Experimental evidence and prediction of soil-structure interaction effects for a masonry tower

EXPERIMENTAL EVIDENCE AND PREDICTION OF SOIL-STRUCTURE INTERACTION EFFECTS FOR A MASONRY TOWER

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Abstract: Remedial actions on historical buildings have to be planned carefully in order to preserve not only the shape and appearance of the monument but also its historical and material integrity. This requires a deep knowledge of the behaviour of the structure, based on long term observations, and quite often the interaction with soil plays a relevant role. In particular, this interaction cannot be neglected when dealing when assessing both seismic vulnerability and long-term behaviour. Although advanced numerical tools are increasingly available, simple approaches still play a fundamental role for the understanding of the actual behaviour of complex structural systems in the engineering practice. In this work a rational framework for soil-structure interaction analysis is proposed and validated by using experimental data from a permanent monitoring system installed on a masonry tower in Italy.

Introduction

It is today well established that the preservation of historical towers requires a multidisciplinary approach to assure that devised remedial measures and intervention techniques preserve the integrity of these monuments. Because the integrity requirement is not only the need to preserve the shape and the appearance of the structure but also its historical and material integrity, remedial actions have to be devised only once a deep knowledge of the behaviour of the monument has been assessed (Brandi, 1977; Calabresi and D'Agostino, 1977; Calabresi, 2013). The example of the Pisa Tower is in this respect an exemplary one.

Collapse events occurred in the past (the Venice Bell Tower and the Civic Tower in Pavia) and recent earthquakes event in Italy (May, 2012) have once again put into evidence the need to assess the long-term behaviour of the monument as well as its seismic vulnerability.

To reach this goal, an accurate modelling of soil-structure interaction represents a significant aspect that cannot be neglected.

Nowadays it is apparently possible to analyse any complex interaction problem using advanced numerical tools, which are increasing available. However, the use of these numerical tools is not easy (they are computationally intensive and require a high level of experience) and they often lead to large variability in the results, especially if different codes are used. Therefore, simple approaches (e.g. Gazetas, 1991; Kausel, 2010) still play a fundamental role for the understanding of the actual behaviour of complex structural systems in engineering practice.

A simple but consistent framework for soil-structure interaction is here proposed based on the case history of the Ghirlandina tower.

The Ghirlandina tower and the Cathedral of Modena (Figure 1) are part of the Unesco site of Piazza Grande.The tower is a square based (side: 11,0 m) structure, 88.82 m high, with a hollow cross section. In the inner part, an open stair runs along the tower from the base to the higher part where the belfry and the spire roof complete the structure. The Tower was started at the same time the Cathedral was by Lanfranco (i.e. 1099) and reached his final stage in 1319. It is argued that the first five floors were successful standing in 1169 or 1184 and the tower reached the height corresponding to the sixth floor in 1216. In 1338 the arches

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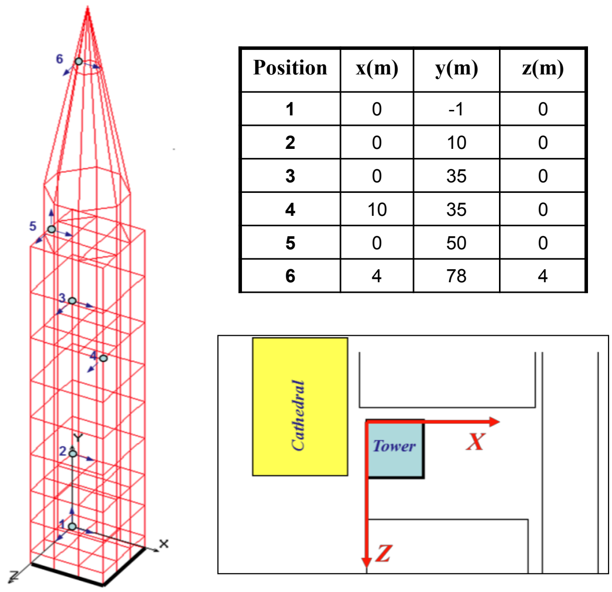
connecting the southern side of the tower to the Cathedral were already in place, eventually to prevent additional tilt of the tower towards the Cathedral, since there is evidence that the tower begun to tilt during the construction and the ancient masons made some corrections accordingly.
The site has been thoroughly characterized from the geotechnical point of view to investigate the stability of the tower and to assess the need of preservation works (Lancellotta, 2013). In the aftermaths of Emilia Earthquake in 2012, which caused several collapses and damages of historical masonry building in the nearby region (Tertulliani et al., 2012), there was a lot of concerns about the seismic behaviour of the tower and this stimulated further investigations on the behaviour of the tower. In particular, dynamic identification of the structure was performed and a permanent dynamic monitoring system was installed (Lancellotta and Sabia, 2013, 2014), so that the available experimental data are here used to validate the proposed approach.

