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SEISMIC EVALUATION OF THREE STORY MASONRY INFILLED REINFORCED CONCRETE FRAME MODEL STRUCTURE

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Abstract

Unrealistic or inadequate assessment of buildings may not identify the true failure modes, leading to unsafe designs or may produce overly conservative retrofits where none is needed to meet the desired performance objective. Therefore, more sophisticated methods that consider both the actual loading and inelastic response buildings experience should be used. This paper provides a performance based seismic evaluation of existing masonry infilled model concrete building. It is performance based: the evaluation and retrofit design criteria are expressed as performance objectives, which define desired levels of seismic performance when the building is subjected to specific levels of seismic ground motion.

So, the approach used in this paper is somewhat reverse: it starts with the observed/measured performance of test building subjected to series of shake table tests. The model structure is a three-story, 1:2,5 scaled reinforced concrete frame building with masonry infill walls subjected to ground motions of increasing intensity in two series. First hollow masonry units were used. After completion of tests, the structure was repaired by replacing hollow masonry units with new solid units. Reinforced concrete end elements were also added along the vertical edges of window and door openings in the first and second stories. The repaired structure was then tested with the same sequence of base motions. The objective of this second test series was to investigate whether the repaired, framed-masonry system would have sufficient capacity to sustain the applied earthquake demands. Acceptable performance is measured by the level of structural and/or non-structural damage recorded during the earthquake shaking.

Once the resulting performance of the test building using the acceptability criteria is defined, the performance point for a demand earthquake is checked. The performance is checked on two levels. First, there are global limits for displacement of the structure for each performance objective. Similarly, individual structural elements were checked against acceptability limits which depend on the global performance goal. The entire process of generating the capacity curve and dealing directly with the interdependence of capacity and demand gives a greatly enhanced understanding of the actual performance of the specific building. This enables the engineer to apply the necessary experience and judgment at a much more refined level then traditional procedures.

Keywords: seismic evaluation; performance; rc masonry infilled frame building; shake table test;

1. Introduction

In the new buildings designed in accordance with the European Standard, EN 1998 [1], the masonry infill is threat as a source of structural additional strength and so called "second line defense" [2]. However, the reduction of input seismic action, as a result of possible favorable infill effects, is not allowed. The problem is even more complex in the case of seismic performance evaluation of the existing reinforced concrete buildings with masonry infill. The influence of the infill is most significant when the structural system itself doesn't possess adequate seismic resistance, which is often case in large number of substandard reinforced concrete buildings constructed before the implementation of seismic codes, as well as in the case of newly designed buildings without respecting capacity design approach. In such buildings the explicitly consideration of infill in analytical model and their verification are necessary [3].

Additionally, due to variation in characteristics of ground movement (earthquakes), which is sensitive to geological conditions, the effects on structure will also differ greatly [4]. The framed-masonry building performance under earthquake event of arbitrary intensity and direction is particularly sensitive to following requirements which should be allowed for in design:

- In-plane and out-of-plane frame-infill interaction e.g. [5]
- Distribution of framed masonry walls throughout the building
- Presence of opening type, size and position in the wall e.g. [6,7]
- High uncertainties related to masonry characteristics (workmanship) e.g. [8]

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 tests of a full scale three-story structure have demonstrated that drift ratios of that magnitude can be achieved by a reinforced concrete frame with solid filler walls provided the columns have the ability to sustain the required shear force under reversals of shear and axial forces (Figure 1).

