In field data to correctly characterize the seismic response of buildings and bridges

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Abstract

The use of in-field acquired data to characterize the seismic response of building and bridges immediately after an earthquake, is discussed in this paper; in particular, we discuss how dynamical tests could help the assessment phase and could indicate if the structure is in condition to continue its regular use.
To this end, two different case-studies will be discussed: the first is concerned with the interpretation of a particular behavior exhibited by a group of four buildings characterized, despite of the similarities among them, by strongly different seismic response (two of them collapsed and the other two survived with a medium damage level) and the second one is concerned with a campaign of dynamical tests finalized to a possible damage detection on a r.c. bridge.
Concerning the first case study it is worth noticing how, during the 6 April earthquake in L'Aquila, two couples of similar (nearly twin) buildings belonging to the same urban intervention, exhibited a dramatically different response. In the two couples, the first couple constituted by two *four-story buildings* and the second couple by two *three-stories buildings*, one element collapsed suffering the well-known soft story mechanism, while the other one survived with a damage pattern not so strong, as expected considering the similarity with the collapsed one. The in situ measurements conducted in the framework of microzoning activities provided the opportunity of gathering useful information about the dynamical properties of the soil, thus evaluating possible site amplification effects. A key role was played in this case by a local amplification of the seismic intensity, due to the geology of the site. On the same time, dynamical tests on the survived buildings enlightened their dynamic characteristics and

[†] Francesco Benedettini passed away on February 15, 2015, at the age of 59. This paper is intended to be a testimonial of affection to a beloved colleague and friend. A.D. and G.M.

permitted the updating of the FE models used in the structural analysis. Linear and nonlinear models of the buildings have been developed, aiming to explain the different behavior. Even the use of simple linear models taking into account the contribution in stiffness of the infilled masonry walls permitted a coherent interpretation of observed phenomena. By using more accurate nonlinear finite elements models, it was possible to reconstruct the evolution of the seismic response and to obtain a more clear unfolding of the collapse mechanism. The key role of the infilled walls has been enlightened putting into evidence how a good construction of such non-structural elements could dramatically influence the global structural behaviour. Some laboratory tests on specimens of infill walls, extracted from the collapsed building, guided the selection of the mechanical parameters of the equivalent truss elements used in the models.
With the second case-study we discuss a different, interesting and effective use of dynamical measures on damaged structures, when the same measures have been conducted in a not-damaged (or reference) condition. In this case, the dynamical tests could enlighten the stiffness reduction and can help to localize the occurred damage. With this purpose some test conducted in L'Aquila (Italy) after the 6 April 2009 earthquake on the r.c. Belvedere Bridge, will be discussed.

Keywords: Soft storey, damage detection, nonlinear dynamic analysis, structural identification, site amplification, seismic collapse.

1. Introduction

The use of dynamical tests on structural and infrastructural systems is worldwide recognized as an effective and reliable mean to help the assessment phase if, for different reasons, the actual response to (dynamical) loads should be ascertained.
On April 6th a Mw 6.3 earthquake, strongly hit the city of L'Aquila (Central Italy) producing more than 300 casualties and diffused damage on residential and public buildings, as well as on infrastructural elements like bridges, roads, tunnels. Some papers are available in literature describing the observed damage. In (Kawashima *et al.* 2010) a reconnaissance on the damage to buildings, bridges, and other structures is reported. Different structures are characterized by a variety of construction techniques, materials, and erection periods. The paper focuses its attention on the use of dynamical tests to understand the seismic response of buildings and bridges, trying both to identify the probable causes of reported damage or structural failure, and to understand the damage evolution and the collapse mechanisms. The dynamical tests are usually used in conjunction with numerical analyses to enlighten the seismic behaviour of selected structures and to help the assessment campaign of infrastructural systems in the ambit of Structural Health Monitoring Programs programmed by the National Civil Protection Department after every important seismic event.
Such a possibility of a quantitative assessment of structures by means of dynamical tests is established by discussing two case studies.
As well known, the presence in a building of a first storey having a lateral stiffness lower than the stiffness of the other ones can generate an anomalous behaviour under strong horizontal excitation, as in case of an earthquake of medium-strong intensity.
If this is the case, the higher part of the building almost behaves as a rigid body moving on a soft support (the archetypal scheme is the inherently unstable inverted pendulum) and the vertical elements (columns) of the soft storey suffer by a generally large relative displacement demand (drift), often well above their capacity.
The so-called soft-storey mechanism has been widely discussed in literature. In

(Chopra *et al*. 1973) this mechanism was studied in view of determining under which conditions a yielding first storey could adequately protect the upper stories from significant yielding. The results demonstrated that a very low yield force level and an essentially perfectly plastic yielding mechanism are required in the first storey to provide effective protection to the superstructure. Moreover, the displacement demand of an effective first storey mechanism was found to be very large. The paper by Ruiz *et al.* 1989 is based on the observation gained during the Michoacán earthquake of September 19, 1985. In the paper, the possible influence of a lateral strength discontinuity on the ductility demand at the first level is studied. A parametric study for five and twelve-storey buildings with weak first storey is presented. The infill walls in the upper stories were brittle in some cases and ductile in others. For certain cases, the results show the existence of a range of values of the ratio of seismic lateral resistance of the upper stories in comparison to the one of the lowest story for which the ductility demand (at the lowest story) can be considerably high. It is shown in the paper that the capacity of energy dissipation in ductile walls also plays an important role in the displacement ductility demands of the first storey. Dolsek *et al.* 2001 report that a large number of multi-storey reinforced concrete frame buildings, with masonry infill walls uniformly distributed over the height of the building, collapsed in the 1999 Kocaeli (Turkey) earthquake, due to complete failure of the first (or first and second) storey. In the paper, it is demonstrated that a soft storey mechanism is activated in such structural systems if the intensity of ground motion is above a certain level. It is likely that collapse will occur if the global ductility capacity of the bare frames, as well as the ductility capacities of the structural elements, are low, and if the infill walls are relatively weak and brittle. In (Decanini *et al*., 2004b) a detailed analysis of two buildings damaged during the 2002 Molise, Italy, Earthquake is reported. The applied methodology is very similar to that used in the present paper. The objective of the paper (Decanini *et al*. 2005) was to reproduce and analyse the response of typical RC frames subjected to the 1999 Athens earthquake in areas where the observed damage was particularly severe but no recordings of the ground motion were available. After a general overview of the seismo-tectonic environment, seismological data, observed macro-seismic intensities, structural typologies and observed building behaviour, an attempt is made to identify representative excitations in the meizo-seismal area. Specifically, design accelerograms were obtained by modifying available records in such a way to reproduce a given global energy content and to be consistent with the observed damage, in particular soft-storey damage. To study the seismic response of RC models, the obtained accelerograms were used to perform nonlinear dynamic analyses. In (Bazzurro *et al*., 2006) a fully probabilistic seismic risk analysis of 48 different configurations of 4-, 6-, 8-, and 12-story reinforced concrete frames with and without masonry infill walls located in four Italian cities is discussed. The results show that, in absence of torsion, the design practice usually adopted of neglecting the contributions of wall panels, is generally conservative for regular frames, and particularly significant in the case of well-constructed, strong infill walls, made using solid bricks and good-quality mortar. Conversely, this practice may not always be conservative if the infill walls are very weak, such as those made with hollow bricks and poor-quality mortar.
The soft storey mechanism has been observed in many cases after the 6 April earthquake in L'Aquila (Celebi *et al*., 2010). In Figure 1, some significant images of (partially) collapsed buildings observable in L'Aquila are reported. It is possible to verify that the *soft storey mechanism* usually affected the first floor,

but also affected a different one, if this is characterized by the lower stiffness in comparison to the other ones.
In the present paper, some soft storey significant collapses of buildings in Pettino (L'Aquila) are discussed and analysed using linear and nonlinear interpretative finite elements models. Because a key role was played in this case by a local amplification due to the geology of the site, the influence of site effect will be analysed as well. The linear and nonlinear models of the buildings, used to explain the different evolution, has been validated by means of dynamical experimental tests on the survived structures.
Dynamical tests can be also devoted to recognize the possible occurrence of a structural damage, even in the case in which a direct observation does not show any evident symptom. The capability of dynamical measures to recognize, quantify and even localize a structural damage is an open research topic and different approaches, based on different damage indicators have been proposed in literature. The easiest and most common quantity used in the damage detection procedures is the frequency lowering of the first structural modes, even if such a direct measures is not reputed to be always meaningful (Armon *et al.* 1994, Capecchi *et al.* 2000, Dilena *et al.* 2009). More sophisticated analyses, based on the (local) modification of modal shapes, seem to be reliable (Gladwell *et al.* 1999, Pandey *et al.* 1991). In particular, a possible discontinuity in the modal curvature appears as a promising way to localize a concentrated damage (Wahab *et al.* 1999, Dilena *et al.* 2011).
Besides the above cited parametric methods, based on a physical or modal models of the structure, non-parametric models, based on the modification of suitable sub-spaces in which the dynamics is confined, appear so far very sensible and promising of success (Yan *et al.* 2006, Dohler *et al.* 2014).
Such a kind of analysis is discussed in the ambit of the second case study, relevant to the identification of the dynamical properties of the Belvedere Bridge, located in L'Aquila downtown.

