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SEISMIC PROTECTION OF THE MESSINA CATHEDRAL BELL TOWER THROUGH VIBRATING BARRIERS

Michele ANDREOZZI[1], Alessandro TOMBARI[2], Andreas LAMPROPOULOS [3] & Pierfrancesco CACCIOLA[3]

Abstract: *Seismic protection of heritage structures is an unresolved research problem that not only is a public safety issue but also encompasses the additional constraint of the protection of the artistic value. In this regard non-invasive techniques appear clearly the most appropriate. In this paper the recently proposed non-invasive vibration control device Vibrating Barrier (ViBa) is scrutinized and extended to reduce the seismic vibrations of the Messina Cathedral Bell Tower in Italy. A detailed study is first undertaken to determine a reliable model of the Cathedral and surrounding area. Original drawings and design calculation has been used to develop a pertinent Finite Element model. The seismic action has been modelled as a zero mean Gaussian process compatible with the response spectrum of the site where the Cathedral is built. In order to design the Vibrating Barrier device a reliable simplified discrete model has been derived and calibrated through an identification procedure. Due to the dynamic behaviour of the Bell Tower a novel design strategy accounting for the rotational coupling of the underlying structure-soil-structure interaction problem has been proposed. Interestingly, the influence of the rocking behaviour of the Tower on the performance of the ViBa has been addressed. The efficiency of the ViBa device has been measured through the reduction of the dynamic response in terms of displacements and internal forces. Reduction of the peak displacements greater than 20% has been achieved without modifying the Bell Tower. Challenges in the implementations of the ViBa and its limitations are also discussed.*

Introduction

The design and the construction material of historic structures in seismic prone areas are generally not adapted to withstand earthquakes and decay occurring over the years. In addition, climate change further modifies the seismic vulnerability of historic areas and current models and technologies do not take into account this effect. Many historic areas have also been affected by past earthquakes making them even more vulnerable. The most recent earthquakes, such as the ones occurring in Central Apennines (Italy) during the 2016-2017 seismic sequence, responsible for the collapse of the Cathedral of S. Benedetto at Norcia, are clear examples of the need for prompt and collective action to protect heritage structures from further natural disaster-induced damage. Historic structures are vital to our understanding of how the cultural, artistic, and technical skills of humanity have developed over time. Up to now the problem of protecting structures by seismic action has been managed using localised solutions such as isolators and dampers. Apart from few attempts to protect existing structures the use of vibration control devices is still restricted to new buildings and/or constructions. One main reason is that the introduction of control devices in existing structures is too invasive, costly and requires the demolishing of some structural and/or non-structural components. For heritage structures clearly such technologies cannot be applied and therefore rarely seismic protection actions are taken to protect such artistic treasure. Bearing in mind the global necessity to protect existing structures from earthquakes and the limitation of current technologies the novel vibrating barrier (ViBa) control strategy has been recently proposed (Cacciola and Tombari 2015). The ViBa device is a massive structure, hosted in the soil, calibrated for protecting structures by absorbing portion of the ground motion input energy. As a difference with other technologies buried in the soil (i.e. trenches, piles, seismic metamaterials) that are focused on surface waves only (see. e.g. Palermo et al. 2016; Craster et al 2018) the ViBa is designed to absorb seismic body waves (i.e. shear waves). In its simplest configuration it is made by a mass spring system buried in the soil and

[1] Visiting Scholar from University of Bologna, University of Brighton, Brighton, UK
[2] Senior Lecturer, University of Brighton, Brighton, UK
[3] Principal Lecturer, University of Brighton, Brighton, UK

able to vibrate. The concept is based on the generally known structure-soil-structure interaction (SSSI) and on the findings of the first works of Warburton et al. (1971) and Luco and Contesse (1973). Up to now the ViBa has been applied to various case studies. Specifically, Cacciola et al. (2015) investigated the potential of ViBa for the seismic protection of monopiled structures, Tombari et al. (2016) considered ViBa to mitigate seismic risk of a nuclear reactor, and Tombari et al. (2018) explored the efficiency of the ViBa to protect a cluster of buildings. This paper investigates the effect of the ViBa on a realistic model of an existing heritage structure, the Bell Tower of the Messina Cathedral. A finite element formulation is used for modelling the Cathedral, the Bell Tower, the soil as well as the ViBa. The design of the ViBA is carried on for the protection of the Bell Tower and seismic analyses are carried out by considering response-spectrum-compatible ground motion processes. Results show a relevant reduction of the structural response and offer potential development of the technology.

Messina Cathedral: modelling and numerical study of the bell tower

The Bell tower of the Messina Cathedral (Figure 1) has been constructed between the 1923 and 1929. At that time, the use of reinforced concrete as structural material was already widely diffused; therefore, the skeleton of this building is composed by a reinforced concrete frame. In Figures 1 and 2 geometrical data are reported. Moreover, the structural frame of the bell tower is covered externally by a 20cm thick marble stone masonry. The various floors are connected though a reinforced concrete staircase organized with two runs for each floor, except for the first two floors where the runs are three.

