Simulation of Auger Displacement Pile Installation

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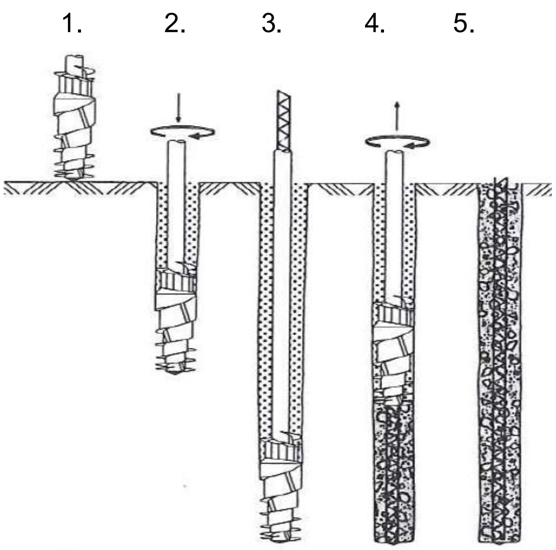
ABSTRACT

Auger displacement piles have been used throughout the piling industry in Australia and worldwide for decades as foundation elements for structures and embankments. The majority of auger displacement piling is carried out in fine-grained soils, with the potential for damage to already completed piles, as well as to adjacent structures. The behaviour of the soil surrounding the auger during penetration, extraction and concrete pumping is simulated using the Finite Element Method, with the soil described by different constitutive models, including the hypo-plastic model. The paper identifies the parameters involved, describes the numerical model, and presents the displacement field, stress field, and pore water pressure distribution obtained. The results provide a prediction for field tests that will be carried out in the near future.

Keywords: displacement piles, hypo-plasticity, numerical modelling, rigid inclusions, soil improvement

1 INTRODUCTION

Auger displacement piles (ADP) are a rotary drilling technique that can be used to construct concrete piles and columns. During the penetration of the drill tool the soil is displaced laterally into the surrounding ground, and the spoil created by ADP installation is minimal. The technique is very economical and environmentally friendly, which has led to its increased global usage during the last decade. The typical ADP installation process is described in Figure 1 (Bottiau 1998).



Installation of Auger displacement piles:

1. Set up auger at pile position and install cap to close concrete outlet at auger tip.
2. Install auger by rotating clockwise and applying vertical pull down force.
3. Drill auger to design depth; the displacement body of the auger pushes the soil cut by the auger tip into the surrounding ground.
4. Pump concrete through hollow auger stem and extract auger while rotating clockwise, always maintaining concrete pressure positive and auger embedded in fresh concrete
5. Install reinforcement into fresh concrete, if required.

Figure 1: Installation process of ADP after Bottiau 1998

Depths of up to 30 m can be achieved with standard piling equipment. Different drilling tools and auger shapes are available and typical diameters range from 200 to 550 mm. Skin friction and end bearing behaviour of different ADP types can vary, depending on soil conditions and the geometry of the drilling tool (Vermeer 2008). Soil behaviour during the entire process, particularly in cohesive soils has not been investigated in detail. ADPs have the potential to damage adjacent structures or freshly cast piles due to displacement effects. Investigations to further understand the behaviour of ADPs in fine-grained soil will be carried out during research at The University of Queensland.

Currently, design approaches for ADPs are commonly based on elastic or elasto-plastic constitutive laws. However, these constitutive laws provide only a rough approximation of the real behaviour of soils, especially for loading / unloading situations. As a result, designs for ADP applications can be conservative and inefficient. The development of a robust numerical model, followed by instrumented field tests will provide a deeper understanding of the stress and displacement behaviour of the soil during drilling tool installation and extraction.

The University of Queensland, Australia, and the Technical University of Dresden, Germany, entered into cooperation in 2010 to develop a suitable numerical model for the penetration and extraction of a typical ADP in fine-grained soil using a hypo-plastic constitutive model for clay (Mašín 2005).

2 HYPO-PLASTICITY

2.1 General

Constitutive models are used to describe the mechanical behaviour of a soil, relating stress and strain or their respective rates. Besides stress and strain (which are both tensorial quantities), additional parameters such as material constants and state variables are used in constitutive equations. Reliable constitutive models are required to describe material behaviour as accurately as possible.

