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RESPONSE SPECTRUM CONSIDERING SOIL-STRUCTURE INTERACTION FOR BUILDINGS WITH SHALLOW FOUNDATION

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ABSTRACT

Soil-structure interaction (SSI) is not taken into account for building design, according to European seismic design code. A numerical investigation on SSI effects is held using the one-directional three-component seismic wave propagation approach in a T-shaped soil domain (1DT-3C). The 1DT-3C approach for SSI analyses consists on modeling the soil fully three-dimensional until a fixed depth and as one-dimensional until the soil-bedrock interface, in a finite element scheme. In this manner, rocking effects can be simulated, and the foundation deformability is considered. Differences in terms of structural response between the cases of soil domain assembled with the frame structure (one-step analysis) and of fixed-base structure subjected to the free-field motion at its base (two-step analysis) are highlighted. A parametric study shows a similar trend of the response spectrum when SSI is taken into account, for different buildings, in a linear-elastic regime.

Keywords: Soil-structure interaction, building design, 3-Component seismic loading, Finite Element Method

1. INTRODUCTION

Soil-structure interaction (SSI) is not taken into account for buildings with shallow foundation in European seismic design codes. The variation in seismic demand due to SSI, influenced by structural dynamic features, soil stratigraphy and input motion characteristics (Saez et al., 2011) should be taken into account.

The one-directional propagation of a three-component earthquake (1D-3C), in a multilayered soil profile with a building at the surface, is simulated by Santisi d'Avila and Lopez Caballero (2018), using quadratic line finite elements with three nodes, and Fares et al. (2017a), using 20-node solid elements having unit area. The three-dimensional (3D) frame structure is meshed using Timoshenko beam elements. The 1D-3C approach for SSI analyses reduces modeling difficulties and computational cost, compared with the case of 3D soil domain.

The seismic wave propagation model where a T-shaped soil profile is used (1DT-3C approach), proposed by Fares et al. (2017b), is a correction for SSI analyses to consider rocking effects and the foundation deformability, and allows structure-soil-structure interaction (SSSI) studies. A fully 3D soil is considered until a fixed depth, beneath which no evidence of SSI is observed. A unit area soil column is considered a good approximation to model the soil domain until the bedrock interface, in the layers where SSI effects are negligible.

A parametric analysis is undertaken using the 1DT-3C modeling technique to estimate the variation in the building seismic response due to SSI. The results obtained, in terms of peak acceleration, by direct solution of the dynamic equilibrium equation for the soil-foundation-structure system (one-step analysis), are compared to those yielded by a fixed-base (FB) building model subjected to the free-field (FF) motion at its base (two-step analysis). A response spectrum, taking into account SSI, is obtained for buildings with shallow foundation, in the linear-elastic regime.

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2. 1D-3C SEISMIC WAVE PROPAGATION IN A T-SHAPED SOIL DOMAIN

The 1DT-3C modeling technique is applied for SSI analyses considering homogeneous material properties in the horizontal directions *x* and *y* (Figure 1). Seismic waves propagate vertically in *z*-direction. The soil model is fully 3D for the top layers until a predefined depth, beneath which the SSI has negligible effects, and a one-dimensional (1D) soil model is used until the bedrock. The proposed T-shaped soil domain allows rocking to be simulated and the foundation deformability to be taken into account. The soil is meshed using 20-node solid finite elements. A constraint equation is used to condense out the degrees of freedom at the base of the 3D soil domain to those at the top of the unit area soil column.

An absorbing boundary condition is applied at the soil-bedrock interface, using dashpots having damping coefficient equal to $A\rho v_{sb}$ in horizontal directions and $A\rho v_{pb}$ in vertical direction, respectively, as proposed by Lysmer and Kuhlemeyer (1969). The bedrock density is ρ, v_{sb}, and v_{pb} are the shear and compressional wave velocity in the elastic bedrock and A is the soil domain area. The seismic load is given at the soil-bedrock interface as forces, which amplitudes are equal to $A\rho v_{sb}\,\dot{u}_{gx}(t)$, $A\rho v_{sb}\,\dot{u}_{gy}(t)$ and $A\rho v_{pb}\,\dot{u}_{gz}(t)$ in *x*-, *y*- and *z*-direction, respectively, where $\dot{u}_g(t)$ is the incident wave velocity at the bedrock level. A periodic boundary condition (tie constraint) is imposed to obtain zero horizontal strains in the horizontally layered soil (Zienkiewicz et al., 1989).

