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Installation effects in the Response of Laterally Loaded Monopiles in Sand- A Numerical-Based Analysis

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ABSTRACT: The rate of construction of offshore wind turbine farms is continuously increasing, with monopiles being the extended type of foundation. During the life period of these infrastructures, the offshore environment induces lateral loads, which are transmitted to the soil. This repetition of the lateral forces results in accumulated deformations in the foundation. Thus, after a certain point of residual displacements, the operation of the wind turbine is compromised, as the serviceability limit state is exceeded. Hence, the research efforts need to focus on the cyclic effects taking place on a large-piled foundation. But for the sake of completeness, it should be stressed that in monopiles, the properties of the soil mass control the response. Different aspects affect the rate of accumulated displacements of monopiles in cohesionless soils, but one of the most important effects is the disruption of the initial soil conditions during their installation. This paper aims to investigate the long-term cyclic response of monopiles taking into consideration the installation effects. In view of such calculations, advanced Finite Element Method (FEM) modelling, adopting the hypoplastic law with intergranular strain, is undertaken in this paper. For the calculation of the lateral cyclic response, an elasto-plastic, stiffness degradation model is considered. Herein, the soil state condition after installation is presented, while afterwards a comparative response analysis between jacked and wished in place piles is carried out.

Keywords: Monopiles; Installation; Jacked; Lateral; Cyclic.

1 INTRODUCTION

Nowadays, monopiles are still the most frequent type of foundations for offshore wind turbines. However, the cyclic behaviour of such systems is not fully understood yet. This is directly related to the large diameter of the monopiles and subsequently to their interaction with the soil. Past research studies were focusing mainly on high aspect ratios (i.e. length of the pile over its diameter - L/D), of 20 to 60, in contrast with the corresponding ratio of monopiles which usually only reach up to 5.

Recent research efforts consider pile installation effects in the analysis of laterally loaded piles. Centrifuge tests prove that different installation methods affect the lateral capacity of single piles [(Fan et al., 2021a); (Maatouk et al., 2021)]. In addition, numerical analyses examine the sand state after installation, with the hypoplastic law as the predominantly employed constitutive model for this purpose, while for the large deformation analysis, the Coupled Eulerian-Lagrangian method is often adopted [(Henke & Grabe, 2006); (Fan et al., 2021b); (Staubach et al., 2022)]. Significant conclusions regarding pile installation by numerical method analyses are highlighted below. Dynamic penetration, such as impact driving, results in significant densification, while jacking of the piles leads to greater radial stresses. This is a remarkable conclusion regarding the volume-stress state of sand after different penetration methods by (Fan et al., 2021b). In addition, arching effects might appear in jacked penetration. For the subsequent lateral response hypoplastic model is employed [(Bienen et al., 2021)] or high-cyclic accumulation law is adopted [(Staubach et al., 2022)]. On the contrary, research efforts have successfully analysed the response of laterally loaded piles without considering the pile installation effects [(McAdam et al., 2019); (Taborda et al., 2019)].

A significant research effort regarding laterally loaded monopiles emphasises the investigation of the accumulation rate due to cyclic load. In view of such calculations, a power law is adopted to represent this rate (Klinkvort & Hededal, 2013), Equation 1.

$$\frac{y_N}{y_1} = N^a \qquad (1)$$

where y: the lateral displacement, N: number of cycles.

This paper aims to investigate the rate of accumulation induced by lateral loads in piles considering also the installation effects. The model is calibrated with existing centrifuge tests on the lateral response of monopiles and later on, extends this analysis to investigate the response of monopiles under different conditions. Firstly, emphasis is given to modelling the installation of monopiles, adopting a hypoplastic law. Secondly, the lateral response of the monopile is investigated by considering the initial conditions of the installation of the pile. In addition, the effect of the diameter is examined during the lateral response, adopting the stiffness degradation method developed by (Achmus et al., 2009).

2 METHODOLOGY

Herein, the validation of numerical models with the centrifuge tests by (Fan et al., 2021a), using loose UWA sand both for pile installation and lateral monotonic response, is presented, followed by an extension of previously reported experimental results by (Wang et al., 2022), who performed centrifuge tests in dense Geba sand with a relative density of D_R=80% at a target g-level of 100. The selected geometrical properties of piles analysed herein are presented in Table 1. It is worth noting that the diameter to thickness ratios, D/t, is relatively small compared to typical values of 80-100 in offshore environments (Fan et al., 2021b), but represent prototype sizes corresponding to the scaled experiments in the centrifuge. The experimental response of piles assumes Wished-In-Place (WIP) conditions as they were jacked in at 1 g.

