LA RISPOSTA DIVERSA DI DUE STRUTTURE APPARENTEMENTE IDENTICHE: UNA LEZIONE DAL TERREMOTO DI MIRANDOLA DEL 20th MAGGIO 2012

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Abstract

Twin structures, that is structures very similar in terms of geometry, materials, mass distribution etc., founded on the same soil and set at very close distance, are rationally expected to have an identical response to earthquakes. When this does not occur, a role is usually played by factors like the interaction with the surrounding structures or by other anomalies hidden behind the apparent similarity. We present the case of two apparently twin towers that showed a very different response to the 2012 Mirandola (Italy) earthquake ground shaking: one remained perfectly intact while the other had a wide set of fractures on secondary walls. This resulted to be the effect of several contributing factors: the stiffness of the two structures, experimentally measured, provided unexpected differences. This reflected into different modal frequencies for the two towers, with the first and second modes of the damaged tower coincident or very close to the soil resonance. The final result was a coupled soil-structure resonance, implying a much higher displacement of one tower compared to the other, under the same input motion. In Italy, insurance against earthquake damage will probably become compulsory in the near future. This case suggests that the specific soil-structure and structure-structure interaction will have to be carefully evaluated since they can critically affect even apparently identical structures.

Introduction

North East off the Old City of Bologna (Northern Italy) there exist two modern structures (tower A and tower B) - identical to the sight - characterized by the same geometry and construction style, set at about 120 m distance one from the other. The two towers are surrounded by much lower rising structures and the whole set of structures constitute a residential aggregate (Figure 1). The height of both towers is 56 m and their basement area is approximately 20 by 40 m. The horizontal section of the towers is a portion of annulus with a common centre. The two structures are located at different distances from the centre, which results in a slightly different curvature, hardly perceptible to the sight (Figure 2). However, the different curvature is mostly an architectonical element since the façade, built with lightweight steel elements, is separated from the main bearing structure. Two reinforced concrete (RC) shear walls and two RC stair cores, together with an irregular layout of steel columns, form the main bearing structure. The geometry and position of the shear walls and stair cores is very similar in the two structures, while the steel columns are located in slightly different positions (Figure 3). The structures were designed at the end of the '90s and built in the early 2000, according to building codes that did not take into account horizontal loads induced by seismic motion but only horizontal loads induced by wind.

The two structures responded in a very different way to the Mirandola (Modena, 44.89°N, 11.23°E), May 20th 2012 $M_L = 5.9$ earthquake, the epicenter of which was located at a distance of about 45 km North. Specifically, tower A did not suffer from any damage while tower B presented extensive damage in the internal walls (only aesthetic damages which are not expected to modify the dynamic characteristics of the tower but which still require the walls to be filled-up, plastered and repainted, Figure 4).

We attempt to provide an explanation for such a different behavior through a dynamic analysis of the structures and of the foundation soil.

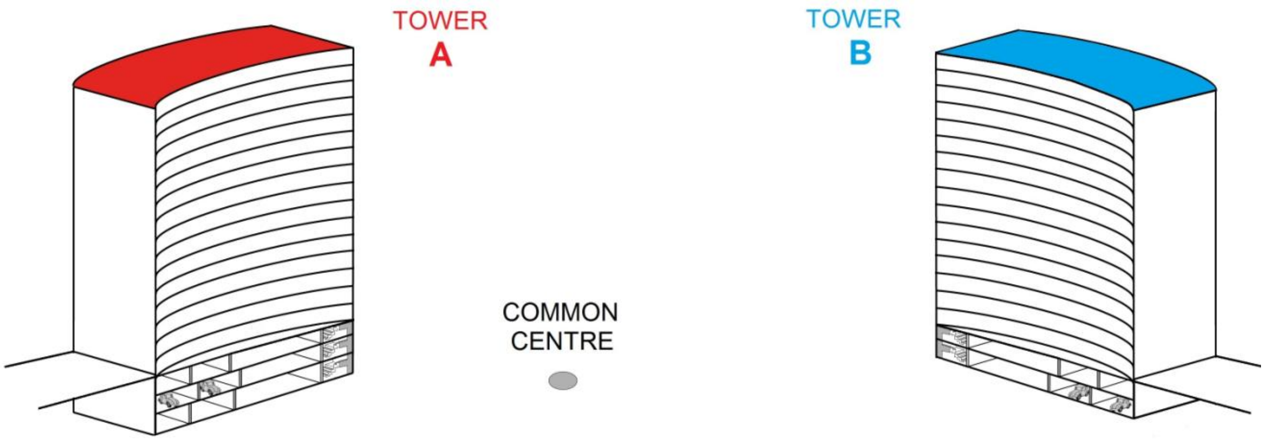


Figure 1. Sketch of the two towers.

