Numerical Simulation of the Installation of Jacked Piles in Sand with the Material Point Method

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Numerical Simulation of the Installation of Jacked Piles in Sand with the Material Point Method


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Abstract

Pile installation has a great impact on the subsequent mechanical pile response. It, however, is not routinely incorporated in the numerical analyses of deep foundations in sand. Some of the difficulties associated with the simulation of the installation process are related to the fact that large deformations and distortions will eventually appear. The Finite Element Method is not well suited to solve problems of this nature. Hence, an alternative procedure is tested herein, by using the Material Point Method to simulate the installation of statically jacked or pushed-in type piles, which has successfully demonstrated its capacity to deal with this simulation. Two constitutive models were also tested, i.e. the Modified Cam Clay (MCC) and the Subloading Cam Clay (SubCam), allowing a clear perception of the great advantage to consider the soil with the SubCam model. The simulations have indeed reproduced some of the important features of the pile installation process, as the radial stress acting around the pile’s shaft or the shaft’s lateral capacity, among other issues. The numerical results were additionally compared to known (semi empirical) methods to derive the lateral capacity of the shaft, with a good and practical outcome.

Keywords: pile installation process, numerical modelling, material point method, subloading cam clay, granular soil.
Introduction

In Brazil, typical calculations for the pile foundation capacity are done via empirical techniques. The effects that pile installation introduce to its capacity have already been studied by Randolph et al. (1979) by using the Cavity Expansion Method. These effects can produce changes in the properties and state of the soil, modifications on the lateral and vertical stresses and therefore changes in the capacity and the stiffness of the pile-soil system. Although this problem is well known, there is still a great deal of empiricism in the estimations of pile installation effects, hence some further experience is needed to do an accurate evaluation of changes in pile load capacity (Doherty and Gavin 2011). Pile installation effects are particularly important for displacement piles, where the soil is radially and vertically displaced creating a total displaced volume of soil of, at least, the nominal volume of the pile. This displacement invariably causes large variations in soil stresses around its shaft. According to Randolph (2003) any scientific approach to predict the shaft capacity of a driven (as well as a pushed-in type) pile must consider the changes that occur during installation, as well as equalizations of excess pore pressures in the soil and the loading phases of the foundation.

Some empirical equations have already been proposed to account for the installation effects of driven piles. Figure 1 presents the idealized exponential profile of shaft’s friction proposed by Randolph et al. (1994). Based in this assumption the authors obtained Eq. (1) to estimate the ratio between the horizontal effective stress after the installation of the pile \( \sigma_f \) and the initial vertical effective stress \( \sigma_{vo} \). This ratio is expressed as a coefficient \( K \) that varies along the depth, as follows:

\[
K(z) = K_{\text{min}} + (K_{\text{max}} - K_{\text{min}}) e^{(-z/d)}
\]  

(1)
\( \kappa_{\text{max}} \) can be estimated as 2\% of the normalized cone resistance \( q_c/\sigma_v' \) for closed-ended pile and 1\% of \( q_c/\sigma_v' \) for open-ended piles (Fleming et al. 2009). \( \kappa_{\text{min}} \) can be linked to the active earth pressure coefficient, \( \mu \) is a coefficient that controls the rate at which the maximum shaft friction is degraded, \( L \) is the embedded length of the pile and \( h \) is the vertical distance from the pile tip to the point of analyses at a depth \( z \).

More recently Jardine et al. (2005) suggest an equation to directly calculate the horizontal effective stress after the installation of a driven pile in sand, as follows:

\[
\sigma_v' = 0.029 q_c \left( \frac{\sigma_v'}{p_a} \right)^{0.13} (h/R)^{0.38}
\]  

(2)

Where \( p_a \) is the atmospheric pressure and \( R \) the radius of the pile.

In general, the effects that the installation of piles introduce are intrinsically related to the displacement of the soil around the pile; the modification of the soil’s properties, and the generation of pore pressures (Gue 1984; Randolph et al. 1979; Slatter 2000). Phenomena such as “friction fatigue” and aging that affect the behavior of the pile are extremely influenced by the installations process (White and Lehane 2004; Zhang and Wang 2009; Zhang and Wang 2014).

For complex geotechnical conditions or initial conditions that are different from the empirically based methods, numerical modeling is mandatory. However, generic finite elements geotechnical codes which incorporates Lagrangian formulations are unsuitable to simulate these types of problems (Dijkstra et al. 2011), given the large strains and deformations that occur around driven piles during the installation process. Numerical methods that combine the Lagrangian and Eulerian descriptions of motion have been preferred for the simulations of penetration problems (Dijkstra et al. 2011; Nguyen et al. 2014; Sheng et al. 2009; Zhang et al. 2014). Very recently the MPM has also been used to
simulate pile installation phenomena. Phuong et al. (2014, 2016) used the MPM with a special approach called a Moving Mesh to simulate a pushed-in pile installation in a hypoplastic soil model. Hamad (2016) used an extended version of the MPM known as Convected Particle Domain Interpolation (CPDI) for dynamically installation of a driven pile also in a hypolastic soil model.