Figure 1. The Ghirlandina tower [photo courtesy of B. Marchetti]

Dynamic response of the tower
The dynamic monitoring program of the Ghirlandina tower started in August 2012, when 12 accelerometers were installed at six locations along the vertical wall of the tower, as shown in Figure 2. The acquisition system operated with a sampling frequency of 100 Hz and allows continuous monitoring of the dynamic response of the structure under ambient vibrations. In particular, time histories of acceleration were recorded during three moderate seismic events, which occurred in: October, 3 2012 (epicentre in Piacenza and magnitude M=4.5), January, 25 2013 (epicentre in Garfagnana, M= 4.8) and June, 21 2013 (epicentre in Alpi Apuane – near Lucca, M = 5.2).



Position	x(m)	y(m)	z(m)
1	0	-1	0
2	0	10	0
3	0	35	0
4	10	35	0
5	0	50	0
6	4	78	4

Figure 2. Layout of measurement points (arrows indicate the coordinate directions along which the acceleration were recorded)

In order to assess the influence of these events on the response of the tower, the frequency response function (FRF) was evaluated from accelerometer records at different levels. As it is shown in Figure 3, a reduction of the first natural frequency of the tower (from 0.74 to 0.69 *Hz*) from the first two events to the one of June 2013 is rather evident. This difference is certainly associated to soil non-linearity, since significant structural non-linearity is not expected for the masonry walls for such a small seismic excitation. Therefore, these differences can be used to estimate the reduction of the soil-foundation stiffness with increasing seismic action, as it will be shown later on.

Note that the expected influence of soil-structure interaction was also previously shown by modal identification analyses performed under ambient excitations (Lancellotta and Sabia, 2014). The number of accelerometers used and their location were carefully evaluated in advance in order to properly capture fundamental and higher modes. They allowed to identify bending and extensional modes (i.e. vibrations along the tower axis – modes 8 and 9), as it is shown in Figure 4 (Lancellotta and Sabia, 2013 and 2014; Sabia et al., 2015).

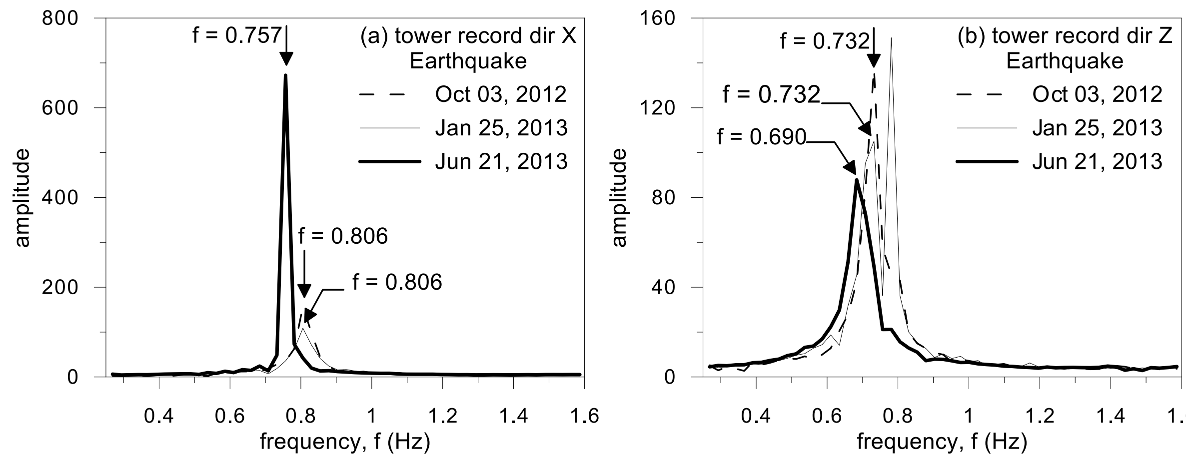


Figure 3. Estimation of cumulative Frequency Response Function of the tower obtained as the ratio between records at the different elevations and the basement record: (a) x-direction and (b) z-direction [Cosentini et al., 2015]

