Invited papers

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Dynamic behaviour of a coupled soil-structure system by means of FEM analyses finalized to the seismic risk mitigation of a school in Catania (Italy)

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ABSTRACT: Seismic design of new structures, as well as retrofitting and improving of existing ones basically depend on many factors, among which the Dynamic Soil Structure Interaction (DSSI). It can lead to significant modifications of the free-field motion. Thus, full-coupled soil-structure analyses should be performed to achieve appropriate structural designs.

The present paper shows the DSSI FEM analysis concerning a school building in Catania, characterized by a high seismic risk. In order to take into soil non-linearity, modified shear moduli and soil damping ratios were evaluated firstly according to EC8 (2003) and secondly considering the effective strain level.

The model was analyzed in the time and frequency domains. The main goals are: to investigate the soil filtering effect; to compare the achieved amplification factors and response spectra with those furnished by NTC (2018); to evaluate the influence of different modeling of soil non-linearity on the dynamic response of the system.

1 INTRODUCTION

The evaluation of the seismic risk in the city of Catania, which is highly subjected to severe seismic events, is the first step for any site response analyses. Since seismic risk is a combination of site hazard and vulnerability of the structures, estimating the seismic input that really impacts the structure is fundamental, because the soil very often has a strategic filtering effect in terms of maximum acceleration at the foundation level and also in terms of fundamental periods of the structures (Bonaccorso et al. 2005; Castelli et al. 2008; Groholski et al. 2010). Studies of the dynamic behavior of coupled soil-structure systems should be consistently encouraged, because the risk of erroneously evaluating the seismic response of a structure is very high without these studies (Mylonakis et al. 2000; Massimino & Scuderi 2009; Maugeri et al. 2012; Gatto et al. 2015). Since the 1970s, Dynamic Soil Structure Interaction (DSSI) has been investigated by means of theoretical approaches (Veletsos & Meek 1974; Gazetas 1983; 1991) and numerical modelling (Gazetas & Apostolou 2004; Massimino 2005; Abate et al. 2015) as well as field and laboratory tests (Faccioli et al. 2001; Kutter & Wilson 2006; Pitilakis et al. 2018). In particular, numerical modelling of coupled soil-structure systems is the most valuable approach, being the nearest to the actual configurations to be analyzed (Abate & Massimino 2016; Abate et al. 2017).

The present paper deals with the 2-D FEM modelling of a full-coupled soil-structure system, i.e. a school building in Catania, characterized by a high seismic hazard (Grasso & Maugeri 2009; Castelli et al. 2016). The building and its subsoil were subjected to investigations in the framework of the POR-FESR Research Project Sicilia 2007-2013, which was aimed at reducing the seismic risk in Eastern Sicily (Abate et al. 2018). The soil non-linearity, extremely important in soil mechanics (Abate et al. 2007; Pecker et al. 2013; Massimino & Biondi 2015), was taken into account adopting: i) degraded shear moduli G and increased soil damping ratios D, according to the EC8 suggestions (EC8-Part 5 2003); ii) G and D

corresponding to the effective reached strain level γ. The response of the full-coupled system was compared with the free-field site response in the time and frequency domains.

2 THE CASE HISTORY

2.1 *The structure*

The analyzed building hosts the Nazario Sauro school in Catania, designed and built before Italy was declared a seismic zone. The building is made of reinforced concrete frames, with isolated footings placed at different depths from the ground floor. The frame chosen for the SSI analyses is shown in Figures 1.a and 1.b, where its dimensions are also reported.

2.2 *The foundation soil*

The adopted soil modeling was based on the geotechnical investigation performed as part of the POR-FESR Project Sicilia 2007-2013 aimed at reducing seismic risk in Eastern Sicily. Two boreholes, S1 and S2, were drilled to depths of 40 m and 30 m, respectively, allowing for a detailed stratigraphy shown in Figure 2.a. Moreover, two seismic dilatometer tests were performed in the S1 borehole obtaining the V_s profiles shown in Figure 2.b. According to this profile, three soil layers were modeled, assuming the bedrock depth equal to 30 m and the soil of type E (EC8-Part 1 2003; NTC 2018). A micro-tremor survey (HVSR test; Figure 3a) was also conducted inside the test area to check the seismic properties of the soil: the fundamental

