# Experimental Investigation of Soil-Pile-Structure Seismic Interaction

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**SUMMARY:**
The cyclic and seismic response of soil-pile-structure systems is assessed through comprehensive experimental tests performed on the 3mx3m shaking table of the Bristol Laboratory for Advanced Dynamics Engineering (BLADE), University of Bristol (UK). Such tests were carried out in the framework of the Seismic Engineering Research Infrastructures for European Synergies (SERIES). The present work discusses preliminary results of the cyclic response of a pile group in a bi-layered soil profile. The outcomes of the test results discussed hereafter emphasize that kinematic effects are of paramount importance for the seismic analysis and design of structural systems with piled foundations. Appropriate combination rules to account for both inertial and kinematic effects are deemed necessary. The experimental results derived with the shaking table tests will be employed to calibrate numerical models, which, in turn, will be utilized to perform comprehensive parametric analysis aimed at providing sound design rules to be implemented in next generation performance-based seismic codes of practice.

*Keywords: single pile, pile group, kinematic interaction, inertial interaction, shaking table testing*

## 1. INTRODUCTION

The dynamic response of the structural systems located in earthquake prone regions can be significantly affected by soil-structure-interaction (SSI), especially when the foundation rests on soft soil. For deep foundations, the SSI effects may be three-fold: (i) the motion at the base of the superstructure will deviate from the free-field ground motion and will include a rotational component; (iii) the vibrational characteristics of the superstructure will be modified, and (iii) the piles will be subjected to additional bending, axial and shear stresses. The pure kinematic bending moments, may be significant especially for piles embedded in soft soils with high stiffness contrast between adjacent layers. The importance of assessing adequately the effects of kinematic and inertial bending moments has been stressed by numerous studies, e.g. Novak, 1991; Pender, 1993 and Gazetas & Mylonakis, 1998, among others. Such studies have also emphasised the need of providing sound combination rules for kinematic and inertial moments, as the aforementioned bending moments are not synchronous.
The present paper illustrates the preliminary results of a comprehensive laboratory tests performed on the 3mx3m shaking table of the Bristol Laboratory for Advanced Dynamics Engineering (BLADE), University of Bristol (UK). Centrifuge and shaking table tests are systematically utilized to investigate SSI effects for piles (Mizuno, 1984; Meymand, 1998; Wei et al., 2001; Chau et al., 2009; Tokimatsu & Suzuki, 2009; Moccia, 2009), as kinematic interaction is difficult to be reproduced in field tests.

The experimental tests performed at BLADE, within the Framework of the Seismic Engineering Research Infrastructures for European Synergies (SERIES), were carried out on a group of piles, with and without pile caps. The sample piles were subjected to various dynamic input motions, namely white noise, sinedwells and earthquakes. The tests were aimed at shedding light into the complex phenomenon of the SSI. To this end, free-field response, kinematic interaction (in both horizontal and vertical directions), foundation-structure interaction, and pile group effects were experimentally investigated. Findings from this investigation will be employed to assess the reliability of existing analytical formulations to predict the inertial and kinematical bending moments along piles, accounting for soil-pile interaction and ground motion characteristics.

## 2. SHAKING TABLE TESTS

The cyclic and earthquake response of pile groups was explored by means of 1-g shaking table tests. Such laboratory tests were aimed at assessing the effects of both kinematic and inertial effects on piles. The test campaign consisted of two series of tests: the preliminary tests were carried out during November 2010 while a comprehensive series of tests, including earthquake loading, was carried out in July 2011. The 6-degree-of-freedom earthquake simulator of BLADE and the equivalent shear beam (ESB) container was utilized to perform the aforementioned series of tests. The ESB is shown in Figure 1; it consists of 8 rectangular aluminium rings, which are stacked alternately with rubber sections to create a hollow yet flexible box of inner dimensions *1.190* m long by *0.550* m wide and *0.814* m deep (Crewe et al, 1995). The rings are made of aluminium box section to minimize inertia while providing sufficient constraint for the $K_0$ condition. The stack is secured to the shaking table by its base and shaken horizontally lengthways (in the x direction). Its floor is roughened by sand-grain adhesion to aid the transmission of shear waves; the internal end walls are similarly treated to enable complementary shear stresses. Internal side walls are lubricated with silicon grease and covered with latex membrane to ensure plane strain conditions.

