

Article

Seismic Evaluation and Strengthening of an Existing Masonry Building in Sarajevo, B&H

Naida Ademović ^{1,*} , Daniel V. Oliveira ² and Paulo B. Lourenço ²

¹ Faculty of Civil Engineering in Sarajevo, University of Sarajevo, Sarajevo 71 000, Bosnia and Herzegovina

² ISE, Institute of Science and Innovation for Bio-Sustainability (IB-S), Dept. of Civil Engineering, University of Minho, Guimarães 4800-058, Portugal; danvco@civil.uminho.pt (D.V.O.); pbl@civil.uminho.pt (P.B.L.)

* Correspondence: naidadem@yahoo.com; Tel.: +387-33-278-498

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Abstract: A significant number of old unreinforced load-bearing masonry (URM) buildings exist in many countries worldwide, but especially in Europe. In particular, Bosnia and Herzegovina has an important stock of masonry buildings constructed from the 1920s until the 1960s without application of any seismic code, due to their nonexistence at that time. With the 1963 Skopje earthquake, this class of buildings were shown to be rather vulnerable to seismic actions, which exhibited serious damage. This article assesses the seismic vulnerability of a typical multi-storey residential unreinforced load-bearing masonry building located in the heart of Sarajevo, which may be exposed to an earthquake of magnitude up to 6 by Richter's scale. The buildings of this kind make up to 6% of the entire housing stock in the urban region of Sarajevo, while in Slovenia this percentage is much higher (around 30%). The analysis of a typical building located in Sarajevo revealed its drawbacks and the need for some kind of strengthening intervention to be implemented. Additionally, many structures of this type are overstressed by one to two additional floors (not the case of the analyzed structure) constructed from 1996 onwards. This was due to the massive population increase in the city center of Sarajevo and further increased the vulnerability of these buildings.

Keywords: unreinforced load-bearing masonry; strengthening intervention; non-linear analysis

1. Introduction

Bosnia and Herzegovina lie in the heart of South Eastern Europe, which is marked as one of the European regions with rather complex tectonic formations. Many researchers investigated this region, i.e., Northwestern Balkans, Northwest Bosnia and Herzegovina. One of the features which makes South Eastern Europe interesting for researchers, is the fact that all the earthquakes that struck this region until now were earthquakes with a shallow focus. Uniform hazard spectra for this region were produced by several researchers [1,2] after the study of the influence of deep and shallow geology. In 2018, the seismic hazard map for Bosnia and Herzegovina, defined in terms of peak ground acceleration (PGA) with a return period of 475 years (Figure 1), as defined in Eurocode 8, was constructed, and is now part of the National Annex in BAS EN 1998-1: 2018 [3].

When considering 1944 recorded earthquakes, the focal depth of 64% of them is only 10 km, while earthquakes with a focal depth in the range from 11 to 20 km make up 30% of the sample and around 4.5% of the earthquakes were with a focal depth in the range from 21 to 30 km; which leaves just 1.7% with a focal depth larger than 30 km [4]. The shallow focus is one of the most destructive features of earthquakes in Bosnia and Herzegovina [5,6].

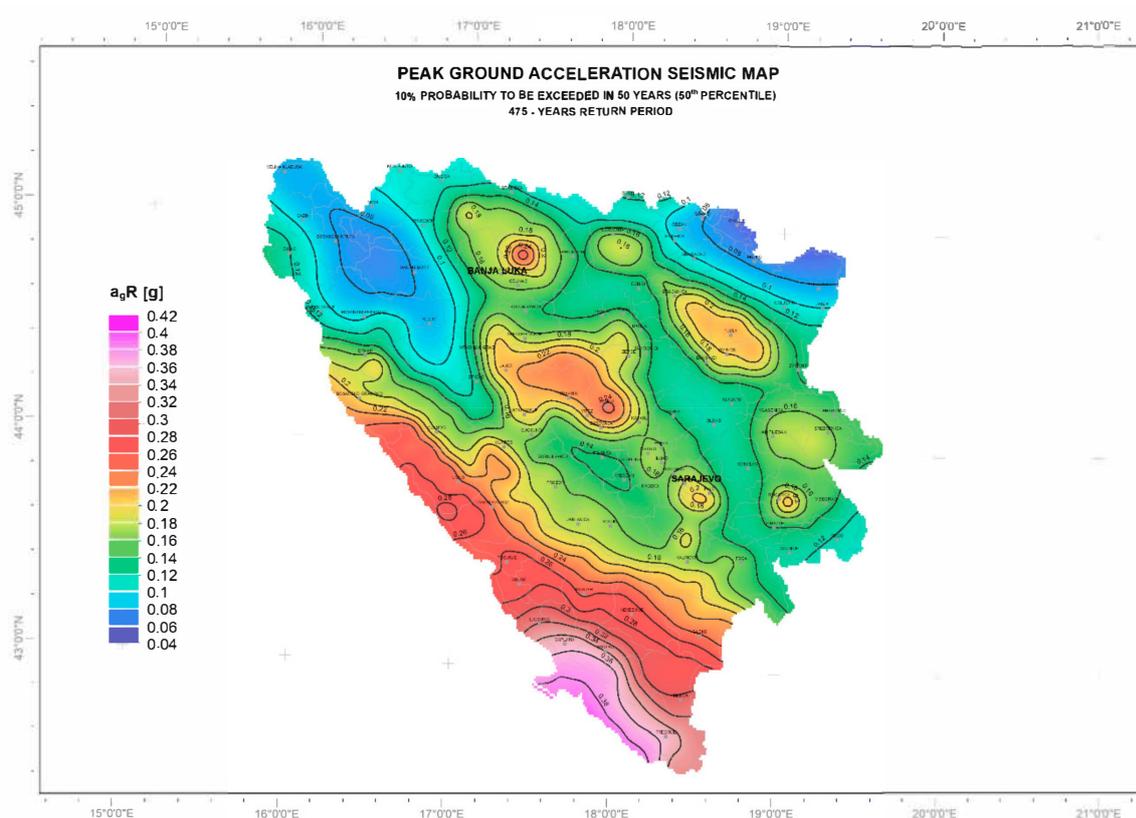


Figure 1. Seismic hazard map for Bosnia and Herzegovina in terms of peak ground acceleration (PGA) (475 years return period) [3].

In this paper, a building structure built in 1957 and typical of the Balkans region is elaborated. These type of buildings can be found in all towns in Slovenia (Figure 2a) and they make up to 30% of the entire housing stock in Slovenia [7]. According to the national census [8], the buildings of this type make up 6% of the housing in the urban part of the Sarajevo city center. This construction type was built from 1920 until 1963 in the entire region of ex-Yugoslavia, especially in the Republic of Macedonia, Montenegro, and Bosnia and Herzegovina, mainly in the urban areas. At that time, no seismic codes existed in this region, so these structures were constructed with no seismic regulations. The 1963 Skopje earthquake showed the drawbacks of this type of structures (Figure 2b), which were severely damaged due to inadequate wall concentration in the predominant earthquake direction, low resistance of the loadbearing system, high height of the unreinforced masonry structure (URM), etc. It is only after this catastrophic event that these type of structures were addressed in the first Temporary Seismic Code produced in 1964 and later upgraded after the 1969 Banja Luka earthquake (which had a focal depth of only 25 km, and a magnitude of 6.4 by Richter's scale). This was one of the most devastating earthquakes registered in this region. Due to the vast devastation caused by the Banja Luka earthquake, micro-zonation of the urban part of the city was done in 1972. According to that data, the expected average ground acceleration for Banja Luka is 0.18g [9]. Regardless of this fact, the area of Banja Luka in the seismological map of Bosnia and Herzegovina was defined as an area of maximum registered intensity of 9 by the Mercalli–Cancani–Sieberg intensity scale [10] for a return period of 500 years. More recently, Lee et al. [11] conducted the micro-zonation of Banja Luka in the light of performance-based earthquake-resistant design.



