Evaluation of monopile embedment using frequency, damping and modal shape analysis

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ABSTRACT

The design of monopiles foundation for Offshore Wind Farms requires the characterisation of the rigidity between the soil and the shaft of the pile. While soil investigation methods are normally used, return of experience with measured data is rarely available. This paper presents a series of simple dynamic tests, performed on two 1:6 scaled monopiles foundation installed in rock as part of a lateral pile test campaign. Results of the dynamic tests can be compared with the theoretical fixed beam using frequency, damping and modal shape parameters with the aim of assessing to rigidity of the soil-structure interaction. The tests were carried out onshore France in 2016 by FUGRO and EDF-RE for offshore wind farms development projects as an add-on to a large-scale pile testing program. Two foundation piles of 1.2 m diameter, partially installed into Calcarenite, received a range of lateral dynamic strokes. The dynamic response of each pile was recorded at high frequency by accelerometers installed along the stick-up length of piles. The results were analysed to identify the lowest resonant frequency and subsequent vibration patterns. In addition, logarithmic decrement analyses and modal deformation analyses were performed to better understand the piles dynamic response. Comparing those results to the theoretical values of a fixed beam provides valuable information and help to assess the as-built connection between the rock and the piles. Such information can be used at design stage to quantify the rigidity of interaction between the soil and the pile. It can also be used during the lifetime of the foundation to monitor the time evolution of this connexion.

Keywords: Monopile, Embedment, Rigidity, Calcarenite, Soil stiffness, Dynamic test, Frequency, Damping, Modal shape.

1. INTRODUCTION

When designing offshore wind turbines supported by a monopile, the foundation can be modelled as an elastic beam of length L, mass per unit length m having an average bending stiffness EI with a top mass M (Tempel and Molenaar, 2002). Its soil-structure connexion is represented by a number of linear springs. The spring stiffness is normally determined from the initial stiffness E^{*}_{p-y} of the non-linear p-y curve formulation suggested by the design regulations (e.g.: API (2000) and DNV (2011)).

The natural frequency of the modelled beam decreases together with the stiffness of its connexion. The evaluation of the frequency of a beam partially casted in soil hence provides insight on the soil-structure interaction stiffness (Alexander, 2011).

The oscillation waves at the pile-soil interface carry away some energy in the form of radiation damping. (Bhattacharya, 2011). The damping measured during the test presented is compared to the structural damping of a fixed beam in an attempt to assess of the stiffness of the soil pile connexion.

The geometry of the piles tested have a L/D ratio of 7.25 and are approximately a 1:6 model for monopiles foundations. However, results presented in this article shall not be related to studies on windfarm natural frequencies and damping such as Andersen (2012) where the total length of the windfarm is considered with a L/D ratio of 25 to 30.

2. FIELD EXPERIMENT

2.1 Test layout

The experiment described below was carried out in November 2016 in Gouvieux, France on two piles of identical geometry as part of a large-scale pile testing experiment. Both piles are 1.2 m diameter and 35 mm in wall thickness. Their total length is 8.7 m and both piles were installed 3.2 m into Calcarenite. The methodology of installation however varies: The first pile (P5) was driven using impact hammer while the second pile (P6) was inserted and grouted in place in a drilled shaft.

The soil stratigraphy at Gouvieux site is characterized by very weak to weak calcarenite (UCS 4 MPa to 10 MPa) from ground level down to 3.6 m, followed by medium strong Calcilutite (two UCS of 30 MPa and 41 MPa). The Calcarenite layer presents similar characteristics to the formation encountered in two wind farm development sites, Offshore France.

Both piles were equipped with a variable quantity of accelerometers, bolted onto the side of the pile along the axis of the dynamic solicitation and distributed around the pile top measuring accelerations perpendicular to the axis of the pile.

Dynamic solicitation of the pile was performed by applying a single and horizontal stroke onto the pile with a sledgehammer. Strokes were performed at 5.25 m (3.25 m for test P5-3) above ground level from a cherry picker. The fig.1 below illustrates the test setup.

