Doctoral School on Engineering Sciences
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Extended summary

# Dynamic response of piles under lateral loading: full scale field test and numerical analyses

*Curriculum: Ingegneria delle Strutture e delle Infrastrutture*

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**Abstract**. In recent years, the question of soil-pile dynamic interaction has received a great deal of attention; modern codes suggest that it should be accounted for in the seismic design of pile foundations and superstructures. The stiffness and damping characteristics of the soil-pile system during earthquake motion depend on the mechanical properties and geometrical characteristics of the soil and piles as well as on their mutual interaction. Soil-pile dynamic interaction is also important in the study of effects due to the impact or the anchorage of vessels against quays with pile foundations. Experimental results of full scale in situ tests are an essential instrument to evaluate the dynamic characteristics of the soil-pile system in order to calibrate the theoretical methods and 3-D finite element models usually used to simulate the soil-pile interaction. Unfortunately, there is a limited database of measured pile performance during earthquakes. In this context, this work presents an extensive experimental programme of full scale field tests on dynamic laterally loaded piles, performed on a group of three steel pipe piles vibro-driven into soft marine clay. This campaign was aimed at detecting the complex dynamic soil-pile interaction taking into account the influence of the specific construction technique that can significantly affect soil properties close to the piles. The experimental programme consisted of two different test campaigns, one performed 1 week after pile vibro-driving and the other after 10 weeks. The dynamic behaviour of the complex soil-water-pile system at different strain levels was investigated by means of three different types of test: impact load test, forced vibration test and free vibration test (release test). Tests were then simulated with two different 3-D finite element models in ABAQUS, modelling the piles with shell or solid elements and calibrating the mechanical parameters on the experimental results. A 3-D model for the kinematic interaction analysis of pile groups, formulated by Dezi et al. (2009), was here specialized to simulate the tests of the experimental campaign.

**Keywords.** Dynamic test, experimental field test, pile foundation, soil-pile dynamic interaction

## 1 Problem statement and objectives

Deep foundations are particularly sensitive to SSI problems since their embedment in soil produces a strong interaction with the soil during seismic wave propagation. Historically, it has been common seismic design practice to ignore or simplify the influence of pile foundations on the ground motions applied to the structure. This has been generally accepted as a conservative design assumption for a spectral analysis approach, as the flexible pile foundation results in period lengthening and increased damping, and consequent decreased structural forces relative to a fixed base case. However, from observations after earthquakes, SSI induced failures reveal that the SSI effect is not always beneficial.

In order to determine soil-pile stiffness and damping under dynamic loads, researchers have conducted different classes of tests on full scale piles and pile groups in field and laboratory tests. In situ tests have the advantage of providing "correct" soil and pile stress conditions, whereas laboratory tests offer the flexibility and economy of making parametric studies in a controlled environment. Field and laboratory tests on the soil-pile interaction are both essential to provide a valuable database to calibrate numerical and analytical models for SSI analysis. Regarding field dynamic tests on piles, three different typologies have been used: the impact load test, free vibration and forced vibration.

General conclusions that can be drawn from observations and comments on tests carried out from 1965 until today (selected from the literature) are that:

- the soil-pile dynamic response is site, frequency and load level dependent;
- the soil-pile non-linear response decreases stiffness;
- pile group effects are frequency, pile spacing, and site dependent, and are more pronounced for stiffness and less for damping.

As described by Poulos and Davis (1980) and Fleming et al. (1992), there are three main approaches for the load-deflection prediction of laterally loaded piles.

The first method is called the *Beam on Elastic Foundation Approach*, where, accepting Winkler's foundation assumption that each layer of soil responds independently to adjacent layers, a beam and discrete spring system may be adopted to model pile lateral loading. In this method, the soil-pile contact is discretized to a number of points where combinations of springs and dashpots represent the soil-pile stiffness and damping at each particular layer.

The *elastic continuum approach* is based on Mindlin's (1936) closed-form solution for the horizontal displacement caused by a horizontal point load within the interior of a semi-infinite elastic-isotropic homogeneous mass. These solutions are generally known as Green's functions, and define the displacement field due to an assumed loading system (pattern) associated with the soil-pile interaction.

Finally, the *finite element method approach* potentially provides the most powerful means for conducting soil-pile-structure analyses, but it has not yet been fully developed as a practical tool. The advantages of a finite element approach include the capability of performing the soil-pile-structure analysis of pile groups in a fully-coupled manner, without resorting to independent calculations of site or superstructure response, or application of pile group interaction factors. It is of course possible to model any arbitrary soil profile, and to study the 3-D effects.