Figure 2. (a) Typical building in Slovenia; (b) devastation of a typical building after the 1963 Skopje earthquake [4].

According to the new seismic hazard code for Bosnia and Herzegovina, defined in terms of peak ground acceleration (PGA) with a return period of 475 years (Figure 1), Sarajevo can experience earthquakes with a peak ground acceleration of $0.18g$ [3]. Such conditions make this structure rather vulnerable as it was constructed without the application of any seismic measures.

As this structure is typical for the wider region of the Western Balkans, the main purpose of this research is to investigate the seismic safety of the building if exposed to ground motions compatible with Eurocode 8 [3]. On the basis of obtained numerical results, possible intervention measures for improving the seismic response of the structure are also analyzed and proposed.

2. Description of the Case Study

The analyzed building is located in the urban part of Sarajevo (Figure 3a,b) and it has a rectangular plan, with length $L = 38$ m and width $B = 13$ m (Figure 3c). The total height of the structure with the basement is 21 m (Figure 3d). The main load bearing walls are located in the Y direction, while the load bearing walls in the X direction are attenuated by many openings. The structure is composed of a basement made of reinforced concrete walls. Dimensions of the walls differ and the inner walls in the Y direction are 38 cm thick, while in the X direction the thickness of the outer walls is 30 cm and of two inner walls is 25 cm. The walls in both directions on all the floors are made of solid brick masonry (dimensions of bricks $250 \times 120 \times 65$ mm) and a façade part (non-load bearing) made of hollow bricks 125 mm thick. The slabs are made of semi-prefabricated “Herbst” concrete hollow elements, joists, and a concrete slab of 6 cm. The total thickness of the slab at the storey’s is 26.5 cm, and the same construction is kept for the roof with thickness increased to 43.5 cm. The structure is an unreinforced masonry building with prefabricated slabs, behaving as rigid diaphragms [5,6].

The vulnerability of the building under study is connected to several issues. One of the essential requirements for adequate seismic response is not satisfied, as there is the lack of load-bearing elements in one of the structure’s main directions. The loadbearing walls are mainly placed in the transversal direction (Y), while walls in the longitudinal (X) direction are weakened by a large number of openings (Figure 2). The structure has a basement, ground floor, and five storeys, and it was constructed as an unreinforced masonry building. The walls are made of solid bricks connected with lime mortar providing a low compressive strength of masonry. By taking into account the EMS 98 [12], the structure is associated with vulnerability class C. This structure (unconfined masonry, at most 60 years old with reinforced concrete floors) can have different damages grades. For a seismic intensity VII, the damage grade would be 2 (moderate damage), and as the intensity increases the damage grade increases by one level: Seismic intensity VIII provides substantial to heavy damage; seismic intensity IX provides very heavy damage and seismic intensity X provides destruction. Additionally, according to the regulations

regarding technical standards for the construction of buildings in seismic areas from 1991 [10], there is a clear limitation regarding the number of storeys for unreinforced masonry structures in respect to the seismic intensity level, as shown in Table 1.

Table 1. Allowed number of storeys for different masonry structures [10].

Type of Masonry Structure	Seismic Intensity Degree		
	IX	VIII	VII
URM	-	G + 1	G + 2
Confined masonry	G + 2	G + 3	G + 4
Reinforced masonry	G + 7	G + 7	G + 7

(G stands for ground floor).

This type of structure is characterized by two longitudinal façade walls with a large number of openings, while in the transverse direction the exterior walls have only one opening at the height of each floor. Other transverse (inner) walls have door openings of 2.3 m²–6.9 m². The total area of the openings in the basement for the longitudinal direction is 19.8%. In the transverse direction, openings in the outer wall occupy only 8.6% of the wall surface. A significant percentage of the aperture is located in the longitudinal walls in the amount of as much as 46% of the wall surface. Thus, the lateral resistance in the longitudinal direction is significantly lower than the lateral resistance in the transverse direction.

Sarajevo is located in the seismic intensity zone VII, thus limiting the number of storeys to G + 2 (see Table 1), while the analyzed structure is G + 5. It is evident that the limitation according to Table 1 is not respected, as this regulation was enforced only in the 1980s.

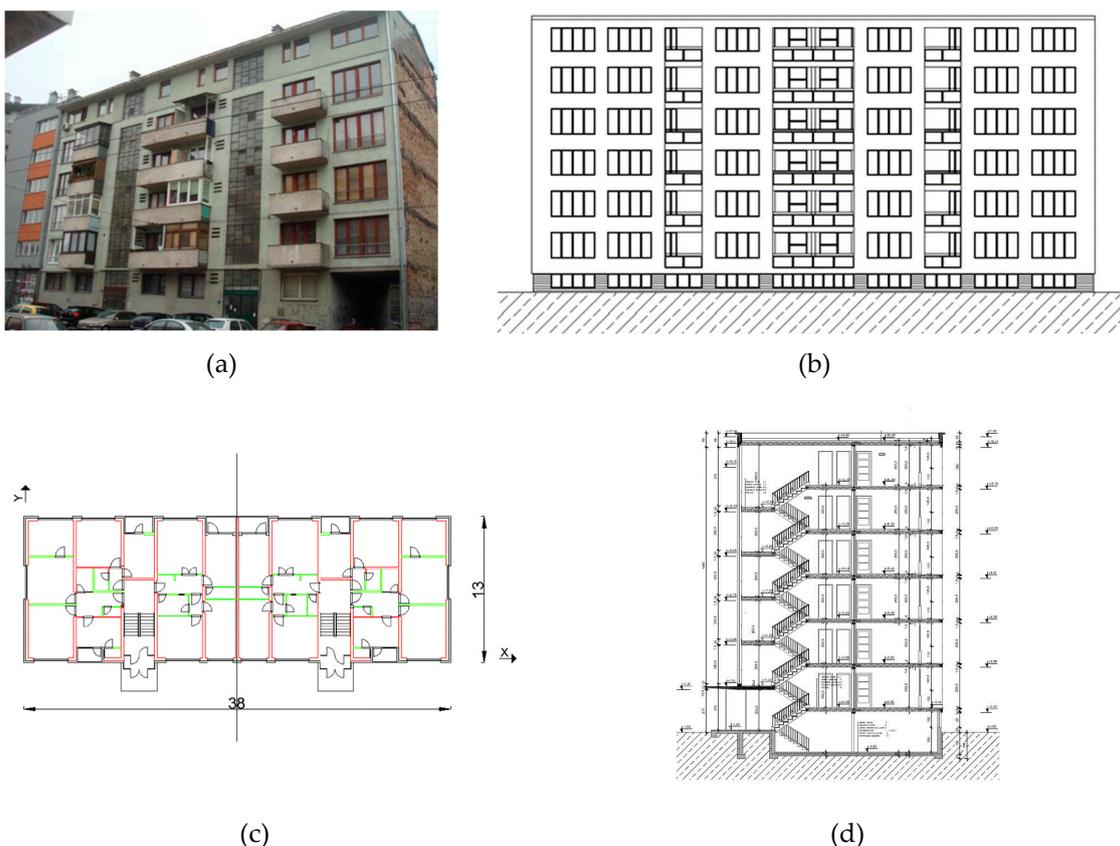


Figure 3. (a) Analyzed building located in Sarajevo; (b) east façade; (c) plan view of the typical floor and labeling of coordinates (X and Y); (d) typical cross section.

