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Jensen, J. Lysebjerg; Augustesen, A.; Sørensen, Carsten S.

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The Influence of Time on the Bearing Capacity of Driven Piles

J. Lysebjerg Jensen

Aalborg University, Denmark, lyseb99@civil.auc.dk

A. Augustesen

Aalborg University, Department of Civil Engineering, Denmark, i5aa@civil.auc.dk

C. S. Sørensen

Aalborg University, Department of Civil Engineering, Denmark, i5css@civil.auc.dk

Abstract: In Danish engineering practice, one of the ways to determine the ultimate bearing capacity of an axially loaded pile is by means of geostatic formulas. In the equation describing the contribution from the shaft friction to the total bearing capacity for piles located entirely or partly in clay, a regeneration factor appears. The regeneration factor accounts for effects of dissipation of pore pressure due to pile driving and true time effects such as ageing on the ultimate bearing capacity. Normally the factor is 0.4 but in this paper, the influence of the undrained shear strength and time on the regeneration factor is investigated. A relation between the quantities is proposed, which in the end may imply an economical benefit in the design of pile foundations.

1 INTRODUCTION

In Denmark, driven squared concrete piles are very commonly used. During pile installation, the soil close to the pile surface is remoulded, and depending on the soil, negative or excess pore-water pressures develop (see for example Tomlinson 1994; Bond & Jardine, 1991). That is, the soil that surrounds the pile will not have the same strength immediately after the installation as it had before pile driving took place.

The excess pore-water pressures dissipate over time, which implies that some of the strength lost during the installation of the pile will be recreated over time. Furthermore, even though the excess pore-water pressures are dissipated the strength of the soil and thereby the ultimate bearing capacity of a pile can increase over time. This is due to true time effects denoted ageing. For further details on the influence of time on soil strength and ultimate bearing capacity of piles, see for example Augustesen *et al.* (2002); Schmertmann (1991); Bergdahl & Hult (1981); Karlsrud & Haugen (1986); Wardle *et al.* (1992); Powell *et al.* (2003); Soderberg (1962); Flaate (1972); Chow *et al.* (1998); Long *et al.* (1999); Chen *et al.* (1999); Axelsson (2000); Fellenius *et al.* (1989) and Skov & Denver (1988).

In connection with geostatic formulas used in Danish engineering practice to calculate the ultimate bearing capacity of an axially loaded pile, time (consolidation and ageing effects) is only assumed to influence the calculated skin friction in clay (DS 415, 1998). In reality, time may also play a role in connection with skin friction in sand and toe resistance in both sand and clay. See for example Axelsson (1998); Chow *et al.* (1997) and Konrad & Roy (1987) for comments regarding skin friction in sand and clay and toe resistance in clay.

In Denmark, the regeneration factor r accounts for the effects of time on the strength of the soil surrounding the pile and thereby the ultimate bearing capacity. Furthermore, it is only incorporated in the equation for the skin friction in clay (see Appendix). The r -factor is the ratio between the shear strength in a given depth at a given time after pile driving and the shear strength in the same depth before pile driving took place. That is, since time influences the shear strength, time also influences the r -factor and thereby the bearing capacity. According to the Danish Code of Practice for foundation engineering (DS 415, 1998) the regeneration factor r equals 0.4 if the value of r is not more precisely specified by means of experiments.

The purpose of this paper is to propose a relation between time, shear strength and the regeneration factor r . This can in the end imply a better estimate on r (instead of using $r = 0.4$) and eventually lead to an economical benefit. The relation is based upon a database (section 2), which is a collection of Danish cases involving static loading tests. In sections 3 and 4, the proposed relation is described and discussed in some details. The geostatic formulas used in Danish engineering practice are mentioned in the appendix.

2 DATABASE

The cases, and thereby the static loading tests from which the relation between time, the regeneration factor, and undrained shear strength is investigated, are listed in Table 1.

