COMPDYN 2025
10th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15-18 June 2025

ASSESSING THE IMPACT OF HORIZONTAL RESTRAINTS ON THE SEISMIC ROCKING OF MASONRY FAÇADES INCLUDING DYNAMIC SOIL-STRUCTURE INTERACTION

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Abstract

This study assesses the beneficial impact of horizontal restraints – such as steel tie-rods or catenas – on the seismic out-of-plane rocking behaviour of masonry façades, accounting for the effects of Dynamic Soil-Structure Interaction (DSSI). To this end, the results of preliminary 3D Finite Element Analyses are discussed, where the mechanical behaviour of the masonry façade, the foundation block, and the soil deposit was described by a linear visco-elastic constitutive model assigned to 10-noded tetrahedra elements (cluster), while the façade-foundation contact was reproduced through interface elements allowing for gapping. The entire system was subjected to 1D-seismic inputs applied at the bedrock depth: hence, the contribution of the dynamic soil response was also taken into account in the analyses.
The same analysis was performed twice: once with the rigid masonry block free to rock (w/o tie-rod), once with masonry rocking limited by the horizontal restraint atop the block imposed by a steel tie-rod, here reproduced with a horizontal spring with stiffness k. The analysis in the presence of the tie-rod was repeated neglecting the presence of the soil deposit to assess DSSI effects. The comparison between the analyses allows to shed some light on the role played by the horizontal tie-rod and the soil deposit in the rocking of the masonry block.

Keywords: Rocking, Masonry, Historic Masonry, Horizontal Restraint, Tie-Rod, Dynamic Soil-Structure Interaction, 3D Finite Element Analysis, Earthquake.

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1 INTRODUCTION

The rocking of rigid-block bodies is a dynamic model which has been extensively used to assess the seismic vulnerability of masonry façades. When wall-to-wall and wall-to-roof connections are of poor quality, macro-elements such as entire façades or parts of them detach from the remaining structure, triggering Out-Of-Plane (OOP) modes. Being these modes more likely to occur rather than In-Plane (IP) modes characterised by a higher inertia, engineers must focus their attention on the former as a primary concern. Masonry walls/arches of different typologies behave as rigid-blocks rocking during the ground motion while interacting with transversal walls, tie-rods, arches, or vaults [1, 2].

Static or dynamic analyses on rigid-rocking blocks may be conducted by analysing the role of different boundary conditions which affect the evolution of motion. Whilst static analyses are more appropriate for a simplified approach and for damage limit states, dynamic analyses are needed when more accurate results are required, also because the impact remarkably influences the oscillatory response, and that aspect can be only considered in a dynamic framework. Such an influence is even higher when earthquake-resistant devices, such as traditional or dissipative tie-rods – recently designed for masonry buildings – are considered in the formulation [3-5].

Normally, the rocking analysis of OOP rigid-blocks simulating masonry façades is developed considering a rigid soil-foundation system over which the block rocks alternatively changing the pivot point. Nevertheless, a finite-stiffness foundation/soil was observed to strongly influence the seismic performance of the wall [6]. The dynamic interaction between soil, foundation and structure was simulated via lumped parameter models (LPM), which were tuned combinations of masses, springs, and dashpots capable of approximating the behaviour of the actual soil-foundation system. Previous research was conducted by Wolf [7] who studied the rocking of a flexible single-degree-of-freedom including SSI and uplift at the soil-foundation interface. He proposed a procedure to calibrate the soil reactions as a function of the reduced contact area between foundation and soil, which in turn depends on the rocking angle. A recent study developed a tool considering SSI to assess the seismic safety of unreinforced masonry buildings against OOP mechanisms [8]. The work proposed a formula to predict the elongation of the fundamental period and the variation of equivalent damping of the soil-foundation-soil system with respect to fixed-base conditions and assessed the probability of exceeding increasing damage levels associated with OOP modes via fragility curves.

This paper presents preliminary research on the Dynamic Soil-Structure-Interaction (DSSI) of masonry façades restrained by a traditional tie-rod. A commercial 3D Finite Element (FE) software modelling the interfaces between wall, foundation, and soil, is used to understand the role of the tie-rod and of DSSI on the behaviour of the rigid block. After showing the problem layout in Section 2, in Section 3 the 3D FE model and the relevant numerical analyses are presented. A brief discussion of results is then provided in Section 4 and, finally, the main conclusions of the study are drawn in Section 5.

