SEISMIC VULNERABILITY, DAMAGE AND STRENGTHENING OF MASONRY STRUCTURES IN THE BALKANS WITH A FOCUS ON BOSNIA AND HERZEGOVINA

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ABSTRACT

Two countries, Bosnia and Herzegovina (B&H) and Croatia, which are in the focus of the article, suffered damages from strong earthquakes in the past. Devastations were evident during earthquakes that struck this region in the past 100 years, of which the most important ones were Skopje earthquake in 1963, Banja Luka in 1969 and Petrovac in Montenegro in 1979. It is clear that structural assessment of these structures and possible seismic strengthening represents a high priority not only in B&H but in the entire Western Balkans as the construction technology and history of construction are the same in the whole region. Available information regarding building stock and building typologies in both countries show that the traditional art of construction are mostly masonry buildings. Distribution of vulnerable building stock is required to assess the seismic vulnerability of the area under consideration. Analytical seismic vulnerability method applied in B&H is presented.

As some experiments in this research area were provided in both countries and some of the experiments are underway, few examples are given in order to gain insight into the behavior of masonry buildings and the selection of strengthening technique. Possible strengthening techniques for an existing structure are presented and elaborated giving possible guidelines for possible strengthening of similar existing structures in both countries.

Keywords: Vulnerability assessment; Masonry; Experimental tests; Strengthening

1. INTRODUCTION

Seismic activity in B&H is connected to the existence of deep lateral and reverse faults. The fact that the second biggest belt (Alpine Belt), going from the Himalayas over Iran, Turkey and Greece, passes through B&H verifies the tectonic activity of this region. Pepeš in 1998 made the most complete picture of the tectonic structure in B&H and identified deep faults passing through B&H as well as 30 tectonic units. The longest is the Sarajevo Fault spreading in the NW-SW direction with a length of 300km, meaning across the entire B&H. The second longest is the Banja Luka Fault, and the third Konjic Fault. All transversal faults are under-passing the Sarajevo Fault. High seismic activity is evident along the transversal deep faults, while low to moderate seismicity along the Sarajevo Fault is noted. According to research conducted by Papeš, Sarajevo and Gradiška Faults may experience earthquakes of magnitude M6 on the Richter's scale or even higher (Papeš 1998). The buildings in Bosnia and Herzegovina are traditionally built as masonry. Large stock of buildings made of brick or stone masonry exists in B&H, some which date to older times and are made of adobe. Traditionally they are unreinforced masonry (URM) structures with wooden floors. After World War II, wooden floors were replaced by reinforced concrete floors. A large number of existing buildings in B&H are multi-storey masonry residential buildings having 4–6 floors, and some now have additional one to

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two stories. It is well known that these structures are relatively stiff and possess limited ductile behavior. According to the seismic vulnerability classification (EMS) they can be regarded as classes B and C. If, as it could be expected, a moderate earthquake occurs some important damages might happen, where as in the case of higher earthquake magnitude heavy and very heavy damages, including partial collapse could be expected.

Devastations were evident during the earthquakes that struck this region in the past 100 years, of which the most important ones are those that occurred in Skopje in 1963, Banja Luka in 1969 and Petrovac in Montenegro in 1979. It is clear that structural assessment of these structures and possible seismic strengthening represents a high priority not only in B&H but in the entire Western Balkans as the construction technology and history of construction are the same in the whole region. As this region had very similar, if not to say the same construction methodology and used materials, this allows comparisons to be conducted leading to a high degree of certainty with respect to the comparison of obtained results. The assessment of some existing masonry structures indicates that some art of rehabilitation is required.

