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SOME EXPERIMETNAL EVIDENCES ON DYNAMIC SOIL-STRUCTURE INTERACTION

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Abstract. *Due to dynamic soil-structure interaction (DSSI), the foundation soil experiences an additional motion which is added to its free-field response and DSSI is ruled by the soil compliance and by the vibration of the structure. In this framework soil non-linear behavior, as well as soil-foundation interface non-linearity, can represent crucial aspects and may govern the seismic response of the overall soil-structure system. Experimental tests on physical models allow identifying the interaction mechanism and also provide a benchmark for theoretical and/or numerical analyses. The present paper describes some experimental evidences concerning dynamic soil-structure interaction. Specifically, the paper deals with the results of two shaking table tests performed on a physical model consisting of a one-storey steel frame and a sand deposit. The steel frame (1.3 x 0.95 x 1.3 m) represents a 1:6 scaled model of a one-storey reinforced concrete building prototype. To reproduce the effects of the foundation soil on DSSI, a special laminar box (5m x 1m x 1.2m) was placed over the table, fixed to it and then filled with dry Leighton Buzzard sand up to a depth of 90 cm. During the tests sine-dwell excitations, with different acceleration amplitudes and frequencies, were applied as input motions and the overall model response was monitored through a large set of accelerometers and displacement transducers. Selected experimental results are presented and discussed in the paper using both time- and frequency-domain representations of the acceleration responses. The obtained results highlight the influence of the frequency and of amplitude of the input motion on the coupled and/or uncoupled response of the considered soil-structure system. In some cases an uplifting of the foundation was clearly observed during the tests and represented a natural isolation for the system; accordingly, the accelerations recorded in the soil underneath the foundation are not completely transmitted.*

1 INTRODUCTION

Seismic response of structures depends on many factors related to both structural and geotechnical issues. Among the latter, the local site effects and the dynamic interaction between the foundation soils and the structure are the more relevant. The cyclic non-linear behavior of the foundation soil affects the soil-structure interaction under seismic condition, produces absorption of the incident wave energy and, generally, leads to a reduction in the energy of the waves which are transferred to the structure. This complex interaction mechanism, generally denoted as dynamic soil-structure interaction (*DSSI*), can be studied through different approaches. Theoretical or numerical approaches (e.g. [1-10]) are powerful tools even if the obtained results are affected by the reliability of the data and by the analysis assumptions. Monitoring of full-scale structures is frequently characterized by unknown boundary conditions and requires a large monitoring period which is a rare occurrence (e.g. [11-15]). Thus, experimental tests on laboratory physical models allow the main aspects of the dynamic soil-structure interaction mechanism to be identified, are useful to calibrate and to validate theoretical and or numerical models and, finally, provide a benchmark for theoretical and or numerical analyses (e.g. [16-23]).

The present paper deals with two shaking table tests performed on a physical model consisting of a one-storey steel frame (hereafter referred to as the "*model structure*") and a Leighton Buzzard sand deposit pluviated into a special laminar box (hereafter referred to as the "*shear stack*") consisting in a large flexible soil container, designed to reproduce, as far as possible, the kinematic response of the soil system. Some of the test results are presented in terms of acceleration responses and are discussed highlighting the influence of the frequency and amplitude of the input motion on the coupled and/or uncoupled response of the soil-structure system.

2 FACILITIES, MODEL SET-UP AND INSTRUMENTATIONS

The adopted shaking table consist of a six-degree-of-freedom shaking table (3x3 m) belonging to the old *Earthquake Engineering Research Centre* (*EERC*) at the University of Bristol. In order to reproduce the effect of the foundation soil the *shear stack* (5x1x2 m) was placed over the table, fixed to it and filled with soil using a special deposition procedure. Figure 1a shows a 3D view of the adopted facilities; further details can be found in [20,24].

The formation of a soil deposit with a controlled uniform relative density is a crucial aspect of the physical modeling since it significantly affects the response of the whole soil-structure system. For the experimental tests described herein the shear stack was filled with dry Leighton Buzzard sand using a special sieve designed for soil pluviation. A relative density D_r of about 50% and a low-strain shear modulus G_o of about 25 MPa can be estimated for the sand pluviated into the shear stack [24]; the reduction of the shear modulus and the increase in the damping ratio with shear strain level γ can be described using the relationships plotted in Figure 2.

The model structure consists of a one-storey steel frame characterized by a longitudinal frame span equal to 111 cm, a transverse span equal to 76 cm and a storey height equal to 130 cm. Figure 1b shows the main geometrical properties of the model structure. The model was designed using appropriate scaling factors [27-28]. A steel roof plate was located on the top of the structure and, to simulate a centered uniformly distributed surcharge, a number of lead blocks were placed and fixed on the roof plate. The significant difference between the weight of the steel roof plate surcharged with the lead blocks and the weight of the model allowed us to schematize the model structure as a single degree of freedom (*SDOF*) system of mass *m* having a natural frequency equal to 5.5 Hz and a flexural stiffness equal to about 555 kN/m. A detailed description of the dimensions and mechanical properties of the prototype can be found in [29].

Once the sand has been completely pluviated into the shear stack, the upper 10 cm were removed from the central area. This allowed us to locate the model structure, ensuring an embedment of about 10 cm to the foundation beams.

