EXTENDED FRAMEWORK FOR PREDICTING THE BEHAVIOUR OF TOLERABLY MOBILE SUBSEA FOUNDATIONS

By

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BEng. MEng

This thesis is presented for the degree of Doctor of Philosophy of The University of Western Australia

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<th>Centrifuge testing of a tolerably mobile subsea foundation on calcareous silt, and comparisons with testing in clay</th>
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Shallow foundations or mudmats are widely used as the foundations for supporting subsea infrastructure, such as pipeline end terminations (PLETs) and flowline termination assemblies (FTAs). Conventionally, they are fixed on the seabed, sometimes with a sliding carriage or a skid that can slide over the fixed mudmat mounted on to accommodate, and to some extent resist, the horizontal periodic thermal forces applied by the supporting pipeline. In recent years, a greater number of offshore oil rigs have been deployed at deep-water sites with very soft ground conditions. In addition, operative vertical loads are increasing, which when combined with the presence of soft soil, results in foundations that are very large and heavy when designed using conventional methods – which in turn dramatically increases the fabrication and installation cost. The tolerably mobile subsea foundation concept, which allows the foundation to slide on the seabed with the pipeline end termination to accommodate the pipeline expansion and contraction and subsequently moderate the axial loads, has previously been proposed as a solution to reduce the foundation size and optimise installation.

This research experimentally and theoretically explores the whole-life behaviour of a mobile foundation. The work follows previous research by Cocjin (2016) and seeks to: (i) better understand the performance of mobile foundation on various soils under various operational conditions through model tests; (ii) improve the sliding foundation model to better simulate the softening and hardening property of soft soil and to reflecting the special soil properties at relatively low stress levels; and (iii) optimise the previously developed theoretical methodology to realise more detailed and better simulation on whole-life sliding resistance and settlement whilst using less back-calibrated inputs.

A number of centrifuge model tests were performed on a reconstituted calcareous silt. Combined with similar tests carried out previously on the kaolin clay commonly used in UWA, the responses in sliding resistance, settlement and rotation were analysed. The results show that the interface soil which determines the base sliding resistance of the mudmat softens from the peak undrained shear strength to the remoulded strength during the first slide due to soil sensitivity, and subsequently increases with reconsolidation cycles towards the drained limit. Due to the low consolidation coefficient, silt is observed to regain strength more slowly than the kaolin clay. Dormant berms are observed to form at the two extremities of the footprint, and exert large additional force on the foundation, leading to peaks in sliding resistance. Incremental shear and consolidation settlements decrease with number of cycles, due to strength regain/compaction of the near-surface soil. The normalised reconsolidation time and loading level are two key factors that influence the consolidation...
and shear settlement rate. Rotation was measured in the experiments presented but was generally less than 2 degrees and thus considered insignificant. In addition, special soil properties, measured at the stress levels relevant to the operational conditions of mobile foundation were observed, including the higher fictional angle (at low vertical stress) and larger compression index.

An extended theoretical framework, based on the model proposed by Cocjin (2016), has been developed to predict the sliding resistance that encompasses both base sliding and berm mobilisation resistance. The berm resistance was calculated by considering both the strength regain and volume accumulation and simulation results show that the strength gain plays a dominant role in the increase in berm resistance. The extended framework can capture the sliding resistance evolution both within and between cycles. The developed soil model has also been used in the calculation of consolidation and shear settlement, with further coupling the inconstant compression index and the unloading-reloading index. Shear and consolidation settlement were derived based on the deformation and strength distribution of the subsoil. The newly developed framework has removed the need to calibrate settlement parameters from an experiment, and overall the predictions generated by the model show good agreement with experimental measurements from two separate campaigns of testing performed in two different soil types.
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NOTATIONS

Roman

\( B \) is the breadth of the mudmat

\( b_b \) is the cross-sectional breadth of the mobilised triangle dormant berm

\( b_{dis} \) is the base length of the top isosceles triangle

\( c_h \) is the horizontal coefficient of consolidation

\( c_{ref} \) is the representative coefficient of consolidation

\( d \) is the drainage path length

\( D_{50} \) is the mean effective particle size

\( D_{T-bar} \) is the diameter of T-bar

\( e \) is the void ratio

\( e_0 \) is the initial void ratio

\( e_N \) is the void ratio intercept of NCL at \( \sigma'_v = 1 \) kPa

\( e_{LL} \) is the void ratio at liquid limit

\( e^*_{LL} \) is the void ratio at the inflection point of NCL

\( e^*_N \) is the void ratio intercept of the back-extrapolation of any point on the NCL at \( \sigma'_v = 1 \) kPa

\( e_{\text{t}} \) is the target error

\( F_{oc} \) is the over-consolidation factor

\( F_{oc0} \) is the initial value of the over-consolidation factor

\( F_{st} \) is the current proportion of the irrecoverable sensitivity

\( F_{st0} \) is the initial proportion of the irrecoverable sensitivity
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

\[ F_{\text{bar}} \] is the ratio of the intact strength to the T-bar measured peak strength

\[ G_s \] is the specific gravity

\[ h \] is the thickness of the mudmat

\[ H \] is the sliding resistance

\[ h_b \] is the height of triangular berm

\[ H_b \] is the berm resistance

\[ H^*_b \] is the maximum resistance given by the dormant berm

\[ H_{\text{base}} \] is the resistance of base plate of the mudmat

\[ h_{btm} \] is the height of the bottom triangle of the dormant berm

\[ H_p \] is the peak resistance

\[ h_s \] is the thickness of the shear band

\[ I_r \] is the rigid index

\[ I_a \] is the vertical stress distribution factor

\[ I_c \] is the shear stress distribution factor

\[ k \] is the strength gradient

\[ k_1 \] is the parameter controlling the slope of the shear settlement rate curve at low loading level

\[ k_2 \] is the parameter controlling the slope of the shear settlement rate curve at high loading level

\[ K_{\text{avg}} \] is the average dimensionless undrained shear strength gradient

\[ L \] is the length of the mudmat

\[ LL \] is the liquid limit

\[ m \] is a constant controlling the consolidation degree


$m_c$ is the moisture content

$M_0$ is the slope of the failure envelope in $\tau$-$\sigma'_v$ plane measured at high stress levels

$M_{\sigma'_v}$ is the strength factor at the effective stress of $\sigma'_v$

$N$ is the number of cycle

$N_c$ is the vertical bearing capacity factor

$N_{t,95}$ is the number of T-bar penetration cycles to attain 95% degradation from the peak undrained shear strength to the residual

$N_{t,i}$ is the number of cycles counted from the start of each packet of T-bar cycles

$OCR$ is the over consolidation ratio

$PL$ is the plastic limit

$R_{\sigma'_v}$ is the multiplier on the strength factor

$R_{cpt}$ is the radius of CPTu

$S_t$ is the soil sensitivity measured by T-bar

$S_{t,r}$ is the real soil sensitivity

$S^*_t$ is the effective sensitivity

$s_u$ is the undrained shear strength of soil

$s_{u,cyc}$ is the undrained shear strength measured by T-bar at the present cycle

$s_{u,n=0}$ is the intact undrained shear strength potentially measured by T-bar at the 0\textsuperscript{th} cycle

$s_{u,n=0.25}$ is the undrained shear strength T-bar measured during the first penetration pass (the 0.25\textsuperscript{th} cycle)

$s_{u,n=0.75}$ is the undrained shear strength T-bar measured during the first extraction pass (the 0.75\textsuperscript{th} cycle)

$(s_u/\sigma'_v)_{NC}$ is the undrained shear strength ratio measured by T-bar in normally consolidated soil
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

\((s_u \sigma'_v)_{NC,r}\) is the real undrained shear strength ratio of normally consolidated soil

\(s_{u,p}\) is the peak undrained shear strength

\(s_{u,p0}\) is the initial peak undrained shear strength mobilised prior to any remoulding event

\(s_{u,r}\) is the fully remoulded strength achieved on RSL

\(s_{u,s}\) is the undrained shear strength on the subloading surface

\(s_{um}\) is the undrained shear strength at mudline

\(t\) is time

\(T\) is the normalised time factor

\(T_{50}\) is the dimensionless factor for 50% of the consolidation settlement to occur

\(t_{con}\) is the consolidation time

\(U\) is the degree of consolidation defined in terms of the normalised time factor

\(V\) is the vertical load

\(V_{eff}\) is the effective vertical load

\(Vol\) is the volume of the dormant berm

\(Vol_{dis}\) is the volume of the displaced subsoil into the dormant berm

\(Vol_h\) is the volume of the heaved soil into the dormant berm

\(V_{op}\) is the vertical operational load

\(V_{ult}\) is the ultimate vertical bearing capacity

\(w\) is the settlement of the mudmat

\(w_{con}\) is the consolidation settlement of the mudmat

\(w_p\) is the shear settlement of the mudmat

\(w_{tot}\) is the total settlement of the mudmat
\( x^* \) is the relative distance to the turning point \( x_0 \)

\( x_0 \) is the turning point dividing the low and high loading level of the shear settlement rate curve

\( z \) is the present depth

\( Z \) is the maximum calculated depth

\( z_f \) is the failure depth

\( z_{fit} \) is the fitting depth

Symbol

\( \sigma_{v,eqm} \) is the equilibrium stress

\( (\sigma'_v)_NCL \) is the effective stress on normal compression line (NCL)

\( (\sigma'_v)_RSL \) is the effective stress on remoulded strength line (RCL)

\( \sigma'_{v,cs} \) is the effective stress achieved at the critical state

\( \sigma'_{v,gen} \) is the effective stress following generation of excess pore pressure

\( \sigma'_{v,dis} \) is the effective stress after dissipation

\( \sigma^*_LL \) is the effective stress at the inflection point of NCL

\( \sigma'_v \) is the vertical effective stress

\( \sigma'_{v,sur} \) is the surcharge pressure

\( \sigma_R \) is a parameter controlling the ultimate initial curvature of the non-linear failure envelope

\( \beta \) is the power of controlling the magnitude of pore pressure generation

\( \gamma \) is the plastic shear strain

\( \gamma_{95} \) is the cumulative plastic shear strain causing 95% remoulding on soil
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

\( \gamma' \) is the submerged unit weight

\( \gamma_w \) is the unit weight of water

\( \delta \) is the sliding distance

\( \delta_{50} \) is the sliding distance that the soil softens to 50% of its peak undrained shear strength

\( \delta_{95} \) is the sliding distance that the soil softens to 95% of its peak undrained shear strength

\( \delta_{\text{max}} \) is the maximum sliding distance of a single slide

\( \delta_p \) is the sliding distance required to mobilise the peak undrained shear strength

\( \delta_r \) is the post-peak remoulding distance

\( \delta_{r,\text{eq}} \) is the equivalent remoulding distance

\( \Delta e_{U=1} \) is the reduction in void ratio when the excess pore pressure fully dissipates

\( \Delta e \) is the reduction in void ratio

\( \Delta \text{Vol}_{\text{dis}} \) is the incremental volume collected into the dormant berm

\( \Delta u_{\text{dis}} \) is the change in pore pressure due to dissipation

\( \Delta z_{\text{fit}} \) is the increment of the fitting depth

\( \zeta \) is the power controlling the rate of accumulation of the equivalent remoulding distance

\( \theta \) is the rotation angle of the mudmat relative to horizontal with anticlockwise being positive

\( \theta_b \) is the base angle of the dormant berm

\( \theta_{\text{mat}} \) is the slope of the skis of the mudmat

\( \kappa \) is the slope of unloading-reloading line derived from the conventional high-stress 1D compression test

XXV
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

$k^*$ is the slope of unloading-reloading line changing with stress level

$\lambda$ is the slope of normal compression line (NCL) derived from the conventional high-stress 1D compression test

$\lambda^*$ is the slope of normal compression line (NCL) changing with stress level

$\lambda_{LS}$ is the slope of normal compression line (NCL) measured at low stress level

$\varphi$ is the friction angle

$\chi$ is the power term controlling the rate of damage of the irrecoverable sensitivity

$\Psi$ is the stress state factor

$\mu$ is the friction coefficient
CHAPTER 1 INTRODUCTION

1.1 Background

1.1.1 Motivation

Shallow foundations are widely used to support subsea structures, including flowline termination assemblies (FTAs) and pipeline end terminations (PLETs) which are used to connect subsea pipelines to the subsea production system, as shown in Figure 1.1. Most subsea shallow foundations (also termed mudmats) are rectangular in plan, and typically have external and internal skirts at the underside of the base plate to increase horizontal resistance (Feng et al., 2014a, Bransby et al., 2015, Bransby et al., 2017).

Subsea foundations are subjected to large vertical loads from pipelines, jumpers and assemblies (which may range from tens to hundreds tons). In addition, they may be required to resist large horizontal forces associated with expansion and contraction of the connected pipeline (Stuyts et al., 2015, Zhou et al., 2015, Hicks et al., 2016), which over the lifespan of the structure may result in many cycles of expansion and contraction (Carr et al., 2003, Carr et al., 2006, Deeks et al., 2014a). Figure 1.2 illustrates the typical movements at the hot and cold ends of an unrestrained pipeline (Rong et al. (2009)) which if restrained would lead to large horizontal forces).

The trend to deeper water means designing shallow foundations for soft soils (Colliat et al., 2010), and when combined with uncertainty (and complexity) in loading, has eventuated in extremely large foundations – which has inevitably escalated fabrication and installation costs (Hicks et al., 2016). To address this, foundation design strives for ever more efficient shallow foundation concepts.

1.1.2 Tolerably mobile subsea foundation

One possible solution to the challenge of reducing subsea shallow foundation footprint is the concept of tolerably mobile subsea foundations (Cathie et al., 2008, Randolph et al., 2011, Bretelle and Wallerand, 2013, Deeks et al., 2014a, Gourvenec and Feng, 2014, Cocjin et al., 2014, Zhou et al., 2015, Wallerand et al., 2015, Corti, 2016, Cocjin et al., 2017). In this case, rather than design the foundation to be fixed on the seabed (with limited displacement), they are allowed to slide within allowable limits – thereby accommodating the thermal forces applied by the pipelines.

In this thesis, the foundation considered does not have skirts, and utilises a sloping edge to promoting sliding. The foundation dimensions used in this project are based on the deployment of sliding foundation on the North-West Shelf of Australia (Deeks et al., 2014a). In this case, they were
designed to have two wings which can fold up along the central line (Zhou et al., 2015), allowing them to be installed with the pipeline and avoid separate installation costs (and complexity), as shown in Figure 1.3. As will be discussed later, a probable difference between the model foundation used in this study and those deployed offshore is the use of a rough base – in engineering practice, it is a trade off depending on what the designer needs – smooth base leads to large horizontal displacement but small settlements while rough base leads to smaller horizontal displacement but larger settlements.

However, there remain challenges to predict the behaviour of mobile foundations, and industry standard approaches for design are not available. For instance, the periodic monotonic motion of the foundation leads to the underlying soil being sheared with intervening resting times between loading stages. Cocjin et al. (2014) explored the behaviour of these mobile foundations through centrifuge model tests on a kaolin clay, with the results showing that sliding resistance increases due to periodic shearing and resting. In addition, soil berms accumulate at two extremities of the footprint, providing additional sliding resistance. It was also observed that settlement (and potentially rotation) accumulates over the lifespan of the foundation, albeit at declining rate. Large sliding resistance and/or embedment could lead to over-stressing of connected spools, increasing the risk of failure.

1.2 Research objectives

The three primary geotechnical design criteria for mobile foundation are described by Cathie et al. (2008) and further discussed by Deeks et al. (2014a), including (a) the foundation must have adequate vertical capacity and stiffness; (b) total settlement must remain within design tolerances; and (c) sliding resistance should be minimised. This thesis focuses on the whole-life sliding behavior of mobile foundation on normally consolidated (NC) and slightly over-consolidated (LOC) clay, which has been divided into the following four research areas (shown in Figure 1.4):

(I) **Base resistance.** Repeated shearing on NC or LOC clay can remould the interface soil and lead to a reduction in strength, while rest periods allow the soil to recover. Methods are required to account for step wise changes in base resistance due to sliding.

(II) **Berm resistance.** Soils are transported and deposited at either end of the sliding footprint forming berms as the foundation scrapes the soil surface. The berms accumulated at two ends provide additional horizontal resistances when the foundation approaches (or leaves) them, leading to high peak resistance. The remobilisation of the berms by the foundation needs consideration in design, as it could lead to over-stressing of the pipeline and/or connection.
(III) **Consolidation settlement.** The shear stress mobilised at underside of the foundation is transferred to the underlying soil, leading to the generation of excess pore pressure for NC or LOC clay. The accumulation and dissipation of excess pore pressure induces consolidation settlement, which must be accounted for in design.

(IV) **Shear settlement.** Shear settlement is induced by the formation of failure zones in the soil as it responds to combining loads (vertical and horizontal load, as well as possible moment loading). This results in incremental shear settlement within each cycle, increasing the accumulated settlement of the foundation.

Each of the above components contribute to whole-life sliding resistance and settlement, and are explored in this project.

### 1.3 Dissertation outline

This dissertation has seven chapters as shown in Figure 1.5, including this introduction.

Chapter 2 provides a comprehensive literature review relevant to the research and design of mobile foundations. Available centrifuge model testing is reviewed, and is followed by the development of theoretical analyses and performance simulations. Relevant geotechnical applications, element testing and penetrometer testing are then presented, with focus on soil berm formation and mobilisation, soil strength evolutions under the periodic monotonic loading, and soil behavior at low (vertical) stress levels.

Chapter 3 presents the results of five (new) centrifuge model tests, which were performed in calcareous silt under various operational conditions. The results are explored in detail, and compared to previous testing (three tests) carried out on a kaolin clay and presented in Cocjin et al. (2014) and Cocjin (2016). The influence of soil properties and operational condition on the evolution of sliding resistance, settlement and rotation is discussed, with Chapter 3 providing a qualitative understanding on the general behavior of mobile foundations that serves as a benchmark for the following theoretical studies. The results of all eight tests are then used in the subsequent chapters to explore berm resistance, sliding resistance and settlement respectively.

Chapter 4 presents a methodology to estimate the soil berm resistance, as the foundation scrapes the subsoil and deposits it at either end of the footprint (to form the dormant berms). The process of soil accumulation is discussed, and equations are proposed to calculate the soil volume. In addition, potential increases in berm strength due to the periodic remobilisation and reconsolidation are explored and accounted for.
Chapter 5 develops an extended theoretical framework based on the previous research (Cocjin et al., 2017) to provide an estimate of whole-life sliding resistance. The extended framework is displacement based, and can predict the strength evolution under periodic remoulding and intervening reconsolidation without involving back-calculated parameters by incorporating a remoulded strength line (RSL) and subloading surface. Resistance from the berm is included, and the model also includes a curved failure envelope to enable higher friction at low effective stress.

Chapter 6 extends the earlier framework to cover the prediction of whole-life settlement. The soil model is extended to include stress-dependent $\lambda$ and $\kappa$, where $\lambda$ and $\kappa$ are the slopes of virgin compression line and unloading-reloading line respectively. Based on the soil model, periodic stresses in the soil column beneath the centre of the foundation are calculated, and consolidation settlement is then derived by accumulating the deformation of each element. A methodology is also proposed to predict shear settlement based on the framework calculated strength profile, which is shown to provide a reasonable approximate of the observed behaviours.

Finally, Chapter 7 provides a brief overview of this research, and summarises the key findings.
Figure 1.1 Illustration of pipeline end terminations (Oil state industries, 2020)

Figure 1.2 Typical axial walking of pipeline that subjected to repeated start-up and shutdown (Rong et al., 2009)
Figure 1.3 Subsea tolerably mobile foundation (Zhou et al., 2015)

Figure 1.4 Challenges related to the design of mobile foundations
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

Chapter 1 (Introduction):
- Provides background and motivation behind this project

Chapter 2 (Literature review):
- Reviews previous work

Chapter 3:
- Discusses the results of model testing on the calcareous silt (new) and the kaolin clay (existing) under various loading levels.
- Generates response benchmark for subsequent prediction

Chapter 4:
- Proposes a methodology to calculate the resistance of berms deposited at two ends of the sliding footprint by considering soil accumulation and hardening induced by repeated remobilisation and reconsolidation.
- Considers the frictional angle at low effective stress levels.

Chapter 5:
- Extends the theoretical framework of Cocjln et al. (2017) to enable prediction of whole-life sliding resistance under periodic sliding events with reducing back-calculated parameters.
- Considers soil properties at low stress levels.
- Undertakes detailed simulation of resistance evolution within individual cycles.

Chapter 6:
- Extends the framework presented in Chapter 5 to enable prediction of the subsoil response.
- Considers soil properties at low stress levels.
- Calculates consolidation settlement based on the extended framework.
- Proposes a methodology to calculate shear settlement based on the framework generated strength profile.

Chapter 7 (Conclusion):
- Summarises key findings and outputs of research (at a minimum), limitations, recommendations for future work etc.

Figure 1.5 Flow chart of research objectives and thesis chapters
CHAPTER 2  LITERATURE REVIEW

2.1 Experiments on mobile foundations

Centrifuge modelling is the most common experimental method used for exploring the behaviour of mobile foundations. In particular, centrifuge tests have greatly improved general understanding on the behaviour of mobile foundations and provided data to benchmark theoretical research.

Cocjin et al. (2014) carried out a series of centrifuge model tests on lightly over-consolidated kaolin clay to research the whole-life response of mobile foundations. A rectangular mudmat with a fully rough base and smooth surrounding skis (to reduce the propensity for the edges of the foundation to dig in during movement), the breadth and length of the base plate was 50mm and 100mm respectively (5mx10m in prototype at 100g). In each test, the vertical load was kept constant, mimicking the self-weight of the foundation. After initial touchdown on the soil surface, the mudmat was slid periodically between the touchdown and operational positions for over 40 cycles. Between cycles, stationary periods (mimicking operation of the development) ranged from 0.25 to 1.5 prototype years. Three tests were carried out in total, with varying duration of operational period and vertical operative loads. These experiments have shown:

(I) The typical whole-life shear resistance is generally as shown in Figure 2.1. In terms of the base resistance, a peak breakout resistance is first reached within a very short sliding distance. Once the initial peak is reached, the resistance gradually decreases towards a residual value. The residual resistance progressively increases towards the drained limit, as a result of the cyclic surface shearing that occurs in subsequent sliding cycles. The soil beneath the footprint is ploughed and accumulates at the two ends of the footprint (in berms). These berms grow cycle-by-cycle and provide extra resistance to movement at the extremities of the sliding footprint.

(II) Mobile foundations experience both shear and consolidation settlement. A typical settlement evolution is plotted in Figure 2.2 (Cocjin et al., 2014), where $w_{con}$ and $w_{p}$ are the consolidation and shear settlements, respectively. The shear settlement occurs during the undrained sliding phase when, under combined loading, the horizontal displacement leads to subsoil failure and a degree of vertical settlement. Consolidation settlement is observed during the operational period, and is caused by dissipation of the excess pore pressure generated in the subsoil due to surface shearing. Both components of settlement
increase over the lifetime of the foundation but at reducing rates. Some minor tilting is also observed.

Wallerand et al. (2015) conducted centrifuge model tests on over-consolidated (OC) and normally-consolidated (NC) kaolin speswhite clay using ‘smooth’, ‘intermediate’ and ‘rough’ foundations. The prototype breadth and length of the base plate of foundations were 5m and 8m respectively in prototype terms. The smooth foundation was made of polyamide with \( R_z = 9.27 \mu m \), where \( R_z \) is the maximum height of the texture profile. The intermediate and rough foundations were fabricated by gluing sand paper with \( R_z \) respectively being 43.9\( \mu m \) and 123\( \mu m \) to the underside of the foundation. In order to accelerate the drainage of excess pore pressure, the model mudmats were perforated with small holes with a total area of 2\% of the area of the base plate. The results generated the following observations:

(I) Under the same soil conditions, the smoother foundation tends to induce less settlement and resistance than its rough counterpart.

(II) The over-consolidation ratio (OCR) significantly influences the evolution of settlement. Keeping all other conditions the same, the settlement yielded on high OCR soil is much reduced compared to normally consolidated soil.

(III) Berm resistance is closely related to the embedment of foundation. Increased embedment causes more subsoil to accumulate in the berms, leading to large berm resistances.

Further information on this set of experiments can be found in Stuyts et al. (2015) and Blanc et al. (2016).

2.2 Theoretical research on mobile foundation

Deeks et al. (2014a) demonstrates the primary aspects that need to be considered in geotechnical design including (i) adequate bearing capacity; (ii) enough small settlement, and (iii) enough low sliding resistance. A framework was then proposed to assess the sliding resistance and settlement that includes (i) primary and secondary settlement; (ii) plastic settlement, and (iii) shakedown settlement. For plastic (or shear induced) settlement, it is stressed that the mechanism-induced (i.e. the failure in subsoil) dominates instead of the permanent deformation incurred due to accumulated permanent shear strain. The framework provides an insight in predicting whole-life behaviour of mobile foundation.

Theoretical research on mobile foundations has predominantly utilised two methods: (i) finite element modelling using existing soil models (e.g. Modified Cam Clay and Tresca) to simulate parts of the
behaviour of mobile foundations (Feng and Gourvenec, 2016, Zhou et al., 2015, Hossain et al., 2020); and (ii) simplified 1D analytical models to generate a practical framework for industrial usage (Cocjin et al., 2017, Corti et al., 2017, Chen and White, 2021).

Feng and Gourvenec (2016) used the small strain finite element method to simulate the evolution of resistance for mobile foundations using a 3D foundation model and the Modified Cam-Clay (MCC) constitutive model. The typical stress path of the top element in the $e - \sigma'$ plane is shown in Figure 2.3. The results of this analysis showed that the subsoil hardens from the undrained shear strength towards the drained limit, at a rate proportional to the ratio of $\kappa/\lambda$, where $\kappa$ and $\lambda$ are the recompression and virgin compression indexes in $e - \sigma'$ plane, respectively. This relatively simple model has some inevitable shortcomings: (i) the softening phase induced by the remoulding event cannot be considered; and (ii) once the stress state lies within the yield surface, the analysis becomes purely elastic with no more excess pore pressure and plastic shear strain generated, and therefore, the evolution of strength in the deeper elements and consolidation settlement cannot be properly simulated.

Corti et al. (2017) employed the Memory Surface Hardening (MSH) model (Corti, 2016, Corti et al., 2015, Corti et al., 2016) to simulate the consolidation settlement of a mobile foundation, and compared the consolidation settlement that was calculated separately via the MCC model and the MSH model. The results showed that the new model overcame deficiencies inherent in the conventional MCC model, namely the purely elastic response within the yield surface. By adding the memory surface, cyclic loading induced pore pressure in the deeper elements was captured, and the MSH model gave a reasonable prediction of whole-life consolidation settlement. However, the simulation was performed based on the assumption that a drained sliding event yields the same consolidation settlement as undrained sliding plus post-sliding consolidation – meaning that the evolution of consolidation settlement was not fully coupled to the evolution of strength in the soil.

Zhou et al. (2015) focused on shear induced settlement of mobile foundations. The Tresca model was employed in the simulations, which demonstrated that shear settlement was mainly induced by subsoil failure under combined loading. A model of the foundation system, including a flowline on one side and a flexible or rigid spool on the other, was built in the commercial FEA software FLAC. The results showed that the different loading conditions associated with pulling and pushing events led to different responses, as the foundation tended to pitch into the spool side. In addition, the flowline and spool were found to be beneficial to the performance of the foundation in terms of shear settlement,
since they provided some additional vertical restraint. However, the published work comprised the simulation of only a single slide.

Hossain et al. (2020) used the coupled Eulerian-Lagrangian (CEL) method to simulate the whole-life trajectory of mobile foundations subject to cyclic loading. An advantage of this method is that it is well suited to simulating the large deformations associated with mobile foundations – although it cannot capture changes in soil strength / behaviour in response to this movement. The results qualitatively showed soil yielding leading to shear induced embedment, and the formation of soil berms at the ends of the sliding footprint. In the simulation, the flowline-mudmat-jumper-seabed system was captured, and the results show that the connected flowline and jumper may reduce embedment of the mudmat, which is consistent with the findings of Zhou et al. (2015).

Cocjin et al. (2015) proposed a methodology to calculate the resistance of the active berm that formed during a foundation sliding event. The cross section of the berm was considered as an isosceles triangle with base angles equal to that of the mudmat ski inclination. It was found that the berm at the mid-way point of the slide generated modest additional resistance. However, for the drained sliding test, the berm resistance increased more significantly.