Experimental results from different countries regarding masonry units, mortar, wallets, unreinforced and strengthened masonry walls were compared and elaborated. During comparative analysis, it was evident that a scatter of results exists regarding some masonry components, which has a clear implication on the results of unreinforced masonry walls as well as strengthened walls in different countries (Slovenia, Albania, Republic of Croatia, Macedonia and B&H). As countries propose similar strengthening procedures, a comparison is more than welcome. One of the strengthening techniques that has been proposed by almost all is jacketing. Different materials for jacketing have been used where it was clear that one leads to excellent results while others made a "concrete" structure. Inadequate knowledge regarding strengthening can lead again to the problems which were clearly visible after the Umbria Marche 1997-98 earthquake. It is clear that inadequate strengthening can lead to worse scenarios than if no strengthening is done at all.

2. SEISMIC ACTIVITY IN BOSNIA AND HERZEGOVINA

Seismic activity in Bosnia and Herzegovina (B&H) is connected to the existence of deep lateral and reverse faults. The fact that the second biggest belt (Alpine Belt), going from the Himalayas over Iran, Turkey and Greece, passes through B&H verifies the tectonic activity of this region (Ademović 2011), (Ademović 2012), (Ademović and Oliveira (2012). As per Euro Mediterranean Seismic Hazard Map, B&H falls in the Moderate Seismic Hazard having peak ground acceleration (PGA) in the range of 0.08 to 0.24g, while the south-west part of the country experiences a High Hazard (PGA>0.24g).

Papeš in 1988 (Papeš 1988) presented the tectonic structure of B&H and identified deep faults passing through the country as well as 30 tectonic units (Figure 1).

![Figure 1. Main deep thrusts (to 25 km): Sarajevo fault (FoFo), Gradiška fault (F1F1), Bihać fault (F2F2), Livno fault (F3F3), Jablanica fault (F4F4) and Mostar fault (F5F5) (Papeš, 1988)](image)

Among the six faults, the longest is the Sarajevo Fault spreading in the NW-SW direction with a
length of 300km. In this way, it crosses the entire B&H. All transversal faults are under-passing the Sarajevo Fault. High seismic activity is evident along the transversal deep faults, while low to moderate seismicity along the Sarajevo Fault is noted. Taking into account the activities in the last 100 years, B&H was divided into five seismic zones and fifty-seven potential seismic structures. According to the tectonic characteristics of the ground and earthquake occurrence, connection and mutual dependency has been detected. The major seismic activity is located on the border belts of the geotechnical units (direction NW-SE). Lower seismic activity is seen in the direction of the longitudinal dislocations (direction NW-SE), whereas the smallest activity is seen at the crossing of the transversal faults (directions NE-SW and N-S) as indicated by Papeš (1988) and Hrvatović (2006) in their work (Figure 2).

![Figure 2. Tectonic Map and Seismic Activity of the B&H (Hrvatović, 2006)](image)

### 3. EXISTING BUILDINGS IN BALKANS

The assessment of the impact of an earthquake on the built environment can only be accomplished by detecting the structural systems of buildings and their performance in past earthquakes, engineering standards adopted during construction and the location and distribution of vulnerable building stock in the shaken area (Jaiswal et al. 2010).

Thus, a catalogue of building typology is a prerequisite for seismic vulnerability assessment of an area or country. This enables one to analyze the vulnerability of each building type, including the influence of the geometric and/or structural modifiers. Unfortunately, a standard building typology catalogue for B&H as well as Croatia has not been generated. This lack of data of current building stock was also pointed out in the project NERA (Report D7.2, 2011), in which six European countries were identified for such an analysis by Cambridge Architectural Research Ltd. (CAR) – Iceland, Switzerland, Serbia, Montenegro, Bosnia and Herzegovina and Croatia.