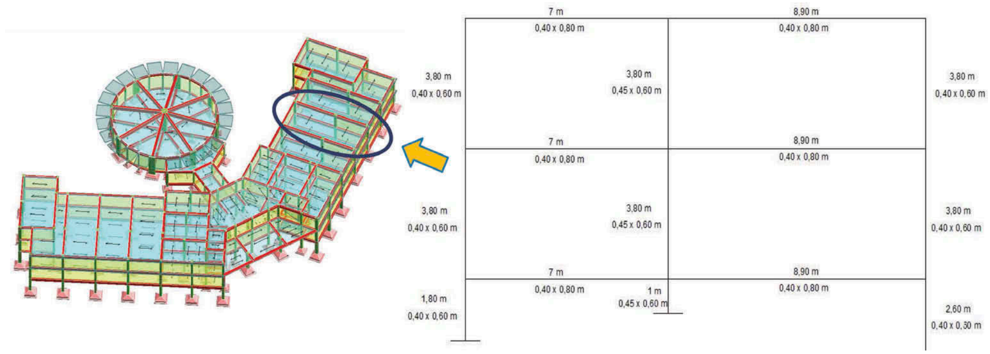

Figure 1. School building: a) isometric view; b) chosen frame.

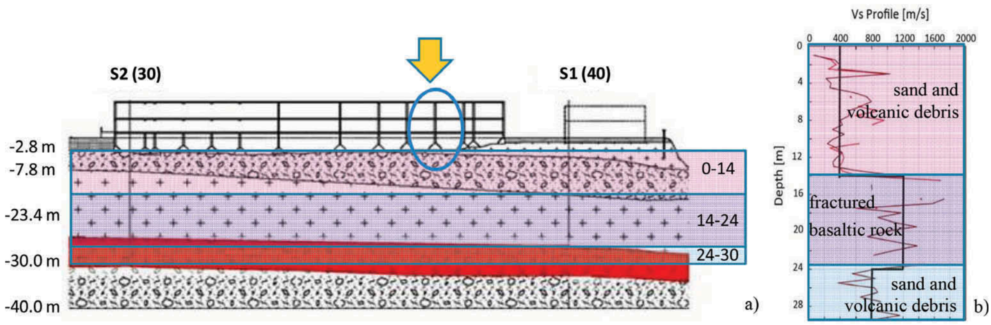

Figure 2. a) Stratigraphy based on the two boreholes (with the localization of the chosen frame); b) V_s profiles achieved by SDMT tests.

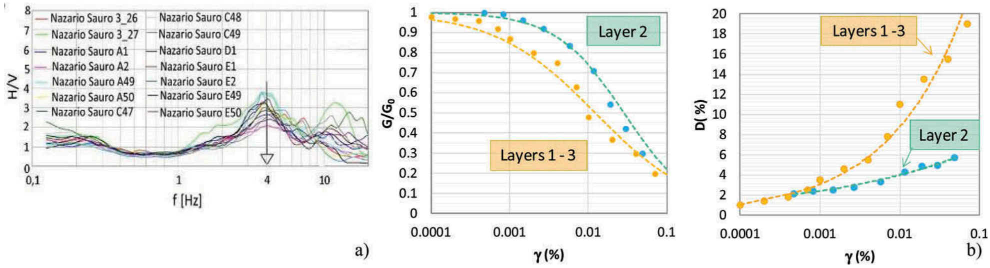

Figure 3. a) Results of the HVSR test. b) *G-γ* and *D-γ* curves adopted for the three soil layers.

frequency of the soil foundation was evaluated as approximately equal to 4 Hz. Resonant column tests (Figure 3b) were performed for the layers of sand and volcanic debris. For the fractured basaltic rock layer, the *G-γ* and *D-γ* curves were adopted with reference to soils characterized by similar V_s values.

