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Effects of 2012 Earthquake on the behavior of Ghirlandina tower in Modena

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ABSTRACT: Collapse events, that occurred in the past (such as the Venice Bell Tower in 1902 and the Civic Tower in Pavia in 1989) claimed for the need to assess the long-term behavior of such monuments. A significant effort has been therefore devoted to clarify the reasons for these collapses after many centuries from the construction date. In addition, recent earthquakes in Italy have once again put into evidence the seismic vulnerability of the cultural heritage. To assess this aspect, in many cases the soil-structure interaction cannot be neglected. In this context, a simple but consistent framework for soil-structure interaction analysis is here presented with reference to a case history. It is discussed how the difference in the fundamental frequency observed during seismic events can be associated to non-linearity in soil response, leading to a rotational stiffness of the soil-foundation system consistent with the shear strain level derived from the seismic ground response analysis. Thereafter, the validated soil-structure interaction model has been used to define an equivalent SDOF model of the structure that explains the differential settlements suffered by the Ghirlandina tower in Modena during the 2012 seismic events as well as its behavior since those events.

1 HISTORICAL NOTES ON THE MODENA CATHEDRAL AND THE GHIRLANDINA TOWER

Established as a Roman colony in 183 BC over previous settlements, the city of Modena owes its current urban centre to the Middle Ages, after numerous and devastating floods that destroyed the ancient monuments and progressively confined the Roman remains under approximately 5 meters of sediments (Cadignani et al 2019; Lugli 2017).

The Piazza Grande is its geographical, religious, political, and economic heart. The medieval city grew from this central spot, which includes the City Hall, the Cathedral and the Ghirlandina tower.

At the time of the investiture controversy between papacy and empire to appoint local bishops, the Modena's bishop's seat was vacant after a time interval in which power had been firmly in the hands of bishop Eriberto. It was a period of growing civic initiative, which lead after a few years to the formation of the free municipality. In this context, the people together with the local clergy but independently from the ecclesiastical and imperial powers decided to build a new great church, as it was happening in other European cities. Therefore, the Cathedral of the architect Lanfranco replaced the previous church, which safeguarded the relics of Saint Geminianus Modena's patron saint, and which lied in a state of great disrepair.

Lanfranco conceived a mighty church with three naves and an annexed bell tower, expression of a new figurative model that influenced the development of the Romanesque in the Po Valley and whose decorative apparatus accompanied the rebirth of sculpture in Europe (Figure 1.a).

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Figure 1. (a) The façade of the Cathedral; (b) Aerial view of the Cathedral and the GhirlandinaTower.

The tower, later named “Ghirlandina” probably because its balconies resembled garlands, soon became the symbol of the city, standing out in the sky above all other buildings, clad in white (Figure 1.b).

Thanks to delays and interruptions that allowed the foundation soil to increase its strength over time, it is argued that the first five floors were successfully standing in 1169 or 1184; the tower reached the height corresponding to the sixth floor in 1216, and the completion date was the 18th of September 1319 by the hand of Enrico da Campione.

In 1338 the arches connecting the southern side of the tower to the Cathedral were already in place, eventually to prevent additional tilt of the tower towards the Cathedral, since there is evidence that the tower begun to tilt during the construction and the ancient masons made some corrections accordingly.

From a typological point of view, the tower recalls the Lombard bell tower model (Figure 2), with a square base of 11 meters per side, which lightens upwards through the progressive increase in the number of windows. Above the square tower shaft, there is a smaller octagonal drum and a high spire crowned by a golden copper sphere topped with a cross, for a total height of 89.32 meters.

The structure is made of brick masonry cladded in slabs of stones of different thickness, sourced from Roman remains up to the 5th floor, while on the upper floors the material (Ammonitico Veronese limestone) was purchased on purpose.