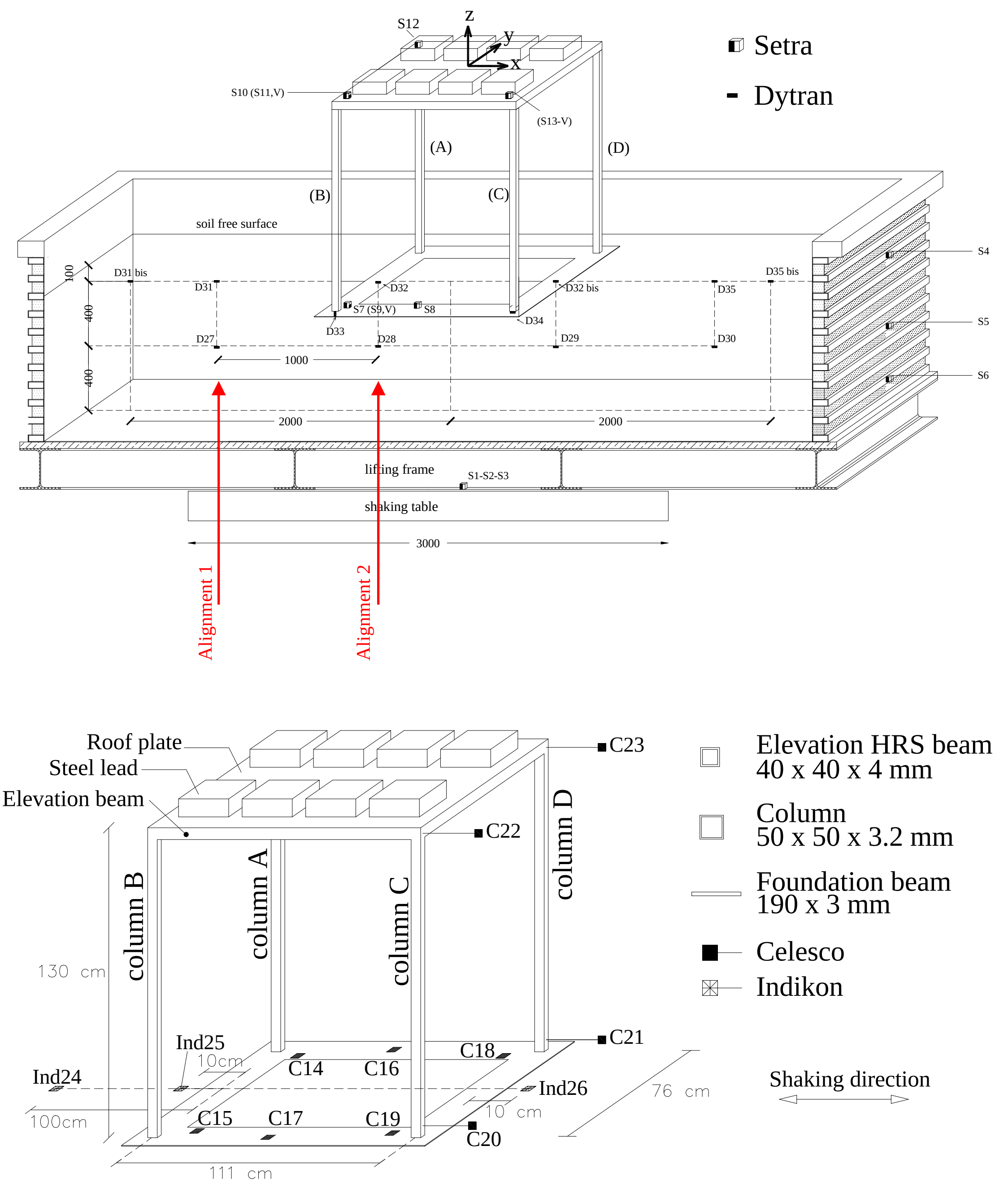


Figure 1: a) 3D schematic view of the physical model and of the adopted facilities and location of the accelerometers; b) schematic of the model structure: geometrical properties and location of displacement transducers.

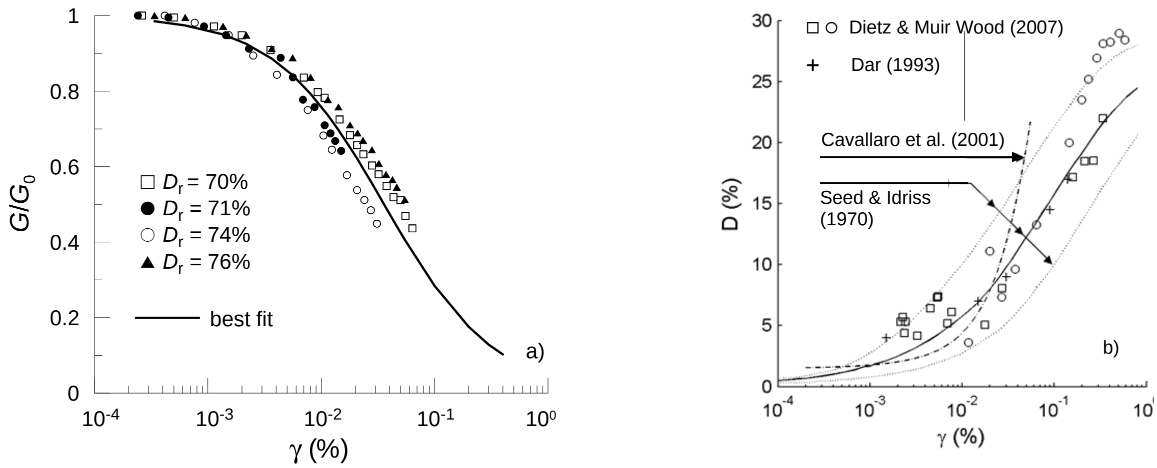


Figure 2: Leighton Buzzard sand: a) reduction of normalized shear modulus with shear strain [24]; b) damping ratio versus shear strain [25].

During the tests the dynamic response of the models was monitored through 10 Dytran accelerometers, 13 Setra piezoelectric-unidirectional accelerometers, 12 Celesco displacement transducers and 3 Indikon no-contact magnetic displacement transducers. The distribution of accelerometers (Fig. 1a) allows detecting the acceleration variation within the depth of the soil deposits. Celesco displacement transducers (Fig. 1b) were used to record the horizontal displacements imposed to the shaking table, the horizontal response of the shear stack and of the columns of the model structure and the vertical displacement of the foundations. Finally, Indikon displacement transducers (Fig. 1b) were used to record the "*free-field*" settlements of the sand deposit due to the densification induced by the motion.

3 EXPERIMENTAL ACTIVITIES

Two physical models (i.e. two soil-structure systems) were prepared and the experimental activities, hereafter identified as *Test 1* and *Test 2*, were carried out on both models.