In order to obtain the necessary information regarding the physical and mechanical characteristics, several on-site and laboratory tests were done. Laser distancemeters and total stations were used for verification of the geometric data and as a result, drawings of the building were produced (Figure 3b–d). Brick units and concrete compressive strengths were determined on samples from the building (Figure 4a). Five series of two bricks were taken out from representative locations in the structure and their compressive strength was tested in accordance with the ex-Yugoslavian standards [13] (Figure 4b). Locations of the extracted brick samples are marked by numbers from 1 to 5 (Figure 4a). The mean value of brick compressive strength was 19.4 MPa, while the minimum value was 13.4 MPa putting it into a class of M15 (15 MPa), which fulfills the requirements for a load-bearing wall [13]. Compressive strength of concrete was determined using six cylindrical samples in accordance with the regulations defined in standard [14]. The mean value of concrete compressive strength was 25.1 MPa, while the minimum value was 22.8 MPa putting it into a class of MB25, which is equivalent to C20/25 in Eurocode 2 [15], and the reinforcement was $\varnothing = 14$ mm, type of steel GA240/360 (smooth bars with yield stress 240 MPa, and 360 MPa ultimate strength). Based on results from the experimental campaign, the calculated compressive strength of masonry according to Eurocode 6 [16] was 4.1 N/mm^2 , which was used as the basis for the calculation of other mechanical characteristics. The slabs were made out of concrete class C20/25 and a reinforcement $\varnothing = 8$ mm and the same type of steel was used as in the basement. All other characteristics were determined from the characteristic compressive strength of the concrete according to Eurocode 2 [15].

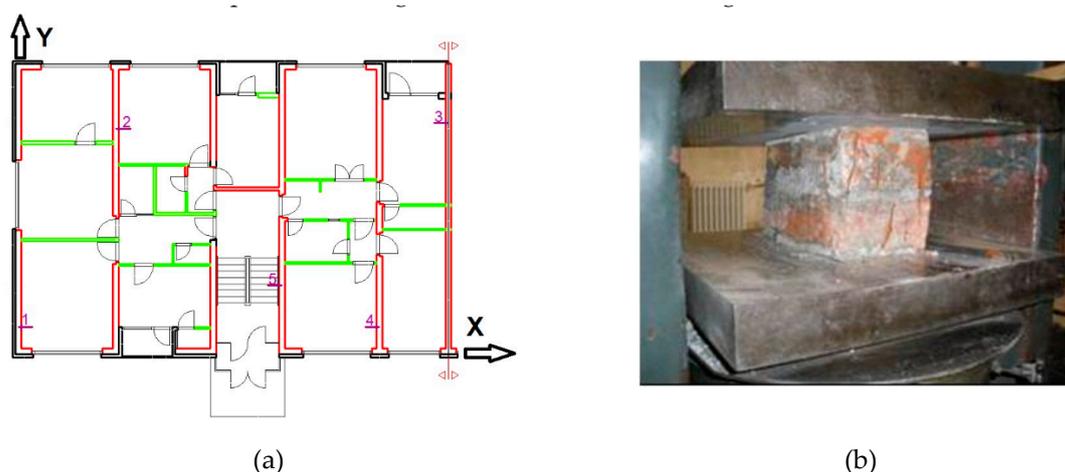


Figure 4. (a) Location of the samples for determination of the compressive strength of bricks; (b) tested sample.

The building, depicted in Figure 3a, was modelled using a finite element model (FEM) and the equivalent frame model (EFM), with the application of DIANA [17] (Figure 5a) and 3MURI [18] (Figure 5b), respectively. DIANA software [19–24], as well as 3Muri [24–30], have been widely applied to model masonry structures. The structure was modeled with a curved shell (quadrilateral element CQ40S type) element. In FEM an element size was 0.25 m having a total of 84,523 nodes and 28,522 elements, while in the EFM, the number of 3D nodes was 218, 34 2D nodes, and 506 elements for the entire structure (Figure 5a,b). In FEM only half of the structure was modeled and analyzed as the structure is symmetric, employing in total 15,759 CQ40S elements and 45,443 nodes (Figure 5c). The applied adequate boundary conditions are indicated in Table 2. Material nonlinearity properties of masonry [31] were taken into account with the application of the total strain fixed crack model, as defined in DIANA [17]. The post-cracked shear behavior was introduced by a very low shear retention factor. The physical and material characteristics taken in the analysis are given in Table 3.

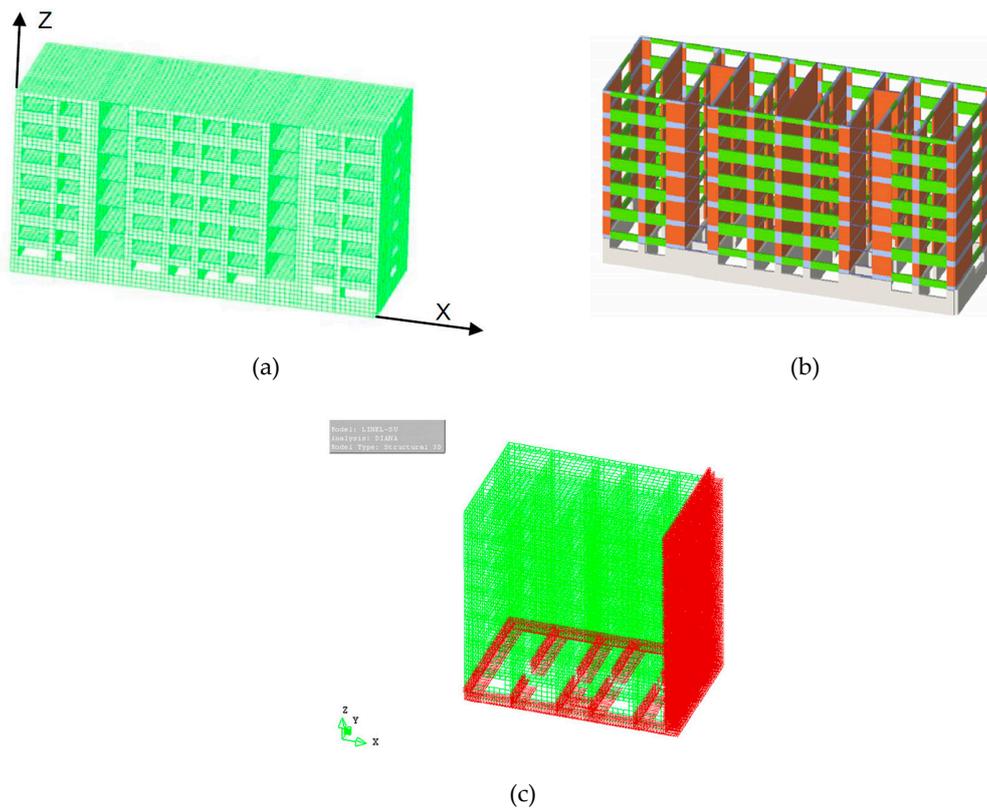


Figure 5. Models used for safety assessment: (a) Finite element model (FEM) in DIANA; (b) EFM in 3MURI; (c) FEM model in DIANA (only half is modelled to keep the computational effort acceptable).

Table 2. Constrain Summary.

Support	Translation	Rotation
Plane of symmetry	X	Y; Z
Base of structure	X; Y; Z	X; Y; Z

Table 3. Physical and material characteristics.