The minimum number of quadratic elements in a soil layer, in *z*-direction, is assumed equal to $p\,f\,h/2v_s$, where $p = 10$ is the minimum number of nodes per wavelength, $f = 15\,\mathrm{Hz}$ is the maximum frequency of interest, *h* is the thickness of the layer, and v_s is the minimum wave velocity in the layer. The Abaqus software (Abaqus User Manual, 2014) is used for this research.

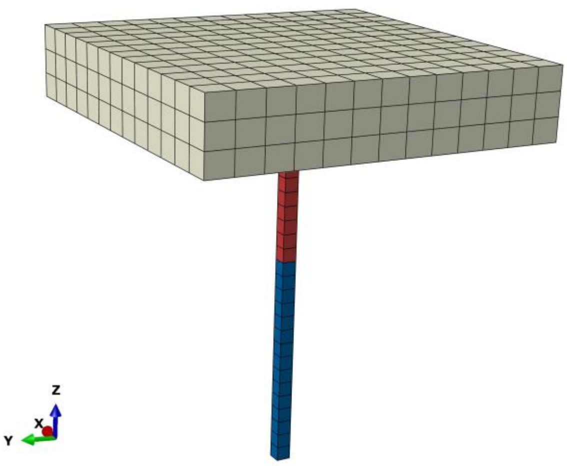

Figure 1. T-shaped soil profile

3. 3D FRAME STRUCTURE

The 3D frame structure is modeled using Timoshenko beam elements. It is rigidly connected at its base, node to node (Figure 2), to the 3D shallow foundation that is meshed using 20-node solid elements. The Timoshenko beam model considers transverse shear deformation as not negligible. Accordingly, the transverse shear stiffness is defined as χGA, where G is the shear modulus, A is the beam cross-sectional area and $\chi = (5(1+\nu))/(6+5\nu)$ is the shear coefficient (Kaneko 1975) for a Poisson's ratio ν. Rayleigh approach (Chopra 1995) is adopted to consider the damping effect of non-structural elements. The damping matrix is estimated as $\mathbf{C} = a\mathbf{M} + b\mathbf{K}$, where $\mathbf{M}$ is the consistent mass matrix and $\mathbf{K}$ is the stiffness matrix. The parameters $a = 2\zeta w_1 w_2/(w_1 + w_2)$ and $b = 2\zeta/(w_1 + w_2)$ are calculated using the first two natural angular frequencies w_1 and w_2 of the structure, and it is assumed a damping ratio $\zeta = 5\%$, as in structural design of typical reinforced concrete buildings.

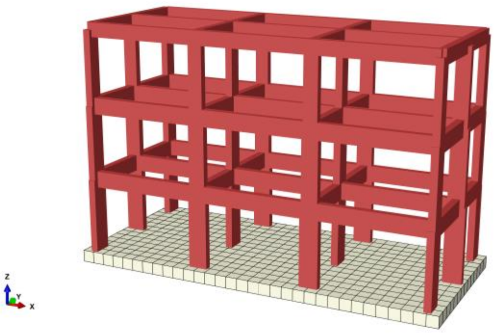

Figure 2. Structure-foundation assembly

4. ASSEMBLY DEFINITION

The discrete dynamic equilibrium equation of the structure-foundation-soil assembly is written as

$$\mathbf{M}\Delta\ddot{\mathbf{d}} + \mathbf{C}\Delta\dot{\mathbf{d}} + \mathbf{K}\Delta\mathbf{d} = \Delta\mathbf{F} \tag{1}$$

where $\Delta\ddot{\mathbf{d}}$, $\Delta\dot{\mathbf{d}}$ and $\Delta\mathbf{d}$ are the increment of acceleration, velocity, and displacement vector, respectively, and $\Delta\mathbf{F}$ is the increment vector of the seismic loading. Equation 1 is solved directly. The implicit dynamic problem is solved step by step using the Hilber-Hughes-Taylor algorithm (Hughes, 1987) that allows to remove high-frequency noise without impacting the meaningful lower frequency response. A slight numerical damping is adopted using the parameters $\alpha = -0.1$ (α needs to verify $-0.5 \le \alpha \le 0$), $\beta = 0.25(1-\alpha)^2 = 0.3025$ and $\gamma = 0.5 - \alpha = 0.6$. The interested reader can refer to Hughes (1987) for the numerical solution of the differential Equation 1 using these parameters.
The soil domain area involved in the SSI has to be defined to reduce the model. Moreover, the soil domain depth involved in the SSI has to be fixed to know where the 1D propagation model can be a sufficient approximation.

