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Experimental behaviour of an industrial building during the 2016-2017 Central Italy seismic sequence

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ABSTRACT
During seismic events in Emilia Romagna in May 2012, several industrial buildings collapsed or were seriously damaged. Most of them were built by assembling pre-cast concrete elements without taking account of seismic actions or according to old codes. Moreover, the steel shelving underwent instability phenomena. From that experience, ENEA and Regione Umbria, in the context of a large research project aimed at monitoring the new Center of Civil Protection Department (DPC) in Foligno, included an industrial building among the instrumented structures. The permanent accelerometric network, installed on an intermediate frame, consists of a K2 Kinemetrics acquisition system, having an internal three-axial accelerometer, and nine uniaxial FBA accelerometers connected to the acquisition system through cables. The monitoring system allowed recording the response of building to the seismic sequence which struck the Center of Italy in 2016-2017. Compared to dynamic characteristics obtained in the ambient vibration tests, significant variations were found due to the increase of the input energy, measured in terms of Arias intensity I_A.

1 INTRODUCTION

On May 20th, 2012, at about 4.00 a.m., a Mw=6.09 earthquake struck the Emilian Po valley. The epicenter was between the cities of Modena and Ferrara. The days after, a number of events affected the same area. The most violent of them, with a magnitude Mw=5.9, occurred on May 29th (Paolini et al., 2012).

After inspections coordinated by the Civil Protection Department (DPC), the total condemned buildings, classified as E or F according to AEDES forms, were more than 16000 and were distributed among three regions: Emilia Romagna, Lombardia and Veneto.

The earthquake struck an area classified as low to moderate seismicity area. However, this hazard level had been formally recognized only a few years before, with the revision of Italian seismic classification, started in 2003. As a consequence, many buildings, although recently built, didn't disclose anti-seismic expedients, which were not required at the time of their construction.

Among them, the industrial buildings were the ones mainly damaged. They were built by assembling precast concrete elements and, therefore, were often devoid of structural continuity and robustness. Indeed, joints without mechanical connections couldn't guarantee the internal force transmission in dynamic conditions, relying on friction for the absorption of horizontal forces, according to a mechanism that is no longer allowed by current codes.

Nevertheless, it is worth reminding that only with the D.M. LL.PP. of December 3rd, 1987, it was recognized that friction connections make the structures labile and subject to lose of vertical supports. However, since the ban concerned only seismic zones, as reported by the classification in force at that time, buildings with this structural deficiency are widespread in some parts of Italy.

As a consequence, the law concerning urgent actions in favour of people affected by the 2012 Emilia Romagna earthquake, stated that the Certificate of Occupancy could be issued only when none of the specified structural deficiencies had been found (Law No. 122, 2012, Gazzetta Ufficiale Serie Generale No. 180, August 3rd, 2012).

In this context the Working Group “Agibilità Sismica dei Capannoni Industriali” drawn up the guidelines for industrial building. These guidelines, although not mandatory, outlined the path to follow, in compliance with existing codes, in order to combine the need for safety in the short medium and long term (DPC, 2012).

Poor details and inadequate connections between structural elements were the main causes of damage to industrial buildings also during other earthquakes, such the 7.4 magnitude earthquake that struck Kokaeli, Turkey, in 1999 (Posada et al., 2002, Senel et al., 2012).

However, there are examples of precast prestressed concrete buildings, which performed remarkably well during a seismic shake, as described by Muguruma et al. (1995) with reference to the 1995 Kobe earthquake (Mw=6.9). Those buildings were designed according to more recent codes, with regular geometry and high quality construction details.

Against this background, within a project organized by ENEA and Regione Umbria for the seismic monitoring of structures at the Civil Protection Centre in Foligno, also an industrial building was included. This structure has already been the subject of a previous study (Bongiovanni et al., 2017) where its dynamic characterization was performed and its responses to several low-energy seismic events, from November 2014 to September 2015, were analyzed.

The results are here supplemented by the analysis of the seismic events recorded by the same permanent instrumentation during the 2016-2017 Central Italy sequence.

2 DESCRIPTION OF THE BUILDING AND OF THE ACCELEROMETRIC NETWORK

The structure under study is a one floor industrial building (Figure 1), made of precast concrete elements, whose layout is shown in Figure 2.

