Improving Urban Seismic Resilience through Vibrating Barriers

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Abstract: The paper addresses the seismic response of structures in urban environments. The novel Vibrating Barriers (ViBa) developed to control the dynamic response of buildings is studied through a simplified model of a city undergoing seismic vibration modelled as Gaussian ground motion processes

1   Introduction

Earthquakes are a well-known natural hazard to urban environments. Recent disasters in Ecuador, Italy and Taiwan manifest the clear need to address the seismic resilience of existing buildings in a different and hopefully more affordable way. Construction industry successfully introduced devices such as isolators, dampers and tuned mass dampers to mitigate dynamic vibrations induced by earthquakes in new buildings, but such devices are rarely used for the protection of existing buildings, as they generally require substantial alteration of the original structure. In the case of heritage buildings, critical facilities and urban areas, especially in developing countries, those traditional localized solutions might become impractical. Therefore, what we are witnessing nowadays is the lack of substantial actions to protect existing cities in seismic prone areas with consequent number of fatalities and loss of historic and artistic heritage.

In order to cope with the global grand challenge of improving seismic resilience of structures in urban environments two main elements need to be addressed and yet fully understood: i) proper definition of ground motion excitation and ii) site-city interaction modelling. Ground motion arising from seismic waves is affected by several factors, i.e. source patterns, path, site effects, etc., that generally cannot be described in a deterministic fashion. Consequently, only a probabilistic approach can provide a rigorous representation of earthquake ground motion. The definition of which methodology and hypothesis better model the seismic ground motion and its effect over the structures, is still an open issue in the scientific community. According to the probabilistic approach, the ground motion acceleration recorded in a given location can be seen as a sample of a stochastic process. In this regard several stochastic models have been proposed in the literature, such as Gaussian, filtered white noise and filtered Poisson processes [1]. Recent studies on the modelling of the seismic action are aimed to bridge the gap between the field of engineering and seismology proposing stochastic models encompassing seismological parameters reflecting the natural variability of earthquakes (see e.g. references [2, 3]). It has to be emphasized, that the approaches currently proposed in the literature for the seismic design of structures focus on the modelling of the ground motion acceleration at the free field without considering the influence of the urban environment. Nevertheless, studies on site-city interaction (see e.g. [4]), showed that in an urban environment the presence of buildings modify significantly the energy of the seismic waves in the underlying soil layers. The consequent ground-motion acceleration at the free-field used for

designing civil engineering structures can be significantly different from the predicted one out-side the urban area.

Due to the difficulties involved in modelling the multiple interactions between structures in an urban environment, numerical approaches based on wave propagation and finite or boundary element analysis are usually preferred [4-6].

Analytical studies on site-city interaction have been also proposed in the literature [7,8]. In [7] the effect of the city is accounted for by modelling the structures as simple oscillators, while in [8] the multiple interactions between buildings are studied through homogenization methods. A recent review of structure-soil-structure interaction problem can be found in [9].

Structure-soil-structure interaction has been proved to be either beneficial or detrimental for structures in the last four decades, but only recently [10] it has been used as a vehicle to control the vibration of structures. Cacciola and Tombari [10] introduced for the first time, a non-localized solution, called Vibrating Barrier (ViBa), hosted in the soil and detached from the structures. Analyses on the efficiency of the ViBa in protecting one building are reported in Cacciola et al. [11] for structures founded on monopile foundation, and Tombari et al. [12] for industrial buildings.

The present study focuses on the study of the efficiency of the Vibrating Barrier as a tool to improve the seismic resilience in urban areas. In this regard, a simplified FE model of a village is first developed, and the response to ground motion in both frequency and time domain is addressed. The ViBa is then designed to reduce the stochastic response of an earthquake. The stochastic response of each individual building protected by the ViBa is studied in the frequency domain and through a pertinent Monte Carlo Simulation.