4.1 Definition of the soil domain area

The analyzed soil stratigraphy and mechanical properties are given in Table 1 where v_s and v_p are respectively the shear and compressional wave velocities.

Table 1. Stratigraphy and mechanical properties of the soil profile

Depth (m)	**Density (kg/m^3)**	**v_s (m/s)**	**v_p (m/s)**
0 - 5	1930	160	1256
5 - 15	1930	170	1275
15 - 30	1930	180	1293
> 30	2100	1000	2449

The floor plan of the three-floor building structure is shown in Figure 3A and the mechanical parameters are listed in Table 2. The dimensions of the rectangular cross-section columns are 30x80 cm², 30x70 cm² and 30x60 cm² for the first, second and third floor, respectively, and that of beams are 30x70 cm² for the first and second floor and 30x60 cm² for the third floor. The interstory height, the adopted elastic modulus in compression, the Poisson's ratio, and the density of the structure, are reported in Table 2. The dead load is converted in consistent mass of beams.

Table 2. Dimensions and mechanical parameters of the structure

Interstory height (m)	**Elastic modulus in compression (N/mm^2)**	**Density (kg/m^3)**	**Poisson's ratio**	**Dead load (kg/m^2)**
3.2	31220	2500	0.2	800

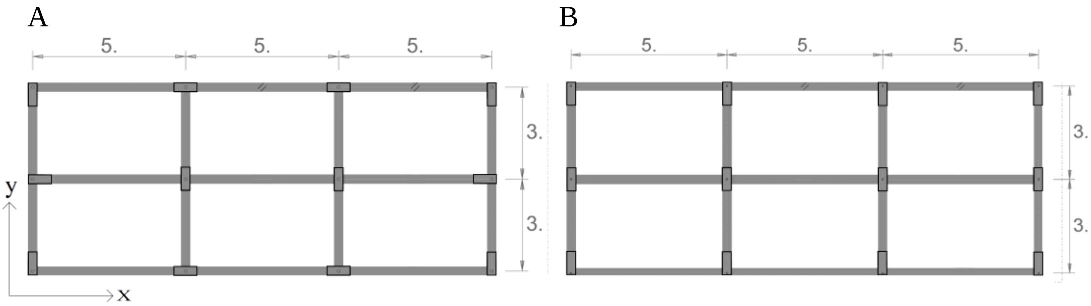

Figure 3. Floor plans of the analyzed structures identified by letters A and B

The soil domain area is defined by evaluating the building base to bedrock transfer function (TF) for different areas. The soil domain area is assumed as the minimum one providing a TF with a peak corresponding to the soil column frequency in the free-field case. Note that assuming a too small soil domain area gives unrealistic results with a soil too much influenced by the building deformation. On the contrary, selecting a too large soil domain area leads to vanish the SSI effect with a soil mass too high and a negligible building deformation effect. Figure 4 shows the obtained TF. The soil area $A = 25 \times 25\,\mathrm{m}^2$ is selected in the case of the softest soil and adopted for all analyses.

The elastic modulus in compression and density have to be corrected as EA and ρA, respectively, in the 1D soil domain that is modeled using unit area solid elements. This correction does not impact the soil frequency since the stiffness to mass ratio remains constant.