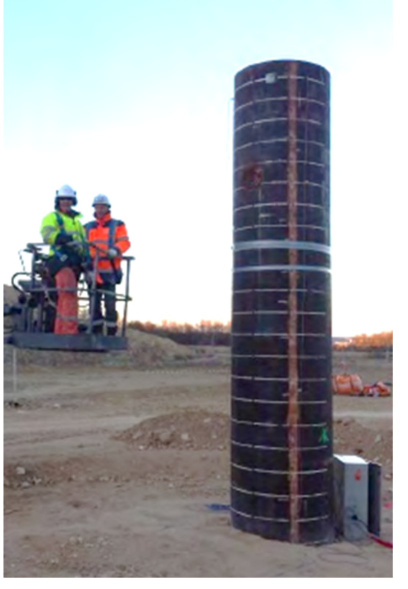

Fig. 1. Test setup with accelerometers and manually applied hammer stroke from a cherry picker

It shall be noted that prior to the dynamic solicitation described above, both piles endured a monotonic horizontal loading up to 2.45 MN applied 5.0 m above ground level as part as the lateral testing program. Horizontal displacement during these tests remained well below the failure criteria (D/10) at ground level in the direction of the loading.

2.2 Measurements

The positions of accelerometers sensors on the piles P5 and P6 are described in Figure 2 and Figure 3.

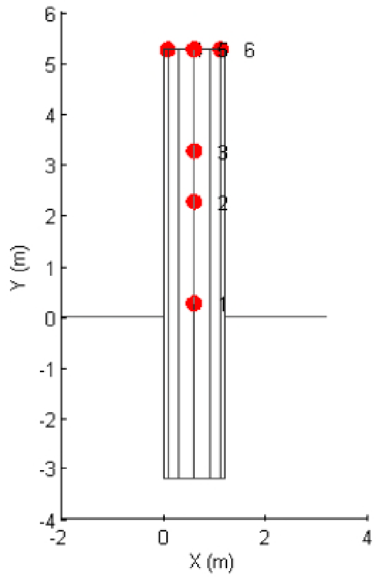

Fig. 2. Test layout for pile P5 and P6 (elevation view). Sensors in red dots.

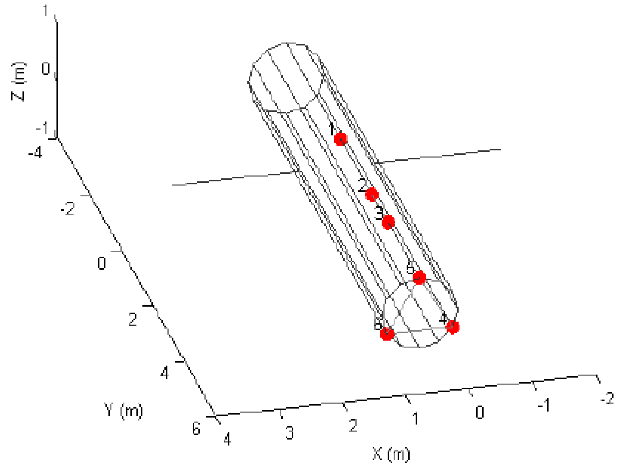

Fig. 3. Test layout for pile P5 and P6 (3D view-horizontal pile). Sensors in red dots.

As per Table 1, the pile P5 was tested with 3 different configurations (tests P5-1 to P5-3) and the pile P6 was tested with 2 different configurations (tests P6-1 and P6-2). The hammer strokes were applied at 5.25 m above ground level except for test configuration P5-3 where the strokes were applied at a lower height of 3.25 m. For each configuration, a low, a medium and a hard stroke were applied.

The Table below indicates the sensors' location for each configuration.

Table 1. Test configurations

Test configuration	P5-1	P5-2	P5-3	P6-1	P6-2
Height of impact [m]	5.25	5.25	3.25	5.25	5.25
Sensor 6	X	X	X	X	
Sensor 5	X	X	X		X
Sensor 4	X	X	X	X	
Sensor 3	X	X	X		X
Sensor 2	X				X
Sensor 1		X	X	X	X

Data from all installed sensors were recorded from a unique datalogger to ensure synchronisation in time. Records are 31 seconds long with a sampling frequency of 9'600 Hz and includes 1 second of pre-trigger recording. The Figure 4 below shows the acceleration recorded versus time from all 5 accelerometers sensors during a stroke.