2 Problem position

Consider the global system depicted in Figure 1 under ground motion excitation at the bedrock $u_g(t)$. The Vibrating Barrier (ViBa) is included aiming to reduce the vibration of the surrounding buildings.
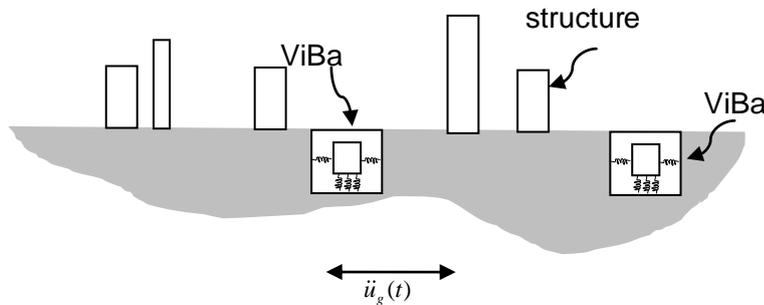

Figure 1. Sketch of the simplified model of structure in urban environment protected by the ViBa.

A simplified mechanical model able to describe the interaction effects among buildings and ViBa is first derived. Full details are given in Cacciola and Tombari [10]. The ViBa is modelled as an internal oscillator device included in a rigid box foundation and globally described by the 2-DOFs (see Figure 2), the internal motion of the oscillator, $u_{ViBa}(t)$ and the displacement of its foundation, $u_{f,ViBa}(t)$. The dynamic governing equations of the global system are derived in terms of absolute displacements, as it is conventional in soil-structure interaction, namely the dynamics of the problem take the form:

$$\mathbf{M}\ddot{\boldsymbol{u}}(t) + \mathbf{C}\dot{\boldsymbol{u}}(t) + \mathbf{K}\boldsymbol{u}(t) = \mathbf{Q}_e u_g(t) + \mathbf{Q}_d \dot{u}_g(t) \tag{1}$$

where **M**, **C** and **K** are the global mass, viscous damping and stiffness matrix; $\ddot{\boldsymbol{u}}(t)$, $\dot{\boldsymbol{u}}(t)$ and $\boldsymbol{u}(t)$ are respectively the absolute acceleration, velocity and. displacement vector. In equation (1), $\dot{u}_\mathrm{g}(t)$ is the first derivative of the ground displacement $u_\mathrm{g}(t)$. The vectors $\mathbf{Q}_e$ and $\mathbf{Q}_d$ are the influence quantities; $\mathbf{Q}_e$ depends on the soil-foundation stiffness values whereas $\mathbf{Q}_d$ depends on the soil-foundation damping coefficients.

The matrices of the global system are partitioned in the sub-matrices defined for the individual buildings and the ViBas; therefore the global mass matrix is as follows:

$$\mathbf{M} = \begin{bmatrix} \mathbf{M}_1 & \mathbf{0} & \cdots & \mathbf{0} & \mathbf{0} \\ \mathbf{0} & \mathbf{M}_\mathrm{i} & & \mathbf{0} & \mathbf{0} \\ \vdots & & \ddots & & \vdots \\ \mathbf{0} & \mathbf{0} & \cdots & \mathbf{M}_n & \mathbf{0} \\ \mathbf{0} & \mathbf{0} & & \mathbf{0} & \mathbf{M}_\mathrm{V} \end{bmatrix} \tag{2}$$

in which the ith sub-block includes the mass of the ith structure, while $\mathbf{M}_\mathrm{V}$ is the mass matrix of the ViBas distributed in the urban environment given by

$$\mathbf{M}_\mathrm{V} = \begin{bmatrix} \mathbf{M}_{\mathrm{V}_1} & \mathbf{0} & \cdots & \mathbf{0} & \mathbf{0} \\ \mathbf{0} & \mathbf{M}_{\mathrm{V}_2} & & \mathbf{0} & \mathbf{0} \\ \vdots & & \ddots & & \vdots \\ \mathbf{0} & \mathbf{0} & \cdots & \mathbf{M}_{\mathrm{V}_\mathrm{i}} & \mathbf{0} \\ \mathbf{0} & \mathbf{0} & & \mathbf{0} & \mathbf{M}_{\mathrm{V}_\mathrm{n}} \end{bmatrix} \tag{3}$$

with

$$\mathbf{M}_{\mathrm{V}_\mathrm{i}} = \begin{bmatrix} m_{\mathrm{ViBa,i}} & 0 \\ 0 & m_{\mathrm{f,ViBa,i}} \end{bmatrix} \tag{4}$$

