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VIBRATION CONTROL OF A CLUSTER OF BUILDINGS THROUGH THE VIBRATING BARRIER

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Abstract

A novel device, called vibrating barrier (ViBa), that aims to reduce the vibrations of adjacent structures subjected to ground motion waves has been recently proposed. The ViBa is a structure buried in the soil and detached from surrounding buildings that is able to absorb a significant portion of the dynamic energy arising from the ground motion. The working principle exploits the dynamic interaction among vibrating structures due to the propagation of waves through the soil, namely the structure–soil–structure interaction. In this paper the efficiency of the ViBa is investigated to control the vibrations of a cluster of buildings. To this aim a discrete model of structures-site interaction involving multiple buildings and the ViBa is developed. In particular, the effects of the soil on the structures, i.e. the soil-structure interaction (SSI) as well as the structure-soil-structure interaction (SSSI) and the ViBa-soil-structures interaction are taken into account in this paper by means of linear elastic springs as in the conventional Winkler approach for a linear elastic soil medium.

Closed-form solutions are derived to design the ViBa in the case of harmonic excitation from the analysis of discrete models. Advanced Finite Element numerical simulations are performing in order to assess the efficiency of the ViBa in protecting one or more buildings. Parametric studies are also conducted to identify beneficial/adverse effects in the use of the proposed vibration control strategy to protect cluster of buildings.

Keywords: structure-soil-structure interaction; cluster of buildings; vibration control; vibrating barrier; dynamics.

1. Introduction

Large magnitude ground motion waves can induce unexpected structural behaviours that lead to rapid deterioration or collapse of buildings. Construction industry recently started to use devices such as isolators, dampers and tuned mass dampers to mitigate dynamic vibrations in new buildings. On the other hand, they are rarely used for protecting existing buildings, as they generally require substantial alteration of the original structure. In the case of heritage buildings and critical facilities or urban areas especially in developing countries, those traditional localized solutions might become impractical. Alternative solutions are to protect the structures introducing trenches or sheet-pile walls in the soil [1]. However, this approach seems to be more effective for surface waves coming from railways rather than seismic or body waves. Therefore, Cacciola and Tombari [2] introduced for the first time, a non-localized solution, called Vibrating Barrier (ViBa), hosted in the soil and detached from the structures. ViBa exploits the structure-soil-structure mechanism as a means of reducing the vibrations of structures due to seismic excitation or ground motion action. Analyses on the efficiency of the ViBa in protecting one building are reported in Cacciola et al. [3] for structure founded on monopile foundation and Tombari et al. [4] for a Nuclear Power Plant.

Furthermore, the ViBa interacts with every adjacent structures place in its area of influence; therefore, it can be used for mitigating the vibrations of a cluster of buildings. During the last two decades, several authors highlighted the effects of the structure-soil-structure interaction by means of studies on the site-city interaction [5-7]. Recently, Alexander et al. [8] developed a discrete model to study the SSSI problem of surface foundations by considering stochastic ground motion excitation and, Aldaikh et al. [9] extended the work of Alexander et al. [8] to the case of three buildings with validation of the discrete theoretical model by means of experimentally shake-table testing.

Based on the same principles, soil-structure interaction (SSI), structure-soil-structure interaction (SSSI) as well as ViBa-soil-structure interaction are simulated in this paper by means of linear visco-elastic springs. Efficiency of the ViBa is investigated in the case of two adjacent buildings with different dynamic characteristics. Results showed a remarkable reduction in terms of maximum harmonic acceleration up to 79.6% and beneficial effects are achieved in both structures.

2. Formulation of the governing equations for the global problem

Consider the global system depicted in Fig. *1* under the ground motion excitation at the bedrock, $u_{\mathrm{g}}(t)$. The proposed Vibrating Barrier (ViBa) is included aiming to reduce the vibration of the surrounding buildings. 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 [2]. Each building is modelled as 2-DOF system with one translational DOF at the top of the building and one at the foundation level, i.e. $u_{\mathrm{i}}(t)$ and $u_{\mathrm{f,i}}(t)$ for i=1,…, n (where n is the number of surrounding buildings). The ViBa is modelled as an internal oscillator device included in a rigid box foundation and globally described by the 2-DOFs, the internal motion of the oscillator, $u_{\mathrm{ViBa}}(t)$ and the displacement of its foundation, $u_{\mathrm{f,ViBa}}(t)$. The dynamic governing equations of the global system are derived in terms of absolute displacements, as is conventional in soil-structure interaction, namely the dynamics of the problem take the form:

$$\mathbf{M}\ddot{\boldsymbol{u}}(t) + \mathbf{C}\dot{\boldsymbol{u}}(t) + \mathbf{K}\boldsymbol{u}(t) = \mathbf{Q}_{\boldsymbol{e}} u_{\mathrm{g}}(t) + \mathbf{Q}_{\boldsymbol{d}} \dot{u}_{\mathrm{g}}(t) \tag{1}$$

where $\mathbf{M}$, $\mathbf{C}$ and $\mathbf{K}$ are the global mass, viscous damping and stiffness matrix; $\ddot{\boldsymbol{u}}(t)$ and $\dot{\boldsymbol{u}}(t)$, are respectively the absolute acceleration and velocity given by the second and first derivative of the displacement vector $\boldsymbol{u}(t)$ defined as:

$$\boldsymbol{u}^{\mathrm{T}}(t) = \begin{bmatrix} u_{\mathrm{i}}(t) & u_{f,\mathrm{i}}(t) & \cdots & u_n(t) & u_{f,n}(t) & u_{\mathrm{ViBa}}(t) & u_{f,\mathrm{ViBa}}(t) \end{bmatrix} \tag{2}$$

Consistently, $\dot{u}_{\mathrm{g}}(t)$ is the first derivative of the ground displacement $u_{\mathrm{g}}(t)$. The vectors $\mathbf{Q}_{\boldsymbol{e}}$ and $\mathbf{Q}_{\boldsymbol{d}}$ are the influence quantities; $\mathbf{Q}_{\boldsymbol{e}}$ depends on the soil-foundation stiffness values as follows:

$$\mathbf{Q}_{\boldsymbol{e}}^{\mathrm{T}} = [0 \quad k_{f,i} \quad \cdots \quad 0 \quad k_{f,n} \quad 0 \quad k_{f,\mathrm{ViBa}}] \tag{3}$$

