THE UPLIFT OF OFFSHORE SHALLOW FOUNDATIONS

By

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THESIS FORMAT AND AUTHORSHIP

In accordance with The University of Western Australia’s regulations regarding Research Higher Degrees, this thesis is presented as a series of papers that have been published, submitted or completed for publication. The contribution of the candidate for the papers comprising Chapters 2 to 8 are hereby set forth.

Paper 1

This paper is presented in Chapter 2, first authored by the candidate, co-authored by Assoc./Prof. Yinghui Tian, Prof. Christophe Gaudin and Prof. Mark J. Cassidy, and published as


The candidate developed the finite element model in collaboration with Assoc./Prof. Yinghui Tian. The candidate prepared the results and wrote the draft paper. Further interpretation of the results and the paper writing were conducted by the candidate with contributions from Assoc./Prof. Yinghui Tian, Prof. Christophe Gaudin and Prof. Mark J. Cassidy.

Paper 2

This paper is presented in Chapter 3, first authored by the candidate, co-authored by Assoc./Prof. Yinghui Tian, Prof. Christophe Gaudin and Prof. Mark J. Cassidy, and completed as


The candidate constructed the finite element model under supervision of Assoc./Prof. Yinghui Tian. The candidate prepared the results and wrote the draft paper. Further interpretation of the results and the paper writing were completed by the candidate in
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Paper 3

This paper is presented in Chapter 4, first authored by the candidate, co-authored by Prof. Christophe Gaudin, Assoc./Prof. Yinghui Tian and Prof. Mark J. Cassidy, and published as


The candidate planned and performed the experimental programme presented in this paper under supervision of Prof. Christophe Gaudin, Prof. Mark J. Cassidy and Assoc./Prof. Yinghui Tian. The experimental results were processed and interpreted by the candidate. The technical report and paper were written by the candidate in collaboration with Assoc./Prof. Yinghui Tian, Prof. Christophe Gaudin and Prof. Mark J. Cassidy.

Paper 4

This paper is presented in Chapter 5, first authored by the candidate, co-authored by Prof. Christophe Gaudin, Prof. Mark J. Cassidy and Assoc./Prof. Yinghui Tian, and completed as


The candidate planned and carried out the experimental programme this paper is based on under supervision of Assoc./Prof. Yinghui Tian and Prof. Christophe Gaudin. The candidate processed the experimental data, prepared the technical report and drafted the paper. Further refinement of the paper was completed by the candidate in collaboration with Prof. Christophe Gaudin, Prof. Mark J. Cassidy and Assoc./Prof. Yinghui Tian.
Paper 5

This paper is presented in Chapter 6, first authored by the candidate, co-authored by Prof. Christophe Gaudin, Assoc./Prof. Yinghui Tian and Prof. Mark J. Cassidy, and published as


The candidate planned and performed the experimental programme this paper is based on under supervision of Prof. Christophe Gaudin and Assoc./Prof. Yinghui Tian. The candidate processed the experimental data and wrote the technical report and draft paper. Further interpretation and refinement of the paper were completed by the candidate with contributions from Assoc./Prof. Yinghui Tian, Prof. Christophe Gaudin and Prof. Mark J. Cassidy.

Paper 6

This paper is presented in Chapter 7, first authored by the candidate, co-authored by Assoc./Prof. Yinghui Tian, Prof. Mark J. Cassidy and Prof. Christophe Gaudin, and published as


The candidate planned and conducted the experimental programme presented in this paper under supervision of Prof. Christophe Gaudin, Prof. Mark J. Cassidy and Assoc./Prof. Yinghui Tian. The candidate analysed the experimental data and wrote the technical report and draft paper. Furtherer interpretation of the data and the final paper were completed by the candidate in collaboration with Assoc./Prof. Yinghui Tian, Prof. Christophe Gaudin and Prof. Mark J. Cassidy.
This paper is presented in Chapter 8, first authored by the candidate, co-authored by Prof. Christophe Gaudin, Assoc./Prof. Yinghui Tian and Prof. Mark J. Cassidy, and published as


The candidate planned and carried out the experimental programme in this paper under the supervision of Prof. Christophe Gaudin and Prof. Mark J. Cassidy. The candidate processed the experiment results, wrote the technical report and prepared the paper with contributions from Prof. Christophe Gaudin, Assoc./Prof. Yinghui Tian and Prof. Mark J. Cassidy.

I hereby declare that, except where specific reference is made in the text to the work of others, the contents of this dissertation are original and have not been submitted in whole or in part for consideration for any other degree of qualification at this, or any other, university.

Xiaojun Li

October 2015
PUBLICATIONS ARISING FROM THIS THESIS

Journal papers


Conference papers


Technical reports


ABSTRACT

Shallow foundations have been widely used in the offshore oil and gas industry as the support of gravity-based structures, fixed platforms or subsea infrastructure such as pipeline end manifolds and terminations. Depending on their application, they may be subjected to uplift loadings due to buoyancy, overturning moments or retrieval operations. Under uplift, negative excess pore pressures (also refer to suctions) may be generated in the soil, which contribute to increasing the uplift resistance. This is beneficial during the operation of offshore structures, e.g., to resist cyclic wave loading, but is impeditive for retrieval of subsea structures. Traditional methods only consider the uplift of shallow foundations under either purely drained or purely undrained soil conditions, in which the pore pressure mechanism is ignored and the soil is simplified as a single-phase material in numerical modelling. This thesis focuses on the uplift resistance of offshore shallow foundations in clay and the underlying fundamental generation and dissipation of pore pressures.

Centrifuge and numerical modelling were performed to investigate the mechanism of uplift. The uplift resistance was found to be a function of the rate of uplift, which governs the suction generation and dissipation in the soil and the occurrence of breakout between the foundation invert and the soil. The suction and the associated uplift resistance are highly rate dependant and closely related to the preloading history prior to uplift. With increasing uplift velocity, the resistance increases due to the generation of a higher magnitude of suction from the reduced rate of excess pore pressure dissipation and viscous enhancement of the soil strength. The application of preloading may decrease or increase the subsequent uplift capacity depending on the magnitude and duration of preloading, from which the soil may be remoulded and strengthened. The breakout, which occurs prior to or after a peak loading, limits the sustainability of suction and consequently the magnitude of the uplift resistance. Breakout is generated by either the vertical effective stress at the foundation invert reducing to zero, or a change in boundary conditions resulting in the separation of the foundation invert from the soil. A combination of eccentric uplift and perforation can facilitate the development of a breakout and significantly reduce the uplift capacity.
The research presented in the thesis provides a comprehensive understanding of the suction generation and dissipation mechanisms in clay, and enables an integrated approach to predict the uplift resistance of shallow foundations. Mitigation measures were developed to establish an optimal strategy to facilitate the retrieval of shallow foundations used to support subsea infrastructure.
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Roman

\( a, b, c \) backbone curve parameters
\( A \) area
\( A_{sh} \) shaded area
\( A_{soilplug} \) area of soil plug
\( B \) width; remoulding rate factor (Chapter 5)
\( B_0 \) width of mudmat foundation
\( c_v \) coefficient of soil consolidation
\( d \) diameter of T-bar; thickness (Chapter 2); backbone curve parameter (Chapter 3); parameter controlling the strength gain due to fully consolidated preloading (Chapter 8)
\( d_0 \) diameter of perforation
\( D \) diameter of foundation; width (Chapter 3 and 4)
\( e \) void ratio; eccentricity (Chapter 2)
\( e_0 \) initial void ratio
\( e_1 \) void ratio at \( p' = 1 \) kPa on the INCL in \( e - \ln p' \) space
\( e_{cs} \) void ratio at \( p' = 1 \) kPa on the CSL in \( e - \ln p' \) space
\( E_s \) secant stiffness
\( f(U) \) function of strength gain due to partially consolidation
\( f_o \) stress influence factor
\( F_{up} \) peak uplift force
\( F_{up, net} \) net peak uplift force
\( g \) gravitational acceleration of 9.8 m\(^2\)/s
\( G' \) submerged self-weight
\( G_s \) specific weight
\( h \) skirt length or embedment
\( i \) number of cycles
\( I_p \) plasticity index
\( k \) gradient of soil strength profile; permeability (Chapter 1, 7 and 8)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>$K_0$</td>
<td>coefficient of geostatic stress</td>
</tr>
<tr>
<td>$L$</td>
<td>image patch size</td>
</tr>
<tr>
<td>$L_0$</td>
<td>length of mudmat foundation</td>
</tr>
<tr>
<td>$m, n$</td>
<td>parameters controlling the strength gain due to partial consolidation</td>
</tr>
<tr>
<td>$m_v$</td>
<td>volume compressibility</td>
</tr>
<tr>
<td>$M$</td>
<td>slope of the CSL in $p' - q$ space</td>
</tr>
<tr>
<td>$N$</td>
<td>specific volume on the NCL at $\sigma_v' = 1$ kPa in $e - \ln \sigma_v'$ space</td>
</tr>
<tr>
<td>$N_c$</td>
<td>bearing capacity factor</td>
</tr>
<tr>
<td>$N_{T-bar}$</td>
<td>bearing capacity factor of T-bar</td>
</tr>
<tr>
<td>$N$</td>
<td>prototype/model scale</td>
</tr>
<tr>
<td>$p$</td>
<td>mean total stress</td>
</tr>
<tr>
<td>$p_1, p_2, p_3$</td>
<td>peak suction monitored by PPT1, PPT2 and PPT3</td>
</tr>
<tr>
<td>$\bar{p}$</td>
<td>average peak suction</td>
</tr>
<tr>
<td>$p'$</td>
<td>mean effective stress</td>
</tr>
<tr>
<td>$p_0'$</td>
<td>initial mean effective stress</td>
</tr>
<tr>
<td>$p_c'$</td>
<td>mean effective stress on initial yield surface at $q = 0$</td>
</tr>
<tr>
<td>$q$</td>
<td>deviatoric stress or resistance</td>
</tr>
<tr>
<td>$q_0$</td>
<td>initial deviatoric stress</td>
</tr>
<tr>
<td>$q_{correct}, q_{net}$</td>
<td>corrected or net resistance accounting for difference between overburden and submerged weight of soil plug</td>
</tr>
<tr>
<td>$q_m$</td>
<td>measured resistance</td>
</tr>
<tr>
<td>$q_p$</td>
<td>preloading</td>
</tr>
<tr>
<td>$q_{T-bar}$</td>
<td>resistance of T-bar</td>
</tr>
<tr>
<td>$q_u$</td>
<td>peak resistance under compression (Chapter 6) and uplift (Chapter 2, 7 and 8)</td>
</tr>
<tr>
<td>$q_{u, ref}, q_{upref}$</td>
<td>undrained uplift capacity as a reference</td>
</tr>
<tr>
<td>$q_{up}$</td>
<td>peak uplift resistance</td>
</tr>
<tr>
<td>$q_{up, correct}$</td>
<td>peak of corrected resistance in uplift</td>
</tr>
<tr>
<td>$q_{u-c}$</td>
<td>undrained bearing capacity in compression as a reference</td>
</tr>
<tr>
<td>$q_{u-u}$</td>
<td>undrained bearing capacity in uplift as a reference</td>
</tr>
<tr>
<td>$Q_m$</td>
<td>measured resistant force</td>
</tr>
<tr>
<td>$Q_u$</td>
<td>undrained bearing force</td>
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<tr>
<td>Symbol</td>
<td>Description</td>
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<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$R, R_d$</td>
<td>uplift capacity ratio</td>
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<tr>
<td>$R_u$</td>
<td>excess pore pressure ratio</td>
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<tr>
<td>$s_u$</td>
<td>soil undrained shear strength</td>
</tr>
<tr>
<td>$s_{u,i}$</td>
<td>soil strength at cyclic number, $i$</td>
</tr>
<tr>
<td>$s_{u,i} = 0.25$</td>
<td>soil strength at cyclic number, $i = 0.25$</td>
</tr>
<tr>
<td>$s_0$</td>
<td>soil undrained shear strength at skirt tip/foundation base level</td>
</tr>
<tr>
<td>$s_{0,ref}$</td>
<td>reference soil shear strength</td>
</tr>
<tr>
<td>$s_{um}$</td>
<td>soil undrained shear strength at mudline</td>
</tr>
<tr>
<td>$s_{umax}$</td>
<td>soil strength after fully consolidated preloading</td>
</tr>
<tr>
<td>$s_{uop}, s_{u-op}$</td>
<td>operative soil strength</td>
</tr>
<tr>
<td>$s_{up}$</td>
<td>soil strength immediately after preloading</td>
</tr>
<tr>
<td>$S_t$</td>
<td>soil sensitivity</td>
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<tr>
<td>$t$</td>
<td>time</td>
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<tr>
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<td>initial time for consolidation</td>
</tr>
<tr>
<td>$t_m$</td>
<td>model time</td>
</tr>
<tr>
<td>$t_p$</td>
<td>prototype time</td>
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<tr>
<td>$T$</td>
<td>normalised time</td>
</tr>
<tr>
<td>$t_{50}, T_{50}$</td>
<td>time and normalised time to 50% consolidation degree</td>
</tr>
<tr>
<td>$\Delta t$</td>
<td>skirt wall thickness</td>
</tr>
<tr>
<td>$u$</td>
<td>excess pore pressure</td>
</tr>
<tr>
<td>$u_{ave}$</td>
<td>average excess pore pressure</td>
</tr>
<tr>
<td>$u_{ave,ref}$</td>
<td>average excess pore pressure generation under fully undrained condition</td>
</tr>
<tr>
<td>$u_i, u_{p0}$</td>
<td>initial excess pore pressure</td>
</tr>
<tr>
<td>$u_p$</td>
<td>excess pressure monitored by PPTs; excess pore pressure due to the change of mean total stress (Chapter 7)</td>
</tr>
<tr>
<td>$u_{PPT0}$</td>
<td>excess pore pressure monitored by PPT0</td>
</tr>
<tr>
<td>$u_q$</td>
<td>excess pore pressure due to the change of deviatoric stress</td>
</tr>
<tr>
<td>$u_t$</td>
<td>excess pressure monitored by TPTs</td>
</tr>
<tr>
<td>$</td>
<td>u_{PPT}</td>
</tr>
<tr>
<td>$\hat{u}$</td>
<td>nominal excess pore pressure generated due to preloading and consolidation</td>
</tr>
<tr>
<td>$\hat{u}_i$</td>
<td>initial nominal excess pore pressure</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>-------</td>
<td>------------</td>
</tr>
<tr>
<td>$U$</td>
<td>degree of consolidation</td>
</tr>
<tr>
<td>$v$</td>
<td>velocity or specific volume</td>
</tr>
<tr>
<td>$v_0$</td>
<td>initial specific volume</td>
</tr>
<tr>
<td>$v_{pc}$</td>
<td>current specific volume during consolidation</td>
</tr>
<tr>
<td>$\Delta v$</td>
<td>change of specific volume</td>
</tr>
<tr>
<td>$V$</td>
<td>normalised velocity</td>
</tr>
<tr>
<td>$V_0$</td>
<td>normalised velocity corresponding to zero strain rate effect</td>
</tr>
<tr>
<td>$V_{50}$</td>
<td>normalised velocity corresponding to 50% full suction generation</td>
</tr>
<tr>
<td>$V_{ref}$</td>
<td>normalised velocity as a reference</td>
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<tr>
<td>$w$</td>
<td>displacement or settlement</td>
</tr>
<tr>
<td>$w_f$</td>
<td>final settlement</td>
</tr>
<tr>
<td>$w_p$</td>
<td>peak uplift distance</td>
</tr>
<tr>
<td>$W$</td>
<td>effective width</td>
</tr>
<tr>
<td>$W_{soilplug}$</td>
<td>submerge weight of soil plug</td>
</tr>
<tr>
<td>$x$</td>
<td>horizontal axis; shortest drainage path (Chapter 2)</td>
</tr>
<tr>
<td>$y$</td>
<td>equivalent width</td>
</tr>
<tr>
<td>$z$</td>
<td>depth</td>
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**Greek**

<table>
<thead>
<tr>
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<th>Definition</th>
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<tbody>
<tr>
<td>$\alpha$</td>
<td>interface frictional ratio; perforated area ratio (Chapter 2)</td>
</tr>
<tr>
<td>$\alpha^\prime$</td>
<td>equivalent perforation ratio</td>
</tr>
<tr>
<td>$\gamma, \gamma_{max}$</td>
<td>principle in-plane shear strain and maximum value</td>
</tr>
<tr>
<td>$\gamma'$</td>
<td>submerged unit weight of soil</td>
</tr>
<tr>
<td>$\gamma_w$</td>
<td>unit weight of water</td>
</tr>
<tr>
<td>$\dot{\gamma}$</td>
<td>strain rate</td>
</tr>
<tr>
<td>$\dot{\gamma}_{ref}$</td>
<td>reference strain rate</td>
</tr>
<tr>
<td>$\delta_{rem}$</td>
<td>soil degradation ratio due to cyclic loading</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>slope of the URL in $e - \ln p'$ space</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>slope of the NCL, ISL and CSL in $e - \ln p'$ space; coefficient of strain rate effect (Chapter 3)</td>
</tr>
<tr>
<td>$\Lambda$</td>
<td>parameter controlling shape of consolidation curves</td>
</tr>
<tr>
<td>$\mu$</td>
<td>coefficient of strain rate effect (Chapter 2); frictional factor</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>----------</td>
<td>---------------------------------------------------------------------------</td>
</tr>
<tr>
<td>( \nu )</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>( \sigma'_{a, \theta, r} )</td>
<td>normal stresses along three cylindrical axes ( a, \theta ) and ( r )</td>
</tr>
<tr>
<td>( \sigma'_{v} )</td>
<td>vertical effective stress; overburden pressure (Chapter 6)</td>
</tr>
<tr>
<td>( \sigma'_{v0} )</td>
<td>initial vertical effective stress</td>
</tr>
<tr>
<td>( \sigma'_{v0, max} )</td>
<td>maximum vertical effective stress, e.g., ( \text{OCR} = \sigma'<em>{v0, \text{max}} / \sigma'</em>{v0} )</td>
</tr>
<tr>
<td>( \sigma'_{vc} )</td>
<td>critical vertical effective stress</td>
</tr>
<tr>
<td>( \sigma'_{vp, \theta, r} )</td>
<td>vertical effective and total stress immediately after preloading</td>
</tr>
<tr>
<td>( \sigma'_{vpc} )</td>
<td>current vertical effective stress during consolidation</td>
</tr>
<tr>
<td>( \phi'_{tc} )</td>
<td>internal friction angle in triaxial compression test</td>
</tr>
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**Abbreviations**

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Definition</th>
</tr>
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<tbody>
<tr>
<td>ASR</td>
<td>Artificially Seeding Ratio</td>
</tr>
<tr>
<td>COFS</td>
<td>Centre for Offshore Foundation Systems</td>
</tr>
<tr>
<td>CSL</td>
<td>Critical State Line</td>
</tr>
<tr>
<td>EPP</td>
<td>Excess Pore Pressure</td>
</tr>
<tr>
<td>ESP</td>
<td>Effective Stress Path</td>
</tr>
<tr>
<td>FE</td>
<td>Finite Element</td>
</tr>
<tr>
<td>INCL</td>
<td>Isotropically Normal Consolidation Line</td>
</tr>
<tr>
<td>ISL</td>
<td>Intact Strength Line</td>
</tr>
<tr>
<td>( K_0-\text{CL} )</td>
<td>( K_0 ) Consolidation Line</td>
</tr>
<tr>
<td>LED</td>
<td>Light-Emitting Diode</td>
</tr>
<tr>
<td>LL</td>
<td>Liquid Limit</td>
</tr>
<tr>
<td>LOC</td>
<td>Lightly Over-Consolidated</td>
</tr>
<tr>
<td>MCC</td>
<td>Modified Cam Clay</td>
</tr>
<tr>
<td>NCL</td>
<td>Normal Compression Line</td>
</tr>
<tr>
<td>OCR</td>
<td>Over-Consolidated Ratio</td>
</tr>
<tr>
<td>PD</td>
<td>Partially Drained</td>
</tr>
<tr>
<td>PIV</td>
<td>Particle Image Velocimetry</td>
</tr>
<tr>
<td>PL</td>
<td>Plastic Limit</td>
</tr>
<tr>
<td>PLEMs/PLETs</td>
<td>Pipeline End Manifolds and Terminations</td>
</tr>
<tr>
<td>PPT</td>
<td>Pore Pressure Transducers</td>
</tr>
<tr>
<td>RP</td>
<td>Reference Point</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>---------------------------------</td>
</tr>
<tr>
<td>TPT</td>
<td>Total Pressure Transducer</td>
</tr>
<tr>
<td>TSP</td>
<td>Total Stress Path</td>
</tr>
<tr>
<td>UD</td>
<td>Undrained</td>
</tr>
<tr>
<td>URL</td>
<td>Unloading-Reloading Line</td>
</tr>
<tr>
<td>UWA</td>
<td>The University of Western Australia</td>
</tr>
</tbody>
</table>

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XXX


CHAPTER 1  INTRODUCTION

1.1 Background and motivation

Offshore shallow foundations consist of concrete or steel bases that rest on the surface of the seabed. They may feature peripheral or internal skirts to embed slightly in the soil and provide additional capacity. Historically, shallow foundations have been widely used to support various gravity-based structures, fixed platforms or as temporary foundations for jacket installations and small subsea infrastructures such as the Pipeline End Manifolds and Terminations (PLEMs/PLETs). Some typical examples of the application of offshore shallow foundations are illustrated in Figure 1.1, and more descriptions of shallow foundations can be found in Andenaes et al. (1996), Humpheson (1998), Neubecker and Erbrich (2004), Watson and Humpheson (2007), Acosta-Martinez (2009), Randolph and Gourvenec (2011) and Mana (2013).

It is notably common for shallow foundations to be subjected to upward tensile loadings in ocean environments. Typical examples include overturning forces that are caused by environmental loadings (e.g., winds, waves and currents) and the buoyancy from empty storage tanks or floating structures. In particular, subsea infrastructures and their underlying foundations (e.g., mudmats – a type of shallow foundations) are required to be removed from the seabed for maintenance, reuse in other drilling fields or at the end of the field development in compliance with environmental regulations. Figure 1.1 also illustrates various tensile loadings that are experienced by offshore shallow foundations.

The application of tensile loadings can generate negative excess pore pressure (which causes the pore water pressure in the soil dropping blow the hydrostatic pressure, also named as suction in the thesis), particularly in low permeable soils. Traditional design methodologies assume that the magnitude of suction generated under undrained conditions is sufficiently high to sustain a reverse end bearing failure mechanism beneath the foundation, which is identical to the failure mechanism in compression but opposite in direction. Thus, a bearing capacity equivalent to the compression can be mobilised. Alternatively, foundation uplift is assumed to occur under fully drained conditions, for which excess pore pressures are not generated, and only the frictional resistance of the
foundation skirt (if any) is considered. In both cases, the soil can be simulated as a single-phase material in numerical modelling.

In fact, the generation of suction in the soil is rather complex and is affected by the geometric configuration of the foundation (e.g., shapes, skirt lengths and perforations), loading conditions during uplift operations (e.g., eccentric, varied rates of transient and varied levels of sustained uplifts) and loading history (e.g., preloading due to self-weight and active installation) prior to the uplift. Ignoring the complexity of suction development in the soil may lead to inaccurate determination of the uplift resistance in designing offshore foundations. Poor estimation of the suction force introduces difficulties in removing subsea foundations in deep waters, where safety and cost are of concern. The development of mitigation measures relies on the reasonable assessment of the suction pressures as a function of soil parameters, foundation geometry and operational conditions.

In summary, the motivation of this project is to estimate more accurately the uplift resistance of offshore shallow foundations for a wide range of conditions by better understanding the generation and dissipation of suction in clay soils. The outcomes of this study improve the understanding of various offshore uplift behaviours and provide guidance to offshore foundation designs and deep-water retrieval operations.

1.2 Aim of research

The overall aim of the project is to provide a better understanding of the suction generation and dissipation mechanisms in clay soils and more accurately predict the uplift resistance of offshore shallow foundations. The specific objectives of the project are as follows:

- To investigate the effect of drainage paths on the suction development and its associated uplift behaviour of shallow foundations, which is primarily attributed to different geometric configurations of the foundations, such as shapes, skirt lengths and perforations;

- To examine the foundation performance under various operational conditions during
the uplift, which include eccentric, varied rates of transient and varied levels of sustained uplift of shallow foundations;

- To study the existence of loading history and resulting initial excess pore pressure field on the subsequent uplift capacity of shallow foundations, which is caused by applying a load (preloading) and consolidation prior to uplift;

- To compare the similarities and differences between compressing and uplifting a shallow foundation, with particular focus on the excess pore pressure generation and its effect on the failure mechanisms;

- To discuss the effects of various soil mobilisation phenomena such as the reverse end bearing failure mechanism, suction relief mechanism and breakout on the uplift performance of shallow foundations.

It is believed that the aspects considered in this project provide the first comprehensive investigation into the suction generation and dissipation mechanisms in clay soils and their implications on the uplift of offshore shallow foundations. The outcomes are significant for understanding the uplift behaviour of other offshore foundations, such as pipelines and anchors.

1.3 Methodology

Both geotechnical centrifuge and finite-element modelling were used to investigate the present problem.

Geotechnical centrifuge is advantageous in modelling offshore soil-structure interaction problems. First, soil behaviour is stress dependant, and the in-situ soil stress state at a depth $z$ in the prototype can be easily replicated in the centrifuge at a depth $z/N$ by creating a radial acceleration equivalent to $N g$ ($g$ is the gravitational acceleration of 9.8 m$^2$/s). Second, the seepage and consolidation process in the soil can be accelerated by $N^2$ times compared to the in-situ time, which is significant in the present study because it reduces the test duration. Scaling laws for centrifuge modelling are summarised in Table 1.1, and more details can be found in Taylor (1995) and Garnier et al. (2007).
All tests in this project were performed in the drum centrifuge at the Centre for Offshore Foundation Systems (COFS) and ARC Centre of Excellence for Geotechnical Science and Engineering, The University of Western Australia (UWA). This centrifuge has the merit of enabling multiple tests to be performed in one soil sample. As shown in Figure 1.2, the drum centrifuge comprises a round U-shape channel and a central tool table. The former is used as the container of the soil sample for full channel tests (Chapters 4, 6, 7 and 8) and can be instrumented with small strongboxes for Particle Image Velocimetry (PIV) analyses as illustrated in Figure 1.3 (Chapter 5). All kaolin clay samples used in the tests were prepared following a standardised rigorous procedure to ensure the uniformity of the entire test programme. The tool table is aligned at the centre of the centrifuge, which can be halted during testing for checking and tool changing. A more detailed technical description of this drum centrifuge can be found in Stewart et al. (1998).

Complementary numerical modelling was performed to validate and compare with the centrifuge test results (Chapter 2 and 3). All analyses were performed using the commercial software Abaqus version 6.11 (Dassault Systèmes 2011). The soil behaviour was represented by the Modified Cam Clay (MCC) constitutive model (Roscoe and Burland 1968), which enabled the coupled analysis accounting for the pore pressure response. As detailed in Chapter 2 and 3, the values of the parameters for the MCC model represent the typical kaolin clay that is used at UWA (Stewart 1992; Chatterjee et al. 2012, 2013). This allows some comparisons between finite-element modelling and centrifuge modelling.

