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Simplified Seismic Assessment of Unreinforced Masonry Residential Buildings in the Balkans: The Case of Serbia

Predrag Blagojević [1], Svetlana Brzev [2,*] and Radovan Cvetković [1]

[1] Faculty of Civil Engineering and Architecture, University of Niš, 18000 Niš, Serbia; predrag.blagojevic@gaf.ni.ac.rs (P.B.); radovan@gaf.ni.ac.rs (R.C.)
[2] Department of Civil Engineering, University of British Columbia, Vancouver, BC V6T 1Z4, Canada
* Correspondence: sbrzev@mail.ubc.ca; Tel.: +381-65-922-1959

Abstract: The paper presents a study on the existing low-rise unreinforced masonry (URM) buildings constructed in the period from 1945 to 1980 in Serbia and neighbouring countries in the Balkans. Buildings of this typology experienced damage in a few earthquakes in the region, including the 2010 Kraljevo, Serbia earthquake and the 2020 Petrinja, Croatia earthquake. The focus of the study is a seismic design approach for Simple masonry buildings according to Eurocode 8, Part 1, which is based on the minimum requirements for the total wall area relative to the floor plan area, which is referred to as Wall Index (*WI*) in this paper. Although the intention of Eurocode 8 is to use *WI* for design of new buildings, the authors believe that it could be also used for seismic assessment of existing masonry buildings in pre- and post-earthquake situations. A study on 23 URM buildings damaged in the 2010 Kraljevo, Serbia earthquake has been presented to examine a relationship between the *WI* and the extent of earthquake damage. Seismic evaluation of a typical 3-storey URM building damaged in the 2010 earthquake was performed according to the requirements of seismic design codes from the former Yugoslavia and Eurocode 8.

Keywords: unreinforced masonry buildings; seismic design; earthquake damage; wall density; wall index; Eurocode 8; simple masonry buildings

Citation: Blagojević, P.; Brzev, S.; Cvetković, R. Simplified Seismic Assessment of Unreinforced Masonry Residential Buildings in the Balkans: The Case of Serbia. *Buildings* **2021**, *11*, 392. https://doi.org/10.3390/buildings11090392

Academic Editors: Stjepan Lakušić, Miroslav Nastev, Josip Atalić and Mario Uroš

Received: 19 July 2021
Accepted: 31 August 2021
Published: 3 September 2021

1. Background

Masonry is a traditional construction technology, which has been widely used for housing construction in European countries, including Serbia and other countries located in the Balkans [1]. Ancient masonry structures in the region were traditionally constructed using stone masonry, but since the second half of 19th century residential and public buildings have been constructed using locally manufactured clay bricks and blocks. Although reinforced concrete (RC) has emerged as a technology of choice for construction of mid- and high-rise buildings, masonry has remained prevalent technology for construction of low-rise single-family dwellings and mid-rise, multi-family residential buildings in the region. The Serbian territory is close to high seismic hazard areas in which several damaging earthquakes occurred in the last 60 years, e.g., the 1963 Skopje earthquake (M 6.1), the 1979 Montenegro earthquake (M 6.9), the 1977 Vrancea, Romania earthquake (M 7.2), and most recently the March 2020 Zagreb, Croatia (M_L 5.5) and the December 2020 Petrinja, Croatia earthquakes (M 6.4). Serbia is located in the low-to-moderate seismic hazard area, and in the last 100 years, more than 10 earthquakes with magnitude 5.0 or higher occurred within the Serbian territory. The most significant earthquake in the 20th century occurred in 1922, had a magnitude 6.0 and epicentre near Lazarevac (approximately 60 km aerial distance from the capital Belgrade). Several other earthquakes affected rural areas, e.g., Rudnik, 1927 (M 5.9), Kopaonik, 1980 (M 5.8), and Mionica, 1998 (M_L 5.7). The World Bank has estimated national annualised capital losses in Serbia resulting from earthquakes on the order of USD 40 million [2]. The country's risk profile reports that an infrequent but intense earthquake event (a 250-year return period) could result in nearly USD 1 billion of capital losses in Serbia (about 3% of the country's GDP in 2015–estimated at USD 36.4 billion).