In the present paper, simulations of the installation process of jacked (pushed-in) piles were done using the Material Point Method (MPM) and two critical state models. For all numerical simulations, the NairnMPM code where no water pressure generation is allowed or considered was used (Nairn 2011). In the end, a parametric analysis is done to evaluate the influence of some key parameters on the derived response of the soil. Finally, an evaluation of considering, or not, the pile installation process in the assessment of the ultimate shaft resistance is done.

**Material Point Method and Constitutive models**

**Brief description of MPM**

In recent years, the finite element method (FEM) has become the standard tool for solving problems in solid mechanics. Nevertheless, this method, in its traditional Lagrangian formulation, is not suitable for the analysis of large deformation problems (Bardenhagen et al. 2000; Wieckowski 2004). According to Beuth et al. (2007) by using this formulation great distortions of the mesh can occur and remeshing may be needed.

To solve issues with the FEM in the simulation of large deformation problems, meshless methods have been developed. For these methods, the generation of the mathematical space (where the governing equations of the problem will be solved) reduces to the generation of material points and its distribution, without any fixed connectivity between them, as in the
FEM. Examples of such methods are the discrete element method (DEM), the smooth particle hydrodynamics (SPH) and the particle in cell method (PIC) (Zabala 2010).

The Material Point Method is a type of PIC (Sulsky et al. 1994). This method combines ideas and procedures of the particle methods and FEM together, and uses the potential of the Lagrangian and Eulerian descriptions of kinematics. With the MPM, a body is modeled as a group of Lagrangian particles. These particles transport the state and other variables needed to solve the problem’s governing equations (e.g. momentum equations). The variables (defined by such equations) are interpolated from particles to the nodes of a fixed mesh (as in FEM) in which the governing equations are solved. After obtaining the overall solution, it is interpolated back to the particles, allowing the state variables and positions to be updated. This procedure is repeated along the whole time domain of the problem, so that, in this way, the fixed mesh has no distortion (Bardenhagen and Kober 2004; Solowski and Sloan 2015).

In the last decade, a generalization to the MPM was done to mitigate numerical noise that arises when particles cross from one cell to another. This method is known as the Generalized Interpolation Material Point (GIMP) and it was introduced by Bardenhagen and Kober (2004). New interpolation shape functions are introduced over each particle’s domain, which may intersect several cells of the mesh, allowing the tracking of any particle when it goes out of its original cell domain. More recently, a modification was introduced to the GIMP method to solve problems that appear when large distortions and tensions occur. This new technique is known as the Convected Particle Domain Interpolation (CPDI) as presented by Sadeghirad et al. (2011).

The numerical code used in the present paper was developed by Nairn (2011) and is denoted as NairnMPM. It allows the establishment of dynamic 2D and 3D analyses, as well as the use of GIMP and CPDI methods. It also allows the addition of a Coulomb friction between
bodies with two contact conditions, one proposed by Bardenhagen et al. (2000), necessary but
not sufficient to define the contact, and other proposed by Lemiale et al. (2010) that
complements the first one. For the simulations done in this research, some models were run
using both CPDI and GIMP versions of the MPM. Negligible differences in the results were
observed when using GIMP or CPDI versions. The CPDI version however consumed more
computational time. Thus, for all the simulations the GIMP method was used.

Brief description of Modified Cam Clay model

The elastoplastic theory is the largest source of constitutive models that are applied nowadays
to geomechanics (Zdravkovic and Carter 2008). One of the most used models based on
critical state is the Modified Cam Clay (MCC) model. This model is relatively simple and
captures reasonably well the main features of the behavior of normally consolidated clays.
Only a few parameters are needed and they are easy to be assessed.

The plastic multiplier for this model is expressed as:

$$\lambda_p = \frac{\frac{\partial f^T}{\partial \sigma} : D^e : d\varepsilon}{\frac{\partial f^T}{\partial \sigma} : D^e : \varepsilon^p - \frac{\partial f}{\partial \sigma} : \varepsilon^p : tr\left(\frac{\partial f}{\partial \sigma}\right)}$$

where \(\varepsilon^p\) is the plastic volume deformation, \(tr\) represent the trace of the entity, \(D^e\) the
elastic constitutive tensor, \(\sigma\) is the Cauchy stress tensor, \(f\) and \(g\) are respectively the yield and
plastic potential.

In spite the fact that the MCC was developed based on tests on clays, very loose and dense
sands also show qualitatively the same stress–strain–dilatancy characteristics as those of
normally and over consolidated clays (Nakai 2013). Thus, for problems where some specific
characteristics of sands, such as liquefaction and particle breakage, are not substantially important, the MCC can be used.