2. First case study: four buildings located in Pettino, L'Aquila

2.1 Damage description

The first case study is relevant to four buildings located in Pettino, belonging to the same urban intervention. The considered structures are two *four-stories* and two *three-stories buildings* erected into two similar couples; considering each couple, after the 6 April 2009 earthquake, it was possible to observe a collapsed and a survived building. In Figure 2 an aerial view of the area is reported. The buildings A and D have 3 stories while the buildings B and E, 4 stories. Moreover the building B (4 stories) is exactly corresponding to the building A (3 storey) being different only the numbers of elevations. The same happens for the building E (4 stories) and D (3 stories).
In Table 1 the damage observed after the 6 April earthquake is synthesized.
Figure 3 presents some view of building A (3 storey) before and after the seismic event. The view after the earthquake shows the collapse of the first storey.
Analogously in Figure 4 the view of the building E, before and after the earthquake, are compared. As in the preceding case, the building suffered the collapse of the first storey.

2.2 Main structural characteristics of the buildings

The following analyses are based on the original technical reports and drawings whose main characteristics were reflected in the study. Also some in situ and laboratory tests helped to select the mechanical parameters of the models (Fig. 5).
All buildings were designed according to the Italian codes Ministry Decree May 3, 1975 and June 26, 1976. The seismic action was considered using equivalent horizontal forces and the checks were based on the allowable stresses method. The equivalent seismic forces were applied to the plane frames constituting the buildings, considering the additional effects due to the plan eccentricity of the seismic action, according to the seismic code.
To correctly characterize the geometric and material parameters needed in the numerical analysis, the following experimental tests have been performed:

- electromagnethical tests on some floors for the definition of the position of the reinforcements in the structural walls,
- endoscopic and penetrometric test on the infill walls for the definition of the thickness and quality of the various layers;
- sclerometric tests on the concrete;
- laboratory test on concrete cylindrical specimens and portion of rebars extracted from the columns and beams;
- diagonal compression tests on masonry specimens extracted from the infilled masonry walls of the collapsed buildings (Fig. 5).

2.3 Seismic local amplification

The buildings are located on a NE-SW oriented gentle slope mainly composed by scree deposits and breccias (hereinafter PBr) that lie over the calcareous outcrops of Mount Pettino. At the foot of this slope runs the Aterno River valley oriented in NW-SE direction and filled by Quaternary deposits (Figure 6). Limestone also outcrops few hundred meters away from buildings in the area of Cansatessa village. The area was included in the microzoning activities promoted by the Dipartimento Protezione Civile (hereinafter DPC) after the April 6th L'Aquila event. The microzoning was aimed to identify, at a detailed scale, areas were local seismic amplification could occur due to the characteristics of surface geology or topography (Working Group MS, 2008, Working Group MS_AQ, 2010). As a result of the process, an amplification factor (Fa) was calculated for all the areas prone to amplification effects on ground motion. One of the activities requested in microzoning study was the installation of temporary seismic stations and/or networks for recording both microtremor and weak motion data. Microtremor data were aimed at deriving information on resonance frequencies for soft soil sites to be used as a constraint to the geological sections and models derived using different approaches. The weak motion data were used to evaluate amplification function to be compared with the results obtained by numerical modeling.
To better study the amplification effects induced by PBr we installed two seismic stations that operated for about 9 days starting from April 30 2009. One station (PC07) was installed on bedrock in the same location where the station AQM of the Italian Strong Motion Network (RAN) operates. A second station (PC10) was installed few meters apart from the building A (Fig. 7). The stations were equipped with 6 channels data loggers Quanterra Q330 with a dynamic range of 24 Db. The recording unit was connected to a Lennartz LE3d-5s seismometer and at a Kinemetrics Episensor accelerometer. During their operation the stations recorded several aftershock of the L'Aquila earthquake with maximum magnitude

of 3.6. The event list was derived from the data recorded by the national seismic network (RSNC) run by INGV (http://iside.rm.ingv.it). The data with a good signal to noise ratio were then selected for the analysis. We applied to the event the conventional spectral approach proposed by Borchardt (1970) and based on the spectral ratio between a sediment site and a rock reference site (SSR). In this analysis it is necessary that the couple of stations is located at shortest distance in comparison with the hypocentral distances. This assumption is necessary in order to assume that the contribution of source and path term in the Fourier spectrum is common for the two stations and can be eliminated in the spectral ratio. In this approach the choice of the reference station is crucial (Steidl *et al.*, 1996; Bordoni *et al*., 2010); the close distance of rock site PC07 (about 400 meters) to PC10 site justify its selection as a reference site. A check on the quality of PC07 as reference site was performed through the evaluation of H/V spectral ratio on microtremor data (Nakamura, 1989), Figure 8. The absence of peaks in the H/V function can be related to the absence of surface layers characterized by low shear waves velocity values. For PC07 H/V spectral ratio is quite flat for frequencies up to 6 Hz. A peak is found for higher frequencies (8-10Hz) to indicate the presence of some amplification related probably to rock fracturing. As a result of this analysis we can conclude that PC07 station can be used as a reference site at least for frequencies up to 6 Hz.

Starting from the selected earthquakes we cut a short time window (10 seconds) starting about 1 second before the S wave arrival. The Fourier spectra were evaluated on the selected time windows along with the geometrical mean of the spectral ratio for the horizontal components. Finally the average on all the evaluated spectral ratios allowed calculating the mean value of the amplification function along with its standard deviation. The results of SSR analysis for PC10 site are shown in Figure 9. It is clear an evident amplification peak centered between 2 and 3 Hz with amplitude values of about 8. The amplification factor found at the site is quite important and indicates the presence of strong local site effects for the area.

The shear wave velocity on the PBr formation was measured during microzoning activities (Figure 10) and resulted to oscillate around the value of 500 m/s at least in the first 30 meters below the surface. The possible impedance contrast between PBr and the Mount Pettino limestone formation at its base can justify some amplification effect at frequencies depending from PBr thickness.

Using a simplified 1D approach, and taking into account both the amplification frequency and the Vs values derived by in situ measurements we can estimate in about 50 meters the depth of bedrock at the PC10 site.

To check for the spatial stability of the amplification function in the area we performed a wide series of microtremor measurements in the sites shown in Figure 7. The microtremor data were collected during a short time window (40-60 minutes). Signals were cut into short duration (40 seconds) windows and processed through a detrigger algorithm (SESAME, 2004) to remove the ones affected by strong transient disturbs. Fourier spectra were evaluated and smoothed using the method proposed by Konno-Omachi (2004). Finally the average spectral ratio, along with the standard deviation, was evaluated for the geometrical mean of horizontal components. The results of the analysis are shown in Figure 7 along with the positions of the measuring points. It is clear that an H/V peak centered at about 2-3 Hz characterizes all the investigated area. The frequency of the peak tends to change moving along a profile oriented in NE-SW direction. All the area surrounding the buildings shows a quite stable H/V values both in frequency and

in amplitude.
As a conclusion of the seismological survey of the area we detect the presence of clear and strong amplification effects in the 2-3 Hz band with a good spatial stability at least in the block where are located the buildings analyzed in this paper.
With the aim of selecting signals to be used in the dynamic analysis of the buildings we examined the April 6th event strong motion records collected in the Pettino area where few permanent station of the Italia Strong Motion Network (RAN) operate. The station AQA (Working Group ITACA, 2010) is installed on gravel bed with a thickness of about 38 meters overimposed to a limestone bedrock. The geological setting of this station cannot be considered really similar to that of area where the investigated buildings are sited. Nevertheless the AQA record is still quite energetic in the frequency band where the amplification occurs in the area investigated in this paper. Considering that also the hypocentral distance is quite similar we believe that AQA record can be used for performing the structural analysis.