Bothe tests consisted of two main phases: *i*) a preliminary white-noise excitation; *ii*) a sequence of sine-dwell excitations with a constant frequency f_i (f_i = 2 and 4 Hz in *Test 1* and *Test 2*, respectively) and peak acceleration amplitude A_i which increased run by run, from 0.08g to 0.53g in *Test 1* and from 0.04g to 0.68g in *Test 2*. A conclusive white-noise excitation was also applied at the end of *Test 2*.

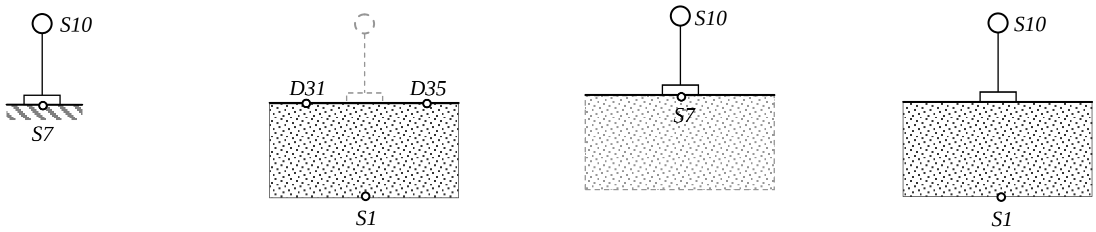
The white-noise consisted of random very low-level excitations having a very large frequency range (0 ÷ 100 Hz) and root mean square acceleration equal to 0.02 g, applied to the table in the direction of the main side of the shear stack. This test was performed in order to investigate the natural frequencies of vibration and the damping ratio of the soil deposit, of the model structure and of the whole soil-structure system. This test

For both *Test 1* and *Test 2* the dynamic excitation consisted of sine-dwell acceleration time-histories with an amplitude which increased from zero to the peak value A_i, during the first five cycles, remained constant in the following 20 cycles and then decreased down to zero in the last 5 cycles. *Test 1* was characterized by an input motion frequency lower than the natural frequency (f_{sf} = 3.5 Hz) of the whole soil-structure system (f_i/f_{sf} = 0.57); conversely for *Test 2* it was f_i/f_{sf} = 1.14.

During the sine-dwell excitation tests several shakes (named *runs*), characterized by different peak amplitudes A_i, were applied to the system. In the present paper, to focus on the effect of the frequency of the input motion on the observed responses, only the results obtained during the runs of *Test 1* (f_i = 2 Hz) and *Test 2* (f_i = 4 Hz) characterized by the same amplitudes A_i will be described and discussed. Table 1 shows the pairs of runs for which the comparison will be presented. Further details on the whole experimental activities can be found in [24].

Test 1 (f_i = 2 Hz)	run	*II*	*VI*	*VII*	*VIII*	*XI*
	A_i (g)	0.11	0.31	0.35	0.40	0.53
Test 2 (f_i = 4 Hz)	run	*II*	*IV*	*V*	*VI*	*VIII*
	A_i (g)	0.12	0.32	0.36	0.40	0.53

Table 1: Selected runs for the comparison between the responses recorded during the sine-dwell sequences of *Test 1* and *Test 2*.



Model structure "fixed" to the table		*Soil deposit*		*Model structure over the soil deposit*		*Soil-structure system*	
f_{ff} (Hz)	D_{ff} (%)	f_s (Hz)	D_s (%)	f_f (Hz)	D_f (%)	f_{sf} (Hz)	D_{sf} (%)
5.1	1.36	12.6	11.12	3.4	3.62	3.5	3.29

Table 2: Fundamental frequencies and damping ratios evaluated through the white-noise excitation tests before the sine-dwell excitation sequence.

4 DYNAMIC IDENTIFICATION OF THE SOIL-STRUCTURE SYSTEM

The main results of the white-noise excitation tests are shown in Table 2 in terms of values of the first natural frequency and of the damping ratio. The data listed in Table 2 refer to the model structure placed on the sand deposit (f_f, D_f), only to the soil deposit (f_s, D_s) and, finally, to the whole soil-structure system (f_{sf}, D_{sf}); for comparison, the values of the first natural frequency (f_{ff}) and of the damping ratio (D_{ff}) computed for the model structure "fixed" to the table, are also listed in the table.

It is apparent that the computed natural frequency for the model structure placed on the sand deposit (f_f = 3.4 Hz) is lower than that computed for the model structure "fixed" to the shaking table (f_{ff} = 5.1 Hz) and is very similar to that corresponding to the whole soil-structure system (f_{sf} = 3.5 Hz). Similarly, the damping ratio D_f = 3.62 % is higher than that estimated for the model structure "fixed" to the table (D_{ff} = 1.36 %) and is very similar to that of the whole soil-structure system (D_{sf} = 3.29 %).

It is worth nothing that the experimental values of the damping ratio obtained through the white noise excitation tests for the model structure fixed to the table (D_{ff} = 1.36 %) and for the whole soil-structure model (D_{sf} = 3.29 %) are lower than conventional values of modal damping ratio usually adopted in dynamic analysis of structures ([24]). However, the obtained value D_{ff} is consistent with the values of structural damping suggested by several authors (e.g.

[30-32]) for very low stress levels, as those imposed during the white noise excitation test. As expected, due to the effect of soil-structure dynamic interaction, the experimental damping ratio D_{sf} (soil-structure model) resultes greater than D_{ff}.

Also the drop in the natural frequency, from f_{ff} = 5.1 Hz to f_f = 3.4 Hz, clearly represents a consequence of dynamic soil-structure interaction. The value f_{ff} = 5.1 Hz is very close to that (f_{ff} = 5.5 Hz) estimated, analytically, with reference to the theoretical scheme of a fixed-base model structure. The difference could be ascribed to the actual constraint conditions of the steel model, which was fixed to the shaking table using bolts.

For the soil deposit, the white-noise excitation tests provide a value of the natural frequency (f_s = 12.6 Hz) greater than those of the frame and of the whole soil-structure system; the latter is essentially governed by the frame natural frequency.