1.4 Thesis outline

Apart from the Introduction and Conclusion chapters (1 and 9 respectively), the remaining chapters of the thesis are presented as a series of technical papers that are published (Chapter 2, 4, 6, 7 and 8) or completed for future publication (Chapter 3 and 5). The thesis presents the results from both centrifuge testing and finite-element modelling, and the influential factors that are considered in the thesis belong in three categories: geometry, operational condition and loading history. The specific factors that are considered in each main chapter are outlined in Table 1.2. The details of the chapters are as follows.
Chapter 2 examines the potential of a coupled analysis of the uplift of shallow foundations using finite-element modelling. The performances of short-term compression and uplift of the shallow foundation are compared. The similarity in undrained capacity and difference in excess pore pressure development between compression and uplift are demonstrated by examining load paths at the element level. Some partially drained uplifts are presented, highlighting the necessity for further study.

Chapter 3 simulates the partially drained uplift of shallow foundations and the effects of the preloading and consolidation on the subsequent uplift capacity using the coupled analysis established in Chapter 2. The backbone curves for both resistance and suction pressure are obtained and compared to the corresponding results from the centrifuge testing in Chapter 4. The centrifuge data obtained in Chapter 6 are also compared with the finite-element results to discuss the consistency and potential discrepancies between them.

Chapter 4 considers the effect of various uplift rates on the suction generation and resistance of both circular and square foundations using centrifuge modelling. The transition from fully undrained to fully drained conditions is quantified. Empirical expressions are also derived to describe the backbone curves of the variation of uplift resistance with the uplift rate, which shows the effects of partial drainage and soil viscosity.

Chapter 5 continues the study in Chapter 4 and examines the failure mechanisms of uplifting square skirted and flat foundations using PIV analyses. The effects of the partially drained condition and soil viscosity on the failure mechanisms are discussed. The breakout process during uplift of the skirted foundation is also visualised and discussed briefly.

Chapter 6 describes an experimental study on the effects of an initial excess pore pressure field on the subsequent uplift capacity of a shallowly skirted foundation, which is formed by applying different magnitudes and durations of preloading onto the foundation prior to uplift. The degree of consolidation under a certain level of preloading is quantified to generalise the results for design purposes. Meanwhile, a theoretical
A framework based on the critical state concept is proposed to describe the variation of the operative soil strength as a function of the preloading level and degree of consolidation.

**Chapter 7** examines another type of failure caused by sustained uplift of a shallowly skirted foundation using centrifuge tests. The displacement rate and breakout time of the foundation under different sustained loading levels are examined. The suction variations during uplift are monitored, which help reveal the reasons of the breakout. The effect of sustained loading on the subsequent undrained uplift is also briefly discussed.

**Chapter 8** investigates a practical problem of retrieving deep-water mudmat foundations using centrifuge modelling. The effects of perforations, skirt length and eccentric uplift on the uplift capacity are examined by associating them with the suction generation and dissipation at the foundation invert. Different soil failure and suction relief mechanisms during concentric and eccentric uplifts are discussed to find an optimal method to remove mudmats from the seabed.

**Chapter 9** summarises the main conclusions of the thesis by highlighting the key findings and implications for the design of offshore shallow foundations and deep-water retrieval operations. Recommendations are also provided for future research.

### 1.5 References


Chapter 1

Introduction


### Table 1.1 Scaling factors in centrifuge modelling

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<th>Scaling relationship (model/prototype)</th>
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<td>Acceleration</td>
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<td>Velocity (dynamic)</td>
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<td>Time (consolidation)</td>
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### Table 1.2 Influential factors considered in main chapters

<table>
<thead>
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<th>Main chapters</th>
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<td>Chapter 8: Perforated mudmat uplift test</td>
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Note: PIV – Particle Image Velocimetry
Figure 1.1 Application of shallow foundations and associated tensile loadings in offshore structures: (a) & (b) gravity-based platforms; (c) jacket platform; (d) tension leg platform; (e) (partially) empty storage tank; (f) wind turbines; (g) removal of subsea infrastructures (modified from Acosta-Martinez (2009) and Mana (2013))

Figure 1.2 Drum centrifuge at COFS, the UWA: (a) with protective cover; (b) without protective cover
Figure 1.3 PIV set-up in the drum centrifuge (modified from Stanier and White (2013))
CHAPTER 2  COMPARATIVE STUDY OF THE COMPRESSION AND UPLIFT OF SHALLOW FOUNDATIONS

Abstract

This paper compares the compression and uplift capacity of a strip foundation from numerical coupled analyses using the Modified Cam Clay (MCC) soil model. The focus is on the failure mechanism and excess pore pressure development in the soil. Triaxial compression and tension tests were first modelled to develop a rigorous understanding of the excess pore pressure responses; then, the compression and uplift of a strip foundation were modelled. The results show that the balance of excess pore pressures due to the changes in mean total stress and deviatoric stress during the compression and uplift of a strip foundation are different, although the ultimate undrained capacities are identical. Furthermore, the resistance and excess pore pressure responses during uplift differ from those in compression under the $K_0$-consolidated condition because of the elastic unloading. Although the failure mechanisms have identical shape and size between undrained compression and uplift, the excess pore pressure distribution in the soil is different and affects the load-displacement behaviours under partially drained compression and uplift.

Keywords: finite-element modelling; shallow foundation; bearing capacity; pore pressure; offshore engineering; soil-structure interaction
Chapter 2  Comparative study of the compression and uplift of shallow foundations

2.1 Introduction

Shallow foundations are commonly used in offshore engineering as foundations of fixed platforms or subsea infrastructures. These foundations are subjected to both compression and uplift loadings, which result from the self-weight of structures, environmental loadings and retrieval operations. Therefore, the estimation of the ultimate capacity of shallow foundations under both compression and uplift is significant for offshore design guidelines.

Traditional solutions focus extensively on the undrained bearing capacity in compression, for which the soil is treated as a single-phase ideal plastic medium, e.g., the Tresca or Mises models, based on the assumption that the soil behaves under fully undrained conditions (Terzaghi 1943; Davis and Booker 1973; Randolph et al. 2004). Following a similar analysis, experimental and numerical studies have indicated that a reverse end-bearing failure mechanism, which is identical to the compression mechanism but opposite in direction, can be mobilised during the undrained uplift of shallow foundations (Craig and Chua 1990; Acosta-Martinez et al. 2008; Mana et al. 2012, 2013; Chatterjee et al. 2013a). The soil underneath the foundation moves upward with the foundation (by sustaining adhesion at the foundation invert), which results in an ultimate capacity that is identical to compression.

However, saturated soils are two-phase materials and may be subjected to partially drained conditions. In this study, the similarities and differences between compression and uplift of a strip foundation are analysed using a coupled finite element method to dissociate pore pressure responses from mechanical soil responses. The soil is represented by the Modified Cam Clay (MCC) constitutive model. First, triaxial compression and tension are modelled to build a basic understanding of the excess pore pressure interaction mechanisms in the soil. Then, a plane strain shallow foundation model is established to characterise the resistance and excess pore pressure responses under undrained compression and uplift. The effect of the excess pore pressure distribution on the partially drained uplift is also briefly discussed.
2.2 Finite element modelling

All analyses were performed with the commercial software Abaqus using the small-strain finite element method (Dassault Systèmes 2011).

2.2.1 Soil property

The standard MCC constitutive model was used to simulate the soil behaviour (Roscoe and Burland 1968). The soil was assumed to be homogenous and normally consolidated. The yield surface is considered isotropic, with an identical value of the parameter $M$ in compression and extension (see Table 2.1), so the model yields the same foundation capacity under compression and uplift. This choice is justified by experimental evidences that demonstrated that, under undrained uplift, a reverse end bearing failure mechanism was developed and the same capacity was mobilised as in compression (Chen et al. 2012; Li et al. 2014a). While the real soil may exhibit anisotropy, the present simplification is appropriate, as a first approximation, to demonstrate different excess pore pressure development between compression and uplift.

Initially, the soil was subjected to a uniform surcharge of 1 kPa to build a uniform initial stress field at the beginning of the analyses and to ensure numerical stability. The parameters of the MCC model are summarised in Table 2.1. The values represent the typical properties of the kaolin clay used at the Centre for Offshore Foundation Systems (COFS) at The University of Western Australia (UWA) (Stewart 1992; Chatterjee et al. 2012, 2013b).

The soil was considered either isotropically or $K_0$-normally consolidated with the coefficient of earth pressure $K_0 = 1$ or 0.6, respectively. The initial size of the yield surface can be determined by

$$p'_c = \frac{q_0^2}{M^2 p_0} + p'_0$$

(2.1)

where $p'_0$ and $q_0$ are the initial mean and deviatoric effective stresses, respectively. The initial void ratio is expressed as
Chapter 2 Comparative study of the compression and uplift of shallow foundations

\[ e_0 = e_1 - \kappa \ln p_0 - (\lambda - \kappa) \ln p_e \]  \hspace{1cm} (2.2)

where

\[ e_1 = e_{cs} + (\lambda - \kappa) \ln 2 \]  \hspace{1cm} (2.3)

The definitions of \( M, \lambda, \kappa \) and \( e_{cs} \) are provided in Table 2.1.

### 2.2.2 Model and mesh

Two numerical models were considered for the present study. First, an axisymmetric model was built to examine the soil behaviour at the “element” scale by modelling the triaxial compression and tension. As shown in Figure 2.1, an axisymmetric model was built to represent a soil cell with unit diameter and unit height (1 × 1 m). Four-node axisymmetric quadrilateral-, bilinear-displacement- and bilinear-pore-pressure elements (CAX4P) were used to create the discrete soil domain. The top and bottom of the cell were assumed to be fully smooth in the tangential direction; thus, uniform distributions of stresses and pore pressure can be expected within the sample. A mesh study on this triaxial model indicated that the results are not sensitive to the density of meshed due to the uniform nature of stress and excess pore pressure distributions in the sample.

Then, a coupled analysis of compression and uplift of a strip foundation was performed by considering a plane strain model. Only half of the model was built because of the symmetry of the problem. As shown in Figure 2.2, the surface strip was assumed to be a rigid body with width \( B = 1 \) m and a reference point at the geometric centre of the bottom. The soil extended up to \( 5B \) from the edges of the foundation in both horizontal and vertical directions to avoid boundary interferences (Gourvenec et al. 2006). The four-node plane strain quadrilateral-, bilinear-displacement and bilinear-pore-pressure elements (CPE4P) were used. Finer meshes around the corner of the strip were used to improve the accuracy of the analyses. The mesh density around the foundation was examined, prior to the formal simulation, to ensure an appropriate balance between accuracy and computational efficiency. In the present model, a bearing capacity factor that is approximately 3% higher than the theoretical value of 5.14 (Prandtl 1921; Reissner 1924) can be achieved. The interface between the foundation and the top of the soil was assumed to be rough in the
Chapter 2 Comparative study of the compression and uplift of shallow foundations
tangential direction and did not allow any separation in the normal direction; thus, the
foundation base and underlying soil were fully bonded in both compression and uplift.
More discussion on the interface setting is given in Section 2.6. All boundaries were
assumed to be impermeable, except for the free soil surface.

2.3 Triaxial compression and tension tests

The procedure to model triaxial compression and extension is as follows: 1) the sample
was assumed to be normally consolidated, and it was subjected to confined pressures in
both axial and radial directions; 2) an upward or downward displacement of \( w = \pm 0.5 \) m
was applied at the top of the sample without drainage at the boundaries.

Figure 2.3a presents the total and effective stress paths for triaxial compression and
tension from the initial isotropic conditions, i.e., \( K_0 = 1 \) (\( q = 0 \)), in \( p' \) or \( p - q \) space. Note
that \( p' = (\sigma_a + \sigma_\theta + \sigma_r)/3 \) and \( p = p' + u \) are the mean effective and total stresses,
respectively, and the deviatoric stress \( q = \sigma_a - \sigma_r \), where \( \sigma_a, \sigma_\theta \) and \( \sigma_r \) are the normal
(and principal) stresses along the three cylindrical coordinate axes, and \( u \) is the excess
pore pressure. The load paths begin at the intersection of the \( q = 0 \) line and the initial yield
surface and end on the expanded final yield surface because the soil hardens. As expected,
the total stress paths (TSPs) for compression and tension have identical slopes but travel
in opposite directions, and the effective stress paths (ESP) for compression and tension
are symmetrical to the \( q = 0 \) line, which results in a difference in development of excess
pore pressures between compression and tension. According to Zdravković et al. (2003),
the generated excess pore pressures can be divided into two parts: the first part is
generated from the change of the mean total stress (\( u_p \)), and the second part is generated
from the change in deviatoric (or shear) stress (\( u_q \)). As illustrated in Figure 2.3a, the
balance between these two parts of excess pore pressures is different for compression and
tension. In compression, \( u_p \) and \( u_q \) are both positive and add together to develop positive
excess pore pressures. In contrast, in tension, \( u_p \) is negative, whereas \( u_q \) is positive, so the
two parts partially cancel each other. This cancellation results in the generation of
negative excess pore pressures in the first stage of tension (\( u_p > u_q \)) but positive excess
pore pressures in the later stage of tension and at failure (\( u_p < u_q \)). This observation
suggests that although the overall resistance, which is defined by the effective stress, is
identical in magnitude in compression and tension, it results from different developments and magnitudes of excess pore pressures.

Figure 2.3b shows the TSPs and ESPs for triaxial compression and tension under the $K_0$-consolidated condition, i.e., $K_0 = 0.6$. The load paths begin at the interaction of the $K_0$ consolidation line ($K_0$-CL) and the initial yield surface and end on the expanded yield surface. Similar to $K_0 = 1$, the TSPs for compression and tension have identical slopes but travel in opposite directions. In compression, both $u_p$ and $u_q$ are positive and contribute to the development of a positive excess pore pressure. However, in tension, the ESP travels inside the initial yield surface (i.e., elastic unloading) before reaching the yield surface in the tension side. During elastic unloading, only the excess pore pressure due to the change in mean total stress $u_p$ is developed, generating negative excess pore pressures. After the ESP reaches the initial yield surface, positive $u_q$ is generated and reduces the magnitude of the negative excess pore pressure generated.

### 2.4 Undrained compression and uplift of shallow foundations

The modelling procedure for undrained compression and uplift of a shallow foundation in this section is as follows: 1) a surcharge of 1 kPa was applied on the soil to establish an initial stress field, and 2) displacement-controlled compression or uplift of the foundation was undertaken at a normalised velocity $V = vB/c_v$ of 10000, which is regarded as sufficiently high to generate fully undrained conditions (Chatterjee et al. 2013b; Li et al. 2014b). The coefficient of consolidation $c_v$ is estimated as

$$c_v = \frac{k}{m_v \gamma_w} = \frac{k}{\gamma_w} \frac{(1 + e_0)}{\lambda} p_0^* \quad (2.4)$$

where $k$ is the soil permeability, $\gamma_w$ is the unit weight of water, and $m_v$ is the volume compressibility, which is determined from the initial void ratio $e_0$ and the initial mean effective stress $p_0^*$.

Figure 2.4 presents the development with normalised displacement $w/B$ of the normalised resistance and normalised average excess pore pressure along the foundation invert for both compression and uplift under undrained conditions. For better comparison, the sign
conventions for compression and uplift are ignored, and only magnitudes are presented. The resistance $q_m$ and average excess pore pressure $u_{ave}$ at the foundation invert are normalised with the initial undrained shear strength $s_{ud0}$ at the foundation base level. For the plane strain conditions, the initial undrained shear strength can be expressed as

$$s_{ud0} = \frac{M}{\sqrt{3}} \exp \left( \frac{e_1 - (\lambda - \kappa) \ln(2) - e_0}{\lambda} \right)$$

(2.5)

where $e_0$ and $e_1$ are the initial void ratio and void ratio at 1 kPa mean effective stress on the normally consolidated line, respectively. $M$, $\lambda$ and $\kappa$ are parameters of the MCC model (Table 2.1).

As shown in Figure 2.4a, the ultimate undrained capacity is identical for both compression and uplift (e.g., $N_c = 5.30$ that is slightly higher than the theoretical value of 5.14) and is independent of the initial stress conditions, which are defined using the coefficient $K_0$ of earth pressure at rest. However, the mobilisation distance to reach the ultimate capacity is different for $K_0 = 0.6$. The load displacement response is stiffer in uplift than in compression because of the elastic unloading, whereas in compression, the soil yields at the beginning of loading, as illustrated in Figure 2.3b. Indeed, the stiffness is identical between compression and tension under isotropically consolidated conditions ($K_0 = 1$) because both exhibit the identical effective stress paths that travel outside the initial yield envelope, as shown in Figure 2.3a.

Figure 2.5 shows the vectors of normalised displacements for compression and uplift of the shallow foundation for both $K_0 = 1$ and 0.6 at the onset of failure, i.e., when $q_m/s_{ud0}$ reaches its maximum value of 5.30, as illustrated in Figure 2.4a. Figure 2.6 shows the corresponding contours of the normalised principal in-plane shear strain ($\gamma/\gamma_{max}$, where $\gamma_{max}$ is the maximum shear strain occurred in the soil), which helps identify the failure mechanism. Both figures indicate that a Prandtl-type mechanism is mobilised for both compression and uplift under undrained conditions regardless of the value of $K_0$. More importantly, the mechanism has almost identical sizes, which validates the generally accepted assumption that the undrained uplift generates a reverse end-bearing mechanism and develops an identical ultimate capacity as compression, as noted in Figure 2.4a and observed experimentally by Chen et al. (2012), Mana et al. (2012) and Li et al. (2014a).
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among others. This result is also corroborated with the effective stress paths at the element level (Figure 2.3), which intercept the same final yield envelope in compression and tension at symmetrical points about the horizontal axis.

Although the identical ultimate undrained capacity is developed for compression and uplift, different excess pore pressure fields are generated, for both \( K_0 = 1 \) and 0.6. Figure 2.4b shows the average excess pore pressure responses underneath the foundation, which were normalised by the initial soil strength for compression and uplift. The magnitude of excess pore pressures generated in compression is higher than that in uplift, for both \( K_0 = 1 \) and 0.6. As evident from Figure 2.4a, the normalised excess pore pressure, averaged across the foundation invert, is identical to the normalised resistance in compression (i.e., all the bearing resistance originates from the generation of excess pore pressures, as expected for undrained behaviour). In contrast, the normalised suction generated during uplift is only about 80% of the normalised uplift resistance. The difference can be interpreted from the stress paths developed at the element level, as shown in Figure 2.3, bearing in mind that the stress paths in triaxial compression and extension tests cannot fully represent the foundation tests. The excess pore pressure due to the change in mean total stress \( u_p \) can be either positive in compression or negative in uplift, but the excess pore pressure due to the change in deviatoric stress \( u_q \) is always positive in both compression and uplift. During compression, both \( u_p \) and \( u_q \) contribute to the generation of positive excess pore pressures to develop bearing resistance. In contrast, during uplift, the generated excess pore pressure \( u_p \) is negative, and balances the positive excess pore pressure \( u_q \) generated after yielding, so the absolute magnitude of suction generated in uplift is lower than the excess pore pressure developed during compression.

Figure 2.7 shows the excess pore pressure distributions in the soil at failure, which was normalised with the ultimate undrained capacity (denoted as \( q_u \)). In compression, only positive excess pore pressures are generated in the entire soil domain, whereas in uplift, both positive and negative pore pressures are generated depending on the stress paths followed by a specific soil element. In uplift, negative excess pore pressures concentrate underneath the foundation, where the soil is primarily subjected to tension in the normal directions and shearing, whereas positive excess pore pressures are developed next to the edge of the foundation, where the soil experiences compression without shearing.
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2.5 Compression and uplift of shallow foundations under partially drained conditions

Using coupled analyses, the uplift and compression capacity of shallow foundations can be investigated under partially drained conditions. This is undertaken by applying a reduced normalised loading velocity $V = 3$ and $V = 0.3$. Li et al. (2014b) demonstrated from centrifuge results that a partially drained condition was generated for a normalised velocity lower than 200.

Figure 2.8 shows the development of normalised resistances and normalised average excess pore pressures with normalised displacement for compression and uplift of the foundation under the two partially drained cases considered. The effect of partial drainage can be assessed by comparing the uplift resistances and excess pore pressures with the fully undrained case ($V = 10000$, shown as marked grey lines in Figure 2.8). To assist in the interpretation, the total and effective stresses (TSPs and ESPs) at three different locations in the soil, which are situated near the slip (or shearing) zones of the undrained failure mechanism (as shown in Figure 2.6), are plotted in Figure 2.9 for the three normalised velocities investigated, for both compression and uplift. It should be noted that the deviatoric stress for TSPs and ESPs is taken as the difference between the vertical and horizontal stresses (denoted as $\sigma'_z - \sigma'_x$), in order to reflect the different shearing directions under compression and uplift, whereas the deviatoric stress for the critical state line and the initial yield surface are derived from the Mises stress (denoted by $q_{\text{Mises}}$).

As illustrated in Figure 2.8, the present modelling is capable of simulating both the partially drained compression and uplift of shallow foundations, and is able to quantify the peak resistance and excess pore pressures generation and dissipation. Overall, the bearing capacity in compression increases with the reduction in normalised velocity due to the increase in effective stresses as the excess pore pressure dissipates during loading. In contrast, the uplift resistance decreases with the reduction in normalised velocity owing to the dissipation of negative excess pore pressure (or suction). Both effects can be related to the stress paths in the soil as drainage occurs. Figure 2.9 shows that the ESPs move towards the TSPs as the velocity reduces for both compression and uplift, which indicates the increasing dissipation of excess pore pressures. The soil underneath the foundation
primarily experienced hardening during compression, and unloading and softening during uplift, especially under partially drained conditions (Point 1 and 2 in Figure 2.9). More particularly, during uplift, the ESPs can potentially cross into the dry side of critical state line, so the soil exhibits behaviour similar to over-consolidated soil. This feature, together with the softening mentioned above, may contribute to the post-peak reduction response of uplift resistance (Figure 2.8b). To satisfy continuity, the soil within the edge of the undrained failure mechanism (Point 3 in Figure 2.9) is likely to be subjected to triaxial tension during compression and triaxial extension during uplift of the foundation.

2.6 Limitations of the model

A full bonded interface between the foundation base and top of the soil was implemented in the present study. This assumption is valid for shallow foundations in compression, since the foundation is expected to remain in contact with the soil during the whole loading process. This is indeed not the case for uplift, for which there are several experimental evidences demonstrating that foundation breakaway occurs (e.g., Chen et al. 2012; Li et al. 2014a). Several parameters influence foundation breakaway, including uplift velocity, loading history and soil degree of consolidation. They are all related to the variation in suction and effective stresses at the foundation invert. The present study focuses on the development of excess pore pressures leading to maximum compression and uplift resistance, which is believed to be reached before foundation breakaway. Nevertheless, a more advanced interface model is necessary to simulate more realistically the foundation breakaway and investigate the conditions it occurs and if it can be facilitated in order to reduce the maximum uplift resistance and facilitate foundation removal for instance.

2.7 Conclusions

This paper presents the results from a finite element coupled analysis of the compression and uplift capacity of a strip foundation. It focuses on identifying the relevant failure mechanism and excess pore pressure distribution under two different geostatic stress conditions ($K_0 = 1$ and 0.6). The analysis was informed with an identical analysis that was
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undertaken at the element level, which allows clear observation of the total and effective stress paths in compression and tension. The following conclusions are reached:

a) Failure mechanisms of identical sizes and shapes but evidently different directions are generated during compression and uplift under fully undrained conditions, which result in identical ultimate undrained capacities, which warrant the use of a unique bearing capacity factor when the ultimate bearing capacity of a shallow foundation is determined under undrained compression and uplift.

b) However, different excess pore pressure fields underneath the foundation are generated because of the balancing contribution of the change in mean total stresses and deviatoric stresses. Only positive excess pore pressures are generated in compression, whereas both positive and negative excess pore pressures are generated during undrained uplift.

c) The coupled analysis on partially drained compression and uplift of shallow foundations demonstrate that the compression resistance increases, but the uplift resistance decreases with a reduced loading velocity. Both are related to the excess pore pressure dissipations in the soil as the effective stress paths moves towards their corresponding total stress paths. The load-displacement responses can be explained by the hardening or softening stress paths of the soil underneath and adjacent to the foundation.

This study has demonstrated the potential to use the MCC model to perform coupled analysis of the uplift of shallow foundations under various drainage conditions. Further work is in progress both numerically and experimentally to (i) develop a more realistic soil-foundation interface to be able to model foundation breakaway, and (ii) thoroughly explore the development of uplift resistance with various uplift rates and after various periods of preloading under self-weight.
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2.8 References


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Table 2.1 Parameters of the MCC model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<td>Slope of the critical state line in $p - q$ space, $M$</td>
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<tr>
<td>Slope of the normally consolidated line in $e - \ln p'$ space, $\lambda$</td>
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</tr>
<tr>
<td>Slope of the unloading-reloading line in $e - \ln p'$ space, $\kappa$</td>
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<tr>
<td>Void ratio at $p' = 1$ kPa on the critical state line in $e - \ln p'$ space, $e_{cs}$</td>
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<tr>
<td>Poisson’s ratio, $\nu$</td>
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</tr>
<tr>
<td>Permeability, $k$ (m/s)</td>
<td>$10^{-9}$</td>
</tr>
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</table>
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Figure 2.1 Model and mesh of the triaxial tests

Figure 2.2 Model and mesh of the foundation tests (RP – Reference Point)
Figure 2.3 Total and effective stress paths for triaxial compression and tension at (a) $K_0 = 1$ and (b) $K_0 = 0.6$ (CSL – Critical State Line, $K_0$-CL – $K_0$ Consolidation Line, ESP – Effective Stress Path, TSP – Total Stress Path; $u_p$ and $u_q$ are the excess pore pressures due to the changes of the mean total stress and deviatoric stress, respectively)
Figure 2.4 (a) Normalised resistance and (b) normalised average excess pore pressure, which vary with the normalised displacement for compression and uplift of the foundation.
Figure 2.5 Vectors of normalised displacement as the foundation is displaced at failure for (a) compression and (b) uplift (the displacements at failure are indicated in Figure 2.4a)
Figure 2.6 Contours of normalised principal in-plane shear strain as the foundation is displaced at failure for (a) compression and (b) uplift
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Figure 2.7 Distributions of normalised excess pore pressure as the foundation is displaced at failure for (a) compression and (b) uplift
Figure 2.8 (a) Normalised resistance and average excess pore pressure generation, which vary with the normalised displacement for partially drained (a) compression and (b) uplift ($K_0 = 0.6, V = vB/c_v$).
Figure 2.9 Total and effective stress paths (TSPs and ESPs) in the soil: (a) typical locations; (b) compression; (c) uplift (CSL – Critical State Line; $K_0 = 0.6$, $V = vB/c_v$)
CHAPTER 3  COUPLED ANALYSIS OF SHALLOW FOUNDATIONS SUBJECTED TO UPLIFT

Abstract

Coupled finite element analyses using the Modified Cam Clay (MCC) constitutive soil model were used to investigate the uplift behaviour of shallow foundations, with a focus on the uplift resistance and the suction development, as a function of uplift velocity and preloaded consolidation prior to uplift. Results show that the uplift resistance of strip and circular foundations increases with uplift rate, due to the increase of suction generation in the soil. The effect of varied rates can be depicted by backbone curves and the level of suction development is fitted by a simple hyperbolic function. The consolidation after a compressive preloading for both strip and circular foundations increases the subsequent uplift capacity due to the dissipation of excess pore pressure and the enhancement of soil strength. As a result, higher levels of preloading and greater degree of consolidation increase the uplift resistance, with the latter having a dominant effect. The trends of soil strength gain are well predicted by general forms of expressions that may potentially provide guidance for offshore foundation designs. The predicted results were also compared with centrifuge test data, indicating an agreement in trends and highlighting the difference due primarily to the soil remoulding effect.