The above studies focus on different aspects of mobile foundation behaviour, and provide a solid basis on which to develop further research. However, the different components are closely related to each other in the prediction of whole-life response. For instance, the cyclic shearing and resting events change the sliding resistance, which in turn influences pore pressure generation and the evolution of strength in the deeper soils, which then further influences the consolidation and shear settlement evolution. The displaced soil due to settlement will form dormant berms that provide extra resistance to mobile foundation. In summary, this is a highly integrated geotechnical problem, requiring a framework that can incorporate every component to predict the whole-life response.

Cocjin et al. (2017) built such a framework to predict the whole-life behaviour of mobile foundations. The soil model proposed in the framework assumed that the Critical State Line (CSL) migrated with the number of cycles in order to reflect the softening process due to the accumulated degradation of soil strength caused by sliding. The typical stress path in $e - \sigma'$ plane for the interface element is described in Figure 2.4. It can be seen that, except for the softening phase (migrating CSL), the stress path is fairly similar to that calculated by the Cam-Clay model reported in Feng and Gourvenec (2016). Through the proposed model, the whole-life consolidation settlement, residual resistance and strength evolution of subsoil can be predicted. For shear induced settlement, a back-calculation method was provided based on an associated flow rule. The framework was verified using the
centrifuge model test conducted on kaolin clay, as described earlier. However, seemingly important features, such as the degradation of resistance within the first slide and the whole-life berm resistance were not addressed. The research outlined in thus thesis explores the importance of these simplifications and attempts to validate a framework using test data for a wider range of soils and operative states.

The framework proposed by Cocjin et al. (2017) was further developed and simplified to predict the settlement and rotation of partially mobile foundations (Chen and White, 2021). Primary consolidation settlement, the whole-life shear and shear induced consolidation settlement were considered. A methodology was proposed to predict the rotation of partially mobile foundations by dividing the foundation to two components and calculating their differential settlement. However, the whole-life strength change in subsoil due to the surface cyclic sliding and the soil softening were ignored.

2.3 Relevant research

Mobile foundations are a relatively novel idea that have not been widely researched to date, besides those studies previously introduced. However, the soil behaviour that governs mobile foundation behaviour has been far more exhaustively explored. Aspects of offshore geotechnical engineering, such as design of subsea pipelines and conventional shallow foundation, T-bar penetrometer test interpretation, and advanced element testing protocols, provide a number of concepts that might be used to further enhance specific aspects of a mobile foundation design framework.

2.3.1 Berm resistance

Similar to mobile foundations, soil berms are also observed at the extremities of a laterally buckling pipeline footprint (Dingle et al., 2008, White et al., 2007). Taking any cross section of the pipeline, due to the thermal forces, it cyclically moves on the soil surface causing the subsoil to displace and form an active berm at the front face of the pipeline (Bruton et al., 2007, White and Cheuk, 2008, White and Randolph, 2007). The process of pipe-berm interaction has been observed via Particle Image Velocimetry (PIV) (White et al., 2003, White et al., 2007, Dingle et al., 2008). Based on further centrifuge experiments, Rismanchian et al. (2012) summarised that the slip surface under the berm mass can be primarily divided into two parts: (i) bearing sliding failure (occurring below the original soil surface); and (ii) sliding failure on the soil surface. Wang et al. (2010) categorised pipelines into two distinct categories: “light” and “heavy”. By running the large deformation finite element (LDFE) analyses, it was found that the “heavy” pipeline tends to dive in forming the bearing sliding failure.
However, in contrast, the “light” pipeline tends to form a slip surface close to the soil surface, with the mechanism remaining very shallow and categorised by a simple planar shear surface between pipe surface and soil surface.

2.3.2 Strength mobilisation and degradation

The softening of clay due to remoulding that occurs beneath sliding foundations on fine grained soils in undrained conditions (Cocjin et al., 2014) is also observed in element tests (Andersen, 1976, Hyodo et al., 1994, Andersen, 2015, Andersen et al., 1988), T-bar penetrometers tests (Stewart, 1991, Stewart and Randolph, 1994) and offshore industrial applications such as the axially and laterally loaded pipeline (Matlock, 1970, Yan, 2013), spudcan foundation (Hossain and Randolph, 2009), gravity offshore foundation (Bjerrum, 1973), riser (Hodder et al., 2008), plate anchor (Zhou, 2020).

Andersen (2015) explored the typical response of soft marine clay subjected to monotonic or cyclic loading through direct simple shear (DSS) testing, although these tests were conducted with higher confining stress than occurs below mobile foundation (typically less than 10kPa). Three tests were compared including a monotonic shear test and two cyclic shear tests, which were firstly cyclically sheared to 2% and 12% of plastic shear strain, respectively, and subsequently sheared further monotonically. In the virgin monotonic shear test, the peak undrained shear strength was firstly mobilised followed by the softening in strength with the accumulation of the shear strain ($\gamma$). The cyclic simple shear tests showed that, after plastic shear strain was accumulated due to cyclic shearing, the post cyclic monotonic stress-strain curve can rapidly re-join the virgin monotonic stress-strain curve but with a smaller peak undrained shear strength being achieved. Three test results are shown in Figure 2.5 (the cyclic phases are omitted for clarity), and the fact that the stress surpasses the monotonic stress-strain curve is due to rate effects. The results demonstrate that the achievable undrained shear strength is influenced by the cumulative shear strain. The more the shear strain is accumulated, the lower the undrained shear strength that can be mobilised.

Typical effective stress paths of monotonic and cyclic shear tests in $\tau$-$\sigma'_v$ plane (Andersen, 2015) are illustrated in Figure 2.6 in schematic form. In the monotonic test, the peak undrained shear strength is first mobilised as the stress path approaches the failure envelope, which is followed by softening along the envelope that results in the generation of additional excess pore pressure. For the cyclic loading test, the undrained strength is not fully mobilised during the early cycles, however, the excess pore pressure is still accumulated cycle-by-cycle. The potential peak undrained shear strength decreases with the build-up of excess pore water pressure. Similar responses were observed for saturated clay soils by Hyodo et al. (1994), Sangrey et al. (1969) and Tsubakihara and Kishida (1993).
T-bar penetrometer tests (Stewart, 1991, Stewart and Randolph, 1994) provide more insights into the soil softening process. In cyclic undrained T-bar penetrometer tests in clay, the softening process can be clearly observed. Soil strength close to the peak undrained shear strength is first mobilised during the initial penetration, and this then reduces cycle-by-cycle towards a residual value. The degradation in undrained shear strength can be related to the accumulation of shear strain, as proposed by Einav and Randolph (2005) as:

$$\frac{s_u}{s_{u,i}} = \delta_{rem} + (1 - \delta_{rem})e^{-\gamma_95/\gamma_95}$$  \hspace{1cm} (2.1)

Where, $s_u$ and $s_{u,i}$ are the softened and initial strength respectively, $\delta_{rem}$ is the fully remoulded strength ratio taken as the inverse of sensitivity, $\gamma_95$ is the cumulative shear strain required to cause 95% reduction from the peak strength to a fully remoulded residual strength. The equation shows that the undrained shear strength decays exponentially with the accumulated shear strain.

Undrained strength regain after softening was explored by White and Hodder (2010), also using the T-bar penetrometer. Tests were carried out in kaolin clay in the centrifuge, in which the T-bar was firstly penetrated and extracted between 15-60mm (3-12D) for 20 cycles at an undrained rate to fully remould the soil. A period of 3.5h (model time) was then observed to allow consolidation prior to the next episode of cycles, which was repeated three times. The result showed that the softened soil regained strength after long-term reconsolidation, and then softened again during the next episode. The normalised strength is plotted against number of cycles in Figure 2.7. The tested sensitivities of the three episodes were 2.5, 1.3 and 1.2, respectively. The remoulded strength increased to 1.5 and 2.0 times in the 2nd and 3rd episodes compared to that mobilised in the 1st episode. The peak strength in the 2nd and 3rd episodes recovered to around 0.8 and 0.9 times that of the 1st episode, respectively. This indicates that both strength and sensitivity are at least partially recoverable due to excess pore pressure dissipation.

A simple framework was proposed by White and Hodder (2010) to simulate the softening and hardening properties of clay soil (Figure 2.8). The remoulded strength line (RSL) and intact strength line (ISL) were proposed to define the fully remoulded state and the intact state of the soil, respectively. The remoulding process was linked to the accumulated shear strain via the “damage” occurring in the surrounding soil. The framework was further developed by Hodder et al. (2013) in order to develop a connection between the undrained shear strength and effective stress instead of just accumulating plastic shear strain or “damage”. The advantage of this is that it allows the
undrained shear strength to recover with consolidation. The back-calculated intact strength line (ISL) that was used to anchor the peak strength of each episode was therefore no longer required.

The framework proposed by Hodder et al. (2013) was further developed by Zhou et al. (2019a). The process of mobilisation towards peak strength was also modelled, taking into account changes in soil stiffness. In addition, the revised framework addressed load-controlled events. A typical stress path simulated by the proposed model is shown in Figure 2.9. It is observed that the strength can recover and soften through successive reconsolidation and remoulding events. The framework is able to simulate the strength evolution observed in the episodically cyclic T-bar test, and has been applied to the other offshore geotechnical applications including the spudcan footing (Zhou et al., 2019a), plate anchor (Zhou et al., 2020b) and steel catenary risers (SCRs) (Zhou et al., 2019b, Zhou et al., 2017).

2.3.3 Soil properties in the low stress range

Mobile foundations operate at relatively low stress ranges (likely less than 10 kPa) where fine grained soils (such as clay) usually exhibit differing properties when compared to high stress levels. Conventional laboratory tests are commonly performed moderate to high stress levels (due to challenges handling low strength soil), and therefore some test parameters are potentially inappropriate for use in the prediction of mobile foundation behaviour. The properties most impacted by low stress levels are as follows:

Curved effective stress failure envelope. Atkinson and Farrar (1985) tested London clay, showing that the effective stress failure envelope was curved. The results, as summarised and fitted by Maksimovic (1989), are shown in Figure 2.10. The envelope was found to be curved below a stress level of around 20kPa. Similar to London clay, the curved effective stress failure envelope is observed at low stress levels for a variety of other clay soils (Boukpeti and White, 2017, Najjar et al., 2007b, Pedersen et al., 2003). The usage of a curved failure envelope is critical to properly capture the response of many offshore geotechnical problems operating at low stress levels, such as the pipeline-soil interaction (Hill et al., 2012).

Curved normally consolidated line or virgin compression line (NCL). The NCL is commonly taken as a straight line in semi-log $e - \sigma'$ plane. However, under low stress levels, the slope of NCL is usually observed to be steeper than under higher stress levels. Nagaraj et al. (1998) published the measured NCL of six different types of soils, with all of them exhibiting such curvatures. Sun et al. (2016) tested the soft clay from three locations near Tianjin and collected more test data from the literature (Berilgen et al., 2006, Burland, 1990, Hong et al., 2010, Hvorslev, 1961, Znidarčić et al.,
1986). It was found that the curved NCL could modelled simply as a bilinear form, with the turning point appearing at approximately the liquid limit. The collected test data and the model fit are shown in Figure 2.11. The same property also has been observed in the kaolin clay used in UWA (Cocjin et al., 2017, Sahdi et al., 2014a). Both of the above related factors likely have an influence on the sliding resistance and settlement predictions generated by a mobile foundation design framework.

### 2.3.4 Shear induced settlement

Insights into the shear induced settlement of mobile foundations can also be gained through review of past studies into lateral buckling of pipelines. Combined physical and numerical modelling has been undertaken by various authors, with the vertical response of the pipe during lateral translation governed by the weight of the pipe and initial embedment. Select previous studies include the use of upper bound theory (Randolph and White, 2008) and finite element analysis (White et al., 2003), and it is suggested that displacement at failure can be simply evaluated through via H-V failure envelope according to the associated flow rule (Prager, 1947).

### 2.4 Summary

This chapter has reviewed the available literature related to mobile foundations, and the areas of offshore geotechnical engineering research that are most relevant to developing a framework for mobile foundation design. Previously reported centrifuge model tests have revealed key behaviours of mobile foundation including base resistance, berm resistance, consolidation settlement and shear settlement. The base resistance changes within and with cycles of sliding due to the cyclic remoulding and reconsolidation of the subsoil. Dormant soil berms are accumulated at two ends of the sliding footpath with providing extra resistances which grow with cycles. The consolidation and shear settlements were observed to accumulate with sliding cycles but with decreasing rate. Smooth surface and high OCR of soil show benefits to reducing settlement. Aiming at the behaviours of mobile foundation observed in centrifuge model tests, based on the relevant research, conclusions are as follows:

**Base resistance:** is determined by the mobilised soil strength around interface. The interface soil behaves similarly to long-distance monotonic shearing as reported in Andersen (2015). In terms of the stress path in the $e - \sigma'$ plane, with the increasing shear distance, the peak strength is first mobilised, followed by the strength decaying exponentially (Einav and Randolph, 2005) along the envelope towards a fully remoulded state. This process is driven by excess pore pressure generation.
After dissipation of these excess pore pressures, the effective stress will increase and the peak undrained shear strength is expected to at least partially recover, according to T-bar test results (White and Hodder, 2010, Zhou et al., 2020b). During the subsequent shearing event, the soil will again soften but with a higher residual strength and lower sensitivity. A series of soil models has been performed to simulate the above properties of soil (White and Hodder, 2010, Hodder et al., 2013, Zhou et al., 2019a, Cocjin et al., 2017). Methods to predict the base sliding resistance evolution of mobile foundations with sliding cycles have been provided by Feng and Gourvenec (2016) and Cocjin et al. (2017).

**Dormant berms:** are formed by displacing the subsoil and ploughing induced accumulation. According to pipeline-berm interaction research, the slip surface of the dormant berm tends to form a simple flat plane (Wang et al., 2010, Dingle et al., 2008, White and Randolph, 2007). Cocjin et al. (2015) assumed that the active berm formed at the mid-way of the footprint of mobile foundation to be an isosceles triangle, and demonstrated that the drainage condition dramatically influenced the berm resistance. A methodology to analyse the accumulation of the dormant berms and their whole-life responses in the application of mobile foundation does not yet exist.

**Consolidation settlement:** is due to the subsoil deformation which is induced by the cyclic generation and dissipation of excess pore pressure under the cyclic shearing. Its evolution is influenced by the interface and the deeper soil in combination. The element tests reported in Andersen (2015) provides insight into the behaviour of soil elements when subjected to various magnitudes of remoulding. Based on the MSH model (Corti et al., 2015, Corti et al., 2016), Corti et al. (2017) simulated the whole-life consolidation settlement of mobile foundations. Cocjin et al. (2017) proposed a theoretical framework based on a proposed softening model to predict the consolidation settlement. Softening is therefore likely to have a significant influence on the generation of settlements.

**Shear settlement:** is induced by failing the near-surface soil. The mechanism was explored by Zhou et al. (2015) using the FEA method and the Tresca soil model. However, in the simulations, the evolution of soil strength under cyclic remoulding and reconsolidation cannot be taken into account by a simple Tresca model. Cocjin et al. (2017) linked whole-life strength changes to shear settlement by proposing a back-calculation methodology to predict the evolution of whole-life shear settlement. However, a pragmatic means of predicting shear settlement without relying on back-calculation from an experiment would be preferred for practical use in design.
In summary, the whole-life behaviour of mobile foundations is a complex problem involving significant interplay between the base sliding and berm resistances, and consolidation and shear settlements. The previous research invariably concentrated on one or a couple of these aspects. However, these four aspects are linked with and influence each other, and therefore, a design framework incorporating all aspects is required to enable a better prediction.

The aim of this thesis is to build upon the framework proposed by Cocjin et al. (2017). First, the sliding resistance generated by the dormant soil berms will be investigated, since these ultimately generate the peak resistance to sliding. Second, the sliding resistance evolution within a cycle will be modelled, capturing the non-linearity of the effective stress failure criterion and NCL. Third, the framework will be developed to reduce the reliance on back-calculated parameters. Finally, the framework will be validated by more tests on an alternate soil type.
Figure 2.1 Normalised shear resistance plotted against sliding distance (Cocjin et al., 2014)

Figure 2.2 Normalised vertical displacement against normalised horizontal displacement (Cocjin et al., 2014)
Figure 2.3 The representative stress path of the interface element in $e-\sigma'_v$ plane (Feng and Gourvenec, 2016)

Figure 2.4 The typical stress path of the interface element using shifting CSL model (Cocjin et al., 2017)
Figure 2.5 Monotonic and combined cyclic and monotonic DSS tests (Andersen, 2015)

Figure 2.6 Effective stress paths for undrained tests with monotonic and cyclic loading (Andersen, 2015)
Figure 2.7 Normalised undrained shear strength evolution of episodically cyclic T-bar test (White and Hodder, 2010)

Figure 2.8 Typical stress path under episodic remoulding and reconsolidation simulated by the framework of White and Hodder (2010)
Figure 2.9 Typical stress path under episodic remoulding and reconsolidation simulated by the framework of Zhou et al. (2019a)

Figure 2.10 Effective stress failure envelope of London clay (Maksimovic, 1989)
Figure 2.11 Curved normally consolidated line (Sun et al., 2016)
CHAPTER 3  CENTRIFUGE TESTING OF A TOLERABLY MOBILE SUBSEA FOUNDATION ON CALCAREOUS SILT, AND COMPARISONS WITH TESTING IN CLAY

3.1 Introduction

In this chapter, the whole-life response of a so-called mobile foundation is explored experimentally. Five model tests have been carried out using the centrifuge and an offshore calcareous silt obtained from the North West Shelf offshore Western Australia (Chow et al., 2019, Zhou, 2020). The silt was reconstituted for the current test campaign, meaning that the in-situ soil structure condition is not fully captured. Four tests were performed with 40 sliding cycles under various operative vertical loads (i.e. $V_{op}/V_{ult} = 0.25$ and 0.36) and with varying intervening consolidation time to achieve different degree of consolidation (i.e. 7% and 32%). The fifth test was performed with 10 sliding cycles, with the position for consolidation taking place (operational position) at the mid of the footprint. The soil sample for the experiments was prepared to a lightly over-consolidated state by scraping a thin layer of surface soil, with OCR generally less than 1.4 after setting up the foundation, which decreases with depth. The moisture contents and soil strength profiles before and after the tests were obtained using the small diameter core sampling (Cocjin et al., 2017) and T-bar penetrometer (White et al., 2010) respectively. The horizontal and representative coefficient of consolidation was derived using the CPTu (Bolton et al., 1999) and piezo-foundation (Cocjin et al., 2014).

The purpose of this testing programme is to obtain a better understanding of the whole-life response of mobile foundations on a typical calcareous silt. This data are then used to develop and benchmark the theoretical model developed in later chapters. The results are also compared to three model tests performed on kaolin clay by Cocjin et al. (2014) to obtain the insight into the behaviour of mobile foundation on different types of soil.

3.2 Backgrounds

Centrifuge modelling is an economical way to simulate large geotechnical engineering problems in the laboratory (Schofield, 1980, Murff, 1996, Taylor, 2018). The centrifuge applies multiples of gravitational acceleration on a soil sample within a container known as a strongbox, effectively increasing the unit weight of the soil mass and increasing the stress gradient with depth within the sample. This maintains stress similitude between the model (scaled down) and prototype (full scale) geometries, preserving the stress-dependent soil response at any given point within the model. For
example, at an acceleration of “n” times Earth’s ambient gravitational acceleration, the model would be a $1/n^{th}$ representation of the prototype foundation geometry, facilitating modelling of relatively large problems in a modestly sized soil sample. In addition, the time required for excess pore pressure dissipation is reduced by a factor of $n^2$ due to the drainage path length being reduced by a factor of “n”. This allows very long-term, or even whole-life (nominally 25 years) response of a foundation to be modelled in a few days. This technique has been widely used in offshore geotechnical applications, including the modelling of pipelines (Bransby et al., 2013), subsea foundations (Cocjin et al., 2014), piles (White and Lehane, 2004) and anchors (Zhou et al., 2020a).

3.3 Experimental apparatus

3.3.1 Centrifuge facilities

All of tests were performed at 100g in a fixed beam centrifuge with a nominal radius of 1.8m (Randolph et al., 1991). Tests were conducted in an aluminium strong box with internal dimensions of 390mm×650mm×300mm in breadth, length and height respectively. A two-axis actuator attached to the strongbox using aluminium crossbars was used to control the load or displacement applied to the foundation during the test. See Figure 3.1(a) for an overview of the experimental setup.

3.3.2 Loading and acquisition system

The loading and measuring system consisted of actuators which controlled the horizontal and vertical movements of the mudmat foundation, sensors and lasers which were used to measure the movement and rotation and a frame connecting the actuator and the mudmat foundation to transmit the applied loads, shown in Figure 3.1(b) in schematic. The mudmat was displacement controlled in the horizontal direction and load controlled in the vertical direction. Through the roller within the loading arm and the hinged joint above the mudmat (Figure 3.1(c)), the rotation of the mudmat was allowed about both the short and long axis. Incorporating a flexible joint to the mudmat, as described in Zhou et al. (2015), will minimise moment applied to the foundation, and is expected to lead to reduces total and differential settlement. Should a non-flexible joint be used in practice, then the effect of higher overturning moment would need consideration. More details of the loading and acquisition system have been introduced in Cocjin et al. (2014).

3.3.3 Model mudmat foundation

The rigid model mudmat foundation used by Cocjin et al. (2014) – which followed the design of Deeks et al. (2014a) and Deeks et al. (2014b) – was used in all the tests reported in this chapter. It is made
from Acetal and is shown in Figure 3.2. The low density of Acetal (1410kg/m$^3$) allows the model self-weight to better represent the weight of a sliding mudmats, while its stiffness is sufficient to be considered as rigid relative to the underlying soil (Cocjin et al., 2014). The base plate was rectangular with 50mm in breadth ($B = 50$mm) by 100mm in length ($L = 100$mm), and with a thickness of 5 mm ($h = 5$mm). Note that while drainage holes in the mudmat may be use in practice, in order to assist touchdown and/or reduce consolidation times (as described further in Wallerand et al. (2015). These were not included for these tests (a) in order to produce comparable results with the existing tests on kaolin clay; and (b) to avoid soil extrusion into the foundation, which could lead to high settlement. The sides of the foundation comprise sloping edges, extending at an angle of 30° from the mudline, so called ‘skis’, ($\theta_{mat} = 30^\circ$) to promote sliding over rather than cutting into the soil sample during the cycles of sliding. Four round targets were attached to corners of the top plate for an array of four laser transducers to measure the settlement, pitch and roll of the foundation during the tests. The loading point was set as low as possible to minimise the loading height (and overturning moment in the direction of loading), with the hinge placed at 5mm above the centre of the top plate.

Although in practice it is expected that a sliding foundation would be designed with a smooth sliding surface, a rough base was adopted in this case. This was done to ensure that the (interface) soil can be fully mobilised by the foundation sliding motions so that soil property, instead of the soil-structure interface property, can be clearly captured. In order to obtain a rough interface, coarse silica sand ($D_{50} = 0.544$mm) was glued to the mudmat base using epoxy resin. Based on previous testing experience, coarse silica sand ($D_{50} = 0.544$mm) was glued to the mudmat base using epoxy resin – to create a rough interface and ensure failure in the underlying soil.

3.4 Penetrometers

3.4.1 T-bar

Miniature T-bar penetrometer (Figure 3.3 (a)) was used to obtain the continuous strength profile, and the strength evolution under cycles of remoulding and reconsolidation, following strategies of Hodder et al. (2013), Cocjin et al. (2014) and Zhou et al. (2019a). The theoretical T-bar factor ranges between 9.4 and 11.9 (Randolph and Houlsby, 1984) determined by its roughness, and 10.5 was recommended for practical use (Stewart and Randolph, 1994). The diameter and the length of the bar are respectively 5mm and 20mm. The penetration rate applied in the T-bar tests was 1mm/s in order to probe the undrained strength profile.
3.4.2 CPTu

Miniature CPTu (Figure 3.3 (b)) was used in the dissipation test to obtain the horizontal coefficient of consolidation, following the strategies of House et al. (2001), Chow et al. (2014), Cocjin et al. (2014) and Chow et al. (2019). The diameter of the piezocone was 5mm. Through the pore pressure transducer mounted right above the tip and on the shaft, the pore water pressure dissipation along the radius direction can be measured.

3.4.3 Piezo-foundation

Miniature piezo-foundation (Figure 3.3 (c)) was used in the dissipation test to obtain the representative coefficient of consolidation, following the strategy of Cocjin et al. (2014). The miniature piezo-foundation used in the tests was a rigid, circular surface foundation with a pore pressure transducer at the centre of the base plate which diameter was 40mm. By imposing specific vertical loads, the dissipation under various stresses can be measured.

3.5 Sample preparation

Calcareous silt, obtained from the North West Shelf of Western Australia (Chow et al., 2019, Zhou, 2020) was initially mixed with water to twice its liquid limit (around 145%). The slurry was subsequently mixed under vacuum over a period of 48 hours before it was poured into the strong box on top of a 10mm thick sand and a porous fabric membrane to facilitate adequate drainage at the base of the sample during consolidation. Before being consolidated in the centrifuge for a further 72 hours, the soil sample in the strong box was left on the laboratory floor for 72 hours to allow the soil to begin deposit naturally, which can make sure that different sizes of soil grains more averagely distributed vertically in the box. Given the potential for small changes in strength to impact the results of testing light weight foundations, it was important to ensure the sample was fully consolidated. Accordingly, T-bar tests were then performed over five days – and model testing only commenced once the increase in the T-bar tip resistance between sequential tests was within 5% (Figure 3.4) (i.e. minor changes in strength).

The basin-shape surface of the soil sample (due to the Coriolis effect) was subsequently scraped by between 4 mm and 24mm in order to achieve a flat surface for subsequent model testing, before being reconsolidated for a period of at least 24 hours prior to further model foundation testing. As the scraping was not uniform across the soil surface, the calculated over consolidation ratio (OCR) will varied slightly across each footprint and the effect of this is expected to be negligible with the
surcharge at the testing footprint lying between around 2kpa and 6kPa, which made the OCR vary between 1.0 and 1.7 after the foundation being set up.

3.6 Soil properties

3.6.1 Review of properties

The basic properties of the calcareous silt are listed and compared with the kaolin clay in Table 3.1. The kaolin clay that has been widely tested on at UWA has been widely tested on its properties by other researchers (Stewart, 1992, Lehane et al., 2009, Cocjin et al., 2014, Sahdi et al., 2014a, Sahdi et al., 2014b, Sahdi et al., 2016, Zhou et al., 2019a). The properties of the calcareous silt are stated in Chow et al. (2019) and Zhou et al. (2020b). The calcareous silt was reported to have a larger internal friction angle $\phi$ of 40°, compared with reported 23.5° for the kaolin clay (Stewart, 1992).

3.6.2 Submerged unit weight

The submerged unit weight $\gamma'$ was calculated from Eq. (3.1) using the moisture content ($m_c$) measured from 20 mm diameter sample cores extracted from the centrifuge sample. Note that moisture content is denoted as $m_c$, not the universally adopted ‘$w$’ in geomechanics, because $w$ will be used for vertical displacement/settlement in this thesis.

$$\gamma' = \frac{(G_s - 1) \gamma_w}{1 + m_c G_s}$$  \hspace{1cm} (3.1)

where, $G_s$ is the specific gravity. $\gamma_w$ is the unit weight of water. The distributions of submerged unit weight along depth for silt and clay are exhibited in Figure 3.5 in prototype scale. The average submerged unit weight in the upper 1B (5m in prototype) calculated from dividing the area under the unit weight curve (Figure 3.5) by depth, obtaining the value of calcareous silt and the kaolin clay is 4.7kN/m$^3$ and 5.5kN/m$^3$ respectively.