5 TEST RESULTS AND DISCUSSION

During the sine-dwell excitations the behavior of the soil-structure system was investigated in terms of accelerations and displacements responses. Herein, the acceleration responses recorded during the selected runs (Table 1) will be presented, using both time- and frequency-domain representations, and analyzed to detect amplification and de-amplification phenomena inside the soil deposit and along the model structure. To this purpose, the *alignments 1* and *2*, indicated in Figure 1b, were considered.

Figures 3 and 4 show a comparison between the acceleration time-histories recorded during *Test 1* and *Test 2* for *alignment 1* and *alignment 2*, respectively. For each pair of runs having the same input acceleration amplitude A_i, the plots in Figures 3 and 4 clearly show that the accelerations recorded during *Test 1* (f_i = 2 Hz) are always larger than those recorded during *Test 2* (f_i = 4 Hz). Since each pair of runs differs only in the input frequency f_i, the influence of this parameter, on both the free-field soil response (*alignment 1* – Fig. 3) and on the coupled soil-structure response (*alignment 2* – Fig. 4), is clearly evident.

For both *alignment 1* (Fig. 3) and *alignment 2* (Fig. 4), at a distance of 40 cm from the table (*D27* and *D28*), lower acceleration values can be observed for all the runs of both tests. This de-amplification, which significantly influenced the response of the whole soil-structure system, is typical in presence of a rigid base (as in the case of a shaking table experiment).

As it moves towards the soil surface, the acceleration starts to grow. Along *alignment 1* (free-field conditions) the acceleration at the soil surface (*D31*) reaches approximately the value A_i of the input motion only for the runs of *Test 1* with A_i = 0.11g and 0.31g; for the other runs of *Test 1* and for the all the runs of *Test 2* the acceleration recorded at the soil surface are lower than A_i. Along *alignment 2* it is possible to identify a clear amplification at the foundation level for all the runs of *Test 1*; however, for input motions with lower amplitudes (A_i = 0.11g and 0.31g) the acceleration recorded in the soil (*D32*) was not completely transmitted to the foundation. Actually, an up-lifting of the foundation occurred during all the runs of *Test 1*; this influenced the time-histories recorded by *D32* and *S7*, which appear clearly different from those recorded by both *D28* and *S10*. Along *alignment 2*, the time-histories recorded during *Test 2* at the foundation level (*S7*) show acceleration amplitudes approximately equal to the peak input value; conversely, lower values were recorded by *D32*. Finally, moving from the foundation of the model structure to its roof, the acceleration increases in the case A_i = 0.11g, is approximately constant in the case A_i = 0.31 g and decreases for all the remaining runs.

The acceleration responses recorded during the selected runs (Table 1) were also analyzed in the frequency-domain. The amplification functions (*AFs*) were computed from the Fourier amplitude spectrum (*FAS*) evaluated for each of the recorded acceleration responses.

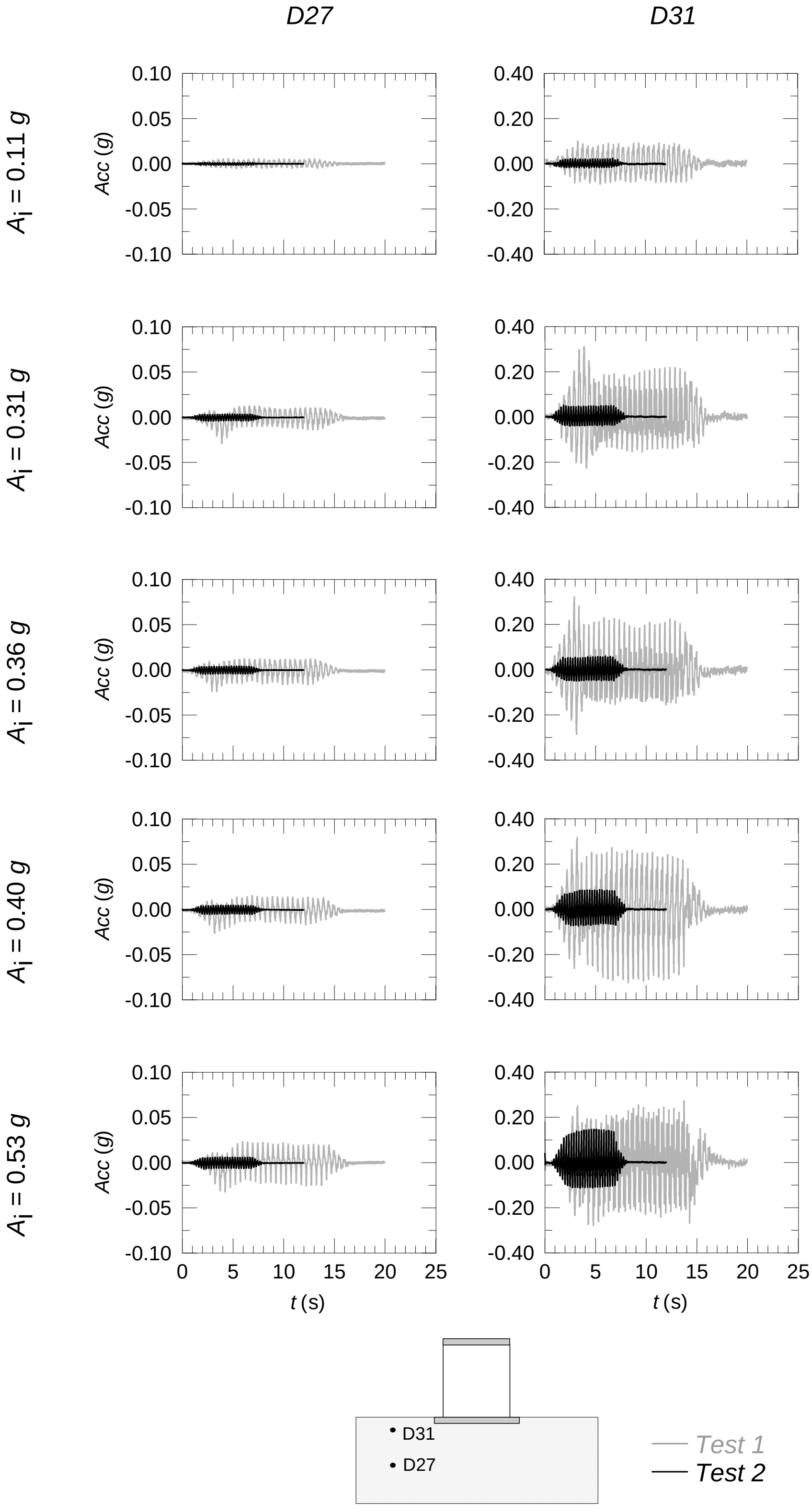


Figure 3: Horizontal accelerations recorded during *Test 1* and *Test 2* by the accelerometers *D27* and *D31* along *alignment 1* (free-field condition).

