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SOIL-PILE-STRUCTURE-INTERACTION: EXPERIMENTAL RESULTS AND NUMERICAL SIMULATIONS

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Abstract. *The complex seismic soil-pile-structure interaction phenomenon is related to the interaction between foundation and structure under seismic and dynamic excitations. An effective way to assess such phenomenon is to analyse the response of scaled physical model in 1-g or n-g devices. In this study some results selected from a comprehensive 1-g shaking table tests are reported and discussed. The extensive experimental campaign was carried out on the 3mx3m shaking table of the Bristol Laboratory for Advanced Dynamics Engineering (BLADE) at the University of Bristol (UK), within the framework of the Seismic Engineering Research Infrastructures for European Synergies (SERIES). The physical model comprises a pile group embedded in a by-layer soil deposit with different pile configurations. This paper focuses on the pile group response generated by the presence of a cantilever system (a single-degree-of-freedom, SDOF) connected at the top of the central pile considering no connection among the other pile heads. The selected input motions consist of a set of sinedwell excitations for SDOFs with different structural masses. The experimental data are used to validate an advanced 2D difference element model using the FLAC2D code. The comparisons between the experimental and the numerical results are presented in terms of both envelope and time histories for the free-field and piles responses. The SDOF response is also assessed in terms of displacement time histories. Comparisons between the numerical and experimental test results appear satisfactory; hence numerical approach can be used for further simulations of the soil-pile-structure interaction phenomena.*

1 INTRODUCTION

Pile foundations are usually adopted when shallow soils are not able to carry the vertical loads imposed by the superstructure and it is necessary to reach stiffer soil layers. When such foundations are utilized, the dynamic response of the system may lead to soil-pile-structure interaction (SPSI). This complex phenomenon can be resumed as the combination of two different mechanisms [1-10]: (i) kinematic interaction (Figure 1a) due to the different response of the pile-soil system from the free-field response; (ii) inertial interaction (Figure 1b) due to the additional loads given to the pile by the superstructure inertial actions.

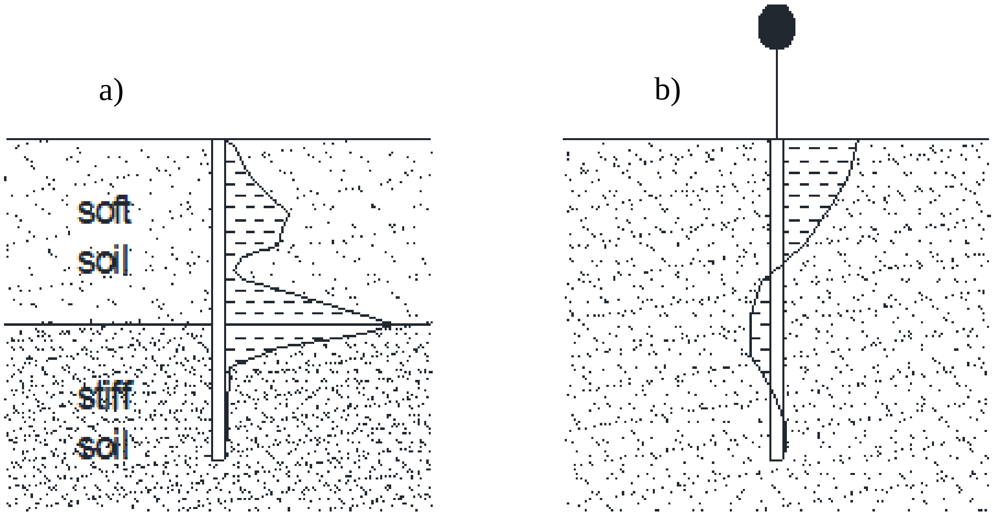

Figure 1. Kinematic (a) and inertial (b) interaction in piled foundations (modified from Simonelli et al., 2013)

The effects of kinematic interaction increase as the stiffness contrast between the crossed layers increases. The maximum bending moment is located in the neighbor of the layer interface.