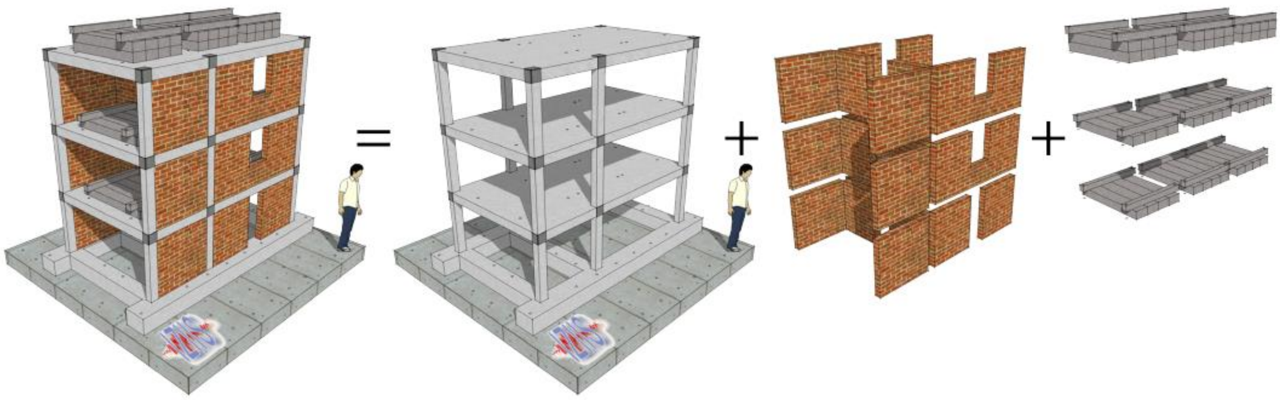

Fig. 1 – General constituent parts of the masonry infilled frame model

The aim of this research was to investigate, through dynamic earthquake simulation tests, the safety and behavior of reinforced concrete frame system containing infill masonry walls since these systems serve both architectural and structural demands efficiently. Moreover, the majority of population in earthquake-prone zones live and will continue to live in such buildings, of which safety rely on frame-wall composites. The research was initiated by tests of regular 1/2.5 scale three-story reinforced concrete framed-masonry building, tested on the shaking table (Figure 2). Two series of tests were considered: in the first series, the structure had unreinforced clay block masonry walls; in the second, the masonry walls in the first and the second stories were replaced by solid brick walls and vertical reinforced confining elements were added around openings. In both series, the structure was subjected to sequences of ten ground movements of

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increasing intensity from 0.05 up to 1.2 g along a single axis. Conclusions are given in the form of force-displacement capacity diagrams of the overall structure as well as of the building performance levels estimation, with respect to ground movement intensity.

Fig. 2 – Series 1 model building during testing on the shaking table

2. Model building

The 1/2.5 scale three-story framed-masonry building was designed and constructed in compliance with EN1992-1-1:2004 [9] and EN1998-1:2005 [1] provisions as moment-resisting frames by considering the medium ductility form of seismic construction detailing. Masonry walls were made of clay block masonry units and masonry mortar which satisfy earthquake resistant design requirements for unreinforced structural masonry. In the first series of tests framed masonry walls were built of clay block masonry units and general purpose masonry mortar. In the second series of tests, the framed masonry walls in the first and second story were replaced by solid clay brick masonry including reinforced concrete confining elements along vertical opening edges while opening sizes and mortar joint thickness were kept the same [10]. The test structure dimensions, cross-sections and some reinforcement details are given in Figure 3 through Figure 6. The in-built material characteristics are described in Table 1.

Table 1 – Chemical composition of cement samples

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			

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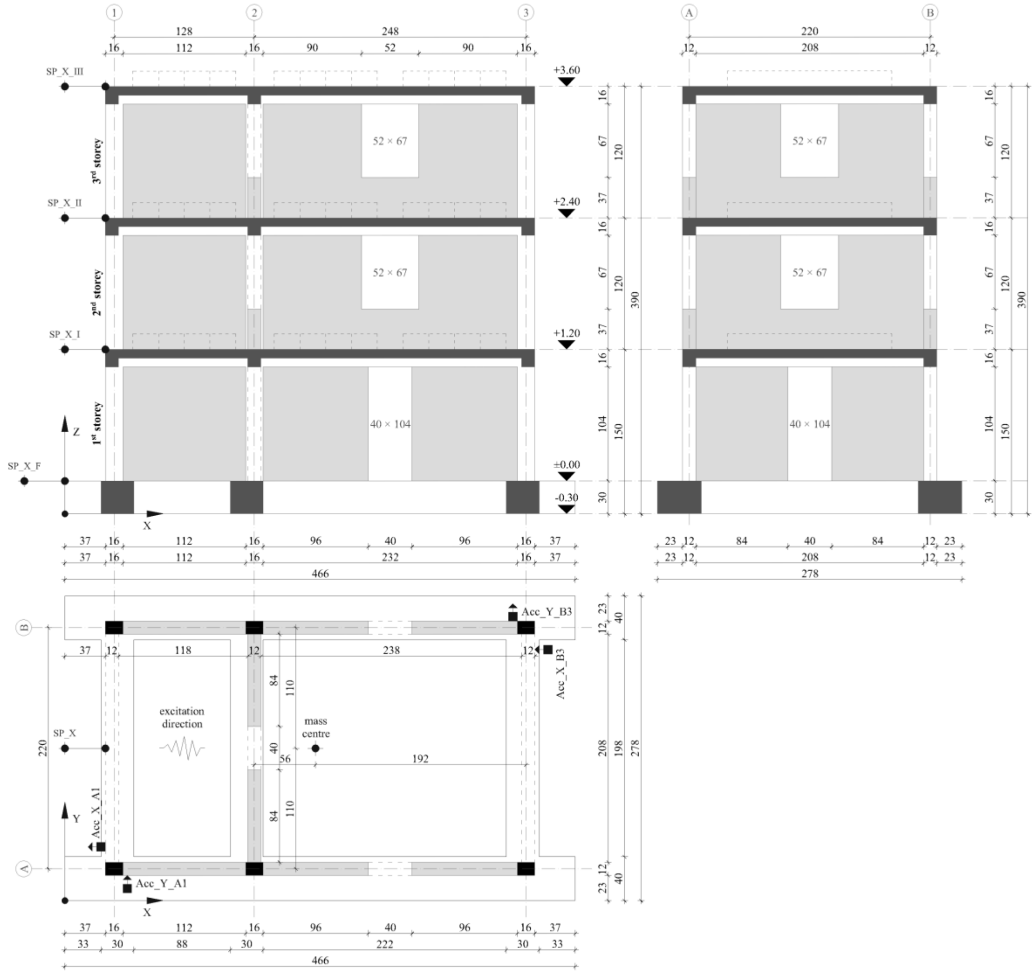