As indicated in Table 1, 7 cases are at present time included in the database. In total 12 piles have been subjected to static compression tests and 1 of the them has been tested more than ones. All the piles are squared concrete piles. The side length varies between 0.2m and 0.4m whereas the embedded length varies between approximately 8m and 31m. Furthermore, the piles are located onshore and the thickness of the clay layers varies at the different locations. In connection with the static loading tests all piles did not fail and the time duration between testing and pile driving lies in the interval 13 – 11600 days. In addition, one of the tests was performed as a Constant Rate of Penetration test (Aalborg H. for the 0.3×0.3 m pile) and the others as tests with stepwise increase in load. The influence of testing procedure in connection with static loading tests on the stress-strain curve for the

pile and the soil that surrounds is not taken into account. For details on this subject for piles located in clay, see Bergdahl & Hult (1981).

Table 1 Overview of the cases used. # piles and # tests denote the total number of piles and the number of tests performed on each pile, respectively. The column “soil” shows the total length of the part of the pile that is located in clay in percentage of the total embedded length. Furthermore, the type of soil that surrounds the tip is indicated. Failure, Comp./Tension and time state if failure occurred during testing, if the pile was loaded in compression or tension and the time between pile driving and testing, respectively. Clay covers soil types with marked cohesive tendencies.

Case	# piles/ # tests	Cross Section [m]	Embedded length [m]	Soil [% Clay/ Tip]	Failure	Comp./ Tension [C or T]	Time [days]
Budolfi	3/1 or 2	0.2×0.2	12.0 – 13.6	50/sand	Yes		22
				91/clay	Yes	C	14/9399
				46/sand	No		18
Fynsværket	2/1	0.3×0.3	25.9 – 26.4	84/clay	No	C	26
					No		27
Fynsværket	2/1	0.35×0.35	25.8 – 29.0	84/clay	No	C	11600
					No		11600
Hanstholm	1/1	0.3×0.3	11.4 – 16.8	39/sand	No	C	13
Aalborg H.	1/1	0.3×0.3	30.8	37/gravel	Yes	C	15
Aalborg H.	1/1	0.35×0.35	28.0	41/gravel	Yes	C	20
Århus Ø.	2/1	0.4×0.4	8.3 – 15.5	63-80/clay	No	C	29
					No		79

3 REGENERATION FACTOR AS FUNCTION OF TIME AND UNDRAINED SHEAR STRENGTH

In this section a relation between the regeneration factor r , time t , and undrained shear strength c_u (determined by means of field vane test), is discussed. The relation is based on experimental load-settlement curves associated with the cases in the database and an approach to reproduce load-settlement curves proposed by Vijayvergiya (1977). First, the assumptions applied in this study related to the Vijayvergiya method are discussed.

3.1 Vijayvergiya method for reproducing measured load-settlement curves

Vijayvergiya (1977) proposes a method to reproduce load-settlement curves for piles by means of equations describing the mobilization of the skin friction and the

base resistance. The method is combined with the geostatic formulas given in DS 415 (1998). Furthermore, the formulas are used to calculate the maximum skin friction and the base resistance. The method is also capable of extending the load-settlement curves for piles, which have not been loaded to failure in connection with a static load test and thereby determine the ultimate resistance.

Vijayvergiya (1977) states that the maximum skin friction and the base resistance are mobilized at a critical movement z_c of the pile skin and the pile base, respectively. The skin friction is fully mobilized at a critical movement $z_{c,skin}$ of 0.2 – 0.3 inches (Vijayvergiya, 1977), whereas Tomlinson (1994) states that $z_{c,skin}$ is 0.3 – 1.0% of the pile diameter. In this analysis $z_{c,skin} = 4\text{mm}$ is used, which is approximately the average of the values stated above for piles normally used in Denmark. In this study it is assumed that the base resistance is fully mobilized at a critical movement $z_{c,base}$ equal to 5% of the pile diameter. For comparison, Vijayvergiya (1977) and Tomlinson (1994) for example postulate that $z_{c,base} = 4 - 6\%$ and $z_{c,base} = 10 - 20\%$ of the pile diameter, respectively.