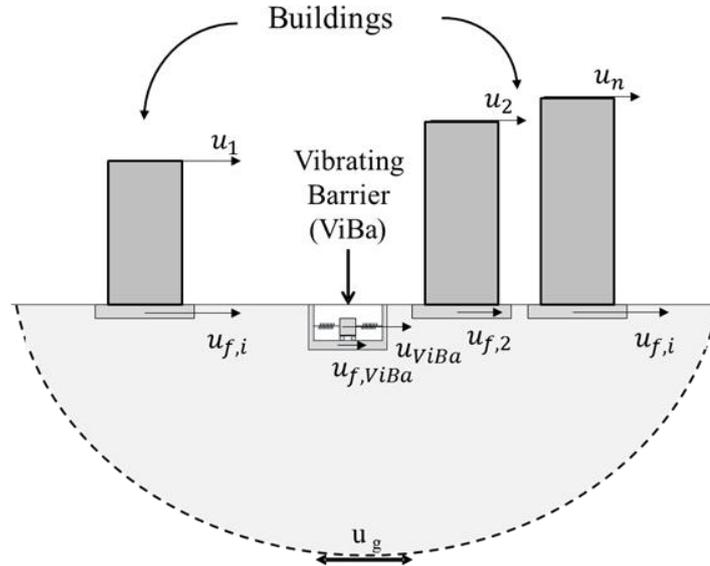composed of the mass of the ViBa, $m_{\mathrm{ViBa,i}}$, and the mass of its foundation $m_{\mathrm{f,ViBa,i}}$.

Figure 2. Vibrating Barrier (ViBa) device embedded in the soil for protecting a cluster of buildings

The global damping matrix **C** and the global stiffness matrix **K** are block-matrices partitioned in the following form:

$$\mathbf{C} = \begin{bmatrix} \mathbf{C}_1 & \mathbf{C}_{1,i} & \cdots & \mathbf{C}_{1,n} & \mathbf{C}_{1,V} \\ \mathbf{C}_{i,1} & \mathbf{C}_i & \ddots & \mathbf{C}_{i,n} & \mathbf{C}_{i,V} \\ \vdots & & \ddots & & \vdots \\ \mathbf{C}_{n,1} & \mathbf{C}_{n,i} & \cdots & \mathbf{C}_n & \mathbf{C}_{n,V} \\ \mathbf{C}_{V,1} & \mathbf{C}_{V,i} & & \mathbf{C}_{V,n} & \mathbf{C}_V \end{bmatrix} \tag{5}$$

for the damping matrix, while:

$$\mathbf{K} = \begin{bmatrix} \mathbf{K}_1 & \mathbf{K}_{1,i} & \cdots & \mathbf{K}_{1,n} & \mathbf{K}_{1,V} \\ \mathbf{K}_{i,1} & \mathbf{K}_i & \ddots & \mathbf{K}_{i,n} & \mathbf{K}_{i,V} \\ \vdots & & \ddots & & \vdots \\ \mathbf{K}_{n,1} & \mathbf{K}_{n,i} & \cdots & \mathbf{K}_n & \mathbf{K}_{n,V} \\ \mathbf{K}_{V,1} & \mathbf{K}_{V,i} & & \mathbf{K}_{V,n} & \mathbf{K}_V \end{bmatrix} \tag{6}$$

for the stiffness matrix. The main diagonal sub-matrices $\mathbf{C}_r$ and $\mathbf{K}_r$ (r =1,…, n) describe the viscous damping and stiffness matrix of the rth-structure and its interaction with the soil. The matrices $\mathbf{C}_V$ and $\mathbf{K}_V$ define the damping and stiffness matrix of the ViBa and its interactions to the other buildings through the soil. Lastly, the off-diagonal sub-matrices $\mathbf{C}_{i,j}$ and $\mathbf{K}_{i,j}$ (i, j=1, …, n) are related to the dynamic coupling between the ith and the jth structures. It is worth mentioning that ground spatial variation of the input motion can be also considered due to the formulation of Eq. (1) in absolute displacements by modifying opportunely the influence quantities $\mathbf{Q}_e$ and $\mathbf{Q}_d$.