A useful datum to better understand the construction process came from the photogrammetric survey performed in 2004 and from the laser scanning survey in 2008, with which the geometry and the inclination were measured, and a three-dimensional model was produced. There is an almost homogeneous inclination of the walls towards the West, while towards the South there is a markedly curvilinear trend (Figure 2), typical of constructions that undergo leaning during the construction

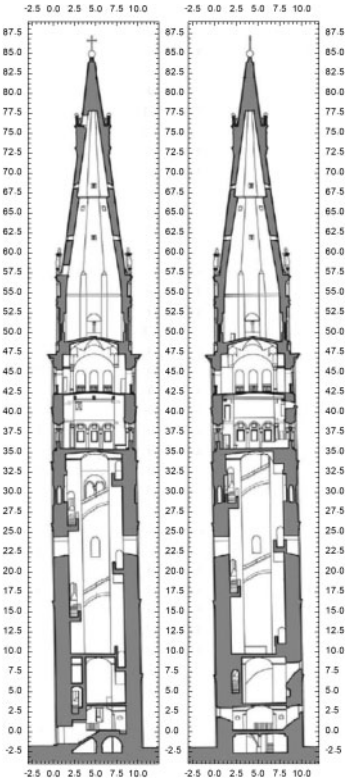

Figure 2. Vertical sections of Ghirlandina tower: from the left, view towards South, view towards East (modified – source: Giandebiaggi et al. 2009).

phase. An overall out of lead of 1.54 m was also measured in the southwest direction, apparent even by eye inspection.

2 THE CONCERNS ABOUT SEISMIC VULNERABILITY

The May 20th, 2012 (02:03:52 UTC) an earthquake (ML = 5.9, with epicenter 44.889N 11.228E close to Mirandola and focal depth 6.3 km), struck the towns of Finale Emilia, San Felice sul Panaro, and Sant'Agostino, involving a wide adjacent area. This event was followed by a series of significant aftershocks, such as the May 29th at 07:00:03 (ML = 5.8, 544.851N 11.086E, focal depth 10.2 km), in the surroundings of Novi di Modena, which caused even more severe and extensive damage on a larger area.

Even if this region was considered of low seismicity, and the population was not very accustomed to earthquakes, in the past the area between Ferrara and Novi di Modena had already been affected by severe seismic events. Some of these were destructive, such as the 1570 earthquake. Several others, though causing less damage, left their traces in the chronicles of the time.

In this respect, it is worth mentioning that on 20 and 29 May 2012 the medieval cathedral in Modena suffered significant damage, as described by Baraccani et al. (2017). The cross vaults, which are a 15th-century addition to the original 12th-century structure of the cathedral, were cut by a complex network of fractures and brick fragments and false ribs fell to the ground. A deep inspection of these fragments, and the absolute dating of lime mortars (using radiocarbon ^{14}C) and bricks (using thermoluminescence) integrated with the pollen record of mortars, recently

performed by Tirelli et al. (2020, 2021), allowed to clarify the construction and restoration history of the vaults and to link the repairs to the earthquake chronology for the area. The original vaults (1404–1454) were built using lime mortar binding with Roman and medieval older bricks. Lime mortar was used also for later repairs caused by earthquakes in the 16th and 17th centuries. Gypsum mortars were then used to rebuild some vaults and to repair others in the 18th and 19th centuries.

Therefore, this approach proved that unexpected damage could be revealed by a detailed chronology of masonry binders, a fundamental step to assess earthquake risk and properly select strengthening measures.

The earthquakes on the 20th and 29th of May 2012 also caused some damages to the tower. Surely the previous work of consolidation of the balconies, the two metal hoops on the balcony and the lower one on the belfry cornice, contributed to mitigate these effects.

On the outside, the most evident damage was a new fracture, parallel to an already restored one, on the sculpture placed in the southwest corner of the first cornice, depicting Samson in the act of smashing the lion's jaw. The damage was caused by the hammering action between the tower and one of the arches connecting it to the Cathedral, which once again points out the relevance of the interaction between the two monuments.

Furthermore, it is worth mentioning that since 1981 the Municipality of Modena promoted a comprehensive study of the induced subsidence, that resulted in a local dense network of geodetic leveling (Castagnetti et al. 2017) and in the execution of geotechnical investigations (Lancellotta 2009). The elevation data resulting from subsequent leveling campaigns allow to identify the trend of vertical displacements suffered over time by the benchmarks of the network, and then by the structures to which they are rigidly connected, and to investigate the related differential components and to evaluate their significance. Figure 3 displays the vertical displacements measured since 1984 in the historic center of Modena, the apses of the Cathedral and the Tower resulting to be characterized by the greater subsidence with respect to the one experienced by the whole city center (mean values are 48 mm for the Tower and 18 mm for the apse of the Cathedral over 32 years).