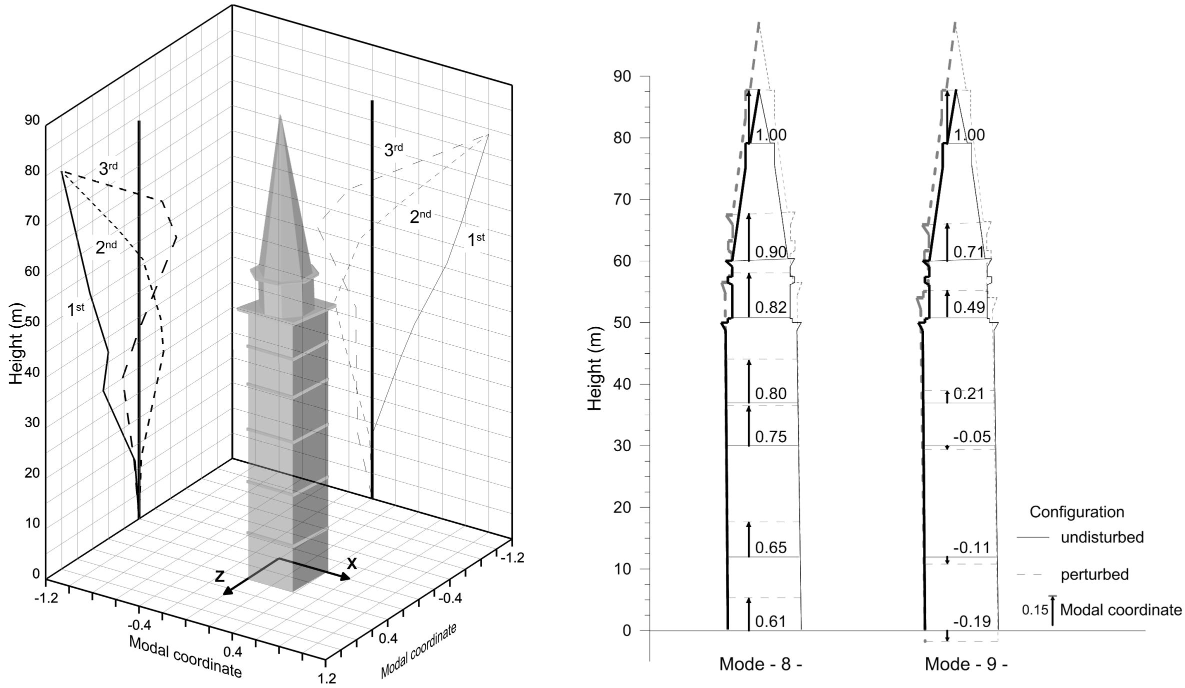


Figure 4. Bending mode shapes (on the right) and axial mode shapes (on the left) [Sabia et al., 2015]

It is apparent how mode shapes show a rotation and displacement pattern at the tower base due to soil deformability.

In particular, referring to the bending modal shapes, with associated frequencies of 0.74 Hz and 0.85 Hz, it can be observed that the first one is associated to the rotation at the tower basis due to soil deformability, whereas the second one reflects for the presence of the arches connecting the Tower and the Cathedral.

Furthermore, the first axial mode (mode 8, with related frequency of 4.51 Hz), highlights that the base moves in phase with the top of the tower, and about 60% of the associated vertical modal displacement is determined by soil deformability. The second axial mode (mode 9 with related frequency 9.81 Hz) shows displacements of the base in opposite phase with respect to the top of the tower.

In conclusion, there is wide evidence that soil–structure interaction cannot be neglected, in contrast to most published structural identification analyses.

