Predicting the foundation performance of offshore jack-up drilling rigs in intermediate soils

by

Raffaele Ragni

B.Sc, M.Eng

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Centre for Offshore Foundation Systems
School of Civil, Environmental and Mining Engineering

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To D.

Behind this not so great a man

is a great woman
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Details of the work:

Location in the thesis:
Chapter 2

Candidate contribution to work:
The candidate contributed to the planning, preparation and execution of the tests, analysed the data and contributed to the drafting of the manuscript.

Details of the work:

Location in the thesis:
Chapter 3

Candidate contribution to work:
The candidate contributed to the planning, preparation and execution of the tests, analysed the data and contributed to the drafting of the manuscript.
Details of the work:

Location in the thesis:
Chapter 4

Candidate contribution to work:
The candidate developed the numerical two-dimensional model in collaboration with the other authors, planned and carried out the numerical parametric study. He analysed the data and had the main role in drafting the paper as corresponding author.

Details of the work:

Location in the thesis:
Chapter 5

Candidate contribution to work:
The candidate developed the numerical three-dimensional model in collaboration with the other authors, planned and carried out the numerical analyses. He also planned and carried out the experimental tests in the geotechnical centrifuge. He analysed the data and had the main role in drafting the paper as corresponding author.

Candidate signature: 

Date: 31/10/2016
I, Britta Bienen, certify that the candidate statements regarding the contribution to each of the works listed above are correct.

Coordinating supervisor signature:  
Date: 28/10/2016

Co-author signature:  
Date: 28/10/2016

Co-author signature:  
Date: 24/10/2016

Co-author signature:  
Date: 28/10/2016

Co-author signature:  
Date: 25/10/2016

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Abstract

Jack-up units are mobile offshore platforms deployed in offshore engineering. The configuration is usually a hull, three retractable trusswork legs and circular foundations located at the end of each leg. Given their extensive use in oil and gas and renewable energy sectors, safety during installation and operational conditions must be assured throughout their lifetime. The entire structure relies on the bearing capacity provided by the circular foundations, named spudcans, which are penetrated for a significant distance into the soil during the installation procedure, under the self-weight of the structure and with the additional vertical loading provided by pumping water into ballast tanks.

When jack-ups are installed in fine-grained to intermediate silty soils, the response of the soil is generally considered undrained. Thus, significant build-up of excess pore pressures can be expected during spudcan penetration and when the structure is loaded by a combination of waves, currents and wind. However, given the relatively high permeability of such soils, consolidation around the footing can take place rather quickly and lead to partial or complete dissipation of such excess pore pressures. Consolidation can occur at different stages and scales: from short pauses in penetration to very long periods of consolidation post-installation. The consequences of this dissipation can be either beneficial or detrimental for the stability of the jack-up. Beneficially, this means having a stiffer, stronger and, most important, reliable soil around the footing, with the chance to reduce the final leg penetration. However, on the other detrimental side, the improvement brought about by consolidation could be only temporary and localised, hence not reliable under cyclic loading and potentially responsible for triggering rapid and uncontrolled leg-penetration. In either case, a site-specific assessment is required to evaluate changes in load-penetration curve due to consolidation and to assess the benefits and risks.

This thesis reports the studies of the effects of consolidation on the spudcan installation in carbonate silty soil, carried out with two complementary approaches: physical and numerical modelling.

Spudcan vertical penetration and the influence of a period of consolidation is studied first. Experimental PIV visual tests combined with full spudcan model penetrations,
carried out both on kaolin clay and natural silty clay in a geotechnical centrifuge, offer a first understanding regarding the change in failure mechanism following a pause in penetration and the soil response upon further penetration. A framework is proposed to link the settlement accumulated in consolidation, the increase in capacity, the severity of capacity reduction and the extent of the improved zone to the length of the pause. The study is then expanded numerically: the implementation of a hypoplastic constitutive model for structured clays in a finite element code along with a Large Deformation Finite Element (LDFE) technique allows, first of all, the effects of installation to be captured. Coupled pore fluid-effective stress analyses also enable an investigation of excess pore pressure build-up/dissipation and resulting void ratio and soil shear strength variation. Different soil sensitivities are demonstrated to affect the increase in bearing capacity post-consolidation and the behaviour upon further penetration.

A period of consolidation does not only affect the response under vertical loading. The increase in spudcan capacity under vertical, horizontal and moment loading is defined through a widened numerical study. The adoption of the same hypoplastic model allows the effects of installation to be modelled, by taking into account the outcome of vertical penetration prior to the three-dimensional investigation. Consolidation is demonstrated to enlarge the combined capacity of the footing, at a rate which is proportional to the consolidation duration. Explanations for such an increase are provided on the basis of excess pore pressure distributions and enlarged failure mechanisms. A comparison with experimental centrifuge tests is also offered, which shows the same qualitative trend in terms of bearing capacity increase.

Finally, an introduction to the soil-spudcan response to vertical cyclic loading is presented, through the implementation of the intergranular strain concept within the frame of hypoplasticity for structured clays. This is to investigate the reliability of the increases in capacity and stiffness under cyclic loading resulting from metocean conditions. Consolidation is demonstrated to affect the very small strain behaviour, by modifying the excess pore pressure distribution and ultimately reducing the amount of spudcan accumulated settlement.

In conclusions, the thesis provides a theoretical framework for a safe installation of spudcan foundations in intermediate soils. Its relevance lies in the investigation of both vertical penetration and three-dimensional loading cases, with regard to the effects of a period of consolidation on the soil-structure interaction.
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Nomenclature

\( \dot{\varepsilon} \)  Euler stretching tensor
\( \dot{\sigma} \)  objective stress rate
\( \dot{L} \)  fourth-order constitutive tensor
\( \dot{N} \)  second-order constitutive tensor
\( \Delta V \)  vertical load cyclic amplitude
\( \Delta q \)  cyclic deviator stress
\( \Delta u \)  excess pore pressure variation
\( \Delta w \)  settlement
\( \beta_r \)  intergranular strain stiffness degradation parameter
\( \gamma' \)  submerged unit weight of the soil
\( \gamma_w \)  unit weight of water
\( \delta \)  intergranular strain
\( \varepsilon \)  strain
\( \dot{\varepsilon}^d \)  damage strain rate
\( \dot{\varepsilon}^s \)  shear strain rate
\( \dot{\varepsilon}^v \)  volumetric strain rate
\( \varepsilon_a \)  axial strain
\( \varepsilon_{som} \)  swept-out-memory strain
\( \theta \)  rotation of the footing
Nomenclature

$\kappa^*$  slope of the unloading-reloading line

$\lambda^*$  slope of the normally compressed line

$\nu$  control parameter on the shear modulus

$\rho$  shear strength gradient

$\sigma'_h$  horizontal effective stress

$\sigma'_v$  vertical effective stress

$\varphi_c$  critical state friction angle

$\chi$  ratio peak tensile over compressive capacity

$\chi_s$  intergranular strain stiffness degradation parameter

$A$  volumetric/shear strain effect on sensitivity degradation

$a$  acceleration level

$A_g$  intergranular strain initial stiffness parameter

$A_s$  spudcan area

$B_q$  excess pore pressure ratio

$c_v$  coefficient of consolidation

$D$  spudcan diameter

$D_e$  deformation direction

$e$  void ratio

$e_s$  yield surface eccentricity in HM plane

$f$  frequency

$f_d$  scalar factor of the hypoplastic constitutive law

$f_s$  scalar factor of the hypoplastic constitutive law

$G$  very small strain stiffness

$g$  gravitational acceleration
Nomenclature

$G_0$  stiffness upon complete strain path reversal

$G_M$  stiffness upon complete intergranular strain alignment

$G_T$  stiffness upon $90^\circ$ strain path change in direction

$H$  horizontal load

$h$  horizontal displacement of the footing

$h_0$  peak ratio horizontal over vertical capacity

$K$  gradient of penetration resistance

$k$  sensitivity degradation rate

$K_0$  earth pressure coefficient at rest

$k_w$  soil permeability

$L$  PIV analysis patch size

$M$  moment load

$m_0$  peak ratio moment over vertical capacity

$\mu_v$  coefficient of volume compressibility

$M_{cs}$  slope of the critical state line

$m_{rat}$  ratio $G_T/G_0$

$N$  intercept of the isotropic compression line at $p' = 1$ kPa

$N_c$  bearing capacity factor

$N'_c$  effective bearing capacity factor

$n_g$  intergranular strain initial stiffness parameter

$N_{ball}$  bearing capacity factor for piezoball penetrometer

$N_{T-bar}$  bearing capacity factor for T-bar

$O_c$  position of the critical state line

$OCR$  over-consolidation ratio
Nomenclature

\[ p \] effective stress
\[ p' \] mean effective stress
\[ p_c \] pressure at critical stress
\[ p_e \] Hvorslev equivalent pressure
\[ p_r \] reference stress
\[ Q \] non-vertical force term
\[ q \] bearing pressure
\[ q_{cons} \] pressure applied in consolidation
\[ q_f \] failure deviator stress
\[ q_{max} \] peak deviator stress
\[ q_{mean} \] mean deviator stress
\[ q_{nom} \] nominal bearing pressure
\[ q_{peak} \] bearing capacity peak recorded in re-penetration
\[ q_{ref} \] pressure applied in continuous penetration
\[ R \] elastic radius
\[ r \] bulk modulus in isotropic compression at NCL over shear modulus
\[ s \] soil sensitivity
\[ s_f \] final sensitivity
\[ s_p \] patch spacing
\[ s_u \] undrained shear strength
\[ s_{ini} \] initial sensitivity
\[ s_{u,ini} \] initial intact undrained shear strength
\[ T \] dimensionless consolidation time
\[ t \] dimensional time

XXVIII
Nomenclature

\( u \)  excess pore pressures

\( V \)  vertical load

\( v \)  penetration rate

\( V_0 \)  vertical bearing capacity

\( V_0^* \)  increased vertical bearing capacity

\( v_f \)  footing penetration velocity

\( v_h \)  horizontal soil velocity

\( v_r \)  resultant soil velocity

\( V_s \)  spudcan normalised velocity

\( v_v \)  vertical soil velocity

\( V_{mean} \)  mean vertical load

\( w \)  penetration depth
Chapter 1

Introduction and background

1.1 Introduction

1.1.1 Jack-up mobile platforms

Jack-ups are widely used in offshore engineering, deployed in shallow waters not usually deeper than 150 m to serve different purposes: from preliminary investigations in oil and gas industry to the installation of other offshore structures such as wind turbines. To date it is estimated that some 275 jack-ups are being used worldwide, making it the most employed offshore mobile platform (Rigzone 2016). These mobile drilling units mainly consist of a buoyant hull that supports the drilling apparatus and is equipped with a number of truss or tubular legs (Figure 1.1), which can be mobilised vertically. The bottom end of each leg presents a footing known as a spudcan (Figure 1.2), with circular section of diameter of approximately 6 m in offshore wind farm installation vessels to 14 – 20 m in oil and gas applications. Several slightly different geometries are used in industry, as shown in Figure 1.3, to facilitate spudcan installation and improve the stability.

Jack-up units can either sail independently or be transported via tug boats to the installation location, with their legs raised up above the water level. Once on location, the process of installation is commenced by lowering the legs to the sea bed level and starting the preloading procedure. At this stage the hull is jacked up above the sea level, before water is pumped into the ballast tanks (Figure 1.4a). The vertical load $V$ provided by the ballast tanks, along with the self-weight of the structure, causes the legs to penetrate into the soil. When the hull approaches the sea level again, water from the tanks is dumped into the sea to allow the hull to be jacked up again (Figure 1.4b) and the procedure is repeated by pumping in more water until the target preload value $V_{preload}$ is reached and held for a certain time (Figure 1.4c). Finally, the ballast tanks are emptied.
Figure 1.1: Example of a jack-up hull equipped with three truss legs
and a gap between water level and hull is established to ensure safety conditions against storms (Figure 1.4d).

### 1.1.2 Spudcan penetration

The spudcan penetration generates excess pore pressures while advancing in the soil. In sandy soils these are quickly dissipated, whereas in clayey and intermediate soils excess pore pressures will be accumulated, due to the lower permeability values. A gradual dissipation of these excess pore pressures is observed if the preloading process is interrupted and the vertical load is held constant (Figure 1.5). During this stage, consolidation in the soil around the footings will occur, with a reduction in void ratio and a consequent increase in strength and stiffness of the soil.

Depending on the length of load-hold and the characteristics of the soil, either detrimental or beneficial consequences can be expected. On one hand, consolidation can be responsible for triggering rapid and uncontrolled leg penetration, as the spudcan rapidly advances through the consolidated soil (red vertical profile upon further penetration in Figure 1.5). A possible scenario, although usually associated with multi-layered soil stratigraphy such as sand-over-clay or strong-over-weak clay (ISO 2012), is known as punch-through. In a single-layered case the footing is sitting on a very stiff and strong layer of soil — described as ‘man made’ in ISO (2012) when
Figure 1.3: Examples of different spudcan shapes adopted in offshore engineering (after ISO 2012)
Figure 1.4: Schematic of jack-up installation procedure
Chapter 1. Introduction and background

Preload, $V$

Excess pore pressures, $\Delta u$

Figure 1.5: Schematic of preload vertical profile involving a load-hold period and following distribution of excess pore pressures
generated by consolidation and not as a consequence of soil stratigraphy — which will offer an enhanced resistance upon further penetration. The risk is that the spudcan may quickly advance into the softer layer underneath, penetrating with no control for several meters in a few seconds through the thin layer of stronger soil. Intuitively, this accident can have dramatic consequences: from the damage of the affected leg to the collapse of the whole structure, with risks for the people working on the structure, the environment and the economic investments. Reports by Brennan et al. (2006), Hunt (2008), Jack et al. (2013), and Osborne et al. (2006) show the high frequency of such accidents, with geotechnical issues being the leading cause behind the failure.

On the other hand though, the strengthening due to consolidation could be proven not to be solely localised on a thin layer beneath the spudcan, but spread instead to a wider region and reliable upon further penetration and under operational loading in time (green vertical profile upon further penetration in Figure 1.5). In this sense, a pause might represent a beneficial solution actively sought to increase the footing capacity at a certain depth, as reported in Amodio et al. (2015) for a jack-up installed on carbonate silty soils in the waters off the north coast of Tasmania, Australia. This implies that the spudcan installation depth could be reduced when the available leg length is a limiting factor in installation.

A monotonic increase of the preload and steady spudcan displacement can only be thought as a hypothetical scenario. In fact, the installation procedure as described above clearly reveals this as a discontinuous process, with an alternation over time of the load applied to the footings (Figure 1.6) that has the potential to create the conditions for either of the scenarios outlined above. Further, less likely storm events and harsh weather conditions should be taken into account as another possible reason to temporarily interrupt the installation.

1.1.3 Spudcan behaviour in operational conditions

Not only are the effects of consolidation relevant for the installation procedure, but also for the operational conditions. Once the installation is completed, the spudcan provides a combined bearing capacity in terms of Vertical, Horizontal and Moment ($VHM$) loading which can be imagined in the $VHM$ space as a function of the bearing capacity $V_0$ (Figure 1.7). As dissipation continues over time and the excess pore pressures are completely dissipated, further void ratio reduction and strengthening of the soil are observed. Hence, when compared to the situation immediately after installation, the footing will have in the long term a higher combined $VHM$ bearing capacity (Figure 1.7). The reliability of this increased capacity, however, needs to be assessed over time against combined cyclic loading arising from metocean actions, such as waves,
Chapter 1. Introduction and background

Figure 1.6: Schematic of a typical spudcan preload force and penetration depth time history during jack up installation
wind and currents.

1.2 Background

Previous studies on the effects of consolidation on the spudcan performance are scarce and did not focus on intermediate soils, leaving a critical gap in the current literature. Also, carbonate materials are often encountered in offshore silty soils (Erbrich 2005); however, specific recommendations in ISO (2012) for installation on carbonate soils are not provided; instead, only potential unexpected behaviours are flagged, inviting to address such soils with care. ISO (2003) also reports the absence of general design procedures for foundations in carbonate soils and highlights that acceptable design methods remain highly site specific and dependent on local experience.

The interest for this study mainly comes from the frequency and high consequence of the accidents in jack-up installations reported (Brennan et al. 2006; Hunt 2008; Jack et al. 2013; Osborne et al. 2006). Brennan et al. (2006) in particular, reported the contribution of set-up in soft clays to punch-through, where a pause in preloading as brief as a few hours was shown to have contributed to the failure. On the other hand, case studies by Erbrich (2005) and Amodio et al. (2015) showed how the static weight of the jack-up was purposely used to allow consolidation and strength gain of the soil, improving the long-term stability assessment. In either case, the phenomenon needs to be understood.

1.2.1 Spudcan installation

Despite the case histories of jack-up operation and set-up effects, relevant research is still scarce. Barbosa-Cruz (2007) carried out a limited number of tests on kaolin clay in a geotechnical centrifuge, offering a first insight on the connection between a pause in spudcan installation and increased bearing capacity upon restart of penetration. Bienen and Cassidy (2013) presented a more extensive experimental study for spudcan penetration on kaolin clay. The footing model was equipped at the bottom tip with a pore pressure transducer to monitor the excess pore pressure buildup during penetration and its dissipation while consolidating. The peak in bearing capacity upon re-penetration was shown to be proportional to the length of the load-hold and a less significant peak was visible when only half of the load was held for the same given time (Figure 1.8a). Interestingly, some of the tests also showed a nose-shaped bearing capacity profile after consolidation, with peak followed by a reduction before the load restarted to increase (Figure 1.8b), indicating a potential risk of punch-through. Although useful to quantify the magnitude of bearing capacity peak and the associated risk of punch-through as the bearing capacity reduces after the peak, Bienen and Cassidy
Chapter 1. Introduction and background

Figure 1.7: Schematic of combined $VHM$ capacity expansion due to a long period of consolidation
(2013) were unable to provide further information regarding the change in failure mechanism associated with a consolidation stage. This becomes possible when a Particle Image Velocimetry (PIV) technique is adopted, which was pioneered by White et al. (2003) in the centrifuge environment. The procedure involves a half footing model sliding along a Perspex window with photos of the event taken at high frequency. Centrifuge studies such as Hossain et al. (2005) adopted this technique to show the failure mechanism in kaolin clay for continuous spudcan penetration. Hossain et al. (2005) revealed an initial surface heave mechanism at shallow depth and the partial formation of a cavity (Figure 1.9a). As the penetration continued, the onset of a second mechanism was observed, with soil beginning to flow around the spudcan edges and further expansion of the cavity (Figure 1.9b). Ultimately, the limiting cavity depth was reached and a third, fully localised mechanism was triggered (Figure 1.9c), where only flow-around could be observed, which limited the depth of the cavity and provided a seal above the spudcan. Hossain and Randolph (2009a) expanded the findings from Hossain et al. (2005), using LDFE analyses, validated against centrifuge data. Different bearing capacity factors to be associated to the change in failure mechanism were proposed, with classical factors related to the shallow mechanism and values that account for penetration depth and soil above the spudcan for the flow around mechanism.

The described experimental techniques can offer information regarding increase in
bearing capacity and changes in failure mechanism. However, a limitation of the physical modelling is represented by the inability to accurately define the gain of soil shear strength due to consolidation. When a numerical approach is adopted instead to model the problem, a close monitoring of the variation of excess pore pressures and void ratio can offer further insight into the changes in shear strength, provided the implementation of coupled effective stress pore fluid analyses. Numerical studies relative to continuous spudcan penetration in kaolin clay, implemented as elastic-perfectly plastic Tresca soil model and including the effects of strain rate and strain softening (Hossain and Randolph 2009b; Zhang et al. 2014), are unable to capture this phenomenon, due to the simulation of total stress analyses. More recently, Wang and Bienen (2016) presented a numerical work for spudcan installation in kaolin clay involving a pause in penetration, using a Large Deformation Finite Element (LDFE) technique, and implementing coupled effective stress-pore fluid analyses, which allowed the variation of excess pore pressures to be evaluated. The results showed the same qualitative trend of the experimental observations of Bienen and Cassidy (2013) in terms of bearing capacity peak post consolidation.

In terms of constitutive models capable of reproducing effective stress-pore fluid behaviour, Mašín (2007, 2014) presented a hypoplastic model for clays with meta-stable structure which can simulate coupled analyses. In contrast to any conventional elasto-plastic model, it also take into account the increasingly non linear behaviour of the soil, i.e. offering more accuracy when in small strain analyses. Further, it also accounts for the effects of sensitivity, to capture the degree of remoulding due to soil straining. Finally, the codes required for the implementation of the model in the finite element code Abaqus were already available free-source online through the soilmodels.info project by Gudehus et al. (2008). This ultimately allowed a large amount of time to be saved. This model was adopted in the present research for the first time to model the response of offshore foundations, for its suitability to describe the behaviour of both carbonate silty clay and kaolin clay.

1.2.2 Combined VHM capacity

Force resultant models to describe the soil-spudcan interaction and the combined bearing capacity under multi-directional loading have been proposed in the past, following either an experimental or numerical approach. So-called yield surfaces were defined in the VHM space to identify the admissible loading states. The studies focused either on the response in sand (Bienen et al. 2006; Cassidy and Houlsby 1999) or in clay (Martin and Houlsby 2000, 2001; Zhang et al. 2011, 2013). No data exist that define the bearing capacity of spudcans under multi-directional loading for intermediate silty soils.
The above mentioned predictive models did not investigate, however, the effects of consolidation on the combined bearing capacity and, in case of numerical studies, neglected the effects of installation. The latter was later addressed by Zhang et al. (2014), with the implementation of a modified Tresca model with strain softening for clay that highlighted a reduction in VHM capacity when the installation effects were taken into account. To date, no investigations of the effects of consolidation on the spudcan combined bearing capacity are available.

### 1.2.3 Soil-spudcan cyclic response

The cyclic behaviour of the soil-spudcan interaction assumes remarkable importance when considering the environment in which jack-up rigs are deployed. ISO (2012) reports crucial aspects related to cyclic loading to be taken into account, such as the displacements accumulated by the foundation, the loss in soil strength due to accumulation of excess pore pressures (liquefaction) and stiffness degradation. Experimental evidences of spudcan cyclic response in a centrifuge environment were provided by Dean et al. (1998) for kaolin clay and Ng and Lee (2002) for sand. The former discussed the effects of slow and rapid loading on the spudcan displacements; the latter showed the influence of pre-load on the accumulated settlement and the changes in stiffness due to densification. Investigations of the effects of consolidation on the cyclic response of spudcan footing are not available in literature. Kohan et al. (2016) investigated the effects of cyclic loading on spudcan extraction experimentally; centrifuge tests on normally consolidated kaolin clay revealed the impact of different mean pull-out loading and cyclic amplitude on the ability to successfully extract a deeply embedded spudcan.

When trying to model the cyclic behaviour numerically, it is extremely important to properly define the response within the yield surface. The assumption of elastic response can be satisfactory for monotonic analyses, but it fails to correctly model the cyclic behaviour. To overcome the problem, Vlahos et al. (2006) presented the development of a non-linear spring based on hyperplasticity, incorporated into a surface plasticity framework and calibrated with cyclic loading experiments on spudcan footing. In this sense, the hypoplastic model represents a valid option to model the cyclic problem, for it considers an increasingly non-linear behaviour within the so-called state boundary surface. However, in problems simulating frequent strain path reversal, such as cyclic loading, the integration of the intergranular strain concept (Niemunis and Herle 1997) is necessary to avoid excessive strain accumulation, phenomenon known as ‘ratcheting’. The idea behind the intergranular strain is to model not only the re-arrangement of the skeleton particles due to straining, but also the interface layer between the grains.
Chapter 1. Introduction and background

Figure 1.9: Evolution of the failure mechanism for a spudcan continuous penetration in kaolin clay: (a) surface heave mechanism; (b) beginning of flow around and further cavity expansion; (c) fully localised mechanism (after Hossain et al. 2015)
Depending on the straining direction, different stiffness values at very small strains are assigned to the soil. The theory was first developed by Niemunis and Herle (1997) for cohesionless soils and later applied to clay by Mašín (2014).

1.3 Aim of the thesis

This thesis aims to investigate the effects of consolidation on the spudcan performance in intermediate soils. Effects on purely vertical penetration are evaluated first, offering an overview of the well characterised kaolin clay before presenting a comprehensive investigation for intermediate soils. Then, the study moves into a three-dimensional analysis of the problem, in order to examine the combined $VHM$ bearing capacity of the spudcan and demonstrate how this is affected by a period of consolidation. An introduction to the cyclic reliability under vertical loading is finally presented. With regard to the vertical case, the following is investigated:

- Soil response to a pause in vertical penetration during installation;
- Increase in vertical bearing capacity in re-penetration post-consolidation;
- Failure mechanism governing the re-penetration post-consolidation;
- Increase in shear strength of the surrounding soil post-consolidation;
- Influence of different soil characteristics;
- Predictive framework for spudcan installation in silty clayey soils.

The capacity under multi-directional loading is explored through the investigation of:

- The increase in $H$ and $M$ capacity due to a period of consolidation, and a comparison to that under $V$ loading only;
- The changes in translational and rotational failure mechanisms due to a period of consolidation;
- The influence of the pore pressure response on the combined $VHM$ bearing capacity;
- The increase of the $VHM$ capacity, defined in terms of an expanded yield surface;

The investigation of cyclic loading behaviour involves:
• The introduction of a numerical constitutive model to describe the soil-spudcan cyclic response;

• The analysis of excess pore pressure variation during cyclic loading;

• The analysis of accumulated settlements by the spudcan, when subjected to different loading paths.

Such goals shall be here pursued with a combination of experimental and numerical approaches, namely geotechnical centrifuge experiments at enhanced gravity and finite element analyses.

1.4 Thesis outline

The thesis is presented according to the ‘series of papers’ format, accordingly to the UWA Graduate Research School guidelines. Therefore, after the present introduction, Chapter 2 to Chapter 5 are presented in the format of journal papers. Chapter 6 offers a preliminary insight into the cyclic problem and for this reason has not been submitted to any journal. A conclusive Chapter 7 summarises the major findings of the thesis and outlines potential future research directions.