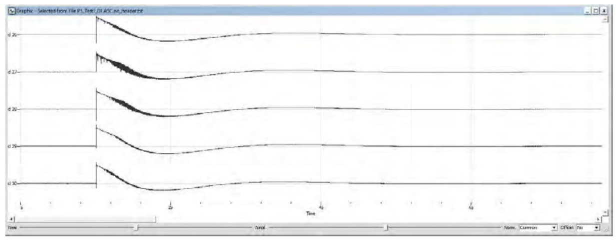

Fig. 4. Recorded accelerations versus time for 5 accelerometer sensors (test P5-1).

3. DATA ANALYSIS

3.1 Theoretical modal frequency of piles

The theoretical frequency of a beam of length L and mass per length m with one fixed end (cantilever beam) can be calculated using the Equation (1) below (Harris and Piersol, 2002):

$$f_i = \frac{{\lambda_i}^2}{2\pi L^2}\sqrt{\frac{EI}{m}} \tag{1}$$

Where λ is a constant related to the mode of vibration, E is the modulus of elasticity, and I is the area moment of inertia calculated as below with R the pile radius and e the pile thickness:

$$I = 2\pi . R^3 . e \tag{2}$$

The theoretical frequencies of 1st to 3rd modes for a fixed beam, considering the pile geometry of P5 and P6 piles are summarized in the Table 2 below.

Table 2. Theoretical modal frequency of piles P5 and P6

Mode of vibration i	1	2	3
λ_i	1.875	4.694	7.855
Theoretical Frequency [Hz]	60	374	1’046

These theoretical frequencies are compared to the measured frequencies of the tested piles.

However, the considered beam is not a theoretical beam but an open-ended cylinder, so local modes can occur and limit the comparison.

3.2 Frequency analysis

Data processing consisted in performing a spectrum analysis using Fast Fourier Transform (FFT) on the recorded data to show the acceleration as a function of frequency.

For each test configuration, the dataset was separated into two windows: The first window was taken during the stroke by selecting the 3 first seconds of the record following the hammer stroke and is referred to as “stroke window”. The Figure 5 below present window selected and the color coding applicable to the stroke window data presented in Figure 8.

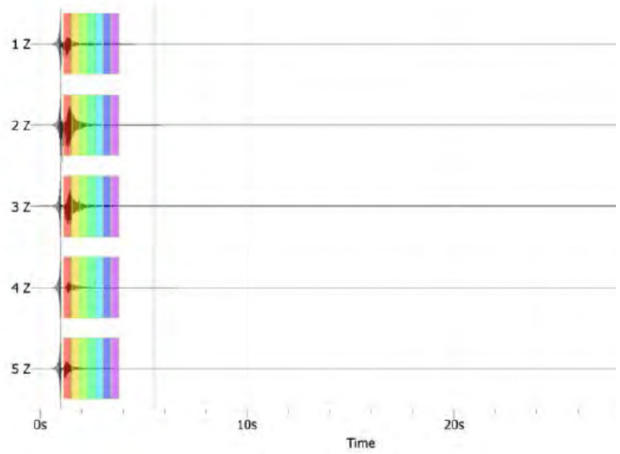

Fig. 5. P5-1 – Window selection for the Stroke window

A second window was selected as the last 10 seconds of the record, hence 20 seconds after the hammer stroke and is referred to as “ambient window”. The Figure 6 below present window selected and the color coding applicable to the ambient window data presented in Figure 7