An effective approach to reliably investigate the SPSI is the testing of physical models and in particular the analysis of the scaled model response through 1-g or n-g devices.

Experimental tests can be targeted to: (i) identify the main aspects that may influence the system response, using different dynamic inputs (i.e. sinusoidal waves, white-noise and earthquakes input); (ii) validate the models utilized in the numerical analyses.

The scope of the present work is to examine the soil-pile-structure interaction referring to a selected set of data coming from high-quality shaking table tests and use these data to validate an advanced difference element numerical model implemented in the computer platform FLAC2D [11]. A cantilever system (a single-degree-of-freedom system, SDOF) connected at the top of the central pile, with no connection among the other pile heads has been considered for the numerical simulations presented herein.

2 SHAKING TABLE TESTS

The experimental campaign was carried out on the 3mx3m 1-g shaking table of the Bristol Laboratory for Advanced Dynamics Engineering (BLADE) at the University of Bristol (UK). The 6-degree-of-freedom earthquake simulator of BLADE and the equivalent shear beam (ESB) laminar container were utilized to this end [12]. The experimental procedure includes the use of: (i) white-noise excitation, a random noise signal of bandwidth 0-100 Hz for the evaluation of the fundamental frequency; (ii) harmonic excitation, sinusoidal wave. Further details on the equipment and the experimental procedure can be found in some of the Authors previous works [13-16].

2.1 Physical model, and instrumentations

The physical model was formed by a group of 5 pile embedded in a bi-layer deposit (Figure 2). The two-layer soil deposit (obtained by means of pluviation) is made of by Leighton Buzzard sand (LB) fraction E for the top layer and by a mix of LB fractions B and E (85% and 15%, respectively) for the bottom layer. The LB sand characteristics has been extensively utilised in previous experimental studies [17-24]. The main details of the two soil layers are listed in Table 1.

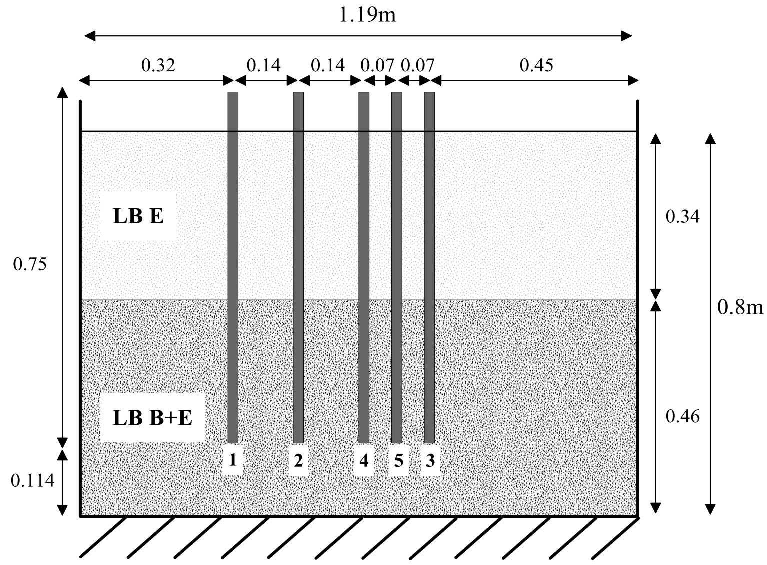

Figure 2. Model details (after Durante et al., 2015)

Soil layers	Thickness *H* (mm)	Relative density *Dr* (%)	Dry unit weight γ_d (kN/m^3)
Top LB(E)	340	28	13.63
Bottom LB(E+B)	460	41	17.46

Table 1. Soil layer details

Piles are formed by an alloy aluminium tube (commercial model 6063-T6) with thickness t = 0.71 mm, outer diameter D = 22.23 mm and length L = 750 mm. Pile 3, 4 and 5 are closer to each other with a relative spacing s=70 mm (s/D ≈ 3); pile 1 and 2 are placed at higher distance, equal to 140 mm. The oscillator is formed by an aluminium column (3x12 mm cross section, 100mm high) with extra masses added to its top to achieve different dynamic response; in the set of tests considered in this work the oscillator is connected through a “foundation” directly to the central pile head (pile 5) and there is no connection among the other piles head; more details about masses are listed in Table 2.