Fig. 3 – Longitudinal (top left), transverse (top right) and instrument plan views of the Series 1 structure (all dimensions in cm) [10,11]

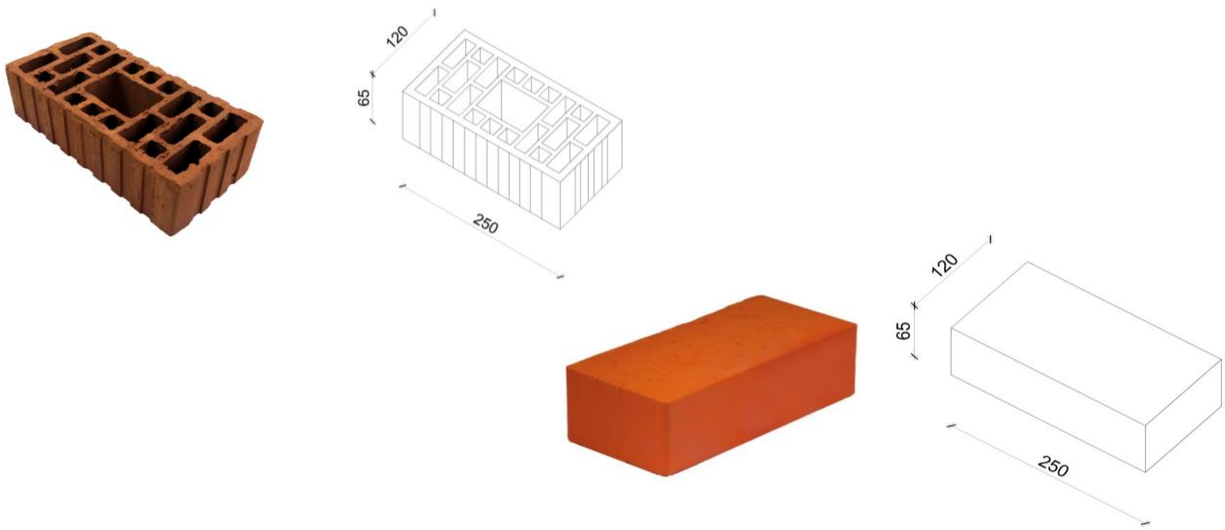

Fig. 4 – Hollow and solid clay masonry units used in tests (all units in mm)

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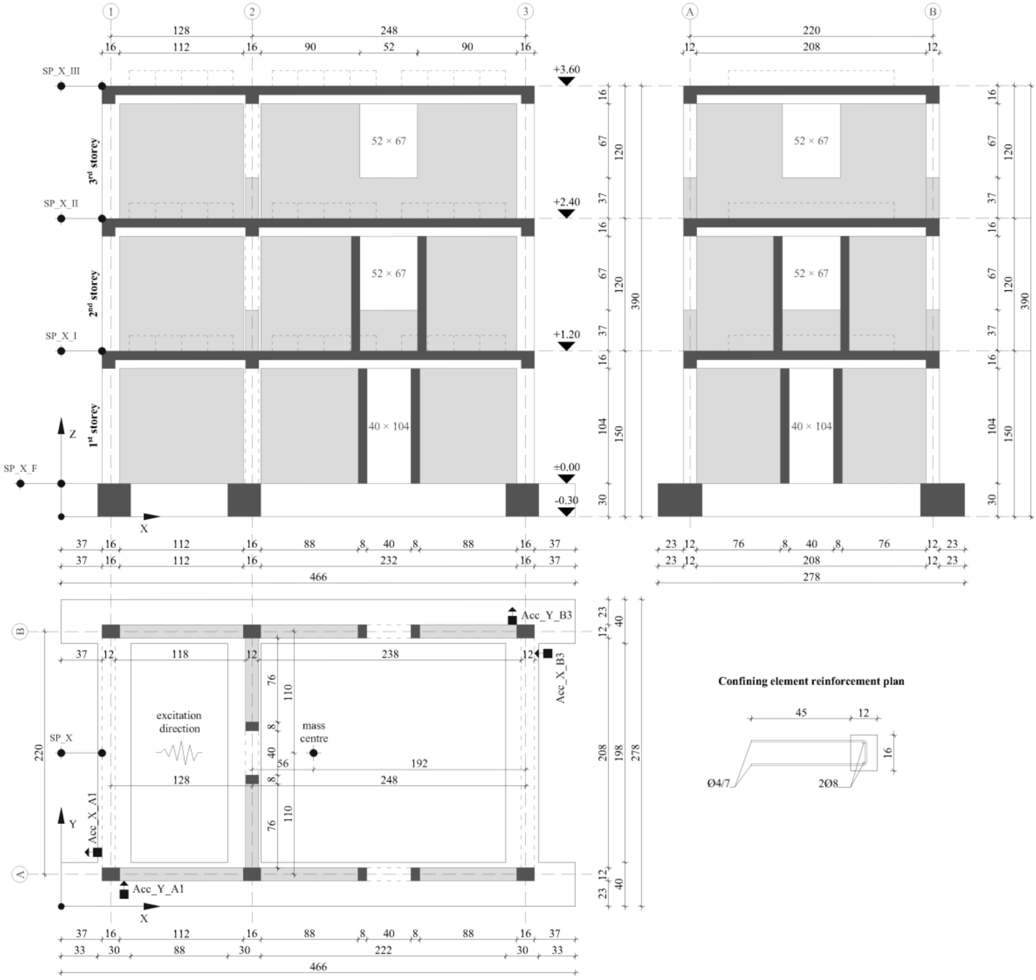