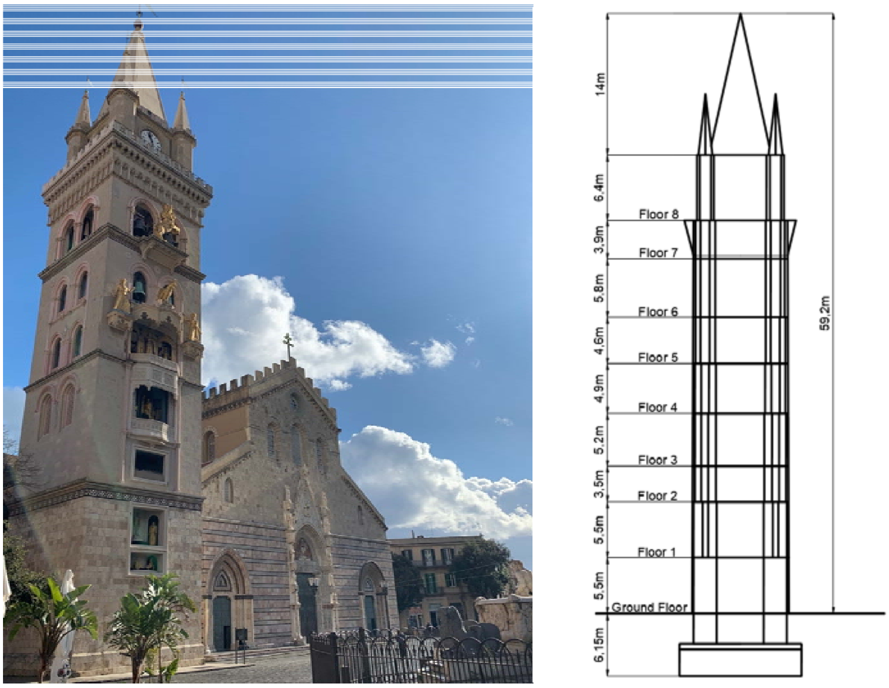

Figure 1: Messina Cathedral and Bell Tower (a), front view of the Bell Tower and relevant dimensions (b)

FE Model

Finite element method is used to create the soil-structure model of the Messina Cathedral as depicted in Figure 3. Structural elements such as 1D frame elements are used to model the columns of the main nave and the crypt and structural parts of the roof such as beams and purlins, whereas 2D shell elements are used to model the walls and roof wooden cladding. Soil deposit and foundation of the cathedral are modelled through 8-node solid elements.

