16TH EUROPEAN CONFERENCE ON
EARTHQUAKE ENGINEERING THESSALONIKI
18 - 21 JUNE 2018

ACCIDENTAL TORSIONAL RESPONSE OF A LARGE-SCALE THREE-STORY FRAMED-MASONRY STRUCTURE

Dalibor BURILO[1], Davorin PENAVA[2], Lucas LAUGHERY[3], Ivica GULJAŠ[4], Santiago PUJOL[5]

ABSTRACT

In designing structures for earthquake resistance, EN1998-1:2005 requires regularity in plan and elevation to limit torsional motion. A regular, 1/2.5 scale three-story masonry infilled reinforced concrete frame structure was built and tested on a unidirectional shaking table. The structure was designed in compliance with EN1992-1-1:2004 and EN1998-1:2005 provisions as a moment-resisting frame by considering the medium ductility form of seismic construction detailing. Interacting masonry infill walls had door openings in the ground story walls and window openings in upper story walls. The 1979 Herceg Novi scaled record was used as seismic action in the longitudinal direction of the structure. The structure was tested under sequences of simulated ground motions in two series. In the Series 1, hollow clay block masonry infill walls were used. After completion of this first series, the masonry infill walls in the first and second story were replaced by solid brick masonry including reinforced concrete confining elements along vertical opening edges. The tests were repeated in the same sequence in Series 2. Torsional response was twice as large in the structure in Series 1 (with hollow clay block masonry walls) as in Series 2 (with solid clay brick masonry walls) as a result of non-uniform structural damage.

Keywords: reinforced concrete frame structure; large-scale; masonry infill wall; accidental torsion; shaking table tests

1. INTRODUCTION

In the designing structures for earthquake resistance in compliance with EN1998-1:2005 (CEN 2004b) provisions the structure should be regular in plan and elevation in order to limit the development of torsional motions. These criteria should also be taken for masonry infilled reinforced concrete frame structures (framed-masonry) where both the masonry and the frame components constitute part of the earthquake-resisting structural system. Typically, masonry walls contain door and/or window openings which should be accounted for in design because they introduce irregularities in plan and elevation. But available mathematical models for design of earthquake resistance of framed-masonry structures are still unable to accurately capture response, particularly when the masonry walls contain openings. By including the high uncertainty related to masonry characteristics, it is easy to misjudge torsional resistance and stiffness of the system. However, if masonry is designed and constructed in compliance with EN1996-1-1:2005 (CEN 2005) and EN1998-1:2005 (CEN 2004b) provisions, the structural performance can be improved and the potential to torsional response reduced.

---

[1]PhD student, Faculty of Civil Engineering Osijek, Josip Juraj Strossmayer University of Osijek, Osijek, Croatia, dburilo@gfos.hr
[2]Dr., Faculty of Civil Engineering Osijek, Josip Juraj Strossmayer University of Osijek, Osijek, Croatia, dpenava@gfos.hr
[3]Visiting Researcher, Department of Architecture & Design, Nagoya Institute of Technology, Nagoya, Japan, eke76296@ict.nitech.ac.jp
[4]Professor, Faculty of Civil Engineering Osijek, Josip Juraj Strossmayer University of Osijek, Osijek, Croatia, iguljas@gfos.hr
[5]Professor, Lyles School of Civil Engineering, Purdue University, West Lafayette, Indiana, United States, spujol@purdue.edu

The driving reason for the tests described here is the lack of consensus among the profession about the effects of masonry infill walls in reinforced concrete frames. Some researchers have suggested that infill walls have led to building collapses (Aschheim 2000; Sezen et al. 2003; Kyriakides & Billington 2008) and that infill walls may affect the response of frames detrimentally (Murty et al. 2006). Others have suggested that masonry infill walls may be beneficial (Akin 2006; Hassan & Sozen 1997; Fardis & Panagiotakos 1997; Henderson et al. 2003; Mehrabi & Shing 1997). The reason for the apparent contradiction may lie in observations made by (Negro & Colombo 1997) and (Hashemi & Mosalam 2006), who stated that masonry infill walls have both positive and negative effects. (Dolšek & Fajfar 2008) captured the essence of the problem stating: "The infill walls can have a beneficial effect on the structural response, provided that they are placed regularly throughout the structure, and that they do not cause shear failures of columns." Contradictions in the views of the research community have led to deconstruction of frame-masonry system as a result of many regional building codes that contain warnings about the interaction of frames and infill walls but are mostly silent on providing recommendations and bounds on their proper proportioning.

In multi-story construction, the most important attribute of the structure is its capability to retain its integrity at story drift ratios on the order of 1.5%-2%. A recent test of a full-scale three-story structure by (Pujol & Fick 2010) demonstrated that drift ratios of that magnitude can be achieved by a reinforced concrete frame with full infill walls provided the columns have the ability to sustain the required shear force under reversals of shear and axial forces. In the new buildings designed in accordance with the EN1998-1:2005, the infill wall is treated as a source of structural additional strength and so-called "second line defense".

However, the reduction of input seismic demand as a result of possible favorable infill wall effects is not allowed. Considering this, design of reinforced-concrete (r-c) buildings with masonry infill according to EN1998-1:2005 errs on the side of safety but it is not rational because it leads to significant increase of reinforcement in the structural r-c elements when compared to the design of bare r-c frames. Problems related to openings, out-of-plane collapse, and column strength under shear and axial load require special attention by structural engineers.