The carrying structure of the almost square plan building, 60.4x59.8 m in plan with a height of 8.7 m, consists of four frames, each of which composed in turn of five columns and beams. The frames are spaced each other of 19.79 m and have equal spans of 14.7 m. In Figure 2Figure 2, where x and y are the transversal and longitudinal directions, respectively, Cij is the label of column j of frame Fi (with i = 1, 2, 3, 4 and j = 1, 2, 3, 4, 5).

Figure 1. View of the industrial building of the Civil Protection Centre at Foligno, Italy.

As typical for this kind of buildings, each column is placed in a pocked plinth and shaped properly at its top in order to house the converging beams. These are connected to the column only by means of weak connector pins (Figure 3). Also the covering beams are precast prestressed concrete beams.

The foundation plinths are linked to each other through reinforced concrete beams, which also serve as supports for the lightweight concrete panels. For the same reason, additional intermediate plinths are placed between two adjacent perimeter columns.

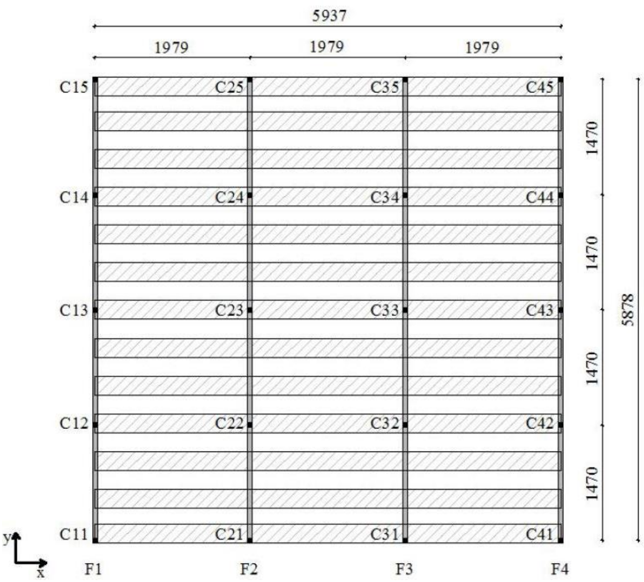


Figure 2. Building layout.

Dynamic instrumentation consists of a K2 acquisition system, having an internal three-axial accelerometer, and nine external uni-axial accelerometers. Sensors are deployed on the frame F2 as shown in Figure 4. The horizontal arrows (A01, A04, A05, A06, A09, A10, A11, A12) and the points (A03, A07, A08) indicate accelerometers in the longitudinal and transversal directions, respectively, whereas A02 indicates the only accelerometer in the vertical direction. The sensor at the foundation plinth of column C23, including A01, A02 and A03, is the already mentioned three-axial sensor.

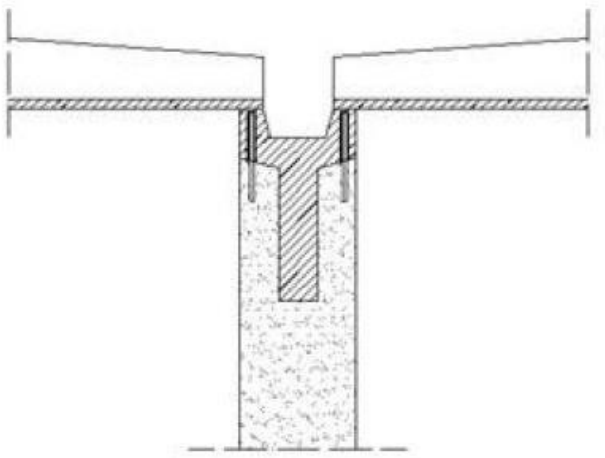


Figure 3. Detail of beam-column connection.

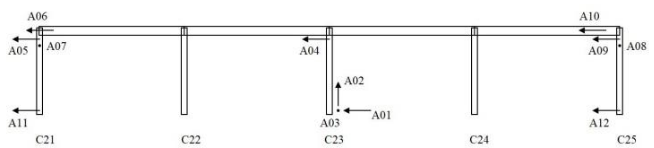


Figure 4. Accelerometers deployment on frame F2.