This type of containers should be ideally designed to match the shear stiffness of the soil contained in it. However, the shear stiffness of the soil varies during shaking depending on the strain level. Therefore the matching between the end-wall and the soil stiffness would be possible only at a particular strain level. The ESB of the BLADE is designed considering a value of strains in the soil close to the failure (0.01–1%). It is thus more flexible than the soil deposit at lower strain amplitudes and, as a consequence, the soil will always dictate the overall behaviour of the container (Bhattacharya et al, 2012). Indeed, the shear stack resonant frequency and damping in the first shear mode in the long direction when empty were measured prior to testing as 5.7 Hz and 27% respectively, sufficiently different from the soil material properties.

The sample test model consists of five piles embedded in a bilayer soil (Figure 2). Pile 3, 4 and 5 are closer to each other with a relative spacing of 70 mm and a diameter of 22 mm (s/d=3); pile 1 and 2 are at a distance of 140 mm.

Accelerometers were used to monitor the accelerations of the shaking table, the shear stack, the soil along a vertical array, the pile heads and the mass of the superstructure. The LVDT transducers were employed to monitor the displacements of the pile in the horizontal and vertical direction. To evaluate the bending response of the piles, 8 strain gauge pairs have been attached on the shafts of pile 4 and 5; additionally, 4 strain gauges are placed on the shaft of pile 1 close to the layer interface. Overall, 63 data channels were employed.

A two layer soil profile was deposited by pluviation. The top layer is made of Leighton Buzzard sand (BS) fraction E, deposited through a 40mm diameter nozzle to achieve a mass density of 1390kg/m$^3$.

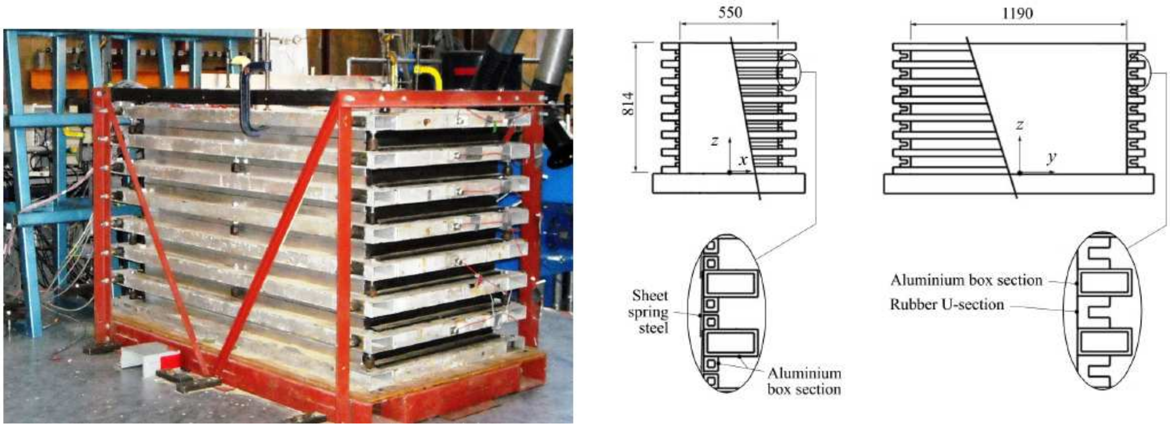


**Figure 1.** Equivalent shear beam container (shear stack)

The bottom layer is a mix between BS fractions B and E (85% and 15%, respectively) pluviated through a 12mm diameter nozzle to achieve a mass density of 1780kg/m$^3$. The free surface of the soil deposit is at the 800mm from the base of the shear stack.
The Leighton Buzzard sand here adopted has been extensively used in the experimental research activity carried out at the BLADE. Numerous density and stiffness data can be found in previous experimental studies (Stroud, 1971; Cavallaro et al., 2001; Lings & Dietz, 2004; Moccia, 2009). Table 1 outlines the properties of the two soil layers used for sample test models.

**Table 1.** Soil layer properties

| Soil layers | Thickness H (mm) | Void ratio e | Relative density Dr (%) | Dry unit weight $\gamma_d$ (kN/m$^3$) | Shear wave velocity Vs (m/s) | | $Vs_2/Vs_1$ | |
|---|---|---|---|---|---|---|---|---|
| | | | | | November 2010 | June 2011 | November 2010 | June 2011 |
| Top layer (LB-E) | 340 | 0.9 | 28 | 13.63 | 45 | 56 | 3.26 | 1.64 |
| Bottom layer (LB-E+B) | 460 | 0.48 | 41 | 17.46 | 150 | 92 | | |