Keywords: finite element method; coupled analysis; shallow foundation; suction; consolidation; uplift
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3.1 Introduction

Shallow foundations represent a type of foundations used to support gravity-based structures and fixed platforms offshore. They also become increasingly used as the base for a variety of subsea infrastructures, such as Pipeline End Manifolds and Terminations (PLEMs/PLETs). These foundations are commonly subjected to tensile loads as the result of overturning forces, structure buoyancy, the cyclical nature of wave and current forces, or during retrieval at the decommission of projects.

Negative excess pore pressures or suction that is lower than the hydrostatic pressure can be generated during uplift. It is commonly considered that under fully undrained conditions, the suction generated at the foundation interface is sufficient to sustain joint soil and foundation movement, leading to a reverse end bearing failure mechanism and the same capacity as compression (Craig and Chua 1990; Puech et al. 1993; Watson et al. 2000; Acosta-Martinez et al. 2008; Mana et al. 2012, 2013). For this undrained case, the total stress method can be used to solve the uplift capacity of foundations analogous to compression by considering the soil as a single-phase material, e.g., Tresca material for clays (Deng and Carter 2000; Song et al. 2008; Chatterjee et al. 2013a). However, under partially drained conditions, the suction generated in uplift has time to dissipate, and consolidation may occur. Furthermore, the consolidation caused by the dead loads and self-weight of infrastructure applied onto the soil during installation and operation has a significant influence on the subsequent uplift capacity (Colliat et al. 2011; Li et al. 2015a). Coupled analysis is required in both situations to study the suction development under a variety of loading history and drainage conditions to predict more accurately the uplift resistance of shallow foundations for a wide range of operational conditions.

In the present study, finite element analyses using the Modified Cam Clay (MCC) constitutive model were performed to simulate the uplift resistance and the associated excess pore pressure responses for both strip and circular foundations. First, the foundations are pulled out over a range of velocities covering seven orders of magnitude to examine fully undrained to fully drained behaviours. Second, the uplift resistance under various magnitudes and durations of preloading is estimated and compared with centrifuge test data. Simple generalised relations are developed to predict the uplift resistance and can be used to provide guidance for offshore foundation design.
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3.2 Finite element modelling

All of the analyses presented here were implemented in the commercial software Abaqus using the small strain finite element method (Dassault Systèmes 2011).

3.2.1 Model description

Both two-dimensional plane strain (strip foundation) and axisymmetric (circular foundation) models were constructed for the present study. As illustrated in Figure 3.1, only half of the domain was modelled due to the symmetrical feature of the models. The foundation, modelled as a rigid body, represents a strip with width $B$ or a circular foundation with diameter $D$. The soil domain is $5 \times 5.5 \times B$ (or $D$) in dimension, which is large enough to eliminate boundary effects. The bottom of the soil is fixed, and the side boundaries are restrained in the horizontal direction but movable in the vertical direction to allow settlement. All of the boundaries except one are assumed to be impermeable; the free soil surface remains permeable to account for the effect of soil drainage conditions. The interface between the foundation and soil is assumed to be fully bonded without allowing separation during uplift. It is acknowledged that breakout (i.e., separation of the foundation from the soil) may occur in reality. The fully bonded interface in this study is a simplified assumption, which is capable of describing the suction generation. Further discussion on this point is detailed in Section 3.3.1.

3.2.2 Mesh validation

Figure 3.1 also illustrates the mesh configuration of the present models. Four-node plane strain or axisymmetric quadrilateral, bilinear displacement and bilinear pore pressure elements (represented as CPE4P or CAX4P in Abaqus) were chosen to mesh the soil domain (Dassault Systèmes 2011). Finer meshes were used close to the foundation to improve accuracy. A mesh sensitivity study was also carried out by comparing the calculated undrained bearing capacity factors with existing theoretical solutions, i.e., 5.14 for strip (Prandtl 1921; Reissner 1924) and 6.05 for circular foundations with a fully bonded interface (Cox et al. 1961). The mesh used in this study was proven to be able to
provide acceptably accurate results (within an error of 3% and 5% respectively compared with the theoretical solutions).

### 3.2.3 Soil property

The standard Modified Cam Clay (MCC) constitutive model was employed to simulate the soil behaviours (Roscoe and Burland 1968). The soil was initially subjected to a surcharge of 1 kPa to build a uniform geostatic stress field and to maintain the numerical stability that is necessary for the MCC model. Although it differs from typical offshore soil conditions, where the shear strength increases linearly with depth, the uniform soil in this study is appropriate for a comprehensive understanding of the uplift mechanism in shallow foundations. The parameters necessary for the MCC model are summarised in Table 3.1. The values of the parameters represent the typical properties of the kaolin clay used at the Centre for Offshore Foundation Systems, The University of Western Australia (Stewart 1992; Chatterjee et al. 2012, 2013b).

The soil was $K_0$-normally consolidated at the beginning of the analysis. The coefficient of the geostatic stress is

$$K_0 = 1 - \sin \varphi_{tc} \quad (3.1)$$

where $\varphi_{tc}$ is the internal friction angle for triaxial compression tests, which is related to the slope of critical state line

$$M = \frac{6 \sin \varphi_{tc}}{3 - \sin \varphi_{tc}} \quad (3.2)$$

in $p' - q$ space where $p'$ and $q$ are the mean and deviatoric effective stress respectively.

The initial size of the yield locus, $p'_c$, is represented by a pre-consolidation pressure

$$p'_c = \frac{q_0}{M^2 p'_0} + p'_0 \quad (3.3)$$

in which $p'_0$ and $q_0$ are the initial mean and deviatoric effective stresses, respectively. The initial void ratio is expressed as

$$e_0 = e_1 - \kappa \ln p'_0 - (\lambda - \kappa) \ln p'_c \quad (3.4)$$
where

\[ e_1 = e_{cs} + (\lambda - \kappa) \ln 2 \] (3.5)

in which \( e_1 \) is the void ratio at \( p' = 1 \) kPa on the normal compression line. The definitions of \( M, \lambda, \kappa \) and \( e_{cs} \) are provided in Table 3.1. The model assumes an isotropic yield surface and a value of the parameter \( M \) identical in compression and tension. This assumption is justified as it can simulate experimental results (Chen et al. 2012; Li et al. 2014a), which demonstrated that the uplift and compression capacities are identical under undrained conditions, and is considered appropriate for a first study on the development of suction under uplift.

### 3.3 Displacement rate effects on the uplift of shallow foundations

The analysis procedure in this section 1) applies a surcharge onto the soil to create the geostatic stress field and 2) applies displacement-controlled uplift of the foundation with a normalised velocity \( V \) varying between 0.001 and 10000, which is assumed to span over fully drained to fully undrained conditions. The normalised velocity is expressed as

\[ V = \frac{vB}{c_v} \frac{vD}{c_v} \] (3.6)

where \( v \) is the uplift velocity, \( c_v \) is the initial value of the coefficient of consolidation

\[ c_v = \frac{k}{m_v \gamma_w} = \frac{k}{\gamma_w} \frac{(1+e_0)}{\lambda} p'_0 \] (3.7)

in which \( k \) is the permeability of the soil, \( \gamma_w \) is the unit weight of water and \( m_v \) is the volume compressibility that can be determined by the initial void ratio \( e_0 \) and the initial mean effective stress \( p'_0 \).

#### 3.3.1 Resistance and excess pore pressure responses

Figure 3.2 and Figure 3.3 present the responses of normalised resistance \( (q_{mu}/s_{u0}) \) and normalised average excess pore pressure \( (u_{ave}/s_{u0}) \) against normalised displacement for
the uplift of strip and circular foundations at different velocities, respectively. The average excess pore pressure \( \mu_{\text{ave}} \) is calculated as the mean value of the excess pore pressure over the foundation base. Both results on the uplift resistance and average excess pore pressure were normalised by the initial undrained shear strength of the soil at the foundation invert, \( s_{u0} \), determined by

\[
s_{u0} = \frac{M}{\sqrt{3}} \exp\left(\frac{e_1 - (\lambda - \kappa) \ln 2}{\lambda} - \frac{\sigma_0}{\lambda}\right)
\]

for the plane strain condition, or

\[
s_{u0} = \frac{M}{2} \exp\left(\frac{e_1 - (\lambda - \kappa) \ln 2}{\lambda} - \frac{\sigma_0}{\lambda}\right)
\]

for the axisymmetric condition. With values of the parameters given in Table 3.1, the undrained soil strength is calculated as 0.29 kPa for the plane strain condition and 0.25 kPa for the axisymmetric condition, resulting in a ratio of the soil strength to the vertical effective stress, \( s_{u0}/\sigma'_v \), of 0.29 and 0.25 respectively (with the vertical effective stress equals to the surcharge, i.e., \( \sigma'_v = 1 \text{kPa} \)).

Overall, the uplift resistance reduces with the decreasing velocity due to the increasing dissipation of negative excess pore pressures at the foundation invert. The normalised resistance approaches the theoretical bearing capacity factor (i.e., \( N_c = 5.14 \) for strip and 6.05 for circular foundations, respectively) when the normalised velocity \( V > 300 \), potentially indicating undrained soil conditions (Figure 3.2a and Figure 3.3a). The drained soil condition can be inferred by the diminishing suction generation as the normalised velocity \( V < 0.1 \) (Figure 3.2b and Figure 3.3b). It is worthy to note that the uplift resistance under drained conditions is far larger than the real case, where the drained resistance should degrade to zero for flat foundations because of zero suction generation and breakout. The latter is not modelled by the present model, leading to the overprediction of drained uplift resistance due to the soil skeleton being fully bonded to the foundation; thus, it can sustain uplift even if no suction exists. The suction, nevertheless, was not affected by the interface setting and demonstrates more consistently its decreasing magnitude with decreased velocity. This study is significant for demonstrating the pore pressure mechanism in uplifting.
The load-displacement behaviours can be related to the stress paths followed under different drainage conditions; these are schematically illustrated in the critical state space (Figure 3.4). All of the stress paths start at the intersection of the initial yield locus and the $K_0$-Consolidated Line ($K_0$-CL) on the compressive side of $p' - q$ space (point O). They then unload from the initial yield locus on the compressive side to the tensile side due to the uplift of the foundation (note that the direction of shearing has changed). Under the fully undrained condition, the unloading path follows a vertical line due to the restraint of zero volume change (path OA, which is superimposed in $p' - e$ space). Under partially drained conditions, the load paths follow the Unloading-Reloading Line (URL) in the $p' - e$ space, depending on the amount of volume change generated by the dissipation of negative excess pore pressures (paths OB, OC and OD). After reaching the initial yield locus, the stress paths move towards the Critical State Line (CSL) in various directions, e.g., path AA1 for the fully undrained condition and DD1 for the fully drained condition in which the negative excess pore pressures dissipate with the continuing shearing of the soil. As mentioned in Li et al. (2015b), the stress paths could go into the dry side of the CSL, under drainage close to fully drained conditions (paths OCC1 and ODD1), such that the soil behaves as a heavily over-consolidated soil element and the resistance displacement curves demonstrate a softening behaviour following a peak value (see Figure 3.2a and Figure 3.3a). The reduction in stiffness of the load-displacement curves in Figure 3.2 and Figure 3.3 can be related to the decrease of shear modulus as the result of the decrease of mean effective stress when drainage occurs in the soil during the initial stage of uplift (see stress paths OB, OC and OD).

### 3.3.2 Backbone curves

Figure 3.5 presents the backbone curves of uplift resistance and suction generation for both the strip and circular foundations, as a function of the normalised uplift velocity. From these curves, the transition from fully undrained to fully drained conditions can be accurately determined. The curves were created by extracting the maximum values of resistance ($q_u$) and average suction ($u_{ave}$) in Figure 3.2 and Figure 3.3, which were normalised by the corresponding values ($q_{u, ref}$ and $u_{ave, ref}$) under the fully undrained condition that is expected to be mobilised at the highest velocity, i.e., $V = 10000$. 

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The backbone curves for resistance reconfirm that the transition from the fully undrained condition to a partially drained condition occurs at a normalised velocity of approximately 300. This is in agreement with the experimental observations by Li et al. (2014b) (CHAPTER 4) for shallow skirted foundations, and about one order of magnitude higher than that for compression tests (Finnie 1993; Chung et al. 2006; Houlsby and Cassidy 2011). This result could be attributed to the shortening of the drainage paths due to the downward movement of the soil around the foundation or the decreasing coefficient of consolidation during uplift of the foundation. However, the transition from the partially drained to the fully drained condition cannot be precisely predicted by the resistance due to the fully bonded interface setting as mentioned above.

The backbone curves for suction are likely to be more accurate to reflect the effect of soil drainage conditions where the fully undrained condition is achieved at $V = 1000$ and fully drained condition mobilised at $V = 0.001$. The former is higher than that indicated by the resistance related backbone curves because of the localised distribution feature of suction pressures, e.g., at the foundation invert. The circular foundation has a quicker dissipation of suction than the strip foundation due to the unconfined and three-dimensional boundary of the circular foundation. The suction related backbone curves can be fitted by a simple hyperbolic equation:

$$\frac{u_{ave}}{u_{ave, \, ref}} = \frac{1}{1+(V/V_{50})^c} \quad (3.10)$$

where $V_{50}$ is the normalised velocity that corresponds to 50% full suction generation in a local area. Parameter $c < 0$ controls the sharpness of the backbone curves. As shown in Figure 3.5, Equation (3.10) enables good fittings to the variation of negative excess pore pressures with values of the parameters given in Table 3.2.

### 3.4 Effects of preloading and consolidation on the subsequent uplift capacity

The procedure for this section is 1) to apply a surcharge onto the soil to simulate the initial geostatic stress field, 2) to preload the foundation with a proportion of the undrained bearing capacity ($q_p/q_{u, \, ref} = 0.1$ to 0.9) and wait for different periods of time to achieve
different degrees of consolidation ($U = 0.1$ to $1$), and 3) to uplift the strip and circular foundations under fully undrained conditions. The preloading levels and consolidation ranges considered here encompass those that have been used in the centrifuge test by Li et al. (2015a) (CHAPTER 6 ), thus enabling a comparison between the finite element results and centrifuge test data.

As mentioned in Li et al. (2015b), identical capacities should be mobilised by compressing and uplifting a shallow foundation under fully undrained conditions. Therefore, the preloading and its associated consolidation should have the same effect on the subsequent undrained uplift capacity as it did in compression (e.g., Fu et al. 2015; Gourvenec et al. 2014). However, the present study is still meaningful as it allows for a direct comparison with centrifuge results and can form the basis for the development of more sophisticated uplift models in the future.

3.4.1 Consolidation curves

Figure 3.6 illustrates the dissipation of excess pore pressure with a normalised time ($T = c_v t/B^2$, $c_v t/D^2$) at the centre of the foundation after application of preloading for strip and circular foundations. The excess pore pressure ($u$) is normalised by its initial value at the beginning of consolidation ($u_i$). It can be observed that the Mandel-Cryer effect (Mandel 1950; Cryer 1963) for the circular foundation is more obvious than for the strip foundation due to its more concentrated boundaries. With a higher preloading level, the Mandel-Cryer effect is restrained, and the excess pore pressure tends to dissipate more quickly for both foundations. The preloading affects the local dissipation rate of excess pore pressures. However, the overall consolidation curves are almost unaffected by the preloading for either the strip or the circular foundation, as shown in Figure 3.7, in which the consolidation was expressed as the normalisation of the settlement of foundations with its final settlement, i.e., $U = w/w_f$. As such, the settlement curves due to different preloading can be fitted by a unique curve for a specific foundation type and the time to achieve a certain degree of consolidation can be inferred by the fits. The consolidation curves can be interpolated by a simple hyperbolic equation

$$U = w/w_f = \frac{1}{1 + (T/T_{50})^\delta}$$  \hspace{1cm} (3.11)

3.43
in which \( T_{50} \) is the normalised time to achieve 50\% consolidation, and parameter \( \Lambda < 0 \) controls the sharpness of the curves. The values of \( T_{50} \) are 0.39 and 0.04 for the strip and circular foundations, respectively, indicating that the consolidation rate of the circular foundation is approximately one order of magnitude faster than that for the strip foundation.

### 3.4.2 Capacity due to fully consolidated preloading

Figure 3.8 illustrates the normalised resistance versus the normalised displacement curves following full consolidation under different levels of preloading for the circular foundation (similar results can be obtained for the strip foundation). In each case, fully primary consolidation was completed prior to undrained uplift, with larger settlement achieved by a higher preloading level. It can be observed that the increased preloading and the subsequent full consolidation increase the undrained uplift capacity proportionally but has no impact on the stiffness responses of resistance-displacement curves before failure.

The strength gain can be expressed as a ratio of consolidated uplift capacity to a reference capacity without applying preloading and consolidation, i.e., \( R = \frac{q_u}{q_{u, ref}} \). Shown in Figure 3.9 are fully consolidated strength gain ratios plotted against the normalised preloading level for both the strip and circular foundations. As would be expected, the higher preloading level results in the greater increase in uplift resistance. The relationship between the strength gain ratio and the preloading level can be fitted by an exponential function:

\[
R = e^{d \cdot \frac{q_p}{q_{u, ref}} f(U)}
\]  

(3.12)

where \( f(U) \) is a function that takes into account the strength gain due to partial consolidation and equals one for full consolidation and zero without consolidation. Parameter \( d \) is a constant relevant to the foundation geometry and boundary, which shows slight differences in values between the strip and circular foundations, as summarised in Table 3.3.

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3.4.3 Capacity following intermediately consolidated preloading

Figure 3.10 demonstrates the normalised resistance versus normalised displacement curves under intermediate consolidation for the preloading level of \(q_p/q_{u,\text{ref}} = 0.8\) for the circular foundation (again similar results can be obtained for the strip foundation). It can be observed that the uplift capacity was gradually strengthened with an increasing degree of consolidation, with few effects on the stiffness response before failure. The strength gain ratios for both the strip and the circular foundations are illustrated in Figure 3.11 as a function of the degree of consolidation for a range of preloading levels of \(q_p/q_{u,\text{ref}} = 0.1\) to 0.9. Equation (3.12) can be extended to consider the strength gain ratio due to partial consolidation, given the function \(f(U)\) as

\[
f(U) = \frac{U}{m+(1-m)U^n}
\]  

(3.13)

where parameters \(m\) and \(n\) are constants for a specific foundation type, with values summarised in Table 3.3. The function \(f(U)\) varies from 0 to 1, so that the uplift resistance varies between the referenced capacity and the fully consolidated capacity, as indicated in Figure 3.9, with a consolidation degree ranging from 0 to 1. As observed in Figure 3.11, a longer consolidation time (i.e., higher consolidate degree) leads to greater gain of the uplift capacity for a specific preloading level. Equation (3.12) combined with Equation (3.13) has the ability to predict accurately the numerical results, with a slight overestimation observed for the cases with levels of preloading higher than 0.8, where plastic deformation occurs around the foundation edges due to the applied preloading.

Figure 3.12 illustrates the comparison of strength variations with a degree of consolidation between the predictions from Equations (3.12) and (3.13) for the circular foundation and the centrifuge data from Li et al. (2015a), who tested a shallow skirted foundation (embedment ratio = 0.2). It has been indicated by the centrifuge tests that the uplift resistance due to the preloading was controlled by the remoulding and strength recovery of the soil due to preloading and subsequent consolidation. As the remoulding effect was not modelled by the present model, it is thus impractical to try to achieve an exact replication of the test results. However, the trend in the variation of the test results are captured by the present predictions based on the finite element modelling. In addition to the soil remoulding, there are other factors that may cause the differences, such as the
spatial variation of the strength ($s_u/\sigma'_v = 0.21$ in the test compared to 0.25 in the present model, as mentioned in Section 3.3.1) and the permeability of real soil in the centrifuge testing and slightly different geometric configurations between the finite element and centrifuge models.

### 3.5 Concluding remarks

Coupled finite element analyses were conducted to investigate the uplift behaviour of both strip and circular foundations, with a focus on the responses of resistance and suction development affected by either varied uplift rates or preloading and consolidation prior to uplift.

The uplift resistance of shallow foundations is closely related to the suction generation and is highly dependent on drainage conditions in the soil. Both the uplift resistance and suction decrease with reduced uplift velocity. The load-displacement curves start to decay due to either the continuous dissipation of suction as the soil is sheared or to the cross of stress paths onto the dry side of the critical state space. The reduction in the stiffness of load-displacement curves with decreasing uplift velocity is attributed to the reduction of the shear modulus when suction dissipates, which is accompanied by the decrease of mean effective stress in the soil. The transition from fully undrained to partially drained conditions can be inferred by backbone curves of both uplift resistance and suction, with the latter fitted by a simple hyperbolic function. The dissipation in the circular foundation is faster than in the strip foundation, due to the former’s unconfined and three-dimensional boundary condition.

The uplift capacity is increased by both the higher preloading level and longer consolidation time, due to the enhanced soil strength as excess pore pressure dissipates. The effects of preloading and consolidation can be incorporated into simple non-dimensional relations. The predictions from the present study successfully capture the trends of the centrifuge test results on uplift of a shallow skirted foundation. Differences were highlighted due to the inability of the present model to simulate soil remoulding, the non-uniformity of strength and permeability in the real soil, and different geometric configurations.
In the present study, the interface between the foundation and soil is assumed to be fully bonded, leading to large overestimation of the drained uplift resistance. More advance interface settings, e.g., controllable separation, are therefore necessary to simulate uplift processes more realistically. Alternatively, a more sophisticated geometry, such as a skirt configuration, is needed to more directly compare with the centrifuge data.

### 3.6 References


Coupled analysis of shallow foundations subjected to uplift


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Table 3.1 MCC model parameters

<table>
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<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Slope of the critical state line in $p-q$ space, $M$</td>
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</tr>
<tr>
<td>(Internal friction angle for triaxial compression tests, $\varphi_{tc}$)</td>
<td>(23.5°)</td>
</tr>
<tr>
<td>Void ratio at $p = 1$ kPa on the critical state line in $e - \ln p$ space, $e_{cs}$</td>
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</tr>
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<td>Slope of the normal compression line in $e - \ln p$ space, $\lambda$</td>
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</tr>
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<td>Slope of the unloading-reloading line in $e - \ln p$ space, $\kappa$</td>
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</tr>
<tr>
<td>Poisson’s ratio, $\nu$</td>
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</tr>
<tr>
<td>Permeability, $k$ (m/s)</td>
<td>$10^{-9}$</td>
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Table 3.2 Parameters of backbone curves for suction

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<th>Foundation type</th>
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</thead>
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<tr>
<td>Strip</td>
<td>-0.63</td>
<td>1.50</td>
</tr>
<tr>
<td>Circular</td>
<td>-0.68</td>
<td>2.89</td>
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Table 3.3 Parameters for strength gain ratios

<table>
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<th>Foundation type</th>
<th>$d$</th>
<th>$m$</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strip</td>
<td>0.60</td>
<td>0.46</td>
<td>1.89</td>
</tr>
<tr>
<td>Circular</td>
<td>0.64</td>
<td>0.61</td>
<td>2.48</td>
</tr>
</tbody>
</table>
Figure 3.1 Model and mesh configuration
Figure 3.2 Uplift of a strip foundation at different velocities: (a) normalised resistance and (b) nominalised average excess pore pressure at the foundation base versus normalised displacement curves
Figure 3.3 Uplift of a circular foundation at different velocities: (a) normalised resistance and (b) normalised average excess pore pressure at the foundation base versus normalised displacement curves.
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Figure 3.5 Backbone curves of normalised resistance and normalised average suction at the foundation base as a function of normalised velocity
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Figure 3.7 Time history of normalised foundation settlement under preloading
Figure 3.8 Normalised resistance versus normalised displacement curves after fully consolidated preloading (circular foundation)

Figure 3.9 Strength gain ratio due to fully consolidated preloading for strip and circular foundations
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Figure 3.10 Normalised resistance versus normalised displacement curves following intermediately consolidated preloading (circular foundation)
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Figure 3.12 Comparison between the present predictions and centrifuge data: (a) \( q_p/q_u, \text{ref} = 0.2 \); (b) \( q_p/q_u, \text{ref} = 0.4 \); (c) \( q_p/q_u, \text{ref} = 0.6 \); (d) \( q_p/q_u, \text{ref} = 0.8 \)
Chapter 3 Coupled analysis of shallow foundations subjected to uplift
CHAPTER 4  RATE EFFECTS ON THE UPLIFT CAPACITY OF SKIRTED FOUNDATIONS ON CLAY

Abstract

Centrifuge tests have been carried out to investigate the uplift resistance of circular and square skirted foundations under increasing displacement rates in lightly over-consolidated kaolin clay. Uplift loads, displacements and pore pressures at the foundation invert were monitored during testing. Results provide insights into the mechanism of development of suction at the foundation invert and its contribution to the uplift resistance in clay. Backbone curves for both circular and square footings were established, accounting for the soil viscous effect generated at high uplift rate.

Keywords: centrifuge modelling; uplift; skirted foundation; rate effect; clay
4.1 Introduction

The effects of penetration rate on the bearing capacity of foundations have been extensively investigated, notably via the research undertaken on the effect of strain rates on the penetration resistance of soil characterisation tools. Backbone curves expressing the normalised penetration resistance as a function of the normalised velocity \( V = \frac{vD}{c_v} \) (where \( v \) is loading velocity, \( D \) the footing dimension and \( c_v \) the coefficient of soil consolidation) have been well established by numerous penetrometer tests, e.g., T-bar, ball and piezocone (House et al. 2001; Randolph and Hope 2004; Chung et al. 2006; Schneider et al. 2007; Houlsby and Cassidy 2011). It is widely recognized that full undrained behaviours are achieved for a normalised velocity greater than 30, while drained conditions are achieved for a normalised velocity lower than 0.01. Partially drained conditions prevail between these two values. At relatively low velocity, the bearing capacity of penetrometers increases with decreasing loading rate, and is dominated by the drainage conditions in soil. However, at higher velocity, the viscous effects of soil become significant and the penetration resistance typically increases with increasing loading rate by 5% – 20% per log cycle (Einav and Randolph 2005). Lehane et al. (2009) built a full velocity-ranged backbone curve, considering both the effects of drainage conditions and the viscous rate effect of soil.

Similar findings have been obtained from studies focusing on penetration of shallow foundations (Finnie 1993) or spudcan (Erbrich 2005; Cassidy 2012 among others). An important aspect to note is that undrained conditions were also achieved for a normalised displacement velocity higher than 30, although the boundary conditions may somewhat be different between penetrometers, spudcans and shallow foundations.