Nevertheless this model is unable to describe some important features of the soil behavior, such as a positive dilatancy during strain hardening, or cyclic loading, or the influence of both density and confining pressure on the soil’s deformation and strength characteristics (Pedroso et al. 2005).

Subloading Cam Clay model

In its original form, the MCC model yields a non-linear elastic behavior under both unloading and reloading paths for overconsolidated clays. However, “real” soils show elastoplastic behavior even in the overconsolidated region (Hashiguchi and Ueno 1977; Nakai 2013). To solve this misconception Hashiguchi and Ueno (1977) introduced the subloading surface concept. This is basically a new surface internal to the yield one which has the same geometrical form. In the stress space, the actual stress point is always on the subloading surface and the evolution of this surface depends on the elastoplastic transition. The models introduced with this concept are known as surface loading type models (Hira et al. 2006). In this way, the yield surface is no longer used to separate elastic and elastoplastic regions. The yield surface acts as a bounding surface and the distance between the yield and the subloading surface defines a parameter that affects the value of the plastic multiplier. When the yield and subloading surfaces match, the plastic multiplier will have the same value as if calculated by the original MCC model, nevertheless, if they do not match, the plastic multiplier will have a lower value that depends of the distance between the surfaces.

The incorporation of the subloading concept to a particular model introduced a new internal variable (ρ) that defines the size of the subloading surface. In the case of sands, it is a measure of the relative density, whereas in the case of clays, it is associated with the
overconsolidation ratio (see Figure 2). The introduction of this variable to a conventional elastoplastic model improves its capacity to reproduce the cyclic behavior of materials, and the transition from elastic to elastoplastic states (Nakai 2013).

The SubCam model proposed by Pedroso (2006) introduces the subloading surface to the well-known MCC. This model considers the influence of density and confining pressure on the deformation and strength of soils and according to Farias et al. (2005); Farias et al. (2009); and Pedroso et al. (2005) it can be used for clay and sands. The SubCam model has been successfully used in finite elements codes for the simulation of sand problems (Farias et al. 2005).

The new internal variable \( \rho \) becomes null when the stress path reaches the normal compression line (NCL). It is therefore related to the overconsolidation ratio as follows:

\[
\rho = (\lambda - \kappa) \ln \left( \frac{p_0}{z_0} \right)
\]  

(4)

where \( z_0 \) is the interception of the subloading surface with the isotropic compression axis, \( \lambda \) is the slope of the normal compression line in the \( \ln p' \) versus void ratio \( (e) \) space, \( \kappa \) is the slope of the unloading-reloading line in the same space \( (\ln p' \text{ vs. } e) \), and \( p_0 \) is the pre-consolidation stress.

The same flow rule defined for the yield surface in the MCC is used in the SubCam. On the other hand, for the subloading surface another flow rule is defined that considers a strain variable known as the subloading plastic strain \( (\varepsilon_v^{p(SL)}) \).

\[
dz_0 = \frac{z_0 (1 + e_0)}{(\lambda - \kappa)} (d \varepsilon_v + d \varepsilon_v^{p(SL)})
\]  

(5)

According to Nakai and Hinokio (2004), \( \varepsilon_v^{p(SL)} \) can be obtained as follows:
where $G(\rho)$ is a function that controls the degradation of $\rho$. This function was proposed by Nakai and Hinokio (2004) to be $G(\rho) = a \delta$. The parameter $a$ is a unique and new parameter added by this model to the MCC model, and can be obtained by a calibration procedure with oedometric tests. The value of $a$ influence in the curvature of the oedometric path near the preconsolidation stress (Nakai 2013).

Considering the consistency condition and $\varepsilon_{\text{v,p}}$, the plastic multiplier for the SubCam model can be obtained as:

$$
\lambda_p^{SL} = \frac{\frac{\partial f^T}{\partial \sigma} : D^p : d\varepsilon_p}{\frac{\partial f^T}{\partial \sigma} : D^p : \frac{\partial f}{\partial \sigma} \frac{1}{\zeta_0} (\lambda - \kappa) \left[ \frac{\partial f}{\partial \sigma} \right]} + L
$$

$$
L = a \frac{(\lambda - \kappa) \ln \left( \frac{p_0}{\zeta_0} \right)}{p}
$$

where $L$ is an auxiliary function.

The consistency condition, in this case, is applied to the subloading surface and not to the yield surface as normally done in elastoplastic models.

If the derivative related to the internal variables are replaced in equations (3) and (7), the plastic multipliers are expressed as:

$$
\lambda_p = \frac{\frac{\partial f^T}{\partial \sigma} : D^p : d\varepsilon_p}{\frac{\partial f^T}{\partial \sigma} : D^p : \frac{\partial f}{\partial \sigma} + \left( M^2 p_0 \frac{1}{\zeta_0} (\lambda - \kappa) \right) \left[ \frac{\partial f}{\partial \sigma} \right]}
$$
where $M$ is the slope of the critical state line.