According to Erbach (1996), a high proportion of Croatia’s building stock, of which the majority were either masonry or concrete, was built after World War II. Within the deliverables of the project NERA (2011), two questionnaire responses were received regarding Croatia’s building stock. Ten different zones (5 urban, 5 rural with a total of 1630 buildings, e.g. 8078 dwellings) were surveyed using Google Street View. Both questionnaire results show that a large proportion of dwellings are masonry constructions. The Street View results show that, particularly in rural areas, a high proportion of masonry buildings, of which newer constructions tend to be confined masonry, whereas older tend to be unit masonry with timber or concrete floors. As stated in Hadzima-Nyarko et al. (2017), a catalogue of building typology has been started for the city of Osijek, the fourth largest city in Croatia (with a population of 107 784 as at 2011). The data collection for the buildings is proposed in Galista and Hadzima-Nyarko (2015) and considered the attributes given by the GEM building typology (Brzev et al. 2013). The current database contains primary schools (82.7 % reinforced concrete structures and 1.3 % masonry buildings), kindergartens (80% masonry buildings and 20% reinforced concrete buildings) and buildings from the oldest part of the city (an eighteenth-century complex containing 106 buildings – unreinforced masonry buildings made of full brick and wooden slabs). The database is further extended with residential buildings typical for every suburb in Osijek, of which 84.62% are confined masonry, 11% are reinforced concrete shear wall (RC SW) dominant buildings, constructed
using a special tunnel form technique, while 4 % are classified as dual systems (RC prestressed frames with RC shear walls - dual systems) (Hadzima-Nyarko et al. 2017). Naturally, these percentages will slightly change when the whole database for the city of Osijek will be completed, which is presumed to be in two years.

A similar situation can be assumed for B&H. Traditionally the buildings in B&H were unreinforced masonry. After World War II, concrete buildings became prominent. Masonry buildings built prior to World War II tended to have timber floor constructions, while the buildings built after World War II tended to have concrete floors. According to Ademović (2013) a typical construction type within Bosnia and the other Balkan countries is medium rise unreinforced masonry constructions with concrete floors. Confined masonry was introduced after the Skopje earthquake of 1963 and became the usual type of masonry construction following the introduction of seismic codes in 1981.

According to results from questionnaires and Google Street View survey from the NERA project (2011), the percentages of dwellings in both countries are presented in Figure 3. It can be concluded from Figure 3 that the nature of construction in Croatia and Bosnia and Herzegovina is similar (as it is in other Balkan countries). Traditional art of construction in Bosnia and Herzegovina as well as in Croatia are mostly masonry buildings. Unreinforced masonry (URM) buildings with wooden floors were built until the mid 1930s, when first half-prefabricated RC floors started to apply. First seismic codes were published after the earthquake in Skopje in 1963 and vertical confining RC elements were introduced in masonry building practice (Ademović et al. 2013). Presently, confining masonry is the common art of masonry structures in both of the mentioned countries.

4. DAMAGES OF MASONRY BUILDINGS CAUSED BY STRONG EARTHQUAKES THROUGH HISTORY

4.1 Zagreb earthquake on 9 November 1880

Systematic data collection on earthquakes in Croatia did not exist until the 19th century. The oldest earthquake in Zagreb, for which is known when it happened, was on March 26, 1502. A stronger earthquake in 26 March 1511 with the epicenter at Idrija in Slovenia, IX – X degree of MCS, was also felt in Zagreb, but caused less damage. The strongest of all the so-called earthquakes in Zagreb occurred on November 9, 1880 (Simović 2000). On 9 November 1880, at 07:33, Zagreb was hit by a powerful earthquake with its epicenter in the area of Medvednica with an estimated magnitude of about 6.2 on the Richter scale. A number of public buildings were destroyed and about 1758 houses were damaged. In Figure 4, the damage of Popov tower – Observatory during the Zagreb 1880 earthquake is presented.
4.2 Skopje earthquake on 26 July 26, 1963

On 26 July 1963, at 04:17 Skopje was struck by earthquake with $M_w$ 6.1. It killed more than 1,070 people, injured around 4,000 and left more than 200,000 people homeless, destroying about 80 percent of the city itself. Public buildings, schools, hospitals and many historical monuments suffered very heavy damage (Figure 5). It was the most destructive event in the history of the Republic of Macedonia and caused losses of about 15% of the gross national product of the Yugoslavian federation for that year (Sinadinovski and McCue 2013). Generally speaking, buildings with brick masonry wall suffered more than any other type and accounted for the highest number of deaths.