In DS 415 (1998) the regeneration factor r , the bearing capacity factor N_m , and the material factor m associated with the geostatic formulas have predefined values (see Appendix). In this study, the values of r and N_m for each layer along the pile shaft are fitted in such a way that the calculated load-settlement curve for a given pile by using Vijayvergiya's method is identical to the load-settlement curve obtained by a static load test performed on the same pile. In this way, different values for r and N_m are obtained for each layer along the pile and the relation between undrained shear strength of the different layers surrounding the pile, the regeneration factor, and time, can be studied.

Vijayvergiya's method has been demonstrated by Jensen (2004); Sørensen & Jensen (1996) and Mosher & Dawkins (2000).

3.2 Regeneration factor vs. Undrained shear strength and time.

By using Vijayvergiya's method on every case mentioned in Table 1, it has been possible to investigate the influence of time and undrained shear strength on the regeneration factor. The results are illustrated in Figure 1.

In Figure 1, every single curve represents the fitted regeneration factor for every single cohesive soil layer surrounding the pile as a function of the undrained shear strength for that given layer. That is, the accentuated points (points marked with \blacktriangle , \blacksquare , \times or \circ) on each curve symbolize the regeneration factor for a given layer with a certain undrained shear strength at a given time after installation of a given pile. Furthermore, the results are divided into time categories depending on the time duration between pile driving and testing. For example, in the case "Budolfi" the undrained shear strengths for the cohesive soil layers surrounding one of the piles

are 25, 110, 120, 140 and 225kPa, respectively. The corresponding regeneration factors (estimated by reproducing the load-settlement curve for the pile by means of Vijayvergiya's method) are 1.10, 0.92, 0.90, 0.88 and 0.85. The pile was tested statically 9399 days after the installation.

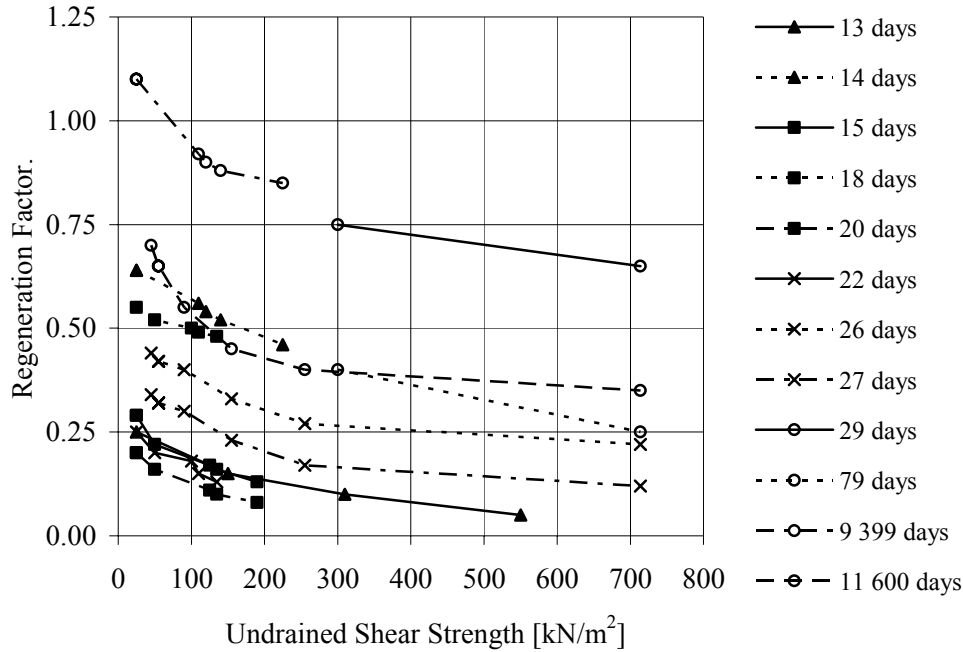


Figure 1 The regeneration factor vs. the undrained shear strength. Time is divided into $t \leq 2$ weeks (\blacktriangle), $2 \text{ weeks} < t \leq 3$ weeks (\blacksquare), $3 \text{ weeks} < t \leq 4$ weeks (\times), $t > 4$ weeks (\circ). (After Jensen, 2004)