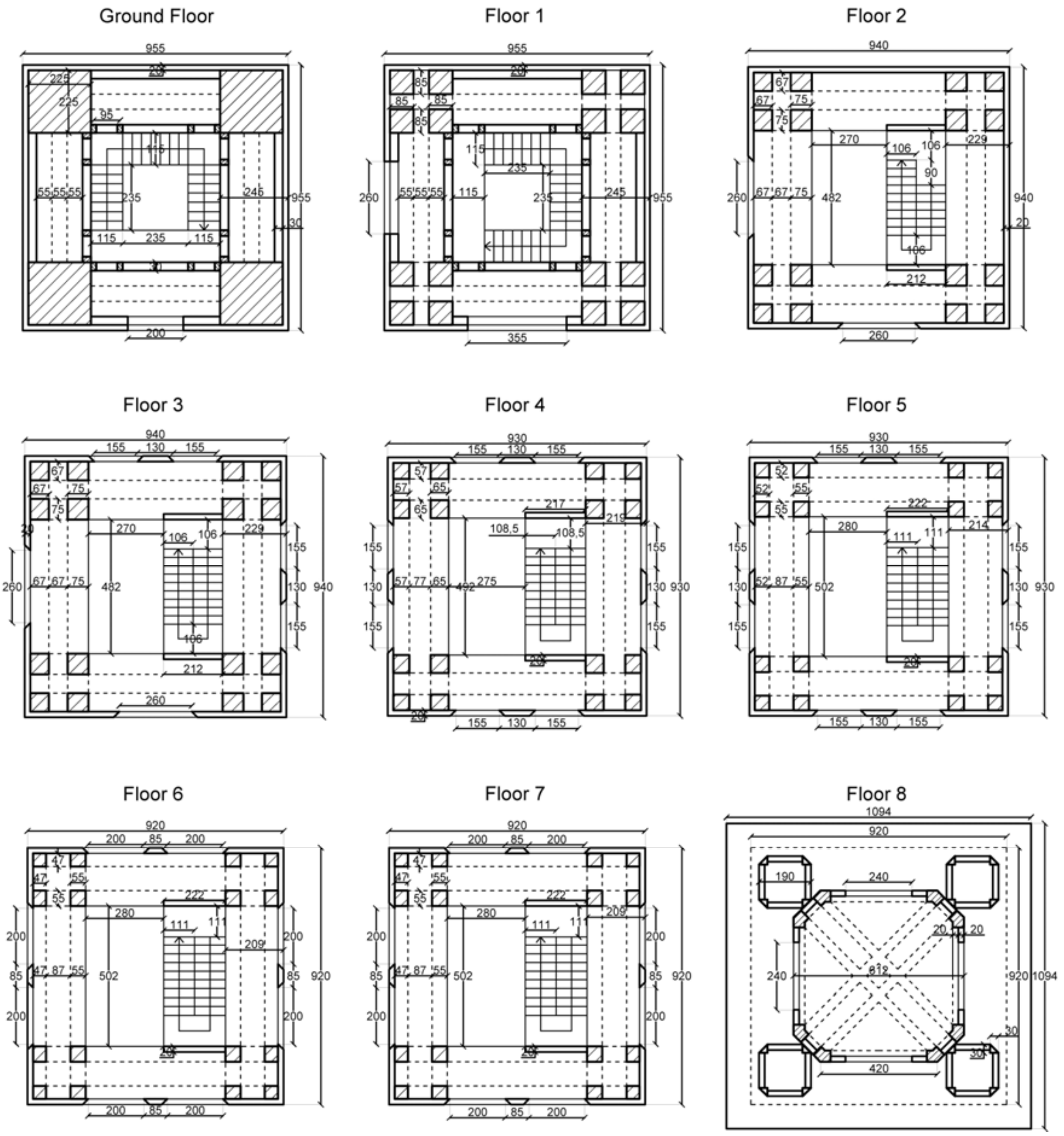

Figure 2: Plan views of each floor of the Bell Tower. Dimensions in cm

The walls of the cathedral are quite irregular, with many decorative elements, statues, and altars, especially on the lateral walls of the naves. In the FE model, the lateral walls have been created with a uniform thickness computed as an average where the real walls are irregular. The definition of the material properties has been done using the data available in the original design and in the laws/codes in place at that period. In particular, the Royal Decree Law of the 4th of September 1927 specifies the characteristics of the materials that had to be used in new constructions. For reinforced concrete, the law recommended to use a density of 2400 kg/m^3 and an Elastic Modulus of 15000 MPa. From the original design calculations it has been observed that the concrete compressive strength used was equal to 3 MPa, interestingly this strength value falls within the expected range of values of the model proposed by Ahmad et al. (2014) for low strength concrete with Elastic Modulus equal to 15000 MPa. Bedrock depth and mechanical characteristics such as soil density and shear modulus are required to fully characterize the elastic dynamic behaviour of the soil deposit. From the seismic characterization carried out for the Messina soil (MS 2014), the 20-meter deep a marine layer stratum is characterized by silty gravel, mixture of gravel, sand and silt; a marine deposit layer with mechanical characteristics reported in Table 1; from 20m downward, a layer consisted of rocks and alternating layers of lithotypes is assumed a a rigid bedrock.The first 20 meters below the ground level are characterized by the same type of soil: silty gravel, mixture of gravel, sand and silt; a marine deposit layer with poor mechanical characteristics. From 20m downward, instead, we can find a layer of much more compact material: rocks and alternating layers of lithotypes. In Table 1, the material properties implemented in the FE Model are reported.

Material	Density (Kg/m^3)	E (Pa)	v
Reinforced Concrete	2400	$1.5 \cdot 10^{10}$	0.2
Soil	1980	35000000	0.3
Stone Masonry Walls	2250	$4.608 \cdot 10^{9}$	0.3
Timber	800	$1.0 \cdot 10^{10}$	0.3
Root Tiles	1200	2000000	0.3

Table 1: Material Properties used in the FE model

The non-structural elements (external walls, bells, etc.) have been also taken into account in the FE Model as additional masses. It is important to highlight that the tower hosts the biggest and most complex mechanical and astronomical clock in the world. Every day, a complex system of counterweights, leverages and gears, determines the movement of the gilded bronze statues located in the façade. In the FEM, the heaviest bronze statues, consisting of a 4m-high roaring lion and a 2m-high rooster, have been introduced as concentrated masses. The specific weight of the bells has been determined through the analysis of ancient documents related to the construction of the bell tower and the installation of the system of bells. In particular, it has been found that the total mass of the set of bells is equal to 16.6 tons in total.