3.6.3 Compression characteristics

The normal compression line in $e$-$\sigma'_v$ plane can be derived from the data of sample cores. The sample cores were taken after scraping, and hence, the void ratio and its corresponding effective stress need to be slightly adjusted by OCR through the equations below

$$e = G_s m_c - \kappa \ln(OCR)$$  \hspace{1cm} (3.2)

$$\sigma'_v = \gamma' z + \sigma'_{v,\text{sur}}$$  \hspace{1cm} (3.3)
where, $\kappa$ is the slope of unloading-reloading line, $\sigma'_{sur}$ is the surcharge pressure, which is estimated to be 5kPa for the amount scraped from the samples taken at around the edge of the strong box. The OCR distribution after scraping, prior to model testing, is shown in Figure 3.6. The normal compression lines of the calcareous silt and the kaolin clay (Cocjin et al., 2017) are shown in Figure 3.7. The calcareous silt showed a linear relationship between void ratio and vertical effective stress (In scale) as depicted in the conventional theory. The slope of the compression line ($\lambda=0.287$) given by Chow et al. (2019) fits well with the measurements derived from the sample core. In comparison, the normal compression line of the kaolin clay is bi-linear, with a higher slope ($\lambda=0.65$) at low stress levels and a lower value ($\lambda=0.26$) at high stress levels (Sahdi et al., 2014a, Cocjin et al., 2017). The “bi-linear” characteristic has been widely observed in many clayed soil (Nagaraj et al., 1998, Sun et al., 2016) with the turning point to be around the liquid limit where the corresponding void ratio is termed as $e_{ll}$. For the kaolin clay, the turning point appears slightly higher than $e_{ll}$. Data from the silt tests did not however reach the range below $e_{ll}$, so the same behaviour was not observed.

### 3.6.4 Consolidation characteristics

Both CPTu and piezo-foundations tests were performed to explore soil consolidation characteristics. CPTu dissipation tests are typically performed at depth and measure the (local) rate of horizontal dissipation. In contrast, a piezo-foundation test, which comprises a pore-pressure transducer mounted on the underside of the round plate and is influenced by drainage in both vertical and horizontal directions, is thought to be more representative of the consolidation scenario of surface foundation.

A CPTu dissipation test was performed at three depths (4.2, 8.2 and 10.2m at prototype scale) to determine the coefficient of consolidation $c_h$. The non-dimensional consolidation time was calculated after Teh and Houlsby (1991) as:

$$T = \frac{c_h t}{R^2 I^2}$$

(3.4)

where, $R = 5\text{mm}$ is the radius of the cone and $I = 100$ is rigid index as provided by Chow et al. (2019). The dissipation tests were interpreted using the methodology given by Sully et al. (1999) and compared with results from finite element analysis as shown in Figure 3.8 (a). The derived $c_h$ values at the three depths were plotted in Figure 3.9 against the vertical effective stress $\sigma'_v$, which was estimated using the average effective unit weight determined in section 3.6.2.

Piezo-foundation tests were used to measure the near-surface dissipation characteristics, and have been considered to be more representative of the drainage conditions beneath a mudmat (Cocjin et al...
The representative coefficient of consolidation \( c_{ref} \) can be obtained by comparing the normalised dissipation curve shown in Figure 3.8 (b) with the result from finite element analysis (Gourvenec and Randolph, 2010). The derived representative coefficient of consolidation \( c_{ref} \) under two different effective stress levels are presented in Figure 3.9, showing that \( c_h \) was around 3 times \( c_{ref} \). The \( c_{ref} \) for kaolin clay is plotted in Figure 3.9 for comparison, with both soils fitted using a power law relationship. The fitting equation of the kaolin clay was given by Cocjin et al. (2014) as

\[
c_{ref} = 2.7\left[0.3 + 0.16\left(\sigma'_{v} \right)_{NCL} \right]^{0.47}
\]  

(3.5)

The similar equation for the silt is

\[
c_{ref} = 0.33\left(\sigma'_{v} \right)_{NCL}^{0.47}
\]

(3.6)

where, \( \left(\sigma'_{v} \right)_{NCL} \) is the effective vertical stress on normal compression line (NCL).

3.6.5 Undrained shear strength characteristics

T-bar tests as suggested by Stewart (1991) and Stewart and Randolph (1994) were performed in vicinity of the proposed mudmat footprints. The soil shear strength profile shown in Figure 3.10 (a) was derived from the T-bar tests using a penetration rate of 1mm/s. Ten cycles of cyclic penetration and extraction were performed between the depths of 6.5m to 8.5m (in prototype scale), equivalent of 13-17\( D_{T-bar} \), where \( D_{T-bar} = 0.5 \)m (in prototype scale) is the diameter of T-bar which. The in-situ strength profile was interpreted according to the methodology of White et al. (2010), showing the gradient strength with depth to be around 2kPa/m – which is consistent with that derived by Zhou (2020). The normally consolidated strength ratio \( (s_u/\sigma'_{v})_{NC} \) was back-calculated from the initial strength profile using Eq.(3.7) as proposed by Wroth (1984). Taking the average submerged unit weight as 4.7kN/m\(^3\), the undrained shear strength stress ratio \( (s_u/\sigma'_{v})_{NC} \) was 0.42.

\[
s_u = \sigma'_{v}\left(\frac{S_u}{\sigma'_{v}}\right)_{NC} OCR^{\frac{\lambda}{\lambda'}},
\]

(3.7)

Strength degradation for the calcareous silt and kaolin clay is compared in Figure 3.10 (c). Sensitivity measured by T-bar (\( S_t \)) is observed to be 4.1 for the calcareous silt, which compares to around 2.5 for the kaolin clay, as reported by Cocjin et al. (2014).

A number of studies have revealed that the initial undrained shear strength measured by T-bar penetrometer is not the true intact undrained shear strength (Zhou and Randolph, 2009b, Zhou and Randolph, 2009a, Sahdi, 2013), because as the T-bar penetrates into the soil, part of the soil around
the penetrometer head has been remoulded to some extent. The undrained shear strength derived by the initial pass of the T-bar may be regarded as the strength of the 0.25th cycle \((s_{u,n=0.25})\). The true intact undrained shear strength of the 0th cycle noted as \(s_{u,n=0}\) needs to be back-calculated through the degradation curve using the methodology of Zhou and Randolph (2009b). The back-calculated undrained shear strength of the 0th cycle was 26% higher than the 0.25th cycle for clay and, 32% higher for silt. Therefore, the ‘real’ strength ratio \((s_{u}/\sigma_{v}^{'}NC)_{r}\) and ‘real’ sensitivity \(S_{tr}\) were 0.57 and 4.8 for silt respectively, and 0.20 and 3.1 for clay. In essence, these higher values are representative of the undrained strength ratios that might be determined from element tests, where mobilisation induced remoulding is less of an issue.

An equivalent long-term cyclic T-bar test was conducted in silt and shown in Figure 3.10 (b). The test comprised 50 cycles of full penetration and extraction with 780s (0.25 years in prototype) consolidation between each cycle for full dissipation of excess pore pressure. The strength variation factor \((s_{u,cyc}/s_{u,n=0.25})\) at the deepest penetration depth is plotted in Figure 3.10 (d) and compared with that of kaolin clay given by Cocjin et al. (2014). It can be observed that the kaolin clay recovers its strength somewhat faster than the silt, eventually reaching a steady value that is around 3 times its initial strength – and which is regarded as its drained state (i.e. no more excess pore pressure generation and dissipation). In contrast, the silt appears to approach an asymptote that being around 1.8 times the initial strength.

### 3.7 Test plan

Five model tests in total (from S1 to S5) were carried out in the calcareous silt sample at the locations which are shown in Figure 3.11. During each test, free water was maintained above the soil surface at a (constant) level that was deep enough to fully submerge the mudmat, but not the attached laser targets that were respectively placed 20mm over the top plate of the mudmat on four corner columns. As the foundation settled, there was a slight change in buoyancy, as the lengths of submerged corner columns supporting the laser targets fell below the water, but this was considered negligible with the vertical load decreased by 0.3%-0.4% for every 1mm settlement.

From tests S1 to S4, the foundation was first placed at the designated position and carefully touched down in-flight (Cocjin et al., 2014). The vertical operational load \((V_{op})\) was kept constant throughout the test, simulating the weight of the mudmat and the section of the pipe / spool it would support in field. Prior to the first slide, the foundation was allowed to consolidate for a period of 4 hours (4.5 prototype years), after which the incremental settlement tends to be minor. The mudmat was then slid
along the sample surface at a rate of 1mm/s for a distance $\delta = 25%L$ to its operational position (OP), where it was then left stationary for periods ranging from 780s to 4680s (0.25 to 1.5 prototype years) to simulate different period of initial operation. The mudmat was then slid back to its original (shutdown) position (SD) at the same rate (1mm/s), representing what happens when the subsea system is shut down, cools and contracts. After 8s of resting (10 prototype days), the foundation was slid to its operational position again, and the process repeated for 40 cycles, after which a long-term consolidation time 4.5h (5 prototype years) was given prior to finally detaching the foundation.

Test S5 was subjected to a different scenario. After the initial consolidation stage, the mudmat was slid for only 12.5mm, before being (immediately) slid back by 25mm, and then slid to its starting position. Long-term consolidation of 4680s (1.5 prototype years) was then allowed at the original position, which occurred between each full cycle of sliding, and the process repeated for 10 cycles. Similarly, a long-term consolidation time 4.5h (5 prototype years) was given prior to finally detaching the foundation. The motivation for this test was to explore the influence of consolidation position on the responses of the mobile foundation.

The intervening operational time between the sliding events for all tests in the silt, along with their clay counterparts (C1-C3) from previous testing (Cocjin et al., 2014), are summarised in Table 3.2.

The operational load and vertical load mobilisation for all eight tests also given in Table 3.2, with tests S1-S3 having the similar vertical load mobilisation as C1-C3. The ultimate bearing capacity prior to consolidation $V_{ult0}$ was estimated based on the initial strength profile as back-calculated from the T-bar testing (as shown in Figure 3.12). The surface intercept of the undrained shear strength profile ($s_{unm0}$) was taken as 1.2kPa for the calcareous silt, and 0.5kPa for the kaolin clay. The bearing capacity factor ($N_c$) was calculated using the methodology given by Feng et al. (2014b), resulting in values of 13.4 and 12.6 for the silt and clay respectively. The value of $V_{ult0}$ can be determined as

$$V_{ult0} = N_c BLs_{unm0}$$

(3.8)

Note that the effective vertical load ($V_{eff}$) applied over the base plate of the mudmat needs adjusting from the operative vertical load $V_{op}$, to account for the fact that the skis increase the footprint area as the foundation settles into the soil, as illustrated in Figure 3.2. In this case, $V_{eff}$ is estimated as follows:

$$V_{eff} = \frac{V_{op}B}{B + 2\min(h, w)\cot\theta_{mat}}$$

(3.9)

where $w$ is the settlement of the mudmat.
3.8 Model test results and discussions

The results from tests S1-S5 are presented in this section, focusing on the foundation settlement, rotation and sliding resistance. Where relevant, the results from the calcareous silt testing are compared with tests on kaolin presented by Cocjin et al. (2014), in order to explore the influence of soil properties on the mudmat response.

3.8.1 Foundation settlement

The vertical and horizontal displacement observed in tests S1-S5 at specific cycles are shown in Figure 3.13. The total settlement comprises the initial settlement (after touchdown) and sliding induced settlement.

The initial settlement varies across the tests from 0.032\(B\) to 0.046\(B\). It is noted that the test S2, had the highest load applied to the foundation, but recorded the smallest initial settlement – which may be due to a higher over consolidation ratio (OCR) at this location, which was in close proximity to the side of the strongbox.

Sliding induced settlement develops with the number of cycles, and can be attributed to two components: the shear settlement \(w_p\) and the consolidation settlement \(w_{con}\), as shown in Figure 3.13. Shear settlement results from combined (\(V-H\) or \(V-H-M\)) loading of the foundation at failure (Zhou et al., 2015), and consolidation settlement is induced by the periodic generation and the subsequent dissipation of excess pore pressure (Cocjin et al., 2017).

Consolidation settlement and incremental consolidation settlement for the tests on the silt (S1-S5) are respectively plotted against the number of cycles in Figure 3.14 (a) and Figure 3.15. For given soil type, it is clear that the time period for reconsolidation and vertical load mobilisation are key influencing factors. The following observations are noted:

- Comparing S1, S3 and S4 – all of which had the same vertical load mobilisation but different intervening reconsolidation time – it can be seen that the longer the consolidation time is, the larger the accumulated consolidation settlement is, with that of S3 (\(t_{con}=1.5y\) in prototype) being three times S1 (\(t_{con}=0.25\) in prototype) at the 40th cycle.
- Test S2 – which was subject to a higher vertical load mobilisation than its counterpart S1 – demonstrated slightly larger accumulated consolidation settlement.
• S3 and S5 had same operational conditions (reconsolidation time in each cycle and loading level), and exhibited the same consolidation settlement even though the consolidation location was different.

• The incremental consolidation, in general, decreases with cycles and asymptotes to zero, reflecting that the generated excess pore pressure reduced cycle-by-cycle.

The normalised consolidation settlement of S1-S3 observed during the first cycle are plotted against normalised consolidation time (time factor) \( T = c_{\text{ref}} \frac{B^2}{t_{\text{con}}} \) in Figure 3.16 (a) to illustrate the reconsolidation process in each cycle. In this case, \( c_{\text{ref}} \) can be calculated through Eqs.(3.5) and (3.6) in which \((\sigma'_v)_{\text{NCL}}\) is calculated by

\[
(\sigma'_v)_{\text{NCL}} = \sigma_{op} \frac{OCR - k}{k} \quad (3.10)
\]

and \( \sigma_{op} \) is the operative stress applied to the base plate. The normalised consolidation curves follow a consistent path – the short normalised consolidation time of S1 and S2 results in their consolidation curves mostly lying on the ‘plateau’, which explains why their consolidation settlement is lower than S3 during a single cycle.

Soil type also influences consolidation settlement (Figure 3.14 (b), (c) and (d)), mainly due to difference in compressibility (Figure 3.7) and coefficient of consolidation (Figure 3.8) that influences the normalised consolidation time as shown in Figure 3.16 (b) and (c). In general, the silt cases generated less consolidation settlement than their clay counterparts due to the lower compressibility and coefficient of consolidation.

The final moisture content measured beneath the centre of the footprint is compared for S1-S5 in Figure 3.17 with the corresponding in-situ values. It can be seen that the consolidation effect is greatest within the upper 0.2\( B \) of the soil.

Normalised shear settlement for the tests on silt (S1-S5) is plotted in Figure 3.18 against the number of cycles. For the shear settlement, it is the load level that has the largest influence and the following is observed:

• The foundation in S2 is 50% heavier than S1, leading to 23% higher shear settlement at the end of the 40\(^{th} \) cycle. The trend is consistent with the previous tests on clay (Cocjin et al., 2014).
The reconsolidation location on the footprint does not have an obvious influence on shear settlement, with S3 and S5 showing only modest differences in shear settlement.

S3 that has longer reconsolidation time was expected to yield smaller shear settlement than S1, due to the larger increase in strength in subsoil. However, the opposite result is observed although the difference tends to be minor—possibly suggesting that other factors, presumably strength distribution with depth, could influence the evolution of shear settlement as well, since $H$-$V$ failure envelope that influences shear settlement development (Zhou et al., 2015, Deeks et al., 2014a) is determined by both its size (determined by the magnitude of subsoil being strengthened) and shape (determined by strength distribution).

Shear settlement appears to develop differently in the two soil types. In general, the incremental shear settlement, defined as the increment of shear settlement within a sliding cycle normalised by the sliding distance of a cycle, decreases with cycle number as shown in Figure 3.19 (a) - however, the rate of reduction is far slower in the silt than the clay (see Figure 3.19 (b), (c) and (d)). For example, the initial incremental shear settlement of S1 is 0.002 which drops by around 50% after 20 cycle to around 0.0009. In comparison, the initial incremental shear settlement of C1 is 0.005 which drops by only 90% after 20 cycles to around 0.0006 (Figure 3.19 (b)). Same trend can be observed in tests C2 and S2 (Figure 3.19 (c)), and C3 and S3 (Figure 3.19 (d)). The reduction in the incremental shear settlement is related to the hardening of the subsoil and the decrease in the vertical load mobilisation (i.e. reduction in $V_{op}/V_{ult}$) following the repeated cycles of remoulding and reconsolidation. Here, the slower decrease for S1, S2 and S3 could be due to the silt having lower degree of strength regain than clay, with reference to Figure 3.10 (d). The final strength profiles of S1-S5 measured at the footprints are compared with the strength profile measured away from footprints in Figure 3.20. The final strength profile is the average of three selected T-bar testing spots across the footprint respectively close to shutdown side, middle and operational side referencing to Figure 3.11. In general, the enhanced zone appears above 0.5B in response to the periodic monotonic surface shearing. The zone extends deeper and the strength increases larger with the longer reconsolidation time and larger loading level.

Finally, shear and consolidation settlement at N=40 is compared for silt tests (S1-S5) in Figure 3.21 (a). It can be seen longer dissipation period results in larger consolidation settlement. The vertical load mobilisation shows the largest influence on shear settlement. Compared to the corresponding tests on clay (as shown in Figure 3.21 (b)), in general, the silt test experiences smaller consolidation settlements as a proportion of the total settlement owing to its lower compressibility and coefficient of consolidation.
3.8.2 Foundation rotation

Rotation changes progressively across all tests, as summarised in Figure 3.22. Tests S1-S4 show a similar trend, with the foundation pitching towards the side not fully covered by the foundation while reconsolidation, with rotation angles less than 2°. Tilting to the shutdown side is thought to relate to the average strength being lower than at the operational side without the cover of foundation during the longer period of consolidation during operation. However, the tests on clay all showed the opposite tilting trend. While not fully understood, this is potentially due to the increased significance of the consolidation settlements at position where reconsolidation happened (OP) in the clay experiments – supported by the fact that as the incremental consolidation settlement for C1-C3 reduces, the trend in rotation appears to reverse (and align with the silt tests).

3.8.3 Sliding resistance evolution

Figure 3.23 shows normalised horizontal resistance versus normalised horizontal displacement as observed in tests S1-S5. The following features are observed:

- A pronounced peak (P1) is observed on initial sliding movement, followed by an exponential decay towards a residual sliding resistance - which is seen part way through the slide, prior to engaging soil at the end of each movement.
- Subsequent peaks (P2 and P4) are mobilised due to the mobilisation of dormant soil berms that accumulate at the extremities of the foundation footprint due to sliding and consolidation.
- On initial movement after being stationary in the operating position, another peak (P3) is observed, which is thought due to the combination of an increase in soil strength on the sliding interface and suction mobilised as the berm separates.
- The residual resistance mobilised during the first slide increases with cycle number, due to an increase in soil strength through periodic monotonic surface shearing and intervening reconsolidation.

The initial peak (P1) is related to the peak undrained shear strength of the soil at the interface, which can be calculated using the undrained strength ratio \( (s_u/\sigma'_v)^{NC,r} \) and Eq.(3.7). The peak horizontal resistance at P1 is then calculated by

\[
H_{p,p1} = \sigma'_r \left( \frac{s_u}{\sigma'_v} \right)^{NC,r} OCR \frac{1}{\lambda BL} \tag{3.11}
\]
The measured normalised peak resistance of P1 for all eight tests are plotted in Figure 3.24 against the calculated values using Eq.(3.11). In general, Eq.(3.11) provides a good prediction of the initial peak resistances – albeit the predicted values are somewhat lower (12% on average). The slight under prediction could potentially be (partly) attributed to resistance generated through side shear on the mudmat and rate effects (Tika et al., 1996, Lehane et al., 2009), neither of which has been accounted for. Comparing the tests on clay and silt, the normalised peak resistances of tests S1-S5 are all larger than those observed in tests C1-C3, which reflects the silt had higher undrained strength ratio.

The normalised horizontal resistance for the first slide of silt test is presented in Figure 3.25 (a), and shows that the peak resistance is mobilised within a very short distance and followed by a rapid softening due to soil remoulding. Comparing the typical case S1 with its counterpart C1 in Figure 3.25 (b), it can be observed that longer shearing distance is needed to fully remould silt than clay with $\delta_{50}$ and $\delta_{95}$ being 8.8 and 3.4 times that of clay. Where, $\delta_{50}$ and $\delta_{95}$ are respectively the sliding distance that the soil softens to 50% and 95% of its peak undrained shear strength. Longer remoulding distance required for silt could be due that the silt has stronger structure than the clay.

The residual resistance increases with number of cycles due to the strength gain of the interface soil as shown in Figure 3.26 (a), which is demonstrated through long-term T-bar testing as shown in Figure 3.10 (d) with the interface soil progressively densified towards its drained state. Comparing tests on silt with the longest (S3, S5), intermediate (S4) and shortest (S1, S2) intervening consolidation time, it can be concluded that the longer the reconsolidation time, the faster the resistance increases. The longer reconsolidation time yields larger degree of consolidation in each cycle, which makes the soil approaches to its drained state faster. In contrast, when the intervening consolidation time is the same, tests with different vertical load mobilisation (S1, S2) or different parking location (S3, S5) tend to show a comparable response in recovery rate. Soil type also exhibits an influence on strength recovery rate – comparing the cases on silt (S1-S3) and clay (C1-C3) that reported in Cocjin et al. (2014), it can be concluded that when the excess pore pressure is allowed to largely dissipate in each cycle, the silt has comparable recovering capacity to final drained state (marginally slower) with clay as shown in Figure 3.26 (d); while for short-term consolidation cases (S1, C1 or S2, C2) the strength recovery rate of silt is significantly lower than that of clay (Figure 3.26 (b) and (c)). As seen in the data, after 40 cycles C1 and C2 have nearly reached a final (steady) strength, while S1 and S2 have only reached around half of their potential final strength. This reflects that the low coefficient of consolidation of the silt controls its recovery rate, which is consistent with the evolution of consolidation settlement as shown in Figure 3.16.
Observing tests S3 and C3 (the foundation off ground after 20 cycles) in Figure 3.26 (d), it is noted that the maximum normalised residual resistances in both soils exceeds the corresponding drained friction coefficients (written as $\mu = \tan \phi$) calculated from the laboratory testing friction angle shown in Table 3.1. This is thought to reflect a higher friction angle at low stress level, associated with a curved failure envelope lying above the inferred straight envelope. This has been widely observed in various soils (Maksimovic, 1989, Pedersen et al., 2003, Najjar et al., 2007b, Boukpeti and White, 2017) - according to the test data for S3 and C3, friction angles of the silt and the clay can be back analysed as $47^\circ$ and $31^\circ$ respectively (at the operating loads), which are larger than $40^\circ$ and $23.5^\circ$ given in Table 3.1.

Dormant soil berms accumulate due to ploughing of the front ski, and provide large additional resistance at positions P2 and P4. The normalised resistance at P2 for all eight tests is plotted against the normalised total settlement in Figure 3.27(a), and it is clear that the berm resistance increases in proportion with the total settlement. The accumulation of soil berms at P2 and P4 also likely relates to rotation of the foundation, with the side pitching into the subsoil tending to create a larger berm and resulting in greater berm resistance. The ratio of the maximum berm resistance of P4 to that of P2 is plotted in Figure 3.27 (b) against the foundation rotation angle $\theta$, which is defined positive for the anticlockwise direction. However, with increasing $\theta$ the berm at position P4 becomes larger than that at position P2, resulting in a bias in berm resistance.

A final peak (P3) is observed when the foundation slides from its operational position. The peak resistance at position P3 of all eight test on both silt and clay is plotted against that observed at position P2 in Figure 3.28, where it can be seen that the berm resistances are practically equal (within a margin of error of 20%). While this suggests the P3 peak relates to remobilization of the berm for a short distance, before detaching, it is likely that reconsolidation of the soil on the interface (when at the operating position) also contributes to the observed peak.

### 3.9 Conclusions

Five centrifuge model tests of a tolerably mobile foundation subjected to various cycles of sliding with intervening consolidation and at different vertical load levels have been conducted on a calcareous silt sample. When compared to kaolin clay, the calcareous silt exhibits four clear distinctions: higher strength ratio and higher sensitivity; lower compressibility and slower regain in strength. These features are reflected in the foundation results, with key findings as follows:
Shear settlement: the development of shear settlement is influenced by the vertical load level, with higher load level inducing greater shear settlement. Dissipation of excess pore pressure due to periodic loading and reconsolidation events strengthen the subsoil, decreasing the incremental shear settlement for subsequent cycles. Compared to silt, tests on clay experience larger incremental shear settlement during early cycles, when then drops rapidly, presumably due to the stronger capacity for strength regain. The position at which the long-term consolidation took place shows no obvious influence on shear settlement.

Consolidation settlement: incremental consolidation settlement decreases with cycle number, because the excess pore pressure generated in each cycle reduces with the soil gradually approaching to its drained state. Higher compressibility of the clay at low stress levels and faster dissipation due to a higher coefficient of consolidation lead to greater consolidation settlement when compared to tests in silt. Again, the position where the long-term consolidation takes place shows no obvious influence.

Rotation: when comparing the results from all eight tests, it is seen that rotation develops slowly, with less than 2° observed over the 40 cycles. Tests on silt show that the mudmat tilts towards the shutdown side where is uncovered by the foundation when the long-term consolidation happened because the average strength is lower in the proximity of the shutdown side. While tests on clay show an opposite trend (tilting to operational side), this is due to the greater consolidation settlement at the operational side. The rotation appears to trend back (to shutdown side) as the incremental consolidation settlement reduces.

Softening and hardening: on first movement, the strength of the soil close to the interface is degraded rapidly over a short sliding distance, with longer sliding distance required for silt than clay to remould the interface soil possibly because of the stronger structure of the silt. The remoulding process is broadly identical for a type of soil. The remoulded strength recovers through a process of periodic remoulding and reconsolidation – eventually reaching the drained limit. Silt and clay show a comparable recovering capacity while under long consolidation time in each cycle. Under relative short consolidation time, silt recovers its strength much slower than clay, since that the smaller consolidation coefficient of silt leads to lower degree of consolidation. At low stress levels, higher friction angles are observed while resistance reaching drained state than from laboratory testing, inferring the presence of a curved failure envelope. Again, the consolidation location shows no obvious influence.

