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Seismic Retrofitting of Mid-Rise Unreinforced Masonry Residential Buildings after the 2010 Kraljevo, Serbia Earthquake: A Case Study

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Abstract: There is a significant building stock of post-WWII low- and mid-rise unreinforced masonry (URM) buildings in Serbia and the region (former Yugoslavia). Numerous buildings of this typology collapsed due to the devastating 1963 Skopje, Yugoslavia earthquake, causing fatalities, injuries, and property losses, as well as experienced damage in a few recent earthquakes in the region, including the 2010 Kraljevo, Serbia earthquake (M_W 5.5) and the 2020 Petrinja, Croatia earthquake (M 6.4). These buildings are three- to five-stories high, have clay brick masonry walls, and rigid floor slabs, usually with an RC ring beam at each floor level. This paper presents a case study of a URM building which was damaged due to the 2010 Kraljevo earthquake and subsequently retrofitted. A comparison of seismic analysis results, including the capacity/demand ratio and displacement/drift values, for the original and retrofitted building according to the seismic design and retrofit codes which were followed in Serbia as well as some of the neighboring countries for several decades and Eurocode 8 has been presented. The results of this study show that the selected retrofit solution that satisfied the Yugoslav seismic code requirements is not adequate according to the Eurocode 8, primarily due to significantly higher seismic demand.

Keywords: unreinforced masonry buildings; earthquake damage; seismic retrofitting; residential buildings

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Citation: Blagojević, P.; Brzev, S.; Cvetković, R. Seismic Retrofitting of Mid-Rise Unreinforced Masonry Residential Buildings after the 2010 Kraljevo, Serbia Earthquake: A Case Study. *Buildings* **2023**, *13*, 597. https://doi.org/10.3390/buildings13030597

Academic Editors: Antonio Formisano and Jorge Manuel Branco

Received: 2 January 2023
Revised: 30 January 2023
Accepted: 17 February 2023
Published: 24 February 2023

1. Introduction

Masonry construction technology has been traditionally used for residential construction in European countries, including Serbia and neighboring countries in the Balkan region [1]. Since the second half of the 19th century, residential and public buildings in Serbia and the region have been constructed using clay brick masonry technology. Reinforced concrete (RC) emerged as the preferred technology for the construction of mid- and high-rise buildings since the 1950s; however, masonry continued to be used for low- to mid-rise residential construction in the region. According to the 2011 Census of Serbia (referred to as "Census" in the following text) [2], low-rise single-family buildings constitute 95% of the national residential building stock, corresponding to 65.9% of the total number of housing units. Multi-family housing accounts for only 2.6% of the housing stock in terms of the number of buildings. Still, the proportion is significantly higher (33%) in terms of the number of housing units. According to the Census, 72% of all residential buildings in Serbia were constructed between 1946 (after WWII) and 1990, when Serbia was a part of the Socialist Federal Republic of Yugoslavia (SFRY), referred to as "former Yugoslavia" in this paper (note that Croatia, Slovenia, North Macedonia, Montenegro, and Bosnia and Herzegovina were also a part of the former Yugoslavia). The majority of pre-1960 multi-family residential buildings were unreinforced masonry (URM) buildings, with load-bearing masonry walls as a structural system for resisting both gravity and lateral loads. Most of URM multi-family residential buildings of post-WWII vintage have semi-prefabricated RC floor systems. Buildings of this type constitute a significant fraction of the building stock in urban areas of Serbia and neighboring countries and are the focus of this study. A few examples of typical urban URM multi-family residential buildings from Serbian cities Belgrade and Niš are shown in Figure 1a and Figure 1b respectively.

(**a**)

(**b**)

Figure 1. Examples of URM multi-family residential buildings in Serbian cities: (**a**) Belgrade and (**b**) Niš.

URM buildings are among the most seismically vulnerable building typologies and are expected to experience severe damage and/or collapse in moderate-to-strong earthquake events and cause a significant number of casualties. Masonry is a composite material characterized by a high compressive strength and at the same time, a very low tensile strength. As a result, load-bearing URM wall structures are able to sustain high gravity loads but are vulnerable to the effects of low-to-moderate earthquakes due to tensile stresses and cracking of the walls induced by seismic actions. Another deficiency of URM structures is associated with buildings with wooden floors and roofs, which act as flexible diaphragms and may cause out-of-plane damage or collapse of URM walls with deficient wall-to-floor and wall-to-roof connections. Numerous older URM buildings experienced damage due to recent earthquakes in the region, e.g., the March 2020 Zagreb, Croatia earthquake [3,4] and the December 2020 Petrinja, Croatia earthquake [5].

URM buildings with rigid diaphragms (e.g., RC floor structures), which are the scope of this study, have performed better in past earthquakes compared to otherwise similar URM buildings with flexible diaphragms. Rigid diaphragms enhance the integrity of masonry structures, which is a very important seismic resilient feature. The results of experimental research studies on URM building models subjected to shaking-table testing confirmed the beneficial effect of rigid diaphragms on the seismic performance of URM buildings [6]; however, a few URM buildings with prefabricated RC hollow core slabs collapsed in the November 2019, Albania earthquake due to inadequate wall-to-floor connections and an absence of RC slab topping [7]. It is also important to ensure a symmetrical wall layout and a sufficient number of walls in each horizontal direction in order to minimize increased seismic demands in the walls due to torsional effects in these buildings.

Several experimental and analytical studies have contributed to the state-of-the-art knowledge related to the seismic behavior of URM buildings with rigid diaphragms, which are the scope of this paper. Comprehensive experimental research studies were performed by Prof. Miha Tomaževič and his team in ZAG, Slovenia, on the seismic response of URM walls subjected to reversed cyclic loading, as well as shaking-table testing of URM building models [8]. A few analytical studies also contributed toward the understanding of global behavior and deficiencies of typical buildings, ranging from simple, practice-oriented seismic evaluation approaches to advanced non-linear seismic analysis procedures [9,10].

Buildings of this typology were exposed to several damaging earthquakes in the region, including the devastating 1963 Skopje, North Macedonia earthquake (M 6.0); the 1979 Montenegro earthquake (M_W 6.9); the 2010 Kraljevo, Serbia earthquake (M_W 5.5), and the 2020 Petrinja, Croatia earthquake (M 6.4). A widely accepted EMS-98 damage scale [11] can be used to classify the extent and type of earthquake damage in a standardized manner. EMS-98 includes the following five Damage States (DSs): DS1 (slight damage), DS2 (moderate damage), DS3 (substantial to heavy damage), DS4 (very heavy damage), and DS5 (destruction). A detailed description of the EMS-98 DSs for these buildings reported after past earthquakes in the region has been presented elsewhere [12].

The most extensive damage related to this typology was reported in the 1963 Skopje earthquake, in which many buildings of this type located in Skopje experienced severe (unrepairable) damage (DS4) or partial/full collapse (DS5) (Figure 2a). The total death toll in the earthquake was estimated at 1.500, and a significant number of fatalities was attributed to the collapse of URM multi-family residential buildings [13]. The affected buildings were designed according to standardized designs, which did not consider seismic effects. Hence, it was acceptable to include load-bearing walls in one horizontal direction only, while the lateral load-resisting system in the other horizontal direction was non-existent.

(**a**) (**b**) (**c**) (**d**)

Figure 2. Seismic performance (DSs) of URM residential buildings in past earthquakes in the Balkan region: (**a**) DS5—a collapsed building in Skopje due to the 1963 Skopje earthquake (credit: Z. Milutinović); (**b**) DS4—a URM building in Petrinja experienced extensive cracking in wall piers due to the 2020 Petrinja earthquake (credit: SUZI-SAEE); (**c**) DS3—a URM building in Petrinja experienced a major shear-induced cracking in piers at the ground floor level due to the 2020 Petrinja earthquake (credit: SUZI-SAEE), and (**d**) DS2—a moderately damaged URM building which experienced moderate shear-induced cracking due to the 2010 Kraljevo earthquake (credit: P. Blagojević).

More recently, buildings of this typology were affected by the 2020 Petrinja earthquake, which was characterized by a higher magnitude than the 1963 Skopje earthquake [5]. Some buildings of this type, located in the epicentral area of the earthquake, experienced damage that could be classified as DS4 (Figure 2b) or DS3 (Figure 2c). There were no reports of severe damage or collapse associated with buildings of this type. Note that URM buildings in the Petrinja area had load-bearing walls aligned in both horizontal directions, unlike the buildings which were severely affected by the Skopje earthquake (due to load-bearing walls provided in one horizontal direction only). The main cause of damage for buildings both in the 2010 Kraljevo and the 2020 Petrinja earthquakes was high in-plane seismic demand which exceeded the shear capacity of URM walls.

Buildings of this typology were also affected by the 2010 Kraljevo earthquake, which was less severe (in terms of magnitude) than the 1963 Skopje earthquake. The damage was mostly due to in-plane seismic effects, and was mostly in the form of shear cracks observed in lower portions of the buildings (usually at the ground floor level) [14]. Structural damage observed in the affected buildings could be classified as DS3 or DS2 (Figure 2d).

Although Serbia is located in an area characterized by moderate seismic hazard, urban areas of the country were not affected by major damaging earthquakes in the past century, with the exception of the 2010 Kraljevo earthquake (M_W 5.5). A few other damaging earthquakes took place in Serbia in the last 40 years, namely the 1980 Kopaonik earthquake (M 5.8) and the 1998 Mionica earthquake (M 5.7), but they affected mostly rural areas of the country. Consequently, design and construction experience in Serbia related to repair and seismic retrofitting of buildings in post-earthquake situations has been rather limited. The 2010 Kraljevo earthquake prompted a need for the repair and retrofitting of a significant number of damaged URM residential buildings. The main objective of the post-earthquake recovery was to restore damaged building infrastructure to its original pre-earthquake condition within a relatively short time frame and with limited financial resources. An additional constraint was to minimize the impact of construction activities on building occupants. As a result, the design and execution of seismic rehabilitation projects related to residential buildings used simple retrofitting techniques which were suitable for easy on-site implementation on a large scale. RC jacketing was selected because it was a well-established technique used for the structural strengthening of URM buildings in Serbia before the 2010 earthquake.

In this paper, the authors have shared lessons on seismic retrofitting of URM buildings damaged in the 2010 Kraljevo, Serbia earthquake. The focus of this study was on the evaluation of the effectiveness of RC jacketing, a common seismic retrofitting technique for URM buildings. This study also provides an insight into differences in the results of seismic evaluation and retrofitting of masonry building according to the seismic codes from former Yugoslavia and Eurocode 8. The PTN-S code for seismic design of new structures and the PTN-R code for seismic retrofitting of existing structures were followed in Serbia and former Yugoslavia for almost 40 years. It should be noted that Eurocode 8 became the governing code for seismic design and retrofitting of buildings in Serbia in 2019. Similarly, neighboring countries in the region are either in the process of adopting Eurocode 8, or have adopted it in the last 5–10 years. A case study on a URM building in Kraljevo, which was damaged due to the 2010 earthquake and subsequently retrofitted, is presented in this paper. A comparison of seismic analysis results, including the capacity/demand ratio and displacement/drift values, has been performed for the original and retrofitted building, according to both the Yugoslav seismic design and retrofit codes and Eurocode 8. Linear elastic seismic analysis was the default procedure for ordinary buildings, such as residential buildings, according to the seismic codes from former Yugoslavia, and was used in this study. The results of this study show that the selected retrofit solution satisfied the Yugoslav seismic code requirements, but it is not adequate according to the Eurocode 8 requirements. The findings of this paper may be of particular interest to engineers in the Balkan countries, which recently adopted Eurocode 8 as the governing code for the seismic design of new buildings and the evaluation/retrofitting of existing buildings.

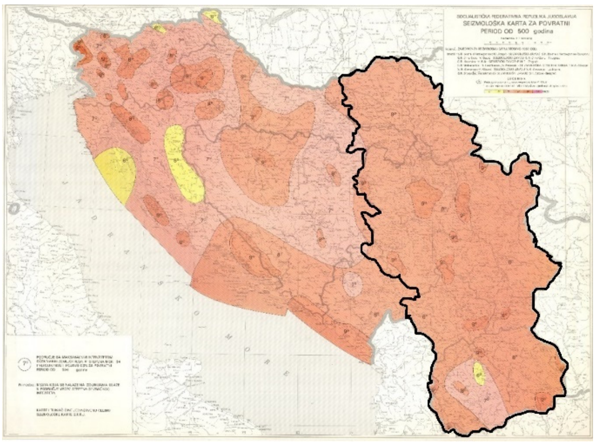
2. Design Code Requirements for Seismic Retrofitting of Masonry Buildings from the Serbian Code and Eurocode 8, Part 3

The first comprehensive seismic design code in the SFRY was published in 1964 [15], after the devastating 1963 Skopje earthquake. Its subsequent edition (PTN-S), issued in 1981 [16], was the governing design code in Serbia until 2019. It was reported that the PTN-S code was as advanced as other international seismic design codes at the time [17].