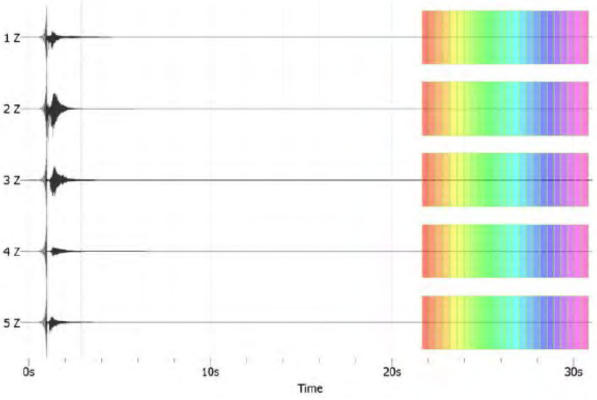

Fig. 6. P5-1 – Window selection for the Ambient window

The ambient window is analysed first as it is considered out of the influence of the hammer stroke with vibrations considered as ambient vibrations. The analysis reveals two clear amplitude peaks at 70 Hz and 200 Hz respectively and consistent for all test configurations.

The Figure 7 shows the recorded accelerations spectrum versus frequency recorded during test P5-1 from twenty to thirty seconds following the hammer stroke and considered as ambient vibration.

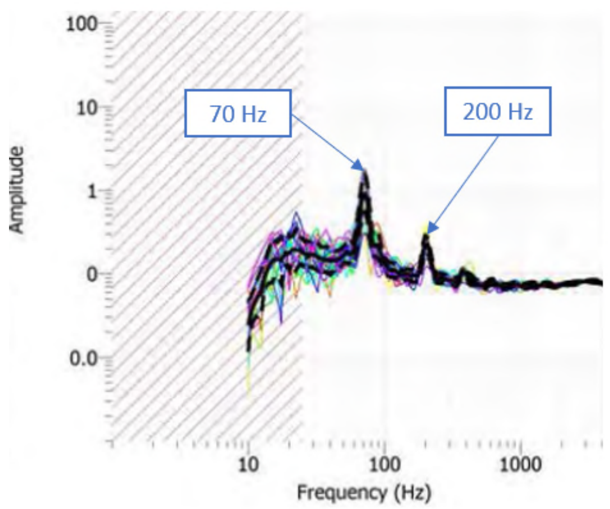

Fig. 7. P5-1 – Ambient vibration for pile P5 (P5-1 test configuration).

Recorded data from the first window corresponds to the hammer stroke. The analysis reveals four amplitude peaks at 70 Hz, 200 Hz, 380 Hz and 615 Hz and consistent for all test configurations.

The Figure 8 shows the frequency spectrum for accelerations recorded during test P5-1 for the first 3 seconds following the hammer stroke.

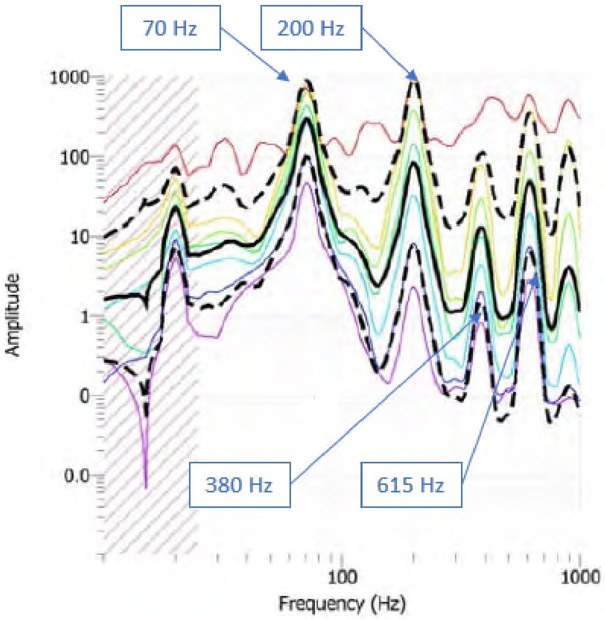

Fig. 8. P5-1 – Stroke recorded for pile P5 (P5-1 test configuration).