For fully undrained conditions, a direct analogy between compression and uplift can be made since for fully sealed conditions, uplift and compression capacities are theoretically equal. There are, however, preliminary evidences that undrained conditions are achieved at significantly higher displacement rates in uplift. Lehane et al. (2008) investigated the rate effects on vertical uplift capacity of embedded anchor in clay, finding that the normalised velocity resulting in fully undrained behaviour for footings subjected to uplift are about two orders of magnitude higher than for footings loaded in compression. Chen et al. (2012) also reported that fully undrained conditions were not fully reached at
normalised uplift velocity of ~ 3000 for uplift of mudmats. Both results relate to the enhanced ability for excess pore pressures to dissipate, due to the development of pore pressure relief mechanisms at the soil foundation interface.

There is an evident need to further examine the effect of loading rate on the uplift capacity of skirted foundations, in order to establish the boundaries delimiting drained, partially drained and undrained conditions, and accounting for the combined effect of drainage, viscous effects and pore pressure relief mechanisms.

This paper presents a series of centrifuge tests on both circular and square skirted foundations in kaolin clay. Loads and pressures at the footing inverts were monitored during uplift. Backbone curves were obtained to investigate the effects of soil drainage conditions and soil viscosity on the uplift of skirted foundations, and corresponding failure mechanisms were also investigated.

### 4.2 Experimental set-up

#### 4.2.1 Facility

The experiment was conducted in the drum centrifuge at the Centre for Offshore Foundation Systems (COFS), The University of Western Australia (UWA). The facility enables multiple uplift tests to be conducted within a single soil sample. The ring channel of the centrifuge has an outer diameter of 1.2 m, an inner diameter of 0.8 m and a channel height (sample width) of 0.3 m. An actuator was mounted onto the central tool table, which can provide both vertical and radial movements by driving the servomotors. The central tool table can rotate independently of the outer channel, allowing halt for check or tool change without affecting the soil sample. A complete technical description of this centrifuge is presented in Stewart et al. (1998). All the tests were performed at a centrifuge acceleration of 200g, which means all model linear dimensions are scaled by 200 and all loads by $200^2$ (see Garnier et al. (2007) for detail).
4.2.2 Model and instrumentation

Both circular and square skirted footing models were investigated. Both have overall dimension of $D = 60$ mm (12 m in prototype) in diameter or width (see Figure 4.1). Note that a curved base with curvature radius of 480 mm was designed to match the curved soil mudline in the drum centrifuge and ensure a perfect contact between the footing invert and the soil. This gives 0.94 mm difference between the centre and the edge of the footing. Both models feature a skirt length of $h = 12$ mm (2.4 m in prototype) giving an embedment ratio $h/D=0.2$. Three Pore Pressure Transducers (PPTs) and two Total Pressure Transducers (TPTs) were mounted onto the model inverts to monitor the pore and total pressure variations during testing. A vent was also drilled to allow air to flow through when installing models onto soil at 1g, which was sealed before ramping up centrifuge. The uplift force was measured by a 2 kN capacity load cell attached onto models, which was connected to the centrifuge actuator by a shaft as illustrated in Figure 4.2.

4.2.3 Clay sample preparation and characterisation

Kaolin slurry with 120% water content (twice the liquid limit) was prepared in a vacuum chamber, and then transferred into the centrifuge channel by a hopper at 20g over a fabric drainage layer. Soil experienced consolidation, by ramping the centrifuge up to 200g, which took four days to be completed. The soil height (including the drainage layer at channel bottom) was 150 mm after consolidation. Then a layer of 30 mm was scraped off the soil surface to create a lightly over-consolidated sample with a strength intercept, in order to ensure a good quality contact between the foundation invert and the soil. Reconsolidation at 200g was undertaken overnight before performing model tests. The overall soil height was 120 mm, with a surface curvature of 480 mm matching the curvature of the invert of the models (Figure 4.1).

T-bar tests were carried out before, during and after model tests. The soil undrained shear strength was calculated by normalising the T-bar resistance with a bearing capacity factor of $N_{T-bar} = 10.5$ (Lehane et al. 2009; Low et al. 2010). The soil profile exhibited an undrained shear strength at mudline of $s_{um} = 2.55$ kPa, and a linear increase in strength with depth, of a gradient of $k = 1.21$ kPa/m. This gives a strength heterogeneity ratio
$kD/\sum$ of 5.7. The soil coefficient of consolidation $c_v$ is taken as $\sim 1.5 \text{ m}^2/\text{year}$, relevant to the stress level experienced by the soil underneath the foundation (House et al. 2001).

### 4.2.4 Test programme and procedure

Eighteen displacement control uplift tests were carried out in the drum centrifuge under an acceleration of 200g at soil surface. The test programme is summarised in Table 4.1. All model tests were carried out in different sites spreading over the centrifuge channel with an edge-to-edge spacing of one and half model width.

Each model test experienced four stages of operation as detailed below:

1) The centrifuge was ramped down to 1g to connect the model with the actuator and it was installed at a velocity of $v = 0.05 \text{ mm/s}$ until full contact with the soil surface was achieved. The valve was subsequently closed to seal the foundation.

2) The centrifuge was ramped up to 200g and the channel was filled up with $\sim 30 \text{ mm}$ water so that the models can be fully submerged. Eventually the load cell reached a stable value, which is close to the submerged weight of models.

3) A preload was applied onto the model, equivalent to $\sim 10\%$ of the vertical undrained bearing capacity ($Q_u$) by assuming a bearing capacity factor $N_c = 10$ (Mana et al. 2012). The model was left consolidated under constant preload for about 10 minutes in model scale, resulting in more than 90% of the consolidation to be completed, as demonstrated by the settlement measurements.

4) Uplift was applied with various displacement rates ranging from $v = 0.0005$ to 3 mm/s ($vD/c_v = 0.6$ to 3784.3, see Table 4.1 for details), hence covering four orders of magnitude. During this stage, the uplift load and the total and pore pressures were monitored. Peak values of force and pressure were also extracted for further study.
4.3 Experimental results

4.3.1 Typical measurements and observation of soil failure mechanisms

Figure 4.3 shows typical measurements of net uplift resistance (i.e., excluding the submerged self-weight of the footing) and excess pore pressure (i.e., excluding the hydrostatic pressure) varying with normalised displacement ($w/D$) for both circular and square footings at three uplift velocities of $v = 0.0005$, $0.05$ and $3$ mm/s ($vD/c_v = 0.6, 63.1$ and $3784.3$) separately. The measured uplift resistance is expressed by normalising the measured uplift load by plane area of models ($q_m = Q_m/A$) and only the excess pore pressures monitored by the PPT at the geometric centre of the foundation base plate ($u_{PPT0}$) are presented for illustration due to the similarity in response with the other PPTs/TPTs. Note that the negative values of excess pressure indicate the generation of suction at the footing invert. Overall, the uplift resistance and the corresponding excess pore pressure variations for circular and square foundations shows the similar variations with slightly lower values monitored for the square footing.

At the slowest rate, i.e., $v = 0.0005$ mm/s ($vD/c_v = 0.6$), the net uplift resistance is in the order of $1$ kPa. The uplift resistance is essentially sustained by the friction mobilised at the outside and inside skirt soil interface, which can be verified by the visual observation on the soil failures after uplift (Figure 4.4). The footing extraction only affects the soil around the skirt under such a low rate, as illustrated in Figure 4.4. It is inferred that nearly fully drained conditions are generated at an uplift rate of $0.0005$ mm/s. This is discussed in more details in Section 4.3.2.

At the highest uplift rate, i.e., $v = 3$ mm/s ($vD/c_v = 3784.3$), significant suction is generated at the foundation invert, with development mirroring the variation of uplift resistance for both footing types. This implies that undrained conditions are potentially generated, with the suction sustaining the internal soil plug and developing a reverse end bearing mechanism. This is demonstrated in Figure 4.4, which shows the soil plug heave, associated with downward soil movement around the foundation.

At intermediate uplift rate, e.g., at $v = 0.05$ mm/s ($vD/c_v = 63.1$), suction is generated at the foundation invert, but at a lower magnitude. Accordingly, both suction and skirt wall
frictions are likely to contribute to the uplift resistance. The soil deformation in Figure 4.4 shows plug heave, but to a lesser extent than at an uplift rate of 3 mm/s, demonstrating that partially reverse end bearing mechanism, and partially drained conditions were mobilised.

Also evident in Figure 4.3 is a breakaway point for uplift velocity of 0.05 and 3 mm/s, at which suction is suddenly released, resulting in a significant reduction in uplift resistance. It is noteworthy that the breakaway happens at a lower vertical displacement for the fastest uplift rate and for both footing types ($w/D = 0.07$ to 0.09 at 3 mm/s against 0.13 to 0.15 at 0.05 mm/s). This is explained by the soil deformations as observed in Figure 4.4. At $v = 3$ mm/s, and under undrained conditions, the soil around the outside skirt moves downward, exposing the tip of the skirt to the free water after less uplift displacement than at the intermediate uplift rate.

Most importantly, for both uplift rates, the breakaway occurs after the peak of suction was mobilised. This indicates that any variation in development of suction at the foundation invert is only due to change in drainage conditions. Suction relief mechanisms, as observed by Chen et al. (2012) and Li et al. (2014), are not generated in the present set of experiments. This enables establishment of the drainage boundary defining partially drained and fully undrained conditions for foundation uplift, without interference of suction relief mechanisms.

Figure 4.5 presents the peak of average excess pore pressures $|u_{PPT}|$ (monitored by the three PPTs) normalised by the corresponding peak net uplift resistance ($q_{up}$) as a function of the normalised velocity. The ratio of suction to uplift resistance reaches a plateau from a normalised velocity of approximately 100, where the suction generated contributes to more than 90% of the uplift resistance. This potentially indicates a transition from partially drained to undrained conditions. However, the averaging of pressure across the three PPTs does not enable an accurate determination of the boundary, and a more rigorous determination of the transition can actually be established from Figure 4.6, as discussed in Section 4.3.2.

The normalised velocity delimiting partially drained to undrained conditions in the present case appears to be one order of magnitude smaller than the one observed by
Lehane et al. (2008) and Chen et al. (2012), with values of ~ 5000 and ~ 3000 respectively. It is however approximately one order higher than the value of 30 generally accepted for foundation in compression. This highlights that, in the absence of suction relief mechanism, undrained conditions for uplift are achieved under faster displacement rates than for compression. This is essentially due to the change in boundary conditions and the associated change in drainage path length. In compression, the foundation embeds deeper into the soil, increasing the length water has to travel to reach zones of hydrostatic conditions. In contrast, under uplift, the drainage paths are shortened as the foundation is pulled out of the soil. The shortening is further enhanced by the downward movement of the soils associated with the reverse end bearing mechanism and consequently, undrained conditions for uplift are achieved under a higher displacement rate than for compression. This conclusion is valid for both the square and the circular footings.

4.3.2 Backbone curves

Figure 4.6 summarised the peak uplift resistance normalised by the soil strength at skirt tips ($s_{u0}$) as a function of normalised velocity.

Both circular and square footings demonstrate the similar trend that the uplift resistance increases with increased normalised velocity. However, the peak uplift resistances for circular and square footings differ slightly at a normalised velocity regime from 10 to 100. Under partially drained conditions, the magnitude of suction depends on the drainage path length, which is different for both foundations as they exhibit two different plane configurations.

Further observation from Figure 4.6 indicates that the rate of increase of uplift resistance varies with the normalised uplift velocity. The uplift resistance remains constant at normalised velocity less than 5, where the soil underneath the footings is experiencing drained conditions and the uplift resistance is generated by interface friction. Assuming an interface friction factor of $\alpha = 0.25$, the skirt friction resistance equals the uplift resistance as illustrated in Figure 4.6.

As the normalised velocity increases, the uplift resistance increases rapidly due to the generation of suction at the footing invert, with the failure mode transitioning from skirt
friction to full reverse end bearing mechanism. The uplift resistance increase experiences a transition point around the normalised velocity of 200 as defined in Figure 4.6, where the normalised resistance is slightly higher than the bearing capacity factors $N_c$ ranging from 9.44 to 10.06 reported by Mana et al. (2010) for smooth and rough skirted circular footings in compression, respectively, under undrained conditions, with an embedment ratio of 0.2 and a heterogeneity ratio $kD/s_{um}$ of 5.7. Figure 4.6 provides a more accurate estimation of the partially drained – undrained boundary.

Beyond a normalised velocity of 200, the uplift resistance continues to increase, but at a much slower rate. This increase is associated with soil viscous effects, as demonstrated below.

In order to build a general framework to predict the uplift resistance of skirted foundations under various rates, the uplift resistance is normalised by a reference resistance ($q_{upref}$ which relates to the undrained uplift at the velocity of $v = 0.1$ mm/s or $vD/c_v = 126.1$) and plotted in Figure 4.7 as a function of normalised velocity. This point corresponds to the experimental data closest to the transition point where $vD/c_v = 200$ and is used to construct the backbone curve presented in Figure 4.7.

The backbone curve is established by interpolating the results with a hyperbolic function of the form:

$$\frac{q_{up}}{q_{upref}} = a + \frac{b}{1 + cV^d}$$

(4.1)

where $a$, $b$, $c$, $d$ are parameters for describing the shape of the backbone curve. $a = 0.03$ represents the drained resistance (equivalent to the normalised skirt wall friction), $(a + b) = 1$ represents the undrained resistance at the reference rate, $c = 500$ and $d = -1.65$ are parameters controlling the sharpness of the backbone curve. Equation (4.1) has been widely used to predict the resistance for penetration tests and can only account for the effect of soil drainage conditions (House et al. 2001; Chung et al. 2006). As illustrated in Figure 4.7, Equation (4.1) fails to predict the increase in uplift resistance under fully undrained conditions.
Chapter 4  Rate effects on the uplift capacity of skirted foundations on clay

The enhanced uplift resistance after undrained conditions is explained by viscous effects. Equation (4.1), therefore, can be modified to incorporate a soil viscous effect term expressed as

\[
\frac{q_{up}}{q_{upref}} = (a + \frac{b}{1 + e^{V_d'}}) \cdot \left\{1 + \frac{\lambda}{\ln(10)} \left[\sinh^{-1}(V/V_0) - \sinh^{-1}(V_{ref}/V_0)\right]\right\}
\]

(4.2)

where \( \lambda \), \( V_0 \), \( V_{ref} \) are parameters controlling soil viscous effects. \( \lambda = 0.16 \) is a rate coefficient, which represents the percentage of soil strength increase per log cycle (Randolph and Hope 2004; Einav and Randolph 2005). \( V_0 = 1 \) is a normalised velocity where rate correction start to reduce to zero (viscous effects can be neglected under this rate). \( V_{ref} = 10 \) is a reference normalised velocity where the rate correction term passes through unity.

As demonstrated in Figure 4.7, Equation (4.2), including a viscous effect parameter, is able to predict the foundation uplift capacity over the full range of drainage conditions.

4.4 Conclusions

A series of centrifuges tests were carried out to investigate the effect of the loading rate on the uplift capacity of skirted square and circular foundations resting on kaolin clay.

The uplift rate investigated covered drainage conditions ranging from fully drained to fully undrained conditions. It was observed that fully undrained conditions were achieved at uplift rate approximately one order of magnitude higher than rates associated with undrained conditions for foundations in compression. The reason is the shortening of the drainage path as the foundation is uplifted from the soil and the surrounding soil is dragged downwards with the development of a reverse end bearing mechanism.

At this higher velocity (compare to foundation in compression), the soil strength is enhanced by viscous effect. This results in an uplift capacity, when undrained conditions are reached, which is higher than the bearing capacity for identical drainage conditions for compression. Note that strength increase due to consolidation post installation has not been accounted for and prevents the determination of accurate bearing capacity factors.
for uplift conditions. This will be investigated further in the second stage of this programme.

4.5 References


Chapter 4  Rate effects on the uplift capacity of skirted foundations on clay


Table 4.1 Test programme

<table>
<thead>
<tr>
<th>Model shape</th>
<th>Velocity, $v$ (mm/s)</th>
<th>Normalised velocity, $vD/c_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circular/Square</td>
<td>0.0005</td>
<td>0.6</td>
</tr>
<tr>
<td>Circular/Square</td>
<td>0.001</td>
<td>1.3</td>
</tr>
<tr>
<td>Circular/Square</td>
<td>0.005</td>
<td>6.3</td>
</tr>
<tr>
<td>Circular/Square</td>
<td>0.01</td>
<td>12.6</td>
</tr>
<tr>
<td>Circular/Square</td>
<td>0.05</td>
<td>63.1</td>
</tr>
<tr>
<td>Circular/Square</td>
<td>0.1</td>
<td>126.1</td>
</tr>
<tr>
<td>Circular/Square</td>
<td>0.5</td>
<td>630.7</td>
</tr>
<tr>
<td>Circular/Square</td>
<td>1</td>
<td>1261.4</td>
</tr>
<tr>
<td>Circular/Square</td>
<td>3</td>
<td>3784.3</td>
</tr>
</tbody>
</table>
Figure 4.1 Sketch of circular and square skirted foundation models (mm in demission)

Figure 4.2 Model layout in drum centrifuge
Figure 4.3 Measurements of force and excess pore pressure versus displacement curves for (a) circular and (b) square footings.
Figure 4.4 Soil failure mechanisms after pulling out for (a) circular and (b) square footings

Figure 4.5 Suction generation normalised by uplift resistance
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Figure 4.6 Uplift resistance normalised by soil strength at skirt tips

Figure 4.7 Backbone curves for uplift resistance
Rate effects on the uplift capacity of skirted foundations on clay
CHAPTER 5 OBSERVATIONS ON THE FAILURE MECHANISM OF SHALLOW FOUNDATIONS SUBJECTED TO UPLIFT

Abstract

Particle Image Velocimetry (PIV) analyses were performed in the centrifuge to investigate the uplift failure and breakout mechanisms of shallow foundations resting on lightly over-consolidated kaolin clay. The effects of skirt length and displacement rate are considered. Excess pore pressure underneath the foundation was monitored to provide information on the soil drainage condition and the breakout of foundations. Results show that a reverse Hill failure mechanism is mobilised in both skirted and flat foundations under fully undrained soil conditions. With increasing soil drainage in the clay, the size of the failure mechanism can be reduced or the failure mechanism can transform into a combined Hill-sliding type. The breakout of the skirted foundation can be attributed to either the decrease of effective stress or dramatical boundary changes such as the loss of embedment or the formation of cracks around the foundation, depending on the soil drainage condition in the clay.

Keywords: centrifuge modelling; Particle Image Velocimetry (PIV); shallow foundation; uplift; clay
5.1 Introduction

Particle Image Velocimetry (PIV) analysis is a nonintrusive technique originally developed to measure flow fields and relevant features in fluid mechanics (Adrian 1991). The analysis is based on a series of image pairs taken on the flow seeded with special particles to visualise the deformation field. The image pairs are then divided into a grid of patches in pixels. The displacement and velocity vectors of identified particles in the patches are calculated by correlating them between two images using correlation functions. With improving accuracy and precision, PIV analysis has been applied into geotechnical modelling to measure soil deformations and further implemented into centrifuge environment (e.g., White et al. 2003, 2005; Stanier and White 2013).

PIV has already been applied to investigating the soil flow mechanism around foundations during uplift. Typical examples include Purwana (2006) on spudcan extraction, Cheuk et al. (2008) on upheaval of a buried pipelines and Tian et al. (2013) on tension of buried plate anchors. All of these are for deep or buried foundations and few studies concentrate on shallow foundations. Mana et al. (2012) investigated the soil flow mechanism around shallow skirted foundations. They compared the failure mechanisms of foundations between compression and uplift under fully undrained conditions. A reverse Hill failure mechanism (Hill 1950) was observed in uplift compared to a Prandtl failure mechanism (Prandtl 1921) in compression. To the authors’ knowledge, no study has revealed the soil failure mechanism of shallow foundations under partially drained conditions. On the other hand, Li et al. (2014b) performed full model (not PIV) tests on the uplift of shallow skirted foundations under a range of soil drainage conditions. It was found that the decrease of uplift resistances with decreasing velocity is related to the transition of soil drainage from undrained to drained condition and the associated reduction of suction at the foundation invert. Nevertheless, the soil failure mechanism in the transition regime is still not clear.

Furthermore, the breakout, which features an abrupt drop of resistance and suction during uplift, plays an important role in controlling the extracting process for offshore shallow foundations. This phenomenon has been studied theoretically by Foda (1982), Mei et al. (1985) and Sawicki and Mierczyński (2003) and numerically by Zhou et al. (2008). They
divided the breakout into a two-stage or three-stage process, based on the assumption that the effective stress at the interface between the foundation and soil governs the breakout. Numerous model experiments (Byrne and Finn 1978; Das 1991; Chen et al. 2012; Li et al. 2014a) and field tests (Bouwmeester et al. 2009) also indicated the importance of the breakout in various uplift events. Nevertheless, there is lack of analysis on visualising the breakout of shallow foundations.

In the present study, PIV analysis has been performed in the centrifuge to investigate the uplift failure and breakout mechanism of shallow skirted and flat foundations. The uplift velocity was varied, extending the study to partially drained conditions. The breakout of the foundations was visualised and discussed briefly.

5.2 Experimental set-up

5.2.1 Centrifuge and PIV apparatus

The drum centrifuge at the Centre for Offshore Foundation System (COFS) at The University of Western Australia (UWA) was employed to perform the tests (Stewart et al. 1998). This centrifuge has a U-shape ring channel with outer diameter of 1.2 m, inner diameter of 0.8 m and channel height of 0.3 m. A tool table is mounted at the centre of the drum, which can rotate independently of the outer channel and thus allow halt for check or tool change during testing. An actuator attached onto the central tool table provides either load or displacement control in vertical and radial direction by driving the servomotors. The centrifuge is equipped with both wired and wireless data acquisition system developed in-house at UWA, which can achieve real-time data transmission at the frequency of 10 Hz (Gaudin et al. 2009). All the tests were operated at a centrifuge acceleration of 200g at the top of the soil sample. Details of the scaling laws can be found in Garnier et al. (2007).

PIV tests were conducted in separated strongboxes that can be screwed into the drum channel (Figure 5.1). The strongboxes are modular and can be assembled with dimension of 80 (width) $\times$ 160 (height) $\times$ 256 (length) mm. During each test, the side plate facing
the camera was replaced with a transparent Perspex window that allows the soil flow mechanism to be observed and captured by the camera in front of the window. The PIV system is equipped with a small format digital camera and a pair of Light-Emitting Diode (LED) panels, both of which can be controlled in flight by software developed in-house. The basic set-up of the PIV system is illustrated in Figure 5.1. More details on the application of PIV technology in the drum centrifuge can be found in White et al. (2005) and Stanier and White (2013).

### 5.2.2 Model and instrumentation

Two half-square model foundations were fabricated for the present study (Figure 5.2). Both have overall width of $D = 60$ mm (12 m prototype), one without skirt (flat) and the other with skirt length of $h = 12$ mm (2.4 m prototype), which lead to a skirt length ratio of $h/D = 0$ and 0.2 separately. These dimensions were chosen to keep consistency with previous studies, particularly Mana et al. (2012) and Li et al. (2014b). Top plates were 10 mm in thickness and the skirt wall 3 mm in width ($\Delta t/D = 0.05$), thus ensuring enough strength and better visualization during testing. Black foam was glued all over the symmetric side of the half models, which was squeezed against the transparent window to provide a seal to ensure no water and soil can flow through. Meanwhile, a special shaft, with the same width as the strongbox, was fabricated. This enabled support from the back plate of the strongbox and thus helped further enhance the seal condition (see also Mana et al. (2012)). One Pore Pressure Transducer (PPT) was instrumented through the top plate of each model to monitor the pore pressure variations at foundation inverts from installation to uplift. One vent on the model plate was drilled to allow air to flow through during installation, which was then sealed by a plastic valve after installation. A load cell was attached onto the top of the shaft to record reaction forces from the actuator during testing.

### 5.2.3 Clay sample preparation and T-bar test

Kaolin slurry with 120% water content (twice the liquid limit) was poured into the pre-fixed strongboxes in the drum channel at 20g. Each box was topped up 2 or 3 times to
accommodate the reduction in volume due to consolidation and settlement. After topping up, the clay was left for consolidation at 200g for a further four days. This final soil height was estimated to be around 120 mm.

T-bar tests were conducted during consolidation to verify if full consolidation was achieved, that is, if a linear soil strength profile was produced. After full consolidation, all the boxes were taken out from the channel and preserved in the water for further use. For each sample, the top 10 mm of clay was scraped off to make a flat surface and Lightly Over-Consolidated (LOC) clay with final soil height of 110 mm. One side plate of the strongbox was removed, and black dyed sand passed through a 300 μm sieve was seeded randomly onto the side surface. This increased the contrast of the soil, a necessity for high quality PIV analysis. The Perspex window was screwed on the side face of the strongbox, and then the box was put back into the channel for further T-bar and PIV tests.

T-bar tests were performed in each strongbox at a standard penetration rate of 1 mm/s to obtain the undrained shear strength (Stewart and Randolph 1991, 1994). Cyclic tests were also included in each T-bar test to correct the drift of the data (Randolph et al. 2007). The penetration resistance was first normalised by a constant bearing factor of 10.5 (Randolph and Houlsby 1984; Low et al. 2010), and then was corrected at shallow depth with lower bearing factors (White et al. 2010). Figure 5.3 summarises the corrected undrained shear strength profile of the clay at the top 50 mm (10 m prototype) of clay for five strongboxes. It can be seen that the strengths for all the boxes are quite uniform and follow a linear variation with depth. The overall strength can be approximated by a linear function with an intercept of $s_{um} = 0.92$ kPa at the top of the clay and a gradient of $k = 1.33$ kPa/m. This is equivalent to a heterogeneity ratio of $kD/s_{um} = 17.3$.

### 5.2.4 ASR calibration

Stanier and White (2013) recommended that the density of the dyed sand seeded onto the surface of the clay sample needed to be calibrated before formal PIV tests, in order to obtain an optimised contrast. The density of the dyed sand can be expressed as an Artificially Seeding Ratio (ASR) varying between 0 (i.e., no seeded sand) and 1 (i.e., fully saturated by seeded sand). During calibration of the tests of this study, the dyed sand was
rained onto the exposed plane of clay sample in stages until the plane was fully saturated with the texture. The images with different seeding densities were captured and analysed by PIV method by comparing an image with its duplicate at different patch sizes of the image, i.e., $L = 25, 50$ and $100$ pixel respectively. As suggested by Stanier and White (2013), a standard error due to different seeding densities can be derived, which changes as a concave parabolic function of ASR, and its minimum value corresponds to the optimised ASR. Figure 5.4 shows the standard error varying against ASR from the present calibration. Although the dyed sand was not evenly seeded, it can still be confirmed from the trend line of the present results that the optimised ASR that gives the best constant is around 0.45. This value was used as benchmark for the formal model tests.