It should be noticed that technical details about experimental and numerical techniques adopted in the thesis are not described in this introductory Chapter 1, for relative information is specifically provided in Chapter 2 to Chapter 6, according to its technical content.

The main body of the thesis addresses each topic as follows:

Chapter 2: Observing the effects of sustained loading on spudcan footings in clay. An experimental investigation of the effects of load-hold on spudcan penetration in kaolin clay is presented. Visual PIV tests carried out in a geotechnical centrifuge reveal the change in failure mechanism caused by a consolidation period. The effect of the duration of the load-hold period on the re-penetration response is also demonstrated.

Chapter 3: Effects of consolidation under a penetrating footing in carbonate silty clay. After the preliminary considerations exposed in Chapter 2 for kaolin clay, Chapter 3 reveals the effect of consolidation on spudcan penetration in intermediate soils through experiments. A thorough investigation of the problem is presented, by combining PIV tests with full spudcan model tests, both carried out in a geotechnical centrifuge. Results for carbonate silty clay are compared with those for kaolin clay presented in Chapter 2.
Chapter 4: Numerical modelling of the effects of consolidation on jack-up spudcan penetration.

Numerical modelling is introduced to enable detailed analysis of the phenomena identified in Chapters 2 and 3. The implementation of a hypoplastic constitutive model for structured clays in a finite element code, combined with LDFE coupled analyses and Remeshing and Interpolation Technique with Small Strain (RITSS) strategy allows a detailed investigation of the entire penetration-consolidation-penetration process.

Chapter 5: Numerical modelling of the effects of consolidation on the undrained spudcan capacity under combined loading in silty clay.

The increase in $H$ and $M$ capacity due to a period of consolidation, following pure horizontal displacement and rotation, is shown. Different failure mechanisms are demonstrated to govern the process and excess pore pressures distributions are proven to affect the combined bearing capacity. A detailed parametric study involving a series of tests with increasing period of consolidation quantifies the increase in combined capacity, in terms of the parameters governing the size and shape of the yield surface.

Chapter 6: An introduction to the cyclic loading response of spudcan foundations in carbonate silty clay.

The hypoplastic model for structured clays introduced in Chapter 4 is expanded with the intergranular strain concept to correctly model the soil response subjected to frequent strain path reversal, such as cyclic loading. The calibration of the numerical model is presented first, before moving into the soil-spudcan cyclic response. Numerical simulations show the effects of consolidation on the accumulated settlements and excess pore pressure distributions, when the footing is subjected to different loading paths.

Chapter 7: Concluding remarks.

The main findings of the thesis are summarised and potential future research arising from this study proposed.
Chapter 1. Introduction and background

References


Chapter 1. Introduction and background


Chapter 2

Observing the effects of sustained loading on spudcan footings in clay

2.1 Abstract

Spudcan foundations of mobile jack-up rigs are penetrated into the seabed under seawater ballast preload, which is shed prior to rig operations commencing. During pauses in the installation process and during operation, soil beneath the spudcan foundations stiffens and strengthens due to consolidation. On the application of further loading or during spudcan extraction, this causes increased resistance, which in extremis can result in punch-though type failure. This note reports results from a series of experiments with Particle Image Velocimetry (PIV) measurements that were performed in a drum centrifuge to facilitate observation of the effects of a load-hold period on the soil movements around a model spudcan during subsequent further loading. The results show that the dimensionless load-hold period dominates the enhancement in the penetration resistance, due to significantly more soil being mobilised following a long load-hold period. These observations might be useful to (i) predict the enhancement in bearing capacity factor due to a load-hold period during installation or operation and (ii) predict the footing extraction resistance during jack-up re-deployment.

2.2 Introduction

Spudcan foundations of mobile jack-up rigs are penetrated into the seabed under seawater ballast preload, which is shed prior to rig operations commencing. This process is often discontinuous, with breaks in preloading caused by inclement weather and other technical problems. Figure 2.1 illustrates schematically a time history of the process of jack-up installation. Soil beneath the footing is subjected to significant loads during
these pauses, causing consolidation of the soil around the footing, resulting in strengthening and stiffening. In practice offshore, hold periods as short as a few hours have been reported to have resulted in ‘punch-through’ type failure (defined as ‘rapid, uncontrolled vertical leg movement due to soil failure in strong overlying weak soil’ in ISO 2012) upon subsequent further loading even in soil where punch-through was not an anticipated risk, such as in normally consolidated clay (Brennan et al. 2006).

For this scenario, Bienen and Cassidy (2013) provided design charts — back-calculated from centrifuge experiments — to allow estimation of the enhancement of the effective bearing capacity factor, \( N_c^* \), for a range of dimensionless load-hold periods. The charts cover the range anticipated for short-term pauses in jack-up installation and longer-term operational periods of up to a few years. Such charts are useful to predict the increase in penetration resistance due to pauses in the installation process and operation prior to spudcan extraction. This note reports results from a series of experiments with Particle Image Velocimetry (PIV) measurements (White et al. 2003) that were performed in a drum centrifuge to facilitate observation of the effects of a load-hold period on the soil movements around a model spudcan, including during subsequent further loading.

### 2.3 Experimental set-up and procedure

#### 2.3.1 Apparatus

The investigation was conducted using the drum centrifuge at the University of Western Australia (UWA) (Stewart et al. 1998). Half of a model spudcan with diameter, \( D = 50 \text{mm} \) and geometry similar to those utilised in the field (Figure 2.2) was used to represent a jack-up footing. Under accelerations of 200 g the model represented a prototype diameter of 10 m.

The half footing was placed against the transparent acrylic window of a strongbox. A closed cell foam seal was bonded to the mating surface of the footing and was lubricated with petroleum jelly. This served to allow the footing to move smoothly along the surface of the window with minimal frictional resistance whilst maintaining a seal between the footing and window of the strongbox.

PIV measurements were captured using the system described by Stanier and White (2013). In brief, this system consists of a machine vision camera (AVT Prosilica GC2450C) with 5-megapixel resolution, large LED panels for illumination, fibre-optic rotary joint for the transfer of data from the centrifuge to the control room in-flight and custom control software. Images were captured at a rate of 5 frames per second.
Chapter 2. Observing the effects of sustained loading on spudcan footings in clay

Figure 2.1: Typical time history of preload force and penetration during a discontinuous jack-up installation process
Chapter 2. Observing the effects of sustained loading on spudcan footings in clay

Figure 2.2: Dimensions of model spudcan footing

Dimensions presented as: (model scale; prototype scale)
RP: Reference Point (for load measurement)
2.3.2 Sample preparation

Commercially available kaolin clay was used to model the soil and two samples were normally consolidated under 200 g in the centrifuge. After consolidation and just prior to testing, the transparent acrylic window was carefully removed from the sample, allowing artificial seeding to be applied to the exposed surface of the model on which PIV measurements were conducted. The density of this seeding, or Artificial Seeding Ratio (ASR) was optimised following the procedure proposed by Stanier and White (2013). This ensured that the PIV measurements were of an optimal precision for all tests.

2.3.3 Test procedure and summary

Figure 2.3 is a schematic of the typical effect of a load-hold period on the bearing pressure-penetration response. The loading path being modelled here, with respect to the in-field condition, is also highlighted in Figure 2.1. The initial stages of the tests were performed under displacement control at a constant rate until the reference point (taken as the lowest depth of the maximal diameter of the spudcan) reached either 0.5 or 1.0 D. The penetration rate, \( v \), was determined such that the normalised penetration rate, \( \frac{vD}{c_v} \), was 95, similar to the corresponding full model tests reported in Bienen and Cassidy (2013) and \( \geq 30 \) to ensure undrained behaviour (Low et al. 2008). The coefficient of consolidation, \( c_v \), was taken as 2.6 m²/year at 0.5D depth and 3.5 m²/year at 1.0D after Bienen and Cassidy (2013).

After reaching the start depth for the load-hold period, the actuator was switched to load control (except for the reference case with no load-hold period). In this mode, feedback from a load cell on the shaft of the footing was used as an input to the actuator control software and the position of the actuator was constantly adjusted to maintain a near-constant reading on the load cell during the load-hold period. Following the load-hold period the actuator was switched back to displacement control at the same constant penetration rate as that used prior to the consolidation. The footing was then further penetrated until it had reached a total depth of 2.0D (100 mm). Table 2.1 provides a summary of the tests performed, including key relevant test data.

Following spudcan testing, T-bar penetrometer (5 by 20 mm) tests were performed at 200 g in the two strongboxes used in the investigation (away from the footing test sites), which are presented in Figure 2.4. By assuming an intermediate T-bar factor, \( N_{T-bar} \) of 10.5 (Low et al. 2008), a linear best fit was found with a mudline strength of 0.5 kPa and a prototype gradient of 1.6 kPa/m.
Chapter 2. Observing the effects of sustained loading on spudcan footings in clay

Figure 2.3: Schematic diagram of the test process adopted and typical effect of a consolidation period on load penetration curve observed in experiments

<table>
<thead>
<tr>
<th>Test</th>
<th>Depth, w/D</th>
<th>$T = c_v t / D^2$</th>
<th>Average Applied Pressure (kPa)</th>
<th>$\Delta w/D$ due to consolidation</th>
<th>Peak $N^*_c$</th>
<th>w/D at peak</th>
<th>Ref. $N^*_c$</th>
<th>Peak $N^<em>_c$ / Ref. $N^</em>_c$</th>
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<td>K1</td>
<td>N/A - Reference test</td>
<td></td>
<td></td>
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<tr>
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<td>0.72</td>
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<tr>
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<td>8.11</td>
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<td>7.45</td>
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</tr>
<tr>
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<td>0.05</td>
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<td>0.19</td>
<td>12.22</td>
<td>1.22</td>
<td>7.33</td>
<td>1.67</td>
</tr>
</tbody>
</table>

Note: Ref. $N^*_c$ is value of reference case at same penetration depth.

Table 2.1: Summary of tests performed
Chapter 2. Observing the effects of sustained loading on spudcan footings in clay

![Graph showing undrained shear strength vs. penetration depth]

Figure 2.4: T-bar penetrometer profiles for the two strongboxes tested assuming $N_T = 10.5$.

### 2.3.4 Data analysis techniques

For the tests with a load-hold period PIV analyses were performed using GeoPIV (White et al. 2003) immediately following completion of the consolidation period and after further penetration equivalent to 0.25 and 0.5 $D$. For comparison, companion analyses were performed at the same depths on images from the reference (no load-hold period) case. A patch size, $L$, equal to 50 pixels was found to give the best compromise between measurement precision and density. Patch spacing, $s_p$, of 10 was adopted in all analyses. Spurious vectors, which typically occurred within the penetration path due to sub-optimal texture caused by the remoulding of soil, were manually removed from the dataset. Image space measurements (pixels) were converted to object space measurements (mm) using close-range photogrammetric correction after White et al. (2003).

### 2.4 Results and discussion

The results shown in this paper are presented in non-dimensional or scale independent forms. The following notation is used throughout: $q_{nom} = V/A_s$ is the nominal bearing pressure where $V$ is the vertical load and $A_s$ is the area of the spudcan in model scale. $w/D$ is the normalised penetration depth where $w$ is the penetration depth and $D$ is the
spudcan diameter. \( N_c = q_{nom}/s_u \) is the effective bearing capacity factor where \( s_u \) is the original undrained shear strength from the T-bar fit at the relevant depth. As the soil undergoes significant changes in undrained shear strength and failure mechanism, as a consequence of a load-hold period, the notion of \( N_c^* \) is adopted to better describe the changes in bearing capacity factor for the tests involving consolidation. \( T = cvt/D^2 \) is the dimensionless consolidation time where \( cv \) is the coefficient of consolidation and \( t \) is the dimensional time. PIV measurements are presented in the form of velocity fields, where \( v_r \), \( v_h \) and \( v_v \) are respectively the resultant, horizontal and vertical velocities of the soil, which are normalised by the footing penetration velocity, \( v_f \).

### 2.4.1 Load-penetration response

Figure 2.5a shows the measured penetration resistance, \( q_{nom} \), with respect to penetration depth, while Figure 2.5b presents the effective bearing capacity factor \( N_c^* \). The penetration resistance increases linearly with depth until the onset of the load-hold period. Correspondingly the effective bearing capacity factor, \( N_c^* \), increases initially very rapidly until a transition from shallow to deep failure occurs between 0.1 and 0.2 \( D \) — as predicted by Hossain and Randolph (2009). During the load-hold periods the footing continues to settle, hence \( N_c^* \) reduces as the soil strength used to back-calculate \( N_c^* \) rises. After the load hold period, tests K2 and K3 where \( T = 0.01 \) at \( w/D = 0.5 \) and 1.0 respectively, show a similar enhancement in \( N_c^* \) of 8-9\% (Peak \( N_c^*/\text{Ref.} N_c^* \) as noted in Table 2.1). In contrast, test K4 where \( T = 0.05 \) at \( w/D = 1.0 \), yielded an enhancement of \( N_c^* \) of 67\%. Figure 2.6 shows that these enhancements are broadly comparable to those measured by Bienen and Cassidy (2013), providing some confidence in the efficacy of the PIV model-based measurements. In test K4, \( N_c^* \) also remained \( \sim 20\% \) higher for the remainder of the penetration than in the reference case of test K1, as occurred in similar tests reported by Bienen and Cassidy (2013).

### 2.4.2 Velocity fields following a load-hold period

The left hand sides (LHS) and right hand sides (RHS) of Figures 2.7, 2.8 and 2.9 compare the normalised velocity fields for tests K1 (no load-hold period) and K4 (the longest load-hold where \( T = 0.05 \) and initial embedment \( w/D = 1.0 \)) at depths following the end of the load-hold period of 0.0, 0.25 and 0.5 \( D \) respectively. Four sub-plots are used to illustrate the velocity fields at each depth: (a) normalised velocity vector field, (b) normalised contours of resultant velocity \( (v_r/v_f) \), (c) normalised contours of horizontal velocity \( (v_h/v_f) \) and (d) normalised contours of vertical velocity \( (v_v/v_f) \). From these figures the following observations can be made:
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Figure 2.5: Profiles of (a) measured nominal penetration resistance ($q_{nom}$) and (b) back calculated effective bearing capacity factor ($N^*_c$)

Figure 2.6: Effective bearing capacity factor ($N^*_c$) from PIV experiments compared to the data of Bienen and Cassidy (2013) and Barbosa-Cruz (2007)
1. Significantly more soil is mobilised by the spudcan immediately following a long load-hold period (K4) than for the reference case with no load-hold period (K1). This results in the greater penetration resistance evident in Figure 2.5a and the initial peak in $N_c^*$ seen in Figure 2.5b;

2. The extent of the soil mobilised in the load-hold period test (K4) reduces with increasing penetration, while for the reference case (K1) it is broadly consistent. The reduction in the extent of soil mobilised by further penetration mirrors the post-peak reduction in $N_c^*$ seen in Figure 2.5b;

3. The similarity in the extent of the velocity fields at a depth of $0.5D$ following the end of this relatively long load-hold period (Figure 2.9), coupled with the remaining 20% enhancement of $N_c^*$, implies that the load-hold period caused a general strengthening of soil in the vicinity of the spudcan. This must be caused by localised consolidation of soil, as indicated by the pore pressure dissipations measured at the spudcan tip by Bienen and Cassidy (2013). Efforts made to measure the volumetric strains (and infer changes in voids ratio) were unfortunately unreliable due to degradation of soil texture immediately beneath the footing, during the load-hold period.

These observations indicate that both the mechanism of soil movement and localised consolidation impact upon penetration resistance. The mechanism of soil movement has the strongest influence on the initial (or peak) enhancement of penetration resistance for this long load-hold period. However, consolidation of a significant volume of soil beneath the footing, causing it to strengthen and stiffen, likely results in the offset in the penetration resistance from the reference case with no load-hold period. This latter effect is only apparent for relatively long dimensionless load-hold periods.

### 2.4.3 Effects of dimensionless time and footing penetration depth

Figures 2.10 and 2.11 present the velocity fields immediately following the end of the shorter load-hold period for tests K2 and K3 ($T = 0.01$ at $w/D = 0.5$ and 1.0 respectively). The significantly reduced extent of the velocity fields and magnitude of the enhancement in $N_c^*$ for both cases compared to test K4 (where $T = 0.05$ at $w/D = 1.0$), indicates that it is the dimensionless consolidation time, $T$, that dominates the initial enhancement of $N_c^*$ over the reference case, as opposed to the initial embedment depth, $w/D$. This is corroborated by the data of Bienen and Cassidy (2013) in Figure 2.6, where no trend with initial embedment is evident.
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Figure 2.7: Comparison of velocity fields immediately following the load-hold period (LHS – no load-hold; RHS – with load-hold) for $T = 0.05$ and $w/D = 1.0$: (a) normalised vectorial velocities; (b) normalised resultant velocity contours; (c) normalised horizontal velocity contours; (d) normalised vertical velocity contours
Figure 2.8: Comparison of velocity fields following $0.25D$ penetration after following the load-hold period (LHS – no load-hold; RHS – with load-hold) for $T = 0.05$ and $w/D = 1.0$: (a) normalised vectorial velocities; (b) normalised resultant velocity contours; (c) normalised horizontal velocity contours; (d) normalised vertical velocity contours
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Figure 2.9: Comparison of velocity fields following 0.5D penetration after following the load-hold period (LHS – no load-hold; RHS – with load-hold) for $T = 0.05$ and $w/D = 1.0$: (a) normalised vectorial velocities; (b) normalised resultant velocity contours; (c) normalised horizontal velocity contours; (d) normalised vertical velocity contours
Figure 2.10: Comparison of velocity fields immediately following the load-hold period (LHS – no load-hold; RHS – with load-hold) for $T = 0.01$ and $w/D = 0.5$: (a) normalised vectorial velocities; (b) normalised resultant velocity contours; (c) normalised horizontal velocity contours; (d) normalised vertical velocity contours
Figure 2.11: Comparison of velocity fields following $0.5D$ penetration after the load-hold period (LHS – no load-hold; RHS – with load-hold) for $T = 0.01$ and $w/D = 1.0$: (a) normalised vectorial velocities; (b) normalised resultant velocity contours; (c) normalised horizontal velocity contours; (d) normalised vertical velocity contours
2.5 Concluding remarks

A short series of centrifuge PIV experiments have been reported investigating the impact of sustained loading on spudcan footings in clay. Short load-hold periods cause a modest increase in penetration resistance compared to a reference case with no load-hold period, due to the mobilisation of additional soil on the application of further loading. Longer load-hold periods exacerbate this effect and in addition cause an offset in penetration resistance of up to 20\%, which remains evident even after further penetration of up to 0.5\,D. This implies that the soil surrounding the footing undergoes significant consolidation during the load-hold period. The observations presented in this note might help to validate methods to predict the enhancement in bearing capacity factor due to a load-hold period during installation or operation (e.g. Bienen and Cassidy 2013).
References


Chapter 3

Effects of consolidation under a penetrating footing in carbonate silty clay

3.1 Abstract

The effects of consolidation under a footing are generally viewed as beneficial due to the resulting increased capacity. Consolidation may also be actively sought because it minimises footing embedment, which can be critical for the installation of mobile offshore jack-ups because available leg length is limited. However, it can also set the platform footing up to subsequently punch through the strengthened zone, with potentially serious consequences. The problem is complex due to the three-dimensional nature of consolidation. Further, footing penetration leaves the soil above heavily remoulded and generates large excess pore pressures below, such that the soil state even prior to consolidation is significantly altered from its in situ conditions. This study has taken an experimental approach to investigate the effects of consolidation around a footing penetrating into carbonate silty clay and, following detailed discussion of the response, offers a framework to predict the changes to the load-penetration curve.

3.2 Introduction

Mobile offshore jack-up drilling rigs are relocated frequently, spending as little as weeks or even days at any one site. At every location, the spudcan footings are penetrated into the seabed until the target bearing capacity is met to support the jack-up. This typically exceeds the self-weight of the platform, with additional preload provided in the form of seawater pumped into ballast tanks, to provide a safety margin against storm loading.
Accident reports (Hunt 2008; Jack et al. 2013; Osborne et al. 2006) alert to the fact that the majority of failures remain related to geotechnical issues, with punch-through (uncontrolled rapid leg penetration) the leading cause. Punch-through is a common risk in layered strata with significant strength differences that may result in extensive damage to the platform. This area has therefore received considerable research attention recently with investigation of both sand overlying clay (Craig and Chua 1990; Hu et al. 2014; Lee et al. 2013a,b; Qiu and Grabe 2012; Teh et al. 2010; Yu et al. 2012) and clay stratification of various strength ratios (Hossain and Randolph 2010a,b).

Punch-through potential, however, can also be man-made (ISO 2012) ‘as a result of soil consolidation occurring during pauses in leg penetration whilst the spudcan is loaded to less than full preload. Such pauses can occur during installation operations or geotechnical investigation from a jack-up prior to full preloading.’ These pauses allow dissipation of excess pore pressures generated during the preceding loading event and hence increase the effective stresses in the soil. However, the effect will be localised such that additional loading, for instance on recommencement of preloading, may lead to the spudcan punching through this strengthened zone. Even relatively short pauses, such as preload holding times of approximately 3 ~ 4 h, have been reported to have contributed to subsequent punch-through failure in the field (Brennan et al. 2006).

The effect of pauses in spudcan penetration and the ensuing consolidation on the subsequent response was investigated experimentally by Bienen and Cassidy (2013) following two preliminary tests with the same apparatus by Barbosa-Cruz (2007). Importantly, these experiments were performed in a geotechnical centrifuge such that stress levels are similar to the prototype situation (Gaudin et al. 2011b). As a result, soil failure mechanisms in the model test are expected to be representative of those in the field, with quantities such as loads and displacements related via established scaling laws (Garnier et al. 2007). In the centrifuge experiments, penetration resistance was found to be enhanced post-consolidation, as expected. The peak resistance as well as the extent of the zone of soil strengthened by the consolidation was demonstrated to depend on the length of the period of consolidation, which is also expected. Punch-through risk, however, is not only determined by the magnitude of peak resistance, but crucially the response following the peak. The gradient of resistance reduction post-peak was identified to be related to the length of the consolidation period and importantly to the load (or pressure) on the spudcan during the pause in penetration. Figure 3.1 illustrates schematically the effect of consolidation on the load-penetration curve of a spudcan footing.

Kaolin clay was chosen in these first series of experiments because of its well-known characteristics because it has been used in centrifuge testing at the University of Western
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Australia for more than two decades. Importantly, the coefficient of consolidation of kaolin clay allows investigation of a range of degrees of consolidation under a penetrating model footing in the centrifuge that represents a large-diameter spudcan in the field. The results were presented in nondimensional form. In particular, the increase in peak bearing capacity, the extent of the improved zone, and the severity of post-peak capacity reduction were provided as design charts relating to the nondimensional time of consolidation, \( T = \frac{c_v t}{D^2} \), where \( c_v \) is the coefficient of consolidation, \( t \) is the consolidation time, and \( D \) is the foundation diameter. This representation was chosen as on the premise of similar soil characteristics, the findings may be viewed as indicative of the response of more permeable soils in the field.

For the range of likely pause durations during in-field jack-up installation (of the order of several hours to approximately 14 days if consolidation is actively sought to minimise penetration depth), the effects of consolidation are expected to influence spudcan behavior in silty soils. Seabed soils offshore Australia often have a high calcium carbonate content, which adds significant challenge to jack-up installation (Erbrich 2005) due to high compressibility and sensitivity to grain crushing, which manifests itself in significant cyclic degradability.

This study therefore investigates the effects of consolidation under a penetrating footing in carbonate silty clay through experimentation in a geotechnical centrifuge. Similarities and differences in behaviour compared with kaolin clay are discussed in detail before design charts are presented that capture the effect of consolidation on the subsequent response of a penetrating footing.

### 3.3 Experimental setup and procedure

Two series of experimental results are drawn on to investigate the problem in detail, the first being model tests with a full spudcan footing in the beam centrifuge (Randolph et al. 1991) and the second tests of a half spudcan model against a Perspex window in the drum centrifuge (Stewart et al. 1998) with a setup that allows Particle Image Velocimetry (PIV) analysis to be performed. The soil sample preparation, testing procedure for both series, and experimental setup for the full spudcan tests is described in the following. For detailed information specific to the PIV setup used in the University of Western Australia drum centrifuge and relevant analysis techniques the reader is referred to Stanier and White (2013), while Stanier et al. (2014) provide details of PIV tests on kaolin clay similar to the carbonate silty clay tests discussed here. All tests were performed at 200 g.
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Figure 3.1: Effect of consolidation on the load-penetration curve of a footing
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3.3.1 Footing model and instrumentation

The model spudcan was the same as used by Zhang et al. (2013, 2014). The spudcan was 60 mm in diameter at its widest section, which is equivalent to 12 m at prototype scale. The footing features five pressure sensors: one pore pressure transducer is integrated in the spudcan tip, and one pair of total and pore pressure transducers each are located on the top and bottom faces of the footing. Because this study focuses on the spudcan response, no attempt was made to model the trusswork leg of a jack-up. Instead, a solid aluminium cylinder was used. A load cell connecting the spudcan and leg with the actuator measured the axial footing loads during the test. Figure 3.2 shows the model spudcan and the experimental setup. A half model spudcan of the same shape but a slightly smaller diameter of 50 mm was used in the PIV tests.

3.3.2 Sample preparation

All new tests reported in this paper were performed on carbonate silty clay, which was recovered from the Laminaria field in the Timor Sea offshore Australia (Erbrich and Hefer 2002; Randolph et al. 1998a,b). For the first series of experiments, with full spudcan penetration, two samples were prepared using minimal added water and consolidated in flight in the beam centrifuge to create normally consolidated profiles. The water table was kept above the sample surface at all times. The samples were consolidated at 200 g for 5 days. Settlement of the sample surface during consolidation was measured using a linear displacement transducer. This confirmed negligible settlement rate after 5 days, and thus near full consolidation prior to commencement of the testing program. The final sample depth was ~150 mm, which allowed spudcan penetration to a depth of $2D$ (where $D$ is the spudcan diameter) without adverse effects.
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from the base of the strongbox (Bienen and Cassidy 2013). The samples for the drum centrifuge PIV tests were prepared in a similar fashion in custom-made strongboxes with a Perspex face.