It is apparent from this leveling the sudden increase in the settlement the tower suffered in May 2012, not experienced by the Cathedral and other surrounding structures, and notably, the differential settlements towards South between benchmarks 18-17 and 16-C1 respectively of the order of 2.8 *mm* and 2.5 *mm*. These differential settlements may be expected to be the results of the seismic events of May 2012, but it remains to prove that this was actually the case, as discussed in the following sections.

3 IDENTIFICATION ANALYSIS AND SOIL-STRUCTURE INTERACTION

To consider the interaction of the structure with the supporting soil, a simple but effective approach considers the soil response as lumped into a series of springs and dashpots, each of them related to a degree of freedom of the soil-foundation system.

It is important to note that the properties of these elements depend not only on the properties of the half-space representing the soil but also on the exciting frequency.

By relying on this approach, if the soil and the foundation are represented by horizontal (K_h) and rocking (K_α) springs, the interaction with the soil will lead to a modified period of vibration T^*, given by:

$$T^* = T_0 \sqrt{1 + \frac{K}{K_h} + \frac{Kh^2}{K_\alpha}} \tag{1}$$

where: $T_0 = 1/f_0$ is the natural period of the structure resting on a fixed-base; K is the stiffness of the structure; h is the height of the lumped mass over the base.

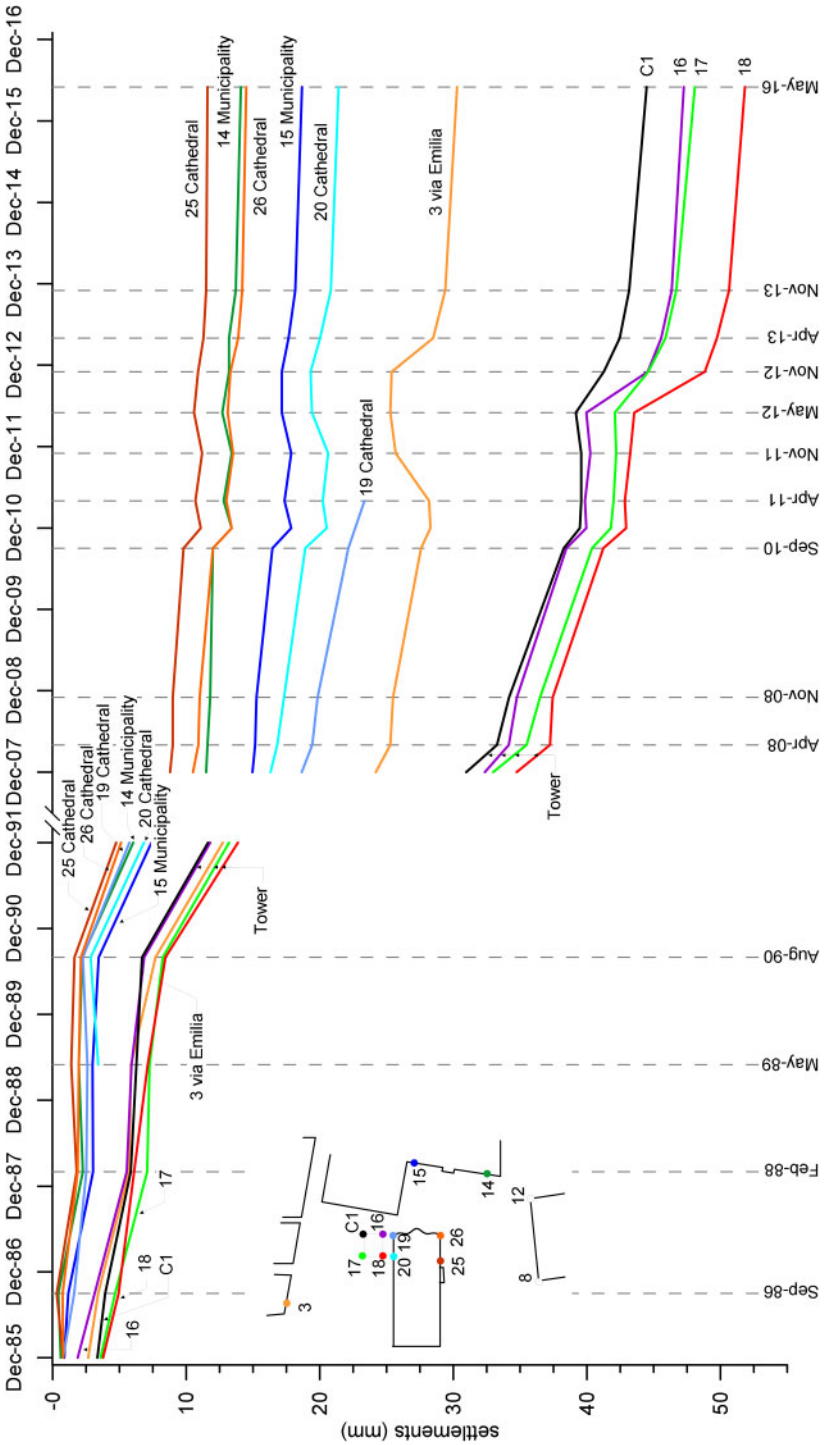

Figure 3. Trends of vertical displacement of the main benchmarks over the periods 1984–1991 and 1991–2016.

Similarly, the equivalent damping ratio (ξ^*) of the single degree of freedom model (SDOF) representing the structure is given by:

$$\xi^* = \xi_{soil} + \left(\frac{T_0}{T^*}\right)^3 \xi \tag{2}$$

Where ξ is the damping ratio of the structure, and ξ_{soil} is the contribution of the soil damping that includes both the radiation and the material damping.

To study the role of soil-structure interaction in the long-term performance and to assess the seismic vulnerability of the tower, a dynamic monitoring program was started in August 2012. The acquisition system operates with a sampling frequency of 100 Hz and allows continuous monitoring of the dynamic response of the tower under ambient vibrations.