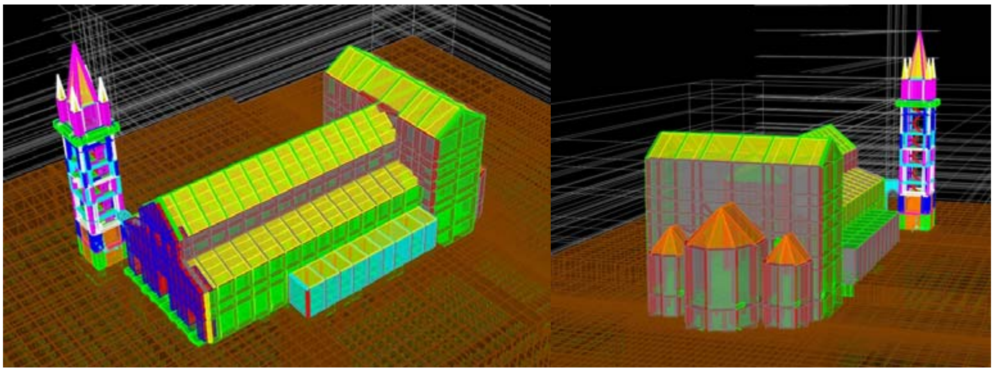

Figure 3: Finite Elements Model of the Messina Cathedral

Modal Analysis

After the model has been implemented in SAP2000, modal analysis has been performed. Figure 4 shows the first three modal shapes with $f_1 = 0.4384\ Hz, f_2 = 0.6087\ Hz$ and $f_3 = 0.6312\ Hz$ respectively. It is noted that the although the design of the ViBa will concern the bell tower only, the Cathedral had to be included in the model due to the strong structure-soil-structure interaction that alters the dynamic behaviour of the bell tower. Moreover it can be seen that second and third mode shapes, in which the bell tower is mainly involved, manifest an evident rocking behaviour.

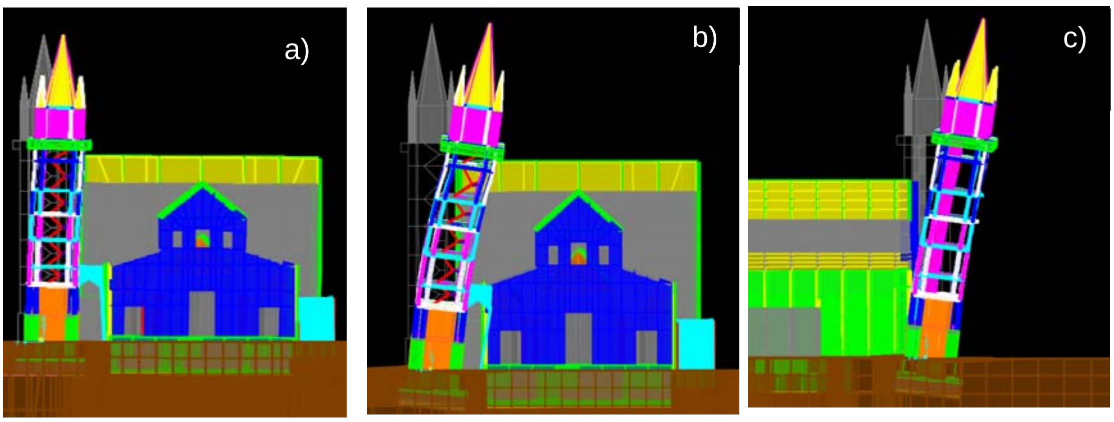

Figure 4: First three mode shapes of the Messina Cathedral and Bell Tower a) First mode, b) second mode and c) third mode.

Design of the Vibrating Barrier device

In this section the design of the Vibrating barrier device is undertaken. The objective is to minimize the dynamic response of the Bell Tower forced by ground motion modelled as a Gaussian stochastic process. The first step in the design of a the Vibrating Barrier for the seismic protection of the Bell Tower of the Messina Cathedral is the construction of a simplified model able to reproduce the dynamic behaviour of the structure which will be used in the optimization procedure. The ViBa's parameters will be determined in order to minimize the response of the bell tower and then, they will be implemented in the FE model in order to verify the results from the optimization procedure.

Seismic input

The seismic input is modelled as quasi-stationary zero mean Gaussian process compatible with the Pseudo Acceleration Response Spectrum (RSA) defined by Italian Code (NTC2008) at the site where the Cathedral is built. In this regard the response-spectrum-compatible power spectral density function $G_{\ddot{u}_g}(\omega)$ has been determined using the procedure proposed by Cacciola et al. (2004). Figure 5 shows the response-spectrum-compatible power spectral density function (after 10 iterations) and the relevant comparison between the average response spectrum (100 samples) and the target one defined by the NTC2008 to satisfy the response spectrum compatibility criteria.

