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SHAKING-TABLE TEST ON A TWO-STOREY RC FRAMED STRUCTURE WITH INNOVATIVE INFILLS WITH SLIDING JOINTS

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ABSTRACT

Within the European FP7 Project "INSYSME" the research unit of the University of Pavia conceived a new seismic-resistant masonry infill system allowing the in-plane damage control in the masonry and the reduction of the detrimental effects of the panel-frame interaction and ensuring the out-of-plane stability of the panel. In this system (Morandi et al. 2016) the masonry panel is subdivided into horizontal strips through suitable "sliding joints"; the infill-frame interface is composed by deformable joints, reducing the stress concentration and the local effects; the out-of-plane stability is governed by the flexural resistance of the masonry strips and adequate out-of-plane supports at the RC columns. An extensive experimental campaign was performed, constituted by characterization tests, in-plane cyclic tests on one-storey one-bay full scale RC bare frame and two configurations of infilled frames followed by dynamic out-of-plane tests, and shake-table tests on a prototype "model building". In this paper, the results of the dynamic test on a full-scale two-storeys RC framed building with the innovative infills are discussed. Due to the mono-directionality of the shake-table, a pentagonal shape in plan was adopted for the structure, in order to investigate the seismic behaviour of the newly proposed infill system subjected to different loading conditions. The disposition of the openings was selected to obtain a regular distribution of the panels in plan, but an irregular layout in elevation. The behaviour of the building (*e.g.* overall response of the RC structure, seismic behaviour of each infill and its damage level, etc.) has been monitored and discussed.

Keywords: Innovative infills, Sliding joints, Shaking table tests, Full scale, infilled RC framed structure.

1. INTRODUCTION

"Traditional" masonry infill solutions, in which panels are built in complete contact with the surrounding RC frame without provision of any gap or connection around the boundaries and after the hardening of the RC members, have evinced a series of critical aspects related to in-plane and out-of-plane seismic response, often observed both in the post-seismic surveys (i.e., Manzini and Morandi 2012) and in the experimental outcomes (i.e.: Calvi and Bolognini 2001, Guidi et al. 2013, Morandi et al. 2018a). Although a series of researches oriented towards possible novel systems have been recently carried out in order to solve, or at least to limit, the aforementioned critical issues (Morandi et al. 2016), a widely recognized solution, which reduces in-plane/out-of-plane seismic vulnerability of masonry infills guarantying, at the same time, a sufficient thermal, acoustic and durability performance, has not been achieved yet.

Within the European FP7 Project "INSYSME", the research unit of the University of Pavia has developed a seismic resistant masonry infill system with sliding-joints with original details. The new infill solution has been conceived uncoupling it from the RC structure through the partition of the panel in horizontal strips separated by suitably shaped plastic sliding joints laid in the mortar bed-

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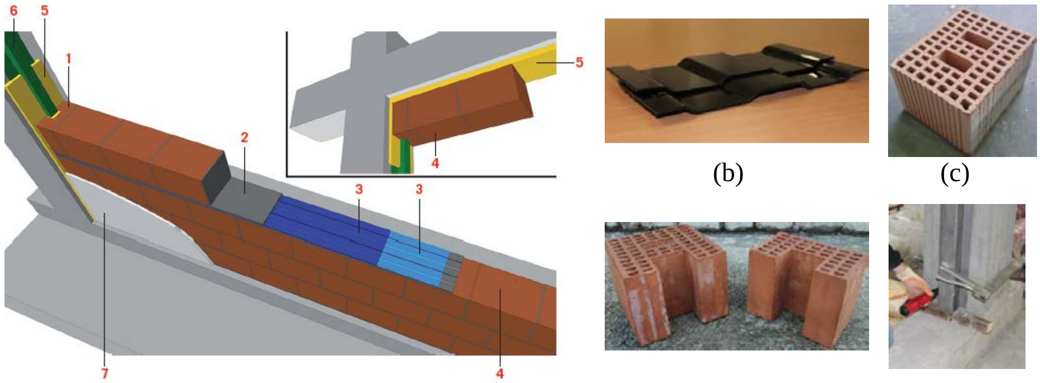
joints. The wall has been subdivided, in height, in four horizontal masonry strips with the application of three levels of sliding joints. A specifically designed deformable material with good thermal and acoustic insulation properties has been placed at the infill-frame interface with the aim of reducing the concentration of the stresses and prevent possible local damages and failures. The out-of-plane strength of the infill is provided by the contribution of a fibre-reinforced plaster and of the transversal mechanical interlocking of the corrugated profiles forming the sliding joints, whereas the out-of-plane stability has been guaranteed by means of steel shear keys suitably anchored at the RC columns and by “C-shaped” clay units placed at the edges of the masonry panel. The construction details of the system are shown, in a schematic way, in Figure 1. Further information on the concepts and the working principles of the system are reported in Morandi et al. (2016). The costs of construction of such system increase of about 25%-30% in comparison with the ones of a “traditional” solution realized with the same masonry typology.



(a) (b) (c) (d) (e)

Figure 1. (a) Details of the innovative masonry infill with sliding joints: 1.(c) C-shape units; 2. mortar bed-joints; 3.(b) sliding joints; 4.(d) clay units; 5. interface joints; 6.(e) shear keys; 7. plaster.

An extensive experimental campaign, constituted by tests of characterization, in-plane cyclic tests on one-storey one-bay full scale RC bare frame and two different configurations of infilled frames (with and without a central opening) followed by out-of-plane shaking-table dynamic tests, and a dynamic test on a prototype “model building”, has been performed (Milanesi 2016, Morandi et al. 2017, Morandi et al. 2018b). In this paper, a description of the specimen, of the instrumentation layout and of the experimental protocol adopted in the shake-table dynamic test on a full-scale two-storeys RC framed building infilled with the innovative solution is reported and the most significant results, in terms of overall response of the RC structure, seismic behaviour of each infill and its damage level, are discussed.