*2.1 Pile and SDOF configurations*

The pile specimens are made of an alloy aluminium tube of thickness $t$ = 0.71 mm, outer diameter $D_0$ = 22.23 mm and length 750 mm. The main properties of the aluminium tube are: unit weight $\gamma$ = 27 kN/m$^3$, Young's modulus $E_p$ = 70 GPa, leading to a bending stiffness $E_pI_p$=.1.95x10$^8$ Nmm$^2$.
Five different oscillators were employed to simulated different SDOF systems. Two types of column were used to study the effect of pier stiffness. All columns had a rectangular cross section (3 mm x 12 mm). The aluminium and steel piers owned a bending stiffness, EI, of 1.89x10$^{-3}$ kNm$^2$ and 5.67x10$^{-3}$ kNm$^2$, respectively.
Seven different model configurations were tested as schematically shown in Figure 2. In configuration 1 (FHP), all pile heads are free to rotate and there are no oscillators on the pile top. Configurations 2, 3 and 4 include free-head piles and one oscillator placed on Pile 1, 4 and 5, respectively. Configuration 5 (SC) has a small cap connecting piles 4, 5 and 3, without any oscillator. Configuration 6 (SC-O) is characterized by the same small cap as before with an oscillator upon. Finally configuration 7 (LC) has a large cap (connecting all the piles) without any oscillator.

## 3. LOADING TYPE

The loading type included white noise excitation, harmonic excitation and earthquake ground motions. Preliminary results for harmonic excitations are described hereafter.

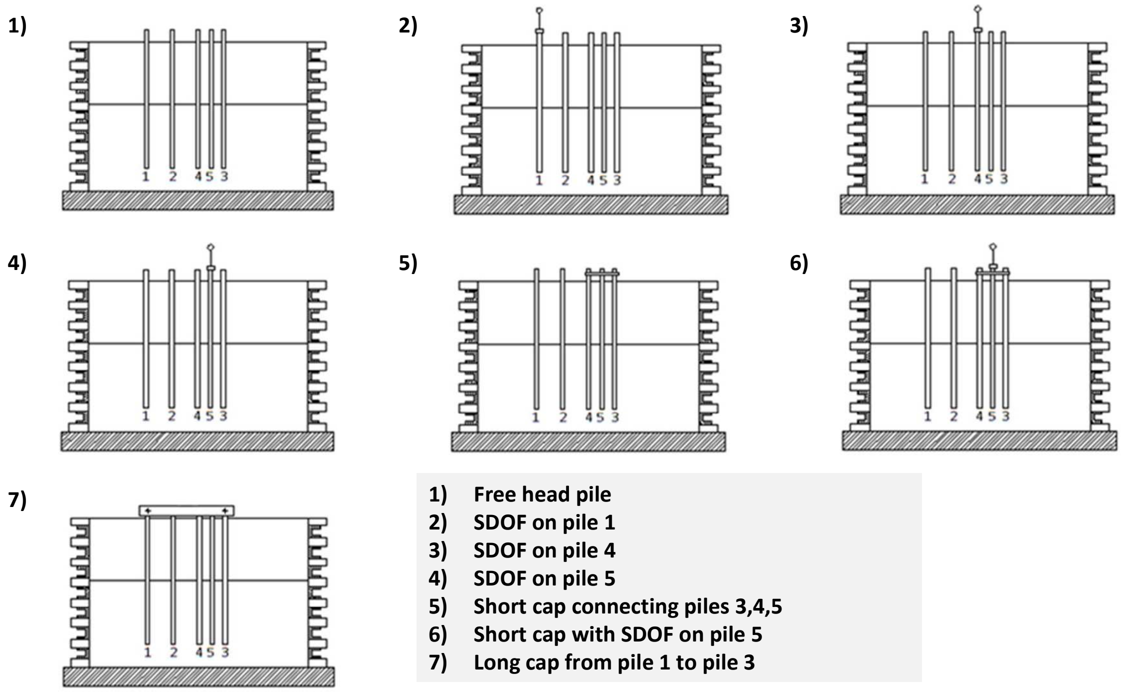


**Figure 2.** Details of model configurations

In the first testing stage (June 2011) the harmonic input motions were typical sine-dwell functions, with 12 steady state cycles; a set of 15 frequencies (from 5 to 30 with an increment of 2.5 Hz; from 30 to 50 with an increment of 5 Hz) was used with acceleration amplitudes varying between 0.01g÷0.18g. In the second testing stage (June 2011) the sinusoidal excitation had 16 steady cycles. A set of 7 frequencies (from 5 to 45 with an increment of 5 Hz) was used, with acceleration amplitudes varying between 0.01g÷0.13g.