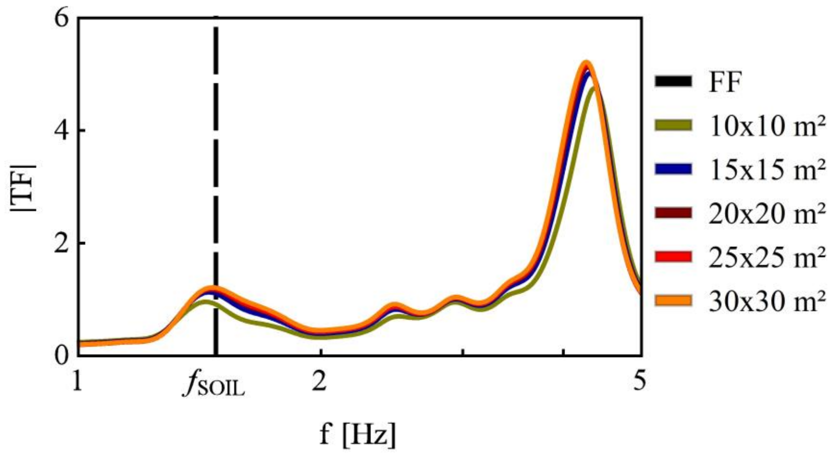

Figure 4. Building base to bedrock Transfer Function for different soil domain areas

4.2 Definition of the 3D soil thickness

The 3D soil thickness is the depth until which a 3D soil domain should be assumed and a 1D propagation model is considered a sufficient approximation for deeper layers. The simulation of vertical propagation of seismic waves is done using a fully 3D soil domain having the previously selected area to verify the assumption of the 3D soil thickness. A synthetic signal is applied. The predominant frequency of the signal f_q is chosen equal to the first frequency of the structure, to incite the structure. The synthetic signal, proposed by Mavroeidis and Papageorgiou (2002), is

$$\dot{u}_0(t) = \dot{u}_{0\max}/2\left[1+\cos(2\pi f_q/n(t-t_0))\right]\cos\left(2\pi f_q(t-t_0)\right) \quad 0 \le t \le 2t_0; \quad \dot{u}_0(t)=0 \quad t>2t_0 \tag{2}$$

where $t_0 = n/2f_q$ is the time of envelope's peak and $n=5$ is the number of cycles. The velocity peak $\dot{u}_{0\max}$ corresponds to an acceleration peak of $0.1 m/s^2$.

The impact of the building deformation on the soil xz shear strain at a fixed time (Figure 5, right) is compared with the free-field case (Figure 5, left). In the analyzed case, the first five meters of soil are influenced by the building. Underneath, no significant change in the shear strain is observed due to the building presence. Consequently, the first five meters of soil are modeled as fully 3D in the following study, due to the influence of the building on the soil.

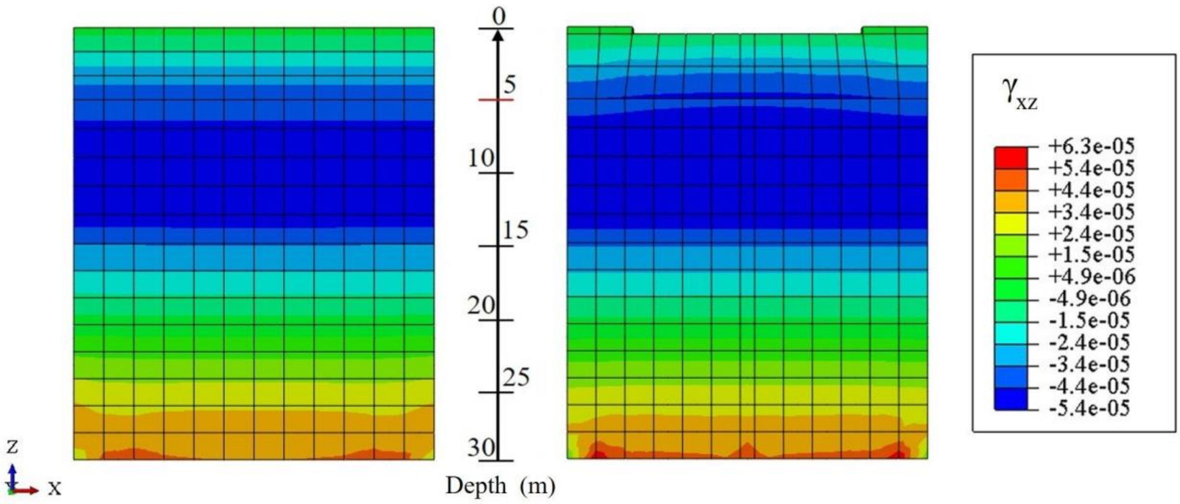

Figure 5. Cartography of *xz* shear strain in the soil for the cases of FF (left) and SSI (right)