In the previous formulation, the structural parameters of the ViBa represent the unknowns of the problem to be determined. Therefore, various optimization criteria can be used to this purpose as a function of the design parameters used in the penalty function $J(\boldsymbol{\alpha})$, that is

$$\min\{J(\boldsymbol{\alpha})\}$$
$$\boldsymbol{\alpha} = \{\mathbf{K}_V, \mathbf{M}_V, \mathbf{C}_V\} \in \mathbb{R}_0^+ \tag{7}$$

where $J(\boldsymbol{\alpha})$ is determined in terms of either relative displacements, internal forces, energy etc. and $\boldsymbol{\alpha}$ is the design parameter vector.

3   Stochastic response to Gaussian stationary ground motion excitation.

Consider the ground motion input $u_g(t)$ modelled as monocorrelated Gaussian zero-mean stationary stochastic process. Accordingly, it is fully defined, from a probabilistic point of view, by the knowledge of the power spectral density (PSD) function $G_{ug}(\omega)$. Under the hypothesis of linear behaving system the stochastic response will be also zero mean and Gaussian and it is fully defined by the knowledge of the power spectral density matrix of the response $G_u(\omega)$, that can be determined via the traditional stochastic analysis in the frequency domain.

The governing equation of motion Eq. (1) can be rewritten in the frequency domain as follows:

$$\mathbf{K}_{dyn}(\boldsymbol{\alpha}, \omega)\mathbf{U}(\omega) = (\mathbf{Q}_e + i\omega\mathbf{Q}_d)\, \mathrm{U}_g(\omega) \tag{8}$$

where $\mathbf{K}_{dyn}(\boldsymbol{\alpha}, \omega) = \mathbf{K}(\boldsymbol{\alpha}) + i\omega\mathbf{C}(\boldsymbol{\alpha}) - \omega^2\mathbf{M}(\boldsymbol{\alpha})$ is the dynamic stiffness matrix, $\boldsymbol{\alpha}$ is the design parameters vector and $i = \sqrt{-1}$ is the imaginary unit. The response in the frequency domain for a single realization $\mathrm{U}_g(\omega)$ can be readily determined as follows

$$\mathbf{U}(\omega) = \mathbf{H}(\boldsymbol{\alpha}, \omega)\, \mathrm{U}_g(\omega) \tag{9}$$

where the transfer function $\mathbf{H}(\boldsymbol{\alpha}, \omega)$ is given by the following equation

$$\mathbf{H}(\boldsymbol{\alpha}, \omega) = \mathbf{K}_{\text{dyn}}^{-1}(\boldsymbol{\alpha}, \omega)(\mathbf{Q}_e + i\omega\mathbf{Q}_d) \tag{20}$$

The power spectral density function is then determined

$$\mathbf{G}_{\text{u}}(\omega) = \mathbf{H}(\boldsymbol{\alpha}, \omega)\,\mathbf{H}^*(\boldsymbol{\alpha}, \omega)\mathbf{G}_{\text{ug}}(\omega) \tag{11}$$

where the asterisk means transpose complex conjugate.

After determining the power spectral density matrix of the response the fractile of order p of the distribution of maxima of the relative horizontal displacements U_r of the structure to be protected is determined through the first crossing problem:

$$X_{Ur}(Ts, p) = \eta_{Ur}(Ts, p)\sqrt{\lambda_{0,Ur}} \tag{12}$$

where Ts is the time observing window; η_{Ur} is the peak factor; $\lambda_{0,Ur}$ is the zero-order response spectral moment. The peak factor determined by Vanmarcke [13] is used:

$$\eta_{Ur}(Ts, p) = \sqrt{2\ln\left\{2N_{Ur}\left[1 - \exp\left[-\delta_{Ur}^{1.2}\sqrt{\pi\ln(2N_{Ur})}\right]\right]\right\}} \tag{13}$$

with

$$N_{Ur} = \frac{T_S}{-2\pi\ln p}\sqrt{\frac{\lambda_{2,Ur}}{\lambda_{0,Ur}}} \tag{14}$$

and

$$\delta_{Ur} = \sqrt{1 - \frac{\lambda_{1,Ur}^2}{\lambda_{0,Ur}\lambda_{2,Ur}}} \tag{15}$$

where the response spectral moments $\lambda_{i,Ur}$ are given by the following equation:

$$\lambda_{i,Ur} = \int_0^{+\infty} \omega^i G_{UrUr}(\omega)d\omega \tag{16}$$

where $G_{UrUr}(\omega)$ is the PSD function of the selected horizontal relative displacements.