## 2 Research planning and activities

The study of soil-pile dynamic interaction was approached by means an experimental programme and its simulation with two different types of model.
The experimental programme was carried out in two different campaigns during August 2009 and October 2009. The test site is located in a lateral area of "Porto Lotti" at the tourist port "Mirabello" in La Spezia, Italy (Fig. 1). Several site explorations were carried out before the construction of the tourist port, investigating a large volume of soil by means of laboratory tests and in situ tests conducted up to a maximum depth of about 50 m. Figure 1 also shows the soil stratigraphy and the CPT performed nearby the test site while the main soil properties of each soil layer, derived from geotechnical investigations, and the evaluation of in situ and laboratory tests are presented in Table 1.

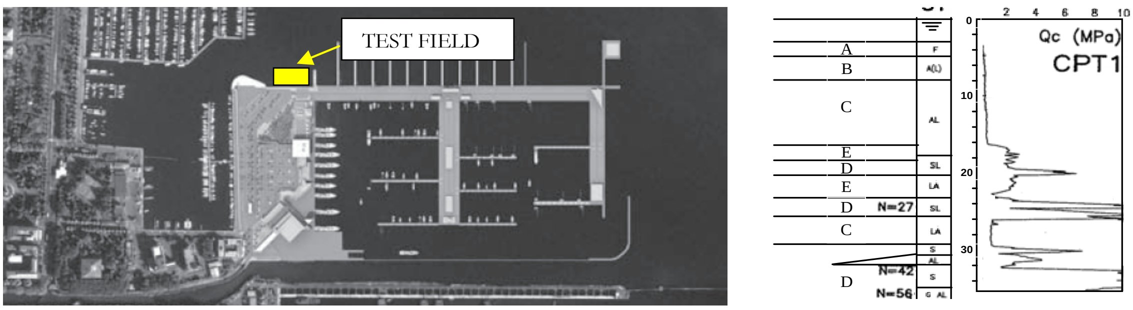


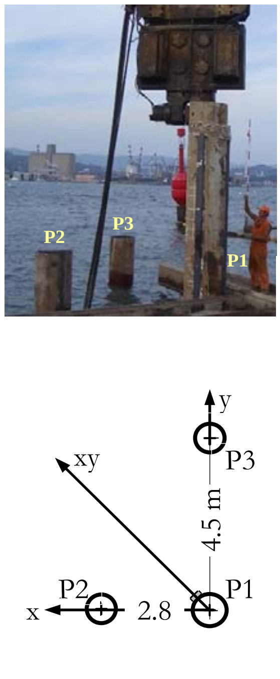

Figure 1. Test site and stratigraphy (with CPT graph)

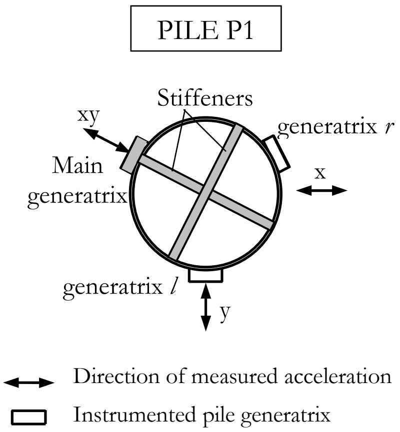
Table 1. Soil properties assigned to each soil layer

| Soil | Borehole | | | CPT | | DMT | | FV | |
|---|---|---|---|---|---|---|---|---|---|
| Type | $N_{SPT}$ | P.P. (MPa) | V.T. (MPa) | Qc (MPa) | Fs (MPa) | Cu (MPa) | OCR | Cu (MPa) | Cur (MPa) |
| A - mud and loose clayey silt | | | | 0.37 | 0.013 | 0.014 | | 0.020 | 0.005 |
| B - slightly silty clay | | | 0.022 | 0.45 | 0.014 | 0.019 | | 0.025 | 0.008 |
| C - slightly sandy, silty clay | | 0.068 | 0.044 | 0.82 | 0.018 | 0.040 | 3.00 | 0.047 | 0.012 |
| D - sand and dense silty sand | 40 | | | 7.90 | 0.100 | | | | |
| E - silty clay and clayey silt | | 0.120 | 0.050 | 2.25 | 0.120 | 0.082 | 5.00 | 0.108 | 0.022 |
| F - clayey sandy gravel | >50 | | | | | | | | |

