Variation in Stiffness of Monopiles in Dense Sand Under Cyclic Lateral Loads

Giulio Nicolai* and Lars Bo Ibsen
Department of Civil Engineering, Aalborg University
Aalborg, Denmark

The stiffness of the soil-foundation system is an important parameter that is taken into account in design. Recent studies of offshore wind turbines have shown that long-term cyclic lateral loading, induced by waves and wind, may lead to an increase in the foundation stiffness during the lifetime of the foundation. This is in contrast to design methods proposed by current guidelines. The present paper represents a further step towards improving the general knowledge regarding such stiffness change issues. Test results from an experimental investigation are presented and discussed. The interpretation focuses on the stiffness variation of a small-scale monopile in dense saturated sand during cyclic tests of approximately 50,000 load cycles.

INTRODUCTION

Offshore wind energy represents an attractive solution among the renewable energies for the large space availabilities and high mean wind speeds of the offshore environment. Nevertheless, wind energy converters are still more expensive offshore than onshore, and therefore a reduction of the costs is necessary for the development of such technology. The foundation represents approximately one third of the offshore wind turbine cost. The monopile, which is the most common foundation for offshore wind turbines, is a hollow steel pile often with a diameter of 4 to 6 m and an embedded length of 20 to 30 m. The design methodology for offshore piles was developed through studies funded by the oil and gas industry and was based on testing slender piles with diameters smaller than 1 m. Such a typology of piles has a flexible behavior that might be different from the stiff behavior typical of monopiles for offshore wind turbines. Furthermore, the current design methodology proposed in API (2010) and DNV (2010) does not account for the number of cycles and was developed through analysis of the experimental results of tests with less than 100 cycles. In real offshore conditions, a wind turbine structure is subjected to millions of cyclic lateral loads that induce an accumulated rotation that might be critical for the turbine. Therefore, further investigations are necessary to fully understand the behavior of monopiles under cyclic lateral loading in order to improve design standards.

The present paper studies the behavior of monopiles under cyclic lateral loading by testing a small-scale monopile model. Recent works on the experimental testing of monopiles showed that the soil-pile stiffness changed after the application of cyclic lateral loads, which might lead to the risk of resonances. The purpose of the present work is to investigate such a phenomenon in order to understand the effects of cyclic lateral loading on the soil-foundation stiffness. The experimental setup used to perform the investigation is a 1-g testing rig capable of applying thousands of load cycles and static loading to the monopile model.

PREVIOUS EXPERIMENTAL WORKS

Cyclic loads are considered to be reduction factors of the lateral resistance of offshore piles in the current design methodology. Such a method relies on experimental results presented in Reese et al. (1974) and O’Neill and Murchison (1983), in which results from full-scale tests of slender piles with no more than 100 load cycles were analyzed. In Long and Vanneste (1994), a review of previous works was presented, and a relationship was proposed where the soil resistance was reduced with a power law as a function of the number of cycles. Such an experimental investigation was based on the analysis of 34 field tests with 5 to 500 load cycles. LeBlanc et al. (2010) performed a series of tests with 8,000 to 65,000 load cycles on a small-scale monopile in loose sand with a 1-g testing rig. The tested pile had a slenderness ratio of 4.5 and a diameter of 80 mm. It was shown that the soil-pile stiffness increased during the cyclic tests. Furthermore, a method to predict the variation of the stiffness was proposed, providing an equation that relies on the cyclic load characteristics and logarithmically on the number of cycles. Other works aimed at studying the change in the soil-foundation stiffness during cyclic loading have been performed by Zhu et al. (2013) and Haigh (2014). The same investigation by LeBlanc et al. (2010) but on a bucket foundation was presented in Zhu et al. (2013), showing that an increase in stiffness was occurring during the cyclic tests. Haigh (2014) presented results from tests with centrifuge modeling conducted on a small-scale monopile, proving that the stiffness of the soil-pile system increased in medium dense sand. Nowadays the knowledge regarding the variation stiffness is still limited, and therefore further tests are required in order to improve the design methodology for monopiles subjected to cyclic lateral loading.

