SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING
AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

NONLINEAR DYNAMIC RESPONSE OF NEAR-SHORE PILES SUBJECTED TO SNAP-BACK TESTING

Francesca DEZI[1], Fabrizio GARA[2] and Davide ROIA[3]

ABSTRACT

This paper presents the results of free vibration tests carried out on a near-shore steel pipe pile vibrodriven into soft marine clay. The tests were carried out by quickly releasing a free head pile from a deformed configuration obtained by applying a quasi-static horizontal loading by means of a hydraulic jack. Different tests were performed at increasing loading levels with the aim of investigating the dynamic soil-water-pile system interaction and investigating how the nonlinear dynamic behaviour develops as the applied load increases. The results of experimental modal analyses, based on the signals recorded by strain gauges applied along the pile, are presented in terms of natural frequencies and damping ratios of the soil-water-pile system and the variations with loading level, due to the nonlinear behaviour of the system, are discussed. The dynamic effects induced on two pipe piles vibrodriven near the loaded pile to form an L-shaped in plan configuration are also discussed on the basis of signals measured by accelerometers applied at the head of the receiver piles; in order to investigate the pile-to-pile dynamic interaction.

Keywords: free vibration test, nonlinear behaviour, pile foundation, pile-to-pile dynamic interaction, soil-pile dynamic interaction.

INTRODUCTION

The performance of a large class of structures which are usually founded on single pile or pile group (e.g. offshore and near-shore platforms, wind turbines, docks, jetties, wharfs, mooring structures and bridge piers) under dynamic lateral excitation strongly depends on complex mechanism regulating the dynamic soil-pile and pile-to-pile interaction. Significant dynamic effects can occur in these structures, as consequence of wind, wave, boat mooring, boat impact forces and also earthquakes, and may lead to early damages and failures.

Dynamic soil-pile interaction problems, especially in the fields of earthquake engineering, have been extensively studied by many researchers by following numerical and analytical approaches, and a large number of theoretical studies have been published. Less works concern experimental tests and even less in-situ real scale tests. Among these, free vibration tests were carried out some decades ago [1,2] and other were performed in the last years [3-5]. This technique is particularly adapt to investigate how the response changes as the input action increases, due to the development of nonlinear phenomena (e.g. soil deformation effects and nonlinear geometrical effects) in soil-pile systems [6].

This paper describes the results of free vibration tests carried out at the “Mirabello” harbour in

---

[1] Assistant Professor, DET, University of San Marino, Rep. of San Marino, e-mail: francesca.dezi@tin.it
[2] Assistant Professor, DICEA, Università Politecnica delle Marche, Italy, e-mail: f.gara@univpm.it
[3] Post Graduate, DICEA, Università Politecnica delle Marche, Italy, e-mail: d.roia@univpm.it

La Spezia (Italy) on 3 near-shore steel pipe piles vibro-driven for a depth of 9.5 m into soft marine clay. The piles were instrumented and subjected to dynamic lateral excitation due to snap-back at increasing loading levels and quick release, with the aim to study the dynamic response of the system. In particular both the response of the single pile and the pile system have been analyzed with the objective of studying the dynamic soil-water-pile system interaction and the pile-to-pile interaction and investigating how the nonlinear dynamic behaviour of the system develops as the applied load increases. The response is analyzed both in time and frequency domain; the modal properties of the soil-water-pile system are identified by means of experimental modal analyses and the values of natural frequencies are estimated from the peaks of Frequency Response Functions (FRFs), whereas the mean values of damping ratios of the system are evaluated by means of the logarithmic decrement and the half power bandwidth methods. The results of the experimental modal analyses are presented, first with regard to the loaded pile (soil-water-pile system) by analysing the measurements of the strain gauges installed along the pulled pile, both in the portion immersed in water and in the portion embedded in soil, then with regard to the pile system (pile-to-pile interaction) by analysing the measurements of the accelerometers placed at the head of the receiver piles. In both cases the effects of nonlinearities developing in the soil for increasing force levels of free vibration test are commented.

2 SITE AND SETUP OF TESTS

In this section a short description of the site characterization, test field and pile instrumentation is reported; for more details readers may refer to [7, 8].

The test site is located in the tourist port “Mirabello” in La Spezia (North-West Italy), where a foundation of about 500 vibro-driven steel pipe piles, with diameters ranging between 609-711 mm, and embedded length from 20 to 45 m, was realized for a new sector of the harbour. Soil stratigraphy and mechanical properties are obtained from geotechnical investigations (laboratory and in-situ) conducted up to a maximum depth of about 50 m during several site explorations (1973, 1981, 1991, 1992 and 2008), are available. The soil stratigraphy and a CPT profile representative of soil conditions in the proximity of the test site is reported in Figure 1whereas the main soil properties of each soil layer, derived by the geotechnical investigations, are presented in Table 1. Based on these results the soil strength profile can be reasonably considered as uniform over the pile embedment length.

A	mud and loose clayey silt
B	normal consolidated slightly silty clay
C	slightly sandy, silty clay
E	overconsolidated silty clay and clayey silt
D	sand and dense silty sand with heterogeneous coarse lenses
E	overconsolidated silty clay and clayey silt
D	sand and dense silty sand with heterogeneous coarse lenses
C	slightly sandy, silty clay
D	sand and dense silty sand with heterogeneous coarse lenses

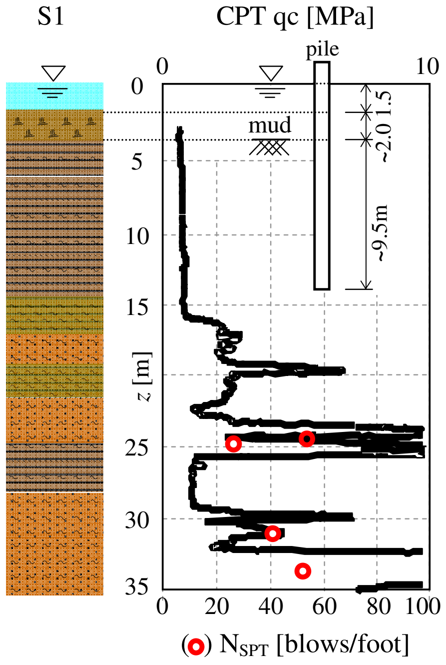


Figure 1. Subsoil profile

Table 1. Geotechnical parameters

Soil	Borehole		CPT		DMT		FVT	
type	P.P. [kPa]	V.T. [kPa]	qc [MPa]	fs[kPa]	Cu [kPa]	OCR	cu [kPa]	cur [kPa]
A			0.37	13	14		20	5
B		22	0.45	14	19		25	8
C	68	44	0.82	18	40	3.00	47	12
D			7.90	100				
E	120	50	2.25	120	82	5.00	108	22

P.P. := resistance to penetration measured by pocket penetrometer, V.T. = shear strength measured by pocket scissometer, cur = undrained residual shear strength, qc = cone resistance, fs = sleeve friction, Cu = undrained shear strength, OCR = over consolidation ratio, cur = undrained residual shear strength.