The frequency peak observed at 70 Hz should be related to the first theoretical modal frequency of a fixed beam (60 Hz). The observed difference is attributed to local modes of vibration of an open-ended tubular pile as discussed in the modal shape analysis section. Moreover, the small length over diameter ratios of piles P5 and P6 can explain differences with the fixed beam model.

The frequency peak observed at 380 Hz can be related to the theoretical frequency of mode 2. However, the peaks at 200 Hz and 615 Hz cannot be related to any theoretical modes.

Comparison of the 3 strokes, which correspond to 3 different stroke amplitudes (medium, low and hard) shows peaks occurring at the same frequencies. Moreover, Comparison between test configuration P5-1 and P5-3, which are related to different impact position, also shows the same frequency spectrum and modes.

Finally, the peaks of frequencies are consistent for piles P5 and P6 indicating that the driven and drilled and grouted methodology of installation did not affect the frequency of oscillations.

3.3 Damping analysis

The theoretical damping ratio of an oscillator can be calculated as per the logarithmic decrement method (Meirovitch, 1997) using the Equation 3 below:

$$\zeta = {}^{D}/_{2\pi} \tag{3}$$

With D the logarithmic decrement of a free oscillation.

The damping analysis was limited to oscillations of modes 1 and 2. To analyse the recorded data, the signal was band pass filtered at the identified frequency of modes 1 and 2 (i.e.: 70 Hz and 200 Hz respectively).

Analysis for the “stroke window” during test P5-1 resulted in a low logarithmic decrement D of 0.0059 and a damping ratio of 0.1 %. Analysis for the “stroke window” during P6-2 resulted in a logarithmic decrement D of 0.012 and a damping ratio of 0.2 %. The Figure 9 presents the amplitude decrement for mode 1 (70 Hz) recorded during the stroke window. Despite consistency of the measured damping for P5-1 and P6-2 stroke windows, the calculated values are surprisingly low compared to usual damping of steel closer to the 1% value.

Analysis performed on higher modes does not show coherent results.

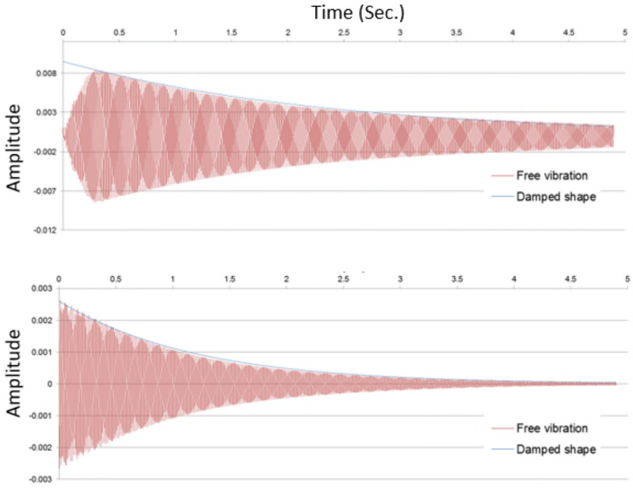

Fig. 9. P5-1 (top) and P6-2 (bottom) – Damping analysis for stroke window. Band pass filtered at 70 Hz, Mode 1

For the ambient window, the random decrement method is used to provide a damping ratio (Rodriguez, 2005). Analyses were performed for sensors 3 of tests P5-1 and P6-2. Figures 10 and 11 present the amplitude decrement for mode 1 (70 Hz) and mode 2 (200 Hz) recorded during the ambient window.

For the ambient window, a damping ratio of 0.97 % and 2.2 % was analysed for test P5-1 and P6-2 respectively for mode 1 at 70 Hz.