## 4. PRELIMINARY EXPERIMENTAL RESULTS

The preliminary experimental results of the comprehensive laboratory tests on cylic response of pile groups carried out on the BLADE shaking table are discussed hereafter.

Figures 3 shows the response histories of the free-field soil accelerations measured for the free-head pile configuration (FHP). The plotted values refer to the application of a sinedwell input motion with three increasing levels of peak ground acceleration (PGA) applied on the shaking table, namely PGA=0.008g, 0.027g and 0.069g. The frequency of the input motion is 30 Hz.

The free-field response increases with the input acceleration level. The recorded accelerations are compliant with the input motion as also shown in the diagrams in the lower part of Figure 3, where the peak acceleration profile in the soil (quoted as $a_{max}$) is plotted.

The diagrams of Figure 3 are also included in Figure 4 to compare these data with the counterparts derived from the tests characterized by PGA=0.013g and 0.041g. The comparison shows that the profiles of $a_{max}$ exhibit quite similar shapes (see Figure 4a). These shapes are better compared on the right side (Figure 4b), where the dimensionless curves obtained by the ratio of $\Delta a_{max}$ [$a_{max}(z)$ minus the minimum value of $a_{max}$ along the vertical axis] to the maximum value of $\Delta a_{max}$ are plotted vs. depth for the five sample tests; it is worth noting that the effect of soil layer interface appears more significant for the lower input acceleration values. As regards the amount of amplification, it is believed that the effect of the subsequent shaking on soil properties may play a key role and should be accounted for in a realistic manner.

Typical sets of measured strain time histories and corresponding bending moments are displayed in Figures 5 and 6; the relationships employed to compute the bending moments (and the axial loads) can be derived from the principles of structural mechanics, as also discussed in Simonelli et al. (2012). The tests have been performed during the first stage of the experimental research; the test configuration is the FHP one, with input PGA at the shaking table of 0.027g and 0.069g respectively.