5. PARAMETRIC INVESTIGATION ON SSI

5.1 Modeling description

The response of a T-shaped soil domain having different mechanical features, with the same building at the surface, is simulated to investigate the effect of soil properties on the SSI effects in the case of buildings with shallow foundation. The analyzed stratigraphies are listed in Table 3 and classified according to the Eurocode 8 soil categories. The shear wave velocity for each layer is arbitrary fixed. According to the relationships presented by Boore (2015), the density and compressional wave velocity are then calculated. The Poisson's ratio is evaluated as $\nu = \left(0.5v_p^2/v_s^2-1\right)/\left(v_p^2/v_s^2-1\right)$. The bedrock parameters are $\rho = 2100\,\mathrm{Kg/m^3}$, $v_{sb} = 1000\,\mathrm{m/s^2}$, and $v_{pb} = 2449\,\mathrm{m/s^2}$.

The same analysis is done for five different buildings having different natural frequencies. The floor plan shown in Figure 3A (named A in Table 4) is adopted for a three-, five- and seven-floor building and the floor plan in Figure 3B (named B in Table 4) is adopted for a three- and five-floor building. The dimensions of the rectangular cross-section of columns are 30x80 cm² and 30x70 cm² for the first and second floor, respectively, and 30x60 cm² for higher floors. The dimensions of the rectangular cross-section of beams are 30x70 cm² for the first and second floor and 30x60 cm² for higher floors. Mechanical parameters are reported in Table 2 and the fundamental frequency f_{BLDG} of these structures, associated to the translational mode shape in *x*-direction, is given in Table 4.

A synthetic signal (Equation 2) is applied in the direction of the first mode shape of the structure. The predominant frequency of the signal f_q is chosen to excite the first frequency of the structure. Then, a recorded signal of the 6 April 2009 Mw 6.3 L'Aquila earthquake (UTC 1:32) is applied to use a multifrequency input loading. This signal is recorded at the Antrodoco (ANT) station of the Italian strong motion network, localized in Lazio region (Italy). It is a FF station in a flat surface, with a slope angle lower than 15°. The soil type is classified as A, according to the Eurocode 8, but the v_{s30} is not measured. The epicentral distance is 26.2 km. Only the North-South (NS) component having peak ground acceleration $0.26\,\mathrm{m/s^2}$ is applied in the direction of the first mode shape of the structure. The time step of input signals is $dt = 0.005\,\mathrm{s}$.

Table 3. Stratigraphy and mechanical properties of the analyzed soil profiles

Depth	**Category**	**ρ**	**v_s**	**v_p**	**f_{SOIL}**	**Depth**	**Category**	**ρ**	**v_s**	**v_p**	**f_{SOIL}**
m		kg/m^3	m/s	m/s	Hz	m		kg/m^3	m/s	m/s	Hz
0 - 5	A	1999	450	1741	7.53	0 - 5	B	1957	360	1601	5.39
5-15		2108	750	2156		5-15		2020	500	1815	
15 - 30		2166	950	2400		15 - 30		2092	700	2091	
0 - 5	B	1937	320	1536	4.51	0 - 5	B	1930	280	1469	4.1
5-15		1976	400	1664		5-15		1957	360	1601	
15 - 30		2058	600	1957		15 - 30		2039	550	1887	
0 - 5	B	1930	250	1417	3.78	0 - 5	B	1930	240	1400	3.36
5-15		1947	340	1568		5-15		1932	310	1519	
15 - 30		2020	500	1815		15 - 30		1994	440	1726	
0 - 5	C	1930	230	1382	3.04	0 - 5	C	1930	220	1365	2.76
5-15		1930	280	1469		5-15		1930	260	1435	
15 - 30		1976	400	1664		15 - 30		1957	360	1601	
0 - 5	C	1930	200	1329	2.47	0 - 5	C	1930	180	1293	2
5-15		1930	240	1400		5-15		1930	210	1347	
15 - 30		1926	300	1502		15 - 30		1930	250	1417	
0 - 5	D	1930	160	1256	1.48						
5-15		1930	170	1275							
15 - 30		1930	180	1293							

Table 4. Fundamental frequency of the analyzed frame structures

Building	**Number of floors**	**Floor plan**	**f_{BLDG}** Hz
1	3	A	3.77
2	3	B	2.77
3	5	A	2.17
4	5	B	1.66
5	7	A	1.49