### 5.2.5 Test programme and procedure

Five model tests were carried out as summarised in Table 5.1. Influential factors considered include: i) uplift velocity $v = 0.001, 0.05$ and $1$ mm/s. This is equivalent to a normalised velocity $vD/c_v = 1.3, 63.1$ and $1261.4$ by assuming an average coefficient of consolidation $c_v = 1.5$ m$^2$/year, which corresponds to the stress level experienced in the clay at the foundation level (House et al. 2001); ii) skirt length ratio $h/D = 0$ and $0.2$. Camera frame rate varies between 1 and 10 frames per second (fps) depending on the displacement rates. This rate is capable of recording a minimum displacement interval of 0.001 mm for the lowest velocity test and 0.1 mm for the highest velocity test, thus enabling a close observation of the interface behaviours. This is much improved compared to previous studies performed in the UWA centrifuges (e.g., White et al. 2005; Hossain and Randolph 2010; Mana et al. 2012).

Each model test was performed in a separated strongbox, which involves vented installation, sealing the drainage hole at the top plates, compressive preloading and uplift at different velocities, as illustrated in Figure 5.5 with the measured resistance monitored by the load cell.

1) The foundation was installed at 1g at a sufficiently slow displacement rate, i.e., 0.05 mm/s, to ensure that the foundation kept contact against the window. The foundation was driven into the sample until full contact between the clay surface and the
foundation base was achieved, which was confirmed by the load cell and pore pressure monitoring during installation and also the visual observation through CCTV cameras. Note that the air valve on the top plate was kept open during installation and was closed after installation.

2) Ramp centrifuge up to 200g and then wait for ~ 30 minutes to allow the clay to settle and any gap between the Perspex window and the clay to be fully closed. Water was then placed on the sample to create an underwater environment.

3) Apply a fraction of compressive preload to ensure full contact between the clay and the foundation base. Wait overnight under constant load to allow the pore pressure underneath the foundation to stabilise, which can be implied by the PPT monitoring and the settlement of the foundation.

4) Uplift the model at different velocities with a displacement-controlled method (see Table 5.1) until the foundation or the skirt tip was clear of the clay surface.

5.3 Experimental results

5.3.1 Interpretation of excess pore pressure responses

Figure 5.6 shows the responses of excess pore pressure (excluding the hydrostatic pressure) monitored by the PPT at the base of the foundation. The negative excess pore pressures suggest the generation of suction at the interface between the foundation base and the top of the clay. For the skirted foundation \((h/D = 0.2)\), the excess pore pressure changes were well captured by the PPT (Figure 5.6a). With decreased velocity, the magnitude of generated excess pore pressure reduces. From test T1 to T2, the peak of excess pore pressure reduces up to 16% with velocity decreasing by a factor of 20, primarily due to the soil viscous effects rather than the drainage condition of the soil. The reason is that in both tests T1 and T2, the foundation broke out at a similar distance of \(w/D \sim 9\%\) (as suggested by grey colour filled circles in Figure 5.6a), indicating that in both tests the clay was almost under fully undrained conditions and the breakout of foundation was unaffected by the slower rate of uplift. The further reduction of excess
pore pressure in test T3 compared to tests T1 and T2, arose from the partially drained condition in the clay as indicated by the earlier breakout at $w/D \sim 5\%$ (see detailed discussion in Section 5.3.3). It is noteworthy that the drainage condition at a specific velocity seems to be shifted compared with the parallel full model tests in Li et al. (2014b), attributed to their different boundary conditions (e.g., models were halved in PIV tests). The distances to the peak of the excess pore pressures are around 2.5\% of the width for all three tests on the skirted foundation.

As shown in Figure 5.6b, the excess pore pressures variations could not be accurately monitored for the flat foundation ($h/D = 0$). However, the little spike monitored by the PPT in test T4 potentially suggests that the peak resistance and the breakout of foundation occurred at $w/D \sim 0.3\%$.

It is noted that excess pore pressures are used here as a default for uplift resistance. This is because that resistances monitored by the load cell can be increased from that of simply soil resistance owing to the side frictional forces between the Perspex window, the back plate and the foundation and its attached shaft. Therefore, it is more reliable to monitor pore pressure variations underneath the foundation, which should not be affected by the friction forces.

### 5.3.2 Observations on the failure mechanism

Figure 5.7 illustrates the displacement vectors of the clay normalised by the foundation displacement at the moment when the peak resistance was mobilised as suggested by the PPT monitoring in Figure 5.6. The normalised displacements of the clay are also presented as contour plots in Figure 5.8. Overall, a certain volume of the clay underneath the foundations moves upward with foundation regardless of the skirted configurations, which indicates the generation of suction pressures although in different levels. The clay around the foundation flows towards the bottom of the foundation to maintain continuity of kinematic failure mechanisms.

For the skirted foundation ($h/D = 0.2$), a clear reverse Hill type failure mechanism was observed in tests T1 and T2, as sketched in the left side of Figure 5.9a and also observed
by Mana et al. (2012). The movement of the soil plug incorporated by the skirt was affected by the progressive soil flow mechanism (Mana et al. 2012). The failure mechanism in test T2 is slightly smaller than that in test T1, which could potentially suggest that the fully undrained condition has not been reached in test T2 (although it could be interpreted as being close to it). In test T3, the soil movement demonstrates a failure mechanism combining Hill failure and sliding along the skirt as partially drained condition in the clay governs the uplift, as observed in Figure 5.8c and sketched in Figure 5.9b. The magnitude of the displacement in the clay along the skirt differs from the foundation displacement by up to 40% (Figure 5.8c), indicating that a significant sliding occurred along the interface between the skirts and the surrounding clay due to the dissipation of suction pressures, although the overall failure mechanism is still a Hill type mechanism. Due to the partially drained conditions, the extent of the failure mechanism in test T3 is much smaller than that of tests T1 and T2.

The Hill type failure mechanism was also observed in both tests T4 and T5, as observed in Figure 5.7d and e, and Figure 5.8d and e. As expected, the flat foundation demonstrates much shallower failure mechanism compared to the skirted foundation (Figure 5.9a). The Hill failure mechanism in test T4 seems to be disturbed by a higher level of preloading prior to uplift (thus more significant embedment and larger volume recovery during uplift). The higher frame rate has proven to be able to capture the failure mechanism for surface foundations where the operational distance is rather short, e.g., only ~ 0.3% of foundation width as mentioned above. However, due to limited cases, the rate effects on the failure mechanism of flat foundations could not be explicitly interpreted.

5.3.3 Observations on the breakout mechanism

The breakout of flat foundation was not properly observed due to the rather short operational distance. However, this phenomenon could be captured accurately for the skirted foundation. As shown in Figure 5.6a, the breakout is clearly indicated by the abrupt drop of suction pressure after a peak. This can be attributed to either the decrease of effective stress at the interface of the foundation base and the top of the clay or the boundary changes during uplift such as the loss of embedment and the formation of cracks around the skirt of the foundation.
Chapter 5  Observations on the failure mechanism of shallow foundations subjected to uplift

Figure 5.10 illustrates three different stages leading to breakout during uplift of the skirted foundation under fully undrained and partially drained conditions (corresponding to tests T1 and T3 respectively). For both cases, the foundation was in full contact with the clay with good seal condition maintained at the beginning of uplift. As uplift is applied, the clay beneath the foundation was lifted up in test T1 due to the fully mobilised suction, whereas in test T2 the clay underneath the foundation began to slide along the skirt. This indicates that under undrained uplift, the suction is fully sustained before breakout, and the breakout is caused by the loss of embedment or formation of cracks where the water can ingress into the foundation bottom in a very short time, resulting in the disturbance of the clay and the blurring of water in the gap, as observed in the stage III of Figure 5.10a. However, under partially drained uplift, both the decrease of effective stresses and boundary changes may contribute to the breakout (Figure 5.10b). First, the gap forms and continues to grow during uplift due to the drainage of water from the far field to the foundation bottom. Second, the clay around the foundation moves downwards, leading to the shortening of skirt embedment and formation of cracks in the clay. Both effects result in the foundation subjected to partially drained uplift breaking out earlier than that for fully drained uplift (i.e., breakout distance $w/D \sim 5\%$ compared to $w/D \sim 9\%$ as demonstrated in Figure 5.6).

Figure 5.11 and Figure 5.12 show the normalised displacement vectors and the corresponding contours in the clay immediately after breakout, respectively. The effects of soil drainage on breakout can be demonstrated. Firstly, the clay surface profile for undrained test T1 is likely to change more dramatically than the partially drained test T3. After breakout, the suction is totally lost in test T1 and the clay underneath the foundation settles under its self-weight. However, there seems to be a certain amount of suction existing around the skirt tip in test T3 after breakout due to the less significant boundary changes.

5.4 Conclusions

PIV technology was employed to investigate the failure and breakout mechanisms of skirted and flat foundations subjected to uplift in lightly over-consolidated kaolin clay.
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The rate of uplift varies to investigate the effect of soil drainage conditions on uplift failure and breakout mechanisms.

A Hill type failure mechanism was observed for uplift of both the skirted and flat foundations under fully undrained conditions. With increasing drainage, the Hill failure mechanism shrinks in size or the failure mechanism transforms into a combination of Hill and sliding type, depending on the level of partial drainage occurring in the clay. A rather shallower Hill failure mechanism is mobilised in the tests for the flat foundation.

The breakout is indicated by the loss of suction pressure underneath the foundation during uplift. During undrained uplift, the breakout of the foundation is controlled by the dramatical boundary change, such as the loss of embedment and the formation of cracks, instead of the decrease of effective stress at the interface of the foundation and clay. Both factors, however, contribute to the breakout of foundations under partially drained uplift.

5.5 References


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### Table 5.1 Summary of PIV tests

<table>
<thead>
<tr>
<th>Test name</th>
<th>Skirt length, $h/D$</th>
<th>Uplift velocity, $v$ (mm/s)</th>
<th>Normalised velocity, $vD/c_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>0.2</td>
<td>1</td>
<td>1261.4</td>
</tr>
<tr>
<td>T2</td>
<td>0.2</td>
<td>0.05</td>
<td>63.1</td>
</tr>
<tr>
<td>T3</td>
<td>0.2</td>
<td>0.001</td>
<td>1.3</td>
</tr>
<tr>
<td>T4</td>
<td>0</td>
<td>1</td>
<td>1261.4</td>
</tr>
<tr>
<td>T5</td>
<td>0</td>
<td>0.05</td>
<td>63.1</td>
</tr>
</tbody>
</table>
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Figure 5.1 PIV layout in the drum centrifuge

Figure 5.2 Models and instrumentations
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Figure 5.3 Soil undrained shear strength profiles

Figure 5.4 Standard error as a function of ASR
Figure 5.5 Time history of load resistance monitored by load cell (test T3)
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Figure 5.6 Excess pore pressure responses monitored by PPTs
Figure 5.7 Soil displacement vectors normalised by foundation displacement (vectors scaled up by a factor of 10)
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Figure 5.8 Normalised soil displacement contours (intervals of 10% of foundation velocity)
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Figure 5.9 Sketch of failure mechanisms during uplift

Figure 5.10 Different stages to breakout during uplift (UD – Undrained, PD – Partially Drained)
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Figure 5.11 Soil velocity vectors normalised by foundation displacement after breakout (vectors scaled up by a factor of 10; UD – Undrained, PD – Partially Drained)

Figure 5.12 Normalised soil velocity contours after breakout (intervals of 10% of foundation velocity; UD – Undrained, PD – Partially Drained)
Chapter 5  Observations on the failure mechanism of shallow foundations subjected to uplift
CHAPTER 6  EFFECTS OF PRELOADING AND CONSOLIDATION ON THE UPLIFT CAPACITY OF SKIRTED FOUNDATIONS

Abstract

Centrifuge tests investigating the effect of compressive preloading and consolidation on the subsequent uplift capacity of shallow skirted foundations that rest on lightly over-consolidated kaolin clay are reported. Both the uplift resistance and total/pore pressures at the foundation invert were monitored during the installation, consolidation and uplift. The results show that uplift performed immediately after the preloading generates a lower capacity due to the remoulding of the clay and the reduction in soil strengths that result from the application of preloading. The soil strength can be recovered and eventually enhanced by the subsequent consolidation process. A theoretical framework, where the variation of the soil operative strength with preloading and consolidation is interpreted using the critical state concept, is proposed. The framework is verified through retrospective prediction of the uplift resistance of the centrifuge model tests with good agreement for a wide range of levels of preloading and degrees of consolidation demonstrated. It will allow for prediction of tensile capacity of deep-water foundations in design.

Keywords: centrifuge; skirted foundation; kaolin clay; preloading; consolidation; remoulding
6.1 Introduction

Skirted foundations are commonly used offshore to support fixed platforms or as subsea manifolds and terminations in soft cohesive soils. They consist of a top plate and a peripheral thin-walled skirt, which embeds the foundation into deeper and stronger soils and provides additional resistance to horizontal loading. Depending on their applications, skirted foundations may be subjected to tensile loads, which need to be resisted during the operation period of the foundation or minimised if removal is required at the decommissioning of the project. The resistance to tensile loads, or uplift, is a function of the soil characteristics and foundation geometry as well as the loading history. During installation and operation, skirted foundations experience preloading due to self-weight and potentially due to ballasting or active suction installation. This preloading results in excess pore pressures, which subsequently dissipate to increase the operative shear strength of the soil and enhance the uplift capacity of the foundation.

The phenomenon of strength increase from consolidation is well characterised and has been recently investigated to predict increase in the foundation bearing capacity as a function of (i) the level of preloading applied (defined as the ratio to the un-preloaded ultimate undrained bearing capacity), and (ii) the degree of consolidation achieved. Field data (Lehane and Jardine 2003) and centrifuge data (Lehane and Gaudin 2005; Fu et al. 2015) indicate an increase in the bearing capacity of up to ~ 40% that depends on the over-consolidation ratio of the soil. Numerical analyses using the Modified Cam Clay (MCC) model confirmed (Zdravković et al. 2003) and extended (Bransby 2002; Gourvenec et al. 2014; Fu et al. 2015) these results to quantify the increase in the bearing capacity with the degree of consolidation.

Parallel studies have recently investigated the parameters that affect the uplift capacity of skirted foundations. Chen et al. (2012) conducted centrifuge tests that measured the uplift capacity as a function of the foundation skirt length and the uplift rate. They demonstrated that the uplift resistance directly correlated with the development of suction at the mat invert and that fully undrained conditions (characterised by a full reverse end bearing mechanism) were achieved at normalised uplift velocities up to three orders of magnitude higher than those usually considered for shallow foundations in compression. Li et al.
(2014a) investigated means of reducing the uplift resistance to facilitate foundation removal by introducing perforation on the mat and eccentricity in the load. The results demonstrated that both methods reduce the uplift resistance by decreasing the suction pressures generated at the foundation invert.

Both studies highlighted the important role played by the pore pressure in the soil underneath the foundation in the development of the uplift resistance. Like the bearing capacity, the loading history of the foundation and notably the excess pore pressure field resulting from preloading are expected to affect the uplift resistance. The present study aims to quantify and predict the uplift resistance of skirted foundations as a function of the level of preloading experienced and the degree of consolidation achieved. Centrifuge tests were performed on a circular skirted foundation with a prototype diameter of 9 m and an embedment ratio of 0.2. The foundation was pulled out under undrained conditions after active preloading of varying magnitudes was applied and maintained over increasing periods of time. The resulting uplift capacity is discussed, and a theoretical framework is proposed to depict the changes in the operative shear strength as a function of the preloading parameters.

### 6.2 Experimental set-up

#### 6.2.1 UWA drum facility

The drum centrifuge at the Centre for Offshore Foundation Systems (COFS), The University of Western Australia (UWA), was employed to perform the tests (see Stewart et al. (1998) for technical descriptions). The centrifuge consists of a round U-shaped channel with an outer diameter of 1.2 m, inner diameter of 0.8 m and channel height (equivalent to soil sample width) of 0.3 m, which is sufficiently spacious to enable multiple tests to be conducted within one soil sample. A tool table is aligned at the centre of the drum, which can be stopped during testing to operate on or change the model, leaving the channel spinning independently. An actuator is attached to the centre of the tool table to provide either load or displacement control in the vertical and radial directions. All tests were performed under an acceleration level of 150 times the level of
gravity (150g). A comprehensive description of the scaling laws for centrifuge modelling was developed in Garnier et al. (2007).

### 6.2.2 Model foundation and instrumentation

The skirted circular model foundation has an outer diameter of $D = 60$ mm, skirt length of $h = 12$ mm and skirt tip thickness of $\Delta t = 1$ mm (Figure 6.1). This configuration is equivalent to a prototype diameter of 9 m under an acceleration of 150g, a skirt length ratio of $h/D = 0.2$ and skirt tip thickness ratio of $\Delta t/D = 0.017$. The foundation was instrumented with two Total Pressure Transducers (TPT1 and TPT2) and three Pore Pressure Transducers (PPT0, PPT1 and PPT2, see Figure 6.1 for configuration). Under undrained behaviour, both transducers monitor changes in the pore pressure at the foundation invert. The axial load was monitored by a 2 kN load cell located on the top of the foundation mat and connected to the loading shaft. A drainage hole, equipped with a mechanical valve, enabled the in-flight installation of the foundation skirt. The valve was closed as foundation invert contacted with the soil surface. The experimental setup is presented in Figure 6.2.

### 6.2.3 Clay sample preparation and characterisation

The soil sample was prepared from dry kaolin clay mixed with water at twice the liquid limit (120%) in a vacuum chamber. The slurry was transferred into the drum channel at an acceleration level of 20g over a layer of fabric to enable two-way drainage. The sample was allowed to consolidate for over four days at the targeted acceleration level of 150 g before a 3 mm layer of clay was scraped off the top to generate an evenly elevated surface. This process produced a Lightly Over-Consolidated (LOC) clay sample that was 140 mm high.

T-bar cyclic tests were performed at a velocity of $v = 1$ mm/s to measure the undrained shear strength of the sample before and after the skirted foundation tests. Cyclic penetration-extraction loading sequences were included in each test to correct the drift of the T-bar resistance (Randolph et al. 2007). The undrained soil strength was inferred using a constant bearing capacity factor of 10.5 (Low et al. 2010), which was corrected at the
Chapter 6  Effects of preloading and consolidation on the uplift capacity of skirted foundations

shallow embedment to account for the near-surface transition from a heave failure mechanism to a full-flow mechanism and for the soil buoyancy (White et al. 2010). The resulting undrained shear strength profiles are presented in Figure 6.3a in prototype dimensions. The sample exhibits strength profiles that are reasonably homogeneous with space and time. Over the depth of interest (e.g., corresponding to the soil likely mobilised by the foundation compression and uplift), the soil strength profiles can be fitted using a linear profile with a mudline strength of $s_{um} = 0.79$ kPa and a gradient of $k = 0.87$ kPa/m. This fitting yields a heterogeneity ratio of $kD/s_{um} = 9.9$, where $D$ is the diameter of the foundation. The degradation of the soil strength due to the cyclic penetration of the T-bar is plotted in Figure 6.3b. An ultimate degradation ratio of $\delta_{rem} = 0.4$ is inferred (equivalent to a soil sensitivity of $S_r = 2.5$), which is consistent with the value obtained by Mana et al. (2013) for the same type of kaolin clay.

The Over-Consolidated Ratio (OCR) profile can be derived from the variation of the vertical effective stresses before and after scraping using an average unit weight of $\gamma = 6$ kN/m$^3$, which was measured by taking core samples after testing. The profile is plotted in Figure 6.3a and indicates an OCR of 1.25 at the skirt tip level.

Additional T-bar tests were performed to estimate the coefficient of consolidation of the LOC clay. The tests featured twitch sequences, for which the penetration rate was progressively halved from $v = 3$ mm/s to 0.0117 mm/s over one diameter (1d) or two diameter (2d) intervals, where $d$ is the diameter of the T-bar penetrometer (House et al. 2001). A constant velocity ($v = 0.375$ mm/s) test was also carried out and served as a reference for undrained conditions. Figure 6.4a shows the penetration resistance profiles for the twitch tests and the reference test. As the penetration velocity decreases below 0.375 m/s, the penetration resistance continuously increases, indicating partially drained conditions. A backbone curve was constructed as the normalised resistance, $q_{T\text{-bar}}/q_{T\text{-bar}}(v = 0.375$ mm/s), which varied with the normalised velocity, $V = vd/c_v$, where $d$ is the diameter of the T-bar and $c_v$ is the coefficient of the consolidation of the soil. The experimental curve was then fitted to an established backbone curve presented by House et al. (2001) by adjusting the value of $c_v$. The best fit, which was obtained using a least square method, yielded a coefficient of consolidation, $c_v$, of 3.28 m$^2$/year, which represented the average value along the tested depth (Figure 6.4b).
6.2.4 Test programme and procedure

Eighteen skirted foundation model tests were performed, as summarised in Table 6.1. The first two tests (B1 and B2) were used as a reference to establish the ultimate undrained bearing and uplift capacities, \( q_{u-c} \) and \( q_{u-u} \), of the foundation without preloading applied prior to the compression and uplift of the foundation. In the subsequent tests, different levels of preloading, \( q_p/q_{u-c} \), were applied and sustained over increasing consolidation times, \( t_m \), as detailed in Table 6.1.

The test procedure for the uplift tests is illustrated in Figure 6.5a (test T3-3) and includes the following three stages:

1) **Installation**: The foundation skirt was penetrated in flight at a displacement-controlled model rate of 0.05 mm/s, with the valve opened to allow the water trapped within the skirt to drain. At the end of the installation stage, the foundation was maintained in place for 5 minutes, which is the time required to close the valve and program the preloading sequence.

2) **Preloading**: Compressive preloading was applied under load control at the targeted level, \( q_p/q_{u-c} \), and maintained over a period \( t_m \). The total and pore pressures at the foundation invert were monitored during the entire stage.

3) **Uplift**: Immediately after preloading, the foundation was pulled out at a velocity of \( v = 0.5 \) mm/s \( (vD/c_v = 288) \) until the skirt tip was clear of the soil surface. The pullout load, total and pore pressures at the foundation invert were monitored during this stage.

For the reference tests B1 and B2, step 2) was ignored, and the foundation was either pushed onto the clay at a rate of \( v = 0.1 \) mm/s \( (vD/c_v = 58) \) or pulled out at a rate of \( v = 0.5 \) mm/s \( (vD/c_v = 288) \) directly after installation until failure was observed. A faster rate is required for uplift than in compression to ensure undrained behaviour, as already demonstrated by Chen et al. (2012) and Li et al. (2014b).
6.3 Experimental results

6.3.1 Ultimate undrained bearing and uplift capacities

Figure 6.6 presents the results of tests B1 and B2 and assesses the ultimate undrained bearing and uplift capacities. The measured and corrected penetration resistance, \( q_m \) and \( q_{\text{correct}} \), and the excess pore pressures are plotted against the normalised penetration depth, \( z/D \), in Figure 6.6a and b, respectively. First, the corrected penetration resistance was obtained by subtracting installation resistance, which was shown to be approximately 2.1 kPa in all the model tests. This corresponds to a friction ratio of approximately 0.3, consistent with those reported by Gourvenec et al. (2007, 2009) and Mana et al. (2013) for a stainless steel interface in LOC kaolin clay. Second, the corrected resistance accounts for the difference between the overburden pressure and the self-weight of the soil plug within the skirt during fully undrained compression or uplift (Tani and Craig 1995; Mana et al. 2012, 2013).

The ultimate undrained bearing capacity was established to be \( q_{u-c} = 26.9 \text{ kPa} \). This value corresponds to a bearing factor of \( N_c = 11.5 \), which falls between the lower and upper bound solutions presented by Martin (2001) (Figure 6.7). The ultimate undrained uplift capacity is slightly higher at \( q_{u-u} = 33.1 \text{ kPa} \), which corresponds to a bearing factor of \( N_c = 14.9 \). This value contradicts the results reported by Chen et al. (2012) and Li et al. (2014a, c), who demonstrated that the uplift capacity could be estimated with the same bearing capacity factor as that during compression if a reverse end bearing capacity is mobilised during undrained uplift. This higher value is believed to result from (i) the 5 minute waiting period that followed skirt installation, during which soil relaxation associated with a reduction of the vertical load measured occurred, and (ii) the higher strain rate effects during uplift, which account for 16% and 6% of the ultimate capacity, respectively. If both effects are ignored, the bearing factor reduces to 11.7, which agrees with the factor obtained during compression (Figure 6.7).
6.3.2 Typical uplift response

The results of test T3-3 are presented in Figure 6.5a (measured penetration resistance) and Figure 6.5b (excess pore pressure). At the contact of the foundation invert with the soil, the normalised installation distance, \( z/D \), is approximately 0.187, which is 7% less than the skirt length ratio \( (h/D = 0.2) \). The difference corresponds to the internal plug heave, as some of the material pushed by the skirt tip is displaced inside the foundation. At the application of preloading, excess pore pressures are generated and begin to dissipate as preloading is maintained constantly. The degree of consolidation was estimated from the pore pressure measurements and is detailed in Section 6.3.3. During uplift, the load reached a peak value over a very short displacement of less than 2% of the foundation diameter before breaking out at displacements ranging from 3% to 15% of the foundation diameter, depending on the parameters of the preloading stage. The peak load is associated with a peak negative excess pore pressure at the foundation invert. Remarkable agreement was observed between all pore pressure measurements, indicating that suction is homogeneously distributed within the area monitored by the sensors. Once the foundation was pulled out of the soil surface, the residual loads and negative excess pore pressures were monitored from the soil plug trapped within the skirt of the foundation.

6.3.3 Degree of consolidation

Figure 6.8 presents the time histories of the excess pore pressures at the centre of the foundation invert (monitored by the PPT0 as illustrated in Figure 6.1) and the settlement of the foundation for tests featuring the longest consolidation time (approximately 17 hours in the centrifuge or 40 years in prototype), i.e., tests T1-4, T2-4, T3-4 and T4-4. The initial time of consolidation, \( t_0 \), was taken as the moment when the preloading reached the targeted level, \( q_p \), as illustrated in Figure 6.8b. This value corresponds to an initial excess pore pressure, \( u_{p0} \), as reported in Figure 6.8a. Evidently, higher levels of preloading generate larger excess pore pressures and greater settlements.

To determine the degrees of consolidation achieved in each test, the experimental results were compared to the Finite Element (FE) results from Gourvenec and Randolph (2010),
in which the excess pore pressure dissipation at the centre of the foundation invert and the settlement for shallowly skirted foundations \((h/D \leq 1)\) were presented as dimensionless forms. The FE results for skirt length ratios of \(h/D = 0.15\) and \(0.3\) are used, which encompass the present case of \(h/D = 0.2\). A dimensionless time to achieve a degree of consolidation of 50\%, \(T_{50} = 0.14\), was interpolated from the FE results. The following procedure was used to determine the experimental degree of consolidation:

1) The excess pore pressure response at the centre of the foundation invert was normalised by the corresponding \(u_{p0}\) and plotted against the dimensionless time, \(T = c_v(t_m - t_0)/D^2\), where \(t_m\) is the time and \(D\) is the diameter of the foundation. The value of the consolidation coefficient, \(c_v\), was adjusted to yield the best fit with the FE results from Gourvenec and Randolph (2010).