Table 1. Geometrical properties of examined piles.

	Pile 1	Pile 2	Pile 3	Pile 4
D, m	1.8	2.5	3.2	5.22
D/L, m	5	3.6	2.8	3.1
e/L, -	0.67	0.67	0.67	1.22
t, m	0.10	0.15	0.10	0.21

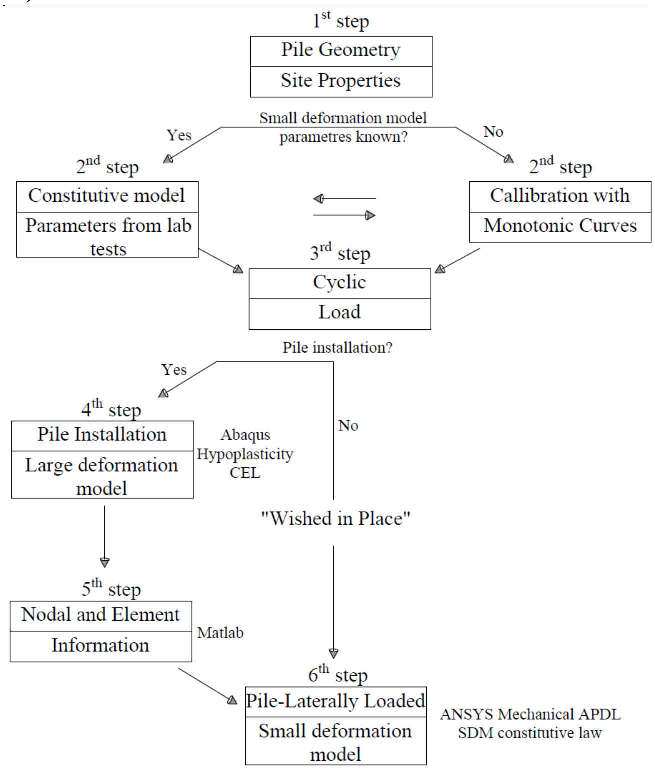

Figure 1. Chart of methodology

The methodology followed in this work is summarised in the chart of Figure 1. First, the geometrical properties of the pile are defined. In the second step, the definition of the soil properties is performed. During the third step of the calculation of cyclic loading, the effects of pile installation can be considered, or the pile can be analysed as WIP. The installation effects are analysed with CEL method and hypoplasticity with intergranular strain law. In the case of the jacked piles the final void ratios, the radial and vertical stresses obtained from the large strain model at the end of the installation of the monopiles, are processed in (MATLAB, 2022) and then imported to the small strain FEM model. The mentioned initial states of the FE model affect the shear modulus of the soil domain and its degradation rate. In the WIP model, all the elements have the same degradation parameters b_1 and b_2, as well as initial stress ratio X (Achmus et al., 2009). This is the main difference with respect to the jacked foundation.

2.1 Large strain- Installation of monopiles

The CEL method for the pile installation is employed herein adopting Abaqus software (Dassault Systèmes Simulia Corp, 2017). The monopile-pile interaction has been modelled using interface elements, considering both normal and tangential contact. Herein, the tangential friction contact is considered with a penalty factor of 0.5. The coefficient of lateral earth pressure is taken as $K_o=1-sin(\varphi_c)$. The width and the height of the selected soil domain are greater than 11D and 25D respectively, see Figure 2 (Fan et al., 2021a), to avoid reflections in the boundaries.