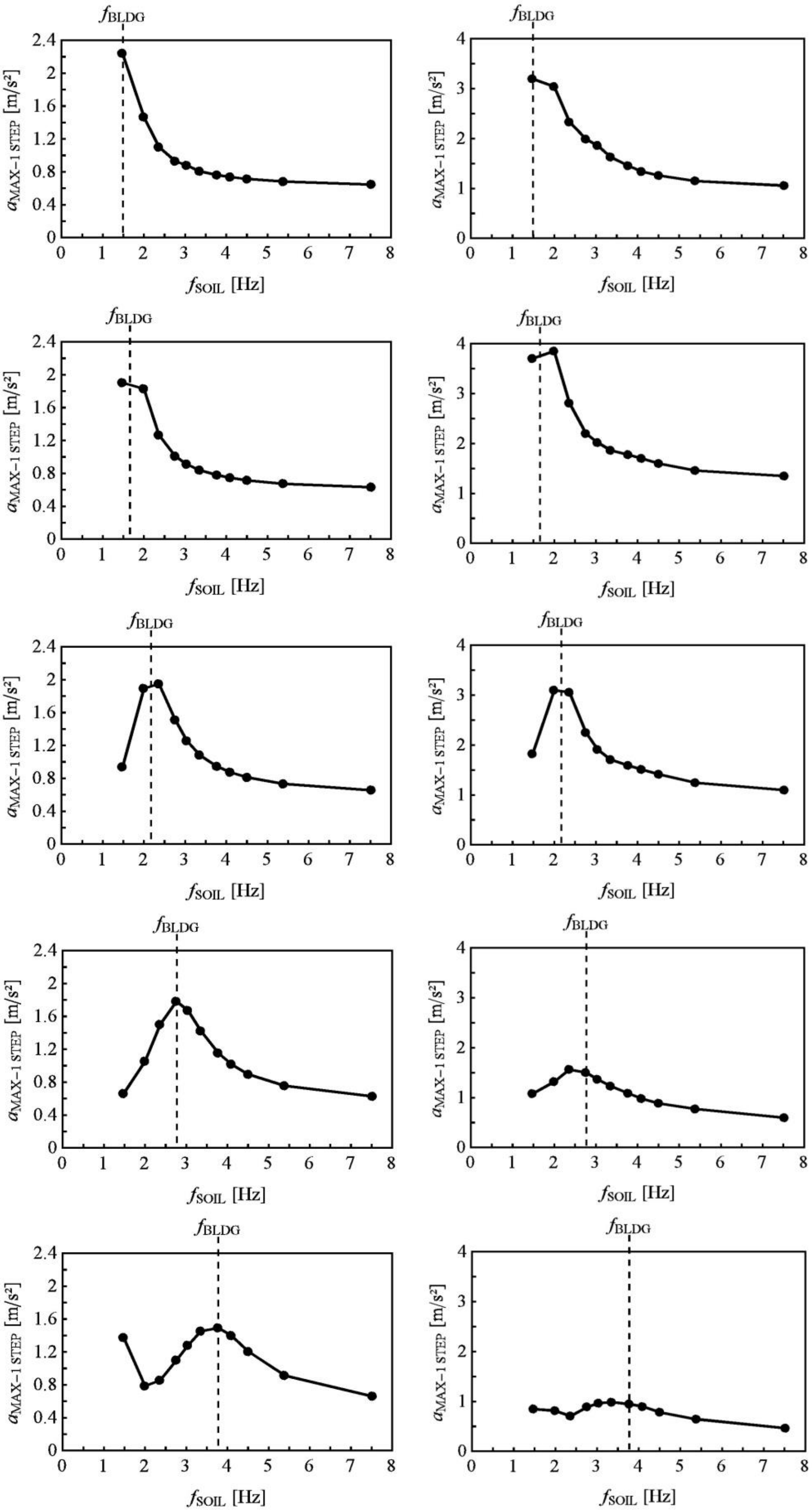

Figure 6. Variation of the peak acceleration at the building top with the soil fundamental frequency, in a one-step analysis: synthetic signal (left) and 2009 Mw 6.3 L'Aquila earthquake (right) as seismic loading