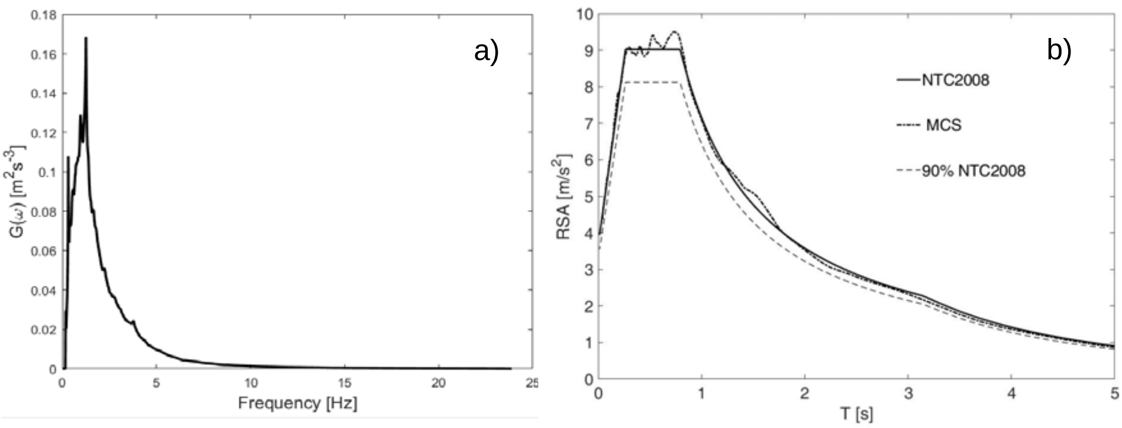

Figure 5: Power Spectral Density function a) and comparison of target and simulated response spectra b) for the Messina Cathedral site.

Location of the Vibrating Barrier

The implementation of the ViBa requires the construction of a foundation able to support this load. Furthermore, it has been demonstrated (see e.g. Cacciola and Tombari 2015, Cacciola et al. 2015) that the dynamic interaction between the foundation and the ViBa increases if they are built very close one to each other. Given these considerations, a possible location of the ViBa is on the east side of the bell tower, in the existing space between the tower, the church's north walls and the building of the rectory, as depicted in **Error! Reference source not found.**a.

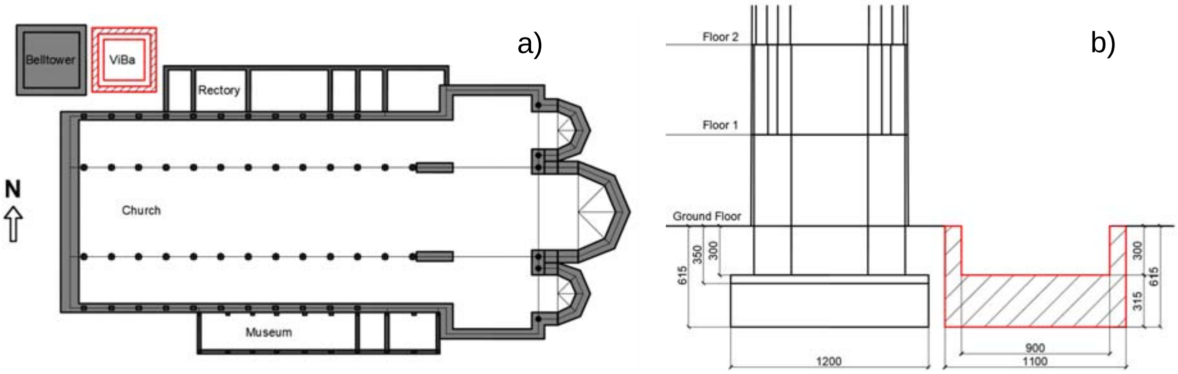

Figure 6: Plan view Location of the ViBa a) and cross section b)

The geometry of the foundation of the ViBa has been chosen in order to be similar to that one of the bell tower. Therefore, the foundation of the ViBa is a square 11x11m reinforced concrete foundation embedded for 6.15m in the ground as depicted in Figure 6b. The foundation has as a principal function to support the oscillating mass of the ViBa; being usually this mass in the same order of magnitude of the mass of the structure to be protected, the base of the foundation has been chosen to be as thick as the concrete platform supporting the bell tower, 3.15m. Moreover, inside the embedded foundation there must be enough space to host the oscillating mass with all its supporting apparatus. The determination of the space needed inside the ViBa's foundation has been carried out considering the side of a steel cube with a mass approximately equal to that of the bell tower. It resulted in the necessity to host a cube of 7m side, plus the space it needs for oscillate. Then, inside the foundation it has been left a room of 9x9m with a depth of 3m. The lateral walls of the foundation have a thickness of 1m, and it has been located 1m from the eastern side of the bell tower, at 1.35 from the rectory, and 3.35m from the northern external wall of the church. The material chosen for the construction of the ViBa foundation is reinforced concrete, for a total mass of 2987805 Kg.

Simplified numerical model of the bell tower and ViBa

In order to design the Vibrating Barrier device, consider the simplified model depicted in Figure 7a. It is a 22 degree of freedom system. The masses are assumed lumped at each node, therefore rotational inertia is neglected apart from the foundation ones as it has a significant impact on the dynamic response due to the rocking effect as shown in the FE modal analysis. Relevant masses are reported in Table 2.