3 ADOPTED SEISMIC INPUTS

Seven seismograms were applied at the bedrock of the soil deposit: three synthetic seismograms evaluated assuming the source to be along the Hyblean-Maltese fault and generating the 1693 seismic ground motion scenario, assumed as a level I earthquake scenario (Grasso & Maugeri 2014); three synthetic generating the 1818 seismic ground motion scenario, assumed as a level II earthquake scenario (Cavallaro et al. 2006); one accelerogram recorded during the 1990 earthquake at the Sortino station. In order to fit these seismograms at the reference area, they were appropriately scaled to a value a_g = 0.245g, which is the expected value at the bedrock at the site where the school is located, for life safety SLV (V_R = 712 years) for building of class of use III, such as the chosen one according to the NTC 2018. The adopted inputs differ in frequency content and significant duration (as shown in Table 1, where the Intensity of Arias I_A and the first two fundamental input frequencies f_1 and f_2 are reported).

4 THE ADOPTED FULL-COUPLED NUMERICAL MODEL

4.1 *The modeling*

The full-coupled soil-structure system was modeled by means of the ADINA FEM code (ADINA 2008). Figure 4 shows the adopted mesh, including the geometry, boundary and loading conditions. The width of the soil deposit was chosen in order to minimize boundary effects as far as possible (equal to 5*B*, where *B* is the width of the structure); the height of the soil deposit was derived from the geotechnical investigations (H = 30 m). The soil was divided into 3 layers, according to the stratigraphy and the V_s profile shown in Figure 2.

As regards the boundary conditions, the nodes of the soil vertical boundaries were linked by "constraint equations" that imposed the same displacements at the same depths (Abate & Massimino 2016); the nodes at the base of the mesh were constrained only in vertical

Table 1. Main parameters of the adopted seismic inputs for the FEM analyses

	1693(1)	1693(2)	1693(3)	1818(1)	1818(2)	1818(3)	1990
I_A	0.798	0.809	0.769	0.801	1.133	0.479	0.607
f_1	0.94	3.86	1.7	1.5	0.67	0.6	1.8
f_2	3.8	0.4	0.7	2.44	1.1	2.2	1.1

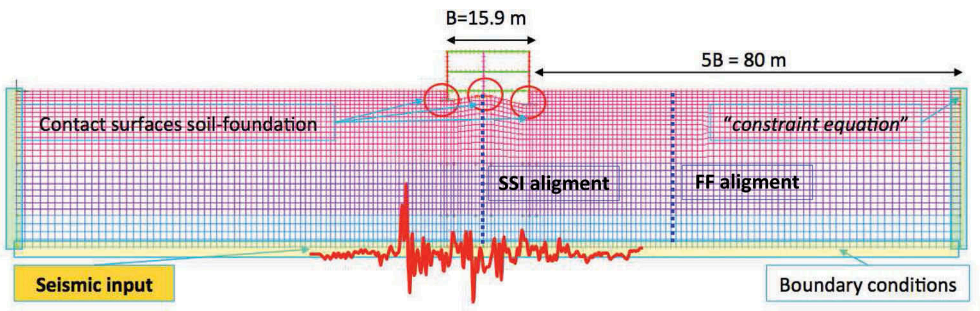

Figure 4. Adopted mesh with geometry, boundary and loading conditions.

direction. In order to model probable uplifting and/or sliding phenomena, special foundation-soil contacts were modeled considering a friction equal to 2/3 φ. About the loading conditions, the weight of the model, non-structural loads and the seven seismograms previously described were applied.

The structure was modeled by means of beam elements, with a linear visco-elastic constitutive model characterized by the conventional properties of reinforced concrete (E = 28500 MPa, v = 0.25, γ = 25 kN/m^3, D = 5%). The soil was modeled by 2D solid elements, by means of a visco-elastic constitutive model, taking into consideration its non-linear behavior by means of two different approaches. By the first approach, the soil was modeled as a linear equivalent visco-elastic material adopting degraded shear moduli G and increased damping ratios D depending on the coefficient $\alpha = S \times a_g(g)$, according to EC8 (2003) suggestions. EC8 (2003) furnishes a range of G values for soil types C and D (Tab.4.1 EC8 2003) and suggests greater G values for stiffer soil profiles. For the analyzed case-history, the expected surface acceleration a_g x S_S (being S_s the stratigraphic coefficient equal to 1.34 for the analyzed site for life safety SLV) was greater than 0.3g, so D was fixed equal to 10% for all the three layers, and G was chosen as equal to $0.36 \cdot G_0$ for soil layers 1 and 3, and a minor degradation ($0.56 \cdot G_0$) was chosen for soil layer 2, because it is a very stiff layer. By the second approach, the values of G and D were chosen according to an iterative sub-routine (Figure 5.a) based on the G-γ and D-γ curves shown in Figures 3.b, considering the effective strain level γ obtained for each soil layer (considering the soil column underneath the structure) and for each different input, as is summarized in Figure 5. As it is possible to see in Figure 5.b, for soil layer 1, the iterative procedure furnished values of G/G_0 and D close to the values fixed by EC8; for soil layer 2, due to the very small values of effective strain level γ, the iterative procedure gave a negligible degradation of the dynamic parameters, different from that suggested by EC8; for soil layer 3, the degradation of G and D was less evident than that fixed by EC8.