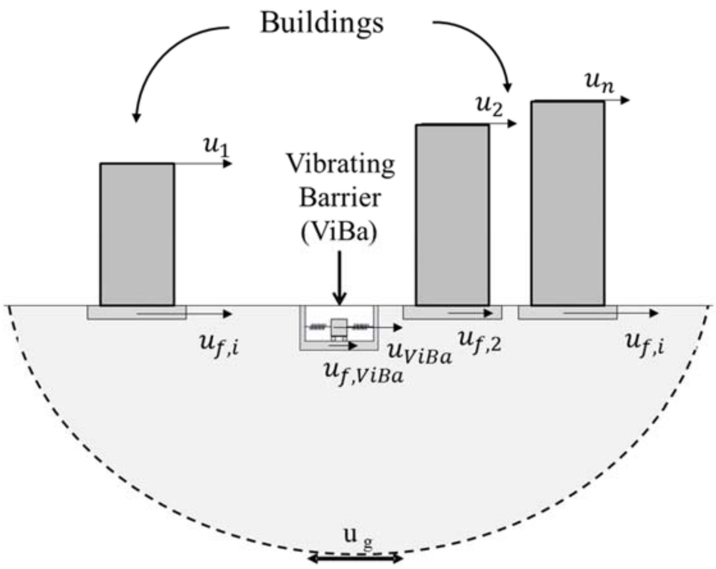

Fig. 1 Vibrating Barrier (ViBa) device embedded in the soil for protecting a cluster of buildings

whereas $\mathbf{Q}_d$ depends on the soil-foundation damping coefficients, which are listed in the following

$$\mathbf{Q}_d{}^{\mathrm{T}} = [0 \quad c_{f,i} \quad \cdots \quad 0 \quad c_{f,n} \quad 0 \quad c_{f,\mathrm{ViBa}}] \tag{4}$$

where T indicates the transpose operator.

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

$$\mathbf{M} = \begin{bmatrix} \mathbf{M}_1 & \mathbf{0} & \cdots & \mathbf{0} & \mathbf{0} \\ \mathbf{0} & \mathbf{M}_{\mathrm{i}} & & \mathbf{0} & \mathbf{0} \\ \vdots & & \ddots & & \vdots \\ \mathbf{0} & \mathbf{0} & \cdots & \mathbf{M}_n & \mathbf{0} \\ \mathbf{0} & \mathbf{0} & & \mathbf{0} & \mathbf{M}_{\mathrm{V}} \end{bmatrix} \tag{5}$$

in which the ith sub-block includes the mass of the ith structure, $\mathrm{m_i}$ and the mass of the ith foundation, $\mathrm{m_{f,i}}$ as follows:

$$\mathbf{M}_{\mathrm{i}} = \begin{bmatrix} m_i & 0 \\ 0 & m_{f,i} \end{bmatrix} \tag{6}$$

while $\mathbf{M}_{\mathrm{V}}$ is the mass matrix of the ViBa given by

$$\mathbf{M}_{\mathrm{V}} = \begin{bmatrix} m_{\mathrm{ViBa}} & 0 \\ 0 & m_{\mathrm{f,ViBa}} \end{bmatrix} \tag{7}$$

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

The global damping matrix $\mathbf{C}$ and the global stiffness matrix $\mathbf{K}$ are block-matrices partitioned in the following form:

$$\mathbf{C} = \begin{bmatrix} \mathbf{C}_1 & \mathbf{C}_{1,\mathrm{i}} & \cdots & \mathbf{C}_{1,n} & \mathbf{C}_{1,\mathrm{V}} \\ \mathbf{C}_{\mathrm{i},1} & \mathbf{C}_{\mathrm{i}} & & \mathbf{C}_{\mathrm{i},n} & \mathbf{C}_{\mathrm{i},\mathrm{V}} \\ \vdots & & \ddots & & \vdots \\ \mathbf{C}_{n,1} & \mathbf{C}_{n,\mathrm{i}} & \cdots & \mathbf{C}_{n} & \mathbf{C}_{n,\mathrm{V}} \\ \mathbf{C}_{\mathrm{V},1} & \mathbf{C}_{\mathrm{V},\mathrm{i}} & & \mathbf{C}_{\mathrm{V},\mathrm{n}} & \mathbf{C}_{\mathrm{V}} \end{bmatrix} \tag{8}$$

for the damping matrix, while:

$$\mathbf{K} = \begin{bmatrix} \mathbf{K}_1 & \mathbf{K}_{1,\mathrm{i}} & \cdots & \mathbf{K}_{1,n} & \mathbf{K}_{1,\mathrm{V}} \\ \mathbf{K}_{\mathrm{i},1} & \mathbf{K}_{\mathrm{i}} & & \mathbf{K}_{\mathrm{i},n} & \mathbf{K}_{\mathrm{i},\mathrm{V}} \\ \vdots & & \ddots & & \vdots \\ \mathbf{K}_{n,1} & \mathbf{K}_{n,\mathrm{i}} & \cdots & \mathbf{K}_{n} & \mathbf{K}_{n,\mathrm{V}} \\ \mathbf{K}_{\mathrm{V},1} & \mathbf{K}_{\mathrm{V},\mathrm{i}} & & \mathbf{K}_{\mathrm{V},\mathrm{n}} & \mathbf{K}_{\mathrm{V}} \end{bmatrix} \tag{9}$$

for the stiffness matrix. The main diagonal sub-matrices $\mathbf{C}_\mathrm{r}$ and $\mathbf{K}_\mathrm{r}$ (r =1,…, n) describe the viscous damping and stiffness matrix of the rth-structure and its interaction with the soil. The matrices $\mathbf{C}_\mathrm{V}$ and $\mathbf{K}_\mathrm{V}$ defines the damping and stiffness matrix of the ViBa and its interactions to the other buildings through the soil. Lastly, the off-diagonal sub-matrices $\mathbf{C}_{\mathrm{i,j}}$ and $\mathbf{K}_{\mathrm{i,j}}$ (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 $\mathbf{Q}_{\boldsymbol{e}}$ and $\mathbf{Q}_{\boldsymbol{d}}$.