Element		**Mass(Kg)**
Foundation of the ViBa	m_{fV}	2987805
Foundation	m_f	4282259
First Floor	m_1	613893
Second Floor	m_2	439357
Third Floor	m_3	303259
Fourth Floor	m_4	297409
Fifth Floor	m_5	253937
Sixth Floor	m_6	247692
Seventh Floor	m_7	233582
Eighth Floor	m_8	447924
Ninth Floor	m_9	158980

Element		**Rotational Inertia (Kgm^2)**
Foundation of the ViBa	I_{rfV}	53267707
Foundation	I_{rf}	92836393

Table 2: Masses and Rotational Inertia implemented in the simplified model

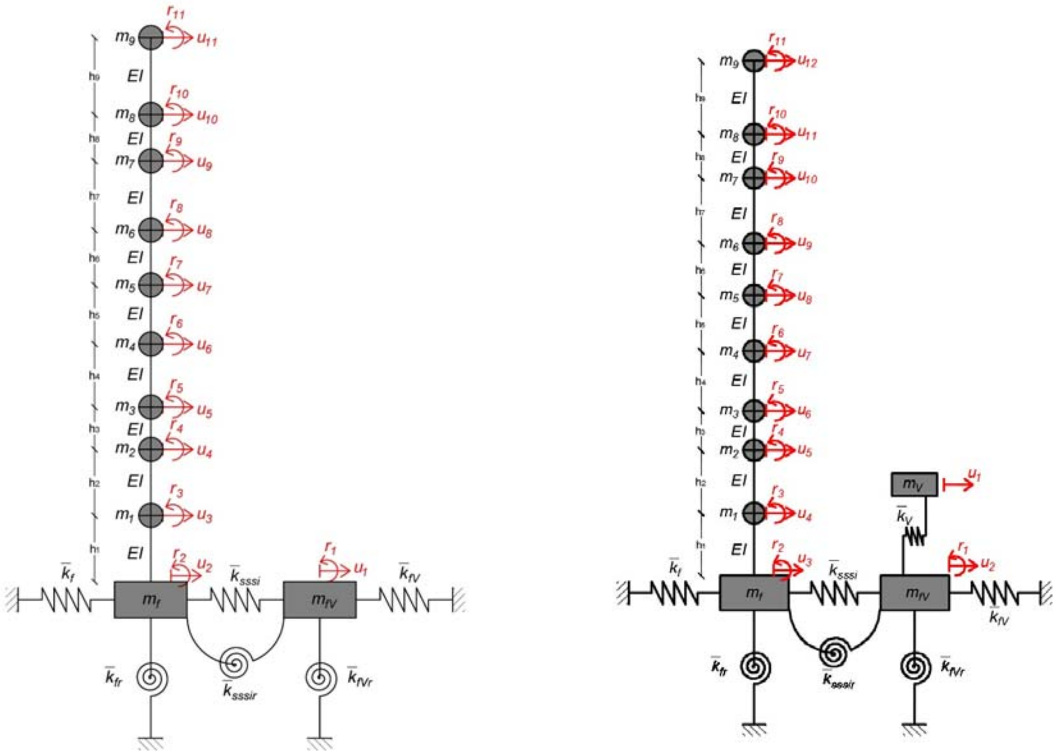

Figure 7: Simplified model of the Bell Tower coupled with: a) the ViBa foundation only and b) the ViBa device

The soil influence is modelled introducing elastic spring following the Winkler approach, i.e. 6 springs in total: 3 rotational and 3 translational. It is noted that the simplified model presented in Figure 7 differs from the traditional one adopted to design the ViBa. The evident rocking effect of the tower (see Figure 4) compelled the adoption of rotational degrees of freedom at the foundation level and the related soil interaction rotational stiffness. The value of the spring stiffness has been determined through the direct stiffness method applied to the FE model. Table 3 shows the values used in the numerical model.

Parameter	**Value**	**Unit**
k_f	$1.0373\ 10^{9}$	*N/m*
k_{sssi}	$4.0664\ 10^{9}$	*N/m*
k_{fV}	$1.4240\ 10^{9}$	*N/m*
k_{fr}	$7.1597E\ 10^{10}$	*Nm*
k_{sssir}	$1.4121\ 10^{10}$	*Nm*
k_{fVr}	$8.3469\ 10^{10}$	*Nm*

Table 3: Masses and Rotational Inertia implemented in the simplified numerical model

The superstructure of the bell tower is considered shear type. The interstorey heights are taken from the geometry (see e.g. Fig 1b) while the flexural stiffness $EI = 9.4767 \cdot 10^{11} Nm^2$, assumed identical for each storey, has been determined through an identification procedure aimed to minimize the differences between the FE and the simplified numerical model transfer function $H(\omega)$ of the translational response. Figure 8 shows the comparison between the transfer functions for both translational and rotational response. Despite the overestimation of the rotational response the simplified numerical model fairly reproduce the dynamic response of the bell tower and will be used as a base for the design of the ViBa device represented in Figure 7b.