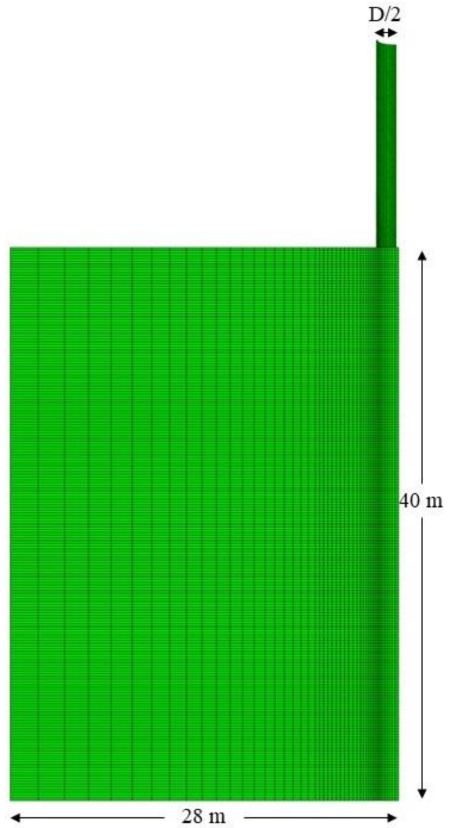

Figure 2. CEL numerical model

The UWA and Geba sands are clean, fine, and uniform, with grain shapes from sub-angular. In Table 2 the hypoplastic parameters are presented for Geba and UWA sands (Azúa-González et al., 2019) and (Fan et al., 2021b) respectively.

Table 2 Hypoplasticity parameters for Geba and UWA sands

Sand	UWA	Geba
Critical friction angle (°), φ_c	30	34
Granular hardness (MPa), h_s	1354	2500
Exponent, n	0.34	0.3
Minimum void ratio, e_{d0}	0.49	0.64
Critical void ratio, e_{c0}	0.76	1.07
Maximum void ratio, e_{i0}	0.86	1.28
Exponent, α	0.18	0.11
Exponent, β	1.27	2.0
Stiff. ratio at change of dir.180°, m_R	5.16	5.5
Stiff. ratio at change of dir. 90°, m_T	3.07	3.9
Max. value of intergranular strain, R	1e-4	1e-4
Exponent, β_R	0.58	0.3
Exponent, χ	5.74	0.7

2.2 Small strain- Lateral response of pile

A small-strain Finite Element Method (FEM) model for the lateral response of WIP and jacked piles is developed (ANSYS, 2019). After a sensitivity analysis, the diameter of this domain is found to be adequate to avoid significant reflection of waves, approximately at 12D, Figure 3. The small strain shear modulus is calculated with Hardin's formula in Equation (2) (Hardin & Black, 1968).

$$G_{max} = \frac{B_g \cdot p_a}{0.3+0.7\cdot e^2}\left(\frac{p}{p_a}\right)^n \quad (2)$$

where, G_{max}: shear modulus in small strains, B_g: soil model parameter, e: void ratio, p: mean effective stress, p_a: atmospheric pressure.

The Stiffness Degradation Method (SDM) with the Mohr-Coulomb yield criterion has been adopted for the estimation of the cyclic (one-way) response of foundations on cohesionless material (Achmus et al., 2009). This specific model is based on experimental results under drained conditions. The estimation of the soil degradation due to cyclic load is based on the accumulation of plastic axial strains obtained from cyclic triaxial test observations. Therefore, the secant stiffness of the Nth cycle can be estimated by Equation (3).

$$\frac{E_{sN}}{E_{s1}} \approx \frac{\varepsilon^{\alpha}_{p,N=1}}{\varepsilon^{\alpha}_{p,N}} \approx N^{-b_1\cdot(X)^{b_2}} \quad (3)$$

where, (X) is the cyclic stress ratio as it was defined by (Huurman, 1996). This ratio is defined as the 1st principal stress at static failure over the 1st principal stress of the considered cyclic, Equation (4).

$$X = \frac{\sigma_1}{\sigma_{1,sf}}, \ \sigma_{1,sf} = \frac{(1+sin\varphi)\sigma_3+2\cdot c\cdot cos\varphi}{1-sin\varphi} \quad (4)$$

The Mohr-Coulomb parameters for the loose UWA sand and dense Geba sand are presented in Table3. The friction angle and dilatancy angle considered for Geba sand can be found in (Li et al., 2022), while for loose UWA sand the critical friction angle considered.

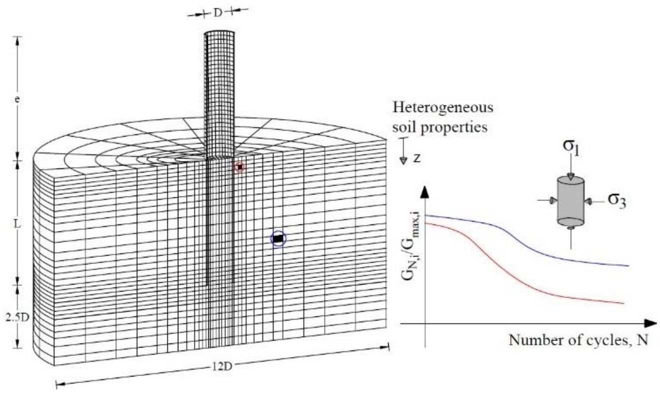

Figure 3. Schematic representation of the FEM model considering soil elements with different soil properties.