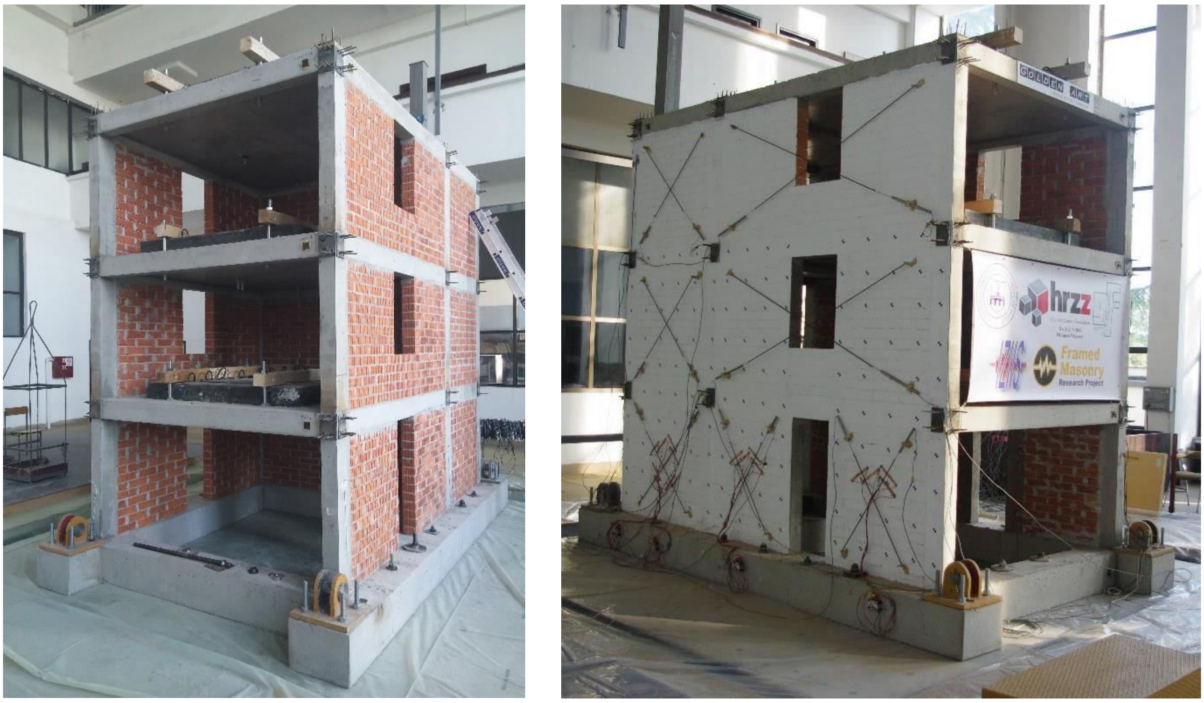

Figure 1. Structures on the shaking table before testing: (left) Series 1, with hollow clay block masonry and no vertical confining elements, and (right) Series 2, with solid masonry and confining vertical elements.

2. LARGE-SCALE TEST STRUCTURE

The 1/2.5 scale three-story masonry infilled reinforced concrete frame structure was designed and constructed in compliance with EN1992-1-1:2004 (CEN 2004a) and EN1998-1:2005 (CEN 2004b) provisions as moment-resisting frames by considering the medium ductility form of seismic construction detailing. It consisted of two longitudinal frames that had two bays with center-to-center span lengths of 128 cm and 248 cm. The three frames in the transverse direction had one bay with a center-to-center span length of 220 cm. Gross dimensions of the structure were 460 cm x 280 cm x 390 cm (length x width x height), including a 30 cm foundation. The columns and beams were 12 cm wide and 16 cm deep. Each story was 120 cm tall including an 8-cm thick reinforced concrete slab. Slabs were reinforced with reinforcement meshes of type Q139 (4.2 mm diameter bars in 100 × 100 mm spacing) in upper and Q196 (5 mm diameter bars in 100 × 100 mm spacing) in lower region.

Masonry infill wall types were built after the frame had hardened and were regularly positioned in plan and elevation. They had 40 cm x 104 cm door openings in the first story and 53 cm x 67 cm window openings in second and third stories. All openings were centered in the bays. In the first series of tests (see section 3) masonry infill walls were built of clay block masonry units measuring 12 cm x 25 cm x 6.5 cm (width x length x height) with volume of voids equal to 68 %, laid with class M5 general purpose mortar with 6 mm thick joints.