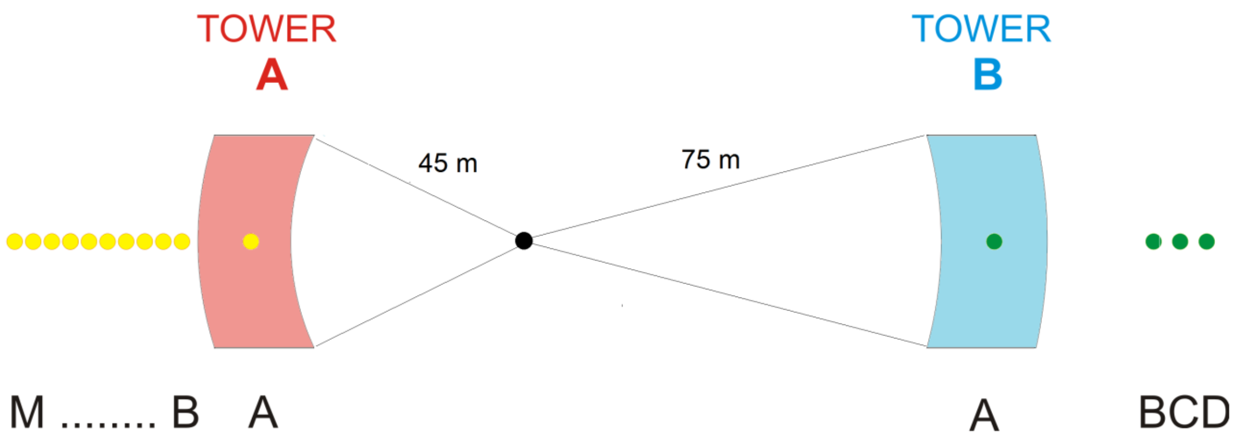
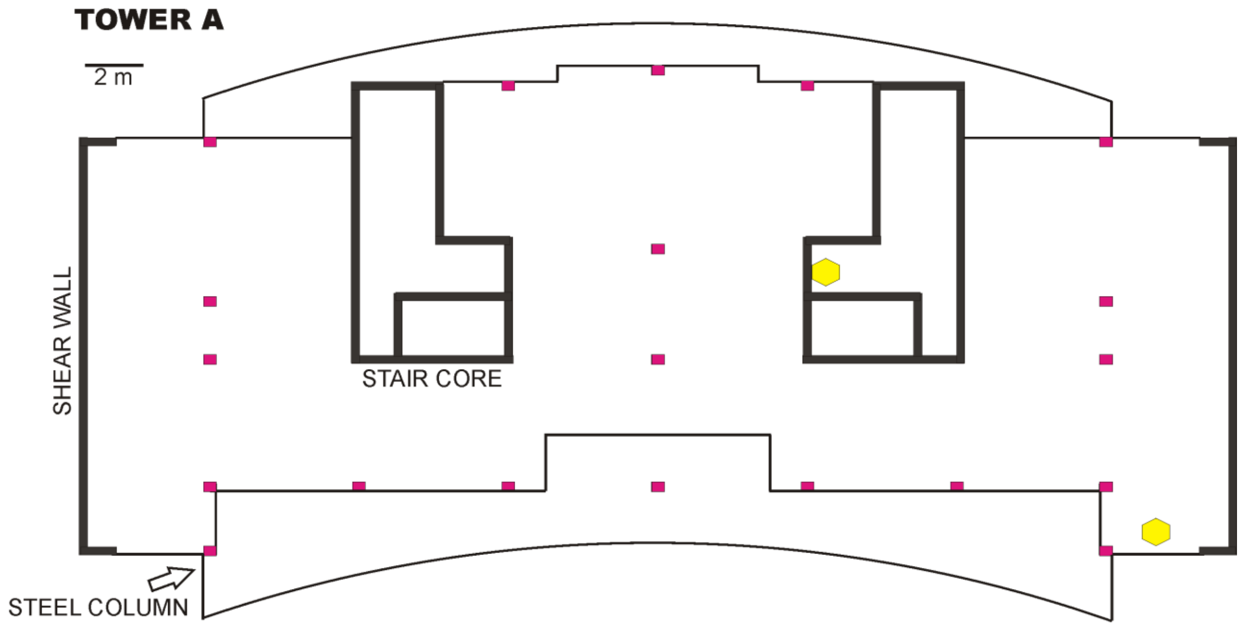


Figure 2. Schematic drawing of the tower plants that highlights their slightly different curvature ray, hardly perceptible to the sight, which is however mostly an architectural rather than a structural feature. The dots (A, B, C… M) indicate the alignment of the measurements on the ground for tower A (yellow) and B (green). See also

Figure 10.



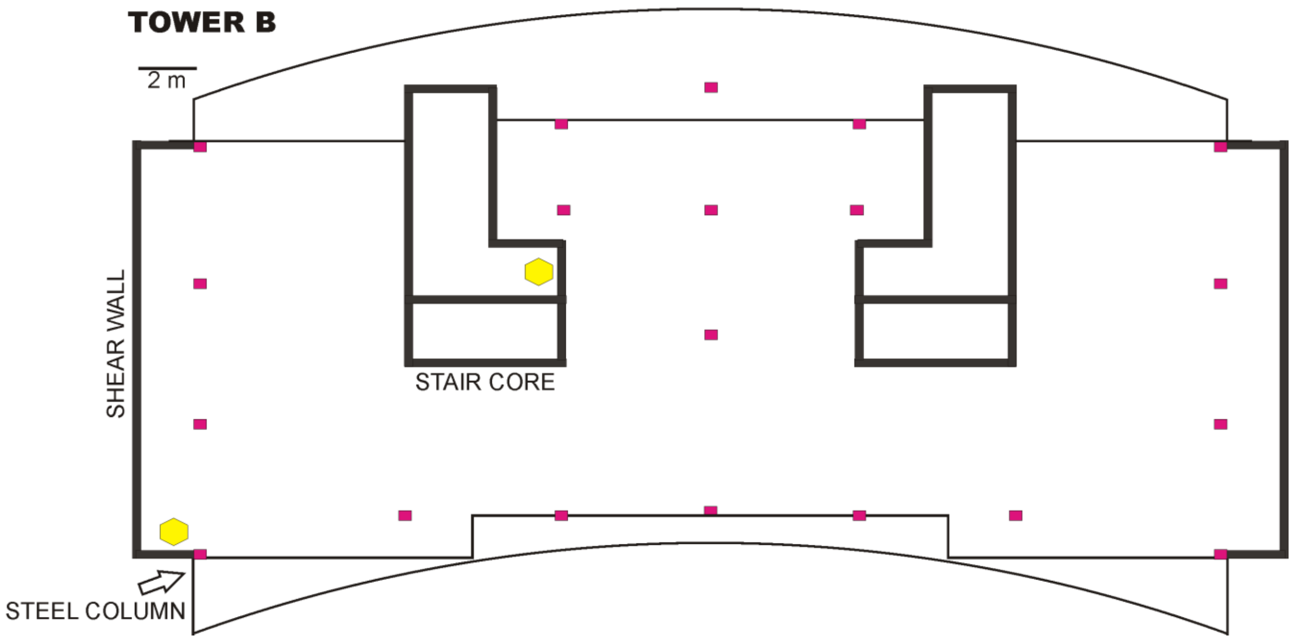


Figure 3. Structural elements of the two towers. Shear walls (thick lines) and steel double-T columns (magenta rectangles). The yellow hexagons indicate the vertical alignment of instruments inside the towers. On each even level two recordings were taken: one approximately in a barycentric position (compatible with the accessibility of the tower at the different levels) and one in a peripheral position.