2.4 Dynamical properties of the buildings

The *in situ* measurements conducted in the framework of microzoning activities provided the opportunity of gathering useful information about the dynamical properties of the studied buildings. The buildings B and D (both not collapsed) have been instrumented with the same equipment used for collecting weak motion seismic data. Several ambient excitation-time histories were recorded and then the dynamic characteristics (frequencies, damping and modal shapes) of each building were identified by using different output-only identification algorithms: the Enhanced Frequency Domain Decomposition (EFDD, Brincker *et al.* 2001) and the data driven Stochastic Subspace Identification (SSI, van Overschee *et al.* 1996); these techniques are available in the commercial program ARTeMIS (SVS 2010). Figure 11 shows a phase of the installation of the instruments from outside of the buildings (at that time the buildings occupants were housed in temporary structures). The main characteristics of the instruments are collected in Tab. 2. The 3 first identified modal shapes belonging respectively to the 4-stories and 3-stories buildings are reported in Figure 12 and Figure 13. In these figures also the locations and channel directions of the triaxial sensors deployed on the buildings are shown. The identified modal characteristics were then used to calibrate (in the sense of model updating) FE element models of the buildings used in the assessment process to better understand the damage scenario. Both a response spectrum linear analysis and a more sophisticated, non linear, direct time integration were used to unfold the collapse mechanism.

2.5 Linear analysis

During the response of the buildings to the 6 April 2009 earthquake a key role, was played by the infilled masonry walls. The two collapsed buildings were characterized by the expulsion of infill walls at the first level, with an overturning local mechanism. After such a sudden expulsion, most likely occurred during the first phases of the earthquake, the walls were not able to contribute any more both to the lateral stiffness of the buildings (stiffening effect) and to the dissipation of the energy injected by the seismic action. This interpretation is strengthened by the observation that one of the overturned infills was observed lying on the ground without showing diagonal cracks (Fig. 3e). Such a circumstance was

probably the trigger activating the soft storey mechanism, ultimately responsible of the building collapse. On the contrary, as we will see in the following, a more regular seismic response and a medium-low global damage level characterized the two survived buildings in which the infills walls remained in place.
Several dynamic linear and nonlinear analyses have been performed, with the aim of proposing a possible explanation of the seismic behaviour of the studied buildings. The in situ measurements of the dynamic properties of buildings B and D (not collapsed) can led to useful information about buildings A and E (collapsed), due to the fact that the buildings belonging to the same couple are similar, having only a different number of floors. Using the structural identification approach some physical parameters (elastic modulus of the concrete, Winkler coefficient of the soil) of buildings B and D were identified, allowing a suitable updating of the relevant FE models.
Even the simple linear dynamic analysis can be used to understand the behaviour of the buildings during the earthquake and the effect on the response caused by the *presence* or the *absence* of infilled masonry walls at the first level. For buildings D and E, the frequency variations of the linear models due to the presence or absence of infills at first level are summarized in Tab. 3. In Figure 14, the response spectrum of the earthquake time history used in the following numerical simulations (recording station AQA) is reported together with the site amplification curve discussed in the preceding paragraph. On the same plot the frequency lines corresponding to the first mode of the building D (3-stories) are located, characterized by a medium level of damage after the earthquake. Similarly for building E (4-stories, collapsed) considering or not considering the presence of the infill walls at the first floor. It is easy to verify that, for building D, the absence of infill walls at the first floor doesn't imply meaningful frequency shift of the first mode toward the peaks of the response spectra (both the accelerometric records and the soil amplification curve). Moreover, the frequency identified by the dynamical tests corresponds to that of the FE model when the infill walls are not considered. This circumstance can be explained by the presence of relevant damage of the infills after the earthquake.
On the contrary, when considering the possible absence (because of expulsion) of the infill walls at the first floor of building A (Fig. 15), a relevant frequency shift is observable toward the maximum amplification zone of the response spectra. Such a circumstance allows a possible explanation concerning the behaviour of the two buildings (D 3-stories and E 4-stories) during the earthquake:

- from the recorded acceleration response spectrum it is possible to observe the presence of strong acceleration amplitudes near the frequency of the first mode of the building E (4-stories, with infills); the site amplification enhances this aspects;
- the infill walls at the first level of building E don't remain within the r.c. frames and a frequency shift takes place; the frequency of the damaged building (after the expulsion of the infill walls) is located in a zone of the response spectrum characterized by a strong amplification factor;
- on the contrary the building D (3-stories), both in the configuration with and without infills, doesn't experience considerable level or strong amplification of the ground motion.

The same analysis can be conducted on the second couple of buildings A (3 stories) and B (4 stories). In this case the results are reported in Figure 15. It is possible to observe how the measured frequency on the survived building (B 4-stories) corresponds to the numerical frequency with intact infills at the first level and is located in a frequency zone of the response spectrum characterized by relatively low acceleration amplitudes. On the contrary, on the building A (3-stories) the absence of infills at ground level was responsible of a frequency shift toward the range characterized by strong earthquake shaking. It is worth noticing that the survived building B, in absence of infills at the ground floor, probably could have experienced a collapse similar to that of building A. This observation leads to the consideration of the importance of the workmanship in the execution of apparently not so important details, like the connection of the infills to the r.c. frame, and especially, to the upper beam.

2.6 Nonlinear analysis

Even if the linear dynamic analysis is useful for the explanation of the observed structural behaviour in occasion of the seismic event, a time history nonlinear numerical integration of the motion equations, can give a deeper insight on the occurred behavior.

The OpenSees code, largely used in the research environment to simulate the behaviour of r.c. buildings under seismic excitation, has been used. The columns and beams sections were modelled using a fibre section model and the infill walls were modelled by means of equivalent trusses with a hysteretic constitutive model (Decanini *et al*., 2004a).

Several runs have been conducted to simulate the behaviour of the buildings in presence or absence of infilled masonry walls at the first level.

The first analysis is concerned with the 4-stories B building having the infill walls at all levels still in place (and working) during and after the earthquake. In Figure 16 an aerial view of the building and the traces on the horizontal motion of the top of the columns at three vertex are reported. The displacement demand, as forecasted by the FE nonlinear model, doesn't exceed 50 mm in every direction, circumstance indicating a quite homogeneous building behaviour with a top drift demand of about 0.5% usually compatible with the capacity of similar existing structures.

In Figure 17, the same kind of plots represents the displacement demand of building A (3 stories) under the earthquake when the infill walls are expelled at the first level. It is quite evident that the condition of *deactivated infilled masonry walls* is the cause of a completely different structural behaviour: a strong displacement demand in the Southwest direction, with amplitude of about 100 mm, is calculated. It is reasonable to argue that the collapse of the building is linked with such a displacement demand significantly higher than the structure capacity.

Figure 18 shows a comparison of time histories (as forecasted by the FE code) for both building A (3-stories, working and not working infill walls at the first level) and building B (4-stories). Again the strong displacement demand, characterizing the time history of the 3-storey building when the *soft storey mechanism* is triggered, is quite evident.

As known, a very meaningful parameter able to synthetically quantify the damage occurred during an earthquake is the inter-storey drift. In Figure19 the drift demand of the three considered cases (building B with working infilled masonry walls and building A in the two different condition of working and deactivated infilled masonry walls) is reported: while in the two cases with working infilled masonry walls, the drift demand is bounded by 1.5% (acceptable value corresponding to a medium level of damage), for building A with *deactivated* infills the drift demand jumps to 3%. This value certainly exceeds the drift capacity and corresponds to a collapse condition.