The undrained soil shear strength $s_u$ was evaluated using a miniature piezoball penetrometer, assuming a factor $N_{Ball}$ of 10.5, as shown in Figure 3.3a. It is well represented by a linear fit with a gradient $\rho = 2.2 \text{kPa/m}$ ($2.9 \text{kPa/m}$ in the PIV tests). This gives a soil strength ratio $s_u/\sigma'_v = 0.4 (0.5)$, considering a submerged unit weight of the soil $\gamma' = 5.4 \text{kN/m}^3$. Episodes of penetrometer cycling were used to establish the sensitivity of the reconstituted soil samples, $s = 2.9$.

Figure 3.3b provides the $c_v$ profile obtained from Rowe cell testing, considering a submerged unit weight of the soil $\gamma' = 5.4 \text{kN/m}^3$. The choice of soil allowed a wide range of degrees of consolidation to be investigated. The results are presented in terms of nondimensional consolidation time $T = c_v t / D^2$, which allows the findings to be viewed independent of the absolute time of consolidation, $t$.

3.3.3 Experimental program and procedure

The full spudcan penetration testing program comprised six tests with varying periods of consolidation at different penetration depths, as summarised in Table 3.1. In addition, one reference test was performed without interruption of the penetration process. The test procedure was as follows:

1. Undrained penetration under displacement control at a rate of $0.1 \text{ mm/s}$ (with the normalised velocity $V_s = vD/c_v > 100$ indicating an undrained response as
discussed by Cassidy 2012; Chung et al. 2006; Finnie 1993) to the depth targeted for the consolidation phase;

2. Consolidation under constant load of the magnitude just reached (this phase was omitted for the reference case); and

3. Further penetration (at a rate of 0.1 mm/s) to the final depth of 2D.

The nondimensional consolidation times, $T$, investigated here ranged from 0.005 to 0.746, with consolidation taking place at 0.5D and 1.0D penetration for three tests each. Not only does this testing program cover a wide range of consolidation phases, but it also allows comparison of the results obtained in the carbonate silty clay with those of similar tests performed in kaolin clay (Bienen and Cassidy 2013).

Corresponding PIV tests were performed for five of the six full spudcan tests, with an additional test at 1.0D penetration and $T = 0.01$, which allows further comparison with tests on kaolin clay (Stanier et al. 2014).

### 3.4 Experimental results and discussion

The experimental results are presented in nondimensional form unless stated otherwise. Table 3.1 summarises the test details and results.

#### 3.4.1 Increase in bearing resistance with consolidation time

Figure 3.4 shows the measured penetration resistance in terms of the nondimensional bearing capacity factor, $N_c = V/(s_u A_s)$, where $V$ is the vertical load on the spudcan, $s_u$ is the in situ undrained shear strength of the soil as inferred from the fitted linear profile at that depth; and $A_s$ is the largest plan area of the spudcan (the choice of $s_u$ for normalisation is the reason the only test performed in Sample 2 has an apparently lower bearing capacity factor (Figure 3.4a) initially). The corresponding results from tests in kaolin clay (Bienen and Cassidy 2013) are included for comparison.

The bearing capacity factor quickly reaches an approximately constant value, indicating the transition to a localised mechanism with a stable cavity depth, or depression, at the soil surface. During consolidation, the bearing capacity factor decreases due to the normalisation by the increasing undrained shear strength corresponding to the spudcan settlement under the constant load in this phase. Following consolidation, a sharp increase in bearing resistance is measured, which, after reaching a peak value, gradually reduces again. This general response follows the schematic shown in Figure 3.1 and is
### Table 3.1: Overview of the Experiments

| Test Sample | Actual depth, w/D | Prototype time, T = cvt/D (years) | Applied pressure, kPa | Δw/D due to consolidation, kPa | Peak Nc/w/D at peak | Reference Nc/w/D | Peak Nc/w/D | Extent of improved zone, Δwez/D | Severity of capacity reduction, K
<table>
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<tbody>
<tr>
<td>SC-1 (PIV)</td>
<td>0.50</td>
<td>0.005</td>
<td>6.4e-5</td>
<td>115.3</td>
<td>0.13</td>
<td>11.58</td>
<td>0.66</td>
<td>7.99</td>
<td>1.45</td>
</tr>
<tr>
<td>SC-2 (PIV)</td>
<td>0.97</td>
<td>0.052</td>
<td>6.4e-5</td>
<td>217.4</td>
<td>0.18</td>
<td>11.40</td>
<td>1.16</td>
<td>9.47</td>
<td>1.94</td>
</tr>
<tr>
<td>SC-3 (PIV)</td>
<td>0.99</td>
<td>(12 years)</td>
<td>0.746</td>
<td>220.1</td>
<td>0.22</td>
<td>16.45</td>
<td>1.27</td>
<td>8.69</td>
<td>1.00</td>
</tr>
<tr>
<td>SC-4</td>
<td>0.49</td>
<td>1</td>
<td>6.4e-5</td>
<td>232.0</td>
<td>0.12</td>
<td>9.64</td>
<td>1.11</td>
<td>8.41</td>
<td>0.49</td>
</tr>
<tr>
<td>SC-5</td>
<td>0.50</td>
<td>1</td>
<td>6.4e-5</td>
<td>203.0</td>
<td>0.10</td>
<td>9.05</td>
<td>1.10</td>
<td>8.33</td>
<td>0.20</td>
</tr>
</tbody>
</table>

Note: PIV refers to corresponding PIV test available.

Ref. Nc is the value of the reference case (SC-1) at same penetration depth.

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similar to that observed in kaolin clay (Bienen and Cassidy 2013). However, there are significant differences as well.

The bearing capacity factor, $N_c$, is slightly lower in the carbonate silty clay than in the kaolin clay. The full spudcan test results cannot provide sufficient insight to resolve the reasons for this difference, though the slight difference in spudcan shape between the model used in testing here and Bienen and Cassidy (2013) is unlikely to be responsible because the measurements of a spudcan penetrating into kaolin by Zhang et al. (2013, 2014) should then also be systematically lower than those by Barbosa-Cruz (2007), which is not the case.

The reasons for the lower bearing capacity factor in the carbonate silty clay are therefore explored using the results from corresponding PIV tests.

Figure 3.5 reveals that the cause of the difference is that the spudcan penetration in the two soils is governed by different mechanisms, which is masked by expression of the response in terms of a bearing capacity factor, $N_c$. The differences are visible in Figure 3.5a, which shows the incremental displacement vectors, but becomes clearer when viewed in terms of contours of incremental displacement (Figure 3.5b,c,d, where $v_r$, $v_h$, and $v_v$ are the resultant, horizontal, and vertical velocities of the soil, normalised by the footing penetration velocity $v_f$). The relatively large extent of vertical displacements underneath the spudcan (Figure 3.5d) with only limited horizontal displacements (Figure 3.5c) in the carbonate silty clay is consistent with the high compressibility expected in the carbonate soil (as evidenced also in Rowe cell testing) and a plunging mechanism seen also in carbonate sand (Dijkstra et al. 2013; Yamamoto et al. 2008). Spudcan penetration into normally consolidated kaolin clay, on the other hand, quickly transitions from general shear to the localised flow-around mechanism seen in Figure 3.5. When expressed in terms of the bearing capacity factor, the relatively larger contribution of the stronger soil below the spudcan that forms part of the mechanism in the carbonate silty clay is balanced by the lower contribution of the remoulded soil above the spudcan due to its higher sensitivity, such that the resulting value of $N_c$ overall is similar or slightly less than in kaolin clay.

Undrained spudcan penetration into the carbonate silty clay generates less excess pore pressures than in the kaolin clay (Figure 3.6 inset, measured at the spudcan tip). The differences in soil characteristics further influence the shape of the consolidation curves as shown in Figure 3.6. In kaolin clay, the excess pore pressures at the spudcan tip in the early stages of consolidation increase before dissipating because the redistribution of stress concentrations and consolidation-induced settlement result in the generation of additional excess pore pressures that initially outweigh the dissipation of excess pore pressures generated during spudcan penetration. This Mandel-Cryer effect is less
pronounced at deeper embedment where soil strength and confining stress are higher. In the carbonate silty clay where $s_u$ increases more strongly with effective stress, the Mandel-Cryer effect was absent in all of the tests. The higher undrained shear strength gradient compared to the kaolin clay results in reduced settlement in the carbonate silty clay (Figure 3.7).

The maximum bearing capacity factor post-consolidation compared with that of the reference case without consolidation at the corresponding penetration depth is broadly similar between the two soil types as shown in Figure 3.8 (with the legend referring to the respective penetration depth prior to consolidation), though perhaps the data points representing the carbonate silty clay plot slightly lower than the corresponding points for the kaolin clay. In the tests involving consolidation, it is perhaps overly simplistic to refer to the bearing capacity factor, $N_c$, because both the undrained shear strength profile and the failure mechanism are changed (which for kaolin clay is discussed in Stanier et al. 2014). Therefore, $N^*_c$ is a better representation of the bearing capacity factor that combines both these influences. This notation is adopted here.

Though the mechanism prior to consolidation differs between the two soil types, the general qualitative change following a significant period of consolidation is similar, as shown for $T = c_v t / D^2 = 0.05$ in Figure 3.9.

The relative differences, therefore, remain similar: $N_c$ was slightly lower in the carbonate silty clay as discussed previously. Immediately post-consolidation, the mechanism is significantly enlarged in both soils, giving rise to the considerable increase in peak $N^*_c$/reference $N_c$ of 1.20 in the full spudcan test (1.34 in the PIV test) in the carbonate silty clay and 1.69 (1.66) in the kaolin, respectively, for $T = 0.05$ (Figure 3.8 shows that the difference is less for other periods of consolidation). In both soils, the extent of the mechanism increased significantly below the spudcan compared with the case without the effects of consolidation, as well as now extending to the soil surface, despite embedment of about $1D$.

### 3.4.2 Evolution of load-penetration response post-consolidation

The most striking difference in behavior between the two soil types is the shape of the load-penetration curve post-peak (Figure 3.4). In the carbonate silty clay, $N^*_c$ gradually reduces before gently curving around to rejoin the response of the reference case without consolidation. In kaolin, however, the post-peak reduction, especially for longer periods of consolidation, is much more pronounced and the curve does not merge with the reference case, it features continuously increased resistance.

While the mechanism governing the response in kaolin rapidly returns to localised flow around the footing (Figure 3.10b, after $0.5D$ of penetration post-peak), the behaviour in
the carbonate silty clay shows significant differences. After penetrating 0.25\(D\) post-peak, the normalised resultant displacement contours extend considerably further below the spudcan (Figure 3.10a), and after 0.5\(D\) the mechanism still involves the soil column above the spudcan up to the soil surface (Figure 3.10b). Even though the behaviour tends to a localised mechanism in the carbonate silty clay, this transition is not complete after 0.75\(D\) of penetration post-peak (Figure 3.10c). Throughout, the response in the carbonate silty clay is limited in its lateral extent, while a wider zone of soil is affected by the discontinuous spudcan penetration in kaolin.

The post-peak behaviour can be described by the (negative) gradient of penetration resistance, \(K\), that describes the overall reduction in \(N^*_c\) from its maximum until no further effect of consolidation is evident and the gradient of the penetration curve returns to be parallel to the reference case (Figure 3.11, inset). Though peak \(N^*_c\) is readily identified, some uncertainty is attached to the determination of \(K\) because the effects of consolidation diminish and thus merging of the curves occurs gradually. This is evident in the scatter in Figure 3.11.

As a general trend, though, \(K\) increases with increasing consolidation, as expected. However, the magnitude of this gradient is consistently and significantly lower in the carbonate silty clay compared with kaolin, and so is the increase with consolidation. This highlights the much gentler post-peak behavior of the former already seen in Figure 3.4. The response is considerably more ductile, and the load-penetration curve
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Figure 3.5: Evidence of differences in mechanism between carbonate silty soil (left-hand side) compared to kaolin clay (right-hand side): (a) normalised vectorial velocities; (b) normalised resultant velocity contours \( v_r = v_f \); (c) normalised horizontal velocity contours \( v_h = v_f \); (d) normalised vertical velocity contours \( v_v = v_f \)
Chapter 3. Effects of consolidation under a penetrating footing in carbonate silty clay

Figure 3.6: Time history of excess pore pressure ratio, $B_q$, measured at the spudcan tip

Figure 3.7: Time history of dimensionless consolidation settlement, $\Delta w' / q_{nom}$
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Figure 3.8: Increase in bearing capacity factor, $N^*_c$, with normalised consolidation time

Figure 3.9: Normalised resultant displacement contours: (a) kaolin clay; (b) carbonate silty clay, $T = 0$ (left-hand side), $T = 0.05$ (right-hand side)
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post-consolidation rejoins that of the reference case (in kaolin the gradient became similar though the penetration resistance retained an offset to the curve without consolidation, see Bienen and Cassidy 2013).

While the response normalised by the undrained shear strength decreases post-peak, the penetration resistance does not (Figure 3.12). Hence, consolidation does not introduce a man-made risk of uncontrolled rapid penetration or punch-through in the carbonate silty clay (at least within the range of experimental conditions investigated) because the penetration resistance curve does not feature a peak or post-peak reduction. In contrast, the bearing pressure continuously increases following a sharp rise immediately post-consolidation. This is an important finding, which sets the effects of consolidation in the carbonate silty clay apart from those in kaolin, where the potential for punch-through, or at least rapid leg run, is evident.

The gradient post-peak, of course, is influenced by the vertical distance between peak $N_c^*$ and the return to the load-penetration curve that is unaffected by consolidation. This vertical distance is termed here the extent of the improved zone.

Because the dissipation of excess pore pressures progresses with time, thus allowing enhancement of effective stresses, the extent of the improved zone increases with the length of the consolidation period for both soils as expected (Figure 3.13). However, the extent of the improved zone increased much more sharply with increasing $T$ in the carbonate silty clay, such that two distinctly different trends emerge for the soils investigated (Figure 3.13). While the peak response is described well in terms of the coefficient of consolidation, dissipation of excess pore pressures that eventually determines the extent of the improved zone is better described as a function of the permeability rather than including stiffness characteristics. Therefore, framing the information in Figure 3.14 in terms of normalised permeability $k_{wt}/D$, where $k_w$ is the permeability (from Rowe cell testing), $t$ is the absolute consolidation time, and $D$ is the maximum spudcan diameter, brings the data for both soils together as illustrated in Figure 3.14 (accepting the limited accuracy of permeability measurements).

The depth at which the installation is interrupted does not have a significant influence on the characteristics of the post-consolidation behavior: the data points representing consolidation at 0.5$D$, 1.0$D$, and 1.5$D$ (the latter data available for kaolin only) all fall within a narrow band, with no specific trends with depth discernible for peak $N_c^*$, $K$, or the extent of the improved zone (Figure 3.8, 3.11 and 3.14).
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Figure 3.10: Mechanism post-peak in terms of normalised resultant displacement contours $v_r = v_f$, after (a) $0.25D$, (b) $0.5D$, (c) $0.75D$, carbonate silty clay (left-hand side), kaolin clay (right-hand side)
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Figure 3.11: Severity of capacity reduction $K$ post-peak

Figure 3.12: Penetration resistance profiles with consolidation at (a) $0.5D$; (b) $1.0D$ penetration
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Figure 3.13: Extent of improved zone with normalised consolidation time

Figure 3.14: Extent of improved zone with normalised permeability
3.5 Proposed framework to predict the effect of consolidation on the load-penetration curve

Based on the findings from the centrifuge testing program, the following framework is suggested to predict the effect of Consolidation on the load-penetration curve:

- Prior to Consolidation, the load-penetration curve is estimated as 
  \[ q_{\text{nom}} = \frac{V}{A_s} = N_c s_u \]
  where \( q_{\text{nom}} \) is the bearing pressure, \( V \) is the vertical load, \( A_s \) is the effective bearing area of the spudcan, and \( s_u \) is the undrained shear strength as determined from site investigation. The bearing capacity factor \( N_c \) has been established for spudcans in kaolin by Hossain and Randolph (2009) while Figure 3.4 provides the values for the carbonate silty clay tested in this study. ISO (2012) provides detailed guidance on the prediction of load-penetration curves;

- During a pause in installation, additional settlement is accumulated, which can be estimated according to Figure 3.15. At the time of testing the level of sophistication of load control was higher in the beam centrifuge tests on the carbonate silty clay, which is the cause of the scatter in the data obtained on kaolin clay in the drum centrifuge. Though it is acknowledged that the settlement depends on the pressure applied during consolidation (relative to the available bearing capacity at that depth), there is insufficient data available at present to include this secondary effect. Ongoing research, including large deformation numerical analyses with coupled pore fluid-stress response, will allow further refinement;

- On recommencement of penetration, the response features high stiffness. The maximum bearing pressure \( q_{\text{peak}} \) can be estimated based on the original undrained shear strength at that depth \( s_u \), the bearing capacity factor \( N_c \) expected without consolidation, and the enhancement factor peak \( N_c^* \) / reference \( N_c \) (Figure 3.9);

- The extent of the improved zone, depending on the soil permeability, can be estimated from Figure 3.14. In carbonate silty soil, this marks the transition to a load-penetration curve unaffected by consolidation, whereas in kaolin the effective bearing capacity factor \( N_c^* \) remains enhanced by approximately 20% compared with the reference case even after 0.5 − 1.0D of further penetration (Stanier et al. 2014).

Prediction of the effects of consolidation on the load-penetration curve requires knowledge of the coefficient of consolidation, \( c_v \), and the permeability of the soil, \( k_w \). It is therefore recommended to perform in situ testing with a penetrometer that is equipped
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The results and discussions in this paper are not only relevant to penetrating footings but also shed light on the state of the soil around a jack-up spudcan immediately prior to its extraction for instance, a problem that has been investigated by Bienen et al. (2009), Gaudin et al. (2011a), and Purwana et al. (2005). The inability to predict $s_u$ post-consolidation has been a major stumbling block in developing a predictive method such as attempted by Purwana et al. (2009).

Figure 3.15: Normalised consolidation settlement with normalised consolidation time to measure pore pressure response or to include odometer or Rowe cell testing on undisturbed (if possible) soil specimen in the site investigation work scope.

The effects of consolidation may be detrimental (e.g., when causing punch-through or rapid leg run, especially if this was not predicted) or advantageous and thus actively sought. The latter may be the case if consolidation offered adequate bearing capacity at much shallower penetration, particularly in deep water where available leg length may become critical. However, reliability of this enhanced capacity both under the general low-level cyclic loading of the ocean environment and under storm loading must be ensured. Therefore, investigation of the reliability of the post-consolidation capacity under cyclic loading will be addressed in further research.
3.6 Concluding remarks

The effects of a period of consolidation interrupting footing penetration in carbonate silty clay have been investigated in a series of centrifuge experiments, complemented by visualisation experiments, also carried out in a geotechnical centrifuge. Parallels to an earlier study on kaolin clay allowed differentiation of the general features of the response against differences in detail when comparing the two soil types. This paper offers graphs useful for the estimation of peak resistance post-consolidation based on the experimental results and proposes a framework to predict the effect of consolidation on the load-penetration curve of the footing.
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References


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Chapter 4

Numerical modelling of the effects of consolidation on jack-up spudcan penetration

4.1 Abstract

The paper addresses the issue of consolidation around jack-up foundations in carbonate silty clay. The problem is tackled with the numerical implementation of a hypoplastic model for structured clays, within the framework of large deformation finite element analyses. Coupled analyses are simulated to account for excess pore pressure build up and dissipation, while sensitivity parameters capture the effects of remoulding. The model implementation is described first, followed by its validation against centrifuge data. The paper concludes with a detailed discussion of beneficial or detrimental effects on spudcan capacity upon re-penetration, depending on the consolidation and soil sensitivity characteristics.

4.2 Introduction

Installation of an offshore jack-up platform typically involves the penetration of circular footings – also known as spudcans – into the soil under the self weight of the structure, followed by progressively pumping sea water into the ballast tasks until the target preload is reached. Although it is desirable for this procedure to be conducted as quickly as possible, interruptions may be experienced during installation, causing the spudcan to sit at a certain depth for a period of time (see Figure 4.1, which provides a schematic illustration of preload and penetration time history). During these pauses in penetration some consolidation can take place in the soil around the spudcan, with a consequent
increase in soil shear strength and stiffness. This can set the conditions for a 'consolidation generated' punch-through, where the spudcan quickly advances through the strengthened zone when penetration restarts. This is recognised in ISO (2012), though no further advice of how to predict it – or mitigate it – is offered. Episodes of field failure have been reported: Brennan et al. (2006) showed that even short pauses in the order of hours contributed to the subsequent punch-through failure. Consolidation can also be actively sought in order to limit spudcan penetration, as detailed in Erbrich (2005). In either scenario, comprehensive understanding of the physical process occurring, and its effects on the geotechnical behaviour, is critical to formulate a detailed installation plan and to mitigate risk. To date, the relevant data base is scarce. Barbosa-Cruz (2007) and Bienen and Cassidy (2013) carried out a small number of centrifuge experiments on spudcan penetration consolidation penetration cycles, with the latter highlighting the punch-through risk created through pauses in footing penetration in kaolin clay. In a natural soil recovered from offshore Australia (a carbonate silty clay) further centrifuge experiments revealed the potential for a beneficial increase in bearing capacity due to consolidation without the elevation of the risk of punch-through (Bienen et al. 2015). The mechanisms of jack-up foundation penetration pre- and post-consolidation were seen to differ in Particle Image Velocimetry (PIV) experiments, as reported in Stanier et al. (2014). Comparable peaks in bearing capacity post consolidation were observed for kaolin clay and carbonate silty clay in all of the experiments reported. However, it has not been possible to isolate the effects of the changing failure mechanism and local changes in undrained shear strength and stiffness from the PIV analyses. In addition, centrifuge modelling, especially on natural soil, is not economic for extensive parametric studies. Numerical modelling can offer the desired insights provided a large deformation approach is used that features coupled pore fluid-stress response and a constitutive model that captures salient features of soil behaviour. This is necessary to model the process of installation of a spudcan footing which involves deep penetration, which can be up to three diameters. The numerical model further needs to account for consolidation. A coupled effective stress-pore fluid analysis is thus required, such that excess pore pressures generated during penetration can dissipate leading to increases in the effective stresses in the soil.

Natural offshore soils can be highly sensitive (with sensitivities up to 20 in the carbonate deposits found offshore Australia; Erbrich 2005), a constitutive model of the hypoplasticity family previously developed for structured clays (Mašín 2007) was incorporated in the numerical analyses of this study. The model has the ability to capture the incrementally non linear behaviour of the soil as well as the effects of remoulding and softening due to soil sensitivity. As this is the first time this constitutive model has
been used in a complex boundary value problem like this, the paper presents both model background and validation in detail. Then, the capabilities of the hypoplastic model are harnessed, which allow aspects of soil response such as the effect of high soil sensitivity to be explored that cannot be captured using Modified Cam Clay, for instance. A detailed investigation of the increase in bearing capacity is presented, through a wide parametric study on the effects of consolidation length, depth and load-hold, as well as the sensitivity properties. In terms of length of consolidation, the study aims to cover different scenarios, from short pauses during installation to longer vertical load-hold periods (i.e. self weight of the structure). On one side, short pauses in penetration in the order of hours can be seen either as beneficial for the jack-up in order to reduce the spudcan installation depth or as a detrimental instigator for punch-through. On the other side, longer periods of consolidation – in the order of months or even years – will not occur during installation, but can enhance the bearing capacity over time. For this reason, the enhancement in bearing capacity has to be seen as potentially beneficial relative to the operational conditions (by improving the fixity of the foundations). Finally, the study on sensitivity parameters illustrates the different behaviours observed when natural, rather than reconstituted soil is modelled, quantifying first the increase in bearing capacity due to consolidation, and then its reliability upon re-penetration, showing once again the possibility of either beneficial or disadvantageous responses.

4.3 Numerical model

4.3.1 Large deformation approach with coupled pore fluid-stress response

The large deformation finite element approach adopted in this paper is based on the Remeshing and Interpolation Technique with Small Strain (RITSS) strategy (Hu and Randolph 1998). The RITSS technique has been extended to model coupled pore fluid-stress response in the commercial code Abaqus/Standard (Dassault Systèmes 2012) as described in Wang et al. (2008). The basic premise of RITSS large deformation finite element analysis is to divide the entire analysis into small incremental steps so that mesh distortion is minimised and consecutive finite strain analyses are executed. A new mesh is generated at the beginning of every step based on the final deformed configuration of the previous one. Then the effective stresses and state variables are mapped to each integration point and the pore pressures to element nodes, with convection achieved by polynomial interpolation. Irrespective of whether the variables are mapped to the new integration points or to the new nodes, the interpolation in RITSS is always conducted.
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Time, \( t \)

Preload force, \( V \)

Penetration depth, \( w \)

Long hold period due to poor weather or a technical problem

Typical short hold period

Water In

Water Out

Path followed in experiments

Figure 4.1: Schematic of a typical spudcan preload force and penetration depth time history during jack-up installation
locally within an old element, an old element patch or a triangle connecting old integration points (Wang et al. 2015). In this study, a modified unique element method proposed by Hu and Randolph (1998) was employed to map effective stresses and material state properties from the old integration points to the new integration points.

4.3.2 Geometry, mesh and boundary conditions

The numerical analyses were conducted using axisymmetric boundary conditions since most spudcan foundations are circular. The spudcan was modelled as a rigid body due to its much higher stiffness compared to the soil. The interaction at the soil-spudcan interface was simplified as frictionless as the interface roughness, even in consolidation problems, has been shown to have little influence on the resulting spudcan penetration resistance (Wang and Bienen 2016). Eight-node elements with biquadratic displacement, bilinear pore pressure and reduced integration (termed CAX8RP in Dassault Systèmes 2012) were used to discretise the soil.