2. SPECIMEN, INSTRUMENTATION AND TESTING PROTOCOL

2.1 Description of the specimen

A full-scale two-storeys RC model building has been constructed and tested at the laboratory of the European Centre for Training and Research in Earthquake Engineering (EUCENTRE) in Pavia, Italy (Calvi et al. 2005).
Due to the mono-directionality of the dynamic testing to be performed, a pentagonal shape in plan of 5.5x4.4 m has been adopted for the structure, in order to be able to investigate the seismic behaviour of the innovative infill solution subjected to different loading conditions (pure in-plane; pure out-of-plane; simultaneous in-plane and out-of-plane actions, respectively), with an inclination of 45°. Moreover, the disposition of the openings in the innovative infills has been studied in order to avoid an irregular distribution in plan of the structure.
After the completion of the construction and transportation of the building specimen onto the shaking table, in order to simulate the presence of constant non-structural and additional loads, additional masses of concrete were cast on the two storey slabs, 9.1 tons each. Taking into account all the

contributions, the masses at the base of the structure (foundation slab; masonry infills and half of RC columns of the first storey), at level 1 (floor and additional masses, RC beams and half of RC columns of the first storey; masonry infills and half of RC columns of the second storey) and at level 2 (floor and additional masses, RC beams and half of RC columns of the second storey) resulted equal to 46.5, 32.4 and 20.4 tons, respectively, corresponding to a total mass of 99.3 tons.

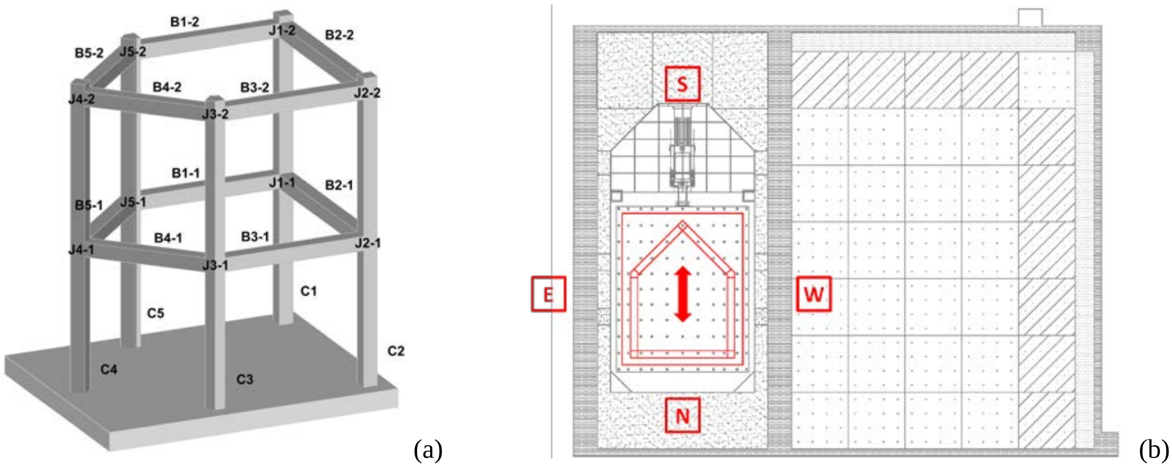


Figure 1. (a) 3D-sketch of the RC structure, with identification of beams (B), columns (C) and joints (J); (b) Plan view of the EUCENTRE laboratory and position of the shake-table and of the model building.

2.2 Design of the specimen

The design of the RC structure (Martuscelli, 2015) has been conducted according to current Italian and European standards (NTC2008 [2008], Eurocode 1 [2002], Eurocode 2 [2004a], Eurocode 8 [2004b]); multimodal analyses with response spectrum of an elastic model of the RC bare frame, in which the infills have been considered only in terms of masses and weight (common approach in current design practice) has been firstly conducted. The design has been performed in high ductility class (DCH), adopting a behaviour factor q equal to 5.85, suitable for regular RC frames. Subsequently, nonlinear static analyses have been pursued in order to check the design and to control the deformation capacity of the RC frame in order to design a structure that could reach at least 1.5% of inter-storey drift. Since the structure has been built outside of the laboratory and has needed to be moved inside and positioned onto the shaking table, the transportation phase has been taken into account and analysed in order to prevent any structural damage to the RC elements and to the infills before the testing phase.

The designed structure resulted at each storey in 300x300 mm RC beams and columns, with a clear span of the beams varied from 2.60 m to 3.80 m and a clear height of the columns of 2.95 m. Due to the pentagonal shape in plan, two RC columns (C3 and C5, at S-E and S-W corners of the structure, respectively, see Figure 1(b)) have been designed with irregular section shape, in order to have thickness, stiffness and resistance similar to those of the square section columns.

A two-way spanning mixed RC joists-hollow clay tile floor slab of 200 mm thickness, with a concrete topping of 50 mm (total thickness 250 mm) reinforced with mesh reinforcement been adopted at the two storeys of the structure.

The disposition of the openings in the innovative infills has been studied in order to avoid irregular distribution in plan of the structure but, at the same time, to provide a wide range of opening configurations and boundary conditions for the panels. In particular, as shown in Figure 2, at ground level two infills with lateral opening have been placed in the enclosures subjected to only in-plane actions, one fully infilled panel to only out-of-plane loading and two fully infilled panels simultaneously loaded by in-plane and out-of-plane seismic actions; the disposition of the partially infilled panels at the first storey was complementary to that at bottom storey.

Further details regarding the design of the building and the construction and transportation phases can be found in Milanesi (2016).