To capture the higher modes of vibration, 12 accelerometers were installed on the tower to record the time histories along three directions (normal to the cathedral nave, vertical, and parallel to the cathedral nave). To select the optimal position of these sensors, a dynamic analysis was performed in advance using a numerical model (FEM) of the tower. Analysis of the results allowed estimation of the natural frequency, mode shapes, and damping parameters of the identified bending mode shapes and extensional mode shapes, i.e., vibrations along the tower axis. In particular, the first bending mode shape, with an associated frequency of 0.74 Hz, shows that the rotation and displacement pattern at the tower base was mainly due to soil deformability. On the contrary, the second bending mode shape, associated with a frequency of 0.85 Hz, reflects the presence of the arches connecting the tower and cathedral, and higher modes are influenced by the masonry deformability (Lancellotta & Sabia 2013).

Based on these results, Lancellotta and Sabia (2013, 2015) estimated a rocking stiffness equal to $K_\alpha = 240\ GN{\cdot}m$ (neglecting in Equation (1) the contribution of the horizontal stiffness $K_h = 2.52\ GN/m$).

This value was proven to be consistent with the one provided by theoretical analysis (Gazetas 1991) with the following assumptions:

(a) the small-strain shear modulus G_o (equal to 28 MPa) in free field condition is obtained from the shear wave velocity ($V_S = 125$ m/s);
(b) the stress level induced by the tower is considered, giving a multiplier 1.57, therefore the operational shear modulus rises to G_o equal to 44 MPa;
(c) the effect of founding depth (equal to 5.65 m), compared to the square base of 12.40 m per side of the foundation, is considered, giving rise to a multiplier factor of 3.19.

An independent approach to highlight the influence of the soil-structure interaction is based on the decomposition of the actual system into two simpler ones. Equation (1), by neglecting the contribution of the soil horizontal stiffness, gives:

$$T^* = T_0\sqrt{1 + \frac{Kh^2}{K_\alpha}} \tag{3}$$

that can be also written as:

$$\frac{1}{(f^*)^2} = \frac{1}{f_0^2} + \frac{1}{f_R^2} \tag{4}$$

where: f^* = measured frequency; f_0 = fundamental frequency of the deformable tower resting on a fixed-base (equal to about 1 Hz); and $f_R = \omega_R / 2\,\pi$ = frequency of the tower, considered as a rigid body resting on the soil represented by a rotational stiffness K_α.

Since f^* and f_0 are known quantities, as well as the mass moment of inertia ($I = 5.4 \cdot 10^9 kg \cdot m^2$, considering the modal mass) of the tower with respect to the horizontal axis at the foundation level, the relation:

$$\omega_R = \sqrt{\frac{K_\alpha}{I}} \tag{5}$$

gives a value of $K_\alpha = 260\ GN{\cdot}m$, consistent with the one obtained through the more sophisticated identification analysis.