3. Second case study: The Belvedere Bridge, L'Aquila downtown

A quite interesting application of dynamical tests is the one devoted to damage detection procedures during Structural Health Monitoring programs. Is this the case of the assessment phase after the 6 April 2009 earthquake in L'Aquila, when several structures were closed to their standard use because of reported or even simply supposed damage. Is this the case of Ponte Belvedere, a r.c. bridge constructed around the 60's and located in L'Aquila downtown (see Fig. 20). After the Earthquake, the bridge was immediately closed and so far is still out of work. The Structural Department at University of L'Aquila (DICEAA, former DISAT), in the framework of a scheduled assessment campaign running around 2002-03, among more traditional different activities (visual inspections, material testing and so on), carried out a dynamical test on Belvedere bridge, using a simple 6-instruments layout (see Fig. 21). The extraction of the modal parameters from output only data was carried out by using two different output-only techniques: the Enhanced Frequency Domain Decomposition (EFDD, Brincker *et al.* 2001) in the frequency domain and the data driven Stochastic Subspace Identification (SSI, van Overschee *et al.* 1996); these techniques are available in the commercial program ARTeMIS (SVS 2010).
The dynamical tests permitted the identification, in operational conditions, of the first 8-9 modes working mainly in time domain (see Fig. 22-up).
This first identification was assumed as the *time-zero* identification and it was used to compare different future identifications. Dynamical measures in the real initial condition, immediately after the bridge opening, were, in fact, not available. Few months after the earthquake, on spring 2010, new dynamical tests and a new identification were performed on the bridge and the relevant results were compared to those obtained in 2002. The relevant results are shown in Fig. 22-down. The more clear 2010 stabilization diagram, if compared with the 2002 one, is essentially due to the longer acquisition time used in 2010, when a more clear identification was needed to try to extract the maximum possible information from the measures.
For the sake of completeness additional dynamical tests were accomplished in 2013 an 2014, using a more sophisticated instrumental setup (see Fig. 23 – year 2014).
The cross-comparison of dynamical characteristics of the bridge obtained during the four assessment campaigns are summarized in the following and clearly show that, while between 2002 and 2010 there are quite strong differences in the identified modal characteristics, the same situation doesn't hold when the 2010 campaign is compared with the ones conducted in 2013 and 2014 (see Table 4). This clearly indicates that, during the 6 April 2009 earthquake a sudden drop of modal properties occurred, caused by the occurrence of a structural damage.
In the above-mentioned case the number of instruments is too small to permit the use of one of the damage localization method decribed in the introduction, e.g. change in modal curvature (Wahab *et al*. 1999, Dilena *et al*. 2011), and the localization cannot be done. However in this particular case the identification of damage state is quite clear, even using only the frequency-lowering criterion.

4. Conclusions

Two different case studies of structures in L'Aquila after the 6 April 2009 earthquake are considered in the paper. In particular, the work is concerned with the use of *in situ* dynamical test, reputed of great help in post-earthquake assessment campaigns. Among several other possible and effective applications, dynamical tests could be conveniently used:

- *Case study 1*- To update FE models used in numerical simulations necessary to understand, quantify and evaluate the main characteristics of the seismic response. As a first case study, the mentioned methodology was applied to determine the triggering effect for a soft storey collapse mechanism characterizing the response of some buildings in Pettino (AQ). The INGV made an extended *in situ* campaign finalized to the determination of characteristics of the local motion of the soil and of site effects. Such informations were used to understand the complex soil-structure interaction mechanism and to quantify the local amplification factor. The modal properties of the survived buildings were identified from *in situ* tests using output only algorithms applied to acceleration time histories. The combined use of experimental tests and numerical analysis permitted a convincing interpretation of the collapse evolution. Even the use of simple linear models, taking into account the contribution in stiffness of the infilled masonry walls, permitted a coherent interpretation of observed phenomena. By using more accurate nonlinear f.e. models it is possible to reconstruct the evolution of the damage obtaining a clearer unfolding of the collapse mechanism. The key role of the infill walls has been enlightened, putting into evidence how a good construction of such non-structural elements could dramatically influence the global structural behaviour under a medium-strong earthquake.
- *Case study 2* - To ascertain the presence of damage in structures, when the visual inspections are not able to correctly determine and quantify the damage level. In the second case study, pertaining the Belvedere Bridge in L'Aquila downtown, a 4-5% frequency-shift of the first identified modes is the first and quite clear indicator that a stiffness reduction of the deck beams was produced by the strong motion undergone by the bridge during the earthquake.

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Building	Number of floors	Damage
A	3	Collapse
B	4	Medium
D	3	Medium-high
E	4	Collapse

Table 1: Observed damage in the analysed buildings (buildings are tagged in Fig. 1).

Measurement	Data Logger	A/D converter	S rate	Sensor	Sensitivity	Band	Installation
Ambient vibration	Reftek 130	24 bits	100 s/s	Lennartz LE3D-5s	400 V/m/s	0.2 – 40 Hz	Free field
Earthquakes	Quanterra Q330	24 bits	200 s/s	Lennartz LE3D-5s	400 V/m/s	0.2 – 40 Hz	Free field - building

Table 2: Characteristics of the instruments used for the measurements of the soil and of the building.

Mode	Bldg. D with infills	Bldg. D w/o infills at first level	Bldg. E with infills	Bldg. E w/o infills at first level
1	.280 s 3.57 Hz	.319 s 3.13 Hz	.350 s 2.86 Hz	.394 s 2.54 Hz
2	.265 s 3.77 Hz	.318 s 3.14 Hz	.326 s 3.07 Hz	.385 s 2.60 Hz
3	.229 s 4.37 Hz	.270 s 3.70 Hz	.287 s 3.48 Hz	.325 s 3.08 Hz

Table 3: Effects, in terms of modal frequency, of the presence or absence of infills at first level in the FE models of buildings D and E.

Mode	Frequency [Hz] 2002	Frequency [Hz] 2010 and – ref 2002	Frequency [Hz] 2014 and – ref 2010
1st flex - symmetric	3.15	3.05 (- 3.5%)	3.05 (0%)
1st torsional	3.92	3.83 (-2.3%)	3.82 (- <1%)
2nd flex - anti-symmetric	6.49	6.45 (-0.7%)	6.42 (- <1%)
2nd torsional	8.20	7.78 (- <5.8%)	7.75 (- <1%)
3th flex - anti-symmetric	9.93	9.54 (- <4%)	9.53 (- <1%)
3th torsional	10.86	10.50 (- <3.9%)	10.45 (- <1%)
4th flex - symmetric	14.65	13.96 (- <4.9%)	13.93 (- <1%)

Table 4: Belvedere bridge, identified frequencies years 2002, 2010, 2014

Figure 1: Soft story collapses observed after the 6 April 2009 L'Aquila earthquake

Figure 2: Aerial view of the area with the investigated buildings. Red arrows point to buildings collapsed (A and E) due to soft storey mechanism.

Figure 3: Building A before (a) and after (b, c, d, e) the seismic event

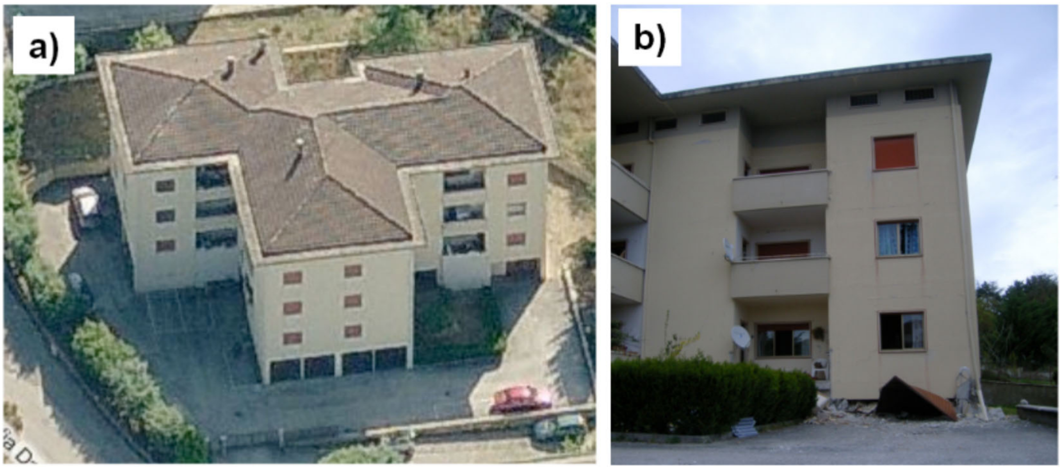

Figure 4: Building E before (a) and after (b) the seismic event

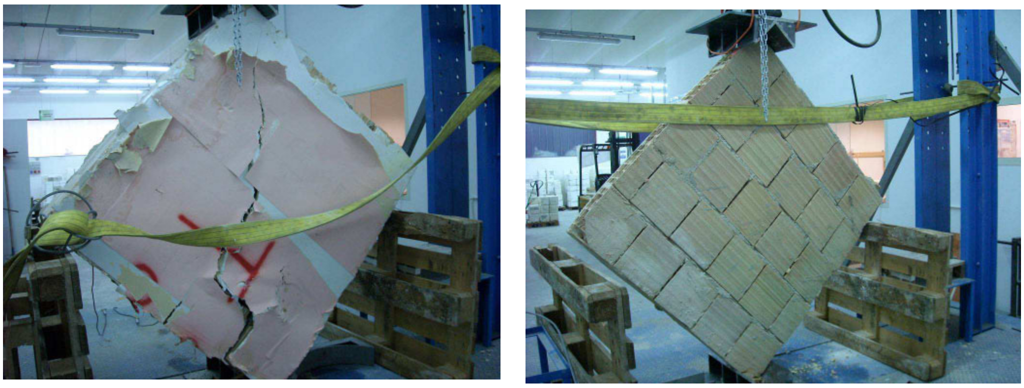

Figure 5: Diagonal compression tests on specimens of infill walls extracted from the collapsed buildings

