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Numerical study on cyclic response of offshore wind turbine foundations

Etude numérique sur la réponse cyclique des fondations d'éoliennes offshore

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ABSTRACT: Around 80% of offshore wind turbines are founded using large-diameter monopiles, with a length reaching up to 60 m due to the lack of easily accessible shallow waters. Thus, there is a natural tendency to seek more economically viable solutions where tripod bucket foundation seems to be especially promising. However, the up-to-date knowledge on the cyclic response of those foundations is still limited mostly due to the lack of the appropriate constitutive laws. To address the above-mentioned issues a series of centrifuge tests for both monopile and tripod foundations were back-analyzed using sophisticated 3D numerical models. The geometry of the foundations, soil-structure interface and properties of various materials were precisely reproduced. An advanced hypoplastic constitutive models were used to simulate the coupled hydro-mechanical response of soils subjected to cyclic loading. The monopile results were interpreted with the emphasis on the accumulation of cyclic rotation, foundation lateral stiffness and pore water pressure changes affected by episodes of reconsolidation. While simulations of the tripod bucket foundation allowed for a qualitative and quantitative assessment of the stiffness recovery effects.

RÉSUMÉ: Environ 80% des éoliennes offshore sont fondées sur des monopiles de grand diamètre, d'une longueur pouvant atteindre 60 m en raison du manque d'eaux peu profondes facilement accessibles. Ainsi, il existe une tendance naturelle à rechercher des solutions plus viables économiquement là où les fondations à godets tripodes semblent particulièrement prometteuses. Cependant, les connaissances à jour sur la réponse cyclique de ces fondations sont encore limitées, principalement en raison de l'absence de lois de comportement appropriées. Pour résoudre les problèmes mentionnés ci-dessus, une série de tests de centrifugation pour les fondations de monopile et de trépied ont été rétro-analysées à l'aide de modèles numériques 3D sophistiqués. La géométrie des fondations, l'interface sol-structure et les propriétés de divers matériaux ont été reproduites avec précision. Les théories récemment développées dans le cadre de l'hypoplasticité ont été utilisées pour simuler la réponse hydromécanique couplée de sols soumis à une charge cyclique. Les résultats obtenus dans le cas du monopile ont été interprétés en mettant l'accent sur l'accumulation de rotation cyclique, la rigidité latérale des fondations et les changements de pression interstitielle affectés par les épisodes de reconsolidation. Alors que les simulations de la fondation du godet tripode ont permis une évaluation qualitative et quantitative des effets de récupération de rigidité.

KEYWORDS: offshore wind turbine foundations, FEM simulations, cyclic loading, implicit approach.

1 INTRODUCTION

The pursuit of clean renewable energy significantly accelerated in the last two decades. A prominent role in renewable resources, besides nuclear power, is played by offshore wind farms. The rapidly developing market of offshore wind farms has certain advantages over the onshore ones. The average wind speed is usually higher and less turbulent offshore than onshore. At the same time, the cost of the offshore structures is higher due to the cost of foundation and installation which can be minimized by seeking areas of shallow water with reasonably strong wind. There are several types of offshore wind turbine (OWT) foundations were monopiles are still the most popular ones and constitute more than 80% of them (EWEA 2019). However, due to the lack of easily accessible shallow waters, their length can reach up to 60 m. Thus, there is a natural tendency to seek more economically viable solutions where tripod bucket foundation seems to be especially promising.

During relatively short design lifetime, OWTs foundations are subjected to loading conditions with peculiar characteristics, i.e. the long periods of calm conditions are interrupted by episodes of extreme cyclic loading (i.e. during storms). In cohesive subsoils, pore water pressure accumulates during an episode of sever loading and dissipates over the long and calm (low-magnitude loading) conditions. A large number of experimental and numerical studies on the cyclic lateral behaviour of monopiles embedded in soft soils can be found in the literature e.g. Matlock 1970, Wang et al. 2015, Hong et al. 2017, Staubach & Wichtmann 2020. However, only few of them analyzed the influence of the reconsolidation. The centrifuge tests carried out by Lai et al. 2020 suggest that lateral soil-pile stiffness degrades during cyclic loading and is fully recovered even exceeding the initial stiffness up to 20% after the reconsolidation episodes. Nevertheless, this positive effect is neglected at the design stage.