2 PROBLEM LAYOUT

The primary challenge in modelling DSSI for masonry façades arises from the differing characteristics of the two subsystems: the rocking rigid façade and the soil-foundation system. Indeed, the façade behaves as a finite and nonlinear system [9], whereas the soil-foundation system is infinite and governed by its natural vibration modes. SSI can significantly impact the seismic rocking response, particularly due to soil flexibility and uplift at the façade-foundation interface. However, the magnitude of these effects depends on the degree of interaction between the subsystems and the properties of the seismic input.

Unlike rigidly supported structures, masonry façades resting on flexible soil experience altered vibrational characteristics, including changes in the dominant oscillation frequency and energy dissipation. This circumstance may lead to increased rocking amplitudes, prolonged oscillations, and a higher probability of overturning during seismic events. The presence of tie-rods further modifies the system response, making it essential to evaluate their effectiveness under different DSSI conditions.

DSSI analysis examines how a structure dynamically interacts with the underlying soil through its foundation when subjected to seismic loading. The effects of DSSI are observed as the difference between the response of a structure supported by a flexible soil-foundation system and that of the same structure assumed to be rigidly fixed at its base. The distribution of contact stresses between the structure and the soil depends on the excitation vibration frequency. Since a structure resting on soil presents a complex boundary condition problem, exact analytical solutions are typically only feasible for highly simplified scenarios.

The façade can be addressed using numerical models of varying complexity, from simplified Single-Degree-of-Freedom (SDoF) systems to Multi-Degree-of-Freedom (MDoF) models and continuum approaches. The continuum model provides the most detailed representation and is analysed using methods like Finite Element or Finite Difference techniques. Soil behaviour is similarly modelled with increasing sophistication, from discrete spring-dashpot systems that approximate stiffness and damping to continuum representations that better capture stress distribution and wave propagation.

This study adopts a continuum finite element model (FEM) for both the façade and the soil-foundation system to accurately simulate their dynamic interaction. An alternative approach is the Lumped-Parameter-Model (LPM) [10,11], which simplifies the soil-foundation subsystem using discrete springs and dashpots. The key distinction between these methods lies in their treatment of soil behaviour. LPMs use frequency-independent coefficients to approximate stiffness and damping, making them efficient for preliminary assessments but potentially neglecting wave propagation and stress redistribution. In contrast, the continuum FEM approach treats soil as a distributed medium with intrinsic properties, enabling a more accurate simulation of stress waves, nonlinear deformations, and energy dissipation.

For a façade-soil-interaction problem, the rocking induced by the seismic input might lead to the uplift at the façade-foundation interface, which introduces nonlinearities in the response. Therefore, it is necessary to adopt advanced numerical techniques to track contact changes and impact forces.

Two primary uplift scenarios can be distinguished (Figure 1), depending on the characteristics of the foundation and soil properties. In the first scenario (Figure 1a), which is more typical of shallow foundations, uplift occurs at the soil-foundation interface, leading to partial detachment of the foundation from the soil. This mechanism allows for significant deformation of the soil beneath the foundation, influencing the rocking motion and energy dissipation through plastic deformation. The second scenario (Figure 1b) assumes an embedded foundation with a perfect connection to the soil, resulting in uplift at the façade-foundation interface. In this case, the soil remains engaged, and the façade undergoes detachment from the foundation.

The approach adopted in the present paper considers an embedded foundation, which is assumed to be perfectly connected to the soil. Consequently, the uplift mechanism occurs at the interface between the façade and the foundation rather than at the soil-foundation interface. This configuration requires the precise implementation of specialised contact rules between block and foundation, along with a thorough assessment of the façade rocking motion, as detailed in the following.