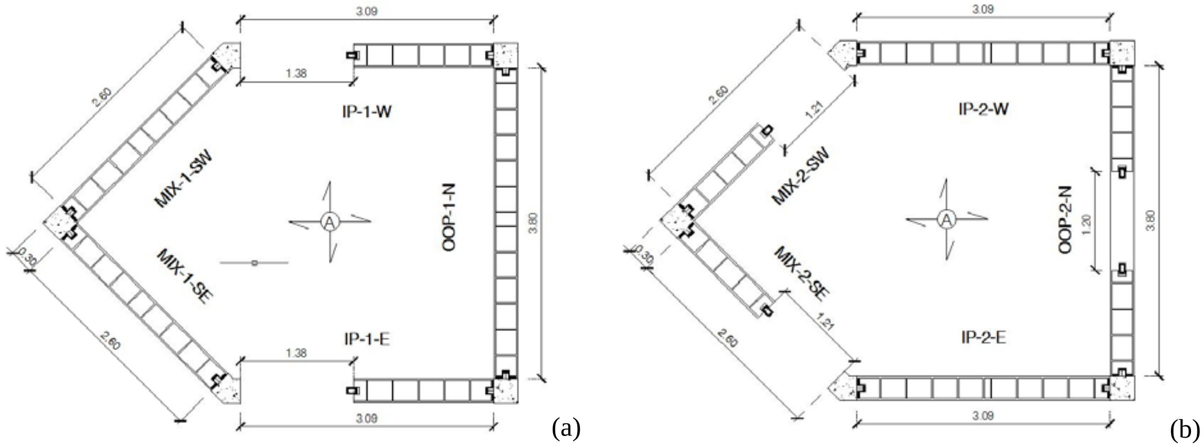


Figure 2. Infills distribution at storey 1 (a) and storey 2 (b) of the model building (dimensions in m).

2.3 Instrumentation

The instrumentation layout has been designed in order both to capture the global behaviour of the RC structure and to focus on the local in-plane and out-of-plane behaviour of the masonry infill panels. A traditional acquisition system, composed by potentiometers and by 1D, 2D and 3D accelerometers, and a 3D optical acquisition system have been adopted (Milanesi 2016). The scheme of the instrumentation layout is illustrated in Figure 3.

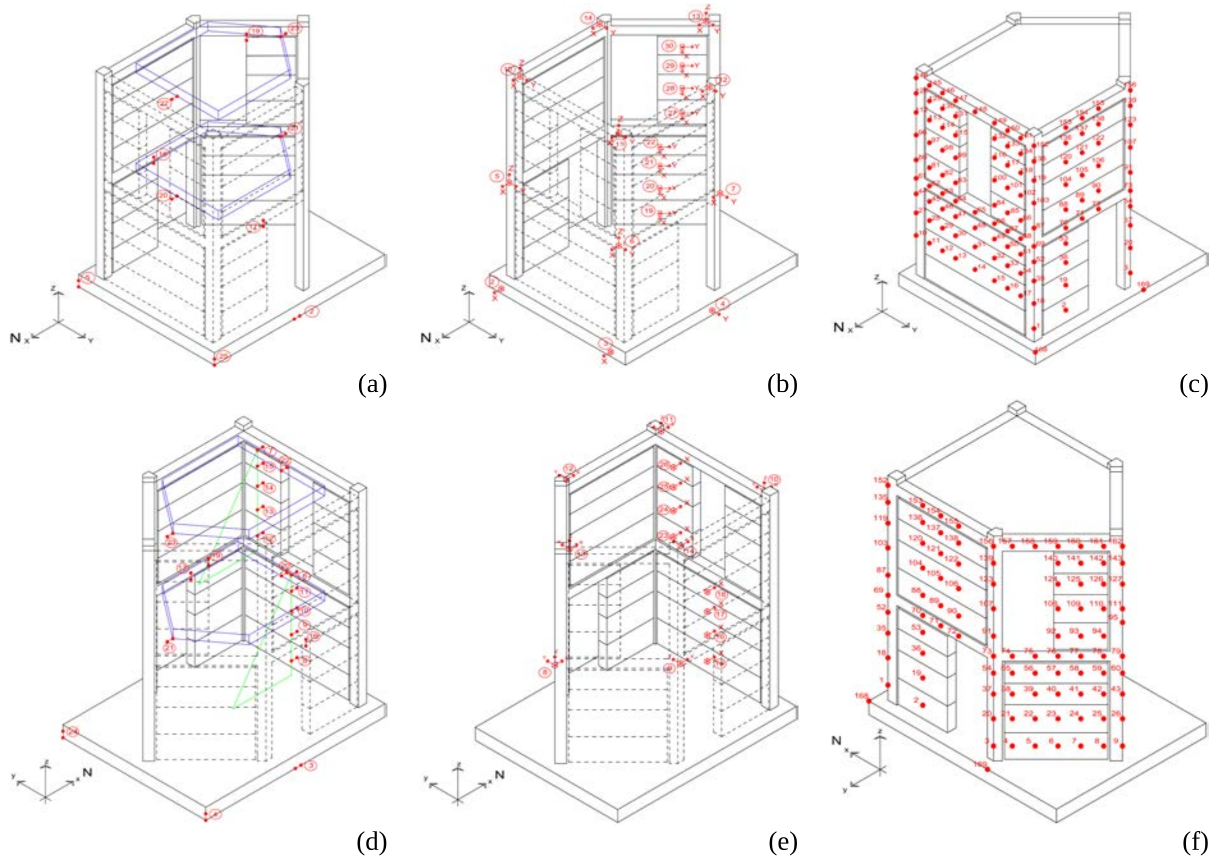


Figure 3. Scheme of the disposition of the instrumentation on the model building: potentiometers (a, d); accelerometers (b, e); optical passive target markers (c, f).

A total of 25 potentiometers have been used to control the eventual sliding and uplift of the foundation with respect to the shaking table and the eventual sliding of the additional masses with respect to the floor slabs and to measure the inter-storey drifts of the structure, the vertical elongation at the top of the telescopic steel post confining the opening of the façades subjected to in-plane actions and the out-of-plane displacement at the centre of each strip of some selected infill panels.

A total of 30 accelerometers has guaranteed an accurate measurement of the accelerations of the shaking table, of the RC foundation, of the RC frame (beam-column joints) and of some infill panels (at mid-height of each strip).