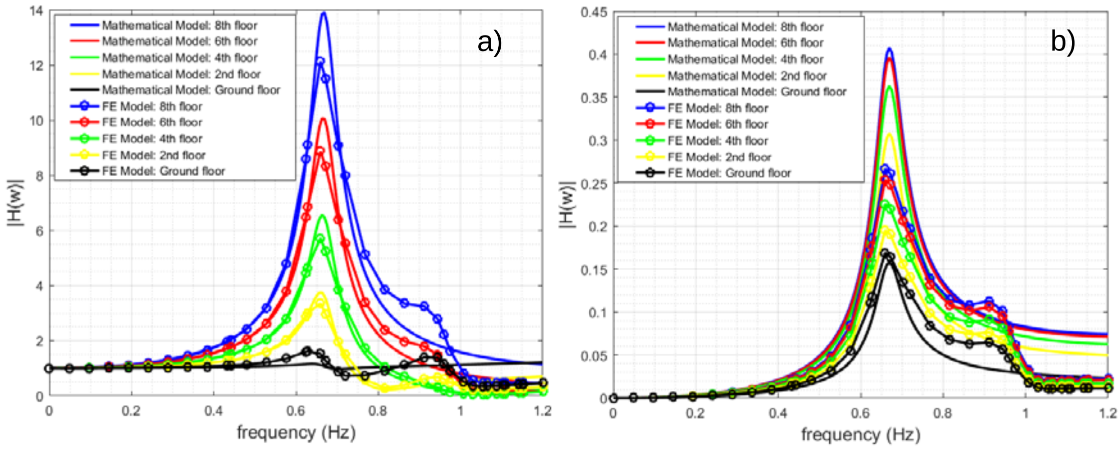

Figure 8: Comparison of the transfer functions determined through the FE model and the simplified numerical model a) translations, b) rotations

Design of the ViBa Device and numerical results

Once defined the simplified numerical model of the Bell Tower coupled with the ViBa device (Figure 7b) it can be seen that unknowns of the problem are the ViBa mass m_V, and stiffness k_V. Due to the basic hypothesis that ground motion is modelled as a Gaussian zero mean stochastic process, owing to the linearity of the system the response of the coupled system will be also zero mean and Gaussian, therefore is fully defined by the knowledge of the response power spectral density matrix:

$$\boldsymbol{G_{UU}}(\omega) = \boldsymbol{H}(\omega) \cdot \boldsymbol{H}^*(\omega) \cdot PSD(\omega) \tag{1}$$

where $*$ is the complex conjugate transpose and the transfer function $\boldsymbol{H}(\omega)$ given by

$$\boldsymbol{H}(\omega) = \widetilde{\boldsymbol{K}}_{\boldsymbol{dyn}}(\omega)^{-1} \cdot \boldsymbol{Q} \tag{2}$$

with

$$\widetilde{\boldsymbol{K}}_{\boldsymbol{dyn}}(\omega) = \widetilde{\boldsymbol{K}} - \omega^2 \boldsymbol{M} \tag{3}$$

where $\widetilde{\boldsymbol{K}}$ and $\boldsymbol{M}$ are the complex stiffness and the mass matrix of the coupled stiffness, respectively and $\boldsymbol{Q}$ is given by

$$\boldsymbol{Q} = \widetilde{\boldsymbol{K}}\boldsymbol{\tau} \tag{4}$$

$\boldsymbol{\tau}$ being the incidence vector. Note that the PSD matrix of the response lists both the response of the ViBa and the structure to be protected, then it can be used to minimize the maximum response of the structure by calibrating the ViBa structural parameters. The fractile of order *p* of the distribution of the maxima of the absolute displacement of a specified degree of freedom of the structure has been selected as the parameter to be minimized, clearly different response parameters can be used (e.g. base shear, bending moment, etc.) . Specifically the displacement at the 8th floor has been selected for this study, that is

$$X_{U_8} = \eta_{U_8}(T_s, p) \cdot \sqrt{\lambda_{0,U_8}} \tag{5}$$

where $\eta_{U_8}(T_s, p)$ is a peak factor depending on the order of the fractile *p* and the time of the observing window T_s, while λ_{0,U_8} is the zero-order response spectral moment. The peak factor is computed by the equation defined by Vanmarcke (1975):

$$\eta_{U_8}(T_s, p) = \sqrt{2ln\left\{2N_{U_8}\left[1 - exp\left[-\delta_{U_8}{}^{1.2}\sqrt{\pi ln(2N_{U_8})}\right]\right]\right\}} \tag{6}$$

with

$$N_{U_8} = \frac{T_S}{-2\pi\, lnp}\sqrt{\frac{\lambda_{2,U_8}}{\lambda_{0,U_8}}} \tag{7}$$

and

$$\delta_{U_8} = \sqrt{1 - \frac{{\lambda_{1,U_8}}^2}{\lambda_{0,U_8}\lambda_{2,U_8}}} \tag{8}$$

where the response spectral moment λ_{i,U_8} are computed with the following equation:

$$\lambda_{i,U_8} = \int_0^{+\infty} \omega^i \, G_{U_8U_8}(\omega) d\omega \tag{9}$$