The recently developed tripod bucket foundation has certain advantages over the monopod. Despite significantly higher stiffness revealed by much lower rotation, the so-called ”healing effect” seems to be promising. The experimental study performed by Wang et al. 2018 shows that rotation of the tripod bucket foundation increases during the first few cycles and then reduces due to the increased stiffness of the foundation-soil system. This effect is probably related to the damage in soil fabric as pointed out by Houlsby et al. 2005. However, a clear explanation of this phenomenon was not given yet.

The crucial question that comes from the design point of view is whether state-of-the-art development in constitutive laws allows to accurately reproduce the aforementioned effects, i.e. stiffness recovery after reconsolidation and stiffness healing effect in tripod. To comprehensively address this issue, sophisticated numerical simulations of the cyclically loaded free-head monopile and tripod bucket foundation inspired by the series of centrifuge tests have been performed. The three-dimensional numerical models with the mechanical behaviour of soil modelled with a non-linear, irreversible and anisotropic hypoplastic constitutive laws for fine and coarse-grained materials have been implemented.

2 MONOPILE FOUNDATION

The centrifuge tests performed by Lai et al. (2020) at the Hong Kong centrifuge facility at 100g conditions serve as the reference for numerical simulations of monopile. The Free FEM code Tochnog professional (www.tochnogprofessional.nl) was used to carry out simulations.

3.1. Model set-up

The geometrical configuration of the model is illustrated in Figure 1. The pile with 60 mm diameter, length of 750 mm and 2 mm wall thickness was embedded into the soil on the depth of 600 mm. The pile was made from an aluminium alloy which mechanical behaviour was reproduced using linear-elastic material with Young's modulus of 72 GPa and Poisson's ratio of 0.33.

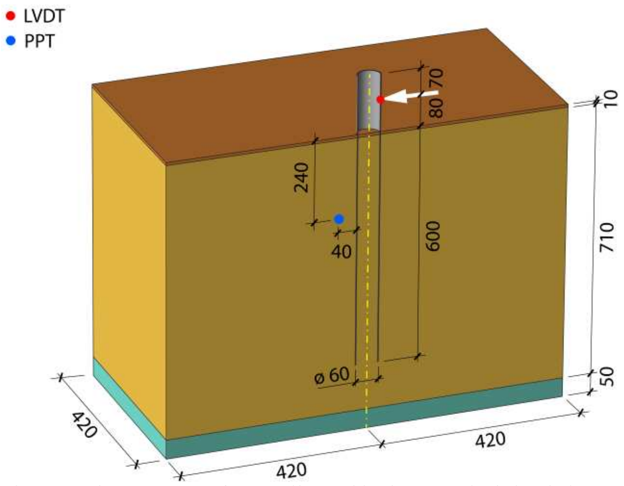


Figure 1. The geometry of monopile used in the numerical simulations

The monopile has been subjected to multistage one-way cyclic loading applied 70 mm from the pile's top end. The loading amplitude in each stage refers to 25% of the failure load. Each loading episode was composed of 100 cycles and followed by reconsolidation stage necessary to dissipate the excess pore water pressures.

The soil-pile interface was modelled using plane interface elements with the linear-elasticity and Mohr-Coulomb failure criterion. The interface stiffness was set to 72 and 7.5 MPa in a normal and tangential direction, respectively. The friction angle of the interface has been set to 14°, dilation angle to 0° and cohesion was set to 0.1 kPa.

The initial stress state has been calculated using relation for K_0 coefficient proposed by Mayne & Kulhawy (1982) for clays, i.e. $K_0 = (1-\sin\varphi_c)\,OCR^{\sin\varphi_c}$, and Jáky (1944) for sands, i.e. $K_0 = 1-\sin\varphi_c$. To initialize void ratio for kaolin layer OCR of 1.32 was used. Furthermore, hydrostatic distribution of pore water pressure with phreatic level at the top model surface was prescribed.