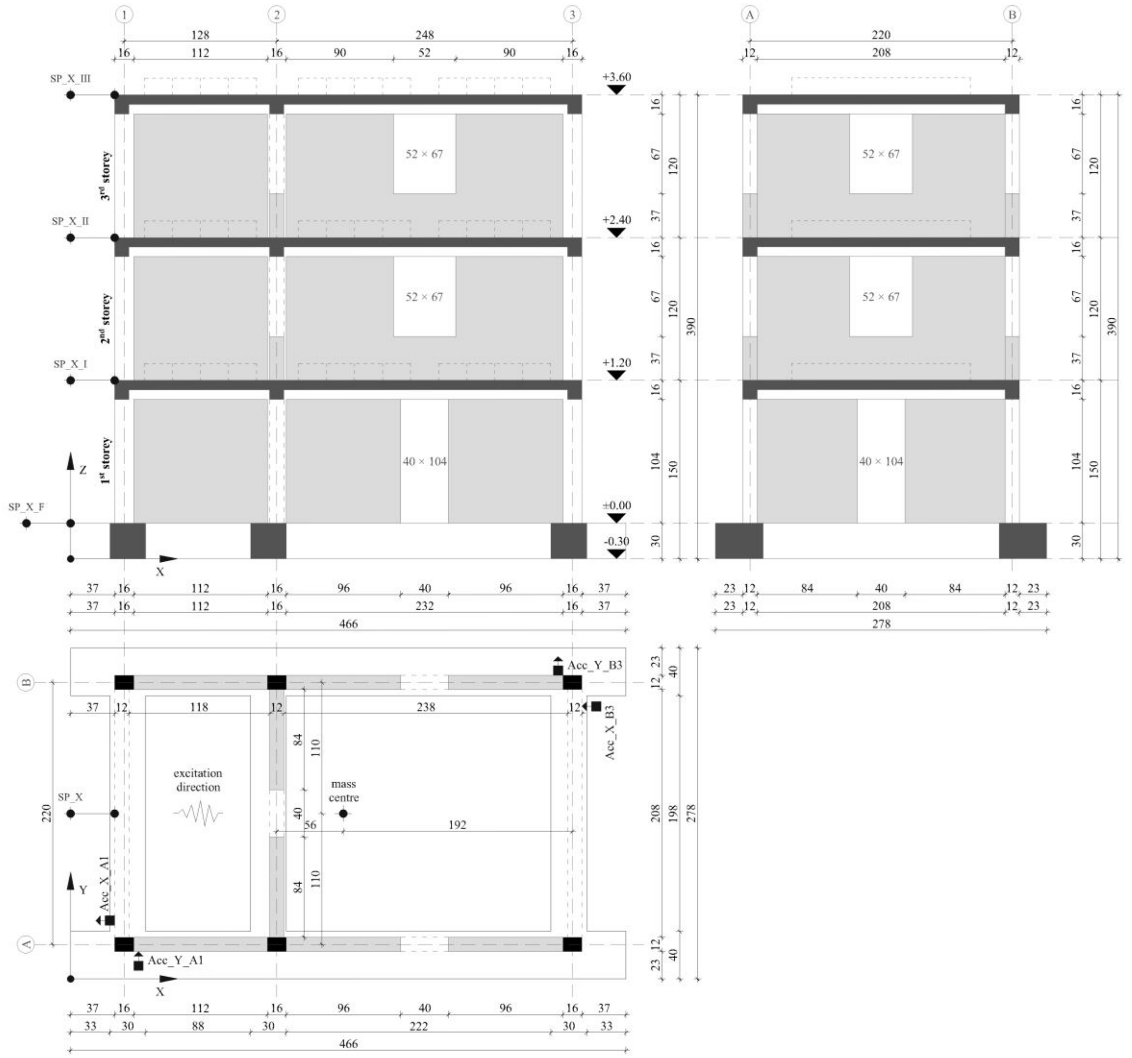

Figure 2. Longitudinal (top left), transverse (top right) and instrument plan views of the Series 1 structure (all dimensions in cm)

The masonry units were cut from full-scale masonry units to preserve the number of bedjoints. Mortar proportions by volume of cement, lime, and sand were 1:1:5. In the second series of tests, the masonry infill walls in the first and second story were replaced by solid clay brick (12 cm x 25 cm x 6 cm) masonry including reinforced concrete confining elements along vertical opening edges. Opening sizes and mortar joint thickness were kept the same.

The test structure appearance, dimensions, cross-sections, reinforcement plans for the beams and columns are given in Figure 1 through Figure 6. All concrete was class C25/30 with a nominal cylinder compressive strength of 25 MPa (nominal cube strength of 30 MPa) and a maximum aggregate size of 8 mm. The reinforcement was of Grade B500B with a nominal yield stress of 500 MPa. The properties of material of construction are given in Table 1.

Additional story masses were installed in the form of steel ingots to increase the period of the specimen by doubling its mass (see Figure 1) and in order to comply with Cauchy-Froude similitude rules. The ground motion such that gm/EL^2 in the prototype and the model were equal (g = gravitational acceleration, m = mass, E = elastic modulus, and L = length). This was done with the intent of matching key frequencies in the ground motion and test specimen. The total masses of the Series 1 and 2 structures were 29.2 and 30.6 tons, respectively. The model building was constructed and tested at the IZIIS laboratory in Skopje, Macedonia (Necevska-Cvetanovska et al. 2015).