According to the 2011 Census of Serbia [3], low-rise single-family buildings constituted 95% of the national residential building stock, corresponding to 65.9% housing units, while multi-family housing accounted for only 2.6% of the housing stock in terms of the number of buildings; however, the proportion was significantly higher (33%) in terms of the number of housing units [4]. Although detailed material-related building exposure data are not available, it is apparent that a significant fraction of single-family and multi-family housing stock was constructed using masonry technology. The Census reported that 72% of all residential buildings in Serbia were constructed between 1946 and 1990, when Serbia was a part of the Socialist Federal Republic of Yugoslavia (SFRY), also known as "former Yugoslavia". The majority of multi-family residential buildings of pre-1964 vintage, mostly 3- to 5-storey high, were constructed as unreinforced masonry (URM) buildings. After the first national seismic design code was issued in 1964, construction of multi-family masonry housing continued at a smaller scale. The period from 1964 to 1980 was characterised by a construction boom. Although RC technology was widely used for both prefabricated and cast-in-situ housing construction, loadbearing masonry continued to be the prevalent construction technology for low-rise dwellings. In the 1960s, solid clay bricks were slowly replaced by modular clay blocks (also known as hollow clay tiles). Confined masonry was introduced for building construction in high seismic intensity areas. Unreinforced masonry buildings also constituted a significant fraction of the housing stock in neighbouring countries which were a part of the former Yugoslavia.

Seismic vulnerability of URM buildings is well recognised, and has been reported after past earthquakes around the globe. These buildings are heavy, due to relatively thick and massive masonry walls, and are also relatively rigid. As a result, spectral accelerations and the corresponding earthquake-induced inertial forces are higher than those in other building typologies. Seismic behaviour of URM walls can be characterised as brittle, since cracks develop when tensile stresses exceed the masonry tensile strength; this damage pattern is typical of URM buildings located in epicentral regions of moderate earthquakes with magnitudes in the range of 5.0 and higher. Although these URM structures may exhibit nonlinear behaviour in the post-cracking stage, their ductility is limited and the overall performance is inferior when compared to otherwise similar RC and steel structures. Research studies related to seismic risk associated with local masonry construction have been carried out in the region, including Slovenia [5,6], Croatia [7–9], Bosnia and Herzegovina [10,11], Northern Macedonia [12], and Montenegro [13]. Post-earthquake reconnaissance after the 2010 Kraljevo earthquake (M 5.4) confirmed seismic vulnerability of URM residential buildings in urban centres of Serbia [14].

This paper presents an overview of historic code provisions related to seismic design of masonry buildings, with a special focus on multi-storey URM residential buildings typical for Serbia. Besides the past seismic design codes, which were followed in the design of existing masonry buildings in Serbia until 2019, the Eurocode 8 provisions pertaining to seismic design of masonry buildings have also been discussed, with the focus on Simple Buildings, which can be designed based on the minimum required Wall Index (*WI*), that is, the total cross-sectional area of all walls aligned in the same horizontal direction relative to the floor plan area. This simplified design approach is applicable to buildings with rigid diaphragms and structural walls subjected to in-plane seismic effects, which are the focus of this study. Numerous multi-storey URM residential buildings were damaged in the 2010 Kraljevo, Serbia earthquake. The paper presents the results of post-earthquake damage assessment for 23 buildings of this type in order to examine a relationship between the *WI* and the extent of earthquake damage. A case study of a typical three-storey URM building in Kraljevo, which was damaged in the earthquake, is also presented. The building was evaluated according to the requirements of seismic design codes from the SFRY and Eurocode 8.

2. An Overview of the Seismic Design Codes

The first comprehensive seismic design code in the SFRY was published in 1964 [15], after the 1963 Skopje earthquake (M 6.1), which caused significant fatalities and economic losses. A subsequent version of the code was issued in 1981 (PTN-S) [16] and it was the governing design code in Serbia until 2019. The PTN-S code was more advanced than its previous version published in 1964, and was similar to other international codes available at the time [17]. A detailed overview of the PTN-S code was presented by Jurukovski and Gavrilović [18].

Eurocodes were officially adopted as governing codes for the design of building structures in Serbia in 2019 [19]. Therefore, Eurocode 8–Part 1 (also referred to as EC8 in this paper) [20] has been adopted for seismic design of new structures in Serbia (SRPS EN 1998-1/NA:2018) [21]. This section presents an overview of relevant seismic design provisions contained in PTN-S and Eurocode 8.