The geometry of the model has been discretized with 45000 hexahedral and prism finite elements of variable sizes (see Figure 2).

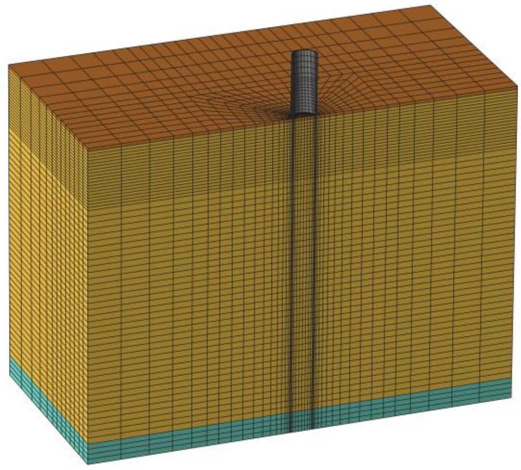


Figure 2. Discretization of the monopile FEM model

3.2. Soil characterization

In the experimental tests, the monopile was embedded in Malaysian kaolin. It can be characterized as highly plastic silt with a specific gravity of 2.71. To reproduce its mechanical response the hypoplastic model for fine-grained materials proposed by Mašín (2013) has been used together with the Intergranular Strain Concept (ISC) by Niemunis and Herle (1997). The model implementation in UMAT ABAQUS™ format is freely available at the soilmodels.info project website.

The response of the anisotropic hypoplastic model from small to large-strain has been calibrated against the monotonic and cyclic triaxial element tests performed on Malaysian kaolin. Undrained monotonic triaxial tests with initial mean effective stress of 200, 300 and 600 kPa have been carried out to evaluate the behaviour of kaolin under static conditions (Figure 3). While, cyclic undrained triaxial tests with 150 cycles and deviator stress amplitude equal to 0.4, 0.6 and 0.7 of undrained deviatoric stress at the critical state for sample initially consolidated to isotropic effective stress of 200 kPa (q_{200}) were taken to calibrate the response of the model in the small-strain range. Here the calibrated response of the model is for brevity shown (Figure 4) only for the test with $0.6 \cdot q_{200}$ and the list of constitutive model parameters is reported in Table 1.

At the bottom of the model, 50 mm thick layer from Toyoura sand was considered as drainage. This layer has been simulated using the hypoplastic model for granular soils by Wolffersdorff

Table 1. Calibrated parameters of hypoplastic constitutive models

Parameters of hypoplastic clay model with ISC for Malaysian kaolin

φ_c [°]	N [-]	λ^* [-]	κ^* [-]	ν [-]	A_g [-]	n_g [-]	R [-]	β_r [-]	χ [-]	m_{rat} [-]
30	1.565	0.13	0.013	0.1	350	0.66	1e-4	0.014	5.25	0.7

Parameters of hypoplastic sand model with ISC for Toyoura sand

φ_c [°]	h_s [MPa]	n [-]	e_{d0} [-]	e_{c0} [-]	e_{i0} [-]	α [-]	β [-]	R [-]	m_R [-]	m_T [-]	β_r [-]	χ [-]
30	2600	0.27	0.61	0.98	1.1	0.14	3.0	2e-5	8.0	4.0	0.1	1.0

(1996) with ISC. Parameters reported in Table 1 were taken from the literature (Ng et. al. 2013).

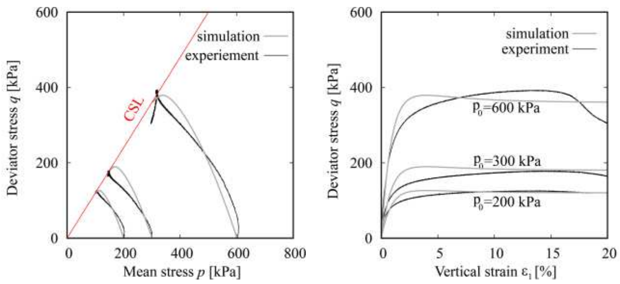


Figure 3. Simulation of monotonic undrained triaxial tests with various initial mean effective stress.