A high number of passive target markers has been placed on the external surface of the N, W and S-W façades of the model building in order to allow high-definition cameras to monitor the in-plane and out-of-plane displacements of defined points of the walls. In particular, lines of markers have been located at mid-height of each infill strip and RC beam; a total of 14 cameras and about 170 markers have been used.

3. TEST PROCEDURE

3.1 Seismic input

A large number of natural accelerograms with characteristics compatible with the Italian source mechanisms was initially taken into account and preselected from ground motion Italian and European databases. The selected input motion was the E-W component of the ground motion recorded at the Ulcinj-Hotel Albatros station during the April 15th, 1979 Montenegro earthquake (PGA = 0.22g). The record was chosen for its time duration, for its broad frequency band and for its relatively regular acceleration response spectrum in the range of periods of interest (Figure 4).

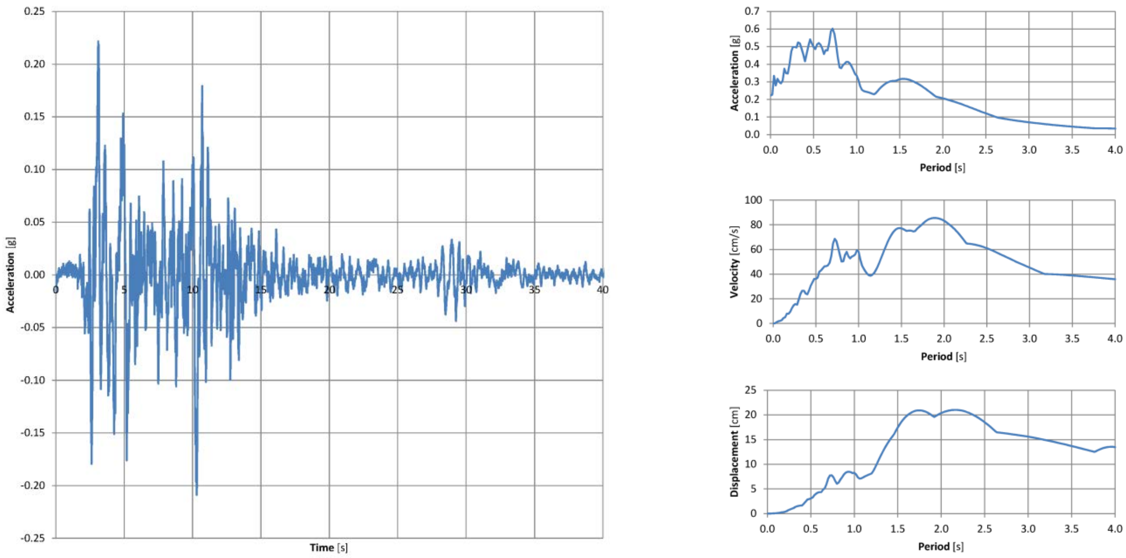


Figure 4. Acceleration time-history (left) and response spectra (right) of the strong-motion selected for the tests.

3.2 Testing protocol

The sequence of dynamic tests has consisted in subjecting the model building to incremental seismic excitations in the N-S direction, namely a series of table motions at increasing intensity.

During the shaking-table tests the selected record was scaled to match peak ground accelerations (PGA) ranging from (about) 0.10 to 1.20 g. Before each main test (“GM”) one or more “white-noise” tests (random signals, “RNDM”) have been performed both for the characterization of the dynamic properties of the model building and for a preliminary tuning of the shaking table. When necessary, series of tuning tests (“TNG”, constituted by opportunely scaled signals of the following main runs) have also been performed for the calibration of the shaking table test parameters. Repetitions of the aforementioned sequence have been performed till the attainment of a collapse/near collapse limit state of the specimen. The complete testing protocol is summarized in Table 1, in which are reported

for each run the date of the test, the sequence number, the typology of the dynamic input and the test name; with respect to each main run, the table also reports the PGA of the input signal (nominal PGA) and the PGA of the actual shaking table movement (actual PGA).
During test GM 120, the actual displacement of the shaking table at the first acceleration peak has exceeded the shaking-table allowable displacement limit and the activation of the interlock system of the shake-table has stopped the test.

Table 1. Testing sequence.

Date	**Test #**	**Dynamic input**	**Test name**	**Nominal PGA** [g]	**Actual PGA** [g]
13/11/2015	1	Random	RNDM 010	---	---
	2	25% sel. GM	TNG 010 25%	---	---
	3	25% sel. GM	TNG 010 25% B	---	---
	4	25% sel. GM	TNG 010 25% C	---	---
	5	**50% sel. GM**	**GM 010**	**0.11**	**0.10**
	6	Random	RNDM 020	---	---
	7	**100% sel. GM**	**GM 020**	**0.22**	**0.22**
	8	Random	RNDM 030	---	---
	9	Random	RNDM 030 B	---	---
	10	**150% sel. GM**	**GM 030**	**0.33**	**0.33**
14/11/2015	11	Random	RNDM 040	---	---
	12	100% sel. GM	TNG 040 100%	---	---
	13	100% sel. GM	TNG 040 100% B	---	---
	14	**200% sel. GM**	**GM 040**	**0.44**	**0.42**
	15	Random	RNDM 060	---	---
	16	**300% sel. GM**	**GM 060**	**0.66**	**0.67**
	17	Random	RNDM 080	---	---
	18	100% sel. GM	TNG 080 100%	---	---
	19	Random	RNDM 080 B	---	---
	20	100% sel. GM	TNG 080 100% B	---	---
	21	100% sel. GM	TNG 080 100% C	---	---
	22	**400% sel. GM**	**GM 080**	**0.87**	**1.14**
18/11/2015	23	Random	RNDM 100	---	---
	24	200% sel. GM	TNG 100 200%	---	---
	25	200% sel. GM	TNG 100 200% B	---	---
	26	200% sel. GM	TNG 100 200% C	---	---
	27	**500% sel. GM**	**GM 100**	**1.02**	**1.32**
	28	Random	RNDM 100 B	---	---
	29	200% sel. GM	TNG 100 B 200%	---	---
	30	200% sel. GM	TNG 100 B 200% B	---	---
	31	200% sel. GM	TNG 100 B 200% C	---	---
	32	**500% sel. GM**	**GM 100 B**	**1.02**	**1.04**
	33	Random	RNDM 120	---	---
	34	200% sel. GM	TNG 120 200%	---	---
	35	200% sel. GM	TNG 120 200% B	---	---
	36	200% sel. GM	TNG 120 200% C	---	---
	37	**600% sel. GM**	**GM 120**	**1.24**	**1.66**
19/11/2015	38	Random	RNDM 100 C	---	---
	39	200% sel. GM	TNG 100 C 200%	---	---
	40	200% sel. GM	TNG 100 C 200% B	---	---
	41	200% sel. GM	TNG 100 C 200% C	---	---
	42	**500% sel. GM**	**GM 100 C**	**1.02**	**1.13**