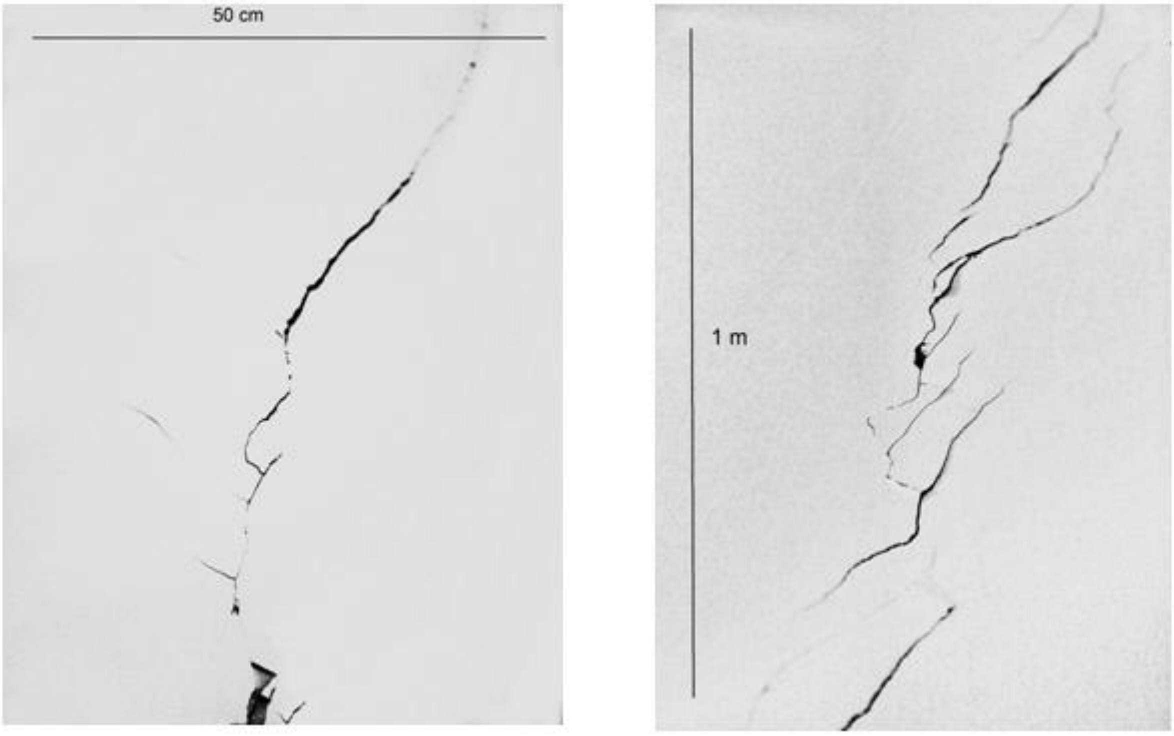


Figure 4. Example of damages (fissures) suffered from tower B as a consequence of the 20th May 2012 earthquake. The fractures have size of the order of meters [courtesy of L. Vigliotti and M. Rovere]. The most part of fractures developed on walls parallel to the transversal axis of the tower, from the 5th floor upwards.

Dynamic analysis of the structures and soil

Experimental structure analysis

As already said, the main structural elements are two RC shear walls and two RC stair cores. An irregular layout of double-T steel columns complete the bearing structure (Figure 3). Both structures are set on a strip foundation (approx. 1 m thick) at 12 m depth under tower A and 10 m depth under tower B. In practice, tower A has one underground level more than tower B, even though the absolute depth of the foundations differ by 2 m only.

The dynamic characterization of the structures was conducted in passive mode, using seismic microtremors as an excitation function. Different methods have been used by different authors to perform dynamic characterization of structures from microtremors (e.g. Snieder and Safak, 2006; Castellaro e Mulargia, 2010; Reynders et al., 2010; Pérez-Gracia et al., 2011; Döhler et al., 2012; Ditommaso et al., 2013; Vidal et al. 2013) and to compare the seismic behaviour of a building subjected to seismic microtremors, weak-motion and strong-motion (Hans et al., 2005). On each tower we recorded the motion at all the even levels on a vertical alignment close to the barycentre (as close as possible, compatibly with the tower accessibility) and on a vertical alignment close to a corner (hexagons in Figure 3). Since in passive surveys the source is spatially distributed and may change with time (e.g. due to wind, traffic etc.) a reference instrument was set at the foundation level of each structure while

moving the other instruments at different levels along the same vertical axis. Eventually, all the measurements were referred to the reference instruments, which therefore acted as a 'normalization'. In order to ensure the stationarity of the excitation, the survey was conducted in a short time span (approximately 3 hours), by sampling microtremors for 15 minutes at each location and continuously at the reference sites. Longer measurements (6 hours) were conducted at the topmost levels of the two towers to characterize their damping.
The experiment was conducted simultaneously on the two structures using four Tromino® digital tromographs (www.tromino.eu, Micromed s.p.a.). Each instrument is a stand-alone unit which can be linked to the others through the built-in radio and which measure both velocity and accelerations. Due to the higher sensitivity of the velocity sensors (instrumental self-noise is approx. $10^{-8.5}$ m/s^2 at frequencies larger than 1 Hz) compared to accelerometers, we use the velocity data. The instruments were simply laid down on the surveyed floors with no particular anchorage other than the instrumental weight (~1 kg) The survey was conducted twice, the first time in July 2012 and the second time in May 2013. The analysis allowed us to obtain: 1) the modal frequencies, 2) the modal shapes and 3) the damping:

The modal frequencies appear as peaks in the horizontal spectra of the recorded motion at each floor. However, in order to normalize for the possible amplitude drifts of the natural input during the measurement, spectra recorded at each floor where divided by the corresponding (in time) spectrum measured by the reference instrument at the foundation floor. This results in plots with adimensional y-axis, since they represent ratios between amplitude velocity spectra. Due to the absence of wind during the surveys, we assumed that the main difference in the excitation function came from the subsoil and this is the reason for putting the reference instruments at the foundation level. Results are shown in Figure 5 and indicate that the first flexural mode of tower A is at 1.1 Hz in the transverse direction and at 1.2 Hz in the longitudinal direction. The modal shapes for the transversal and longitudinal direction are shown in Figure 6. This tower was analyzed also 3 years before (Castellaro e Mulargia, 2010) and the modal frequencies and shapes obtained from the present survey coincide with those obtained from the previous survey, thus indicating that neither earthquake nor ageing have caused any structural damage to the tower. The difference in the amplitude of the modal shape between the two surveys is only apparent and due to the fact that the previous survey used the ground floor as a reference site instead of the foundation floor. Correcting for this difference leads to the same absolute amplitude of the modal shape.
Measurements on the B tower had unexpected features: the first flexural mode in the transversal direction appeared at 0.9 Hz (i.e 20% lower than the twin tower A) and at 1.1 Hz along the longitudinal axis (i.e. 8% lower than the twin tower B).

From the above comparison we observe that the dynamics of the two towers is rather similar along the longitudinal axis (in terms of both frequency and amplitude) while - as anticipated - the transversal behavior is very different for two supposedly almost identical structures. Since the mass distribution of the two towers is very similar, a lower stiffness of tower B compared to tower A must be responsible for this difference. The slightly different curvature of the two towers (Figure 2) is mostly an architectural element, therefore it is not expected to play a role higher than a very small percentage in this difference (Figure 2). The different depth of the foundation embedment could instead play a bigger role but, as we will see later from the results of a numerical model, can only account for about 6% of the difference. Other factors must therefore determine this difference: variations in structural layouts, structural details and material properties are the best candidates. In fact, the dissimilar layout of steel columns might also explain why the variation between natural frequencies is larger among the transversal rather than among the longitudinal modes.

For what refers to higher modes, the modal analysis revealed that the second mode (1.6 Hz for tower A and 1.4 Hz for tower B) is a torsion mode. This emerges from the correlation of the intra-floor (corner to centre) measurements which emphasizes the torsion modes. The second flexion modes appear at frequencies of approximately 4-4.5 Hz and shapes the structures as illustrated in Figure 7.

Concerning modal shape amplitudes, we observe that under the excitation of ambient noise, tower B has an excursion which is 40% larger than tower A in the transversal direction while in the longitudinal direction the dynamics is identical both in frequency and amplitude. The displacement of tower B is 40% larger than tower A also for the first torsion mode.

The main hypotheses that could explain the differences in amplitudes are the same proposed to explain the difference in the natural frequencies, i.e., a) differences in the foundation embedment and b) differences in the structural stiffness, due to several possible factors.

As a last step, the damping associated to the first flexural mode has been quantified for both towers through the non-parametric method proposed by Mucciarelli and Gallipoli (2007), which consists after the offset removal, in the integration of the signal (recorded in terms of velocity) to get the displacement. The local maxima are then sought and a matrix of amplitude maxima $x(t_i)$ vs. time of occurrence t_i is built. For each couple of data, if $x(t_i) > x(t_i + 1)$, then the pseudofrequency and damping ratio are calculated. This technique is less accurate than the classical RandomDec (Vandivier *et al.*, 1982) but allows an approximate estimation of the damping even from short microtremor recordings, as in this case. The median of the damping distribution for tower A is estimated in 2.5% while for tower B appears to be lower than 2%, which is in line with the above considerations.

We emphasize that since tower B suffered from many but luckily only aesthetical damages, we do not expect its modal pattern to be substantially different now from what it was before the earthquake.

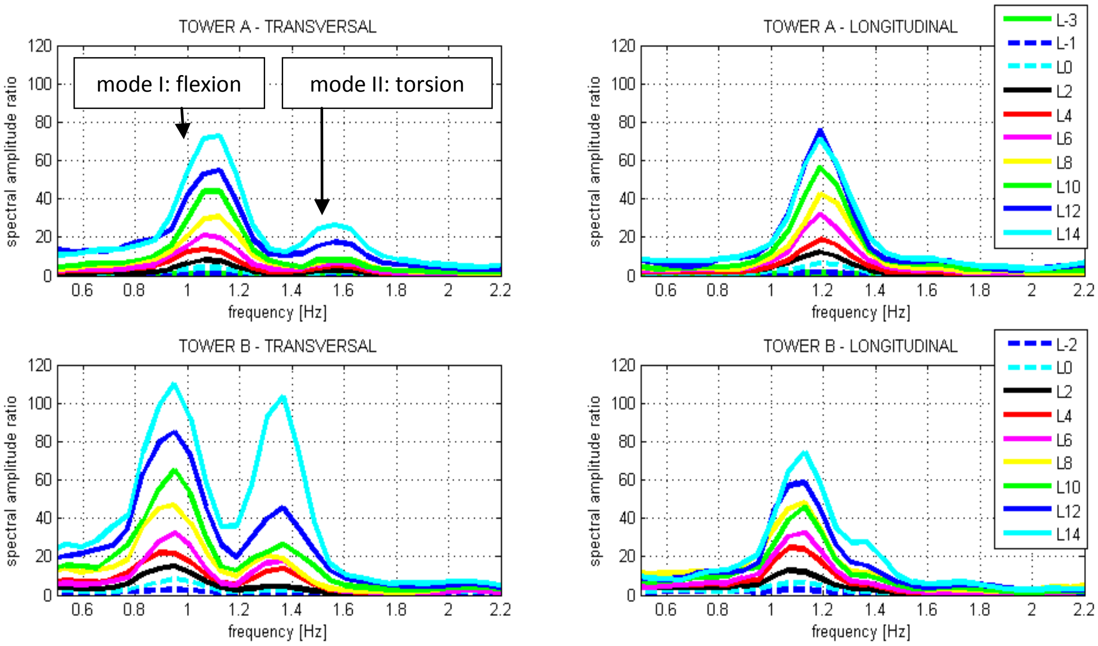


Figure 5. Modal frequencies of towers A and B obtained from the ratio of the spectra recorded at the different levels and those at the reference level (foundation floor). We observe that the behavior along the longitudinal axis is rather similar, while a substantial difference (both in terms of frequency and amplitude) appears on the transversal axis.