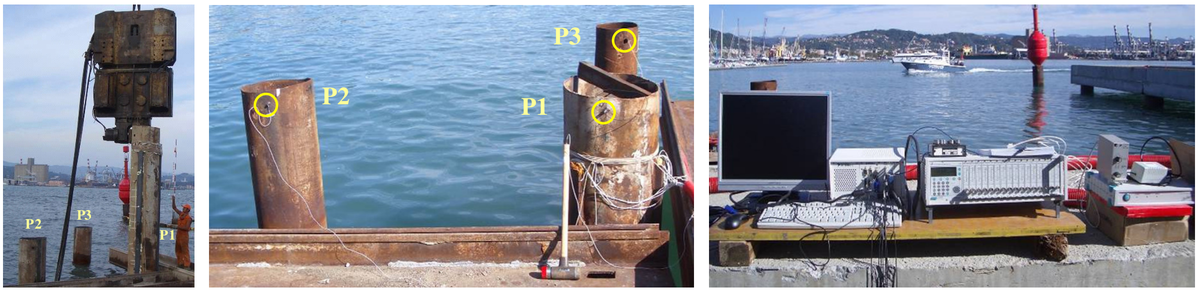
The test field consists of 3 steel pipe piles vibro-driven (Figures 2a and b) for a depth of 9.5 m into soft marine clay. Piles are 15.5 m long and the pile head elevation is 1.0 m above mean sea level (m.s.l.) (Figure 3a). Pile P1 has a diameter of 711 mm and thickness of 11 mm whereas piles P2 and P3 have a diameter of 609 mm and thickness of 10 mm; the head of the piles is kept free and the head of pile P1 is stiffened with two steel profiles, welded in a crux shape. The piles are arranged in an “L” shaped horizontal layout characterized by different distances between pile P1, located at the “L” corner, and piles P2 and P3 (Figure 3b).

As regards the instrumentation, a total of 19 strain gauges (SG) are used to measure the longitudinal strains along the pile P1. Strain gauges are placed along three generatrices of the pile (Figures 3a and c), spaced 120 degrees apart, to capture the cross section average strains (elongation and curvature of the pipe), i.e. 11 strain gauges are located along the main generatrix and 4 along each of the two secondary generatrices *l* and *r*. Narrower measure points are considered for the pile section where maximum curvatures are expected (Figure 3a). Furthermore, to measure the horizontal acceleration at the head of the two receiver piles uniaxial piezoelectric accelerometers (A) are used. Accelerometers (circled in yellow in Figure 2b) are located at 0.3 m from the top of the receiver piles: two accelerometers on the pile P2 along *x* and *y*-direction and another one on the pile P3 along *y*. The measurement chain also includes amplifiers, signal conditioners, one spectrum analyzer, two data acquisition systems, and a computer with dedicated software (Figure 2c).

3 IN SITU TESTS

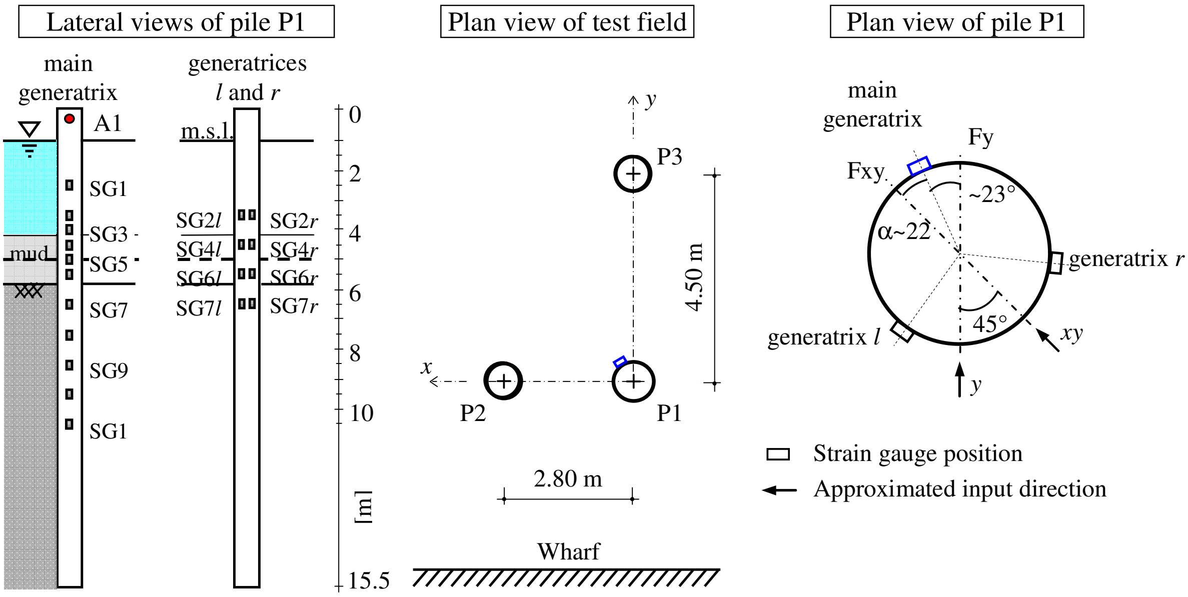
3.1. Test description

The free vibration tests allow investigating the dynamic behaviour of the system at different strain levels, by simply varying the level of force which is usually imposed with a standard hydraulic actuator. For test described in this paper the load was applied using a double acting jack with a capacity of about 400 kN, placed between the pile and the quay and connected to the pile P1 by means of steel cables. The quick release of the load was achieved thanks to the failure for traction of a steel pin placed along the steel cable between jack and pile (Figure 4a). The cross section of the pin was opportunely calibrated to achieve the desired load level. After the pin failure, the pile undergoes a number of steadily decreasing oscillations around its equilibrium position.