4. TEST RESULTS

4.1 Crack pattern

The sequence of the damage of the model building has been recorded after each test run and the evolution of the cracking patterns in both the outer and the inner side of each façade has been studied. According to the observation of the damage propagation, no crack, neither in the RC elements nor in the masonry panels, occurred up to GM 80 (actual PGA=1.14 g) has been noticed, with the only exception of horizontal light cracks in correspondence of the sliding joints and at the frame-panel interface, at the first and at the second storey, in all of the five plane frames (in plane, out-of-plane and at 45° angle) which form the structure.
Starting from GM 100 (actual PGA=1.32 g), two diagonal cracks developed in the bottom strip of the panels at the ground storey close to the steel studs confining the openings in both the infills parallel to the direction of the seismic action; in the W-side frame, a diagonal crack, almost parallel to that below, also appeared in the second bottom strip. Few thinner diagonal cracks at the edges of the strips formed in the 45° degree solid infills of the ground storey (MIX-1-SW, MIX-1-SE) at the level of the plaster.
At GM 100-B (actual PGA=1.04 g), other diagonal cracks occurred in the infills at the ground storey, mainly concentrated in the panels close to the openings (forming cracks with adverse angle respect to the previous ones) and at the 45° angle ("MIX") infills. First flexural cracks appeared in the RC columns C1, C2 and C4 (the corner one located at South).
At GM 120 (actual PGA=1.66 g), few new diagonal cracks appeared in the strips of the ground panels (E- and W-sides and SE- and SW- sides); the flexural cracking in RC columns C1, C2 and C4 increased in number and width and formation of few horizontal cracks at the ground storey in the columns C3 and C5 occurred.
The last run, GM 100-C (actual PGA=1.13 g), caused a development of new diagonal cracks and the enlargement of the existing ones in the panels close to the openings of the ground storey with the failure of the connection of the steel plates that support the confining studs and the dislodgement of the stud in the E-side; diagonal cracks in the masonry on these short panels formed and spalling of small areas of plaster occurred also in the 45° degree solid infills at the ground storey; few increase of the flexural cracks in the columns was noticed, without involving any shear/diagonal cracks in the RC elements; no damage was evident in the RC members of the upper storey. The N-side infills placed perpendicular to the seismic motion and all the infills at the upper storey did not develop any other cracks but the very light ones in the sliding joints, without affecting the quality of the masonry.
Figure 5 sketches the evolution of the cracking pattern in the outer side of the E, S-E and N façades of the model building, subjected to pure in-plane, pure out-of-plane and simultaneous in-plane and out-of-plane loading, respectively.

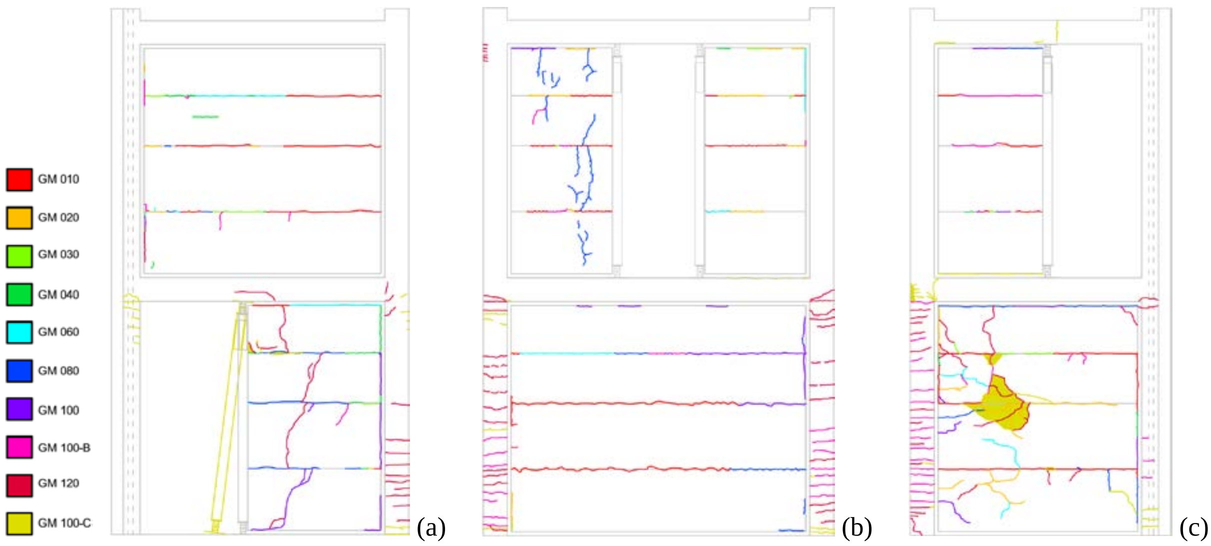


Figure 5. Cracking pattern of outer side of the E (a), N (b) and S-E (c) façades of the model.