Hypo-plasticity is a nonlinear constitutive theory that is able to describe dissipative behaviour, plastic flow and nonlinear effects with a single tensorial equation (Niemunis 2002).

Several hypo-plastic constitutive models express the stress rate as a function of a given strain rate and of the state variables stress and void ratio. Provided **T** denotes the tensorial quantity stress, **D** denotes the tensorial quantity stretching tensor and e denotes void ratio, this can be expressed by the general form of the hypo-plastic equation (Kolymbas 2001):

$$\overset{\circ}{\mathbf{T}} = h(\mathbf{T},\mathbf{D},e) \tag{1}$$

The basic stress–strain rate relationship is expressed as (Gudehus 1996):

$$\overset{\circ}{\mathbf{T}} = f_s \left(\mathbf{L} : \mathbf{D} + f_d \, \mathbf{N} \, \lVert \mathbf{D} \rVert \right) \tag{2}$$

with **L** and **N** being fourth- and second-order constitutive tensors and f_s and f_d representing two scalar factors, known as the barotropy (influence of stress level) factor and the pyknotropy (influence of density) factor, respectively.

The hypo-plastic relationship (2) seems simple, as there is only one equation for loading and unloading. It assumes non-elastic deformations occurring from the beginning of the loading process. Hypo-plasticity does not distinguish between elastic and plastic deformations.

2.2 Hypo-plasticity after Mašín

Research on hypo-plastic models started more than 20 years ago. Many of the early models were developed by researchers of the University of Karlsruhe in Germany, focussing on coarse-grained granular materials (von Wolffersdorff 1996; Niemunis and Herle 1997). However, in recent years hypo-plastic models for clays were developed as well (Niemunis 2002; Herle and Kolymbas 2004, Mašín 2005).

Mašín's (2005) hypo-plastic constitutive model for clays is based on the general approach to hypo-plasticity developed at Karlsruhe. It has been used for modelling the clay for this research project. Mašín's basic model requires only five constitutive constants, which can be determined from standard laboratory tests. Despite this simple and practical approach, several effects of the non-linear soil behaviour are described by the model: (i) the variation of stiffness during loading and unloading, (ii) the influence of relative density (overconsolidation ratio) on the stiffness, contractancy and dilatancy in the volumetric behaviour, and (iii) the dependence of peak friction angle on the state.

The basic model comprises several principles of critical state soil mechanics. It also uses a similar set of five constitutive constants determined by standard laboratory tests. The interpretation of the parameters is similar to the modified Cam Clay Model (Roscoe and Burland 1968).

Parameters N and λ^* define, respectively, the position and the slope of the isotropic normal compression line (Figure 2), following the formulation of Butterfield (1979):

$$\ln(1+e) = N - \lambda^* \ln(p/p_r) \quad (4)$$

The parameter, κ^*, controls the slope of the isotropic unloading line (Figure 2).

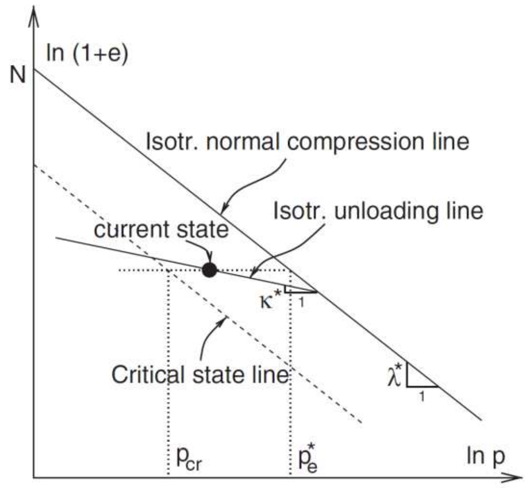


Figure 2: Definition of parameters N, λ and κ* (Mašín 2005)*

Further information about Mašín's hypo-plastic constitutive model for clays and further refinements of this model can be obtained from Mašín (2005) and Mašín and Khalili (2008).