Element	Thickness (m)	Compressive Strength f_c (N/mm ²)	Compressive Fracture Energy G_{fc} (N/mm)	Tensile Strength f_t (N/mm ²)	Tensile Fracture Energy G_t (N/mm)	Modulus of Elasticity E (N/mm ²)	Poisson Ratio ν	Density ρ (kg/m ³)
Masonry								
Façade walls	0.375	4.07	6.51	0.20	0.10	4070	0.20	2700*
Inner walls	0.250							1900
Element	Thickness (m)	Compressive Strength f_c (N/mm ²)	Tensile Strength f_{ct} (N/mm ²)		Modulus of Elasticity E (N/mm ²)	Poisson Ratio ν	Density ρ (kg/m ³)	
Concrete								
Façade walls	0.380	24	2.2		30,000	0.20	2400	
Inner walls	0.250						2190	
Floor	0.265						2050	
Roof	0.435							

* On the basis of experimental test, the value of density of this type of masonry is 1900 kg/m³, however, in order to take into account the non-bearing façade walls in respect of the mass, the value has been proportionally increased, while keeping the thickness of $d = 25$ cm enabling the stiffness to remain intact.

Dynamic characteristics of the structure are important for understanding its dynamic behavior under earthquake actions. The natural frequencies and mode shapes of the entire structure were determined by modal analysis. The first eigen frequency in FEM was 2.19 Hz, while in EFM it amounted to 1.96 Hz and the first mode in both cases was the translation in the X direction [5,6,29]. Furthermore, the values of frequencies are consistent with the data provided by Tomažević [32], indicating: “For higher structures even up to 11-storeys the value are close to 2 Hz even though buildings have been built with different materials”. This was used as a verification parameter of the model as no ambient vibration tests were conducted on the structure.

Figure 6a shows the labeling of the walls in the structure in the two directions as well as the numbers of the nodes on the top floor of the structure. The node used to monitor the top displacement is located at the axis of symmetry in the center of the structure (referred to as node 44014) (Figure 6b). For more details regarding the numerical model of the structure and material characteristics the reader is referred to references [5,6].

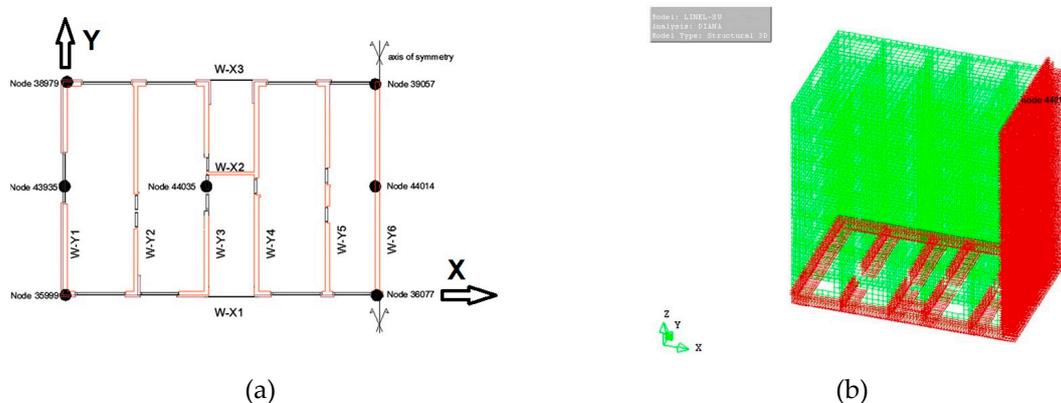


Figure 6. (a) Location of the nodes; wall labeling; (b) location of the control node 44014.

3. Results of the Unstrengthened Configuration

The FEM model was used to perform pushover and time history analysis, while the EFM model was only used to conduct pushover analysis. Comparing the pushover curves obtained from the two models, the difference in the maximum load coefficient was in the range of 6.4% to 6.9%, which is considered as an acceptable range [5,6]. Here, the load coefficient is the ratio between the base shear forces due to pushover (y direction) and the sum of all vertical loads (z direction).

Results show that the EFM model is stiffer in comparison to the FEM model. The difference in the stiffness could be attributed to the rigid connections between the spandrel and the pier elements. Spandrels mainly influence the boundary conditions of piers [33]. The time history analysis in FEM was compared with pushover analysis in EFM just in the view of distribution of pattern damage and it was evident that it follows the same sequence [5,6]. The location of the cracks were around the opening and, as the windows are very close to the corner of the walls (75 to 100 cm), this is weakening the connection between the walls causing additional concentration of stresses and crack development. The major damage is concentrated on the ground floor. The two models gave results that were in excellent correlation, however for the sake of computational efficiency, it was decided to continue the further analysis with the EFM model.

In order to generate the response of the structure to various PGA values, the structure was exposed to the short-period 1979 Petrovac strong earthquake motion registered at Montenegro. This acceleration record is commonly used for seismic structural assessment throughout the region [34,35]. In order to take into account the ground motion in Sarajevo, according to the new seismic hazard map, the accelerogram from the Petrovac earthquake (with a PGA of 0.43 g) was scaled down to 0.1 g (Figure 7) and 0.2 g. Seismosignal software [36] was used for scaling and filtering the acceleration record. The code response spectrum used in the EFM model is presented in Figure 8 for 0.1 g.

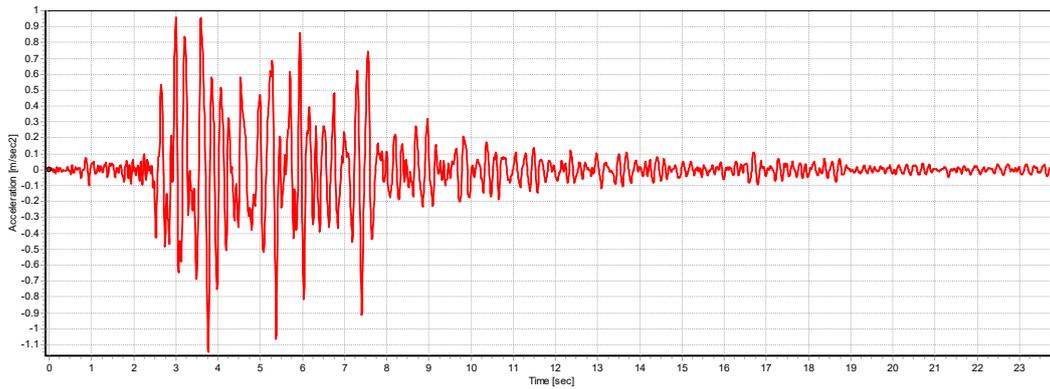


Figure 7. Petrovac earthquake acceleration scaled to 0.1 g [36].

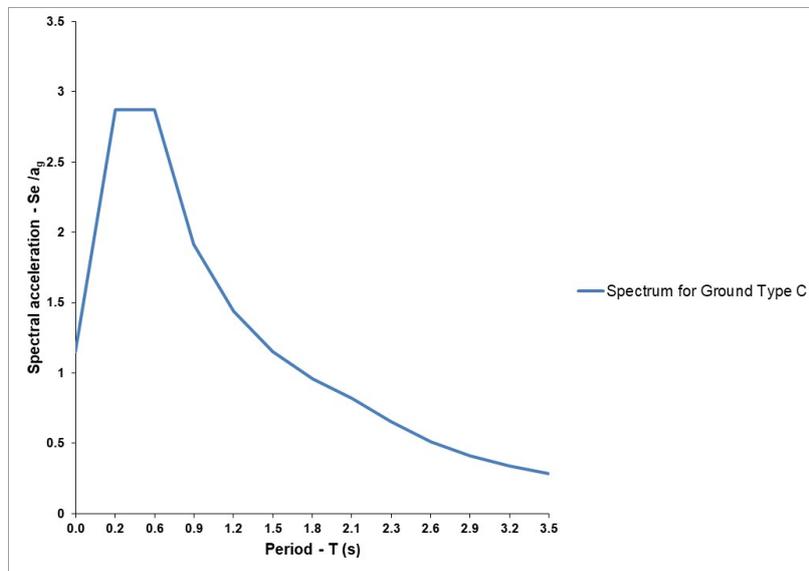


Figure 8. Elastic response spectra for Sarajevo according to Eurocode 8 for 0.1 g.