Peak resistances: four obvious peaks in resistance (P1-P4) appear at the extremities of the footprint. P1 reflects mobilisation of the initial peak undrained shear strength, with silt
showing larger normalised value than clay owing to its larger strength ratio; P2 and P4 are associated with mobilisation of dormant soil berms, the magnitude of which is related to settlement and tilt of the foundation; P3 is attributed to two aspects, i.e. consolidation related strength gain and temporary adherence of the berm to the mudmat ski.
Figure 3.1 Experimental setup: (a) apparatus setup; (b) schematic of loading and measuring system (O’Loughlin et al., 2018); (c) model foundation and the connected loading arm

Figure 3.2 Photo of model foundation

Figure 3.3 Photo of penetrometers: (a) T-bar; (b) CPTu; (c) Piezo-foundation
Figure 3.4 Tip resistance of T-bar tests performed during sample consolidation

Figure 3.5 Distribution of the saturated unit weight along depth
Centrifuge testing of a tolerably mobile subsea foundation on calcareous silt, and comparisons with testing in clay

Figure 3.6 Over consolidation ratio (OCR) distribution after scraping, prior to model testing

Figure 3.7 Void ratio plotted against vertical effective stress
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

Figure 3.8 Normalised dissipation curve: (a) dissipation curves of the calcareous silt tested by CPTu at 4.2, 8.2, and 10.2m in prototype; (b) dissipation curves of the calcareous silt tested by piezo-foundation under the vertical stress of 10 and 20kPa respectively

Figure 3.9 Coefficient of consolidation of the calcareous silt and the kaolin clay
Figure 3.10  Cyclic T-bar penetration tests: (a) strength profile of continuous cyclic T-bar tests in calcareous silt; (b) strength profile of T-bar tests with intervening consolidation in calcareous silt; (c) Comparison of strength degradation of calcareous silt and kaolin clay; (d) Comparison of strength enhancement of calcareous silt and kaolin clay
Figure 3.11 Schematic of locations of tests

Figure 3.12 Distribution of the intact undrained shear strength $s_{u,n=0}$ for the calcareous silt and the kaolin clay
Figure 3.13 Normalised vertical displacement against the normalised horizontal displacement
Figure 3.14 Evolution of the normalised consolidation settlement with number of cycles: (a) S1-S5; (b) S1 versus C1; (c) S2 versus C2; (d) S3 versus C3
Figure 3.15 Incremental consolidation settlement plotted against number of cycles for S1-S5
Figure 3.16 Evolution of the normalised consolidation settlement with normalised consolidation time (time factor) $T = c_{ref} t_{con}/B^2$ during operation of the first cycle: (a) S1-S3; (b) S3 versus C3; (c) S1 versus C1
Figure 3.17 The moisture content distributions of S1-S5 measured before and after the model testings
Figure 3.18 Evolution of normalised shear settlement with number of cycles for S1-S5
Figure 3.19 Incremental shear settlement plotted against number of cycles: (a) S1-S5; (b) S1 versus C1; (c) S2 versus C2; (d) S3 versus C3
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

Figure 3.20 The undrained shear strength distributions of S1-S5 tested before and after the model testings
Figure 3.21 Comparison on the normalised consolidation and shear settlements at the last cycle: (a) S1-S5; (b) comparison of tests on the kaolin clay (C1, C2, C3) and the calcareous silt (S1, S2, S3)

Figure 3.22 Evolution of rotation versus number of cycles
Figure 3.23 Normalised horizontal resistance against the normalised horizontal displacement
Figure 3.24 Normalised measured initial peak resistance versus the calculated (H_p,P_1/V_eff)^{exp}:

\begin{equation}
\frac{H_p}{V_{eff}}:(-)
\end{equation}

Predicted normalised peak resistance (P1), \( \left( \frac{H_p}{V_{eff}} \right)_{Fw} \):

\begin{equation}
\frac{H_p}{V_{eff}}:(-)
\end{equation}

S1
S2
S3
S4
S5
C1
C2
C3
LoE
Silt
Clay

Figure 3.25 Normalised resistance evolution during the first slide: (a) S1-S5; (b) S1 versus C1

\begin{equation}
\frac{H_p}{V_{eff}}:(-)\quad \frac{\delta}{B}:(-)
\end{equation}

S1 \( (V_{op}/V_{ult}=0.25, t_{con}=0.25y) \)
C1 \( (V_{op}/V_{ult}=0.29, t_{con}=0.25y) \)
Figure 3.26 Normalised residual resistance plotted against number of sliding cycles: (a) S1-S5; (b) C1 versus S1; (c) C2 versus S2; (d) C3 versus S3
Figure 3.27 Berm resistance evolution: (a) normalised berm resistance at P2 plotted against normalised settlement; (b) the ratio of berm resistances of P4 to P2 plotted against mudmat rotation angle

Figure 3.28 Normalised peak resistance at P3 plotted against that of P2
Table 3.1 Properties of Kaolin clay and calcareous silt reported in previous literatures

<table>
<thead>
<tr>
<th>Property</th>
<th>Clay</th>
<th>Silt</th>
</tr>
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<tbody>
<tr>
<td>Internal friction angle, $\varphi:(^\circ)$</td>
<td>23.5</td>
<td>40</td>
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<tr>
<td>Strength factor of high stress level, $M_0=6\sin \varphi/(3-\sin \varphi):(-)$</td>
<td>0.92</td>
<td>1.62</td>
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<td>Slope of normal compression line in $e$-$\ln \sigma'_v$ plane, $\lambda:(-)$</td>
<td>0.205-0.31</td>
<td>0.287</td>
</tr>
<tr>
<td>Slope of swelling line in $e$-$\ln \sigma'_v$ plane, $\kappa:(-)$</td>
<td>0.044-0.06</td>
<td>0.036</td>
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<td>The void ratio intercept of NCL at $\sigma'_v=1$kPa, $e_v:(-)$</td>
<td>3.40-3.72</td>
<td>4.00</td>
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<td>Liquid limit, $LL:(-)$</td>
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<td>Plastic limit, $PL:(-)$</td>
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<td>0.39</td>
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<td>Specific gravity, $G_s:(-)$</td>
<td>2.6</td>
<td>2.71</td>
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</table>

Table 3.2 Summary of the operational conditions of centrifuge tests on kaolin clay and calcareous silt (model scale)

<table>
<thead>
<tr>
<th>Operational load (in prototype), $V_{op}:(kN)$</th>
<th>Vertical load mobilisation, $V_{op}/V_{ult}:(-)$</th>
<th>Pause time at operational position in each cycle, $t_{con}:(s)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1 200</td>
<td>0.25</td>
<td>780</td>
</tr>
<tr>
<td>S2 300</td>
<td>0.36</td>
<td>780</td>
</tr>
<tr>
<td>S3 200</td>
<td>0.25</td>
<td>4680</td>
</tr>
<tr>
<td>S4 200</td>
<td>0.25</td>
<td>4680 for the 7th, 14th, 28th, … cycles, 780 for the rest</td>
</tr>
<tr>
<td>S5 200</td>
<td>0.25</td>
<td>4680</td>
</tr>
<tr>
<td>C1 92.5</td>
<td>0.29</td>
<td>780</td>
</tr>
<tr>
<td>C2 154.5</td>
<td>0.50</td>
<td>780</td>
</tr>
<tr>
<td>C3 92.5</td>
<td>0.29</td>
<td>4680</td>
</tr>
</tbody>
</table>
CHAPTER 4  PREDICTION OF BERM RESISTANCE

4.1 Introduction

As mobile foundations move across the seabed, they will tend to settle due to ploughing and generation and dissipation of shear induced excess pore pressure, leading to displacement of soil. The soil beneath the foundation is ploughed into an active berm defined as soil that is transported at the leading edge of the foundation. This soil is subsequently deposited into dormant berms defined as the soil that remains at the ends of the foundation footprint, and which accumulates in volume during repeated cycles of start-up and shut-down over the operating life of the subsea system. Rough foundations and normally consolidated subsoil will generate most excess pore pressure and therefore greatest settlement and larger berms according to centrifuge tests (Stuyts et al., 2015, Wallerand et al., 2015). The volume of these dormant berms grows on a cycle-by-cycle basis, contributing a significant amount of resistance against sliding as the active and pre-existing dormant berms combine, leading to repeated mobilisation of the dormant berm and associated hardening. Mobilisation of the dormant berm could potentially lead to the generation of sufficient resistance to impede mudmat sliding, which could lead to overstressing of the pipeline connections that the mudmat is designed to support. Therefore, predicting the sliding resistance generated by the accumulated dormant berms is critical in ensuring a safe sliding foundation design.

This chapter sets out a methodology of predicting the resistances induced by the dormant berms. The mechanism of accumulation and mobilisation is first analysed and idealised. Equations are then proposed to calculate the accumulated soil volume encapsulated in each dormant berm and the resistance generated on their mobilisation in sliding. A simple soil model capturing the berm hardening process under periodic remobilisation and reconsolidation, based on critical state soil mechanics principles, is incorporated in the model.

The methodology is validated using the eight centrifuge model tests (five on silt and three on clay) reported in Chapter 3 and shows good prediction of dormant berm resistance at both the shut-down side (the side without long-term parking of the foundation) and operational side (the side with relatively long-term parking of the foundation) of the foundation footprint. The sensitivity of the model to the assumed berm shape was explored, indicating minimal sensitivity to this assumption for a plausible range of triangular berm shapes. Combined with the prediction of residual sliding resistance (Chapter 5), this unlocks the ability for sliding foundations to be designed where sliding throughout the design life cycle can be ensured with confidence.
4.2 Berm states

Figure 4.1 demonstrates the process of accumulating and mobilising the dormant berm in a single undrained sliding event, resulting in three states:

- State 1 (St1) is where an active berm is formed on the leading edge of the mudmat as it settles into (and scrapes) the seabed, without engaging the dormant berm. This generates a negligible amount of additional sliding resistance relative to the sliding resistance generated at the mudmat-soil interface (Cocjin et al. 2015).
- State 2 (St2) is where the active berm engages with soil that collapsed from the dormant berm when the mudmat moved away during the previous cycle. The sliding resistance begins to rise rapidly through this phase.
- State 3 (St3) is where the active and dormant berms have fully combined and the peak resistance is being mobilised. This often constitutes the peak resistance in the life cycle of the mudmat.

The horizontal resistance evolutions of tests C1 and S1 (respectively performed on UWA kaolin clay and calcareous silt) are taken as representative examples to illustrate the mudmat-berm interactions, as shown in Figure 4.2 – for more test data and details please refer to Chapter 3. The peak resistances induced by the dormant berms appeared at positions P2 and P4 (as defined in Figure 4.2(a)) when the berm is in State 3, which is the scenario for which a simple analytical solution is derived in this chapter.

4.3 Model derivation

The soil within the dormant berm is considered to come from two components: the underlying soil displaced by settlement of the mudmat during sliding, and the soil layer ahead of the leading edge of the mudmat. The magnitudes of each of these components of soil accumulation are directly caused by settlement and rotation of the mudmat, hence the analytical model for predicting dormant berm resistance is derived directly from the mudmat settlement and rotation.

4.3.1 Failure modes

For simplicity, the two-way sliding action of mobile foundation is assumed to be plane strain conditions in the out of plane direction. This assumption is justified from observations made during testing, where it was noted that little soil heave occurred adjacent the edges parallel to the sliding direction which can be seen in the test photos (Figure 4.1). From a design perspective, this assumption
is more conservative, leading to a larger dormant berm and higher resistance. Under this assumption, the undrained interactions between the mudmat and the subsoil can be divided approximately into five modes (see Figure 4.3):

- Mode M1 represents pure vertical loading, resulting in pure vertical settlement.
- Mode M2 represents pure horizontal shearing under zero vertical load, which (assuming associated flow holds), means there will be no shear settlement.
- Mode M3 represents the operating mode of mudmat foundation, comprising combined loads (vertical and horizontal). Again using associated flow assumptions, this results in both horizontal movement and vertical embedment. The soil displaced by settlement of the foundation is assumed to accumulate in front of the foundation footprint (in the direction of movement), prior to being ploughed into the active berm and eventually forming the dormant berms at either end of the mudmat footprint.
- Mode M4 represents a generalised case where consolidation settlement results in an expanded yield surface and additional soil being displaced into the dormant berms by increased active berm formation.
- Mode M5 represents a more comprehensive case where foundation rotation and the resulting bias in dormant berm formation is explicitly accounted for in the predictive calculations.

Modes 3 to 5 most commonly happen during the service of mobile foundations.

### 4.3.2 Berm volume approximation

The typical trajectory of the mudmat during a cycle of operation and shut-down is illustrated in Figure 4.4 schematically, with the coordinate ‘S’ indicating the shut-down end of the foundation, the coordinate ‘C’ representing the centre of the foundation, and the coordinate ‘O’ representing the operational end of the foundation and the subscript representing the cycle number notation with ‘N’ representing the number of cycles, N =1, 2, 3 etc and ‘c’ representing the state after consolidation. The surface soil is displaced into the dormant berm during shearing events. In another word, the volume of the shear induced settlement is considered to be transferred into dormant berms. According to the function of calculating the area of the polygons, the incremental volume collected into the dormant berm at shut-down side within a single cycle noted as $\Delta V_{dis,P4}$ can be derived simply according to the corner coordinates of the polygon as:

\[
\Delta V_{dis,P4} = f(x_1, y_1, x_2, y_2, x_3, y_3, x_4, y_4)
\]
\[ \Delta V_{\text{dis},P4} = |C_{(N)}C_{(N-0.5c)}| + |C_{(N-0.5c)}S_{(N-0.5c)}| + |S_{(N-0.5c)}S_{(N-0.5)}| + |C_{(N-1+c)}C_{(N-0.5c)}| + |S_{(N-0.5c)}S_{(N-0.5)}| + |S_{(N-1+c)}S_{(N-0.5c)}| + |S_{(N-1+c)}S_{(N-0.5)}| \]

\[ \Delta V_{\text{dis},P2} = |C_{(N-0.5)}C_{(N-1+c)}| + |C_{(N-1+c)}O_{(N-1+c)}| + |O_{(N-1+c)}O_{(N-1)}| + |C_{(N-1.5c)}C_{(N-1.5c)}| + |C_{(N-1.5c)}O_{(N-1.5c)}| + |O_{(N-1.5c)}O_{(N-1)}| + |O_{(N-0.5c)}O_{(N-0.5)}| \]

where, the subscript ‘P4’ stands for the shut-down side (as defined in Figure 4.2(a)). Similarly, the incremental volume of displaced soil accumulated in the dormant berm on the operational side can be derived as:

where, the subscript ‘P2’ stands for the operational side (as defined in Figure 4.2(a)). In application of this methodology, when \(N=1\), the settlements for the steps of \((N-1), (N-1.5+c)\) are set to be zero. The three coordinates required for these volume calculations, ‘S’, ‘C’ and ‘O’, can be derived directly from the position of the midpoint of the mudmat and the rotation as follows:

\[
[S, C, O] = \left[ \begin{array}{ccc} \delta - \frac{L}{2} \cos \theta & \delta & \delta + \frac{L}{2} \cos \theta \\ w + \frac{L}{2} \sin \theta & w & w - \frac{L}{2} \sin \theta \end{array} \right]
\]

where, \(\delta\) is the horizontal displacement of the mudmat (taken as 0 and \(\delta_{\text{max}}\) at shut-down and operational positions, respectively), \(w\) is the settlement at the mid-point of the mudmat, \(\theta\) is the rotation angle of the mudmat relative to horizontal with anticlockwise being positive, and \(L\) is the length of the mudmat.

The volumes of the displaced subsoil into the two berms at the operational and shut-down positions (\(V_{\text{dis},P2}\) and \(V_{\text{dis},P4}\)) can be derived by summing up the incremental volumes as follows:

\[
V_{\text{dis,P2(4)}} = \sum_{N=1,2,3...} \Delta V_{\text{dis,P2(4)}}
\]

**4.3.3 Dormant berm mobilisation**

Figure 4.5 illustrates schematically the process of the mobilisation of the dormant berm at the extremity of the mudmat foundation sliding footprint. An inclined linear slip surface is assumed to form between the berm and the underlying soil, similar to assumptions previously applied in the analysis of berms adjacent to partially buried pipelines (Wang et al., 2010, Rismanchian et al., 2012).

The total soil volume involved in a dormant berm referred to as \(V_{\text{ol}}\) consists of two components: the
displaced soil depicted above (top isosceles triangle) and the heaved original soil layer (the triangle below the soil surface). The base length of the top isosceles triangle \( b_{\text{dis}} \) can be calculated as:

\[
b_{\text{dis}} = 2 \sqrt{\frac{\text{Vol}_{\text{dis}}}{B \tan \theta_b}}
\]  (4.5)

where, \( \theta_b \) is the base angle berm as illustrated in Figure 4.5, which was assumed to be equal to the base angle of the ski of the mudmat \( \theta_{\text{mat}} \) (Cocjin et al., 2015), and \( B \) is the breadth of the mudmat.

The volume of the heaved component can be derived by

\[
\text{Vol}_{h} = \frac{B(w + 0.5L \sin \theta)b_{\text{dis}}}{2}
\]  (4.6)

Vol then can be expressed as the sum of two components:

\[
\text{Vol} = \text{Vol}_{h} + \text{Vol}_{\text{dis}}
\]  (4.7)

According to the geometry relationship, the corresponding cross-sectional breadth of the mobilised triangle berm \( b_{b} \) can be calculated as:

\[
b_{b} = \sqrt{h_{\text{btm}}^2 + (h_{\text{btm}} \cot \theta_b + b_{\text{dis}})^2}
\]  (4.8)

where, \( h_{\text{btm}} \) is the height of the bottom triangle, expressed as

\[
h_{\text{btm}} = w + 0.5L \sin \theta
\]  (4.9)

**4.3.4 Soil strength evolution**

In addition to the volume of soil and shape of the berm, the evolution of soil strength has a significant influence on the dormant berm resistance mobilised on contact with the active berm from the current slide. It can be observed that some soil from the dormant berm collapses onto the footprint area when the foundation moves away. The collapsed soil will be recollected and interact with the dormant berm during the next cycle. This can be observed in Figure 4.2, where it is noted that the mobilising process (St2) starts from around 0.1-0.2B away from the final peak (St3) , which is assumed to be long enough to sufficiently mobilise the current undrained shear strength of the soil around the slip surface of the the dormant berm. The mobilisation and the subsequent resting time leads to the periodic generation and dissipation of excess pore water pressure, which leads to changes in soil strength along the slip surface of the dormant berm.
The mean undrained shear strength at the slip surface of the berm can be approximated through the equation suggested by Wroth (1984) as:

\[ s_{u,b} = \left( \frac{s_u}{\sigma'_r} \right)_{NCL} \left( \frac{s_v}{\sigma'_v} \right)_{NC,r} \]  
(4.10)

where, \( s_u / \sigma'_v \) is the normally consolidated strength ratio which can be calculated from the T-bar measured value according to Zhou and Randolph (2009b). \( \sigma'_v \) refers to the effective stress on the NCL given by:

\[ (\sigma'_v)_{NCL} = \exp \left( \frac{e_N - e}{\lambda} \right) \]  
(4.11)

where, \( e_N \) is the intercept void ratio at \( \sigma'_v = 1 \) kPa on the normal compression line (NCL) in \( e-\sigma'_v \) plane. \( e \) is the void ratio, and \( \lambda \) is the slope of NCL in \( e-\sigma'_v \) plane.

The generated excess pore pressure is considered to fully dissipate before the next cycle due to the very short drainage path within the soil berm and the relatively long operational time between shutdown cycles. Under this assumption, the mean void ratio of the slip surface after consolidation can be calculated as:

\[ e = \min \left[ e_{(N-\Delta N)} - \kappa \ln \left( \frac{\sigma'_{v,app}}{\sigma'_v} \right) + \lambda \sigma'_{\text{app}} \right] \]  
(4.12)

where, \( \kappa \) is the slope of the unloading-reloading line. If \( N = 0.5 \) or 1, the subscript ‘\( N-\Delta N \)’ stands for the pre-mobilising state. The first expression in the above minima function represents the consolidation along the unloading-reloading line prior to reaching the NCL and the second expression represents a stress path following the NCL (i.e. normal compression). Figure 4.6 illustrates these two types of consolidation stress path in the \( e-\sigma'_v \) plane in schematic. \( \sigma'_{v,cr} \) is the effective stress achieved at the critical state, which can be expressed as:

\[ \sigma'_{v,cr} = \frac{s_{u,b}}{0.5M |_{\sigma'_{v,cr}}} \]  
(4.13)

where \( M |_{\sigma'_{v,cr}} \) is a stress-dependent strength ratio. Various researchers have shown that the effective stress failure envelope for clay soils is non-linear at very low stress levels, lying somewhat above the conventional straight failure envelope (Maksimovic, 1989, Pedersen et al., 2003, Najjar et al., 2007b, Boukpeti and White, 2017, Najjar et al., 2007a). A simple method is used here to describe such a
curved failure envelope, utilising a multiplier $R|_{\sigma'}$ on the strength ratio $M_0$ which is measured in conventional tests at higher stress levels (e.g. triaxial tests). The strength factor at any vertical effective stress of $\sigma'$, noted as $M|_{\sigma'}$ can be derived as follows:

$$M|_{\sigma'} = R|_{\sigma'} M_0 \quad (4.14)$$

where, $R|_{\sigma'}$ is the multiplier, which is expressed as a function of the vertical effective stress $\sigma'$ as:

$$R|_{\sigma'} = 2 - \exp \left( -\frac{3 \sigma_R}{\sigma'} \right) \quad (4.15)$$

where, $\sigma_R$ is a parameter controlling the ultimate initial curvature of the non-linear failure envelope.

$\sigma_{v,eqm}$ in Eq.(4.12) is the mean equilibrium stress at the slip surface of the berm, which is determined by the weight of the mobilised soil berm, as follows:

$$\sigma_{v,eqm} = \frac{h_b \gamma'}{2} \quad (4.16)$$

where, $\gamma'$ is the submerged unit weight of soil and $h_b$ is the height of triangular berm, which can be derived as:

$$h_b = \frac{2\text{Vol}}{Bb_h} \quad (4.17)$$

The maximum resistance of the dormant berm noted as $H_{b}^{*}$ can be calculated by

$$H_{b}^{*} = s_{u,b} b_B B \quad (4.18)$$

The peak resistance ($H_p$) achieved at the dormant soil berm can be estimated as the sum of berm resistance and the resistance of base plate ($H_{base}$) as:

$$H_p = H_{b}^{*} + H_{base} \quad (4.19)$$

### 4.4 Input parameters

The input parameters adopted to simulate the berm soil strength evolution in the eight centrifuge model tests presented in Chapter 3 are provided in Table 4.1. The non-linear failure envelope can be tested through triaxial or simple shear test, however, since lacking of lab tests, here, it was calibrated using the 20th cycle of test C3 and the 40th cycle of tests S3, because these two tests had the long
intervening consolidation period and had reached the ‘drained limit’ where no further consolidation hardening was occurring. In other words, no excess pore pressure is generated by further foundation sliding, so the bearing pressure generated by the weight of the foundation is essentially the vertical effective stress and the sliding resistance is the equivalent shear stress, therefore this cycle must lie directly on the non-linear failure envelope. Appropriate values of $\sigma_R$ were derived on this assumption, as shown in Figure 4.7.

4.5 Results

Predictions of the peak berm resistances were obtained by using the measured residual sliding resistance, foundation settlement and rotation of the mudmat, as inputs for the model derived in the previous section for each cycle. The predictions of the peak resistances at positions P2 and P4 are compared with the experimental observations in Figure 4.8, illustrating good predictive capacity of the proposed methodology. The residual sliding resistance and foundation settlement are key inputs to the methodology, which have been simulated by Cocjin et al. (2017) and further advanced in the later chapters 5 and 6, largely controlling the magnitude of the peak berm resistances and their evolution on a cycle-by-cycle basis. The foundation rotation – which is considered more difficult to predict – is less critical in that it is only required to correctly predict the bias in berm resistance generated at either end of the mudmat footprint. In all tests, the side of the foundation that pitched deeper into the seabed generated a larger berm.

The measured peak resistances at positions P2 and P4 (referring to Figure 4.2) are plotted against those predicted for all tests for the two soil types tested in Figure 4.9. A cumulative distribution (CDF) of the ratio of predicted to measured peak resistance is shown in Figure 1.10. This indicates that 71% of data falls within 10% of the measurements and 97% falls within 20% of the measurements, with a slight tendency for over-prediction ($P_{50}$ is larger than 1). This tendency to over-predict the peak resistance could be due to any or a combination of the following reasons:

- The berm soil strength was assumed to be fully mobilised in each slide. However, the fact that the resistance is still rising at positions P2 and P4 in Figure 4.2 suggests that the undrained strength is only partially mobilised by the berm interaction.
- The 2D plane strain assumption likely leads to an over-estimate of the berm volume and peak berm resistance. In reality, a small proportion of soil ploughed into each berm could flow out along the two long sides of the mudmat as shown in Figure 4.1. However, the conservatism might be offset by the assumption no shearing resistance across the parallel (long) edges.
• The assumed shape of the berm which was effectively controlled by the ski angle of the mudmat according to Cocjin et al. (2015). The assumption may not be accurate. The influence of the assumed shape of the berm will be further explored later.

All of the above factors could lead to over-prediction of the peak resistance, which is thought to be conservative (under-prediction of peak berm resistance would be more hazardous).

In order to explore the influence of the assumed berm shape, the berm base angle, \( \theta_b \), was varied over the range of 20\(^o\)-50\(^o\), with the results shown in Figure 4.11 for tests C1 and S1. It can be seen that the influence of berm shape is minor, with the peak resistances calculated for the 40\(^{th}\) cycle using \( \theta_b = 50^o \) being only 9\% and 4\% less than those with \( \theta_b = 20^o \) for test C1 and S1 respectively.

Figure 4.12 illustrates the relative importance of the following on the peak berm resistance at P2 for tests C1 and S1: (i) base sliding resistance; (ii) berm volume gain due to the accumulation of displaced subsoil and more heaved soil as the mudmat embeds deeper; and (iii) strength gain due to generation and dissipation of excess pore pressures on repeated mobilisation of the berm. The berm clearly provides a significant additional resistance that must be accounted for in the generation of a robust sliding foundation design, with the dormant berm accounting for approximately 50\% and 36\% of the total resistance at the 40\(^{th}\) cycle for tests C1 and S1, respectively. The strength gain due to periodic consolidation accounted for the majority of the additional increase in the resistance of dormant berm in both tests C1 and S1, highlighting the importance of strength evolution being captured by the model. The ratio of the mobilised maximum berm resistance at the operational side (P2) calculated by both considering the effect of periodic remobilisation and reconsolidation and ignoring it (\( H^*_{b,P2} / H^*_{b,P2(Nocon)} \)) is shown in Figure 4.13. This ratio increases from unity to around 3.5 and 2 for tests C1 and S1, respectively, with a dramatic increase during the first 15 cycles of sliding, before slowly reaching an asymptotic state that implies that the soil on the slip surface of the dormant berm had reached the ‘drained limit’ (i.e. constant volume undrained shearing with no generation of excess pore pressure) by this stage.

4.6 Conclusions

The soil berms generated at either end of the footprint of a sliding foundation have been demonstrated to contribute significantly to the peak resistances generated by a sliding foundation. A methodology for estimating the berm resistance has been proposed in this chapter, using a simple failure mechanism based on the associated flow rule and critical state soil model to capture the cycle-by-cycle evolution
of peak resistance. The methodology proposed was validated using eight centrifuge model tests encompassing two types of soil. The main conclusions of this chapter are that:

- Dormant berms are accumulated due to two aspects:
  1. Soil displaced by settlement of the foundation during sliding; and
  2. Soil heaved up in front of the leading edge of the foundation.

- The foundation settlement and rotation directly determines the volumes of the dormant berms at either end of the sliding foundation footprint.

- Periodic remobilisation and reconsolidation of the soil within the berms significantly enhances the berm strength, which leads to the increased berm resistance. The berm strength develops from an initially undrained resistance towards the ‘drained limit’ (i.e. constant volume undrained shearing with no generation of excess pore pressure).

- The increase from an initially undrained resistance to the ‘drained limit’ occurred during the first 15 cycles for the experiments analysed in this chapter, however, the timescale of this transition would be dependent on operational time periods (i.e. time elapsed between shutdown cycles) and the dissipation characteristics of the soil in real world applications.

- Application of the berm resistance model proposed in practice requires accurate prediction of:
  1. residual sliding resistance (on the base);
  2. sliding foundation settlement; and
  3. (ideally) foundation rotation.