Table 3. Mohr-Coulomb parameters

Sand	UWA	Geba
Soil model parameter, B_g	280	440
Friction angle, φ, º	30	38
Dilation angle, ψ, º	1	8

Accumulation parameters for Achmus et al. (2009) model

The accumulation of permanent strains in the first cycle over the N-th cycle is determined and the stiffness degradation throughout the whole cyclic load is assumed to be analogous. The accumulation rate of strains depends on different factors, such as the relative density D_R, the mean particle size d_{50}, the uniformity index C_u, the mean stress p, the cyclic stress ratio η=q/p, the number of cycles N, etc. [(Qian et al., 2019); (Wichtmann & Triantafyllidis, 2015)]. (Achmus et al., 2009) account degradation by means of the stress ratio and the degradation parameters, b_1 and b_2. Existing cyclic triaxial tests from the literature are adopted to obtain a range of b_1 and b_2 parameters.

Firstly, cyclic triaxial tests performed in clean quartz sand (Wichtmann and Triantafyllidis, 2015) are considered. The first set of tests had the same mean uniformity index C_u (equal to 1.5), varying the mean particle size d_{50} from 0.35mm to 2.0mm (in Figure 4 noted as Wichtmann et al. (2015)-1). The second set of cyclic triaxial tests considered herein have the same d_{50}, 0.6 mm, but varying C_u from 2 to 5 (in Figure 4 noted by Wichtmann et al. (2015)-2). Moreover, the experiments in Toyoura sand with d_{50} of 0.16mm and 0.20mm by (Qian et al., 2019) and (Sun et al., 2020) respectively, are also employed in this research. Finally, the tests by (Lentz & Baladi, 1961) carried out tests in Highway subgrade sand with mean particle size d_{50} of 0.4mm. The investigation of parameters considered for all the aforementioned tests is undertaken for a number of cycles, N, up to 10^4.

The accumulated permanent strains of the described cyclic triaxial tests were matched with Equation (3) adopting a least square method. Hence, the produced parameters are presented in Figure 4 (a) and (b) normalised by the relative density of the sample, mean particle size and index of uniformity. The limitation of this analysis refers to the small stress range of the adopted tests. Further information regarding the b1 and b2 can be found in (Spyridis, 2023).

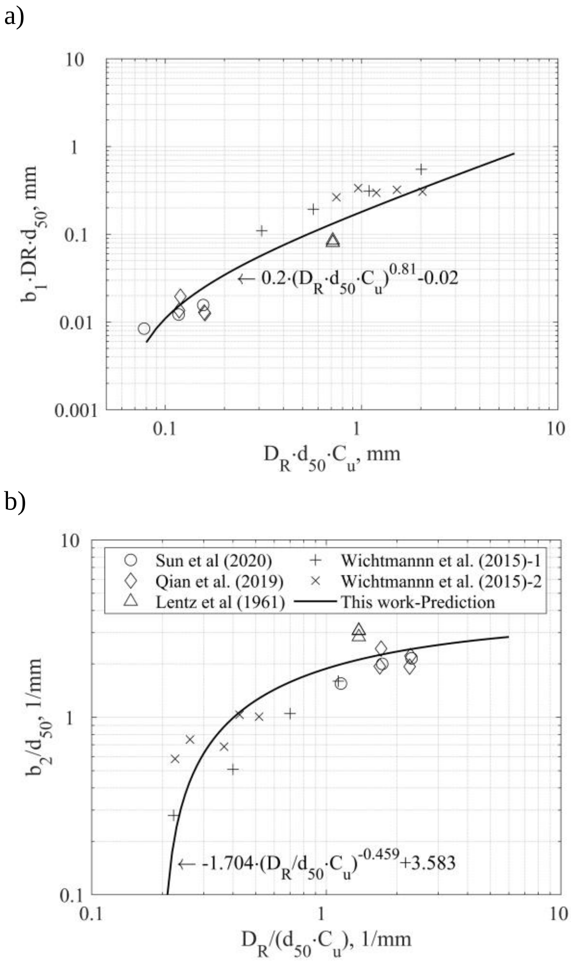

Figure 4. Prediction of accumulation rate parameters for Achmus et al. (2009) constitutive model: a) b1 b) b_2.