Due to the earthquake action of 0.1 g, in the Y direction, formation of typical diagonal cracks (Figure 9a) were noticed with concentration of the damage at the ground level (Figure 9b) (connection between the concrete basement and masonry structure) after 24 seconds.

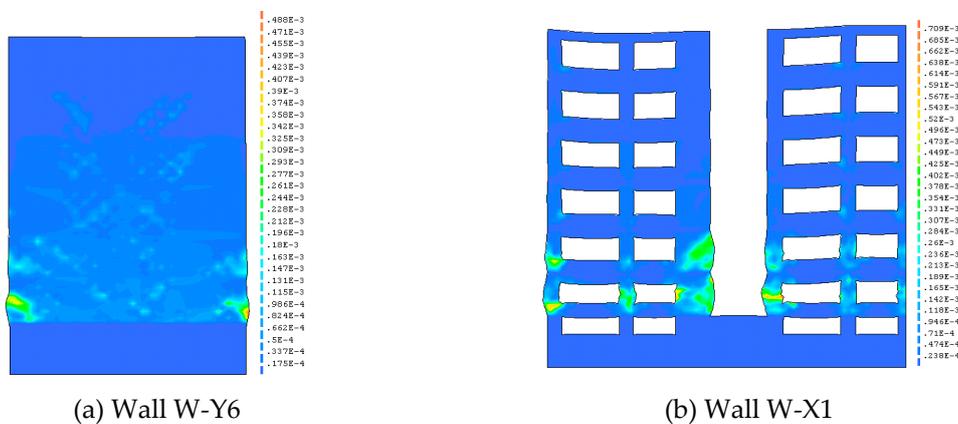


Figure 9. (a) Typical diagonal X-shaped crack due to shear in the perpendicular wall; (b) concentration of damage at the ground floor in the longitudinal wall (0.1g scaled accelerogram).

On the other hand, it was only after 3.89 seconds that the structure showed significant damage, which caused the collapse of the structure, when the structure was exposed to 0.2 g of the same earthquake (Figure 10a,b).

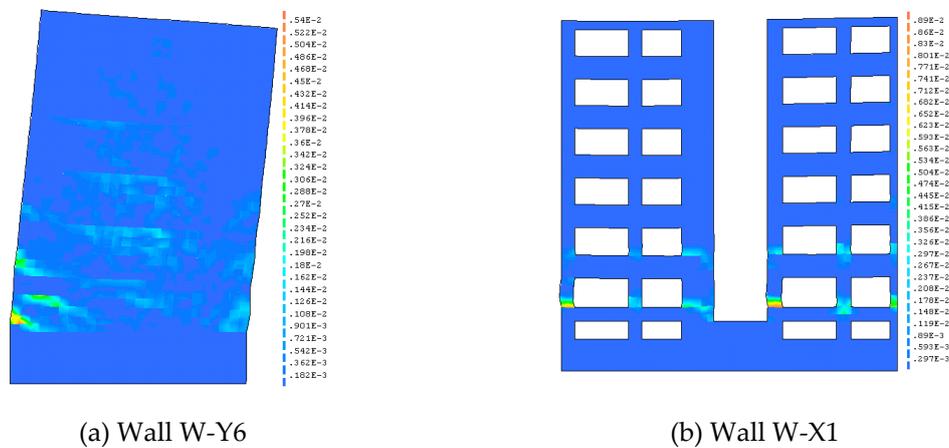


Figure 10. (a) Damage of the perpendicular wall; (b) longitudinal wall due to 0.2 g scaled accelerogram.

From Figure 11 it is evident that, once the structure is exposed to the Petrovac ground motion scaled to 0.2 g, it undergoes very large displacements with a maximum of 29.33 cm after only 3.89 s, which is 2.6 times larger than when the structure was exposed to a 0.1 g scaled accelerogram. It is evident that these types of structures are highly vulnerable to such seismic actions and strengthening measures should be proposed.

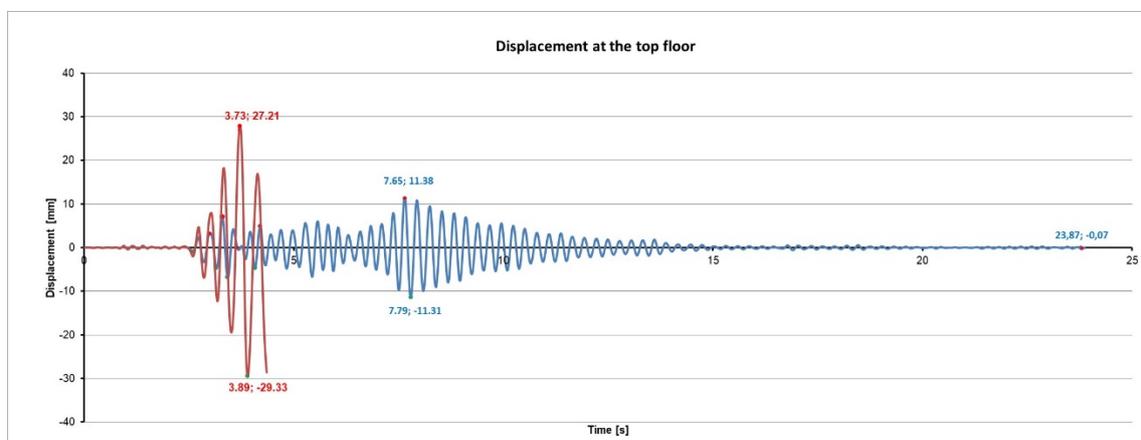


Figure 11. Displacement of the node 44014 at the top floor due to action of 0.1 g (indicated by a blue color) vs 0.2 g scaled accelerogram (indicated by the red color).

The maximum displacement in the case of 0.1 g is rather small and it amounts to 11.38 cm while for the case of higher seismic activity (0.2 g) the displacement increased to 29.33 cm, which is 2.6 times larger. The maximum inter-storey drift per each floor was calculated separately (as it does not happen simultaneously). The maximum value of inter-storey drift of each floor was identified, and an envelope of the inter-storey drift was created. Further, the maximum inter-storey drift (envelope) moved from 0.8 % to 5.8 % (Figure 12) as the PGA increased from 0.1 g to 0.2 g, making the structure more vulnerable, and showing that severe damage will occur if exposed to an earthquake of higher PGA (0.2 g).

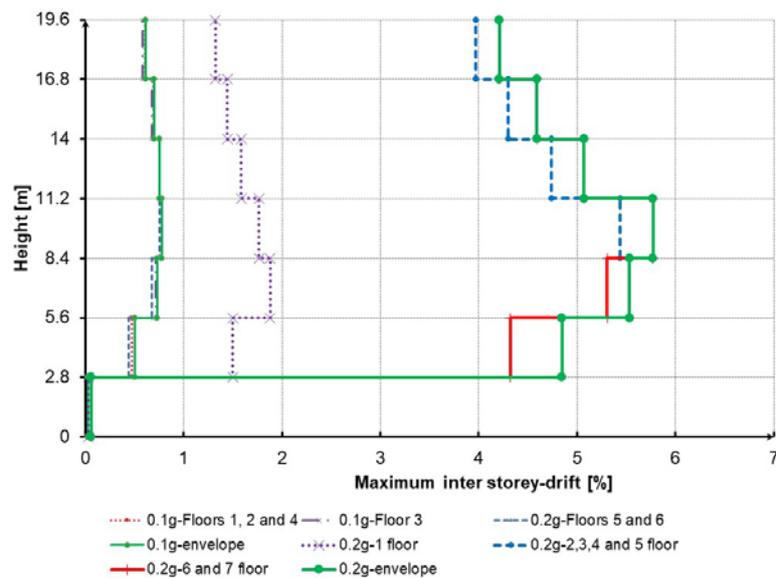


Figure 12. Maximum inter-storey drift for PGA 0.1g and PGA 0.2g scaled accelerograms.