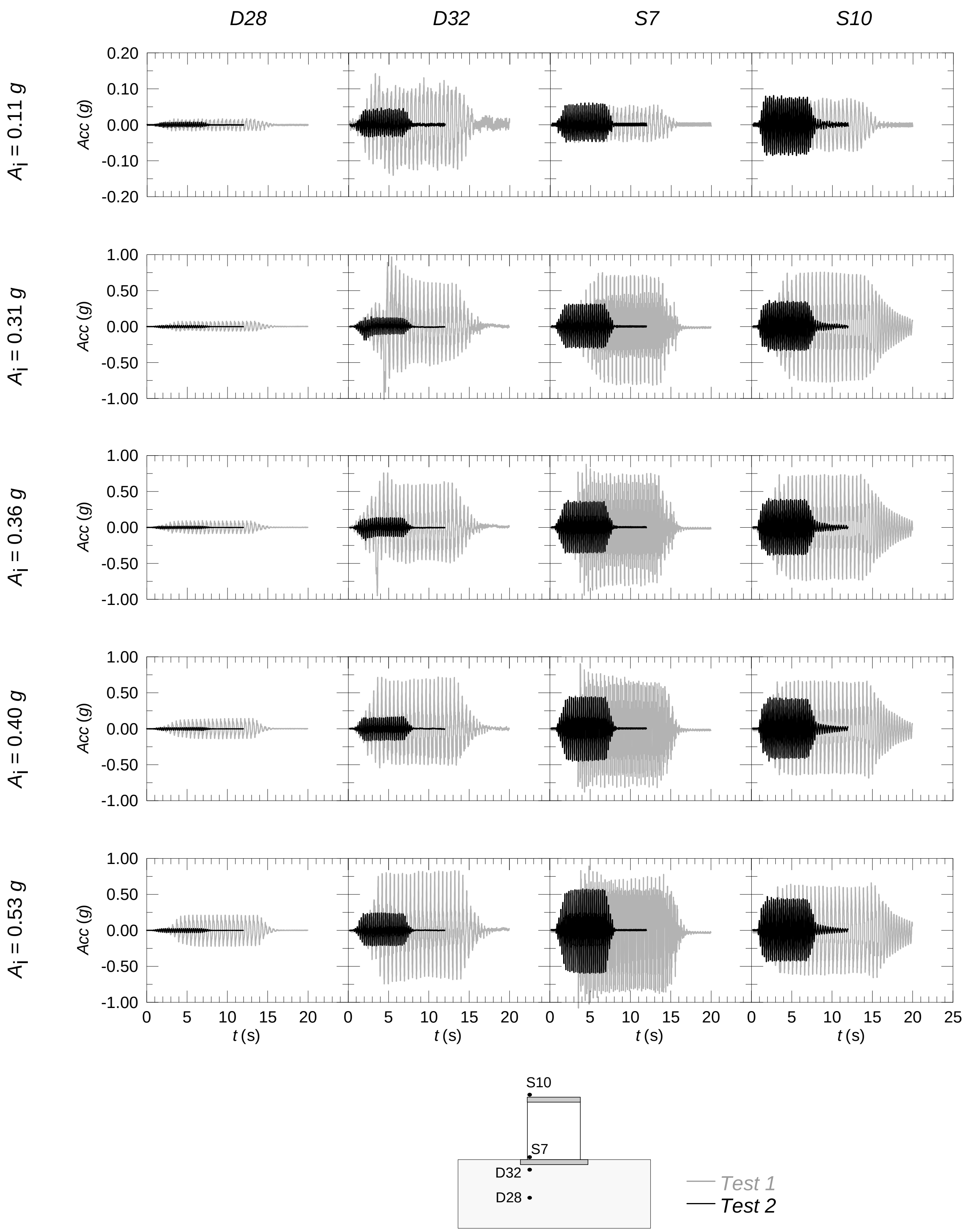


Figure 4: Horizontal accelerations recorded during *Test 1* and *Test 2* by the accelerometers *D28* and *D32* along *alignment 2* (involving the soil and the model structure).

The results are presented in Figures 5 and 6 for some of the runs listed in Table 1. Details on the procedure adopted to estimate the *AFs* can be found in [24]. During the runs characterized by low input motion amplitudes (A_i = 0.11 *g*; Figs. 5a and 6a), the *AF*s computed for both tests clearly show a narrow band of amplification at frequencies very close to the natural frequency of the model structure (f_f = 3.4 Hz; Tab. 3). As the amplitude of the motion increases (Figs. 5 b,c and 6 b,c) the bandwidth of the computed amplification functions increases and the frequency corresponding to the peak of *AFs* decreases.