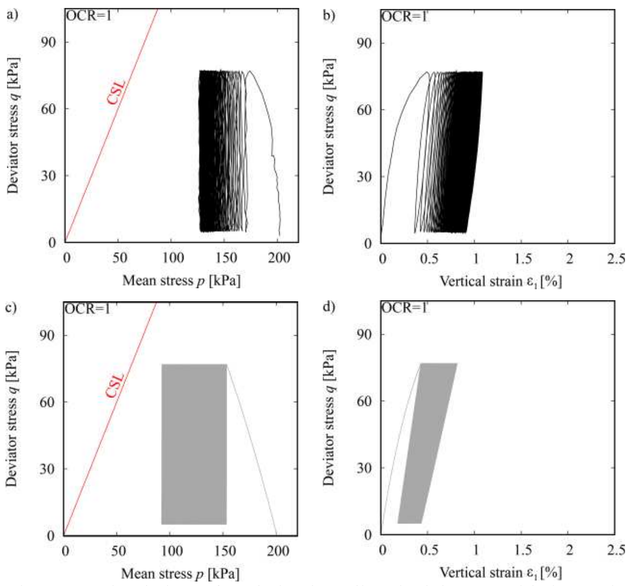


Figure 4. Simulation of undrained cyclic triaxial tests with isotropic consolidation p_0 = 200 kPa and cyclic stress amplitude $q^{amp} = 0.6{\cdot}q_{200}$: a) and b) experiments, c) and d) simulations.

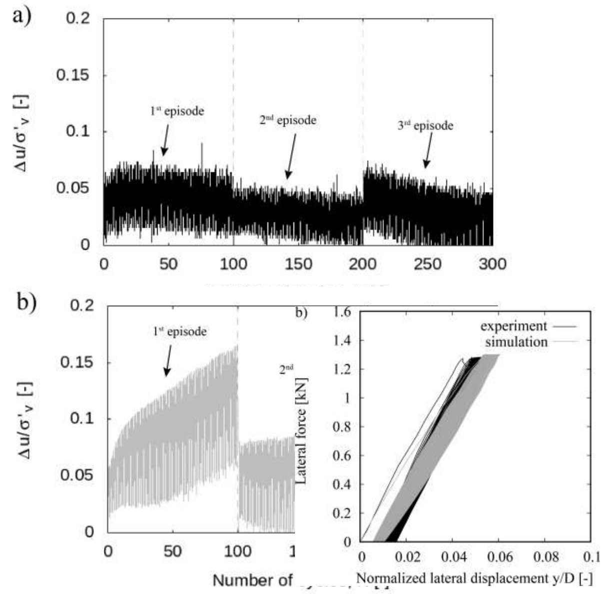
3.3. Simulation results

The behaviour of the monopile subjected to cyclic loading is analyzed by means of excess pore water pressure and pile head displacements read out from specific points (see PPT and LVDT in Figure 1).

The observed and computed changes of pore water pressure ($\Delta u/\sigma'v$) with the number of cycles are illustrated in Figure 5. The dissipation of pore water pressure occurring between episodes has been for brevity filtered out. From the experiments, it can be seen that the increase of excess pore water pressure reveals only during the first few cycles for each loading episode. After, it remains almost constant (ep. 1) or even slight reduces (ep. 2 and 3) with the number of cycles. The simulation results show a noticeably different response. Excess pore water pressure increases significantly during the first few cycles and then continue through the remaining ones. However, simulation results are in the same order of magnitude as the experimental observations.

Comparison of the monopile head displacement between the experiment and simulation is presented for each cyclic loading episode in Figure 6. For the first loading episode numerical model presents well-reproduced behaviour during the first cycle and the following one. However, residual displacements after unloading are slightly underestimated. The experiment shows that after reconsolidation for second and third episodes monopile has a stiffer response, i.e. lower recorded displacements for first cycles. This stiffness increase after reconsolidation was not caught by numerical simulations. However, peak and residual displacements were still considerably well reproduced by simulations.

Figure 5. Comparison of a) experimentally and b) numerically obtained normalized excess pore water pressure.