Deterministic seismic hazard maps used in SFRY in conjunction with the PTN-S code were published in 1982. The entire territory of the country was divided into zones VI to IX based on the macroseismic intensity according to the MCS-64 scale. The PTN-S code did not explicitly prescribe a design response spectrum; however, it contained coefficients K_s and K_d to quantify seismic hazards at the site, depending on the dynamic characteristics, type of soil, and seismic intensity. The coefficient K_s depends on the seismic intensity zone and ranges from 0.025 for zone VII to 0.1 for zone IX. The dynamic response coefficient K_d is a function of the soil category and dynamic properties of a structure (fundamental period of vibration). According to the soil classification, Category I includes rock and hard soils (dense sand or stiff clay), Category II includes medium-dense and dense soils, while Category III includes soft soils.

Seismic hazard for design of structures according to the Eurocode 8 is estimated using a probabilistic seismic hazard assessment approach. A design earthquake for a "no-collapse" performance of a structure is based on 10% probability of exceedance in 50 years, also known as "475-year return period earthquake". Eurocode 8 quantifies seismic hazard for a building site by means of a reference Peak Ground Acceleration (PGA) for type A ground, a_{gR} (rock). Figure 1 shows the seismic hazard map for Serbia according to Eurocode 8. It can be seen that the a_{gR} value for the capital Belgrade is 0.1 g, while the value for the city of Kraljevo, which is the scope of this study, is 0.2 g. Note that, according to the previous seismic hazard map, which was used in conjunction with the PTN-S code, Belgrade and Kraljevo were located in zones VII and VIII, respectively.

Eurocode 8 elastic response spectrum is characterised by zero period acceleration and spectral values $S_e(T)$ calculated at three characteristic periods T_B, T_C, and T_D, which depend on the ground type (A to E) and other parameters. The zero period acceleration is equal to a_g x S, where a_g is the design PGA for the type A ground, which is equal to the product of reference peak ground acceleration a_{gR} and the building importance factor γ. Soil coefficient S accounts for soil amplification, depending on the ground type, while the correction factor η accounts for damping ratios different from 5%. Eurocode 8 prescribes two types of spectra with different shapes and spectral accelerations, depending on the potential seismic hazard sources for a specific site. The Type 2 spectrum should be used for the sites that are potentially exposed to the effects of earthquakes with a surface wave magnitude M_s not greater than 5.5, while the Type 1 spectrum is intended for seismic design at sites, which are potentially affected by earthquakes with higher M_s levels. The design spectrum for elastic analysis $S_d(T)$ is determined by dividing the elastic spectral values by the behaviour factor q, which reflects the expected ductility level for a structure and varies depending on the construction technology and structural system. For example, Eurocode 8 prescribes the lowest q value of 1.5 for URM structures designed without seismic provisions, which are the focus of this study. Figure 2 shows Eurocode 8 and PTN-S spectra for Kraljevo, Serbia. Note that the PTN-S curve is based on the prescribed shape for the K_d coefficient assuming soil Category II, and the values reflect the total seismic coefficient obtained as a product $K_o \times Ks \times K_d \times K_p$, using applicable values for the seismic design of URM buildings and building Category I (residential buildings), as follows: K_o = 1.0 (Category I), K_s = 0.1 (seismic intensity zone VIII), and K_p = 2.0 (URM building).