When dynamic instrumentation were placed for the first time, in 2014, also a temporary seismometer array, composed of 24 Kinemetrics SS-1 connected to a Granite acquisition system, was installed in order to test the correct working of the accelerometer network. For this reason, ambient vibrations were recorded, lasting 20 min and with a sampling interval $\Delta t = 0.005$ s, by both accelerometer and seismometer arrays. A good agreement was obtained at the top of the columns, whereas some differences at their bases were attributed to the distances between the two types of sensors.

From the ambient vibration records obtained, the first structural frequencies were determined by means of spectral analysis. These were equal to $f_{1x} = 1.92$ Hz ($T_{1x} = 0.52$ s) and $f_{1y} = 2.05$ Hz ($T_{1y} = 0.49$ s) in transversal and in longitudinal direction, respectively (Figure 5). Even if some variations of resonance frequencies came out during the analysis of the building response to the recorded events, after each shake the structure recovered its original values demonstrating that it had not been damaged. Indeed, natural frequencies are sensitive indicators of the structural integrity and their periodic measurement can be used to monitor the structural condition (Salawu, 1997).

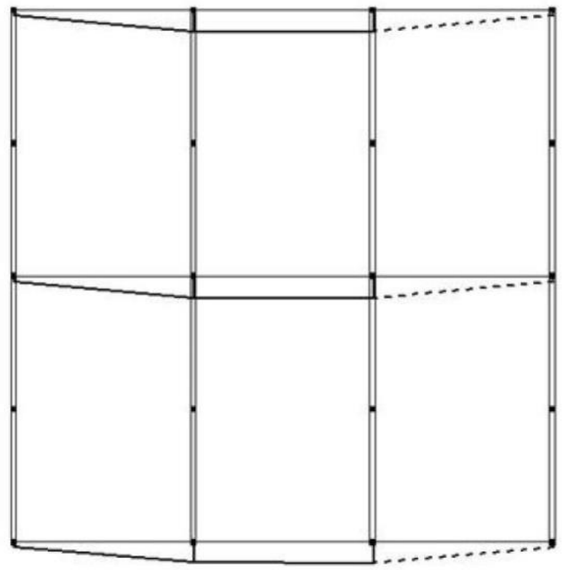


Figure 5. First longitudinal mode shape ($f_{1y} = 2.05$ Hz).

3 EXPERIMENTAL DYNAMIC ANALYSIS

The 2016-2017 seismic sequence that struck the Center of Italy was characterized by high magnitude events and interested significantly four Regions along the Apennines chain.

In more detail, the earthquakes recorded in this area by the INGV National Seismic Network since August 24th, 2016, to January 2017, were many tens of thousands. Among those, 67 events had a magnitude $4.0 \leq M < 5.0$, whereas 9 events had a magnitude ≥ 5.0.

Figure 6 shows the map of epicentres of earthquakes with $M \geq 4$, updated to January 2017, detected by ENEA instrumentation located at the Civil Protection Centre at Foligno.

All these seismic events were recorded, so the monitoring system installed by ENEA allowed to have a valuable database of the response of the building to the seismic sequence.

First of all, in order to assess the signal energy content at the site of the building, the events were classified in terms of the Arias intensity (Arias, 1970):

$$I_A = \frac{\pi}{2g}\int_0^{t_0}\left(a_x^2 + a_y^2 + a_z^2\right)dt \qquad (1)$$

In equation (1), a_x, a_y and a_z are the ground acceleration components recorded by the sensors at the foundation plinth of column C23.

For events with magnitude $M \geq 5$, the main characteristics, i.e., the date, the magnitude M, the epicenter distance, the Peak Ground Accelerations in the three directions and the Arias intensity I_A, are summarized in Table 1.