4.2 Dynamic characteristics of the specimen

Peak Picking (Erwin 2000) and Enhanced Frequency Domain Decomposition (Brincker et al. 2000) modal identification techniques have been used to survey the building specimen, making reference to the low intensity random vibration tests performed at different stages of the testing sequence. A summary of the results obtained is presented, focusing on the frequencies of vibrations of the fundamental mode of the structure in the direction of motion.
The variation of the fundamental frequency is depicted in Figure 6, showing a trend characterized by values initially very slightly reducing from 11.83 Hz to 10.83 Hz up to test #15 (RNDM 060) and then rapidly decreasing to 2.42 Hz (corresponding to 0.204 times the initial value) as the damage level increases in the structure.

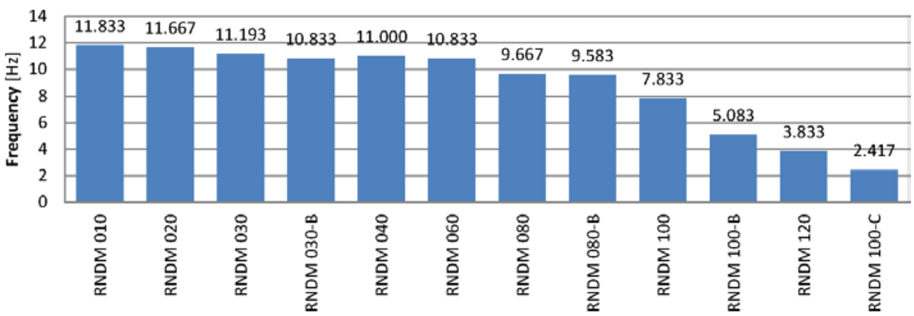


Figure 6. Variation of the fundamental frequency.

Up to test #15 (subsequent to GM 040), the fundamental mode of vibration of the specimen in the direction of motion remains essentially unchanged with respect to that identified during test #1, while after test #15 a significant change in modal shape has been observed, showing a torsional component, with the E-W façades of the building characterized by different levels of in-plane displacement.

4.3 Global response of the structure

The displacement at each storey of the building has been evaluated as the average of the displacement history of the optical markers placed on the N, W and S-W façades at middle height of the RC beams, deducted of the absolute displacement of the foundation (which resulted equal to that of the shaking table, since no sliding has been registered by the instrumentation); inter-storey displacement has been consequently computed.
The total drift of the model building has been evaluated as the top displacement relative to the foundation divided by the height of the structure, assumed equal to 6350 mm. The inter-storey drift has been computed assuming a total storey height of 3100 mm and 3250 mm, respectively, at first and second storey, being 2950 mm the effective storey height and 300 mm the height of the RC beam at each level of the building.
Few critical issues occurred during the tests, which have influenced the evaluation of the building displacements. During test GM 120, the actual displacement of the shaking table at the first acceleration peak has exceeded the allowable displacement limit of the shaking-table and the activation of the interlock system has stopped the test. Moreover, during the last test (GM 100-C) the optical acquisition system failed in recording the displacement of a significant amount of markers placed both on the N and on the W façade of the building. Further post-processing of the recorded data is therefore needed and ongoing; at the moment, the displacement at each storey of the model building has been evaluated from the data recorded by the traditional acquisition.
During the testing sequence, the displacement demand has been mostly localized in the bottom storey, with a fundamentally symmetric behaviour of the model building when excited in positive and negative direction (S-N and N-S direction of motion of the shake-table, respectively) up to test GM 060; from test GM 080, the inter-storey drift in the positive direction has resulted larger than in the negative direction (with the exception of test GM 120). During the last two tests, the inter-storey drift

at the bottom storey has exceeded 1.0%. In particular, during tests GM 120 and GM 100-C the maximum recorded inter-storey drift resulted equal to 1.66% (negative direction) and 3.13% (positive direction), respectively.

The data recorded by the accelerometers located on the RC frame and on the foundation have been averaged in order to evaluate the acceleration histories at each level of the structure (i.e.: foundation, first storey, second storey). The results, illustrated in Figure 7, show how the acceleration is continuously increasing up to test GM 100. The acceleration demands in the first and in the second storey appear to be rather similar with a slight increase of the second floor acceleration compared to the one at the first floor. The amplification of the accelerations as respect to the Peak Ground Acceleration is limited for all the runs, with a maximum acceleration at the top of the building of about 1.60 g, corresponding to test GM 120.

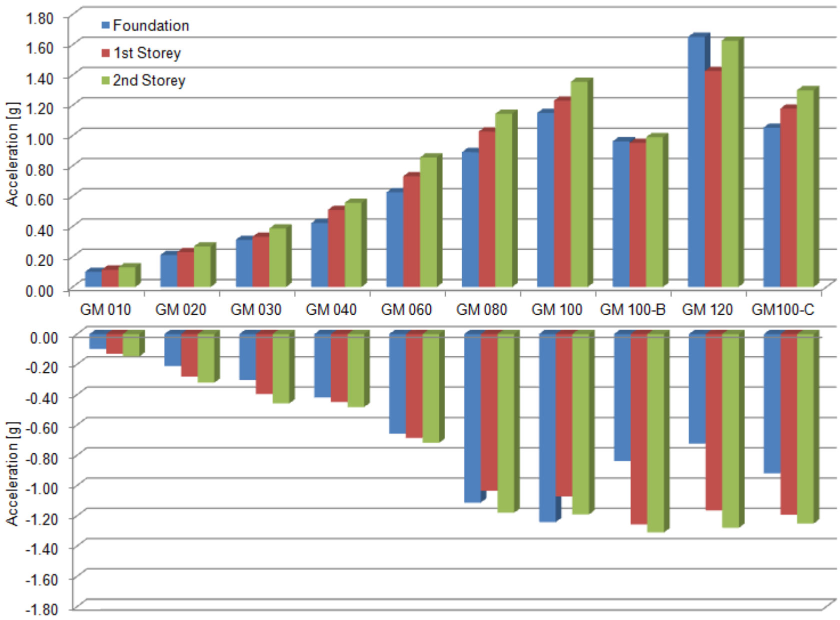
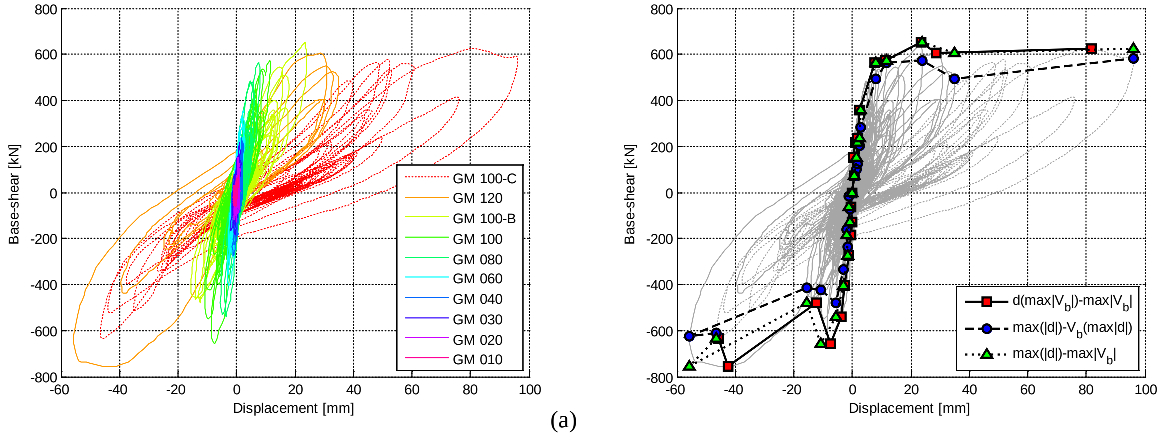