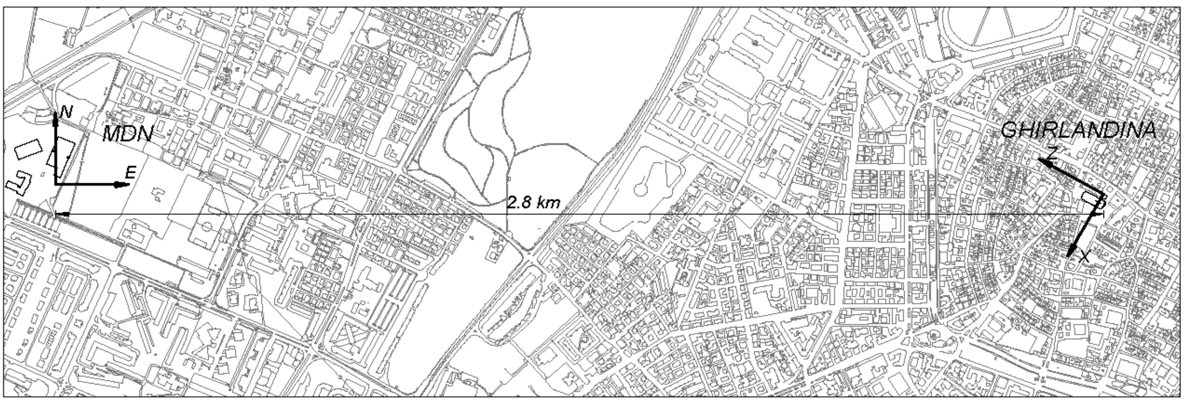


Figure 5. Position of the Modena station of the Italian accelerometer network with respect to the Ghirlandina site [Cosentini et al., 2015]

Independent prediction of ground motion

Specific analyses were performed to study the influence of soil non-linearity on seismic site response. The previously mentioned seismic events were recorded in the free-field by one station of the Italian accelerometric network, which is located not far from the site of the tower (Figure 5). Considering the epicentral distance these records can be used as input for studying ground response at the site of the tower.

The recorded ground motions were deconvolved to account for the local stratigraphic conditions at MDN station (Foti et al., 2011) and used in equivalent linear elastic analyses to estimate the ground response at the site of the Ghirlandina Tower (Cosentini et al., 2015). The results in terms of maximum shear strain profile and maximum acceleration profile are reported in Figures 6. The obtained results suggest that the third earthquake caused significant shear strains in the zone just beneath the tower foundation, whereas very small strains are associated to the two previous earthquakes.

In particular, the largest deformations are attained in the zone below the foundation, where a weak soil horizon is encountered (for details about soil profile of the Ghirlandina site see Lancellotta, 2009) This layer is likely to have a major impact on the rocking response of the tower. Finally, to assess the influence of the kinematic interaction, by using data from June earthquake, the seismic motion at the interface between the first and second layer (at 3 m depth from the ground surface) was considered (Figures 7) and for the sake of comparison, the records from the permanent monitoring systems were converted in N–S and E–W components according to the relative orientation of the sensors (see Fig. 5).