EXPERIMENTAL SETUP

Test Equipment

The tests were performed in the laboratory of Geotechnical Engineering at Aalborg University. The experimental investigation is carried out by means of a 1-g testing rig consisting of a steel sandbox $S$, a loading frame $F$, a loading lever $L$, and three weight hangers (see Figs. 1 and 2). The sandbox is a cylindrical container of diameter 2.00 m and height 1.20 m and is filled with sand at the top to 0.90 m and with gravel at the bottom to 0.30 m. The gravel layer is separated from the sand by means of a geotextile membrane. The soil is filled with water through a system of perforated pipes placed in the gravel that is used as drainage material. The pipes are
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Fig. 1 Photograph of the testing rig

![Fig. 1 Photograph of the testing rig](image)

Fig. 2 Sketch of the testing rig. $F_1$ and $F_2$ are the load cells, $D_1$, $D_2$, and $D_3$ are the horizontal displacement transducers, $m_1$ and $m_2$ are the masses used to apply the cyclic loading, and $m_3$ is the mass used to counterbalance the left side of the rig. $S$, $F$, $L$, $B$, and $W_1$ and $W_2$ are the sandbox, loading frame, loading lever, steel bar, and steel wires, respectively. Dimensions are in m.

equally distributed at the bottom of the sandbox in order to pour the water uniformly into the soil.

The water is slowly poured from a tank that is placed in a higher position of the sandbox. The gravel ensures proper drainage conditions and a homogenous inflow. A water level of 20 to 40 mm is kept over the sand surface in order to have the whole soil sample saturated. Aalborg University Sand No. 1 (Baskarp Sand No. 15) is the sand used to carry out the presented tests. The sample is prepared dense with a relative density of 80% to 90%, and its properties are listed in Table 1. The permeability of the Aalborg University Sand No. 1 was assessed by tests described in Sjelmo (2012). The permeability coefficient is measured by means of the falling head method described in the standard BS 1377-5 (1990).

A homogenous soil along the sand box is the goal that has to be achieved in order to have consistent soil parameters. The soil is the same from test to test and is prepared with a standard procedure in order to ensure conditions of repeatability. The soil preparation consists of different steps that are repeated prior to each test and have been widely described in Sørensen et al. (2015) and Sørensen and Ibsen (2011). The soil is loosened by raising the water level approximately 100 mm above the soil surface with an upward hydraulic gradient equal to 0.9. The intensity of the gradient is chosen in order to prevent piping in the sand. A wooden plate with equally distributed holes is then placed on the top of the sand box, and the soil is vibrated with a rod in a systematic manner. The sand is vibrated by introducing the rod in established holes with a specific order to ensure a uniform compaction of the soil. The water level is kept above the soil surface during vibrating in order to prevent air from entering into the sand. The vibration enables us to reach the density that is pursued for the investigation. Finally, three small-scale Cone Penetration Tests (CPTs) are performed at three different spots of the soil to check the uniformity of the sand along the sandbox. Typical results from the CPTs are reported in Fig. 3, in which CPT 2 is performed at the center of the sandbox, while CPTs 1 and 3 are performed at 500 mm from CPT 2 on both sides. The CPTs are also used to extrapolate the sand parameters and to calculate the relative density of the sand according to the procedure shown in Ibsen et al. (2009). Afterwards, the installation is performed by driving the pile into the sand at the center of the sandbox with a screw jack driven by an electric motor. A steel bar $B$ is then bolted to the top of the pile and connected to the rig with two steel wires $W_1$ and $W_2$, as shown in Fig. 4. We applied the loads to the pile by pulling the wires connected to the steel bar that represents the loading arm.

Static and cyclic tests are performed within the present experimental investigation. A static test is performed by pulling the steel bar monotonically with a steel wire at a constant velocity of 0.02 mm/s by means of a screw jack that is placed on the left side of the testing rig. A cyclic test is performed by utilizing two wires that are connected to the steel bar, one on each side. As shown in Fig. 2, the steel bar is connected on the left side of the rig to the loading lever that supports the mass $m_1$ and on the right side of the rig to the loading lever that supports the mass $m_2$. A third mass $m_3$ is used as a counterweight to the loading lever. The hanger of $m_2$ is

![Fig. 3 Example of typical results from CPTs](image)