The test field, set up at the end of July 2009, consisted of 3 steel pipe piles (Fig. 2a) with the geometric characteristics reported in Table 2. 15.5 m piles were vibro-driven for a depth of 9.5 m into the soft marine clay, with pile head elevation of 1.0 m from the mean sea level (Fig. 2c). An L -shaped arrangement ,in plan view, was set up (Fig. 2a) in order to study the interaction between piles in two directions for different pile spacing.
The following measuring instruments were used: a total of 19 strain gauges placed along three generatrices of pile P1, spaced by 120° (Fig. 2b-c) to capture the cross-section average strains (elongation and curvatures of the pipe): 11 strain gauges were located along the main generatrix and 4 along each of the two secondary generatrices l and r; narrower points were considered for the pile section where maximum curvatures were expected (Fig. 2c); a piezoresistive pressure transducer was located 2.5 m below the soil surface in a blind hole

on the surface of pile P1; a uniaxial accelerometer was located 0.3 m from the top of each pile and was conveniently moved to different places for the different tests.



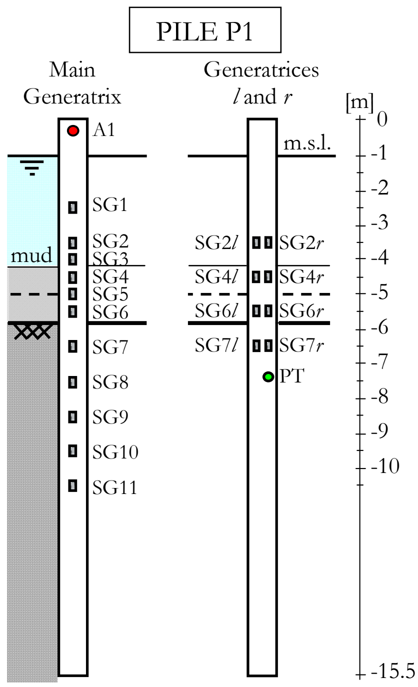




Table 2. Pile geometry

| Pile | Diameter [mm] | Thickness [mm] | Length [m] |
|---|---|---|---|
| P1 | 711 | 11 | 15.5 |
| P2, P3 | 609 | 10 | 15.5 |

a) b) c)

Figure 2. a) Test field; b) Pile P1 plan view and geometric characteristics of the piles; c) pile P1 instrumentation

Three different test types were carried out, the impact load test and forced vibration test during the first campaign and the impact load test (repeated) and free vibration test during the second campaign.

The *Impact load test* performed with an instrumented hammer is commonly used in dynamic testing due to its simplicity of execution. Furthermore, it allows the investigation of a wide range of frequencies with few impact loads. However, since a low amount of input energy is supplied to the system at each frequency, this test allows the system behaviour to be analysed only at very small strains and thus it is not suitable for investigating the non-linear behaviour of the system which generally occurs at higher strain levels. A maximum impact force of about 50 kN was reached with the hammer used in the tests (Fig. 3a). Several test configurations were considered, varying the direction of the impact and the direction of the measured accelerations.

The *free vibration test* allows stresses to be induced on the soil which are greater than those of the impact load test. In this type of test, the load is applied using a double acting jack with a capacity of 200 kN, placed between the pile and the quay and connected to the system by means of steel cables (Fig. 3b). The quick release of the load is achieved thanks to a "calibrated shear pin", placed along the steel cable between the jack and the pile. The tests were carried out for different load amplitude, gradually increased up to about 40 kN, and for two different configurations releasing pile P1 along the Y and XY direction.

Finally, the *forced vibration test* is the best technique to provide input with a high energy content and allows an accurate assessment of the characteristics of the system even for highly damped modes. The forced vibration test can be carried out in two different ways. In the stepped-sine test the command signal supplied to the exciter is a discrete sinusoid with a fixed amplitude and frequency. The sine sweep test, instead, involves the use of a sinusoidal command signal, with frequency varying slowly but continuously through the range of interest. In both methods, it is necessary to check that steady-state response conditions are attained before measurements are made. The vibrator used is not an experimental shaker but a commercial exciter usually adopted for driving and extracting piles and sheet-piles. The vibrator was placed horizontally, suspended by a system of steel cables (Fig. 3c), in order to excite the pile laterally. The tests were carried out for two different configurations exciting pile P2 in the X direction and then pile P1 in the XY direction.