The Rayleigh damping factors α and β adopted in the numerical modeling were computed as $\alpha = D \cdot \omega$ and $\beta = D/\omega$ (Lanzo et al. 2004), being D the damping ratio and $\omega = 2\pi f$ the angular frequency of the soil or of the structure, computed by the following frequencies of the soil and the structure: f_{soil} = 2.95 Hz, according to the well-known expression $T = V_s/4H$; $f_{STRU,FB}$ = 2.69 Hz, according to the expression: $T = C_1$ x h$^{3/4}$ (NTC 2008).

4.2 *The results*

The main achieved results are shown in terms of amplification functions and acceleration ratios and they are compared with reference to: i) two different vertical alignments (SSI and FF alignments shown in Figure 4), below and far from the structure; ii) the two different approaches adopted for modeling the soil non-linearity.

As for the amplification functions, Figure 6 shows: in the first two columns, the soil amplification functions $A(f)$ for the seven seismic inputs, for the two alignments and for the two approaches adopted for modeling soil non-linearity; in the third column, the ratios between

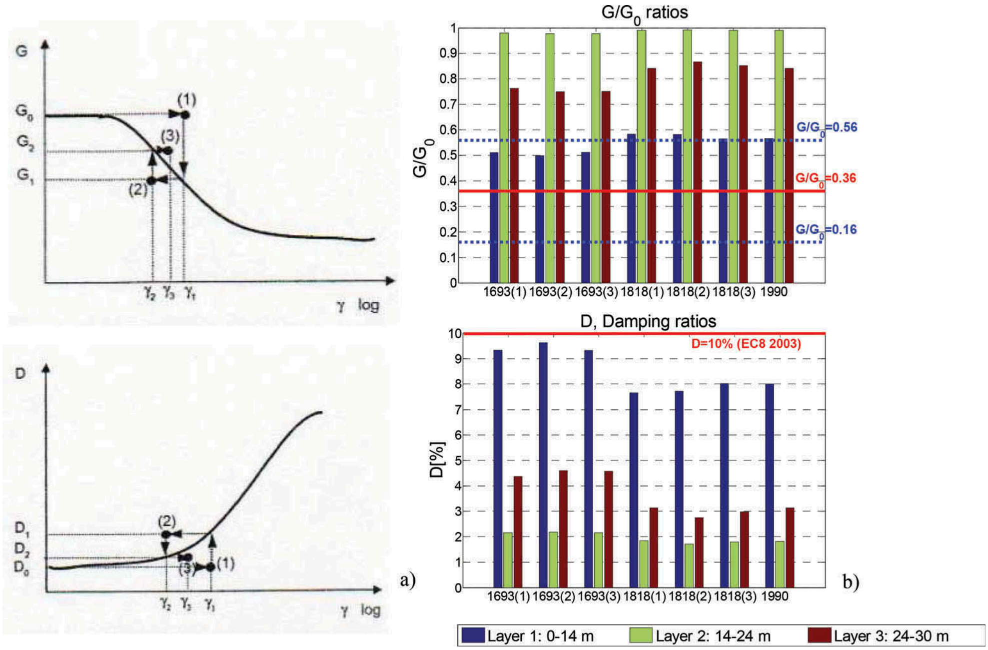

Figure 5. a) Iterative procedure for defining the G and D values according to the effective strain level γ; b) Adopted seismic parameters for each soil layer and for each different input.