Total added mass [g]	Fixed base frequency (f_{fix}) [Hz]	Damping [%]
75	38.0	0.7
125	30.5	1.2
175	26.5	0.9
275	20.5	1.4
475	15.0	1.2
975	10.4	1.5

Table 2. Fixed base frequency for oscillator considered

Accelerometers were employed to monitor the shaking table, the shear stack, the soil along a vertical array (free-field response), the pile head and the superstructure. Linear Variable Displacement Transformer (LVDT) were adopted to assess the pile head and shear stack displacements. For evaluating bending moment a set of 8 pairs of strain gauges were employed for each sample pile to be monitored (i.e. pile 4 and 5). More details of the instrumentation used in the experimental campaign can be found in Durante et al. [25].

2.2 Shear wave velocity profile

An inhomogeneous shear wave velocity profile is adopted for the bi-layer deposit according to the equations:

(1)

where $V_{s1\text{-}2,B}$ are the values of the shear wave at the bottom of the deposit; z is the depth starting from the surface and b is an inhomogeneity coefficient.
$V_{s1,B}$ and $V_{s2,B}$ are obtained employing the procedure proposed in Durante et al. [26], considering the inhomogeneity coefficient (b) equal to 0.5. Towards this end, the white-noise test results are employed to evaluate the fundamental frequencies of the top layer and of the overall deposit. The initial shear wave profile of the deposit is characterized by to $V_{s1,B}=81.7m/s$ and $V_{s2,B}=100.6m/s$, as pictorially depicted in Figure 3.

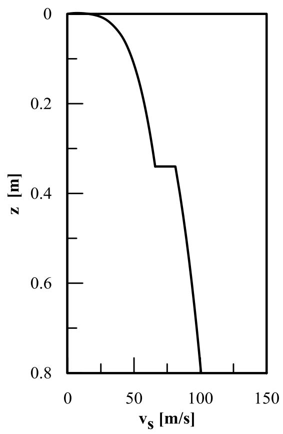

Figure 3. Initial shear wave profile

3 NUMERICAL ANALYSIS

Numerical analyses are carried out at the model scale by means of the advanced difference element code FLAC2D [11]. Soil deposit is modeled considering the hysteretic Ramberg-Osgood constitutive model [27]; the structural elements are modeled by means of pile (for the foundation) and beam elements (for all the SDOF components) implemented in FLAC2D.

3.1 Modeling of soil deposit

The shear wave velocity profile shown in Figure 3 was used. In order to simulate the hysteretic soil behaviour, a multiaxial formulation of the Ramberg-Osgood stress-strain relationship was employed [27], implemented into FLAC as a user-defined constitutive model [28]. The calibration of the two model parameters requires fitting of the Ramberg-Osgood predicted soil behaviour against the experimental $G(\gamma)$ and $D(\gamma)$ curves: in this work the Vucetic and Dobry curves for zero plasticity index were employed [29]. The values used in the modelling can be derived from the curves shown in Figure 4. To compensate for the damping response of the model and the ESB effect at small strain levels, an additional 5% Rayleigh damping is added.

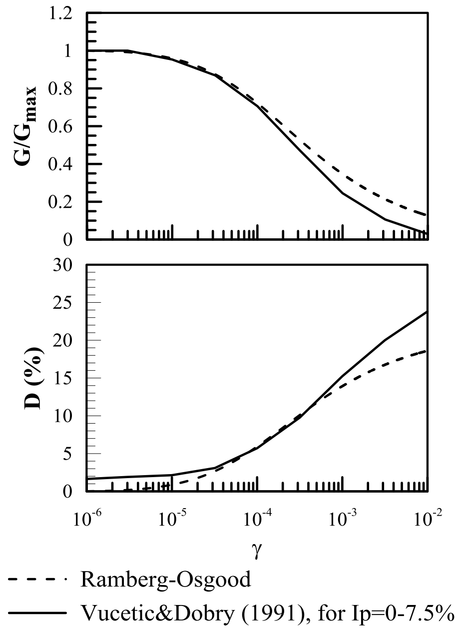

Figure 4. Shear modulus reduction and damping ratio curves adopted (after Durante et al., 2015)