5.2 Response spectrum considering SSI

The variation with the soil fundamental frequency of the peak acceleration at the building top $a_{MAX-1STEP}$, in a one-step analysis (building-foundation-soil assembly), is shown in Figure 6 for different soil profiles and the same structure, in the two cases of input seismic loading corresponding to a synthetic signal (having predominant frequency f_q close to the building fundamental frequency) and to the 2009 Mw 6.3 L'Aquila earthquake. The frequency associated to the highest energy content is $f_q = 1.82\,\text{Hz}$ for the 2009 L'Aquila earthquake. The peak acceleration at the building top is maximum when $f_{SOIL} = f_{BLDG}$ both for the case of synthetic signal, where $f_{BLDG} = f_q$, and for the 2009 Mw 6.3 L'Aquila earthquake. This is expected since it is the resonance case. A decreasing acceleration peak is obtained for f_{SOIL} higher and lower than f_{BLDG}. The same trend is obtained for all the structures, independently of the input motion.

Figure 7 shows the variation of peak acceleration at the building top in a one-step analysis, normalized with respect to its maximum, with the building to soil fundamental frequency ratio. In this way, a similar result is achieved in all the cases. This result suggests a response spectrum that takes into consideration the soil-profile heterogeneity and the SSI.

The variation of the one-step to two-step peak acceleration ratio $a_{MAX-1STEP}/a_{MAX-2STEP}$ (for motion at the building top) with the building to soil fundamental frequency ratio is shown in Figure 8. In the analyzed cases, the influence of SSI can reduce the acceleration peak at the top of the building between 30 and 40%, or increase of about 5%. The similarity of all the cases is maintained. Consequently, an average curve for all analyzed cases could provide an acceleration ratio $a_{MAX-1STEP}/a_{MAX-2STEP}$ to quantify the SSI effect for any structure, known the building to soil fundamental frequency ratio f_{BLDG}/f_{SOIL}. In other words, the SSI effect could be predicted knowing the structure and soil dynamic features, to correct the result obtained using a two-step analysis approach, according to

$$a_{MAX-1STEP} = a_{MAX-1STEP}/a_{MAX-2STEP} \times a_{MAX-2STEP} \tag{3}$$

where $a_{MAX-1STEP}/a_{MAX-2STEP}$ is read in the response spectrum considering SSI (as those in Figure 8).

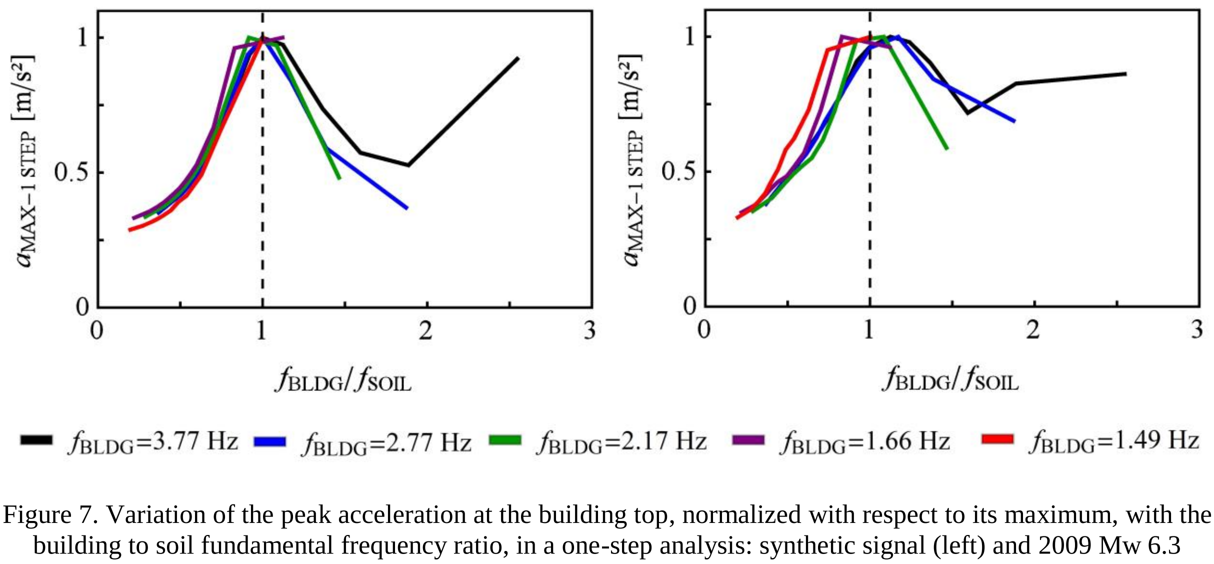

Figure 7. Variation of the peak acceleration at the building top, normalized with respect to its maximum, with the building to soil fundamental frequency ratio, in a one-step analysis: synthetic signal (left) and 2009 Mw 6.3 L'Aquila earthquake (right) as seismic loading