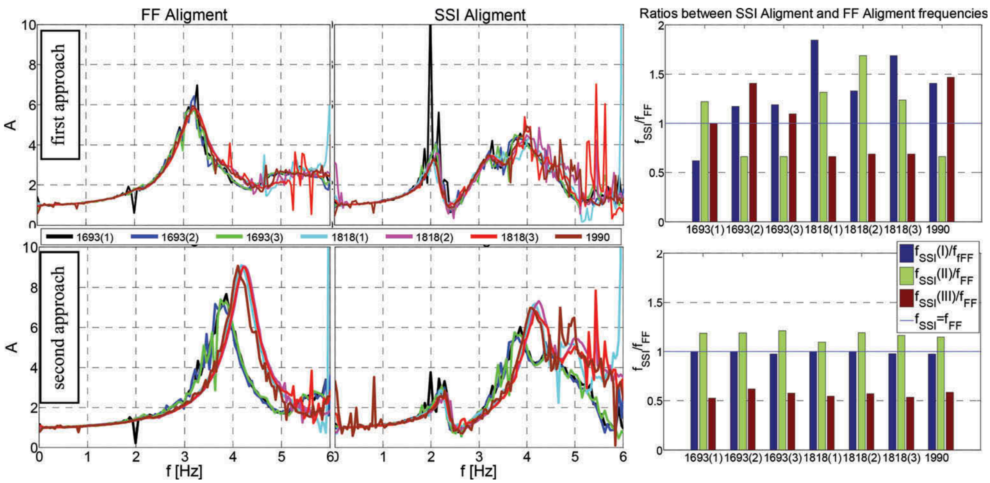

Figure 6. Amplification functions achieved by both the adopted approaches for modeling soil non-linearity: along the FF (first column) and SSI (second column) alignments. Ratios between SSI alignment and FF aligment frequencies (third column) for the adopted inputs and for the adopted approaches.

SSI alignment and FF aligment frequencies for the seven inputs achieved by both the adopted approaches. $A(f)$ was evaluated as the ratio between the Fourier amplitude spectrum computed at the foundation level and the Fourier amplitude spectrum computed at the bedrock level, i.e. referring to the soil only, considering both the SSI alignment and the FF aligment. By the first approach, the average natural frequency of the soil in free-field condition was about 3.2 Hz; this value was similar to the value obtained by the HVSR test (4 Hz), nevertheless there was a slight difference. Along the SSI alignment, three natural frequencies of the soil are evident: $f_{SSI}(I)$ = 2 Hz; $f_{SSI}(II)$ = 4 Hz; f_{SSI} III) = 5.4 Hz. Considering the second approach, the average natural frequency of the soil in free-field condition was 4 Hz and it coincided with that calculated by HVSR. Along the SSI alignment, the following three natural frequencies of the soil are evident: $f_{SSI}(I)$ = 2.2 Hz; $f_{SSI}(II)$ = 4 Hz; $f_{SSI}(III)$ = 5.4 Hz. These values were very similar to those achieved by the first approach: this was due to the predominant role of the structure in the soil response. The different results achieved by the two approaches were due to the rough estimation of the soil non-linearity suggested by EC8; the gap was overcome by the second approach, which fixed G and D in relation to the stress-strain level reached. The ratios between the three soil natural frequencies for the SSI alignment and the soil natural frequencies f_{FF} for the FF alignment for each accelerogram and for the two soil non-linearity modelling are often far from unit value: the natural frequency of the soil was strongly influenced by the presence of the structure. So performing site response analyses in FF conditions to estimate the design acceleration of structures could not be sufficient.

Finally, Figure 7 shows the amplification ratios R_a for the two alignments and for each adopted input, both as profiles R_a-z and as values at the ground surface. All the accelerograms were subjected to an evident amplification within the shallow layer with almost vertical trends at deeper layers. The comparison between the SSI alignment and the FF alignment shows that the presence of the structure generated a strong amplification at the ground surface compared to free-field conditions. Therefore, in these cases, taking into account soil-structure interaction for the seismic safety of buildings is of fundamental importance. Moreover, the values obtained by the iterative procedure were always greater than the values reached by the EC8 suggestions. This latter result was due to the lower values of D estimated according to the second approach (Figure 6), as was observed earlier. Finally, the values of R_a at the