Figure 3. a) Impact load test; b) free vibration test; c) forced vibration test

In this study, two different numerical models (Fig. 4) were used to simulate the soil-water-pile system and the pile-soil-pile dynamic interaction problems and to fit the experimental results of the impact load tests and the free vibration test at low strain.
The first model is a 3D finite element model created in ABAQUS, which consists of two parts, the pile domain and the soil domain. The soil domain is modelled by means of 8-node brick elements and 8-node infinite elements at the boundaries to avoid wave reflection. For the pile, two different models were used, 20-node brick elements to minimize the local effects and emphasize the bending modes, and 4-node shell elements to capture the contribution of the pile radial-circumferential modes and reproduce the experimental response of the loaded pile.
In the second model, proposed by Dezi et al. (2009), the piles are modelled as Euler-Bernoulli beams embedded in a layered Winkler-type medium. The pile–soil–pile interaction and the radiation problem are accounted for by means of elastodynamic Green's functions The model is adopted, with minor modifications to numerically simulate the test results.
In both models the hydrodynamic effects of the water surrounding the piles is taken into account by considering an added mass to the piles.

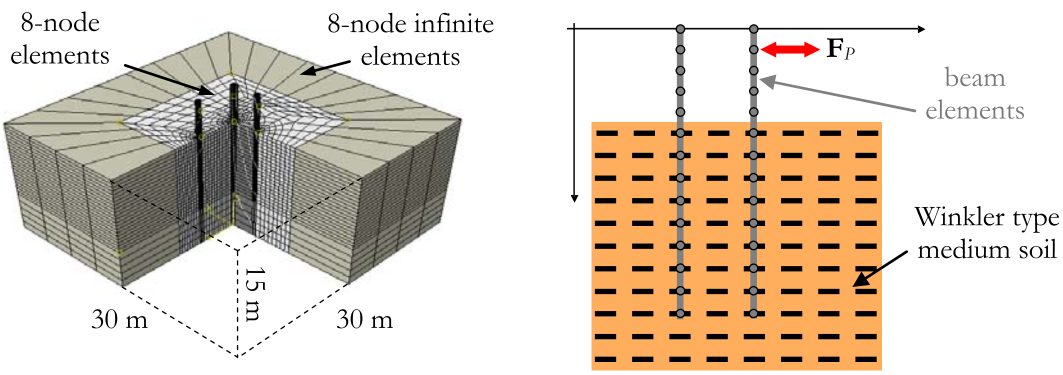



Figure 4. 3-D ABAQUS model on the left and model proposed by Dezi et al. (2009) on the right

## 3 Analysis and discussion of main results

With reference to the impact load test, Figure 5 shows the acceleration time histories measured at the pile heads. In particular, the raw signals and those filtered by a Butterworth low-pass filter with a cut-off frequency of 100 Hz are reported in the graphs on the left and on the right, respectively. Signals are filtered to nearly eliminate the effects due to cross-sectional deformation on loaded pile P1, and also the high frequency content on the receiver piles P2 and P3 due to wave propagation in the water. The acceleration at the head of pile P1 (A1x) is characterized by very high values, high frequency and damping due to the effects of the radial-circumferential modes which are particularly evident in the pile section near the hammer impact point.

With respect to the signal of the source pile, the acceleration time histories of the receiver piles are two order of magnitude smaller; they show an initial time delay and are both characterized by a first part with high frequency content followed by a second transitory segment, in which the maximum acceleration is achieved in pile P3, and a final damped part of the signal with oscillating frequency corresponding to the first bending mode of the soil-water-pile system. This may be attributed to different types of waves propagating from the source pile to the receiver piles. The first part is primarily due to propagation of compression and surface (Scholte) waves and thus their effects are more significant in pile P2; the peak values that are evident in the acceleration of pile P3 are due to the propagation of shear waves that, by propagating orthogonally to the impact direction, only a secondary effect on pile P2.

The L-shaped layout, characterized by different pile spacing (Fig. 6a), makes it possible to achieve an estimation of wave propagation velocity through the first layer of soil starting from the time delays of the recorded signals in different directions (Fig. 4b). The time delays (Fig. 4c) for test configurations in which the receiver piles are placed orthogonally to the direction of the load are calculated by means of the Cross Correlation Function (CCF). The difference in time delay is due to the different pile spacing and consists in the S-wave travel time through the portion of soil of length ($d_{13}$ - $d_{12}$). A shear wave velocity of about 55 m/s was estimated.