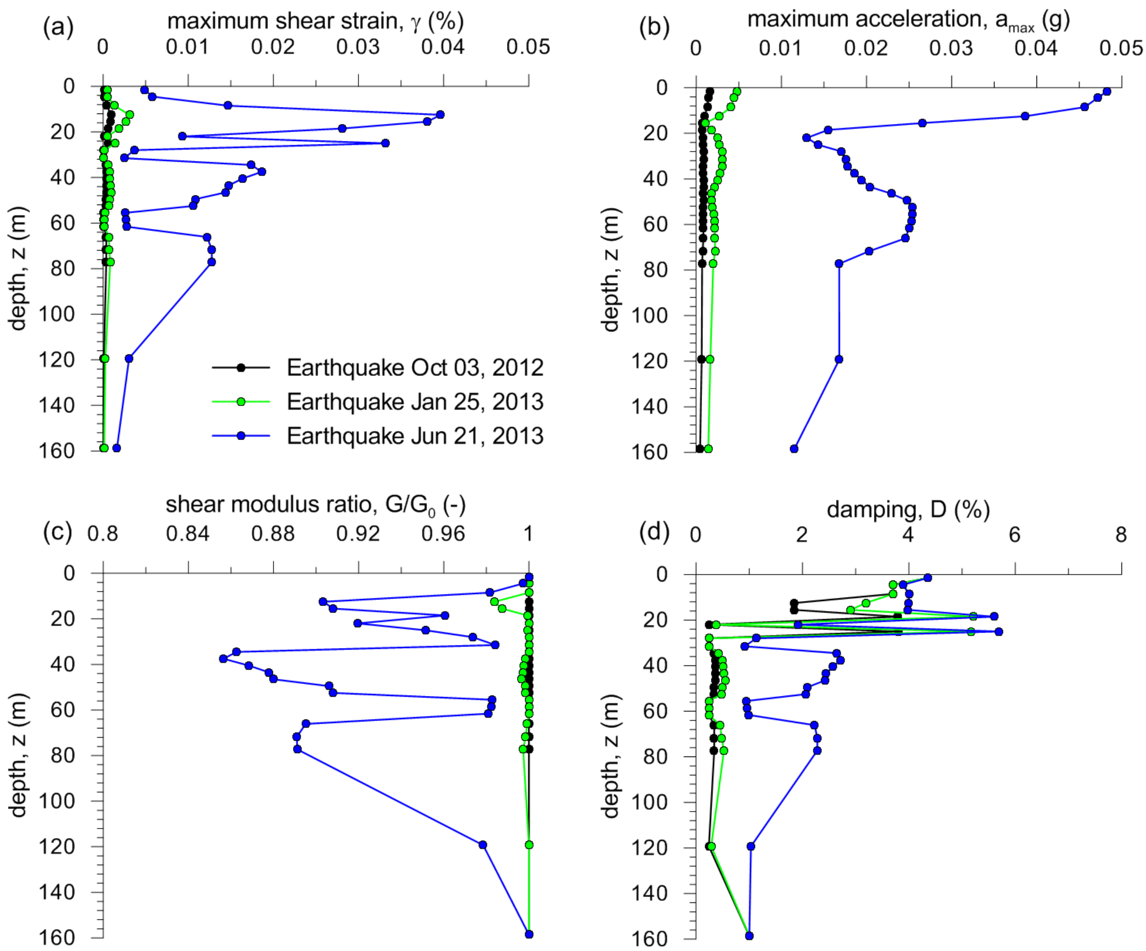


Figure 6. Seismic ground response of the Ghirlandina site for the seismic events recorded by MDN accelerometric station (only the results for the N-S component are shown): (a) profile of maximum shear strains; (b) profile of maximum acceleration; (c) profile of equivalent shear modulus ratio and (d) profile of equivalent damping ratio

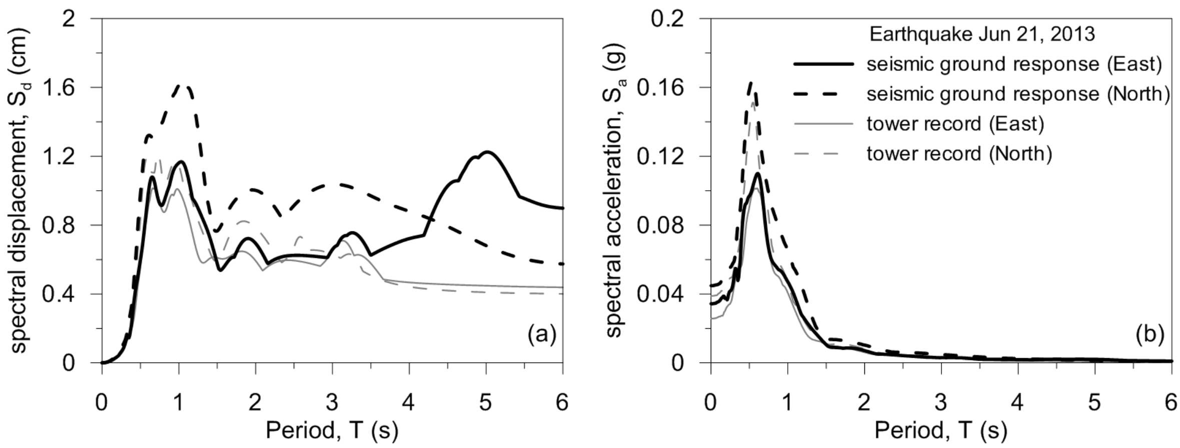


Figure 7. Comparison of response spectra from the ground response spectra and response spectra of the recordings at the basement level of the tower for the earthquake of January 2013: (a) displacements and (b) accelerations [Cosentini et al., 2015]

The comparison in Figures 7 highlights that the results of the site response analysis provide a reasonable estimate of the foundation input motion to be used for the analysis of the structure.

Soil-structural interaction

The above analyses show the need to consider not only the soil-structural interaction but also the non-linearity of soil response. Using a simple classic approach of vibrations of foundations (Gazetas, 1991) it is possible to evaluate the rocking stiffness from the following equation:

$$K_\alpha = \frac{3.6Gb^3}{1-\upsilon} \cdot f_D \tag{1}$$

where υ is the Poisson ratio, G is the shear modulus and f_D is the correction for embedment of the foundation given by:

$$f_D = \left\{1 + 1.26\frac{d}{b}\left[1 + \frac{d}{b}\left(\frac{D}{d}\right)^{0.2}\sqrt{\frac{b}{l}}\right]\right\} \tag{2}$$

in which 2b and 2l are the dimensions of the foundation, D is the foundation depth, d is the fraction of D that contributes to the constraint.

