconsolidation.

In the southern part of the Hising bridge, 45 m long piles, (ϕ 0,610, t = 12,5 mm) were driven. No plugging occurred, which indicates an α-value less than 0,06.

In the northern part of the bridge, where the shear strength of the clay is considerably higher, 69 m long piles, (ϕ 0,403, t = 12,5 mm), were driven. An approximately 20 m long plug was observed, (even though the plug appeared continuously during driving).

With the actual shear strength this corresponds to an α-value of appr. 0,04. These piles were, however, installed with considerable interruptions, (partly due to welded joints), which may have enabled the clay to reconsolidate.

A summary of the α-values obtained from observed plugging of steel piles is shown in Table 1.

Table 1. α-values back-calculated from observed plugging of steel tubes

Site	α-value
Marieholm, (ϕ ≈ 1500 mm)	< 0,14
Platinan, (ϕ = 323 mm)	0,025
Hising bridge, south, (ϕ = 610 mm)	< 0,06
Hising bridge, south, (ϕ = 323 mm)	0,02
Hising bridge, north, (ϕ = 406 mm)	0,04

3 RECONSOLIDATION OF CLAY – INCREASE OF SHEAR RESISTANCE

In the Norwegian handbook *Peleveiledningen*, the maximum shaft resistance which can be mobilized at different times after installation is presented, see figure 5.

However, these figures are design values, and therefore conservative in the case a best estimate of the load bearing capacity is to be determined.

Back in the seventies, HE 180 A steel beams were installed as anchors in soft clay in central Gothenburg, (Hansbo et al, 1974). The 14+9 m long anchors were test loaded 5-42 days after installation, (ML tests), and the results showed that 75 % of the capacity "after long time" was reached after just 5 days, see figure 6.

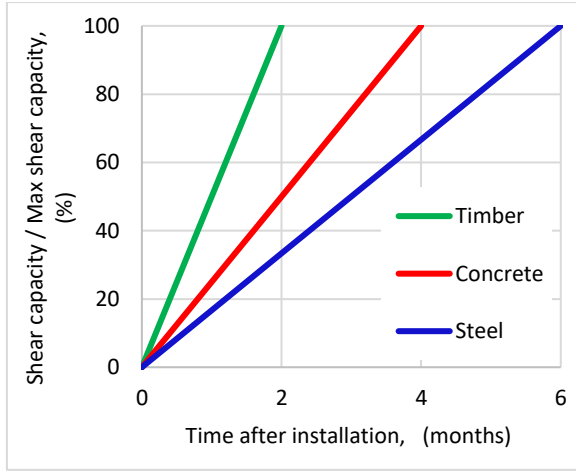

Figure 5 Shear capacity at different times after pile installation, (Peleveiledningen)

Fellenius, (1955), installed a number of piles at Gothenburg central station. The piles where then test loaded after 1 – 1140 days. Piles 1, 2 and 3 where 13 m long concrete piles with B = 0,2-0,25 m, (square piles and an "X-pile"), while Pile 4 was a 12 m long steel I-profile, (300 x 125 mm), see figure 6.

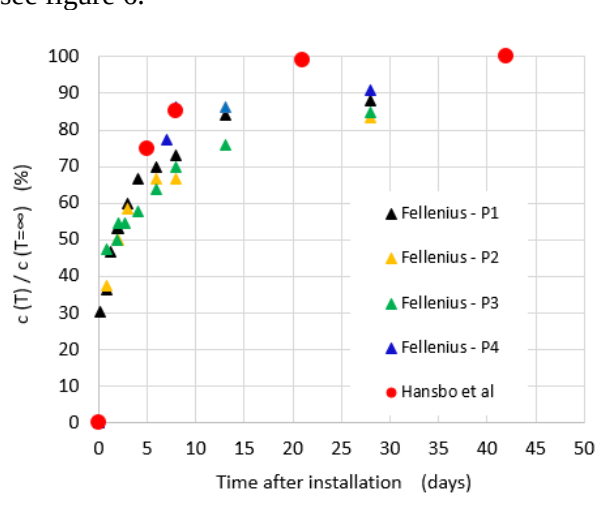

Figure 6 Shear resistance at different times after installation, (after Fellenius, 1955, and Hansbo et al, 1974)

As can be seen in figure 6, the shear resistance increases rather rapidly after installation, and after 5 days, 60-70 % of the final shear resistance is obtained. Somewhat unexpectedly, the highest values are obtained for the two cases with steel profiles, i.e. Hansbo et al and Fellenius Pile 4.

This is contradictory to the relations presented in *Peleveiledningen*, cf. figure 5, where steel piles are assumed to take longer time for re-establishing the shear resistance. Any obvious explanation for this is hard to see. A steel pile will definitely not suck any water during reconsolidation, and will also give the least normal stresses against the pile shaft, but on the other hand it will probably cause least disturbance of the clay during driving.

Further, it can be observed that the results reported by Hansbo et al indicates a somewhat higher shear resistance at a given time after driving compared to the results presented by Fellenius. This might, at least to some extent, be explained by the fact that the tests carried out by Fellenius are carried out on the same piles, i.e. piles 1-4 are tested many times, which may have caused a disturbance of the clay at each test.

4 CONCLUSIONS

The shaft resistance during driving of piles in soft clay is found to be considerably reduced due to remoulding of the clay. Practical experience show that an α-value in the order of 2-5 % can be expected.

Vibratory hammers seem to be more effective than drop hammers when it comes to reducing the shaft resistance.

Further, interrupted installation, e.g. for time consuming jointing work, seems to cause some reconsolidation of the clay, and therefore also increased shear resistance.

Reconsolidation of the remoulded clay occurs rather rapidly during the first period after installation, and the cohesion between pile and clay may be as high as 75 % of the undisturbed shear strength after one week.

Thus, according to the findings presented here, the increase of shaft resistance after installation can be expected to follow figure 7, where the initial value is 2-5 %.

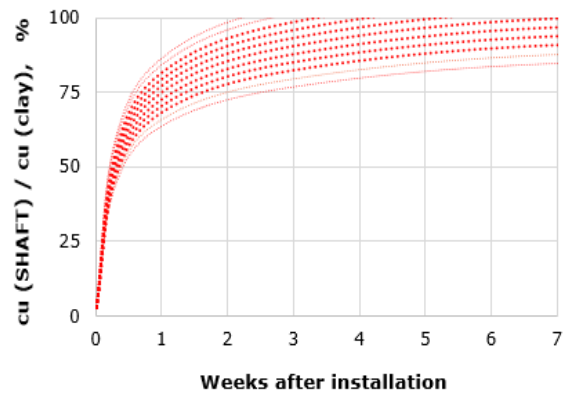

Figure 7 Increase of shaft resistance after driving

5 LITERATURE

Fellenius, B., (1955), Resultat från pålprovningar vid Göteborg C, *Meddelande No 5, kungliga järnvägsstyrelsens geotekniska avdelning*.

Hansbo, S., Sällfors, G., Widing, S, (1974), Bakåtförankring av sponter vid mycket stora lerdjup – praktiska och teoretiska erfarenheter av belastningsförsök, *Väg- och vattenbyggaren, No 8-9*

Peleveiledningen, (1987), Den Norske Pelekomite

Torstensson, B-A, (1973), Kohesionspålar i lös lera – En fältstudie i modellskala, *PhD Thesis, Chalmers University of technology.*