For each time interval the results presented in Figure 1 have been fitted with a curve and depicted in Figure 2. It appears that the curves for the different time intervals are similar in shape. Furthermore, Figure 1 and Figure 2 indicate that the regeneration factor increases when the undrained shear strength decreases. In addition, it seems like the regeneration factor increases with time. If the pile is left untouched for 4 weeks or more, the regeneration factor will exceed 0.4 for undrained shear strengths below 714 kPa. In that case $r = 0.4$ as proposed by DS 415 (1998) may be too conservative.

The tendencies illustrated in Figure 2 can approximately be described by the following equation:

$$r = 2.31c_u^{-0.26} \left(\frac{t}{t_{\text{ref}}} \right), \quad t \leq 70 \text{ days} \quad (1)$$

where t is time in days, c_u is the undrained shear strength determined by means of field vane test, and t_{ref} is the reference time. The reference time t_{ref} is found to be

70 days in order to get the best fit. The curves in Figure 2 do almost coincide with curves based on Eq.(1) for $t = 14, 21, 28$ and 70 days, respectively.

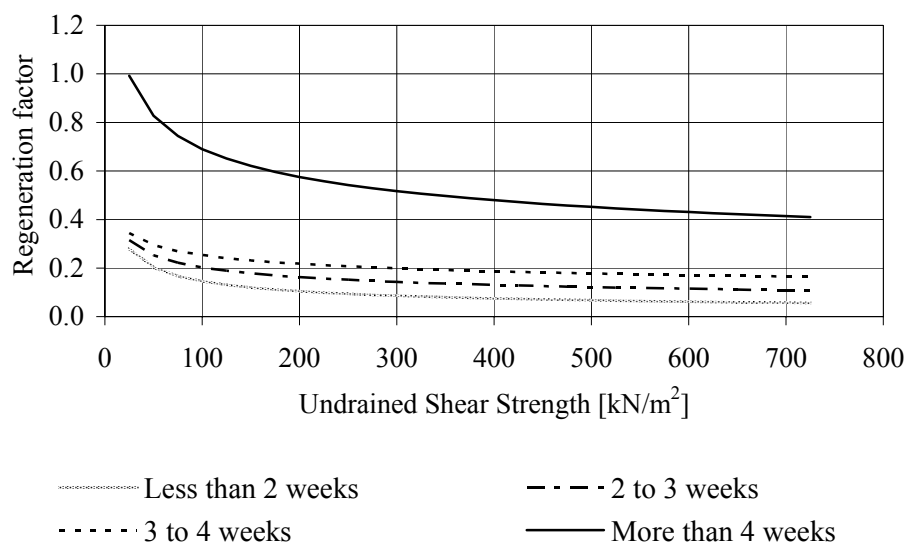


Figure 2 The regeneration factor as a function of undrained shear strength and time. (After Jensen, 2004)

4 DISCUSSION

The above mentioned results should be viewed upon as preliminary investigations on how to use existing data to develop a relation between time, strength and capacity of a pile. In the future, more tests should be subjected to such an analysis in order to calibrate and verify the model. If it turns out that the relation described by Eq.(1) is able to describe the regeneration factor as a function of time, it may imply an economical benefit if the piles can be left untouched for more than 4 weeks.