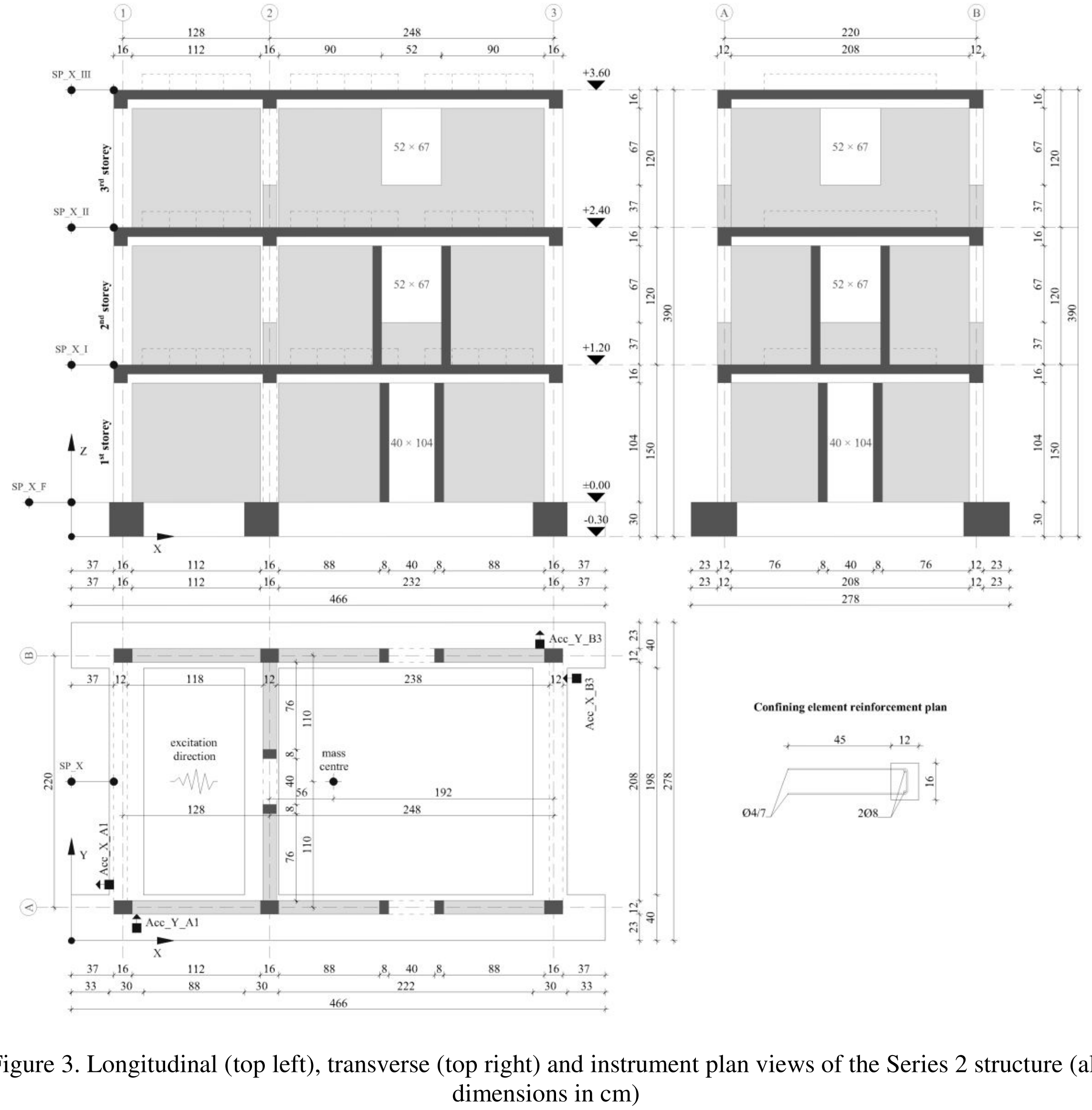

Figure 3. Longitudinal (top left), transverse (top right) and instrument plan views of the Series 2 structure (all dimensions in cm)

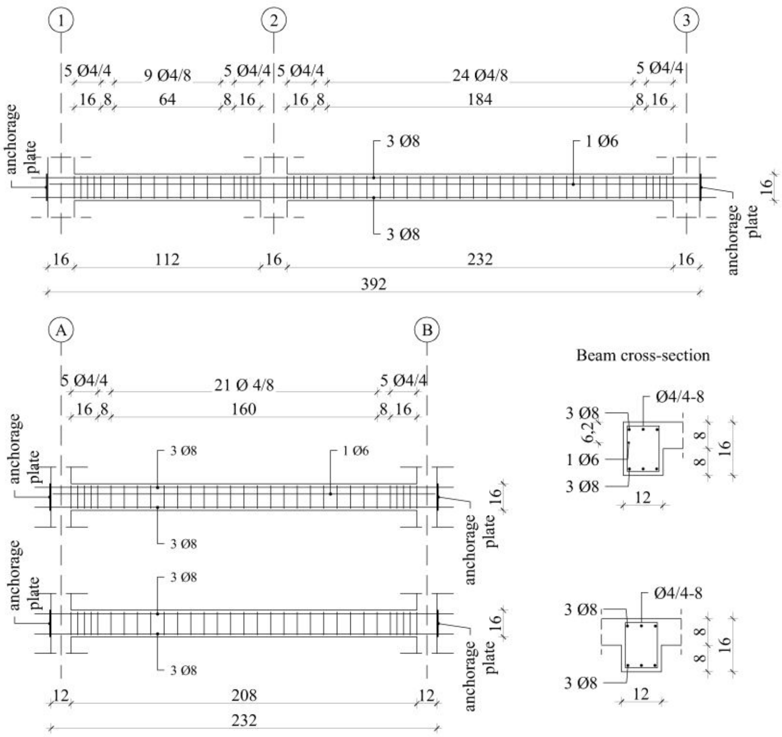

Figure 4. Beam reinforcement details (units in cm except for bar sizes)