It is apparent from these results that a key factor in the evaluation of the dynamic stiffness representing the soil-foundation response is the selection of an appropriate value of the shear modulus.

In this respect, Cosentini et al (2015) proved that a value of G consistent with the shear strain level can be computed from a seismic site response analysis. Specifically, they analyzed the records of three seismic events: October the 3rd, 2012 (epicenter in Piacenza and magnitude ML = 4.5), January the 25th, 2013 (epicenter in Garfagnana, ML = 4.8) and June 21st, 2013 (epicenter in Alpi Apuane, near Lucca, ML = 5.2). In the following, a summary is reported.

Details of site investigation are reported in Lancellotta (2009, 2013). The soil profile (Figure 4) down to the investigated depth of 80 m can be described as composed of a first horizon of medium to high plasticity inorganic clays, with an abundance of laminae of sands and peat, only millimeters thick. The upper portion of this horizon, whose thickness ranges from 5 to 7 meters, is known as Modena Unit and it is linked to the flooding events (of post-Roman age) produced by minor streams (Fossa-Cerca stream). The underlaying horizons, ranging in depth from 22 to 54 m, represent the result of a complete transgressive-regressive cycle: the fine-grained sediments, belonging to the horizon known as Niviano Unit, were deposited during the penultimate interglacial cycle; the superimposed coarse-grained materials, belonging to the Vignola Unit, are linked to transport activities of the Secchia River. A second horizon of coarse-grained materials is encountered at depths ranging from 54 to 63 m, and thereafter a fine-grained horizon is found down to a depth of 78 m, here again, characterized by a diffuse presence of laminae of sand.

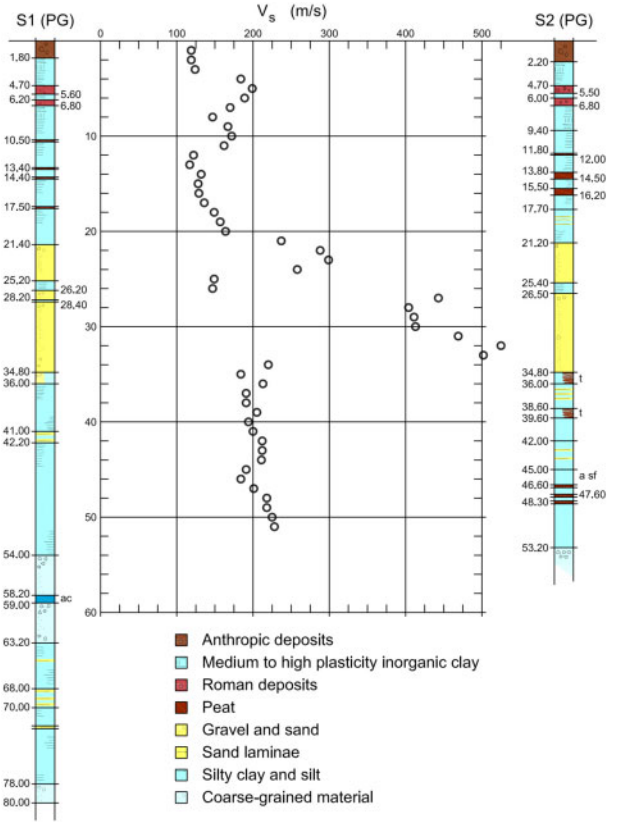

Figure 4. Soil profile and shear wave velocity from cross-hole tests.

Figure 5 shows the change of the normalized shear modulus reduction and the increment of damping ratio with shear strains, as measured on undisturbed samples with resonant column tests for the clayey layers.

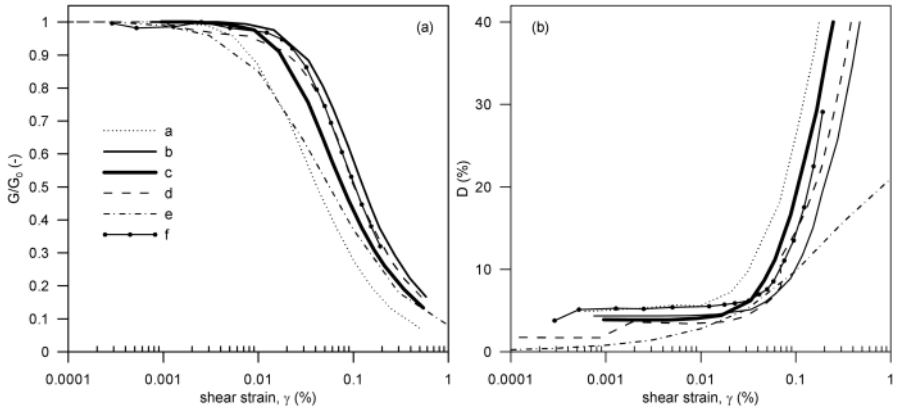

Figure 5. Dependence of shear modulus (a) and damping ratio (b) on strain level.