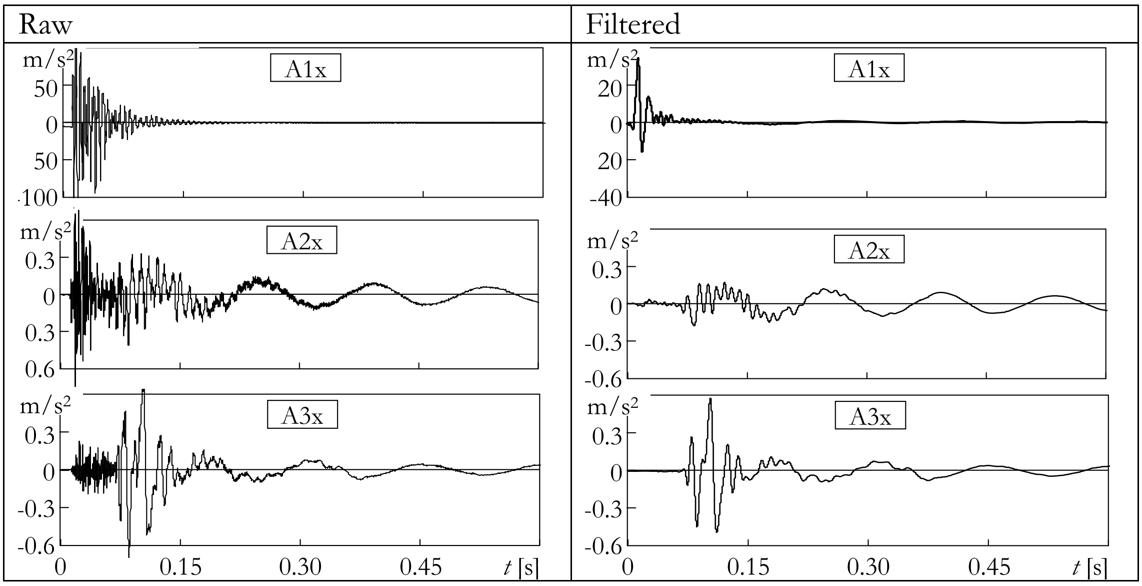



Figure 5. Hx-x-e test: accelerometer signals

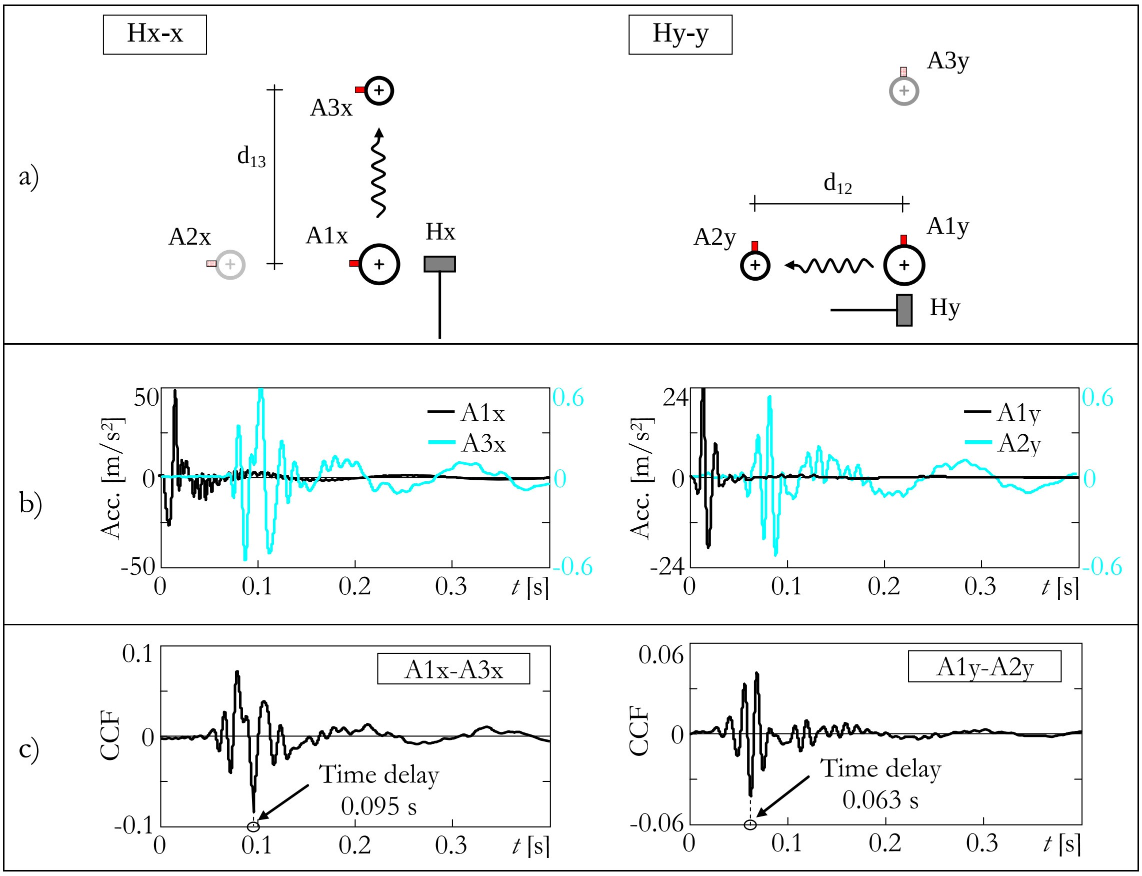



Figure 6. a) Test configurations; b) Time histories of acceleration on source and receiver piles; and c) Cross-Correlation Functions

Tables 3 and 4 list the values of the first and second natural frequencies in Hz obtained from strains along the pile excited in the different directions ($f_x$, $f_y$ and $f_{xy}$) and for the two campaigns. The discrepancy between the frequency values, in the different directions X, Y and XY, are mainly due to the UPN profiles that determine a dissymmetry of the pile cross-section. The values obtained from the test carried out in October are greater than those obtained in August due to soil reconsolidation subsequent to the pile vibro-driving.

Table 3. Natural frequencies obtained during the test carried out 1 week after the vibro-driving

| | | SG1 | SG 3 | SG 4 | SG 5 | SG 6 | SG 7 | SG 8 | SG 9 | SG 10 | SG 11 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| I mode | $f_x$ | 6.7 | 6.1 | 6.5 | 6.5 | 6.5 | 6.5 | 6.5 | 6.7 | 6 | 6.3 |
| | $f_y$ | 6.9 | 6.9 | 6.9 | 7 | 7 | 7 | 7 | 7 | 6.9 | 6.9 |
| | $f_{xy}$ | 6.9 | 7 | 7 | 7 | 7 | 7 | 7 | 7.1 | 7.1 | 7 |