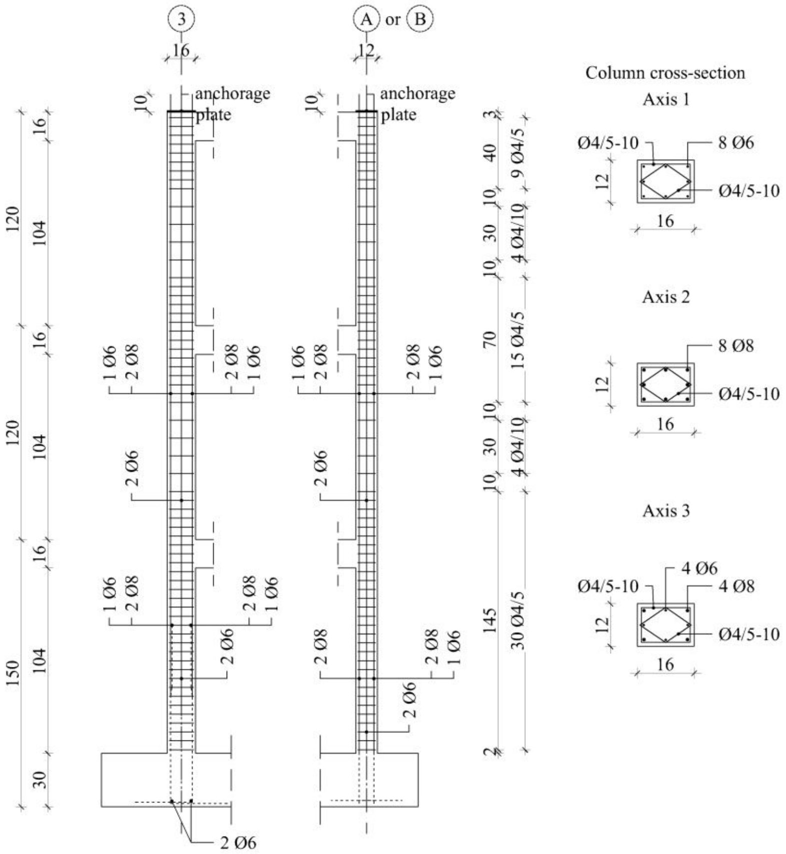

Figure 5. Column reinforcement details (units in cm except for bar sizes)

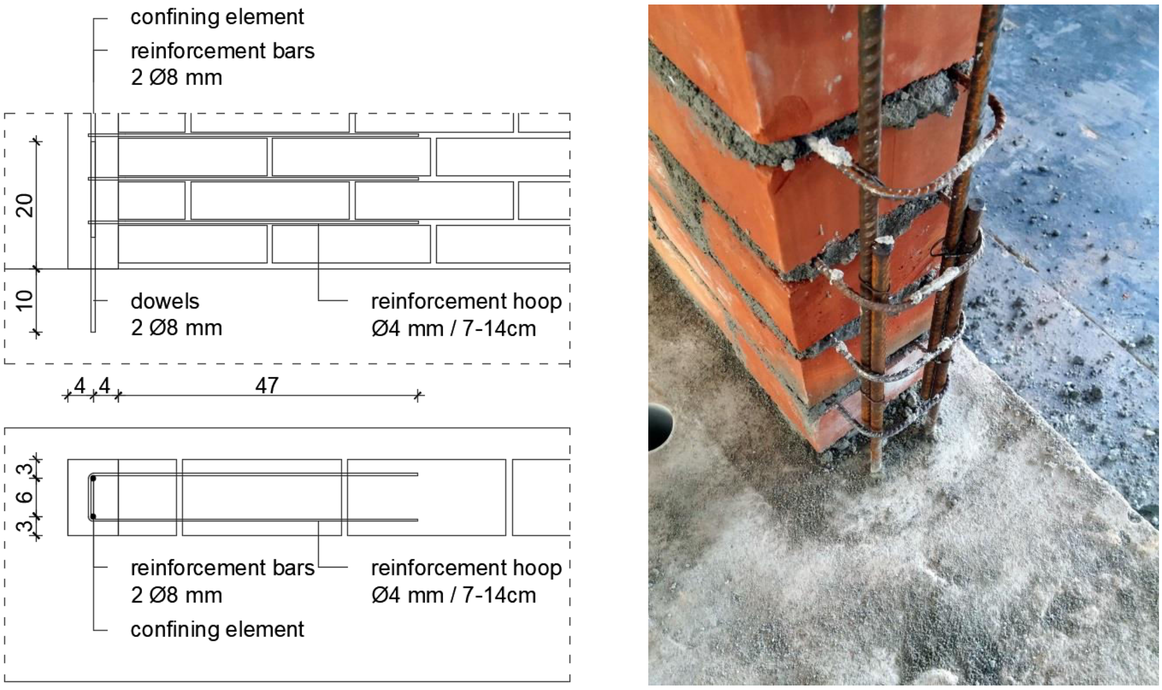

Figure 6. Confining element anchorage and reinforcement details in the cross-section (left bellow), in elevation (left above) and in structure (right)

Table 1. Mean values of material properties

Property	Value		Units
Concrete cylinder strength	36.6		MPa
Secant modulus of elasticity of concrete	38840		MPa
Reinforcing steel yield / ultimate tensile strength	Ø 4 mm	753 / 780	MPa
	Ø 6 mm	564 / 589	MPa
	Ø 8 mm	591 / 621	MPa
Masonry Units & Mortar	Series 1*	Series 2*	
Masonry unit net compressive strength	31.2	20.0	MPa
Masonry mortar compressive strength	10.6	10.6	MPa

*Note: Series 1- clay block masonry, Series 2- Solid clay brick masonry

3. SHAKING TABLE TESTS AND RESULTS

The north-south component of the record obtained at Herceg-Novi station during the 1979 Montenegro Earthquake was used as input excitation. To comply with common similitude practices, the duration of the excitation was reduced by dividing the time step by √2.5. The amplitude of excitation was scaled down to different peak ground accelerations (a_g / g). The base excitation was applied to the structure along its longitudinal (or x) direction in increasing intensity a_g / g: 0.05 g, 0.1 g, 0.2 g, 0.3 g, 0.4 g, 0.6 g, 0.7 g, 0.8 g, 1.0 g, 1.2 g (see Figure 7). An extra excitation at 1.4 g was included in the second series. Before and after each test run, cracks and damage were marked and recorded, and a free vibration test was conducted. To measure in-plane drift response, string potentiometers were installed and attached to the centerline of the slabs at each floor (Figure 3). To measure torsional response, pairs of accelerometers were installed in-plane on each floor slab on diagonally-opposite sides (near columns A1 and B3, see Figure 3).