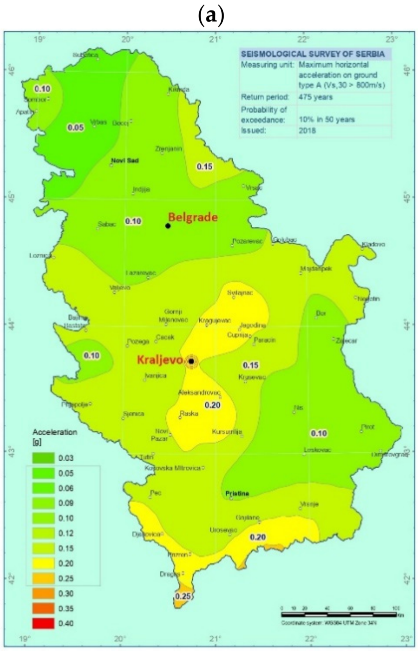
Eurocodes were adopted as official codes for the design, construction, and maintenance of building structures in Serbia in 2019 [18]. As a result, Eurocode 8—Part 1 [19] (also referred to as EC8-1 in this paper) is currently used for the seismic design of new buildings

(SRPS EN 1998-1/NA:2018) [20], while Eurocode 8—Part 3 [21] (also referred to as EC8-3 in this paper) has been followed in projects related to seismic assessment and retrofitting of existing buildings (SRPS EN 1998-3/NA:2018, 2018) [22].

Figure 3 presents seismic hazard maps for Serbia. Deterministic seismic hazard maps used in the SFRY were originally published in 1982 as a companion to the PTN-S code and were updated in 1987 (Figure 3a). The territory of Serbia was divided into zones VI to IX based on the MCS-64 seismic macrointensity scale (note that the borders of Serbia are shown in black color on the map). Figure 3b shows the current official seismic hazard map for Serbia developed for the design according to Eurocode 8 [23]. According to the map, Peak Ground Acceleration (PGA) values for design-level earthquake with 10% probability of exceedance in 50 years (corresponding to a 475-year return period) are largest for Southern and Central Serbia (0.25 g and 0.20 g, respectively), while other localities in the country have been assigned lower PGA values. Seismic design requirements for masonry buildings from Serbian codes and a comparison with the corresponding Eurocode 8 provisions were outlined by the authors in an earlier paper [1].



(**a**)



(**b**)

Figure 3. Seismic hazard maps: (**a**) seismic intensity map for SFRY (including Serbia) according to the MCS-64 scale published in 1987 for the 500-year return period earthquake (in compliance with the PTN-S code) and (**b**) seismic hazard map for Serbia showing the design PGA values for an earthquake with 10% probability of exceedance in 50 years, according to Eurocode 8, Part 1 (EC8-1).

Based on the experience gained after the 1979 Montenegro earthquake, the first national design code for repair, rehabilitation, and retrofitting of existing buildings was issued in 1985 (PTN-R) [24]. The code addressed structural rehabilitation and seismic retrofitting of masonry and RC buildings and the foundations.

Structural rehabilitation of existing masonry buildings was performed to improve their load-bearing capacity, and it was mandatory in one of the following cases: (i) when the walls have experienced structural damage, (ii) when the number of walls in each horizontal direction (expressed in terms of the total cross-sectional area) is inadequate, (iii) when allowable stresses in the walls have been exceeded, (iv) when there is a change of building function, and the total mass is increased by more than 10%, or (v) in case of a building renovation/extension.

According to the PTN-R code, the main performance objective was the same for retrofitted existing buildings and new buildings, that is, structural damage due to a major damaging earthquake was acceptable, but collapse had to be avoided. Seismic retrofitting provisions for a specific building were dependent on its configuration, type, and quality of the materials, the extent of damage, as well as the expected seismic performance. The code prescribed the following approaches for structural rehabilitation or seismic retrofitting of existing masonry buildings: (a) rehabilitation/retrofitting of the existing load-bearing structure, (b) reconstruction of the existing walls, (c) construction of new structural walls to enhance the seismic capacity of the existing lateral load-resisting system, and (d) retrofitting of existing floor structures and wall-to-floor connections (discussed in Section 3.7).

The PTN-R code prescribed that damaged clay brick/block masonry walls had to be either replaced (if they experienced heavy damage) or repaired by injecting cracks using cement-based grout. Masonry walls could also be strengthened by means of the following alternative techniques: (a) by applying one- or two-sided RC jackets (3–5 cm thick); (b) by constructing new RC confining elements, or (c) by post-tensioning of existing walls. In the context of RC jacketing, the code prescribed the minimum amount of distributed horizontal and vertical reinforcement, concentrated reinforcement at the wall ends, and the anchorage of vertical reinforcement into the floor/roof structure. A discussion on the wall retrofitting techniques is presented in Section 3.

A key seismic analysis requirement was to simulate the effect of both original and new (or retrofitted) structural elements by considering their deformation characteristics. The code provided prescriptive provisions related to the modelling of masonry walls retrofitted by means of RC jacketing. The thickness of a retrofitted wall had to be equal to the original wall thickness plus four times the thickness of each RC jacket. The intent of this provision was to approximately take into account the effect of RC jackets on increasing wall stiffness.

Seismic retrofitting solutions could be designed either according to the Allowable Stress Design approach or the Ultimate Limit States design approach. When the structural safety of a wall retrofitted using the RC jacketing technique was verified using the Allowable Stress Design approach, the thickness of the retrofitted wall was taken equal to the sum of thicknesses for the original masonry wall and RC jackets.

Similarly to the PTN-R code, which was developed in former Yugoslavia, Eurocode 8, specifically EC8-3 (Annex C), outlined provisions related to seismic retrofitting of masonry buildings. Acceptable seismic analysis approaches include linear static and dynamic analysis procedures. The numerical model of a masonry structure needs to consider the cracked stiffness of structural elements/walls, which may be taken as 50% of their uncracked values. It should be noted that the PTN-R code did not prescribe any stiffness reduction; that is, gross (uncracked) section properties were considered in the analysis.

EC8 prescribed nonlinear seismic analysis (nonlinear static or dynamic procedure) for irregular structures, while linear elastic seismic analysis was permitted for regular structures. It should be noted that the PTN-R code did not contain provisions related to the application of nonlinear seismic analysis procedures for the evaluation of existing buildings.

Repair and retrofitting techniques for masonry buildings, prescribed by the EC8-3 (Annex C), are similar to those prescribed by the PTN-R code and include the repair of cracks, repair and retrofitting of wall intersections, retrofitting and stiffening of horizontal diaphragms, provision of tie beams, retrofitting by means of steel ties, retrofitting of walls by means of RC jackets or steel profiles, and fiber-reinforced polymer (FRP) jackets.

In addition to the PTN-R code, which was the governing code for seismic retrofitting of buildings in Serbia until 2019, all masonry structural components (e.g., walls) had to be designed or evaluated according to the PTN-Z code which was issued in 1991 [25]. Similarly, Eurocode 6 (EN 1996-1-1:2004) [26], which is currently used in Serbia, contains design provisions for masonry buildings.

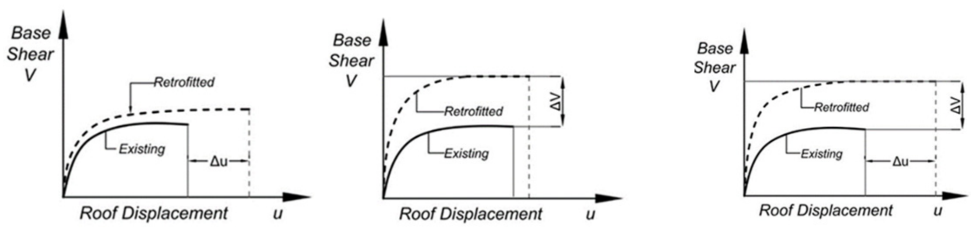
3. Seismic Retrofitting Techniques for URM Buildings

3.1. Seismic Retrofitting Objectives

Seismic retrofitting solutions should be effective in enhancing the performance of existing structures to achieve predetermined performance objectives. Performance objectives for a specific structure are either set by the design code or project-specific design criteria. In some countries, technical codes/standards for existing buildings may permit relaxed seismic performance objectives for the evaluation and retrofitting of existing buildings relative to the design of new structures, e.g., ASCE/SEI 41-17 code in the USA [27]. However, seismic design codes in former Yugoslavia did not make a distinction between new and existing buildings in terms of performance objectives.

One of the key design aspects of a seismic retrofitting project is to identify retrofitting goals. After the seismic evaluation of a building is performed and the deficiencies have been identified, a designer should be able to determine the retrofitting goals. Is the main goal of the retrofitting to enhance the lateral load-resisting capacity and/or stiffness and/or ductility of the existing structure—or perhaps a combination of those structural characteristics? An appropriate seismic retrofitting solution may be selected after the goals have been established.

Figure 4 illustrates different seismic retrofitting goals [28,29]. In many cases, the primary goal of retrofitting is to enhance the ductility of the existing structure, which may be feasible for the retrofitting of older RC structures (Figure 4a). Alternatively, stiffness and capacity enhancement (Figure 4b) may be feasible for retrofitting of an existing non-ductile structure. For example, the provision of new RC shear walls or steel bracings are common retrofitting techniques for enhancing both the capacity and stiffness of existing buildings. Stiffness, capacity, and ductility enhancement (illustrated in Figure 4c) may be feasible for existing buildings with high seismic demand, which prompts a need for increased lateral load-resisting capacity. On the other hand, stiffness may be increased due to inherent features of a selected retrofitting technique, e.g., provision of new shear walls or braces. In the context of URM structures, it is important to note that it is unlikely for a retrofitting solution to achieve a significant increase in ductility due to the brittle nature of masonry. It is expected that a typical global retrofitting solution for a URM structure should primarily be effective in increasing its lateral load-resisting capacity. Several researchers have studied different seismic retrofitting techniques for RC and masonry buildings and compared their effectiveness [10,30–36].



(a) (b) (c)

Figure 4. Seismic retrofitting goals for existing buildings: (**a**) ductility enhancement; (**b**) stiffness and strength enhancement, and (**c**) stiffness, strength and ductility enhancement [30].

3.2. Seismic Retrofitting Techniques—An Overview

Seismic retrofitting projects in the Balkan region were initiated after the 1979 Montenegro earthquake (M 6.9), which caused the damage and collapse of buildings in coastal areas of Montenegro and Croatia. Engineers and academics from all parts of the former Yugoslavia participated in the planning, design, and construction supervision of post-earthquake recovery. The earthquake also prompted a few relevant regional projects, which engaged experts from neighboring countries, such as the UNIDO-sponsored project "Building Construction Under Seismic Conditions in the Balkan Region". A series of comprehensive technical resources were produced as a result of the project, including the guidelines for seismic retrofitting of existing RC and masonry buildings [37]. Notable experimental research studies and field applications of seismic retrofitting on existing masonry buildings were performed by Prof. Miha Tomaževič and his colleagues at ZAG, Slovenia [32,38]. Comprehensive technical guidelines have recently been developed for the repair and retrofitting of masonry buildings affected by the March 2020 Zagreb, Croatia earthquake [39]. A valuable resource is available in Serbia for engineers engaged in the structural and seismic rehabilitation of buildings [40].

The most common retrofitting techniques for URM structures include: (i) retrofitting of existing masonry walls, (ii) construction of new RC walls attached to the existing masonry walls, (iii) retrofitting of wall connections, as well as the wall-to-floor connections, and (iv) retrofitting of the existing floor and/or roof structures. In some cases, retrofitting of existing foundations may be required, when shear and/or flexural capacity of the retrofitted wall have increased as a result of the retrofit.

A brief overview of selected seismic retrofitting techniques for URM structures is presented in the following text.

3.3. RC Jacketing—A Wall Retrofitting Technique

The RC jacketing technique (Figure 5) consists of constructing one- or two-sided RC jackets which need to be attached to the exterior and/or interior wall surfaces [30]. A jacket consists of a 3 to 5 cm thick concrete overlay with reinforcement in the form of a steel mesh (usually small size bars, 4 to 8 mm diameter). RC jackets are usually attached to an existing masonry wall via steel anchors inserted in pre-drilled holes, which are subsequently filled with cement- or epoxy-based grout. The required size and spacing of anchors depend on seismic demand and the required jacket thickness [41,42]. Either cast-in-place concrete or sprayed concrete (shotcrete) can be used for RC jackets.

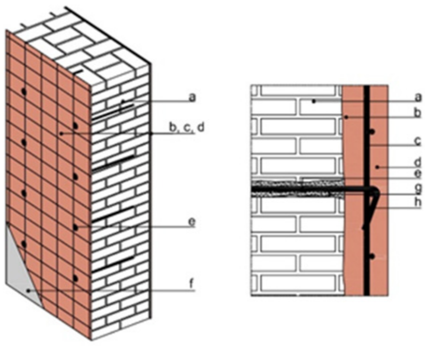


Figure 5. A schematic diagram of a retrofitted URM wall with RC jacketing (Legend: (a) existing masonry wall; (b) first layer of concrete; (c) steel wire mesh; (d) second layer of concrete; (e) steel anchors; (g) a grouted hole; (f) cement-based plaster; and (h) a steel anchor with a 90-degree hook).

The design of masonry walls retrofitted by RC jackets is performed by considering the stiffness of the retrofitted wall as the sum of the stiffnesses of the original masonry wall and the RC jackets. As a result, the internal shear force in the jacket is proportional to its stiffness relative to the total wall stiffness. It is worth noting that the stiffness of the jacket is influenced by the jacket thickness and mechanical properties of concrete (modulus of elasticity).