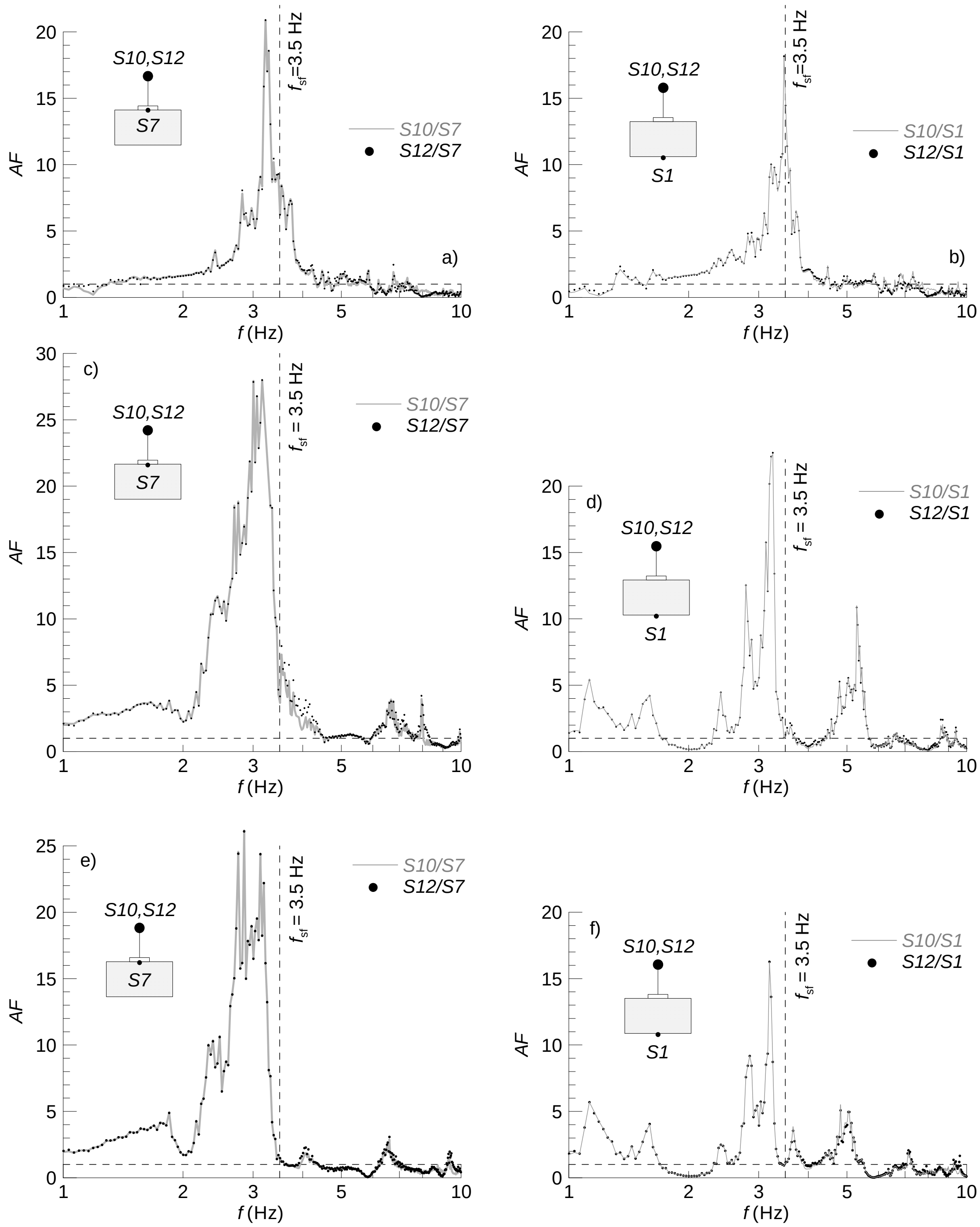


Figure 5: Amplification functions computed for some of the acceleration time-histories recorded during some selected runs of *Test 1*: a,b) A_i = 0.11 *g*; c,d) A_i = 0.36 *g*; e,f) A_i = 0.53 *g*.