As one can readily notice in equations (9) and (10), the only difference between them is the function $L$. When the subloading surface reaches the yield surface the value of $L$ becomes zero and the SubCam model behaves as an MCC model. When the parameter $a$ is very small, the plastic multiplier becomes very large and plastic strains appear at the onset of loading.

The incorporation of the variable $\rho$ in the SubCam model improves the behavior of the critical state model in the transition from elastic to elastoplastic behavior. This transition is soft and not sharp as in the MCC case, resulting in better results in boundary problems.

The SubCam model uses the failure criterion of Matsuoka and Nakai (Matsuoka and Nakai 1985). In this form, the triaxial compression behavior of the model is different from the triaxial extension case, taking into account the differential strength of the soils along extension or compression paths.

The MCC and SubCam models were implemented in the NairnMPM code and are used in the numerical analyses to be presented next. Some simulations of laboratory experimental tests with different stress-strain paths were already published by the authors to show the advantages of considering, or not, the SubCam model as replacement for the MCC one (Lorenzo et al. 2015). Pedroso (2006) and Pedroso et al. (2005) presents results of simulated cyclic triaxial tests using the SubCam model. This results match the experimental ones obtained for the Fujinomori clay and Toyoura sand. Demonstrating the capability of the SubCam model to simulate cyclic loading.
Simulation of the installation of a model pile in sand

With the aim to validate and assess the capacity of the MPM and the implemented constitutive models MCC and SubCam to reproduce the installation process of a pile foundation during penetration, a numerical simulation of an existing small-scale experimental chamber test with sand was carried out. The test was originally published by Jardine et al. (2013a).

Figure 3 shows the dimensions of the calibration chamber and the mini-ICP pile used in the test together with the levels of the measuring sensors inside the soil mass. The chamber was filled with a fine NE34 Fontainebleau sand by using the technique of air pluviation, which gave an initial average void ratio of 0.62 to the pluviated deposit, with an equivalent relative density of 72%. The authors also reported a $K_o = 0.45$ and an internal friction angle at critical state between $35.2^\circ$ and $32.8^\circ$ with an overconsolidation ratio (OCR) of 1 – normally consolidated state. More details of the complete experiment are found in Jardine et al. (2013b) and Yang et al. (2014).

Tsuha et al. (2012) presented ring-shear interface tests that reproduced the interface conditions between the pile and the sand used by Jardine et al. (2013a). They obtained an average interface friction angle of $\delta' = 26^\circ$. This implies an interface friction coefficient of $\mu' = 0.49$. The values of $\delta'$ obtained in this type of shear test are more suitable to represent the condition of a soil-pile shear interface, rather than adopting values from standard direct shear tests.

The values of $\lambda$ and $\kappa$ needed to complete the parameters of the MCC and the SubCam models used in the simulations were interpreted from an oedometric compression test on the virgin NE34 sand, as presented by Yang et al. (2010). The obtained results for $\lambda$ and $\kappa$ were 0.15 and 0.013, respectively. The Poisson ratio value considered was $\nu = 0.3$. This value was
selected based on the results obtained for the Fontainebleau sand by De Gennaro and Frank (2002) for stresses on the order of 100kPa (as in the initial state in the current analysis). Table 1 summarizes the mechanical parameters used in this simulation.

The mini-ICP2 test carried out by Jardine et al. (2013a) was simulated here in. In this test the sand deposit was overloaded by a top membrane that had a central internal diameter of 200mm, just before the installation process. A base-pressurized membrane was also used, with a surcharge pressure of 152kPa, which has generated an initial vertical stress along the sand mass of approximately $\sigma_v = 150kPa$. A mini cone penetration test (CPT) was pushed in the sample before the driving process, reproducing a quasi-constant tip cone resistance of $q_c = 21 \pm 2MPa$. A constant jacking rate of around 0.5mm/s was adopted during the semi static (pushed-in) installation process. The pile diameter is D=36mm.

In the numerical simulations, the pile was considered to be a rigid material, greatly reducing the computing time. Thus, the pile was introduced in the simulation as a moving boundary condition. A constant vertical velocity of 20mm/s was applied to the pile until it was completely installed. A Coulomb frictional contact type was used between the pile and the surrounding soil with the interface friction coefficient obtained based on Tsuha et al. (2012) results. Details of the contact algorithm used can be found in Bardenhagen et al. (2001) and Lemiale et al. (2010).