A significant increase in the shear capacity and stiffness of the masonry walls retrofitted using RC jacketing has been reported based on experimental studies on masonry wall specimens subjected to monotonic and/or reversed cyclic lateral loading [41,43–46]. The tests revealed that RC jacketing was able to increase the lateral capacity of the specimens by a factor of 2.0 to 3.0. Specimens with two-sided jacketing exhibited higher ductility and energy dissipation capacity. A few researchers performed experimental studies on the shear capacity of masonry wallets reinforced with ferrocement [47,48]. Shaking table testing of a four-story masonry building model retrofitted with RC jacketing was also performed [49].

In most cases, researchers used steel reinforcement (welded wire mesh) for RC jacketing; however, a few researchers tested specimens with jacketing consisting of fiber-reinforced concrete with glass or steel fibers, which exhibited superior performance compared to specimens retrofitted with RC jacketing [50].

3.4. A New RC Overlay/Wall Attached to the Existing One—A Wall Retrofitting Technique

When an existing URM wall has deficient gravity and lateral load-resisting capacity, it can be strengthened by constructing a new RC shear wall which is attached to the existing URM wall [51]. This is essentially a similar concept to RC jacketing, except that the thickness of the new wall is 10–15 cm, while the thickness of an RC jacket is usually on the order of 5 cm. Addition of a new RC wall results in a significant increase in lateral stiffness, shear and flexural capacity of the existing wall. A new RC wall is attached to the existing masonry wall in the same manner as previously explained for RC jacketing, except that the amount of wall reinforcement and anchors may be different. Retrofitting of wall foundations is usually required due to a significant increase in the shear and flexural capacity of a retrofitted wall.

This technique has proven to be effective for seismic retrofitting of damaged URM walls. Figure 6a shows a conceptual retrofitting scheme, which consists of constructing a new RC wall attached to the existing masonry wall by means of steel anchors. Figure 6b shows a possible location for a new RC wall attached to an existing exterior URM wall.

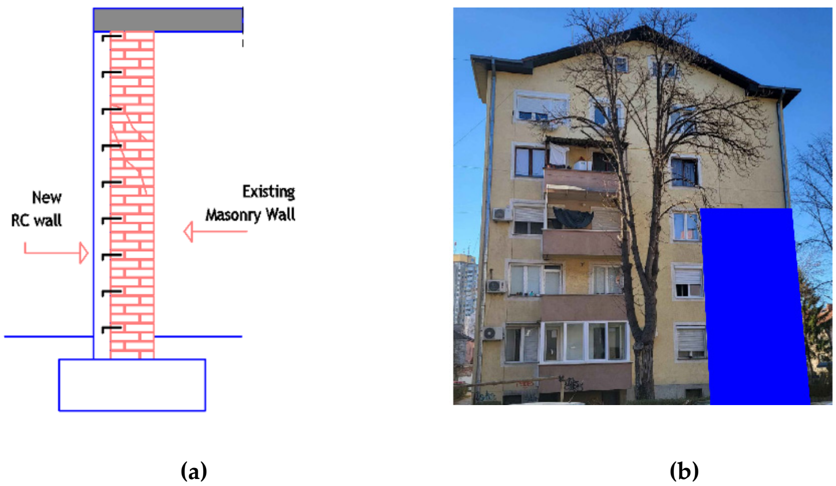
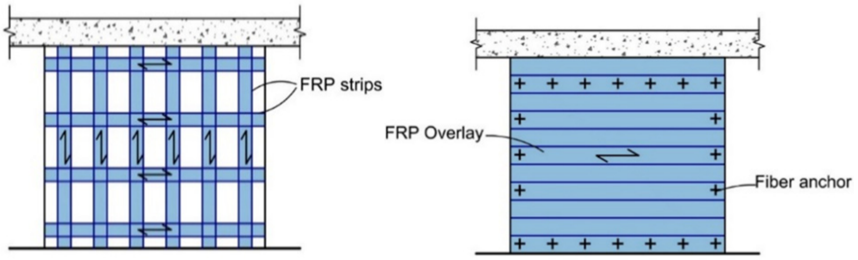


Figure 6. New RC walls: (**a**) a conceptual seismic retrofitting solution showing new RC wall and existing URM wall, and (**b**) a building elevation showing retrofit location.

3.5. FRP Overlays or Strips—A Wall Retrofitting Technique

Seismic retrofitting can also be achieved by applying fiber-reinforced polymer (FRP) overlays or strips on wall surfaces that were previously saturated by epoxy resin (or an alternative). FRP overlays may cover the entire wall surface (Figure 7a); alternatively, they could be applied in the form of strips aligned in horizontal, vertical, or diagonal directions (Figure 7b). FRP overlays and strips can be used either as one-sided or two-sided applications. To ensure an adequate anchorage, these FRP overlays/strips can be either wrapped (extended) at the wall ends, or custom-designed fiber anchors can be installed along the wall perimeter [30] (Figure 7a).



(a) (b)

Figure 7. Seismic retrofitting of masonry walls using FRPs: (**a**) a FRP overlay applied over the entire wall surface, and (**b**) horizontal and vertical FRP strips [30].

FRP overlays act in a similar manner to RC jackets and are suitable for application to existing masonry walls with a deficient shear capacity. The fibers act as tension reinforcement for the wall and should be aligned in the direction of tensile stresses. The required effective area of fibers per unit width and the FRP contribution to shear capacity of the retrofitted wall are governed by the bond and anchorage strength at the FRP-to-wall interface. Design procedures for FRP-based retrofitting of masonry structures are well established [52].

The FRP technology has been used in the last 30 years, initially for structural rehabilitation purposes (e.g., existing bridges) and later on for seismic retrofitting of RC and masonry structures before and after earthquakes. Numerous experimental research studies on masonry specimens retrofitted with FRP overlays have been performed since the 1990s. Early experimental research studies were focused on testing small specimens, such as masonry triplets with bonded GFRP fabrics (with glass fibers) which are subjected to monotonic shear loading. The results showed that the mode of failure was governed by the FRP tensile strength [53].

One of the earliest experimental research studies involved the testing of a URM wall specimen retrofitted with a GFRP overlay on one side and vertical strips on the other side of the wall. The specimen was subjected to reversed cyclic lateral loading, and the results showed a capacity increase by a factor of 2.2 and a ductility increase by a factor of 2.5 compared to an otherwise similar URM specimen [54]. Delamination of the GFRP overlay was observed in the region of high tensile stresses in the middle portion of the wall. A study on URM piers retrofitted with horizontal and vertical GFRP strips showed a significant increase in the shear capacity by a factor of more than 2.0 for specimens subjected to reversed cyclic loading [33]. The behavior was governed by the delamination of vertical GFRP strips. A significant loss of shear capacity was observed after delamination (debonding) of the FRP from the wall surface. A study on seven scaled URM wall specimens with FRP overlays showed that one-sided retrofitting resulted in significantly enhanced lateral capacity, stiffness, and energy dissipation of the test specimens [55]. The results show that an increase in the lateral capacity was proportional to the amount of FRP axial stiffness and that high FRP axial stiffness led to a brittle failure. A comprehensive experimental study comprised testing 28 full-size brick masonry wall specimens with four different FRP layouts [56]. The results of the study emphasized the importance of a careful design of FRP retrofit scheme as well as material compatibility in developing seismic retrofitting solutions. In another study, a URM wall specimen with a CFRP overlay (with carbon fibers) and custom-designed fiber anchors installed along vertical and horizontal wall edges was subjected to reversed cyclic loading [57]. The results showed a 50% increase in the shear capacity compared to the URM wall specimen, and a significant drift capacity of the retrofitted wall (4.5%).

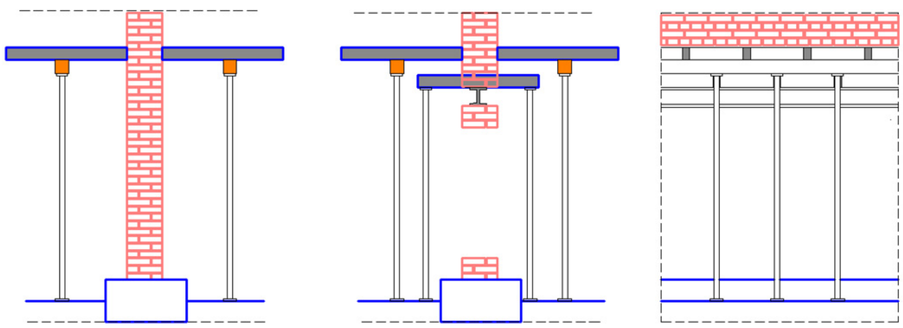
The effectiveness of vertical GFRP strips (with glass fibers) for enhancing the seismic performance of masonry piers was studied on a single-story masonry building model subjected to pseudo-dynamic loading [58]. The results show that GFRP strips were effective in increasing the pier lateral capacity and stiffness but did not cause a change in the original failure mechanism (pier rocking).

Shaking table tests on five half-scale masonry wall specimens retrofitted by means of single-sided FRP configurations with glass, aramid, and carbon fibers were also performed [59,60]. The results show that retrofitting with GFRP fabrics improved the shear capacity of masonry walls by a factor of about 2.5. In another study, shaking-table tests of five masonry walls retrofitted using GFRP strips in four different configurations (including a full-surface overlay and a combination of strips) showed that all retrofitted specimens performed well during the design-level shaking, and three out of four GFRP configurations also performed well during the extreme-level shaking [61]. The tests showed that the use of vertical GFRP strips alone is able to improve the in-plane performance of URM walls. A shaking-table testing on a scaled URM building model retrofitted with CFRP strips was performed at ZAG, Slovenia [8]. The model retrofitted using CFRP strips resisted to 3.5 times stronger shaking compared to the control model and did not experience a collapse.

Experience related to the application of FRP technology in Serbia and the region is limited; however, this technology has been recently used for the retrofitting of masonry buildings after the 2020 Zagreb, Croatia earthquake [62].

3.6. Replacement of Existing Masonry Walls

In some cases, it may be necessary to demolish an existing damaged wall and replace it with a new masonry or RC wall or frame, with enhanced gravity and lateral load capacity. The same procedure can be applied when it is required to enlarge an opening in an existing load-bearing wall. Figure 8 illustrates the reconstruction process. Initially, scaffolding and formwork are installed as preparation for the demolition of a lower portion of the wall (Figure 8a). Subsequently, supports and cross beams are provided to support the upper portion of the wall (Figure 8b). A side view of the arrangement for supporting the upper portion of the wall during the intervention on its lower portion is shown in Figure 8c).



(a) (b) (c)

Figure 8. Demolition of a damaged URM wall and construction of a new wall: (**a**) scaffolding and formwork prepared for wall demolition; (**b**) setup for supporting upper portion of the wall, and (**c**) a side view of the arrangement for supporting upper portion of the wall.

This approach was used to increase the seismic resistance of the Diocese in Pančevo, Serbia, an existing URM building of cultural and historical importance which was constructed in 1832 (Figure 9) [63]. Loadbearing URM walls were 62 cm thick and were constructed in lime mortar. Due to earlier interventions, significant portions of load-bearing walls were removed and openings were formed in transverse walls (shown in solid red color on the floor plan, Figure 10). As a result of these interventions, the seismic integrity of the building in the transverse direction was jeopardized and a structural intervention was required.

Figure 9. The Diocese in Pančevo, Serbia—a view of the front façade.

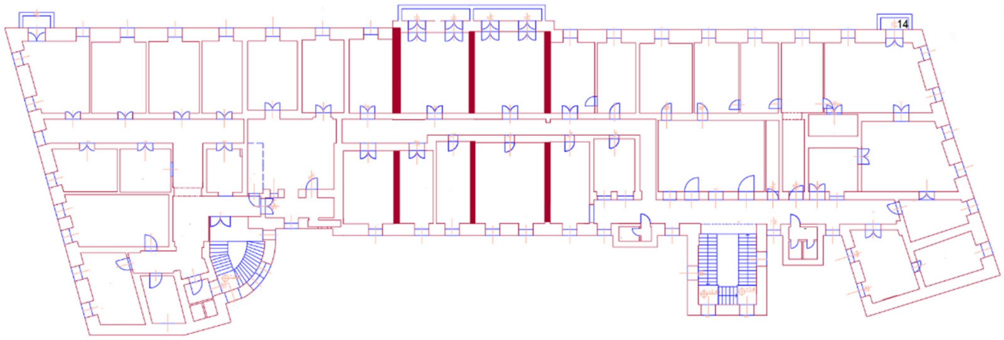
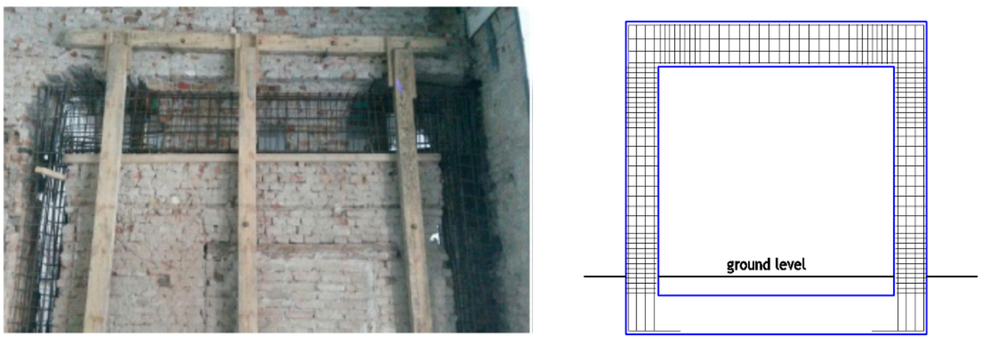


Figure 10. Floor plan of the Diocese in Pančevo.