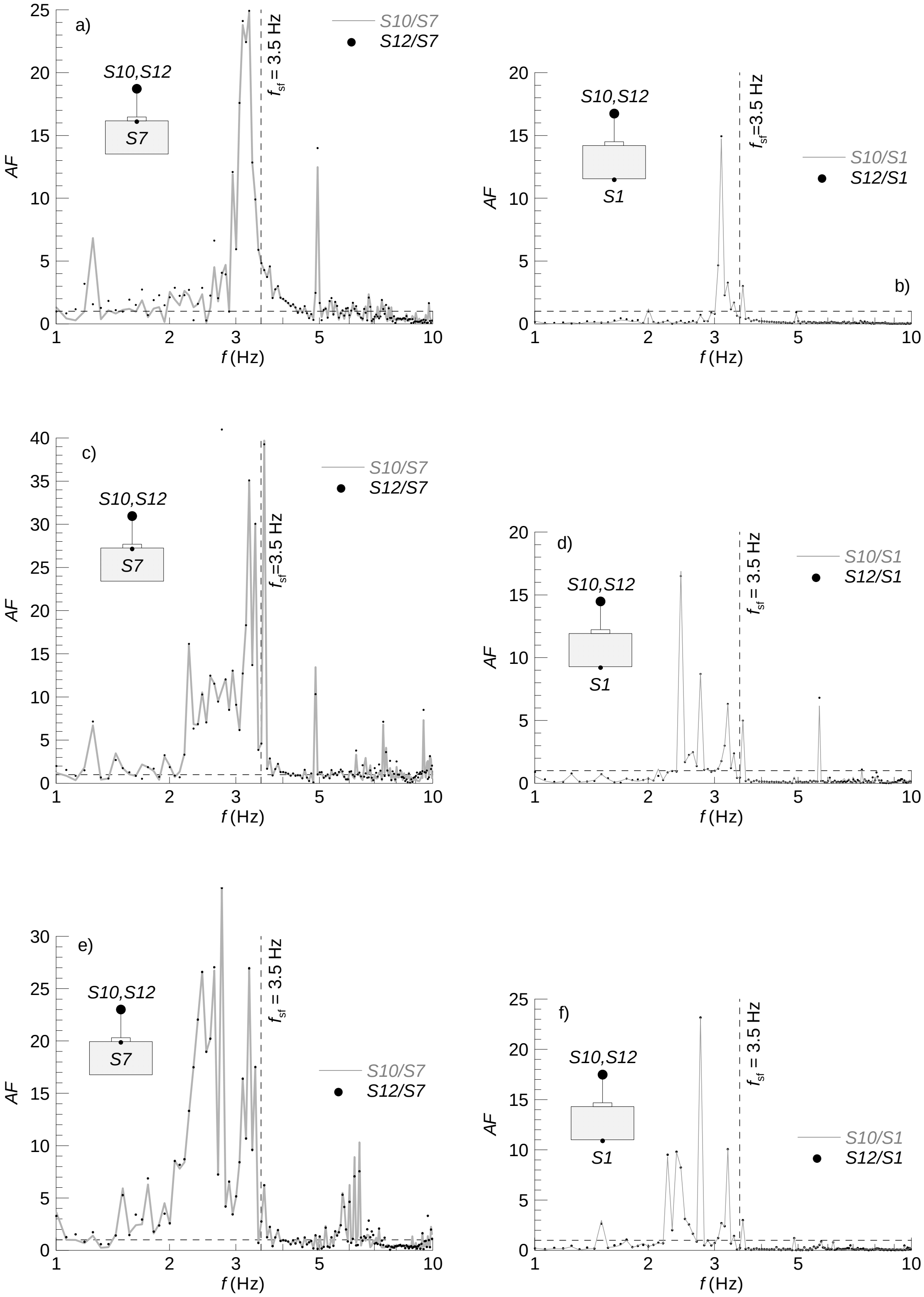


Figure 6: Amplification functions computed for the acceleration time-histories recorded during some selected runs of *Test 2*: a, b) A_i = 0.11 g; c,d) A_i = 0.36 g; e,f) A_i = 0.53 g.

Since the response of the steel model was elastic, this shifting of the amplification band can be attributed only to the tilting motion which was observed during those runs characterized by larger amplitude of the input motion.

As regards the coupled soil-frame system, during *Test 1* (Figs. 5 b,d,f) a narrow amplification band can again be observed whatever is the amplitude of the input motion. In the case A_i = 0.11g (Fig.5b) the peak of the *AF* occurs at about 3.5 Hz, which coincides with the natural frequency of the soil-frame system (Tab. 2). Moreover, as the amplitude of the input motion increases, the frequency corresponding to the peak of the *AF*s reduces (Figs. 5 d,f), i.e. it becomes closer to the input frequency f_i = 2 Hz in *Test 1* and farther from f_i = 4 Hz in *Test 2*. This shifting of the amplification band could be attributed to the effect of the non-linear soil behavior. In fact, as the amplitude of the imposed motion increases, larger strains arise in the soil deposit leading to a reduction of the soil shear modulus. Accordingly, the natural frequencies of the soil deposit reduce. Similar conclusions can be drawn with reference to the amplification functions computed for the selected runs of *Test 2* (Figs. 6 b,d,f); in this case the shifting of the amplification band is more evident.

Due to the shifting of the natural frequency of the coupled soil-structure system toward the value f_i = 2 Hz, the *AF* amplitudes computed for *Test 1* are generally higher than those evaluated for *Test 2* (Figs. 5 and 6). This result is consistent with those obtained analyzing the acceleration response in the time domain (Figs. 3 and 4).

6 CONCLUDING REMARKS

The paper describes the results of two sets of shaking table experiments (*Test 1* and *Test 2*) performed on a physical model consisting of a Leighton Buzzard sand deposit and a model structure consisting of a one-storey steel frame. Both tests included preliminary white-noise excitations and sine-dwell excitations having a constant input frequency (f_i = 2 Hz in *Test 1*; f_i = 4 Hz in *Test 2*) and input acceleration amplitude A_i ranging in the intervals 0.08-0.53 g during *Test 1* and 0.04-0.68 g during *Test 2*.

The white noise excitation tests allowed us estimating the natural frequency of the soil deposit (f_s = 12.6 Hz) and the natural frequency for the model structure placed on the sand deposit (f_f = 3.4 Hz); this is lower than the natural frequency of the model structure "*fixed*" to the shaking table (f_{ff} = 5.1 Hz) and is very similar to that of the coupled sand-frame system (f_{sf} = 3.5 Hz). The input frequencies of the two sets of sine-dwell excitations were selected to be somewhat lower and somewhat higher than f_{sf} .

The results obtained during some selected runs of *Test 1* and *Test 2* were analyzed using both time- and frequency-domain representations of the acceleration responses.

Due to non-linear effects of soil cyclic behaviour a failure was observed at a depth of about 40 cm from the shaking table, producing a natural isolation for both the soil deposit and the model structure. Moving towards the soil surface the acceleration starts to grow, even if the de-amplification observed at a depth of 40 cm influences the response of the whole soil-structure system. In some cases the accelerations recorded in the soil underneath the foundation were not completely transmitted to it as a result of uplifting phenomena, which were clearly observed.

Horizontal accelerations recorded during *Test 1* (f_i = 2 Hz) are definitely higher than those recorded during *Test 2* (f_i = 4 Hz) even if the input acceleration amplitudes are the same for each pair of the selected runs (Table 1). This confirms the significant influence of the frequency of the input motion and highlights the importance of the coupling with the natural frequency of the system.

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REFERENCES

[1] J.P. Stewart, G.L. Fenves, R.B. Seed, Seismic soil-structure interaction in buildings. I: analytical methods. *J. Geotechnical and Geoenvironmental Engineering*, **125** (1), 26-37, 1999.

[2] J.P. Stewart, G.L. Fenves, R.B. Seed, Seismic soil-structure interaction in buildings. II: empirical findings. *J. Geotechnical and Geoenvironmental Engineering*, **125** (1), 38-48, 1999.

[3] M.R. Massimino, Experimental and numerical modelling of a scaled soil-structure system. Maugeri M. Editor. *Seismic Prevention of Damage for Mediterranean Cities, a case History: the City of Catania (Italy)*, Wit Press, 2005.

[4] G. Abate, M.R. Massimino, M. Maugeri, Finite element modeling of a shaking table test to evaluate the dynamic behaviour of a soil-foundation system. *AIP Conference Proceedings*, **1020**, 569-576, 2008.

[5] R. Paolucci, M. Shirato, M.T. Yilmaz, Seismic behaviour of shallow foundations: shaking table experiments vs numerical modelling. *Earthquake Engineering and Structural Dynamics*, **37**, 577-595, 2008.

[6] A. Cavallaro, M.R. Massimino, M. Maugeri, Noto Cathedral: soil and foundation investigation. *Construction and Building Materials Journal.* **17**, 533-541.