The soil domain was extended $20D$ (where $D$ is the spudcan diameter) both vertically and horizontally to avoid boundary effects, modelling a large expanse of single layer soil in the field. The minimum element size around the spudcan was $0.025D$ and it had minimal effect on the simulated penetration resistance when reduced to $0.015D$. The numerical model is shown in Figure 4.2, where vertical and horizontal degrees of freedom were constrained for the bottom of the region and the two vertical sides respectively. Drainage of excess pore pressures was allowed through the soil surface. The vertical spudcan displacement for each incremental step of the RITSS analyses was taken as $0.01D$, small enough to avoid excessive mesh distortion, after which the mesh was periodically regenerated.

The numerical analyses were performed at model scale rather than at prototype dimension, in order to make a straightforward comparison with the centrifuge tests that are discussed in the validation section. Penetration of the spudcan from the soil surface is challenging to perform numerically especially for normally consolidated soil profiles, as heaving soil around the foundations changes geometry suddenly upon failure caused by its self-weight; hence an initial embedment of the spudcan of $0.5D$ was prescribed to ease convergence.

4.3.3 Hypoplastic constitutive model for structured clays

Model description

Numerous studies (Atkinson 2000; Gasparre 2005) have demonstrated the reduction in soil stiffness from very small to very large strain and the incrementally non-linear
Figure 4.2: Numerical soil domain and mesh strategy around the spudcan
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behaviour of soil. Such behaviour cannot be modelled using conventional elasto-plastic constitutive models such as Modified Cam Clay, where a yield surface in the stress space bounds the admissible states and the behaviour inside it is reversible. It is instead necessary to introduce irreversible strains inside the state boundary surface, as in soil behaviour the elastic range is extremely limited. Therefore, the choice to adopt a hypoplastic model is a step beyond the conventional elasto-plasticity.

Hypoplasticity models have been developed to describe the behaviours of clays and sands. In its general form (Gudehus 1996) it may be written as

\[ \dot{\sigma} = f_s (L : \dot{\varepsilon} + f_d N \parallel \dot{\varepsilon} \parallel) \]  (4.1)

where \( \dot{\sigma} \) and \( \dot{\varepsilon} \) represent the objective stress rate and the Euler stretching tensor, respectively, \( L \) and \( N \) are the fourth- and second-order constitutive tensors, and \( f_s \) and \( f_d \) are two scalar factors. In hypoplasticity, the stiffness predicted by the model is controlled by tensor \( L \), whereas the strength (and asymptotic response in general) is governed by a combination of \( L \) and \( N \). Earlier hypoplastic models, such as those of Von Wolfersdorff (1996) and Mašín (2005), did not allow arbitrarily changing of the \( L \) formulation as any modification of tensor \( L \) undesirably influenced the predicted asymptotic states. This hypoplasticity limitation was overcome by Mašín (2012) who developed an approach enabling the specification of the asymptotic state boundary surface independently from tensor \( L \). The efficacy of this approach was demonstrated by proposing a simple hypoplastic equivalent of the Modified Cam clay model. Based on this approach, Mašín (2014) developed an advanced hypoplastic model for clays, which is adopted in the present work as an updated version of the Abaqus/Standard user subroutine (Dassault Systèmes 2012) by Gudehus et al. (2008).

The basic hypoplastic clay model requires five material properties \( \phi_c, N, \lambda^*, \kappa^* \) and \( \nu \). The parameters have the similar physical interpretation as the parameters of the Modified Cam clay model, and they are thus easy to calibrate using standard laboratory experiment. The model parameters \( N \) and \( \lambda^* \) define the position and the slope of the isotropic normal compression line in the \( \ln(1+e) \sim \ln(p'/p_r) \) plane. The isotropic normal compression line is described using the equation

\[ \ln (1+e) = N - \lambda^* \ln \left( \frac{p'}{p_r} \right) \]  (4.2)

where \( p_r = 1 \text{kPa} \) is a reference stress, \( e \) the void ratio and \( p' \) the mean effective stress. Parameter \( \kappa^* \) controls the slope of the isotropic unloading line in the same plane and also of the isotropic compression line of overconsolidated soil. \( \phi_c \) is the critical state friction angle, with identical meaning to that in any other critical state soil
mechanics-based model. Finally, the parameter $\nu$ controls the shear modulus and reads

$$\nu = \frac{3r(\lambda^* + \kappa^*) - 4\kappa^*}{6r(\lambda^* + \kappa^*) + 4\kappa^*} \quad (4.3)$$

where $r$ is the ratio of the bulk modulus in isotropic compression at the isotropic normally consolidated state and the shear modulus in undrained shear.

Apart from the stress, the most important state variable controlling the response of the model is the void ratio $e$. In the numerical simulation, this parameter can be directly initialised as a constant. However, such an approach has a significant drawback as the degree of over-consolidation with depth is increased. In this case, the void ratio is initialised in an alternative manner, ensuring a constant over-consolidation ratio $OCR$ with depth, consistently with the conditions found in a normally consolidated deposit. The $OCR$ in the hypoplastic model is defined as

$$OCR = \frac{p_e}{p^f} \quad (4.4)$$

where $p_e$ is the Hvorslev equivalent pressure, that is, the mean effective stress at the isotropic normal compression line calculated using Equation 4.2 at the current void ratio. Note that this definition of $OCR$ is slightly different from the traditional soil mechanics definition, where $OCR$ is a ratio of pre-consolidation pressure and mean stress. It is also important to point out that in cases like the present work, where the soil is in normally consolidated (i.e. oedometric) conditions, $OCR$ is not equal to 1, as the oedometric normal compression line is below the isotropic normal compression line in the $\ln(1 + e) \sim \ln(p^f/p_r)$ plane. To initialise the state for $K_0$ consolidated conditions, with the earth pressure coefficient at rest $K_0 = 1 - \sin(\varphi_c)$ (assuming a constant $\varphi_c$ after triaxial simulations of Figure 4.6) governing the ratio of horizontal over vertical stresses (Rott et al. 2015), a value of $OCR(K_0)$ must be calculated from the model equations. The basic hypoplastic model was also enhanced by the effects of sensitivity to capture the degree of remoulding in the soil, using a procedure developed in Mašín (2007). This enhancement requires three additional parameters, $k$, $A$ and $s_f$, and the sensitivity $s$ as a state variable. Sensitivity $s$ is here defined in the way as proposed by Cotecchia and Chandler (2000): it is representative of the ratio of both the undrained shear strengths and the ratio of pre-consolidation pressures for undisturbed and reconstituted soils. The parameter $k$ controls the rate of the sensitivity degradation

$$\dot{s} = -\frac{k}{\kappa^*} (s_{ini} - s_f) \dot{\varepsilon}^d \quad (4.5)$$

where $s_{ini}$ and $s_f$ represent the initial and final values of sensitivity, and $\dot{\varepsilon}^d$ is the damage strain rate. The parameter $A$, which only has a minor effect on the results, controls the
relative importance of the volumetric strain $\varepsilon^v$ and shear strain $\varepsilon^s$ components

$$\dot{\varepsilon}^d = \sqrt{(\dot{\varepsilon}^v)^2 + \frac{A}{1 - A} (\dot{\varepsilon}^s)^2}$$  \hspace{1cm} (4.6)$$

The normal compression line shown in Figure 4.3 then reads

$$\ln(1 + e) = N - \lambda^* \ln \left( \frac{p'}{p_r} \right) + \lambda^* \ln (s)$$  \hspace{1cm} (4.7)$$

and the over-consolidation ratio is defined using $OCR = p_e / p' = sp_e^* / p$, where $p_e$ is the Hvorslev equivalent pressure on the isotropic normal compression line of the structured soil and $p_e^*$ is the same equivalent pressure of the reconstituted soil.

The clay hypoplastic model, in its default form, assumes fixed relative position of critical state line in the $\ln(1 + e) \sim \ln(p'/p_r)$ plane with respect to the isotropic normal compression line. For the given void ratio, the pressure on the critical state line $p_c$ is equal to $p_e / 2$. This ratio is consistent with the Modified Cam clay formulation and it is suitable for most standard soils. However, the carbonate silty clay investigated here is non-standard and it has been found that the relative positions of the critical state and isotropic normal compression line are different from the default values. The model formulation has for this reason been enhanced by a parameter $O_c$, which controls the position of the critical state line in the following way

$$p_c = \frac{p_e}{O_c}$$  \hspace{1cm} (4.8)$$

With $O_c = 2$ the default model is recovered. The parameter $O_c$ affects the shape of the state boundary surface, and consequently, also the undrained stress paths, undrained stress-strain curves and undrained shear strength. Figure 4.4a,b show the shape of the Asymptotic State Boundary Surface (ASBS) and undrained stress paths and stress-strain curves of $K_0$ normally consolidated Laminaria soil. From the shape of ASBS and the definition of sensitivity the undrained shear strength corresponding to the given degree of sensitivity $s$ predicted by the hypoplastic model is given by

$$s_u = \frac{OCR p' M_{cs}}{2O_c} s$$  \hspace{1cm} (4.9)$$

where $M_{cs}$ is the slope of critical state line in the $q \sim p$ plane, which can be calculated from $M_{cs} = 6 \sin \varphi_c / (3 - \sin \varphi_c)$ for triaxial compression. Although the model does not explicitly feature a residual state friction angle, the critical state strength and current $OCR$ do not imply a lower bound to the undrained strength of the soil. This is because the model considers sensitivity as a state variable: soil reaches critical state stress ratio with sensitivity larger than one, i.e. in natural conditions. Further shearing and
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$$\lambda^* \ln s$$

$$\kappa^*$$

$$NCL$$ for $$s = 1$$

Figure 4.3: Hypoplastic bi-logarithmic compression law for structured clays

consequent strain increase causes sensitivity to decrease and the soil to reach a fully remoulded residual state, represented by sensitivity equal to one.

Parameter calibration

The soil used in the experiments is a carbonate silty clay recovered from the Laminaria field, in the Timor Sea off the Northern Coast of Western Australia. It has a particle distribution consisting of about 10% sand, 70% silt and 20% clay and a specific unit weight expressed in kN/m$^3$ as a function of the depth $w$ in m: $\gamma' = 5.6 + 0.022w$ (Randolph et al. 1998). The elevated calcium-carbonate content up to 85% and grain crushing at increasing stress level give this soil characteristic of ‘clay-like’ elevated plasticity (Erbrich and Hefer 2002). Further studies on similar carbonate soils also revealed this silt to be a ‘difficult’ and ‘challenging’ soil (Amodio et al. 2015; Erbrich 2005).

The basic material properties of the Laminaria soil were evaluated through standard laboratory tests. Throughout the paper, ‘Exp’ refers to experimental results, ‘Num’ to numerical results. In particular, $N, \lambda^*$ and $\kappa^*$ were fitted from experimental oedometer test results of Figure 4.5, carried out starting from soil in a fully remoulded state, i.e.

4-10
with $s_{ini} = 1$. Triaxial tests on $K_0$ consolidated samples (Figure 4.6) revealed a slope $M_{cs} = 1.375$ when results are plotted in the $q \sim p'$ plane and a consequent $\varphi_c = 34^\circ$ (Figure 4.6b). $\nu = 0.1$ was identified as appropriate through numerical parametric study of triaxial tests. Given the observation of shear bands in the samples and the inability of the model to capture this phenomenon, the response of the simulations is quite different to the experimental response (this discrepancy is revisited in a later discussion at the end of this section). However, the slope of the critical state line $M_{cs}$ was nonetheless captured.

Back analysis of the results from the centrifuge tests of Bienen et al. (2015) underpinned the calibration of the remaining parameters. A series of experiments involving full spudcan penetrating into carbonate silty clay was carried out in a beam centrifuge at the University of Western Australia (UWA) at enhanced gravity level of $a = 200\, g$. Before starting the testing program, the undrained soil shear strength was evaluated as $s_u = 2.2 \, \text{kPa}$ using a miniature piezoball penetrometer, assuming a factor $N_{Ball} = 10.5$.

The spudcan had a diameter $D = 60 \, \text{mm}$ and was equipped with five pressure transducers, three on the bottom surface and two on the top surface, in order to monitor the pore pressure variations.

Cyclic T-bar tests, reported in Figure 4.7, showed an average value of sensitivity $s = s_{u,ini}/s_u = 2.9$ for the carbonate silty clay, where $s_{u,ini}$ is the initial, intact undrained shear strength of the sample and $s_u$ the remoulded value. Although these centrifuge tests were carried out on real soil recovered from the Laminaria field, the sample used for centrifuge tests in Bienen et al. (2015) was reconsolidated under its self weight only. For this reason it is very likely that part of the original natural sensitivity – intended by Baudet and Stallebrass (2000) as a combination of fabric and bonding – was lost during
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Figure 4.5: Results of oedometric test on carbonate silty clay from reconstituted sample and corresponding numerical simulation

Figure 4.6: Results of triaxial tests on carbonate silty clay on (a) stress-strain plane, (b) mean effective deviator stress path
Figure 4.7: Episodes of cyclic T-bar penetration in centrifuge environment showing sensitivity degradation

sampling and reconstitution. In particular, the high degree of disturbance due to sampling would erase the bonding forces between soil particles, which are achieved with extremely long periods of sedimentation and diagenesis, whereas at least part of the fabric can be recovered during centrifuge consolidation. Erbrich (2005) reported sensitivity field values for carbonate soils as high as $s = 20$ and indicated that ‘the laboratory samples have been substantially disturbed at some stage, leading to re-consolidation at lower void ratio’. Also, there are uncertainties on the rate of degradation of such sensitivity when under natural conditions.

Figure 4.8 reveals that $O_c = 1.5$ offered the best overall fit when trying to replicate the bearing capacity profile of ‘Exp Ref’, with all the simulations starting from $OCR(K_0)$ conditions. The value was also verified by comparing the model-predicted undrained shear strength (Equation 4.9) with the distribution of undrained shear strength with depth as estimated using a miniature piezoball penetrometer. For the Laminaria soil parameters (Table 4.1) and the intact value of sensitivity $s_{ini} = 2.9$ (Figure 4.7), the back-analysed values of $O_c = 1.5$ and $k = 0.05$ led to an increase of $s_u$ with depth of 2.1 kPa/m, which was close to 2.2 kPa/m measured by piezoball penetrometer. $k = 0.05$ also offered the
best fitting for the re-penetration curves post-consolidation, as detailed in the following parametric study. $s_f = 1$ reflects the absence of any residual sensitivity after large remoulding and $A$ was shown to be in the range of 0.1 to 0.5 for most clays (Mašín 2007), however, it has only a minor influence, and a value of 0.2 was chosen here. Even though both laboratory and centrifuge samples are considered to be in $K_0$ conditions, the two consolidation procedures occurring in the centrifuge and the triaxial tests are in fact different from each other. In particular, the centrifuge samples and the triaxial samples may potentially present different initial sensitivities, as the sample preparation processes involved are very different. Moreover, the formation of shear bands and strain localisation at the peak in the stress-strain path of triaxial tests was observed (Figure 4.6a), which make such tests unreliable for the calibration of parameters related to post-peak softening, particularly for large scale boundary value problems incorporating very large deformations as modelled here. Although already treated within hypoplasticity (Maier 2004), the modelling of such localisations was nonetheless outside the scope of this paper. The set of parameters adopted in the simulations is given in Table 4.1.
Basic Model Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \varphi_c )</td>
<td>34°</td>
</tr>
<tr>
<td>N</td>
<td>1.697</td>
</tr>
<tr>
<td>( \lambda^* )</td>
<td>0.114</td>
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Sensitivity Parameters

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Consolidation Parameters

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Table 4.1: Hypoplastic model parameters for Laminaria carbonate silty clay

4.3.4 Validation

The validation of the numerical model involved the retrospective simulations of centrifuge model data on spudcan installation with a pause during penetration from Bienen et al. (2015). Each centrifuge test involved (i) a stage of penetration to the target depth \( \frac{w}{D} = 1.0 \) at a constant rate of 0.1 mm/s, with the normalised velocity \( V_s = \frac{vD}{c_v} > 100 \) to guarantee undrained conditions, as discussed by Cassidy (2012), Chung et al. (2006), and Finnie (1993), (ii) a period of consolidation holding the load achieved at \( \frac{w}{D} = 1.0 \) for a normalised time \( T = \frac{c_v \ell}{D^2} \), and (iii) re-penetration at the same rate of 0.1 mm/s to the target depth \( \frac{w}{D} = 2.0 \).

The permeability governing the rate of excess pore pressure build up/dissipation was assumed to be isotropic and estimated as

\[
k_w = \gamma_w m_v c_v = \frac{\gamma_w c_v \lambda^*}{\sigma'_v} \tag{4.10}
\]

where \( \gamma_w \) is the unit weight of water, \( m_v \) is the coefficient of volume compressibility of soil, \( c_v \) the coefficient of vertical consolidation and \( \sigma'_v \) the vertical effective stress. The coefficient of consolidation \( c_v \) was fitted from the experimental oedometer test data as shown in Figure 4.9 with the following empirical equation.

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Chapter 4. Numerical modelling of the effects of consolidation on jack-up spudcan penetration

\[ c_v = \frac{\sigma_v'}{10 + 0.478\sigma_v'} \]  

The units of \( c_v \) and \( \sigma_v' \) are \text{m}^2/\text{year} and \text{kPa}, respectively. The numerical simulations generated using this relation show generally good comparability with the experimental interpretations in Figure 4.9.

Each test was labelled ‘Exp \( T = \ldots \)’. A further reference test with continuous penetration to \( w/D = 2.0 \) was also performed and labelled ‘Exp Ref’. The same procedure was adopted in the numerical analyses. The reference case is labelled ‘Num Ref’, the other analyses are labelled ‘Num \( T = \ldots \)’. All numerical analyses and centrifuge tests are summarised in Table 4.2.

In order to widen the investigation and offer a better comparison with numerical results, a second series of Particle Image Velocimetry (PIV) tests was conducted in the UWA drum centrifuge (see Stanier and White (2013) for details on the apparatus and modelling techniques adopted). A half spudcan model sliding against a Perspex window was used, with geometry similar to the above mentioned footing but \( D = 50 \text{mm} \) and no pore pressure transducers available (Bienen et al. 2015). This technique enabled the different failure mechanisms to be observed, expanding the set of information made available with the first series of tests, which only provided the histories of penetration resistance and excess pore pressures.

Figure 4.10 reports the bearing capacity \( q = V/A_s \) profiles, where \( V \) is the vertical load acting on the spudcan and \( A_s \) the area at its widest section, for both experimental and numerical cases. In particular, a reference test with continuous penetration and three other tests involving a period of consolidation at depth \( w/D = 1.0 \) with an increasing length \( T = 0.005; 0.052; 0.746 \) are shown. Although the numerical cases start from an initial embedment of 0.5 \( D \), the reference case quickly realigns with its experimental counterpart, hence without affecting the consolidation stage at \( w/D = 1 \) and showing a rapid transition to a fully localised mechanism (Hossain et al. 2005). In addition, the experimental and numerical reference cases provide similar resistance profiles, since the undrained shear strength profile deduced from the hypoplastic model, 2.1 kPa/m, is close to the 2.2 kPa/m measured using the T-bar. The settlements observed during the consolidation stage at depth \( w/D = 1.0 \), are significantly larger than those predicted numerically. The causes of such a difference are not fully understood. However, the normalised penetration rate immediately before consolidation in the experimental case is \( V_s = 120 \) and well beyond the threshold of \( V_s = 80 \) necessary to guarantee undrained conditions. The increased penetration rate at the onset of the load-hold period in the experiments can result in rate effects (Barbosa-Cruz 2007) leading to the onset of drainage being delayed whilst the normalised penetration rate falls from \( \sim 120 \) to \( \sim 80 \),
before which excess pore pressures are precluded from dissipating. For this reason, the larger experimental settlement may be seen as a combination of rate effects \( V_s > 80 \) and consolidation \( V_s < 80 \) components (Bienen et al. 2010), whereas the numerical analysis only accounts for the latter, as the model adopted is rate independent. For example, on average, in the experiments, 18% of the total load-hold induced settlement had already occurred before \( V_s < 80 \). Upon re-penetration after consolidation, a peak in the experimental curves is observed, followed by a continuous increase of the bearing capacity as the penetration advances towards \( w/D = 2.0 \), behaviour already described by Bienen et al. (2015). The numerical analyses were able to reproduce the same trend, illustrating once again low risk for a potential consolidation-generated punch-through, as no bearing capacity reduction post peak is observed, in contrast for example with the kaolin clay responses studied in Bienen and Cassidy (2013). Similar numerical results (dotted lines of Figure 4.10) were achieved when adopting the centrifuge Boundary Conditions (BC) in terms of sample depth (from \( 20D \) to \( 3.5D \)) and drainage conditions (halved drainage path), confirming the validity of centrifuge modelling in simulating real field conditions.

Figure 4.11 shows the dissipation during the consolidation stage of the excess pore pressures \( \Delta u \) normalised by the bearing pressure \( q, Bq = \Delta u/q \), plotted against the normalised consolidation time \( T \), where \( c_v \) is taken as the value defined by Equation 4.11 at the depth of the load reference point of the foundation at the onset of the load-hold period. The numerical dissipation curve ‘\( \text{Num } T = 0.746 \)’ and the three experimental dissipation curves present similar shape and they all start to dissipate at \( T \approx 0.001 \), with consolidation almost completed \( (Bq \approx 0) \) when \( T = 0.746 \) is reached. The longer the period of consolidation, the more excess pore pressures dissipated, which means a reduction in the void ratio around the spudcan and a consequent increase in shear strength.

Numerical reproduction of centrifuge PIV tests carried out with a slightly different spudcan shape of diameter \( D = 50\text{mm} \) at model scale enabled better visualisation. Figure 4.12 compares the numerical profiles of maximum undrained shear strength for current sensitivity \( s_c \), calculated using Equation 4.9, immediately before (LHS) and after a consolidation period of \( T = 0.05 \) (RHS). On the LHS a linear increase of undrained strength with depth at far field is observed, while the soil above the spudcan is highly remoulded. Instead, on the RHS, the excess pore pressure dissipation and consequent reduction in void ratio implies a significant increase in \( s_u \) below the footing for approximately \( 1D \), as well as laterally.

The change in \( s_u \) will affect the failure mechanism, as Figure 4.13 shows by comparing the normalised resultant velocity contours for experimental PIV data (LHS) and
numerical simulations (RHS). Immediately before consolidation at $w/D = 1$ (Figure 4.13a), both numerical and PIV results show a mechanism of comparable size, with soil flowing around the spudcan (Hossain et al. 2005). However, when restarting penetration after a pause of $T = 0.05$ (Figure 4.13b), more soil under the spudcan is mobilised. Although there is a discrepancy in the volume of mobilised soil, the numerical analysis captured the soil above and below the spudcan moving downwards at a rate similar to the spudcan. Even upon further re-penetration of $0.5D$ (Figure 4.13c) and $0.75D$ (Figure 4.13d) the mechanism is still far from realigning with the localised flow around in Figure 4.13a. Although it is gradually reducing in size, the soil volume with higher undrained strength trapped below the spudcan is obvious. This slow realignment with the reference case also contributes to explain the gaps observed in Figure 4.10 between the reference case and the post-consolidation profiles.

### 4.4 Parametric study

#### 4.4.1 Influence of $w/D$, $T$, $q$

Unless consolidation is actively sought and hence planned for, neither (i) the depth at which the consolidation stage is going to take place (ii) duration of consolidation nor (iii)
Figure 4.10: Vertical profiles of spudcan bearing capacity $q = V/A_z$ for centrifuge experimental data and numerical retrospective simulations.

$T = 0.746$

$T = 0.005$

$T = 0.052$
### Table 4.2: Overview of numerical analyses and experimental tests

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magnitude of preload which is held during the consolidation stage are known a priori. In order to provide a relevant database and facilitate the understanding of the effects of a pause during installation, a parametric study was performed. This included investigation of

- the effects of different consolidation depth \((w/D) = 1.0; 1.5; 2.0\);
- normalised consolidation time \((T = 0.001; 0.005; 0.01; 0.05; 0.1; 0.5)\);
- load ratio \((q_{cons}/q_{ref} = 0.5; 0.75; 1.0)\), where \(q_{cons}\) is the pressure held during the consolidation stage and \(q_{ref}\) refers to the pressure applied when the spudcan is penetrated continuously to a particular depth.

This parametric study was designed to expand the existing experimental database in a systematic manner, covering a wider range of different types of consolidation stage, whilst still remaining representative of the qualitative trends already identified in the validation section.

The soil properties and spudcan dimensions are the same as in the validation section. All results are presented in terms of normalised time \(T\), so that the results are indicative for different soils presenting different coefficients of consolidation – provided their
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Figure 4.12: Undrained soil shear strength contour plot for current sensitivity from numerical simulation immediately before (LHS) and immediately after (RHS) a period of consolidation $T = 0.05$
Figure 4.13: Normalised resultant velocity contours for experimental PIV data (LHS) and numerical simulations (RHS) (a) immediately before consolidation $T = 0.05$, (b) immediately after consolidation, (c) $0.5D$ after consolidation, (d) $0.75D$ after consolidation
characteristic behaviours are similar. In particular, the wide range of $T$ and $q_{cons}/q_{ref}$ investigated, aimed to cover possible field scenarios.

This parametric study allowed quantification of the expected trends on increasing spudcan capacity ($q_{peak}/q_{ref}$, where $q_{peak}$ represents the bearing capacity recorded approximatively at the point where the re-penetration profiles change their curvature) with increasing consolidation time $T$, and larger gains for higher load ratios during the consolidation period. The enhancements of peak pressure at different consolidation depths are plotted in Figure 4.14 (black dots for $q_{cons}/q_{ref} = 1.0$) against the normalised consolidation time $T$. The ratio is increased nearly linearly to the logarithm of $T$.

Moreover, comparison of Figure 4.14a, b, c shows that the consolidation depth $w/D$ does not have an influence on the magnitude of such peak, given the same $T$ and $q_{cons}/q_{ref}$, with $q_{peak}/q_{ref}$ values always ranging from 1% to nearly 50%. This confirms the independence of the ratio $q_{peak}/q_{ref}$ from the consolidation depth, within a realistic range of $w/D$, already shown by Bienen and Cassidy (2013) for kaolin clay and Bienen et al. (2015) for carbonate silty clay, and also validates the experimental observations in kaolin clay of similar failure mechanisms for different consolidation depths of Stanier et al. (2014).