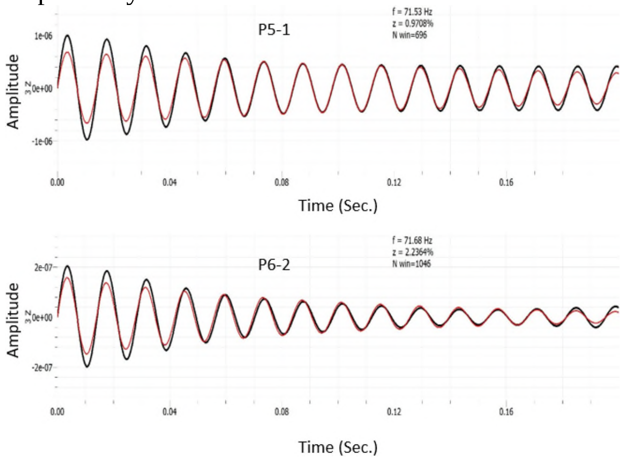

Fig. 10. P5-1 (top) and P6-2 (bottom) – Damping analysis for ambient window. Band pass filtered at 70 Hz, Mode 1

A damping ratio of 2.2 % and 1.3 % was analysed for test P5-1 and P6-2 respectively for mode 2 at 200 Hz.

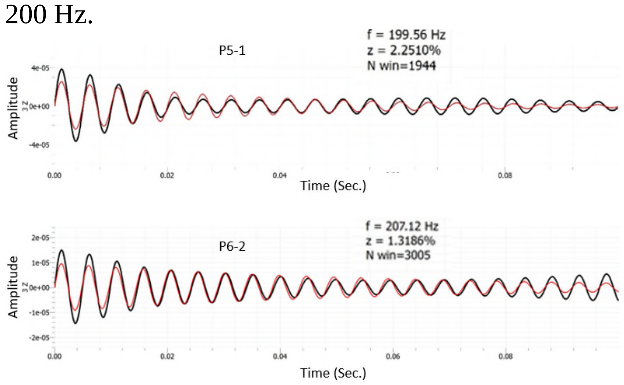

Fig. 11. P5-1 (top) and P6-2 (bottom) – Damping analysis for ambient window. Band pass filtered at 200 Hz, Mode 2

Damping ratios calculated during the stroke window are surprisingly low. Damping ratios for the ambient window are close to the one generally known for steel (around 1%). Considering a rigid connexion, the measured damping should not be representative of energy dissipation in soil connexion (which is rigid) but representative of damping of the material. However, in order to relate damping ratio to the rigidity of the soil-pile connexion, additional studies such as laboratory test on scaled piles of various soil-pile connexion rigidity are required.

3.4 Modal shape analysis

Analysis is performed with Artemis software (SViBS) with a Frequency Domain Decomposition analysis. Analysis in frequency is performed during the stroke (between 1 and 6 seconds of the stroke) and during ambient vibration towards the end of the record (between 15 and 30 seconds).

For mode 1 (70 Hz), the modal shape analysis shows that both stroke and ambient vibrations windows provides the same modal shape with a mix of first beam mode and local mode of cylinder.

For mode 2 (200 Hz), the modal analysis shows a local mode of cylinder. Both stroke and ambient vibrations windows provide the same modal shape.

For mode 3 (380 Hz), modal shape for the stroke vibration window shows a local mode of cylinder.

For mode 4 (615 Hz), modal shape for the stroke vibration window shows a mix of first beam mode and local mode of cylinder as for mode 1.

Based on the above, the theoretical first modal frequency of fixed beam (60 Hz) should be related to the first observed mode at 70 Hz with the difference related to the presence of local mode of cylinder. Higher theoretical modes of fixed beam don't fit to higher observed modes showing mainly local deformations.

Test configurations P6-1 and P6-2 assembled with common sensor 1 position shows modal shape consistent to test P5-1 and P5-2 for stroke vibration.

Figures 12 illustrates the modal shape analyses for pile P5 during stroke and ambient vibrations for modes 1 and 2.