The change of the unloading stiffness (Figure 7) due to reconsolidation can be derived from the above presented cyclic-loading displacement curves. It is defined as ratio k_{epi}/k_{ep1} being unloading stiffness of the first cycle for specific loading episode divided by the unloading stiffness of the first cycle obtained from the first loading episode. The experiment reveals an increase of unloading stiffness after reconsolidation, while simulation results show almost no change. However, lack of model capability to reproduce this effect is conservative. Figure 6. Comparison of normalized lateral displacement of monopile's head vs lateral force for a) first episode b) second episode c) the third episode.

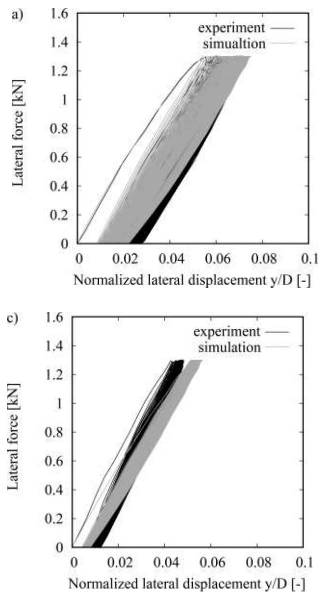
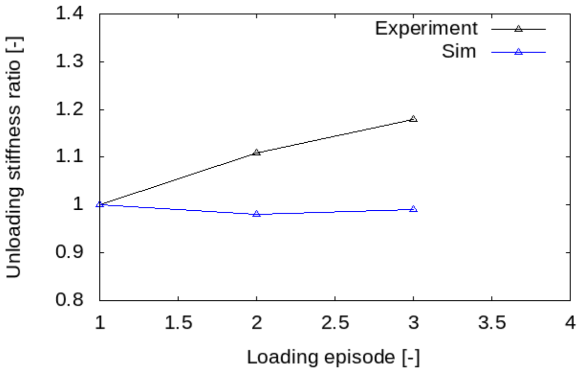


Figure 7. Comparison of experimentally and numerically obtained unloading stiffness for all three loading episodes.

3 TRIPOD SUCTION BUCKET FOUNDATION

Numerical simulation of tripod suction bucket foundation reproduces centrifuge tests carried out at Zhejiang University (Wang et al. 2018) at 100g conditions. Tochnog professional FEM code was also used to perform numerical simulations.



3.1. Model set-up

The tripod geometry consists of the three buckets with a centre-to-centre distance of 30 m connected by each other with a triangular base plate. At its centroid, a rigid loading tower is attached. Figure 8 shows detailed geometry reduced by symmetry. The foundation was made from an aluminum alloy with the Young modulus of 72 GPa and Poisson's ratio of 0.3.

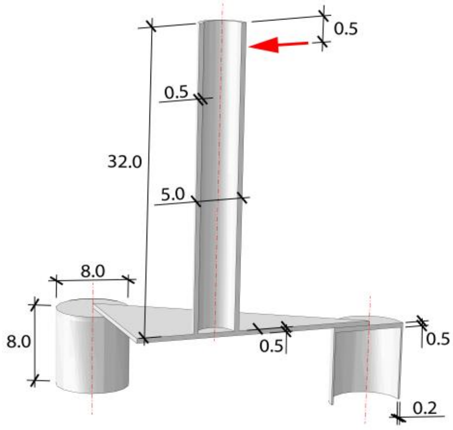


Figure 8. Geometry of the tripod bucket foundation

The tripod has been subject to six loading episodes with increasing loading amplitude (from 10% up to 60% of failure load – 13.33 MN). To reduce the computational effort, in each episode 100 loading cycles have been simulated (from 1000 to 2000 per each episode in the experiment). The load has been applied at a height of 31.5 m as shown in Figure 8. Numerical simulations have been performed separately for each episode neglecting stress history effects. This assumption is acceptable as proven by Kallehave et al. (2015).

Simulation of each episode has been preceded with the calculation of the initial stress state using K_0 procedure with Jáky (1994) relation, i.e. $K_0 = 1 - \sin\varphi$. Moreover, for simplicity increase of gravity from 1g to 100g has been omitted thus the initial state refers directly to 100g conditions.