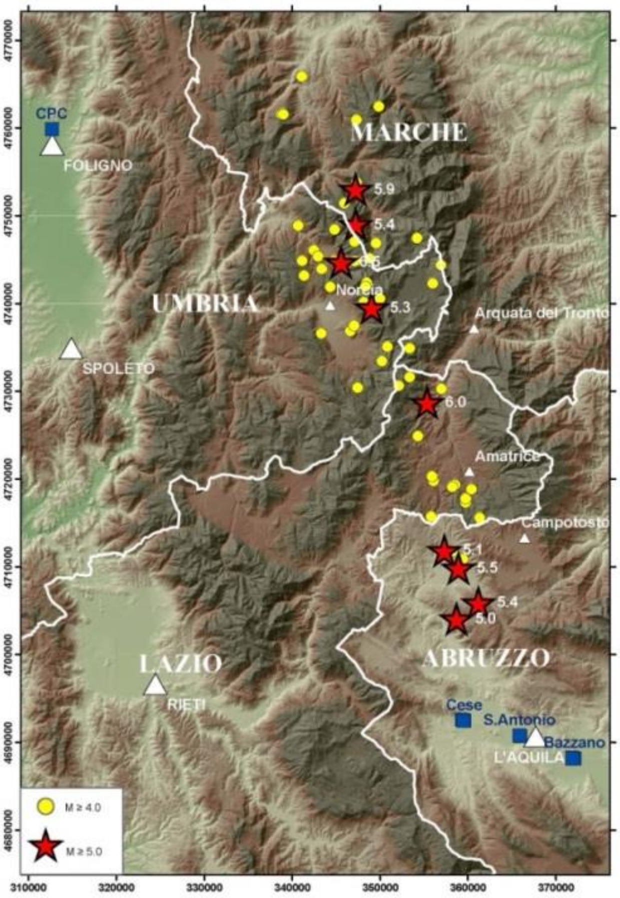


Figure 6. Seismic sequence in Central Italy: epicentres of the earthquakes with M ≥ 4 detected by ENEA in Foligno.

Table 1. Seismic events with Mw ≥ 5 recorded at Foligno during the 2016-2017 Central Italy Earthquake.

Earth	Mw	Epic. Dist. (km)	PGA (g) A01	A02	A03	I_A (cm/s)
Aug 24	6.0	53	.084	.028	.071	13.0
Aug 24	5.3	41	.025	.011	.028	2.0
Oct 26	5.4	36	.093	.022	.067	7.2
Oct 26	5.9	32	.068	.025	.076	12.5
Oct 30	6.5	36	.115	.052	.207	53.7
Jan 18	5.1	66	.038	.010	.035	1.7
Jan 18	5.5	68	.022	.006	.019	1.1
Jan 18	5.4	70	.015	.006	.010	0.4
Jan 18	5.0	72	.022	.004	.018	0.5

4 THE OCTOBER 30TH, 2016, EARTHQUAKE

Two months after the first violent earthquake, whose epicentre had been very close to the little town of Accumoli, on October 30th a 6.5 magnitude event was recorded with epicentre about five kilometres from the town of Norcia. It was the strongest event of the entire 2016-2017 seismic sequence and also the strongest event recorded in Italy after the 1980 Irpinia Earthquake (M = 6.9).

In Figure 7 the shake map elaborated by INGV, using the conversion relations by Faenza and Michelini (2010, 2011) between ground motion parameters and the MCS intensity scale, is shown. Red triangles indicate the accelerometric and velocimetric stations of INGV, while blue triangles indicate the accelerometric stations of DPC. Among the latter, the one that is circled indicates the Foligno station, located 36 km far from the epicenter. According to the map, in this area, the perceived shaking was from "strong" to "severe", corresponding to an Instrumental Intensity from VI to VIII.

In Figure 8 the horizontal response spectra obtained from recordings of A01 and A03 are superimposed to the ones provided by the Italian technical code for a 5%, 10% and 63% probability of exceedance in the reference period, which has been assumed equal to 100 years, the building being for strategic use. The values of probability of exceedance correspond to the collapse limit state (SLC), the life safeguard limit state (SLV) and the damage limit state (SLD), respectively.

As one can see, both the acceleration components exceed the SLD spectral values in a narrow range of the period. The transversal component (A03) exceeds also the SLV spectral values. In both cases the peak corresponds approximately to a period T = 0.15 s.