3 RESULTS

The installation force for the penetration of Pile 4 by the described numerical model has a very good aggrement against the centrifuge test carried out by (Fan et al., 2021b) in UWA sand, see Figure 5. Secondly, the comparison of the centrifuge tests with the small strain numerical model for laterally loaded piles is presented in Figure 6. An excellent agreement is achieved between the WIP experimental result and the numerical model. It should be noted that the WIP numerical model underestimates the initial stiffness. The procedure to calculate the jacked piles with MC is explained briefly herein. The post-installation sand state are imported in the small strain model, namely, the radial stresses, vertical stresses and the void ratio around the pile. The solution fails to capture with high accuracy the actual response of jacked monopile. For the monotonic response, an advance constitutive model is required, such as the one used in PISA project (Taborda et al., 2019).

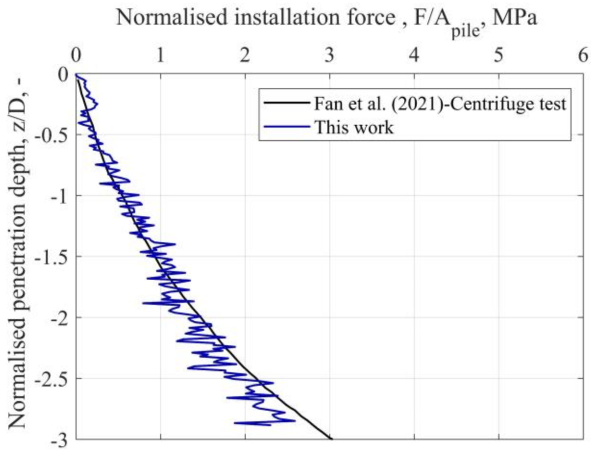

Figure 5. Validation of the pile installation numerical model.

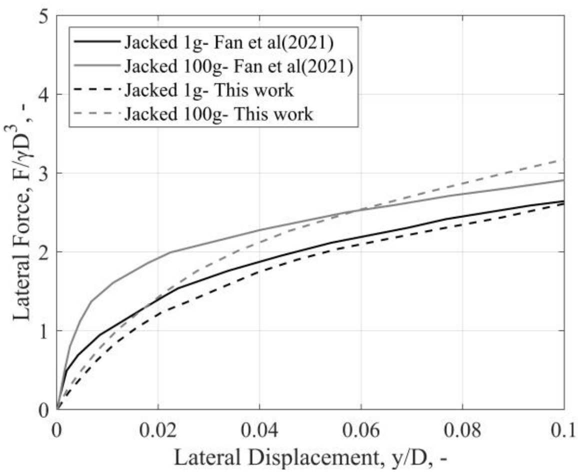

Figure 6. Validation of the pile installation numerical model.

3.1 Installation of Monopile in Geba sand

The initial conditions of the soil domain have significant effects on the lateral response of monopiles. In view of this investigation, the soil conditions after the installation of a 1.8m diameter pile are plotted in Figure 6 and 5. The initial void ratio (e_0=0.726) of the dense Geba sand D_R=80% after installation is significantly changed, especially in the surrounding soil mass of the pile. A zonation at the outer surrounding soil mass of the pile of dilated material is shaped, see Figure 7. This zone has an average thickness 0.8m measuring from the outer diameter of the pile. The cohesionless material is dilated by 12% to 32%. At the tip of the pile, the sand is densified by 13%. In the inner soil of the open-ended pile, dilation is calculated. Specifically, the cohesionless material is dilated by 10% to 20% in the inner soil.

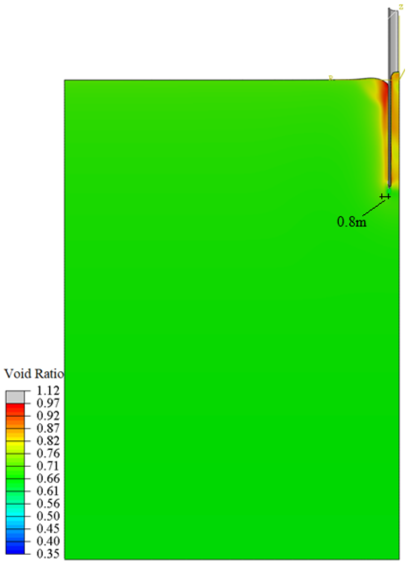

Figure 7. Void ratio at the end of installation-D=1.8m

3.2 Cyclic Lateral response of monopiles

In this analysis, a comparison of laterally loaded WIP and jacked piles is attempted. The comparison of this analysis examines the effect of installation in the lateral response of monopiles, but also the effect of the diameter in the lateral response of piles. The WIP numerical model is calibrated against (Wang et al., 2022) in order to adopt the profile of shear modulus small strains.