(a) (b) (c)

Figure 2. (a) Test field; (b) pile layout, accelerometers and instrumented hammer; (c) measuring chain



(a) (b) (c)

Figure 3. (a) Pile P1 instrumentation; (b) plan view of test field; (c) plan view of pile P1

An analogical pressure transducer was used to sense the hydraulic oil pressure in the pump actuating the jack. Unfortunately, due to the low sensitivity of the analogical transducer and to the friction between ram and cylinder (amplified by environmental conditions), the adopted measuring system was affected by raw approximation. However, a more precise estimation of the actual loading exerted by the jack has been derived from the values recorded by two strain gauges (ε_{SGi} and ε_{SGj}) located in the portion of the pile free-standing above the ground surface where the bending moment (and therefore the longitudinal strain) varies linearly with depth. The actual applied load is obtained from the following formula

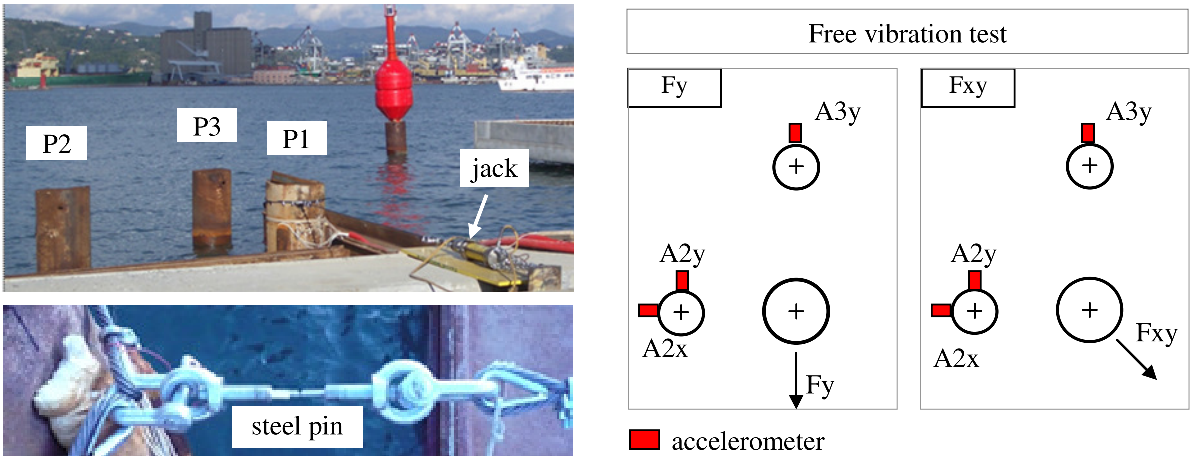
$$F = \frac{EW}{\Delta h \cos\alpha}\left(\varepsilon_{SGj} - \varepsilon_{SGi}\right) \tag{1}$$

where E is the elastic modulus, W is the section modulus, α is the angle between the direction of the load F and the line connecting the main generatrix to the centre of the cross section and Δh is the distance between SGi and SGj. The angle α (i.e. the relative direction of the input respect to the sensors position) is obtained from the strains measured by SG, SGl, and SGr, located at the depth h respect to the loading point, according to the following formula

$$\alpha = \arctan\left[\frac{\left(\varepsilon_{SGr} - \varepsilon_{SGl}\right)}{\sqrt{3}\,\varepsilon_{SG}}\right] \tag{2}$$

3.2. Test configurations

The free vibration tests were carried out according to two different configurations shown in Figure 4b, Fy and Fxy tests concerning the release of the pile P1 along y- and xy-direction, respectively. It should be pointed out that test along x-direction was not carried out because it was impossible to set up the traction system on the wharf. A time acquisition of 4 s, including a pre trigger of 2 s, and a sampling rate of 5 kHz were used. Tests were repeated for different load levels: 5 tests in y-direction and 4 tests in xy-direction for which different pins opportunely calibrated were used. In Table 2 the values of the traction force applied to the pile head calculated both from the measurements of the analogical pressure transducer applied to the jack and by means of equation (1) are reported for each test.



(a) (b)

Figure 4. (a) Test field and particular of the calibrated pin; (b) test configurations

Table 2. Free vibration tests: measured and calculated forces

	Fy-1	Fy-2	Fy-3	Fy-4	Fy-5	Fxy-1	Fxy-2	Fxy-3	Fxy-4
Force measured [kN]	3.0	5.5	12.1	16.5	35.9	9.4	11.3	15.7	20.8
Force calculated from SGs [kN]	2.8	6.9	18.0	24.0	58.1	2.7	6.7	16.5	26.6

4. SOIL-PILE INTERACTION

In this section the results of free vibration tests performed at the lowest force level are presented in order to discuss the dynamic behaviour of the soil-water-pile system, first with reference to the tests at the lowest force levels Fy-1 and Fxy-1 (behaviour at small strain) and then considering the tests at higher force levels. Besides the measured strains along the pile, the modal properties of the soil-pile system in terms of frequencies and damping ratios will be discussed.

In Figure 5 the time histories of the longitudinal strains recorded along the pile P1 by SG1, SG3, SG5, SG7, SG9, and SG11, for test Fy-1 are reported. The time histories show nearly constant values before the release, due to the quasi-static manner of the loading. After the quick release, free damped oscillations of the pile are manifest at nearly the first natural frequency of the soil-water-pile system (which is clearly identified by the higher peak of the Fourier Response Function that are shown in Figure 6). From a qualitative point of view, a part from the amplitude, the graphs are very similar and the damped harmonic oscillations are almost proportional. The maximum values of strain amplitude, proportional to pile bending moment, are attained in the pile section located just below the soil surface, measured by SG7.

The modal properties of the soil-water-pile system, such as natural frequencies and damping ratios, are obtained by means of experimental modal analyses using the measured excitation applied to the pile head and the system response measured by strain gauges at various locations. Both the excitation and response time histories are transformed into the frequency domain to define Frequency Response Functions (FRFs), i.e. the Fourier transforms of the response measurements normalized by the Fourier transform of the input. Natural frequencies are obtained by means of the peak picking method, selecting the frequencies corresponding to the peak values of the FRF amplitude.

As an example, the FRFs relevant to SG5 for test Fy-1 and Fxy-1 are reported in Figure 6. Only one peak, defining the first natural frequency, is clearly defined at 7.4 Hz and 7.3 Hz respectively, whereas the peaks relevant to the higher natural frequencies are not evident due to the fact that the pile is released from a deformed shape close to the first mode shape and, consequently, superior modes are only slightly excited.

The damping ratios of the soil-water-pile system at small strain are estimated from the strain gauge signals by means of the logarithmic decrement, working in time domain. The procedure is applied fitting the first eight peaks to obtain a mean value representing the damping of the system during almost the entire oscillation of the pile. The damping has been also estimated by fitting a lower number of peaks, e.g. the first four peaks or the second four peaks: the results, not reported here for brevity, are obviously more scattered but the mean value practically agrees with that obtained by fitting eight peaks. This confirms that the system behaves in a linear manner at these force levels and justifies the choice to consider eight peaks.

For each strain gauge of the main generatrix, Figure 7 shows the values of the first natural frequency (Figure 7a) and damping ratio (Figure 7b), obtained by considering the measurements (opportunely filtered) of the tests Fy-1 and Fxy-1, with black and blue lines, respectively. Some interesting information can be drawn by comparing results obtained for the two directions. As regards the frequencies, slight differences are evident among the values obtained from test Fy-1 in *y*-direction (mean value equal to 7.47 Hz) and test Fxy-1 in *xy*-direction (mean value equal to 7.31 Hz). It is worth noting that, since the test Fxy-1 was performed after the series of Fy tests, the system had experienced nonlinear behaviour (increasing the free length of the pile) and some modifications might still affect the system when tests Fxy started.