The choice of the shear modulus in Equation 1 is a key factor in the evaluation of the dynamic stiffness, and by considering the highly non-linear soil behaviour, the operational shear modulus has to be consistent with the expected shear strain.

At very low strain level, it is possible to assume the initial value of shear modulus, for example the value obtained by wave propagation in the cross-hole test.

Cross-hole test were performed during the geotechnical investigation of Ghirlandina site in 2007 (Lancellotta, 2009). A representative value of shear wave velocity for the foundation soils is V_S = 125 m/s, from which the small-strain shear modulus is estimated as G_0 = 28 MPa. This value refers to free-field conditions so that, if correction is made to account for the stress level induced by the tower, by considering a representative soil element at a depth of 3b/2 as suggested by the stress path method (Lambe and Whitman, 1969) and assuming a power law relationship (Viggiani and Atkinson, 1995), the operative value becomes G_0 = 44 MPa.

The foundation basement of the tower is a square-based structure of side 12.40 m, with founding depth D = 3.50 m. Therefore, assuming d = D, the rocking stiffness obtained by equation (1) is K_α = 240 GN m.
This value is consistent with that provided by the structural identification analysis based on ambient vibrations (Lancellotta and Sabia, 2013 and 2014).
In both the identification analysis and the shear wave propagation in the cross-hole test the strain level is very low, so the obtained value of dynamic foundation stiffness is deemed to be appropriate only for a low intensity seismic motion. In strong motion case, it is necessary to take in account the consequences of soil non-linear response, assuming a value of shear modulus consistent with the expected shear strain level.
An estimate of the operational shear modulus can be obtained directly from the seismic ground response analysis. Considering the ground response analysis for Ghirlandina site previously shown (Figure 6), consistently with the shear strain level induced by the seismic motion, an operational value of G/G_0 of about 0.85 is obtained within the zone of influence beneath the foundation for the event of June 21, 2013 (Figure 6c).
To validate the suggested procedure, an independent estimate of the reduction of foundation stiffness with increasing seismic action has been made on the basis of data collected through the permanent monitoring system of the tower.
As shown in Figure 3, a change in natural frequency of the structure was observed in correspondence of the earthquake more intense (June 2013). The difference in natural frequency are associated to soil non-linearity and they can be converted into an estimate of the reduction of the foundation stiffness.
It is assumed the tower to be represented by an equivalent single degree of freedom model, with a mass lumped at an height h over the base of the foundation and a structural stiffness equal to K_S. If T_0 is the fundamental period of the structure on a rigid base, it can be proven that the period of the soil-structure system increases when the flexibility of the soil is taken into account and it is given by

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

where K_α is the rocking stiffness of the soil-foundation system.
Provided that the value of T_0 can be obtained by a numerical model calibrated on the structural identification process (in the present case T_0 = 1.01 s), the ratio between the mobilized soil stiffnesses during two different seismic events can be obtained by using the inverse formula

$$\frac{K_{\alpha 2}}{K_{\alpha 1}} = \frac{\left(\frac{T_1}{T_0}\right)^2 - 1}{\left(\frac{T_2}{T_0}\right)^2 - 1} \tag{3}$$

From the difference in fundamental frequency observed in Figure 3(a) (the X-direction is not considered because it is influenced by the arches connecting the tower to the cathedral), a stiffness ratio equal to 0.8 is obtained, consistently with the shear strain level derived from the seismic ground response analysis (Figures 6c).
The proposed procedure can also be used to estimate an operational value of the damping ratio (Figure 6d) to be used for the calibration of a dashpot macro-element (e.g. according to the formulation proposed by Gazetas, 1991).

Conclusion

A simple but consistent method is proposed to include soil-structure interaction in routine seismic analyses accounting for the non-linearity of soil response. The stiffness is evaluated using classical approaches of soil dynamics, with the equivalent elastic modulus assumed from the seismic ground response analysis. The approach is validated using experimental data recorded by a permanent monitoring system on a masonry tower. Different seismic events have indeed induced a different behaviour of the structure that can be associated to non-linearity in soil response, as confirmed by a seismic ground response analysis based on independent data.

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