The friction angles of the two layers were obtained based on the well known Bolton [30] and Cavallaro et al. [31] formulations, due to the absence of ad-hoc laboratory tests. Friction angles equal to *35°* and *38°* are adopted for the LB fraction E (upper layer) and fraction B+E (bottom layer), respectively.

For the equilibrium analysis the model was fixed in both the directions at the bottom and only in the horizontal direction at its lateral borders. In the dynamic analyses the lateral restrains were imposed in a simplified way, with the aim of reproducing the boundary conditions imposed by the ESB.

3.2 Modeling of pile and oscillator

Piles were modeled by means of pile elements as implemented in the computer program FLAC [11]. Pile elements interact with the 2D plane strain grid via normal and shear coupling springs that represent nonlinear connectors able to transfer force and motion between pile elements and the grid in correspondence of the nodes. The calibration of the normal springs is obtained by the force per unit length – relative displacement (p-y) response of the piles based on the analytical method by Georgiadis et al. [32]; the values for piles in cohesionless soils have been used for modelling the sample piles. The values of k increases proportionally with depth by means of a subgrade modulus (n_h), which depends on the sand density. In the present work, values of n_h equal to *1500* and *2000* kN/m^3 for the top and bottom layers respectively, according to Table 3 [33], are employed. The ultimate soil resistance p_u depends on the unit weight of the soil, the friction angle of sand, the earth pressure at rest and the active earth pressure coefficients and the pile diameter. The shear coupling springs were obtained starting from the soils characteristics, as suggested in Itasca [11].

Relative Density	Loose	Medium	Dense
n_h [kN/m^3]	1100 - 3300	3300 - 11000	11000 - 23400

Table 3. Coefficient of initial subgrade modulus n_h (derived from Terzaghi, 1955)

The foundation, the oscillator column and the mass were realized as beam elements; the different masses were modelled considering different density. The damping ratio for the oscillator is considered assuming the in-built *combined* damping, a variation on *local* damping available in FLAC; in fact the *combined* one is more efficient than the latter, for the significant uniform motion like the sinusoidal waves adopted in these analyses. Local damping is a non-viscous damping, in which the damping force on a node is proportional to the magnitude of the unbalanced force. The values assumed for the damping ratio were obtained from the experimental data with the logarithmic decrement method. A 4.6% value is assumed for the damping in the analyses carried out for the sample models.
The numerical model adopted for the dynamic simulations is shown in Figure 5.

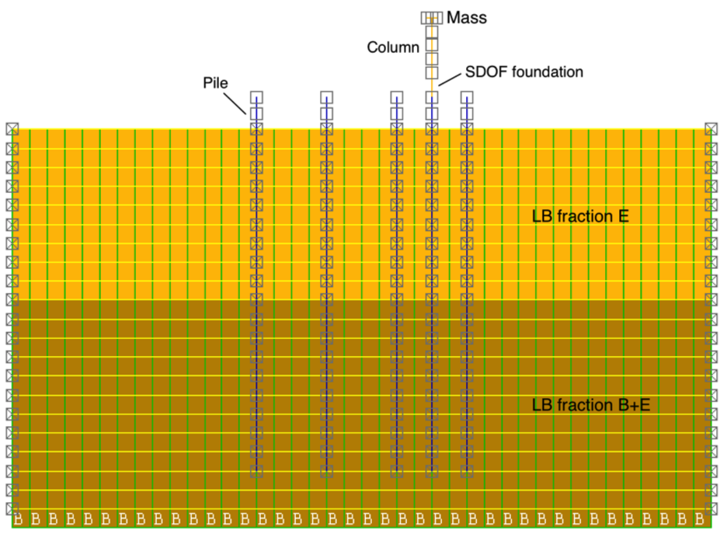

Figure 5. FLAC2D model

4 ANALYSIS RESULTS

The set of data considered in this work refer to the same harmonic input, i.e. a *20 Hz* sinusoidal wave with amplitude equal to *0.1g*.