2) The time to reach 50\% of the consolidated degree, \(t_{50}\), was calculated using \(t_{50} = T_{50}D^2/c_v\), where \(c_v\) is the adjusted coefficient of consolidation and \(T_{50} = 0.14\), as suggested above. Once \(t_{50}\) was established, the corresponding settlement and the final settlement of the foundation can be inferred from the settlement curves (as suggested by the circles in Figure 6.8b). The degree of the consolidations for each test is obtained by normalising the settlements to the corresponding final settlement, i.e., \(U = w/w_f\).

Figure 6.9 presents the comparison between the experimental and FE results from Gourvenec and Randolph (2010) following the procedure above. Both the excess pore pressures and the foundation settlements agreed well with the FE results when coefficients of consolidation of 5.25 to 6.50 \(m^2/\text{year}\) were used. The adjusted coefficient of consolidation is in relatively good agreement with that established from the twitch tests (Figure 6.4b), considering that the twitch tests yield an average value over the depth of the sample, e.g., over a larger range of stress values.

This procedure is necessary because no tests were conducted until 100\% of consolidation was achieved; thus, the final settlements and final pore pressures are unknown (variations in the water table during the test prevent an accurate estimation of the hydrostatic pore pressure within the sample). The resulting degrees of consolidation for all tests are
presented in Table 6.1. They fall into four groups, i.e., $U \leq 1\%$ (almost no consolidation), $15\% - 31\%$, $45\% - 53\%$, and $91\% - 93\%$, as illustrated in Figure 6.9b.

### 6.3.4 Effects of preloading and consolidation on uplift resistance

The entire set of experimental results for all preloaded uplift tests is presented in Figure 6.10 (measured penetration resistance) and Figure 6.11 (excess pore pressure at the centre of the foundation invert). The results of test B2 are also plotted for comparison. The figures are divided into four groups according to the levels of preloading applied, e.g., $q_p/q_{u-c} = 20\%$, $40\%$, $60\%$ and $80\%$. In each group, the degree of consolidation varies from $0\%$ (no consolidation) to slightly more than $90\%$.

All tests exhibited an excellent repeatability during the installation and preloading stages. The settlements of the foundation increase with both the level of preloading and the degree of consolidation, with the highest settlement being $0.065D$ for $q_p/q_{u-c} = 80\%$ and $U = 91\%$ (test T4-4 in Figure 6.10d). The excess pore pressures generated during the application of preloading increase with the level of preloading and exhibit a steady reduction as dissipation occurs (Figure 6.11).

The measured peak uplift resistances, $q_{up}$, and peak excess pore pressures from either the total pressure transducers, $u_t$, or pore pressure transducers, $u_p$, during uplifting were extracted and normalised by the corresponding values obtained from test B2. This normalisation defines a capacity ratio, $R_q$, and an excess pore pressure ratio, $R_u$, which are plotted against the degree of consolidation and the level of preloading in Figure 6.12 and Figure 6.13, respectively. Two fundamental observations emerged from Figure 6.12 and Figure 6.13:

1) The undrained uplift capacity of the circular skirted foundation increases linearly with the degree of consolidation. This increase is more pronounced for higher levels of preloading, reaching $50\%$ for a level of preloading of $80\%$ and a degree of consolidation of $91\%$. In contrast, this increase is approximately $30\%$ for the same degree of consolidation but a level of preloading of $20\%$. Although the data are relatively scattered, a trend of dissociation is apparent between the ratios $R_q$ and $R_u$.  

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6.114
as the degree of consolidation increases. However, the totality of the capacity increase for lower degrees of consolidation can be attributed to the increase in suction at the foundation invert associated with the increase in soil strength due to consolidation. For higher degrees of consolidation, the increase in suction measured at the centre of the foundation invert seems to contribute to only 50 to 80% of the increase in capacity. This phenomenon may be attributed to a change in the distribution of the suction at the foundation invert as the degree of consolidation increases, which results in the measurements at the centre to no longer be representative of the average suction developed, or to a change in the contribution of the overburden stresses generated by the large settlements of the foundation.

2) For the highest level of preloading (i.e., $q_p/q_u = 80\%$), a reduction in the uplift capacity is evident for degrees of consolidation lower than 30% (Figure 6.13a and b). This reduction results from the remoulding of the clay generated by the application of preloading and the associated reduction in shear strength, whose magnitude increases with the level of preloading. With consolidation, the strength of the clay increases from the remoulded value to values greater than the intact shear strength measured by the T-bar. For low levels of preloading, the reduction of the shear strength due to the application of preloading is either marginal or immediately compensated by the increase due to consolidation. For levels of preloading equal or higher than 80\%, e.g., when the foundation nearly mobilises a full failure mechanism, the reduction in strength is such that 30\% of the consolidation is required for the strength to regain its initial value. This requirement does not prevent higher levels of preloading exhibiting the higher increase in capacity after full consolidation. Figure 6.12 shows that the rate of uplift capacity increase with degree of consolidation positively correlates with the level of preloading.
6.4 Analytical modelling and discussion

6.4.1 Theoretical framework

The prediction of the ultimate undrained uplift capacity requires the determination of a bearing capacity factor and an operative shear strength that represents the average strength of the soil mobilised during uplift. As discussed above, the bearing factor can be considered as the bearing factor for uplift, e.g., $N_c = 14.9$. Here, the relaxation and rate effects discussed in Section 6.3.1 are taken into account, but a standard value of $N_c = 11.7$ can be used if these effects are absent. The determination of the operative shear strength necessitates the consideration of the load history and the weakening and strengthening from the applied preloading and subsequent consolidation.

To this end, a simple framework based on critical state theory is proposed, where the changes in undrained shear strength during preloading and consolidation are correlated to the changes in vertical effective stress level of the soil. The framework is presented in Figure 6.14 and is based on the well-established relationship between the specific volume and the vertical effective stress, which was used for interpreting the behaviour of risers under episodic cyclic loading (White and Hodder 2010; Hodder et al. 2013). The path ABCD illustrates the change in the effective stress and specific volume during preloading and consolidation. A is located on an Intact Strength Line (ISL) that represents the effective stress – specific volume state in LOC clay. Undrained preloading generates excess pore pressures, which reduces the effective stress from A to B at a constant volume. Because only a fraction of the ultimate capacity is applied, B is located between the ISL and the Critical State Line (CSL). During the subsequent consolidation, the effective stress follows the Unloading and Reloading Line (URL) from B to C and may merge with the Normal Compression Line (NCL) and pursue its path to point D at full consolidation. The path from B to D may vary depending on the magnitude of the preloading relative to the initial effective stress and the degree of consolidation achieved.

The NCL is defined as in the Modified Cam Clay (MCC) model:

$$\nu = N - \lambda \ln \sigma'_v$$

(6.1)
with $N$ being the specific volume at $\sigma'_v = 1$ kPa and $\lambda$ being the slope of NCL (also ISL and CSL). The ISL is defined as follows:

$$\nu = \Gamma - \lambda \ln \sigma'_v$$  \hspace{1cm} (6.2)

which corresponds to the initial stress state after installing the foundation (i.e., without remoulding due to preloading). $\Gamma$ is the specific volume at $\sigma'_v = 1$ kPa on the ISL. The URL depends on the initial stress, $\sigma'_v0$, and the initial specific volume, $\nu_0$:

$$\nu = \nu_0 - \kappa \ln (\sigma'_v / \sigma'_v0)$$  \hspace{1cm} (6.3)

in which $\kappa$ is the slope of the URL and $\nu_0$ is the initial specific volume that can be determined by the following:

$$\nu_0 = N - \lambda \ln (OCR\sigma'_v0) + \kappa \ln OCR$$  \hspace{1cm} (6.4)

where OCR = $\sigma'_{v0,max} / \sigma'_v0$, in which $\sigma'_{v0,max}$ is the maximum vertical effective stress experienced by the soil. The parameters $N$ and $\Gamma$ are related by the following equation:

$$N = \Gamma + (\lambda - \kappa) \ln OCR$$  \hspace{1cm} (6.5)

White and Hodder (2010) suggested that the soil strength can be related to the vertical effective stress by a constant frictional factor, $\mu$, as follows:

$$s_u = \mu \sigma'_v$$  \hspace{1cm} (6.6)

Thus, the change in the undrained shear strength of the soil is equivalent to the change in the vertical effective stress. The remoulded soil strength that results from the application of preloading can be assumed to be similar to that caused by cyclic disturbances (Einav and Randolph 2005) by relating the damage parameters, $\delta_{rem}$, to the level of preloading:

$$\frac{s_{up}}{s_{uo}} = \frac{\sigma'_{vp}}{\sigma'_v0} = A \delta_{rem} + (1 - A \delta_{rem}) e^{-B \frac{q_p}{q_u-c}}$$  \hspace{1cm} (6.7)

where $\sigma'_{vp}$ is the vertical effective stress immediately after the application of preloading, $q_p$ is the preloading, and $q_u-c$ is the ultimate undrained bearing capacity without preloading
as in test B1). \( \delta_{\text{rem}} = 0.4 \) represents the ratio of the full remoulded soil strength to the intact strength (Figure 6.3b). Because the disturbance from the monotonic preloading is expected to be less significant than that of the cyclic disturbance, in which the soil is repeatedly sheared, a parameter, \( A \), is introduced to account for the reduction in soil strength due to a single shearing event. The values of \( A \) range from 1 (no remoulding) to \( S_t \) (full remoulding). \( B \ (> 0) \) is the parameter that controls the rate of strength reduction.

The total stress after applying the preloading is \( \sigma_{vp} = \sigma_{v0} + f_{\sigma} q_p \), where \( f_{\sigma} \) is a stress influence factor that accounts for the vertical stress change caused by the preloading; its value less than a unit due to its decay along the depth of soil (as that for Boussinesq solution). Therefore, a nominal excess pore pressure immediately after application of preloading can be obtained as follows:

\[
\hat{u} = \sigma_{vp} - \sigma_{vp}' = \sigma_{v0}' + f_{\sigma} q_p - \sigma_{v0}' \left[ A\delta_{\text{rem}} + (1 - A\delta_{\text{rem}})e^{-B\frac{q_p}{q_u-c}} \right] \quad (6.8)
\]

The operative soil strength after consolidation is related to the excess pore pressure dissipation in the soil. The current level of pore pressure can be assumed to be linearly related to the degree of consolidation:

\[
\hat{u} = \hat{u}_i (1 - U) \quad (6.9)
\]

The vertical effective stress at any time during the consolidation can be determined by considering the difference between the applied total stress and the current pore pressure:

\[
\sigma_{v}' = \sigma_{v} - \hat{u} = \sigma_{v0}' + f_{\sigma} q_p - \hat{u}_i (1 - U) \quad (6.10)
\]

By substituting Equation (6.8) into Equation (6.10), the current operative soil strength due to any preloading and consolidation degree can be expressed as

\[
\frac{s_{u-op}}{s_{u0}} \sigma_{v0}' = \frac{s_{u-op}}{s_{u0}} \sigma_{v0}' = \frac{s_{\text{umax}}}{s_{u0}} - \left[ \frac{s_{\text{umax}}}{s_{u0}} - \left( A\delta_{\text{rem}} + (1 - A\delta_{\text{rem}})e^{-B\frac{q_p}{q_u-c}} \right) \right] (1 - U) \quad (6.11)
\]

where \( s_{\text{umax}} \) represents the fully consolidated soil strength, which can be determined as follows:
\[
\frac{s_{u,\text{max}}}{s_{u,0}} = 1 + \frac{f_u q_p}{\sigma'_{v,0}}
\]  

(6.12)

During consolidation, the effective stress path follows the URL and may merge with the NCL if the current vertical effective stress is higher than a critical value:

\[
\sigma'_{v,c} = e^{\frac{N - \nu_0 - \kappa \ln \sigma'_v}{(\lambda - \kappa)}}
\]  

(6.13)

Therefore, the change in the specific volume due to preloading and the subsequent consolidation can be determined as follows:

\[
\Delta \nu = \nu_{pc} - \nu_0 = -\kappa \ln \left( \frac{\sigma'_{vpc}}{\sigma'_{vp}} \right) \text{ for } \sigma'_v \leq \sigma'_{v,c}
\]  

(6.14)

or,

\[
\Delta \nu = -\kappa \ln \left( \frac{\sigma'_{v,c}}{\sigma'_v} \right) - \lambda \ln \left( \frac{\sigma'_{vpc}}{\sigma'_{v,c}} \right) \text{ for } \sigma'_v > \sigma'_{v,c}
\]  

(6.15)

The proposed framework can be used to derive an operative shear strength to predict the undrained capacity of a skirted foundation while accounting for the varying magnitude of preloading and consolidation.

### 6.4.2 Back calculated operative shear strength – validation of the framework

The operative soil shear strength can be back-calculated from the experimental results and used to adjust the parameters of the proposed framework. A constant bearing capacity factor of \( N_c = 14.9 \), as established from test B2, was assumed for all uplift tests. By accounting for the difference between the weight of the soil plug and the overburden pressure, the operative soil strength can be obtained by dividing the corrected ultimate uplift capacities by the bearing capacity factor, i.e., \( s_{u,\text{op}} = q_u / N_c \).

Figure 6.15 presents the comparison between the back-calculated operative soil strength normalised by the intact strength obtained from the T-bar tests and the operative shear strength ratio predicted by the framework as a function of the level of preloading (Figure 6.15a) and the degree of consolidation (Figure 6.15b). The parameters used in the
Chapter 6  Effects of preloading and consolidation on the uplift capacity of skirted foundations

Framework are presented in Table 6.2. The parameters $\delta_{rem}$, $\Gamma$, OCR and $\mu$ can be determined by combining the T-bar tests (Figure 6.3), and moisture content tests on core samples in which the specific volume – in-situ vertical effective stress relationship can be derived. The parameters $\lambda$ and $\kappa$ are inferred from the typical properties of kaolin clay (Stewart 1992; Chatterjee et al. 2012, 2013). Parameters $A$, $B$ and $f_\sigma$ are related to the soil disturbance and stress non-uniformity due to preloading, which can be adjusted to fit the test results. This yields $A = 1.2$, $B = 0.06$ and $f_\sigma = 0.3$, which falls into the range mentioned in Section 6.4.1.

Figure 6.15 demonstrates that the framework can capture the general variation of the operative soil strength resulting from remoulding and consolidation; notably, it also captures the reduction in the operative shear strength (and hence uplift capacity) observed for a high level of preloading and low degrees of consolidation. At the beginning of consolidation, the operative strength is dominated by the remoulding effects and degrades with higher levels of preloading. In the subsequent consolidation, the soil strength increases linearly as the degree of consolidation increases, as observed in the centrifuge tests. Furthermore, the rate of the recovery is proportional to the preloading level, with higher levels of preloading resulting in higher operative soil strength when full consolidation is achieved.

Figure 6.16 shows the back-calculated specific volume changes obtained by substituting the operative vertical stresses (inferred from the back calculated operative soil strengths) into Equation (6.14) and Equation (6.15). The stress paths deduced from the framework for the relevant levels of preloading are also plotted. The figure indicates that the test conditions covered the full range of consolidation that should have resulted in the NCL being reached in some cases.

$A$, $B$ and $f_\sigma$ are empirical parameters, which depend on the soil strength degradation characteristics (for $A$ and $B$) and the geometry of the foundation (for $f_\sigma$). The values presented are only valid for the case considered. However, they are believed to be appropriate for all surface loading problems on clays for which strength degradation characteristics are similar to those of kaolin clay. Further work is required to establish these parameters for a wider range of soil conditions.
Centrifuge tests have been conducted to examine the effects of the magnitude and duration of preloading on the subsequent uplift responses of a shallowly skirted foundation. The preloading was expressed as a percentage of the unconsolidated undrained bearing capacity during compression. The degree of consolidation was quantified by comparing the excess pore pressure dissipation and settlements of the foundation with the FE results from Gourvenec and Randolph (2010).

The undrained bearing capacity factor for the compression of the foundation in the undisturbed clay is approximately 11.5 and consistent with the bound solutions reported by Martin (2001), while the bearing factor for undrained uplift is 30% higher than that during compression. This is due to the relaxation between the end of the preloading and the beginning of the uplift and to the higher strain rate effects resulting from the higher uplift rate.

The application of compressive preloading results in a remoulding of the soil and a reduction in the operative shear strength, whose magnitude positively correlates with the level of preloading. This relationship results in an uplift capacity immediately after the application of preloading that is lower than the compression capacity. With consolidation, the soil regains strength, and the uplift capacity progressively increases to reach values up to 1.5 times the benchmarked capacity during uplift for a preloading level of 80% and almost full consolidation.

A simple critical state framework was proposed to describe the variation of the operative soil strength with preloading and consolidation. The framework can capture the key aspects of the soil behaviour and predict the uplift capacity of a circular skirted foundation for a wide range of preloading levels and degrees of consolidation.

This paper provides a comprehensive study of the effect of preloading and consolidation on the undrained uplift capacity of shallow foundations. The outcomes of the study, including the proposed predictive framework, can be used for the stability design of offshore foundations subjected to tensile loadings and to provide guidance to the retrieval of deep-water foundations.
6.6 References


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Effects of preloading and consolidation on the uplift capacity of skirted foundations


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### Table 6.1 Model test programme

<table>
<thead>
<tr>
<th>Test name</th>
<th>Preloading, $q_p$ (kPa)</th>
<th>Preloading level, $q_p/q_{u-c}$</th>
<th>Consolidation model time, $t_m$ (s)</th>
<th>Consolidation prototype time, $t_p$ (year)</th>
<th>Normalised consolidation time, $T$</th>
<th>Normalised consolidation degree, $U$</th>
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<tr>
<td>B1*</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>B2*</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>T1-1</td>
<td>5.39</td>
<td>20%</td>
<td>14</td>
<td>0.01</td>
<td>0.0007</td>
<td>1%</td>
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<td>T1-2</td>
<td>5.39</td>
<td>20%</td>
<td>971</td>
<td>0.69</td>
<td>0.05</td>
<td>31%</td>
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<td>T1-3</td>
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<td>2.29</td>
<td>0.16</td>
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<td>T1-4</td>
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<td>62149</td>
<td>44.34</td>
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<td>0.00</td>
<td>0.00</td>
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<td>55038</td>
<td>39.27</td>
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<td>60%</td>
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<td>0.00</td>
<td>0%</td>
</tr>
<tr>
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<td>60%</td>
<td>906</td>
<td>0.65</td>
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<td>23%</td>
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<td>60%</td>
<td>2851</td>
<td>2.03</td>
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<td>50%</td>
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<td>T3-4</td>
<td>16.17</td>
<td>60%</td>
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<td>43.69</td>
<td>3.51</td>
<td>93%</td>
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<td>0.00</td>
<td>0.00</td>
<td>0%</td>
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<td>2.78</td>
<td>91%</td>
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* Reference tests on undrained bearing capacities under compression B1 and uplift B2
Table 6.2 Parameters of proposed predictive method

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>CSL: slope</td>
<td>λ</td>
<td>0.205</td>
</tr>
<tr>
<td>URL: slope</td>
<td>κ</td>
<td>0.044</td>
</tr>
<tr>
<td>Full remoulded strength ratio</td>
<td>δ_{rem}</td>
<td>0.4</td>
</tr>
<tr>
<td>ISL: specific volume at $\sigma'_v = 1$ kPa</td>
<td>$\Gamma$</td>
<td>2.84</td>
</tr>
<tr>
<td>Over-Consolidated Ratio</td>
<td>OCR</td>
<td>1.25</td>
</tr>
<tr>
<td>Frictional strength parameter</td>
<td>$\mu$</td>
<td>0.21</td>
</tr>
<tr>
<td>Remoulding amount parameter</td>
<td>$A$</td>
<td>1.2</td>
</tr>
<tr>
<td>Remoulding rate parameter</td>
<td>$B$</td>
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</tr>
<tr>
<td>Non-uniform stress influence factor</td>
<td>$f_o$</td>
<td>0.3</td>
</tr>
</tbody>
</table>
Figure 6.1 Model configuration and instrumentation: (a) side view; (b) upward view
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$V = vd/c_v$

Best estimation: $c_v = 3.28 \text{ m}^2/\text{year}$
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Figure 6.11 Development of excess pore pressures at the centre of the foundation invert for different levels of preloading: (a) $q_p/q_u = 20\%$; (b) $q_p/q_u = 40\%$; (c) $q_p/q_u = 60\%$; (d) $q_p/q_u = 80\%$
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Figure 6.13 Capacity and excess pore pressure ratios as a function of the level of preloading for different degrees of consolidation: (a) $U \leq 1\%$; (b) $U = 15\% - 31\%$; (c) $U = 45\% - 53\%$; (d) $U = 91\% - 93\%$
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Figure 6.15 Comparison of soil strength ratios varying against (a) preloading level and (b) consolidation degree.
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CHAPTER 7  SUSTAINED UPLIFT OF SKIRTED FOUNDATION IN CLAY

Abstract

A series of centrifuge tests have been performed to investigate the uplift behaviour of a shallow skirted foundation resting on clay subjected to various sustained loadings expressed as a fraction of the undrained bearing capacity in compression. Displacements, uplift loads, total and pore pressures underneath the foundation were monitored during testing to provide insight on the development of negative excess pore pressure (suction) at the foundation base and on the mechanism triggering breakout. The results indicate that the displacement rate and the time the uplift load can be sustained depend on the magnitude of the load, but also on the time the foundation experienced consolidation prior to uplift. Breakout was not observed at a relatively large time scale under low sustained loadings, while the foundation was pulled out directly under high loading levels. At intermediate loadings, breakout was inferred from the sudden increase in displacement rate. The breakout mechanism of the foundation is also discussed based on the observation of pore pressure variations underneath the foundation.

The outcomes of the present study have the potential to provide a better understanding of the suction mechanism for offshore foundations and to provide guidelines for offshore retrieval operations.

**Keywords**: centrifuge test; skirted foundation; kaolin clay; sustained loading; breakout; suction


7.1 Introduction

Skirted foundations represent a type of shallow foundation that comprises a top plate and peripheral or sometimes internal skirts. The use of skirts is beneficial to increase sliding resistance by transmitting the loads on the foundation into deeper and usually stronger soil. Skirted foundations are used to anchor floating-type platforms or to support temporary subsea facilities. It is common for these foundations to be subjected to upward sustained loadings due to buoyancy or to requirement of retrieval after project decommission (Chen et al. 2012; Li et al. 2014). Therefore, the study of the sustainability of skirted foundations is of significance in offshore operations.

The sustained uplift of skirted foundations was first investigated by Byrne and Finn (1978) in a 1g test to relate foundation breakout to drainage conditions in the soil. Das (1991) proposed an empirical expression considering the effect of soil thixotropy and in-situ time on the sustainability of shallow embedded foundations. Centrifuge tests were also performed that focused extensively on the effects of the skirt length, gap formation and eccentric uplift on the capacity of the foundation under various sustained upward loadings (Acosta-Martinez et al. 2008, 2010, 2012; Gourvenec et al. 2009; Mana et al. 2012a, 2013b). These results provide useful guidelines regarding offshore skirted foundations under sustained loadings. The phenomenon of shallow foundation breakout was also studied theoretically by Sawicki and Mierczyński (2003) and further numerically developed by Zhou et al. (2008), suggesting that the effective stress at the interface between the soil and foundation base could be a crucial factor governing the breakout process.

This paper presents centrifuge test results on the uplift of a shallowly skirted foundation under various sustained loadings. The sustained loading levels are expressed as a fraction of the undrained bearing capacity of the same foundation in compression. The displacement, load and pressure variation at the foundation invert were monitored. The effect of sustained loading on the subsequent transient uplift capacity was considered. The breakout mechanism of the foundation under sustained uplift is also discussed in this study.
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7.2 Experimental set-up

7.2.1 Facility and model

All model tests were performed in a drum centrifuge at the Centre for Offshore Foundation Systems (COFS) at The University of Western Australia (UWA) (Stewart et al. 1998). The drum centrifuge was selected because it enables multiple tests to be conducted within a single soil sample. The ring channel of the centrifuge has an outer diameter of 1.2 m, an inner diameter of 0.8 m and a channel height (soil sample width) of 0.3 m. A tool table is mounted at the centre of the drum, which can rotate independently of the outer channel and thus be halted for verification or a tool change without disturbing the soil sample. An actuator attached to the central tool table can provide both load and displacement control in the vertical and radial directions by driving the servomotors. The centrifuge is equipped with a wireless data acquisition system developed in-house at UWA, which can achieve real-time data transmission (Gaudin et al. 2009). In the present study, all tests were performed at a centrifuge acceleration of 200g. The details of the scaling laws relevant to the modelling can be found in Garnier et al. (2007).

One circular model was fabricated for the present study (Figure 7.1). The model has an overall diameter of \( D = 60 \) mm (12 m prototype) and a skirt length of \( h = 12 \) mm (2.4 m prototype, excluding the thickness of the base), yielding an embedment ratio of \( h/D = 0.2 \). The reaction force was measured using a 2 kN capacity load cell attached to the centre of the model top, and any movement of the model was recorded by an actuator that was fixedly connected to the model. Three Pore Pressure Transducers (PPTs) and two Total Pressure Transducers (TPTs) were instrumented onto the model invert to monitor the pressure changes during testing. A vent was drilled through the model plate to allow air to flow when installing the model onto the clay; the vent was sealed by a plastic air valve after installation. Additionally, the model was made with a curved base (curvature radius = 480 mm on model scale) to ensure regular contact between the model invert and the clay surface. This configuration results in a 0.94 mm elevation difference between the centre and edge of the model (Figure 7.2).
7.2.2 Clay sampling and characterisation

Commercial kaolin powder was mixed with 120% water (twice the liquid limit) in a vacuum chamber. The mixed slurry was transferred into a hopper and poured into the centrifuge by a delivery nozzle at an acceleration of 20g. A fabric drainage layer was placed at the channel bottom to achieve two-way consolidation. After three top-ups, the channel was full of clay. The centrifuge was spun at 200g for four days to allow full consolidation of the clay. The height (including the drainage layer) of the normal consolidated sample was ~ 150 mm. A further ~ 30 mm of the top layer was scraped off to form an evenly elevated surface and to match the curvature of the foundation invert. Reconsolidation at 200g was performed overnight to allow the sample to settle down. The final height of the Lightly Over-Consolidated (LOC) clay was confirmed to be 120 mm.

The properties of the clay have been well-characterised (Stewart 1992; Chen 2005; Acosta-Martinez and Gourvenec 2006): specific gravity of $G_s = 2.6$, liquid limit of $LL = 61\%$, plastic limit of $PL = 27\%$ and plasticity index of $I_p = 34$. The soil coefficient of consolidation is taken to be $c_v = 1.5 \text{ m}^2/\text{year}$, corresponding to the stress level experienced by the clay underneath the foundation during testing (House et al. 2001).