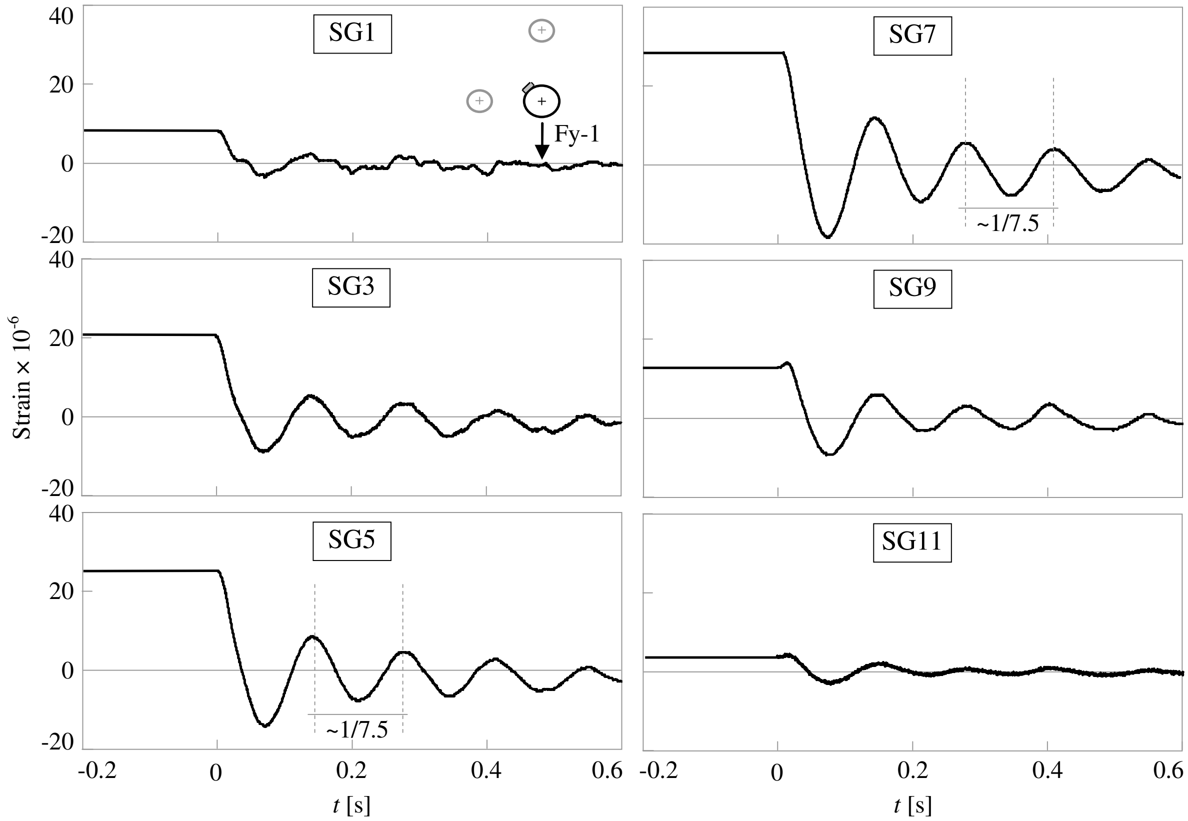


Figure 5. Time histories of SGs signals relevant to the test Fy-1

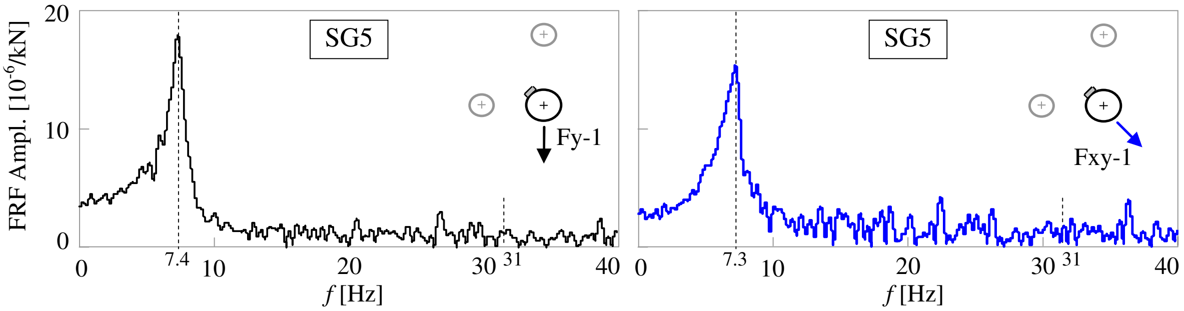
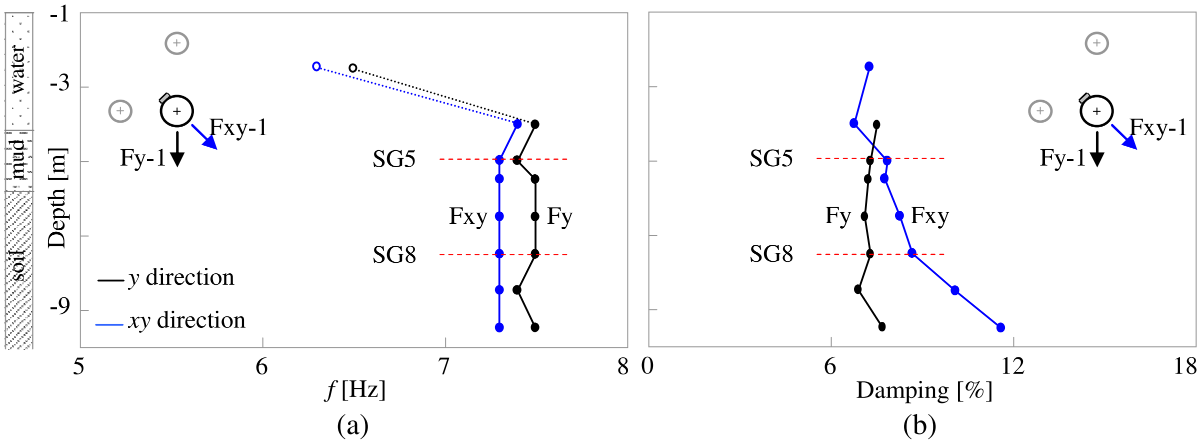


Figure 6. FRFs of SG5 signals relevant to the tests Fy-1 and Fxy-1



(a) (b)

Figure 7. First natural frequencies and damping ratios evaluated from SGs along the pile for tests Fy-1 and Fxy-1

Regarding to the damping ratios, the mean values, averaged among values obtained from all the SGs, are quite different for the two different directions (7.3 % and 8.7 % for tests Fy-1 and Fxy-1, respectively) most likely due to the modification of the soil induced by the series of tests Fy. However, it can be noted that the values of damping ratio tend to increase with depth; this trend, more evident from the results of test Fxy, can be due to the higher effects of soil radiation damping at deeper levels.