In order to restore the lateral load-resisting capacity of the structure in the transverse direction, the width of new RC frames constructed at the locations of original walls was the same as wall thickness (Figure 10). RC beams were 72 cm high, while column dimension in the plane of the frame was 50 cm. Seismic analysis and design of new frames in the transverse direction at the ground floor level were carried out according to the PTN-S and PTN-R codes (and applicable codes for the design of RC structures). Figure 11a shows a scaffolding arrangement for transferring gravity loads from the upper floors to the ground, as well as reinforcement for the RC frame. Reinforcement detailing for a typical RC frame is shown in Figure 11b.



(a) (b)

Figure 11. New RC frames were constructed as a part of the seismic retrofitting of the Diocese in Pančevo: (**a**) formwork and scaffolding arrangement and (**b**) frame reinforcement detailing.

3.7. Retrofitting of Flexible Floor Structures

The retrofitting of existing flexible floor structures in masonry buildings, usually made of timber, is often required, and it was prescribed by the PTN-R code. The following approaches were permitted by the code: (a) steel ties provided parallel with the walls on both sides (underneath the floors or roof); (b) diagonal bracing of existing timber floors in horizontal plane, and (c) replacement of an existing timber floor by a new RC floor [38]. Additionally, wall-to-floor connections may need to be strengthened. Figure 12 shows the application of the approach (b) for strengthening of the floors in a URM building after the 2010 Kraljevo earthquake [42]. Seismic assessment of the building showed that wall retrofitting would not be required provided that floor and roof structures acted as rigid diaphragms. A portion of the floor structure was a rigid RC floor, while another portion was a flexible timber floor (since the original building was extended in the horizontal direction). The design engineers decided to retrofit a flexible portion of the floor structure by means of a horizontal steel truss. The truss elements were anchored into RC ring beams which existed at each floor level.

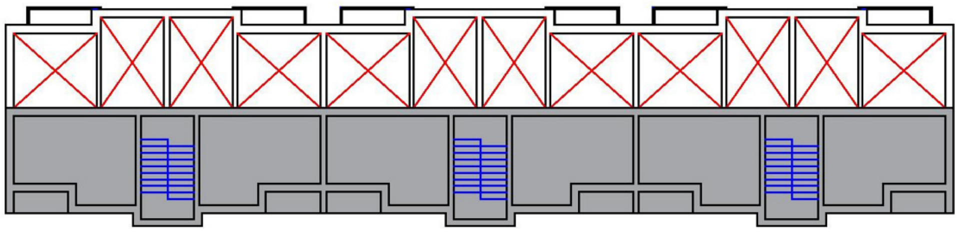


Figure 12. Retrofitting of an existing flexible floor structure by means of horizontal steel truss after the 2010 Kraljevo, Serbia earthquake.

3.8. Comparison of Seismic Retrofitting Techniques for Masonry Buildings

A comparison of seismic retrofitting techniques for masonry buildings is summarized in Table 1 [30]. The following criteria are deemed relevant for selecting the most suitable seismic retrofitting technique: (i) local availability of construction materials, (ii) required level of construction skills, (iii) construction cost, (iv) disruption to the occupants, and (v) required maintenance.

RC jacketing is often considered a more feasible solution compared to the retrofit performed using FRP technologies due to lower construction cost. Another advantage of RC jacketing is that advanced construction skills are not required for its implementation; however, disturbance to the occupants during the construction is significantly higher compared to the FRP retrofit. Advantages of FRPs include ease and speed of application, thus resulting in minimal disruption to the occupants. It should be noted that FRP technologies were not widely used for the retrofitting of buildings affected by the 2010 Kraljevo earthquake, but are currently used for various structural and seismic retrofitting projects in Serbia. Unit construction costs for different retrofitting techniques are included in Table 1 (column 6). These costs are based on the seismic retrofitting projects of URM buildings performed after the 2010 Kraljevo earthquake. Unit costs are expressed in EUR/m^2 and represent cost per m^2 of the wall area.

Table 1. A comparison of seismic retrofitting techniques for masonry walls.

Retrofitting Technique	Advantages	Disadvantages	Local Availability of Construction Materials	Required Level of Construction Skills	Construction Cost	Disruption to the Occupants	Required Maintenance
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**
RC jacketing	(i) One of the most cost-effective retrofitting techniques. (ii) Able to enhance flexural and/or shear capacity and/or ductility.	(i) Adds mass/weight to the structure. (ii) Drilling holes through the existing walls may be required.	High	Low	Low (approximately 99 EUR/m^2 based on 5 cm thick RC jacket)	Moderate to high	Low
FRP overlays and strips	(i) Increases shear and/or flexural capacity of the existing wall. (ii) Lightweight. (iii) Rapid installation.	Requires fire and UV protection.	Low	Moderate to high	Moderate (approximately 110 EUR/m^2 for CFRP strips)	Low	Low
Replacement of an existing masonry wall with a new masonry or RC shear wall	One of the most effective retrofitting techniques.	(i) May increase seismic forces at the wall-to-floor slab interface. (ii) Requires new foundations. (iii) Need to drill holes through the existing RC floor slabs.	High	Medium	Moderate	High	Low

4. Seismic Retrofitting of Damaged URM Buildings after the 2010 Kraljevo Earthquake: A Case Study

4.1. The Earthquake and Its Consequences

The most damaging earthquake in Serbia since the beginning of the 21st century occurred on 3 November 2010, and had a magnitude (M_W) of 5.5 [64]. The epicenter was located close to the Sirča village, approximately 4 km north of Kraljevo (a town with a population of 68,000). The earthquake caused two fatalities and more than USD 100 million in damages [65]. Out of 16,000 buildings that experienced damage or collapse due to the earthquake, approximately 25% were found to be unsafe to occupy [66]. A large number of single-family dwellings, as well as some multi-family residential buildings, educational, and health facilities, were affected by the earthquake. Masonry buildings, which accounted for more than 90% of the building stock in the earthquake-affected area, were most severely affected by the earthquake [67].

Several multi-family URM buildings (three- to five-stories high) constructed after WWII (1945–1963) were damaged due to the earthquake and required repair and retrofit [42,67,68]. The masonry walls in these buildings were typically constructed using solid clay bricks, and their thickness ranged from 25 cm (interior walls) to 38 cm (exterior walls). Floor structures were in the form of ribbed RC slabs or semi-prefabricated clay and concrete floors, and RC tie-beams (ring beams) were provided at each floor level. In most cases, the walls experienced moderate damage in the form of cracks due to in-plane or out-of-plane seismic loads [14,42,67]. Some of the damaged buildings had vertical extensions (additional floors), and it was reported that the extensions which were not designed and constructed according to the existing technical regulations were damaged in many cases [69].

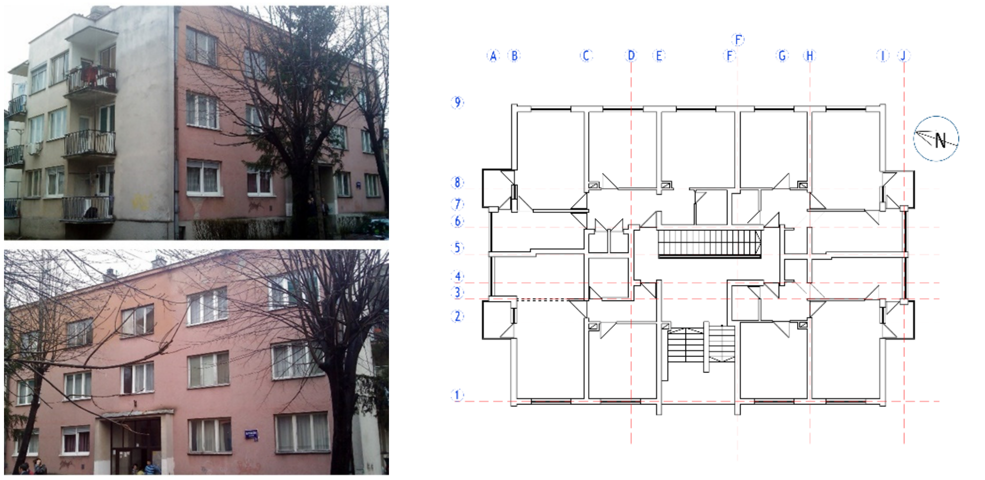
A URM residential building in Kraljevo was retrofitted before the 2010 earthquake due to the upcoming vertical extension, which prompted the need for seismic evaluation and retrofitting. The building was retrofitted using RC jacketing, and it did not experience any damage in the earthquake [69]. The building was located in Jug Bogdanova Street in Kraljevo, in the vicinity of a few other similar URM buildings which experienced moderate-to-severe structural damage and had to be repaired and retrofitted after the earthquake (see Figure 13).

This section discusses in detail a URM building in Kraljevo that was damaged due to the 2010 earthquake and was retrofitted after the earthquake by applying a seismic retrofitting approach, which was prescribed by the Serbian code PTN-R and was used in Serbia and the region since 1985.

4.2. Case Study Building: Description and Earthquake Damage

The case study building is located in Njegoševa Street No. 2 in Kraljevo, and was constructed around 1950 as a three-story residential building with a basement and a half-floor at the top (typical story height was 2.8 m), see Figure 14. The building had a rectangular plan shape with 22.2 m length and 16.0 m width for all floors, except for the top floor (extension) with smaller plan dimensions (11.0 m length and 10.9 m width). The walls at the lower three floors were constructed using 25 cm solid clay bricks in cement–lime mortar, while the walls at the top floor were constructed using modular (multi-perforated) clay blocks with 120 mm thickness. Floors and roofs were constructed using semi-prefabricated composite masonry and a concrete system which were considered to act as rigid diaphragms. The walls were constructed using URM construction, but RC tie-beams were provided at each floor level.

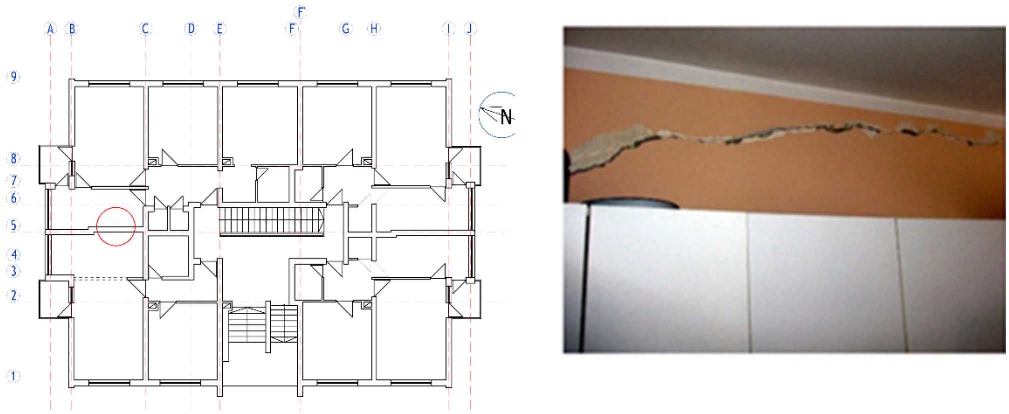
Figure 13. Jug Bogdanova Street in Kraljevo, Serbia after the 2010 earthquake: examples of damaged and undamaged masonry buildings.



(**a**) (**b**)

Figure 14. URM building located in Njegoševa Street No. 2, Kraljevo: (**a**) exterior views and (**b**) typical floor plan.

As the building was constructed around 1950, seismic effects were likely not considered in the original design. The building was damaged in the 2010 earthquake. Structural damage in the lower portion of the building was mostly in the form of inclined cracks due to in-plane seismic effects. Cracking was most prominent in longitudinal walls (N–S direction); for example, wide cracks were observed along the masonry-to-tie-beam interface in a longitudinal wall along gridline 5 at the second-floor level (Figure 15). The most extensive damage was observed in the extended portion of the building at the top floor level. Refer to [1] for more details related to the seismic performance of the building in the 2010 Kraljevo earthquake and a seismic evaluation of the damaged structure according to the PTN-S code and Eurocode 8.

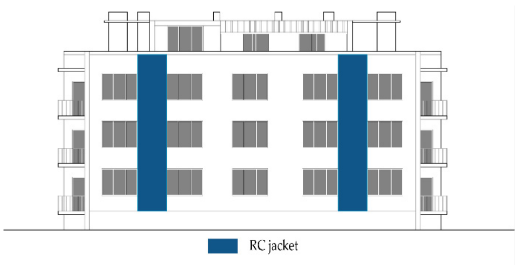


(**a**) (**b**)

Figure 15. Example of earthquake damage in a case study building: (**a**) floor plan at the second-story level showing a damaged wall (red arrow) and (**b**) severe cracking in a longitudinal wall (gridline 5).