### 4.4.2 Influence of $s, k$

The strength of the numerical model featuring the hypoplastic soil constitutive model lies in the fact that soil behaviour with higher sensitivity, such as that expected from intact soil in the field, can now be investigated, and this is done here.

In order to cover the range of sensitivity expected in the field from carbonate deposits offshore Australia, the solid lines in Figure 4.15 present the bearing capacity profiles of four cases with increasing sensitivity $s_{ini} = 2.9; 5.8; 8.7; 20$ respectively, supported by observations of Erbrich (2005) of $s$ values as high as 20. The simulations involved a consolidation stage of $T = 0.052$ commencing at $w/D = 1.0$ and used the same $k = 0.05$ and $OCR = 1.205$. According to Figure 4.3 and 4.4, a reduced capacity would be expected in penetration from the tests with higher sensitivity at same $OCR$, as the final critical state is reached at a lower $q/p_c$ level. However, this behaviour is not observed here for two reasons: first, the very low rate of sensitivity degradation ($k = 0.05$) contributes to mitigate the effect of softening, i.e. softening does not have the chance to fully develop; second, higher values of $s$ imply soils existing at higher void ratio for a given stress (this follows from Equation 4.7), leading to partially drained conditions in penetration. This second reason is also supported by observation during the consolidation stage, by looking at the excess pore pressure dissipation curves of Figure 4.11. Soils with higher $s$ dissipate more quickly than the benchmark case with
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Figure 4.14: Increase in bearing capacity with normalised time $T$ for (a) $w/D = 1.0$; (b) $w/D = 1.5$; (c) $w/D = 2.0$
$s_{ini} = 2.9$, as the higher void ratio results in a higher permeability. In turn, this will have an effect on the re-penetration curves, with the $s_{ini} = 20$ case presenting the highest excess pore pressure dissipation and as a consequence the largest bearing capacity profile, followed by $s_{ini} = 8.7$, $s_{ini} = 5.8$ and $s_{ini} = 2.9$.

Sensitivity is not the only factor; the rate of degradation with accumulated shear strain plays an important role. While a value of $k = 0.05$ fits the reconstituted carbonate silty clay used in the laboratory tests, values as high as 0.35 and 0.4 have been cited for Bothkennar and Pisa clays, respectively (Mašín 2007). The dashed lines of Figure 4.15 illustrate the effect a degradation rate of $k = 0.2$ has on the behaviour. In this case the effect of softening becomes predominant over the enhanced bearing capacity due to consolidation. As a consequence, the case with lowest sensitivity presents the highest bearing capacity at $w/D = 1.0$. Importantly, the shape of the penetration profile is significantly affected by the value of $s$. The spudcan load-penetration curves now show a ‘nose’, followed by a substantial reduction in resistance post peak. In these cases, consolidation has created punch-through risk that was not evident in the original soil profile, with the possibility of the spudcan penetrating rapidly at least another $0.8D$ until bearing capacity similar to the peak value is met again. While the case of $s_{ini} = 2.9$ is not characteristic of punch-through, rapid leg run would be expected due to the almost constant resistance with depth.

Figure 4.16 helps to further demonstrate the influence of strength softening rate, comparing sensitivity $s$ contour plots immediately before the consolidation stage at $w/D = 1.0$ for $k = 0.05$ (LHS) and $k = 0.2$ (RHS), with both cases starting from an initial intact value of $s_{ini} = 2.9$. While no soil reached the fully remoulded value with $k = 0.05$ on LHS, a large portion of soil above and immediately below the spudcan reaches fully remoulded state $s = 1$ on the RHS, and with a larger volume of soil that undergoes a more severe degradation process, according to Equation 4.5. The gap between the bearing capacity profiles generated using $k = 0.05$ and $k = 0.2$ is a result of the significant difference in distribution and concentration and softening generated during the penetration of the foundation.

Finally, Figures 4.17 and 4.18 show the effects of different $q_{cons}/q_{ref} = 0.5; 0.75; 1.0$ and $T = 0.005; 0.052; 0.746$ on a soil in potentially natural conditions, with $s_{ini} = 20$ and $k = 0.05; 0.2$. In particular, Figure 4.17 shows that for $k = 0.05$, a given period of consolidation ($T = 0.052$ in this case) will always be beneficial regardless of the fraction of load held in consolidation, with $q_{peak}$ proportional to the ratio $q_{cons}/q_{ref}$ and $q$ increasing constantly in the following penetration post-peak. On the other hand, a higher degradation rate of $k = 0.2$ will set the conditions for a potential punch-through, although with its severity mitigated by lower $q_{cons}/q_{ref}$ ratios, as both $q_{peak}$ and the extent of the
Figure 4.15: Effects of sensitivity $s$ and degradation rate $k$ on the vertical bearing capacity profiles for numerical simulations

Chapter 4. Numerical modelling of the effects of consolidation on jack-up spudcan penetration
Figure 4.16: Sensitivity degradation contour plot for $k = 0.05$ (LHS) and $k = 0.2$ (RHS), starting from intact sensitivity $s_{ini} = 2.9$
uncontrolled leg penetration are reduced. The observed decrease in punch-through distance in particular, of roughly 50% from $q_{cons}/q_{ref} = 1.0$ to $q_{cons}/q_{ref} = 1.0$, is in line with the experimental observations of Bienen and Cassidy (2013) for kaolin clay, soil that presents the same degradation rate $k = 0.2$ (Ragni et al. 2015). The three cases also lay on a common path after a given penetration, differing from the $k = 0.2$ profiles where the influence of a different $q_{cons}/q_{ref}$ ratio is still present at $w/D = 2.0$.

Figure 4.18 shows the effects of $T$ on the re-penetration profiles, where intuitively both $k = 0.05$ (Figure 4.18a) and $k = 0.2$ (Figure 4.18b) present $q_{peak}$ values proportional to $T$. An increasing gap between the re-penetration profiles and the reference case (dashed line) is observed when approaching $w/D = 2.0$, confirming the long-lasting effect of enhanced bearing capacity post-consolidation described experimentally by Bienen et al. (2015) for the same carbonate silty clay. For $k = 0.05$, an increment in $T$ leads to higher $q_{peak}$ values and more pronounced long-term beneficial effects, whereas for $k = 0.2$ the severity of the punch-through is worsened by higher $T$, with the extent of uncontrolled leg penetration increasing from $0.25D$ for $T = 0.005$ to more than $0.7D$ for $T = 0.746$.

### 4.4.3 Implications for practice

The preceding figures illustrated the effects of a range of parameters influencing post-load-hold spudcan behaviour. In particular, the potential impact of high rates of the degradation of soil sensitivity emphasises the need to understand the soil conditions – in situ and undisturbed, as reconstituted samples may show differing behaviour. The parametric study presented in this paper indicates the range of behaviour that may be expected during spudcan penetration in soils where excess pore pressures (at least partially) dissipate during pauses in installation.

As a recommendation for practice it follows then, in addition to standard procedures to calibrate the parameters of the hypoplastic family of model, one needs to perform:

- Penetrometer (T-bar or ball) tests in situ during site investigation. This will provide an indication of the shear strength profile;
- Episodes of cyclic (T-bar or ball) penetrometer testing. This will provide an indication of the sensitivity;
- Penetrometer tests with pauses during penetration. This will provide information regarding the behaviour post-consolidation.

The findings presented in this paper allow the identification of general trends in behaviour but cannot cover all possible parameter combinations. Specific predictions
Figure 4.17: Effects of different load sustained in consolidation $q_{cons}/q_{ref}$ on the vertical bearing capacity profiles of natural soil with $s_{ini} = 20$
hence require analysis. Penetrometer data such as that recommended above allows calibration of:

- $O_c$ based on the initial penetration resistance;
- $s$ based on the cyclic penetrometer tests;
- $k$ based on the behaviour during further penetration following a pause.

The implementation of the effects of sensitivity revealed the importance of modelling the correct soil characteristics: the results showed distinct behaviours when reconstituted rather than natural soil is modelled, and different sensitivity degradation rates led to different behaviours post-consolidation. This numerical study is a demonstration of a viable and cost effective approach to revealing aspects of natural soil behaviour for spudcan penetration consolidation penetration problems, when used in combination with reliable field data on natural soil and standard laboratory tests on reconstituted samples.

### 4.5 Concluding remarks

The implementation of a hypoplastic constitutive model for structured clays in a large deformation finite element code was presented. The performance of the implementation has been validated through retrospective simulation of centrifuge test data, carried out on
a carbonate silty clay recovered from the north west shelf of Australia. The analyses reproduced the effects of pauses during spudcan installation, showing the increase in bearing capacity generated by such interruptions. A parametric study expanded the limited experimental database to cover a wide range of depth and length of load-hold period, during which consolidation occurs due to dissipation of excess pore pressures. The implementation of sensitivity allowed soil in reconstituted and natural conditions to be modelled, revealing contrasting behaviours post-consolidation, that is dependent on the amount of sensitivity and its degradation rate.
References


Chapter 4. Numerical modelling of the effects of consolidation on jack-up spudcan penetration


Chapter 5

Numerical modelling of the effects of consolidation on the undrained spudcan capacity under combined loading in silty clay

5.1 Abstract

The paper shows the increase in vertical and combined horizontal and moment bearing capacity of jack-up spudcan installed in silty clay, when a load-hold period is accounted for. The numerical implementation of a hypoplastic model for structured clays, combined with large deformation coupled analyses allowed the modelling of the spudcan installation process. Results were mapped into three-dimensional small strain analyses, conducted to investigate the combined loading capacity and describe the yield surface. The underlying failure mechanisms were investigated and increases in capacity due to consolidation determined. Experimental centrifuge data on carbonate silty clay validated the qualitative trend revealed numerically.

5.2 Introduction

Foundations for offshore jack up platforms, also known as spudcans, are subjected to different loading conditions during their lifetime. The installation phase involves mainly vertical loading, as the installation is achieved by penetrating the spudcan into the soil under the self-weight of the structure and additional seawater ballast. Once in service, however, the spudcan is required to provide the necessary resistance to lateral and
Figure 5.1: Three dimensional yield surface and its expansion due to consolidation

overturning loads which arise from metocean actions, such as waves, currents and wind (ISO 2012). For this reason, the combined bearing capacity under multi directional vertical, horizontal and moment loading of the foundation must be demonstrated in a site-specific assessment.

Models that describe the interaction of vertical, horizontal and moment degrees-of-freedom in the resulting combined foundation capacity were first proposed by Roscoe and Schofield (1956), with the introduction of a yield surface concept. Further developments from Butterfiled and Ticof (1979) and Schotman (1989) led to the first complete incremental force-resultant model based on plasticity theory for spudcan footings. Nova and Montrasio (1991) introduced the idea of macro-element to evaluate settlements and rotations of shallow foundations. The formulation of full macro-element models involves a yield surface to describe the combined bearing capacity of the footing in the vertical $V$, horizontal $H$ and moment $M$ loading space ($VHM$, Figure 5.1).

Intuitively, the size of this surface is a function of the vertical load $V$, which in turn depends on the spudcan embedment $w$.

Bienen et al. (2006), Cassidy and Houlsby (1999), Dean et al. (1997), and Salciarini and Tamagnini (2009) proposed macro-element models for spudcans and shallow circular foundations in sand, where fully drained behaviour can be assumed. Martin and Houlsby (2000, 2001) and Zhang et al. (2011, 2013) focused instead on the response in clay,
where undrained soil behaviour is observed. Models that describe the $VHM$ capacity of foundations with other geometries (strip, rectangular, skirted) have also been proposed (Bransby and Randolph 1998; Gourvenec 2007; Houlsby and Puzrin 1999).

An important aspect not considered in previous research is the consolidation that can take place in intermediate soils (i.e. silty soils that display intermediate drainage characteristics), as a consequence of excess pore pressure dissipation. This may occur during jack up installation, as a consequence of pauses in leg penetration, and during the phase of operation, as the self-weight of the jack-up platform maintains a mean vertical stress on the foundation. Bienen and Cassidy (2013), Bienen et al. (2015), Ragni et al. (2016), Stanier et al. (2014), and Wang and Bienen (2016) demonstrated that maintaining the vertical load, or a fraction of it, on the spudcan for a certain time results in excess pore pressure dissipation around the spudcan, a reduction in void ratio and increased shear strength. A consequent $V$ peak upon further penetration, following this consolidation phase, is schematically illustrated in Figure 5.1. Consolidation is expected to result in increases in the combined bearing capacity as well, though the improvement may not be uniform in all directions such that the yield surface may change shape as it expands. In the case of spudcan footings, the combined capacity will be tested under storm conditions. When considering the rate at which metocean actions are applied, the foundation response under $VHM$ loading, even in intermediate soils, can be expected to be undrained.

This paper investigates the increase in undrained $VHM$ capacity of a spudcan in silty clay following a period of consolidation under vertical load by means of numerical analyses. In contrast to most of the previous numerical analyses (Templeton 2009; Templeton et al. 2005; Zhang et al. 2011), where the footing was pre embedded at a target depth with the soil undisturbed, the installation phase was modelled here prior to consolidation and determination of the $VHM$ capacity. This therefore accounts for the remoulding effect and entrapment of soil under the spudcan during the initial installation.

5.3 Numerical model

An appropriate description of the problem as illustrated in Figure 5.1 requires first of all the modelling of spudcan penetration to a certain depth and the following consolidation stage. This involves large vertical penetration (here one footing diameter $D$, or 12 m, was chosen) and results in a high degree of shearing and remoulding in the soil. Therefore, a two-dimensional Large Deformation Finite Element (LDFE) approach was employed. Secondly, the hypoplastic constitutive model for structured clays adopted was able to simulate the soil remoulding experienced in the large-amplitude penetration. The model
was suitable for the full simulation of the installation, consolidation and the following undrained \textit{VHM} loading, with the variations of excess pore pressures and void ratio captured.

Determination of the combined bearing capacity usually requires Small Strain Finite Element (SSFE) analyses only, given the limited soil strains and consequent mesh distortion, compared to the large deformation occurring during penetration. In contrast to the installation phase, which can be modelled using axisymmetry, determination of the combined \textit{VHM} capacity requires a three-dimensional model. To account for the effects of installation, a rigorous mapping procedure of the soil properties and state variables is required, when passing from the axisymmetric LDFE model to the SSFE analyses.

The following sections explain in more detail the above mentioned characteristics of the numerical model.

### 5.3.1 Large deformation analyses of spudcan penetration and consolidation

The process of spudcan installation involves large deformations and results in a significant amount of straining and remoulding in the soil (Hossain et al. 2005; Hu et al. 2015; Yi et al. 2014; Zhang et al. 2014b), exceeding traditional small strain numerical analysis capabilities. For this reason, this research adopted a Remeshing and Interpolation Technique with Small Strain (RITSS, Hu and Randolph 1998; Wang et al. 2015) strategy, where the entire penetration of the spudcan was divided into a sequence of multiple steps, each small enough to avoid excessive mesh distortion. At the end of each step, the distorted mesh was regenerated, while state variables and excess pore pressures are mapped from the previous step to the next. Once the target depth \( w \) was reached, the consolidation stage was performed by holding the load, or a fraction of it, for a given time. Simulation of coupled effective stress pore fluid analyses was hence necessary to track the generation of excess pore pressures during penetration and following dissipation during consolidation.

Following this approach, Ragni et al. (2016) showed increases in vertical bearing capacity resulting from pauses in penetration, with the magnitude of such increments being proportional to the length of the consolidation phases (shown in Figure 5.6, which is discussed in the results section). The implementation of soil sensitivity also revealed contrasting behaviours post consolidation, either beneficial or detrimental for the stability of the structure, depending on the amount of sensitivity and its rate of degradation.

A thorough description of the effects of consolidation on spudcan penetration, modelled
with RITSS analyses, can be found in Ragni et al. (2016) and Wang and Bienen (2016).

### 5.3.2 Hypoplastic constitutive model for structured clays

In contrast to conventional elasto plasticity, where a clear distinction is made between elastic and plastic deformations, hypoplastic models introduce irreversible strains inside what is known as the state boundary surface, for the elastic range is known to be extremely limited in soils. In the present work, an advanced hypoplastic constitutive model for clays was adopted, developed by Mašín (2014) and implemented in Abaqus/Standard as an updated version of the user subroutine proposed by Gudehus et al. (2008). (The coded models are available for free download from www.soilmodels.info).

The basic hypoplastic clay model presents a framework similar to that of any other critical state soil mechanics-based model. Figure 5.2 shows the bi-logarithmic compression law in the $\ln(1 + e) \sim \ln(p'/p_r)$ plane. The isotropic normal compression line is described using the equation

$$\ln (1 + e) = N - \lambda^* \ln \left( \frac{p'}{p_r} \right)$$  \hspace{1cm} (5.1)

where $e$ is the void ratio, $p'$ the mean effective stress and $p_r = 1$ kPa a reference stress. The parameters $N$ and $\lambda^*$ define its position and slope respectively. The parameter $\kappa^*$ controls the slope of the isotropic unloading line in the same plane and the isotropic compression line of over-consolidated soil. The slope of the critical state line $M$ is defined through the critical state friction angle $\varphi_c$:

$$M = \frac{6 \sin \varphi_c}{3 - \sin \varphi_c}$$  \hspace{1cm} (5.2)

with identical meaning to other critical state soil mechanics based models. To correctly represent the position of the critical state line, parameter $O_c = 1.5$ controlling relative positions of isotropic normal compression line and critical state line has been included in the model; for more details see Ragni et al. (2016). Finally, the description of the basic model is completed by the parameter $\nu$, which controls the shear modulus and reads:

$$\nu = \frac{3r(\lambda^* + \kappa^*) - 4\kappa^*}{6r(\lambda^* + \kappa^*) + 4\kappa^*}$$  \hspace{1cm} (5.3)

where $r$ is the ratio of the bulk modulus in isotropic compression at the isotropic normally consolidated state and the shear modulus in undrained shear.

In order to model the effects of soil softening and remoulding due to plastic straining, the model was enhanced with the implementation of soil sensitivity, which required a
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Figure 5.2: Bi-logarithmic compression law for structured clays
sensitivity state variable $s$ and three parameters $k$, $A$ and $s_f$ to be introduced (Mašín 2007). Sensitivity $s$ is quantified by the ratio of intact and reconstituted soil shear strength, as defined in Cotecchia and Chandler (2000). $s_f$ is the value of sensitivity in reconstituted conditions and can generally be assumed as unity. The parameter $k$ governs the rate of sensitivity degradation:

$$
\dot{s} = -\frac{k}{\lambda^*} (s_{ini} - s_f) \dot{\varepsilon}^d
$$

(5.4)

where $s_{ini}$ and $s_f$ represent the initial and final values of sensitivity, and $\dot{\varepsilon}^d$ is the damage strain rate. The parameter $A$, which only has a minor effect on the results, controls the role of volumetric strain $\varepsilon^v$ and shear strain $\varepsilon^v$ components:

$$
\dot{\varepsilon}^d = \sqrt{(\dot{\varepsilon}^v)^2 + \frac{A}{1-A} (\dot{\varepsilon}^s)^2}
$$

(5.5)

The equation for the normal compression line shown in Figure 5.2 can be then updated, accounting for sensitivity, as follows

$$
\ln(1+e) = N - \lambda^* \ln\left(\frac{p'_{cr}}{p'_{cr}}\right) + \lambda^* \ln(s)
$$

(5.6)

5.3.3 Soil description

The hypoplastic constitutive relation for structured clays was adopted for its suitability to investigate the effects of spudcan penetration and consolidation while allowing the characterisation of the resulting undrained VHM capacity. The choice of parameters describes a natural soil encountered in the Laminaria field, off the Northern Coast of Western Australia. This carbonate silty clay has a particle distribution of about 10% sand, 70% silt and 20% clay and a specific unit weight as a function of the depth $w$ (in unit of m of $\gamma' = 5.6 + 0.022 w\, \text{kN/m}^3$ (Randolph et al. 1998). Experimental investigation in a geotechnical centrifuge provided an undrained shear strength profile $s_u = 2.2w\, \text{kPa}$ using T-bar tests (Ragni et al. 2016). Previous studies by Amodio et al. (2015) and Erbrich (2005) on a similar soil highlighted a high degree of in situ sensitivity, from 3.5 up to 20, due to its high content of calcium carbonate and grain crushing at increasing stress level. Cyclic T-Bar episodes in a geotechnical centrifuge revealed a sensitivity $s = 2.9$; this is considerably lower than the values reported in Erbrich (2005), due to the continuing testing and remoulding of the silty clay used in the centrifuge.

A constant value of the over-consolidation ratio $OCR = 1.205$ was calculated from the model equations to initialise the void ratio in the soil for $K_0$ normally consolidated conditions. It should be noticed that, differently from traditional soil mechanics
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definition, in hypoplasticity a normally consolidated profile is characterised by $OCR > 1$, as $OCR$ represents the ratio of Hvorslev equivalent pressure over mean effective stress $p$. Jacky (1948) provided the earth pressure coefficient at rest $K_0 = 1 - \sin \phi_c = 0.441$.

Soil permeability $k_w$ governs the excess pore pressure build-up in penetration and subsequent dissipation rate during the consolidation stage. Following basic laboratory tests, described in Mahmoodzadeh et al. (2015) and Ragni et al. (2016), an isotropic permeability was assigned to the soil as follows

$$ k_w = \gamma_w m_v c_v = \frac{\gamma_w c_v \lambda^*}{\sigma'_v} $$

(5.7)

where $\gamma_w$ is the unit weight of water, $m_v$ is the coefficient of volume compressibility of soil, $c_v$ the coefficient of consolidation and $\sigma'_v$ the vertical effective stress. Due to the bi-logarithmic nature of the compressive law illustrated in Figure 5.2, the coefficient of compressibility is calculated as $m_v = \lambda^*/\sigma'_v$. The coefficient of consolidation $c_v$ measured via the oedometer test was

$$ c_v = \frac{\sigma'_v}{10 + 0.478\sigma'_v} $$

(5.8)

The units of $c_v$ and $\sigma'_v$ are m$^2$/year and kPa, respectively.

The parameters related to the basic model and sensitivity are listed in Table 5.1. These match the values adopted in Ragni et al. (2016), where the same carbonate silty clay was investigated, with the implementation of the same numerical model. The parameter calibration is described in Ragni et al. (2016), where basic laboratory tests (triaxial and oedometer tests) and retrospective simulations of boundary value problems (spudcan vertical penetration in geotechnical centrifuge) underpinned the procedure. All the necessary steps towards an accurate model calibration were taken in Ragni et al. (2016): for this reason, the model was also assumed to be suitable for the description of the problem here investigated.

### 5.3.4 From two-dimensional to three-dimensional analyses

The entire process of spudcan penetration with a phase of consolidation, followed by excursions of combined displacements, is a difficult problem to model numerically. Due to the mono-directional geometry of the problem, the penetration and consolidation were analysed in an axisymmetric model, without the need to recur to a three-dimensional model which would affect the efficiency of the analyses. LDFE model with coupled pore fluid-stress response allowed large strains generated during installation to be modelled and excess pore pressure variations to be captured. Moreover, the implementation of soil
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### Basic Model Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi_c$ CS friction angle</td>
<td>34°</td>
</tr>
<tr>
<td>$p'$ = 1kPa NCL slope</td>
<td>1.697</td>
</tr>
<tr>
<td>$\lambda^*$</td>
<td>0.114</td>
</tr>
<tr>
<td>$\kappa^*$</td>
<td>0.013</td>
</tr>
<tr>
<td>$\nu$ Control on shear modulus</td>
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</tr>
</tbody>
</table>

### Sensitivity Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_{ini}$ Initial sensitivity</td>
<td>2.9</td>
</tr>
<tr>
<td>$k$ Sensitivity degradation rate</td>
<td>0.05</td>
</tr>
<tr>
<td>$A$ Vol/shear strains effect</td>
<td>0.2</td>
</tr>
<tr>
<td>$s_f$ Final sensitivity</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 5.1: Hypoplastic model parameters for Laminaria carbonate silty clay

Sensitivity contributed to the description of the installation effects. Eight-node elements with biquadratic displacement, bilinear pore pressure and reduced integration (termed CAX8RP in Dassault Systèmes 2012) were used to discretise the soil. However, simulation of multi-directional VHM problems required to move from the axisymmetric to a three-dimensional model. Due to the geometry of the problem under consideration, only half of the three-dimensional model (i.e. 180°), rather than a full 360° domain, needed to be considered. Twenty node elements, with triquadratic displacement, trilinear pore pressure and reduced integration (C3D20RP in Dassault Systèmes 2012) allowed the simulation of coupled analyses and were used to discretise the soil.

The spudcan geometry is shown in Figure 5.3. It was modelled as a rigid body, given its much higher stiffness compared to the soil. The load displacement response of the spudcan is represented by three degrees of freedom (vertical, horizontal and rotational) referring to the Reference Point (RP) and with the sign convention illustrated in Figure 5.3.

The three-dimensional soil domain was chosen as 3D vertically and 6D horizontally (see Figure 5.4), in order to minimise boundary effects and computational cost. This is a “regular” domain size for undrained bearing capacity analyses using small-strain finite element method. In contrast, the soil extensions were as large as 20D in the axisymmetric LDFE analyses of penetration and consolidation, to avoid any potential boundary effect. Three-dimensional analyses involving a larger soil domain (20D vertically and 40D horizontally) offered a solution extremely close to the smaller domain (an average of -1.6% in the loading response), while at the same time dramatically increasing the computational cost by $\sim 400\%$. Since the aim of the paper was to isolate the bearing capacity offered by the footing alone, the jack up leg connected to the spudcan was not modelled intentionally in the three-dimensional
analyses (RHS of Figure 5.5a, b, c), whereas it was involved in the LDFE study to capture the effects of the installation process properly (LHS of Figure 5.5a, b, c).

