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Impact of installation on the recovery of the bearing capacity of displacement piles in sensitive clay

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Abstract. Deep foundations are essential for the soft soil deposits that are ubiquitous in the Nordic countries. Precast driven displacement piles are commonly used, due to the ease of installation and their low cost. The stiffness and strength of the clay surrounding the piles will change during and after the pile installation process. This paper presents an effective stress based numerical framework that elaborates the pile set-up and pile ageing mechanisms observed empirically. For the trial case studied, it is demonstrated that the magnitude of the increase in axial bearing capacity over time is strongly linked to the pile installation stage. Furthermore, it is found that classic effective stress based soil mechanics concepts readily describe the observed behaviour.

1. Introduction

Deep foundations form a necessary foundation solution in regions with soft soils, such as the sensitive clays that are common in the Nordic countries. Pre-cast driven piles remain economical due to controlled (mass) production and ease of installation. The large shear distortions from pile driving combined with the low hydraulic conductivity of clays, however, lead to significant soil disturbance and excess pore water pressures that subsequently dissipate. The generation of excess pore water pressures during constant volume mass-displacement and their subsequent dissipation lead to additional volume change over time. Consequently, the stiffness and strength of the clay surrounding the pile is evolving as well. Finally, additional creep might be triggered in the soil, especially in sensitive clays that have a large degree of initial bonding [1]. Within the discipline the increase in pile capacity over time after pile installation is referred to as ‘setup’ [2].

Time-dependent effects for foundation elements such as piles are often studied separately from the soil behaviour. This leads to a multitude of interpretations on the underlying mechanisms that underpin the observed pile head response over time. Alternatively the mechanisms are simply ignored altogether and empirical relations to quantify the increase in bearing capacity with time due to pile setup effects are formulated instead. Some of the most important contributions regarding the topic of pile setup, and pile ageing comes from Scandinavian researchers, *i.e.*, respectively from Denmark, Norway and Sweden [3, 4, 5]. These are mostly derived from pile tests where the same pile is loaded several times within a certain time period (staged testing) or different piles are loaded once after certain time (unstaged testing). The ‘setup’ phase where the pile capacity is evolving includes, (i) the dissipation of excess pore



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water pressures and associated volume change, hence recovery of effective stress and (ii) the change in strength independent of changes of effective stress. These mechanisms have different time scales, *i.e.*, thixotropy is rather fast [6], whilst creep continues after the dissipation of pore water pressures already has finished. In this subsequent (ongoing) phase stress relaxation and creep mechanisms in the soil surrounding the pile shaft continue. It is difficult to fully capture the pile response from pile load tests only, as those are dependent on pile type, installation method and site conditions.

In this paper an alternative approach will be used where the pile setup and pile ageing mechanisms are linked to the pile installation stage using concepts based on a basic understanding of soil behaviour with advanced modelling techniques. After introducing the method where the Shallow Strain Path Method (SSPM) is combined with an advanced constitutive model for soft soils [7, 8], the applicability of the approach is demonstrated by modelling a case study for time effects on the capacity of pile foundations [4] on a well documented test site with natural sensitive clay [9] using a novel methodology to incorporate pile installation effects in a general purpose Finite Element code [10].

2. Methodology

2.1. Modelling strategy

The proposed modelling strategy overcomes the mesh distortion issues in general purpose small strain finite element codes by decoupling the pile installation problem and the constitutive response of the clay. In the implementation considered here, the Shallow Strain Path Method [11], which is a further evolution of the original Strain Path Method [12] by considering a transient source moving from the free surface into deep layers, is used to calculate the deformations near a pile during installation. In this method only the strain paths for an incompressible flow of soil, *i.e.*, undrained response of clay, around a penetrating object are obtained. Subsequently, the strain paths are used to calculate the complementary increment in effective stress using Creep-SCLAY1S implemented in a strain driver [13]. After obtaining the conjugate pairs of stress and strain for several points adjacent to the pile shaft the stress equilibrium, strain compatibility and consolidation are enforced in a generalised 2D axisymmetric model in COMSOL. All steps are schematically shown in Figure 1, for a more detailed discussion on the method and intermediate results is referred to [10].