The hysteresis curve originated by the 0.2 g scaled accelerogram was determined for the control node 44014 and the relationship between the seismic coefficient (ratio between base shear and gravity forces) and displacement has been obtained as shown in Figure 13. The maximum seismic coefficient of 0.37 was reached while the maximum displacement was 29.3 mm. From Figure 13, energy dissipation was observed due to flexural and shear phenomena. However, in order to understand the behavior of a structure besides the amount of dissipated energy, it is important to know the equivalent viscous damping ratio and dissipated energy ratio.

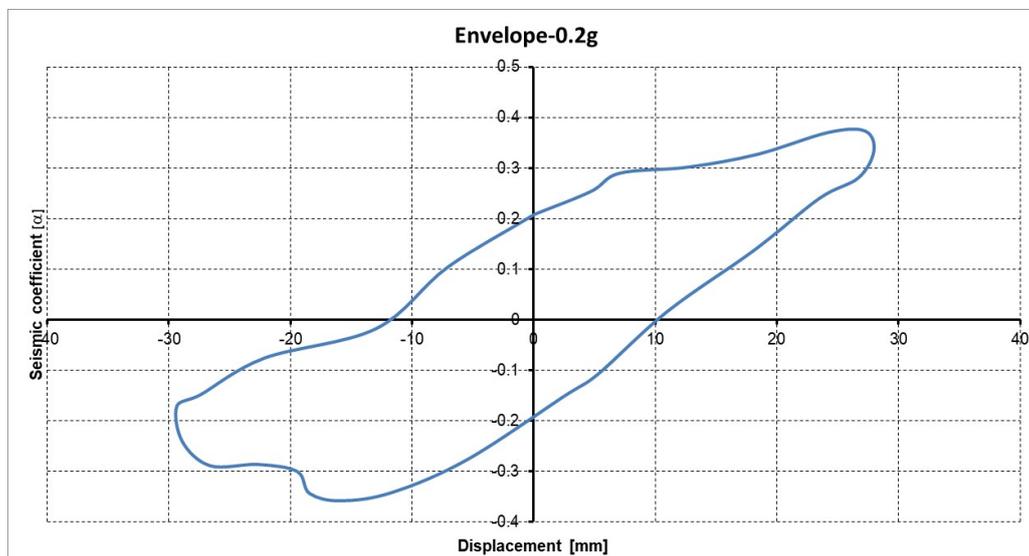


Figure 13. Envelope of the hysteresis curve in the transversal direction.

The structural analysis of the case study was also performed with the EFM (additional information could be found in references [5,6,29]). Here, only the results in the X direction of the façade walls will be shown. Damage was again concentrated in the lower floor of the structure with bending and compression failure at the ground level of the structure (Figure 14). The observed damage shown in Figure 14 (damage in masonry piers) is consistent with the damage observed on the building after the Skopje earthquake (Figure 2b). As results obtained by both FEM and EFM approaches regarding the pushover analysis were in a very good agreement, it was decided not to report the respective results

here. In addition, due to the much less computational time required, it was decided to conduct further analysis in the X direction utilizing only the EFM approach. As stated before, one of the major flaws of this type of building is the nonexistence of adequate number of load bearing walls in the longitudinal (X) direction and the large number of openings causing a lower lateral stiffness in this direction with respect to the transversal (Y) direction.



Figure 14. Façade walls damage of the original structure after exposure to $PGA = 0.1$ g.

4. Strengthening Intervention (EFM Model)

The proposed strengthening method followed here is a combination of traditional and modern strengthening techniques. This envisaged solution includes the construction of four new walls (Figure 15a) and strengthening the ground floor with fiber reinforced polymers (FRP) where concentration of damage was observed, see Figure 14. In this way, the structure moved from inadequate to the adequate positioning of the structural walls in plan [32]. The walls were positioned in such a way as to keep the symmetry of the structure and in that way circumvent possible undesirable torsional effects. The new walls are made of solid clay bricks connected with lime mortar in order to be compatible with the existing walls. The four new structural walls were built in place of existing separation walls in order to not affect the serviceability of the structure, besides being the most convenient location. The walls are founded on new continuous wall footings connected to the existing foundation. The two longitudinal walls surrounded with a red rectangle in Figure 15b were reinforced at the ground floor with carbon FRP bidirectional strips, having thickness of 2 mm and a width of 200 mm and spaced every 0.5 m, see S_d in Figure 15c. The used FRP material has the following mechanical properties: $E = 240,000$ MPa (modulus of elasticity), $f_{td} = 3182$ MPa (tensile strength) and $\epsilon = 1.45\%$ (ultimate elongation), area of the corresponding strips is 2×200 mm².

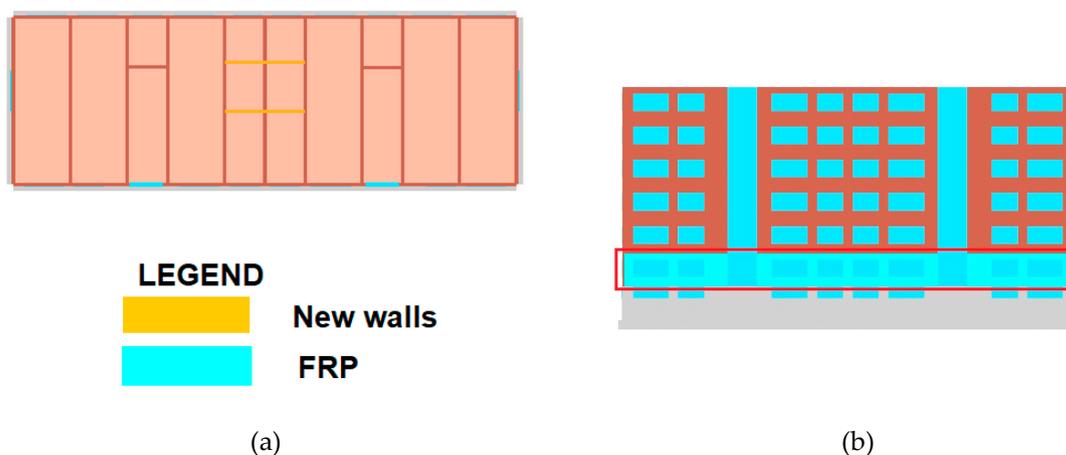
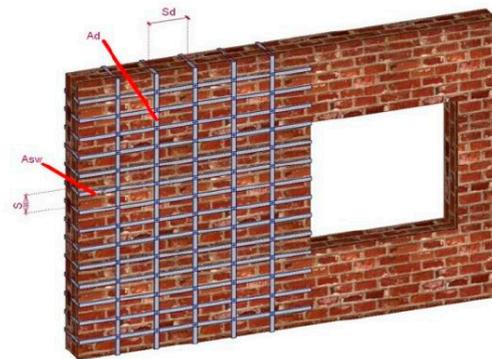


Figure 15. Cont.



(c)

Figure 15. Strengthened structure (a) 4 additional walls; (b) FRP at the ground level; (c) FRP modeling in 3MURI [18].