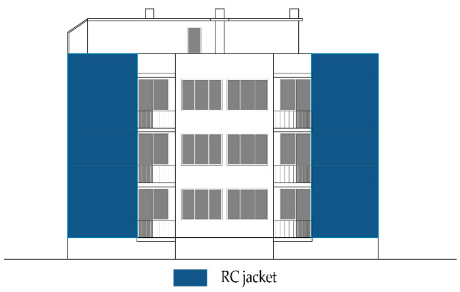
4.3. Seismic Retrofitting Approach

The building was retrofitted according to the PTN-R code. The main goal of seismic retrofitting was to enhance the overall structural integrity by constructing vertical RC jackets along the façade (embellished in navy blue color in Figure 16). The main reason for performing retrofitting at the exterior was to minimize the disruption to the building occupants.



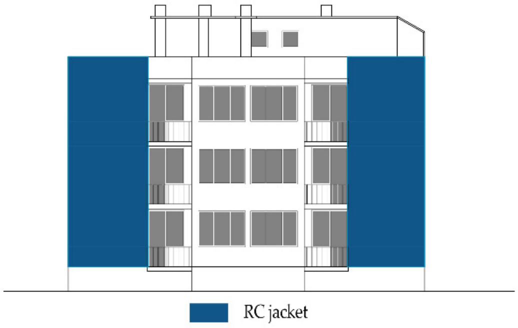
(**a**)

(**b**)



(**c**)

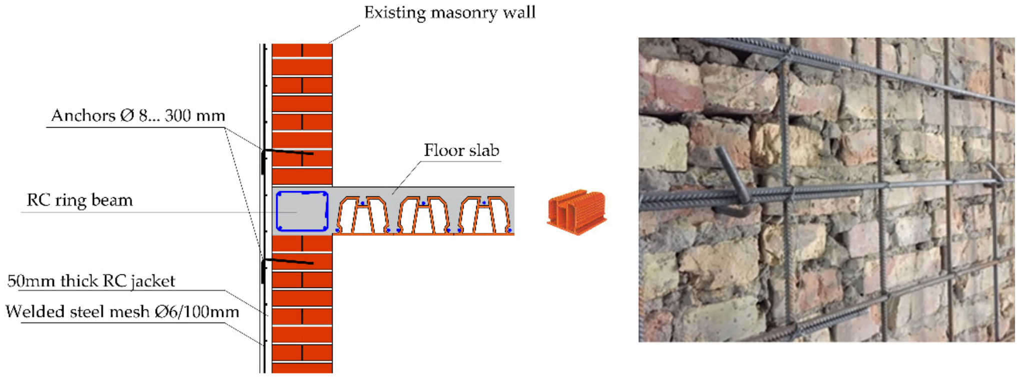
Figure 16. *Cont.*



(**d**)

Figure 16. Elevations of the retrofitted building: (**a**) east façade; (**b**) west façade; (**c**) north façade, and (**d**) south façade.

RC jackets were constructed using 50 mm thick concrete reinforced by means of 6 mm diameter welded steel mesh at 100 mm spacing (Figure 17). The jackets were attached to the existing wall by means of 8 mm diameter steel anchors spaced at 300 mm horizontally and vertically. Prior to the retrofit, the wall surface was prepared by removing plaster and sandblasting. The structural cost of the retrofitting (based on m^2 of the built-up area) was approximately 8.4 EUR/m^2.



(**a**) (**b**)

Figure 17. RC jackets: (**a**) vertical section of the wall and (**b**) an example of RC jacketing under construction [30].

4.4. Seismic Analysis and Numerical Models

Seismic evaluation and retrofit design of earthquake-damaged buildings in Kraljevo was performed in line with the technical regulations which were enforced in Serbia at the time of the 2010 earthquake, that is, the PTN-S code (which prescribed seismic design parameters and analysis procedure) and the PTN-R code (which contained provisions related to retrofit design). These codes prescribed linear elastic analysis for both the original and retrofitted structures. In situ testing of masonry materials was not performed after the earthquake, and material properties were assumed based on the original design specifications. As a result, the effect of non-linear material behavior on seismic response in the post-cracking stage of URM walls was not considered. It should be noted that the effect of non-linear seismic response of cracked URM walls for the case study building was considered in line with the EC8-3 provisions for masonry buildings (by reducing the wall stiffness).

Equivalent static seismic analysis according to the PTN-S code was performed using the following parameters: seismic intensity coefficient K_s of 0.05 (seismic intensity zone VIII according to the map shown in Figure 3a), building category coefficient K_o of 1.0

corresponding to Category I, dynamic response coefficient K_d of 1.0, and the ductility and damping coefficient K_p of 2.0 (corresponding to URM building). The soil was classified as Category II according to the PTN-S code. It should be noted that seismic hazard parameters for Kraljevo were revised after the earthquake; hence, the building site is currently located in seismic intensity zone IX.

Multi-modal seismic analysis was performed for both the original and retrofitted structure according to EC8-1. The design ground acceleration for soil type A was 0.2 g, while ground type B was considered for the site. Spectral accelerations for the elastic design spectrum $S_d(T)$ according to Eurocode 8 were divided by the behavior factor q of 1.5 for URM structures designed without seismic provisions (for original structure) and q = 2.5 (for retrofitted structure). Type 1 spectrum was deemed appropriate given the seismic hazard setting for the building site. Design response spectra for Kraljevo, Serbia, based on the PTN-S code and Eurocode 8 are presented in Figure 18.

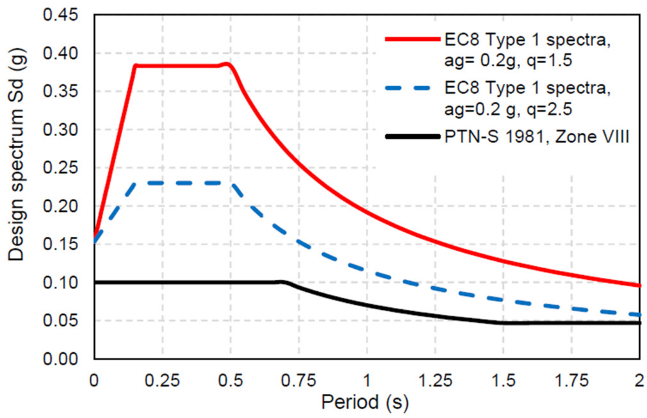
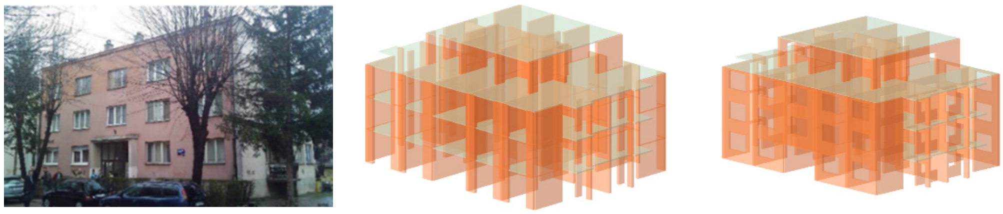


Figure 18. Design response spectra for Kraljevo, Serbia, according to Eurocode 8 (a_g = 0.2 g, ground type B, q = 1.5 and q = 2.5) and PTN-S code (seismic intensity VIII, soil Category II—valid at the time of the 2010 earthquake).

Due to time constraints and limited resources, it was not possible to perform detailed material testing in order to determine the mechanical properties of masonry. Hence, material properties were assumed to be equal to the values specified at the time of the original design. Similar values were also obtained by testing sample bricks extracted from buildings in Kraljevo after the earthquake. Masonry walls were constructed using solid clay bricks, with the estimated characteristic compressive strength of 2.5 MPa. Cement:lime:sand mortar with the mix proportions 1:3:9 was used, and its characteristic compressive strength was estimated at 2.0 MPa. Based on these characteristics, a masonry compressive strength of 2.41 MPa was obtained based on the PTN-Z code provisions, and the modulus of elasticity for masonry was taken as 2410.0 MPa. Since the masonry walls in the extended building portion (top floor) were constructed using modular clay blocks, their characteristic compressive strength was estimated at 10 MPa. The verification of load-bearing capacity for masonry structural components for the seismic retrofitting purposes was performed according to the PTN-Z code [25]. Eurocode 6 [26,70] requirements were not followed in the retrofit project; however, relevant provisions were used in this study.

RC jacketing was performed using low-strength concrete overlay (M20 grade with 20 MPa characteristic compressive strength based on the cube specimens), while GA240/360 steel grade (yield strength 240 MPa) and MAG500/560 steel grade (yield strength 500 MPa) were used for wall anchors and welded wire mesh, respectively.

A 3D numerical model of the building was created in the Tower finite element software package, which was developed in Serbia and has been widely used by academics and practicing engineers [71]. The walls were modelled as shell elements, while the slabs were modelled as plate elements. Floor and roof structures were modelled as rigid diaphragms. The foundations were simulated as fixed-base restraints. Two numerical models were developed for the original structure: (a) a model taking into account wall piers (no parapets) and (b) a model with parapets and spandrels, accounting for the stiffness of horizontal elements below and above the openings (Figure 19). The first model, in which wall piers are the main vertical elements of the lateral load-resisting system, may also be referred to as the "Cantilever model". Since the model ignores the effect of spandrels, the effect of the wall–slab interaction may not be accurately simulated in the seismic analysis. On the other hand, the second model is similar to the "Equivalent Frame Model" (EFM), which has also been used to model masonry structures for seismic analysis purposes [72]. A discussion on the pros and cons of both models is provided elsewhere [73].



(**a**) (**b**) (**c**)

Figure 19. 3D numerical models: (**a**) actual building; (**b**) Cantilever model, and (**c**) Equivalent Frame Model (EFM).

The following three models were considered to account for the effect of cracking on the original and retrofitted structure: (a) Model 1, which considered uncracked (gross) properties of the original structure (referred to as Original 1 and Retrofitted 1); (b) Model 2, which considered the effect of cracking (20% stiffness reduction), which is referred to as Original 2 and Retrofitted 2, and (c) Model 3, which considers a 50% stiffness reduction (Original 3 and Retrofitted 3). Note that the PTN-S code did not include a provision related to the stiffness reduction (or an alternative provision to account for the effect of cracking in masonry and RC structures); hence, Model 1 is in line with the PTN-S requirements. On the other hand, Model 3 reflects the EC8-3 requirements, which prescribe a 50% stiffness reduction. Finally, Model 2 (20% stiffness reduction) reflects a situation wherein URM walls have experienced moderate cracking, which was true for many buildings affected by the 2010 Kraljevo earthquake.

Dynamic properties of various numerical models were obtained from the modal analysis. It should be noted that the total seismic weight calculated according to the PTN-S was 14,032 kN. Table 2 presents fundamental periods for the original structure in both directions (N–S and E–W). A comparison of the two models (Cantilever versus EFM) can be made based on the fundamental period values. It can be seen from the table that fundamental periods are consistently higher for the Cantilever model compared to the EFM model, and the difference is on the order of 15 to 20%.

Table 2. Fundamental periods for the original structure.

	Original 1		**Original 2**		**Original 3**	
	Cantilever Model	**EFM**	**Cantilever Model**	**EFM**	**Cantilever Model**	**EFM**
	(1)	(2)	(3)	(4)	(5)	(6)
N–S direction	0.304	0.240	0.332	0.266	0.401	0.335
E–W direction	0.288	0.237	0.316	0.264	0.381	0.332

The modelling of RC jacketing is an important aspect of the project. According to the PTN-R code, a retrofitted masonry wall with an RC jacket is modelled as an equivalent masonry wall with the thickness equal to the thickness of the original wall plus additional thickness (equal to four times the thickness of an RC jacket), as discussed in Section 2. This concept is illustrated in Figure 20. Figure 20a shows a horizontal section of an actual 25 cm thick masonry wall retrofitted by a 5 cm thick RC jacket, while Figure 20b shows a model recommended by PTN-R, where a 5 cm thick RC jacket is represented by an equivalent 20 cm thick masonry wall. As a result, a 45 cm thick masonry wall is used for seismic analysis purposes. Unfortunately, the PTN-R code did not have a commentary; hence, the basis for this provision is not provided as a part of the code. Based on the fundamental mechanic principles for a composite masonry and RC section such as the one shown in Figure 20a, it can be assumed that the stiffness is determined as a sum of the stiffnesses for the masonry and RC components, which are characterized by different mechanical properties (modulus of elasticity E and modulus of rigidity G), as well as different thicknesses. The stiffness of the composite section can be assumed to have the mechanical properties of masonry. The equivalent thickness for the composite section can be determined by estimating a ratio of E_c/E_m, corresponding to the moduli of elasticity for concrete E_c and masonry E_m. The equivalent thickness according to the PTN-R code can be obtained when the E_c/E_m ratio is approximately equal to 4.0, which is a reasonable assumption when concrete with low-to-moderate characteristic compressive strength is used for RC jackets, as is the case with the retrofit solution for the case study building. According to the EC8-3 code, the designer is expected to simulate the effect of an RC jacket by modelling it as a separate shell layer, or a part of a composite equivalent column section, where masonry and concrete materials would be simulated using appropriate mechanical and geometric properties (see Figure 20c).