Figure 7. Maximum/Minimum accelerations recorded at each level of the model building.

The hysteretic response of the model building during each test, in terms of base-shear versus average top displacement, has been investigated as well. The base-shear has been evaluated by summing the inertia forces of the two storey of the specimen, which have been computed as the average acceleration history of the accelerometers placed at each RC joints times the total tributary mass at each storey. Figure 8(a) shows the hysteretic response of the model building during the complete sequence of tests performed (the curve related to GM 100-C in plotted in dashed line). In Figure 8(b) the “maximum base-shear versus top displacement”, the “base-shear versus maximum top displacement” and the “maximum base-shear versus maximum top displacement” envelopes of the experimental curve are also reported.

In following Table 2 and Figure 9, the “maximum base-shear versus top displacement” envelope of the total hysteretic response of the model building is reported. The model building seems to have maintained during the testing sequence an elastic behaviour up to a value of lateral force of about 600 kN in both positive and negative direction (i.e. S-N and N-S direction of motion of the shaking table), corresponding to approximately 11.5 mm and 7.5 mm of top displacement, respectively. Subsequently, in case of positive direction of excitement the force has slightly increased up to a displacement of about 23.5 mm and finally, after a slightly decrease at about 600 kN, it has remained unvaried up to a maximum top displacement of about 97 mm. In case of negative direction of excitement, a sudden decrease of the lateral force to about 480 kN has been recorded; subsequently, the force has increased to about 750 kN, corresponding to a top displacement of about 42.5 mm, and finally has decreased again to about 600 kN, corresponding to about 46 mm of top displacement. Maximum top displacement in negative direction resulted equal to about 55 mm.



(a) (b)

Figure 8. Total hysteretic response of the model building (a) and corresponding envelopes (b).

Table 2. Maximum base-shear vs top displacement envelope of the hysteretic response of the model building.

	Positive direction (S-N)		**Negative direction (N-S)**	
Test	**Displacement** [mm]	**Force** [kN]	**Displacement** [mm]	**Force** [kN]
GM 010	0.7	70.5	-0.4	-62.5
GM 020	0.4	153.0	-0.2	-127.0
GM 030	1.1	218.0	-0.6	-183.0
GM 040	1.7	238.0	-1.1	-273.0
GM 060	2.4	360.0	-2.5	-403.0
GM 080	7.6	563.0	-3.6	-542.0
GM 100	11.5	573.0	-7.6	-655.0
GM 100-B	23.3	651.0	-12.2	-479.0
GM 120	28.7	604.0	-42.5	-757.0
GM100-C	81.8	624.0	-46.0	-635.0

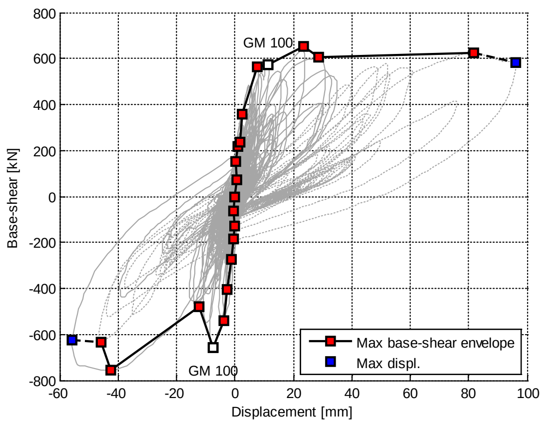


Figure 9. Maximum base-shear envelope of the total hysteretic response of the model building.

The deformed shape of the model building has been evaluated for each ground motion, in terms of relative displacement of the joints of the RC frame with respect to the displacement of the RC foundation, which shows almost pure translational displacements of the structure up to tests GM 060-GM 080 and, afterwards, an in-plan torsional anti-clockwise mode up to GM 120 and a torsional clockwise mode at the last run (GM 100-C), also affecting the distribution of the cracking pattern in the last runs of the tests. In Figure 10 the deformed shape of the model building corresponding to the maximum and minimum displacements of the shaking table during GM 100-C is reported.