The simulations took into consideration aforementioned sand properties for an OCR of 1. Axisymmetric conditions were also considered, which have been recently presented for GIMP by Nairn and Guilkey (2013). The background grid used the traditional uniform square mesh with mesh cell of 0.25D, with 4 particles per cell. It was defined after a mesh-particle discretization analysis resulting in a grid of 67x167 and a total of 44756 material points. The boundary condition at the bottom side of the model was considered vertically fixed, and the
lateral edge was horizontally fixed. An appropriate global damping factor (500/sec) was added to the model since the beginning of the simulation, allowing to approach quasi-static results without over damping.

The first loading stage was modeled using a distributed surface pressure of 150kPa. Only after this initial case was the pile jacked process simulated. In the experimental setup, the stresses inside the sand mass were measured with sensors located at different depths ($z$) at certain radial distances from the pile’s axis ($r$). Figure 4 shows a schematic diagram of the geometric variables that improves the understanding of the outcome results. The “x” in the chart indicates experimental measurement points.

Figure 5 presents the radial displacement and the deformed shape obtained by the simulations when using GIMP and the SubCam constitutive model.

Figure 6 shows the predicted radial stress at specific points inside the sand mass during the installation process of the pile when simulated by using GIMP with either the MCC or the SubCam models. The experimental values published by Jardine et al. (2013b) and Yang et al. (2014) are also shown. The predicted radial stress values were normalized by $q_c = 21\text{MPa}$, as presented by Jardine et al. (2013a). In the vertical axis, values of $h/R<0$ represent stress results for a specific sensor when the pile’s tip is above its particular depth level. In the case of the sensor located at a depth of $z=550$mm, these authors only published measured results for $-5<h/R<5$ (Figure 6c and 6d).

The results obtained using the MCC model differ from the experimental reference values, albeit having similar trends. The maximum values of radial stress are experienced when the pile tip is near the measuring sensor, decreasing once the pile passes by. The stresses for $r/R=3$ (Figure 6a) are higher than those for $r/R=8$ (Figure 6b), exposing a tendency of reduction of stresses as the horizontal distance to the pile axis increases. Simulations using
the SubCam model tended to matched experiments better, although still with differences in magnitude between experimental and numerical values. The simulated maximum radial stresses were as much as 25 times higher than the initial stresses. For the four points studied (Figure 4), the radial stresses were higher than the initial ones at the end of the installation process. The radial stresses, at the end of the installation process, were as much as 3.5 times higher than the initial radial stresses, as noticed for the point close to the pile (r/R=2). The theoretical consideration of the densification process of the sand, as built in the SubCam model, has indeed improved the match. The higher output of maximum stresses of this latter model, when compared to output using the MCC, is associated to the increase in the rigidity of the soil by the densification phenomenon that happens after pile penetration. Hence, for an equal value of deformation the generated stresses in the SubCam model are generally higher.

The fact that the SubCam model uses the Matsuoka and Nakai failure criterion (Matsuoka and Nakai 1985) also contributed to the slightly better results obtained with this model. When the pile’s tip is penetrating into the soil, some extension stresses appear around it, which is undoubtedly better reproduced with this particular criterion.

Figure 7 shows the vertical stress at specific points inside the sand mass during the pile installation process. The measured values obtained by Jardine et al. (2013a) and the simulated ones using GIMP with both MCC and SubCam models are presented. Similar to the radial stress case, the vertical stress increased when the pile’s tip was near the depth level of the measuring sensor. The maximum value is observed at h/R = -5. Once the pile passes by the sensor the stress tends to decrease. However, contrary to what occurs in the radial case, the final vertical stress at the end of installation is lower than the initial value for the data point that is radially closer to the pile, thus indicating an upward movement of the soil around the pile’s tip.
For the vertical stresses, the SubCam model also gives better results than the MCC model, especially near the peak value, when the pile’s tip is in the nearest position of the sensor level.

Figure 8 presents the earth pressure coefficient ($K$) for the point located at $z = 300\text{mm}$ and $r/R = 3$ during the pile installation. At the beginning of the installation process, the value of $K$ is not affected at the depth of the sensor ($z = 700\text{mm}$) and is very close to $K_0 = 0.45$. When the pile’s tip is at $h/R = -5$, the horizontal stress increases more than the vertical stress and exceeds this last one, leading to a value of $K > 1$. After the pile tip passes the depth level of the measuring sensor the value of $K$ rapidly decreases, still reaching values greater than 1 at the end of the installation process. This outcome characterizes a pattern of earth pressures that resembles the development of passive pressures around the pile’s shaft. Again, the SubCam model led to results that are closer to the experimental values.

Figure 9 shows the stress path during the pile installation process for a point at $z = 700\text{mm}$ and $r/R = 3$, obtained using the MCC and the SubCam models. This path resembles those from oedometric loading tests, at least until the pile’s tip has reached the level of the measuring sensor level ($h/R = 1.7$ for the MCC and $h/R = -1.1$ for the SubCam). After this stage is reached, a rapid unloading path follows. Sheng et al. (2005) also showed similar behavior in their numerical analyses with a critical state model, where a sharp decrease in stress is observed soon after a peak value is reached. They observed that such a phenomenon is associated with the formation of a plastic softening zone around the pile.