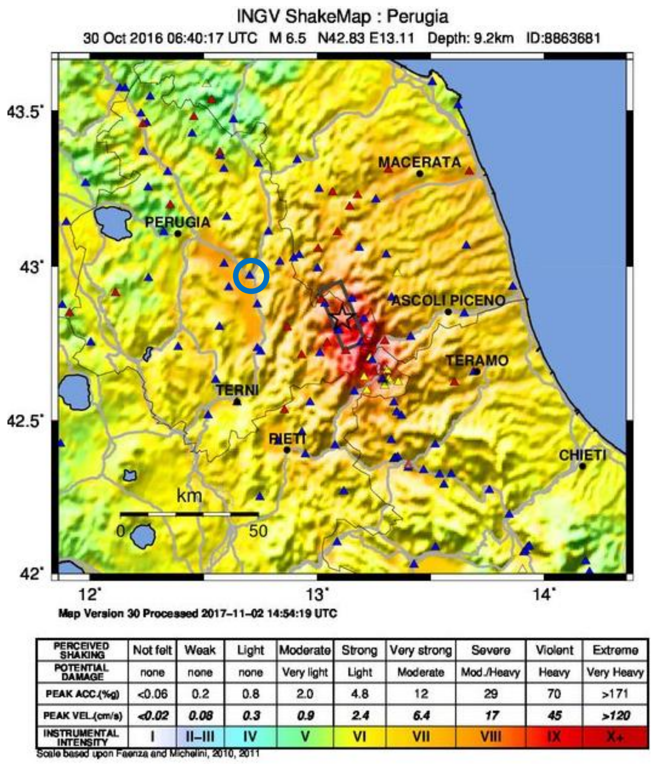


Figure 7. October 30th, 2016, earthquake: Shake Map (http://shakemap.rm.ingv.it/shake/8863681/intensity.html, INGV).

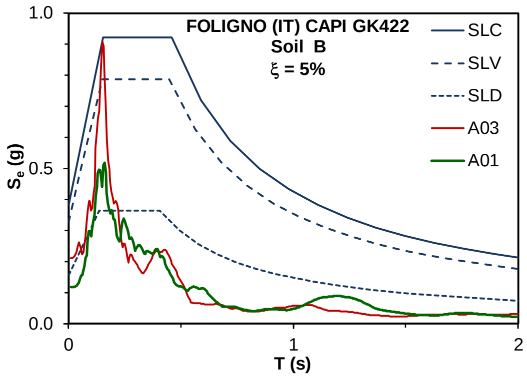
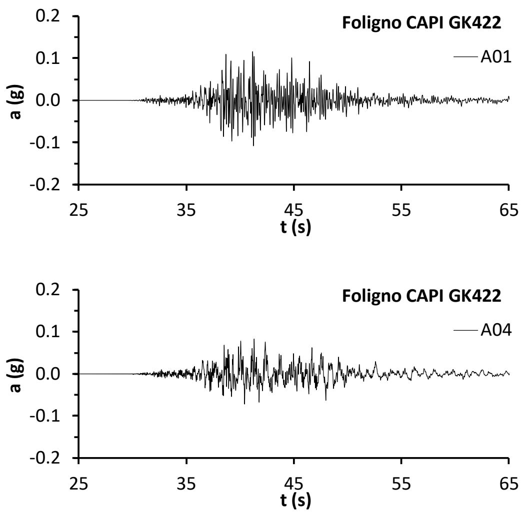


Figure 8. October 30th, 2016, earthquake: horizontal response spectra compared with the elastic response spectra of the code for different values of probability of exceedance in the reference period, equal to 100 years.

In Figure 9 the horizontal time histories, recorded in the longitudinal direction at the foundation plinth and the top of column C23, respectively, are plotted. No amplification in terms of peak acceleration can be seen. This occurrence is consistent with the fact that the building has a first longitudinal period (T = 0.49 s) which corresponds to a maximum acceleration approximately equal to PGA.



(a)

(b)

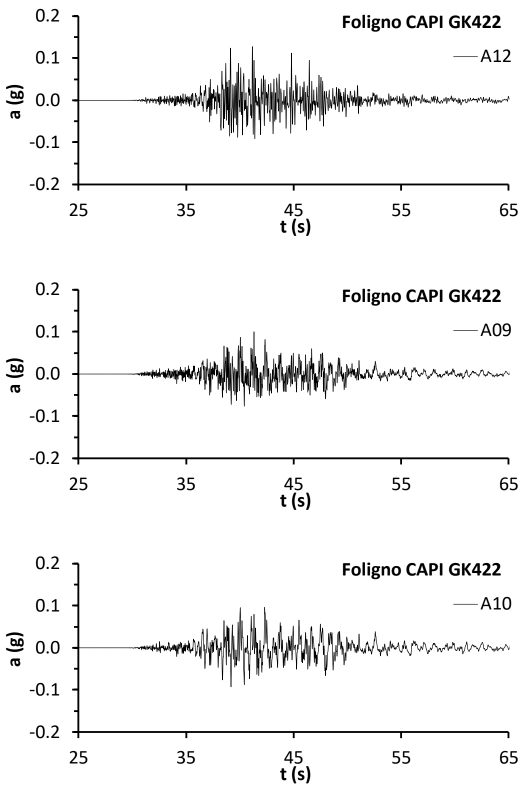
Figure 9. October 30th, 2016, earthquake: recordings (a) at the at the foundation plinth and (b) at the top of column C23.