3 FIELD AND LABORATORY TESTS

The field test site is located in Lawnton, Queensland, Australia. In February 2011, two cone penetration tests (CPT) and two undisturbed, continuous soil samples were taken to bedrock at two locations. The soil samples were taken about 0.5 m from the CPTs in order to get a close correlation between CPT values and the soil profile.

Index properties were determined in the laboratory for each of the three different soil layers encountered. The typical soil profile, classification and index properties are summarised in Table 1.

Table 1: Soil profile, index properties and soil classification of Lawnton clays

Soil Description	BH1	BH2	Fines Content <0.15mm	Liquid Limit (LL)	Plastic Limit (PL)	Plasicity Index (PI)	Classification DIN 18196	Classification USCS
Stiff, light brown, **ocher clay**	0.00 m to -2.20 m	0.00 m to -1.50 m	79%	0.43	0.25	0.18	Medium plasticity	Low plasticity
Stiff, **grey clay** with coloured inclusions	-2.20 m to -6.80 m	-1.50 m to -6.20 m	91%	0.53	0.21	0.32	High plasticity	High plasticity
Stiff, **grey sandy clay** with coloured inclusions	-6.80 m to -8.00 m	-6.20 m to -7.20 m	57%	0.44	0.19	0.25	Medium plasticity	Low plasticity

The second clay layer, which was defined as grey clay, will be the material in which the ADPs will be founded. Toe levels of the test piles will be located at -5.5 m. Characteristics of the grey clay will be critical for the ADP performance.

Laboratory oedometer tests and consolidated undrained (CU) triaxial tests were carried out on undisturbed and remoulded soil samples to determine the basic five hypo-plastic soil parameters.

Parameters N, λ and κ*:* These parameters were determined from a single loading / unloading oedometer test. Isotropic loading must exceed the pre-consolidation pressure in order to find the position of the slope of the normal compression line. Parameter κ* should be calibrated from the slope of the isotropic unloading line of an oedometer test close to the normally compressed state.

Parameter φ_c: The critical state friction angle was found using a linear regression through the critical state points of all shear tests available, determined from linear regression of triaxial CU test results on remoulded and consolidated soil.

Parameter r: Parameter r may be evaluated directly, using the definition as the ratio of the bulk to the shear moduli, for tests starting from the isotropic normally compressed stress state (Figure 3).

A summary of the calculated hypo-plastic parameters for grey clay are as follows:

φ_c	= 24.7°	(critical state friction angle)
N	= 1.105	(Isotropic compression)
λ*	= 0.1034	(slope isotropic normal compression line)
κ*	= 0.0127	(slope isotropic unloading line)
r	= 0.2	(ratio of bulk modulus to shear modulus)

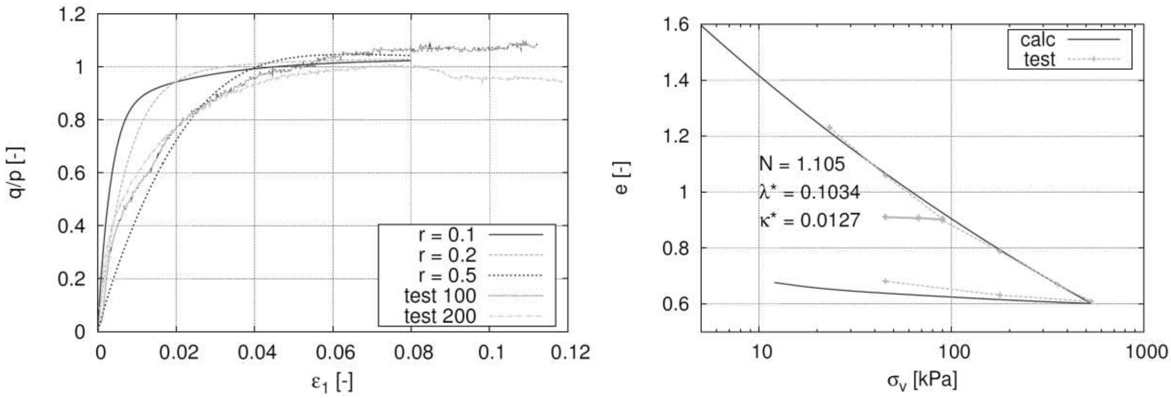


Figure 3: Calibration of parameter r (left), comparison of calculated oedometer results and test data for grey clay (right).