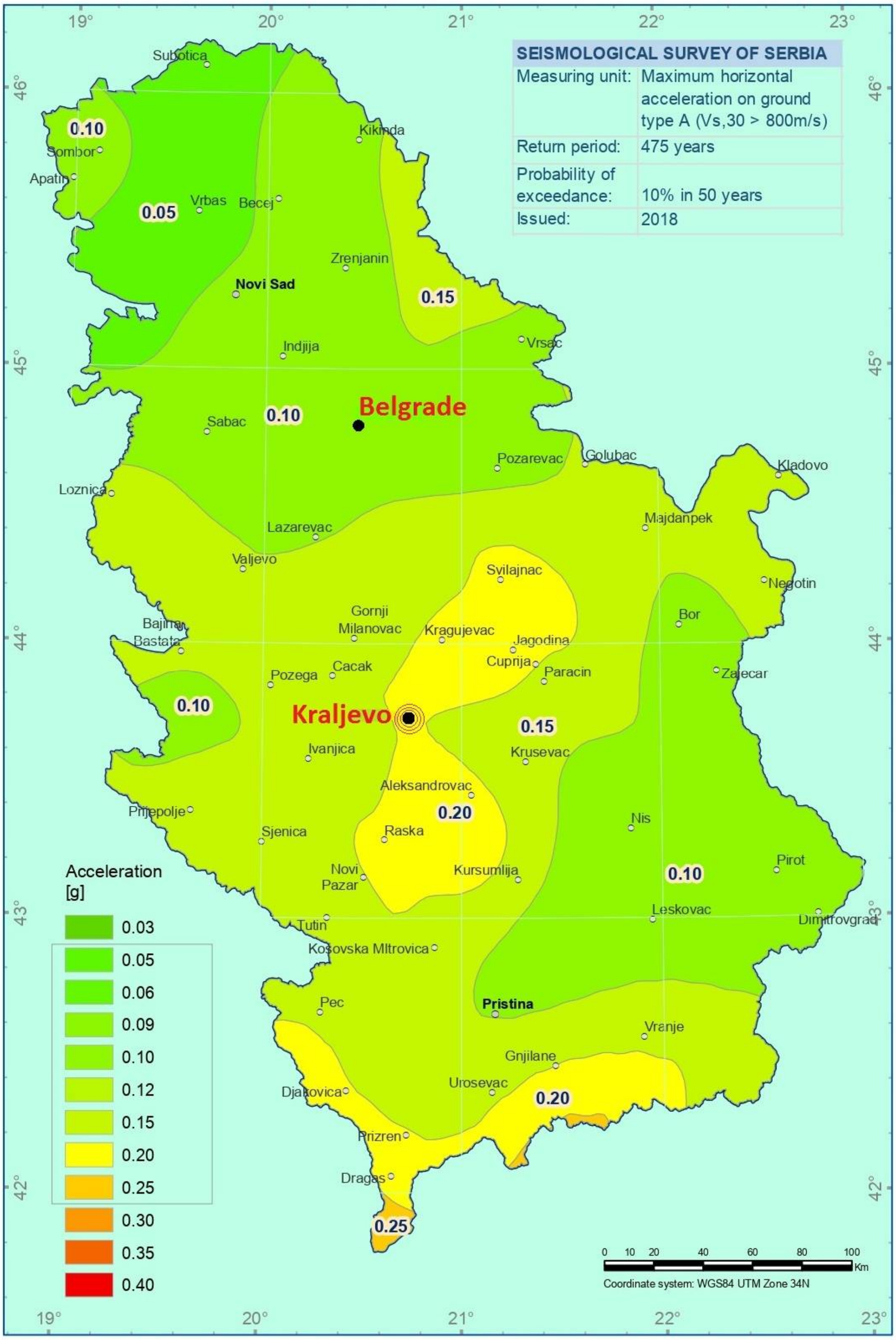


Figure 1. Seismic hazard map for Serbia showing the design PGA values for an earthquake with 10% probability of exceedance in 50 years, according to Eurocode 8 [22].

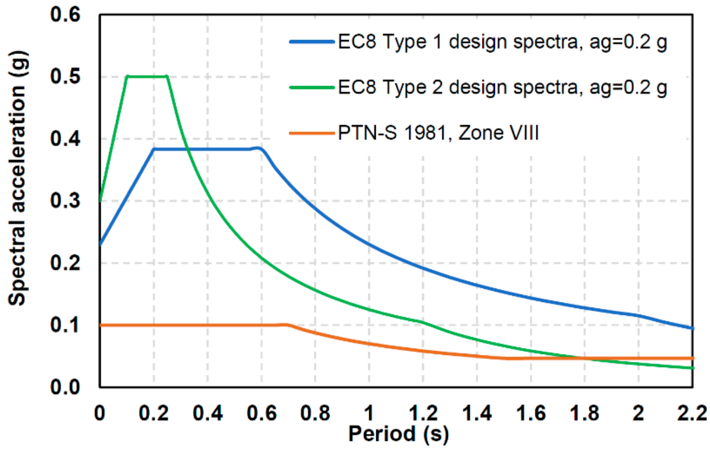


Figure 2. Design response spectra for Kraljevo, Serbia according to PTN-S (seismic intensity VIII, soil Category II) and Eurocode 8 (a_g = 0.2 g, ground type C, q = 1.5).