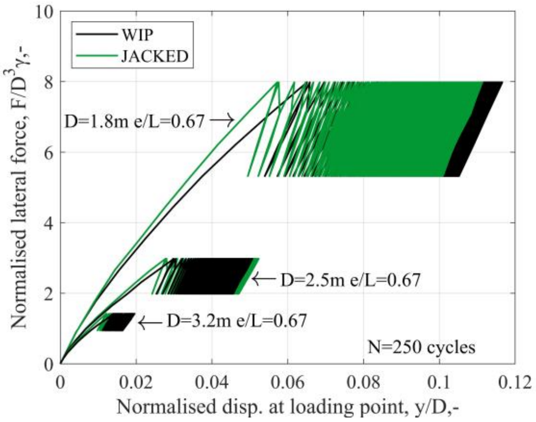

Figure 8. Comparison of WIP and jacked pile in Geba sand.

The piles are examined under the WIP assumption, but also considering the installation effects. For the WIP the initial conditions are produced by the first step of the analysis, that is to say, the static analysis considering only gravitational forces. On the other hand, the initial conditions during installation are significantly affected, as presented in a previous section and thus are imported as initial conditions. The applied lateral force at the top of the pile is a one-way low-frequency (f=1 Hz) cyclic load. The maximum and the minimum amplitude of the applied lateral forces are 720 kN and 480 kN respectively.The resultant response of the piles under the different initial conditions is depicted in Figure 8. Firstly, it can be noticed that the initial stiffness of the jacked pile, during the monotonic part of the load, is greater than the analysed foundation assuming WIP. Similar conclusions in a comparative analysis between a WIP pile and a jacked pile excited by the monotonic load are drawn by (Fan et al., 2021a).

After 250 cycles, the rate of accumulation in the WIP piles is greater than jacked piles, see Figure 9. This is induced by the dilation of the material around the pile. The parameter α of the power law in Equation 1 for each numerical analysis is presented in Table 4. As the diameter of the pile increases the accumulation rate reduces.

Table 4. Parameter α of power accumulation law.

	WIP	Jacked
Pile 1	0.10	0.12
Pile 2	0.09	0.11
Pile 3	0.05	0.08

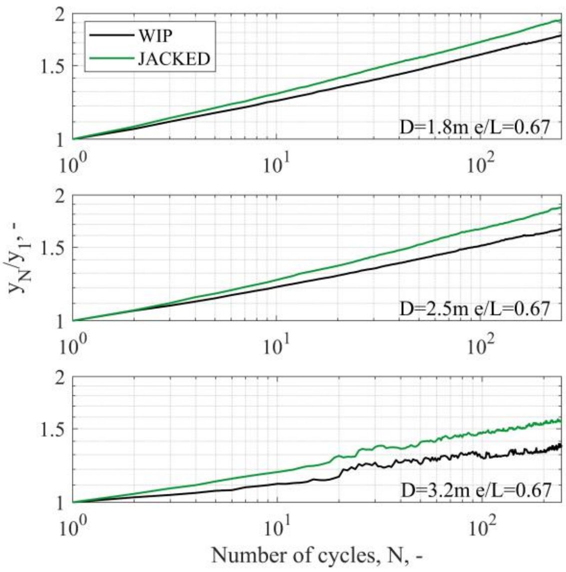

Figure 9. Accumulated displacements in Geba dense sand.

4 CONCLUSIONS

This paper deals with the lateral response of monopiles in sand. Firstly, the jacked installation of the pile is investigated by adopting a hypoplastic law. In the dense Geba sand, the dilation after installation at the target depth is calculated. Three piles with different diameters, but with the same embedded length are examined. This analysis investigates the effect of the diameter on the lateral response of the piles. A comparative analysis between a jacked pile and a WIP pile is carried out herein. The initial stiffness of the examined jacked piles is greater than the corresponding WIP piles. Nevertheless, the rate of accumulation of displacements of jacked pile is greater than the WIP pile in the analysed cases.

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