Some uncertainties and limitations are associated with the analysis of the cases in Table 1. First, the regeneration factor includes effects of pore pressure dissipation and “true” time effects. Since the drainage conditions for the soils associated with the different cases are not the same, the magnitude of the regeneration factor will vary from case to case even though the dimensions of the piles, the time for testing, and the undrained shear strengths of the layers considered, are the same. Secondly, failure did not occur in all the tests associated with the cases in Table 1. Therefore, some uncertainty is connected to the reproduction of these load-settlement curves and thereby the estimation of the regeneration factor. In addition, in the analyses it is chosen that the skin friction and the base resistance for every pile segment are fully mobilized corresponding to deformations equal to 4mm and 5% of the pile diameter, respectively (section 3.1). It can be discussed whether these values are “correct”. It seems like the values of the critical deformations do not affect the magnitude of the regeneration factor for the different cohesive soil layers. Thirdly, piles driven within close distance of the tested pile may influence the regeneration factor. This has not been taken into account, but it could be the case in connection

with “Fynsværket”, where the distance between the test piles and the surrounding piles are approximately 1.6m. As mentioned earlier, the type of test (Constant Rate of Penetration or tests with stepwise increment in load) influences the load-settlement curve and thereby the regeneration factor. It should also be mentioned that Eq.(1) is based on undrained shear strengths measured by field vane test. At last, pore pressures were not measured during driving, i.e. whether negative or excess pore pressures were present during driving is not known. This might also affect the magnitude of the regeneration factor at a given time after pile driving and thereby Eq.(1).

5 CONCLUSION

This paper deals with the influence of time and undrained shear strength on the regeneration factor, which is the quantity in connection with Danish engineering practice that accounts for the effects of time on the bearing capacity of an axially loaded pile. 7 cases (13 static pile load tests) have been investigated by means of the approach by Vijayvergiya (1977) and the geostatic formulas used in Danish engineering practice. A relation (Eq. (1)) between time, undrained shear strength and the regeneration factor has been found. If a pile is left untouched for approximately 4 weeks after pile driving, it seems like the Danish Code of Practice for foundation engineering (DS 415, 1998) are too conservative compared to the relation (Eq.(1) and Fig. 2) proposed in this paper regarding the magnitude of the regeneration factor. That is, the proposed relation may imply an economical benefit in the design of pile foundations.

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APPENDIX

In Denmark the bearing capacity of a single axially loaded pile can be determined by means of geostatic formulas as given in DS 415 (1998). The characteristic value of the total bearing capacity R_{ck} is calculated as the sum of the skin friction R_{sk} and the toe resistance R_{bk} :

$$R_{ck} = R_{bk} + R_{sk} \quad (2)$$

The characteristic value of the skin friction R_{sk} is calculated by means of empirical equations:

$$R_{sk} = \sum_{i=1}^n q_{sik} A_{si} \Rightarrow q_{sik} = \frac{1}{1.5} m r c_u \quad \text{for cohesion soil} \quad (3)$$

$$q_{sik} = \frac{1}{1.5} N_m q'_m \quad \text{for cohesionless soil}$$

where q_{sik} is the characteristic value of the skin friction per unit area in the i^{th} layer, A_{si} is the pile skin area in the i^{th} layer, m is a material factor, r is the regeneration factor, c_u is the undrained shear strength, N_m is a bearing capacity factor, and q'_m is the effective vertical stress at the middle of the i^{th} layer.

The characteristic value of the toe resistance R_{bk} is:

$$R_{bk} = q_{bk} A_b \Rightarrow q_{bk} = \frac{1}{1.5} 9 c_u \quad \text{for cohesion soil} \quad (4)$$

where q_{bk} is the characteristic toe resistance per unit area and A_b is the cross area of the pile toe. In till with cohesive tendencies, the factor 9 can be increased to 18.

In cohesionless soils, it is recommended not to use the geostatic formulas to calculate the toe resistance, but to use the Danish Driving Formula.

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