Figure 10 shows the radial stresses in two horizontal sections at different depths after the pile installation process is completed. The maximum values for each section are near $r/R=3$. The far the points are from the pile axis the smaller the stresses are. Besides, the stresses increase with depth until the pile’s tip region. Near the shaft ($r/R=1-3$) the radial stresses show a
decrease when compared with the trend. Such reduction near the shaft was linked by Yang et al. (2010) with a) with some particular factors, as (a) strain path reversals that occur as the soil flows through the shoulder of the pile’s tip. This reduces the stresses and leads to arching with $\sigma_{\theta} > \sigma_{r}$ near the shaft, (b) the development of an annulus of intensely compressed and sheared sand around the shaft. The simulations captured relatively well the reduction of stresses around the shaft, sustaining the first hypothesis advocated by Yang et al. (2010).

Lower values of radial stresses were also obtained when they were compared to the measured ones. Also, a faster reduction of the magnitude of the stresses can be observed. For points that are far from the pile’s axis ($r/R=13$), the stresses are almost equal the initial values.

Figure 11 shows the displacement path obtained using GIMP and the SubCam model during the installation process. Most of the displacements occur when the pile’s tip has a relative depth between $h/R=-10$ and 10. For the point closer to the pile’s axis ($r/R=3$), after the pile’s tip pass through the point level ($h/R=-0.1$) the vertical displacement changes in direction and the point goes up. This explains the decrease that appears in the vertical stress once the pile’s tip passes through the sensor level (Figure 7a). The upwards movement is actually a consequence of the flow of soil around the shoulder of the pile’s tip (White and Bolton 2004). For the point farther from the pile’s axis ($r/R=8$), this features doesn’t appear, so the vertical displacement is always downwards. For both points, the horizontal displacement increases until the pile’s tip pass through the point level ($h/R=-0.1$), after that, a little decrease occurs. This movement contributes to the decrease in the radial stress when the pile’s tip pass the point level (Figure 6).

Volumetric strain paths obtained during the installation process are shown in Figure 12. For the point nearest to the pile’s axis ($r/R=3$), compression is observed when the pile tip gets closer to the point level ($-10<h/R<-6$), this is followed by a strong dilation when the pile tip is
within a distance of 6 radii. Once the pile’s tip passes the point level, a small compression is observed until \( h/R = 10 \). A net increase in volume is then obtained for this point (12%). This behavior is similar to the results presented by White and Bolton (2004) for points nearest to the pile’s axis \( (r/R < 2) \), inside the zone defined as very near field behavior zone.

For the point far from the pile’s axis \( (r/R = 8) \), a monotonically increase of compression occurs during the pile installation process. This trend is in good agreement with the experimental results of White and Bolton (2004) for points located inside the zone defined by the authors as far field behavior zone.

The end points of the volumetric strain paths indicate the variation in density after the installation process. The strong dilation in the near field creates a zone of soil close to the pile shaft that is less dense than the more distant soil.

The simulated results can be considered in good agreement with the experimental ones for applications in foundation engineering. However, improvement can be done to the model to perfectly match with the experimental outputs. Some important constitutive characteristic can be added such as, anisotropy, grain breakage, strain dependency and friction angle stress dependency.

**Parametric analysis and effect of friction fatigue**

A parametric analysis was performed to evaluate the influence of the installation process on the bearing capacity of a typical pushed-in pile in granular soil. The SubCam constitutive model was used for the soil, and the pile was considered infinitely rigid. The analysis was performed using axisymmetric conditions starting with a \( K_0 \) geostatic stress field. The installation process was simulated by applying a constant vertical velocity (20mm/s) to the
pile until it was completely driven. This feature reflects better the (pushed-in) installation of a
jacked pile.

The differences between the behavior of a jacked pile and a driven pile have been presented
by Yang et al. (2006) and Zhang and Wang (2009). They concluded that the shaft resistance
of jacked piles is generally stiffer and stronger than that of driven ones.

Figure 13 shows both geometry and mechanical parameters common for all simulated cases.
Table 2 presents the material and the normalized geometric variables considered for all
analyses. In this table, the numerical cases are coded by an identification number (L/D- \( \phi' \) -
OCR), followed by the slenderness of the pile, the friction angle and the overconsolidation
ratio of the soil. The mesh and number of particles per cell used in these simulations was the
same of the previous presented case (mesh cell dimension = 0.25D and 4 particles per cell).
The boundary condition at the bottom side of the model was considered vertically fixed, and
the lateral edge as horizontally fixed.