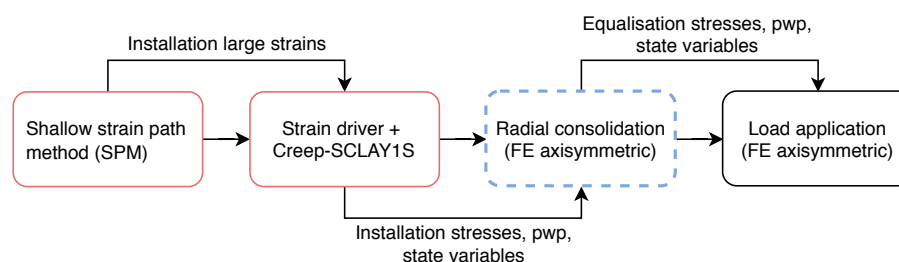


Figure 1. Schematic methodology of modelling steps.

2.2. Constitutive model

Soft sensitive soils have specific features that need to be incorporated in the constitutive model in order to capture the appropriate mechanical response. Here, we follow the work of Karstunen and co-workers who developed the SCLAY1(s) family of models that incorporate anisotropy (strength/stiffness), destructuration (bond degradation) and creep. The rationale and model details of this anisotropic extension of the Modified Camclay model have been discussed in

length elsewhere *e.g.*, [14, 15, 7]. Two advanced main model features and their relevance to the long-term pile response are further discussed below. Those are destructuration and creep (sometimes called rate-dependency). The 3D effective stress model is implemented in a single element strain driver [13].

Creep-SCLAY1S has a third hardening rule that describes the degradation of bonding χ as function of plastic volumetric and deviatoric strains (Equation (1)). It is closely linked to the first isotropic hardening rule in the classic critical state models, *e.g.*, [16]. The relative size of the intrinsic yield surface, *i.e.*, soil without any structure left, is related to the yield surface of the intact soil (natural soil that is not disturbed) with a quantity χ that reflects the amount of bonding (Equation (2)). Therefore, the natural yield surface (which size is governed by the initial in-situ stress state of intact soil) will shrink and the intrinsic surface (which describes the size of the yield envelope of a material without bonds left) will expand or shrink (depending on the flow rule) until there is no bonding left and they have the same size.

$$d\chi = -\xi\xi(\|d\varepsilon_v^p\| + \xi_d d\varepsilon_d^p) \quad (1)$$

$$p'_m = (1 + \chi)p'_{mi} \quad (2)$$

where $d\varepsilon_v^p$ and $d\varepsilon_d^p$ are the volumetric and deviatoric plastic strains, respectively, ξ is the parameter controlling the absolute rate of bond degradation and ξ_d is the parameter controlling the relative bond degradation between the volumetric and deviatoric component of plastic strain.

It becomes apparent that this formulation only incorporates the reduction of strength and stiffness due to plastic strains. In laymen terms it reflects the ‘degree’ of remoulding. Although the current hardening law incorporates the increase in (intrinsic) strength as function of time due to creep effects, it doesn’t explicitly model thixotropic effects. The latter could be an probable addition to include the time dependent effects from thixotropy in the formulation. However, thixotropy is seen as a mechanism of secondary importance compared to the dissipation of pore pressures (hence effective stress build up) and creep.

In addition to anisotropy and structure, soft clays also exhibit viscous behaviour (rate dependency/creep). This additional property incorporates the time dimension in the material description. Creep-SCLAY1S is an anisotropic elasto-viscoplastic constitutive model capable of modelling rate dependency and destructuration [7, 8]. Creep-SCLAY1S is a special extended over-stress model that uses a generalised empirical formulation obtained from one dimensional observations to model the rate dependent behaviour of the soil. As a result the additional viscous parameters can be readily obtained from standard incremental loading oedometer tests.

It is important to mention that the mechanism in the soil adjacent to the pile shaft result mainly in distortion, *i.e.* shear strain. This is something the model automatically resolves without the need for model calibration with data from special deviatoric creep tests.

Including a creep formulation in the modelling response will enable to capture the on-going time-dependent gain in pile capacity beyond the effects of thixotropy (days) and pore pressure dissipation (months), as observed in the field tests [4].