After creating the three-dimensional geometry by revolving the axisymmetric model around a 180° angle, particular attention was dedicated to the mapping strategy. Once the spudcan was penetrated to the target depth with LDFE analyses, the resulting effective stresses and material properties were mapped to the integration points of the three dimensional model, whereas the excess pore pressures were mapped to the nodes. The mapping strategy was such that every integration point and node of the three-dimensional model was searched in the axisymmetric model, by conversion of its three dimensional coordinates to the axisymmetric corresponding counterpart. Then, each field variable was interpolated within the eight node axisymmetric element which contains the integration point or the node. A mesh density similar to that of the axisymmetric model is desirable to minimise the error in interpolation. For the limited volume of soil replacing the leg which was present in LDFE analyses, it was assumed that the field variables were equal to those at the nearest integration point or node. Figure 5.5 shows an example of the results obtained when mapping (a) excess pore pressures, (b) sensitivity and (c) void ratio from the LDFE analyses (LHS) in the three-dimensional model (RHS). The minimal discrepancy between the vertical force applied to the spudcan in the two-dimensional and three-dimensional model after mapping (limited to 4%) confirmed the validity of the mapping procedure.
Figure 5.4: Mesh density and soil domain dimensions
Figure 5.5: Results of mapping (a) excess pore pressures, (b) sensitivity, (c) void ratio, from 2D (LHS) to 3D (RHS) analyses
5.3.5 Definition of the yield surface

In order to interpret the results, an equation to describe the shape of the yield surface in the $VHM$ space must be first defined. It is here outlined in the form proposed by Martin and Houlsby (2000) for heavily overconsolidated clay and later expanded by Vlahos et al. (2008) to account for tensile capacity. It reads:

$$f = \left( \frac{H}{h_0 V_0} \right)^2 + \left( \frac{M/D}{m_0 V_0} \right)^2 - \frac{2e_sHM/D}{h_0m_0V_0^2} - \left( \frac{4}{(1+\chi)^2} \right)^2 \left( \frac{V}{V_0} + \chi \right)^2 \left( 1 - \frac{V}{V_0} \right)^2 \quad (5.9)$$

where $h_0$ is the peak of horizontal over vertical capacity in the $VH$ ($M = 0$) plane, $m_0$ is the peak of moment over vertical capacity in the $VM$ ($H = 0$) plane, $e_s$ represents the eccentricity of the surface in $HM$ plane, $V_0$ is the vertical bearing capacity and $\chi$ is the ratio of peak tensile over compressive capacity.

If a non-vertical force term $Q$ is defined as

$$Q^2 = \left( \frac{H}{h_0} \right)^2 + \left( \frac{M/D}{m_0} \right)^2 - \frac{2e_sHM/D}{h_0m_0} \quad (5.10)$$

then Equation 5.9 can be simplified and re-written as

$$f = \left( \frac{Q}{V_0} \right)^2 - \left( \frac{4}{(1+\chi)^2} \right)^2 \left( \frac{V}{V_0} + \chi \right)^2 \left( 1 - \frac{V}{V_0} \right)^2 \quad (5.11)$$

and the results presented in the non-dimensional $Q/V_0 - V/V_0$ plane. The advantage of such simplification is that, provided the optimal set of parameters, all the tests should lay on the same curve. Zhang et al. (2014a) showed Equation 5.9 to be applicable also for spudcans experimentally penetrated into soft normally consolidated kaolin clay, with recommended parameters values at a penetration depth $w/D = 1.0$ included in Table 5.2.

5.3.6 $VHM$ investigation strategy: three-dimensional analyses

Investigation of the undrained $VHM$ capacity of a spudcan following penetration and consolidation in silty clay is the main focus of this paper. For numerical stability and computational efficiency, the spudcan-soil interaction was simplified with a tie constraint, which tends to overestimate the capacity. However, further analyses involving frictionless contact showed a marginal effect (2.5% difference in pure rotation) on the magnitude of the combined bearing capacity.

Roughly 15000 elements were used to discretise the soil domain. The minimum element size around the spudcan was $0.045D$; further simulations reducing this value to $0.02D$
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<table>
<thead>
<tr>
<th>$T$</th>
<th>$h_{0,num}$</th>
<th>$h_{0,exp}$</th>
<th>$m_{0,num}$</th>
<th>$m_{0,num}$</th>
<th>$e_s$</th>
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<tbody>
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<td>0 (Zhang et al. 2014a)</td>
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<td>-</td>
<td>0.092</td>
<td>0.244</td>
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<tr>
<td>0.01</td>
<td>0.309 (+4.0%)</td>
<td>0.215 (+15.6%)</td>
<td>0.151 (+2.7%)</td>
<td>0.085 (+13.3%)</td>
<td>0.1</td>
<td>0.5</td>
</tr>
<tr>
<td>0.05</td>
<td>0.387 (+30.3%)</td>
<td>0.268 (+44.1%)</td>
<td>0.152 (+3.4%)</td>
<td>0.089 (+18.6%)</td>
<td>0.1</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 5.2: Yield surface parameters for soft normally consolidated clay (Zhang et al. 2014a) at $w/D = 1$. Percentage increases refer to corresponding numerical or experimental $T = 0$ case

proved to have a marginal effect of 1% while, at the same time, increasing the computational effort by 700%. Further reduction of the element size in relation to the spudcan displacements is thought to generate excessive mesh distortion (Yi et al. 2014) and cause analysis instability.

After penetrating the spudcan to a target depth $w$, allowing for a period of consolidation under constant vertical load and mapping the state variables to the three-dimensional model, probe and swipe tests were carried out in order to investigate the combined bearing capacity of the spudcan. In case of probe tests (Gottardi et al. 1999; Tan 1990), either a horizontal displacement of the spudcan $h$, or a rotation $\theta$ was applied at a constant value of $V$, which was assumed as a fraction of the vertical bearing capacity. These tests were used to investigate the underlying failure mechanisms and the effect of different excess pore pressure distributions. In the swipe tests, firstly proposed by Tan (1990), a horizontal displacement $h$, a rotation $\theta$, or a combination of both was applied, while the spudcan depth $w$ was held constant, so as to generate load paths where $V$ gradually reduced as the response in $H$ and $M$ increased. Provided a sufficiently high ratio of elastic over plastic stiffness of the soil (Martin 1994; Martin and Houlsby 2000; Tan 1990), such load paths are thought to closely track the yield surface. All the swipe tests started from the same $V$ (with $H = M = 0$) and moved along different paths of the three-dimensional surface, according to the $u : D\theta$ ratio, so to determine the capacity surface in the $VH$ and $VM$ planes. Swipe tests were preferred to probe tests to describe the yield surface, for the latter start within the surface and would not offer a straightforward identification of the yielding point, due to accumulation of plastic deformations from the very onset of the test (as real soil does not have the perfect elasticplastic yield surface boundary we attempt to model it by). When the $HM$ response
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of each swipe was plotted at constant \( V \) values (i.e. sectioning the yield surface parallel to the \( H - M \) plane at regular intervals along the \( V \) axis), the surface eccentricity was observed, which ultimately offered a comprehensive picture of the yield surface in the \( VHM \) space.

5.4 Results

5.4.1 Vertical installation and consolidation

Results from the parametric study presented in Ragni et al. (2016) in terms of increasing bearing pressure \( q \) with depth are shown in Figure 5.6. The dashed line represents the load-penetration curve of a continuous spudcan penetration with no stoppage. Figure 5.6a and b respectively show how penetration is affected by different lengths of consolidation and vertical loads held during the pause, at increasing consolidation depths \( w/D = 1.0 ; 1.5 ; 2.0 \). In particular, increasing dimensionless consolidation time \( T = c_v t / D^2 \) (varying from 0.001 to 0.5, where \( t \) is the dimensional time) is demonstrated to generate a higher peak in bearing capacity \( q \) upon further penetration. A reduction of the load (i.e. the pressure) held in consolidation \( q_{cons} = 1.0 ; 0.75 ; 0.5 q_{ref} \) (where \( q_{ref} \) is the pressure achieved in continuous penetration) leads to the opposite effect. These data simulated the installation effects and represented the starting point of the following \( VHM \) investigation.

The results presented in the next section compare the combined bearing capacities of a spudcan installed to \( w/D = 1 \), followed by a period of consolidation \( T = 0 ; 0.01 ; 0.05 \) and where the load held during the pause was \( q_{cons} = 1.0 q_{ref} \) (tests reported in black in Figure 5.6a).

5.4.2 Example load paths: capacity increase under pure translation or rotation

It is common practise in jack-up installation to increase the vertical load by pumping sea-water into specifically designed ballast tanks to facilitate the spudcan penetration. This vertical load is then reduced during operational conditions once the installation is completed and ballast tanks emptied. The combined capacity of the footing (i.e. the size of the yield surface) can be considered as a function of the vertical capacity \( V_0 \) (with \( T = 0 \)) achieved during installation, as schematically illustrated in Figure 5.7. However, if this load is held for a certain amount of time \( t \), an increase in vertical capacity is observed due to consolidation, leading to a higher \( V^*_0 = V_0(T) \). As a result, also the combined capacity will increase accordingly.
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Figure 5.6: Load-penetration curves at increasing depth $w/D = 1.0; 1.5; 2$ showing the effects on further spudcan penetration of different (a) length of consolidation $T = 0.001; 0.005; 0.01; 0.05; 0.1; 0.5$ at constant $q_{cons}/q_{ref} = 1$ and (b) fraction of load held in consolidation $q_{cons}/q_{ref} = 0.5; 0.75; 1.0$ at constant $T = 0.5$

The probe tests presented in Figures 5.8 and 5.9 investigated the response to pure horizontal displacement $u$ and pure rotation $\theta$ of a spudcan installed to a depth $w/D = 1.0$ and were simulated at a model scale, at enhanced gravity of 200g and $D = 60$ mm. A reference case without consolidation was compared against two cases involving increasing consolidation $T = 0.01; 0.05$. In order to simulate real case scenarios, the tests were carried out by reducing $V$ to $0.5; 0.75$ and $0.9$ of the value achieved in installation (and held for consolidation, when this was included), labelled as $V_0(T = 0)$ in Figure 5.7.

All results are presented in a dimensionless fashion, unless differently stated. When normalising $H$ and $M$, the adopted ultimate vertical capacity $V_0^*$ refers to the expanded yield surface for the cases involving consolidation, i.e. $V_0(T = 0.01)$ and $V_0(T = 0.05)$ in Figure 5.7. Such $V_0^*$ values were obtained from Figure 5.6a, where the recorded peak in the load-penetration curves after consolidation at $w/D = 1$ revealed an increase of the original bearing capacity $V_0 = 665.0N$, equal to 23% for $T = 0.01$ ($V_0^* = 818.1N$) and 37% for $T = 0.05$ ($V_0^* = 914.4N$). As shown in Figure 5.6, a period of consolidation causes a certain amount of settlement of the footing, which increases with increasing $T$ and $q_{cons}$. Consequently, it should be noticed that when referring to an increase in combined bearing capacity, such an improvement should be regarded as a combination of the effect of consolidation itself and increased depth $w/D$ due to settlement, which is also responsible for an increase in capacity (Wang and Bienen 2016; Zhang et al. 2011).
In the tests reported in Figure 5.8, a final horizontal displacement $h/D = 0.03$ was targeted (please note that some of the tests failed to converge before reaching the final displacement), while rotation $\theta$ was not allowed. Similarly, Figure 5.9 involved a rotation $\theta = 4^\circ$, while no horizontal displacement was allowed. Regardless of the fraction of vertical load at which the tests were carried out ($V = 0.5; 0.75$ or $0.9$ times $V_0(T = 0)$), a higher initial stiffness is observed for increasing $T$, which degrades at increasing strain levels.

As summarised in Table 5.3, longer periods of consolidation result in larger increases in $H$ and $M$ capacity, as expected and already observed for the $V$ capacity. Towards the capacity surface apex, i.e. with $V$ approaching $V_0$, the $HM$ cross-section becomes smaller (see Figures 5.1 and 5.7), such that the $H$ and $M$ capacity is lower even for long periods of consolidation (Figures 5.8 and 5.9).

![Table 5.3](image)

<table>
<thead>
<tr>
<th>$V/V_0$</th>
<th>Test</th>
<th>$T = 0$</th>
<th>$T = 0.01$</th>
<th>$T = 0.05$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>$h (H)$</td>
<td>N/A</td>
<td>22.60%</td>
<td>52.90%</td>
</tr>
<tr>
<td></td>
<td>$\theta (M)$</td>
<td>N/A</td>
<td>14.30%</td>
<td>43.00%</td>
</tr>
<tr>
<td>0.75</td>
<td>$h (H)$</td>
<td>N/A</td>
<td>32.20%</td>
<td>69.60%</td>
</tr>
<tr>
<td></td>
<td>$\theta (M)$</td>
<td>N/A</td>
<td>20.70%</td>
<td>56.30%</td>
</tr>
<tr>
<td>0.9</td>
<td>$h (H)$</td>
<td>N/A</td>
<td>37.30%</td>
<td>79.10%</td>
</tr>
<tr>
<td></td>
<td>$\theta (M)$</td>
<td>N/A</td>
<td>32.00%</td>
<td>77.40%</td>
</tr>
</tbody>
</table>

Table 5.3: Percentage increase in $H/V_0$ and $M/DV_0$ for pure translation and rotation tests shown in Figures 5.7 and 5.8

The next sections explore the reasons behind the increase in combined capacity observed in Figures 5.8 and 5.9, highlighting differences in failure mechanism and excess pore pressure distribution caused by consolidation.

**Failure mechanism**

Figures 5.10 and 5.11 show the resultant displacement contour plots observed in the $uw$ plane, with an applied vertical load $V = 0.5V_0(T = 0)$, following $h/D = 0.03$ and $\theta = 4^\circ$ respectively (load-displacement curves in Figures 5.8a and 5.9a), for a) the case with no consolidation and b) a consolidation period $T = 0.05$.

When observing Figure 5.10 ($h/D = 0.03$), similarities can be found with the study presented by Zhang et al. (2011) (see Figure 5.10 inset) on kaolin clay, with a prevalent sliding mechanism at the base and scoop mechanism above the spudcan. The position of the centre of the scoop mechanism is also similar for the depth $w/D = 1.0$ analysed.
shallow mechanism can be observed in both cases, which extends to the soil surface as in Zhang et al. (2011). Differences with Zhang et al. (2011) lie in the increased asymmetry of the failure mechanisms presented here. The test described is such that passive and active zones are activated as a result of the footing sliding horizontally. Consequently, the adoption of an advanced constitutive model can better describe the different stiffness in loading and unloading, as well as the different strength in compression and extension.

On the other side, the adoption of a simpler constitutive model adopted in Zhang et al. (2011) (Tresca model and total stress analyses) can be responsible for an incorrect symmetry. The higher heterogeneity in the silty clay due to higher shear strength (2.1 kPa/m compared with 1.2 kPa/m in kaolin) may also be responsible for the increased asymmetry. Also, when the footing is also idealised as wished-in-place (Templeton 2009; Templeton et al. 2005; Zhang et al. 2011), the presence of a soil cavity above the spudcan is denied. While the general shape of the failure mechanism is similar for cases with and without a consolidation period, consolidation is shown to increase the size (Figure 5.10).

Similar conclusions can be drawn from Figure 5.11 (θ = 4°), with the enlarged failure mechanism of comparable shape. As already observed in Zhang et al. (2011) (Figure 5.11 inset), the mechanism is rotational, with centre of rotation close to the RP at \( w/D = 1 \). Differently from Zhang et al. (2011) though, in this case the failure mechanism extends to the soil surface, as a consequence of the correct modelling of the installation procedure and the improved constitutive model implemented.

It is clear that although the failure mechanisms presented in Figures 5.10b and 5.11b (\( T = 0.05 \)) are slightly more extended than their counterpart without consolidation, this mobilisation of larger portions of soil is not sufficient on its own to justify the increase in

---

**Figure 5.7:** Schematic of increase in yield surface size and \( V_0 \) magnitude due to a period of consolidation
Figure 5.8: Load-displacement curves \( (h/D - H/V_0) \) following pure horizontal displacement at increasing \( T \) for (a) \( V = 0.5V_0(T = 0) \); (b) \( V = 0.75V_0(T = 0) \); (c) \( V = 0.9V_0(T = 0) \)
Figure 5.9: Moment-rotation curves ($\theta - M/DV_0$) following pure rotation at increasing $T$ for (a) $V = 0.5V_0(T = 0)$; (b) $V = 0.75V_0(T = 0)$; (c) $V = 0.9V_0(T = 0)$
combined capacity observed in Figures 5.8 and 5.9. For this reason, the next section investigates the influence of dissipation of excess pore pressures on the foundation capacity under combined loading.

**Excess pore pressure distribution**

The penetration process causes the generation of excess pore pressures in the region of soil around the spudcan, for undrained conditions govern the process. However, with a load-hold period following the penetration, dissipation of excess pore pressures is observed. Bienen et al. (2015) experimentally showed that holding the full vertical load mobilised at \( w/D = 1.0 \) for a period of time as long as \( T = 0.05 \), the excess pore pressure ratio \( B_q = \Delta u/q_{nom} \) (being \( \Delta u \) the excess pore pressures at the base of the spudcan and \( q_{nom} \) the bearing pressure during consolidation) reduces about 40% from the value recorded before consolidation. As a consequence, the void ratio will also reduce, as shown in Figure 5.12, where the void ratio distribution for the case without consolidation (LHS) and with consolidation \( T = 0.05 \) (RHS) are compared. The reduction in void ratio is accompanied with the increase in the undrained shear strength \( s_u \). The enhancement of undrained strengths ultimately affects the combined capacity of the footing, as observed in the increased responses given by the numerical scenarios involving consolidation in Figures 5.8 and 5.9.

The assumption of undrained conditions can be formulated during the VHM investigation. Figures 5.13 and 5.14 confirm the behaviour described above by showing the distribution of excess pore pressures \( \Delta u \) (kPa) when \( h/D = 0.03 \) and \( \theta = 4^\circ \) are applied a) immediately after penetration or b) after a consolidation period \( T = 0.05 \). Figures 5.13a and 5.14a show higher levels of \( \Delta u \) developing due to the elevated excess pore pressures sustained from the spudcan penetration process. In contrast, Figures 5.13b and 5.14b show much lower \( \Delta u \) as a result of the dissipation during the consolidation stage. Regions with negative \( \Delta u \) are also observed below the base of the footing, generated when \( h \) or \( \theta \) are applied starting from \( \Delta u \sim 0 \). The negative excess pore pressures contribute to the generation of suction, causing increase in capacity.

### 5.4.3 Quantification of yield surface expansion due to consolidation

**Size and shape along the vertical load axis**

Figure 5.15 reports the results of the numerical SSFE simulations of swipe tests by plotting the load paths in terms of \( Q/V_0 - V/V_0 \) for scenarios (a) with no consolidation \( (T = 0) \) and with consolidation of (b) \( T = 0.01 \) and (c) \( T = 0.05 \). All the results were normalised by the respective increased bearing capacity \( V_0^* \), which was also the starting
Figure 5.10: Comparison of resultant displacement contours following pure horizontal displacement (a) without consolidation \( T = 0 \); (b) including a period of consolidation \( T = 0.05 \). *: Zhang et al. (2011) inset not in scale.
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**Figure 5.11:** Comparison of resultant displacement contours following pure rotation (a) without consolidation $T = 0$; (b) including a period of consolidation $T = 0.05$. *: Zhang et al. (2011) inset not in scale
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Figure 5.12: Void ratio distribution (a) without consolidation and (b) with consolidation $T = 0.05$
Figure 5.13: Comparison of excess pore pressure contours following pure horizontal displacement (a) without consolidation $T = 0$; (b) including a period of consolidation $T = 0.05$.
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Figure 5.14: Comparison of excess pore pressure contours following pure rotation (a) without consolidation $T = 0$; (b) including a period of consolidation $T = 0.05$
point of the test (see inset Figure 5.15). In order to offer a straightforward comparison with available centrifuge experimental data later presented, the tests were simulated at a model scale, at enhanced gravity of 100g (swipe tests carried out are summarised in Table 5.4). Numerical simulations of the penetration-consolidation-penetration were necessary to determine the increased $V_0^*$. Increments of 32% and 69% against $V_0(T=0) = 305.6\text{N}$ were obtained for $T=0.01$ ($V_0^* = 403.6\text{N}$) and $T=0.05$ ($V_0^* = 518.8\text{N}$), respectively.

<table>
<thead>
<tr>
<th>Swipe Tests (ST)</th>
<th>Name</th>
<th>$u$ (mm)</th>
<th>$u_{exp}$ (mm/s)</th>
<th>$\theta$ (°)</th>
<th>$\theta_{exp}'$ (°/s)</th>
<th>$u/D\theta$ (-)</th>
<th>Num T</th>
<th>Exp T</th>
</tr>
</thead>
<tbody>
<tr>
<td>ST1</td>
<td>12</td>
<td>0.4</td>
<td>0</td>
<td>0</td>
<td>$\infty$</td>
<td>0; 0.01; 0.05</td>
<td>0; 0.01; 0.05</td>
<td></td>
</tr>
<tr>
<td>ST2</td>
<td>6</td>
<td>0.4</td>
<td>6</td>
<td>0.4</td>
<td>0.95</td>
<td>0; 0.01; 0.05</td>
<td>0; 0.05</td>
<td></td>
</tr>
<tr>
<td>ST3</td>
<td>-0.6</td>
<td>-0.04</td>
<td>6</td>
<td>0.4</td>
<td>-0.095</td>
<td>0; 0.01; 0.05</td>
<td>0; 0.05</td>
<td></td>
</tr>
<tr>
<td>ST4</td>
<td>0</td>
<td>0</td>
<td>6</td>
<td>0.4</td>
<td>0</td>
<td>0; 0.01; 0.05</td>
<td>0; 0.01; 0.05</td>
<td></td>
</tr>
<tr>
<td>ST5</td>
<td>-3.6</td>
<td>-0.24</td>
<td>6</td>
<td>0.4</td>
<td>-0.57</td>
<td>0; 0.01; 0.05</td>
<td>0.05</td>
<td></td>
</tr>
<tr>
<td>ST6</td>
<td>-7.2</td>
<td>N/A</td>
<td>6</td>
<td>N/A</td>
<td>-1.14</td>
<td>0; 0.01; 0.05</td>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.4: Summary of swipe tests simulated numerically and experimentally

For each consolidation time, an equation of the yield surface (black curve) was determined to best fit the experimental results, according to Equation 5.11. The fitting exercise concentrated on the optimisation of $h_0$, $m_0$ and $e_s$ through least squares regression. Centrifuge data of spudcan penetration followed by immediate extraction provided a ratio of tensile to compressive vertical capacity of $\chi = 0.5$. The same value was adopted for the cases involving consolidation, for no sufficient experimental data were available to determine an exact value. The value $\chi = 0.5$ is slightly lower than presented in Zhang et al. (2014a) for soft clay ($\chi = 0.6$), possibly due to the higher sensitivity of the silty clay ($s = 2.9$ for silty clay and 2.2 for soft clay), which causes the remoulded soil above the spudcan to offer a lower resistance in extraction.

The fitting exercise returned $h_0 = 0.297$ and $m_0 = 0.147$ for $T = 0$, offering the good agreement shown in Figure 5.14a. As a period of consolidation was taken into account, $h_0$ were observed to increase to 0.309 (+4.0%) and 0.387 (+30.3%) (Figure 5.15b and 5.15c), whereas $m_0$ presented only a marginal increase to 0.151 (+2.7%) for $T = 0.01
and 0.152 (+3.4%) for \( T = 0.05 \). Interestingly, not only is the absolute response in terms of \( H \) and \( M \) enhanced after consolidation, but also the normalised response increases in terms of \( h_0 \) and \( m_0 \). This means that consolidation does not simply scale up the size of the yield surface, with ratios of vertical over combined capacities unchanged, but also modifies its shape, in line with the observations of Figures 5.10 and 5.11 in terms of different sizes of the failure mechanism.

Zhang et al. (2014b) adopted a slightly different equation to describe the yield surface in the \( VHM \) space. Nonetheless, the parameters \( h_0 \) and \( m_0 \) only determine the size of the yield surface and can thereby be compared with the present study. Numerical simulations for soft clay in Zhang et al. (2014b) resulted in lower values of \( h_0 = 0.224 \) and \( m_0 = 0.120 \) for the same embedment depth \( w/D = 1 \), \( T = 0 \) and a sensitivity increased to \( s = 3 \) (close to \( s = 2.9 \) for silty clay). The reasons for the difference with the silty clay values can be found in the lower shear strength profile (1.2 kPa/m compared with 2.1 kPa/m) and slightly higher sensitivity assumed (3 compared with 2.9), which would result in a lower \( HM \) response. The increasing consolidation period generates an effect similar to the increasing embedment depth observed in Zhang et al. (2014b). In particular, a significant increase with the embedment depth of spudcan is observed for \( h_0 \) (+39% from \( w/D = 1 \) to \( w/D = 3 \)), whereas a lower increase is observed for \( m_0 \) (+13% from \( w/D = 1 \) to \( w/D = 3 \)).

**Size and shape in planes of constant vertical load**

The least square regression introduced above determined a constant eccentricity \( e_s = 0.1 \) for \( T = 0; 0.01; 0.05 \). The complete set of parameters \( (h_0, m_0, e_s, \chi) \) was used to plot sections of the yield surfaces in the normalised plane \( H/V_0 - M/DV_0 \), at constant vertical load \( V \) of 0.5; 0.75 or 0.9 times \( V_0 \) (or \( V_0^* \)), for increasing \( T = 0; 0.01; 0.05 \), as illustrated in Figure 5.16. Accordingly to the rugby ball-shaped yield surface schematically outlined in Figure 5.7, a reduction in \( HM \) capacity is observed for increasing \( V \) (from red to green), since the loading state is moved incrementally closer to the ultimate vertical capacity \( V_0 \) (or \( V_0^* \)). Having assumed the swipe tests to move along the yield surface, the intercepted response of each swipe test at the corresponding \( V/V_0 \) was also plotted as a dot in the same plane, to illustrate the parameter fit obtained through least squares regression.