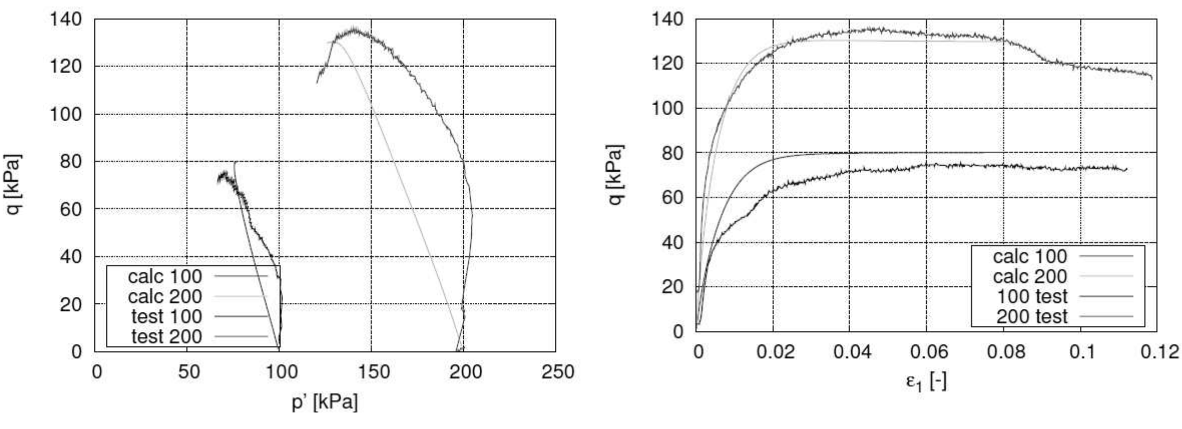


Figure 4: Test results from triaxial CU tests on grey clay

4 NUMERICAL SIMULATION

For the numerical simulation of the penetration and extraction of an ADP, the Finite Element (FE) code Abaqus Standard was used. Hypo-plastic soil behaviour for fine-grained soil (clays) after Mašín was implemented using UMAT routine. An axisymmetric, two dimensional FE model was developed to simulate the process. The model greatly simplifies the real geometry and process.

The drill head of the ADP is modelled as a cone-shaped rigid body, with a 60 degree cone angle and a total auger height of 1.5 m, representing a typical ADP drill head. It is assumed that the auger flights stay completely filled with soil throughout the process, as the soil cutting and soil transport process of the auger has not been modelled. To avoid excessive mesh distortion at the beginning of the penetration process, the cone is partly pre-installed into the soil and the soil and the cone are modelled to be in full contact.

In the past, a particular method called the “zipper technique” was used successfully (Cudmani 2001, Henke 2010) to model pile penetration into a soil continuum. A smooth rigid tube with a diameter d = 1 mm is discretised at the axis of penetration. The cone-shaped, rigid ADP body slides over the rigid tube and separates the soil from the tube. The cone establishes contact with the soil and is able to deform the meshed continuum, thus simulating penetration and the resulting soil displacement. The surface-to-surface contact between the penetrating object (ADP auger) and the surrounding soil is based on the master-slave principle. The friction coefficient between the deformable soil and the rigid piling auger is assumed to be tan (φ_c/3), based on Coulomb’s friction law.

The diameter of the displacement auger head is taken to be 450 mm and the penetration depth into the soil continuum is taken to be 4 m. Pile installation is modelled using constant penetration and extraction rates of 0.03 m/s. However, since the constitutive model is rate-independent, the rates are of no impact. Soil behaviour has been assumed to be undrained during penetration (constant volume), as penetration occurs too rapidly to allow substantial drainage.

The boundary conditions and FE mesh of the model are shown in Figure 5. Four-node bilinear axisymmetric elements (CAX4) have been used to represent the soil continuum, and unacceptable mesh distortions are avoided due to the use of an adaptive meshing technique (re-meshing rule). The model could not be used with elastic-plastic constitutive laws as mesh distortion proved excessive and the simulation was aborted by the program after a penetration of less than 10 mm. Figure 5 shows the stress field, soil displacements, and pore water pressures obtained during ADP installation at the point of reaching the design depth.