To study the dynamic behaviour of the soil-water-pile system at higher strain and to investigate the effects of soil nonlinearities, free vibration tests were performed at higher load levels, with a maximum force (58.1 kN) about 20 times the minimum one (2.8 kN), as reported in Table 2. In this case, being expected a non linear behaviour of the soil-pile system at higher force levels, the natural frequencies are evaluated from peaks of the FRFs (peak picking method) and the damping ratios are estimated with the half power bandwidth method, working in the frequency domain. These methods furnish mean values of frequency and damping for each single acquisition, which are representative of the entire oscillation phenomenon. For non linear systems, other methods like crossing time or logarithmic decrement, respectively for natural frequencies and damping ratios, furnish different values depending on the length and the part (initial or final) of the signal considered for the estimation procedure; these values cannot be properly used here because tests at different force levels are compared. Figure 8 shows the strains (and bending moments that are proportional to strains) measured along the pile by SGs for different loads just before the quick release. The continuous line is obtained, for each level of force, by interpolating the experimental data (reported with dots) and allows obtaining an estimation of the maximum value of strain attained along the pile and its location (Table 3). The red dashed line connects points where the maximum strains are attained permitting to observe a clear lowering of these points as the load level increases. This can be due to the formation of gap at the soil-pile interface and the consequent increase of the free length of the pile. Analogous considerations can be drawn from Figure 9 where the SGs strains along the pile normalized by the SG3 strain are shown.

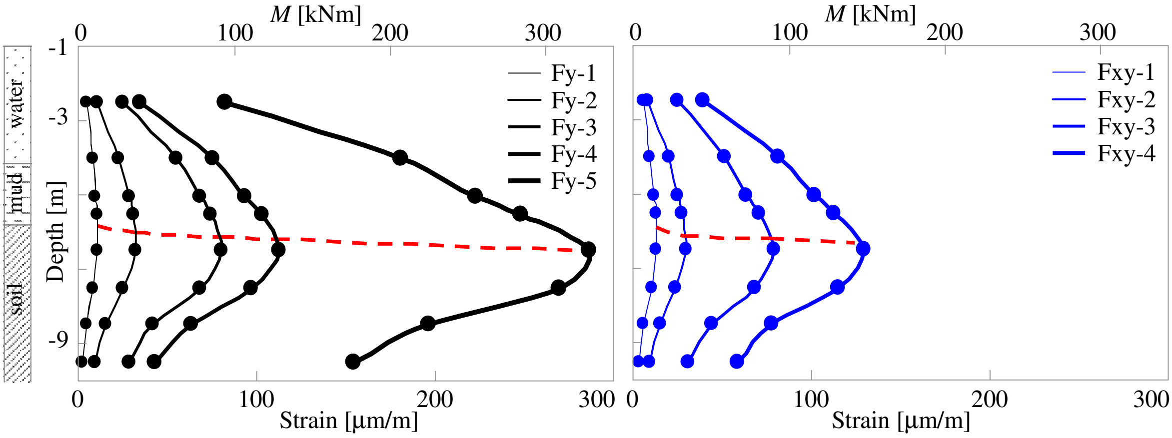
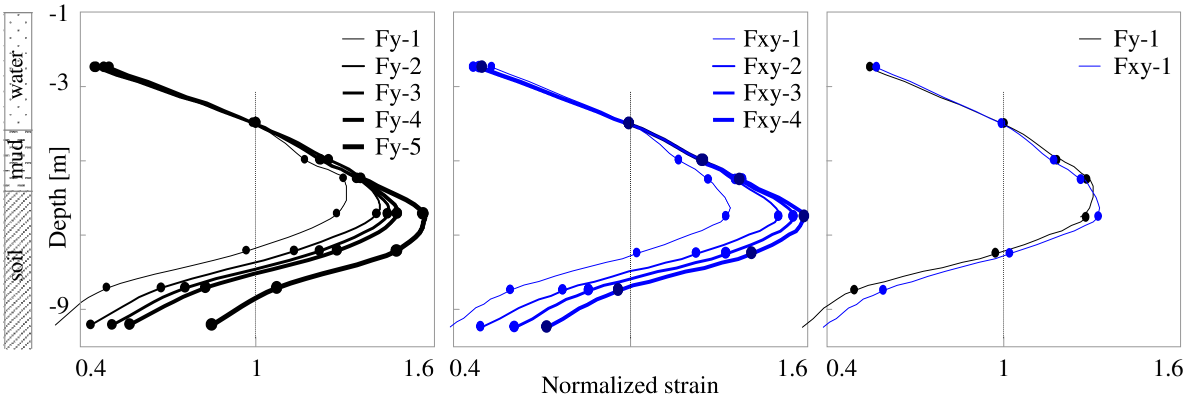


Figure 8. Strains and bending moments along the pile for different load levels along *y*- and *xy*-direction



(a) (b) (c)

Figure 9. Normalized strains along the pile; comparison among tests: (a) at increasing force levels in *y*-direction; (b) at increasing force levels in *xy*-direction; (c) at the lowest force level in *y*- and *xy*-direction

Table 3. Free vibration tests: applied forces, maximum strains and their locations along the pile

Free vibration test	Fy-1	Fy-2	Fy-3	Fy-4	Fy-5	Fxy-1	Fxy-2	Fxy-3	Fxy-4
Force calculated from SGs [kN]	2.8	6.9	18.0	24.0	58.1	2.7	6.7	16.5	26.6
Maximum pile strain [μm/m]	11.3	32.2	79.4	110.3	278.1	13.2	29.1	76.8	124.7
Location of maximum strain [m]	-5.8	-6	-6.1	-6.2	-6.5	-5.9	-6.1	-6.2	-6.3

As expected, the same normalized values are attained by SGs located on the pile portion above the ground, due to the linear elastic behaviour of this portion of the system. On the contrary, higher normalized values are attained by SGs located in the embedded pile portion for tests at higher force levels; higher normalized deformations (and, therefore, displacements) in a larger portion of soil confirm the developing of nonlinear phenomena in the soil, increasing with the level of input force. Same considerations may be drawn from Figure 9b that shows the normalized strains attained at different force levels during test in *xy* direction. However, a difference can be noted among the tests in *y*- and *xy*-direction. As shown in Figure 9c, where the normalized strains of the two tests at the lowest force level (Fy-1 and Fxy-1) are compared, the values in the elastic portion of the pile (above the ground) are practically coincident, while the values along the embedded portion are greater for test in *xy*-direction than in *y*-direction. This difference can be again attributed to the soil modifications produced by the Fy tests, performed just before the Fxy-1 ones, during which the system experienced nonlinear behaviour.