4.3 Banja Luka earthquake on 27 October 1969

The database of 2058 damaged masonry residential structures constructed without seismic strengthening, caused by the earthquake in 1969 in Banja Luka enabled the presentation of results in relation to the main structural and strength characteristics of masonry structures (Stojković 2009). The evaluation of damage degree has been done using the MSK-64 scale. The main impact of this earthquake had a magnitude of 6.6 degrees per Richter, an epicenter about 10 - 15 km north of the town and a focal point 25 km deep. A day earlier, somewhat north and deeper, it was preceded by a stronger stroke of magnitude 5.9 degrees per Richter. Studies on the degree of damage to masonry dwellings in the area of Banja Luka included 2058 objects, mainly with 1 and 2 floors, which were uniformly distributed around 20 km² of the city's surface. In Figure 6, damage grades 3, 4 and 5 are presented for one storey houses.

4.4 Classification of damage to masonry buildings
Classification of damage quantity is a very difficult task and very few recommendations are currently available. The distributions of building damage, reported in surveys after an earthquake, serve as the statistical basis of empirical curves. On the other hand, analytical vulnerability curves use damage distributions obtained from analytical simulations of structural models under increasing earthquake loads. The damage scale limit states need to be unambiguous with respect to the damage expected in the structural and non-structural elements of buildings with different lateral load resisting systems (Hadzima-Nyarko and Kalman Šipoš 2017). Some of the most frequently used damage scales are EMS-98 (Grünthal et al. 1998), the US HAZUS method/damage scale (FEMA 2003) and ATC-14 (1987).

The indicators of the damage grades of masonry buildings according to the European Macroseismic Scale 1998 (EMS-98) (Grünthal et al. 1998) are summarized in Table 1.

Table 1. Classification of damage to masonry buildings according to EMS-98 (Grünthal et al. 1998)

<table>
<thead>
<tr>
<th>Damage grade</th>
<th>Classification of damage to buildings of masonry buildings according to EMS 98</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Negligible to slight damage (no structural damage, slight non-structural damage) Hair-line cracks in very few walls. Fall of small pieces of plaster only. Fall of loose stones from upper parts of buildings in very few cases.</td>
</tr>
<tr>
<td>2</td>
<td>Moderate damage (slight structural damage, moderate non-structural damage) Cracks in many walls. Fall of fairly large pieces of plaster. Partial collapse of chimneys.</td>
</tr>
<tr>
<td>3</td>
<td>Substantial to heavy damage (moderate structural damage, heavy non-structural damage) Large and extensive cracks in most walls. Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).</td>
</tr>
<tr>
<td>4</td>
<td>Very heavy damage (heavy structural damage, very heavy non-structural damage) Serious failure of walls; partial structural failure of roofs and floors.</td>
</tr>
<tr>
<td>5</td>
<td>Destruction (very heavy structural damage) Total or near total collapse.</td>
</tr>
</tbody>
</table>

5. SEISMIC VULNERABILITY

According to Coburn and Spence (2002), vulnerability is defined as the degree of loss to a given element at risk resulting from a given level of hazard. The vulnerability of an element is defined as a ratio of the expected loss to the maximum possible loss on a scale from 0 to 1. In a large number of elements, like building stocks, vulnerability may be defined in terms of the damage potential to a class of similar structures subjected to a given seismic hazard.

There are two methods for the analysis of building vulnerability; namely empirical and analytical methods (Figure 7). Empirical (or observed) assessment method is based on the observation of damage suffered during past seismic events and is based upon the statistical evaluation of past earthquake damage.

The analytical method is based upon the numerical analysis of the structure. The buildings with the same material and construction type are grouped into one class. The performance of the buildings is predicted based upon design specifications and construction details. For example, vulnerability assessment can be analyzed through incremental dynamic analysis with interstory drift ratio as damage measure. Fragility curves for every building typology will give the probability of exceedance of a
specific limit state at different levels of seismic scenarios. Fragility curves will result in the measure of the performance of the structures for estimation of seismic risk at an acceptable level of certainty. Using these curves, damage description is provided for each building class in each damage state.