The model is calibrated for the site investigation data of the Onsøy test site leading to the parameters in Table 1.

2.3. Numerical model for field test at Onsøy

The SSPM and follow up 2D numerical model in COMSOL Multiphysics version 5.3 are designed to capture the normalised increase in tension capacity of the tubular piles installed at the Onsøy field test [4]. Onsøy is a well documented test site with natural sensitive clay (high plasticity and medium sensitivity) [9]. The pile ageing test comprised a series of steel open-ended piles with an outer pile radius R_{pile} of 254 mm a wall thickness t of 6.3 mm and a length of 19 m. The

Table 1. Model parameters for the natural Onsøy clay.

| Parameter | Definition | unit | value |
|---------------|--|------|-------|
| λ_i^* | Modified intrinsic compression index | [-] | 0.076 |
| κ^* | Modified swelling index | [-] | 0.011 |
| ν | Poisson's ratio | [-] | 0.15 |
| M_c | Stress ratio at critical state in triaxial compression | [-] | 1.23 |
| M_e | Stress ratio at critical state in triaxial extension | [-] | 0.80 |
| ω | Rate of rotation | [-] | 200 |
| ω_d | Rate of rotation due to deviator strain | [-] | 0.56 |
| ξ | Rate of destructuration | [-] | 10 |
| ξ_d | Rate of destructuration due to deviator strain | [-] | 0.30 |
| OCR | Over-consolidation ratio | [-] | 1.1 |
| e_0 | Initial void ratio | [-] | 1.80 |
| α_0 | Initial anisotropy | [-] | 0.47 |
| χ_0 | Initial amount of bonding | [-] | 10 |
| μ_i^* | Modified intrinsic creep index | [-] | 0.005 |
| τ | Reference time | [d] | 1 |

capacity, in tension, as function of time was tested on independent piles, to isolate the ageing effects at the pile-soil interface without the impact of the pile base distorting the measurements at the pile head. Therefore, the measured increase in pile head capacity in tension reported is directly related to the increase of undrained shear strength in the soil adjacent to the pile shaft.

The SSPM equations are solved for the closed ended pile with radius R_{pile} and for the open ended pile (unplugged) with outer radius R_{pile} and wall thickness t . The domain is chosen sufficiently large at $200 R_{pile}$. The open-ended piles plugged during pile installation, as a result a weighted average of the results for the open-ended and close-ended solution for the SSPM (and follow up analyses) using a reported plugging ratio of 0.2 is used in the analyses. A steady state solution of the strain paths is reached with SSPM for a penetration depth of 5 m. Subsequently, Creep-SCLAY1S is used in the strain driver to obtain the complementary stress increment with the model parameters reported in Table 1 before solving the 2D axi-symmetric consolidation and creep problem. Hence the increase in strength is calculated for a slice of soil near the pile shaft at a depth of 7 m. This is the depth for which most high quality laboratory data on block samples is available.

3. Results

3.1. Development of effective stress

Figure 2 shows the mean effective stress p' and excess pore water pressures Δu as function of time for a point in the clay at a depth of 7 m and a distance of $1 R_{pile}$. The data starts directly after pile installation and is normalised with the far field value for the mean effective stress p'_0 . This value represents the mean effective stress at rest at a depth of 7 m is not influenced by the pile installation. Hence, at the start of the pile setup (and pile ageing) stage directly after pile installation the excess pore water pressures, sometimes called pressurisation is ca. 120% of $p'_0 = 35.3$ kPa. Simultaneously, p' is 40% lower than prior to pile installation. Another interesting observation is that initially Δu increases sharply to $1.6p'_0$ in the first 30 min before dissipation starts. This is the well known Mandel-Cryer effect which can be significant in soils with Poisson ratios ν that fall below 0.2 [17]. The Mandel-Cryer effect is only fully resolved in the analyses when using the complete 3D formulation of the Biot consolidation equations and the particular boundary conditions with only the far-field radial boundary being open. Furthermore, for the simplified case of radial consolidation the total duration for the dissipation of Δu is ca. 120 d. This value agrees well with experience from piling works in soft soils in Scandinavia.