Discretized geometry of foundation together with soil block is shown in Figure 9. FE mesh consist of around 260000 hexahedral and prism elements of variable sizes.

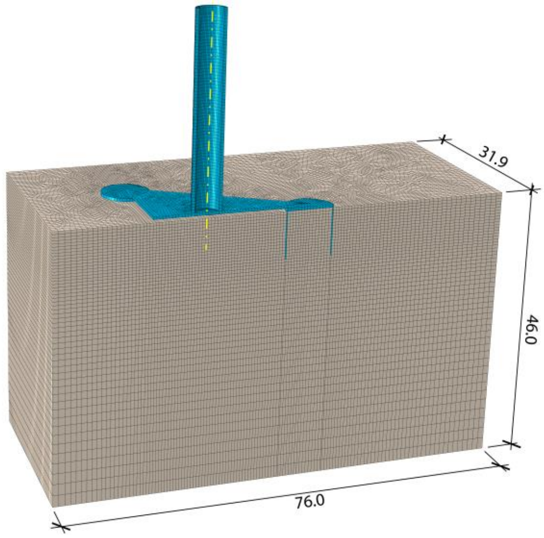


Figure 9. Discretized geometry of the tripod foundation

The soil-structure interaction was modelled using plane interface elements with the linear-elasticity and Mohr-Coulomb failure criterion. The interface stiffness was set to 1000 and 10 MPa in a normal and tangential direction, respectively. The friction angle of the interface has been calculated as *2/3* $\tan(\varphi_c)$, while cohesion was set to 0.1 kPa.

3.2. Soil characterization

Soil used in the experiment is medium dense dry Fujian sand. Its mechanical response was reproduced using the advanced rate-independent hypoplastic model for granular materials. The large-strain response was simulated using von Wolffersdorff (1996) model characterized by 8 parameters (φ_c, h_s, n, e_{d0}, e_{c0}, e_{i0}, α and β). The small-strain response and effects of past history were described by Intergranular Strain Concept (Niemunis and Herle 1997) characterized by another 5 parameters (R, m_R, m_T, β_r and χ).

This constitutive model has been calibrated based on the drained cyclic triaxial tests with various stress path reversal (0°, 90°, -90° and 180°) prior cyclic shearing. The calibrated response correlated with the experimental one is shown in Figure 10 and 11. Parameters of the calibrated model are given in Table 2.

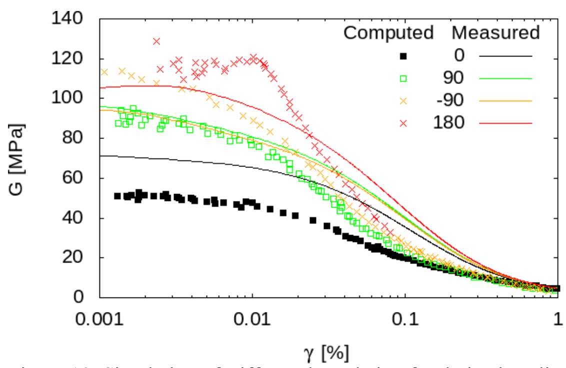


Figure 10. Simulation of stiffness degradation for drained cyclic triaxial tests.

Table 2. Calibrated parameters of hypoplastic constitutive model (von Wolffersdorff 1996, Niemunis and Herle, 1997) for Fujian sand.

φ_c [°]	h_s [MPa]	n [-]	e_{d0} [-]	e_{c0} [-]	e_{i0} [-]	α [-]	β [-]	R [-]	m_R [-]	m_T [-]	β_r [-]	χ [-]
32.5	25000	0.31	0.607	0.952	1.14	0.08	1.8	1e-4	3.9	2.6	0.1	0.8

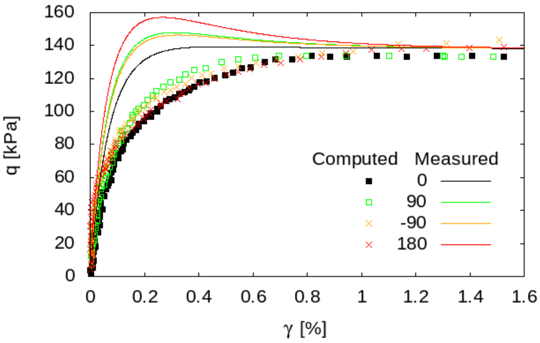


Figure 11. Simulation of deviatoric stress change for drained cyclic triaxial tests.