In the previous formulation, the structural parameters of the ViBa represent the unknowns of the problem, as they have to be determined in order to reduce the dynamic response of the adjacent structures. The objective of the ViBa is to reduce the vibrations of the adjacent structures and the consequent stresses related to the relative displacements. Therefore, the optimization problem is established as:

$$\begin{gathered} \min\left\{u_\mathrm{i}^{\mathrm{r,max}}(\mathrm{t},\boldsymbol{\alpha})\right\} \;\; \mathrm{i} = 1,\dots,n \\ \boldsymbol{\alpha} = \{\mathbf{K}_\mathrm{V}, \mathbf{M}_\mathrm{V}, \mathbf{C}_\mathrm{V}\} \in \mathbb{R}_0^+ \end{gathered} \tag{10}$$

where $u_\mathrm{i}^{\mathrm{r,max}}(\mathrm{t},\boldsymbol{\alpha})$ is the maximum displacement of the ith structure relative to its foundation:

$$u_\mathrm{i}^{\mathrm{r,max}}(\mathrm{t}) = \max(u(\mathrm{t}) - u_{f,\mathrm{i}}(\mathrm{t})) \tag{11}$$

and $\boldsymbol{\alpha}$ is the design parameter vector. It is worth emphasized that various objective functions of the optimization problem of Eq. (10) can be selected (e.g. L^1-norm, L^2-norm or weighted norms). The solution of the optimization problem in Eq. (10) is usually obtained numerically; however, closed-form expressions can be derived in some particular cases as described in the following sections.

2. Optimal ViBa tuning for controlling two buildings

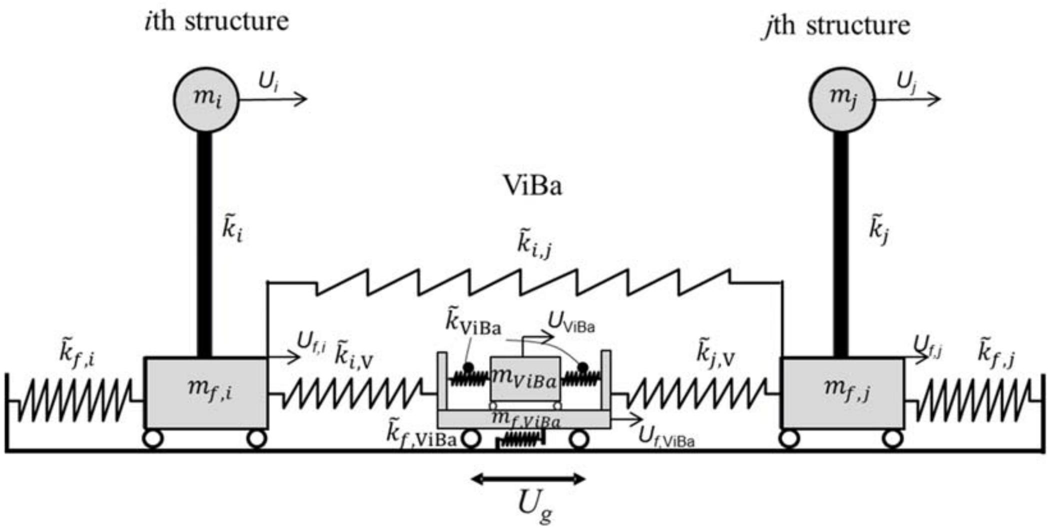

Fig. 2 Simplified Mechanical model of two structures protected by ViBa device

Consider the global system composed of two buildings protected by the Vibrating Barrier as illustrated in Fig. *2* with i=1 and j=2 where damping effects are taken into account according to the hysteretic damping model given by $\tilde{k} = k(1 + i\eta)$ where η is the loss factor and $i = \sqrt{-1}$ is the imaginary unit. Note that the hysteretic damping used in this section assumes the same values of the viscous damping when $c = \frac{k\eta}{\omega}$.

The governing equation of the motion Eq. (1) can be analysed in the frequency domain by applying the Fourier Transform as follows:

$$\widetilde{\mathbf{K}}_{\mathrm{dyn}}(\boldsymbol{\alpha}, \omega)\mathbf{U}(\omega) = \mathbf{Q}\,\mathrm{U_g}(\omega) \tag{12}$$

where $\widetilde{\mathbf{K}}_{\mathrm{dyn}}(\boldsymbol{\alpha}, \omega) = \widetilde{\mathbf{K}}(\boldsymbol{\alpha}) - \omega^2\mathbf{M}(\boldsymbol{\alpha})$ is the dynamic stiffness matrix and $\boldsymbol{\alpha}$ is the design parameters vector. If the shape of two foundations of the buildings is identical, the interaction with the soil is identical as well and the following relations occur: $\tilde{k}_f = \tilde{k}_{f,1} = \tilde{k}_{f,2}$ and $\tilde{k}_{\mathrm{SSSI}} = \tilde{k}_{1,V} = \tilde{k}_{2,V}$. Therefore, the dynamics of the problem of Eq. (9) is rewritten in the expanded form:

$$\left\{\begin{bmatrix} \tilde{k}_1 & -\tilde{k}_1 & 0 & 0 & 0 & 0 \\ -\tilde{k}_1 & \tilde{k}_1 + \tilde{k}_f + \tilde{k}_{1,2} + \tilde{k}_{\mathrm{SSSI}} & 0 & -\tilde{k}_{1,2} & 0 & -\tilde{k}_{\mathrm{SSSI}} \\ 0 & 0 & \tilde{k}_2 & -\tilde{k}_2 & 0 & 0 \\ 0 & -\tilde{k}_{1,2} & -\tilde{k}_2 & \tilde{k}_2 + \tilde{k}_f + \tilde{k}_{1,2} + \tilde{k}_{\mathrm{SSSI}} & 0 & -\tilde{k}_{\mathrm{SSSI}} \\ 0 & 0 & 0 & 0 & \tilde{k}_{ViBa} & -\tilde{k}_{ViBa} \\ 0 & -\tilde{k}_{\mathrm{SSSI}} & 0 & -\tilde{k}_{\mathrm{SSSI}} & -\tilde{k}_{ViBa} & \tilde{k}_{\mathrm{ViBa}} + \tilde{k}_{f,\mathrm{ViBa}} + 2\tilde{k}_{\mathrm{SSSI}} \end{bmatrix} - \omega^2 \begin{bmatrix} m_1 & 0 & 0 & 0 & 0 & 0 \\ 0 & m_{f,1} & 0 & 0 & 0 & 0 \\ 0 & 0 & m_2 & 0 & 0 & 0 \\ 0 & 0 & 0 & m_{f,2} & 0 & 0 \\ 0 & 0 & 0 & 0 & m_{ViBa} & 0 \\ 0 & 0 & 0 & 0 & 0 & m_{f,ViBa} \end{bmatrix}\right\} \begin{bmatrix} \mathrm{U_1}(\omega) \\ \mathrm{U_{f,1}}(\omega) \\ \mathrm{U_2}(\omega) \\ \mathrm{U_{f,2}}(\omega) \\ \mathrm{U_{ViBa}} \\ \mathrm{U_{f,ViBa}} \end{bmatrix} = \begin{bmatrix} 0 \\ \tilde{k}_f \\ 0 \\ \tilde{k}_f \\ 0 \\ \tilde{k}_{\mathrm{f,ViBa}} \end{bmatrix} \mathrm{U_g}(\omega) \tag{13}$$

Eq. (12) is analysed by resorting to the transfer function representation that provides a basis for determining system response characteristics. The transfer functions of the system are defined as the ratio of the output $\mathbf{U}$ and input displacement U_g:

$$\begin{aligned}\mathbf{H}(\boldsymbol{\alpha},\omega) &= \tilde{\mathbf{K}}_{\mathrm{dyn}}^{-1}(\boldsymbol{\alpha},\omega)\mathbf{Q} \\ &= [\mathrm{H}_1(\boldsymbol{\alpha},\omega)\quad \mathrm{H}_{f,1}(\boldsymbol{\alpha},\omega)\quad \mathrm{H}_2(\boldsymbol{\alpha},\omega)\quad \mathrm{H}_{f,2}(\boldsymbol{\alpha},\omega)\quad \mathrm{H}_{\mathrm{ViBa}}(\boldsymbol{\alpha},\omega)\quad \mathrm{H}_{f,\mathrm{ViBa}}(\boldsymbol{\alpha},\omega)]^{\mathrm{T}}\end{aligned} \tag{14}$$

Assuming the ground motion excitation modelled by a harmonic signal with the frequency ω_0. The adopted procedure consists in minimizing the transfer functions related to the structures at the input frequency ω_0. The vector of the design parameters is reduced to $\boldsymbol{\alpha} = \{k_{\mathrm{ViBa}}, m_{\mathrm{ViBa}}, \eta_{\mathrm{ViBa}}\}$, see e.g. Cacciola and Tombari [2]. From Eq. (14), the optimization problem is stated as:

$$\begin{gathered}\min\{\mathrm{H}_1(\boldsymbol{\alpha},\omega_0), \mathrm{H}_2(\boldsymbol{\alpha},\omega_0)\} \\ \boldsymbol{\alpha} = \{k_{\mathrm{ViBa}}, m_{\mathrm{ViBa}}, \eta_{\mathrm{ViBa}}\} \in \mathbb{R}_0^+\end{gathered} \tag{15}$$

Clearly, the solution of the optimization problem will be straightforward if it is possible to assign a variable. It is noted that the mass of the ViBa m_{ViBa} is restrained by engineering criteria (e.g. bearing capacity of the soil, volumetric restraint, etc...). Therefore, by assigning m_{ViBa} as a known quantity, from Eq. (15) the stiffness $k_{\mathrm{ViBa}}^{\mathrm{optimal}}$ and the damping $\eta_{\mathrm{ViBa}}^{\mathrm{optimal}}$ are derived in closed form by determining the zeros of the transfer functions $\mathrm{H}_1(\boldsymbol{\alpha},\omega_0)$ and $\mathrm{H}_2(\boldsymbol{\alpha},\omega_0)$. Following simple algebra, the following formula is derived:

$$\tilde{k}_{\mathrm{ViBa}}^{\mathrm{optimal}}(\omega_0) = \frac{(\omega_0^2 m_{\mathrm{ViBa}})\left[\tilde{k}_{f,\mathrm{ViBa}} + \tilde{k}_{\mathrm{SSSI}}\left(2 + \frac{\tilde{k}_{f,\mathrm{ViBa}}}{\tilde{k}_f}\right) - \omega_0^2 m_{f,\mathrm{ViBa}}\right]}{\tilde{k}_{f,\mathrm{ViBa}} + \tilde{k}_{\mathrm{SSSI}}\left(2 + \frac{\tilde{k}_{f,\mathrm{ViBa}}}{\tilde{k}_f}\right) - \omega_0^2\left(m_{f,\mathrm{ViBa}} + m_{\mathrm{ViBa}}\right)} \tag{16}$$

From Eq. (16), the stiffness $k_{\mathrm{ViBa}}^{\mathrm{optimal}}$ and the damping $\eta_{\mathrm{ViBa}}^{\mathrm{optimal}}$ are derived as follows:

$$\begin{aligned}k_{\mathrm{ViBa}}^{\mathrm{optimal}} &= \mathrm{Re}\left\{\tilde{k}_{\mathrm{ViBa}}^{\mathrm{optimal}}(\omega_0)\right\} \\ \eta_{\mathrm{ViBa}}^{\mathrm{optimal}} &= \frac{\mathrm{Im}\left\{\tilde{k}_{\mathrm{ViBa}}^{\mathrm{optimal}}(\omega_0)\right\}}{\mathrm{Re}\left\{\tilde{k}_{\mathrm{ViBa}}^{\mathrm{optimal}}(\omega_0)\right\}}\end{aligned} \tag{17}$$

where $\mathrm{Re}\{\cdot\}$ and $\mathrm{Im}\{\cdot\}$ indicate the real and imaginary component of complex value $\tilde{k}_{\mathrm{ViBa}}^{\mathrm{optimal}}$. Therefore, after proper tuning, the Vibrating Barrier is designed for mitigating the dynamic response of both two buildings.