The analysis presented in this chapter has shown that it is possible to accurately predict the formation of soil berms, and associated lateral resistance, as the foundation slides repeatedly across the seabed. The model has been developed for a rough foundation underside, which it expected to lead to the greatest settlement and sliding resistance. In practice, sliding foundations might be constructed with a smoother underside, which will tend to reduce both resistance to sliding and settlement – with the latter expected to generate smaller soil berms. This should be considered when adopting the current practices in design.
Figure 4.1 Schematic of the mudmat mobilising soil berm and the corresponding photos

Figure 4.2 Normalised horizontal resistance versus normalised horizontal displacement for C1 and S1: (a) C1; (b) S1
Figure 4.3 Soil translation modes under combined loads

Figure 4.4 Cycle number notations

Figure 4.5 Mechanism of mobilising dormant berm
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

\[ e_{(N-1)} - \kappa \ln \left( \frac{\sigma_{v,equ}}{\left(\sigma'_v\right)_{CSL(N-1)}} \right) \]

\[ e_N - \lambda \sigma_{v,equ} \]

Figure 4.6 Schematic of stress path evolution in \( e - \sigma'_v \) plane

Figure 4.7 Fitted curved failure envelopes of the kaolin clay and the calcareous silt: (a) the kaolin clay; (b) the alcalareous silt
Figure 4.8 The measured and predicted peak resistances at P2 and P4 normalised by effective vertical force plotted against number of cycles: (a)-(h) are respectively the tests C1, S1, C2, S2, C3, S3, S4, S5.
Figure 4.8 Continued
Prediction of berm resistance

Figure 4.9 The normalised measured against the normalised predicted peak resistance at P2 and P4:
(a) tests C1-C3; (b) tests S1-S5

Figure 4.10 Cumulative distribution of the ratios of predicted to measured peak resistances (P2 and P4)
Figure 4.11 Comparison on the predicted peak resistance at P2 with using various assumed $\theta_b$ (20º, 30º, 40º, 50º): (a) C1; (b) S1

Figure 4.12 Resistance constitution at P2: (a) C1; (b) S1
Figure 4.13 Ratio of the predicted berm resistance at P2 with considering the effect of remobilisation and reconsolidation to that without plotted against number of cycles.
## Table 4.1 Framework parameters

<table>
<thead>
<tr>
<th>Property</th>
<th>Kaolin clay</th>
<th>Calcareous silt</th>
<th>Source of selected values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Breadth of the base plate of mudmat, $B$ (model scale)</td>
<td>50mm</td>
<td>50mm</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td>Length of the base plate of mudmat, $L$ (model scale)</td>
<td>100mm</td>
<td>100mm</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td>Base angle of ski, $\theta_{\text{mat}}$</td>
<td>30°</td>
<td>30°</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td>Maximum single sliding distance, $\delta_{\text{max}}$</td>
<td>25%L</td>
<td>25%L</td>
<td>Test parameter reported in Chapter 3</td>
</tr>
<tr>
<td>Critical state friction constant tested from high stress level, $M_0$</td>
<td>0.92</td>
<td>1.62</td>
<td>Calibrated through triaxial test, here is given by Stewart (1992) and Chow et al. (2019)</td>
</tr>
<tr>
<td>Parameter to control the shape of the failure envelope, $\sigma_R$</td>
<td>0.16</td>
<td>0.40</td>
<td>Calibrated from triaxial test, here is calibrated through model tests C3 and S3 reported in Chapter 3</td>
</tr>
<tr>
<td>Intercept void ratio at $\sigma'_c = 1$ of the Normal Compression Line (NCL) in $e$-$\sigma'_c$ plane, $\varepsilon_N$</td>
<td>2.45</td>
<td>3.30</td>
<td>Calibrated from core samples reported in Chapter 3 (the value for clay is from the the extension cord of the second-phase line of NCL)</td>
</tr>
<tr>
<td>Slope of normal consolidation line, $\lambda$</td>
<td>0.260</td>
<td>0.287</td>
<td>Calibrated from core samples reported in Chapter 3 (the value for clay is from the the extension cord of the second-phase line of NCL)</td>
</tr>
<tr>
<td>Slope of swelling line, $\kappa$</td>
<td>0.056</td>
<td>0.036</td>
<td>Calculated according to the ratio of $\kappa/\lambda$ given by Stewart (1992) and Chow et al. (2019)</td>
</tr>
<tr>
<td>Effective unit weight, $\gamma'$</td>
<td>5.5 kN/m³</td>
<td>4.7 kN/m³</td>
<td>Calibrated from core samples reported in Chapter 3</td>
</tr>
<tr>
<td>T-bar measured normally consolidated undrained strength ratio, $\left( s_u / \sigma'<em>c \right)</em>{\text{NC}}$</td>
<td>0.16</td>
<td>0.42</td>
<td>Calibrated from undrained T-bar test reported in Chapter 3</td>
</tr>
<tr>
<td>Normally consolidated undrained strength ratio, $\left( s_u / \sigma'<em>c \right)</em>{\text{NC,r}}$</td>
<td>0.20</td>
<td>0.57</td>
<td>Calculated from $\left( s_u / \sigma'<em>c \right)</em>{\text{NC}}$ using the methodology given by Zhou and Randolph (2009b)</td>
</tr>
</tbody>
</table>
CHAPTER 5 PREDICTION OF SLIDING RESISTANCE

5.1 Introduction

It is important to be able to accurately predict the peak sliding resistance of a mobile foundation in order to ensure that it will slide throughout its operational life, to ensure that any connection that could be subject to pipeline expansion loads will remain within design limits. The previous chapter demonstrated that it is possible to accurately predict the dormant berm induced peak resistance for a surface foundation if the residual sliding resistance – when the influence of berm formation is the least – can be accurately predicted. Further, it has been shown that sliding resistance evolves (cycle-by-cycle) as a function of over consolidation ratio, strain-softening due to sensitivity, consolidation induced hardening and berm accumulation due to both consolidation (during operation periods) and shear induced settlement (during shutdown and startup periods).

This chapter presents a theoretical framework – expanding on the work of Cocjin et al. (2017) – to predict sliding resistance over the whole-life of a mobile foundation. The soil model reported in Cocjin et al. (2017) is further simplified and developed in the framework, with the migrating critical state line (CSL) adopted by Cocjin et al. (2017) replaced by a unique remoulded strength line (RSL) (White and Hodder, 2010, Hodder et al., 2013, Zhou et al., 2019a, Zhou et al., 2019b, Zhou et al., 2020b) and the ‘subloading surface’ concept (Hashiguchi, 1978, Hashiguchi et al., 2002), through which the achievable peak and remoulded undrained shear strengths can be linked to the pre-shearing stress state. In addition, a non-linear failure envelope is adopted to reflect that many fine grained soils exhibit a higher friction angle at low stress levels (Maksimovic, 1989, Pedersen et al., 2003, Najjar et al., 2007b, Boukpeti and White, 2017). The methodology for estimating the berm resistance proposed in Chapter 4 is incorporated into the framework and the predictions are compared for eight centrifuge tests (reported in Chapter 3) encompassing two different soils; a kaolin clay and a carbonate silt.

5.2 Review on previous work

5.2.1 Centrifuge model test: features of a typical experiment

The whole-life sliding resistance of mobile foundations has been explored through the centrifuge model tests introduced in Chapter 3, and is first described by Cocjin et al. (2014). Figure 5.1(a) provides a representative schematic of the sliding movements for a typical experiment. In general, the mudmat was first set in the shutdown position (SD), which was followed by a long waiting period
allowing full consolidation of the subsoil. Following this, the mudmat was slid into the operational position (OP) under displacement control, mimicking pipeline thermal expansion during operation, and a period of rest time was observed representing the operational time (typically several months in prototype terms). The mudmat was then slid back to the shutdown position and held there for a comparatively brief shutdown period (a matter of hours in prototype terms). The above process repeats throughout the operational lifetime of the mudmat, under constant vertical load representing the self-weight of the mudmat and the attached equipment in the field. The above episodic actions lead to the mudmat settling in the soil surface, and the displacement of subsoil into dormant berms that form at two ends of the footprint as shown in Figure 5.1(b) and (c).

The sliding resistance evolves both during and between the sliding events, as shown in Figure 5.2 from the work of Cocjin et al. (2017), where ‘Exp’ stands for experiment and ‘Fw’ for theoretical framework. Four peaks and an increasing residual resistance are clearly apparent, with the framework described in the remainder of this chapter attempting to capture all of these features. The initial breakout peak (P1) is mainly attributed to the peak undrained shear strength of the soil at the mudmat-soil interface, which is mobilised within a very short sliding distance. With further movement, the sliding resistance dramatically softens towards a residual value that reflects the sensitivity of the soil and the generation of excess pore pressure, where the sliding distances of mobilising and softening strength do not need to scale from the centrifuge model to prototype. Due to the periodic remoulding and reconsolidation, the residual soil strength (and hence sliding resistance) will increase from the initial residual value on a cycle-by-cycle basis. The sliding events also displace the subsoil, forming active berms which eventually accumulated at the two extremities of the footprint as dormant berms. These provide significant additional resistance to sliding, resulting in another two resistance peaks (P2 and P4) that can be predicted using the methods described in Chapter 4. It appears that during reconsolidation at the operational position, the subsoil strength recovers, and the dormant berm effectively bonds to the mudmat. This is surmised because when the mudmat slides away from the operational position, the recovered base resistance and the adhered soil berm lead to another peak (P3) that mirrors the former one (P2).

5.2.2 Existing theoretical research

A simple 1D theoretical framework was proposed by Cocjin et al. (2017) to capture the cycle-by-cycle softening or hardening beneath a tolerably mobile foundation due to episodic sliding with intervening consolidation. Here, the evolution of sliding resistance is divided into two components (CP1 and CP2) as summarised in Figure 5.3. CP1 models pore pressure generation and dissipation
(hardening), while CP2 models changes in the critical state line (aka the CSL) due to the remoulding and restructuration of soil (softening). During periodic shearing events, damage in the soil structure is assumed to accumulate, reflected by the CSL permanently migrating towards the left in $e^{-\ln \sigma'_v}$ plane. This framework was compared to centrifuge model test data obtained using the kaolin clay which is commonly used in UWA. However, the following aspects were not considered and have been targeted as areas of potential improvement:

- Episodic undrained T-bar tests have shown that, even following full remoulding, at least part of the sensitivity of the soil is recovered after reconsolidation (Hodder et al., 2013, Zhou et al., 2020b).
- At very low stresses – such as those relevant to a surface sliding foundation being investigated here – a higher friction angle is often observed for fine grained soils (Maksimovic, 1989, Pedersen et al., 2003, Najjar et al., 2007b, Boukpeti and White, 2017).
- Ignoring the peak resistance, including the initial and berm induced peaks shown in Figure 5.2, could lead to overloading of connections to the mudmat.
- The model parameter $N_{95}$, used to control the softening process and referring to the number of cycles required for the current spacing ratio to be equivalent to 95% of the value of the final spacing ratio, must be back-fitted using experimental data.

To address these potential shortcomings, a new framework has been developed that incorporates: (i) pore pressure driven softening and hardening with partial recovery of sensitivity; (ii) a simple non-linear failure envelope in $\tau-\sigma'_v$; (iii) an incremental approach for calculating the sliding resistance time-history; and (iv) soil parameters that can all be calibrated using existing site investigation or laboratory tests. The model is subsequently validated using data from eight centrifuge model tests that were performed under various operational conditions and on two types of soils (kaolin clay and calcareous silt).

### 5.3 Key features of the new framework

Expanding on the discussion above, the new framework features the following improvements:

- It employs a unique fully remoulded strength line (RSL) that is parallel to the normal compression line (NCL) and anchors the fully remoulded strength. The RSL is clear in its physical meaning and easy to calculate through the intact state and soil sensitivity. The RSL
has been widely used to simulate soil strength degradation under the reloading events (White and Hodder, 2010, Hodder et al., 2013, Zhou et al., 2019a, Zhou et al., 2019b, Zhou et al., 2020b).

- It employs the subloading surface proposed by Hashiguchi (1978) to anchor the peak undrained shear strength after reconsolidation, replacing the migrating CSL in the framework of Cocjin et al. (2017). This links the achievable peak undrained shear strength to the pre-shearing stress state, such that the sensitivity can recover with the dissipation of excess pore pressure, and the extra back-calculated parameter $N_{95}$ that controls evolution of the CSL can be ignored.

- It employs a curved effective stress failure envelope in order to reflect higher friction angles at low stress. A simple equation is used to describe the curved failure envelope lying above the conventional straight-line envelope, and is coupled into the developed soil model.

- It includes the methodology of calculating the maximum dormant berm resistance, as proposed in Chapter 4. The significant extra resistance on top of the base resistance can be effectively captured at P2, P3 and P4 by the new theoretical framework.

- Finally, the new framework is built in step-by-step manner to capture more detailed features within individual cycles.

5.4 Theoretical derivation

5.4.1 Overview

The soil is discretised as a 1D column of elements beneath the foundation via a technique sometimes referred to as the “oedometer method”, following (Cocjin et al., 2017). The vertical equilibrium stress in each element within the column is defined:

$$\sigma_{v, eqm} = \sigma_{op} I_{\sigma} + \sigma'_{v0}$$

(5.1)

where $I_{\sigma}$ is a stress influence factor to be defined and $\sigma_{op}$ is the operative stress applied by the foundation to the soil, which is calculated from the effective vertical load $V_{eff}$ as:

$$\sigma_{op} = \frac{V_{eff}}{BL}$$

(5.2)
where \( B \) and \( L \) are respectively the breadth and length of the base plate. \( \sigma_v' \) is the in-situ effective stress calculated by

\[
\sigma_v' = \gamma'z
\]  

(5.3)

Similarly, the shear stress \( \tau \) mobilised in each element is defined as:

\[
\tau = \tau_{op} I_t
\]  

(5.4)

where \( \tau_{op} \) is the shear stress at the mudmat-soil interface and \( I_t \) is also a stress influence factor to be defined.

The stress influence factors \( I_\sigma \) and \( I_\tau \) are taken from the work of Poulos and Davis (1974), based on elastic solutions for the stress distribution beneath the centreline of a rectangular, uniformly loaded area on the surface of a semi-infinite mass:

\[
I_\sigma = \frac{2}{\pi} \left[ \frac{b}{r_2} \left( 1 + \frac{z^2}{2r_2^2} \right) \arctan \left( \frac{l}{r_3} \right) + \frac{l}{r_3} \left( 1 + \frac{z^2}{2r_3^2} \right) \arctan \left( \frac{b}{r_3} \right) + \frac{blz^2}{2r_3^2} \left( \frac{1}{r_3^2} + \frac{1}{r_1^2} \right) \right]
\]  

(5.5)

\[
I_\tau = \frac{2}{\pi} \left[ \arctan \left( \frac{b}{z} \right) - \frac{z}{r_1} \arctan \left( \frac{b}{r_1} \right) + \frac{b}{z} \left( \frac{b^2}{r_2^2} - \frac{b^2 + l^2}{r_3^2} \right) \right]
\]  

(5.6)

where \( I_\sigma \) and \( I_\tau \) are respectively the vertical and shear stress distribution factors, \( l = 0.5L \) and \( b = 0.5B \) with \( L > B \), and \( z \) represents the target depth. The parameters \( r_1, r_2, r_3 \) are given as:

\[
r_1 = \left( l^2 + z^2 \right)^{0.5}
\]

\[
r_2 = \left( b^2 + z^2 \right)^{0.5}
\]

\[
r_3 = \left( l^2 + b^2 + z^2 \right)^{0.5}
\]

(5.7)

### 5.4.2 Normal Consolidation Line (NCL) and Remoulded Strength Line (RSL)

The undrained shear strength of the soil is modelled based on critical state soil mechanics principles, defined via a normal consolidation line (NCL) and a remoulded strength line (RSL) in the \( e^{- \ln \sigma'_v} \) plane. The RSL that represents the ultimate remoulded state of the soil under undrained conditions was proposed in White and Hodder (2010), and further developed in Hodder et al. (2013), Zhou et al. (2019a), and comprehensively used in Zhou et al. (2019b) and Zhou et al. (2020b) to simulate the soil responses under episodic remoulding and reconsolidation events (e.g. the evolution of SCR-seabed stiffness and plate anchor capacity under cyclic loading and reconsolidation).
The normal consolidation line (NCL) in the $e - \ln \sigma_v'$ plane is taken as:

$$e = e_N - \lambda \ln (\sigma_v')_{NCL}$$

(5.8)

where $\lambda$ is the slope of NCL and $e_N$ is the void ratio intercept of NCL at $\sigma_v' = 1$ kPa. $\left(\sigma_v'\right)_{NCL}$ is the effective stress on the NCL, which can be expressed through $\sigma_v'$ and OCR as:

$$\left(\sigma_v'\right)_{NCL} = \sigma_v' OCR^\frac{\lambda - \kappa}{\lambda}$$

(5.9)

The remoulded strength line (RSL) is assumed to be parallel to NCL. Therefore, the effective stress on the RSL can be derived from:

$$\left(\sigma_v'\right)_{RSL} = \left(\sigma_v'\right)_{NCL} \left[ \frac{2}{M_0 S_t} \left( \frac{s_u}{\sigma_v'} \right)_{NC} \right]$$

(5.10)

where $\kappa$ is the corresponding slope of the unload-reload line, $M_0$ is the slope of the effective stress failure envelope in the $\tau - \sigma_v'$ plane at high stress levels (e.g. $\sigma_v' > 50$ kPa), $(s_u / \sigma_v')_{NC}$ is the normally consolidated strength ratio, and $S_t$ is the sensitivity. Note that both $(s_u / \sigma_v')_{NC}$ and $S_t$ can be evaluated from standard penetrometer testing (e.g. T-bar).

### 5.4.3 Failure envelope

As addressed in Chapter 4, a simple method is used here to describe such a curved failure envelope, utilising a multiplier $R_{\sigma_v'}$ on the strength ratio $M_0$ which is measured in conventional tests at higher stress levels (e.g. triaxial tests). The strength factor at any vertical effective stress of $\sigma_v'$ noted as $M_{\sigma_v'}$ can be derived as follows:

$$M_{\sigma_v'} = R_{\sigma_v'} M_0$$

(5.11)

where, $R_{\sigma_v'}$ is the multiplier, which might be expressed as a function of the vertical effective stress $\sigma_v'$ as:

$$R_{\sigma_v'} = 2 - \exp \left( -3 \frac{\sigma_R}{\sigma_v'} \right)$$

(5.12)
In Eq.(5.12) $\sigma_R$ is a parameter controlling the ultimate initial curvature of the non-linear failure envelope. While many other forms could have been used to describe this envelope curvature – this simple form provides a good fit to the data in Chapter 4.

### 5.4.4 Undrained shear strength

The sliding induced softening response of fine-grained soils under large amplitude movements is similar to the response of mudmat-soil interface elements, and has been explored by Andersen (2015) through direct simple shear (DSS) tests. A typical effective stress path in $\tau - \sigma'$ plane has been proposed for the normally consolidate (NC) or lightly over-consolidated (LOC) soil, indicating that the peak undrained shear strength is first mobilised, before a reduction in strength that is driven by the generation of excess pore pressure as shown in Figure 5.4.

The dissipation of these excess pore pressures induces hardening of the soil, as has been observed through episodically cyclic undrained T-bar tests performed in kaolin clay by White and Hodder (2010), and in calcareous silt by Zhou et al. (2020b). By performing episodes of undrained T-bar cycling in between periods of excess pore pressure dissipation, these tests show that the undrained shear strength decays from a peak towards a residual value over a number of penetration and extraction cycles. The residual strength increases between events due to reconsolidation, and will subsequently soften again during the next episode of cycling with a reduced apparent sensitivity. A typical undrained shear strength evolution as measured through the episodically cyclic T-bar tests (Zhou et al., 2020b) is shown in Figure 5.5. These tests show that soil sensitivity is partially recoverable after periods of excess pore pressure dissipation.

In order to simulate such properties, the achievable peak and fully remoulded undrained shear strengths need to be first anchored. NCL and RSL are used herein to represent the intact and the fully remoulded states respectively. If the normally consolidated soil has not experienced any disturbance prior to shearing (i.e. the pre-shearing effective stress is located on the NCL), then the peak undrained shear strength should be reached at a stress defined by the conventional critical state. However, if the soil has been partially remoulded with the pre-shearing effective stress lying between the NCL and RSL, a smaller peak undrained shear strength would be expected. To simulate this in a simple manner, the subloading surface concept (Hashiguchi, 1978, Hashiguchi et al., 2002), originally proposed for separating the ‘normal state’ representative of the intrinsic soil property and the over-consolidated state, is adopted here to moderate the peak undrained shear strength as a function of the pre-shearing stress state.
In order to reflect the position of the present effective stress relative to the NCL and RSL, the normalised stress state factor $\psi$ is used, which is expressed as

$$\psi = \frac{\sigma'_v - \sigma'_v^{\text{RSL}}}{\sigma'_v^\text{NCL} - \sigma'_v^{\text{RSL}}}$$

(5.13)

If the stress state lies on the NCL, the soil is in its ‘intact’ state with $\psi = 1$. Conversely, if the stress state lies on the RSL, the soil is regarded as fully remoulded with $\psi = 0$.

In continuous undrained shear, $\psi$ has been assumed to decrease exponentially with cumulative plastic shear strain, $\gamma$ (Hodder et al., 2013):

$$\psi = \exp\left(-3\frac{\gamma}{\gamma_{95}}\right)$$

(5.14)

where $\gamma_{95}$ refers to the cumulative plastic shear strain causing 95% remoulding on soil.

Einav and Randolph (2005) suggested that the achievable undrained shear strength similarly decays exponentially with $\gamma$ according to:

$$\frac{s_{u,s}}{s_{u,r}} = 1 + (S^*_v - 1)\exp\left(-3\frac{\gamma}{\gamma_{95}}\right)$$

(5.15)

where $s_{u,s}$ is the undrained shear strength on the subloading surface, $s_{u,r}$ is the fully remoulded strength on the RSL, and $S^*_v$ is the effective sensitivity – defined as the present sensitivity excluding the already damaged irrecoverable sensitivity, which considers both current irrecoverable sensitivity and recoverable sensitivity components, and the influence of the curved failure envelope.

For a normally consolidated state ($\psi = 1$), $s_{u,s}$ is simply the peak undrained shear strength which can be calculated following Wroth (1984) as:

$$s_{u,s} = \left(\frac{s_u}{\sigma'_v}\right)^{\text{NCL}}\frac{s_u}{\sigma'_v^{\text{NC,r}}}$$

(5.16)

where $\left(\frac{s_u}{\sigma'_v}\right)^{\text{NC,r}}$ is the real strength ratio of the normally consolidated soil, corrected from T-bar measured according to the approach reported by Zhou and Randolph (2009b). For intermediate stress states ($0 < \psi < 1$), by combining and rearranging Eq.(5.14) and Eq.(5.15), the ‘achievable’ undrained shear strength on the subloading surface $s_{u,s}$ can be estimated as a function of the pre-shearing effective stress state as follows:
\[ s_{u,s} = s_{u,r} \left[ 1 + (S^*_t - 1) \psi \right] \] (5.17)

The relationship between the pre-shearing effective stress and the achievable undrained shear strength on subloading surface is illustrated in Figure 5.6 (a) in schematic form. Finally, the remoulded undrained strength \((\psi = 0)\), \(s_{u,r}\), is taken as:

\[ s_{u,r} = 0.5M_{\sigma_v} (\sigma'_v)_{RSL} \] (5.18)

5.4.4.1 Sensitivity

The sensitivity of the soil is assumed to consist of two components: a recoverable part representing the influence of excess pore pressure generation and dissipation, and an irrecoverable part representing the influence of structure or fabric on the undrained strength that once damaged, does not recover due to reconsolidation (Hodder et al., 2013). The effective sensitivity, \(S^*_t\), can be defined to account for both aspects of sensitivity as a function of the sensitivity measured at conventional effective stress levels by a suitable test – such as a cyclic undrained T-bar test – as follows:

\[ S^*_t = \frac{(1 - F_{s0} + F_s)S_{t,r}}{R(\sigma'_v)_{RSL}} \] (5.19)

where \(F_{s0}\) and \(F_s\) are the initial and current proportion of the irrecoverable sensitivity, respectively, and \(R(\sigma'_v)_{RSL}\) accounts for the curvature of the failure envelope at the effective stress of \((\sigma'_v)_{RSL}\). \(S_{t,r}\) is the real sensitivity of normally consolidated soil, corrected from the T-bar measured according to the approach reported by Zhou and Randolph (2009b). If \(F_{s0}\) is taken as zero, then all of the sensitivity is recoverable. The proportion of irrecoverable sensitivity throughout sliding evolves as a function of strength mobilisation \((\tau / s_u)^\chi\) as follows:

\[ F_s = F_{s(N-\Delta N)} \left[ 1 - \max \left( \frac{\tau}{s_u} \right)^\chi \right] \] (5.20)

where \(\chi\) is a power term controlling the rate of degradation, and \(F_{s(N-\Delta N)}\) is the value of \(F_s\) in the previous increment, i.e. \(N-\Delta N\), of the calculation.

5.4.4.2 Peak undrained shear strength

The peak undrained shear strength \((s_{u,p})\) is determined by both the strength on the subloading surface \((s_{u,s})\) and the effect of over consolidation, which leads to a further increase in strength. Although the degree of over consolidation expected for sliding foundations is small, it is important to account for
this because the soil on which they are expected to operate is unlikely to be perfectly normally consolidated. In the centrifuge tests described in Chapter 3, the over-consolidation ratio was in the range of 1~2.1.

Figure 5.6(b) illustrates the relationship between the undrained shear strength on the subloading surface and the peak undrained shear strength. The peak undrained shear strength $s_{u,p}$ is estimated as a function of the over-consolidation factor ($F_{oc}$), reflecting the additional undrained strength in excess of the subloading surface strength, $s_{u,s}$. This can be expressed as:

$$s_{u,p} = (1 + F_{oc}) s_{u,s} \quad \text{(5.21)}$$

For normally consolidated conditions, $F_{oc}$ may be taken as zero, with increasing values indicating an increasing degree of over consolidation. The initial value of $F_{oc}$ (denoted as $F_{oc0}$) is back-calculated using Eq.(5.21) as:

$$F_{oc0} = \frac{s_{u,p0}}{s_{u,s}} - 1 \quad \text{(5.22)}$$

In this equation, $s_{u,p0}$ is the initial peak undrained shear strength mobilised prior to any remoulding event. According to Wroth (1984), $s_{u,p0}$ of over-consolidated soil can be estimated as:

$$s_{u,p0} = \sigma_{v,eqm} \left( \frac{s_u}{\sigma_{v,NC,r}} \right)^{\frac{\lambda-\kappa}{\lambda}} \text{OCR} \quad \text{(5.23)}$$

Shearing events are assumed to break down the over consolidation induced structure (Asaoka et al., 2000, Najjar et al., 2007b). In the same manner as was assumed for the irrecoverable component of sensitivity, the degradation of the enhanced strength due to over consolidation induced structure is assumed to be determined by the strength mobilisation $\left( \frac{\tau}{s_u} \right)^{\epsilon}$. Therefore, the evolution of $F_{oc}$ can be expressed similarly as:

$$F_{oc} = F_{oc(N-\Delta N)} \left[ 1 - \max \left( \frac{\tau}{s_u} \right)^{\epsilon} \right] \quad \text{(5.24)}$$

where $F_{oc(N-\Delta N)}$ is the value of $F_{oc}$ in the previous increment of the calculation.
5.4.4.3 Undrained shear strength mobilisation and degradation

Prior to mobilisation of the peak undrained shear strength, the achievable undrained shear strength is expected to be equal to the peak value, $s_u,p$. After the peak undrained shear strength is attained, further shearing induces strength softening due to the sensitivity of the soil. A simple exponential form (Einav and Randolph, 2005) is used to express the softened undrained shear strength ($s_u$) as:

$$s_u = \min \left( \frac{s_u,r}{s_u,p} + \left( 1 - \frac{s_u,r}{s_u,p} \right) \exp \left( \frac{-3 \frac{\delta_r}{\delta_{95}}}{1} \right) \right) s_u,p$$

(5.25)

where $\delta_{95}$ represents the shearing distance required to generate a strength reduction of 95% from the peak towards the residual value and $\delta_r$ is the post-peak remoulding distance. $\delta_r$ may be taken as:

$$\delta_r = \delta - \delta_p$$

(5.26)

where $\delta$ is the sliding distance in a single slide, and $\delta_p$ is the sliding distance required to mobilise the peak undrained shear strength.

A simple linear mobilisation is assumed because, compared to the large sliding distance of mobile foundations (in reality usually several meters), the distance required to mobilise the peak undrained strength is very small (likely a few millimetres). Although linear mobilisation is assumed, this process is still regarded as fully plastic. Accepting this, the operative shear stress $\tau_{op}$ is taken as:

$$\tau_{op} = \min \left( \frac{\delta}{\delta_p}, 1 \right) \min \left( \frac{s_u}{I_r} \right)$$

(5.27)

This conservatively assumes that the mudmat-soil interface is moderately rough, and as such the shear stress developed at the interface is limited by the undrained shear strength of the underlying soil. This chapter is concerned only with the behaviour of the weakest element, which is almost always the surface element, where the magnitude of $I_r$ is unity.

5.4.5 Pore pressure generation

Figure 5.6 (c) demonstrates the stress path of mobilisation and degradation with generating excess pore pressure. The generated excess pore pressure is determined by $\left( \tau / s_u \right)^\beta$ following Cocjin et al. (2017). Where $\beta$ is the power of controlling the magnitude of pore pressure generation. The effective stress following generation of excess pore pressure, denoted as $\sigma_{v,gen}'$, is taken as:
\[ \sigma'_{v,gen} = \min \left[ \sigma' - \left( \sigma'_{v,cs} \left( \frac{\tau}{s_u} \right)^\beta \right) \right] \] (5.28)

where \( \sigma'_{v,cs} \) is the effective stress on the failure envelope in \( \tau - \sigma'_v \) plane, which changes along the failure envelope as calculated by:

\[ \sigma'_{v,cs} = \frac{s_u}{0.5 M |\sigma'_{v,cs}|} \] (5.29)

### 5.4.6 Pore pressure dissipation

Figure 5.6 (d) demonstrates the stress path during excess pore pressure dissipation. The effective stress after dissipation (\( \sigma'_{v,dis} \)) can be calculated as:

\[ \sigma'_{v,dis} = \sigma'_{v,gen} - \Delta u_{dis} \] (5.30)

where \( \Delta u_{dis} \) is the change in pore pressure due to dissipation. This in turn can be calculated by:

\[ \Delta u_{dis} = \left[ \exp \left( \frac{\Delta e}{\kappa} \right) - 1 \right] \sigma'_{v,gen} \] (5.31)

where \( \kappa \) is the slope of the unloading-reloading line in \( e - \ln \sigma'_v \) plane. \( \Delta e \) is the reduction in void ratio induced by pore pressure dissipation, written as:

\[ \Delta e = \Delta e_{U=1} U \] (5.32)

where \( \Delta e_{U=1} \) is the reduction in void ratio when the excess pore pressure fully dissipates, as given by:

\[ \Delta e_{U=1} = \kappa \ln \left( \frac{\sigma'_{v,gen}}{\sigma'_{v,gen}} \right) \] (5.33)

The \( U \) in Eq.(5.32) is the degree of consolidation defined in terms of the normalised time factor of the soil horizon, which may be expressed as:

\[ U = 1 - \frac{1}{1 + \left( \frac{T}{T_{50}} \right)^m} \] (5.34)

where \( T_{50} \) is the dimensionless factor for 50% of the consolidation settlement to occur and \( m \) is a constant. \( T \) is the normalised time factor given by:
\[ T = \frac{c_{ref} t}{d^2} \] (5.35)

where \( t \) is the consolidation time, \( d \) is the drainage path length (which is taken as the width \( B \) for a rectangular mudmat) and \( c_{ref} \) is the reference coefficient of consolidation as defined by Cocjin et al. (2014).