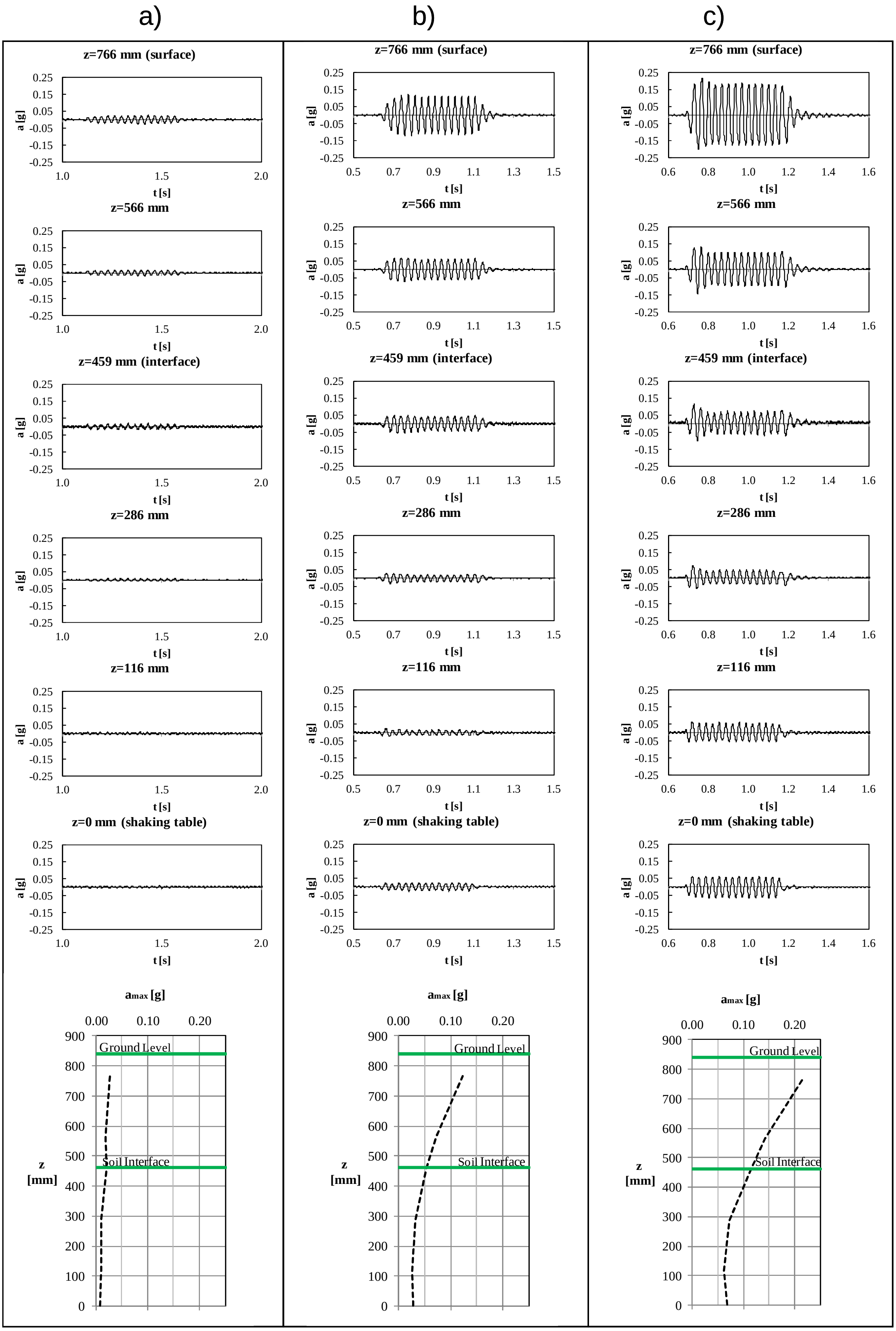


**Figure 3.** Free-field accelerations for the FHP configuration for different amplitudes of the input acceleration at the shaking table: (a) PGA= 0.008g; (b) PGA= 0.027g; (c) PGA=0.069g

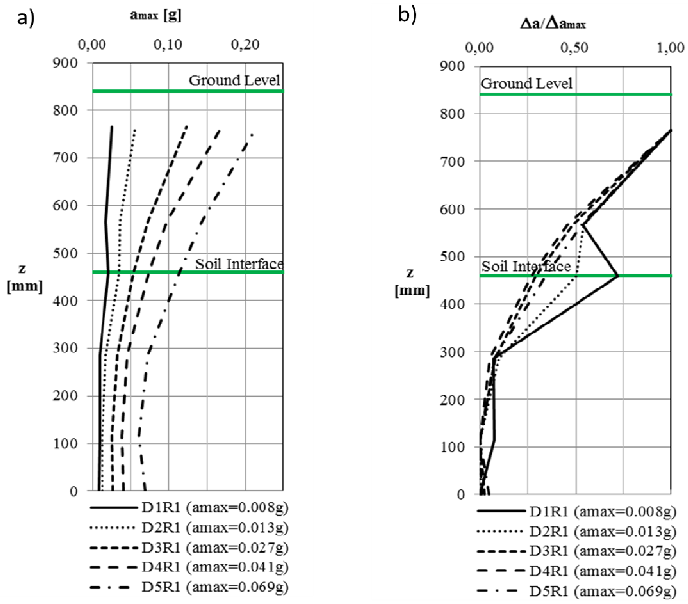


**Figure 4.** Peak acceleration (a) and dimensionless acceleration profiles (b) vs. z for different amplitudes of input acceleration

All data refer to pile 4. A significant effect of kinematic interaction is observed in proximity of the layer interface (z=440 mm), where high values of strain amplitude are detected.
These two sets of experimental data have been selected as they are representative of typical response of the sample pile groups.
In fact, in the first case (Figure 5) the strains appear almost symmetric with respect to the horizontal axis and in opposite phase, which suggests that the pile has mainly been subjected to pure bending. Additionally, when the excitation stops, the bending strains return to zero; the pile returns to the initial configuration, with no residual moment.
On the contrary, in the second case (Figure 6), the strain time histories on the opposite sides show an offset, indicating a residual deformed configuration of the section, with a residual bending moment. This moment is slightly lower than the maximum moment induced during the excitation.

The comparison of the different pile response is further presented in Figure 7, where the time history of normal stress is also plotted. It is evident that in the second case (PGA=0.069g) the pile section at z=440 mm experiences a residual normal stress. The strain-gauge time histories are, indeed, not symmetric with respect to the horizontal axis.
By taking the absolute maxima from the time histories of bending moments, the envelopes of bending moment profiles in the soil were computed (Figure 8) for piles 4 and 5, for the FHP configuration and the five tests with input $a_{max}$ increasing from 0,008g and 0,069g (the same tests utilised for the free-field analysis). The bending moments increase with input motion level, the peak is located at the layer interface; such bending moments are, in fact, generated by kinematic interactions.