In Figure 10 the recordings of sensors in the longitudinal direction along the column C25 (A12, A09) and on the corresponding beam (A10) are shown. Also in this case, amplifications from the base to the top of the column were not found. Furthermore, a comparable signal amplitude on the beam and on the column was found, even though a difference in the frequency content was already apparent in the time histories. This is indicative of the fact that a not rigid constraint is between them.

Finally, in Figure 11, the recordings obtained in the transversal direction at the foundation plinth and the tops of the two external columns of frame F2, C21 and C25, respectively, are plotted. In this case, an amplification in terms of peak acceleration from the base (A03) to the tops (A07, A08) is apparent.

The analysis of the Fourier's spectra pointed out the following results:

– A significant change in the frequency content is apparent in the longitudinal direction (Figures 12a and b). While the recordings obtained at the feet of the columns show an important content between 4 and 8 Hz, in the recordings at the top of the same column the resonance frequencies of the frame are apparent. These are between 1 and 2 Hz. The highest frequencies are present also at the top, but they are not amplified with respect to the recordings at the feet.
– The recording at the base of column C23 in the transversal direction (A03, Figure 12c) presents significant amplitudes between 4 and 8 Hz. This contents is amplified in the structure. Instead, no amplification can be seen at the lower frequencies in the transversal direction.



(a)

(b)

(c)

Figure 10. October 30th, 2016, earthquake: recordings (a) at the foot of column C25 (b) at the top of column C25 and on the corresponding beam.

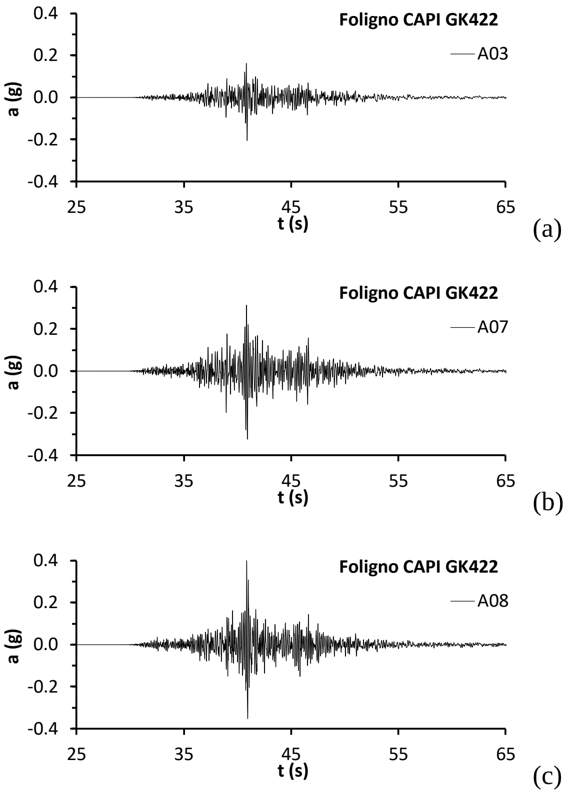


Figure 11. October 30th, 2016, earthquake: recordings at the foot of column C23, at the top of columns C21 and C25, respectively, in the transversal direction.