Figure 6a shows the maximum soil accelerations in the free-field conditions. The results demonstrate that soil response is independent from the mass attached at the oscillator, ensuring that the distance from the piles is large enough to prevent interaction effects to occur.

The bending moment envelope along instrumented piles (pile 4 and pile 5) are reported in Figure 6b and 6c, respectively. The inertial effects are significant at the pile head up to a depth depending on mass value: considering the dynamic response of the system, the increasing of the maximum bending moment at the pile head is related to the resonant condition between the input and the elongated period of the oscillator.

It is found that the oscillator with the maximum SSI effect is the SDOF with a *175g* mass. Utilizing the transfer function between the accelerometer at the top of the oscillator and the accelerometer on the shaking table, a *42%* of period elongation ($T_{SSI}/T_{fix} = 1.42$) was computed, due to the soil-pile-structure interaction ($f_{SSI} = 18.6\ Hz$); the measured elongation is higher than the ones computed with typical literature formulas [34-37], as reported in Table 4.

	Jennings&Bielak [34] Veletsos&Nair [35]	Wolf [36]	Kumar&Prakash [37]
T_{SSI} / T_{fix}	≈1.00	≈1.00	≈1.15

Table 4. Period elongation computed with literature formulas

Figure 6b shows that pile 4 is influenced by the oscillator even if it is not connected to it; such response depends, indeed, on the short distance between piles 4 and 5 ($s/D \approx 3$). Additionally, the maximum value of the bending moment is not located close to the interface ($z=460\ mm$); furthermore, it depends on the mass oscillator. This behavior may probably stems from the fact that the oscillator on the central pile (and its resonance condition) tends to influence the pure kinematic interaction of the close pile.

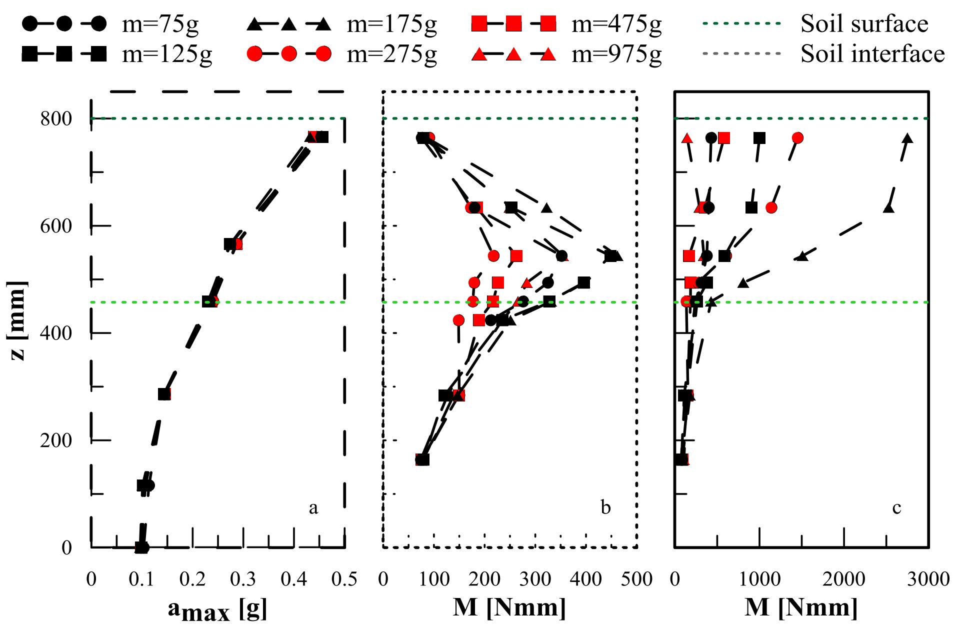

Figure 6. Experimental results: maximum soil acceleration vs. depth (a), maximum bending moment vs. depth for pile 4 (b) and pile 5(c)

Numerical simulations of the above experimental tests were performed by means of FLAC2D code [11]. For brevity, only two typical results are reported in the present paper, namely the tests with oscillator masses equal to 175g and 975g, i.e. the cases corresponding to the lower and upper bound responses.