Fig. 5 – Longitudinal (top left), transverse (top right) and instrument plan views of the Series 2 structure (all dimensions in cm) [10,11]

The Cauchy-Froude similitude rules were applied in which the prototype and the model construction materials are the same. The total masses of the Series 1 and 2 buildings were 29.2 and 30.6 tons, respectively. The model building was built and tested at the IZIIS laboratory in Skopje, Macedonia [12].

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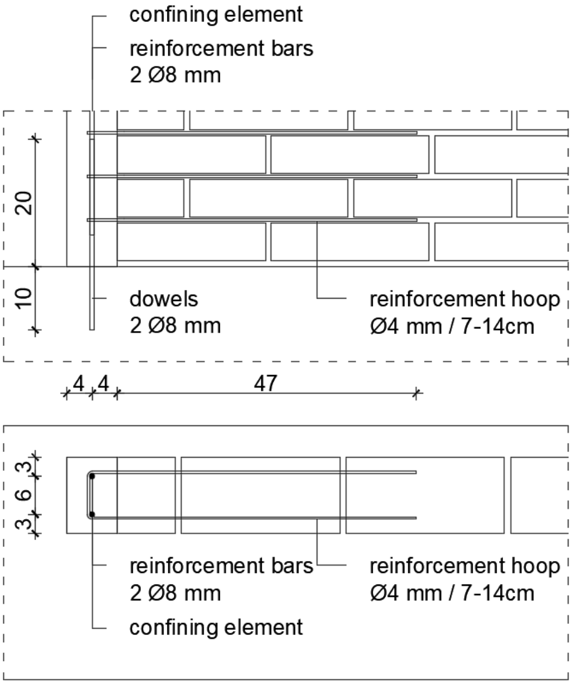

Fig. 6 – Confining element anchorage and reinforcement details in the cross-section (left bellow), in elevation (left above) and in structure (right) [10,11]

3. Ground motion

The ground motion record used for the shake table test was the ground motion recorded at the Herceg-Novi station during the 15th April 1979 Montenegro earthquake. The earthquake had a moment magnitude of 6.9 and a hypocentral depth of 12 km. To account for the fact that the structure is constructed at 1:2.5 scale, the record was scaled in time by reducing the duration by a factor $2.5^{1/2}$. The record was base line corrected and then scaled to match the different levels of peak ground acceleration (PGA) that were used as input signals for the shake table test. Shaking table testing of the scaled model has been performed by applying the same excitation record with the gradual increase of the intensity of input earthquake excitations. It was aimed at monitoring the progressive development of cracks and phases of dynamic behavior of the model, i.e., defining the elasticity limit (occurrence of the first cracks) until failure of the infill walls and/or RC frame.