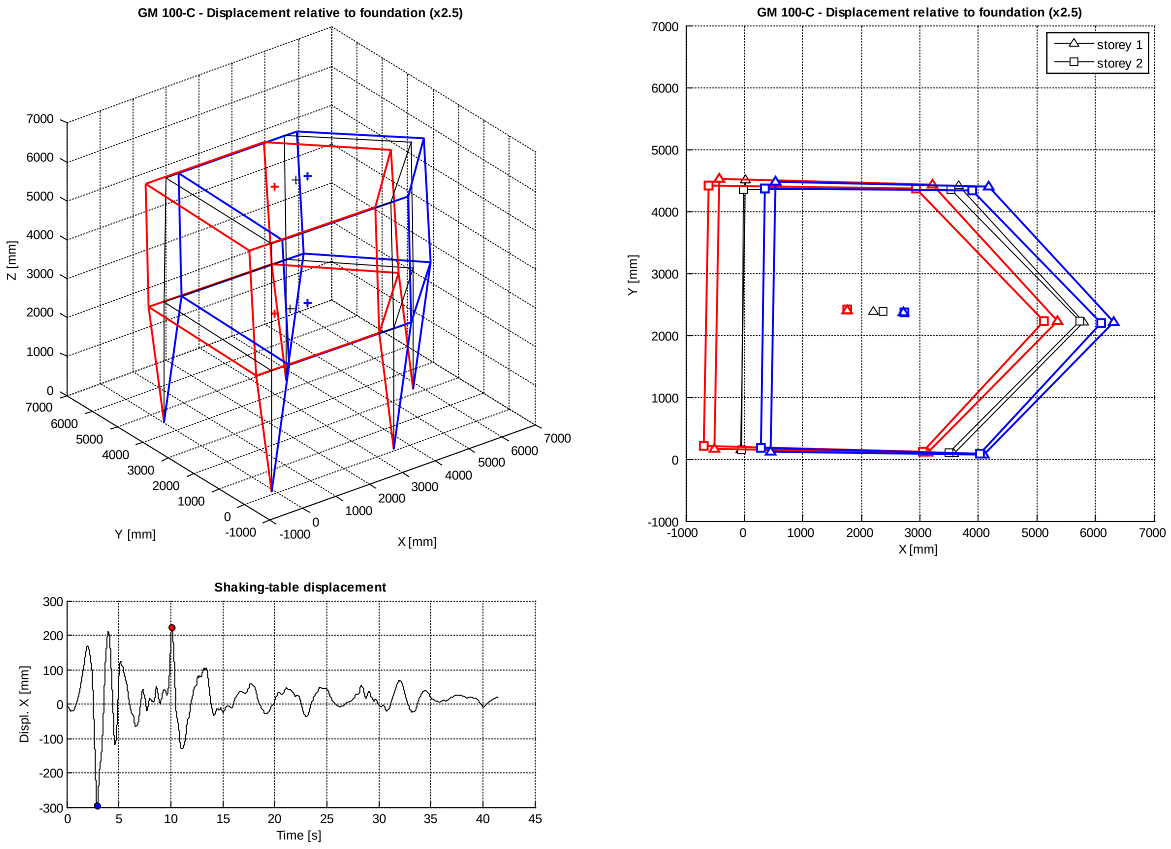


Figure 10. GM 100-C: Deformed shape of the model building at maximum (red lines) and minimum (blue lines) displacement of the shaking table.

5. CONCLUSIONS

Within the European FP7 Project "INSYSME", a dynamic test on a full-scale two-storeys RC model building infilled with the innovative infill system developed by the research unit of the University of Pavia was performed on the shake-table of the laboratory of EUCENTRE, Pavia, Italy. The RC structure of the building, designed in accordance to current Italian and European standards, has an in-plan pentagonal shape in order to investigate the pure in-plane, the pure out-of-plane and the simultaneous in-plane and out-of-plane responses of the infilled system, given the mono-directional nature of the action of the shake-table. The disposition of the openings in the innovative infills has been studied in order to avoid an irregular distribution in plan of the structure but, at the same time, to provide a wide range of opening configurations and boundary conditions for the panels.
After the completion of the construction and transportation of the building specimen onto the shaking table, displacement transducers and optical passive target markers have been installed on the specimen for the acquisition of the displacements, along with accelerometers for the measurement of the accelerations.
The sequence of dynamic tests has consisted in subjecting the model building to incremental seismic excitations in the N-S direction, using the E-W component of the ground motion recorded at the Ulcinj-Hotel Albatros station during the April 15th, 1979 Montenegro earthquake (PGA = 0.22g). During the shaking-table tests, the selected record was scaled to match peak ground accelerations (PGA) ranging from (about) 0.10 to 1.20 g; the effective (recorded) ones resulted generally in good agreement with the exception of few runs (i.e. GM 120 reaching an actual PGA = 1.66 g).
The cracking pattern in both the RC structure and in the masonry enclosures has been registered during the tests and several results regarding the global response of the structure have been obtained; the variation of the fundamental frequency of vibration of the specimen has been evaluated conducting dynamic identification tests before each main ground motion; the displacement, inter-storey drift and

acceleration demand at each storey have been monitored, as well.
Although further investigations are still ongoing, regarding the evaluation of the interaction between the RC elements and the infills, according to the reported results of the shaking-table tests (and, in general, also at the light of the experimental evidence deducted from the whole experimental campaign), the overall seismic performance of the newly proposed infill system seems to be very promising for future possible real application, both in the construction of new RC structures and in the seismic upgrade of existing buildings. In fact, very light or negligible amount of damage was developed in the RC elements and in the infilled masonry up to very large level of seismic action (PGA larger than 0.80 g); the type of cracking is such limited that it appears very easy and cheap to be repaired being the damage mainly lumped in the sliding joints at the plaster level. Only after the last very demanding two runs (GM 120: PGA=1.66 g, max drift=1.66%; GM 100-C: PGA=1.13 g, max drift=3.13%), the extent of the damage in the infills has increased with the creation of diagonal cracks in the short panels at the ground storey and spalling of small areas of plaster in some of the infills of the ground storey. However, this damage level still provides a lot of residual capacity before the attainment of an ultimate limit state (ULS) condition that could represents risk for the safety of people.

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