Figure 7 shows the comparison between the numerical and the experimental data for the free-field (Figure 7a), pile 4 (Figure 7b) and pile 5 (Figure 7c) responses. Figure 7a ensures that soil is adequately simulated in the numerical analyses. The resonance effects are also reliably modeled for pile 5. It is evident, from Figure 7c, that the numerical maximum bending moments at the pile head and at the interface provide close matches with outcomes of the experimental data. Conversely, Figure 7b shows that, for pile 4, the numerical model is able to simulate the maximum bending moment due to the kinematic interaction, nevertheless it cannot reproduce the resonance effect due to the oscillator on the other pile. The latter response is still under investigation by the Authors.

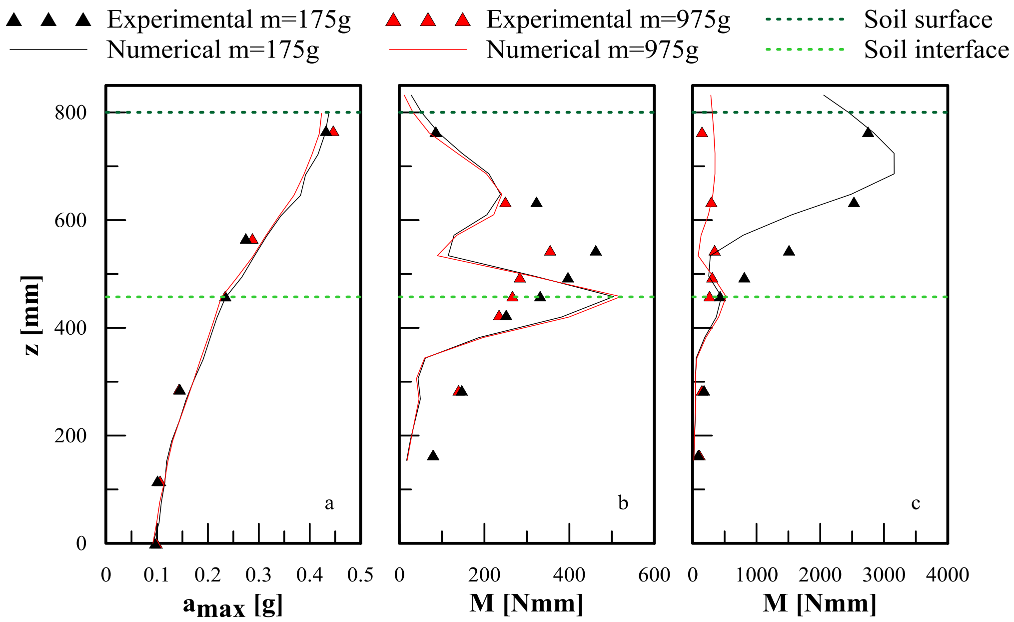

Figure 7. Experimental vs. numerical results: maximum soil acceleration vs. depth (a), maximum bending moment vs. depth for pile 4 (b) and pile 5(c)

Figure 8 shows the comparison in terms of time histories for the test with the mass oscillator equal to 175g. On the left hand side, the comparison for the free-field vertical array at three different elevations (bottom, interface and surface) is provided. It is noticed that there is a full match between computed values and test results. It is worth mentioning that the input was applied to the sample model in terms of accelerations, simulated through an artificial sinusoidal wave (generated directly with a FLAC routine). On the right hand side of Figure 8, the responses close to the pile head (bending moment) and at the top of the oscillator (displacement) are reported. The results confirm that numerical model implemented in FLAC may simulate accurately the experimental data of the shaking table tests. Figure 8 shows a small mismatch of the numerical time histories for the structural elements, that appears after several cycles.