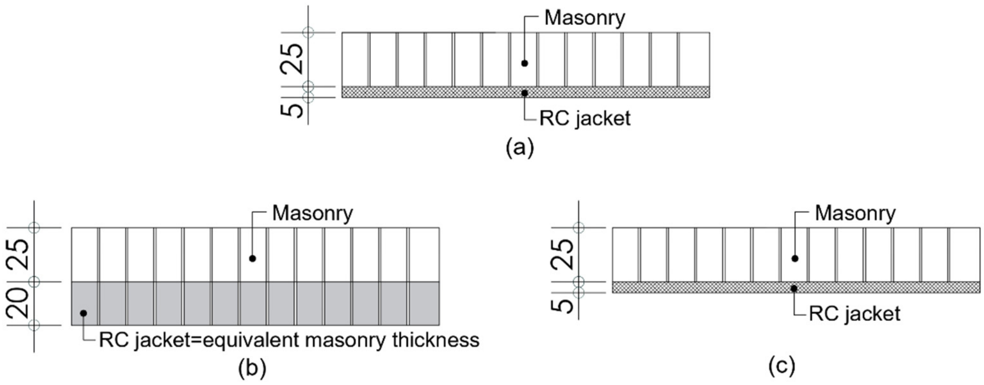


Figure 20. Numerical models for simulating RC jacketing: (**a**) actual wall section; (**b**) equivalent masonry section according to PTN-S, and (**c**) numerical model according to EC8-3.

Numerical models for the original and retrofitted structure are shown on Figure 21. It is worth noting that the vertical RC jackets are embellished in a darker color (Figure 21b).

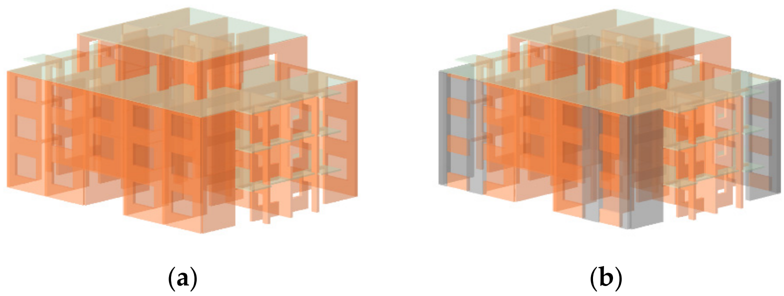


Figure 21. Numerical models: (**a**) original structure and (**b**) retrofitted structure.

Fundamental periods for the retrofitted structure, which were obtained using three different numerical models which take into account the extent of cracking and two different models for simulating masonry wall characteristics (Cantilever and EFM), are summarized in Table 3. It can be seen from the table that the periods for the Cantilever model are higher than for the EFM model; a similar phenomenon was observed for the original structure (Table 2). It can be noticed that the difference ranges from 18 to 23% for the N–S direction, while the difference for the E–W direction is smaller (on the order of 10%); this can be explained by a larger number of openings (windows) in the N–S direction (see Figure 14), which can be better simulated by the EFM model. It should be noted that the periods for both models are within the "plateau" range of the EC8-1 design spectra.

Table 3. Fundamental periods for the retrofitted structure.

	Retrofitted 1		**Retrofitted 2**		**Retrofitted 3**	
	Cantilever Model	**EFM**	**Cantilever Model**	**EFM**	**Cantilever Model**	**EFM**
	(1)	(2)	(3)	(4)	(5)	(6)
N–S direction	0.299 s	0.231 s	0.327 s	0.258 s	0.396 s	0.326 s
E–W direction	0.253 s	0.227 s	0.281 s	0.253 s	0.352 s	0.319 s

The effect of retrofitting and the extent of cracking on fundamental periods (based on the results presented in Tables 2 and 3) is illustrated in Figure 22. It can be seen from the chart that fundamental period values are consistently smaller for the retrofitted structure (for all models), but the difference is insignificant (less than 2%); this indicates a minor effect of RC jacketing on the stiffness increase, which can be attributed to one-sided jacketing applied at critical exterior locations in the building. An important observation is related to the effect of extent of cracking on the fundamental period values. It can be seen from the chart that the fundamental period of 0.401 s for Model 3 (50% stiffness reduction in line with the EC8-3 requirements) is by approximately 30% higher than the corresponding period of 0.304 s for Model 1 (no stiffness reduction—in line with the PTN-S requirements).

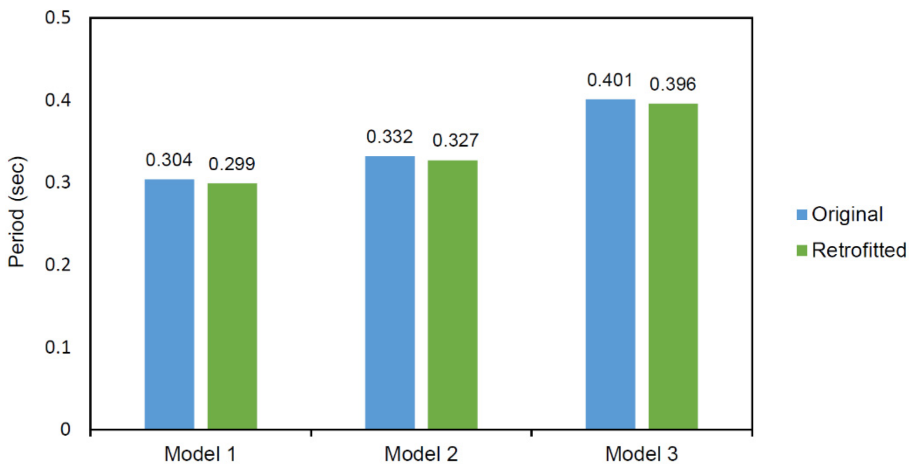


Figure 22. The effect of cracking on the fundamental periods in the N–S direction for the original and retrofitted structure (Cantilever model).

4.5. Results and Discussion

The results of the seismic analysis enabled a comparison between the seismic demand and capacity for individual walls, as well as the entire structure, according to the PTN-S code and Eurocode 8 (Parts 1 and 3). Note that majority of the results presented in this section were obtained using the Cantilever model, due to higher seismic forces and a more conservative design in this case.

The seismic base shear forces for the original building, according to the PTN-S code and Eurocode 8, Part 1, were previously reported by the authors [1]. It was observed that seismic forces corresponding to the PTN-S code were significantly smaller than the corresponding values obtained according to the Eurocode 8 requirements; this can also be seen from the design response spectra presented in Figure 18. A difference in the magnitude of seismic forces for the case study building according to the PTN-S and EC8-1 codes is illustrated in Figure 23. At the time of the 2010 earthquake, Kraljevo was located in seismic intensity zone VIII, but seismic hazard zonation has been subsequently revised to seismic intensity zone IX, thereby resulting in higher seismic forces. On the other hand, EC8-1 seismic forces were determined based on the latest seismic hazard map for Serbia shown in Figure 3b. A significant difference, both in magnitude and distribution of seismic force up the building height, can be seen from the chart. The difference is particularly notable at the third-story level (elevation 8.4 m), at which the applied seismic forces are 442.2 kN, 844.4 kN, and 1668.4 kN for PTN-S (intensity zones VIII and IX) and EC8-1, respectively. It can be concluded that the applied force according to EC8-1 (1668.4 kN) is almost 3.8 times higher than the corresponding force (442.2 kN) used for seismic evaluation and retrofitting of the building after the earthquake (PTN-S, zone VIII).

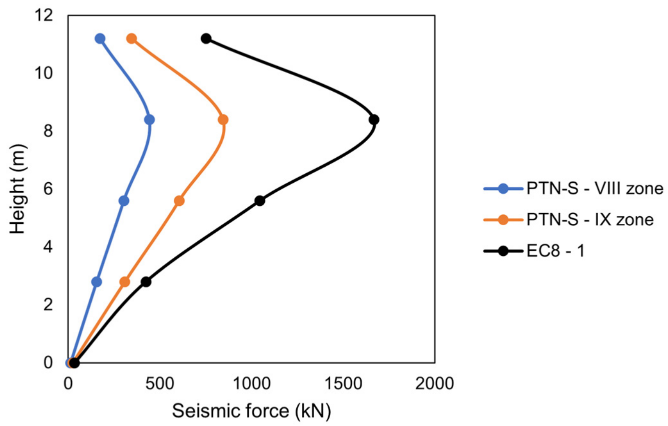


Figure 23. Applied seismic forces for the case study building at story levels determined according to the PTN-S code (seismic intensity VIII and IX) and EC8-1 (a_g = 0.2 g, ground type B, q = 1.5, and q = 2.5).

To illustrate the effectiveness of retrofitting, seismic base shear force V_{Ed} (kN) (seismic demand) was compared with the shear capacity at the ground floor level V_{Rd} (kN), which was taken as equal to the sum of capacities for all walls aligned in the same direction (N–S or E–W).

The results for the longitudinal (N–S) direction are summarized in Tables 4 and 5 and illustrated in Figure 24. It can be seen from the chart that the capacity of the building was satisfactory according to the PTN-S code, since the capacity (C) versus demand (D) ratio, C/D, is larger than 1.0 both for the Original 1 (uncracked) model which is in line with the PTN-S code and the Original 2 model (cracked, 20% stiffness reduction), but it is not satisfactory for the Original 3 model (cracked, 50% stiffness reduction—in line with EC8-3), as shown in Figure 25.

Table 4. Comparison of seismic capacity (C) and demand (D) at the ground floor level of the building in N–S direction (for the original structure).

	Original 1		**Original 2**		**Original 3**	
	PTN-S	**EC8**	**PTN-S**	**EC8**	**PTN-S**	**EC8**
	(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**
C: Shear Capacity $V_{Rd}(kN)$	1231.57	2052.61	985.25	1642.09	615.78	1026.30
D: Design Shear Force $V_{Ed}(kN)$	930.59	2301.70	930.59	2301.70	930.59	2301.70
$\frac{C}{D} = V_{Rd}/V_{Ed}$	1.32	0.89	1.06	0.71	0.66	0.44

Table 5. Comparison of seismic capacity (C) and demand (D) at the ground floor level of the building in N–S direction (for the retrofitted structure).

	Retrofitted 1		**Retrofitted 2**		**Retrofitted 3**	
	PTN-S	**EC8**	**PTN-S**	**EC8**	**PTN-S**	**EC8**
	(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**
C: Shear Capacity $V_{Rd}(kN)$	1477.88	2463.13	1180.08	1970.51	738.94	1231.56
D: Design Shear Force $V_{Ed}(kN)$	969.88	2398.88	969.88	2398.88	969.88	2398.88
$\frac{C}{D} = V_{Rd}/V_{Ed}$	1.52	1.03	1.22	0.82	0.76	0.51

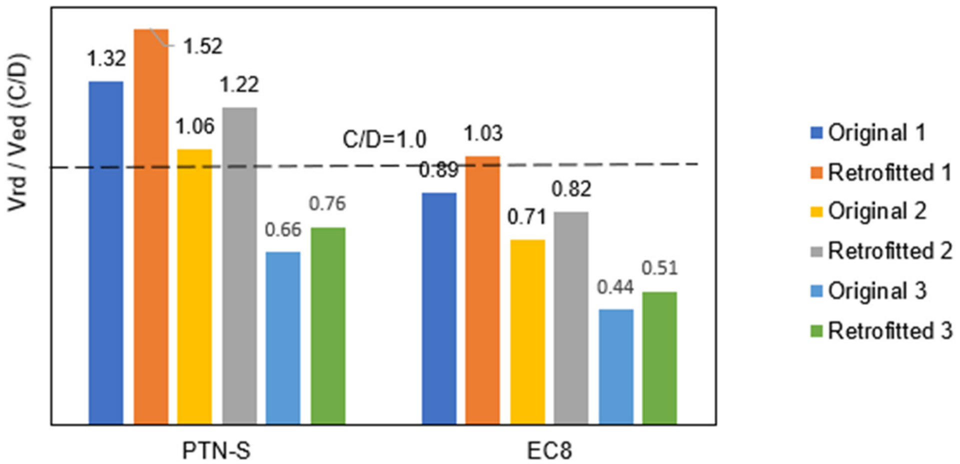


Figure 24. Seismic capacity versus demand (C/D) ratio for the ground floor of the building in N–S direction (for the original and retrofitted building).

The results also indicate that, according to the PTN-S code, the retrofit has resulted in an increased C/D ratio for the building to 1.52, 1.22, and 0.76 for Models 1, 2, and 3, respectively. The results shown in Table 5 indicate that the Retrofitted 3 model (which considers a 50% stiffness reduction) is not satisfactory, since the corresponding C/D value is less than 1.0. The results of the analysis performed according to the EC8-1 requirements have shown that the capacity of the structure is not satisfactory even after the retrofit for Models 2 and 3, since the corresponding C/D values are less than 1.0; however, the retrofit seems to be effective for Model 1, since the corresponding C/D ratio is 1.03 (in line with the PTN-S code).