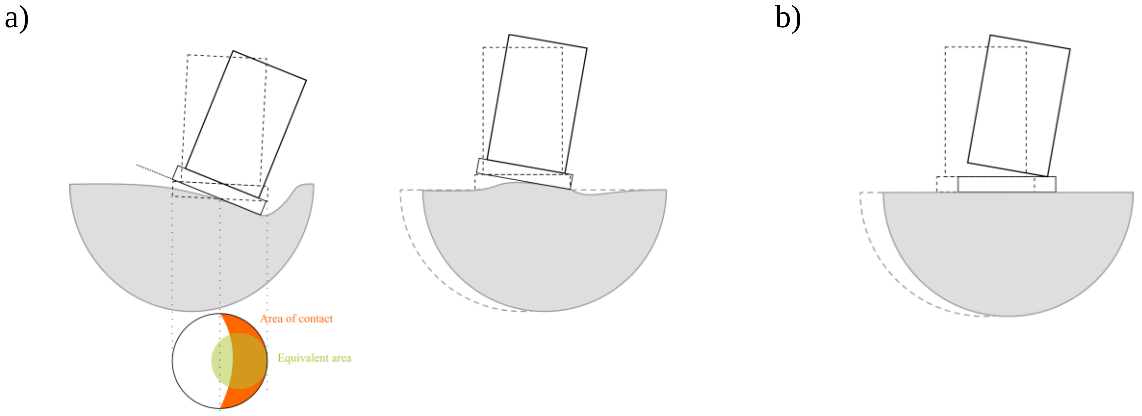

Figure 1: Foundation-structure connection: a) strong enough to cause the partial uplift at the soil-foundation interface and plastic deformation at the soil surface [7]; b) weak enough to cause the uplift of the block at the block-foundation interface [10].

3 3D FINITE ELEMENT NUMERICAL MODELLING

The problem considered in the present paper is that of a rigid block resting on a foundation embedded in a homogeneous soil stratum, subject to a horizontal acceleration time history applied along the direction parallel to the wall thickness at the bedrock depth. The geometry of the problem is reported in Figure 2, where the 3D FE model, which was implemented in the FE software *Plaxis CE v22.01* [12], is shown. The model is characterised by 16362 elements and 28700 nodes, where the soil deposit, the foundation and the block were all simulated through 10-noded and 4-Gauss point tetrahedral FE (*cluster* in the following), with a second-order shape functions for displacement and first-order interpolation of strains, while the tie-rod atop the rigid block was modelled through a horizontal elastic *beam element* (magenta element in Figure 2). *Interface* elements were introduced at the block-foundation contact to allow for gapping during rocking, whereas the embedded foundation is assumed to be perfectly connected to the soil (no detachment, Figure 1b). The model size was preliminary designed to both limit the influence of the static boundary conditions during the wished-in-place activation of the foundation and the rigid block, and to reproduce 1D free-field wave propagation towards the centre of the model when applying the seismic input and imposing the free-field boundary conditions ($C_1 = C_2 = 1$) along the vertical *y-z* planes located at the edges of the numerical domain. The FE size was determined to limit the numerical distortion of the waves travelling through the model, following the well-known requirement reported in [13]. As for time integration, the unconditionally-stable average-acceleration Newmark scheme was adopted [14] with a time step equal to the sampling time interval of the seismic input, $\Delta t = 0.005$ s.

The mechanical behaviour of all the *cluster* elements, namely the block, the foundation, and the foundation soil, was described through a linear viscous-elastic material, in order to get a preliminary glimpse into the physics of the problem at hand. The mechanical parameters are listed in Table 1, where γ is the unit weight, E and ν are the Young modulus and the Poisson ratio, respectively, and $G = E/[2\cdot(1+\nu)]$ is the shear modulus. A damping ratio $\xi = 3$ % was assigned to the structural material representative of the block and the foundation, while the value $\xi = 5$ % was attributed to the soil stratum. A Rayleigh formulation was adopted, whose controlling frequencies f_m and f_n were derived following the procedure presented in [15] from preliminary 1D free-field analyses carried out with the Linear Equivalent (L.E.) [16] approach implemented in the software MARTA [17].

As for the *beam* element representing the tie-rod, a length l = 1.6 m was selected to reach the symmetry axis of the façade, while a diameter d = 24 mm (cross section A = 455.6 mm^2) was adopted so that an axial stiffness $k = EA/l \approx 60$ MN/m was obtained, which is representative of a typical steel tie-rod (E = 210 GPa).