The PTN-S code prescribed equivalent static analysis procedure for seismic analysis of regular structures, while dynamic analysis was prescribed for the design of special structures, e.g., tall buildings (more than 25 storeys high) and/or irregular structures. Eurocode 8 has prescribed the following two approaches for linear elastic analysis of building structures: modal response spectrum analysis approach and lateral force method. The modal response spectrum analysis approach, also known as multi-modal analysis, is the reference method of analysis, which accounts for the effects of higher vibration modes on the dynamic response. The lateral force method, also known as the equivalent static analysis procedure, can be followed for seismic design of regular structures for which the contribution of fundamental vibration mode is predominant in the dynamic response (Cl. 4.3.3.2). It is considered that the requirements for the equivalent static analysis procedure are satisfied for regular buildings with fundamental periods, which were generally less than 1.0 s. Since masonry buildings are rigid and characterised by low fundamental periods they can be analysed using the equivalent static procedure. An overview of relevant seismic design provisions for URM buildings contained in the PTN-S and Eurocode 8, Part 1, are presented in Table 1.

Table 1. Overview of relevant seismic design provisions for masonry buildings (1981–present).

Design Code	**PTN-S [16]**	**EN 1998-1:2005 [20]**
Expected seismic performance	Life safety: structural damage acceptable at the design earthquake level, but collapse should be avoided.	1. Ultimate limit state: the structure as a whole must have adequate resistance and stability. 2. Damage limitation state: an adequate degree of reliability against unacceptable damage to be ensured by satisfying the deformation limits.
Total horizontal seismic base shear force	$S = K \times G$ [1] $K = K_o \times K_s \times K_d \times K_p$	$F_b = S_d(T_1)m\lambda$ [2] (original) $F_b = \left(0.85\frac{S_d(T_1)}{g}\right)W$ (buildings taller than two storeys)
Distribution of seismic forces up the building height	$S_i = S \times \frac{G_i \times H_i}{\sum G_j \times H_j}$ [3]	$F_i = F_b \times \frac{z_i \times m_i}{\sum z_j \times m_j}$ [4]
Seismic weight	G = self-weight of structural elements and permanent load (G_k) + 50% variable load (Q_k) at all levels (Cl. 19) $G = \sum G_{ki} + 0.5 \sum Q_{ki}$	W = self-weight of structural elements & other permanent loads (G_k) + loads due to variable actions (Q_k) multiplied by the combination coefficient for variable action (Cl.3.2.4) $W = \sum G_{ki} + \sum \phi \times \psi_2 \times Q_{ki}$

Table 1. *Cont.*

Design Code	PTN-S [16]	EN 1998-1:2005 [20]
Rigid diaphragm assumption	Yes	Yes
Combination of the effects of horizontal components of the seismic action	Does not consider simultaneous action of seismic loading in two orthogonal directions.	Considers simultaneous action of seismic loading in two orthogonal directions, e.g., 100% action effects in X-direction and 30% action effects in Y-direction: $E_{Edx} + 0.3E_{Edy}$ (Cl.4.3.3.5.1)

[1] K = total seismic coefficient; K_o depends on the building category (I to IV); K_p = coefficient of ductility and damping which accounts for the type of structural system; [2] $S_d(T_1)$ = design spectral acceleration corresponding to fundamental period T_1; m = total mass of the superstructure; λ = correction factor equal to 0.85 for three-storey plus buildings for which $T_1 < 2T_c$; [3] H_i = height of level i measured from the base of the building; G_i = seismic weight at level i; [4] z_i = height of level i measured from the base of the building; m_i = seismic mass at level i.

3. An Overview of Seismic Design Provisions for Masonry Buildings

Design of masonry structures in Serbia and other countries within the territory of former Yugoslavia has been governed by applicable codes, starting with the 1949 design code [23] to the latest code issued at the time of breakdown of SFRY in 1991 [24,25]. Eurocode 6 (EN1996-1-1:2004) [26] was recently adopted as the official code for the design of masonry structures in Serbia [27]. According to the Yugoslav codes, masonry buildings were classified into the following three types: (i) URM buildings, which are referred to as ordinary masonry (OM); (ii) confined masonry (CM); and reinforced masonry buildings. Note that since the 1949 code, URM (OM) buildings were required to have horizontal RC confining elements (also known as ring beams or tie-beams) at floor/roof levels. These confining elements were integrated with RC floor/roof slabs, which were used in buildings of post-1945 construction. CM buildings have both horizontal and vertical RC confining elements; hence, the main difference between OM and CM buildings is in the provision of vertical confining elements. Finally, reinforced masonry buildings have horizontal reinforcement placed in mortar bed joints, in addition to horizontal and vertical confining elements. Table 2 presents a summary of building height restrictions applicable to OM and CM buildings. It can be seen from the table that the PTP-12 code permitted construction of taller OM buildings (up to 4-storey high) in seismic zone VIII, while other two codes limited building height to maximum two storeys. It is interesting that the PTN-S and EN 1998 codes have the same height limits for OM buildings. As a consequence of this provision, there are numerous, up to four-storey high URM buildings (mostly multi-family housing) throughout the territory of former Yugoslavia, which were constructed in the period 1945–1980. Buildings of that type experienced damage in the December 2020 Petrinja, Croatia earthquake [28]; note that the epicentral zone of the Petrinja earthquake was classified as seismic intensity zone VIII, according to the PTP-12 and PTN-S codes.