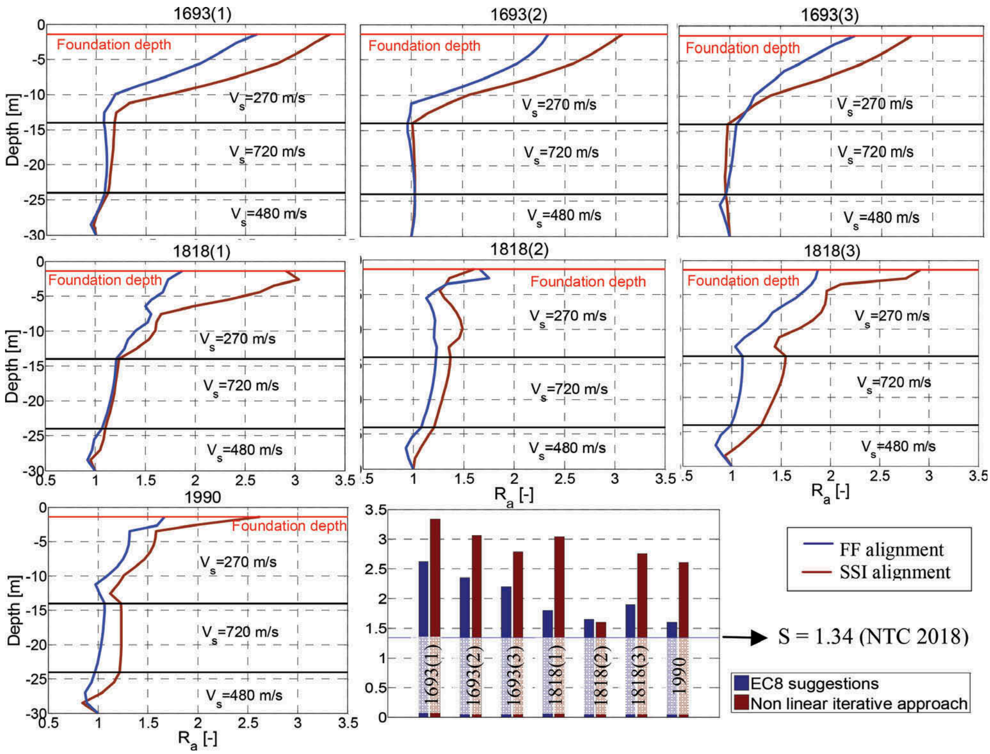

Figure 7. Amplification ratios profiles along the two analyzed alignments and achieved by both the approaches for modeling soil non-linearity.

foundation level were always greater than the value provided by the Italian technical code (NTC 2018) for soil type E: $S = 1.34$.

5 CONCLUSIONS

The paper deals with 2-D FEM analyses of the dynamic behaviour of a full-coupled soil-structure system, performed in order to highlight the importance of considering DSSI and to evaluate the influence of different modelling of soil non-linearity on the dynamic response of the system. So, the dynamic response was analysed comparing both two different alignments, below the structure (SSI) and far from it (FF), and two different approaches for taking into account soil-nonlinearity: the EC8 suggestions regarding the expected acceleration at the ground level and an iterative procedure according to the strain level reached in the soil.

The presence of the structure clearly modified the frequency content of the soil, showing basically three soil fundamental frequencies along the SSI alignment, for both the two approaches. This result is also observable by the ratios between soil frequencies for the SSI alignment and soil frequency for the FF alignment: they were often far from unit value. Moreover, the presence of the structure generated a strong amplification at the ground surface for some accelerograms which in free-field conditions were subjected to a much lower amplification. These results underline the importance of taking into account the soil-structure interaction for the seismic safety of buildings. Modeling soil non-linearity by G and D values chosen considering the effective reached strain level γ, the $A(f)$ peaks move towards greater frequencies, in comparison with the $A(f)$ peaks achieved modeling soil non-linearity by values of G and D chosen according to EC8 suggestions. This was due to higher G values and lower D values estimated using the iterative procedure. However, the first adopted approach had a great computational advantage for the designer who wants to take into account the soil non-linearity without adopting onerous procedures. The numerical amplification ratios R_a found at the ground surface were greater than the stratigraphic amplification value S_s calculated by NTC 2018, and moreover the values obtained by the second approach were always higher than the values achieved by the first one. This latter result was due to the lower values of D estimated according to the iterative procedure.

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