Table 1 Characteristics of Aalborg University Sand No. 1

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{50} = 50%$ - quantile</td>
<td>0.14</td>
<td>[mm]</td>
</tr>
<tr>
<td>$C_U = d_{50}/d_{10}$</td>
<td>1.78</td>
<td>[-]</td>
</tr>
<tr>
<td>Specific grain density $d_s$</td>
<td>2.64</td>
<td>[-]</td>
</tr>
<tr>
<td>Maximum dry unit weight, $\gamma_{\text{max}}$</td>
<td>17.03</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>Minimum dry unit weight, $\gamma_{\text{min}}$</td>
<td>14.19</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>Permeability coefficient, $v_s$</td>
<td>0.692</td>
<td>$10^{-4}$ [m/s]</td>
</tr>
</tbody>
</table>
steady, while an electric motor enables the hanger of \( m_1 \) to make a circular rotation that induces the loading lever to oscillate vertically. Such an oscillation, which is applied to the steel bar and thus to the pile, generates the cyclic loading. The period of the cyclic load is represented by the time it takes \( m_1 \) to complete a circular rotation and is set to 10 seconds. Two load cells \( F_1 \) and \( F_2 \) are used to measure the loads applied to the pile during a test, while \( D_1 \), \( D_2 \), and \( D_3 \) are the transducers used to measure the displacement and rotation of the pile (see Fig. 2). The monopile model used in the present work is scaled to approximately 1:50 in relation to a typical monopile and is an open-ended aluminum pile with a diameter, length, and thickness equal to 100 mm, 500 mm, and 5 mm, respectively (see Fig. 4). No pore pressures are developed during the tests, meaning that the response of the pile is drained.

Further details regarding the test setup and soil preparation are shown in Roesen et al. (2012). The approach proposed by LeBlanc et al. (2010) relies on two parameters that define the cyclic loading features: \( \zeta_b \) and \( \zeta_c \), respectively, and \( K_b \) and \( K_c \) are nondimensional parameters that depend on \( \zeta_b \) and \( \zeta_c \), respectively, and \( K_c \) is defined as \( K_c (\zeta_c = 0) = 1 \).

In order to determine the parameter \( K_b \), it is necessary to carry out a number of tests with \( \zeta_b = 0 \). As shown in Fig. 6, the parameter \( k_\theta \) represents the stiffness during the first cycle and varies with \( \zeta_b \) and \( \zeta_c \). The parameter \( K_b \) is now equal to \( k_\theta \), as \( K_c \) is equal to 1 for \( \zeta_c = 0 \).

Therefore, it is possible to determine \( K_b \) for different values of \( \zeta_b \). Once \( K_b \) is determined, other tests with different \( \zeta_c \) may be performed. As \( k_\theta \) and \( K_b \) can be determined, \( K_c \) is now achievable for different \( \zeta_c \) through the inversion of Eq. 4. Further details regarding the presented method are discussed in LeBlanc et al. (2010). The parameter \( A_\theta \) achieved in LeBlanc et al. (2010) was found to be 8.02 and was considered to be independent of the loading features and relative density of the sand. Note that the data are presented in this paper with real dimension and are not scaled as in LeBlanc et al. (2010). The reason for this choice is that the authors are interested in interpreting qualitative and not quantitative results. Furthermore, the scaling law presented in LeBlanc et al. (2010) is highly dependent on the characteristics of the setup and model and might affect the results if used under different conditions.

### Stiffness Parameters

An increase in the soil-pile stiffness has been found in each of the cyclic tests that have been carried out in the present experimental investigation. This outcome confirms that even dense sand may become stiffer during cyclic loading. These findings are in contrast to the general assumption that the stiffness of the soil-pile system by increasing the number of cycles.

In Ref. 34, LeBlanc et al. (2010) suggested the following equation to fit the experimental data:

\[
k_N = k_0 + A_k \ln(N)
\]

where \( N \) is the number of cycles, \( k_0 \) represents the initial stiffness and may be evaluated as \( k_N (N = 1) \), and \( A_k \) represents the slope of the evolution of \( k_N \) with respect to \( N \). The parameter \( k_0 \) can be expressed as:

\[
k_\theta = K_b (\zeta_b) K_c (\zeta_c)
\]

where \( K_b \) and \( K_c \) are nondimensional parameters that depend on \( \zeta_b \) and \( \zeta_c \), respectively, and \( K_c \) is defined as \( K_c (\zeta_c = 0) = 1 \).

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to current design guidelines that consider cyclic loads by reducing the soil-foundation stiffness. The soil-pile stiffness during a cyclic test is plotted in Fig. 7 with the fitting curve of Eq. 3. Such increase in stiffness is due to a compaction of the sand during the pile oscillation induced by the cyclic loading. As a consequence of this process, the sand becomes stiffer and more resistant, as shown in Nicolai and Ibsen (2014). Figures 8 and 9 depict the parameter $K_b$ achieved in the present study and in LeBlanc et al. (2010), respectively. Note that in the present work, $K_b$ is expressed in Nm/Deg and therefore is not quantitatively comparable to $K_b$ scaled in LeBlanc et al. (2010). Nevertheless, a qualitative comparison is possible, and it turns out that in both works $K_b$ decreases when $\zeta_b$ increases. The parameter $K_c$ as a function of $\zeta_c$ is plotted in Fig. 10 with the results of LeBlanc et al. (2010). The figure shows an increase of $K_c$ as $\zeta_c$ reaches zero in both works. Furthermore, the parameter $A_k$ has been found not to be a constant in the present experimental investigation and in particular varies between 10 and 25 Nm/Deg.