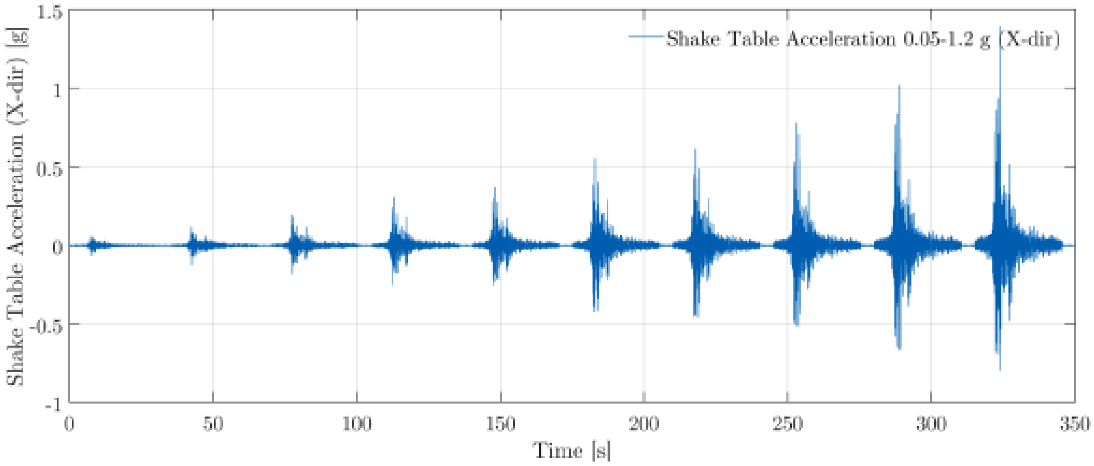

Fig. 7 – Shake table acceleration (N–S direction) at 10 different intensity levels

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The ground motion was applied to the building along its longitudinal (or x / N-S) direction in following sequence using following a_g / g values: 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 sequence, cracks and damage were marked and recorded. Measuring equipment in the building consisted of string potentiometers attached to the centreline of the slabs at each floor, and accelerometers mounted on each floor slab.

4. Tests and results

The shake table test on the three-story framed-masonry building structure constructed at 1:2.5 scale allowed studying the performances of such a structure for a large range of ground motion intensities. The first runs with small peak ground accelerations caused only small and few cracks. The final run, on the contrary, brought the structure very close to collapse as all in-plane loaded unreinforced masonry walls of the first and second story were heavily damaged. RC frame structure had few negligible damages. They were mostly concentrated on the beams and joints that had very dense reinforcement. None of them endangered the vertical stability of the structure. Figure 10 shows damage to the masonry 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. The observations were made and documented after each test. The behaviour of the structure showed that the masonry infill wall had a significant effect on the behaviour of the RC frame structure. Their effect diminished as the damage grade achieved higher levels.

Table 2 – Summary of selected peak response values for Test Series 1

Run / PGA	Max. Measured Roof Displacement (mm)	Max. Measured Roof Drift Ratio (%)
0,05 g	0,71	0,02
0,10 g	1,74	0,04
0,20 g	2,88	0,07
0,30 g	4,02	0,10
0,40 g	6,08	0,16
0,60 g	8,99	0,23
0,70 g	16,06	0,41
0,80 g	19,92	0,51
1,00 g	32,70	0,84
1,20 g	45,32	1,16

Table 3 – Summary of selected peak response values for Test Series 2 (after repair)

Run / PGA	Max. Measured Roof Displacement (mm)	Max. Measured Roof Drift Ratio (%)
0,05 g	0,85	0,02
0,10 g	1,72	0,04
0,20 g	2,56	0,07
0,30 g	3,09	0,08
0,40 g	3,97	0,10
0,60 g	5,35	0,14
0,70 g	6,78	0,17
0,80 g	9,37	0,24
1,00 g	23,37	0,60
1,20 g	31,73	0,81