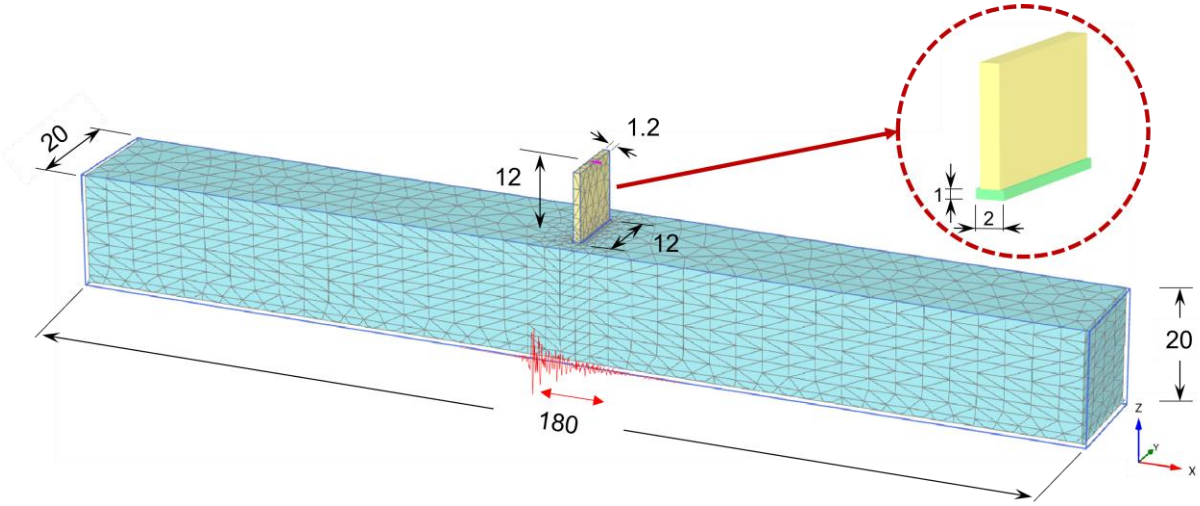

Figure 2: 3D FE model adopted in the time domain numerical analyses (units: m).

parameter	block	foundation	soil
γ (kN/m^3)	19.0	25.0	20.0
E (MPa)	30000.0	30000.0	26.6
ν (-)	0.15	0.15	0.33
G (MPa)	13040.0	13040.0	10.0
f_m (Hz)	0.90	2.63	2.63
f_n (Hz)	9.63	9.63	9.63

Table 1: Mechanical properties assigned to the *clusters* adopted in the 3D FE analyses.

The seismic input adopted in the analyses is given in Figure 3, both in terms of the horizontal acceleration time history (Figure 3a) and the relevant Fourier Amplitude spectrum (Figure 3b). The ground motion is characterised by a peak acceleration $a_{max,b} = 0.177g$, a predominant and mean period $T_{p,b}$ = 0.32 s and $T_{m,b}$ = 0.37 s [14], respectively, a strong-motion duration $D_{5\text{-}95}$ = 6.96 s [15], and an Arias Intensity $I_{A,b}$ = 0.22 m/s [16], where subscript b means acceleration time history at the bedrock depth. This seismic input was obtained following the steps reported below:

- extracting the original, natural seismic horizontal acceleration time history recorded at station CMI (Campi, Perugia, Italy, 40 km away from Amatrice) on 2016, October 26th during the 2016 Central Italy earthquake;
- scaling the above time trace to a peak acceleration = 0.30g, for which the tie-rod considered in the numerical analyses was designed against;
- applying the linear viscous-elastic deconvolution up to the bedrock depth;
- applying a low-pass, 8th-order Butterworth filter (f_{max} = 9 Hz);
- imposing a 3rd-order baseline correction to obtain zero displacement and velocity at the end of the seismic event.

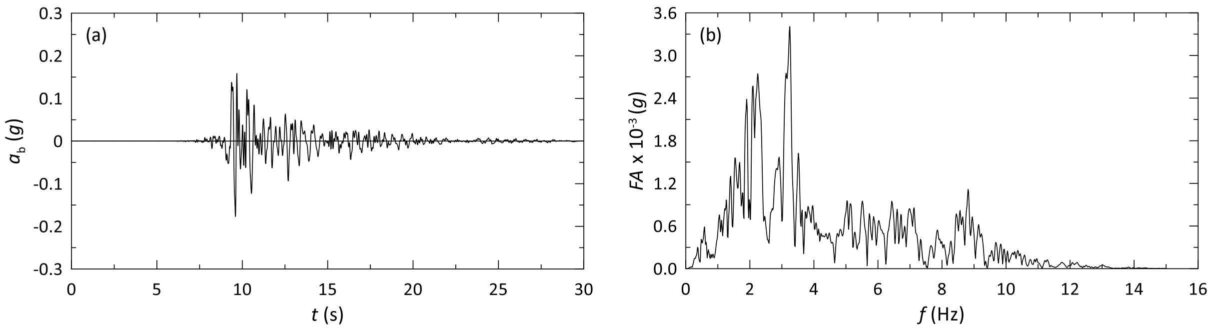

Figure 3: Seismic input adopted in the 3D FE analyses: (a) horizontal acceleration time history; (b) Fourier Amplitude spectrum.