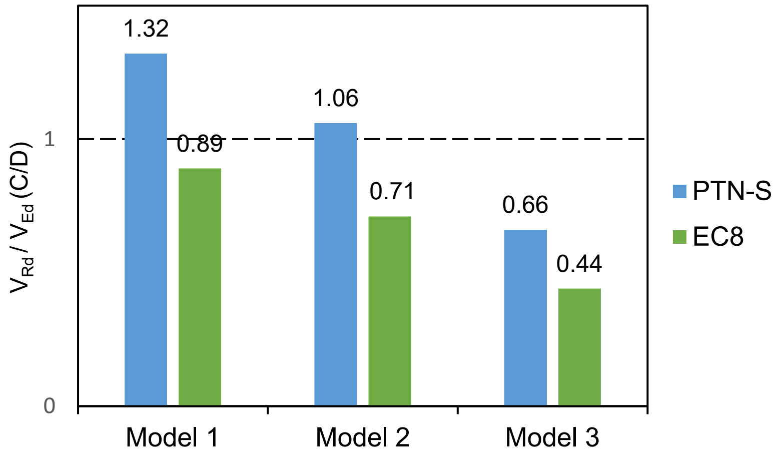


Figure 25. Seismic capacity versus demand (C/D) ratio for the ground floor of the building in N–S direction (for the original building).

A similar comparison is presented for the transverse (E–W) direction of the building, as shown in Tables 6 and 7 and Figure 26. The results (Table 6) have shown that the original building has satisfactory capacity (C/D ratio greater than 1.0), except for the Original 3 model, which is in line with the EC8-3 requirements (50% stiffness reduction); this is also illustrated in Figure 27. The results (Table 7) indicate that, according to the PTN-S code, the retrofit is effective in enhancing the seismic safety, expressed in terms of the C/D ratio (Retrofitted 1 model), but it was not deemed effective according to EC8-3 code requirements (Retrofitted 3 model) since the corresponding C/D ratio is 0.94.

Table 6. Comparison of seismic capacity (C) and demand (D) at the ground floor level of the building in E–W direction (for the original structure).

	Original 1		**Original 2**		**Original 3**	
	PTN-S	**EC8**	**PTN-S**	**EC8**	**PTN-S**	**EC8**
	(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**
C: Shear Capacity $V_{Rd}(kN)$	2088.35	3480.58	1670.68	2784.46	1044.18	1740.29
D: Design Shear Force $V_{Ed}(kN)$	930.59	2127.16	930.59	2127.16	930.59	2127.16
$\frac{C}{D} = V_{Rd}/V_{Ed}$	2.25	1.64	1.79	1.31	1.12	0.82

Table 7. Comparison of seismic capacity (C) and demand (D) at the ground floor level of the building in E–W direction (for the retrofitted structure).

	Retrofitted 1		**Retrofitted 2**		**Retrofitted 3**	
	PTN-S	**EC8**	**PTN-S**	**EC8**	**PTN-S**	**EC8**
	(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**
C: Shear Capacity $V_{Rd}(kN)$	2506.02	4176.69	2001.04	3341.36	1253.01	2088.34
D: Design Shear Force $V_{Ed}(kN)$	969.88	2216.97	969.88	2216.97	969.88	2216.97
$\frac{C}{D} = V_{Rd}/V_{Ed}$	2.58	1.88	2.06	1.51	1.29	0.94

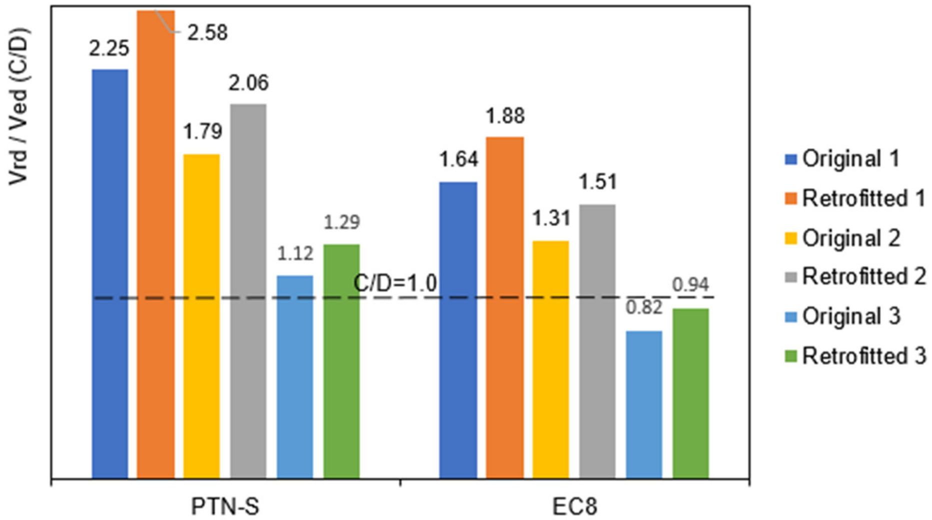


Figure 26. Seismic capacity versus demand (C/D) ratio for the ground floor of the building in E–W direction (for the original and retrofitted building).

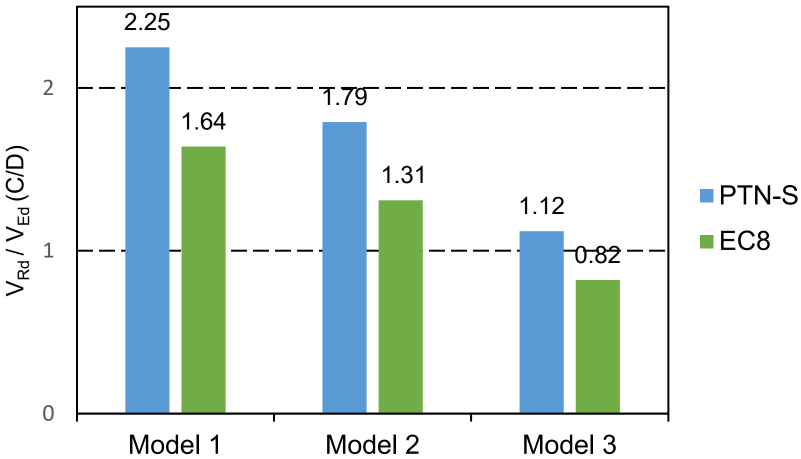


Figure 27. Seismic capacity versus demand (C/D) ratio for the ground floor of the building in E–W direction (for the original building).

The C/D ratio values for longitudinal (N–S) direction are generally lower than for the transverse (E–W) direction; this is in line with the findings of the previous study related to the same building [1], which was focused on examining the wall index (WI) as an indicator of seismic safety, based on the number of walls in each horizontal direction of a masonry building. The building under consideration is characterized by a significantly higher WI per floor ratio value for the transverse (E–W) direction (1.76%) compared to the longitudinal (N–S) direction (1.03%); this indicates that the lateral load-resisting capacity of the building for the N–S direction may not be adequate (but needs to be verified through design calculations).

The effect of retrofitting was also studied on an example of an interior wall in the transverse (E-W) direction (embellished in red color in Figure 28). Note that the wall was not retrofitted since only exterior walls were retrofitted at critical exterior locations (shown in blue color on the floor plan). The wall was located at the ground floor level of the building and was subjected to seismic shear demand V_{Ed} (kN), which was compared with the corresponding shear capacity V_{Rd} (kN). The procedure for determining the shear capacity for URM walls according to the PTN-S code and Eurocode 8 was explained in [1]. It should be noted that the masonry design code from former Yugoslavia, PTN-Z [25], as well as Eurocode 6 [26,70] were used for the wall capacity calculations.

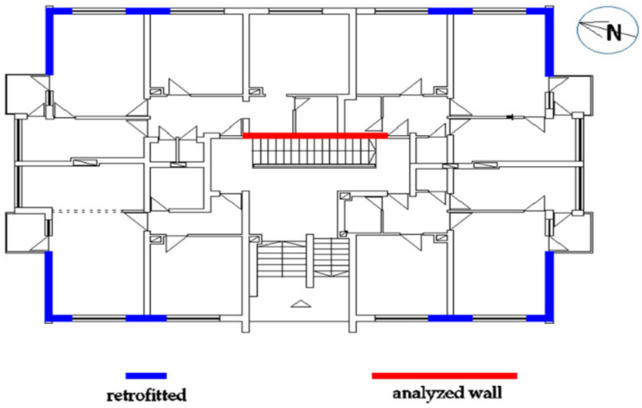


Figure 28. The floor plan showing retrofitted exterior walls (blue color) and the analyzed wall (red color).

The results are summarized in Tables 8 and 9 and illustrated in Figure 29. It can be seen from the chart that before retrofitting the capacity of the wall was inadequate since the corresponding C/D ratios were less than 1.0, according to both the PTN-S code as well as the EC8-1 code for all models. The analysis has shown that the retrofit has resulted in an increased C/D ratio for the wall according to the PTN-S code to 1.24, but only for the Retrofitted 1 model (uncracked), which is in line with that code. However, when the effect of cracking was considered, the corresponding C/D ratio was less than 1.0.

Table 8. Comparison of the seismic capacity (C) and demand (D) for the analyzed wall at the ground floor level (for the original structure).

	Original 1		**Original 2**		**Original 3**	
	PTN-S	**EC8**	**PTN-S**	**EC8**	**PTN-S**	**EC8**
	(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**
C: Shear Capacity $V_{Rd}(kN)$	177.00	295.00	141.6	236.00	88.50	147.50
D: Design Shear Force $V_{Ed}(kN)$	181.00	431.00	181.00	431.00	181.00	431.00
$\frac{C}{D} = V_{Rd}/V_{Ed}$	0.98	0.68	0.78	0.55	0.49	0.34

Table 9. Comparison of the seismic capacity (C) and demand (D) for the analyzed wall at the ground floor level (for the retrofitted structure).

	Retrofitted 1		**Retrofitted 2**		**Retrofitted 3**	
	PTN-S	**EC8**	**PTN-S**	**EC8**	**PTN-S**	**EC8**
	(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**
C: Shear Capacity $V_{Rd}(kN)$	212.40	354.00	169.6	283.20	106.20	177.54
D: Design Shear Force $V_{Ed}(kN)$	171.75	408.97	189.20	450.54	189.64	455.23
$\frac{C}{D} = V_{Rd}/V_{Ed}$	1.24	0.86	0.90	0.63	0.56	0.39

Based on the example of this wall and other similar walls in the building, it can be concluded that the presented seismic retrofit solution, which was developed based on the PTN-S and PTN-R codes from former Yugoslavia, does not meet the EC8-3 requirements for retrofitted masonry buildings.

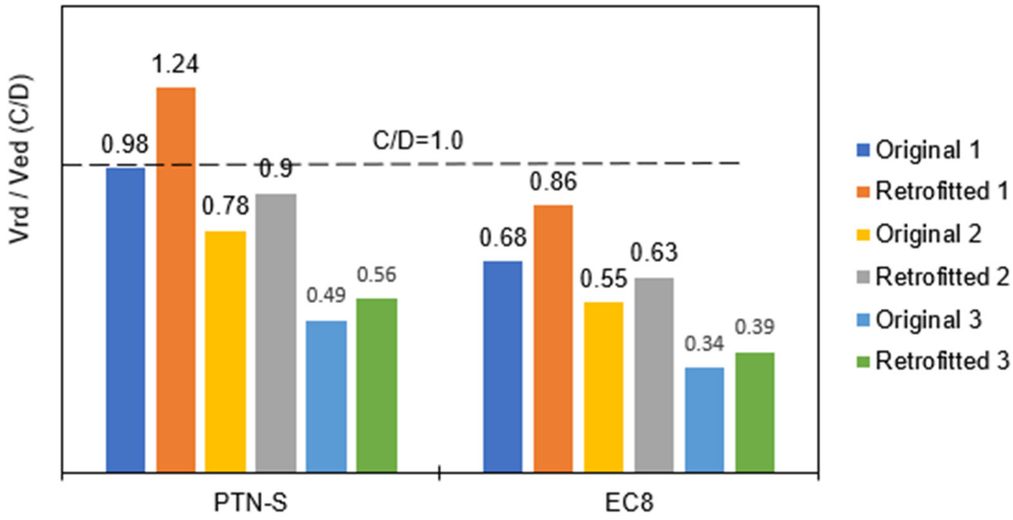


Figure 29. Seismic capacity versus demand (C/D) ratio in the analyzed wall for the original and retrofitted building.

Maximum lateral displacements and total drift ratio were determined for the original and retrofitted structure (all models) and presented in Tables 10 and 11. The results include elastic displacements Δ_e, design displacements Δ_d, and total drift ratio d, which is a ratio of the maximum design displacement Δ_d and the building height (11.2 m). It should be noted that the design displacements Δ_d were obtained by multiplying elastic displacement values by the behavior factor. In the case of EC8 calculations, the behavior factor (q) of 1.5 was used for the original structure (Table 10), while q = 2.5 was used for the retrofitted structure (Table 11). It can be seen from the tables that the displacement values are significantly higher for the analyses performed according to the EC8 code in comparison with the results according to the PTN-S code. For the Cantilever model, which was used in majority of the analyses presented in this section, drift ratio (d) ranges from 0.048% to 0.08% for the original structure and from 0.0079 to 0.130% for the retrofitted structure. It can be seen that drift values for the retrofitted structure are higher than for the original structure; this may not seem logical since the application of RC jackets results in the increased stiffness for the masonry structure. However, this result is not surprising when elastic displacements (Δ_e) are considered—displacements for the PTN-S model are similar for the original and retrofitted structure. For example, the Δ_e value for the Original 1 model is 3.59 mm (Table 10), while the corresponding displacement for the Retrofitted 1 model is 3.54 mm (Table 11); therefore, an increase in the drift value for retrofitted structure can be attributed to a higher q value. In the case of the results corresponding to analyses performed according to EC8, values of drift ratio are higher, ranging from 0.186 to 0.310% for the original structure (drift values for the retrofitted structure are almost the same). Elastic displacements (Δ_e) for the retrofitted structure are lesser compared to the original structure (as expected). For example, the Δ_e value for the Original 3 model is 23.17 mm (Table 10), while the corresponding displacement for the Retrofitted 3 model is 9.268 mm (Table 11); however, the drift ratios are based on design displacements (Δ_d) and reflect different q values used in these two cases (1.5 and 2.5 for the original and retrofitted structure, respectively). Overall, all lateral displacements and drift values are very low, as expected for a low- to mid-rise URM building.