The dynamic characteristics of the structure can be estimated in terms of the transfer function from the excitation at the base to the response measured on the structure at a given level.

The *Frequency Response Function* (FRF) is therefore defined as follows:

$$H(\omega) = \frac{Y(\omega)}{X(\omega)} \quad or \quad H(\omega) = \frac{S_{XY}(\omega)}{S_{YY}(\omega)} \tag{6}$$

where $Y(\omega)$ and $X(\omega)$ are the Fourier transforms of the output and the input, respectively; $S_{XY}(\omega)$ is the cross-spectra of the input $x(t)$ and the output $y(t)$; $S_{YY}(\omega)$ is the auto-spectra of the output $y(t)$.

The FRF is a complex-valued function which amplitude has a maximum at the resonance frequency, and the output is 90° out of phase with respect to the input.

Figure 6 compares the FRF, evaluated using the acceleration time history at the base as input, and the time history measured at 78 m as output, for the three seismic events (October 2012, January 2013, and June 2013). It is apparent a reduction of the first natural frequency of the tower (from 0.73 to 0.69 Hz) moving from the first two events to the one of June 2013.

This difference is certainly associated with soil non-linearity since significant structural non-linearity is not expected for the masonry walls for such a small seismic excitation.

Therefore, the observed difference in natural frequency can be converted into an estimate of the reduction of the foundation stiffness by considering that the ratio between the mobilized soil stiffness during two different seismic events can be derived by using the inverse of equation (3)

$$\frac{K_{\alpha 2}}{K_{\alpha 1}} = \frac{(T_1/T_0)^2 - 1}{(T_2/T_0)^2 - 1} \tag{7}$$

From the difference in fundamental frequency observed in Figure 6, a stiffness ratio equal to 0.78 was obtained, consistently with the shear strain level derived from a seismic ground response analysis (for more details the reader is referred to Cosentini et al. 2015).

The effect of soil non-linearity on seismic response of the Tower can also be highlighted through an analysis of the Power Spectral Density (PSD) of some acceleration time-histories recorded at the height of 78 m. Four acceleration time histories (Figure 7) in direction parallel to the cathedral nave were analyzed: a signal recorded 50 minutes before the June 2013 seismic event; the signal during the seismic event; one recorded just 60 minutes after the earthquake; and a signal recorded

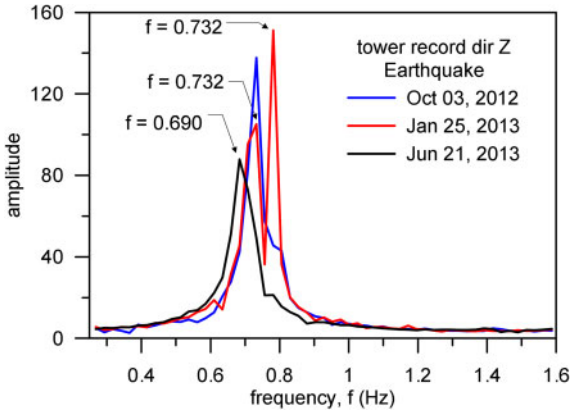

Figure 6. Frequency Response Function FRF for three seismic events.