It is noted that for the design of the the ViBa's structural parameters the solution of equations (7) and (12) will provide the parameters that will minimize the p fractile of the response maxima. However, the numerical solution might be computationally challenging. An alternative option, useful for the preliminary design, is to consider the ViBa as a tuned mass damper embedded in the soil and part of the site-city structural system. In such a way, by the knowledge of the response power spectral density function a first tuning can be done through control of the frequencies pertinent to the peaks of the response PSD. It has to be emphasized, that the linear approach can be seen as a preliminary design strategy to be followed by Monte Carlo study with the eventual consideration of soil nonlinearity and/or soil uncertainties.

4 Numerical results

In this section, the stochastic response of buildings in urban environments is addressed. In this regard the small village of Vathia in the Mani Peninsula (Greece) is considered as case study. Figure 3 shows the village and its plan representation [14].

Figures 3. Plan representation of Vathia village with indication of the studied section.

A simplified Finite Element (FE) model was created from the 2D-section of the village indicated in Figure 3.All the analyses were performed using the FE software ADINA 9.2.1. Figure 4 displays the FE model mesh with indication of building labels considered in the following analyses. It is important to highlight that all the materials considered in the FE model (stratum, buildings, etc.) had isotropic linear elastic behaviour and the only global degree of freedom considered was horizontal translations (X-Direction).
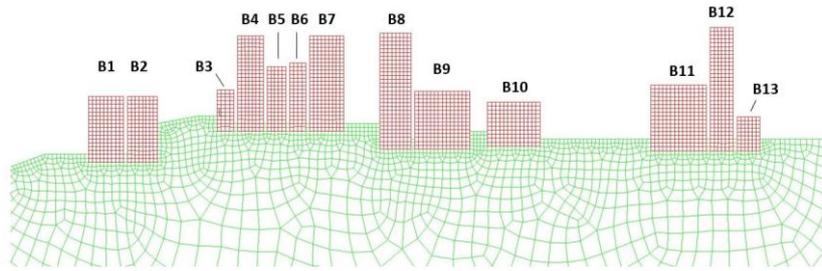

Figure 4. FE mesh of the Vathia village section.

Shear wave velocity (Vs) of 1100 m/s and mass density of 2200 kg/m^3 were considered to be a realistic assumption for the rock stratum parameters. Furthermore, making use of a detailed model of building seven (B7) the mass of the other buildings was also determined. A constant mass per unit area (2D-model) of 4896.03 kg/m^2 was used for all the buildings. The determination of the modulus of elasticity was performed by an identification process to match a target value of the fundamental frequency of B7 computed according EUROCODE 8, that is

$$T_1 = C_t \cdot H^{3/4} \qquad (17)$$

where T_1 is the fundamental period of the structure, H is the height of the building, in m, from the foundation, C_t is a factor depending on the type of structure, for structures with masonry shear walls this value can be estimated as follows

$$C_t = 0.075/\sqrt{A_c} \qquad (18)$$

where A_c is the total effective area of the shear walls in the first storey of the building, in m^2 calculated as follows:

$$A_c = \sum[A_i \cdot (0.2 + (l_{wi}/H)^2)].\tag{19}$$

In Eq. (19), A_i is the effective cross-sectional area of the shear wall i in the direction considered in the first storey of the building, in m^2. l_{wi} is the length of the shear wall i in the first storey in the direction parallel to the applied forces, in m.