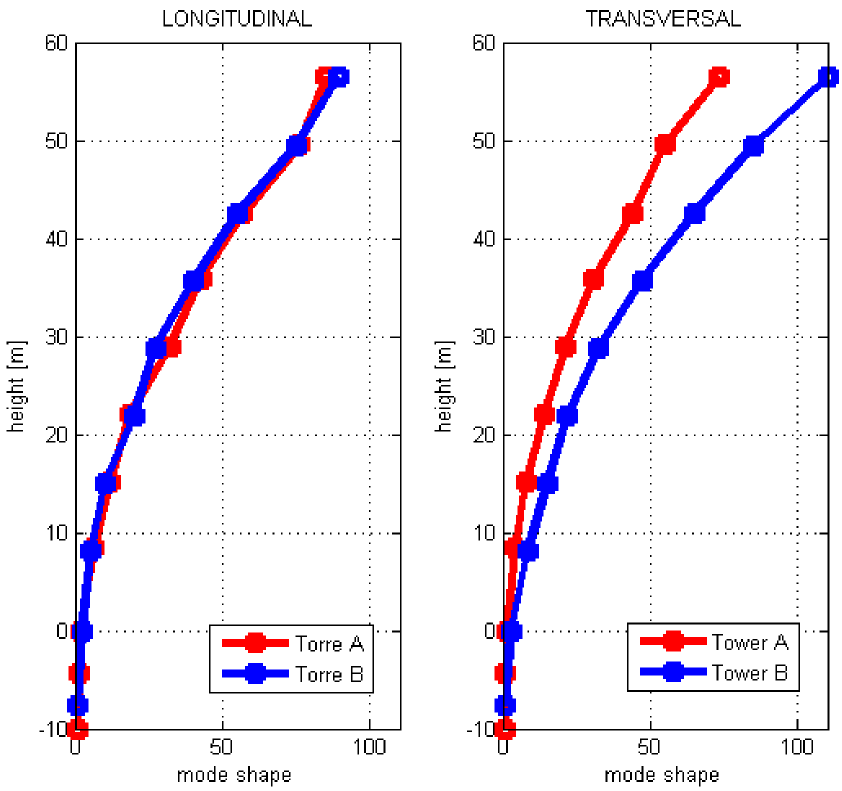


Figure 6. First flexion mode shapes (normalized to the foundation level) of tower A and B in the longitudinal and transversal direction.

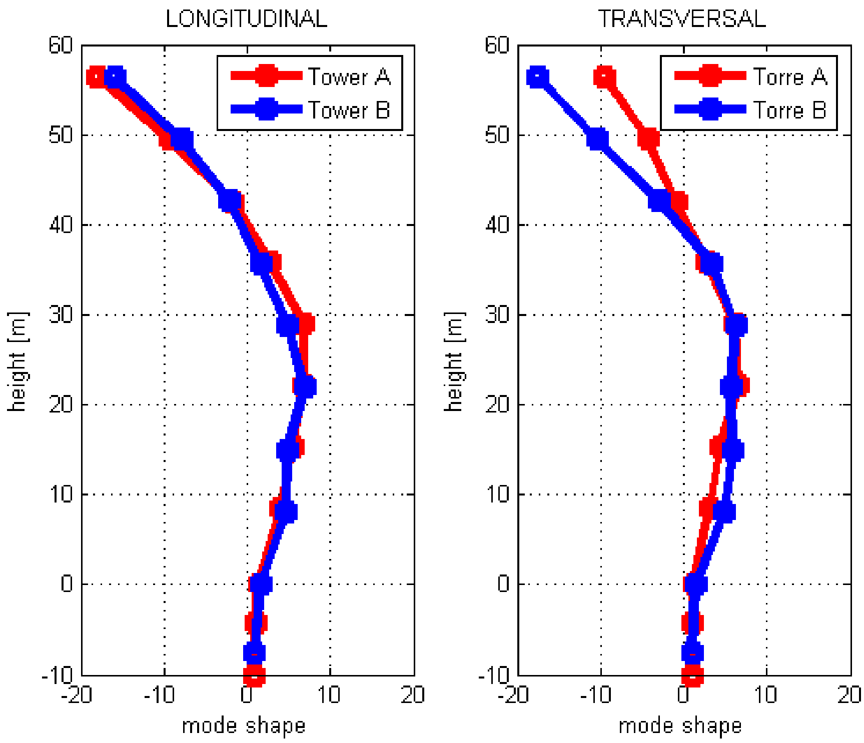


Figure 7. Second flexion mode shapes (normalized to the foundation level) of tower A and B in the longitudinal and transversal direction.

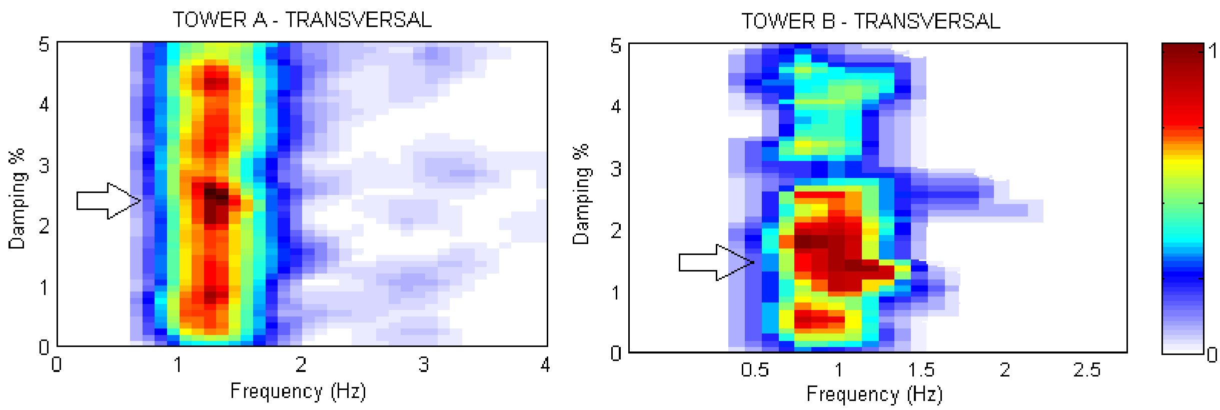


Figure 8. Damping estimated for the first flexural mode in the transversal direction for tower A (left) and B (right).