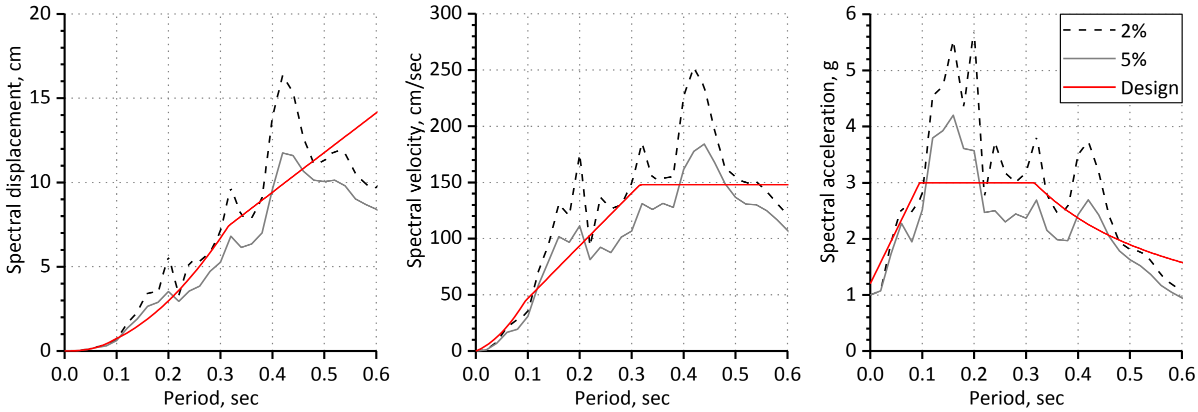

Figure 7. Linear response spectra for ground motion at PGA=1g (2% critical damping)

On the roof, two out-of-plane accelerometers also were installed at these locations. All sensors were sampled at 1000 Hz. To obtain out-of-plane displacements at the roof, the relative acceleration histories between the roof and foundation were double integrated. When determining the relative out-of-plane roof acceleration, the foundation acceleration histories in this direction were assumed to be zero because excitation was applied in-plane only.

Before integration, these relative acceleration histories were filtered using a fourth order bandpass filter using cutoffs 0.5 Hz and 400 Hz. Table 2 summarizes minimum and maximum roof displacements in both the out-of-plane and in-plane directions. In-plane displacements in this table were obtained from the difference between the string potentiometer at the roof and at the foundation. Also shown in Table 2 are the ratios of peak out-of-plane displacement measured in Series 1 to peak out-of-plane displacement in Series 2. On average, peak out-of-plane displacements were twice as large for the same motion in Series 1 as in Series 2. Some of this difference can be attributed to the fact that the test structure in Series 1 reached larger in-plane drifts than in Series 1. But even for motions that produced similar in-plane peak drifts, the test structure in Series 1 experienced larger out-of-plane demands. Consider the 20% amplitude run: both test structures reached a peak in-plane drift of 0.3 cm, but the peak out-of-plane drift was nearly twice as large in Series 1 (0.5 cm) as in Series 2 (0.26 cm). A similar comparison can be made between the 30% and 40% amplitude tests in Series 1 and the 70% tests in Series 2. During the 30% and 40% runs, the Series 1 structure reached a peak in-plane drift of 0.61 cm and peak out-of-plane drifts ranging from 0.6 to 0.93 cm. For comparison, the Series 2 structure reached a peak in-plane drift of 0.65 cm during the 70% run, but out-of-plane drifts were as small as 0.2 to 0.56 cm. The difference in out-of-plane motion can be attributed to differences in wall damage between the two longitudinal frames.

The test structures experienced negligible-to-slight damage in the first story walls (based on the definitions in (ATC 1998; Grünthal et al. 1998)) before 0.4 g. At 0.4 g in both test series, the separation of the infill wall and the surrounding frame was intensified. At 0.8 g in Series 1, in the first and second story infill walls inclined cracks appeared in several locations. They were formed in both directions in the shorter infill wall as a sign of its participation as a seismic resisting element. At the same intensity, in Series 2, cracking in the first and second story infills was more pronounced. The third story in both series had negligible damage during the testing. At the end of Series 1, the infill wall adjacent to the door opening had collapsed. Otherwise, the damage patterns in the infills were similar to what had been observed after the 0.8 g test. Figure 8 shows damage to the walls in frame A on the first floor after the final tests in both series. In Series 1, the masonry wall in frame A collapsed, while simultaneously the masonry wall in frame B separated from the frame. In contrast, the masonry walls in Series 2 had similar damage levels and stayed in place because of the vertical confining elements that were added during the repair. Not only were overall drift demands smaller in Series 2, but the more uniform damage state of the walls helped to reduce out-of-plane response (Figure 9).