5.1 Analytical vulnerability assessment applied in Bosnia and Herzegovina

The importance of seismic vulnerability assessment including evaluation of possible strengthening and retrofit measures has been emphasized in the few last decades. Among analytical methods which were carried out in this area, a numerical analysis of a typical building, not only in B&H but the entire Balkans, was selected (Figure 8a). The structure was modelled in (DIANA, 2009), see Figure 8b. One of the characteristics is that the load bearing walls are located only in one direction (here Y direction). More details can be found in (Ademović 2011), (Ademović 2012) and (Ademović et al 2013). Pushover analysis had proven to be an applicable method in the process of determination of the vulnerability zones from steel, RC structures to masonry structures (Ademović et al. 2013). The drawback of this procedure is that for masonry structures, the best pattern of loads has not yet been determined.

Figure 8. a) Considered building, built in the year of 1957 b) Numerical model (Ademović, 2012)

Physical non-linear behavior of the masonry walls is defined through the Total Strain Fixed Crack model detailed in Diana (DIANA, 2009). In this way the cracks are fixed in the direction of the principal strain vectors being unchanged during the loading of the structure. For hysteretic behavior of masonry parabolic stress-strain relation for compression, based on Hill-type yield criterion, was chosen with no lateral confinement and no lateral crack reduction. Tension path, based on Rankine-type yield criterion, was described by an exponential tension softening diagram (Figure 9).

Figure 9. Hysteretic Behavior of Masonry (Ademović 2011, Ademović 2012) (Mendes, Lourenço 2010), (Ademović 2017)

The post-cracked shear behavior was defined by taking into account the retention factor of its linear behavior, which reduces its shear capacity according to the following equation:

\[ G_{ct} = \beta_r \cdot G \]  

(3)

where \( \beta_r \) is the retention factor \( 0 < \beta_r \leq 1 \) and \( G \) is the shear modulus of the un-cracked material. The structure was exposed only to horizontal acceleration in the ‘‘±Y’’ directions, as the building would not be able to resist earthquakes with the ground motion applied in the weak direction of the building. The horizontal load was applied in a stepwise fashion proportional to the mass of the structure. Figure 10 shows the appearance of the cracks at the last step equivalent to 51.8% of the force. The largest amount of cracks is located at the walls W-Y6 and W-Y5 caused by shear, followed
by the wall W-Y4, while the walls W-Y1, W-Y2 and W-Y3 had almost the same and the smallest amount of strains at this stage. Walls in Y direction exhibit shear damage which would further lead to shear failure. Bending damage is seen above the openings. Evidently this has implications on the development of the cracks on the facade wall W-X1 and W-X3, (Figure 10), but to a smaller extent, where evident compression (seen from the principal compressive stresses, not presented here) damage and even failure at the ground level is noticed, with most probably later on local falling out of masonry. This type of damage has been observed on a similar structure type affected by the Skopje earthquake in 1963 (Figure 5). The concentration of damage is located on the ground floor with diagonal cracks between the openings, probably caused by shear and in one of the corners, falling of the masonry is evident. (Ademović et al. 2013). The level between the basement and the ground floor is seen as the weakest point in the structure caused by a large change of stiffness clearly forming this to be a vulnerable zone for these types of structures.

Figure 10. Principal tensile strains (a = 0.518) depicted on the incremental deformed meshes for load bearing walls: (a) W-Y1; (b) W-Y2; (c) W-Y3; (d) W-Y4; (e) W-Y5; and (f) W-Y6; (g) W-X1; (h) W-X3. (Ademović 2011, Ademović 2012, Ademović et al. 2013)

On the basis of damage pattern and vulnerability zone and being that this building belongs to the unconfined masonry, younger than approx. 60 years with reinforced concrete floors it can be concluded that for the 7th degree of seismic intensity (corresponding to Sarajevo region where the building is located) moderate to heavy damages (Grades 3–4) could be expected. This has been seen during the time history analysis (Ademović 2012).