The comparison of the performance of the unstrengthen and strengthened structure exposed to the elastic response spectra as defined in Eurocode 8 [37] for PGA equal to 0.1 g and 0.2 g and C soil type (Figure 16), shows that the strengthening procedure adopted could not overcome the problem of seismic vulnerability of this type of structures for larger earthquake motions. The structure has an adequate behavior if exposed to a smaller PGA, however bending failure occurs at the lower floors of the structure with the increase of PGA.

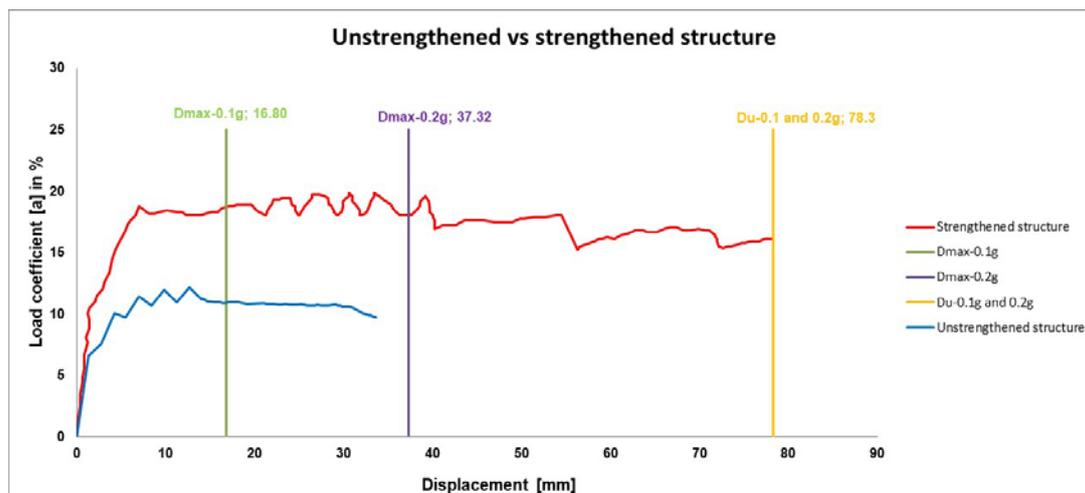


Figure 16. Pushover curves of the unstrengthen and strengthened structures, with performance points for PGA = 0.1 g and PGA = 0.2 g (D_{max} —demand displacement and D_u —ultimate capacity displacement).

Both analyses satisfied the Damage Limit State (DLS) check according to Eurocode 8 [37]. However, the structure exposed to the excitation of 0.2g did not pass the Ultimate Limit State (ULS) check, where the obtained value of q^* (the relationship between the elastic response force and the yield strength of the equivalent system) as defined in reference [38] was 4.90, which is larger than the maximum acceptable value of 3 [38], while in the case of 0.1g this value was equal to 2.45. The available ductility (the ratio between the ultimate displacement and the elastic limit displacement) was 16.78 for the two analyzed cases. If the structure is to be exposed to an earthquake with PGA of 0.1g this intervention would be acceptable (Figure 17). However, this cannot be stated for an earthquake of a higher magnitude, where the concentration of damage due to bending is seen on the lower levels of the strengthened building (Figure 18) causing major damage on the masonry spandrels.

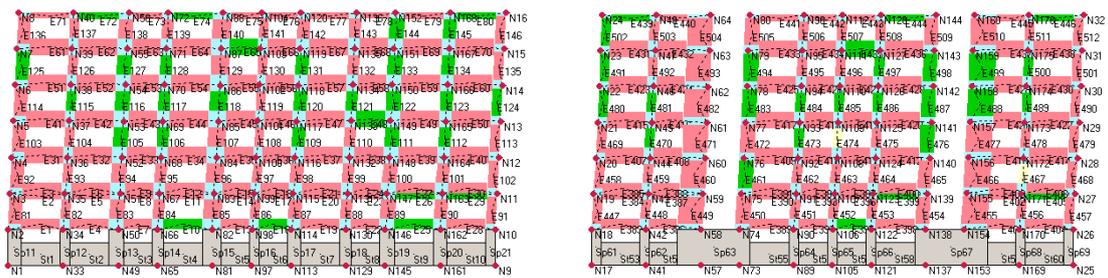


Figure 17. Wall damage after exposure to PGA = 0.1 g—satisfactory behavior of the structure.

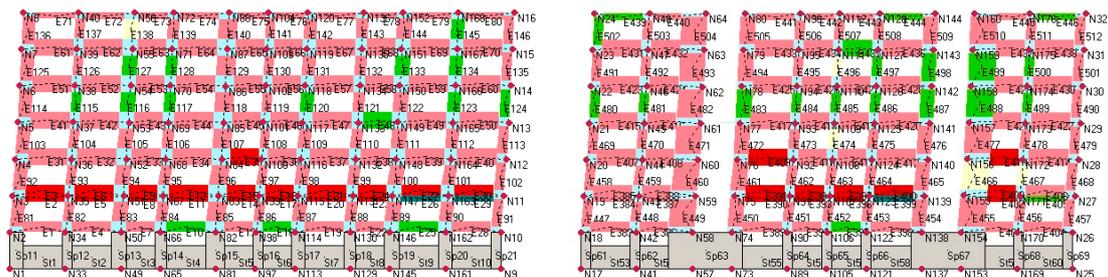


Figure 18. Wall damage after exposure to PGA = 0.2 g—inadequate behavior of the structure.

5. Conclusions

This article addresses the assessment of the seismic vulnerability of a typical URM building in Sarajevo, which is characteristic of the Balkan region, from Macedonia to Slovenia. Assessment of existing structures and determination of their vulnerability to seismic actions and possible intervention methods should be seen as a preventive measure for seismic risk mitigation. According to the hazard seismic map of Bosnia and Herzegovina, Sarajevo can expect earthquakes of up to 0.2g, giving a good basis and reason for assessment of structures that were built from 1920 to 1965 without the application of a seismic code, as such codes did not exist in this territory at that time. The vulnerability of these typologies is more than evident and adequate strengthening procedures should be planned and implemented in order to be in line with preventive measures. Focus should be put on earthquake awareness and mitigation of structural vulnerability so that in the case of future earthquakes, seismic losses are minimized and lives saved. It is an unfortunate fact that this type of structure has not been strengthened and, on the contrary, additional floors have been added on to it, which makes them even more vulnerable to future seismic actions.

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References

1. Bulajić, B.Đ.; Manić, M.I.; Ladinović, Đ. Effects of shallow and deep geology on seismic hazard estimates: A case study of pseudo-acceleration response spectra for the northwestern Balkans. *Nat. Hazards* **2013**, *69*, 573–588. [[CrossRef](#)]
2. Lee, V.W.; Herak, D.; Herak, M.; Trifunac, M.D. Uniform hazard spectra in western Balkan Peninsula. *Soil Dyn. Earthq. Eng.* **2013**, *55*, 1–20. [[CrossRef](#)]
3. BAS. EN 1998-1:2018 Eurocode 8: Design of Structures for Earthquake Resistance—Part 1: General Rules, Seismic Actions and Rules for Buildings—National Annex; Institute of Standardization of Bosnia and Herzegovina: Sarajevo, Bosnia and Herzegovina, 2018.