## 4. CONCLUSIONS

A comprehensive experimental testing program has been performed at BLADE, within the Framework of the Seismic Engineering Research Infrastructures for European Synergies (SERIES). Tests on groups of piles, with and without pile caps and SDOF oscillators were carried out, under various dynamic input motions. Here some results obtained during the preliminary phase (November 2010) are illustrated, with reference to a free-head pile configuration.

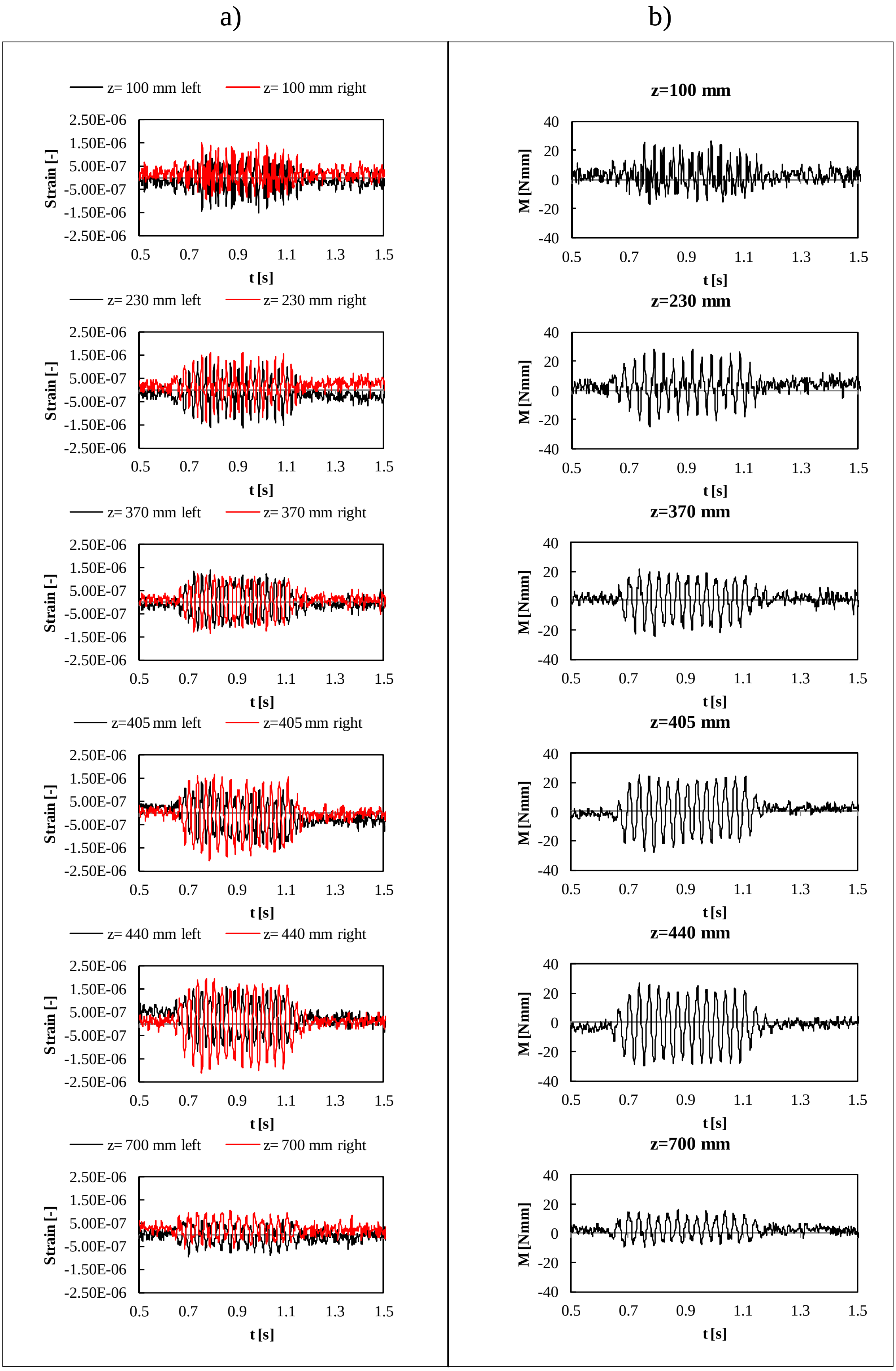


**Figure 5.** Time histories of strains (a) and bending moments (b) at different elevations along pile 4 for the FHP configuration (PGA=0.027g) (test n. 101115_D3R1)