### 5.4.7 Sliding resistance

The total horizontal sliding resistance (\( H \)) is comprised of two parts: the resistance on the mudmat-soil interface, and the resistance generated in moving the soil berms that form ahead of the leading edge of the sliding mudmat. A methodology was proposed in Chapter 4 to calculate the maximum resistance generated by the dormant berm (\( H_{b,P}^* \)) as a function of the mudmat trajectory, where strain-softening was ignored for simplicity. By substituting the undrained shear strength used in Chapter 4 with the peak undrained strength, \( s_{u,p} \), defined above, the influence of strain-softening can also be captured.

Some of the soil in the dormant berm is dragged into the footprint due to it becoming ‘attached’ to the foundation during reconsolidation during operation. This deposited soil will be pushed back into the dormant berm during the next slide. This process is modelled simply by assuming that it begins after 70% of the maximum sliding distance, \( \delta_{\max} \), has occurred. Therefore, the berm resistance can be estimated as:

\[
H_{b,P2(4)} = H_{b,P2(4)}^* \left[ \max \left( \frac{\delta - 0.7\delta_{\max}}{0.3\delta_{\max}}, 0 \right) \right]^2
\] (5.36)

where \( H_{b,P2} \) and \( H_{b,P4} \) are the additional resistances induced by soil berms at locations P2 and P4, respectively. After reconsolidation, the soil berm bonds to the mudmat and imposes a pulling force on the mudmat as it moves away during shutdown. The force due to the dormant berm, denoted as \( H_{b,P3} \), is assumed to be equal to that generated by mobilisation of the dormant berm at location P2 during the transition from the operational position. In other words, decoupling of the mudmat and the berm from operational position is assumed to mirror the combination of the mudmat and the dormant berm during the mobilisation process. Therefore the mobilisation process, \( H_{b,P3} \) can be written as:

\[
H_{b,P3} = H_{b,P2(N+1)}^* \left[ \max \left( \frac{0.3\delta_{\max} - \delta}{0.3\delta_{\max}}, 0 \right) \right]^2 \min \left( \frac{\delta}{\delta_p}, 1 \right)
\] (5.37)
Prediction of sliding resistance

Where, $H_{b,P2}^{*}$ is the maximum berm strength at P2 after reconsolidation. When the mudmat slides forward from the shutdown to the operational position, the total horizontal resistance can be calculated as:

$$H = \tau_{op}BL + H_{b,P2}$$  \hspace{1cm} (5.38)

When the mudmat slides back from the operational position, the total horizontal resistance is:

$$H = \tau_{op}BL + H_{b,P4} + H_{b,P3}$$  \hspace{1cm} (5.39)

5.5 Calibration of input parameters

5.5.1 Operational conditions

To validate the framework defined in the preceding subsections, the eight centrifuge model tests reported in Chapter 3 have been simulated. Table 5.1 summarises the operational conditions of the two sets of tests (C1-C3 and S1-S5). Note that the effective load on the base plate is determined from the operative vertical load by accounting for embedment of the mudmat into the subsoil, whereby the skis along the two long edges of the mudmat contribute to the vertical resistance, as explained in Figure 5.7 (see Chapter 3 for further detail). The effective load on base plate can therefore be approximated by:

$$V_{eff} = \frac{V_{op}B}{B + 2 \min(h, w) \cot \theta_{mat}}$$  \hspace{1cm} (5.40)

where $V_{op}$ is the operative load, $w$ is the embedment of the mudmat, and $h$ and $\theta_{mat}$ are respectively the thickness and the slope angle of the skis of the mudmat.

5.5.2 Consolidation coefficient

The consolidation coefficient is widely reported to be stress-dependent (House et al., 2001). An expression for the representative consolidation coefficient ($c_{ref}$) of the kaolin clay, as measured by surface piezo-foundation tests, was given by Cocjin et al. (2014) as:

$$c_{ref} = 2.7\left[0.3 + 0.16\left(\sigma'_v\right)_{NCL}^{-0.47}\right]$$  \hspace{1cm} (5.41a)

Similarly, $c_{ref}$ for the carbonate silt was reported in Chapter 3 as:
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

\[ c_{ref} = 0.33 \left[ \left( \sigma'_v \right)_{NCL} \right]^{0.47} \]  \hspace{1cm} (5.41b)

5.5.3 Sensitivity and strength ratio

Sensitivity and strength ratio are calibrated using an episodic cyclic undrained T-bar test, as introduced by Hodder et al. (2013) and reported more recently in Zhou et al. (2020b). In this test, a T-bar is penetrated into the soil and cycled continuously within a fixed depth range to measure degradation from the intact to the remoulded undrained shear strength. The T-bar is then extracted before a pause to allow full dissipation of the excess pore pressure accumulated in the remoulded soil. This process is repeated for multiple packets of cycles, which yields data that allows the rate of softening, sensitivity and recovery of sensitivity to be approximated.

The remoulding range of depth was 1-3m and 4.5-11.5m (prototype scale) in the clay and silt, respectively. The sample of clay was scraped for 2.25m before the T-bar test. The average unit weights of clay and silt are 5.5 and 4.7 respectively, referring to Chapter 3. The strength ratio is back-calculated according the measured strength profile through the equation (Wroth, 1984):

\[ s_u = \sigma'_v \left( \frac{s_u}{\sigma'_v} \right)_{OCR} \frac{1-K}{\lambda} \] \hspace{1cm} (5.42)

The measured strength ratios of clay and silt are respectively 0.16 and 0.42, and their sensitivities measured by T-bar are 2.5 and 4.2.

It is noted that the T-bar measured strength ratio and sensitivity need to be corrected to the ‘true’ undisturbed condition, reflecting that the initial penetration is considered representative of the strength after 0.25 cycles of remoulding (Randolph et al., 2007). To account for this, Randolph et al. (2007) proposes that the ratio of the intact strength \( (s_{u,n=0}) \) to the T-bar measured peak strength \( (s_{u,n=0.25}) \) can be estimated by

\[ F_{T-bar} = \frac{s_{u,n=0}}{s_{u,n=0.25}} \approx \left( \frac{s_{u,n=0.25}}{s_{u,n=0.75}} \right) \] \hspace{1cm} (5.43)

where \( s_{u,n=0.75} \) is the T-bar measured strength during the first extraction pass.

The intact strength ratio \( \left( \frac{s_u}{\sigma'_v} \right)_{NC,r} \) can then be expressed as

\[ 5-15 \]
The predicted sliding resistance is given by

\[
\left( \frac{S_u}{\sigma'_{vry}} \right)_{NC,r} = F_{\text{thar}} \left( \frac{S_u}{\sigma'_{vry}} \right)_{NC}
\]

(5.44)

and the real sensitivity of a normally consolidated soil (at high stress level) \( S_{t,r} \) can be expressed as

\[
S_{t,r} = F_{\text{thar}} S_t
\]

(5.45)

In order to simulate the episodic cyclic T-bar tests, the subloading surface method proposed earlier (referring to Eq.(5.21)) is again employed to calculate the peak undrained shear strength after reconsolidation assuming full dissipation between each packet of cycles. Einav and Randolph (2005) proposed an exponential degradation law to describe the strength softening process within a packet of undrained cycles in a T-bar test:

\[
s_u = \left[ \frac{S_{u,r}}{S_{u,p}} + \left( 1 - \frac{S_{u,r}}{S_{u,p}} \right) \exp \left( -3 \frac{N_{t,i}}{N_{t,95}} \right) \right] S_{u,p}
\]

(5.46)

where \( N_{t,95} \) is the number of cycles to attain 95% degradation from the peak undrained shear strength to the residual, and \( N_{t,i} \) is the number of cycles counted from the start of each packet of cycles. The first penetration of each episode is taken as the 0.25th cycle and the increment is 0.5th cycle for a single extraction or penetration pass (Zhou and Randolph, 2009b).

The measured in-situ strength profiles of the kaolin clay and calcareous silt are compared with the simulations in Figure 5.8(a) and (b), while the measured and simulated evolution of undrained shear strength with number of penetration cycles is compared in Figure 5.9(a) and (b). Observing the measured degradation during the first packet of silt and clay cycling, the strength continues to decrease over 20 cycles of penetration with the early phase decreasing exponentially and the latter phase linearly. As surmised in Chapter 3, the linear-degradation phase may be influenced by other mechanisms ignored in the relatively simple framework model, such as gradual homogenization of the undrained strength above and below the depth of the discrete measurements. Therefore, the sensitivity is defined solely by the phase exhibiting exponential degradation – and the framework appears able to simulate the strength evolution of the episodically cyclic T-bar tests.

\( F_{\text{thar}} \) is simulated to be 1.25 and 1.35 respectively for clay and silt which are close to values given by Eq.(5.43). Therefore, \( \left( \frac{S_u}{\sigma'_{vry}} \right)_{NC,r} \) becomes 0.20 and 0.57 for the clay and silt respectively. Similarly, while \( S_{t,r} \) becomes 3.1 and 5.7 for clay and silt.

5-16
The irrecoverable proportion of sensitivity ($F_a$) is 0% for the kaolin clay and 43% for the carbonate silt. This is in broad agreement with an alternative analysis performed on the same data by Singh et al. (2021). The significant proportion of the sensitivity that is irrecoverable for the carbonate silt is attributed to the contained calcareous material such as the fragments of shell being broken by remoulding actions.

5.5.4 Other parameters

The remaining input parameters used in the framework are provided in Table 5.2. The components relevant to strength mobilisation and degradation, and pore pressure generation, require 3 further parameters - including $\delta_p$ and $\delta_{95}$ that control the evolution of the shape of the sliding resistance curve, and which are not expected to scale according to centrifugal acceleration. In practice, where the anticipated sliding distance will be of the order of several meters, the magnitude of these values is inconsequential - although they could be calibrated from an interface shear box test if needed for specific cases. The power $\beta$ and $\chi$ have influences on the sliding resistance response only if the peak undrained shear strength is not mobilised (i.e. when $\delta < \delta_p$). Therefore, once again, in most practical circumstances, the selection of values for $\delta_p$, $\delta_{95}$, $\beta$ and $\chi$ has very limited impact on the evolution of the mobilised sliding resistance.

The settlement and rotation of the mudmat, which is required to estimate the additional resistance generated due to soil berm accumulation, are taken directly from the measurements. Their prediction is considered separately in Chapter 6.

5.5.5 Summary

The framework comprises of 16 input parameters, excluding those related to foundation geometry and loading condition. These parameters can be divided as three types:

- Parameters that fall within stable ranges and are largely independent of soil type, including $\gamma'$, $T_{50}$ and $m$. Of these – $\gamma'$ can be taken as 5kN/m$^3$ if measurements are not available; while $T_{50}$ and $m$ are recommended based on finite-element analysis as 0.0043 and 1.05 respectively.
- Parameters that are soil specific, including $(s_u / \sigma_v)'_{NC}$, $S_0$, $M_0$, $\sigma_R$, $c_{ref}$ and $\lambda / \kappa$. Of these – $(s_u / \sigma_v)'_{NC}$, $S_0$, $M_0$ and $\sigma_R$ may be measured using shallow penetrometers such as the hemiball, toroid and ring penetrometers (Yan, 2013, Randolph et al., 2018, Schneider et al., 2020a, Schneider et al., 2020b), which are designed to measure the strength properties of soil at
shallow; $c_{ref}$ can be measured using a piezofoundation; and $\lambda/\kappa$ can be determined through laboratory tests such as the odometer test. An estimation of $\lambda/\kappa$ based on large data sets is also acceptable for practical use.

- Parameters that are considered to have only minor influence on the prediction of sliding resistance, including $e_N$, $\delta_p$, $\delta_{\theta_5}$, $\beta$ and $\chi$. The values of clay listed in Table 5.2 can be taken as a universal use.

### 5.6 Validation and discussion

The evolution of horizontal sliding resistance for the 1st, 10th and 40th cycles is plotted against the normalised horizontal displacement ($\delta/B$) for each test in Figure 5.10 showing good predictions. Note that:

- Test C3 (Figure 5.10 (e)) shows the 20th cycle instead of the 40th cycle due to the mudmat partially losing contact with the footprint after around 25 cycles (Cocjin et al., 2014).
- Test S5 (Figure 5.10 (h)) shows the 1st and 10th cycles since only 10 cycles were performed in this scenario.

The stress paths of the interface element in $e$-$\ln\sigma'$ plane for two typical cases (tests C1 and S1) are shown in Figure 5.11(a) and (b). In general, the peak undrained shear strength is mobilised during the first slide, following by further softening towards the remoulded undrained shear strength as defined by the RSL. With intervening reconsolidation between sliding events, the undrained strength partially recovers, and then softens again when subjected to the next sliding event. The stress path zigzags downwards towards the drained limit, which is where the effective stress state is equal to the operative stress, thus implying that no excess pore pressure is generated during further undrained shear. The remoulded strength and the corresponding residual resistance evolves along the RSL via periodic remoulding and reconsolidation, resulting in the cycle-by-cycle hardening observed in the experiments.

The normalised measured residual sliding resistance is plotted against that predicted by the framework in Figure 5.12. The discrepancy for test C3 during later cycles, which was caused by partial loss of contact between the mudmat and the footprint, is clearly visible. According to the cumulative distribution figure (CDF) (Figure 5.13), 86% of the framework predictions fall within 20% of the measurements. Also highlighted on Figure 1.13 is the observation that if the curved failure envelope was not considered, the residual sliding resistance will be significantly underestimated.
These results illustrate that the combined use of the RSL parallel to NCL and the curved failure envelope can simulate the evolution of the remoulded undrained strength and of the corresponding residual sliding resistance of a mobile foundation.

The predicted initial peak resistances (P1) of the eight experiments are compared to the measurements in Figure 5.14. The predictions are around 12% lower than the measurements on average. This phenomenon could be caused by two reasons:

- First, the mudmat may have settled into the soil somewhat during touching down, potentially resulted in more soil being mobilised on initial movement of the foundation.
- Second, rate effects could lead to the peak strength being somewhat higher than estimated here (Tsubakihara and Kishida, 1993, Tika et al., 1996, Lehane et al., 2009).

The predicted peak resistances at P2 and P4 are evaluated in Figure 5.15. Again, the framework generates reasonable predictions on the berm induced peak resistance, with 78% of points falling within 20% of the experimental measurement. Compared to using the hardening model reported in Chapter 4, where the berm resistance was moderately over-predicted, by incorporating the softening aspect of soil behaviour a modest reduction in the bias in predicted berm resistance has been achieved.

The predicted peak resistance at P3 is evaluated in Figure 5.16. The existence of P3 can be attributed to both the recovery of base resistance and the adhesion of the mudmat to the dormant berm due to reconsolidation. The results show that the current framework can relatively well predict this aspect of the sliding response with 74% of points falling within 20% of the measurements.

5.7 Conclusions

This chapter has presented a theoretical framework that extends the work of Cocjin et al. (2017) to predict the whole-life sliding resistance of a tolerably mobile subsea foundation subjected to undrained episodic sliding with intervening consolidation. The framework developed has been shown to be capable of predicting the evolution of both residual and peak sliding induced resistances using a set of eight centrifuge model tests performed on two types of soils; a kaolin clay and a carbonate silt. Key improvements captured in the new framework can be summarised as:

- A displacement based rather than cycle-by-cycle methodology has been developed to simulate the sliding resistance of a tolerably mobile foundation. This allows the initial peak, mid-cycle
residual sliding resistance and peak resistances due to dormant berm mobilisation to be simulated.

- A model for undrained soil strength evolution that encapsulates a means of simply but robustly simulating softening, sensitivity, and reconsolidation induced recovery of sensitivity has been proposed. This has resulted in the ability to model the full evolution of the sliding process, over multiple cycles of sliding, with varying degrees of reconsolidation between sliding events.

- The strength model is characterised by a remoulded strength line (RSL) that is assumed to be parallel to the NCL. The position of the RSL can be calculated simply via the intact strength and sensitivity as measured by a T-bar test. A curved effective stress failure envelope has also been adopted in the framework, and has been shown to have a significant influence on the accuracy of the sliding resistance predictions.

- The mobilisable peak undrained shear strength is connected to the pre-shearing stress state in the developed soil model. The subloading surface concept was used to replace the migrating CSL adopted by Cocjim et al. (2017), which provides a simpler way to estimate the peak undrained shear strength at any pre-shearing stress state, and has been modified to incorporate the flexibility to model the partial recovery in sensitivity due to reconsolidation.

- The additional sliding resistance at location P2, P3 and P4 due to mobilisation of dormant berms at either end of the sliding foundation footprint has been incorporated in the framework. The addition of these almost always resulted in the largest resistances across the whole-life of the sliding foundation, so being able to model this aspect of the response is a critical contribution to ensuring that foundations intended to slide and are able to do so without damaging the connections carrying the loads (from flowline to foundation).
Figure 5.1 Schematic and photos of the mobile foundation: (a) schematic of the mudmat on sliding footprint; (b) photo of the mudmat at shutdown position; (c) photo of the footprint after the testing.

Figure 5.2 Measured and predicted sliding resistance of mobile foundation at the 1st and 20th cycles (Cocjin et al., 2017)
### List of key components of the advanced theoretical framework proposed by Cocjin et al. (2017)

<table>
<thead>
<tr>
<th>Analysed domain</th>
<th>Components</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Target element</strong></td>
<td><strong>CP1</strong></td>
</tr>
<tr>
<td></td>
<td>• Pore pressure generation</td>
</tr>
<tr>
<td></td>
<td>• Pore pressure dissipation</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Analysed domain</strong></td>
</tr>
<tr>
<td></td>
<td><strong>CP1</strong></td>
</tr>
<tr>
<td></td>
<td>• Pore pressure generation</td>
</tr>
<tr>
<td></td>
<td>• Pore pressure dissipation</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* $N_{95}$ is the number of cycles required for the current spacing ratio to be equivalent to 95% of the value of the final spacing ratio.

---

**Figure 5.3** List of key components of the advanced theoretical framework proposed by Cocjin et al. (2017)

---

**Figure 5.4** Typical effective stress path in $\tau - \sigma'$ plane of a NC and LOC soil under monotonic loading (Andersen, 2015)
Figure 5.5 The normalised undrained shear strength evolution measured through typical episodically cyclic T-bar tests (Zhou et al., 2020b)
Figure 5.6 Flowchart of the strength mobilisation and degradation and, the pore pressure generation and dissipation

Figure 5.7 Schematic of mudmat embedding into subsoil
Figure 5.8 Comparison on the T-bar measured and simulated in-situ strength profile in (a) the kaolin clay; (b) the calcareous silt
Figure 5.9 Comparison on the T-bar measured and the simulated strength evolution of the episodically cyclic T-bar test plotted against number of cycles: (a) the kaolin clay; (b) the calcareous silt
Figure 5.10 Comparison on the evolution of the normalised horizontal resistance ($H/V_{eff}$) with the normalised horizontal displacement ($\delta/B$): (a)-(h) are respectively C1, S1, C2, S2, C3, S3, S4 and S5.
Figure 5.10 Continued
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

Figure 5.11 Stress path of the interface element plotted in $e - \sigma'_v$ plane: (a) C1, (b) S1

Figure 5.12 Measured against predicted normalised residual resistance
Figure 5.13 Cumulative distribution of the ratio of the measured to the predicted residual resistance with using the present soil model and that based on the linear failure envelope.

Figure 5.14 Measured against predicted normalised peak resistance (P1)
Figure 5.15 Cumulative distribution of the ratio of the measured to the predicted peak resistance at P2 and P4 with the berm resistance calculated using the present soil model and that of Chapter 4

Figure 5.16 Cumulative distribution of the ratio of the measured to the predicted peak resistance (P3)
Table 5.1 Summary on operational conditions of centrifuge model tests C1-C3 and S1-S5

<table>
<thead>
<tr>
<th></th>
<th>Operational load, $V_{op}$ (N)</th>
<th>Initial settlement, $w_0$ (mm)</th>
<th>Consolidation time, $t_{con}$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>9.3</td>
<td>1.4</td>
<td>780</td>
</tr>
<tr>
<td>C2</td>
<td>17</td>
<td>2.3</td>
<td>780</td>
</tr>
<tr>
<td>C3</td>
<td>9.3</td>
<td>1.9</td>
<td>4680</td>
</tr>
<tr>
<td>S1</td>
<td>20</td>
<td>2.0</td>
<td>780</td>
</tr>
<tr>
<td>S2</td>
<td>30</td>
<td>1.6</td>
<td>780</td>
</tr>
<tr>
<td>S3</td>
<td>20</td>
<td>2.2</td>
<td>4680</td>
</tr>
<tr>
<td>S4</td>
<td>20</td>
<td>2.3</td>
<td>4680 for the 7th, 14th, 28th, ... cycles 780 for the rest</td>
</tr>
<tr>
<td>S5</td>
<td>20</td>
<td>1.5</td>
<td>4680</td>
</tr>
</tbody>
</table>
### Table 5.2 Framework parameters

<table>
<thead>
<tr>
<th>Framework components</th>
<th>Parameter</th>
<th>Description</th>
<th>Clay</th>
<th>Silt</th>
<th>Source of selected values</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Boundary conditions</strong></td>
<td>$B$</td>
<td>Foundation breadth (model scale)</td>
<td>50mm</td>
<td>50mm</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$L$</td>
<td>Foundation length (model scale)</td>
<td>100mm</td>
<td>100mm</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$h$</td>
<td>Foundation height (model scale)</td>
<td>5mm</td>
<td>5mm</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\theta_{\text{mat}}$</td>
<td>Base angle of ski</td>
<td>$30^\circ$</td>
<td>$30^\circ$</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\delta_{\text{max}}$</td>
<td>Maximum single sliding distance</td>
<td>$L/4$</td>
<td>$L/4$</td>
<td>Test parameter reported in Chapter 3</td>
</tr>
<tr>
<td><strong>Initial condition</strong></td>
<td>$\sigma_{\text{sur}}$</td>
<td>Surcharge</td>
<td>3.5kPa</td>
<td>5.0kPa</td>
<td>Estimated according to the thickness of the scraped soil reported in Chapter 3</td>
</tr>
<tr>
<td><strong>Basic properties</strong></td>
<td>$\gamma'$</td>
<td>Submerged unit weight of soil</td>
<td>5.5kN/m$^3$</td>
<td>4.7kN/m$^3$</td>
<td>Calibrated from core samples reported in Chapter 3</td>
</tr>
<tr>
<td><strong>Compression and swelling line</strong></td>
<td>$e_N$</td>
<td>Intercept void ratio at $\sigma'_v = 1$ of the Normal Compression Line (NCL) in $e - \sigma'_v$ plane</td>
<td>2.45</td>
<td>3.30</td>
<td>Calibrated from core samples reported in Chapter 3 (the value for clay is from the the extension cord of the second-phase line of NCL)</td>
</tr>
<tr>
<td></td>
<td>$\lambda$</td>
<td>Slope of NCL at high stress level</td>
<td>0.26</td>
<td>0.287</td>
<td>Calibrated from core samples reported in Chapter 3 (the value for clay is from the the extension cord of the second-phase line of NCL)</td>
</tr>
<tr>
<td></td>
<td>$\kappa$</td>
<td>Slope of swelling line at high stress level in $e - \ln \sigma'_v$ plane</td>
<td>0.056</td>
<td>0.036</td>
<td>Calculated according to the ratio of $\kappa/\lambda$ given by Stewart (1992) and Chow et al. (2019)</td>
</tr>
<tr>
<td><strong>Failure envelope</strong></td>
<td>$M_0$</td>
<td>Strength factor of high stress level</td>
<td>0.92</td>
<td>1.62</td>
<td>Calibrated through triaxial test, here is given by Stewart (1992) and Chow et al. (2019)</td>
</tr>
</tbody>
</table>
### Prediction of sliding resistance

<table>
<thead>
<tr>
<th>Framework components</th>
<th>Parameter</th>
<th>Description</th>
<th>Clay</th>
<th>Silt</th>
<th>Source of selected values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_k$</td>
<td>Parameter to control the shape of the failure envelope</td>
<td>0.16</td>
<td>0.40</td>
<td>Calibrated from triaxial test, here is calibrated through model tests C3 and S3 reported in Chapter 3</td>
</tr>
<tr>
<td>Strength mobilisation and degradation and pore pressure generation</td>
<td>$\delta_p$</td>
<td>sliding distance for fully mobilising the peak strength</td>
<td>1mm</td>
<td>2mm</td>
<td>Observed from model tests C3 and S3 reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\delta_{95}$</td>
<td>Sliding distance for 95% remoulding the peak strength</td>
<td>2mm</td>
<td>10mm</td>
<td>Observed from model tests C3 and S3 reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\beta$</td>
<td>Excess pore pressure parameter controlling the effective stress path in $\tau-\sigma'_v$ plane</td>
<td>2</td>
<td>2</td>
<td>Chosen for a proper stress path ($\beta$ has no influence on resistance evolution for present cases)</td>
</tr>
<tr>
<td></td>
<td>$\chi$</td>
<td>Power controlling the damage of irrecoverable sensitivity and over-consolidation factor</td>
<td>2</td>
<td>2</td>
<td>Chosen for properly describing the degree of loading ($\chi$ has no influence on resistance evolution for present cases)</td>
</tr>
<tr>
<td>Consolidation</td>
<td>$T_{50}$</td>
<td>Dimensionless time factor for 50% of the consolidation settlement to occur</td>
<td>0.043</td>
<td>0.043</td>
<td>Obtained from a finite-element analysis of a rectangular mudmat (Feng and Gourvenec, 2015)</td>
</tr>
<tr>
<td></td>
<td>$m$</td>
<td>Constant</td>
<td>1.05</td>
<td>1.05</td>
<td>Obtained from a finite-element analysis of a rectangular mudmat (Feng and Gourvenec, 2015)</td>
</tr>
</tbody>
</table>
CHAPTER 6  PREDICTION OF SETTLEMENT

6.1  Introduction

Chapters 4 and 5 have demonstrated the importance of accurately predicting the settlement of sliding foundations. The berm formation is directly linked to settlement, and in turn governs the peak resistance generated upon sliding. If a sliding foundation is to be designed such that it slides freely throughout its design life without overstressing connectors, it is therefore critical that a method is developed that can predict the whole-life settlements accurately.

The evolution of whole-life settlement after set-up is illustrated in Figure 6.1(a) and (b) for centrifuge model tests on kaoling clay (C1) and carbonate silt (S1), (these and the remaining tests are presented in detail in Chapter 3). The figure shows evolution of the shear, consolidation and total settlement with number of cycles of sliding. Note that the settlement happening during set-up is not discussed here, and it is assumed to be fully finished prior to sliding. In the current work, w is the total settlement; B is the breadth of the sliding foundation; N is the number of operational cycles; and Δwp and Δwcon are the incremental shear and consolidation settlement, respectively. Periodic shear settlement is induced by subsoil failure under a combination of vertical and horizontal loads, and potentially (modest) overturning moment associated with lateral loading above the base of the foundation (Zhou et al., 2015). Periodic consolidation settlement, sometimes referred to as ‘shakedown’, is caused by the periodic generation and dissipation of excess pore pressure induced by surficial shearing of soil at the foundation-soil interface (Deeks et al., 2014a, Cocjin et al., 2017). In general, both shear and consolidation settlement rates are observed to decrease with increasing number of cycles, which is due to the compaction and hardening of the soil. However, the settlement rate and the relative contribution of the accumulated shear and consolidation settlements for test C1 and S1 are quite different – in spite of the tests being identical besides the soil type, which highlights that this strongly influences foundation performance. Further discussion of the experimentally observed whole-life shear and consolidation settlement, in both soil types and under various operational conditions, has been provided in Chapter 3.