Table 2. Code-prescribed height limits (maximum number of storeys) for URM (OM) and confined masonry (CM) buildings (1945–present).

Seismic Intensity (MCS)	PTP-12 [15] (1964–1980)		PTN-S [16] (1981–2019)		EN 1998-1:2005 [20] [1] (2020–Present)	
	OM	CM	OM	CM	OM	CM
VII	5	6	3	5	3	4
VIII	4	6	2	4	2	3
IX	3	5	n/a	3	n/a	2

[1] Simple masonry buildings according to Cl 9.7; n/a—not acceptable.

An overview of seismic design provisions for URM (OM) buildings from various codes is presented in Table 3.

Table 3. Overview of seismic design provisions for URM (OM) buildings according to different codes (1964-present).

Provision	PTP-12 [15] (1964–1980)	PTN-S [16] (1981–2019)	PTN-Z [24] (1991–2019)	EN 1998–1:2005 [20] (2020–Present)
Design method	Allowable Stress Design	Allowable Stress Design; Ultimate Limit States Design (strength only)	Allowable Stress Design; Ultimate Limit States Design (strength only)	Ultimate Limit States Design (strength plus serviceability)
Materials	Cement:lime: sand mortar mandatory, except single-storey buildings in seismic intensity zones VII and VIII	Cement:lime: sand mortar mandatory; grade M25-M50 (2.5–5.0 MPa)	Cement:lime:sand mortar grade M2 (2.0 MPa) or cement mortar grade M10 (10.0 MPa)	Min. mortar compressive strength 5.0 MPa
		Masonry units with horizontal holes not permitted	Solid clay bricks, modular clay blocks permitted	Detailed classification of masonry units
		Min. strength for solid clay bricks MO100-MO150 (10–15 MPa); modular clay blocks MO150 (15 MPa)	Min. strength for solid clay bricks and modular clay blocks 10.0 MPa	Min. unit compressive strength 5.0 MPa
Wall thickness	25–38 cm	Min. 19 cm	Min. 24 cm (exterior walls) and 19 cm (interior walls)	Min. 24 cm (effective thickness $t_{ef,min}$)
Minimum effective slenderness ratio $(h_{ef}/t_{ef})_{max}$	Not prescribed	Not prescribed	10–20 (depending on the strength of masonry units and mortar)	12.0

According to Section 9.6 of Eurocode 8, Part 1 [20], seismic design of masonry buildings can be performed by one of the following approaches: (i) prescriptive approach called "Rules for Simple Buildings", which is applicable to regular low-rise buildings and different seismic hazard levels, or (ii) engineered analysis and design approach, which requires a verification of safety against collapse for each structural element in a building, where the resistance is determined according to Eurocode 6 provisions [26]. The latter approach needs to be followed in design of all buildings, which do not meet the requirements for Simple Buildings, as specified in Sections 9.2, 9.5, and 9.7.2 of Eurocode 8, Part 1. It should be noted that masonry walls designed according to either approach need to meet seismic detailing requirements prescribed in Section 9.5 of Eurocode 8, Part 1.

4. Seismic Design of Simple Masonry Buildings According to Eurocode 8, Part 1

Simple Buildings addressed by Section 9.7 of Eurocode 8, Part 1, are characterised by specific features outlined in Sections 9.2, 9.5, and 9.7.2 [20]. It is required that floors and walls are connected in horizontal and vertical directions, and that floors/roofs act as rigid diaphragms. Buildings need to have a regular plan shape and a symmetrical wall layout. The walls need to conform to certain geometric requirements, related to height/thickness and height/length ratios.