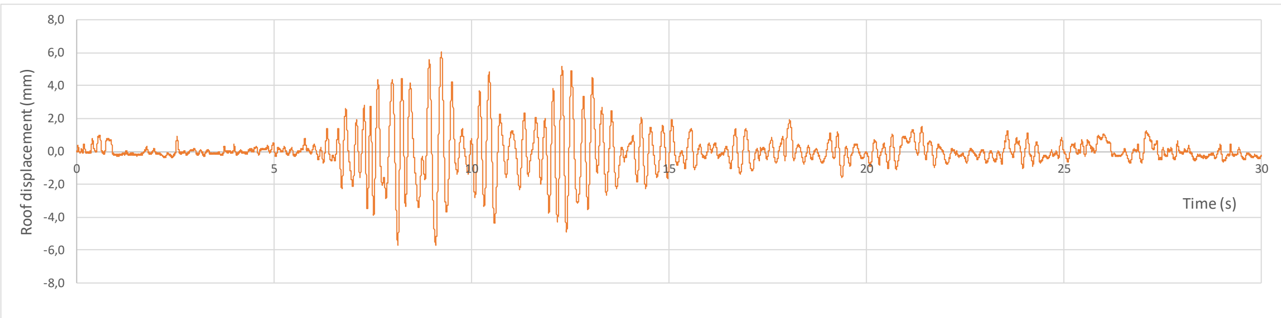

Fig. 8 – Time history of the roof displacement for PGA=0,40g, Series 1

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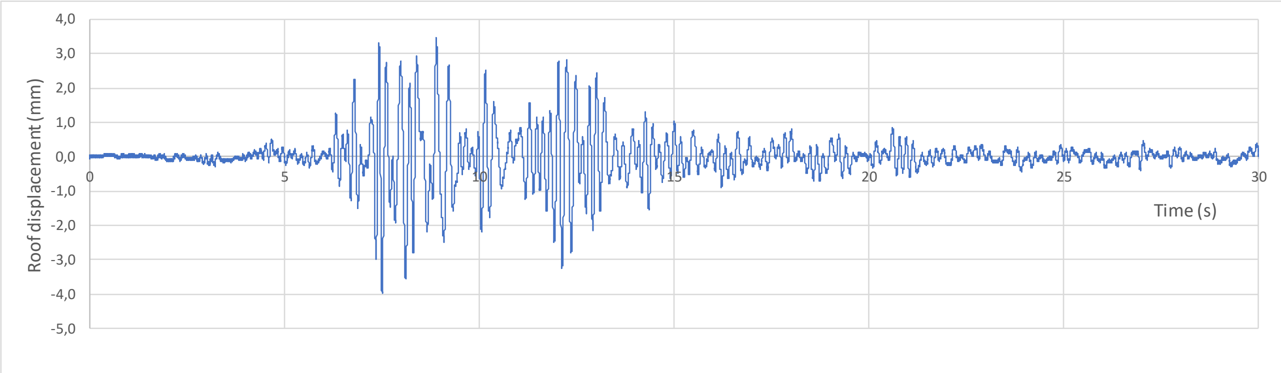

Fig. 9 – Time history of the roof displacement for PGA=0,40g, Series 2

Fig. 10 – Observed damage to frame A on the first floor after the final tests in both series: (left) Series 1, (right) Series 2

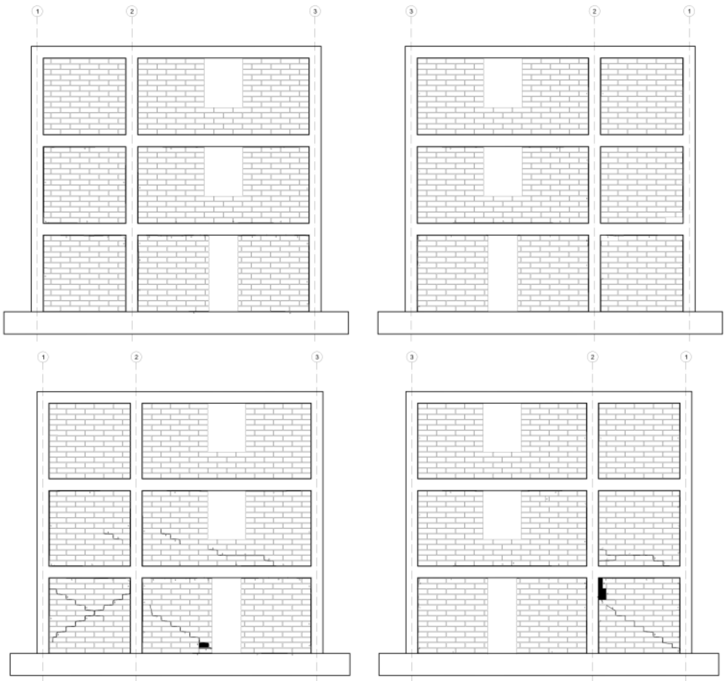

Fig. 11 – Observed in-plane damage after the 0.4 g and 0.8 g in frames A (left) and B (right) in Series 1

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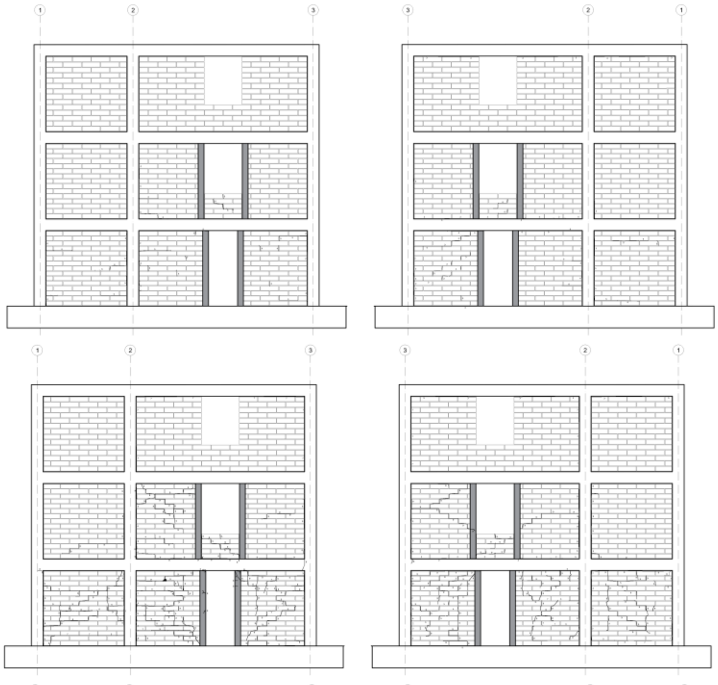