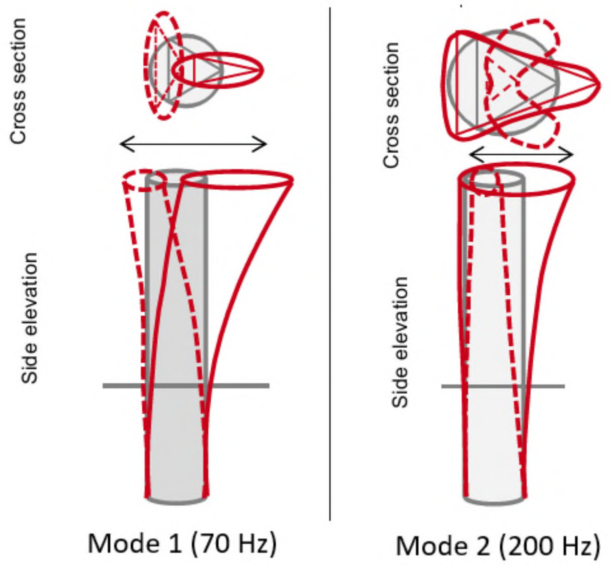

Fig. 12. Modal shape analyses of pile P5 for modes 1 and 2 during stroke data.

Figures 13 illustrates the modal shape analyses for pile P5 during and for stroke vibrations for mode 3 and 4.

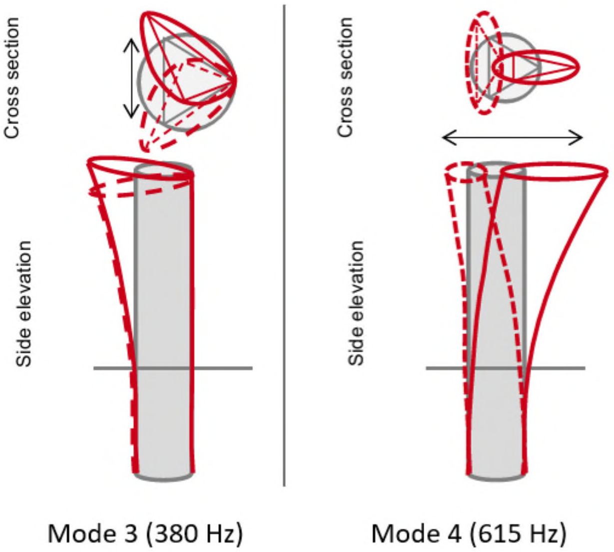

Fig. 13. Modal shape analyses of pile P5 for modes 3 and 4 during stroke data.

4. DISCUSSION AND CONCLUSIONS

This paper presents results of frequency, damping and modal shape analyses recorded on two 1.2 m diameter steel open ended test piles installed in calcarenite and excited in their free-standing length by a hammer stroke. A comparison is made with theoretical values calculated for a fixed beam. Results of the comparison allow, to some extent, for an evaluation of the rigidity of the soil-structure interaction. The following observations can be captured:

The frequency analysis identifies 4 modes of vibrations at 70 Hz, 200 Hz, 380 Hz and 615 Hz consistent for all test configuration both during the stroke vibrations window. The first mode (70 Hz) and second mode (200 Hz) are also identified in the ambient vibration window.

Damping analyses performed for mode 1 (70 Hz) and two (200 Hz) show a surprisingly low damping ratio (0.1 % to 0.2 %) during the stroke window a damping ratio (1% to 2%) close to the value for steel during the ambient window. However, no relation between damping ratio and rigidity of the embedment is proposed in this paper.

Modal shape analyses show fixed beam flexion mode coupled to local cylinder mode for mode 1 (70 Hz) and 4 (615 Hz); and local cylinder mode for mode 2 (200 Hz) and 3 (380 Hz).

Observed frequencies for mode 1 (70 Hz) is considered as representative of first theoretical mode of a fixed beam (60 Hz) with the observed difference attributed to local mode vibration of the cylinders. The soil-structure connexion of the foundation is therefore considered as close to a rigid connexion and similar for P5 and P6.

Ambient and stroke windows records show very good constancy in frequency with for modes 1 and 2. Higher modes are not observed with ambient vibrations due to lack of energy at this frequency range.

Considering a rigid connexion, the measured damping should not be representative of energy dissipation in soil connexion (which is rigid) but representative of damping of the material. Damping ratio measured during ambient vibration window correlate this conclusion.

Further analyses are required in order to quantify the rigidity of the soil-structure interface and relates this connexion to Geotechnical parameters of the Calcarenite.

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