The system was then forced by ground motion white noise process to identify the relevant peak responses. The response PSD of the top of each building are presented in Figure 5. As can be seen from Figure 5, all the buildings have a predominant peak at 2.50 Hz. this frequency corresponds to the fundamental frequency of the ground. This behaviour is a direct consequence of the soil-structure cross interaction phenomena in which if the structure-subsoil natural frequency is in the higher range with respect to the stratum fundamental frequency, the fundamental frequency of the soil-structure system will be in the vicinities of the fundamental frequency of the stratum [15]. As a consequence, the design of the ViBa needs to target this frequency instead of being designed for individual buildings. Figure 6 depicts the proposed arrangement of three ViBas with an internal mass of 600 Tonnes, each calibrated at the system's fundamental frequency.
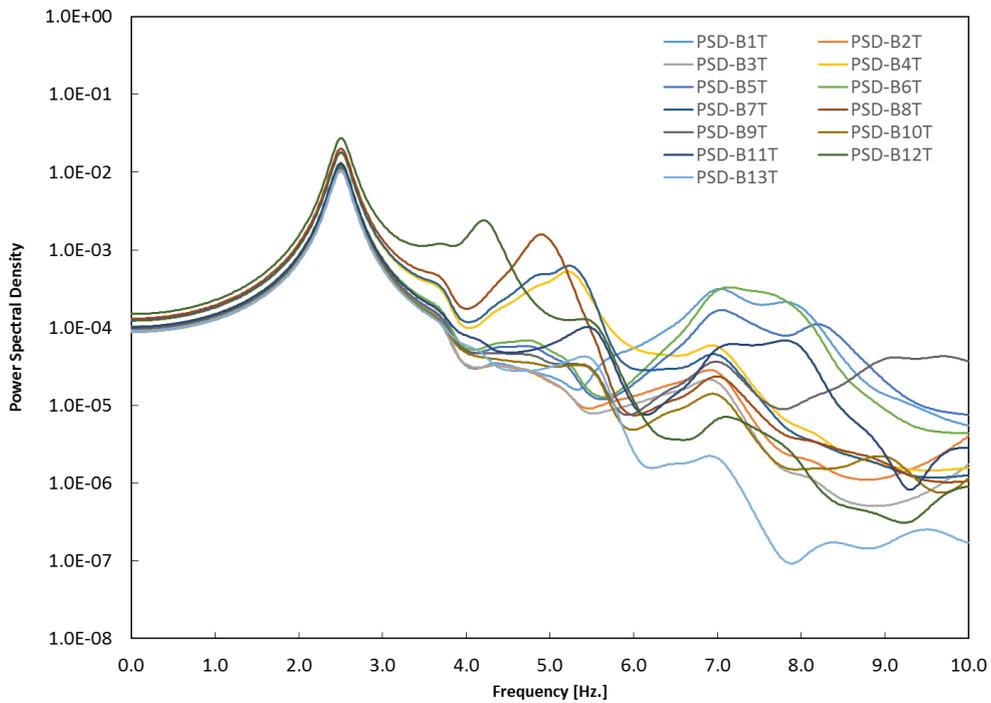

Figure 5. Power spectral density functions of the buildings top absolute displacements.
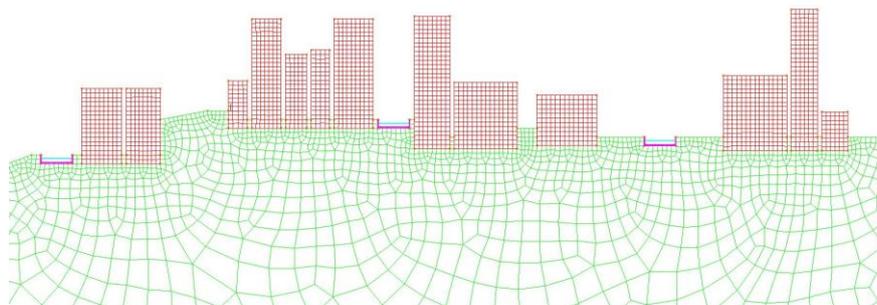