Soil analysis

The foundation soil of the tower has been surveyed several times in the past and presents shear wave velocity values (Vs) of about 200 m/s and a main resonance frequency, well know in this part of the river Po Plain, of 0.8 Hz. The results of the multichannel passive/active surface wave based surveys, of the H/V curves and their joint fit and interpretation are shown in Figure 9.

The first observation is that tower B is in resonance with the soil: the first flexion mode is coincident and the first torsion mode is very close to the soil resonance frequency. This does not occur for tower A (red and blue shades in Figure 9).

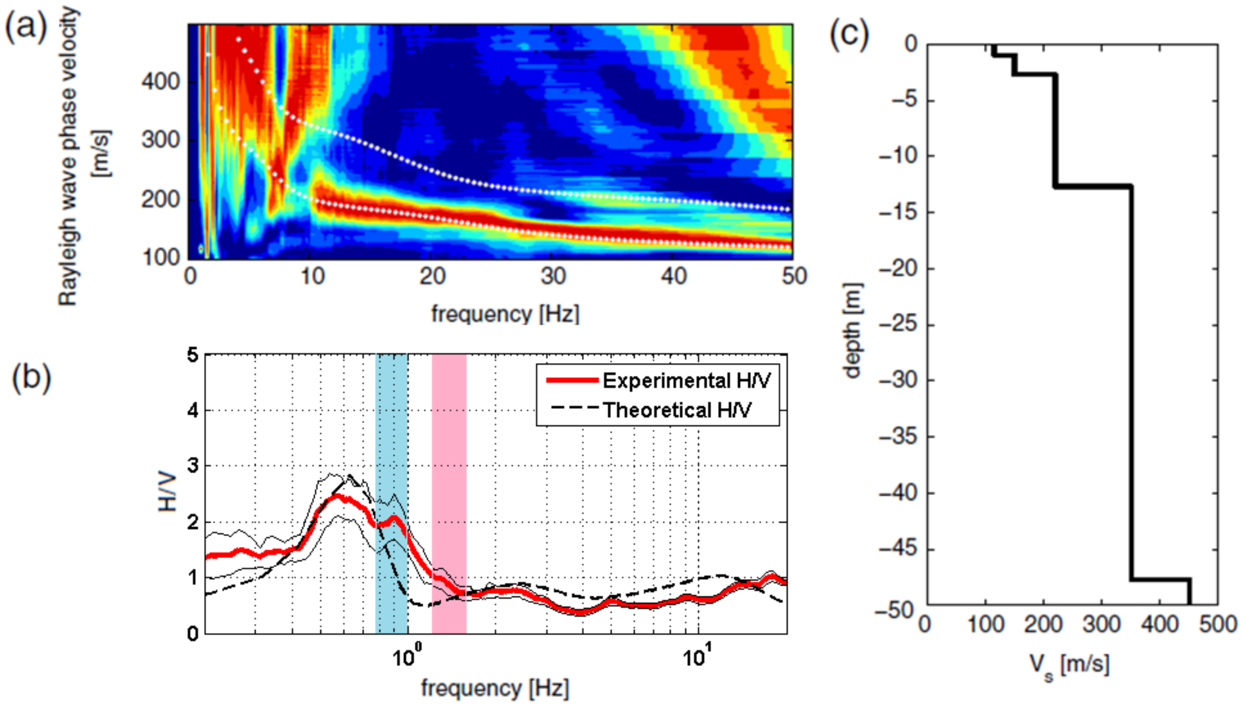


Figure 9. A) Rayleigh phase velocity spectra recorded on the free-field surrounding the towers, B) average H/V curve (red) ± standard deviation (thin black line) of the site. The shaded areas represent the frequency of the first flexion mode of tower A (red) and B (light blue).
The joint fit of A) and B) suggests the Vs model illustrated in C). The theoretical dispersion curve (first 2 modes) is given by the white circles in panel A) and by the black dashed line in panel B). The investigation depth reached by the survey A) is limited to 15 m. The remaining part of the model was obtained by the joint fit of A) and B) beyond this depth.

Soil-structure interaction

In a former study (Castellaro and Mulargia, 2010) we evaluated the soil-structure interaction in the case of large structures and we concluded that the radiation of the structure motion to the soil, which decays as the inverse of the distance, is more evident when the soil and structure resonances coincide.

As an example, in the case of the leaning tower of Pisa (with the first flexural mode at 1 Hz, which coincides with a secondary resonance of the soil), the radiation of the tower motion to the soil was perceptible up to distances of 12-14 m from the tower.
In the case of the Asinelli tower in Bologna, the first flexural mode (~0.3 Hz) can be recognized on the soil just within very few meters from the tower and the second mode (~0.8 Hz) can be unexpectedly observed up to the same distance because it coincides with the resonance of the soil.

In the case of the San Marco bell tower in Venice, which is a massive structure, the first flexural mode (~0.7 Hz) can hardly be measured on the surrounding area which instead shows the resonance frequency of the soil at 0.25 and 4 Hz.

In the case of the towers under analysis we expect that the motion of the undamaged tower A, which has resonances far from the soil resonance, is much less visible on the surrounding soil compared to the motion of the damaged tower B, which is in double-resonance with the soil. This is in fact what we observe: at 7 m distance from the foundation perimeter (point C in

Figure 10 and Figure 2, absolute distance from the center of tower A: 16.5 m) the effect of the radiation of the tower A to the soil is negligible.
In the case of tower B, where the first and second modes occur at frequencies very close to the soil one, the tower motion is radiated to the soil up to at least 40 m from the center of the tower (

Figure 10).