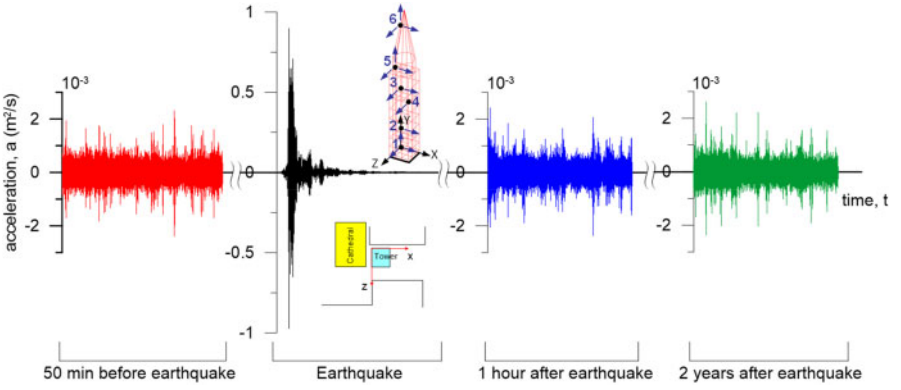

Figure 7. Acceleration time histories recorded at the height of 78 m (sensor 6) before the event of June 2013, during the earthquake, 1 hour and 2 years after the event.

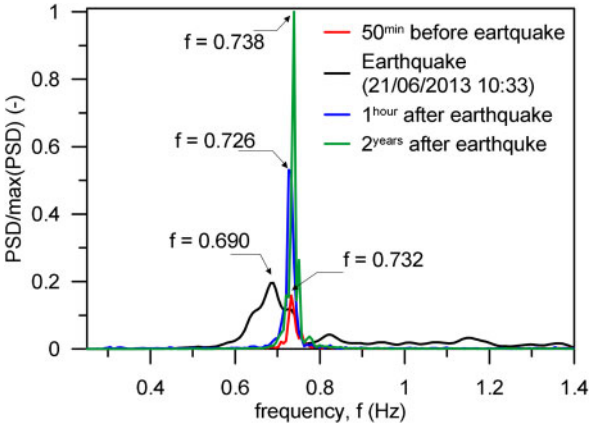

Figure 8. Comparison of normalized PSD of the time histories showed in Figure 7.

2 year later. The Power Spectral Density was computed for each signal. As it is shown in Figure 8, in terms of a normalized PSD, the reduction of the first natural frequency of the tower (from 0.74 to 0.69 Hz) observed during the earthquake disappeared already one hour after the event, results that prove that the behavior of the tower after the seismic event is the same as it was before that event.

4 DIFFERENTIAL SETTLEMENTS DURING SEISMIC EVENT OF MAY 2012

Consider now the tower to be represented by a single degree of freedom model (SDOF) consisting of a mass (M) lumped at a height (h) over the base. This model can be thought to represent the behavior of the tower in its first fundamental mode of vibration and, accordingly, if x_i is the modal (or even the assumed) horizontal displacement at the height y_i with a lumped mass m_i, then the modal mass associated to the first fundamental mode is given by:

$$M = \frac{\left(\sum m_i \cdot x_i\right)^2}{\sum m_i \cdot x_i^2} \tag{8}$$

and the height of the centroid of the inertial forces is given by:

$$h = \frac{\sum m_i \cdot x_i \cdot y_i}{\sum m_i \cdot x_i} \tag{9}$$

In the present case, the $M = 4868 \cdot 10^3$ *kg* (i.e., 64% of the total mass) and $h = 38.39$ *m*.

The stiffness of the structure resting on a rigid base can be estimated as $K = (4\pi^2 M)/T_0^2 = 192$ *MN/m*, and by considering that the shear force at the base of the tower can be obtained either in terms of displacement (u) of the structure or in terms of the displacement (u^*) of the equivalent SDOF model, the following relation applies:

$$u = \left(\frac{f^*}{f_0}\right)^2 \cdot u^* \tag{10}$$

If during the motion the structure rotates of the quantity $\alpha(t)$, by considering the dynamic equilibrium of horizontal forces and moment of the center of the foundation, the following relation is obtained:

$$\alpha \cdot h = \frac{Kh^2}{K_\alpha} \cdot u \tag{11}$$

Therefore, if $b = 11$ *m* is the dimension of the tower at the level of the benchmarks, the differential settlements suffered by the tower can be predicted from the spectral displacement by properly combining Equations 10 and 11, i.e.

$$\Delta w = \frac{Kh^2}{K_\alpha} \cdot \frac{b}{h} \cdot \left(\frac{f^*}{f_0}\right)^2 \cdot u^* \tag{12}$$

Figures 9 and 10 are the spectral acceleration and the spectral displacement related to the acceleration records of seismic events of May 20 and 29, 2012, together whit those of June 21, 2013.

The acceleration time histories were recorded in the free field by the Modena station (MDN) of the Italian accelerometric network (D'Amico et al. 2020), located to relatively small distance (about 2.8 km) from the Ghirlandina Tower.