3.3. Validation

Comparison of the observed and computed rotation angle of the tripod is shown in Figure 12. Rotation angle is defined as a rotation of the whole foundation being positive if directed counterclockwise. From the experimental study, it can be seen that the rotation angle increases during the first few cycles for each loading episode and then turns into a slight decrease for the remaining ones known as a shakedown response. This trend of rotation angle change is called “self-healing” effect caused by irreversible damage in soil fabric (Houlsby et al. 2005, Andersen 2015). The numerical simulations show that the magnitude of peak and residual rotation angle can be accurately reproduced. However, the backward tilt of foundation is observed only for the second and the subsequent episodes.

3.4. Deformation mechanism

Figure 13 shows the deformation mechanism for the last loading episode (60% Fu) after 10 and 100 cycles. It is illustrated by the displacement vectors together with maps of mobilized friction angle (φ_{mob}). Initially, after the first 10 cycles, a slight drop of mobilized friction angle is observed around skirts of both suction buckets. The active wedge is formed behind the pulled one as φ_{mob} increases noticeably. At the same time, displacement vectors indicate that the whole foundation undergoes counterclockwise rotation which is consistent with the direction of the applied load. At this stage, damage in soil fabric around the pulled bucket is not extensive enough to affect the direction of rotation.

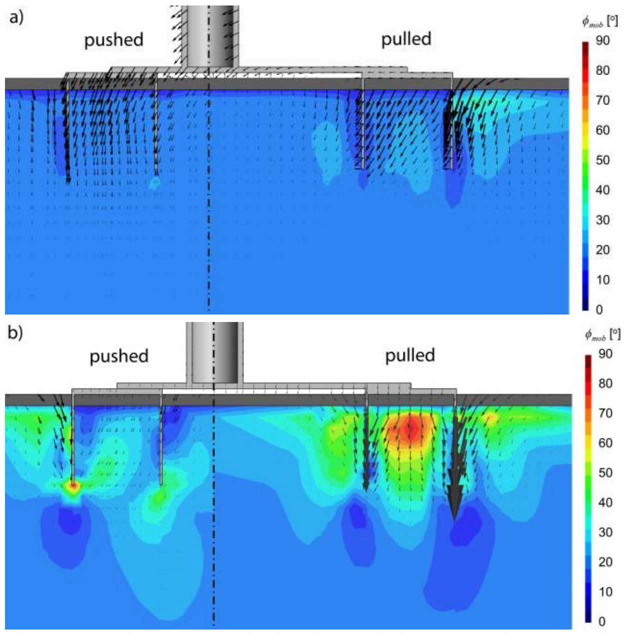


Figure 13. Deformation mechanism induced after a) 10 and b) 100 cycles for last loading episode (60% Fu).

The subsequent cycles leads to drastic stress change around foundation which is revealed by significant increase of mobilized friction angle and excessive deformations. The φ_{mob} increases up to 50º in the zones where active wedges are formed, i.e. at the front and at back of pulled caisson and at the front of the pushed one. Inside pulled bucket φ_{mob} increases almost up to 80º which is possible due to the significant stress reduction and the resulting increase of void ratio. With the increasing number of loading cycles and progressive damage in soil fabric being more extensive around the pulled bucket, the settlement rate of the pulled bucket overcome that for the pushed one. Furthermore, when the cumulative settlement of the pulled bucket overcome that for the pushed one, the whole foundation starts to change the direction