The increase in \( h_0 \) due to consolidation is reflected in Figure 5.16, with the yield surface widening along the \( H/V_0 \) axis as the consolidation period increases. In contrast, the intercept on the \( M/DV_0 \) axis (i.e. \( H/V_0 = 0 \)) is almost unchanged, reflecting the minimal increase in \( m_0 \) due to consolidation. As a consequence, an increasingly different shape of the yield surface section is observed as consolidation time increases.
Figure 5.15: Results from numerical simulation of swipe tests in the normalised $Q/V_0 - V/V_0$ plane for increasing consolidation (a) $T = 0$; (b) $T = 0.01$; (c) $T = 0.05$.
On the other hand, the distribution of load path intercepts at constant $V/V_0$ ratios shows how the eccentricity is not affected by a period of consolidation, as a constant value $e_s \sim 0.1$ fits the results throughout.

### 5.4.4 Supporting evidence from centrifuge testing

A series of experimental swipe tests was carried out on carbonate silty clay in the geotechnical beam centrifuge at the University of Western Australia at enhanced gravity level of 100 g. The sample was prepared mixing the soil with minimum amount of added water and consolidated in flight for five days to achieve normally consolidated conditions. T-bar tests measured an average shear strength profile of 1.14 kPa/m prior to the spudcan tests and 2.01 kPa/m immediately after. The spudcan tests lasted approximately three days. The spudcan diameter was $D = 60$ mm with geometry as illustrated in Figure 5.3 and it was equipped with five pressure transducers, in order to monitor excess pore pressure variations throughout the test. Only a limited number of previous studies modelled the lattice leg above the spudcan (Li et al. 2012, 2014; Springman and Schofield 1998; Yang et al. 2014); in this case a cylindrical aluminium shield protected the leg equipped with axial and bending strain gauges. A soft silicon sealant was used to seal the gap between the shield and the spudcan; this ensured minimal load transfer between the two, allowing in this way to record only the loading acting on the spudcan. A detailed description of the apparatus can be found in Zhang et al. (2013, 2014a).

In the experiments, the footing was first penetrated to the target depth $w/D = 1.0$ at a rate of $0.2$ mm/s, with the normalised velocity $V = vD/v_c > 100$ to guarantee undrained conditions (Cassidy 2012; Chung et al. 2006; Finnie 1993). Then, a reference set of four tests involved displacement and/or rotation immediately after vertical penetration to $w/D = 1.0$, whereas in the remaining seven tests, a period of consolidation (either $T = 0.01$ or 0.05) was allowed between the end of the penetration and the swipe test. All $u : D\theta$ paths were applied at a rate such that undrained conditions were guaranteed. Table 5.4 summarises all the swipe tests carried out.

Figure 5.17 reports the load paths in terms of $Q/V_0 - V/V_0$: all the tests started from $V_0$ reached prior to consolidation (see inset Figure 5.16), as the increased $V_0^*$ was unknown prior to the swipe tests. The $V_0^*/V_0$ ratios determined numerically were adopted to normalise the experimental results.

In the same way as the numerical case, a constant values of $\chi = 0.5$ was used. A constant $e_s = 0.1$ was also assumed from the numerical results, for the experimental tests only covered part of the yield surface in the $H/V_0 - M/DV_0$ plane and thus could not offer a precise estimation of the eccentricity.
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Figure 5.16: Sections of the yield surfaces obtained with numerical simulation in the normalised $H/V_0 - M/DV_0$ plane for increasing $V = 0.5; 0.75; 0.9 V_0$ (or $V_0^*$) and consolidation (a) $T = 0$; (b) $T = 0.01$; (c) $T = 0.05$
The parameters $h_0$ and $m_0$ estimated through least square regression appear to be consistently lower than the numerical counterparts (Table 5.2) as well as the values presented in Zhang et al. (2014a) for soft clay. This can be attributed to a consolidation process of the silty clay sample only partially completed in the centrifuge, as demonstrated by the increase over time of the shear strength profiles from T-bar tests. Fully consolidated conditions are only achieved by spinning the sample for a long time; for this reason the dedicated five days of consolidation may not have been sufficient. When modelling the sample numerically, a fully consolidated profile is automatically achieved. This leads to a shear strength profile (2.1 kPa/m) higher than the experimental case and potentially to the discrepancy in the results. Also, the experimental loading paths involving consolidation initially lie inside the expanded yield surface, as it was not possible to experimentally determine $V_0^*$ a priori. This leads to uncertainties on when they rejoin and begin to track the yield surface, noting that the concept of pure elastic-plastic response of soil in a macro element model is just a numerical approximation. On the other hand, although it represents a reasonable assumption, the hypoplastic model may not strictly track the yield surface during the vertical unloading observed during swipe tests (see Mašín and Herle 2007).

Nonetheless, a similar qualitative trend was captured in terms of $h_0$ and $m_0$ increase with $T$. In particular, the original $h_0 = 0.186$ for $T = 0$ increased significantly to $h_0 = 0.215$ (+15.6%) for $T = 0.01$ and $h_0 = 0.268$ (+44.1%) for $T = 0.05$; whereas $m_0 = 0.075$ for $T = 0$ showed a more moderate increase to $m_0 = 0.085$ (+13.3%) for $T = 0.01$ and $m_0 = 0.089$ (+18.6%) for $T = 0.05$. Table 5.2 offers an overview of the increases in $h_0$ and $m_0$ with $T$ for both the numerical and the experimental cases.

The following section provides a comparison with the state-of-the-art procedure to estimate the combined capacity offered in the current international guidelines for jack-ups (ISO 2012), before the outcomes of the paper are summarised in the final section.

### 5.5 Recommendations for ISO (2012) guidelines

In the current guidelines offered in ISO (2012):

1. the ultimate vertical bearing capacity $V_0$ is establish according to the vertical preload;
2. only further penetration can generate an increase in $V_0$;
3. spudcan geometry and soil properties at the installation depth determine the maximum horizontal and moment capacity, which define the size of the yield
Figure 5.17: Results from experimental swipe tests in the normalised $Q/V_0 - V/V_0$ plane for increasing consolidation (a) $T = 0$; (b) $T = 0.01$; (c) $T = 0.05$
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4. guidance is only offered for clayey and sandy soils, not for silty material.

With regard to points 1) and 2), Figure 5.6 demonstrated that $V_0$ is indeed established according to the depth reached in installation, but also that it can be substantially modified due to the installation process, remoulding and through consolidation. As described in point 3), ISO (2012) does not take into account the effects of consolidation in the determination of combined capacity, as this is determined as a function of the installation depth. This paper demonstrated the effect of consolidation on the vertical and, consequently, the combined capacity. Such findings are important as they reveal the potential to achieve the targeted capacity at a shallower embedment through a planned consolidation period, should the leg length be a limiting factor in installation. Future research will establish the reliability of the enhanced capacity under cyclic loading from metocean actions. The findings can also guide probabilistic considerations on the reduced failure probability (i.e. capacity to withstand storm events with increasingly higher return period) as a function of the degree of consolidation achieved under the vertical load held over time. Finally, ISO (2012) does not offer any guidance in terms of assessment of the $VHM$ capacity for spudcan installed in silty clay. In this sense, the paper offers insights aimed at easing the difficult and challenging installation process in these soils.

5.6 Concluding remarks

A numerical investigation of the influence of consolidation on spudcan combined capacity in silty soil was presented. An increased combined capacity in terms of $H$ and $M$ was shown in the tests where a period of consolidation followed the initial vertical penetration. In particular:

- The results demonstrated the importance of simulating the entire process of spudcan installation and potential consolidation stage before a $VHM$ investigation is carried out. Similar conclusions were drawn in Zhang et al. (2014a,b) for spudcans penetrated into soft clay. On the other hand, a load-hold period is demonstrated to dramatically affect the excess pore pressure distribution, as not only does it have an influence on the vertical (Ragni et al. 2016) but also on the combined capacity (Figures 5.8 and 5.9). It is thus important to treat the problem as a whole, rather than as a sequence of independent stages from installation to operation. Since the present study is limited to one embedment $w/D = 1$, future
research shall take into account the effect of different installation depths, as it is demonstrated in Zhang et al. (2014a,b) to have an influence on the size and shape of the yield surface;

- The adoption of an advanced constitutive model revealed asymmetric failure mechanisms related to pure translation or rotation, in contrast to the results generated by the assumption of simple constitutive models, such as Tresca, where symmetric failure mechanisms can be incorrectly predicted;

- Not only does consolidation lead to an enhanced absolute response in terms of $H$ and $M$, but also to increased values of $h_0$ and $m_0$. As these relate the size of the surface to the (increased) uniaxial vertical capacity, increases in $h_0$ and $m_0$ signal a disproportionately larger increase in horizontal and moment capacity.
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References


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Chapter 6

An introduction to the cyclic loading response of spudcan foundations in carbonate silty clay

6.1 Abstract

Chapter 6 presents the implementation of the hypoplastic model with intergranular strain concept, within the framework of spudcan performance when subjected to cyclic loading. Due to the extremely complex nature of the problem, it is the aim of this Chapter to only provide an overview of the capability and the potential of the model to describe such problem. A comprehensive picture of the response of spudcan foundations to vertical and multidirectional cyclic loading will follow in a future work. The calibration of the intergranular strain parameters through laboratory tests is presented first, followed by experimental observations of spudcan cyclic loading. Finally, preliminary results from numerical simulations of spudcan vertical cyclic loading in silty clay and how the response is affected by a period of consolidation are presented. Although the results can already offer insights into the spudcan cyclic response, some numerical issues are still present; possible explanations for such model instability are provided and guidelines towards the improvement of the model are outlined.

6.2 Introduction

The installation of spudcan foundations for jack-up platforms involves the increase of vertical loading \( (V) \) until the target value is reached. In order to guarantee a safe installation, the initial vertical penetration should be completed in a short period of time, in calm waters and mild weather conditions, while horizontal \( (H) \) and moment \( (M) \)
loading are not significant. Chapters 2, 3 and 4 investigated the process of spudcan penetration in clay and silty clay, showing the effects of a period of consolidation on further penetration. Once in operating conditions, the structure is subjected to a combination of vertical, horizontal and moment loading \((VHM)\), which arise from the self-weight of the structure as well as metocean actions, such as wind, waves and currents. Chapter 5 investigated the soil-spudcan interaction in the three-dimensional \(VHM\) loading space and showed the effects of consolidation on the combined response of the footing.

The nature of metocean actions is typically cyclical, especially when considering loading generated by waves. When considering the loading frequency for geotechnical purposes, an average period of around 10 s can be normally assumed for waves in water depths relevant to jack-ups. Previous studies on carbonate sandy silts and silty sands have shown transitional drainage conditions during jack-up installation (Erbrich 2005). Due to the cyclic loading rate though, which is higher than the spudcan installation rate, undrained conditions can be assumed within the application of a single cycle (Amodio et al. 2015a,b). The permeability characteristics of the silty clay studied in this work (lower coefficient of consolidation than silty soils presented in Amodio et al. 2015a,b) are such that undrained conditions can be expected when cyclic loading is applied. However, when the applied mean vertical load is compressive, some degree of consolidation and consequent excess pore pressure dissipation can be expected over an increasing number of cycles (Byrne and Cassidy 2002). In light of this, the considerations and the findings arising from previous chapters must be integrated with an investigation of the soil-spudcan behaviour under cyclic loading, in order to assess the reliability of the consolidated zone. Interest lies in minimum additional settlement following consolidation, due to stiffening and strengthening of the soil. Risk of rapid uncontrolled leg penetration on the other hand would render the increased capacity unreliable.

The study presented in this chapter aims to provide a glimpse of the response to vertical cyclic actions of spudcan installed in silty clay, in terms of generated settlement and variation of excess pore pressures. In this sense, a numerical approach to the problem offers the necessary flexibility to estimate the effects of different loading paths on the soil-spudcan response. The effects of installation and consolidation on the cyclic behaviour are taken into account through a rigorous mapping of the field variables and soil properties from previous penetration analyses (Chapter 4). Only a general introduction to the problem is offered in this chapter; future research will focus on expanding the breadth of analysis of the vertical case as well as assessing the three-dimensional cyclic behaviour.
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6.3 Hypoplastic model with intergranular strain

The reference hypoplastic model is well suited to describe large strain problems, where rearrangements of the grain skeleton are involved. However, when modelling problems involving frequent strain path reversal such as under cyclic loading, a model capable of describing the soil response at very small strains is required. An excessive strain accumulation, known as ‘ratcheting’, is usually observed when this behaviour is not properly modelled.

A solution to the problem was proposed by Niemunis and Herle (1997) with the introduction of the intergranular strain concept within the frame of hypoplasticity for cohesionless soils. The theory was later applied to clays by Mašín (2014), incorporating the intergranular strain concept in the original model from Mašín (2013). In this way, an anisotropic form of the stiffness tensor allows the correct prediction of the behaviour at very small strains. The model presented in Mašín (2014), in combination with the hypoplastic model for structured clays (Mašín 2007) used in Chapters 4 and 5 was here adopted to investigate the cyclic behaviour of spudcan footings in silty clay.

The intergranular strain concept involves an increasingly non-linear stiffness of the soil even at very small strains $\varepsilon$ (with the exception of an extremely limited initial elastic range), formulated in the strain space rather than stress space. This is achieved by describing not only the skeleton particles rearrangement (as in basic hypoplasticity) but also the deformation of the interface layer between the grains. The very small strain stiffness $G$ mainly depends on the deformation history: provided a deformation $\varepsilon$ is constantly applied in the same direction $D_\varepsilon$, the intergranular strain $\delta$ will ultimately align with $D_\varepsilon$ and provide the lowest stiffness value ($G_M$, Figures 6.1a and 6.2); the maximum stiffness is instead achieved for a complete strain path reversal, i.e. when $D_\varepsilon$ is rotated 180° from $\delta$ ($G_0$, Figures 6.1b and 6.2); intermediate stiffness conditions arise from a 90° rotation ($G_T$, Figures 6.1c and 6.2). The asterisks in Figure 6.1 denote the fact that different stiffness values can arise from identical deformation states.

Figure 6.2 schematically shows the gradual stiffness decrease from an initial value $G_M$ for monotonic deformation (Figure 6.1a, its value is calculated from basic hypoplastic model equations); after a 90° reversal of the strain path the stiffness increases to a value of $G_T$ whereas a 180° reversal causes the stiffness to increase to $G_0$. In clay hypoplasticity, the initial stiffness $G_0$ can be expressed as a function of the mean stress $p$ (Mašín 2014; Wroth and Houlsby 1985):

$$G_0 = p_r A_g \left( \frac{p}{p_r} \right)^{n_g}$$

(6.1)

where $p_r$ is a reference pressure of 1 kPa; $A_g$ and $n_g$ are model parameters. With the
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Figure 6.1: Different deformation paths in the strains plane related to different intergranular strain directions (after Niemunis and Herle 1997)

Figure 6.2: Stiffness degradation with strain accumulation along the same direction (after Niemunis and Herle 1997)
 exception of an initial elastic range $R$, the stiffness degradation for increasing $\varepsilon$ is controlled by two parameters $\beta_r$ and $\chi_g$.

When the swept-out-memory threshold $\varepsilon_{som}$ is reached through a prolonged deformation along the same direction, the less recent load history is lost and the stiffness values converge to a final common value, which matches the basic hypoplastic model stiffness (Figure 6.2).

The implementation of the intergranular strain concept in a finite element code requires the expansion of the reference hypoplastic model for structured clays presented in Chapters 4 and 5. In particular, an extra state variable, the intergranular strain $\delta$, is introduced to describe the deformation of the interface layer. The model also requires the following six extra parameters, summarised in Table 6.1: $A_g$ and $n_g$ to define the initial stiffness $G_0$; $R$ to quantify the initial elastic range; $\beta_r$ and $\chi_g$ to control the stiffness degradation; $m_{rat}$ to define the ratio $G_T/G_0$.

The calibration of the intergranular strain parameters for the silty clay that is the object of this study is illustrated in the following section; for an exhaustive description of the model the reader is instead referred to Mašín (2014) and Niemunis and Herle (1997).

<table>
<thead>
<tr>
<th>Intergranular Strain Parameters</th>
<th>$A_g$</th>
<th>$n_g$</th>
<th>$R$</th>
<th>$\beta_r$</th>
<th>$\chi_g$</th>
<th>$m_{rat}$</th>
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<td>Control on Initial stiffness</td>
<td>6500</td>
<td>0.5</td>
<td>$10^{-4}$</td>
<td>0.27</td>
<td>3</td>
<td>0.7</td>
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<td>Control on Elastic radius</td>
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<tr>
<td>Control on stiffness degradation</td>
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<td></td>
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</tr>
<tr>
<td>Control on stiffness degradation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Control on Ratio $G_T/G_0$</td>
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<td></td>
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</tr>
</tbody>
</table>

Table 6.1: Intergranular strain parameters for carbonate silty clay

### 6.3.1 Intergranular strain calibration

The soil modelled in the analyses is a carbonate silty clay recovered from the Laminaria field, on the North-West Shelf of Australia. The same soil was already modelled with hypoplasticity and used for the numerical simulations presented in Chapters 4 and 5. Chapter 4 details the calibration of the parameters for the reference hypoplastic model.

The calibration of the intergranular strain parameters requires several laboratory tests to be carried out. The initial (or upon $180^\circ$ strain path reversal) stiffness $G_0$ can be measured by means of bender element measurements. By plotting the results in terms of stiffness as a function of the mean stress $p$ (Figure 6.3), the parameters $A_g$ and $n_g$ can be calibrated according to Equation 6.1.
$m_{rat}$ identifies the ratio $G_T / G_0$ and is usually difficult to calibrate as $G_T$ cannot be measured with bender element tests. The implementation of local strain transducers to accurately measure the local strains represents a complex option to measure $m_{rat}$.

The remaining parameters $R$, $\beta_r$ and $\chi_g$ can be calibrated by means of parametric studies, fitting the stiffness degradation curve obtained from local strain measurements.

The calibration procedure as described above suits problems with constant strain direction or several strain path reversals. However, when the hypoplastic model with intergranular strain is implemented to describe the cyclic behaviour of clays, the calibration is preferably underpinned by cyclic loading tests. For this reason, cyclic triaxial tests carried out at the University of Western Australia (UWA) soil laboratory (Fahey and Bhattarai 1997) were used to calibrate the model.

Bender element tests carried out on samples consolidated anisotropically (ratio of horizontal over vertical stress $K_0 = 0.4$) revealed a small strain stiffness $G_0$ of 92 MPa and 88 MPa at $p = 75$ kPa and 102 kPa respectively (Fahey and Bhattarai 1997, Figure 6.3). Bender element tests carried out on the triaxial samples presented in Chapter 4, which served for calibration of the reference hypoplastic model, were also used: values of $G_0 = 32.5; 39.2; 104.2$ MPa emerged for $p = 60; 114; 210$ kPa (Figure 6.3). It should be noticed how the complete soil remoulding involved in the preparation of the samples presented in Chapter 4 may be responsible for the lower $G_0$, when compared to the less disturbed samples tested by Fahey and Bhattarai (1997). Figure 6.3 shows the
interpolation of bender element tests results according to Equation 6.1, which provided \( A_g = 6500 \) and \( n_g = 0.5 \).

The cyclic triaxial tests were carried out on undisturbed push-tube samples of 72 mm nominal diameter (Fahey and Bhattarai 1997). The samples were consolidated in two consecutive stages to avoid failure during consolidation: a stage of isotropic stresses was applied first, followed by anisotropic stress conditions \( K_0 = 0.4 \) (\( \sigma'_h = 82 \text{kPa} \); \( \sigma'_v = 180 \text{kPa} \); mean effective stress \( p = 108 \text{kPa} \) and deviator stress \( q = 108 \text{kPa} \)). The cyclic loading was applied under undrained conditions: in the first test (LMTX02) a cyclic deviator stress \( \Delta q = 24 \text{kPa} \) was applied at a mean deviator \( q_{\text{mean}} = 108 \text{kPa} \), in the second test (LMTX03) a \( \Delta q = 40 \text{kPa} \) was applied at \( q_{\text{mean}} = 108 \text{kPa} \). In both cases the cyclic loading was applied at a frequency \( f = 0.1 \text{Hz} \) until failure of the sample was reached.

A parametric study to determine \( \beta_r \) and \( \chi_g \) was carried out simulating numerical analyses of triaxial tests LMTX02 and LMTX03 in the finite element code Abaqus/Standard. Given the complexity of the calibration procedure and the limited amount of experimental data available, \( R = 10^{-4} \) and \( m_{\text{rat}} = 0.7 \) were assumed as material independent constants. Figures 6.4 and 6.5 compare the experimental (black) and numerical (green) results: the numerical simulations started from the end of anisotropic consolidation, i.e. immediately before cyclic loading. The attention of the parametric study mainly focused on the correct estimation of axial strain \( \varepsilon_a \) (Figures 6.4c and 6.5c) and excess pore pressure \( \Delta u \) accumulation (Figures 6.4d and 6.5d). For the former, in particular, it should be noticed that the assumption of \( \beta_r = 0.27 \) and \( \chi_g = 3 \) allowed the correct estimation of \( \varepsilon_a \) for both small and large cyclic amplitude (\( \Delta q = 24 \text{kPa} \) for LMTX02 and \( \Delta q = 40 \text{kPa} \) for LMTX03). Nonetheless, Figures 6.4c and 6.5c show how the calibration only concentrated on the initial quasi-linear strain accumulation, as it was beyond the scope of this study to capture the formation of shear bands and consequent sample failure.

The set of parameters for the reference hypoplastic model adopted for the numerical analyses of triaxial tests (Table 6.2) remained unchanged from that assumed in Chapter 4, with the exception of the over-consolidation ratio \( OCR \). The hypoplastic model does not allow \( p - q \) states to fall outside the State Boundary Surface (SBS). For this reason, the size of the SBS was artificially increased to include the excursions of deviator stress from the available tests. The experimental triaxial LMTX03 presented a peak in deviator stress \( q_{\text{max}} = 148 \text{kPa} \), which exceeded the failure deviator \( q_f = 121 \text{kPa} \) predicted by the model for the given set of parameters and \( p \); \( OCR \) was increased from 1.205 (Chapter 4) to 1.6 in order to contain such \( q_{\text{max}} \). The assumption of \( OCR = 1.6 \) was extended to LMTX02 for homogeneity in the analyses. Despite the increase in the size of the SBS,
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Figure 6.4: Results from triaxial test LMTX02 in terms of (a) $q$ - Cycle Number; (b) $p$ - Cycle Number; (c) $\varepsilon_a$ (%) - Cycle Number; (d) $\Delta u/\sigma'_c$ - Cycle Number; (e) $q - p$; (f) $q - \varepsilon_a$ (%)

$\sigma'_c$: consolidation stress
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Figure 6.5: Results from triaxial test LMTX03 in terms of (a) $q$ - Cycle Number; (b) $p$ - Cycle Number; (c) $\varepsilon_a$ (%) - Cycle Number; (d) $\Delta u/\sigma_c'$ Cycle Number; (e) $q - p$; (f) $q - \varepsilon_a$ (%)

$\sigma_c'$: consolidation stress
the parameters obtained through calibration are still thought to be indicative of the stiffness degradation rate.

### Basic Model Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_c$</td>
<td>CS NCL for frictions $p' = 1$ kPa slope</td>
</tr>
<tr>
<td>$N$</td>
<td>NCL Unloading reload slope</td>
</tr>
<tr>
<td>$\lambda^*$</td>
<td>Unloading reloading slope</td>
</tr>
<tr>
<td>$\kappa^*$</td>
<td>Control on shear modulus</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Control on shear modulus</td>
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</table>

### Sensitivity Parameters

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<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
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<td>$s_{ini}$</td>
<td>Initial sensitivity</td>
</tr>
<tr>
<td>$k$</td>
<td>Sensitivity degradation rate</td>
</tr>
<tr>
<td>$A$</td>
<td>Vol/shear Strains effect</td>
</tr>
<tr>
<td>$s_f$</td>
<td>Final sensitivity</td>
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</tbody>
</table>

### Consolidation Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_0$</td>
<td>Earth CSL Pressure position coeff. at rest</td>
</tr>
<tr>
<td>$O_c$</td>
<td>CSL Over Consolidation Ratio</td>
</tr>
<tr>
<td>OCR</td>
<td>1.6 (calibration)</td>
</tr>
</tbody>
</table>

Table 6.2: Hypoplastic model parameters for carbonate silty clay

Table 6.1 summarises all the parameters derived from the calibration of the intergranular strain, which were adopted for the numerical simulations of spudcan cyclic loading presented in the following section.

### 6.4 Spudcan response to vertical cyclic loading

#### 6.4.1 Experimental evidences on kaolin clay

A series of experimental vertical cyclic loading tests on kaolin clay was carried out in a beam centrifuge at UWA, at enhanced gravity 100 g. The experiments were carried out on kaolin clay, rather than the carbonate silty clay, due to insufficient soil sample
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volume. However, previous chapters demonstrated that kaolin and silty clay presented a qualitatively comparable behaviour in terms of vertical penetration and multi-directional response. The results obtained for kaolin clay can be thus still expected to be indicative of the silty clay behaviour.

The soil sample was consolidated in flight for four days to achieve normally consolidated conditions. T-bar penetration measurements indicated an undrained shear strength profile of 0.9 kPa/m, adopting a T-bar factor $N_{T-bar} = 10.5$ (Randolph et al. 1998). As will be discussed in Section 6.3.3, this profile is lower than what usually found in previous studies using UWA kaolin clay (Bienen and Cassidy 2013; Colreavy et al. 2016). The spudcan model diameter $D$ was 60 mm.