A miniature T-bar was used to characterise the shear strength of the clay sample (Stewart and Randolph 1991, 1994). The undrained shear strength was first obtained by normalising the T-bar resistance by a bearing capacity factor of $N_{T-bar} = 10.5$ (Lehane et al. 2009; Low et al. 2010). However, a lower bearing capacity factor was used at shallow depth to account for soil buoyance and soil heave (White et al. 2010). A cyclic test was also included in each T-bar test to eliminate the offset of the data recorded (Randolph et al. 2007). As demonstrated in Figure 7.3, the LOC clay sample exhibited a linearized undrained strength against depth, with an average intercept of $s_{um} = 2.88 \text{ kPa}$ at the surface. The gradient of the soil profile was $k = 1.23 \text{ kPa/m}$, corresponding to a strength heterogeneity ratio of $kD/s_{um} = 5.8$. T-bar tests were performed before and after model tests. It is indicated in Figure 7.3 that during test, the soil strength became slightly stronger with time, due to further consolidation within the clay and drying at the top of the clay.
7.2.3 Test programme and procedure

Overall, six sustained loading tests were performed at an acceleration of 200g. The tests were performed in consideration of multiple uplift loadings, which were compared with a benchmark test by penetrating the same foundation at a fast enough displacement rate to be considered undrained. Displacement-controlled tests were also performed in the same clay sample and reported in another publication (Li et al. 2014). Some tests are referred to in the present study for comparison. The test sites were spread over the centrifuge channel with an edge-to-edge spacing of one and half model width, which were expected to be wide enough to avoid interference between tests.

Each model test underwent four stages of operation. The time histories of displacement and measured force (normalised by the foundation cross sectional area) are presented in Figure 7.4.

1) The foundation was first installed into the clay at 1g with the vent opened at a velocity of \( v = 0.05 \text{ mm/s} \) until the foundation base was in full contact with the clay surface. The touchdown of the foundation was monitored by the load cell reading (Figure 7.4a).

2) The vent was subsequently closed to seal the foundation. The centrifuge was then ramped up to the targeted 200g acceleration level, and the channel was filled with water until the footing was fully submerged. The load cell experienced tensile loads during this process and attained a stable value indicating the submerged weight of the foundation (Figure 7.4b).

3) A compressed preload, equivalent to \( \sim 10\% \) of the undrained bearing capacity in compression (according to the post analysis), was applied to the foundation to accommodate the settlement of clay during ramping up and establish full contact between the clay top and the foundation base. The clay was consolidated under the preload, and the degree of consolidation was monitored by the movement of the actuator to which the foundation was attached (Figure 7.4c).

4) The foundation was finally pulled under various sustained loadings, where the exact levels are expressed as a fraction of the undrained bearing capacity of the same
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Sustained uplift of skirted foundation in clay  

foundation in compression, as detailed in Section 7.3.1. It should be noted that the centrifuge was preset with a protecting velocity of \( v = 0.01 \text{ mm/s (model scale)} \), i.e., the fastest possible velocity that the actuator is allowed to reach under load control mode. When breakout happened, the foundation was extracted with this velocity (Figure 7.4d).

7.3 Experimental results

All results are presented at the prototype scale unless stated otherwise.

7.3.1 Determination of undrained bearing capacity and sustained loading levels

The measured resistance \((q_m)\) is defined as the measured force \((Q_m, \text{ excluding the submerged weight of the model})\) divided by the cross sectional area of the foundation \((A)\). The net resistance \((q_{net})\) is corrected by considering the difference between the submerged weight of the soil plug incorporated by the peripheral skirt and the overburden pressure provided by an undrained failure mechanism that can be mobilised in both compression and uplift (Tani and Craig 1995; Mana et al. 2012b, 2013a):

\[
q_{net} = q_m - \sigma' v + \frac{W_{soilplug}}{A_{soilplug}} \quad (7.1)
\]

where \(\sigma' v\) is the overburden pressure above the skirt tip level, \(W_{soilplug}\) is the submerged weight of the soil plug incorporated by the skirt and \(A_{soilplug}\) is the cross area of soil plug which is approximated by the cross sectional area of the foundation. Figure 7.5 shows both the measured and net resistance of the circular foundation in the displacement-controlled compression and uplift tests. Overall, the corrected net resistance is smaller than the measured resistance in both compression and uplift.

The net resistances of the foundation are also normalised by the soil undrained shear strength at the current skirt tip level \((s_{ud})\) to account for the changes in embedment (Figure 7.6). These values provide benchmarking information on the bearing capacity of the foundation in both compression and uplift. The normalised velocity \(V\) (defined as \(vD/c_v\))
7.1 Sustained uplift of skirted foundation in clay

~ 126.1 (corresponding to the model velocity $v = 0.1$ mm/s) was expected to be high enough to achieve an undrained bearing failure mechanism for compression (Finnie and Randolph 1994) and a full reverse end bearing failure mechanism for uplift (Li et al. 2014). As illustrated in Figure 7.6, the bearing capacity factor $N_c$ (defined as the peak of the normalised resistance) was equal to 10.9 for compression and 10.6 for uplift, with both mobilised at a normalised displacement of $w/D$ of approximately 4%. The bearing capacity factors for both compression and uplift, nevertheless, are slightly higher than those calculated numerically by Gourvenec and Mana (2011). The difference is most likely due to viscous enhancement and preloading effect in the centrifuge test, both of which were not accounted for in the numerical model.

The measured undrained bearing capacity for compression is probed as $q_u = 80.8$ kPa, which mobilises at 4% normalised displacement, as observed in Figure 7.5. The applied sustained stress levels, excluding the submerged weight of the foundation, were measured to be 1.6 kPa, 7.2 kPa, 17.5 kPa, 27.8 kPa, 35.1 kPa and 46.3 kPa, which correspond to 2%, 9%, 23%, 36%, 46% and 58%, respectively, of the undrained bearing capacity of the same foundation in compression. This is summarised in Table 7.1.

7.3.2 Sustained uplift displacement versus time responses

Figure 7.7 summarises the time history of the sustained uplift displacement normalised by the foundation diameter under various sustained loading levels. Overall, the displacement responses under sustained loading are linear up to foundation breakout (if occurring), which is expected to be governed by the negative excess pore pressure (suction) at the foundation base. For test C1 with a lower loading level, breakout was not explicitly observed within the prototype 227 days of testing (Table 7.1). In contrast, in tests C2, C3, C4 and C5, which were subjected to intermediate sustained loading levels, breakout occurred at different times, depending on the sustained loading level. Once the foundation became unsustainable, it broke out at a protecting velocity preset by the centrifuge, as shown for test C3 in Figure 7.4d. It is noteworthy that the rate of the displacement and time to breakout are also relevant to the elapsed time before sustained uplift was applied, i.e., the consolidation time (Table 7.1). As indicated in Figure 7.7, the foundation in test C3 moved approximately 20% slower and broke out approximately 41
days (prototype scale) later than test C2 although the former was subjected to 14% higher sustained loading than the latter. This result could have been caused by the ten-fold longer consolidation time in test C3. The foundation tends to be more stable with longer elapsed time prior to uplift, due to the higher degree of consolidation in clay.

Due to the protecting velocity in the centrifuge, the foundation broke out directly when the sustained loading reached more than 58% of the undrained bearing capacity in compression, and the responses reveal a single linear variation with time, which may not be the real situation in practice (test C6).

**7.3.3 Effect of sustained loading on subsequent transient uplift capacity**

To investigate the effect of sustained loading on the subsequent uplift capacity of the foundation, test C1, without breakout observed after 227 days (Table 7.1), was extracted transiently at a model velocity of 1 mm/s. This displacement rate was expected to be higher enough to mobilise a reverse end bearing failure mechanism (Li et al. 2014). The test that experienced transient uplift without prior sustained uplift was employed for comparison.

Figure 7.8 illustrates the responses of the measured uplift resistance against the normalised displacement. It can be observed, although not obviously, that the introduction of sustained uplift had an adverse effect on the subsequent transient uplift capacity. The amount of 2\%q_u sustained loading (test C1) lead to an approximately 4% decrease of the uplift capacity compared with the test without sustained loading in advance. The reason for this difference could be that the suction generated was partially dissipated under sustained loadings, thus making the foundation easier to extract. In addition, the foundation in test C1, which was affected by sustained loading, also broke out earlier than the foundation in the test without sustained loading due to the shortened skirt embedment during sustained stage in test C1 (Figure 7.8).

The introduction of sustained uplift loadings has significant implications in real operations. Offshore floating foundations, which are subjected to consistent tensile loadings, e.g., buoyance, may be more vulnerable to extraction. In contrast, this effect is beneficial in retrieving temporary subsea structures to reduce the unexpected large forces
from bottom suction. This is in contrast with compression, for which preloading typically result in strength increase and additional bearing capacity.

### 7.3.4 Excess pore pressure response and its implication on breakout failure mechanism

Figure 7.9 summarises the excess pore pressure responses at the foundation invert during the sustained loading stage for tests in which breakout could be explicitly observed, i.e., tests C2, C3, C4 and C5. The sustained loadings (normalised by area) applied to the foundation are also plotted in Figure 7.9 (short dashed lines) which can be assumed to be the total stress added at the interface between the clay and foundation base. The negative values of the excess pore pressure indicate the generation of suction underneath the foundation. The breakout of the foundation can be expected from the sudden drop of suctions (see circles in Figure 7.9).

As observed in Figure 7.9, when the foundation was subjected to sustained upward loadings, negative excess pore pressures (suctions) were generated, which were higher in magnitude than their corresponding applied sustained stresses. This effect triggered water seepage toward the bottom of the foundation. As the loading was consistently sustained, the suction gradually dissipated with time, and the foundation continued to move upward, which was explicitly observed in test C2. Furthermore, the breakout of the foundation appears to coincide with the moment when the magnitude of the suction equals to the applied total stress. This finding could potentially suggest that the breakout of the foundation is caused by zero effective stress at the interface between the clay and foundation base. However, the seal condition around the foundation appears to be also vital to the sustainability of the foundations. In the tests with higher sustained loadings, e.g., C3, C4 and C5, an abrupt loss of suction was observed before excess pore pressure dissipation can be observed obviously, which may suggest the appearance of a preferred drainage path, such as cracks, around the external skirts, allowing water to rapidly flow into the foundation base, leading to the sudden loss of suction and sustainability.
7.4 Conclusions

Centrifuge tests have been performed to investigate the uplift behaviour of a shallowly skirted foundation under various sustained loadings in lightly over-consolidated kaolin clay. The levels of sustained loading were expressed as a fraction of the undrained bearing capacity of the same foundation in compression.

The foundation moves linearly with time when the load remains constant. The rate of displacement increases with increasing sustained loading level but may also decrease due to a prolonged consolidation time before uplift. The foundation broke out directly with a mechanical protecting velocity in the centrifuge once losing its sustainability.

The introduction of sustained uplift has adverse effects on the subsequent transient uplift capacity of the foundation and the breakout time. Further study is needed to better understand this problem.

Based on the observation of excess pore pressure variations underneath the foundation, it can be inferred that the breakout of the foundation under sustained uplift could depend on either the effective stress at the interface of the clay and foundation base or the seal condition around the foundation. Numerical modelling is in developing to simulate the process to breakout and verify the proposed failure mechanism.

7.5 References


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### Table 7.1 Summary of sustained loading test

<table>
<thead>
<tr>
<th>Ref.</th>
<th>Preloading stage</th>
<th>Sustained loading stage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stress, $q_m$ (kPa)</td>
<td>Elapsed time, $t$ (day)</td>
</tr>
<tr>
<td>C1</td>
<td>7.2</td>
<td>28.5</td>
</tr>
<tr>
<td>C2</td>
<td>10.5</td>
<td>31.6</td>
</tr>
<tr>
<td>C3</td>
<td>9.0</td>
<td>342.7</td>
</tr>
<tr>
<td>C4</td>
<td>7.6</td>
<td>34.3</td>
</tr>
<tr>
<td>C5</td>
<td>9.1</td>
<td>35.1</td>
</tr>
<tr>
<td>C6</td>
<td>6.8</td>
<td>38.8</td>
</tr>
</tbody>
</table>

* $q_m$ expressed as percentage of $q_u$

Note: all dimensions are in prototype scale.
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Figure 7.1 Model configuration

Figure 7.2 Curved design at foundation base (dimensions on the model scale)

Figure 7.3 Undrained shear profile of clay sample
Chapter 7 Sustained uplift of skirted foundation in clay

![Diagram](image)

- **(a)** Soil mudline
- **(b)** Acceleration level: 1g
- **(c)** Acceleration level: 200g

*Installation rate: \( v = 0.05 \text{ mm/s} \)*

*Submerged weight of model*

*Water filling*

*Settlement*

*Preloading level*
Figure 7.4 Typical test sequences: (a) installation of foundation; (b) ramping centrifuge up to 200g and filling in water; (c) preload and consolidation; (d) sustained uplift (* model scale)

Figure 7.5 Measured and net resistance versus normalised displacement responses
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Figure 7.6 Normalised net resistance versus normalised displacement responses

Figure 7.7 Normalised sustained uplift displacement versus time responses during sustained loading stage
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Figure 7.8 Transient uplift resistance following sustained uplift (* model scale)

Figure 7.9 Excess pore pressure versus time responses during sustained loading stage
CHAPTER 8  EFFECT OF PERFORATIONS ON UPLIFT CAPACITY OF SKIRTED FOUNDATIONS ON CLAY

Abstract

The retrieval of deep-water subsea installations resting on soft soil, such as “mudmat” shallow foundations, can be a difficult and costly operation if significant resistance to uplift is experienced. At the mudmat invert, suctions may develop, increasing the uplift resistance to greater than the weight of the mat. In this paper, a series of centrifuge model tests are performed to determine the uplift resistance of rectangular mudmats resting on lightly over-consolidated kaolin clay. The study investigates the influence of perforation, in combination with skirt length and eccentric uplift, on the uplift resistance and suction generation at the foundation invert. The outcomes demonstrate that the central and eccentric uplift of mudmats have different failure mechanisms, resulting in a different distribution of excess pore pressure at the foundation invert. In contrast, perforations do not change the failure mechanism and only alter the magnitude of suction generated. The two different configurations of perforation investigated significantly reduce the suction at the mat invert and the uplift resistance, and may potentially shorten the operating time for centred uplift. The combination of perforation and eccentric uplift has the most beneficial effect on the reduction of the uplift resistance.

Keywords: centrifuge modelling; mudmat; clay; perforation; uplift resistance; suction
8.1 Introduction

Mudmats are a type of shallow raft foundation used to support various temporary and semi-permanent subsea structures, such as Pipeline End Manifolds and Terminations (PLEMs/PLETs). They are an easily installed and economical solution commonly used in deep water oil and gas developments. In order to provide sufficient resistance to withstand horizontal loads from the thermal expansion of pipelines and jumpers, mudmats are usually designed with skirts, which are embedded into the seabed by a fraction of the mudmat width. Upon completion of the project, and in some instances to comply with environmental regulations, mudmats must be decommissioned and removed from the seabed.

The standard removal procedure for mudmats is to attach cables to the load points on the structures. These cables are then pulled by a lift vessel at sea level to extract the mudmat from the seabed. The uplift forces required for removal from the seabed are resisted not only by the self-weight of the submerged mudmat, but also by the suction forces potentially developing at the mudmat invert. In soft soil with low permeability, such as the clays or silts commonly encountered in deep waters, these suction forces can be equal to twice the submerged weight of the mudmat (Bouwmeester et al. 2009). In extreme cases, the suction forces may be greater than the lifting capacity of the vessel and lead to hazards during removal (Reid 2007).

Various mitigation measures to reduce the generation of negative pressures (or suction) at mudmat inverts have been investigated, using both in-situ data and laboratory experiments. It was expected that perforations would limit the development of suction at the mudmat invert by shortening the drainage path. Lieng and Bjørgen (1995) reported that even a small perforation (with respect to the total mudmat area) can lead to a significant reduction in peak uplift resistance. During field trials, a reduction of about 50% of the uplift resistance was observed for a perforation ratio of 3.1% (defined as the plane area of perforating holes with respect to the total area). White et al. (2005) demonstrated that a large number of small perforations were more efficient in reducing the uplift ratio than a small number of large perforations. Their results can be used to maximize the ratio of vertical compression to uplift resistance. An alternative mitigation solution involves applying the uplift load with an eccentric movement to facilitate breakaway at the mudmat.
invert and hence reduce the magnitude of the suction forces generated. From small-scale model tests, Reid (2007) reported a reduction up to 66% (compared to the centred uplift resistance) by applying the pullout load at the edge of the mudmat. Water jetting at the invert is also a proven method to reduce uplift forces for offshore jack-up rigs embedded foundations, as demonstrated by Gaudin et al. (2011). However, the logistics associated with the jetting method are significantly more complex and costly than typical lifting devices.

Chen et al. (2012) presented a comprehensive investigation of the uplift resistance of mudmats, combining the effects of eccentric uplift, loading rate and skirt length in a model test programme performed in a geotechnical centrifuge. Chen et al. (2012) demonstrated that the uplift resistance was directly correlated to the development of suction at the mat invert and that fully undrained conditions (characterised by a full reverse end bearing mechanism) were achieved at normalised uplift velocities three orders of magnitude higher than those usually considered for shallow foundations in compression. This is because a suction relief mechanism develops at the foundation-soil interface during uplift, but while the system is in compression, the pore pressure dissipation mechanism is governed by pore pressures in the far field (Lehane et al. 2008). In contrast, fully drained conditions that would lead to low uplift resistance require uplift rates too slow to be practically undertaken in-situ and partially drained conditions may be prevalent during uplift of prototype mudmats. Accordingly, the prediction of uplift resistance is hindered by difficulties in assessing the relevant drainage conditions and the associated bearing capacity factor. Additional results from Chen et al. (2012), associated with eccentric uplift, indicated that a different failure mechanism was taking place, favouring the suction relief mechanism and hence contributing to a significant reduction in the uplift resistance.

In this paper, a series of model mudmat tests performed in a geotechnical drum centrifuge is presented. The research aims to advance Chen et al. (2012)'s study by linking perforation and uplift eccentricity in order to (i) further understand the mechanism governing suction development at the invert of a perforated mudmat, and (ii) provide recommendations to optimise a retrieval strategy in order to minimise the uplift resistance and the associated risk and cost. In particular, the generation of suction at the mat invert and the uplift force versus displacement curves were monitored during centrifuge model
Effect of perforations on uplift capacity of skirted foundations on clay tests, and were considered as a function of the effective width, the mudmat skirt length and the uplift eccentricity.

8.2 Determination of uplift capacity

As detailed in Chen et al. (2012), the ultimate uplift resistance of mudmats is controlled by the operative shear strength of the soil and the failure mechanism during uplift. The failure mechanism can be assumed to be either a reverse end bearing type (Craig and Chua 1990; Acosta-Martinez et al. 2008; Gourvenec et al. 2009; Randolph et al. 2011; Mana et al. 2012) or a breakout hemispherical type (Yu 2000; Rattley 2007) depending on the level of suction mobilised at the mat invert. Following the compression convention, the uplift capacity of mudmats in clay can be expressed as

\[ u_c = N_c s_{uop} - \gamma' h \]  

(8.1)

where \( N_c \) is the bearing capacity factor, \( s_{uop} \) the operative shear strength of the soil at the skirt tips, \( \gamma' \) the submerged unit weight of the soil and \( h \) the skirt length, which accounts for the embedment of the foundation. The second term on the right hand side of the equation is the correction for overburden. For skirted foundations, the overburden stress is cancelled by the weight of soil column incorporated by the skirts (see Figure 8.1). Therefore, the uplift capacity of a skirted mudmat, regardless of the failure mechanism, can be determined by

\[ u_c = N_c s_{uop} \]  

(8.2)

Rigorous solutions to determine the bearing capacity factor of a strip footing on homogeneous clay under vertical loading have been developed by Prandtl (1921) and Reissner (1924) and yielded a value of \( N_c = 5.14 \). In non-homogeneous soil, \( N_c \) increases with the soil heterogeneity \( kB_0/s_{um} \), where \( B_0 \) is the width of the footing, \( k \) the gradient of the soil profile and \( s_{um} \) the initial soil undrained strength at the mudline (Davis and Booker 1973; Randolph et al. 2004), as illustrated in Figure 8.1. The full reverse end bearing capacity can be assessed using solutions derived from the undrained compression
capacity (Equation (8.2)), since for fully undrained conditions, uplift and compression capacities are theoretically equal.

The ultimate bearing capacity of two or more parallel strips has also received attention from Martin and Hazell (2005), Gourvenec and Steinepreis (2007) and Bransby et al. (2010), providing insights into the effect of perforations on the bearing capacity of shallow foundations under undrained soil conditions.

The operative shear strength of the soil, \( s_{u\text{op}} \), is taken as the shear strength at the skirt tip, \( s_{u0} \), which can be determined using the standard T-bar test (Stewart and Randolph 1991; 1994), potentially enhanced by soil strain rate effects. Einav and Randolph (2005) and Lehane et al. (2009), among others, reported that the soil strength increases with strain rate by approximately 5% – 20% per log cycle of increasing strain rate. This can be expressed as

\[
\frac{\sigma_{u\text{op}}}{\sigma_{u\text{ref}}} = 1 + \mu \log\left(\frac{\dot{\gamma}}{\dot{\gamma}_{\text{ref}}}\right)
\]

where \( \sigma_{u\text{op}, \text{ref}} \) is the soil shear strength at a reference strain rate \( \dot{\gamma}_{\text{ref}} \) (which can be taken as \( \sigma_{u0} \) from the T-bar test) of 0.0001 s\(^{-1}\), and \( \mu \) is a rate parameter of approximately 0.1 for normally consolidated kaolin clay (Randolph et al. 2005). Atkinson (2000) suggested that the average operational strain rate underneath a rectangular shallow foundation subjected to vertical loading can be approximated as \( \nu/3B_0 \) (where \( \nu \) is uplift velocity and \( B_0 \) is the width of the mat). Assuming that full contact is maintained between the foundation and the soil during uplift, a similar approach may be assumed for the present scenario.

The uplift force during model tests can therefore be expressed as

\[
F_{\text{up}} = N_c s_{u\text{op}} A + G'
\]

where \( F_{\text{up}} \) represents the peak uplift force and \( G' \) the submerged self-weight of the mudmat. Note that in the present study, the gross area \( A \) is used to calculate uplift force regardless of the configuration of perforations.
8.3 Experimental set-up

8.3.1 Facility

The drum centrifuge at the Centre for Offshore Foundation Systems (COFS), The University of Western Australia (UWA) was used to carry out the described tests, as it enables multiple mudmat uplift tests to be conducted in one single soil sample. The ring channel of the centrifuge has an outer diameter of 1.2 m, an inner diameter of 0.8 m and a channel height (sample width) of 0.3 m. A servo-controlled actuator was mounted on the central tool table to provide both vertical and radial movements. The tool table can be coupled to the channel or may rotate independently of it, allowing it to be stopped for examination or changing the tool, without affecting the soil sample. A complete technical description of this centrifuge is presented in Stewart et al. (1998). Tests were performed at a centrifuge acceleration of 150g, i.e., all model linear dimensions are scaled by 150 and all loads by 150² (see Garnier et al. (2007) for details on similitude principles).

8.3.2 Model configurations, instrumentation, and calculation of effective widths

Three types of model mudmats were fabricated using aluminium plates, with dimensions of 5 mm in thickness (d), 100 mm in length (L₀) and 50 mm in width (B₀). This represents a prototype mudmat 15 m long and 7.5 m wide. The overall dimensions are identical to models tested by Chen et al. (2012).

One non-perforated model (labelled B) and two types of perforated models (labelled P1 and P2) were considered (see Figure 8.2). Model P1 featured large perforations with 36 circular holes 6.0 mm in diameter (Figure 8.2b). The second model, P2, featured small perforations, comprising 171 circular holes 2.7 mm in diameter (Figure 8.2c). Both perforated models had the same perforation ratio, α, of 0.19, defined as the ratio of the area of the holes to the gross area. Each perforated model was made with removable skirts with a length (h) of 0 mm, 5 mm and 10 mm (0 m, 0.75 m and 1.5 m in embedment prototype, respectively), while the non-perforated mudmat models were fabricated with
the same skirt lengths for benchmarking. Both model and prototype dimensions are summarised in Table 8.1.

The models were equipped with three Pore Pressure Transducers (PPTs), as illustrated in Figure 8.2 and Figure 8.3, to monitor variations in excess pore pressure at the foundation invert. As the PPTs’ housing was too large to be fitted between perforations, they were installed in place of a single perforation as illustrated in Figure 8.2b and c. Note that extra houses were mounted onto mats P2 to install the PPTs, thus resulting in slightly heavier weights than mats P1, as indicated in Table 8.1. In order to examine the effect of central and eccentric uplifts, three small holes (illustrated in Figure 8.2a) were drilled to allow a vertical ball shaft to be screwed onto the model plate and connected to a loading cell by a tong (illustrated in Figure 8.3). In each test, only one hole was used to mount the ball shaft and the other two were glued to avoid extra perforations. Uplift was applied via the ball shaft and the uplift resistance was measured by a 500 N capacity load cell.

The effective width \( W \) of each model was defined to represent the average length of drainage paths between perforations (White et al. 2005). For mudmats with circular perforations, the effective strip width \( W \) was calculated as

\[
W = \frac{x + \sqrt{2}(x + d_0) - d_0}{2}
\]  

where \( d_0 \) is the diameter of circular perforations and \( x \) represents the shortest drainage path between the perforations, as illustrated in Figure 8.4. From Equation (8.5), it was calculated that \( W \) is 6.07 mm (0.91 m in prototype) for mudmat P1 and 3.34 mm (0.5 m in prototype) for mudmat P2. For the non-perforated mudmat B, \( W \) is simply taken as the average of length and width, e.g., \( (B_0 + L_0)/2 = 75 \) mm (11.25 m in prototype).

### 8.3.3 Soil sample preparation and characterisation

Two soil samples were prepared for the present study. Kaolin slurry, prepared at a water content of ~ 120% (approximately twice the liquid limit), was poured into the centrifuge channel under an acceleration of 20g, over a preplaced 10 mm thick drainage blanket at the bottom. Self-weight consolidation under two-way drainage was achieved by spinning
the centrifuge at 150g for approximately four days. The degree of consolidation was monitored by measuring excess pore pressure dissipation via PPTs located at the bottom of the channel and settlement of the top surface of the soil. After full consolidation was achieved, a soil layer 5 to 15 mm thick was scraped off the surface to create a lightly over-consolidated soil sample with an evenly elevated surface, enabling a good contact between the model and the soil. The final height of both samples was 150 mm (including the drainage layer).

T-bar tests were performed in both soil samples to evaluate the undrained shear strength. Tests were carried out by using a 5 mm diameter, 20 mm long T-bar at a standard penetration rate of 1 mm/s, ensuring undrained soil conditions (Stewart and Randolph 1991; 1994). As a first approximation, a constant bearing factor $N_{T-bar} = 10.5$ derived from plastic solution (Randolph and Houlsby 1984) and experimental calibration (Low et al. 2010) were adopted to convert the measured T-bar resistance into undrained shear strength. Following the procedure proposed by White et al. (2010), lower bearing factors were applied to characterise the T-bar penetration resistance at shallow depths, where full flow of soil around the T-bar cylinder cannot occur. A cyclic test was also included as part of each penetration test to obtain accurate calibration data for soil penetration resistance (Randolph et al. 2007).