This chapter presents a methodology to predict the whole-life evolution of settlement for sliding foundations. The soil is modelled as a 1D column of elements, with simple approximations of the shear stress profile then used to simulate excess pore pressure generation
due to each sliding event. A model for the shear settlement rates was derived from finite element simulations, while dissipation solutions are proposed to approximate the consolidation settlements that occur as a result of operating cycles. The total settlement is then the sum of the shear and consolidation settlement components. The model proposed has been validated against data from the eight centrifuge model tests reported in Chapter 3, and also compared to predictions using the model proposed by Cocjin et al. (2017).

6.2 Theoretical derivation

6.2.1 Overview

The soil beneath the sliding foundation is idealized as a simple 1D column of elements subjected to a vertical stress and shear stress, following the approaches of Cocjin et al. (2017) and Corti et al. (2017). The model is predicated on the assumption that the maximum sliding distance is relatively small relative to the overall foundation length (limited to 0.25L in the test campaigns reported in this thesis) such that spatial variations in stress can be ignored. The soil model in Chapter 5 is further extended by incorporating the stress-dependent parameters $\lambda^*$ and $\kappa^*$ (where $\lambda^*$ and $\kappa^*$ respectively represent the slopes of the curved ‘NCL’ and ‘unloading-reloading line’), which capture the effect of the non-linearity observed in the behaviour of the soil when subjected to 1D compression at low stress levels, as seen in Figure 3.7 of Chapter 3.

The shear and consolidation settlements are respectively calculated based on the distributions of strength and void ratio, where:

- The periodic shear stress generated at the interface between the sliding foundation and the underlying soil is assumed to propagate in a diminishing manner through the column of soil elements, resulting in the generation of excess pore pressure, which in turn results in consolidation settlements throughout the column of elements during operation or shutdown periods.

- The shear settlement is predicted under the combined loading caused by sliding foundation self-weight and flowline expansion or contraction using the evolving in-situ strength profile, which is simplified to a linear function over the depth of a typical combined loading induced failure mechanism. This is achieved using a family of finite element solutions for a range of typical non-dimensional undrained strength profiles.
6.2.2 Derivation of consolidation settlement

6.2.2.1 Distributions of vertical and shear stress

Unlike the surface element analysed in Chapter 5, for most deepwater offshore soil profiles the undrained strength of the deeper elements is unlikely to be fully mobilised by the applied shear stress. This results in reduced excess pore pressure generation with depth, in turn causing reduced consolidation settlements in deeper soil elements. To approximate this, the vertical and shear stress profiles are defined based on the elastic solution beneath the centre of a uniformly loaded rectangular area in a semi-infinite mass derived by Poulos and Davis (1974), the appropriateness of which was demonstrated by Corti et al. (2017) via a comparison with finite element simulations:

\[ I_\sigma = \frac{2}{\pi} \left[ \frac{b}{r_2} \left( 1 + \frac{z^2}{2r_2^2} \right) \arctan \left( \frac{l}{r_2} \right) + \frac{l}{r_1} \left( 1 + \frac{z^2}{2r_1^2} \right) \arctan \left( \frac{b}{r_1} \right) + \frac{blz^2}{2r_1^2} \left( \frac{1}{r_2^2} + \frac{1}{r_1^2} \right) \right] \] (6.1)

\[ I_\tau = \frac{2}{\pi} \left[ \arctan \left( \frac{b}{z} \right) - \frac{z}{r_1} \arctan \left( \frac{b}{r_1} \right) + \frac{b}{z} \left( \frac{b^2 - b^2 + l^2}{r_1^2} \right) \right] \] (6.2)

where \( I_\sigma \) and \( I_\tau \) are respectively the vertical and shear stress distribution factors; \( l = 0.5L \) and \( b = 0.5B \) with \( L > B \); \( L \) is the length of mudmat; and \( z \) represents the target depth. The parameters \( r_1, r_2, r_3 \) are given as:

\[ r_1 = \left( l^2 + z^2 \right)^{0.5} \]
\[ r_2 = \left( b^2 + z^2 \right)^{0.5} \]
\[ r_3 = \left( l^2 + b^2 + z^2 \right)^{0.5} \] (6.3)

In combination with the self-weight of soil, the vertical stress applied to each element in the column of soil is therefore:

\[ \sigma_{v,eqm} = \sigma_{v,op} I_\sigma + \gamma' z \] (6.4)

where \( \gamma' \) is the submerged unit weight of soil and \( \sigma_{v,op} \) is the operative stress applied to base plate which is calculated by:

---

1 An exception to this may be profiles comprising a surficial crust, which has stronger strength to the underlying soil; or profiles otherwise comprising interbedded weak layers.
\[
\sigma_{v,op} = \frac{V_{eff}}{BL}
\]  

where \(V_{eff}\) is the effective weight applied on the base plate, which is expected to be smaller than the applied vertical load since the skis carry part of the load after the foundation starts to embed into the soil.

Similarly the profile of shear stress is approximated as:

\[
\tau = \tau_{op} I_{\tau}
\]  

where \(\tau_{op}\) is the operative stress generated by the sliding foundation at the interface, which can be computed as:

\[
\tau_{op} = \min\left(\delta \frac{\delta}{\delta_p}, 1\right) \min\left(\frac{s_s}{I_{\tau}}\right)
\]

This conservatively assumes that the mudmat-soil interface is fully rough, and as such the shear stress developed at the interface is limited by the undrained shear strength of the underlying soils (Cocjin et al., 2017).

6.2.2.2 NCL and RSL

The normal compression line (NCL) is conventionally assumed to be a straight line in \( e - \ln \sigma_v' \) plane, and is determined in the laboratory via an oedometer test under stress levels of over 50kPa (ASTM, 1999). However, the stresses in the surficial soil relevant to sliding foundations are typically far lower than this threshold. At such low stress levels, the NCL is thought to be non-linear, and in this study is idealised using a bilinear function with larger slope at low stress levels (Nagaraj et al., 1998, Sun et al., 2016). The bilinear NCL can be broadly separated into two phases that are divided by the liquid limit (LL). The slope of the second phase of NCL in \( e - \ln \sigma_v' \) plane is defined as \( \lambda \), which corresponds to the parameter derived from the conventional high-stress 1D compression test. The corresponding swelling index is \( \kappa \). The void ratio intercept of the back-extrapolation of the second-phase line at \( \sigma_v' = 1 \text{kPa} \) is defined as \( e_N \). The larger NCL is observed at very low stress level and is referred to as \( \lambda_{LS} \).

In order to avoid the discontinuity at the inflection point of the bilinear curve, the curve is smoothed around the inflection point using an ‘arctan’ formula. The curved NCL is illustrated
in schematic in Figure 6.2. The relationship between void ratio \( e \) and the vertical effective stress \( \sigma'_v \) can be expressed as:

\[
e = e^*_N - \lambda^* \ln(\sigma'_{v,NCL})
\]

(6.8)

where \( \lambda^* \) is the slope of NCL (changing with stress level), and \( e^*_N \) is the void ratio intercept of the back-extrapolation of any point on the NCL at \( \sigma'_v = 1 \)kPa. \((\sigma'_{v,NCL})\) is the effective stress on the NCL, which can be expressed through \( \sigma'_v \) and \( OCR \) as:

\[
(\sigma'_{v,NCL}) = \sigma'_{v,OCR} \lambda^* \]

(6.9)

\( \lambda^* \) is written as a function of \((\sigma'_{v,NCL})\) as follows:

\[
\lambda^* = \lambda_{LS} - (\lambda_{LS} - \lambda) \left\{ \frac{1}{\pi} \arctan \left[ \frac{1}{C} \ln \left( \frac{\sigma'_{v,NCL}}{\sigma^*_{LL}} \right) \right] + \frac{1}{2} \right\}
\]

(6.10)

where \( \sigma^*_{LL} \) refers to the effective stress at the inflection point. The constant \( C \) controls the curvature around this inflection point and can be best fitted to data such as that provided in Figure 3.7 of Chapter 3. The parameters \( e^*_N \) and \( \kappa^* \) are given by:

\[
e^*_N = e^*_{LS} + \frac{\lambda^*}{\lambda} \left( e^*_N - e^*_{LS} \right) - (\lambda_{LS} - \lambda) \frac{C}{2\pi} \ln \left[ C^2 \ln^2 \left( \frac{\sigma'_{v,NCL}}{\sigma^*_{LL}} \right) + 1 \right]
\]

(6.11)

\[
\kappa^* = \frac{\lambda^*}{\lambda}
\]

(6.12)

where \( e^*_{LS} \) is the void ratio at the inflection point. Assuming that at such low stress levels the remoulded strength line (RSL) is parallel to the NCL, the effective stress on the RSL can be derived from:

\[
(\sigma'_{v,RSL}) = (\sigma'_{v,NCL}) \left[ \frac{2}{M_0 S_i \left( \sigma'_{v,NC} \right)} \right]
\]

(6.13)

where \( M_0 \) is the slope of the failure envelope in \( \tau - \sigma'_v \) plane at high stress levels (normally > 50kPa), and \( \left( s_u / \sigma'_{v,NC} \right) \) and \( S_i \) are the in-situ strength ratio for normally consolidated soil and
the sensitivity as measured by a T-bar, respectively. Based on the given equations, if taking $\lambda_{LS} = \lambda$, the curved NCL and RSL would regress to a simple straight line.

6.2.2.3 Undrained shear strength

A simple model for the undrained shear strength at any degree of remoulding was proposed in Chapter 5. The formulation here remains largely the same, except that the remoulding distance $\delta_r$ is substituted for an equivalent remoulding distance $\delta_{r,eq}$ in order to reflect the partial mobilisation of shear stress at depths other than at the interface, giving:

$$s_u = \min \left[ \frac{s_{u,r}}{s_{u,p}} + \left(1 - \frac{s_{u,r}}{s_{u,p}}\right) \exp\left(-3 \frac{\delta_{r,eq}}{\delta_{95}}\right) \right] s_{u,p}$$

(6.14)

where $s_{u,r}$ is the remoulded strength achieved on RSL, $s_{u,p}$ is the achievable peak undrained shear strength, and $\delta_{95}$ is the shearing distance required for a 95% reduction in strength from the peak towards a residual value. Detailed derivations for $s_{u,r}$ and $s_{u,p}$ were given in the section of 5.4 in Chapter 5. The equivalent remoulding distance $\delta_{r,eq}$ is assumed to decrease with depth as a function of the stress ratio $\tau/s_u$, as follows:

$$\delta_{r,eq} = \sum_{i=0}^{\delta_{max}/\Delta\delta} \left( \frac{\tau}{s_u} \right)^{\zeta}$$

(6.15)

where $\tau$ is shear stress, $\Delta\delta$ is the incremental sliding distance of a calculation step, $\delta_p$ is the sliding distance required to mobilise the peak operative shear stress and $\delta_{max}$ is the maximum sliding distance of a slide. The power $\zeta$ controls the rate of accumulation of the equivalent remoulding distance around the shear band– with the higher $\zeta$ leading to reduced depth of impacted soil.

6.2.2.4 Consolidation settlement

The consolidation settlement is then simply taken the sum of the volume change in each element beneath (the centreline of) the sliding foundation. This is calculated from the change in void ratio of each element, which is defined as $\Delta e$. The generated excess pore pressure is determined by the loading level $(\tau/s_u)^{\theta}$ following Cocjin et al. (2017). The effective stress following generation of excess pore pressure, denoted as $\sigma'_{gen}$, is taken as:
\[ \sigma'_{gen} = \min \left[ \sigma'_v - \left( \sigma'_v - \sigma'_{v,cs} \right) \left( \frac{\tau}{s_u} \right)^\beta \right] \]  

(6.16)

where \( \sigma'_{v,cs} \) is the effective stress on the failure envelope in \( \tau - \sigma'_v \) plane, which changes along the failure envelope as calculated by:

\[ \sigma'_{v,cs} = \frac{s_u}{0.5 M \theta'_{v,cs}} \]  

(6.17)

The reduction in void ratio induced by pore pressure dissipation \( \Delta e \) is written as:

\[ \Delta e = \Delta e_{U=1} U \]  

(6.18)

where \( \Delta e_{U=1} \) is the reduction in void ratio when the excess pore pressure fully dissipates, as given by:

\[ \Delta e_{U=1} = \kappa^* \ln \left( \frac{\sigma_{v,op}}{\sigma'_{gen}} \right) \]  

(6.19)

The \( U \) in Eq.(5.32) is the degree of consolidation defined in terms of the normalised time factor, and is expressed as:

\[ U = 1 - \frac{1}{1 + \left( \frac{T}{T_{50}} \right)^m} \]  

(6.20)

where \( T_{50} \) is the dimensionless factor for 50% of the consolidation settlement to occur and \( m \) is a constant describing the shape of the dissipation response. \( T \) is the normalised time factor given by:

\[ T = \frac{c_{ref} t}{d^2} \]  

(6.21)

where \( t \) is the consolidation time, \( d \) is the drainage path length (which is taken as the length of the short edge \( B \) for a rectangular mudmat for it is the shortest drainage path, following Cocjin et al. (2017)) and \( c_{ref} \) is the representative coefficient of consolidation as defined by Cocjin et al. (2014).

Noted previously, the consolidation settlement can then be calculated by summing the volume change of each element within the calculation depth over the number of cycles, as follows:
The shear settlement rate \( \frac{d\omega_p}{d\delta} \) (where \( d\omega_p \) is the increment in shear settlement over the incremental sliding distance \( d\delta \)) has been determined via finite element analysis for a range of simple linear strength profiles and normalised vertical load, \( V/V_{ult} \) (where \( V_{ult} \) is the ultimate vertical bearing capacity), with a simple model then fit to the results of these simulations. The method proposed is predicated on the hypothesis that a simple linear fit to the strength profile of the near-surface soil is able to predict the shear settlement rate with sufficient accuracy for design purposes.

### 6.2.3.1 Numerical model

In order to acquire the shear settlement rate curves for a range of potential linear undrained strength profiles, a series of small strain finite element analyses were conducted in ABAQUS. The sliding direction is assumed to be parallel to the long dimension of the foundation, which for the experiments reported in this thesis is rectangular with an aspect ratio \((L/B)\) of 2. Taking account of symmetry, only half of the domain was modelled, as shown in Figure 6.3. For simplicity, the sliding foundation was modelled at prototype scale (half width of 2.5m, length of 10m) as a rectangular foundation without skis. The size of the constructed soil body was 10m×30m×15m in width, length and depth, with the sliding foundation placed at the soil surface along the symmetrical centerline. Due to the small embedment, the sliding foundation is regarded as a surface foundation. C3D8H elements were used to model the soil body and the sliding foundation was modelled as a rigid body with the reference point (RP) located at the interface, and a modest degree of mesh refinement was adopted in the expected failure zone. The Tresca model was used to model the soil, with Young’s modulus taken as a large value \((30,000\sigma_0)\) in order to isolate the settlement rate at failure, and Poisson’s ratio of the soil taken as 0.495 in order to simulate undrained conditions. The foundation-soil contact was modelled as rough with no breakaway, mimicking the rough undrained interface on the centrifuge model.
6.2.3.2 Vertical bearing capacity

The vertical bearing capacity factor \( N_c \) for various normalised strength gradients were calculated via FEA using the aforementioned model. The concept of an average dimensionless undrained shear strength gradient (Chatterjee et al., 2012, Stanier and White, 2014) was adopted because this allows the parameter space, over which the shear settlement rate is to be defined, to be bounded (assuming possible strength profiles range from uniform to linearly increasing with depth). The average dimensionless undrained shear strength gradient is thus:

\[
K_{\text{avg}} = kB / \left( s_{um} + 0.5kB \right)
\]

(6.23)

where \( k \) is the strength gradient expressed as \( \Delta s_u / \Delta z \). Simulations were performed over a range of \( 0 \leq K_{\text{avg}} < 2 \), covering the range of practical interest that may occur immediately beneath the foundation as a result of local strain-softening and hardening due to consolidation. The ultimate bearing capacity generated by the foundation under uniaxial vertical loading (ignoring any heave) can be expressed simply as:

\[
V_{ult} = N_c s_{um} BL
\]

(6.24)

A vertical displacement of 0.02\( B \) was applied at the reference point with rotation precluded in order to fully mobilise the vertical bearing capacity. The results of these simulations, as presented in Figure 6.4, illustrates that the bearing capacity factor, \( N_c \), is well fitted by the following expression:

\[
N_c = 5.63 + \frac{K_{\text{avg}}}{1 - 0.45K_{\text{avg}}}
\]

(6.25)

6.2.3.3 Shear settlement rate curve

In order to obtain the shear settlement rate for a given target vertical loading level, the vertical load was first imposed on the reference point of the numerical model and kept constant throughout the remainder of each simulation. The reference point was then moved horizontally by 0.01\( B \), whilst constraining the rotation about the long axis (in the Y-Z plane) and freeing the rotation in the X-Z plane, mimicking the experimental conditions. The corresponding shear settlement rate was then calculated as the ratio of the vertical to horizontal displacement.

Figure 6.5 presents the shear settlement rate plotted against the horizontal displacement for a range of varying mesh refinements. Where \( \delta \) is the cumulative sliding distance in a single slide. The simulated shear settlement rate is somewhat sensitive to the size of the elements close to
the interface, due to the shallow failure zone developed under the relatively low loads relevant to this problem. Therefore the depth within 0.01L was refined by dividing the depth into 2 to 5 elements with keeping the bias ratio as 2 and the surface element finer. Figure 6.5 also compares the results of various meshing strategies for simulating the uniform soil under a typical loading level of 0.4V_{ult}, which shows that refining the mesh around the interface modestly increases the steady shear settlement rate. The set of analysis with 5 elements within 0.01L depth was chosen for all subsequent simulations since the result was largely convergent with the mesh with the next smallest surface element.

The shear settlement rate is initially higher during the mobilisation phase (Phase I), but rapidly falls to a steady state value with increasing horizontal displacement as failure occurs (Phase II), which appears fairly independent of the mesh refinement adopted. This research only focuses on the failure induced shear settlement (Phase II) which is regarded as the main component contributing to long-term foundation shear settlement (Deeks et al., 2014a, Cocjin et al., 2017). For a typical sliding distance (2.5m) given by Cocjin et al. (2017), the difference in shear settlement calculated by the presented model with and without considering the mobilisation phase is only 0.9% although the magnitude is somewhat influenced by Young’s modulus.

The simulated shear settlement rates of various $K_{avg}$ under various loading levels are shown in Figure 6.6. For each $K_{avg}$, the shear settlement rates, within the range of $V/V_{ult}$ of 0.1 to 0.7 (the suggested range of practical interest) were simulated for an increment of 0.05. The FE results were fitted using Eq. (6.26) which was adapted from the bilinear arctan equation:

$$\frac{\Delta w}{\Delta \delta} = \left\{ \frac{k_i + k_2}{\pi} \left( x^* + x_0 \right) - \frac{k_i - k_2}{\pi} \left[ x^* \arctan \frac{x^*}{C_1} - \frac{C_1}{2} \log \left( x^* + C_1^2 \right) \right] + const \right\}^2$$

(6.26)

where $x^*$ is the relative distance to the turning point $x_0$, which is defined as:

$$x^* = \frac{V}{V_{ult}} - x_0$$

(6.27)

$C_1$ is the parameter of controlling the curvature of the shear settlement rate curve, and is taken as 0.03 for the best fit in this instance. The constant referred to as $const$ is invariant to the value of $V/V_{ult}$, and is expressed as:
\[ const = \frac{k_1 - k_2}{\pi} \left[ x_0 \arctan \frac{x_0}{C_i} - \frac{C_1}{2} \log \left( \frac{x_0^2 + C_i^2}{x_0^2} \right) \right] \]  

(6.28)

where \( k_1 \) and \( k_2 \) control the slopes of the shear settlement rate curve at low and high loading levels, written as:

\[ k_1 = -0.025K_{avg} + 0.048 \]  

(6.29)

\[ k_2 = -0.14K_{avg} + 1.56 \]  

(6.30)

The value of \( x_0 \) defines the turning point dividing the low and high loading level regions of the curve fit, and can be approximated as:

\[ x_0 = \frac{K_{avg}}{100(2 - K_{avg})} + 0.5 \]  

(6.31)

The model fits are compared with the FE simulation results in Figure 6.7. The shear settlement rate increases with loading level and decreases with increasing dimensionless undrained shear strength gradient \( K_{avg} \) (see Figure 6.6). The residual error for the majority of the data points is within 10% of the value observed in the simulations. Note that these functions are well-suited to rapid automated computation of shear settlement rate, whereas full FEA of each specific loading case cannot be conducted within a reasonable timescale on demand.

6.2.3.4 Failure depth

It is thought that the failure depth developed during combined loading can be estimated from the corresponding shear settlement rate, which is a concept that has been utilised in the current study. Green (1954) suggested the failure mechanism for the indentation of a strip into a material that demonstrated plastic failure when subjected to \( H-V \) combined loads. According to the given failure mechanism, the failure depth that can be estimated by

\[ z_f \approx \frac{\Delta \nu}{\Delta \delta \times L} \]

Assessment of a simulation with uniaxial vertical loading on a strip footing showed that the extent of the displacement field with resultant displacement magnitude \( \delta < 0.25\delta_{max} \) (where \( \delta_{max} \) is the maximum resultant displacement within the failure mechanism, which occurs immediately beneath the centreline of the foundation for uniaxial vertical loading) agrees remarkably well with the geometry of the classical exact solution given by Prandtl (1920) (Figure 6.8(a)). To investigate this, numerical modelling has been undertaken for a rectangular mudmat with \( V/V_{ult} = 0.6 \) in Figure 6.8(b), with the displacement magnitude contours delimited
to $\delta < 0.25\delta_{\text{max}}$ – this agrees well with the finding of Green (1954), which is illustrated by the red line.

To incorporate this into the current model, the estimated failure depth for $K_{\text{avg}}$ ranging from 0 to 1.7 and $V_{\text{ult}}$ ranging from 0.1 to 0.7 have been determined from FE analysis, and are compared with the Green (1954) solution in Figure 6.9. The analysis suggests that for practical purposes, $z_f = \Delta w_p / \Delta \delta \times L$ may be used as an initial estimate for the failure mechanism depth for a rectangular foundation under $H-V$ combined loads.

6.2.3.5 Shear settlement rate

The process to derive shear settlement rate is illustrated in Figure 6.10, and may be used for any possible strength profile and for vertical loads (V) less than 0.7$V_{\text{ult}}$. This assumes that the soil beneath the assumed failure depth does not have a significant influence on the result, and that the use of a linear strength profile (fitted to the typically mildly non-linear strength in the failure zone) can provide a reasonable estimate of the response – the latter assumption being needed to make use of the shear settlement rate curves developed via numerical modelling. In undertaking this analysis, it is considered important to match the fitting depth ($z_{\text{fit}}$) and the failure depth ($z_f$) because:

- If the depth over which the linearisation is performed is too small, then the influence of the underlying soil could be erroneously ignored.
- On the other hand, if the depth over which the linearisation is performed is too large, then the soil at depth – which is far from the influence of the combined loading failure mechanism – will have an unreasonably large influence on the calculations.

Consequently, an iterative approach has been developed to define the appropriate depth over which to linearise the soil strength profile, which is believed to be adequate (for practical purposes) due to the relatively shallow failure mechanisms mobilised when sliding foundations are subjected to combined loading. A flow chart illustrating this process is shown in Figure 6.11, with a schematic presented in Figure 6.12 that outlines this iterative process, which encompasses the following steps:

1. The failure depth $z_{\text{fit}}$ is firstly assumed as a small value ($z_{\text{fit}} = 1 \times 10^{-6} B$ in this instance).
2. The calculated strength profile within the assumed failure depth is fitted via least squares best fit.
3. The shear settlement rate \( \frac{d\delta w_p}{d\delta} \) is calculated from the \( V/V_{ult} \) and the \( K_{avg} \) derived from the fitted linear profile of undrained shear strength.

4. A new estimate for the failure depth is calculated as \( z_f = \Delta w_p / \Delta \delta \times L. \)

5. The error between the assumed failure depth and the resulting failure depth from the Green (1954) approximation is calculated (denoted as \( er \)).

6. The calculated error magnitude is then compared to the target error \( er_t \) (\( 2 \times 10^{-7} B \) was used in the present calculation, which showed minor influence on predicted results), and if it is smaller than the target error, the linear fit is accepted. The accumulated error over the whole-life is therefore less than \( 2N\delta_{max}(er/L) \).

7. If the calculated error is larger than the target error a new approximation of the failure depth is derived as follows in steps 8, 9 and 10:

8. If the ratio of the current and previous errors is greater than 0, go to step 10.

9. Otherwise if the ratio of the current and previous errors is less than 0, divide the assumed incremental depth \( \Delta z_{fit} \) by 2.

10. Assume the updated failure depth \( z_{fit(i+1)} = z_{fit(i)} + er/(|er|/|\Delta z_{fit(i+1)}|) \). Where the subscript \( i \) and \( i+1 \) are the present and next iteration number.

Using this approach, which has been fully automated and usually converges in around 5 iterations, the shear settlement is accumulated incrementally as follows:

\[
W_p = \sum_{i=0.5}^{N/0.5} \sum_{j=1}^{\Delta \delta} \left( \frac{\Delta w_p}{\Delta \delta} \right) \Delta \delta_j \tag{6.32}
\]

6.3 Framework parameters

6.3.1 Input parameters

This above settlement model has been validated against the eight centrifuge model tests conducted on UWA Kaolin clay (C1-C3) and calcareous silt (S1-S5), which are documented in earlier chapters. The input parameters are listed in Table 6.1. The operational conditions of each test has been documented in Chapters 3 and 5.

The reference consolidation coefficient for clay, according to Cocjin et al. (2014), is expressed as:

\[
c_{ref} = 2.7 \left[ 0.3 + 0.16 (\sigma'_{NCL})^{0.47} \right] \tag{6.33a}
\]

while for silt it is given in Chapter 3 and Chapter 5 as:

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\[ c_{ref} = 0.33 \left[ (\sigma'_v)_{NCL} \right]^{0.47} \]  

(6.33b)

The parameters \( \beta \) and \( \chi \) (which influence the consolidation settlement and strength distribution) and \( h_s \) and \( \zeta \) (which influence shear settlement) are relatively stable, which suggested that initial values could be provided here by calibrating from existing tests. The following is noted:

- The parameter \( \beta \) controls the shape of the stress path in the \( \tau - \sigma'_v \) plane which can be roughly measured through triaxial test or direct simple shear (DSS) test as suggested by Andersen (2015). Figure 6.13 shows the simulated normalised stress paths assuming \( \beta \) in the range of 1~4. \( \beta = 1.6 \) is suggested here, which is close to the value \( (\beta = 2) \) used in Cocjin et al. (2017).

- The parameter \( \chi \) in Eq.(5.20) in Chapter 5 controls the degradations of the irrecoverable component of sensitivity and the over-consolidation factor, which in essence reflects the rate of permanent damage accumulation in subsoil. \( \chi = 2.5 \) is suggested here, which is the same with that used in Cocjin et al. (2017).

- The parameter \( h_s \) is taken to be the size of the top element used in the framework Figure 6.10, and is assumed to represent the thickness of the shear band formed at the foundation-soil interface. Muir Wood (2012) reported that the shear band thickness may be approximated as \( 10-20D_{50} \), where \( D_{50} \) is the mean grain diameter of the soils - which was 1 \( \mu \)m for the calcareous silt (Chow et al., 2019) and 0.6 \( \mu \)m for the kaolin clay (Doan and Lehane, 2020). The thickness of shear band can be observed through the experimental technic suggested by Martinez and Stutz (2018).

- The parameter \( \zeta \) controls degree of remoulding induced by the post-peak continuous surface sliding, which influences the elements that the peak strength is fully or nearly mobilised. It is considered that there is minor influence on those elements where the imposed shear stress is far less than its peak undrained shear strength, therefore, this surface sliding induced remoulding only happens within and around shear band, and a large value of \( \zeta \) is appropriate. \( \zeta = 50 \) is suggested here.

### 6.3.2 Summary

The framework comprises of 20 input parameters, excluding those related to foundation geometry and loading condition. These parameters can be divided as three types:

- Parameters that fall within stable ranges and are largely independent of soil type, including \( \gamma' \). Of these – \( 
\gamma' \) can be taken as 5kN/m\(^3\) if measurements are not available;
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while \( T_{st} \) and \( m \) are recommended based on finite-element analysis as 0.0043 and 1.05 respectively; \( \beta, \chi, h_s \) and \( \zeta \) have been suggested as per above.

- Parameters that are soil specific, including \( \left( \frac{s_u}{\sigma_v'} \right)_{NC}, S_t, F_{st}, M_0, \sigma_{kr}, c_{ref}, \lambda, \lambda^{**}, \kappa, \varepsilon_{el} \).