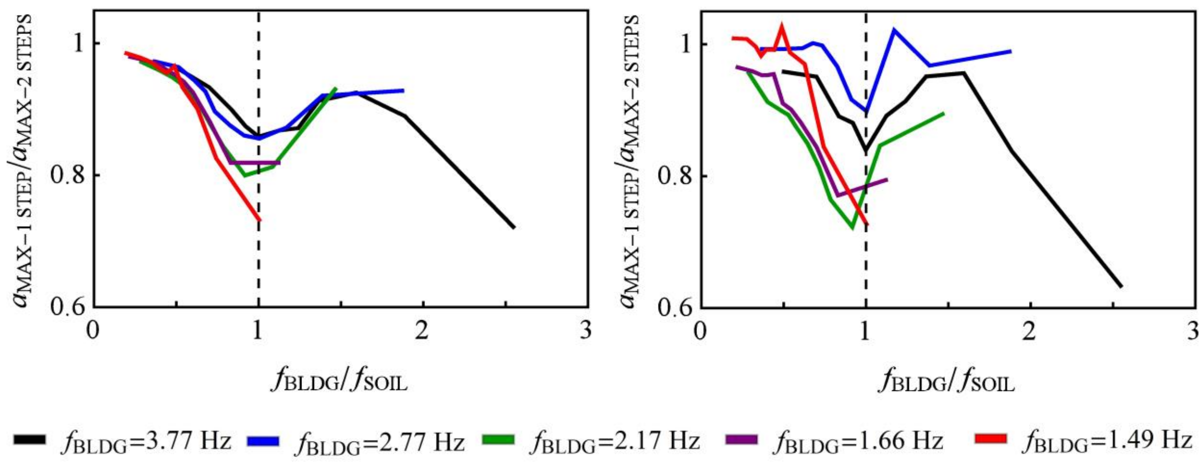

Figure 8. Variation of the one-step to two-step peak acceleration at the building top with the building to soil fundamental frequency ratio: synthetic signal (left) and 2009 Mw 6.3 L'Aquila earthquake (right) as seismic loading

6. CONCLUSION

A parametric study has been held for the analysis of the seismic response of a building having shallow foundation, taking into account soil-structure-interaction (SSI).
The finite element model consists of a 3D frame structure, meshed using Timoshenko beam elements, rigidly connected to the foundation, the latter embedded in a T-shaped soil domain. Soil and foundation are discretized using 20-node solid finite elements. The 3D foundation slab is rigidly connected, node to node, to the 3D soil domain. A reliable soil domain for SSI analyses demands the definition of the soil domain area (that is selected after analysis of the building base to bedrock transfer function for different areas) and the thickness of the near-surface soil domain modeled as 3D.
The proposed 1DT-3C modeling approach allows simulating the response of soil and building to a three-component seismic motion, accounting foundation deformability, rocking effect and SSI.
The undertaken parametric analysis considers different soil profiles, characterized by their average shear wave velocity for the first 30 m depth, and five different structures. A synthetic signal having predominant frequency close to the building fundamental frequency and the 2009 Mw 6.3 L'Aquila earthquake are used as input seismic loading. A one-step analysis, where SSI is considered using a soil domain assembled with the frame structure, is compared with a two-step analysis, where a fixed-base structure is subjected at its base to the free-field motion.
The similarity of building response from the point of view of SSI effect is obtained for different dynamic features of the soil profile. The impact of SSI, dependent of the building to soil fundamental frequency ratio, is estimated. The same trend is obtained for all analyzed buildings having frame structure and shallow foundation. Consequently, a response spectrum that accounts SSI could be provided.
The impact of SSI in the acceleration peak at the building top appears variable between about 30-40% of reduction and 5% of increase, in a linear elastic regime. A correction factor for a two-step analysis approach, considering the effect of SSI, is provided.
Further work is necessary to introduce the nonlinearity of soil and structure in this parametric analysis.

7. ACKNOWLEDGMENTS

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