6. EXPERIMENTAL RESULTS

In the following section, although experiments of masonry buildings and walls were provided in both countries, several examples of recently conducted experiments have been shown to get an insight into the behavior of masonry buildings and the selection of the strengthening methods. In order to get as much as information regarding existing masonry structures two full scale masonry walls were constructed and tested under cyclic loading. Both walls were made of solid bricks and cement-lime mortar. The bricks were of standard dimensions $b/h/t = 25/12/6.5$ cm (where $b$, $h$ and $t$ are the length, height and width of the unit, respectively). As most of the existing masonry structures are composed with a mortar of an approximate compressive strength being equal to 2.5 N/mm² the composition of mortar was chosen to be lime: cement: sand = 1:0.5:4 (grades by volume). The thickness of the mortar was approximately 1.5 cm. The walls were exposed to constant vertical compression equal to 0.4 N/mm² and cyclic lateral loads. Based on experimental data several features were obtained from hysteresis loops, failure modes and resistance envelope curves. The cracks first started at the lower corner of the wall and as the cyclic loading exchanges, they slowly started propagating towards the middle of the wall. As the cracks formed in the form of a “X” form width of the cracks especially through the head joints increased. Dominant diagonal cracks were formed (Figure 11) at the end of the testing procedure.
Today confined masonry is the common art of masonry structures in both countries. However, the crack propagation from the wall to the tie-column depended on the connection type, which was investigated in work of Matošević et al. (2015). For this reason, nine confined masonry wall specimens, with three different connection details between the masonry and the tie-columns and one unreinforced masonry wall, were tested and hysteresis loops, failure modes and resistance envelope curves were obtained. The masonry walls were tested under constant vertical load of 133 kN and cyclic lateral loads. Clay masonry units V-5 with the dimensions \( bl/t = 25/19/19 \) (cm) that belong to Group 2 according to EN 1996-1-1:2005 with a nominal compressive strength of 15 MPa were used. They were cut to dimensions \( bl*/t = 25/13/19 \) cm (where \( b, h* \) and \( t \) are the length, height and width of the unit, respectively). The concrete used in the ties was C30/37 and the reinforcement was B500B. The general purpose mortar was made in situ in volume proportions cement: hydrated lime:sand = 1:1:5. The horizontal tie-beams were reinforced by 4φ8 mm with stirrups of 6 mm spaced at 14 cm, while the tie-columns were reinforced with a longitudinal reinforcement of 4φ8 mm (anchored to the foundation beam) with closed stirrups of 6 mm spaced at 10 cm. All the specimens were produced in three groups: A1, B1 and C1 then A2, B2 and C2 and finally A3, B3, C3 and D. Group B specimens, presented in Figure 6, had a toothed connection, as it is prescribed in EN 1996-1-1:2005. For specimens in group C (Figure 12), anchoring between the masonry wall and tie-columns was provided by φ6 mm dowel stirrups anchored in every bed joint.

For all the specimen groups, diagonal cracks were dominant, while the crushing of the masonry units at the wall corners was not observed. In the specimens of Group B and Group C, cracks propagated from the wall into the tie-columns, indicating their composite shear failure. In the Group A specimens, horizontal cracks appeared at the outer edges of the tie-columns, indicating their tensile failure. The number and depth of the cracks increased with drift increasing. Slight damage occurred at drifts of 0.16−0.18% for all the specimens. The occurrence of moderate and heavy damage grades, as well as the failure type (brittle or ductile) depended on the connection type. The initial stiffness and average elastic stiffness of all three confined wall types (Groups A, B and C) were almost the same and were 10−26% higher than in the masonry wall (Group D).

7. STRENGTHENING OF MASONRY STRUCTURES

The structural risk that the existing structures represent may be enlarged due to several reasons: deterioration of structural elements due to bad maintenance, reduction of the load-bearing capacity due to removal of inner elements for obtaining more space, construction of additional storey levels and further damages cause by earthquake actions (Ademović 2011).