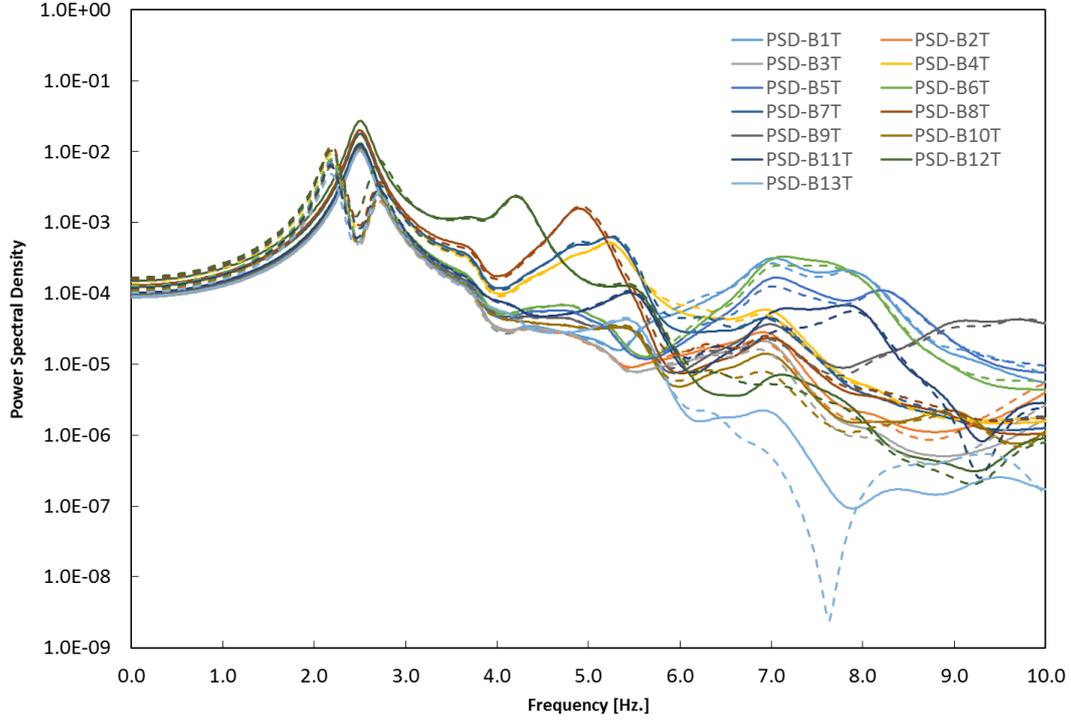*Figure 6. FE mesh of the array of ViBas in the Vathia village.*

Figure 7. Power spectral density functions of the buildings top absolute displacements with the ViBas [Dashed line] and without [Solid line].

The PSD of the city protected with the array of ViBas was calculated in the same manner as the city's response without the ViBas. As can be seen from Figure 7 there is an obvious and significant effect of the array of ViBas in the city by reducing the amplitude of the peak at the frequency that the devices were designed (the fundamental frequency of the system) this indicates a redistribution and absorption of the ground motion energy.

Finally, a pertinent Monte Carlo study has been performed generating 50 quasi-stationary ground motion time histories with power spectral density

$$S_{CP}(\omega) = \frac{\left(\omega_K^4 + 4\zeta_K^2 \omega_K^2 \omega^2\right)S_W}{(\omega_K^2 - \omega^2)^2 + 4\zeta_K^2 \omega_K^2 \omega^2} \cdot \frac{\omega^4}{(\omega_P^2 - \omega^2)^2 + 4\zeta_P^2 \omega_P^2 \omega^2} \tag{20}$$

with

$$S_W = \frac{0.141 \cdot \zeta_K \cdot \ddot{u}_{g0}^2}{\omega_K \sqrt{\left(1 + 4\zeta_K^2\right)}} \tag{21}$$