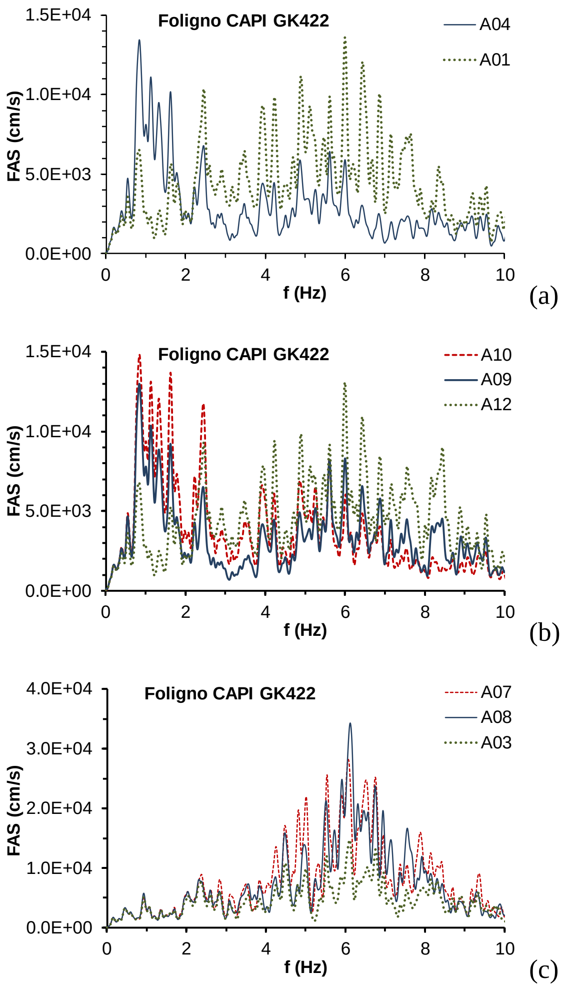


Figure 12. October 30th, 2016, earthquake: Fourier's spectra.

After the correction for instrument response, data were bandpass filtered and double integrated in frequency domain, in order to obtain the displacement time histories.

In Figure 13 (where 0° and 90° correspond to the transversal x and longuitudinal y direction, respectively) the absolute and the relative particle motions at the top of column C25 are plotted. The absolute motion (Figures 13a and b) is quite irregular without a preferential direction and with a maximum value of about 3 cm. In Figures 13c and 13d, the particle motion of A09-A08, relative to the motion of A01-A03 is shown. In this case a preferential direction can be seen, which corresponds to the longitudinal direction of the frame F2. The relative displacements are much lower than the absolute ones.

This occurrence is related to the surface waves, whose components of frequencies lower than 0.6 Hz are transmitted to the structure without any amplification. The effect is the presence of displacements almost constant along the height and equal to about 3 cm. This behaviour is not present in the relative displacement diagrams. Anyway, relative displacements can be seen also in the transversal direction.

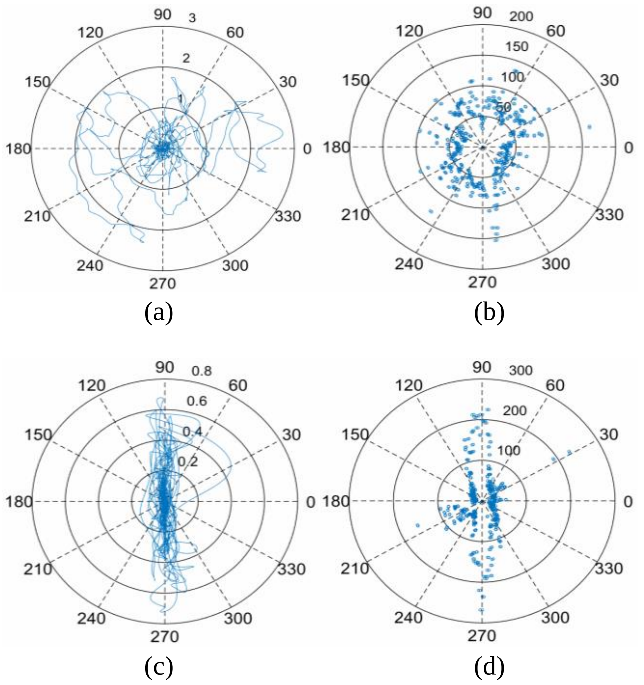


Figure 13. October 30th, 2016, earthquake: (a) absolute particle motion at A08-A09 and (b) corresponding angle distribution, (c) particle motion at A08-A09 relative to the base (A01-A03), and (d) corresponding angle distribution.