$G_{U_8U_8}(\omega)$ being the power spectral density of the displacement of the 8th floor and the parameters T_S and p are taken respectively as 15s and 0.5. Hence, the optimization process has as a target to compute the optimal ViBa's stiffness of the ViBa for a selected mass so that to reduce the maximum displacement fractile at the 8th floor, X_{U_8}:

$$\left.\begin{array}{c} \min\{X_{U_8}(\alpha)\}, \\ \alpha = \{k_{ViBa}\} \in \mathbb{R}_0^+ \end{array}\right\} \tag{10}$$

Table 4 shows the results of the reduction for different values of the mass of the ViBa. Pertinent Power spectral density function $G_{U_8U_8}(\omega)$ of the displacement at the 8th floor are presented in Figure 9a. Comparison with the FE model for a selected mass of the ViBa is reported in Figure 9b.

$\frac{m_V}{m_S}$	m_V $[Kg]$	k_V $[N/m]$	X_{U_8} reduction [%]
3	8988099	$1.3797 \cdot 10^8$	-10.1330
5	14980165	$2.2976 \cdot 10^8$	-14.7279
7	20972231	$3.2192 \cdot 10^8$	-18.2304
9	26964297	$4.1257 \cdot 10^8$	-20.9897
11	32956363	$5.0114 \cdot 10^8$	-23.1777

Table 4: ViBa mechanical parameters and displacement reduction

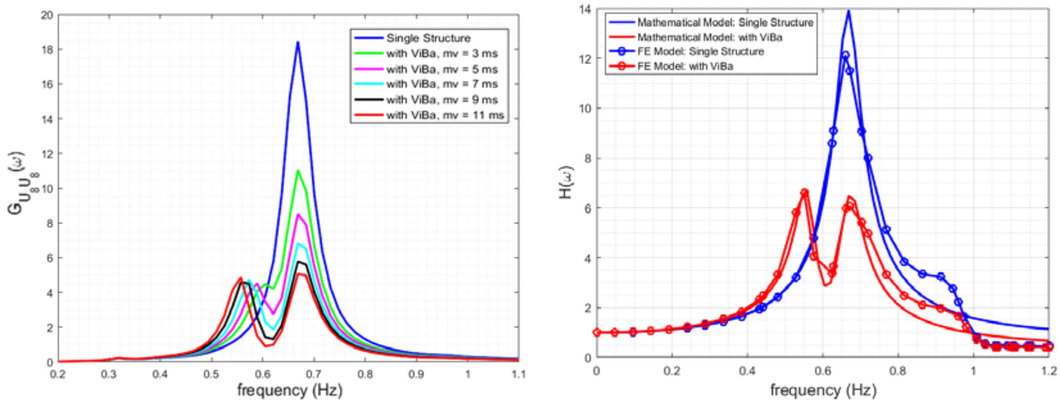

Figure 9: Comparison of the response in terms of PSD Function of 8th floor X-direction Displacement between Single Structure and Structure Coupled with ViBa with different ViBa Mass/Structure Mass ratio a) Comparison with the FE model b).

To better appreciate the influence of the ViBa on the dynamic response, the trajectory of response of the 8th floor displacement (with and without the ViBa) to a ground motion sample simulated from the response-spectrum-compatible process is presented in Figure 10. The reduction of about 30% the response is in line with the results presented in Table 4 (m_V=11ms).

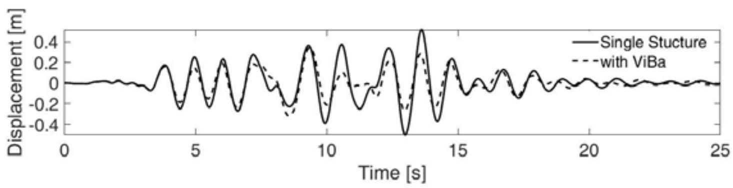

Figure 10: Trajectories of the displacement response at the 8th floor with and without the ViBa

Concluding Remarks

In this paper the design of the Vibrating Barrier for the Bell Tower of the Messina Cathedral has been performed. A FE model has been first determined using data from original drawing and technical reports. A novel discrete model able to reliably represent the coupled response of the Bell Tower and the ViBa has been proposed in this paper. Significant reductions of the response have been observed for various values of the mass of the ViBa device. Clearly, in absence of a calibration/validation of the FE model with experimental results from the full scale structure the results presented in this paper need to be taken as academic and cannot be used for designing the ViBa for the Bell Tower. Moreover due to the rocking mechanism manifested in the fundamental modes the mass of the ViBa appears to be quite larger than those used in all the previous case studies reported in literature (generally of the order of magnitude of the mass of the structure to be protected). In this regard ViBa can be enhanced by the adoption of inerters, as shown in Cacciola et al. 2018, in which it has been shown that it was possible to reduce the mass of the ViBa of about 70% keeping the same level of performance. Future works will focus on alternative configurations of the ViBa exhibiting both translation and rotational behaviour so to be optimized for structures exhibiting rocking behaviour.

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