Figures 14 to 16 show the radial stress at the shaft of the pile after its installation. As one can
see, the variation of stress along depth is almost linear until the tip of the pile (0.7L for the
shortest piles and 0.9L for longest ones). At this particular region there is a sudden increase in
stress followed by a sharp decrease. This behavior resembles the idealized pattern shown in
Figure 1, which encourages the authors to affirm that the analyses tended to approach the
“real” experimental phenomena. The increase of the radial stress ratio near the pile’s tip
occurs because this zone experiences less shear cycle than other depths during the installation
process. One can also notice that the radial stress increment in relation to the initial stress
\( \left( \sigma_{ri} / \sigma_{n} \right) \) is higher for the models with \( \phi' = 40^\circ \) than those with \( \phi' = 20^\circ \). For the former
ones, the weighted mean increment stress along depth is around 5, whereas for the latter ones
it drops to values in the range of 1.5.
Figure 17 depicts the large influence of the friction angle on the value of the radial stress after installation of the pile. This phenomenon can be explained by observing the stress path followed by a point near the pile’s shaft during penetration. The plastic strains are much higher than the elastic ones, hence the direction of the deformations are controlled by the plastic strains. For the constitutive model used herein, the plastic volumetric strains are higher for the soil with higher $\phi'$ values (Figure 18). The higher strength granular materials, idealized herein, tend to have higher values of normal stress, consequently, for this case, higher values of radial stress.

Figure 19 shows the influence of the length of the pile, on the radial stresses at the end of the installation process. The relative increment in the radial stress to increases in the friction angle tends to be higher for the shortest pile (L/D=5) than for the longest one (L/D=25). Also, the longer is the pile, the lower will be the radial stress at equivalent depth. By noticing that, at a particular depth, the value of $h/R$ is larger for the pile with L/D=25, one concludes that a “friction fatigue phenomenon” (or $h/R$ effect) is probably the cause of such a result. Experimental evidence tends to back up this observation, which can be attributed to the gradual densification of the soil adjacent to the pile’s shaft under the cyclic shearing action of installation (White and Bolton 2004). As commented before, Sheng et al. (2005) also associated this phenomenon with the softening of the soil one radius around the pile, because of a plastic expansion of the material in this particular zone.

The effect of the overconsolidation ratio (OCR) in the final radial stress mobilized at the pile’s shaft is negligible, as clearly shown in Figure 20. The authors did not find any experimental data that confirms this phenomenon, but neither the equation proposed by Randolph et al. (1994) (Eq. 1) nor the one by Jardine et al. (2005) consider the OCR effect. Sheng et al. (2005) evidenced that the general pattern of the stress path followed by a soil
point near the pile is not significantly influenced by the OCR during pile installation. A possible explanation is given by the fact that the stress path inside the yield surface is very small compared to the one on the yield surface itself. Hence, one can expect the stress response of the soil to be similarly close at distinct conditions of overconsolidation ratio.

Figure 21 shows the profile of the earth pressure coefficient along depth for piles with distinct lengths (or slenderness ratios L/D). The presented coefficient \( K = \sigma'_h / \sigma'_v \) is actually a ratio between the horizontal effective stress at the end of the installation process and the constant vertical effective geostatic stress. A similar pattern to the radial stress profile is also noticed for this coefficient, which is logical. Also, for the soils with a higher granular strength \( \phi'_v \), a higher \( K \) is obtained, because as mentioned before the horizontal stress is strongly influenced by the friction angle of the soil.

**Effect of installation process on ultimate shaft resistance**

One of the most important variables that can be derived from the radial stress around the pile’s shaft is the ultimate shaft resistance \( Q_s \) of the pile – or lateral bearing capacity. Considering that \( Q_s \) is related to the ultimate shaft friction \( \tau_s \), and this last one correlates to both the horizontal effective stress \( \sigma'_h \) acting on the shaft and the effective (remoulded) angle of friction between soil and pile \( \delta'_v \), one can write:

\[
Q_s = A_0 \int_0^L \tau_s \, dz = A_0 \int_0^L \sigma'_h \tan \delta'_v \, dz
\]  

(12)

With equation (12) one notices that by considering that \( \delta'_v \) does not change throughout the installation of the pile, the relation between ultimate shaft resistance considering the installation effects \( Q'_s \) and not considering \( Q'_s \) only depends on the horizontal effective
stress at the pile’s shaft immediately before failure (lateral shearing). Therefore, new
simulations were done similar to those previously shown, by applying a further prescribed
displacement of D/30 on the pile once its installation was finalized. This added displacement
effect tended to simulate a lateral friction failure.

Figure 22 shows the relation $\frac{Q_s'}{Q_s^0}$ for the simulated cases. As one can see, the increase in
shaft resistance is higher for short piles (models 1, 2 and 5) than for long ones. This
difference is caused by aforementioned friction fatigue effect on longer piles. The increase is
also higher for the piles immersed in a high strength soil (models 2, 4, 5 and 6). As the
previous case, the OCR effect is negligible.