3. Numerical Results

In this section, the proposed optimization procedure for tuning the Vibrating Barrier (ViBa) parameters is applied to investigate the dynamic response of the scenario consisting of two simple scaled buildings, Structure 1 and Structure 2, supported on embedded mat foundations. A finite element representation of the model is depicted in Fig. 3a. The ground soil is defined by a linear visco-elastic material. Each structure has on the top of the roof an added mass of 0.1 kg. The characteristics of the materials used in the numerical analysis are reported in Table 1. The structural walls have a rectangular cross section of 40 mm width and 0.12 mm thick. The height of the Structure 1 is 170 mm; this value is kept as constant while the height of the Structure 2 changes according the values reported in Table 2. In particular three different case studies are analysed starting from the case of identical structures and then by decreasing the height of the Structure 2 in order to modify the natural frequencies. It is worth mentioning that the natural frequency of the Structure 1, kept as constant, slightly varies with the cases due to the structure-soil-structure interaction effect.

The described model is analysed numerically by adopting a finite element formulation through the software SAP2000 [10]; for each of the three case studies, two models, with and without the ViBa, are investigated. The Vibrating Barrier is modelled as a single oscillator consisting of mass, spring and dashpot placed in the embedded box foundation.

The aim of the study is to reduce the maximum accelerations of both Structure 1 and Structure 2 at a design frequency. Harmonic accelerations are applied to the soil base in order to analyse the frequency response function of the overall system.

Table 1 Properties of the materials used in the model

Element	Young's elastic modulus (kPa)	Poisson coefficient	Unit weight (kN/m^3)
Soil	470.66	0.47	12.29
Structural walls	69637055	0.33	26.61
Foundation/roof	2452000	0.35	11.67

Table 2 Characteristics of the investigated structures and first natural frequencies

	Structure 1			**Structure 2**		
Case Study	**A**	**B**	**C**	**A**	**B**	**C**
Height [mm]	170			170	160	150
1st freq. [Hz]	12.07	12.11	12.11	12.07	13.03	14.06
1st freq. with trench [Hz]	12.07	12.19	12.2	12.07	13.14	14.18

It is worth mentioning that the trench realized for containing the ViBa slightly alters the natural frequencies of the buildings as reported in Table *2*.

3.1 Case Study A

Case study A is related to the scenario with two equal structures as illustrated in Fig. *3*. In order to protect them from seismic loadings, a Vibrating Barrier (ViBa) device is embedded on the soil between the two structures (see Fig. *4*) at the relative distance of 40mm from each building. The mass of the ViBa is assigned as 0.63 kg while the remaining parameters, that is the stiffness and the damping coefficient is obtained by using Eq. (16) and Eq. (17). The ViBa is tuned in order to protect the structures at the frequency ω_0 corresponding to the "resonant" or first natural frequency of both buildings. The soil interaction stiffness coefficients are derived by conventional numerical technique such as the matrix stiffness method since the soil behaviour is assumed as linear elastic. The ViBa spring is simulated by means of a two-joint link while a lumped mass is assigned to a free node place at the end of the link. It is worth mentioning that the optimal damping coefficient derived from Eq. (17) can lead to numerically negative solution as already shown in Cacciola and Tombari [2]. Therefore, in order to simulate a realistic behaviour, a constrained optimisation problem is performed by using a physical constrained boundary damping ratio of 0.01. Finally, a damping ratio of the soil equal to 0.05 is applied for illustrative purpose.

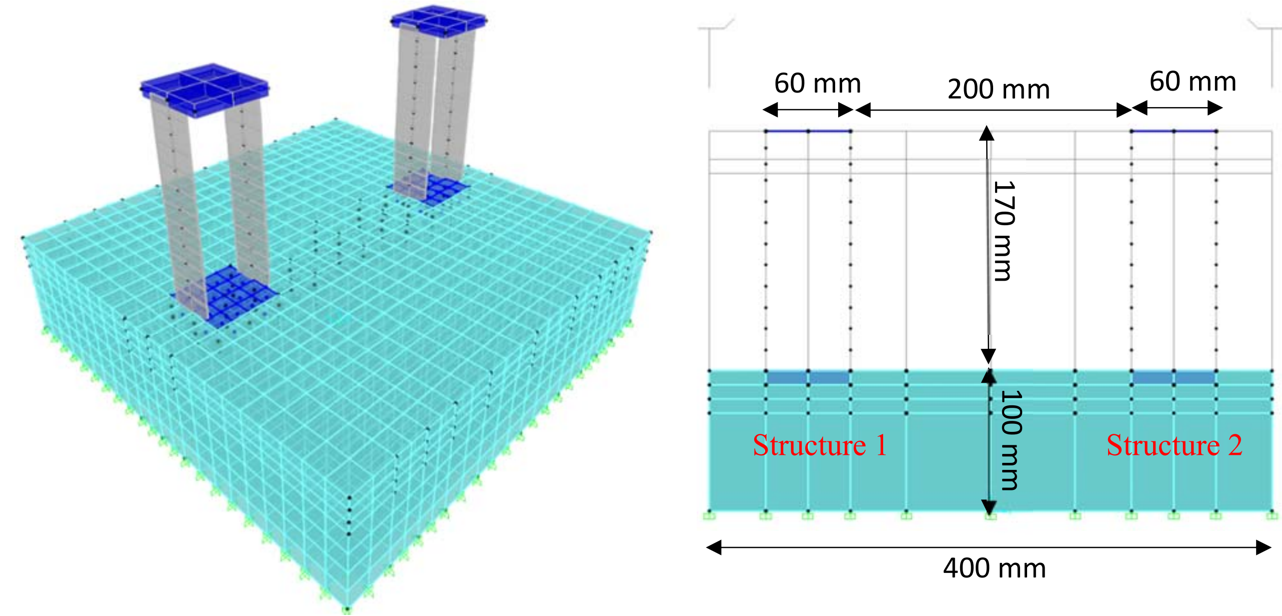

Fig. 3 Finite Element Model of the case study A (equal structures); a) 3D view; b) section view (pre-meshing) before construction of the ViBa