| II mode | $f_x$ | 27.8 | 29.9 | 29.9 | 32.4 | 30.8 | 30.9 | 31 | 26.3 | 29.4 | 29.7 |
| | $f_y$ | 31.8 | 31.8 | 31.3 | 31.5 | 32 | 32.1 | 30 | 30.1 | 32.1 | 26.3 |
| | $f_{xy}$ | 31.8 | 31.1 | 30.1 | 30 | 31.3 | 31.3 | 30.8 | 31.3 | 26.8 | 26.7 |

Table 4. Natural frequencies obtained during the test carried out 10 weeks after the vibro-driving

| | | SG 1 | SG 3 | SG 4 | SG 5 | SG 6 | SG 7 | SG 8 | SG 9 | SG 10 | SG 11 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| I mode | $f_x$ | 7.2 | 6.3 | / | 6.2 | 6.7 | 6.7 | 6.9 | 6.7 | 7.1 | 6.4 |
| | $f_y$ | 7.3 | 7.3 | / | 7.4 | 7.4 | 7.4 | 7.4 | 7.5 | 7.5 | 7.5 |
| | $f_{xy}$ | 7.4 | 7.5 | / | 7.5 | 7.5 | 7.5 | 7.5 | 7.5 | 7.6 | 7.5 |
| II mode | $f_x$ | 32.1 | 32 | / | 32.3 | 32.1 | 30.2 | 30.3 | 30.3 | 28.6 | 28.8 |
| | $f_y$ | 31.8 | 31.7 | / | 31.7 | 31.4 | 31.4 | 30.6 | 29 | 30.8 | 29.5 |
| | $f_{xy}$ | 32.2 | 31.8 | / | 31.6 | 32.2 | 32 | 29.5 | 28.7 | 28 | 28 |

The damping ratio of the soil-water-pile system is evaluated, in the time domain, by means of the decrement logarithmic method. Table 5 shows the damping ratios relevant to the first natural frequency obtained from the strains along the pile excited in the different direction ($\xi_x$, $\xi_y$ and $\xi_{xy}$) and for the two campaigns; the values are very scattered.