By defining the relationship between the ultimate shaft resistance of a pushed-in and of an
excavated pile as $\frac{Q_s^{pushed}}{Q_s^{excavated}}$, one can also reach interesting conclusions. This ratio can be
empirically defined by Brazilian (SPT pile design) methods that somehow take into account
the installation process (from experimental load tests on long piles), as for instance the
methods of Aoki and Velloso (1975) and Décourt and Quaresma (1978). These latter
empirical approaches indicate, for piles installed in the same soil profile, ratios
$\frac{Q_s^{drivenpile}}{Q_s^{excavated}} = 1.18$ for clay and 2 for sand (long or high L/D piles). So, considering in a
simplified manner that $Q_s^0 = Q_s^{excavated}$, the relation $\frac{Q_s'}{Q_s^0}$ can be directly compared to the ratio
$\frac{Q_s^{pushed}}{Q_s^{excavated}}$. With such an exercise one derives empirical values of $\frac{Q_s^{drivenpile}}{Q_s^{excavated}}$ that
are in the same order than those numerically predicted by both models 3 and 4, which again is
an encouraging (experimental vs. numerical) outcome.

Conclusions

The installation process of single jacked (push-in) piles in granular soils can be modelled
with the generalized interpolation material point method, or GIMP, by adopting a Coulomb
friction model at the contact soil/pile. The adopted numerical method was able to handle this large deformation problem in a robust and practical way.

Two constitutive models have been implemented and tested. The final response of the driven pile with the modified cam clay model (MCC) did unfortunately not agree well with the small scale experimental results. Because only a few adjustments were necessary to turn the MCC into the subloading cam clay model (SubCam), the latter was also implemented and tested herein. The SubCam model clearly provides a better match between numerical and experimental outputs, being therefore advantageous to use it in the simulation of problems of this kind. Indeed, the SubCam model does consider in a better and proper way the granular soil densification after pile insertion, and can differentiate the response from triaxial or compression loading paths.

Furthermore, for typical geometries adopted in jacked piles in granular soils, the post installation lateral stress coefficient $K$ reaches values as high as 2 times the original earth pressure coefficient $K_0$. From the spatial distribution of stresses after pile installation, it seems that horizontal stresses in the soil can be very high when compared to the initial geostatic horizontal values ($K_0$ times the vertical effective stress). For instance, at a radial distance around 4 times the pile diameter from the pile’s axis ($r/D=4$), horizontal mobilized stresses can be as much as 2 times higher than the original effective values. The analyses have also attested some observations in literature that the overconsolidation ratio (OCR) does not significantly affect the output response of the pile’s installation process.

The simulations were able to capture the effect of friction fatigue in long slender piles installed in sands. The results suggest that the friction fatigue effect shows the effect of stress history in the soil element resulting in evolving strength/stiffness properties.
A final and broad conclusion of the study can be derived from the importance demonstrated by the analyses to effectively consider the penetration process of driven (push-in type) piles in sands. The installation phenomena clearly changes some post execution geotechnical variables of the granular material, enhancing for instance the bearing capacity of the pile, densifying the surrounding media, or increasing the stress levels all around the foundation – generally in a simultaneous manner. It is then concluded that to properly understand the post installation behavior of driven piles in sands, one must account for the installation effect itself, either by numerical or by analytical simplified approaches.

Acknowledgements

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(Jardine et al., 2013a)
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<table>
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<th>$Z_{(mm)}$</th>
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<tr>
<td>550</td>
<td>x</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>700</td>
<td>-</td>
<td>x</td>
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SubCam Material

\[ \lambda = 0.1 \quad c_0 = 1.8 \]
\[ \kappa = 0.01 \quad \gamma = 20 \text{kN/m}^3 \]
\[ v = 0.3 \quad a = 200 \text{kN/m}^3 \]
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a) $\phi' = 20^\circ$

b) $\phi' = 40^\circ$

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Table 1. Model parameters used for the simulation of the model pile in sand

<table>
<thead>
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<th>Model parameters</th>
<th>Parameter</th>
<th>Value</th>
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<tr>
<td></td>
<td>( \gamma ) (kN/m(^3))</td>
<td>16.3</td>
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<tr>
<td></td>
<td>( K_0 )</td>
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</tr>
<tr>
<td></td>
<td>( \nu )</td>
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<tr>
<td></td>
<td>( \lambda )</td>
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</tr>
<tr>
<td></td>
<td>( e_0 )</td>
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<tr>
<td></td>
<td>( \phi' )</td>
<td>35°</td>
</tr>
<tr>
<td></td>
<td>( a )</td>
<td>2000</td>
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<tr>
<td></td>
<td>( \mu' )</td>
<td>0.49</td>
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Table 2. Models Parameters and identification code used in the parametric analysis

<table>
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<tr>
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<th>OCR</th>
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<td>1</td>
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D: Pile diameter = 80cm for all models.