Which type of strengthening method will be applied depends on type of the structure, quality of the
material of the existing structure, availability of adequate equipment and workers' skills and knowledge, and above all its seismic resistance. The decision will be made according to the required degree of improving the structure's resistance. The state of the structure before the intervention greatly influences the selection of the technical procedure, its applicability and effectiveness. It is important to ensure a good global behavior of the structure due to earthquake action. So, the entire structure has to possess an adequate resistance, ductility and energy dissipation. The correct application of strengthening method is not only of a technical issue but of an economic justification as well (Ademović 2011). Traditional strengthening of concrete structures (Winokur and Rosenthal 1982), (Mourad and Shannag 2012) with the application of ferrocement fount its application in masonry structures as well. Ferrocement in the masonry structures showed its advantages in the increase in ductility as well as improvement of crack resistance (Kaushik et al 1994) and (Ahmed et al. 1994). During the experimental investigations (Papanicolaou et al. 2001) and (Yardim and Lalaj 2016) showed that this type of strengthening increased the stiffness and load carrying capacity as well as increase of in-plane resistance. Strengthening of brick and block masonry walls by RC jacketing has been experimentally investigated only in a few cases (Sheppard and Tomaževič 1986), (Sheppard and Terčelj 1980), (Penazzi et al. 2001), (ElGawady et al. 2004). The improvement in strength depends on the strengthening layer thickness, the cement mortar strength, the reinforcement quantity and the means of its bonding with the retrofitted wall, and the degree of masonry damage. If the connection was not adequate to prevent splitting, the coating separated from the wall at the occurrence of cracks in the masonry wall and buckled. Inadequate strengthening can lead to unfavorable behavior of structures during earthquake actions as seen in Figure 13 due to the incompatibility of the new materials or techniques with the existing materials or techniques in respect to the old traditional strengthening techniques (Binda et al. 2011, Ademović 2017*).

![Figure 13. Inadequate strengthening techniques (Binda et al. 2011)](image13)

The structure presented in 4.2 could be strengthened in several ways. By building additional walls in X direction (weak direction of the structure) while keeping uniform distribution in plane and in height, in order to avoid unwanted torsion effects. As this was not proved to be adequate enough ties were proposed at the location of the ground level. However, the failure mode moved from compression failure to shear failure so this proved to be inadequate. The third proposal was to include FRP locally on the ground floor level with the aim to solve the problem of shear failure and local crushing at the ground floor. Finally, with this kind of strengthening bending damage was observed (Ademović et al 2014).

![Figure 14. Pushover curve – different strengthening proposals (Ademović et al 2014).](image14)

Inappropriate interventions can lead to further damage and even collapse of the structure, as was the case after the Umbria earthquake (Penazzi et al. 2001). Until today many techniques have been exploited for strengthening of the existing unreinforced masonry structures. Decision of the adequate strengthening method has to be done for each structure individually.
8. CONCLUSION

The Balkan and Mediterranean countries, as well as Turkey, are all at higher risk of earthquakes than many other parts of Europe, according to a map produced by EU researchers. It was concluded that the construction in Croatia and Bosnia and Herzegovina is similar, if not to say the same (as it is in other Balkan countries). The traditional art of construction in Bosnia and Herzegovina as well as in Croatia are mostly masonry buildings. The fact that more than 50% of all the dwellings are masonry structures and the fact that they are prone to earthquake actions makes them rather interesting for research. Traditional strengthening of concrete structures was applied to masonry structures. It was noted that the improvement in strength depends on the strengthening layer thickness, the cement mortar strength, the reinforcement quantity and the means of its bonding with the retrofitted wall, and the degree of masonry damage. So, it is not a straight forward process. As the structures are of the same type their results can be used in these countries as well. However, due to the variety of masonry elements and the complexity of its behavior further investigations are necessary.

9. REFERENCES


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