Table 5. Damping ratios obtained from impact load tests

| | | SG 1 | SG 3 | SG 4 | SG 5 | SG 6 | SG 7 | SG 8 | SG 9 | SG 10 | SG 11 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 week | $\xi_x$ | 7.6 | 20.3 | 11.8 | 13.4 | 14.2 | 13.7 | 13.1 | 13.1 | 12.0 | 11 |
| | $\xi_y$ | 6.7 | 6.7 | 6.8 | 6.7 | 6.9 | 7.2 | 7.4 | 7.5 | 10.8 | 8.8 |
| | $\xi_{xy}$ | 6.4 | 7.0 | 7.5 | 7.4 | 7.6 | 7.9 | 8.3 | 8.6 | 8.9 | 10.1 |
| 10 weeks | $\xi_x$ | 5.3 | 5.7 | / | 6.4 | 6.1 | 6.8 | 6.8 | 7.2 | 5.1 | 6.5 |
| | $\xi_y$ | 6.5 | 6.3 | / | 6.4 | 6.5 | 6.8 | 7.0 | 7.1 | 10.2 | 8.8 |
| | $\xi_{xy}$ | 9.9 | 8 | / | 8 | 8.1 | 8.5 | 8.8 | 9.7 | 13 | 13.7 |

Figure 7 reports the values of natural frequency and the damping ratio in function of the load release obtained from the free vibration test. The natural frequency decreases as the load release increases, while the damping ratio increases. This is due to the non-linearity of the soil for high load release.

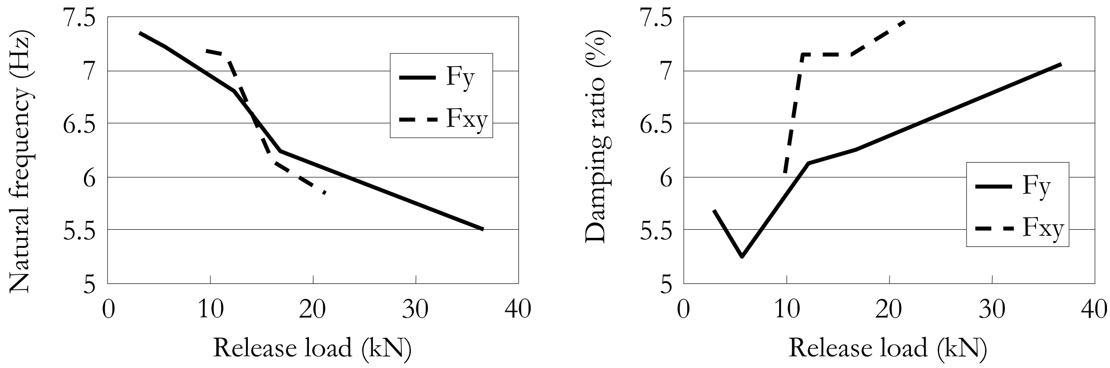



Figure 7. Natural frequency vs load release and damping ratio vs load release

Figure 8 shows the time histories recorded during the forced vibration test, in particular those relevant to sweep sine. During these tests a few problems occurred due to difficulty in vibrator control. Nevertheless, the time history clearly highlights the difference in the behaviour of the receiver piles, due to different pile spacing and incident wave types.