Figure 8. Observed damage to frame A on the first floor after the final tests in both series: (left) Series 1, (right) Series 2

Table 2. Out-of-plane drift (from y-axis accelerometers) and in-plane-drift (from string potentiometers).

a_g / g		Series 1			Series 2			Series 1 / 2	
		Out-of-plane drift, cm		In-plane drift, cm	Out-of-plane drift, cm		In-plane drift, cm	Ratio of out-of-plane drifts, Series 1/2	
		Acc A1	Acc B3	SP	Acc A1	Acc B3	SP	Acc A1	Acc B3
10%	min	(0.30)	(0.29)	(0.18)	(0.15)	(0.13)	(0.21)		
	max	0.24	0.28	0.16	0.18	0.13	0.17		
	absmax	0.30	0.29	0.18	0.18	0.13	0.21	1.7	2.2
20%	min	(0.40)	(0.45)	(0.30)	(0.20)	(0.19)	(0.27)		
	max	0.50	0.44	0.29	0.26	0.16	0.30		
	absmax	0.50	0.45	0.30	0.26	0.19	0.30	1.9	1.6
30%	min	(0.59)	(0.60)	(0.61)	(0.27)	(0.16)	(0.42)		
	max	0.68	0.56	0.37	0.31	0.16	0.29		
	absmax	0.68	0.60	0.61	0.31	0.16	0.42	2.2	3.7
40%	min	(0.72)	(0.71)	(0.61)	(0.28)	(0.19)	(0.41)		
	max	0.93	0.69	0.57	0.47	0.17	0.34		
	absmax	0.93	0.71	0.61	0.47	0.19	0.41	2.0	3.7
60%	min	(0.69)	(0.71)	(1.07)	(0.34)	(0.22)	(0.45)		
	max	0.63	0.55	0.85	0.66	0.19	0.35		
	absmax	0.69	0.71	1.07	0.66	0.22	0.45	1.0	3.2
70%	min	(0.58)	(0.46)	(1.33)	(0.34)	(0.20)	(0.65)		
	max	0.79	0.46	1.70	0.56	0.19	0.52		
	absmax	0.79	0.46	1.70	0.56	0.20	0.65	1.4	2.3
80%	min	(0.68)	(0.54)	(1.64)	(0.34)	(0.18)	(0.99)		
	max	0.74	0.47	1.94	0.48	0.25	0.74		
	absmax	0.74	0.54	1.94	0.48	0.25	0.99	1.5	2.2
100%	min	(0.52)	(0.50)	(2.15)	(0.33)	(0.31)	(2.18)		
	max	0.88	0.47	3.36	0.60	0.28	2.30		
	absmax	0.88	0.50	3.36	0.60	0.31	2.30	1.5	1.6
120%	min	(0.64)	(0.50)	(3.80)	(0.45)	(0.51)	(2.72)		
	max	0.79	0.50	4.63	0.53	0.53	3.14		
	absmax	0.79	0.50	4.63	0.53	0.53	3.14	1.5	0.9

*absmax designates maximum absolute value between minimum and maximum drift vaules

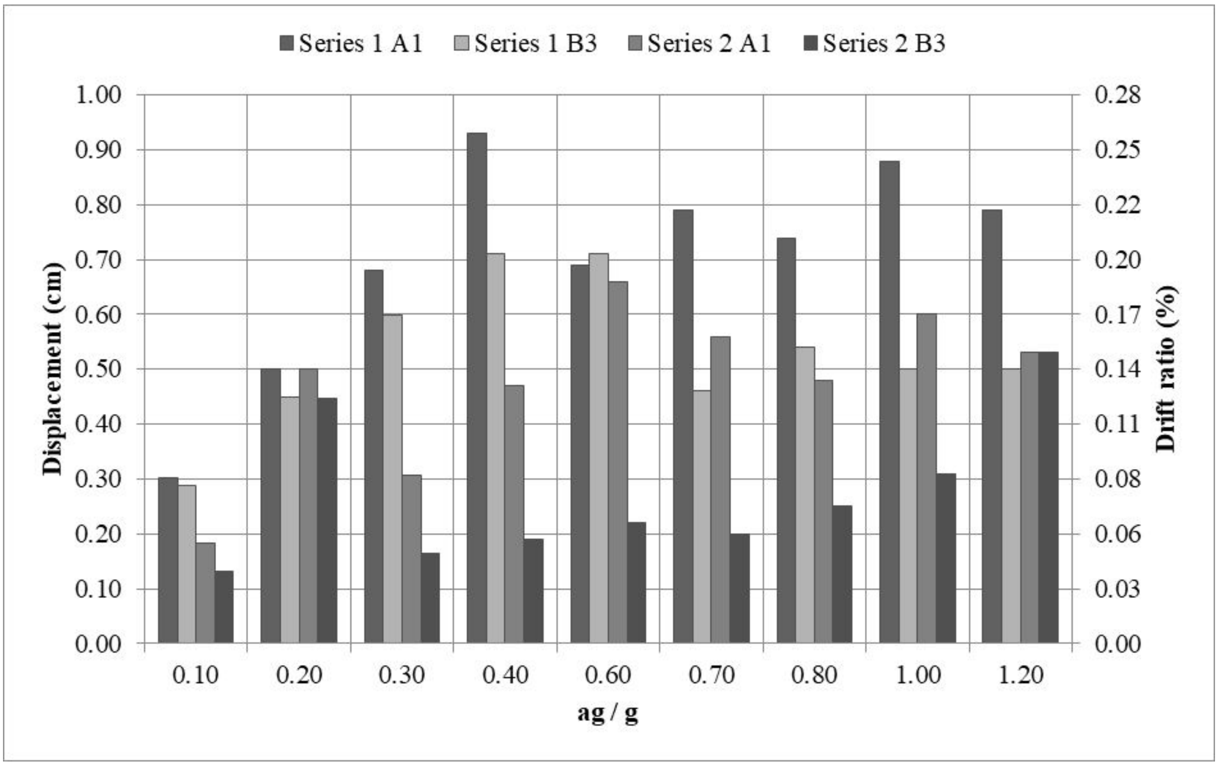

Figure 9. Out-of-plane displacements, cm (from y-axis accelerometers)