where $\ddot{u}_{g0}$ is the peak ground acceleration, taken as 0.3g and $\omega_K = 15.0$, $\zeta_K = 0.6$, $\omega_P = 1.5$, $\zeta_P = 0.6$.
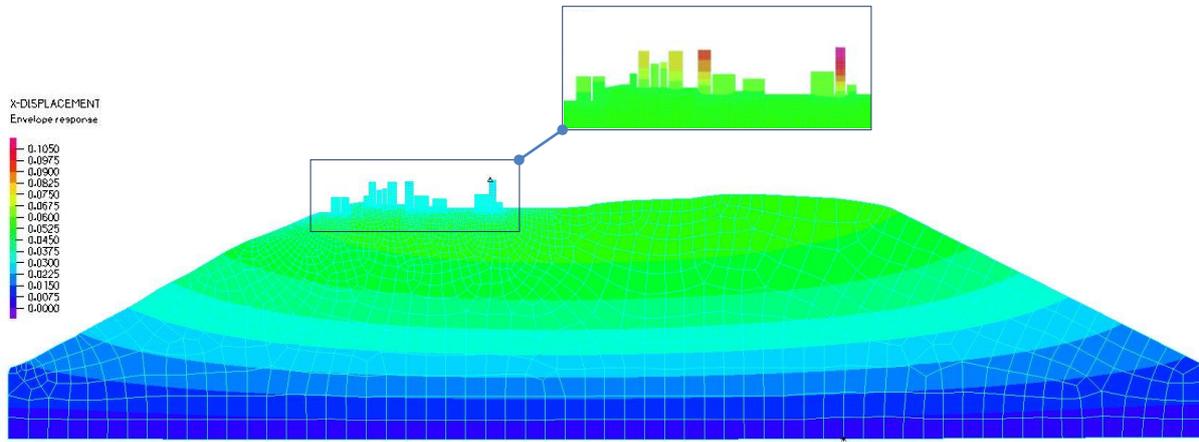
Figure 8. Band plot of maximum absolute displacements of the city without the ViBa.
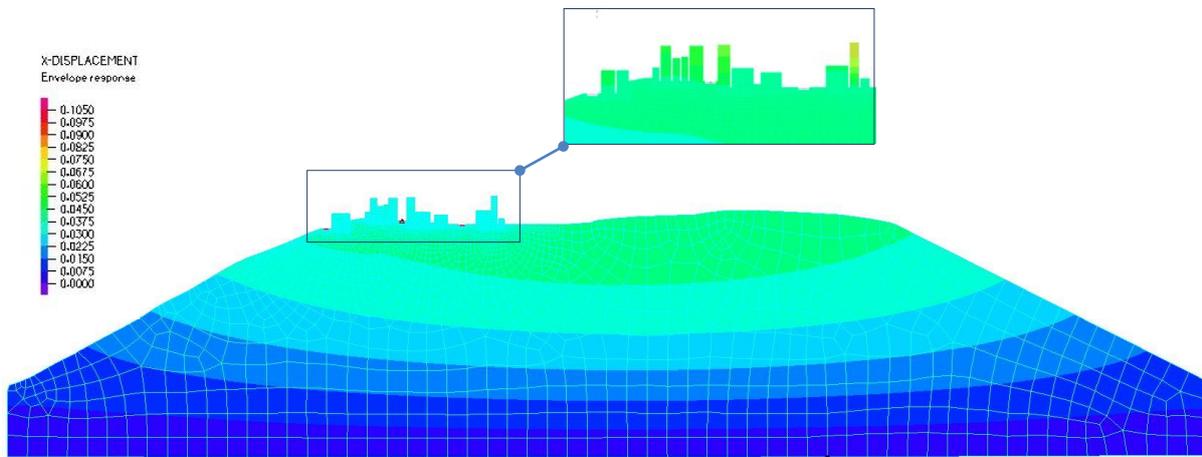

Figure 9. Band plot of maximum absolute displacements of the city with the array of ViBas.

Figures 8 and 9 show the band plots of the city model for a selected sample of simulated ground motion time history with and without the ViBas. It can be observed the beneficial effects of the ViBa in the reduction of the maximum displacements. Also Figure 9 shows that the ViBa is experiencing, as expected, the maximum displacements, undergoing "resonance" to absorb part of the seismic energy. It was evidenced form the Monte Carlo study that each building had a beneficial effect by the inclusion of the ViBas in the soil yielding a minimum average reduction of 12.22% and maximum reduction 26.15% of the maximum relative displacement. It is noted that additional masses/ViBas will further improve the beneficial effects.

5 Concluding remarks

The global grand challenge to improve urban seismic resilience through vibrating barriers has been addressed. A procedure for the design of the ViBa in case of linear behaving system and Gaussian stationary ground motion input has been presented. A simplified model of the village of Vathia has been developed and used as a case study to explore the efficiency of the ViBa to reduce the seismic induced structural vibrations. The adoption of the ViBa has been shown to be beneficial by reducing the maximum average peak displacement of every single building analysed. Despite the simplified model it is relevant to note that ViBa can be considered a novel promising alternative for protecting cities from earthquakes in the case in which

other techniques cannot be applied. Clearly further studies are necessary to explore additional pros and cons of this developing strategy.

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