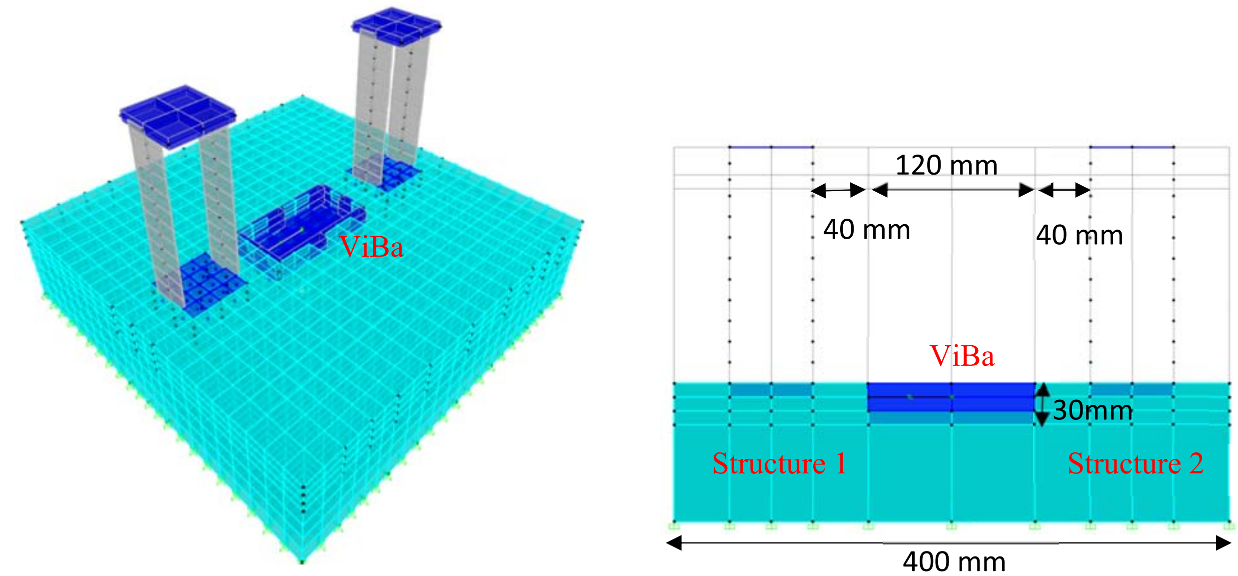

Fig. 4 Finite Element Model of the case study A (equal structures); a) 3D view; b) section view (pre-meshing) with ViBa

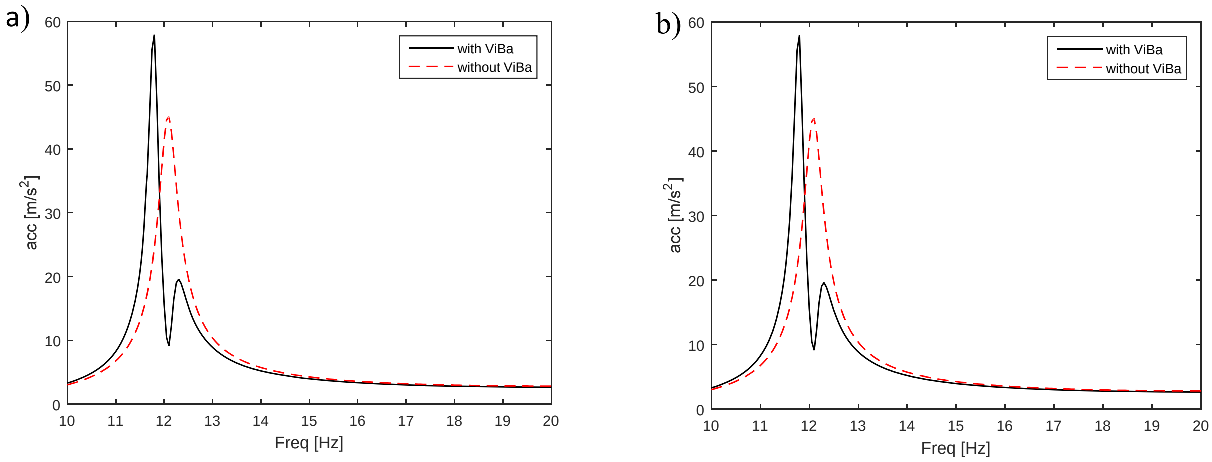

Fig. 5 Frequency response acceleration of a) Structure 1 and b) Structure 2 with and without the coupling with ViBa for the case study A

Steady state analyses are performed for both scenarios of structures without and with protection of the ViBa. The structural response in acceleration in the frequency domain is reported in Fig. 5. It is worth noting that the two structural responses are the same since the two buildings are equal. Moreover, the ViBa acts as a similar tuned mass damper by strongly reducing the structural response at the tuned input frequency of 12.1 Hz; remarkably, a relevant reduction of 79.6% of the maximum acceleration for both structures is accomplished by using the ViBa. It is worth mentioning that in case of broad-band signals, another procedure for tuning the ViBa parameters may be used in place of Eq. (16) as done for stochastic analysis in Cacciola et al. [3-4].

3.2 Case Study B

Case study B is related to the scenario with the same Structure 1 used for the case study A whereas Structure 2 has an reduced height of 0.15 m, as illustrated in Fig. 6a. The other geometric characteristics of the soil, structure and ViBa are unchanged. It is worth noting that the tuning formula of Eq. (17) is independent of the structural characteristics. Therefore, the same ViBa, previously tuned, can be used for this case. This is remarkable since the performance of the ViBa is still effective even if buildings will be modified onwards.

Steady state analyses are performed for both scenarios, of structures without and with protection of the ViBa. The structural response in acceleration in the frequency domain is reported in Fig. 7. For the harmonic input at the frequency 12.2 Hz, corresponding to the natural frequency of the Structure 1, still relevant reductions are recorded; reduction over 74.6% of the maximum acceleration is accomplished for Structure 1 by using the ViBa. The beneficial effects are obtained even for Structure 2 where it is achieved a relevant reduction of 75.5% at the working frequency of the ViBa at 12.2 Hz; furthermore, at the resonance, the structural response of Structure 2 is reduced of about 36% without being tuned for it.

3.2 Case Study C

In Case Study C, as depicted in Fig. 6b, the height of the Structure 2 is 0.15 m while the same Structure 1 of case study A is used. The other geometric characteristics of the soil, structure and ViBa are kept unchanged. Therefore, this case investigates the performance of the ViBa when buildings with different natural frequency are located in its area of influence. The aim is to protect the Structure 1 by investigating the effects on the surrounding Structure 2.