In particular, a maximum spectral displacement (u^*) of 1.8 *cm* can be associated with the measured period of $T^* = 1.45$ *s*, and the corresponding differential settlement (Δw) is about 2.6 *mm*, in agreement with the measured values (see Figure 3).

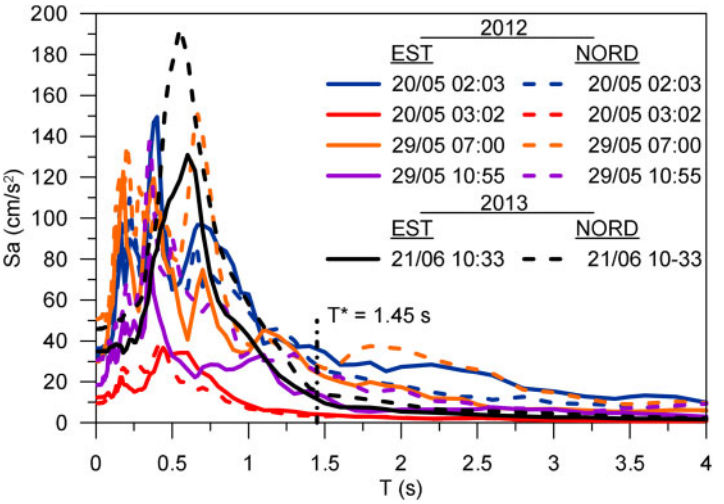

Figure 9. Spectral acceleration of May 2012 and June 2013 seismic events.

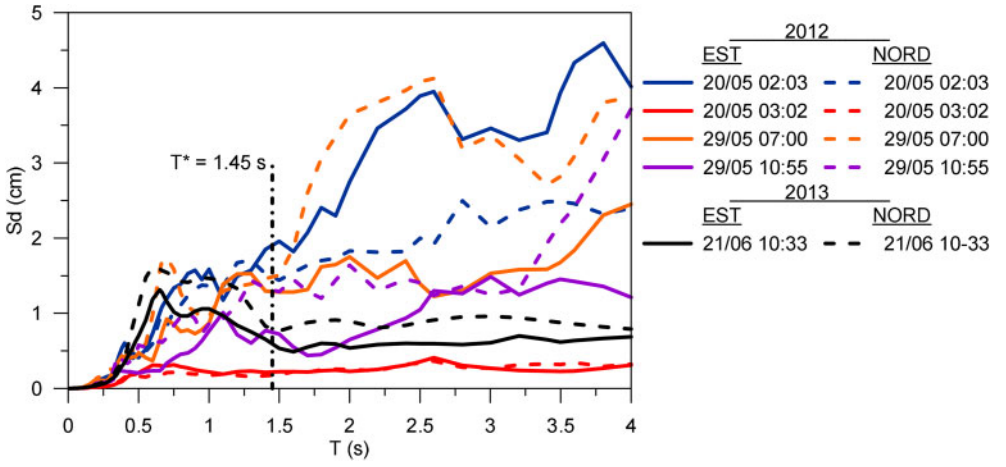

Figure 10. Displacement spectra of May 2012 and June 2013 seismic events.

However, a deeper insight into the displacement spectra would also suggest that since the event of May 2012 was a sequence of many shocks, the differential settlement suffered by the tower could be the result of an accumulation of the irreversible portion of the total displacement suffered by the tower during each event.

5 FINAL REMARKS

Recent earthquakes event in Italy (May 2012) have once again put into evidence the seismic vulnerability of the cultural heritage and there were many cases showing the role played by soil-structure interaction.

Taking into account this interaction is not a straightforward task, even when using a simplified lumped parameters model, because the selection of soil modulus requires to consider many aspects:

soil nonlinearity; dependence on stress level induced by the structure; effectiveness of the contact between the foundation and the surrounding soil; dependence on the exciting frequency.

In this respect, identification analyses are certainly powerful tools to characterize the behavior of the structure interacting with the soil and to properly define the properties of simplified lumped models, that can be used to forecast the response of the structure, as it was the case with the Ghirlandina tower.

In addition, long-term continuous dynamic monitoring allows to identify the causes of movements and their potential increase in rate, distinguishing the normal physiological behavior from deviations.

In many cases, monitoring can help in avoiding unnecessary intervention measures, preserving the historic integrity of the structure.

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