Fig. 12 – Observed in-plane damage after the 0.4 g and 0.8 g in frames A (left) and B (right) in Series 2

5. Performance based seismic evaluation of the tested model structure

Two key elements of performance based evaluation procedure are demand and capacity. Demand is a representation of the earthquake ground motion. Capacity is a representation of the structure's ability to resist the seismic demand. The performance is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist the demands of the earthquake such that the performance of the structure is compatible with the objectives of the design/evaluation. Simplified nonlinear analysis procedures using pushover methods require determination of three primary elements: capacity, demand (displacement) and performance [13].

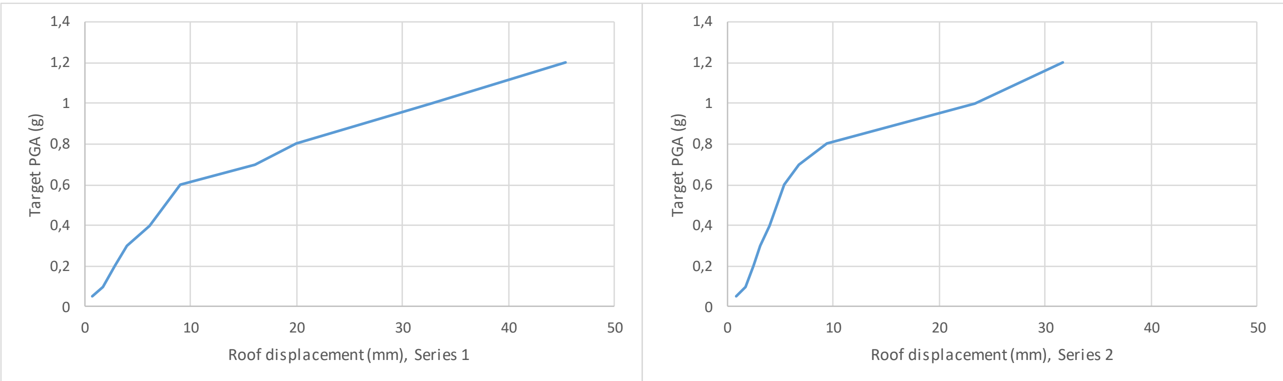

Fig. 13 – Force displacement curves for Series 1 and Series 2 model structure, respectively

The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. In order to determine capacities beyond the elastic limits, some form of nonlinear analysis, such as the pushover procedure, is required. In our case, the pushover curve is constructed by joining the points of maximum top story drifts for a given ground shaking intensity lined up in ascending order. Every next step of this procedure continues to the previous one taking into account for

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reduced resistance of yielding components (if any). Basically, this procedure uses a series of sequential maximum top displacements joined to approximate a force-displacement capacity diagram of the overall structure (Figure 13). As for the demand (displacement), for a given model structure and ground motion, it is a measured maximum response of the building during the ground motion. Once a capacity curve and demand displacement are determined, a performance check can be done. A performance check verifies that structural and nonstructural components are not damaged beyond the acceptable limits of the performance objective for the forces and displacements implied by the displacement demand (Figure 14).

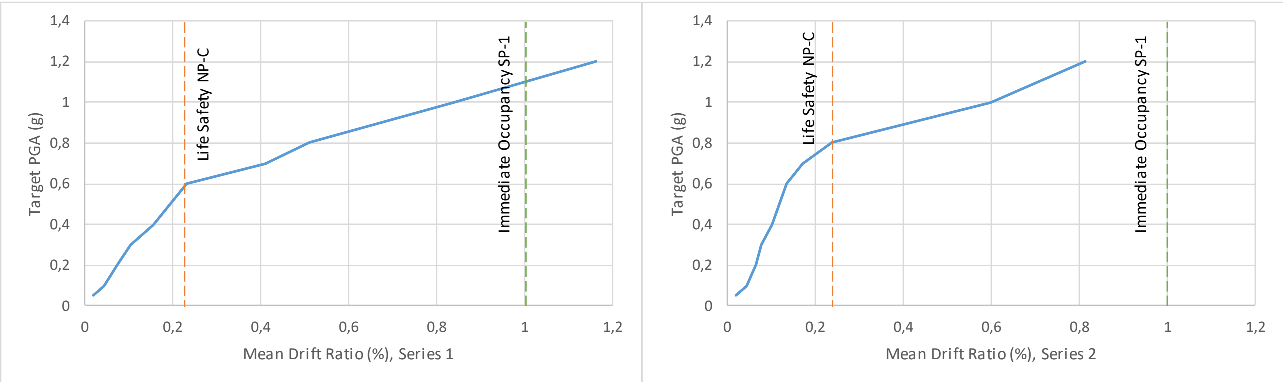

Fig. 14 – Force displacement curves in terms of MDR for Series 1 and Series 2 model structure, respect.