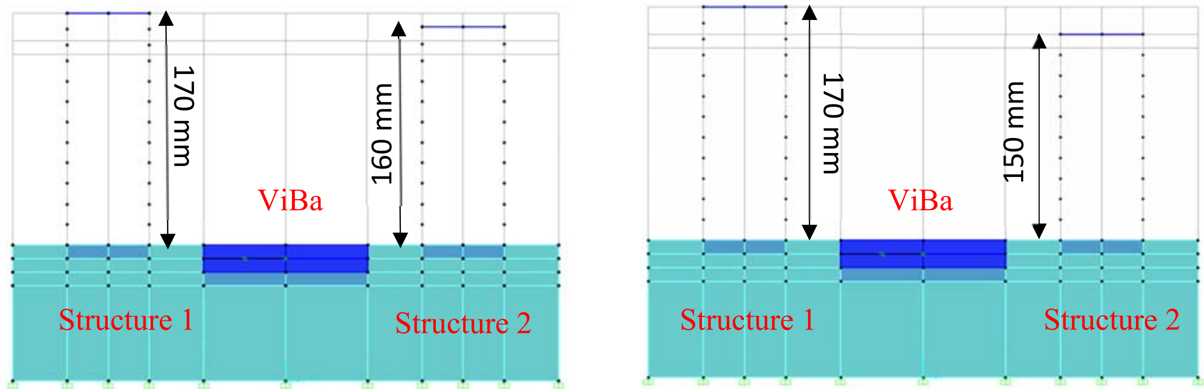

Fig. 6 Finite Element Model of the case study a) B and b) C (pre-meshing) with ViBa

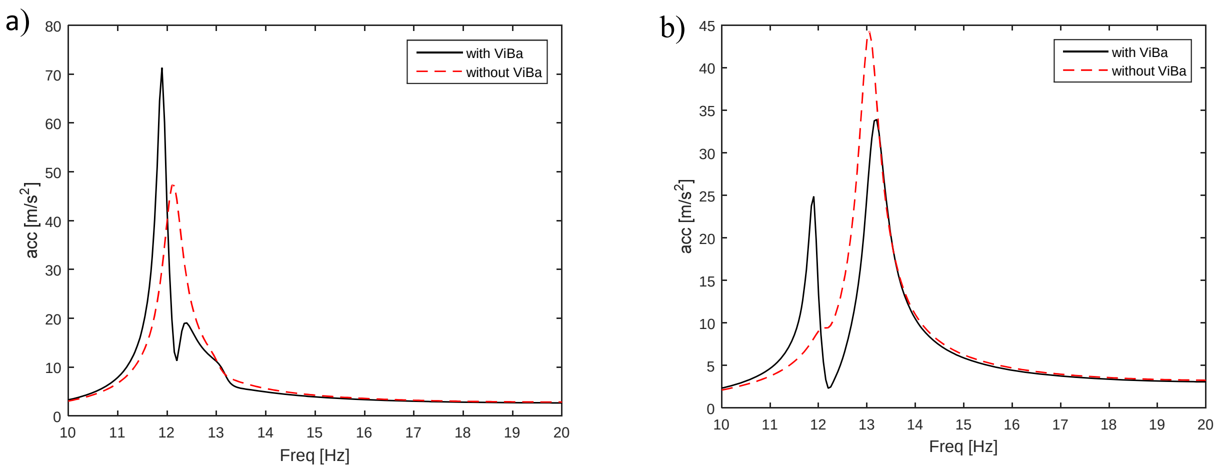

Fig. 7 Frequency response acceleration of a) Structure 1 and b) Structure 2 with and without the coupling with ViBa for the case study B

Steady state analyses are performed with and without the coupling to the ViBa device. The structural acceleration response in the frequency domain is reported in Fig. *8*. For the harmonic input at the frequency 12.2 Hz, corresponding to the natural frequency of the Structure 1, relevant reductions are recorded; reduction over 74.3% of the maximum acceleration is accomplished for Structure 1 by using the ViBa. The beneficial effects are obtained even for Structure 2 where it is achieved a relevant reduction of 70.1% at the working frequency of the ViBa at 12.2 Hz; furthermore, the structural response of Structure 2 is reduced of about 29.8% for the resonant frequency of Structure 2.

Finally, Fig. *9* shows the effect of the trench on the structural responses of both buildings for case B and C; slightly differences are observed due to the alteration of the structure-soil-structure interaction caused by the excavation. This manifests clearly the role of the vibrating component of the ViBa in comparison of the static counterpart (i.e. the trench realized to embed the ViBa) that slightly alters the structural frequency response by increasing the soil stiffness.

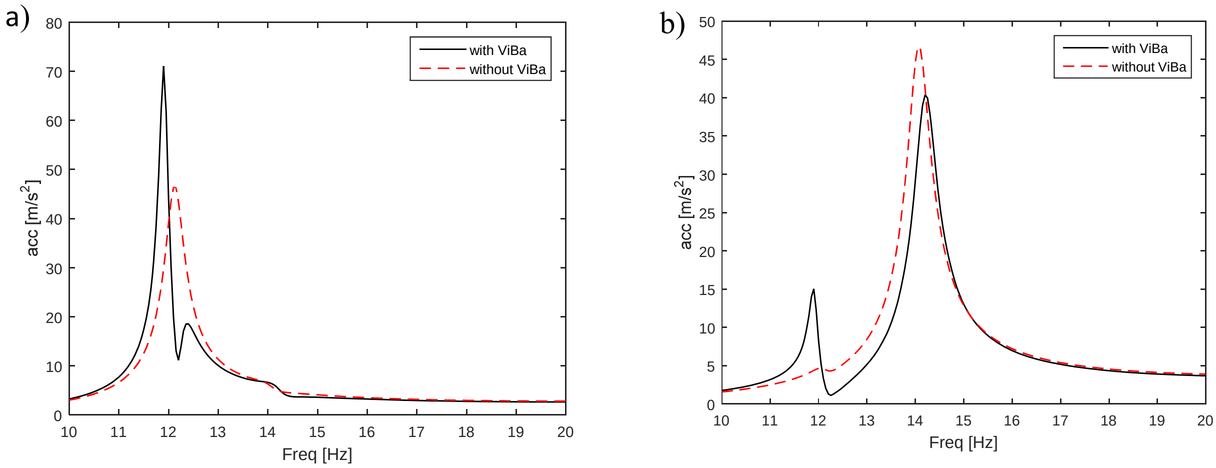

Fig. 8 Frequency response acceleration of a) Structure 1 and b) Structure 2 with and without the coupling with ViBa for the case study C