Figure 10 shows the measured pile-head horizontal Frequency Response Functions (FRFs), obtained from the ratio between the complex spectrum of pile response in terms of strains along the pile and the complex spectrum of the load applied at the pile head. From graphs of Figure 10 relevant to tests in *y*-direction three observations can be done. (*i*) The resonance frequency gradually decreases as the load level increases, from initial test Fy-1 to the final one Fy-5. This is due to a reduction of the stiffness of the soil-pile system which can be attributed to a nonlinear behaviour of the soil surrounding the pile (likely formation of soil-pile gap) that produces a variation of the distribution of the soil reaction along the pile; the resultant of soil reactions is thus located at deeper position and, consequently, the free length of the pile increases while the embedded length decreases, making the system more flexible and reducing the first resonance frequency. This confirms the comments already presented on Figure 8. However, it is worth noting that the gap could not be experimentally observed because of the presence of the sea water. (*ii*) The peak width increases as the load level increases, as a consequence of the increase of hysteretic damping with load. (*iii*) The static value of FRF (amplitude at 0 Hz) must be constant for linear systems. For signals from SGs located along the free length of the pile (pile section above the ground) the static values of FRFs remain constant with loading as can be observed from the graph relevant to SG5, whereas the static value of FRFs relevant to SGs placed in the embedded portion of the pile, e.g SG8, increases with loading; this can be attributed to the non linear behaviour of soil.

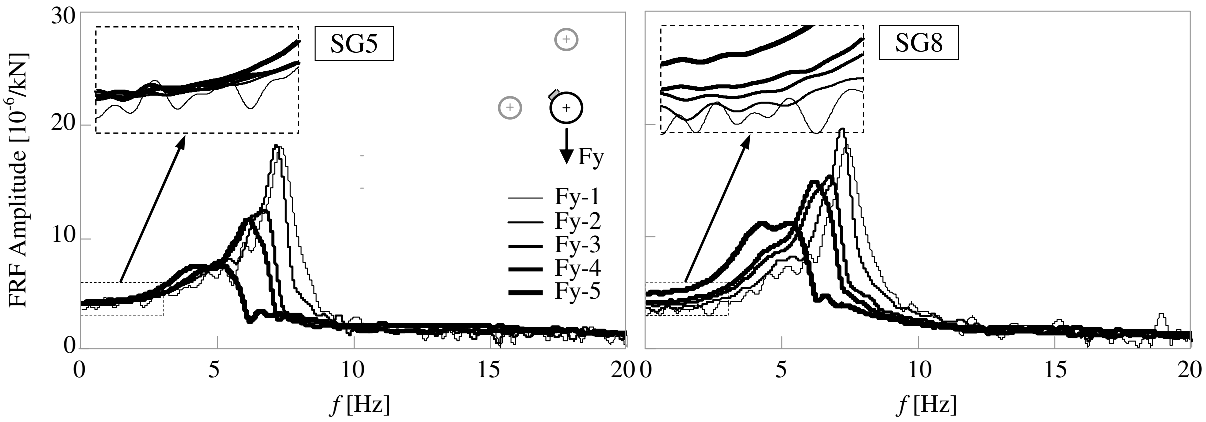


Figure 10. FRFs of SG5 and SG8 obtained from tests along *y*-direction

Furthermore, it can be noticed that the amplitude of the FRF peaks of all SGs generally decrease as the loads increase. This effect can be due to a higher damping which causes the pile oscillation (at about the first natural frequency of the system) to be much more rapidly decreasing even if starting from a higher value of displacement due to soil plasticization.

Figure 11 shows the values of the first natural frequency and damping ratio obtained at different load levels from SGs along the pile, for tests in *y*-direction (left graphs) and *xy*-direction (right graphs). For each test, the frequency values obtained from different SGs are in good agreement, excepting for SG1 values which in several cases sensibly differ from the others. Being SG1 close to the load point, this behaviour is probably caused by the whiplash effect due to the quick release. It is worth noting that the mean values of the fundamental natural frequency decrease with increasing loads (graphs at the top of Figure 11), as a consequence of the nonlinear behaviour of the soil (e.g soil plasticization, soil-pile gap formation, soil cave-in) that increases the flexibility of the soil-pile system. The graphs at the bottom of Figure 11 show the damping ratios obtained by signals of SGs along the pile. The increase with load of the damping ratios confirms a pronounced nonlinear behaviour of the soil. Furthermore, it is interesting noting that higher damping ratios are obtained from SG located at deeper level, due to the higher contribution given by the soil to the damping of the soil-pile system. The same considerations hold for both tests along *y*- and *xy*-direction.

Figure 12 shows the mean values and standard deviations of the first natural frequencies and damping ratios obtained from free vibration tests at different loading levels; these values are obtained considering all SGs of the main generatrix, except SG1 which furnished not fully reliable values in several tests, as already mentioned. With increasing loads the decrease of frequencies and increase of damping ratios, due to soil nonlinearities, are particularly evident from these graphs both for tests in *y*- and *xy*-direction (left and right graphs of Figure 12, respectively). Furthermore, tests performed in *xy*-direction, respect to those in *y*-direction, show lower frequencies and higher damping ratios. This difference, as previously observed, is consequence of the soil modifications produced by the tests in *y*-direction. It is also worth noting that the standard variation of the frequencies is very small while, as expected, that of damping ratios is larger but still acceptable considering the uncertainties usually involving this parameter.

5. PILE-TO-PILE INTERACTION

The pile-soil-pile dynamic interaction is investigated by analyzing the recorded accelerations at the head of each pile (source pile P1 and receiver piles P2 and P3). As an example, the time histories of accelerations recorded on the receiver piles P2 and P3 during the first test Fy-1, at the lowest load level, are shown in Figure 13. Raw and filtered signals are reported with thin light grey and black lines, respectively; experimental signals are filtered with a Butterworth low-pass filter with a cut-off frequency of 100 Hz to nearly eliminate the effects due to noise which are mainly characterized by high frequency content. The filtered signals show an oscillation at nearly the first frequency of the soil-pile system, with amplitude slightly greater for pile P2 (mainly subjected to s-waves) than pile P3 (mainly subjected to P-waves) due to its minor distance from the source pile. However, the oscillation is not regular and seems to be affected, also after several cycles, by the oscillation of the source pile.