4 DISCUSSION OF RESULTS

Capability of the 3D FE model to properly capture the wave propagation was first checked by comparing the main outcomes obtained at some specific alignments in the absence of the structure, namely at the symmetry plane (x = 0) and at a free-field alignment (x = 20 m), with those resulting from the above-mentioned 1D L.E. free-field analyses. The comparison is shown in Figure 4 in terms of profiles of the peak acceleration ratio, $a_{max}/a_{max,b}$, and of the peak shear strain, γ_{max}. From the Figure it is apparent that 3D model accurately reproduces the benchmark 1D L.E. results, and that it could be therefore adopted for the advanced 3D DSSI analyses in the presence of the rigid block and its embedded foundation.

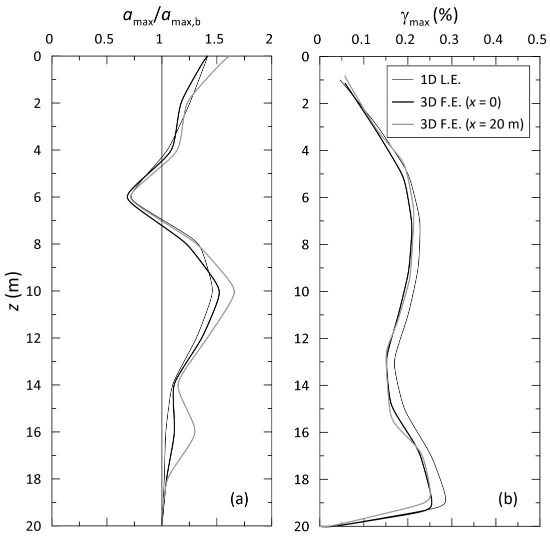

Figure 4: Profiles of the (a) peak acceleration ratio; (b) peak shear strain.

The dynamic response of the façade is expressed in terms of the dimensionless rigid rotation θ/α, plotted in Figure 5. θ is the angle of rigid rotation of the block, whose time trace reads:

$$\theta(t) = \frac{u_{\text{top}}(t) - u_{\text{bottom}}(t)}{h} \tag{1}$$

with u_{top} and u_{bottom} being the horizontal displacements at the top and at the bottom of the rigid block, and h = 12 m being the height of the block, while

$$\alpha = \arctan\left(\frac{b}{h}\right) = 5.71° \tag{2}$$

is the angle between the diagonal of the *x-z* plane of the rigid block and the vertical, b = 1.2 m being the façade thickness.

From the plot in Figure 5 it is evident that, in the presence of DSSI, the constraint imposed by the tie-rod implies a strong reduction of the rigid block rotation, as expected. In particular, a noticeable reduction of the rotation was computed over the entire duration of the ground motion; also, a second frequency of oscillation of the block was introduced in the system, while keeping almost the same eigen frequency (f = 0.9 Hz, T = 1.1 s), the latter coinciding with the fundamental period of the free-field soil column. Here it is worth noting that the above differences may be attributed to the tie-rod only, since the kinematic and inertial interaction effects did not affect the acceleration time history atop the foundation block (Figure 5b), due to its low embedment [21, 22].

A seismic performance index could then be defined from the peak rotation θ_{max}, as θ_{max}/α, whose values are listed in Table 2: from the Table it follows that a reduction of about 70 % was computed in the presence of the tie-rod atop the rigid block.

The analysis with the tie-rod was then repeated excluding the soil deposit contribution (*w/o DSSI*), which implied applying the horizontal acceleration time history computed at the free-field ground surface straight at the bottom of the foundation along the same *x-direction* as above, while fixing the remaining DoFs ($u_y = u_z = 0$). From Figure 6 and Table 3 it is evident that DSSI effects contribute to reduce the seismic performance index θ_{max}/α, in the presence of the tie-rod, by an amount of about 20 %, which is a non-negligible contribution to the seismic performance of the masonry façade.