4. CONCLUSIONS

A 1/2.5-scale, three-story masonry infilled reinforced concrete frame structure was built and tested on a unidirectional shaking table. The structure was designed in compliance with EN1992-1-1:2004 and EN1998-1:2005 provisions. Interacting masonry infill walls had door openings in the ground story walls and window openings in upper story walls. The structure was tested under sequences of simulated earthquakes in two series. In Series 1, hollow clay block masonry infill walls were used. After completion of first series, the masonry infill walls in the first and second stories were replaced with solid brick masonry, including reinforced concrete confining elements along vertical opening edges. The tests were then repeated in the same sequence in Series 2. Displacements obtained by double-integrating out-of-plane acceleration histories showed that out-of-plane response of the test structure occurred in both series, even though the motion was applied along a single axis. This out-of-plane response was twice as large in Series 1 as in Series 2, owing in part to uneven damage to the masonry walls in the first story of the two longitudinal frames. During the test program, these walls detached from the adjacent columns and became ineffective at resisting the applied demand. In contrast, the additional confining boundary elements in Series 2 helped the walls stay active in resisting the applied demand and helped keep the damage state more uniform between the two frames. The result was a smaller torsional response. The tests highlight that vertical confining elements around openings in masonry infill walls can not only help reduce in-plane drift demands, but also can help minimize torsional response.

6. ACKNOWLEDGMENTS

The research presented in this article is a part of the research project FRAmed-MAsonry composites for modelling and standardization (HRZZ-IP-2013-11-3013) supported by Croatian Science Foundation and its support is gratefully acknowledged. The authors are grateful to Professor Vladimir Sigmund (1956 – 2016) for his significant contribution and direction towards the research described in this paper.

7. REFERENCES

Akin, L.A., (2006). *Behaviour of reinforced concrete frames with masonry infills in sesmic regions. PhD thesis*. Purdue University, West Lafayette, United States.

Aschheim, M.A., (2000). Kocaeli, Turkey, Earthquake of August 17, 1999: Reconnaissance Report. *Earthquake Spectra*, 16(S1), pp.1–461.

ATC, (1998). *Evaluation of earthquake damaged concrete and masonry wall buildings: basic procedures manual (FEMA 306)*, Redwood City, California.

CEN, (2004a). *Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings (EN 1992-1-1:2004)*, Brussels: European Committee for Standardization.

CEN, (2005). *Eurocode 6: Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures (EN 1996-1-1:2005)*, Brussels: European Committee for Standardization.

CEN, (2004b). *Eurocode 8: Design of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings (EN 1998-1:2004)*, Brussels: European Committee for Standardization.

Dolšek, M. & Fajfar, P., (2008). The effect of masonry infills on the seismic response of a four-storey reinforced concrete frame — a deterministic assessment. *Engineering Structures*, 30(7), pp.1991–2001.

Fardis, M.N. & Panagiotakos, T.B., (1997). Seismic Design and Response of Bare and Masonry-Infilled Reinforced Concrete Buildings Part I: Bare Structures. *Journal of Earthquake Engineering*, 1(1), pp.219–256.

Grünthal, G. et al., (1998). *European macroseismic scale 1998*, Luxembourg.

Hashemi, A. & Mosalam, K.M., (2006). Shake-table experiment on reinforced concrete structure containing masonry infill wall. *Earthquake Engineering and Structural Dynamics*, 35(14), pp.1827–1852.

Hassan, A.F. & Sozen, M.A., (1997). Seismic vulnerability assessment of low-rise buildings in regions with infrequent earthquakes. *Structural Journal*, 94(1), pp.31–39.

Henderson, R.C. et al., (2003). Summary of a Large- and Small-Scale Unreinforced Masonry Infill Test Program. *Journal of Structural Engineering*, 129, pp.1667–1675.

Kyriakides, M.A. & Billington, S.L., (2008). Seismic Retrofit Of Masonry-Infilled Non-Ductile Reinforced Concrete Frames Using Sprayable Ductile Fiber-Reinforced Cementitious Composites. In *14th World Conference on Earthquake Engineering*. Beijing, China, p. Paper Number 05-04-0063.

Mehrabi, A.B. & Shing, P.B., (1997). Finite Element Modeling of Masonry-Infilled RC Frames. *Journal of Structural Engineering*, 123(5), pp.604–613.

Murty, C.V.R. et al., (2006). *At Risk: The Seismic Performance of Reinforced Concrete Frame Buildings with Masonry Infill Walls*,

Necevska-Cvetanovska, G., Sendova, V. & Apostolska, R., (2015). *FRAme - MAsonry Composites for Modelling and Standardization (Framed-Masonry). Report IZIIS 2015 - 31*, Skopje, Macedonia.

Negro, P. & Colombo, A., (1997). Irregularities induced by nonstructural masonry panels in framed buildings. *Engineering Structures*, 19(7), pp.576–585.

Pujol, S. & Fick, D., (2010). The test of a full-scale three-story RC structure with masonry infill walls. *Engineering Structures*, 32(10), pp.3112–3121.

Sezen, H. et al., (2003). Performance of reinforced concrete buildings during the August 17, 1999 Kocaeli, Turkey earthquake, and seismic design and construction practise in Turkey. *Engineering Structures*, 25(1), pp.103–114.