[7] G. Abate, C. Caruso, M.R. Massimino, M. Maugeri, Evaluation of shallow foundation settlements by an elasto-plastic kinematic-isotropic hardening numerical model for granular soil. *Geomechanics and Geoengineering Journal*. **3**(1), 27-40, 2008.

[8] M. Maugeri, G. Abate, M.R. Massimino, Soil-structure interaction for seismic improvement of Noto Cathedral (Italy). Sakr M.A., Ansal A. Eds. *Special Topics in Earthquake Geotechnical Engineering in Geotechnical, Geological and Earthquake Engineering*. Springer, 2012.

[9] G. Abate, M.R. Massimino, M. Maugeri, Numerical modelling of centrifuge tests on tunnel-soil systems. *Bulletin of Earthquake Engineering*, 2014.

[10] F. Grassi, M.R. Massimino, The evaluation of kinematic bending moments in a pile foundation using complete 3D finite element modelling. S. Syngellakis Editor. *Earthquake ground motion: Input Definition for Aseismic Design.* Wit Press, 147-161.

[11] D.E. Hudson, Dynamic tests of full-scale structures.Wiegel, R.L. Editor. *Earthquake engineering*, Prentice Hall, 127-149, 1970.

[12] M.D. Trifunac, M.I. Todorovska, Recording and interpreting earthquake response of full-scale structures. *NATO Workshop on Strong Motion Instrumentation for Civil Engineering Structures*, Istanbul, June 2-5, 1999.

[13] E. Cascone, G. Biondi, A case study on soil settlements induced by preloading and vertical drains. Geotextiles and Geomembranes, **38**, 51–67, 2013.

[14] E. Cascone, V. Bandini, A. Galletta, G. Biondi. Acceleration of the consolidation process of a clay soil by preloading and vertical drains: field measurements and numerical predictions. Karstunen & Leoni Ed.s, *2nd International Workshop on Geotechnics of Soft Soils - Focus on Ground Improvement*. Glasgow, September 3-5, 2008.

[15] M. Maugeri, F.Castelli, M.R. Massimino, G. Verona, Observed and computed settlements of two shallow foundations on sand. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE,* **124 (**7), 1998.

[16] A.W. Crewe, M.L. Lings, C.A. Taylor, A.K. Yeung, R. Andrighetto, Development of a large shear-stack for resting dry sand and simple direct foundations on a shaking table. Proc. *5th SECED Conference on European seismic design practice*, Balkema, 1995.

[17] M. Maugeri, G. Musumeci, D. Novità, C.A. Taylor, Shaking table test of failure of a shallow foundation subjected to an eccentric load. *Soil dynamic and Earthquake Engineering*, **20**, 435-444, 2000.

[18] G. Biondi, P. Capilleri, M. Maugeri, Dynamic response analysis of earth-retaining walls by means of shaking table tests. *8th U.S. National Conference on Earthquake Engineering*, San Francisco, April 18-22, 2006.

[19] I. Anastasopoulos, V. Drosos, N. Antonaki, A. Rontogianni, The role of soil-foundation-structure interaction on the performance of an existing 3-storey building: shaking table testing. *15 WCEE*, Lisboa, 2013.

[20] G. Biondi, M.R. Massimino, M. Maugeri, C. Taylor. Influence of the input motion characteristics on the dynamic soil-structure interaction by shaking table tests. Fourth International Conference on Earthquake Resistant Engineering Structures, ERES IV, Ancona, Italy, September 22, 2003.

[21] G. Abate, M.R. Massimino, M. Maugeri, Finite element modelling of a shaking table test to evaluate the dynamic behaviour of a soil-foundation system. Proc. Seismic Engineering International Conference Commemorating the 1908 Messina and Reggio Calabria Earthquake, MERCEA 2008, Reggio Calabria, Italy, July 8-11, 2008.

[22] G. Abate, M.R. Massimino, M. Maugeri, D. Muir Wood, Numerical modelling of a shaking table test for soil-foundation-superstructure interaction by means of a soil constitutive model implemented in a FEM code. *Geotechnical and geological engineering*, **28**, 37-59, 2010.

[23] M.R. Massimino, M. Maugeri, Physical modelling of shaking table tests on dynamic soil-foundation interaction and numerical and analytical simulation. *Soil Dynamic and Earthquake Engineering Journal*. **49**,1–18, 2013.

[24] G. Biondi, M.R. Massimino, M. Maugeri, Influence of frequency content and amplitude of input motion in DSSI investigated by shaking table tests. *Bulletin of Earthquake Engineering*, 1-35, 2014.

[25] A. Cavallaro, M. Maugeri, R. Mazzarella, Static and dynamic properties of Leighton Buzzard sand from laboratory tests. *4th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamic and Symposium in honour of Prof. WD Liam Finn*, San Diego, March 26–31, 2001.

[26] M. Dietz, D. Muir Wood, Shaking table evaluation of dynamic soil properties. *4th International conference on earthquake geotechnical engineering*, June 25–28, 2007.

[27] S. Iai, Similitude for shaking table tests on soil-structure-fluid model in 1g gravitational field. *Soils and Foundations*, **29**(1), 105-118, 1989.

[28] S. Iai, T. Sugano, Soil-structure interaction studies through shaking table tests. *Earthquake Geotechnical Engineering*, Balkema, 927-940, 1999.

[29] M. Maugeri, D. Novità, C. Taylor, Unidirectional shaking table tests of 1:6 reduced scale steel model. *12th European Conference on Earthquake Engineering*, London, 2002.

[30] G.W. Housner, R.R. Martel, J.L. Alford, Spectrum analysis of strong-motion earthquakes. *Bulletin of Seismological Society of America*, **43**: 97-119, 1953.

[31] N.M. Newmark, W.J. Hall, Seismic design criteria for nuclear reactor facilities. *4th World Conference on Earthquake Engineering*, Santiago, Chile, 1969.

[32] J.D. Stevenson, Structural damping values as a function of dynamic response stress and deformation levels. *Nuclear Engineering and Design*, **60**: 211-237, 1980.