4.6. Limitations of the Study: Seismic Analysis Procedure

It may be considered that a limitation of the study is associated with the use of a linear elastic seismic analysis procedure—as opposed to modern nonlinear analysis approaches. Note that application of linear elastic seismic analysis was justified at the time when retrofitting of the earthquake-damaged buildings in Kraljevo was performed (2010). Linear elastic seismic analysis was the default procedure for ordinary buildings, such as residential buildings, as per the Serbian technical regulations, including the PTN-S code for seismic design of new structures and the PTN-R code for seismic retrofitting of existing structures. It should be noted that linear elastic seismic analysis is an acceptable approach by the current European seismic code for existing buildings (EC8-3).

Table 10. Lateral displacements and total drift ratio for the original structure in N–S direction.

		Original 1		**Original 2**		**Original 3**	
		Cantilever Model	**EFM**	**Cantilever Model**	**EFM**	**Cantilever Model**	**EFM**
		(1)	(2)	(3)	(4)	(5)	(6)
PTN-S	Δ_e (mm)	3.590	2.270	4.240	2.690	6.010	3.900
	Δ_d (mm)	5.385	3.405	6.36	4.035	9.015	5.850
	d (%)	(0.048)	(0.030)	(0.056)	(0.036)	(0.080)	(0.052)
EC8	Δ_e (mm)	13.94	7.440	16.440	6.650	23.17	2.09
	Δ_d (mm)	20.91	11.16	24.66	9.975	34.755	3.135
	d (%)	(0.186)	(0.099)	(0.220)	(0.089)	(0.310)	(0.138)

Table 11. Lateral displacements and total drift ratio for the retrofitted structure in N–S direction.

		Retrofitted 1		**Retrofitted 2**		**Retrofitted 3**	
		Cantilever Model	**EFM**	**Cantilever Model**	**EFM**	**Cantilever Model**	**EFM**
		(1)	(2)	(3)	(4)	(5)	(6)
PTN-S	Δ_e (mm)	3.540	2.020	4.190	2.390	5.930	3.440
	Δ_d (mm)	8.850	5.05	10.475	5.975	14.825	8.600
	d (%)	(0.079)	(0.045)	(0.093)	(0.053)	(0.132)	(0.076)
EC8	Δ_e (mm)	8.330	1.090	9.840	1.290	9.268	1.050
	Δ_d (mm)	20.825	2.725	24.600	3.225	34.675	4.625
	d (%)	(0.186)	(0.024)	(0.219)	(0.029)	(0.309)	(0.041)

The nonlinear seismic response of masonry structures has been extensively studied in recent decades, and the corresponding numerical modelling and analysis approaches have been developed and incorporated into international seismic design codes. Non-linear static (pushover) analysis (NSA) is an internationally accepted analysis approach for simulating the nonlinear seismic response of building structures. NSA has been recommended for the seismic evaluation of existing buildings in the USA since the 1990s [74], and it has been incorporated into the current code for seismic evaluation of existing buildings ASCE/SEI 41-17 [27]. One of the main results of the NSA is a lateral force versus displacement curve (also known as pushover curve), which characterizes the response of a structure subjected to incrementally increasing monotonic lateral loading. The NSA results enable a designer to compare the lateral displacement/drift capacity for a structure under consideration and the corresponding displacement demand, which depends on the seismic hazard level and dynamic characteristics of the structural model. The displacement capacity can be determined for different performance limit states, which are established by the design codes, e.g., damage limitation and no collapse, according to EC8-1. NSA is one of the key components of performance-based earthquake engineering, and it can be used to help predict structural and non-structural damage and losses for buildings at different earthquake hazard levels.

Nonlinear analysis of masonry structures has been particularly challenging due to the composite nature of masonry structures, consisting of masonry elements, mortar, and reinforcement (in some cases). One of the early numerical models for nonlinear static analysis of masonry buildings was based on a simplified "story model", which considered the non-linear force-displacement behavior of individual walls idealized through a bilinear elastic–plastic model, and was developed in the 1970s by Tomaževič [75]. The non-linear modelling of URM structures at the micro level was studied by Lourenço [76], while the macro-modelling approach was extensively studied by several researchers due to its suitability for practical design applications. One of the most popular macro-models for simulating nonlinear seismic response of masonry structures is the equivalent frame model (EFM), proposed by Magenes [77], and further developed and implemented in the form of a TREMURI computer analysis program [72,78]. According to the EFM model, a masonry wall with openings is idealized as a moment frame consisting of the rigidly connected pier and spandrel elements. The TREMURI program enables the user to simulate the non-linear response of both piers and spandrels in a masonry wall subjected to in-plane seismic loading by means of a non-linear macro-element model [79]. The non-linear seismic response of URM walls obtained using the EFM model has been validated using the results of experimental studies [73]. The EFM approach for the non-linear analysis of masonry buildings is in line with the code provisions in the USA [27] and Europe (EC8-3) [21].

NSA of a typical six-story residential URM building from Sarajevo, Bosnia and Herzegovina was performed by Ademović et al. [9]. The building is representative of the typology which was the subject in this study. The NSA results were obtained for a micro-model (FEM) developed using the DIANA software package, while a macro-model was developed using the TREMURI software. Both models took into account material non-linearity characteristic for masonry walls. The results of the analysis (pushover curve for the transverse direction) show that the structure demonstrated a linear elastic behavior until the inter-story drift of approximately 0.07%. Under a further deformation increase, the structure experienced non-linear behavior until the failure took place. It should be noted that the case study building from Sarajevo is similar to the building considered in the current study, except for a very few walls provided in the longitudinal direction.

The application of NSA would be possible in the context of the current study; however, it may be argued whether the results of such an analysis would be useful in the context of the seismic retrofit project of the case study building. Since the case study building experienced damage in the 2010 Kraljevo earthquake, it can be expected that its seismic response to ground shaking was non-linear. Seismic evaluation of the building was performed according to the local codes, based on the given seismic hazard parameters, and the results indicated that the building was safe for seismic demand determined based on the PTN-S code. These results evidently did not reflect the actual seismic performance of the building; however, acceleration records for the Kraljevo earthquake were not available. As a result, seismic hazard parameters corresponding to the earthquake were not known, and it would be difficult to determine realistic displacement demand which is required for NSA. On the other hand, precise values for the mechanical properties of masonry materials were not available due to the lack of in situ material testing after the earthquake. Consequently, it was not possible to determine strength and deformation characteristics for nonlinear characterization (backbone curve) of masonry piers and spandrels, which are required as input for an NSA. It can be concluded that the results of an NSA would neither contribute toward an improved understanding of non-linear seismic behavior of the case study building nor be useful for optimizing the seismic retrofit solutions.

It is unfortunate that more precise data related to the material characteristics and seismic hazard for URM buildings affected by the 2010 earthquake were not available; however, limited information related to material characteristics and seismic hazard parameters is a reality in many post-earthquake situations. For that reason, the authors believe that simplified seismic analysis and numerical modelling approaches that account for material non-linearity characteristics will continue to be used in practice and are relevant for the engineering community.

5. Conclusions

This paper presents a study on seismic retrofitting of URM mid-rise residential buildings damaged due to the 2010 Kraljevo earthquake. Seismic evaluation and retrofitting of the buildings were performed according to the codes from the former Yugoslavia, which were enforced in Serbia until 2019, such as the seismic design code PTN-S and the code for seismic retrofitting of buildings PTN-R. A comparison of the results for the application of pertinent Yugoslav codes and Eurocode 8 was presented for a case study building in Kraljevo. The following relevant conclusions have been drawn based on this study:

(1) The case study building is a mid-rise URM building typical of residential construction in former Yugoslavia after WWII. Buildings of this type are vulnerable to earthquake effects and were exposed to a few damaging earthquakes, which caused moderate-to-severe structural damage in these buildings.

(2) The retrofit solution which was applied in the case study building consisted of RC jackets applied to exterior walls. The retrofit solution is acceptable by Eurocode 8, Part 3.

(3) The results of the seismic analysis and the design checks for the retrofitted structure, which were performed according to the pertinent Yugoslav codes, show that all walls in the retrofitted structure were satisfactory in terms of seismic safety.

(4) A comparison of the results of seismic analysis for the case study building performed according to the Yugoslav codes and Eurocode 8 has shown that the seismic demand according to Eurocode 8 is significantly higher compared to the seismic codes from former Yugoslavia, which were used for seismic evaluation and retrofitting design. The key finding is that the retrofit design performed according to the Yugoslav code does not meet the seismic safety requirements of Eurocode 8. It would be required to perform more extensive retrofit, likely for the interior walls (in addition to the exterior walls), in order to satisfy the seismic demand requirements according to Eurocode 8.

(5) There is a difference in stiffness estimation for the retrofitted walls for the numerical model developed according to the Yugoslav code PTN-R and Eurocode 8, Part 3. According to the PTN-R code, the thickness of a retrofitted wall needs to be increased to account for RC jackets by considering an equivalent masonry section with the RC jacket thickness increased four times. On the other hand, Eurocode 8, Part 3 prescribes a reduction in the wall stiffness to account for the effect of cracking.

The results of this study are relevant for Serbia, and other countries in the region which followed seismic design and retrofit codes developed in the former Yugoslavia in the past and are currently using Eurocode 8 as the main code for seismic design and retrofit of masonry structures.

The authors believe that this paper contributes to the knowledge base related to field applications of cost-effective and practical seismic retrofitting techniques for URM buildings which are suitable for application in countries with limited human and financial resources, compounded by limited experience related to seismic retrofitting.

6. Recommendations for Future Research Studies

The findings of studies related to seismic evaluation and retrofitting of earthquake-damaged buildings, such as the one presented in this paper, are useful for seismic risk and resilience studies on local and/or regional levels. The authors believe that future research studies related to the topic of this study should be integrated with seismic risk studies, which will provide useful information for predicting losses due to future earthquakes and enhancing the resilience of communities at risk. Unfortunately, seismic risk studies in Serbia are practically non-existent; however, an ongoing initiative to undertake a seismic risk study for Serbia is underway by members of the Serbian Association for Earthquake Engineering (SUZI-SAEE). Notable past initiatives at the regional level (which included Serbia) resulted in the development of the European seismic risk model in the framework of the SERA project [80]. A World Bank's study on the seismic risk associated with urban residential buildings in Eastern Europe and Central Asia [81] provided useful information related to Serbia's capital Belgrade.

There has been a limited effort to determine empirical fragility functions based on building damage data after past earthquakes in Serbia. Notable empirical seismic risk studies were performed on masonry buildings after the damaging earthquakes in Italy [82–84] and a similar methodology could be applied in Serbia and the neighboring countries. A unique study of this kind was performed after the 2010 Kraljevo earthquake on a sample of 1.193 damaged masonry buildings, and it resulted in the development of empirical fragility functions which will be useful for future seismic risk studies in Serbia and the region [66,85]. Expert-based fragility functions were recently developed for masonry buildings in Serbia [12].

Seismic risk studies on portfolios of existing urban masonry buildings have been performed in a few countries, including Italy [86]. A methodology for rapid seismic assessment of urban masonry and RC buildings in the countries surrounding the Adriatic and the Ionian Sea has been recently developed within framework of the Interreg's Adriseismic project [87]. The Serbian team participating in the project performed an initial pilot study, in which the proposed methodology was applied on typical masonry and RC buildings in the center of Serbia's capital Belgrade.

One of the important aspects of seismic risk studies is the estimation of costs associated with the recovery after possible future earthquakes. Methodology developed in recent Italian studies [88] can be applied to estimate retrofit cost for masonry buildings after future earthquakes in Serbia.

Author Contributions: Conceptualization, P.B. and S.B.; methodology, P.B. and S.B.; software, P.B; validation, P.B. and R.C.; formal analysis, P.B.; investigation, S.B., P.B. and R.C.; resources, P.B. and S.B.; data curation, R.C.; writing—original draft preparation, P.B. and S.B.; writing—review and editing, S.B. and R.C.; visualization, P.B. and R.C; supervision, P.B.; project administration, R.C.; funding acquisition, P.B. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding.

Data Availability Statement: Data related to this study is archived in the City of Kraljevo administration office, because the 2010 Kraljevo earthquake took place more than 10 years ago.

Acknowledgments: The authors acknowledge the City Administration of the City of Kraljevo for providing information related to the rehabilitation of buildings after the 2010 Kraljevo earthquake. The authors also acknowledge Nikola Blagojević, doctorand at the Swiss Federal Institute of Technology (ETH) at Zurich, Switzerland, for reviewing the manuscript and providing valuable comments.

Conflicts of Interest: The authors declare no conflict of interest.

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