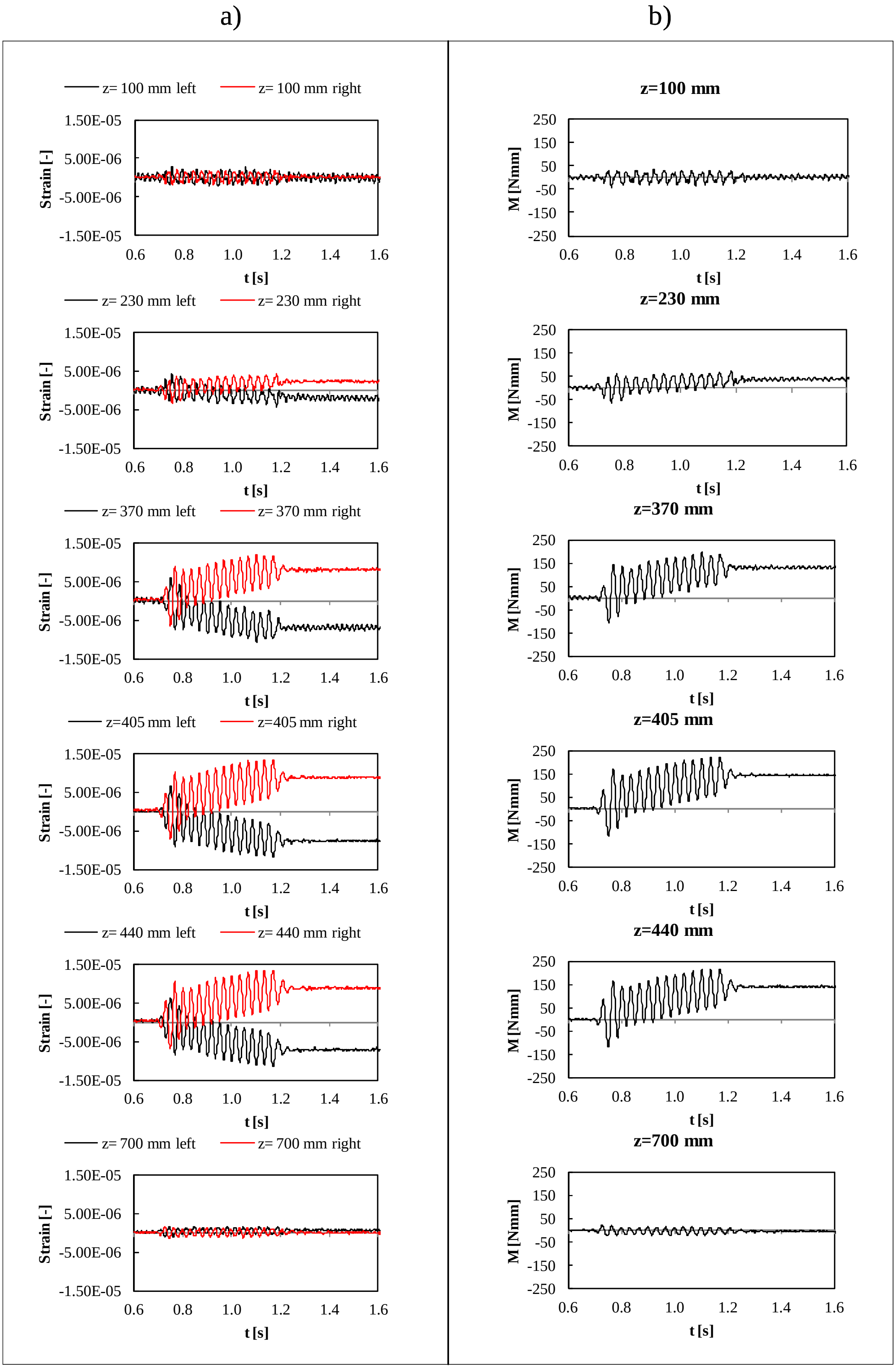


**Figure 6.** Time histories of strains (a) and bending moments (b) at different elevations along pile 4 for the FHP configuration (PGA=0.069g) (test n. 101115_D5R1)