**Observations**

In the following section, seven tests are analyzed in more detail, and the main data are shown in Table 2. $\Delta k_{fin}$ represents the variation between the stiffness of the soil-pile system at the end of the cyclic test, $k_{fin}$, and at the first cycle, $k_0$, as follows:

$$\Delta k_{fin} = \frac{k_{fin} - k_0}{k_0}$$  \hspace{1cm} (5)

The last column of Table 2 shows that the stiffness of the soil-pile system has increased from the beginning to the end of the tests in a range from 13% to 49%. In LeBlanc et al. (2010), an example

<table>
<thead>
<tr>
<th>Test</th>
<th>$N$</th>
<th>$\xi_b$</th>
<th>$\xi_c$</th>
<th>$A_k$</th>
<th>$k_0$</th>
<th>$k_{fin}$</th>
<th>$\Delta k_{fin}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50810</td>
<td>0.31</td>
<td>-0.15</td>
<td>11.8</td>
<td>980</td>
<td>1108</td>
<td>13%</td>
</tr>
<tr>
<td>2</td>
<td>57437</td>
<td>0.32</td>
<td>-0.95</td>
<td>15.7</td>
<td>434</td>
<td>606</td>
<td>40%</td>
</tr>
<tr>
<td>3</td>
<td>49970</td>
<td>0.31</td>
<td>-0.11</td>
<td>10.3</td>
<td>671</td>
<td>782</td>
<td>16%</td>
</tr>
<tr>
<td>4</td>
<td>49920</td>
<td>0.29</td>
<td>-0.92</td>
<td>22.1</td>
<td>614</td>
<td>853</td>
<td>39%</td>
</tr>
<tr>
<td>5</td>
<td>50092</td>
<td>0.29</td>
<td>-0.27</td>
<td>24.7</td>
<td>689</td>
<td>957</td>
<td>39%</td>
</tr>
<tr>
<td>6</td>
<td>23801</td>
<td>0.28</td>
<td>-0.09</td>
<td>20.1</td>
<td>408</td>
<td>610</td>
<td>49%</td>
</tr>
<tr>
<td>7</td>
<td>32441</td>
<td>0.25</td>
<td>-0.19</td>
<td>12.6</td>
<td>665</td>
<td>796</td>
<td>20%</td>
</tr>
</tbody>
</table>

Table 2 Tests features. $A_k$, $k_0$, and $k_{fin}$ are in [Nm/Deg]
where was given to demonstrate the use of Eq. 3 to calculate the variation of the soil-foundation stiffness during the expected lifetime of a foundation (e.g., 10^2 load cycles). It turns out that the first hundred cycles play an important role in the lifetime of the foundation and that after 10^4 cycles a variation of the stiffness might be negligible or might not even be expected.

CONCLUSIONS

The present paper provides the results of an experimental investigation aimed at studying the long-term variation of the stiffness of a soil-pile system in dense sand. The setup used for carrying out the tests described in the present work is a 1-g small-scale testing rig. First, the stiffness variation is evaluated and compared with results achieved in LeBlanc et al. (2010), where an increase in the soil-pile stiffness was shown during cyclic tests in loose sand. The presented results show that an increase in stiffness occurred during the cyclic tests in this work. An example is provided that shows that an increase in stiffness of 38% in dense sand occurred in the present work compared to that of 60% in loose sand in LeBlanc et al. (2010), with the same loading configuration used in both works. This means that an increase in stiffness is expected to occur even in dense sand, but it is less than that in loose sand. A thorough study of the stiffness variation rate per number of cycles is then reported, proving that almost half of the
total stiffness increase occurs within the first hundred cycles, while a non-relevant increase occurs after $10^4$ cycles. Furthermore, the accumulated rotation of the pile during the cyclic tests is evaluated and analyzed in terms of the variation rate. Indeed, the pile rotation does not uniformly accumulate during a cyclic test but mainly accumulates in the first hundreds of cycles. It is shown that the rotation and stiffness variation rates have the same trend so that the rotation accumulation decreases when the soil becomes consistently stiffer after 100 load cycles. Nevertheless, further work is required to validate the present results, in particular full-scale tests.

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