The tests involved an initial penetration to $w/D = 1$ at a rate of 0.2 mm/s in order to guarantee undrained conditions (Cassidy 2012; Chung et al. 2006; Finnie 1993). Once the target depth of $w/D = 1$ was reached, a normalised period of consolidation $T = c_v t / D^2 = 0.05$ was allowed to take place (coefficient of consolidation $c_v = 3.5 \text{m}^2/\text{year}$ at $w/D = 1$ after Bienen and Cassidy 2013; $t$ is the dimensional time), where the entire load previously achieved in penetration was held constant. A reference test further penetrated the spudcan to $w/D = 2$ following consolidation, so to capture the peak in vertical bearing capacity due to consolidation $V_0^*$. For the remaining tests, a vertical cyclic loading path was applied to the spudcan following the consolidation stage at a frequency of 0.5 Hz, with a target of 50 cycles. The loading paths involved different mean vertical loads $V_{mean}$ and cyclic amplitudes $\Delta V$, expressed as a function of the increased bearing capacity $V_0^*$. The paths are summarised in Table 6.3.

<table>
<thead>
<tr>
<th>$T$</th>
<th>$V_{mean}$</th>
<th>$\Delta V$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>$0.4V_0^*$</td>
<td>$0.05V_0^*$</td>
</tr>
<tr>
<td></td>
<td>$0.5V_0^*$</td>
<td>$0.25V_0^*$</td>
</tr>
<tr>
<td></td>
<td>$0.7V_0^*$</td>
<td>$0.4V_0^*$</td>
</tr>
<tr>
<td></td>
<td>$0.75V_0^*$</td>
<td>$0.15V_0^*$</td>
</tr>
</tbody>
</table>

Table 6.3: Summary of experimental cyclic tests carried out on kaolin clay

Figure 6.6 reports the results from the five tests in terms of normalised accumulated settlement $\Delta w/D$ during cyclic loading. Despite the limited set of data, the results are nonetheless indicative of the amount of vertical load required to trigger any significant settlement. In particular, it can be noticed that for $V_{mean} = 0.4V_0^*$ and $\Delta V = 0.05V_0^*$ (i.e. with a peak of $V = 0.45V_0^*$) no relevant settlements were observed; in fact, the footing was slightly uplifted by 0.001 $D$. Similar response was observed for $V_{mean} = 0.5V_0^*$ and
Figure 6.6: Spudcan settlements accumulated during experimental cyclic loading in kaolin clay

\[ \Delta V = 0.25 V_0^* \text{ (peak of } V = 0.75 V_0^*) \], with minimal settlement of \( \Delta w/D = 0.012 \). An increasingly higher accumulation of settlements was observed in the remaining cases, where \( V_{mean} \) was increased to 0.75 \( V_0^* \) and \( \Delta V \) to 0.4 \( V_0^* \), with peaks in vertical load of 0.9 \( V_0^* \).

The cyclic response is thought to be governed by mean load and cyclic amplitude, rather than the peak values in vertical load (Andersen 2009). From such results, however, it can be concluded that the loading paths must be pushed incrementally close to \( V_0^* \), i.e. representative of extremely harsh conditions in a field scenario, in order to observe any significant settlement. A similar behaviour was observed by Kohan et al. (2016) when studying the effects of cyclic loading on the extraction of an embedded spudcan in kaolin clay; a peak load (intended as a sum of \( V_{mean} \) and \( \Delta V \)) of 0.45 of the monotonic tensile capacity failed to extract the spudcan, which was instead achieved by increasing such peak to 0.7 of the tensile capacity. The former case also showed reducing accumulation of displacement with number of cycles, due to excess pore pressure dissipation and consequent consolidation.

The large amount of \( \Delta w/D \) accumulated in the relatively small number of cycles shown in Figure 6.6 could potentially represent a risk for the structure, in contrast to lower \( V \)
levels which appeared to produce nearly irrelevant amount of settlement. For this reason, the numerical study on carbonate silty clay presented in the following sections focused on cyclic paths sitting in proximity of the vertical bearing capacity (Figure 6.7).

### 6.4.2 Numerical model

The effects of installation and consolidation prior to cyclic loading were taken into account by mapping the results of Large Deformation Finite Element (LDFE) analyses presented in Chapter 4. Due to the symmetry of the problem and the direction of the load applied, the cyclic simulations were performed as two-dimensional axisymmetric analyses, with a soil domain size of \(20D\) (\(D\) is the spudcan diameter). The analyses were carried out within the framework of Remeshing and Interpolation Technique with Small Strain (RITSS, Hu and Randolph 1998) strategy adopted in Chapter 4; however, due to the limited amount of strains accumulated during cyclic loading, the RITSS strategy was only required to account for the effects of installation and consolidation, whereas the entire cyclic stage was simulated without remeshing. A mesh density comparable to Chapter 4 was assumed; an average of 2100 eight-node, with biquadratic displacement, bilinear pore pressure and reduced integration elements (termed CAX8RP in Dassault Systèmes 2012) was used to discretise the soil. The contact interface soil spudcan was modelled as frictionless.

The hypoplastic model parameter \(p_t\), which is responsible for shifting the mean effective stress due to cohesion, was increased from \(10^{-5}\) kPa to 5 kPa and 10 kPa for tests with
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and without consolidation respectively, leading to a 9.6% and 6.8% reduction in the accumulated settlements when compared to \( p_t = 10^{-5} \). This change was required for numerical stability of the analyses and its assumption is explained in the following section. The remaining hypoplastic model parameters (summarised in Table 6.2) and the characteristics of the numerical model are described in Chapter 4.

Different loading paths have been applied to the spudcan foundation, accounting for either no consolidation \((T = 0)\) or a period of consolidation \(T = 0.01; 0.05\) \((c_v = 1.58 \text{ m}^2/\text{year} \text{ for carbonate silty clay at } w/D = 1)\). Figure 6.7 schematically illustrates the cyclic paths in terms of \( V_{\text{mean}} \) and \( \Delta V \), with reference to the vertical bearing capacity \( V_0 \) and \( V_0^* \), for \( T = 0 \) and \( T = 0.05 \). The vertical cyclic loading was applied around \( V_{\text{mean}} = 0.8; 1.0V_0(V_0^*) \) with amplitudes \( \Delta V = 0.1; 0.2V_0(V_0^*) \) (see Table 6.4). All loading paths were applied at a frequency \( f = 0.1 \text{ Hz} \), which is thought to be a close approximation of the frequency of wave action.

<table>
<thead>
<tr>
<th>( T )</th>
<th>( V_{\text{mean}} )</th>
<th>( \Delta V )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>( 0.8V_0 )</td>
<td>( 0.1V_0 )</td>
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<td></td>
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</tr>
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<td>( 0.2V_0^* )</td>
</tr>
<tr>
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<td>( 0.1V_0^* )</td>
</tr>
<tr>
<td></td>
<td>( 0.2V_0^* )</td>
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</tr>
<tr>
<td></td>
<td>( 1.0V_0^* )</td>
<td>( 0.1V_0^* )</td>
</tr>
<tr>
<td></td>
<td>( 0.2V_0^* )</td>
<td></td>
</tr>
</tbody>
</table>

Table 6.4: Summary of numerical cyclic tests carried out on carbonate silty clay

### 6.4.3 Discussion of the numerical results

Figure 6.8 reports the accumulation of normalised spudcan settlement \( \Delta w/D \), as a function of the number of cycles, for \( T = 0.05 \). \( V_{\text{mean}} \) appears to be responsible for the amount of settlement accumulated in the first part of the loading stage (roughly the first 30 cycles). The assumption of \( V_{\text{mean}} = 1.0V_0^* \) caused the vertical loading to be close to, or even exceed, the maximum size of the expanded yield surface defined by \( V_0^* \). This had the effect of producing a more plastic response than \( V_{\text{mean}} = 0.8V_0^* \), where the vertical loading was instead confined within the yield surface and a quasi-elastic response can be
Chapter 6. An introduction to the cyclic loading response of spudcan foundations in carbonate silty clay

thus expected (hypoplasticity and intergranular strain theory do not contemplate purely elastic behaviour, Mašín 2014). As a result, the $V_{mean} = 1.0V_0^*$ cases started to accumulate settlement from the very first cycles, whereas the reduction of the vertical load to $V_{mean} = 0.8V_0^*$ resulted in an initial average uplift of the footing $\Delta w/D = -0.0001$. The long-term trend (after 30 cycles) in terms of rate of settlement accumulation is instead governed by $\Delta V$, where cases with different $V_{mean}$ were observed to accumulate settlement at the same rate. As expected, the cyclic component of the loading is also responsible for the amplitude of the oscillations in $\Delta w/D$. It is clear from Figure 6.8 that higher values of $V_{mean}$ and $\Delta V$ contribute to a larger settlement accumulation.

The observations regarding Figure 6.8 were underpinned by the normalised excess pore pressure distribution $\Delta u/q_{cons}$ ($q_{cons}$ is the pressure held during consolidation stage) below the spudcan tip. Results for the analyses involving $T = 0.05$ are illustrated in Figure 6.9. The initial $\Delta u/q_{cons} = 0.236$ was eventually dissipated to a mean value close to zero ($\Delta u/q_{cons} = 0.025$ for $\Delta V = 0.1V_0^*$ and $\Delta u/q_{cons} = 0.05$ for $\Delta V = 0.2V_0^*$). Differently from the cases involving $V_{mean} = 0.8V_0^*$, an initial peak in $\Delta u/q_{cons}$ was
observed when $V_{\text{mean}} = 1.0V_0^*$ was applied, which is responsible for the larger settlement accumulated in the early stage of the analyses. Overall, the high soil stiffness due to a period of consolidation and the limited additional plastic penetration, in combination with the low initial $\Delta u/q_{\text{cons}} = 0.236$, allowed the excess pore pressures to be completely dissipated in all the cases.

The response for the cases without consolidation ($T = 0$) is shown in Figure 6.10. The case with $V_{\text{mean}} = 0.8V_0$ and $\Delta V = 0.2V_0$ showed a final $\Delta w/D = 0.0014$ (Figure 6.10a). This was supported by the evolution of excess pore pressures shown in Figure 6.10b, where the initial value of $\Delta u/q_{\text{cons}} = 1$ was nearly completely dissipated ($\Delta u/q_{\text{cons}} = 0.02$) after roughly 40 cycles. However, for the remaining cases the model was observed to continually increase or maintain the excess pore pressures (Figure 6.10b), rather than dissipate them. In turn, large settlement accumulations were predicted by the model, as reported in Figure 6.10a. This phenomenon of rapid accumulation of excess pore pressures is not sustainable for the numerical model and it leads to non-convergence of the analyses. The causes behind this instability can be found first of all in the elevated initial excess pore pressures ($\Delta u/q_{\text{cons}} = 1$), which contributed to the reduction of the effective stresses to extremely low values. As a confirmation, the problem was less pronounced when simulating the same loading paths for $T = 0.05$, 
where instability occurred only for the cases with larger $\Delta V$ and in the late stage of analysis. The assumption of $p_t = 10\, \text{kPa}$ (larger than the consolidated case $p_t = 5\, \text{kPa}$) was thus necessary to provide some degree of stability to the analyses. Also, increased plastic settlement might have been caused by soil stiffness lower than the consolidated case, leading to further development of excess pore pressures. Finally, the implementation of the intergranular strain can lead to rapid and dramatic excursions in stresses even for very small strains, which can cause localised phenomena of temporary zero effective stress and consequent numerical instability. This final hypothesis, in particular, is validated by the $\Delta V = 0.1V_0$ case compared to $\Delta V = 0.2V_0$ (for $V_{\text{mean}} = 0.8V_0$), as the induced lower straining would cause a more frequent assumption of the highest stiffness value $G_0$ (Figures 6.1 and 6.2).

Nonetheless, the results obtained from $T = 0$ are still indicative of the larger amount of settlement that can be accumulated in the early stage of cyclic loading, when $\Delta u/q_{\text{cons}}$ is still close to one or, in general, when elevated excess pore pressure distributions occur.

In an effort to quantify the effects of consolidation on the accumulated settlement and assess the reliability of the increased bearing capacity, the results in terms of $\Delta w/D$ for three different periods of consolidation $T = 0; 0.01; 0.05$ were compared in Figure 6.11; a $V_{\text{mean}} = 0.8V_0 (V_0^*)$ and $\Delta V = 0.2V_0 (V_0^*)$ were applied. The curves show a reduction in the accumulated $\Delta w/D$ for increasing $T$, with a reduction of 57% from $T = 0$ to $T = 0.05$ after 70 cycles. It should be kept in mind that $p_t (T = 0) = 10\, \text{kPa}$ leads to a larger overestimation of the settlement than $p_t (T = 0.05) = 5\, \text{kPa}$, so that such a gap could be even larger. The reason for reduced settlements can be understood analysing the consequences of a period of consolidation; this has the effect of increasing the mean effective stress in the soil, by means of $\Delta u$ dissipation. The SBS defined in the $q-p$ plane also results expanded, potentially with increased ratios $p/p_{\text{cr}}$ (being $p_{\text{cr}}$ the critical mean effective stress). Although $V_{\text{mean}}$ and $\Delta V$ are also scaled up according to $T$, i.e. different mean and cyclic amplitude of deviator stress are applied for different $T$, it can be imagined that the critical state is reached less rapidly by the soil experiencing consolidation.

This is particularly relevant, as it indicates that consolidation allows the spudcan to withstand larger vertical loads (i.e. harsher storm conditions) in return for a reduced amount of settlement, when compared to the case without consolidation. From this preliminary investigation, the increased capacity due to consolidation appears, thus, to be reliable under cyclic loading, for no rapid spudcan settlements were observed in the analyses.

It should be noticed that the order of magnitude of settlements accumulated in numerical simulations appears small and is consistently smaller than that observed in the
Figure 6.10: (a) Normalised spudcan settlements accumulation and (b) normalised excess pore pressure variation during cyclic loading for $T = 0$
Figure 6.11: Spudcan settlements accumulated during cyclic loading $V_{\text{mean}} = 0.8 V_0 (V_0^*)$, $\Delta V = 0.2 V_0 (V_0^*)$ for increasing $T = 0; 0.01; 0.05$
experiments on kaolin clay. Explanations for this gap, however, can be found in the fact that two different soils were used and different loading paths were applied. Further, the undrained shear strength of the kaolin clay sample of 0.9 kPa/m flagged the possibility that the sample tested was not fully consolidated (see the higher 1.1 kPa/m in Bienen and Cassidy 2013, ~ 1.17 kPa/m in Colreavy et al. 2016, 1.06 kPa/m in Kohan et al. 2016). Due to limitations imposed by the apparatus, the sample was tested at 100 g, rather than the usual 200 g; this may have been responsible for a slower consolidation process, a lower final stiffness and consequently larger observed settlements. In this sense, Byrne and Cassidy (2002) provided further experimental evidences on spudcan vertical cyclic loading in normally consolidated kaolin clay (with undrained shear strength profile of 1.25 kPa/m), which showed a reduced amount of settlement for comparable tests. For instance, when Byrne and Cassidy (2002) simulated a large number of cycles with $V_{\text{mean}} = 0.5 V_0$ and increasing cyclic amplitude up to $\Delta V = 0.5 V_0$, spudcan settlement of $\Delta w/D \sim 0.12$ was recorded, roughly 80% lower than $\Delta w/D = 0.75$ shown in Figure 6.6 for $V_{\text{mean}} = 0.5 V_0^*$ and $\Delta V = 0.4 V_0^*$. On the other hand, smaller numerical settlements were also predicted during the consolidation phase compared with centrifuge experiments in the same soil, with the causes of this difference not fully understood (Chapter 4). Finally, the calibration of the intergranular strain was based on a limited number of experimental tests and the large loading amplitudes applied to the samples introduced a degree of uncertainties over the determination of the parameters.

6.5 Further research on cyclic loading required

The opportunities for expanding the investigation of spudcan response to cyclic loading following a period of consolidation are significant. In particular, a more accurate calibration of the intergranular strain is recommended to be carried out, with retrospective analyses of laboratory tests to more robustly determine the values of the parameters, e.g. cyclic triaxial tests with loading amplitude not exceeding $q_f$ predicted by the model, such that the tests can be accurately reproduced numerically. Cyclic centrifuge tests for spudcan on carbonate silty clay are also recommended to assist the numerical model calibration, through direct validation of the numerical model. With regard to vertical cyclic loading, future research should focus more deeply on the influence of $V_{\text{mean}}$ and $\Delta V$. It was here demonstrated experimentally and numerically how different loading paths led to changes in the distribution of the excess pore pressures and consequently on the accumulated spudcan settlement. For this reason, a more detailed numerical parametric study is required to determine with accuracy the effects of
Chapter 6. An introduction to the cyclic loading response of spudcan foundations in carbonate silty clay

$V_{\text{mean}}$ and $\Delta V$, as well as the influence of different periods of consolidation. In particular, attention should be drawn to loading scenarios more realistic and relevant for practical applications, where $V$ would ideally reduce to operational values, i.e. $\sim 0.5 V_0 (V_0^*)$ and $\Delta V$ from 0.05 to 0.1 $V_0 (V_0^*)$. Ultimately, a framework similar to that defined by Andersen (2009) can be proposed, to relate the mean loading $V_{\text{mean}}$ and cyclic amplitude $\Delta V$ to the number of cycles required to fail the footing, provided the definition of an appropriate failure settlement.

Finally, the attention should be directed towards the three-dimensional case, with a combination of cyclic $VHM$ loading applied to the footing. As per the vertical case, a proper investigation of the problem will involve the application of different loading amplitudes at different $V$ levels and, in this case, different ratios $H : M / D$. The idea is to investigate the effects of cyclic loading on the accumulated displacements, when the loading is applied within, close to, or beyond the current yield surface. The reliability of the expanded yield surface will be assessed by taking into account different periods of consolidation.

### 6.6 Concluding remarks

The results presented in this chapter are preliminary and aimed at offering a glimpse of the soil-spudcan response to vertical cyclic loading in carbonate silty clay. The analyses relied on the capabilities of the hypoplastic numerical model with the intergranular strain concept to describe the problem.

Experimental observations from tests on kaolin clay in a centrifuge environment indicated the need to apply cyclic vertical loading close to the vertical bearing capacity of the footing, in order to observe any significant settlement.

When accounting for consolidation, the accumulated spudcan settlement determined numerically was demonstrated to be a function of both $V_{\text{mean}}$ and $\Delta V$. The former in particular, controlled the initial stage of the cyclic loading, whereas the latter governed the long-term accumulation as well as the cyclic amplitude of the settlement. The excess pore pressures were shown to be dissipated within a limited number of cycles, leading to a following reduction of the settlement accumulation rate. On the other hand, the application of cyclic loading without a period of consolidation led in most of the cases to continuous increase of excess pore pressures, dramatic reduction of the effective stresses and consequent instability of the numerical model.

Overall, a period of consolidation was demonstrated to offer a reliable increase in the bearing capacity of the footing, as reduced settlements were observed when applying the same loading paths for cases with increasing periods of consolidation.
Current shortcomings of the numerical model were finally highlighted, particularly in terms of difficulty to deal with extremely low effective stress, along with recommendations for future investigation of the cyclic response.
References


Chapter 7

Concluding remarks

7.1 Summary

The thesis investigated the effects of consolidation on the response of spudcan foundations installed in intermediate soils, more specifically, in a carbonate silty clay. Experimental and numerical methods have been used to reveal the following different aspects of the process.

Firstly, the change in failure mechanism from continuous spudcan penetration to a case involving a period of consolidation was highlighted. For completeness, results obtained from kaolin clay – a widely investigated soil for similar studies – were presented first, followed by a comprehensive study on carbonate silty clay, which represents the main objective of this thesis.

Secondly, the study focused on the connection between consolidation and several factors relevant to the spudcan installation process, such as the increase in undrained shear strength in the soil around the footing, the peak observed in vertical load upon re-penetration and the response post-peak.

Thirdly, the study moved from the purely vertical to the multi-directional case, in order to explore the spudcan combined bearing capacity. Numerical simulations, validated by experimental evidences, demonstrated the increase in combined bearing capacity due to consolidation. Different distribution of the excess pore pressures with consolidation and changes in the failure mechanisms were demonstrated to govern the problem.

Finally, an introduction to the reliability of the increased yield surface under vertical cyclic loading was presented. The implementation and calibration of the intergranular strain concept for numerical analyses was presented first, before conducting a parametric study that showed the influence of mean vertical load and cyclic amplitude on accumulated settlement and excess pore pressure variation.
7.2 Main findings

The following goals were achieved in this thesis, in line with the aims set out in Chapter 1:

1. Predictive framework for spudcan penetration:

The thesis offers a predictive framework for spudcan penetration, providing a set of information to guide potential users through a safe spudcan installation. The framework offers assistance in terms of:

- Vertical bearing capacity in continuous penetration;
- Settlements occurring during consolidation;
- Increase in vertical bearing capacity following consolidation;
- Behaviour post-consolidation upon further penetration;
- Extent of the improved zone.

The effects of consolidation were studied for installation in kaolin clay and, for the first time in a consolidation-related spudcan study, in an intermediate soil such as the carbonate silty clay.

2. Implementation of an advanced constitutive model:

An advanced hypoplastic constitutive model for structured clays was successfully implemented in a large deformation numerical analysis software. For the first time, such an advanced model was adopted to tackle this kind of boundary value problems and it clearly represents a step forward from traditional constitutive models usually adopted, such as Tresca or Modified Cam Clay.

The model revealed the effects of soil sensitivity on the spudcan bearing capacity following consolidation. High sensitivities associated with fast sensitivity degradation rates led to potential punch-through scenarios, with drastic reductions in bearing capacity post-peak, whereas lower degradation rates offered a continuous bearing capacity increase after consolidation. Coupled effective stress-pore fluid analyses allowed the quantification of increase in undrained shear strength due to a period of consolidation. Finally, the implementation of such an advanced constitutive model showed asymmetric failure mechanisms for spudcan subjected to multi-directional loading paths, differently from previous studies were simplified constitutive models were adopted.
3. Increase in combined bearing capacity:

A detailed relationship between length of consolidation and increase in combined bearing capacity was proposed, with the reasons behind such increases explained.

Recommendations for the use of ISO (2012) in the installation and operation of spudcan in silty soils were proposed, with limitations of the current guidelines outlined.

4. Cyclic reliability:

The implementation of the hypoplastic model enhanced with the intergranular strain concept was presented. Consolidation was demonstrated to reduce the accumulated spudcan settlement during cyclic loading and thus to provide a reliable increase in capacity. Mean vertical loading and cyclic amplitude were correlated to the amount of accumulated settlement and to the variation of excess pore pressures in the soil.

Although it has already served a preliminary investigation of the soil-spudcan response, the calibration of the model set out the conditions for future research, in terms of numerical investigation of the vertical and multi-directional cyclic behaviour.

7.3 Recommendations for future research

Although the above mentioned achievements represent a significant step forward towards a complete understanding of the phenomenon, there are other areas of the problem that could not be investigated within this work, but require further research. Particular attention should be dedicated to:

1. The reliability over time of the increased bearing capacity.

A quantification of how the expanded yield surface proposed in this thesis modifies in size or shape after the footing is subjected to cyclic loading would be of extreme interest for the jack-up industry. In this sense, Chapter 6 merely offered a glimpse of a future exhaustive investigation of the problem, where the soil-spudcan response to metocean actions, such as waves, currents and wind, should be thoroughly assessed.

It is recommended that a combination of numerical and experimental approaches be used. On one side, centrifuge testing would give an insight into the response of natural soil when the footing is subjected to small, multi-directional cyclic
loading. Small-amplitude cycles of vertical loading in kaolin clay were already presented in Chapter 6, revealing the effects of mean load and cyclic amplitude on the accumulated spudcan settlement. However, to date, the available apparatus at the University of Western Australia presents some limitations in terms of simultaneous application of three-dimensional loading directly to the spudcan. The difficulties rise from the necessity to apply loading paths (rather than displacements) directly to the spudcan, which requires an extremely sophisticated feedback system operating between the footing and the control panel. On the other side, the experimental data would serve the validation of the numerical model, which would provide a wider set of information, with different cyclic loading scenarios investigated. In this sense, a thorough parametric study is recommended, so to account for different cyclic loading scenarios, installation depths and load-hold periods;

2. Easing the calibration procedure for the numerical hypoplastic model introduced in Chapter 4.

Although the calibration is already underpinned by simple laboratory tests, such as oedometer and triaxial test, recovery of soil samples can be sometimes extremely difficult and expensive, especially when dealing with offshore natural soils. For this reason, future research could base the calibration procedure on in-situ tests, such as penetrometer tests. Potentially, one single penetrometer test would be able to provide information regarding (i) shear strength profile, (ii) soil drainage properties, (iii) soil sensitivity, (iv) sensitivity degradation rate, provided the right procedure and equipment are developed;

3. The application of the Remeshing and Interpolation Technique with Small Strain (RITSS) strategy – here adopted in Chapter 4 to simulate vertical penetration – to the investigation of the multi-directional VHM case.

Chapter 5 proved small strain three-dimensional analyses to be barely sufficient to provide the combined bearing capacity, for severe mesh distortion would be observed if larger displacement paths were simulated. The adoption of RITSS for three-dimensional analyses would drastically reduce such distortion, giving the user the freedom to explore larger displacement paths and possibly reach the tensile side of the yield surface. It should be mentioned that computational time would also increase dramatically, for RITSS analyses require rather small increments. However, with the ongoing improvement in computer performance, this issue could be easily overcome in the future;

4. Access to field data from in-situ monitoring campaign.
A combination of experimental and numerical strategies represents a valid option to minimise the unknown of the consolidation problem in spudcan performance. However, a long-term monitoring campaign of a real jack-up unit would guarantee further validation of the results, as well as the possibility to improve the predictions of the numerical model. In particular, data regarding metocean cyclic actions, spudcan accumulated settlement and excess pore pressure variation would be used to fine-tune the cyclic behaviour described in Chapter 6 and assess the reliability of the increased combined $VHM$ capacity under cyclic loading;

5. The introduction of a wave-soil-structure interaction software.

Ultimately, all the findings of the thesis could be combined in a software package where the increase in combined $VHM$ capacity is included in a reliability context. The most severe storm loading conditions (i.e. the highest storm return period) that the jack-up can withstand would be identified by means of a probabilistic study. The ultimate goal would be to provide a user-friendly interface, where only basic soil and structure parameters must be provided to carry out the analyses.
Chapter 7. Concluding remarks

References