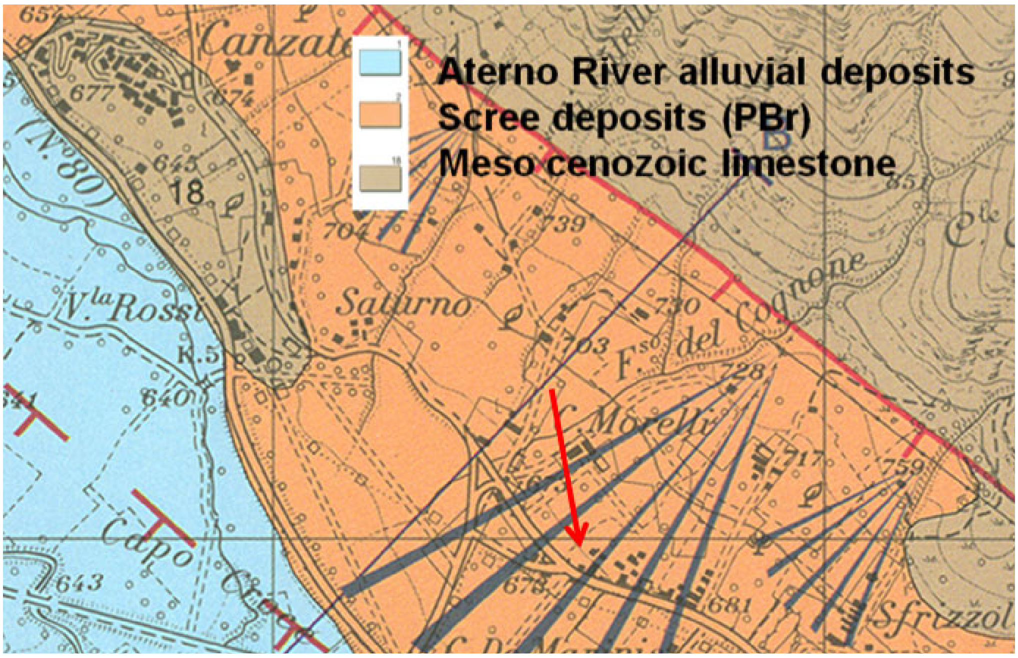

Figure 6: Geological map of the investigated area; the arrow points toward the location of the case study buildings

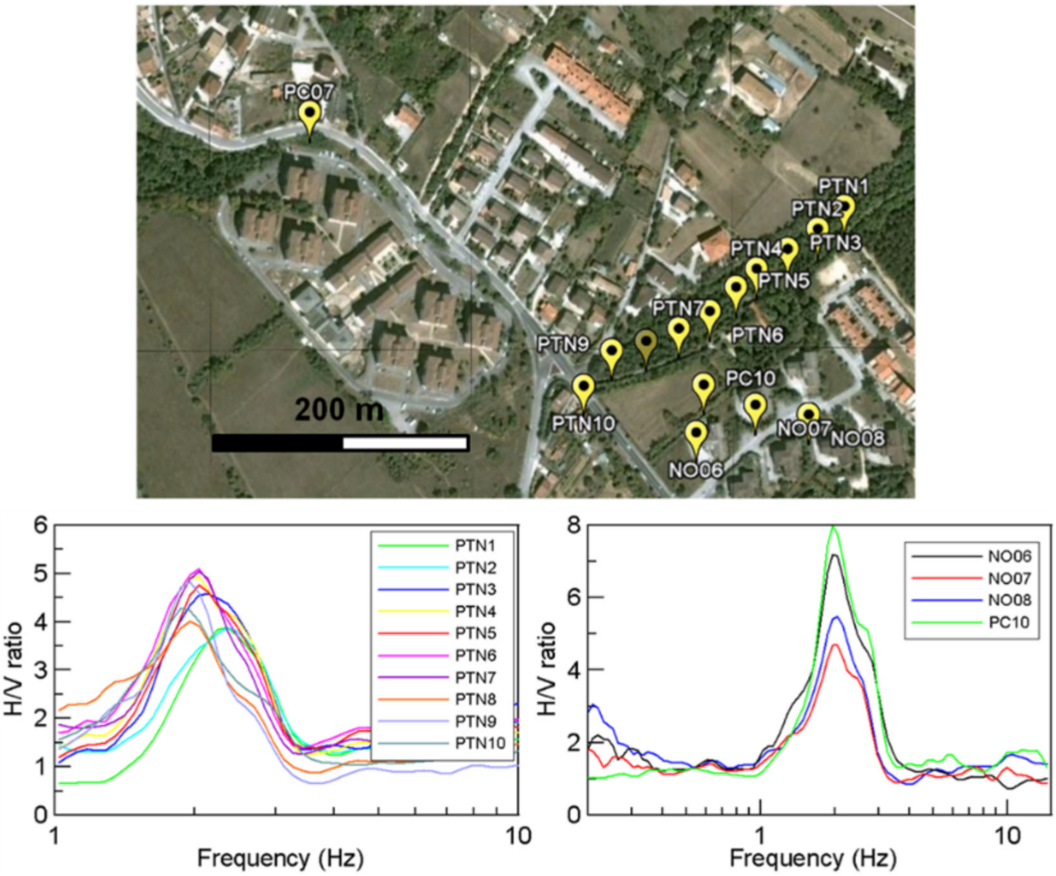

Figure 7: Location of seismic measuring points (top panel) along with HVNSR results (bottom panels). Building A is located between measuring points NO06 and PC10.

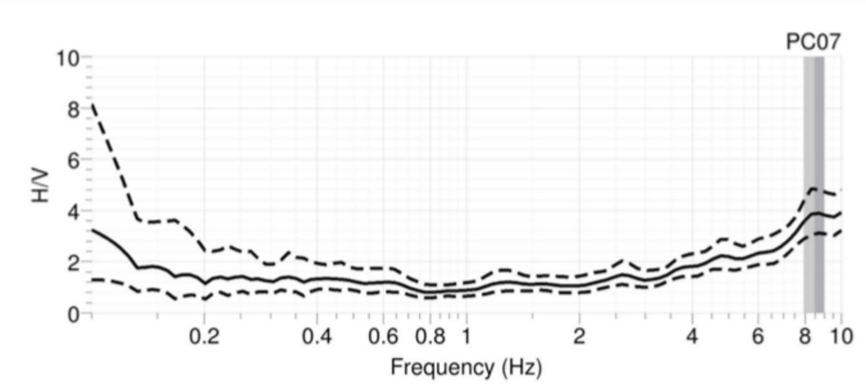

Figure 8: Ambient noise HV spectral ratio for the reference site PC07

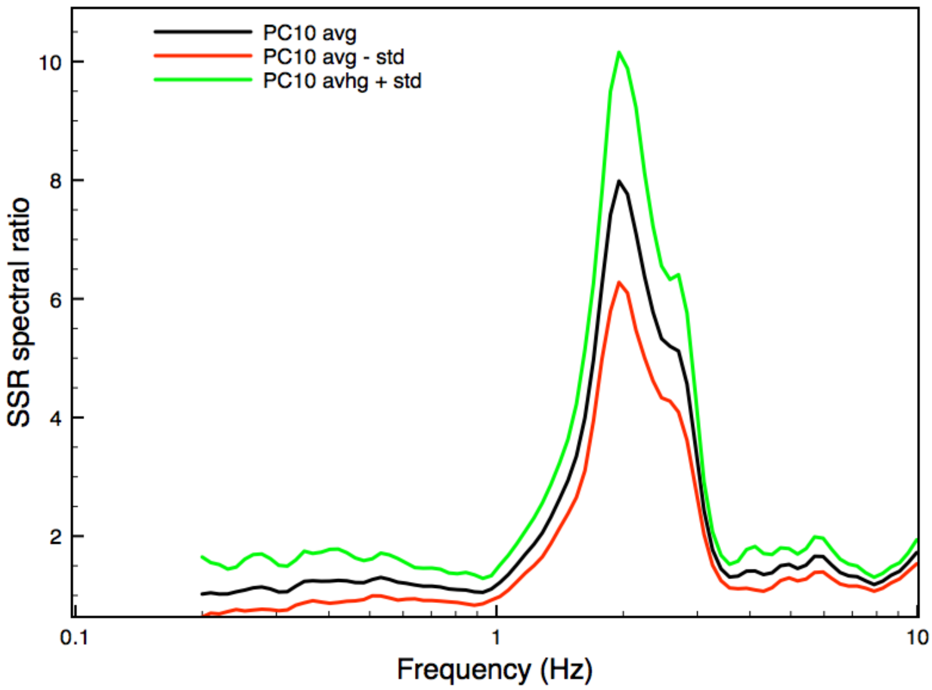

Figure 9: SSR spectral ratio for PC10 station. Middle line (black curve) represents the mean value; standard deviation is also shown (upper and bottom lines).