  Of these – \( \left( \frac{s_u}{\sigma_v'} \right)_{NC}, S_t, F_{st}, M_0, \sigma_{kr} \) may be measured using shallow penetrometers such as the hemiball, toroid and ring penetrometers (Yan, 2013, Randolph et al., 2018, Schneider et al., 2020a, Schneider et al., 2020b) which are designed to measure the strength properties of soil at shallow; \( c_{ref} \) can be measured using a piezofoundation; and \( \lambda, \lambda^{**}, \kappa, \varepsilon_{el} \) can be determined through laboratory tests such as the odometer test. An estimation based on large data sets is also acceptable for practical use.

- Parameters that are considered to have only minor influence on the prediction of sliding resistance, including \( e_N, \delta_p, \delta_{\beta} \). The values of clay listed in Table 6.1 can be taken as a universal use.

The inputs used in the framework are listed in Table 6.1, while sensitivity studies on the selection of \( \beta, \chi, h_s \) and \( \zeta \) are provided in the next section.

6.4 Prediction and discussion

Based on the methods outlined above, the predicted consolidation (\( w_{con} \)), shear (\( w_p \)) and total settlement (\( w_{tot}=w_{con}+w_p \)) is compared with measurements from centrifuge testing in Figure 6.14 - Figure 6.21. It is clear that both consolidation and shear settlement are captured reasonably well. The measured total settlement is also plotted against the respective predictions in Figure 6.22, showing that the residual errors (during each test) are largely within 20%, which is expected to be sufficient for practical purposes. The results suggest that the response during early cycling for both consolidation and shear settlement is somewhat under-estimated, which may be due to:

- The effect of over-consolidation on consolidation coefficient was ignored leading to the error on consolidation settlement; and
- The simplification noted in Section 6.2.3.3 (and evident on Figure 6.5) whereby higher immediate settlement rate was ignored leading to the error on shear settlement.

The following sections provide further discussion on the approach used and observations.
6.4.1 Sensitivity study of key model parameters

The optimal values of parameters $\beta$, $\chi$, $h$, and $\zeta$ may change somewhat for different soils, and therefore a sensitivity study was performed to understand the impact of varying these parameters. A range of $\pm 40\%$ around the initially suggested value was respectively applied to each parameter with keeping the others as the suggested to explore the influences on the total settlement. Figure 6.23 compares the average normalised change in the predicted total settlement against the normalised change in each of the above parameters. The subscript ‘s’ stands for the suggested value reported in Table 6.1, and the analysis was undertaken by changing only one parameters at a time. Within the range considered for analysis, the parameters $\chi$, $h$, and $\zeta$ have only minor influence (<10%), while $\beta$ shows a larger influence as its value approaches to or even smaller than 1 (where the stress path in $\tau$-$\sigma_v$ plane tends towards a linear form).

6.4.2 Comparisons with NCL and RSL

The maximum deformation of each element influencing the consolidation settlement is determined by the spacing between the NCL and the RSL, which in turn is influenced by their respective slopes. Figure 6.24 illustrates that properly taking account of the higher slope of the NCL is important if the model is to accurately capture the consolidation settlement of a mobile foundation on Kaolin clay, for which the normal compression response is strongly bilinear. In contrast, the silt can be well captured using a conventional linear model (as is generated when $\lambda_{LS} = \lambda$). The R-squared values for the adopted NCL fits are 0.996 for clay and 0.977 for the silt, respectively. According to the position of the NCL, the position of the RSL can be derived via Eq. (6.13).

Tests C1 (on clay) and S1 (on silt) are shown alongside the NCL and the RSL in Figure 6.25 and may be used to demonstrate (simulated) stress paths of select sub-elements in the 1D soil profile. Two typical positions are shown in the figure, namely: (i) at the foundation-soil interface; and (ii) at a depth of $0.1B$. The void ratio of each sub-element decreases due to periodic remoulding and reconsolidation, but with a reducing rate at greater depths - reflecting the diminishing propagation of shear stress with depth. For the deeper elements, the accumulation of excess pore pressure is somewhat attenuated because it takes a number of cycles for the soil above to harden and generate greater shear stresses at depth. Correspondingly, in terms of the strength evolution, the top element softens to the remoulded
strength within the first slide and exhibits hardening thereafter, while for deeper elements the strength softens over several cycles (before later beginning to harden).

6.4.3 Comparisons of soil strength profiles

Both the in-situ and final (end of testing) strength profiles were measured in the free field and within the model test footprint using a T-bar, and are compared with predictions from the proposed analysis framework in Figure 6.26 - Figure 6.30. C1-C3 are correspondingly plotted with their counterparts S1-S3 for comparison. Where, the measured final strength profile of the silt cases is from the location close to the shutdown side for consistency with the comparison reported in Cocjin (2016). The published T-bar derived strength profiles were all interpreted using the methodology given by White et al. (2010). Note that the calculated undrained shear strength needs to be (slightly) moderated to obtain the T-bar measured strength according to Zhou and Randolph (2009b), written as:

\[
s_u = \left[\frac{s_{u,r}}{s_{u,p}} + \left(1 - \frac{s_{u,r}}{s_{u,p}}\right) \exp\left(-3\frac{0.25}{N_{t,95}}\right)\right]s_{u,p}
\]  

(6.34)

where \(N_{t,95}\) is the number of cycles to attain 95% degradation from the peak undrained shear strength to the residual. After cycles of remoulding and reconsolidation, the near surface soil beneath the foundation is strengthened, which appears well captured by the framework.

6.4.4 Changing failure envelopes

The evolution of shear settlement is directly related to the size of the \(H-V\) failure envelope. The failure envelope at specific cycles (0\(^{th}\), 10\(^{th}\), 20\(^{th}\) and 40\(^{th}\)) has been explored using the ‘swipe’ test concept introduced by Tan (1990), using the aforementioned FE model and framework generated strength profile. The curved strength profile (that evident before simplification to a linear profile) calculated by the extended framework was incorporated into the soil elements of the FE model. In order to obtain the \(H-V\) failure envelope, a vertical displacement of 0.002\(B\) was firstly imposed at the reference point of the foundation (with its rotation restrained) in order to fully mobilise the vertical bearing capacity. Then, whilst maintaining the vertical position, the mudmat was moved horizontally by 0.01\(B\) with rotation in the X-Z plane allowed. In conjunction with the result of a horizontal sliding simulation with zero vertical load (i.e. pure sliding), the failure enveloped could then be determined.
The resulting failure envelopes for typical cycles in test C1 and S1 are shown in Figure 6.31, as examples of the numerical modelling process. In clay, the failure envelope evolves slowly from the initial failure envelope (Cyc 0) – and after the 10th cycles is still close to the initial envelope in shape and size. With increasing cycles, the failure envelope then quickly expands due to the hardening induced by the intervening periods of reconsolidation, with the envelope after 40 cycles being (roughly) 50% larger than the original envelope. In contrast, the envelopes associated with the silt tests reduce in size significantly with number of cycles, decreasing to less than half the initial state after 10 cycles. While hardening does occur, after 40 cycles the envelope remains well within the initial envelope. This slower expansion in failure envelope of test S1 helps to explain why the predicted shear settlement rate decreases more slowly than its clay counterpart test C1.

The failure envelopes in Figure 6.31 are further normalised by the ultimate capacities of $H$ and $V$ in Figure 6.32. Subtle differences are observed in the normalised envelope shapes, illustrating that it is difficult to predict the evolution of shear settlement based on any one specific normalised failure envelope. Of particular note, the operative vertical loads are relatively low, and coincide with the ‘flat’ part of the envelope – explaining why the shear settlement is small relative to the sliding distance.

### 6.4.5 Comparisons with the previous framework

The reported extended framework demonstrates a straightforward methodology to predict the settlement of mobile foundation. Compared with the framework released by Cocjin et al. (2017), the present framework shows the improvements:

- A measured $\lambda/\kappa$ of the kaolin clay given by Stewart (1992) can be directly used, since the stress-dependent $\lambda$ and $\kappa$ have been considered.
- The mechanism of inducing shear settlement is analysed through FEA giving a more robust evidence.
- The previous back-calculated parameters obtained by performing model tests have been removed or replaced by more insensitive suggested parameters.

### 6.5 Conclusions

This Chapter presents a framework that builds on the one in Chapter 5, but incorporates the soil beneath the mudmat-soil interface to predict whole-life settlement of mobile sliding
foundations. The framework was validated against the results from eight centrifuge model tests (reported in Chapter 3), with highlights of the proposed framework including:

- Stress-dependent $\lambda^*$ and $\kappa^*$ have been incorporated into the soil model to better capture the one-dimensional consolidation properties at low stress level, which improves resulting consolidation settlement predictions for the clay tests.
- Deeper elements appear to remould more slowly than surface elements, due to attenuated propagation of surficial shear stresses, with softening during early cycles and hardening thereafter.
- The $H-V$ failure envelope, which is related to the shear settlement rate, typically shrinks during early cycles (due to remoulding) and then later expands (due to dissipation-induced hardening). The shape of the envelope also changes subtly, impacting the estimated settlement.
- The shear settlement rate is derived – using a model developed via FE analyses – from a linear strength profile fitted to the curved strength profiles determined by the proposed model. This methodology provides a means of simulating the whole-life settlement of a sliding foundation with minimal fitted parameters.
- A comparison between the framework derived predictions and the measurements from the experiments is generally good, with the vast majority of the predictions falling within 20% of the measurements – which is considered adequate for practical applications.
Figure 6.1 The evolution of normalised shear and consolidation settlement in (a) C1 on the kaolin clay and (b) S1 on the calcareous silt.

Figure 6.2 Schematic of bilinear NCL
Figure 6.3 Finite element mesh adopted in the finite element analyses (FEA).

Figure 6.4 Vertical bearing capacity factor of finite element analysis result and the fitted plotted against normalised strength gradient.
Figure 6.5 Shear settlement rate evolution plotted against the sliding distance for various mesh refinement strategies.
Figure 6.6 Shear settlement rate curves of various linearly distributed strength profiles for surface footing with \(L/B=2\).

Figure 6.7 Fitted versus the finite element simulated shear settlement rates.
Figure 6.8 Failure mechanism of strip footing under pure vertical (a) and the rectangular mudmat foundation under combined $H$-$V$ loading (b).

Figure 6.9 Evaluated failure depth under combined loads plotted against the finite element results.
Figure 6.10 Flow chart of the developed methodology for calculating shear settlement
Set initial fitting depth $z_{fit(0)}$, increment $\Delta z_{fit(0)}$, target error $e_{r(t)}$ and initial error $e_{r(0)}=e_{r(t)}$

1. Linearise the strength profile in the fitting zone
2. Calculate the shear settlement rate ($dw_p/d\delta$) according to $V_{eff}/V_{ult}$ and $K_{avg}$
3. Calculate the new failure depth, $z_{f(i)}=\Delta w_p/\Delta \delta^*L$
4. Calculate the error, $e_{r(i)}=z_{f(i)}-z_{fit(i)}$

Update the fitting depth,

5. $z_{fit(i+1)}=z_{fit(i)}+e_{r(i)}/|e_{r(i)}|\Delta z_{fit(i+1)}$
6. $\Delta z_{fit(i+1)}=\Delta z_{fit(i)}$
7. $e_{r(i)}/e_{r(i-1)}>0$
8. $e_{r(i)}<e_{r(t)}$
9. $\Delta z_{fit(i+1)}=\Delta z_{fit(i)}/2$
10. Reduce the increment, $\Delta z_{fit(i+1)}=\Delta z_{fit(i)}/2$

Yes

Shear settlement rate

Figure 6.11 Flow chart illustrating the process for ensuring failure zone

Figure 6.12 Schematic illustrating the process for ensuring failure zone.
Figure 6.13 Typical stress paths for various $\beta$ values in $\tau-\sigma'_v$ plane.

Figure 6.14 Measured whole-life consolidation (a), shear (b) and total (c) settlements of C1 versus the predictions calculated by the extended framework
Figure 6.15 Measured whole-life consolidation (a), shear (b) and total (c) settlements of C2 versus the predictions calculated by the extended framework.

Figure 6.16 Measured whole-life consolidation (a), shear (b) and total (c) settlements of C3 versus the predictions calculated by the extended framework.
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

Figure 6.17 Measured whole-life consolidation (a), shear (b) and total (c) settlements of S1 versus the predictions calculated by the extended framework

Figure 6.18 Measured whole-life consolidation (a), shear (b) and total (c) settlements of S2 versus the predictions calculated by the extended framework
Figure 6.19 Measured whole- consolidation (a), shear (b) and total (c) settlements of S3 versus the predictions calculated by the extended framework.

Figure 6.20 Measured whole-life consolidation (a), shear (b) and total (c) settlements of S4 versus the predictions calculated by the extended framework.
Figure 6.21 Measured whole-life consolidation (a), shear (b) and total (c) settlements of S5 versus the predictions calculated by the extended framework.

Figure 6.22 Normalised measured total settlement plotted against the predicted.
Figure 6.23 Average normalised change in the predicted total settlement of eight presented cases at the last sliding cycle plotted against the normalised change of the parameters $\beta$ (a), $\chi$ (b), $h_s$ (c) and $\zeta$ (d)
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

Figure 6.24 Measured compression data of clay and silt plotted versus the fitting NCL

Figure 6.25 Stress path of elements at $z=0$ and $z=0.1B$ for (a) C1 and (b) S1
Figure 6.26 Measured and predicted in-situ (on free field) and final (on footprint) strength profiles of C1 (a) and S1 (b)

Figure 6.27 Measured and predicted in-situ (on free field) and final (on footprint) strength profiles of C2 (a) and S2 (b)
Figure 6.28 Measured and predicted in-situ (on free field) and final (on footprint) strength profiles of C3 (a) and S3 (b)

Figure 6.29 Measured and predicted in-situ (on free field) and final (on footprint) strength profiles of S4
Figure 6.30 Measured and predicted in-situ (on free field) and final (on footprint) strength profiles of S5

Figure 6.31 Evolution of $H$-$V$ failure envelopes in (a) C1 and (b) S1
Figure 6.32 Normalised $H-V$ failure envelopes in (a) C1 and (b) S1
Table 6.1 Framework parameters

<table>
<thead>
<tr>
<th>Framework components</th>
<th>Parameter</th>
<th>Description</th>
<th>Clay</th>
<th>Silt</th>
<th>Source of selected values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boundary conditions</td>
<td>B</td>
<td>Foundation breadth (model scale)</td>
<td>50mm</td>
<td>50mm</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>L</td>
<td>Foundation length (model scale)</td>
<td>100mm</td>
<td>100mm</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>h</td>
<td>Foundation height (model scale)</td>
<td>5mm</td>
<td>5mm</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\theta_{mat}$</td>
<td>Base angle of ski</td>
<td>$30^\circ$</td>
<td>$30^\circ$</td>
<td>Foundation geometry reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\delta_{max}$</td>
<td>Maximum single sliding distance</td>
<td>L/4</td>
<td>L/4</td>
<td>Test parameter reported in Chapter 3</td>
</tr>
<tr>
<td>Initial condition</td>
<td>$\sigma_{sur}$</td>
<td>Surcharge</td>
<td>3.5kPa</td>
<td>5.0kPa</td>
<td>Estimated according to the thickness of the scraped soil reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\gamma'$</td>
<td>Submerged unit weight of soil</td>
<td>5.5kN/m$^3$</td>
<td>4.7kN/m$^3$</td>
<td>Calibrated from core samples reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$D_{50}$</td>
<td>Mean effective particle size</td>
<td>1$\mu$m</td>
<td>0.6$\mu$m</td>
<td>Given by Chow et al. (2019) and Doan and Lehane (2020)</td>
</tr>
<tr>
<td></td>
<td>$S_t$</td>
<td>T-bar measured sensitivity</td>
<td>2.5</td>
<td>4.2</td>
<td>Calibrated from episodically cyclic T-bar test reported in Chapter 5</td>
</tr>
<tr>
<td></td>
<td>$F_{st}$</td>
<td>Proportion of the irrecoverable sensitivity component</td>
<td>0</td>
<td>0.43</td>
<td>Calibrated from episodically cyclic T-bar test reported in Chapter 5</td>
</tr>
<tr>
<td></td>
<td>$(s_u / \sigma'_v)^{NC}$</td>
<td>T-bar measured strength ratio</td>
<td>0.16</td>
<td>0.42</td>
<td>Calibrated from episodically cyclic T-bar test reported in Chapter 5</td>
</tr>
<tr>
<td>Basic properties</td>
<td>$e_{nv}$</td>
<td>Intercept void ratio at $\sigma'_v = 1$ of the Normal Compression Line (NCL) in $e-\sigma'_v$ plane</td>
<td>2.45</td>
<td>3.30</td>
<td>Calibrated from core samples reported in Chapter 3 (the value for clay is from the the extension cord of the second-phase line of NCL)</td>
</tr>
<tr>
<td>Compression and swelling line</td>
<td>$\lambda$</td>
<td>Slope of NCL at high stress level</td>
<td>0.26</td>
<td>0.287</td>
<td>Calibrated from core samples reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\kappa$</td>
<td>Slope of swelling line at high stress level in $\epsilon - \ln \sigma'_v$ plane</td>
<td>0.056</td>
<td>0.036</td>
<td>Calculated according to the ratio of $\kappa / \lambda$ given by Stewart (1992) and Chow et al. (2019)</td>
</tr>
<tr>
<td></td>
<td>$\lambda_{**}$</td>
<td>Slope of NCL at low stress levels</td>
<td>0.65</td>
<td>0.287</td>
<td>Calibrated from core samples reported in Chapter 3</td>
</tr>
</tbody>
</table>
## Extended framework for predicting the behaviour of tolerably mobile subsea foundations

<table>
<thead>
<tr>
<th>Framework components</th>
<th>Parameter</th>
<th>Description</th>
<th>Clay</th>
<th>Silt</th>
<th>Source of selected values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drained failure envelope</td>
<td>$e'_{Ll}$</td>
<td>Void ratio at the inflection point of NCL</td>
<td>1.80</td>
<td>1.71</td>
<td>Calibrated from core samples reported in Chapter 3</td>
</tr>
<tr>
<td>Strength mobilisation and degradation</td>
<td>$M_0$</td>
<td>Strength factor of high stress level</td>
<td>0.92</td>
<td>1.62</td>
<td>Calibrated through triaxial test, here is given by Stewart (1992) and Chow et al. (2019)</td>
</tr>
<tr>
<td></td>
<td>$\sigma_r$</td>
<td>Parameter to control the shape of the failure envelope</td>
<td>0.16</td>
<td>0.4</td>
<td>Calibrated from triaxial test, here is calibrated through model tests C3 and S3 reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\delta_p$</td>
<td>Sliding distance for fully mobilising the peak strength</td>
<td>1mm</td>
<td>2mm</td>
<td>Observed from model tests C3 and S3 reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\delta_{95}$</td>
<td>Sliding distance for 95% remoulding the peak strength</td>
<td>2mm</td>
<td>10mm</td>
<td>Observed from model tests C3 and S3 reported in Chapter 3</td>
</tr>
<tr>
<td></td>
<td>$\beta$</td>
<td>Excess pore pressure parameter controlling the effective stress path in $\tau - \sigma'$ plane</td>
<td>1.6</td>
<td>1.6</td>
<td>Suggested according to Cocjin et al. (2017)</td>
</tr>
<tr>
<td></td>
<td>$\chi$</td>
<td>Power to control the degradation of irrecoverable sensitivity and over-consolidation factor in deeper soil</td>
<td>2.5</td>
<td>2.5</td>
<td>Suggested according to Cocjin et al. (2017)</td>
</tr>
<tr>
<td>Shear settlement parameters</td>
<td>$\zeta$</td>
<td>Controlling the rate of accumulating equivalent remoulding distance in soil around shear band</td>
<td>50</td>
<td>50</td>
<td>Suggested according to its definition</td>
</tr>
<tr>
<td></td>
<td>$h_s$</td>
<td>Thickness of shear band</td>
<td>$15D_{50}$</td>
<td>$15D_{50}$</td>
<td>Suggested according to Muir Wood (2012)</td>
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<tr>
<td>Consolidation parameters</td>
<td>$T_{50}$</td>
<td>Dimensionless time factor for 50% of the consolidation settlement to occur</td>
<td>0.043</td>
<td>0.043</td>
<td>Obtained from a finite-element analysis of a rectangular mudmat (Feng and Gourvenec, 2015)</td>
</tr>
<tr>
<td></td>
<td>$m$</td>
<td>Constant</td>
<td>1.05</td>
<td>1.05</td>
<td>Obtained from a finite-element analysis of a rectangular mudmat (Feng and Gourvenec, 2015)</td>
</tr>
</tbody>
</table>
CHAPTER 7  CONCLUSIONS

7.1 Introduction

This thesis improves the understanding of the behaviour of mobile foundation, and provides design methodologies to support their use in design for offshore projects.

The results from a series of experiments are presented in Chapter 3, which facilitate the understanding of mobile foundation response to periodic monotonic shearing, and provide a benchmark for the subsequent theoretical analysis. Whole-life berm resistance, sliding resistance and settlement are discussed in Chapter 4, 5 and 6 respectively. Each element extends the earlier framework (Cocjin et al., 2017) for predicting response of mobile foundations under periodic monotonic loading with intervening consolidation. The enhanced framework, which has been validated against the centrifuge results, is shown to capture the evolution of sliding resistance within and between cycles, while also providing reasonable predictions of consolidation and shear settlement. A brief summary of each research topic is schematically listed in Figure 7.1 and is further demonstrated below.

7.2 Centrifuge model tests

Five centrifuge model tests were carried out on a calcareous silt (recovered from the North-West Shelf) with different operational conditions (i.e. vertical load mobilisation, intervening consolidation time, reconsolidation time and sliding scenario). The results were compared to those performed on a kaolin clay reported in Cocjin et al. (2014). A number of observations are made:

- A significant degradation in resistance takes place during the first slide. The peak resistance is first mobilised within a few millimetres (at model scale), corresponding to the peak undrained shear strength of interface soil. With further shearing, the soil at the foundation-soil interface softens towards the fully remoulded undrained shear strength which is the undrained shear strength obtained when the soil is fully remoulded (the structure between grains is completely destroyed), resulting in a smaller residual sliding resistance than the peak resistance. The ratio of the peak (corresponding to peak undrained shear strength) to residual (corresponding to fully remoulded undrained shear strength) sliding resistance is close to the measured soil sensitivity.

- The residual resistance (or fully remoulded undrained shear strength) increases with sliding cycles, and the longer the intervening consolidation time, the greater the strength increases—
with the resistance eventually approaching the drained strength at the interface. At low vertical effective stress (relevant for mobile foundations), the soil may be expected to have a higher friction angle than that measured under high stresses. This effect has been included.

- Berms deposited at either end of the sliding footprint lead to peaks in sliding resistance. The magnitude of the berm resistance is related to the observed settlement and rotation of the foundation, and larger settlement leads to more soil accumulating in the dormant berms.

- Settlement can be divided into shear and consolidation settlement. The magnitude of consolidation settlement is related to soil compressibility, coefficient of consolidation, consolidation time and vertical load mobilisation. The evolution of shear settlement developed under periodic undrained surface sliding is affected by the magnitude of strength softening and regain, and the vertical load mobilisation.

- Tilting of the foundation is observed to be minimal, less than 2° over all the level of cycles applied. This reflects the design, with lateral loads applied close to the sliding surface.

- The location where the reconsolidation happens (during operation) does not appear to influence resistance and settlement.

### 7.3 Prediction of the whole-life sliding resistance

The proposed framework for predicting sliding resistance encapsulates two components: berm resistance at extremities of the sliding footprint, and base resistance of the foundation. The procedure for predicting berm resistance is described in Chapter 4, while the base resistance is addressed in Chapter 5. Key conclusions include:

- The resistance due to the (dormant) berms deposited at either end of the sliding footprint increases with number of cycles of loading with intervening consolidation, which can be attributed to the volume gain as the foundation ploughs in the soil, and strength gains due to periodic remobilisation and reconsolidation. The volume gain is related to the settlement of the foundation, and also reflects foundation tilting (albeit this is shown to be small in this study). Equations are provided to calculate the volumes of berms at two sides of the foundation – those in the direction of sliding. The strength gain of berms is also calculated based on the critical state theory. In order to better reflect a (higher) friction angle at low vertical stress, a curved failure envelope in $\tau-\sigma'$ plane was employed. The results show that the strength gain contributes a large proportion to the increase in berm resistance.

- A displacement based (rather than cycle-by-cycle based) methodology by which the sliding process within any single slide can be described, has been developed to simulate the sliding
resistance of a tolerably mobile foundation, which allows for the coupling of the berm component.

- A methodology has been developed to simulate the response of the interface soil to periodic remoulding and reconsolidation. The subloading surface and RSL (parallel to the NCL) were employed to anchor the peak and fully remoulded, undrained shear strengths, respectively. The proposed framework can realistically capture the hardening and softening process under cyclic loading, which has been validated through episodically cyclic T-bar tests performed in kaolin clay and calcareous silt. Again, a curved failure envelope in $\tau-\sigma'$, plane was adopted to better reflect the soil strength so that the sliding resistance under low stress range where mobile foundations usually lie in.

7.4 Prediction of the whole-life settlement

The framework presented in Chapter 5 was extended to include the prediction of whole-life settlement (consolidation and shear induced). Key conclusions include:

- A curved NCL and RSL with stress-dependent $\lambda$ and $\kappa$ were introduced, to better simulate the deformation at low stress levels. This improves the predicted consolidation settlement of the foundation.
- Shear settlement is considered as failure of the subsoil, and was calculated based on the framework generated strength profile. A methodology is proposed to linearise the curved strength profile generated by the extended framework. In addition, a group of shear settlement rate curves for various linearly distributed strength profiles was fitted from FE simulations, allowing the shear settlement rate of any strength profile (and under any practical loading level) to be calculated.
- Through simulation with the theoretical framework, the deeper soil elements were observed to soften and harden much slower than the interface element. Along with sliding cycles and intervening consolidation time, the $H-V$ failure envelope initially shrinks, before expanding with a changing shape. The expansion in magnitude and change in shape slow down the development of shear settlement.

7.5 Limitations and future research

All centrifuge model tests presented in this thesis are based on a fully rough foundation interface. In reality, sliding foundations are expected to be designed with a smooth interface – which would lead to reduced lateral resistance and settlement. Further physical modelling with fully smooth to partially
rough interfaces are suggested for future research, which would all the framework to be modified to account for variations in interface roughness. In practice, this may be achieved by adding a reduction factor to the strength mobilisation process used to predict the response of the interface element – although further consideration is warranted.

The mudmats researched in this thesis do not have drainage holes in the base plate, which may be used in practice to facilitate touchdown and/or improve drainage times. This could be explore through additional physical modelling, with improvements in consolidation time (and vertical capacity) potentially offset by the risk of soil extrusion (from beneath the base) leading to higher settlement.

The test results available for this study come from small scale modelling in a geotechnical centrifuge. Further confidence in the design of mobile foundations would be generated by performing field tests at larger scale. It would also be informative to understand how these foundations have worked in practice.
Extended framework for predicting the behaviour of tolerably mobile subsea foundations

Figure 7.1 Summary of research topics

Chapter 3 (Experiment)

Chapter 4, 5 (Sliding resistance)
Conclusions

Figure 7.1 Summary of research topics (Continued)

Chapter 6 (Settlement)

Ensuring shear settlement rate

Calculate strength profile through Eq.(6.34)

Discretise calculated domain

Iteration until $z_f = z_{fit}$

$z_{fit}$ (fitting depth)

New profile

Linearize the curved profile

Calculate the settlement rate and its failure zone

$V_{eff}/V_{ult}$

Exp (Cocjin et al. 2017)

$H/V_{eff}$

Failure envelope evolution

Cyc 0

Cyc 10

Cyc 20

Cyc 40

C1

$H$

$V$

NCL

RSL

'Sf' for softening

'Hd' for hardening

Initial peak

'Sf' for softening

'Hd' for hardening

Consolidation

settlement

Shear settlement

C1

Present simulation

Exp (Cocjin et al. 2017)

Exp (Hodder et al. 2013)

$\delta H/V_{eff}$

$w_{con}/B$

$w_{p}/B$

Consolidation settlement

Shear settlement

C1

Exp

Present simulation

C1

Hodder et al. 2013

Ev (Hodder et al. 2013)

N

Exp

Present simulation

Shear settlement

C1

Exp

Present simulation

C1
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