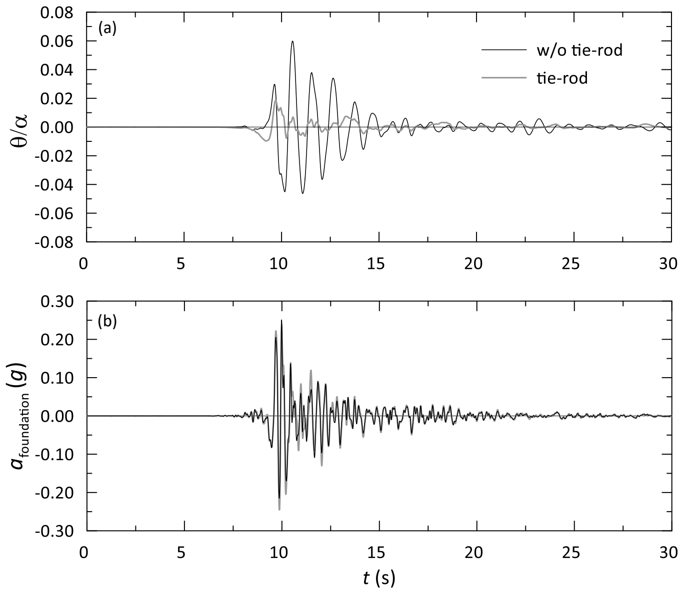

Figure 5: Influence of the tie-rod on the time histories of the (a) dimensionless rotation block and the (b) foundation acceleration, considering DSSI.

parameter	tie-rod (1)	w/o tie-rod (2)	(1)/(2)
θ_{max}/α	0.0184	0.0599	0.31

Table 2: Influence of the tie-rod on the seismic performance index of the rocking rigid block considering DSSI.

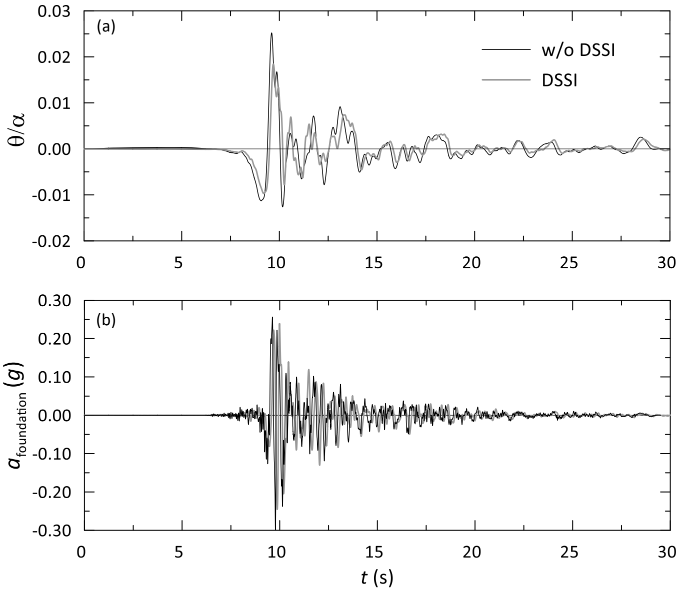

Figure 6: Influence of DSSI on the time histories of the (a) dimensionless rotation block and the (b) foundation acceleration, in the presence of the tie-rod.

parameter	DSSI (1)	w/o DSSI (2)	(1)/(2)
θ_{max}/α	0.0184	0.0235	0.78

Table 3: Influence of DSSI on the seismic performance index of the rocking rigid block with tie-rod.

5 CONCLUSIONS

In this paper, the influence of horizontal restraints on the seismic rocking of masonry façades has been assessed, including Dynamic Soil-Structure Interaction. To this end, a 3D FE model, including the homogeneous soil stratum, the rigid block, and its foundation, has been developed, to evaluate the rigid rotation of the structural system, subjected to a horizontal acceleration time history at the bedrock depth. The same analysis was performed twice, once in the absence and once in the presence of the horizontal tie-rod located at the top of the masonry block, which was simulated through a horizontal spring with stiffness *k*. The comparison of the two analyses provided an estimate of the influence of the horizontal restraint, which turned out to be remarkable, since the peak rotation was reduced by about 70%.

Also, the role played by the soil deposit was assessed by performing an additional dynamic analysis, in the presence of the tie-rod, where the masonry block + foundation system was fixed at the bottom, except for the direction along which the seismic input was applied (*w/o DSSI* analysis). The comparison with the analysis where the DSSI effects were explicitly taken into account revealed that they contributed to reduce the peak rotation by an amount of about 20%.

Further development of this paper would require extending the study to different geometry of the block, mechanical soil characteristics and seismic input properties. Also, the influence of the damping provided by the tie-rod will be included in the analysis, since this is expected to further influence the seismic performance of the system at hand.

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