A performance objective specifies the desired seismic performance of the building and may include consideration of damage states for several levels of ground motion and would then be termed a dual or multiple level performance objective. A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post-earthquake serviceability of the building. Target performance levels for structural and nonstructural systems are specified independently. Building Performance Levels are a combination of a structural performance level and a nonstructural performance level and are designated by the applicable number and letter combination such as 1-A, 3-C, etc., as shown in Table 4.

Table 4 – Combinations of Structural and Nonstructural Performance Levels to Form Building Performance Levels (according to ATC 40) [13]

Building Performance Levels						
	Structural Performance Levels					
Nonstructural Performance Levels	SP-1 Immediate Occupancy ↓	SP-2 Damage Control (Range) ↓	SP-3 Life Safety ↓	SP-4 Limited Safety (Range) ↓	SP-5 Structural Stability ↓	SP-6 Not Considered ↓
NP-A Operational →	1-A Operational	2-A	NR	NR	NR	NR
NP-B Immediate Occupancy →	1-B Immediate Occupancy	2-B	3-B	NR	NR	NR
NP-C Life Safety →	1-C	2-C	3-C Life Safety	4-C	5-C	6-C
NP-D Hazards Reduced →	NR	2-D	3-D	4-D	5-D	6-D
NP-E Not Considered →	NR	NR	3-E	4-E	5-E Structural Stability	Not Applicable

Legend

(white)	Commonly referenced Building Performance Levels (SP-NP)
(grey)	Other possible combinations of SP-NP
NR	Not recommended combinations of SP-NP

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6. Conclusions

The three-story 1:2,5 scaled framed-masonry building was tested on a shaking table under sequences of ground motion of increasing intensity. Tests were divided in two series from which Series 1 included clay block and Series 2 clay brick masonry walls and confining elements at opening edges in the first and the second story (existing walls were replaced).

The consideration was given to the measured response of the regular building through resulting performance of the test building using the acceptability criteria. The first runs with small peak ground accelerations caused only small and few cracks. The final run, on the contrary, brought the structure very close to collapse as all in-plane loaded unreinforced masonry walls of the first and second story were heavily damaged. RC frame structure had few negligible damages. They were mostly concentrated on the beams and joints that had very dense reinforcement. None of them endangered the vertical stability of the structure. Performance objectives are summarized in Table 5 and Figure 14, and following observations can be made:

- Reinforced concrete frames, designed and constructed in compliance with EN1992-1-1:2004 [9] and EN1998-1:2005 [1] provisions as moment-resisting frames by considering the medium ductility form of seismic construction detailing, behaved well, with only a few minor damages while keeping its drift ratios bellow 1,0%; that way they maintained the structural performance level Operational.
- The infill used contributed significantly in limiting the overall drifts to less the 1,0% of building height. However, due to quite brittle behavior of the infill, for high seismicity, building performance level is mainly in the Damage Control range.
- In Series 2, not only were overall displacements smaller, but the more uniform damage state of the walls helped to increase the overall building performance level because of the higher ground motion intensity.

Table 5 – Estimated Building Performance Levels for characteristic PGA based on top floor drift ratio

Series 1	Structural Performance Levels	
Nonstructural Performance Levels	SP – 1 Immediate Occupancy	SP – 2 Damage Control
NP-A Operational	1-A (0,2g) Operational	
NP-B Immediate Occupancy	1-B (0,4g) Immediate Occupancy	
NP-C Life Safety	1 – C (0,6g)	
NP-D Hazards Reduced		2 – D (>0,6g)

Series 2	Structural Performance Levels	
Nonstructural Performance Levels	SP – 1 Immediate Occupancy	SP – 2 Damage Control
NP-A Operational	1-A (0,2g) Operational	
NP-B Immediate Occupancy	1-B (0,6g) Immediate Occupancy	
NP-C Life Safety	1 – C (0,8g)	
NP-D Hazards Reduced		2 – D (>0,8g)

The estimations given in table 5 are only informative focusing mainly on drifts measured during the series of tests and need to be improved by more detailed (and quantified) descriptions of the particular nonstructural performance levels and ranges. However, given the fact that the majority of population in earthquake-prone zones live and will continue to live in such buildings, of which safety rely on frame-wall composites, both constitutive elements deserve to be considered jointly and carefully, as a whole. Also, based on recent earthquakes in our region, in moderate seismicity zones and in low and medium height buildings, majority of injuries and economic losses were due to collapse of masonry infill walls.

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