For the same sensors, A09 and A08, the wavelet transforms are plotted in Figure 14 with the same scale (Kanasewich, 1981). In the longitudinal direction (A09) the low frequencies between 1 and 2 Hz are always present, as well as those above 4 Hz. It is apparent that the resonance frequencies of the lower range change during the recording due to a significant nonlinear behaviour of the structure, as already pointed out in the previous study (Bongiovanni et al., 2017).

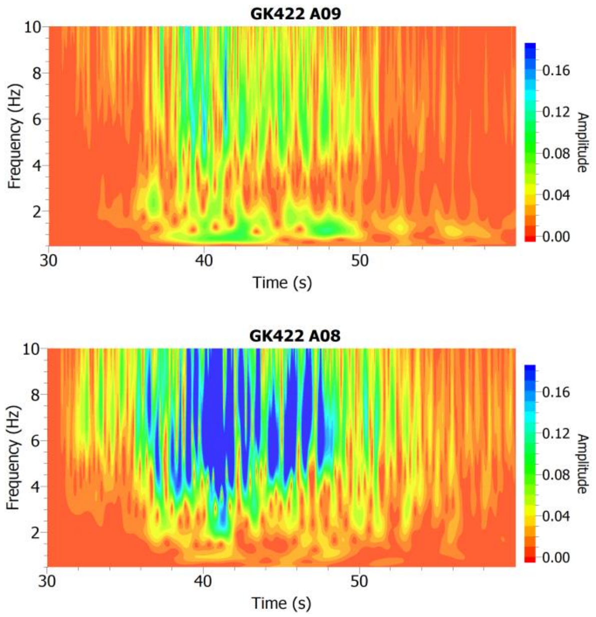


Figure 14. October 30th, 2016, earthquake: wavelet transforms of recording obtained at A09 (longitudinal direction) and A08 (transversal direction).

In the transversal direction (A08), the frequency components above 4 Hz are dominant while the lower ones are occasional; in particular, between 40 and 42 s, the higher frequencies, with high amplitude in this direction, increase also in the longitudinal direction and frequencies around 2 Hz become significant in the transversal direction. This occurrence seems to testify a certain interaction between the actions in the two directions, probably due to the high frequency motion along the transversal direction.

5 CONCLUSIONS

The experimental dynamic response of a precast prestressed concrete building to the strongest event (Mw=6.5) of the 2016-2017 Central Italy seismic sequence has been here presented.

The building is located 36 km far from the epicenter. The Arias intensity measured at its basement reached a value I_A = 53.7 cm/s.

A permanent instrumentation made of a three-axial and nine uni-axial accelerometers, deployed on a single frame of the building, has been used.

Acceleration time histories in the longitudinal and transversal directions at the central foundation plinth as well as at the top of the columns and at the beams were recorded.

Displacement time histories were computed by double integration of sensor records. Absolute and relative particle motions were obtained, together with the corresponding angle distributions. These allowed establishing that the preferential direction of motion corresponds to the longitudinal direction of the frames.

Response spectra and Fourier’s spectra at the foundation were computed and compared with the Fourier’s spectra at the top of the structure. Moreover wavelet transforms at the top of columns in longitudinal and transversal directions have been drawn in order to analyse how the signal frequency content changes over time.

In the longitudinal direction, the resonance frequencies of the frame were evident and located between 1 and 2 Hz. These values are lower than those obtained from ambient vibration records (f_{1y} = 2.05 Hz). This occurrence can be related to the nonlinear response of the structure, as confirmed also by the analysis of the wavelet transforms. As a matter of fact, before the strong phase of the earthquake, the building response is mainly confined around 2 Hz.

In the transversal direction, no amplification can be seen in the lower frequency range. Indeed the input response spectra is narrow around the peak and declines rapidly without stimulating building at its lowest frequency, which is the one obtained from ambient vibration records (f_{1x} = 1.92 Hz). On the other hand, a scattering of seismic energy to high-frequency structural modes is evident in the wavelet transform plots. This occurrence could explain the value of the peak acceleration at the top of columns, which is about twice the peak acceleration at the foundation.

Future developments of this study must be the improvement of the model, with particular reference to the perimeter lightweight concrete panels, to the building cover and to the connections between structural elements as well as between structural and non-structural ones.

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