4. Ademović, N.; Kalman Šipoš, T.; Hadzima-Nyarko, M. Multidimensional Protocol for Evaluation of Seismic Risk for Bosnia and Herzegovina. *Bull. Earthq. Eng.* **2018**. under review.
5. Ademović, N. Structural and Seismic Behavior of Typical Masonry Buildings from Bosnia and Herzegovina. Master's Thesis, University of Minho, Braga, Portugal, 2011.
6. Ademović, N. Behavior of Masonry Structures in Bosnia and Herzegovina at the Effect of Earthquakes from the Viewpoint of Modern Theoretical and Experimental Knowledge. Ph.D. Thesis, Faculty of Civil Engineering, University of Sarajevo, Sarajevo, Bosnia and Herzegovina, 2012.
7. Lutman, M.; Tomazevic, M. *Unreinforced Brick Masonry Apartment Building*; Housing Report; Earthquake Engineering Research Institute and International Association for Earthquake Engineering: Ljubljana, Slovenia, 2002.
8. Bureau of Statistics CBS. *National Population and Housing Census 2013*; National Report; Agency for Statistics of Bosnia and Herzegovina: Sarajevo, Bosnia and Herzegovina, 2013.
9. Stojković, M.; Mihailov, V. *Seismic Microzonation Map of Banja Luka of the City Urban Area of Banja Luka*; Institute of Earthquake Engineering & Engineering Seismology: Skopje, Macedonia, 1972.
10. Regulations Regarding Technical Standards for the Construction of Buildings in Seismic Areas. Official Gazette SFRJ No. 31/81, 49/82, 29/83, 21/88 and 52/90. 1990.
11. Lee, V.W.; Manić, M.I.; Bulajić, B.Đ.; Herak, D.; Herak, M.; Trifunac, M.D. Microzonation of Banja Luka for performance-based earthquake-resistant design. *Soil Dyn. Earthq. Eng.* **2015**, *78*, 71–88. [[CrossRef](#)]
12. Grünthal, G. (Ed.) *European Macroseismic Scale 1998, EMS-98*; Center Europeen de Geodynamique et de Seismologie: Luxemburg, 1998; Volume 15, pp. 1–101.
13. JUS.B.D1.011. *Collection of Yugoslav Regulations and Standards for Construction Structures, Masonry Structures*; Book 5; Faculty of Civil Engineering of the University of Belgrade: Belgrade, Serbia, 1995.
14. JUS. U.M1.048. Regulations Regarding Technical Standards for Concrete and Reinforced Concrete. Official Gazette of SFRY 48/85. Belgrade. 1985.
15. CEN.Eurocode 2. *EN 1992-1-1:2004. Design of Concrete Structures—Part 1-1: General Rules and Rules for Buildings*; Committee European de Normalization: Bruxelles, Belgium, 2004.
16. CEN.Eurocode 6. *EN 1996-1-1:2005. Design of Masonry Structures. Part 1-1: General Rules for Reinforced and Unreinforced Masonry Structures*; Committee European de Normalization: Bruxelles, Belgium, 2005.
17. DIANA 9.4, TNO. Displacement method ANALyser 9.4. Finite element analysis. User's Manual, release 9.4. Netherlands: S.n., Vol. Release 9.4. 2009. Available online: <https://dianafea.com/> (accessed on 15 November 2018).
18. S.T.A. DATA 3 Muri. Manual, STA DATA srl, Torino. 2010. Available online: <http://www.stadata.com/> (accessed on 15 November 2018).
19. Haach, V.G.; Vasconcelos, G.; Lourenço, P.B. Parametrical study of masonry walls subjected to in-plane loading through numerical modeling. *Eng. Struct.* **2011**, *33*, 1377–1389. [[CrossRef](#)]
20. Dolatshahi, K.M.; Aref, A.J. Two-dimensional computational framework of mesoscale rigid and line interface elements for masonry structures. *Eng. Struct.* **2011**, *33*, 3657–3667. [[CrossRef](#)]
21. Ghiassi, B.; Marcari, G.; Oliveira, D.V.; Lourenço, P.B. Numerical analysis of bond behavior between masonry bricks and composite materials. *Eng. Struct.* **2012**, *43*, 210–220. [[CrossRef](#)]
22. Marques, R.; Pereira, J.M.; Lourenço, P.B.; Parker, W.; Uno, M. Study of the Seismic Behavior of the “Old Municipal Chambers” Building in Christchurch, New Zealand. *J. Earthq. Eng.* **2013**, *17*, 350–377. [[CrossRef](#)]
23. Araujo, A.S.; Lourenço, P.B.; Oliveira, D.V.; Leite, J. Seismic Assessment of St James Church by Means of Pushover Analysis—Before and After the New Zealand Earthquake. *Open Civ. Eng. J.* **2012**, *6*, 160–172. [[CrossRef](#)]
24. Ademović, N.; Oliveira, D.V. Seismic Assessment of a Typical Masonry Residential Building in Bosnia and Herzegovina. In Proceedings of the 15th World Conference on Earthquake Engineering, Lisbon, Portugal, 24–28 September 2012; pp. 1–10.
25. Marques, R.; Lourenço, P.B. Unreinforced and confined masonry buildings in seismic regions: Validation of macro-element models and cost analysis. *Eng. Struct.* **2014**, *64*, 52–67. [[CrossRef](#)]
26. Formisano, A.; Iaquinandi, A.; Mazzolani, F.M. Seismic retrofitting by FRP of a school building damaged by Emilia-Romagna earthquake. *Key Eng. Mater.* **2015**, *624*, 106–113. [[CrossRef](#)]
27. Paparo, A.; Beyer, K. Modelling the seismic response of modern URM buildings retrofitted by adding RC walls. *J. Earthq. Eng.* **2015**, *20*, 587–610. [[CrossRef](#)]

28. Cattari, S.; Lagomarsino, S. Seismic assessment of mixed masonry-reinforced concrete buildings by non-linear static analyses. *Earthq. Struct.* **2013**, *4*, 241–264. [[CrossRef](#)]
29. Ademović, N.; Hrasnica, M.; Oliveira, D.V. Pushover analysis and failure pattern of a typical masonry residential building in Bosnia and Herzegovina. *Eng. Struct.* **2013**, *50*, 13–29. [[CrossRef](#)]
30. Maio, R.; Vicente, R.; Formisano, A.; Varum, H. Seismic vulnerability of building aggregates through hybrid and indirect assessment techniques. *Bull. Earthq. Eng.* **2015**, *13*, 2995–3014. [[CrossRef](#)]
31. Lourenço, P.B. Computational Strategies for Masonry Structures. Ph.D. Thesis, Technische Universiteit Delft, Delft, The Netherlands, 1996.
32. Tomažević, M. *Earthquake-Resistant Design of Masonry Buildings*; Tom, I., Ed.; Imperial College Press: London, UK, 1999.
33. Cattari, S.; Lagomarsino, A. Strength criterion for the flexural behaviour of spandrel in un-reinforced masonry walls. In Proceedings of the 14th World Conference on Earthquake Engineering, Beijing, China, 12–17 October 2008; p. 41.
34. Tomažević, M.; Iztok, K. Verification of Seismic resistance of confined masonry buildings. *Earthq. Eng. Struct. Dyn.* **1997**, *26*, 1073–1088. [[CrossRef](#)]
35. Tomažević, M.; Bosiljkov, V.; Polona, W. Structural behavior factor for masonry structures. In Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, BC, Canada, 1–6 August 2004; p. 2642.
36. Seismosignal. Earthquake Engineering Software Solutions, Version 4.2.1. 2010. Available online: <https://www.seismosoft.com/> (accessed on 15 November 2018).
37. CEN. Eurocode 8. *EN 1998-1-2004: Design of Structures for Earthquake Resistance—Part 1: General Rules, Seismic Actions and Rules for Buildings*; Committee European de Normalization: Bruxelles, Belgium, 2004.
38. OPCM 3274 Norme tecniche per il progetto, la valutazione e l'adeguamento sismico degli edifici testo integrato dell'Allegato 2—edifici—all'ordinanza 3274 come modificato dall'OPCM 3431. 2005. Available online: <http://zonesismiche.mi.ingv.it/pcm3274.html> (accessed on 15 November 2018).



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