We remind that the radiation of a structure motion to the soil appears as a sharp peak in the spectra recorded on the soil at the resonance frequency(ies) of the structure. This peak is more evident on the horizontal components but can also be evident on the vertical component, as a function of the rocking. As a further evidence, the peak fades while moving away from the structure. This cannot be confused with the soil resonances because in microtremor recordings the latter are identified by a local minimum in the vertical spectral component while the horizontal ones usually do not show any peak or, when they do it (as an effect of SH or Love waves), a) its amplitude does not decay with the distance from the structure, b) it has a much broader 'gaussian' shape than the sharp peaks typical of the structures (cfr. Figure 18 in Castellaro and Mulargia, 2010).

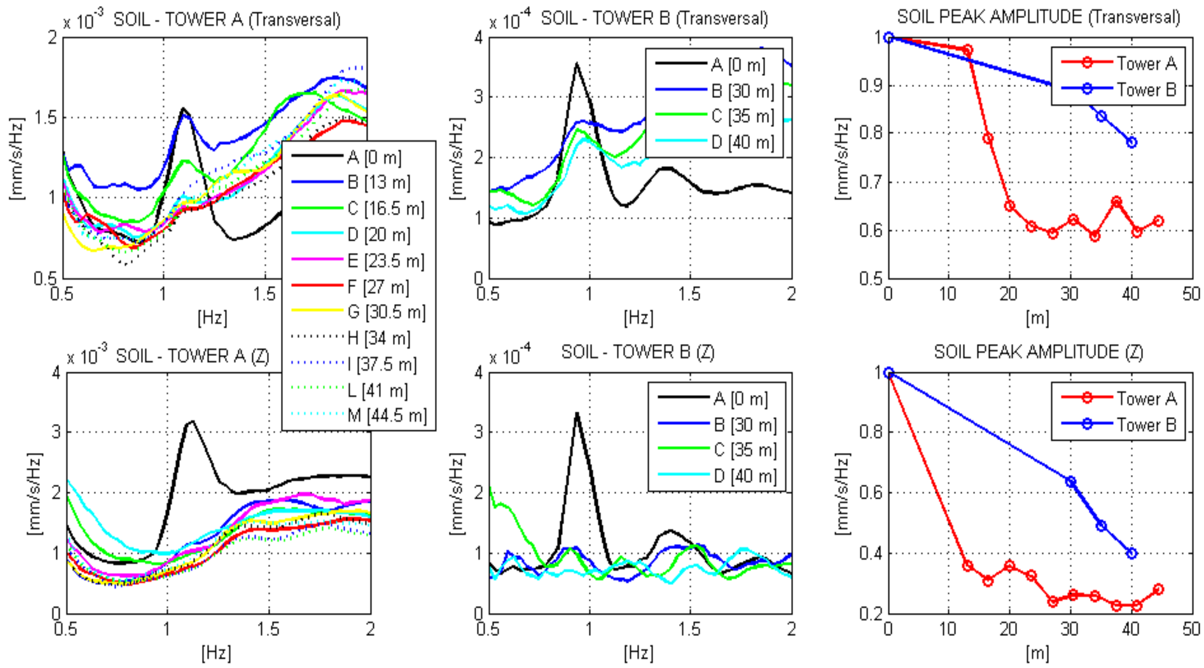


Figure 10. ***Left column:*** **horizontal (top) and vertical (bottom) spectra recorded on the ground at different distances from tower A (dots in Figure 2).** ***Center column:*** **same for tower B. The distance is calculated from the center of each tower (point A, 0 m) and increase outwards. The first measurement point outside the foundation of tower A is point C, at 16.5 m distance from the center of the tower. The first measurement point outside the foundation for tower B is point B, at 30 m distance from the center of the tower.** ***Right column:*** **spectral amplitude decay of the peak induced by the tower motion on the ground at the frequency of the tower first flexural mode frequency. The decay is much more evident for tower A than for tower B, due to the resonance of the latter with the soil.**

Numerical modeling

Finally, in order to understand whether the difference in foundation embedment can explain the differences observed in the fundamental frequencies of the two structures, a simplified numerical analysis was carried out taking into account the soil-structure interaction. The multidomain Boundary Element Method code in the frequency domain described in Maeso *et al.* (2002) has been used to this end. This section studies the problem as an idealized block model, where the building, foundation and soil are modeled as homogeneous viscoelastic media. Nine-noded quadratic elements are used to mesh the boundaries, and the element size is set to be much smaller than half the shortest wavelength. The length of the discretized free surface equals the height of the superstructure, which corresponds to 4.6 times the embedment of the deepest foundation. The excitation is defined by a vertically incident S wave causing displacements in the direction of the plane of symmetry of the problem. Due to the simmetry of the geometry, only one half of the problem needs to be discretized (Figure 3).
The foundation, which is expected to be very stiff, is characterized by the properties of concrete (with 20% of its density), while the properties of superstructure have been tuned to match the fundamental frequency f_0 = 1.1 Hz measured for tower A. The first two transversal mode shapes obtained from this model are illustrated in Figure 12. It can be seen that they

agree very well with the experimental modal shapes shown in Figure 6 and Figure 7, which suggests that this simplified model is able to capture the general behavior of the system.

The properties that allow the model to fit the experimental results found for tower A are – at this stage - expected to fit also the experimental results found for tower B. However, this is not the case: by using the same parameters and accounting only for the geometrical differences we observe a reduction of the fundamental frequency of the order of only 6% (against the experimental 20%), of which around 3% is due to the foundation being closer to the V_S=350 m/s stratum in the case of tower A (see the soil shear wave velocity profile in Figure 9c). This means that the foundation embedment alone is not capable to explain the experimentally observed differences, and that other factors must come into play. Different structural stiffness due to variations in the structural layouts, structural details and material properties, as well as local variations in the subsoil just below the foundations, must be responsible for these differences. In any case, and concerning the seismic design of the structures, very detailed dynamic analyses, taking into account soil-structure interaction, would have had to be performed for both structures in order to be able to anticipate the measured differences and the associated seismic risks.