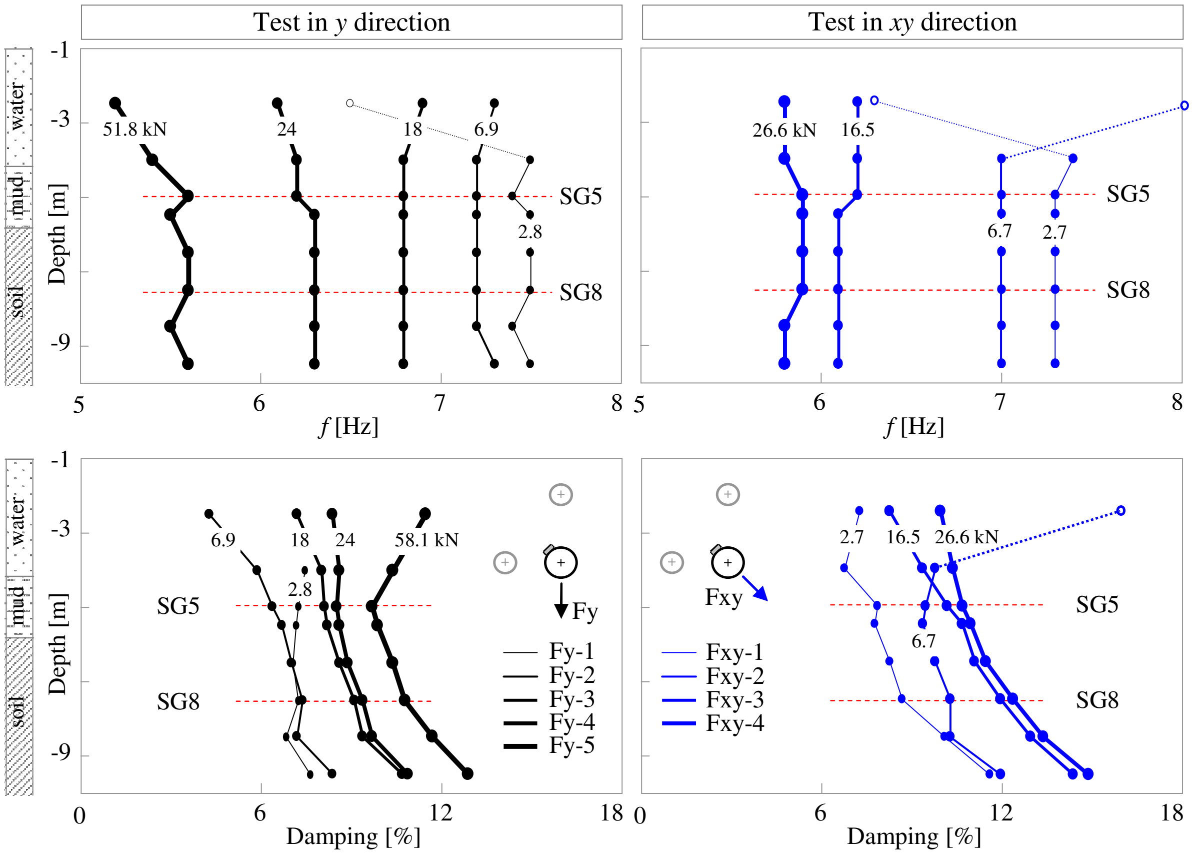


Figure 11. First natural frequencies and damping ratios at different location along the pile for different load levels of free vibration tests along *y*- and *xy*-direction

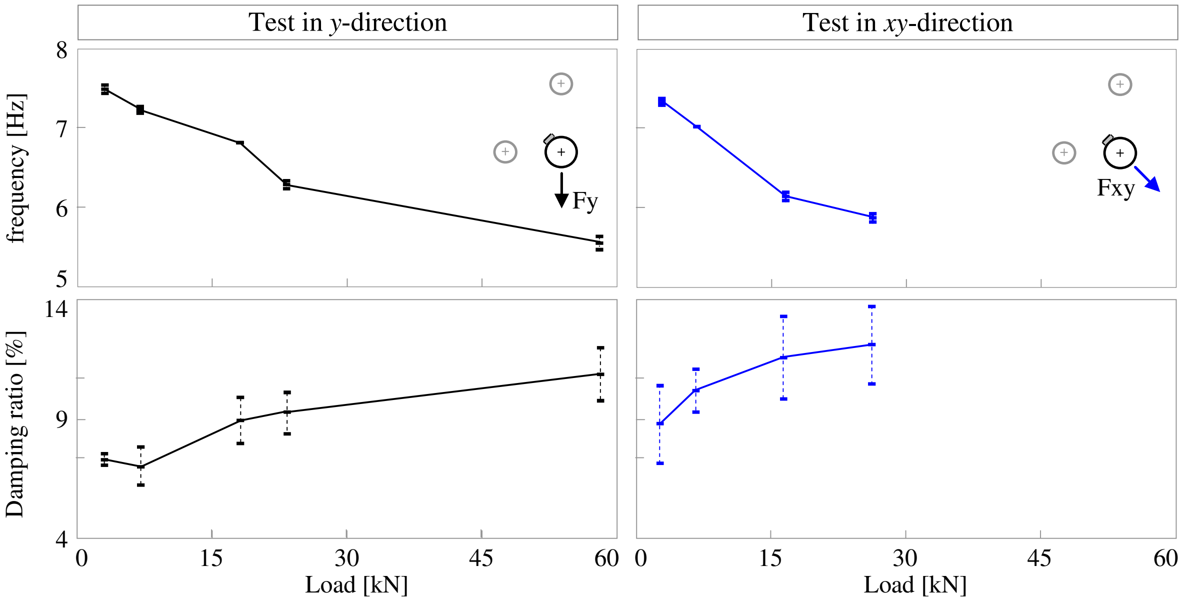


Figure 12. Mean value and standard deviation of natural frequencies and damping ratios for tests at increasing load levels along *y*-and *xy*-direction

As regards the frequency domain, Figure 14 shows the measured pile-head horizontal Frequency Response Functions (FRFs), obtained from the ratio between the complex spectrum of pile response in terms of acceleration at the pile head and the complex spectrum of the load applied at the pile head. As an example, only the FRF of the acceleration in *y*-direction at the head of the piles P2 and P3 during the first test in *y*-direction (Fy-1) are reported.

Some information about the interaction between piles when the source pile is subjected to higher strain levels may be drawn by analyzing the peak amplitude of the FRFs of accelerations measured at the head of the receiver piles P2 and P3 during the tests at higher load levels. Figure 15 shows the peak amplitude of FRFs relevant to all the tests in *y*-direction (left graph) and in *xy*-direction (right graph). An interesting trend can be observed, with a decrease of the amplitude for increasing levels of lateral loading. This behaviour is due to nonlinear phenomena developing in the soil surrounding the source pile and becomes more significant as the load increases.