Figure 8.5 summarises the corrected undrained shear strength profiles at prototype scale in both soil samples. In general, soil strength profiles in both soil samples exhibited an excellent repeatability, with sample two featuring a more linear increase in strength with depth. The corrected soil strength for both samples can be idealised as bilinear profiles. At shallow depths ($z \leq 0.75$ m and $z \leq 0.6$ m for samples one and two, respectively), the soil samples were over consolidated following the trimming process and exhibited a constant shear strength with depth, with values of $s_u \sim 3.26$ kPa and $s_u \sim 1.68$ kPa for samples one and two, respectively. Soil strength at higher depths can be idealised by linear profiles with gradients of $k \sim 1.01$ kPa/m for sample one and $\sim 1.06$ kPa/m for sample two, resulting in a heterogeneity ratio of $kB_0/s_{um} \sim 2.3$ and $\sim 4.7$, respectively.
8.3.4 Testing programme and procedure

Nine central uplift tests were performed in soil sample one and nine eccentric uplift tests were performed in sample two, both under a centrifuge acceleration level of 150g, as summarised in Table 8.2. Model mudmats were installed on the soil surface at 1g and consolidation under the weight of the foundation was achieved at 150g. During consolidation, the tiny gap (~ 0.7 mm) caused by the curved clay top and flat model base can be closed during installation and consolidation, thus ensuring full contact at the interface between soil and foundation. A constant uplift velocity of \( v = 3 \text{ mm/s} \), which coincided with that adopted by Chen et al. (2012), was applied to the model once all excess pore pressures at the mat invert were fully dissipated, indicating that full consolidation under self-weight had been achieved. A constant water table of 50 mm above the soil surface was maintained during each test.

8.4 Test Results

8.4.1 Typical measurements of uplift force and excess pore pressure

Typical central uplift load/displacement and excess pore pressure/displacement curves are presented for tests S1-1 and S1-4 in Figure 8.6. The general patterns were consistent with Chen et al. (2012)’s observations that uplift resistance experienced a sudden increase to reach a peak value \( F_{up} \) over a short distance \( w_p \), then reduced to a semi-residual value which was slightly higher than the submerged self-weight \( G' \) of the model mudmats due to the soil attached at the model invert. \( G' \) differed between tests due to the different skirt lengths and configurations of perforation (see Table 8.1). \( G' \) also changed slightly with uplift displacement due to the changing acceleration level along the radius in the centrifuge (see dashed line in Figure 8.6), and this has been accounted for in the analysis.

The excess pore pressure displacement curves exhibit the same pattern as the load displacement curves, indicating a close correlation between excess pore pressure generation at the foundation invert and the uplift resistance. The negative values indicate the generation of suction at the mudmat invert, with peak values represented by \( p_1, p_2 \) and
Effect of perforations on uplift capacity of skirted foundations on clay

$p_3$ being coincident with the peak uplift resistance, indicating that uplift resistance is sustained by the development of suction at the mudmat invert. It is noteworthy that the uplift force for perforated mudmats, e.g., S1-4 and the associated suction at the mat invert is less sustainable compared to that for non-perforated mudmats, e.g., S1-1. This is attributed to shortening of the drainage path resulting from perforation and the associated acceleration in the dissipation of excess pore pressures. More details on the effects of perforations will be provided in Section 8.4.2.

The peak values of the uplift forces ($F_{up}$), the peak value of the excess pore pressures monitored by the three PPTs ($p_1$, $p_2$ and $p_3$) and their average values $\overline{p} = (p_1 + p_2 + p_3)/3$, and the distance travelled to reach the peak uplift force ($w_p$) for all the eighteen uplift tests are summarised in Table 8.2 for further interpretation.

The distances required to reach peak uplift forces, $w_p$, are presented in Figure 8.7 as a function of the skirt length. The operational distance decreased with decreasing skirt length, regardless of the configuration of perforations, and both perforated mudmats exhibited a significantly lower operational distance during uplift compared to the non-perforated mudmat. Figure 8.8 provides some insight into the secant stiffness ($E_s$) of the soil under vertical uplift, calculated as the normalised peak extraction resistance $F_{up}/A$ divided by the normalised skirt displacement $w_p/B_0$. Mudmats with perforations generated a stiffer response than non-perforated mudmats, while the stiffness for all mats was reduced with increased skirt length (Figure 8.8). This occurred because mudmats with perforation and shallower skirts generate a much shallower compatible strain filed. As shown in Figure 8.7, peak uplift force occurred faster for the perforated mudmats for the same skirt length (while uplifted at the same velocity), suggesting that the perforated design could be a promising method for saving uplift expenses by reducing operating time in the field.

### 8.4.2 Effect of perforation combined with skirt length

Figure 8.9 presents the net peak uplift forces, $F_{up,net} (= F_{up} - G)$, normalised by the gross area (i.e., $F_{up,net}/A$) and the corresponding peak values of average excess pore pressures ($\overline{p}$) varying with the effective width for central uplift tests. It is evident that the peak
uplift force decreases with reducing effective width and shallower skirt embedment. The peak uplift forces for tests on the perforated mudmat (P1) were reduced by almost half compared to the non-perforated mudmat (B), indicating that the perforation had beneficial effects in reducing the uplift resistance of mudmats. For a same perforation ratio of 0.19, the reduction in effective width resulted in a further reduction of uplift resistance of about 30%. For a specific configuration of perforation, the reduction in skirt length resulted in a reduction of the uplift force by up to 50% for the largest effective width. This improvement significantly reduced, however, with reduced effective width. As anticipated, this reduction of peak uplift force either was associated with a concomitant reduction in average peak suction, due to the shortening of the drainage paths by perforations or decreased skirt embedment, which accelerated the dissipation of the negative excess pore pressure generated by the uplift mechanism.

The net peak uplift forces (normalised by gross area $A$) are also plotted against the associated average suctions in Figure 8.10. Figure 8.10 demonstrates that the uplift resistance was essentially sustained by the suction at the foundation invert, independent of the skirt length and the perforation. Consequently, the mudmat failure mode was a reverse end bearing failure mechanism (see illustration in Figure 8.10), as observed and described by Craig and Chua (1990), Acosta-Martinez et al. (2008), Gourvenec et al. (2009), Randolph et al. (2011) and Mana et al. (2012) rather than a breakout contraction type mechanism (Yu 2000; Rattley 2007). This demonstrates that fully undrained conditions are experienced by the soil during uplift. Drainage conditions may be assessed by calculating the dimensionless velocity $vB_0/c_v$ (Finnie and Randolph 1994; Chung et al. 2006), where $c_v$ is the coefficient of consolidation of the soil, typically equal to 1.5 m$^2$/year for kaolin clay at a stress level of about 10 kPa (House et al. 2001).

In the present study, the dimensionless velocity for the non-perforated mudmat was about 3000, where undrained soil conditions for uplift can be assumed according to Chen et al. (2012). The dimensionless velocity for perforated mudmats was approximately one order less than for non-perforated mudmats if normalised by the effective width $W$, i.e., $vW/c_v$ ~ 400 and ~ 200 for P1 and P2, respectively. This potentially leads to partially drained conditions within the soil that would explain the lower uplift capacity. This is however inconsistent with observations from Figure 8.10, and will be discussed further in the following paragraphs.
To provide further insights into the drainage conditions and failure mechanisms associated with skirt length and the configuration of perforations, the bearing capacity factors for central uplift tests have been calculated from Equation (8.4) and are summarised in Table 8.3. Figure 8.11 presents the bearing capacity factors for non-perforated mudmats (B) as a function of skirt length in comparison with limit analysis results from Randolph et al. (2004) for circular foundations and experimental results from Chen et al. (2012). Results from Randolph et al. (2004) are presented for a soil heterogeneity of $kB_0/s_{um} = 0$, 3 and 10, encompassing the heterogeneity of the soil samples. The bearing capacity factors for non-perforated mats ranged from 6.84 to 8.37 with skirt length ratio ($h/B_0$) varying from 0 to 0.2. This agrees well with those obtained by Chen et al. (2012) in soil samples of a similar heterogeneity ratio (ranging from 3.38 to 3.61) indicating good repeatability of the present tests. They also compare reasonably well with the limit analysis solutions of Randolph et al. (2004), although there is a trend for an overestimation of the bearing capacity factor for flat foundations (i.e., $h/B_0 = 0$).

Figure 8.12 presents the bearing capacity factors for all the three model mudmats as a function of the effective width. There is an evident trend of reduction of bearing factors with reduced effective width. It is also noteworthy that the effect of the embankment, which increases bearing capacity factors (see Randolph et al. (2004)), reduced as the effective width decreased. As mentioned previously, the reduction in bearing capacity factors could be attributed to an accelerated dissipation of excess pore pressures with increased occurrence of perforations. However, the load/displacement curves in Figure 8.6, and pullout stiffness in Figure 8.8, indicate that perforated mats exhibited a stiffer load displacement response, and a faster generation of suction at the foundation invert. Both observations demonstrate that the drainage conditions for perforated mats were also undrained, and that the reduction in uplift capacity (and associated bearing capacity factors) was essentially due to an earlier onset of suction breakaway at the mat invert caused by the perforations.

No theoretical solutions have been established to determine bearing capacity factors for perforated mudmats. The closest solution is the one developed by Martin and Hazell (2005), who established bearing capacity factors using the method of characteristics for two-dimensional surface multi-strip footings subjected to downward vertical loadings under undrained conditions. Results from Martin and Hazell (2005) are plotted in Figure
8.13 for soil heterogeneity ratios ranging from 0 to 5. They indicated a trend of reducing bearing capacity factor with increasing perforation ratio, beyond a value that depends on the strength heterogeneity ratio.

The perforation ratio used by Martin and Hazell (2005) in Figure 8.13 is defined under two-dimensional plane strain condition as the ratio of total footing spacing to the total width, so a distinction cannot be made between the perforation ratio and the effective width, as for the three-dimensional models. In order to enable a direct comparison with the experimental results, an equivalent perforation ratio $\alpha^*$ was calculated for mudmats P1 and P2, as illustrated in the inset in Figure 8.13. The equivalent perforation ratio $\alpha^*$ was calculated by converting the shaded area $A_{sh}$ into an equivalent width $y$, resulting in values of 0, 0.28 and 0.23 for mudmat B, P1 and P2, respectively. Bearing capacity factors for the three mudmats are plotted in Figure 8.13, considering the equivalent perforation ratio, for comparison with results from Martin and Hazell (2005).

Bearing capacity factors for the non-perforated mat agreed reasonably well with results from Martin and Hazell (2005), accounting for the effects of heterogeneity ratio and skirt length. The agreement was also satisfactory for the perforated mat P1, but not for the perforated mat P2, as Martin and Hazell (2005) only modelled two-dimensional strips that cannot account for the effect of different perforated patterns. Nevertheless, the results suggest that Martin and Hazell (2005) might be used as a first approximation to evaluate the effect of perforation on uplift capacity, provided that the effective width is not reduced by more than a factor of 10, compared to a plain foundation of identical overall dimensions.

### 8.4.3 Effect of eccentric loading combined with perforation

Figure 8.14 presents typical variation of uplift force and excess pore pressures with uplift displacement for the eccentric uplift test S2-7. The mudmat experienced a rotational soil failure mechanism (as illustrated in Figure 8.14) about a point located close the centre of the mudmat. This resulted in positive excess pressures being mobilised at the end farthest from the lifting side (instrumented with PPT1) and negative excess pore pressures at the lifting side (instrumented with PPT3). The peak excess pore pressures at the mat for all the eccentric uplift tests (refer to Table 8.2) is presented in Figure 8.15.
Figure 8.15 presents the approximate excess pore pressure profile along the length of the mudmats ($L_0$) at failure. Note that PPT3 in test S2-6 ceased to function during the test, so no data is available (Figure 8.15c). It can be seen that the perforation did not change the general failure mechanism as detailed above, which remained rotational. The perforation lead to lower suction being generated at the uplift side, but is unlikely to have significantly affected the excess pore pressure on the opposite side, indicating that they are most likely generated by the increase in bearing pressure resulting from the self-weight of the foundation being applied on a smaller section of the mat as it is being uplifted. It is also noteworthy that the centre of rotation of the mudmat moves away from the lifting point with increasing skirt length. As the skirt length increases, a deeper failure mechanism is generated, with breakaway at the mudmat invert occurring later during uplift. Compared to the central uplift cases, the distance to reach peak loads for eccentric uplift cases are more sensitive to skirt configuration of mudmats. As shown in Table 8.2, the peak uplift distances was enlarged by 2.6 to 6 times for perforated mudmats with the longest skirt ($h/B_0 = 0.2$).

### 8.5 Discussion and recommendations

Table 8.4 summarises the ratio of measured uplift resistance to central uplift resistance for all tests. The reported values enable comparison between eccentric and centric uplift tests, and configurations of perforations. For the non-perforated mudmat, the eccentric uplift considered reduced the peak resistance by 66% to 79% compared to central uplift tests, decreasing with increasing skirt length due to the deeper failure mechanisms mobilised with longer skirts. When a large number of small perforations were introduced (mudmat P2), the peak uplift resistance was reduced by about 74% compared to the central uplift of non-perforated mudmats, with less effect from the skirt length. In contrast, a small number of large perforations (mudmat P1) yielded a reduction in uplift capacity of only 45% indicating that the effective width is the relevant parameter when determining the effect of perforation. In summary, the results indicate that eccentric uplift appears to be more efficient in reducing the uplift resistance than perforation ratio (for the range considered in this study), although both reduce the uplift capacity by generating an early breakaway at the foundation invert. Eccentric uplift is indeed more efficient in reducing the uplift capacity, as the breakaway can propagate more rapidly along a larger
Chapter 8  Effect of perforations on uplift capacity of skirted foundations on clay surface. However, the efficiency of eccentric uplift is hindered by higher skirt embedment, whereas the central uplift capacity of perforated mudmat is less affected by skirt lengths.

When eccentric uplift and perforations are combined, the mudmats experience the highest reduction in uplift resistance, with a reduction of ~ 76% for mudmat P1 and ~ 93% for mudmat P2 (i.e., the uplift force is only slightly larger than the self-weight of the mat), with skirt length having only a relatively small effect.

8.6 Conclusions

A series of centrifuge tests were undertaken to assess the effect of perforations and loading eccentricity on the uplift capacity of subsea mudmats. The results demonstrated that the uplift capacity in all cases is essentially sustained by the generation of suction pressures at the mudmat invert, and that undrained soil conditions prevailed for all tests, regardless of the configuration of perforation. The reduction of uplift capacity, which can reach up to ~ 80%, results from the breakaway of suction at the foundation invert, which can be generated either by perforations or by eccentric uplift. Eccentric uplift was observed to have a much greater effect in reducing the uplift capacity than perforations, although the benefit reduces with increasing skirt embedment.

8.7 References


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### Table 8.1 Characteristics of model mudmats

<table>
<thead>
<tr>
<th>Mudmat type*</th>
<th>Perforated ratio, $\alpha$</th>
<th>Skirt length, $h$</th>
<th>Perforated diameter, $d_0$</th>
<th>Effective width, $W$</th>
<th>Submerged weight, $G'$</th>
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<td></td>
<td>(mm)</td>
<td>(m)</td>
<td>(mm)</td>
<td>(m)</td>
<td>(N)</td>
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<tr>
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<td>75.00</td>
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<td>6.07</td>
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<td>2.7</td>
<td>3.34</td>
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* B = mudmat without perforations; P1 = mudmat with large perforations; P2 = mudmat with small perforations
Table 8.2 Summary of mudmat tests

<table>
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<th>Soil sample &amp; test no.</th>
<th>Mudmat type</th>
<th>Skirt length, ( h ) (mm)</th>
<th>Uplift eccentricity, ( e ) (mm)</th>
<th>Peak uplift force, ( F_{up} ) (N)</th>
<th>Peak suction, ( p_1 ) (kPa)</th>
<th>Peak suction, ( p_2 ) (kPa)</th>
<th>Peak suction, ( p_3 ) (kPa)</th>
<th>Average suction, ( \bar{p} ) (kPa)</th>
<th>Peak uplift distance, ( w_p ) (mm)</th>
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Table 8.3 Bearing capacity factors inferred from central uplift tests

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Table 8.4 Ratio of uplift resistance to central uplift resistance for all mudmat tests

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<th>P1 $e/L_0 = 0.4$</th>
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</tbody>
</table>

Note: $e/L_0$ = eccentricity
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Figure 8.1 Symbols and notations
Figure 8.2 Model mudmats (a) without perforations (B), (b) with large perforations (P1) and (c) with small perforations (P2)
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Figure 8.4 Calculation of effective width between perforations
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Figure 8.9 Net uplift resistance and average suction varying with effective width for central uplift tests (Mudmat type: B = no perforation; P1 = big perforations; P2 = small perforations)

Figure 8.10 Net uplift resistance varying with average suction pressures (Mudmat type: B = no perforation; P1 = big perforations; P2 = small perforations)
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Figure 8.11 Comparison of bearing capacity factors for varying skirt length

Figure 8.12 Bearing capacity factors varying with effective width (Mudmat type: B = no perforation; P1 = big perforations; P2 = small perforations)
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(S2-7: small perforations, no skirt, eccentricity = 40 mm)
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(Mudmat type: B = no perforation; P1 = big perforations; P2 = small perforations)
CHAPTER 9 CONCLUDING REMARKS

9.1 Summary of major outcomes

This research is concerned with the generation and dissipation of suction in clay soils associated with the uplift of offshore shallow foundations. Various influential factors, e.g., foundation geometry, operational condition and loading history, have been investigated to develop a thorough understanding of the uplift mechanism and predict the uplift resistance more accurately. The major outcomes of the research are:

- The uplift resistance of shallow foundations is highly dependent on the suction generation level at the foundation invert, which is affected by the soil drainage conditions.

- The uplift velocity required to mobilise fully undrained condition is approximately one order of magnitude higher than in compression due to the changes in boundary conditions associated with the shortening of the drainage paths during uplift.

- The uplift capacity of shallow foundations is highly dependent on the initial effective stresses and excess pore pressures field. The magnitude of preloading and the degree of consolidation achieved during the preloading stage govern the remoulding and subsequent enhancement of the soil strength, which further affects the subsequent uplift capacity.

- The breakout of shallow foundations is caused by either the reduction in effective stress at the foundation/soil interface (from drainage and dissipation) or the development and propagation of a crack at the interface (from a change in boundary conditions around the foundation). It further controls the magnitude and the sustainability of the uplift resistance.

- The combination of perforation and eccentric uplift leads to a significant reduction in uplift capacity due to early breakout. This potentially provides an optimal strategy to retrieve subsea mudmat foundations.
These outcomes of the research have been achieved through complementary centrifuge testing and finite-element modelling. The remaining part of this chapter summarises the detailed findings of the research and makes recommendations for further research.

9.2 Detailed findings

9.2.1 Uplift of shallow foundations under a range of soil drainage conditions

While previous studies have primarily focused on the undrained uplift of shallow foundations, the present study extends the existing research into considering the effect of different soil drainage conditions on the uplift resistance and suction development. The shallow skirted foundations were uplifted with velocities across four orders of magnitudes. As such, the suction generation and uplift resistance under fully undrained, partially drained to fully drained conditions can be observed. Both circular and square shaped foundations were tested, showing only minor discrepancies between them. The transition of the uplift resistance from fully undrained to partially drained conditions was identified around a normalised velocity \( (vD/c_v) \) of 200, which is approximately one order of magnitude higher than that for foundations under compression. This is attributed to the shortened drainage path as the foundation was uplifted and the soil around the foundation tended to move downwards to maintain volume continuity. The uplift resistance increases with increasing velocity, and the trends are described as backbone curves, where the effects of partially drained soil conditions and viscous effects are quantified. The soil drainage condition has significant effects on the uplift failure mechanism of shallow foundations as well. A reverse Hill-type failure mechanism is mobilised under fully undrained conditions. However, the failure mechanism can be reduced in size and transfer into a Hill-sliding failure type with increasing drainage in the soil.

A coupled finite element modelling has been performed to simulate the uplift of shallow foundations under fully undrained, partially drained to fully drained conditions. The load-displacement response demonstrates a softening in stiffness and starts to decrease after a peak under partially drained uplift, which can be interpreted within a critical-state framework. The established model can well capture the suction generation under various
uplifting rates. The transition from the fully undrained to partially drained condition is consistent with that obtained by the centrifuge test. The excess pore pressure dissipation in the circular foundation is slightly faster than that in strip foundation because the former is unconfined and has three-dimensional boundaries.

9.2.2 Effect of preloading and consolidation on the subsequent uplift capacity

The uplift capacity of shallow foundations is highly dependent on the preloading and its associated consolidation prior to uplift, which can be caused by the self-weight of structures, ballasting or active installation. The effects of different magnitudes and durations of preloading on the subsequent uplift capacity of a shallow skirted foundation were examined experimentally and numerically, with the focus on the generation and dissipation of the excess pore pressures in lightly over-consolidated clay. The uplift capacities were presented as ratios to the reference value of no preloading degree of consolidation and as a function of the preloading level and degree of consolidation. The results imply that the application of a compressive preloading decreases the subsequent uplift capacity because of the remoulding and strength degradation of the clay. The strength can be recovered and enhanced by the subsequent consolidation under preloadings. Both the extent of remoulding and the rate of recovery are amplified by higher preloading levels. The variation in the shear strength of clay was interpreted using a critical-state framework as a function of the generation and dissipation of the excess pore pressure during the preloading and consolidation stage. The framework is able to capture the main features of the strength variations obtained from the centrifuge test.

In addition, the effects of preloading history on the subsequently uplift capacity were simulated with the established finite-element model as mentioned in Section 9.2.1. Compared with experimental tests, more comprehensive parameter studies were conducted, and the results were presented as dimensionless forms and interpolated by empirical relations. The outcomes have established the benchmark for future development of more realistic model on uplift of offshore shallow foundations.
9.2.3 Breakout failure mechanism of shallow foundations

Breakout is a common phenomenon when uplifting shallow foundations. It controls the magnitude and the sustainability of uplift forces. The breakout failure mechanism of shallow foundations under both transient and sustained uplift has been investigated. It is found that the breakout can occur prior to or after a peak resistance is mobilised, depending on the soil drainage and boundary conditions around the foundation. Under sustained uplift, the foundation moves linearly with time, and breaks out as the result of the loss of suction pressure underneath the foundation. The observation on suction pressure variations beneath the foundation indicates that the breakout of shallow foundations could be attributed to either the reduction in effective stress at the interface of the soil and the foundation or the crack propagation at the foundation invert.

9.2.4 Comparison between compression and uplift

Both the centrifuge testing and finite-element modelling indicate that a reverse end bearing failure mechanism can be mobilised in the undrained uplift of shallow foundations, which results in a similar ultimate capacity as in compression. This warrants the use of a unique bearing capacity factor for compression and uplift of shallow foundations under fully undrained conditions. However, finite-element modelling results demonstrate that different excess pore pressure generation levels and distributions are observed between compression and uplift, because of the different interactions of excess pore pressures due to the change of mean total stress and deviatoric stress in the soil. The difference in the excess pore pressure development further affects the foundation behaviour under partially drained compression and uplift.

9.2.5 An optimal strategy to retrieve subsea foundations

The retrieval of deep-water subsea mudmat foundations can be challenging and costly because of the suction generation underneath the foundation during uplift. Centrifuge tests were performed to provide a first insight into the mechanism of suction generation and dissipation during uplift, from which recommendations were established. Typical configurations of the mat perforation and skirt length were considered along with the eccentricity of uplift. The results demonstrated that both perforation and uplift
eccentricity significantly reduced the uplift resistance by developing a suction relief mechanism beneath the foundation. The eccentric uplift appears more efficient than the perforation but more sensitive to the skirt length. The eccentric uplift near the edge of the foundation could reduce the peak uplift resistance by 66 – 79% compared to the corresponding central uplift cases, whereas the investigated perforations yielded a reduction of up to 74% relative to the non-perforated cases. The combination of perforation and eccentric uplift results in the highest reduction of lifting resistance, accounting for up to 93% reduction compared to the central uplift of non-perforated mudmats, with a smaller effect from the skirt length. This means the uplift force is reduced to slightly larger than the self-weight of the mudmats, potentially indicating an optimal strategy to remove subsea mudmat foundations from the seabed.

9.3 Recommendations for future research

9.3.1 Complex interface setting and post failure behaviour

In the present finite-element modelling, the interface between the foundation invert and the soil is perfectly bonded, so that no separation is allowed regardless of how the drainage condition changes during the uplift. This setting is reasonable for investigating foundation uplift under fully undrained conditions, because the suction is sufficiently high to enable the soil to move with the foundation. However, this setting may overestimate the uplift resistance when excess pore pressure dissipation dominates the foundation behaviour during partially drained uplift. Therefore, a more advanced interface model is required to simulate the progressive sliding along the skirts of the foundations and separation of foundations under partially drained uplift.

Furthermore, the centrifuge tests imply that an unexpected loss of suction may occur as dramatic boundary changes because of the perforation or shortened skirt length. This loss of suction can result in a foundation breakout earlier than when an ultimate bearing capacity can be mobilised. To more realistically simulate the foundation breakout mechanism, an interface model is required, which can describe the dramatic changes in boundary conditions and reduction in suction. Such a model could also simulate post failure behaviour.
9.3.2 Validation and extension of finite element modelling

The finite-element analyses in this thesis are based on some benchmark cases, in which the foundation was simplified as a strip or circular surface foundation. This simplification hindered comparisons with the centrifuge results in a straightforward manner. As the first step of further development, the skirt configuration and lightly over-consolidated soil profile should be incorporated into the present model to allow the validation of the finite-element modelling using centrifuge data and the wider application into practical engineering designs.

In addition, the offshore shallow foundations can have various shapes that require three-dimensional modelling. The centrifuge tests also indicate that some features could not be considered by two-dimensional modelling, such as the spatial arrangement of the perforations on the mudmats under identical area ratio. Therefore, a three-dimensional analysis is also a direction for future study with the aid of improved hardware and solving techniques.

9.3.3 Combined loading analyses that incorporate the tensile component

Only the centred and normal vertical uplift of shallow foundations was considered in the present research, except for a brief discussion on the recovery of mudmat using an eccentric uplift. The uplift resistance may be affected by the horizontal and moment components of the uplift load, and this is an important aspect of potential future work. Most of the frameworks developed to investigate the combined capacity of shallow foundation consider a vertical compressive load. The development of a combined yield envelope considering a tensile vertical load and accounting for suction relief mechanisms would be of particular significance for pipelines and plate anchors.

9.3.4 Pore-pressure-related cyclic-loading behaviours

Cyclic loadings are commonly encountered offshore because of the repeatable nature of winds, waves and currents and on-off loading sequences. Nevertheless, only monotonic loadings were investigated in this thesis. It is widely acknowledged that displacements
accumulate under continuous cyclic loadings, and is accompanied by soil strength and stiffness degradation. The displacement accumulation through cyclic loading can also potentially facilitate foundation extraction. Furthermore, their relationships with the pore pressure response in the soil have been less investigated. Particularly, it is worthy examining the cyclic behaviours under partially drained conditions, where the accumulation level of excess pore pressures is highly diverse.

9.4 Summary

The work in this thesis provides a comprehensive study on the suction generation and dissipation in clay soils and its implications on the uplift behaviour of shallow foundations. Both fundamental and practical aspects of the problem were addressed. The results were obtained in general forms, so that they can be used to design offshore foundations that are subjected to uplift loadings and provide guidance for deep-water retrieval operations. The outcomes of this research can be applied to the uplift of other shallowly embedded foundations such as pipelines and anchors.