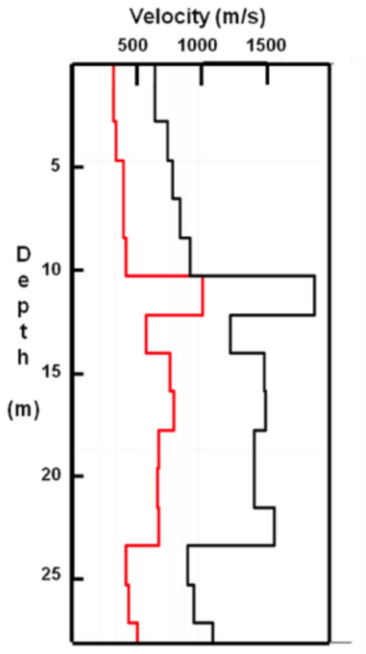

Figure 10: Downhole measurements performed in Via Dante Alghieri. Red line refers to Vs profile, black line to Vp profile

Figure 11: a phase of the installation of the instruments from outside of the buildings using a Fire Brigades truck

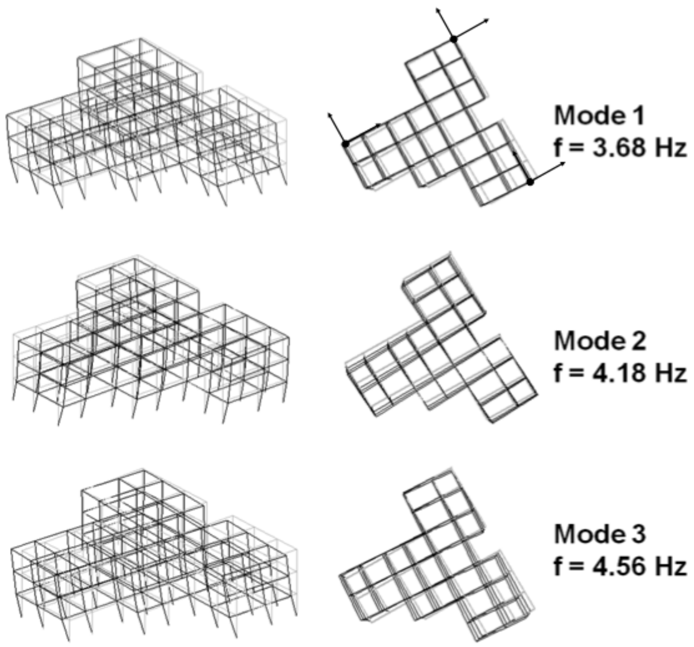

Figure 12: Modal shapes and fundamental frequencies of building B (four stories); the roof level was not monitored; points and arrows show the locations and directions in plan of the 9 triaxial sensors deployed at first, second and third level of the building.

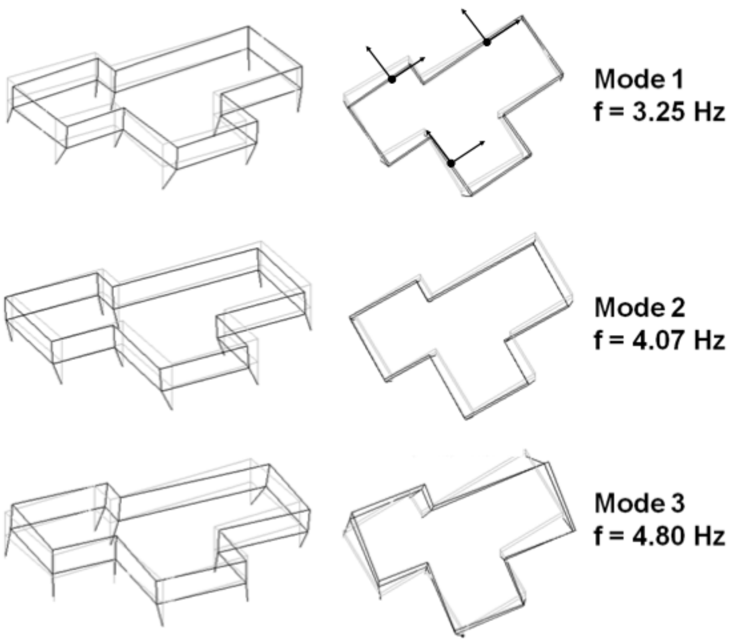

Figure 13: Modal shapes and fundamental frequencies of building D (three stories); the roof level was not monitored; points and arrows show the locations and directions in plan of the 6 triaxial sensors deployed at first and second level of the building.

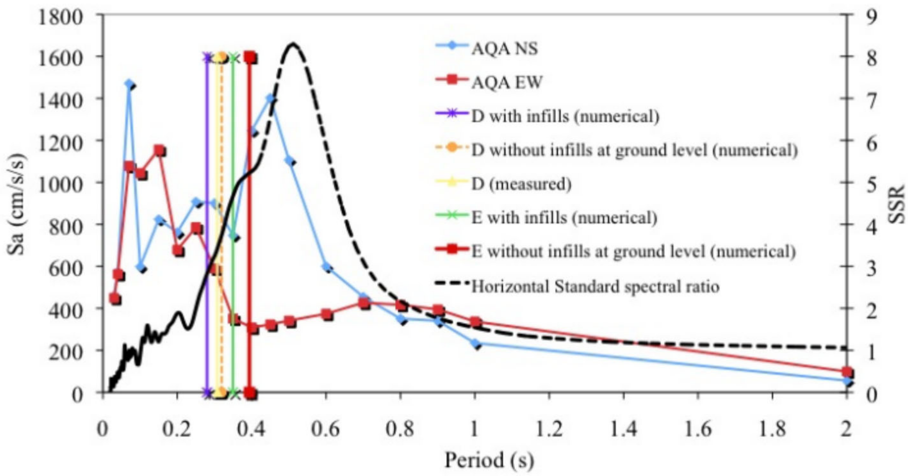

Figure 14: Acceleration response spectra relevant for the analysis and fundamental periods of buildings D and E with different structural configurations of the infill walls.

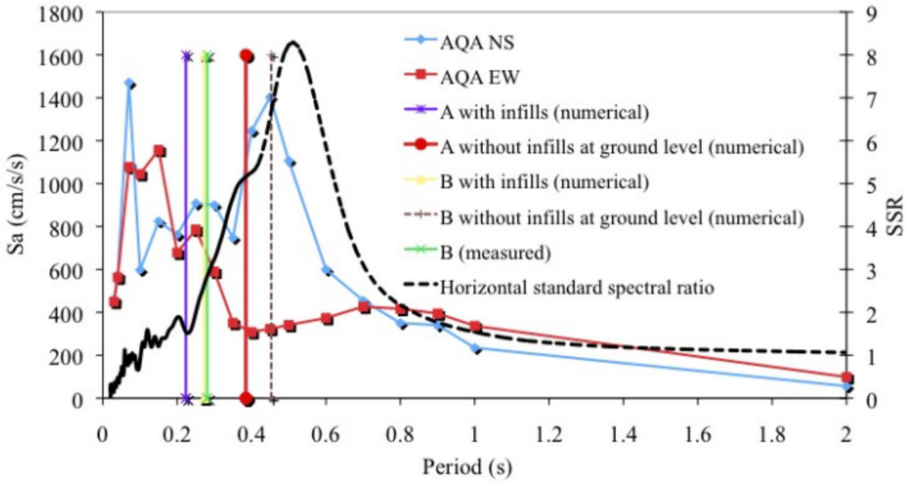

Figure 15: Acceleration response spectra relevant for the analysis and fundamental periods of buildings A and B with different structural configurations of the infill walls.