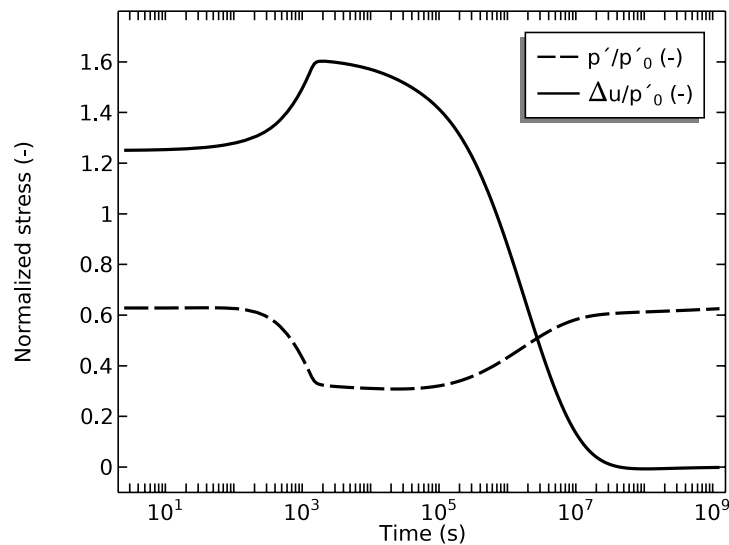


Figure 2. Evolution of normalised excess pore water pressures Δu and mean effective stress p' . p'_0 is the far field mean effective stress that is unaffected by pile installation.

3.2. Evolution of strength

Figure 3 plots the predicted (line) and measured (points) [4] increase in shaft capacity. Each data point represents a new pile load test on an untested pile. The reference time $t_{ref} = 30$ d is taken similar to the field measurements. The numerical results is the weighted average from the open-ended and closed-ended SSPM, using the plugging ratio reported. Given the somewhat crude method the agreement with the measured values is very good. The differences remaining are associated to simplifications in the modelling geometry (unit length in simulations, full pile length in the measurement data from the field test) and variations between the various piles tested.

The numerical results allow further investigation into the effects of pile installation method, *i.e.*, comparing the increase in capacity Q_{sf} between open-ended and close-ended piles (full displacement piles commonly used in soft soils). Figure 4 shows the increase in shaft capacity over time as function of installation method. Furthermore, the results are normalised with the shaft capacity for intact clay $Q_{sf;0}$, *i.e.*, the strength of the clay prior to pile installation. An alternative interpretation of the y-axis of Figure 4 is the α used in total stress pile design methods, *e.g.*, [2].

These unique results clearly show that directly after installation the shaft capacity reaches a minimum. Subsequently, the shaft capacity recovers as a function of the dissipation of excess pore water pressures, and continuous to decrease at a lower rate after the excess pore water pressures are fully dissipated (3 months for the closed ended pile). An important notion is that for the sensitive clay studied, the increase in pile capacity is primarily a recovery of strength lost during pile installation. Pile systems that disturb the soil least also have the smallest strength recovery, but have a higher capacity. Furthermore, values of 1 are reached after 100 months for the open-ended pile, whereas the close-ended pile never fully recovers a shaft capacity that reflects the intact strength of the clay. This finding indicates that for maximum utilisation of the bearing capacity in sensitive clays the disturbance from pile installation should be minimised, so that a larger long-term strength ratio $Q_{sf}/Q_{sf;0}$ is maintained. On the contrary, pile systems that disturb the surrounding soil less might be more sensitive to strength degradation in subsequent (cyclic) loading.