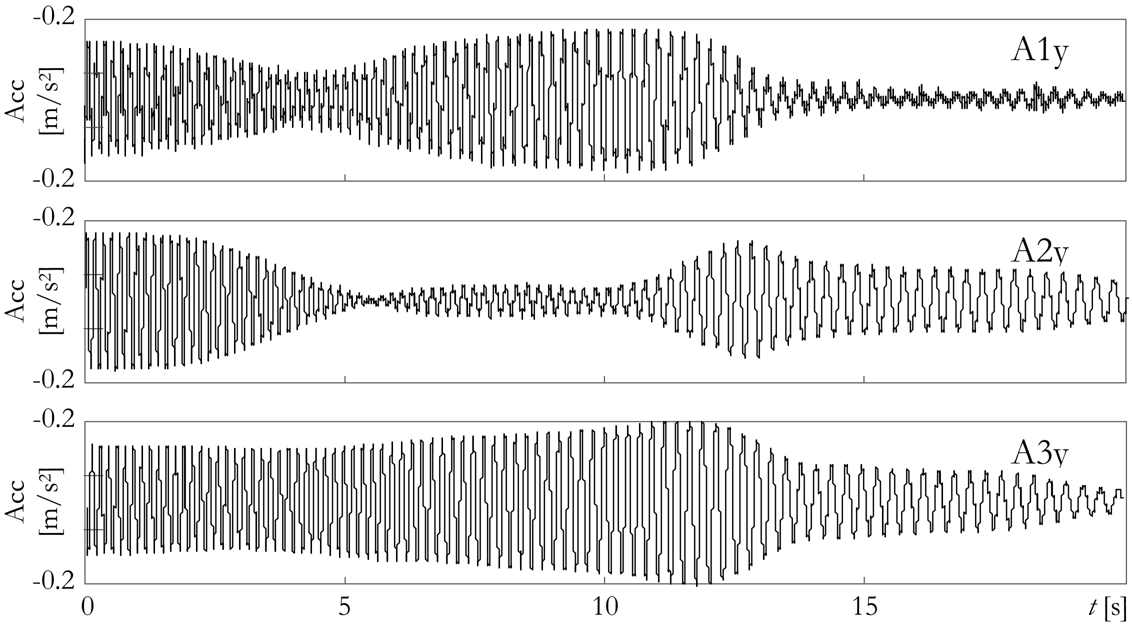



Figure 8. VAxy-y-ee.sweep test : accelerometer signals

In the following figures, some comparisons between the experimental and numerical results are shown. Figure 9 reports the time histories of strains of the impact load test and free vibration test (for low release load); both models show a good agreement with experimental results. Figure 10 reports the time histories of accelerations recorded at the head of each pile during the impact load test; the numerical models closely follow the experimental results in the oscillatory part, while less so in the transitory part.

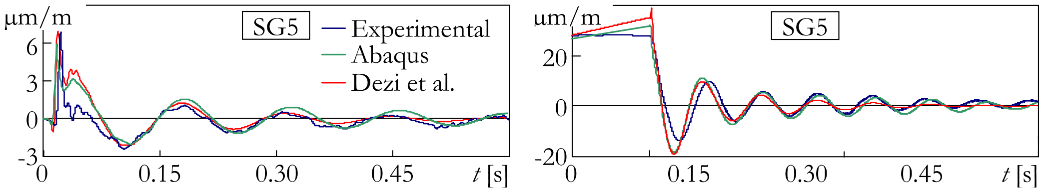


Figure 9. Time histories of strains

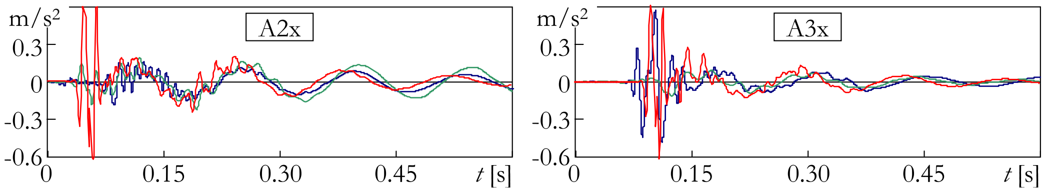


Figure 10. Time histories of accelerations on the receiver piles

## 4 Conclusions

The test results allowed us to highlight the different nature of the waves propagating in the soil medium, as induced by the inertial soil-pile interaction. In particular, based on the results, the following useful information can be derived: a) the shear wave-propagation velocity, estimated by means of cross-correlation methods, b) the dynamic properties of the soil-water-pile system at low strain, by means of the impact load test, and at higher strains, by means of the free-vibration test.
The test results were compared with those obtained from two numerical models developed in ABAQUS and with the method formulated by Dezi et al.. The results in terms of response of the soil-water-pile system, are in good agreement with the experimental results, although some minor discrepancies were observed in the simulation of the pile-soil-pile interaction effects.
Future development will be undertaken in two different ways: the development of a standard methodology for a dynamic test on laterally loaded piles and the use of non-linear models for soil in order to simulate the free vibration test (at high strain) and the forced vibration test.

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