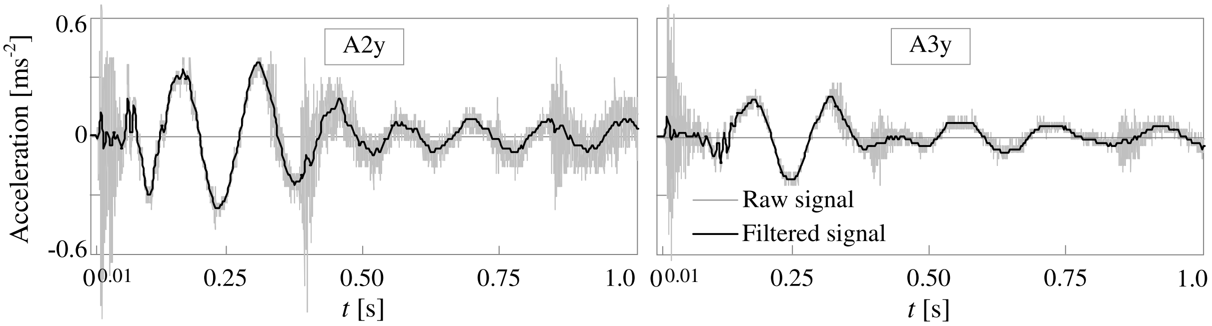


Figure 13. Time histories of accelerations on receiver piles during the test Fy-1

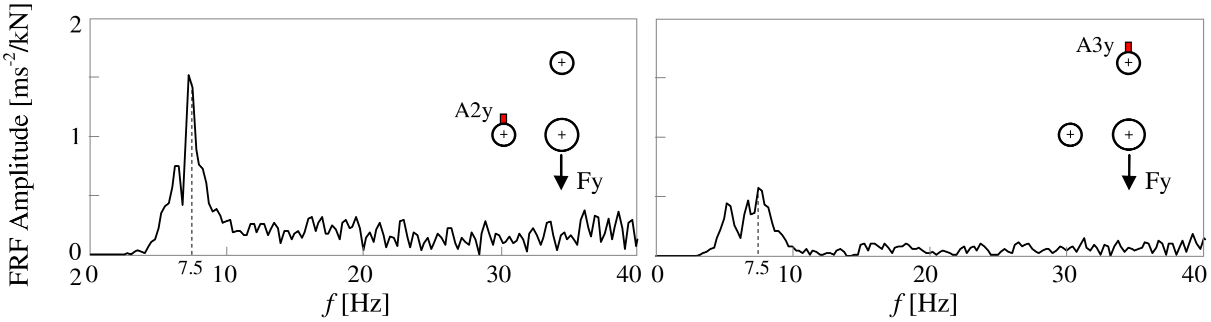


Figure 14. FRFs of acceleration signals of receiver piles during the test Fy-1

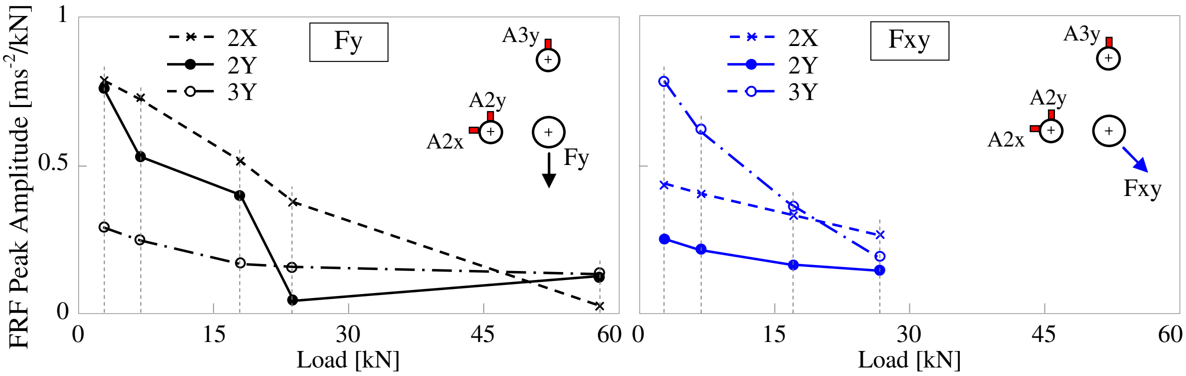


Figure 15. Peak amplitude of FRFs for different pile spacing and input direction for tests in *y*-and *xy*-direction

6 CONCLUSIONS

A full scale in-situ investigation, regarding free vibration tests carried out at increasing levels of lateral loading on a group of 3 steel pipe piles, in a “L” configuration, vibro-driven into soft marine clay, has been presented. The results have been analyzed both in time and frequency domain (determining frequencies and damping ratios) by processing signals registered by strain gauges placed along the source pile, at the corner, and accelerometers at the head of the receiver piles.

Obtained results have demonstrated that the experimentation was effective to study the dynamic behaviour of the single pile (soil-pile interaction) and the pile group (pile-to-pile interaction) both at low strains (elastic behaviour) and at increasing levels of lateral loading, investigating the nonlinear phenomena affecting the dynamic response. For increasing levels of loading, a reduction of the first natural frequency and an increase of the hysteretic damping of the soil-water-pile system together with the decrease of pile-soil-pile interaction have been observed.

REFERENCES

[1] Alpan I. The dynamic response of pile foundations to lateral forces. In: Proceedings of the 5th World Conference on Earthquake Engineering. Rome, Italy, 1973; 2: 256-271.

[2] Crouse C., Cheang L. Dynamic testing and analysis of pile-group foundation, in Dynamic Response of Pile Foundations - Experiment, Analysis, and Observation. Geotech. Spec. Pub. 11, ASCE, 1987, 79-98.

[3] Algie, T. B., Pender, M. J. and Orense, R. P. (2010) Large scale field tests of rocking foundations on an Auckland residual soil. In “Soil Foundation Structure Interaction” (R Orense, N Chouw, and M Pender (eds)), CRC Press / Balkema, The Netherlands, pp. 57- 65.

[4] M.Sa’Don, N. M., Pender, M. J., Orense, R. P. and Abdul Karim, A. R. (2010) Full-scale pile head lateral vibration tests. In “Soil Foundation Structure Interaction” (R Orense, N Chouw, and M Pender (eds)), CRC Press / Balkema, The Netherlands, pp. 33 – 39.

[5] Pender, M. J., Algie, T., M.Sa’Don, N. and Orense, R. P. (2010) Snap-back testing and estimation of parameters for nonlinear response of shallow and pile foundations at cohesive soil sites. Proc. 5th International Conference on Earthquake Geotechnical Engineering, Santiago, January.

[6] Carrubba P., Maugeri M. Non linear effects during dynamic loading on piles, In: Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, 1996; Paper No. 2074.

[7] Dezi F, Gara F, Roia D. Dynamic response of a near-shore pile to lateral impact load. Soil Dynamics and Earthquake Engineering 2012; 40: 34–47.

[8] Dezi F, Gara F, Roia D. Experimental study of near-shore pile-to-pile interaction. Soil Dynamics and Earthquake Engineering 2013; 48: 282–293.