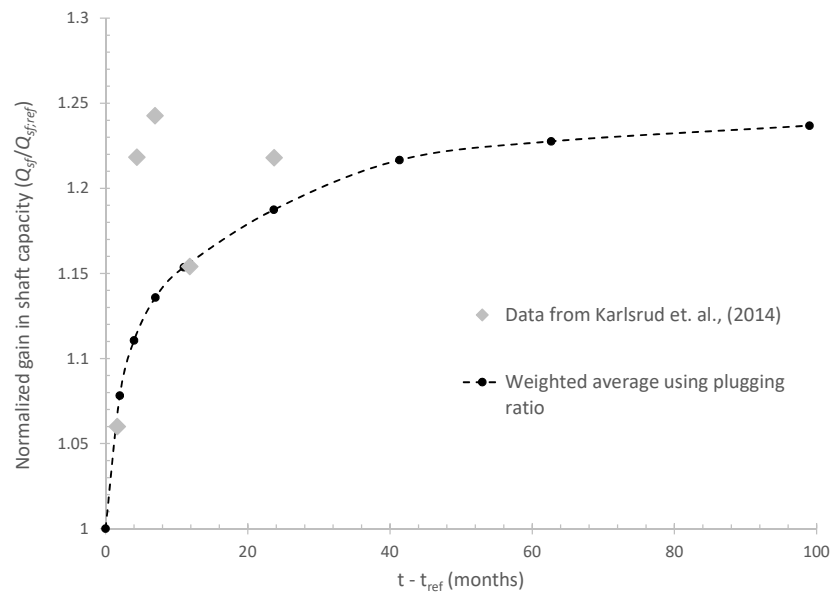


Figure 3. Evolution of normalised increase in shaft capacity as function of time after pile installation. $Q_{sf;ref}$ is the shaft capacity one month after pile installation.

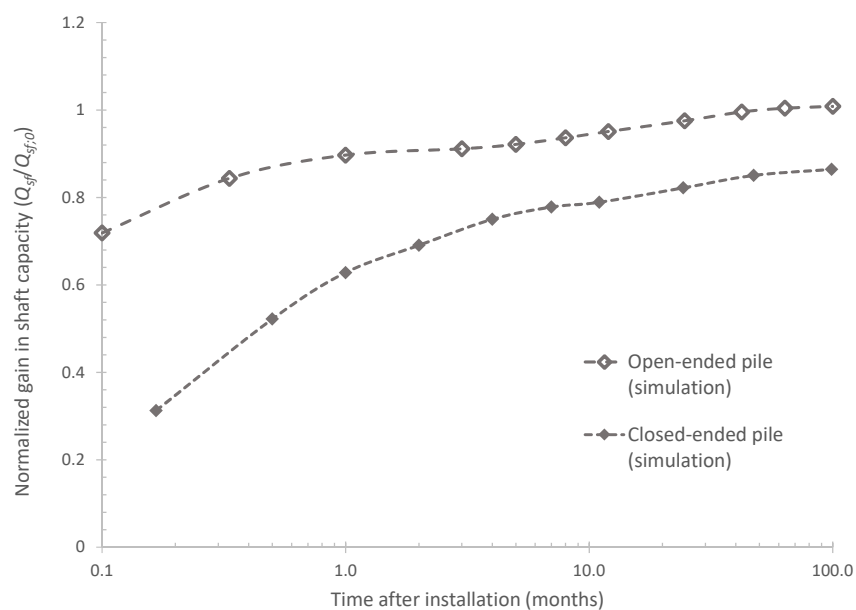


Figure 4. Simulated increase in shaft capacity Q_{sf} with time. The data is normalised with shaft capacity for intact clay $Q_{sf;0}$ corresponding to the intact strength of the clay before pile installation.

4. Conclusions

The effects of pile ageing, the ongoing increase in shaft capacity after pile installation and the equalisation of excess pore water pressures in soft soils can be modelled successfully by combining the Shallow Strain Path Method (SSPM) to model pile installation with Creep-SCLAY1S to include advanced soft soil features in the analyses. The results for a well documented case study demonstrate that a large part of the pile ageing, on-going increase in pile capacity, is governed by the creep in the soft soil surrounding the pile. The increase in the shaft capacity is best described as a strength recovery process towards the original intact strength before the pile is installed into the soil. Finally, similarly to pile setup effects that are linked to an increase of effective stress, resulting from the dissipation of excess pore pressures, pile ageing is also affected by other factors in the soft soil. Those include the degree of remoulding (as proxy for soil disturbance), hence destructuration during pile installation and the anisotropy. Finally, it is shown that the degree of strength reduction during pile installation is directly linked to the magnitude of the subsequent increase in shaft capacity over time. Pile systems that disturb the pile to a smaller degree during installation preserve more strength, which in turn leads to smaller strength recovery over time.

Acknowledgments

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