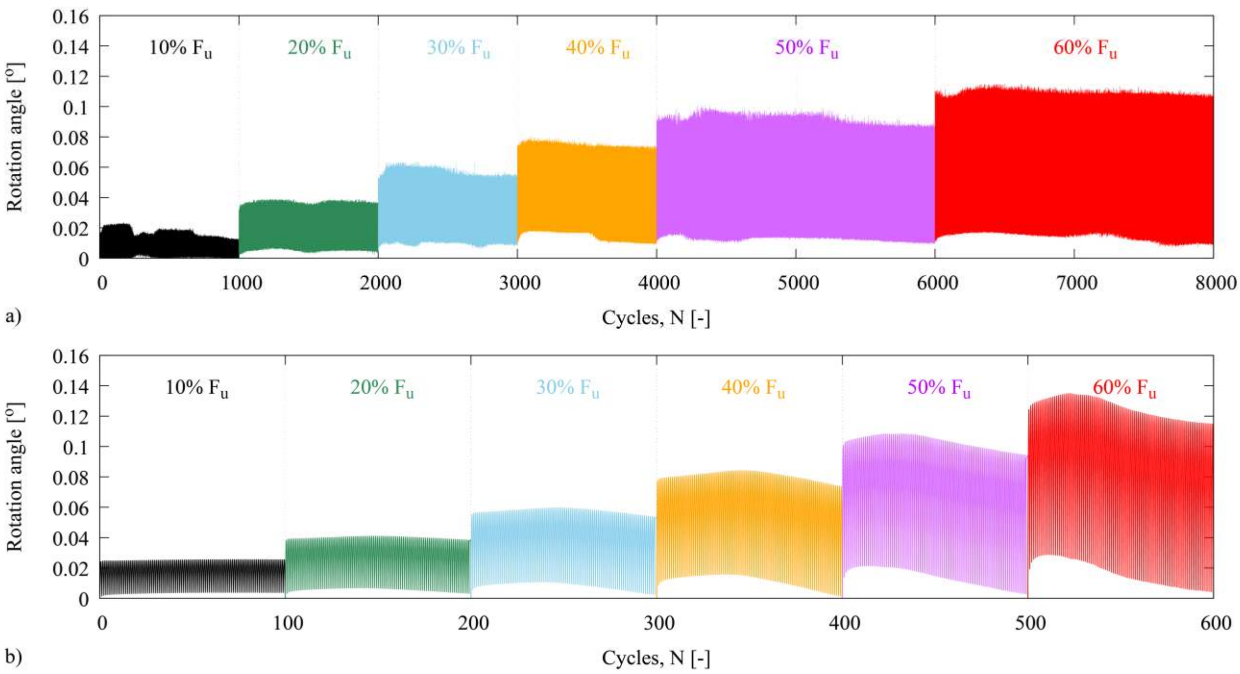


Figure 12. Rotation angle of tripod bucket foundation: a) observed, b) calculated

of rotation from counterclockwise to clockwise as observed in Figure 12.

4 CONCLUSIONS

Reliability of numerical simulations to prediction cyclic response of OWTs foundations is essential to take benefits from their peculiar behaviour.

Numerical simulations of monopile in saturated fine-grained soil subjected to alternating episodes of cyclic loading and reconsolidation shown that:

- The increase of cumulative peak and residual displacements of pile's head during cyclic loading was well predicted for the first lading episode. However, they are significantly affected by stiffness increase during each reconsolidation stage which was also well reproduced.
- The model is not capable to predict an increase of unloading stiffness after each reconsolidation stage.
- Finally, the excess pore water pressure generated during cyclic loading was overestimated by the model. The experiments show that after an initial increase of pore water pressure during each episode of cyclic loading, there is a phase of stabilization or even reduction of pressure during the next loading cycles. While the numerical simulations predicted typical shakedown response.

The main findings from the numerical simulations of tripod suction bucket foundation in coarse-grained material are given below:

- Numerical simulations well-reproduced rotation of foundation and deformation mechanism.
- For all loading stages after the first few cycles foundation rotates counterclockwise. The progressive and uneven damage in soil fabrics induced by subsequent loading cycles change this trend.
- When the cumulative settlements of the pulled bucket overcome those for the pushed one rotation of foundation is reversed from counterclockwise to clockwise producing "stiffness recovery effect"

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