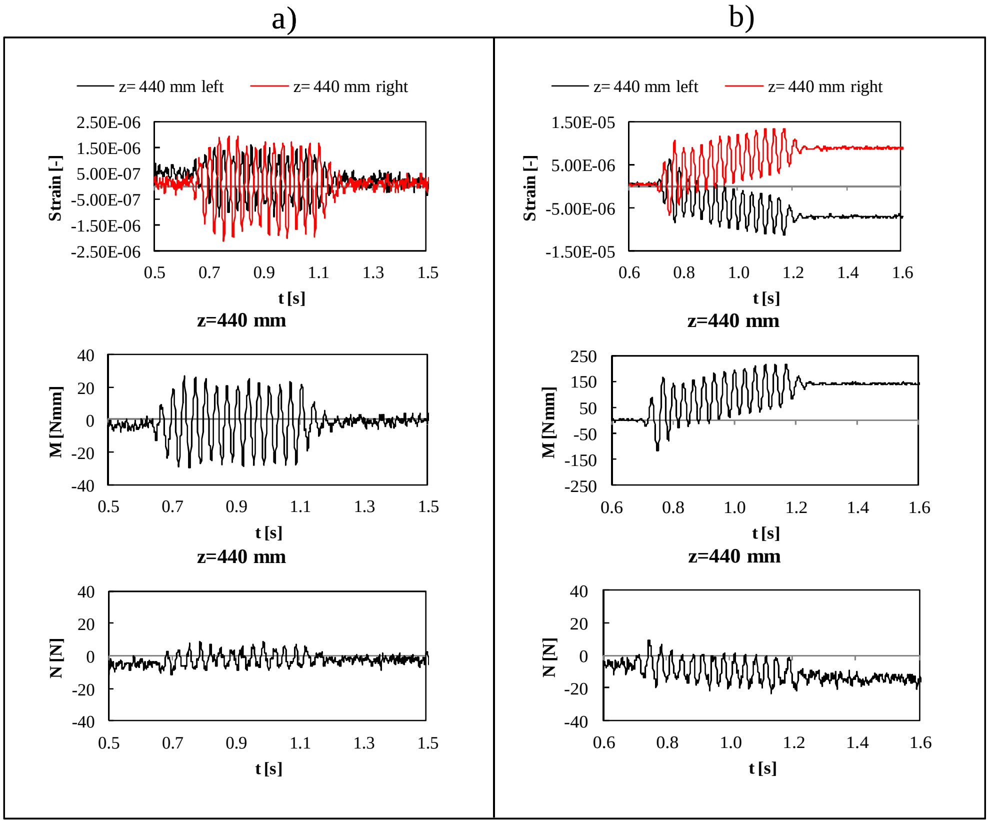


**Figure 7.** Strains and bending moment for a given pile elevation (z=440mm). In (a) results refer to the test with PGA=0.027g; in (b) to a PGA=0.069g at the shaking table

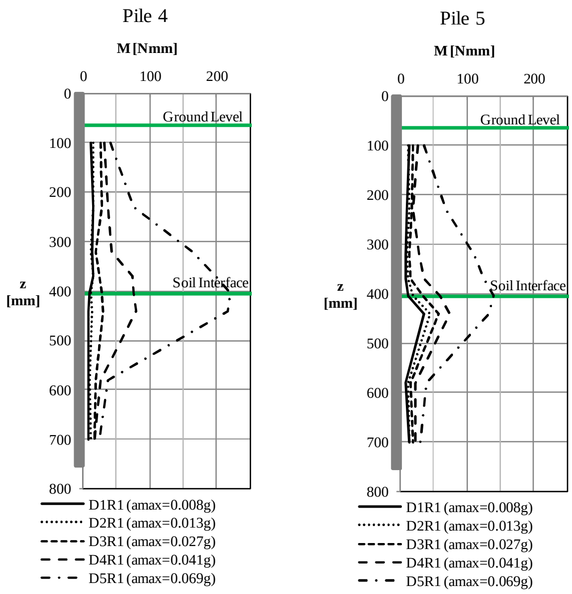


**Figure 8.** Envelope of bending moments vs. depth for Pile 4 and Pile 5 in different model configurations

First the free-field soil response was examined, for five tests with increasing input accelerations.
The results confirm the effectiveness of the shear stack as container of soil models: the response of the subsoil is not affected by the boundary conditions; further the comparison among the free-field responses of the different tests shows that they are consistent. For the same tests, the response of piles has been investigated. The strain gauge measurements were very effective for determining both the bending and the normal stresses along the piles, allowing to evaluate both the time-history and the residual deformations of piles at any level. The bending moment diagrams put in evidence the significant kinematic interaction effects on piles, which reach the maximum about the interface between the soil layers, even in the FHP configuration with no moments at the pile top and bottom.
The results of this extensive investigation are still under assessment, for achieving the multiple goals of the research program. Results will be reported in subsequent publications.

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