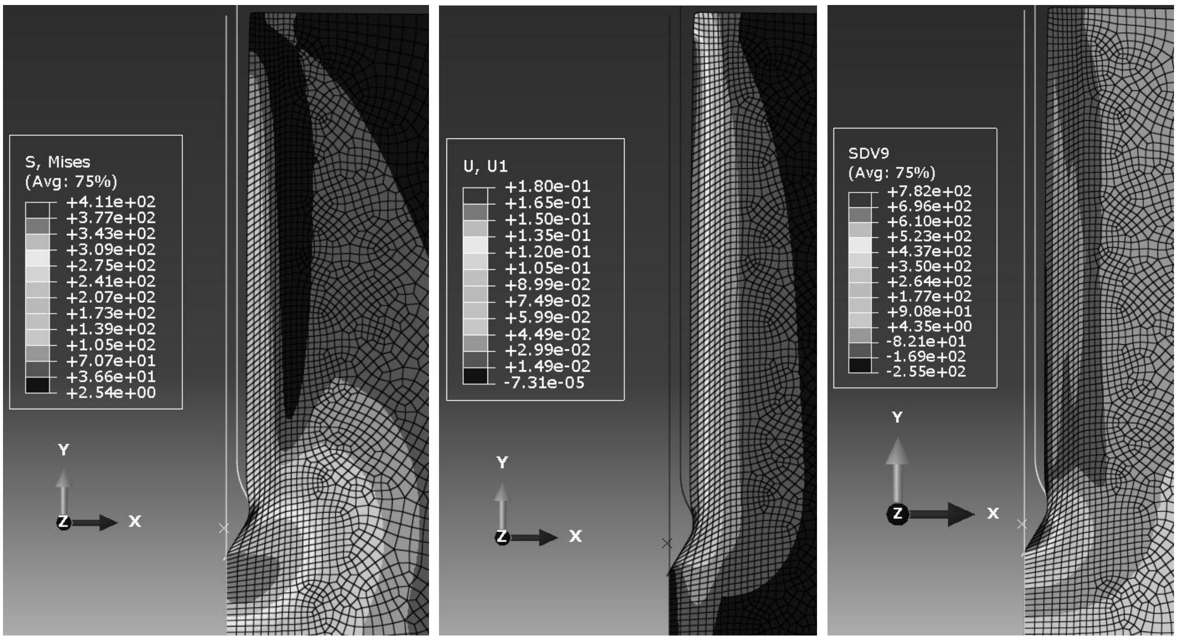


Figure 5: Expected effective stress field (left), soil displacement (centre), and pore water pressures (right) of ADP to be installed in Lawnton clay modelled using Abaqus Standard

It can be observed that the effective stress bulb below the auger tip is expected to reach about 1.5 pile diameters from the pile centre line, and about one pile diameter below the design depth. Minor stress changes in the soil continuum are expected as far as three pile diameters away (horizontally and vertically). Soil displacements are expected to reach up to 2.5 pile diameters from the pile centre line; however most of the displacements are expected to occur within 1.5 pile diameters from the pile centre line. The simulation of auger rotation, soil cutting and the transport processes are beyond the capabilities of the continuum methods and thus have not been considered.

5 CONCLUSION

Hypo-plastic soil models are an attractive option to model non-linear mechanical soil behaviour. They are relatively simple and the constitutive constants can be determined by standard laboratory tests.

The undrained penetration of a rigid element (auger displacement pile) into Lawnton clay was modelled using the finite element code Abaqus Standard. The hypo-plastic constitutive model for fine-grained soil was successfully used in Abaqus Standard and has simulated stress and displacement fields during penetration and extraction of the drilling tool. The results of the FE simulation will be used as a prediction for field ADP installation tests to be carried out during 2012, which will in turn be used to calibrate and further refine the FE model used in this research project.

Due to the complexity of the ADP installation process, details such as the detailed auger shape, rotations, different torque, and pull-down configurations were not implemented in the FE model. However, it is expected that the current model will deliver good predictions of soil behaviour during the ADP installation process in fine-grained soils.

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