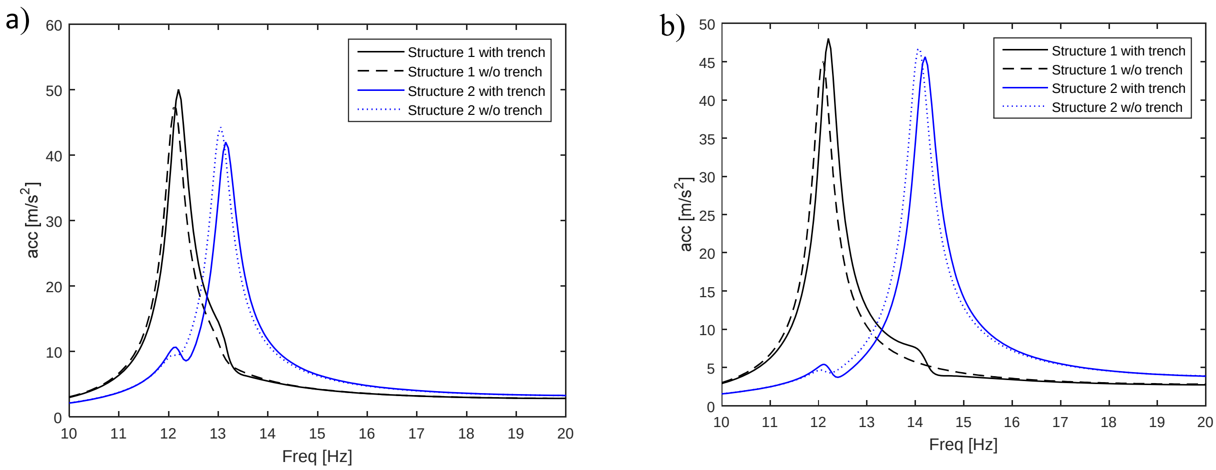

Fig. 9 Frequency response acceleration of Structure 1 and Structure 2 before and after realizing the trench for case B a) and C b)

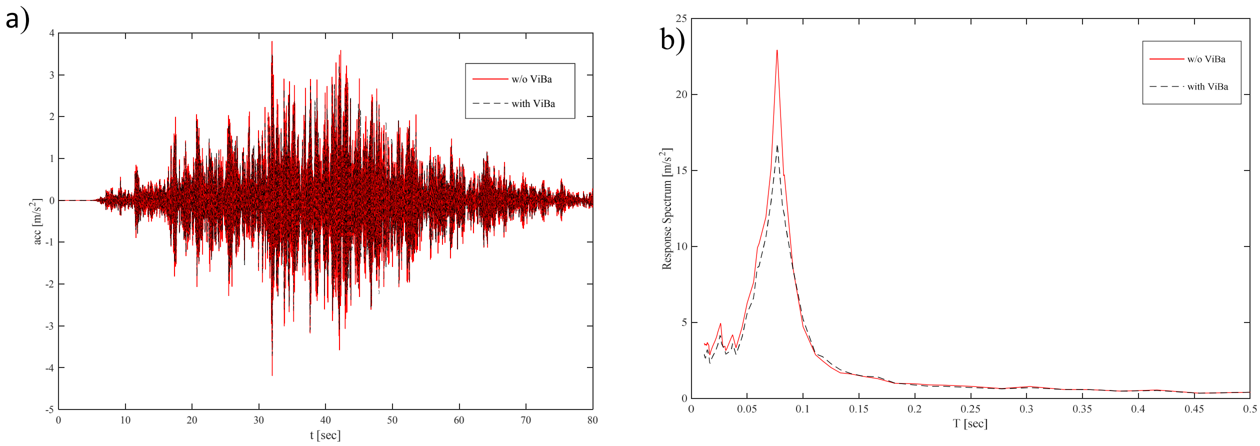

Fig. 10 Dynamic response of Structure 2 subjected to the Chile Coquimbo 2015 event in terms of a) time-history acceleration and b) elastic response spectrum with and without the coupling with ViBa

3.4. Response to seismic excitation

Finally, the performance of the ViBa is tested for a wide-band signal. The real earthquake event of Chile Coquimbo of 8.3Mw, occurred in the 2015 is thus applied to the system. The considered signal is the component 360° recorded at the Torpederas station near the city of Valparaiso. It is worth mentioning that the same parameters used to tuning the ViBa under harmonic signals, are here adopted; an optimal tuning procedure for wide-band signal can be found in [3-4]. By investigating the Case Study C, a reduction of about 20% of the maximum structural acceleration is obtained as depicted in Fig. *10*a. Fig. *10*b shows the elastic response spectrum curves for the same structure before and after being protected by the ViBa. It can be observed that the peak of the structural response is mitigated of about 30%. On the other hand, due to the frequency content of the real earthquake, there are neither any significant beneficial nor detrimental effects are observed for the Structure 1.

4. Conclusion

The paper presents the application of the Vibrating Barrier (ViBa) to protect a cluster of two buildings. A numerical finite element model is realized by considering both case of unprotected structures and structures under the dynamic protection of the Vibrating Barrier. The ViBa parameters are designed by using a closed-form solution. The effectiveness and the efficiency of the ViBa is highlighted by analysing three different scenarios in which the buildings have equal and different dynamic characteristics. In every case, relevant reductions of over 70% and up to 76.6% (at the structure's natural frequency) of the maximum harmonic acceleration are achieved. Moreover, beneficial effects have been observed for both structures in every investigated case. It is worth mentioning that further investigations will involve more refined models with larger number of degrees of freedom, material nonlinearities and uncertainties. Moreover, in case of seismic excitation the ViBa should be designed according a different optimization procedure as outlined in [3].

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