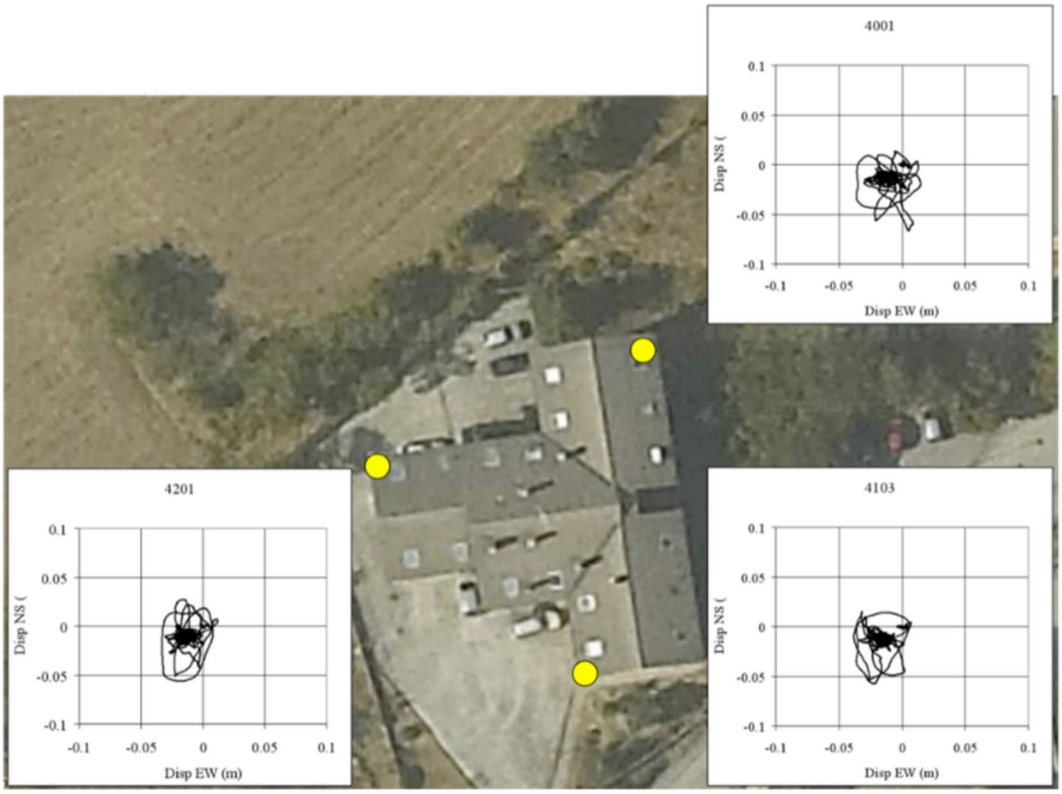

Figure 16: Displacement time histories obtained with the nonlinear model for three points of the roof of building B (4 stories).

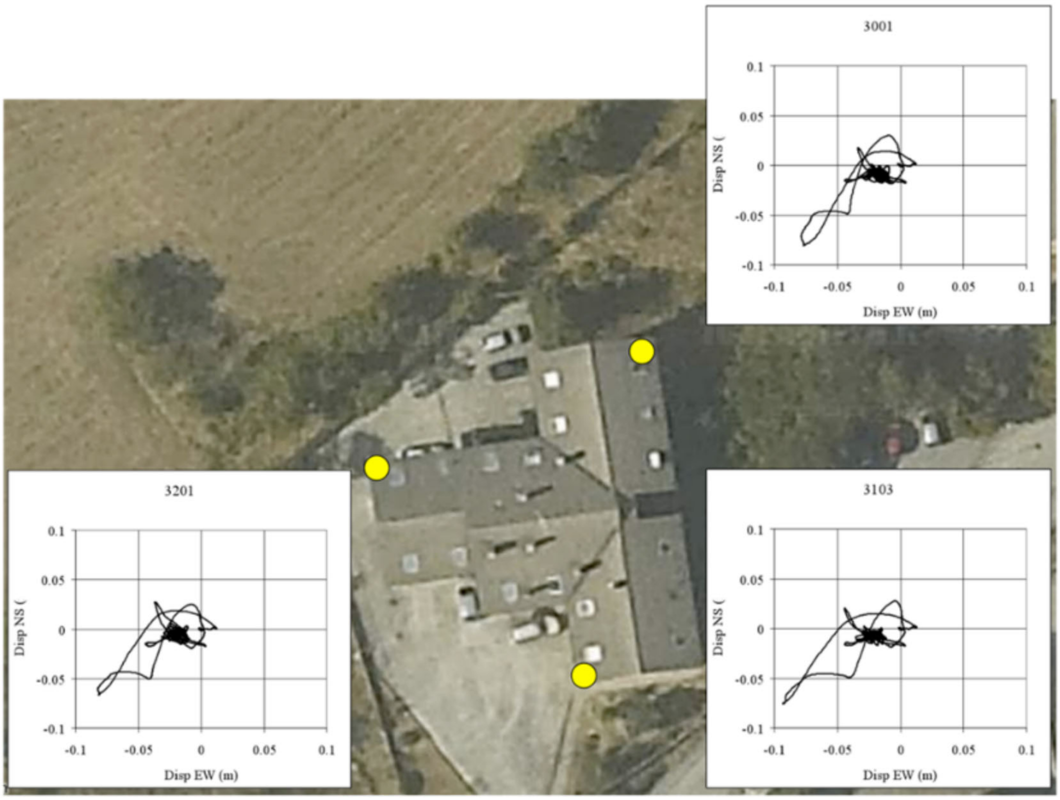

Figure 17: Displacement time histories obtained with the nonlinear model for three points on the top of building A (4 stories) without infill walls at ground level.

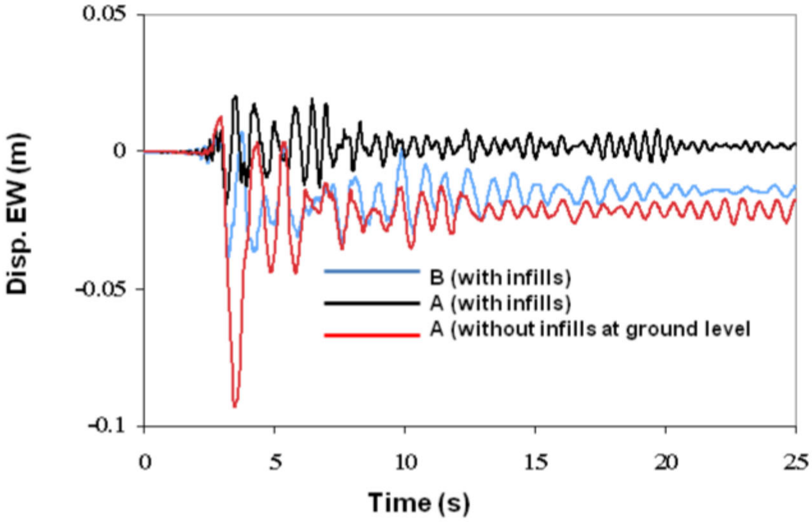

Figure 18: Displacement time histories obtained with the nonlinear model on the top of buildings A and B with different configurations of the infill walls