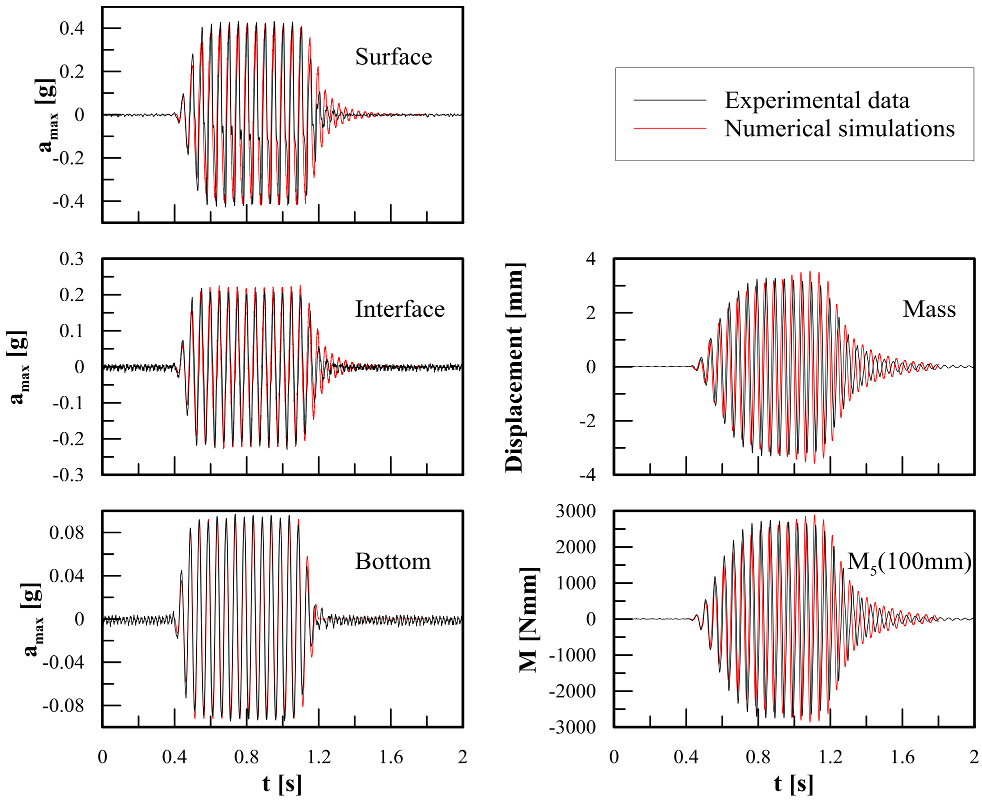

Figure 8. Time histories comparison for the SDOF with mass of 175g

5 CONCLUSIONS

Numerical calibrations were performed by means of an advanced two-dimensional model in FLAC. The numerical model reproduced a scaled model tested on the shaking table of the BLADE laboratory at the University of Bristol. The physical model is formed by a group of five pile embedded in a bi-layer deposit with and oscillator with different masses connected at the top of the central pile. The model accounts for a hysteretic constitutive model for the soil; pile elements for the foundation and beam elements for all the oscillator components were adopted. The interface between piles and soil was simulated by elasto-plastic springs in both shear and normal direction. The preliminary analysis results presented herein have indicated that:

- the numerical model is able to adequately reproduce the free-field response;
- the resonance effects and the bending moment can be reproduced fairly accurately with the numerical model for the pile connected to the oscillator, both in terms of envelope and time history response;
- the soil-pile-structure interaction gives a measured period elongation of 42%: this elongation, which is usually underestimated by the simplified literature formulas is well reproduced by the numerical analysis;
- the numerical model is able to catch effectively the bending moment values due to kinematic interaction; while the maximum bending location is slightly different probably because the influence of the SDOF is not well accounted for.

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