	Shear modulus G [N/m²]	Density ρ [kg/m³]	Poisson's ratio	Damping ratio
Stratum	$8.38\ 10^{7}$	1900	0.4	0.02
Half-space	$2.45\ 10^{8}$	2000	0.4	0.02
Foundation	$1.15\ 10^{10}$	380	0.2	0.02
Superstructure	$2.89\ 10^{8}$	380	0.2	0.05

Table 1. Domain properties.

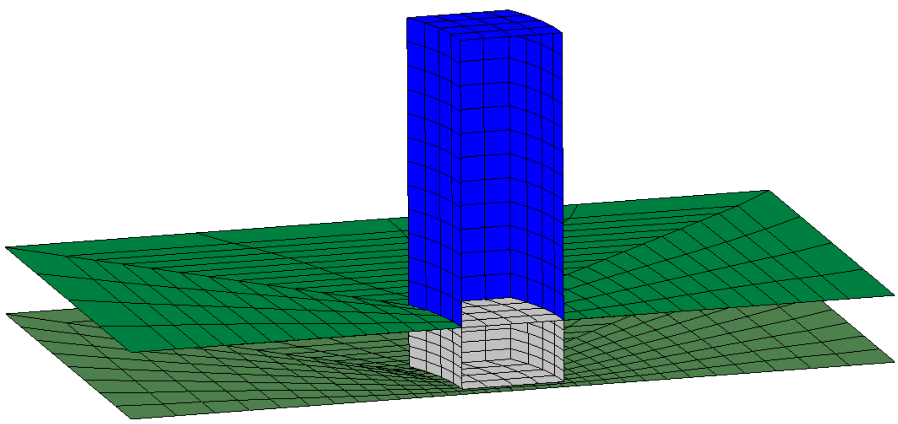


Figure 11. Example of the meshes used for the analysis. Due to symmetry, only half problem needs to be meshed.

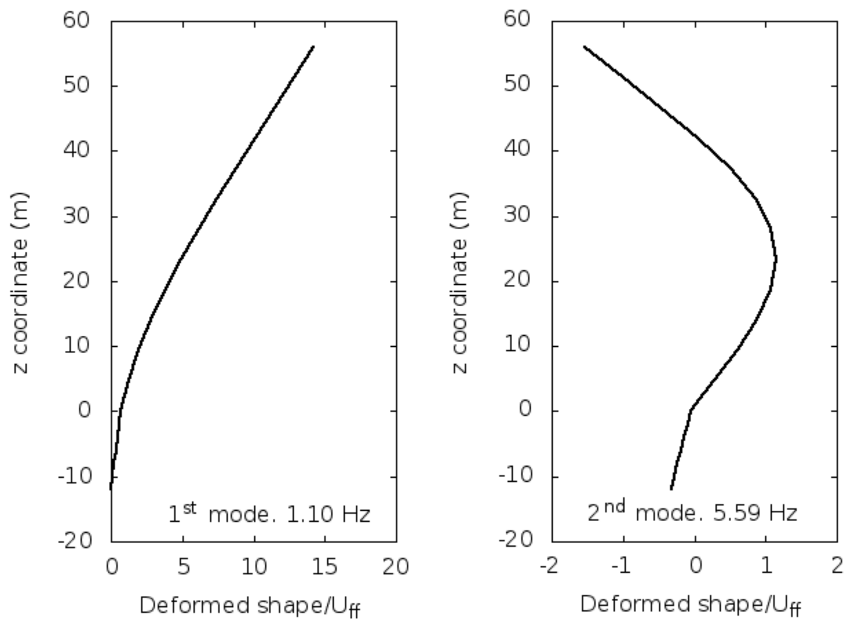


Figure 12. First and second flexion mode for tower A according to the numerical model.

Discussion and conclusions

As a consequence of the Mirandola 20th May 2012 $M_L = 5.9$ earthquake, we observed a very different damage level on two esthetically identical residential structures located at 45 km from the epicenter. Through a passive modal analysis of the structures and of the foundation soil and specific numerical modeling, we have tried to identify the reasons for such a different behavior.

The detailed analysis showed that the two structures (tower A: undamaged, tower B: damaged) have approximately the same mass distribution and therefore the same rotation inertia. They have a slightly different ray of curvature, which is mostly an architectural feature, and which would potentially result in a slightly larger rotation stiffness for the more curved structure (tower A). They have also a slightly different foundation depth (2 m). Experimental evidence shows that the fundamental frequencies of the two towers differ of 20% in the transversal direction while numerical analyses suggest that the geometric difference can only explain a difference of the order of 6%. It follows that the two structures must have a different intrinsic stiffness, most probably explained by their complexity and irregularity and the fact that they were designed by independent engineers.
As a consequence of the lower stiffness, tower B is in full double resonance with the soil, while tower A is not. Double resonance is a well known factor that induces much larger amplitudes in the structure than expected when the structure and soil eigenfrequencies are not far one from the other.

An evidence supporting the importance of the soil-structure interaction in this case is that the undamaged tower A does not radiate its motion to the surrounding soil in an appreciable way while the tower motion radiation is well measurable on the soil around the damaged tower B. This phenomenon is common under coincidence of resonances between soils and structures.

In conclusion, we have observed a very different dynamic behaviour in two esthetically and apparently identical structures, that led one of the two to suffer from damages during a recent seismic event, while the other was completely unaffected. A detailed analysis revealed that the damaged tower has lower natural frequencies, in such a way that the first transversal flexural mode coincides with the subsoil resonance. As a consequence of the double-resonance, the damaged tower has a mode displacement amplitude 40% larger than the undamaged tower even under ambient vibration excitation.

This case shows that two structures that would be defined identical to the sight (and for any insurance inspector) can react in a very different way for at least two "hidden" factors: a) structural stiffness and b) soil-structure coincidence of resonances. For this reason detailed experimental dynamic analyses taking into account also soil-structure interaction should be performed to anticipate possible differences and associated seismic risks.
The above issues cannot be considered as 'details' in design since in practice they led a modern building to be damaged by a M < 6 earthquake at 45 km distance.

Acknowledgments

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