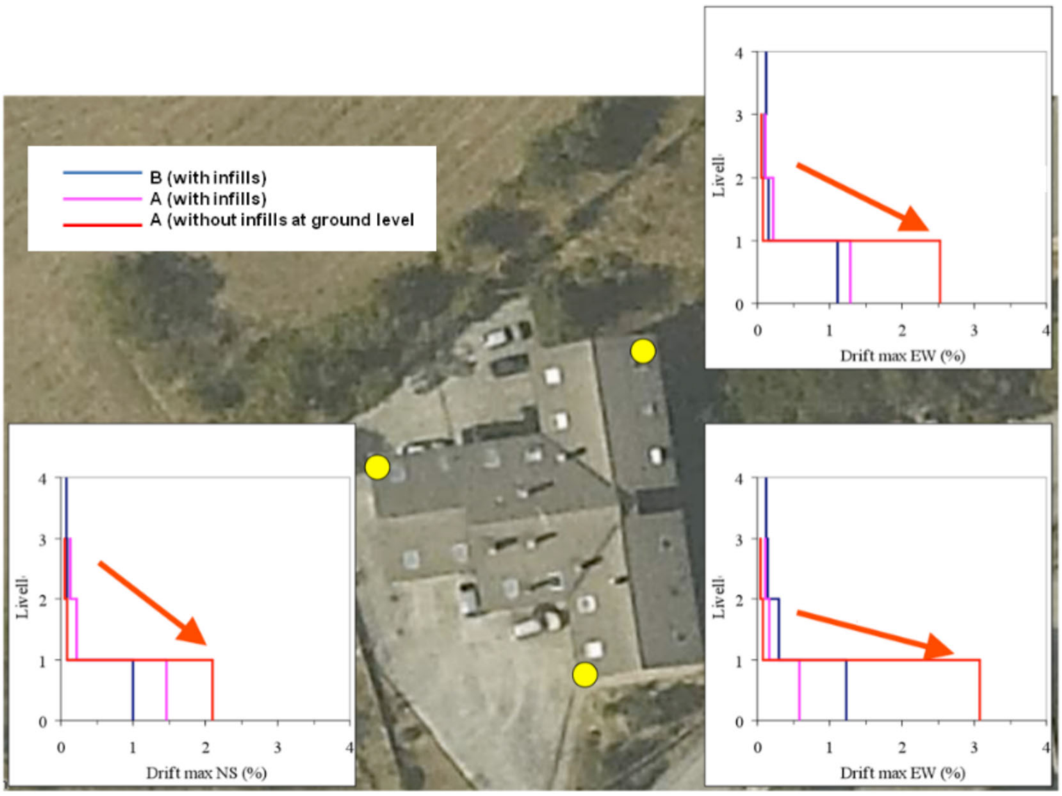

Figure 19: Maximum interstorey drifts obtained with the nonlinear analysis of buildings A and B with different configurations of the infill walls.

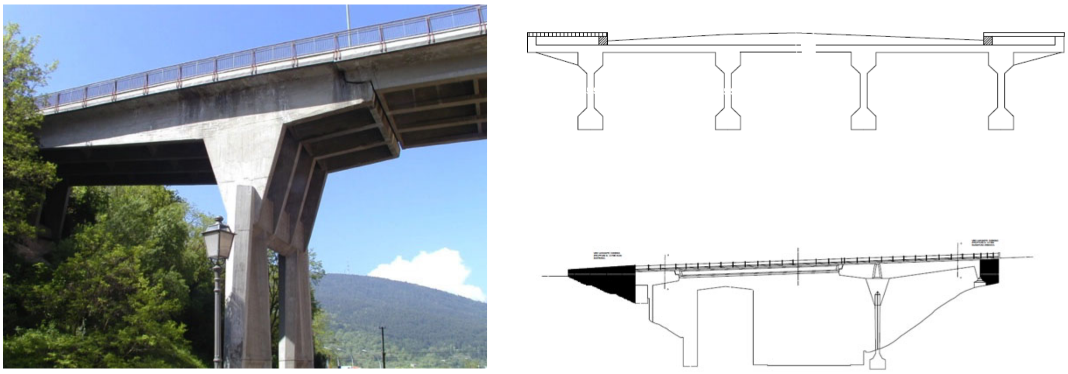

Figure 20: The Belvedere Bridge in L'Aquila downtown (left), technical drawings (right).

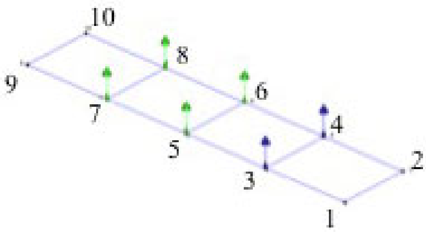

Figure 21: The 2002 6-axis instrumental setup on Belvedere bridge.

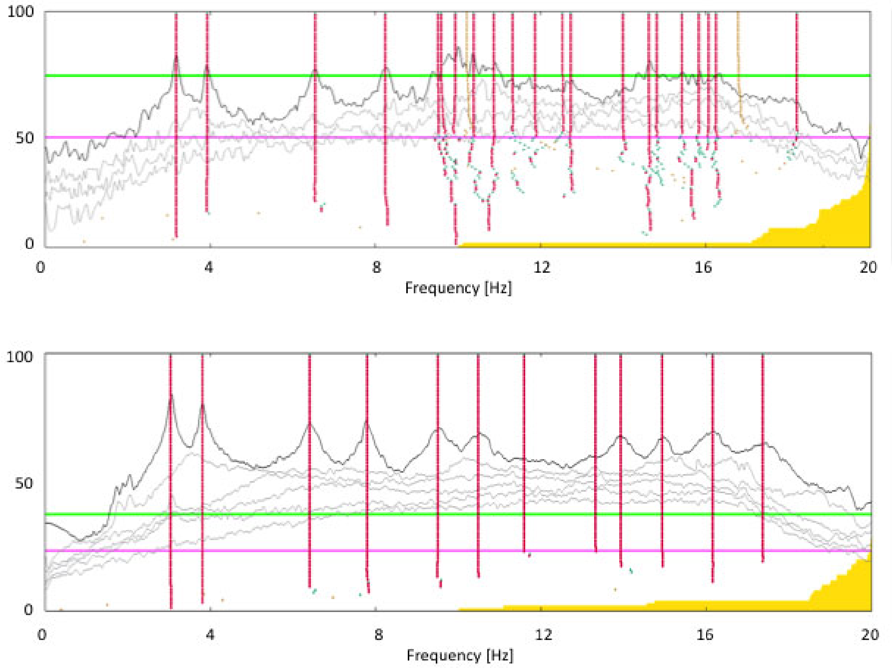

Figure 22: The 2002 (up) and 2010 (down) identifications in time domain: stabilization diagrams.

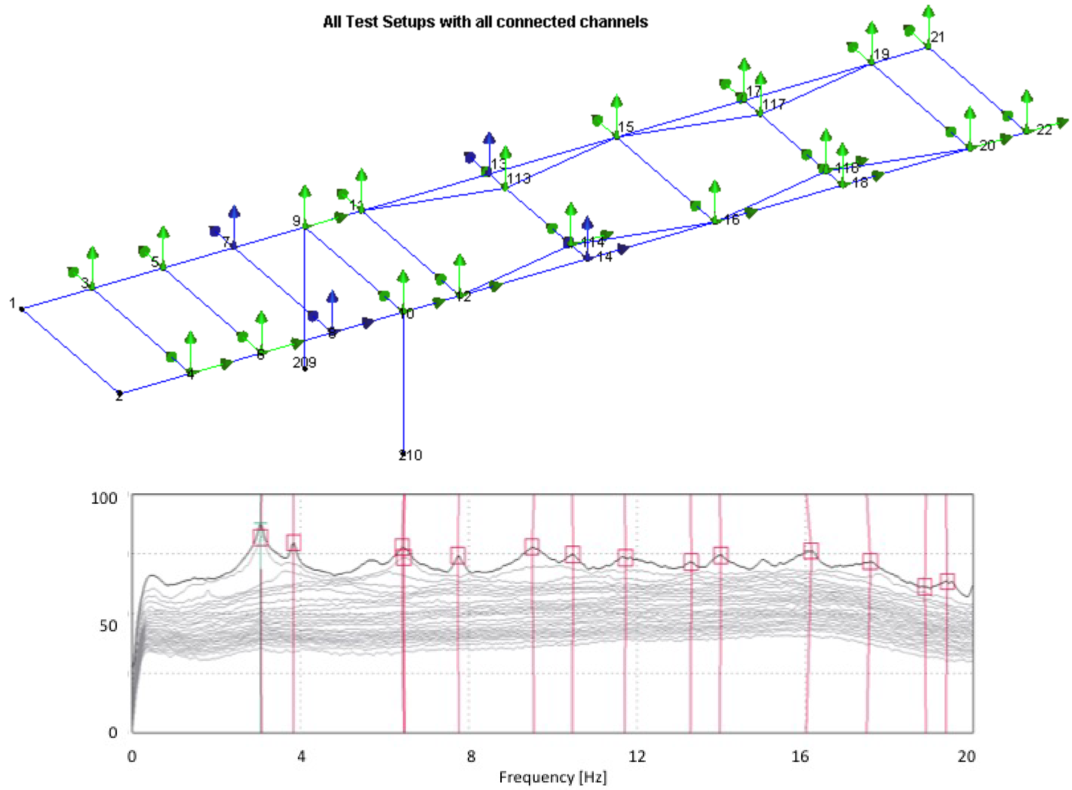

Figure 23: The 2014 instrumentation setup (up) and the identified modes (down).