Simple buildings are deemed to satisfy seismic design requirements of the code, provided that the minimum required amount of walls is provided in each horizontal direction of a building plan relative to the floor plan area, referred to as Wall Index (*WI*) in this paper (note that an alternative term "wall density" is also used in technical publications). Figure 3 shows a sample floor plan of a masonry building. The *WI* value for a given direction of a floor plan (X or Y) is a ratio of the sum of cross-sectional areas for all walls in the direction of considered earthquake action and the ground floor plan area, that is,

$$WI = \frac{A_w}{A_p} \tag{1}$$

where A_W is the cross-sectional area of walls with their lengths parallel to one direction at the ground floor level, and A_P is ground floor plan area. The required *WI* value for a specific building increases with the number of storeys and seismic hazard level, which is expressed as a product of design site acceleration (a_gS) and a correction factor k. Note that the k value ranges from 1.0 to 2.0, depending on the average wall length. For example, k = 1.5 when an average wall length is 4.0 m. Table 4 contains the minimum required *WI* values for ordinary (URM) buildings according to Eurocode 8, Part 1 [20] (Table 9.3); note that the code uses term $p_{A,min}$ instead of *WI*. The code-prescribed values were determined assuming the minimum compressive strength for masonry units (e.g., modular clay blocks) of 5.0 MPa. For a three-storey URM building, *WI* values range from 3.0 to 5.0 % when ground acceleration *agS* increases from 0.07 to 0.15 g (provided that k = 1.0). Note that the *WI* values presented in Table 9.3 are recommended values; however, different values can be determined by a country-specific National Annex.

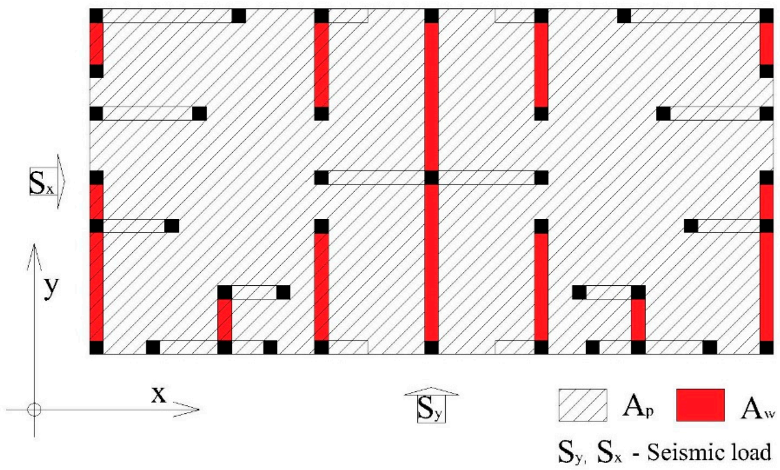


Figure 3. Input parameters for the Wall Index (*WI*) calculation.

Table 4. Minimum required *WI* ($p_{A,min}$) values for URM buildings based on Eurocode 8, Part 1.

Number of Storeys	**Acceleration at Site a_gS**							
	≤0.07 $k\times g$		**≤0.10 $k\times g$**		**≤0.15 $k\times g$**		**≤0.20 $k\times g$**	
	WI	***WI/n***	***WI***	***WI/n***	***WI***	***WI/n***	***WI***	***WI/n***
1	2.0%	2.00%	2.0%	2.00%	3.5%	3.50%	n/a	-
2	2.0%	1.00%	2.5%	1.25%	5.0%	2.50%	n/a	-
3	3.0%	1.00%	5.0%	1.67%	n/a	-	n/a	-
4	5.0%	1.25%	n/a	-	n/a	-	n/a	-

Notes: n/a—not acceptable.

It is useful to normalise the *WI* values in order to compare buildings with different heights (number of storeys); this can be accomplished by dividing the *WI* value by the number of storeys (n), and use *WI/n* value as "Wall Index per floor".

A *WI* value indicates lateral load-resisting capacity of a masonry building in which walls are subjected to seismic action in the direction under consideration. A building must have sufficient shear capacity in each horizontal direction (X and Y) to resist the seismic forces at each storey level. The shear capacity depends on the number of shear walls in each horizontal direction, and the capacity of each wall to resist seismic forces. On the other hand, seismic demand; that is, seismic storey shear force at ground floor level, is resisted by the floor/roof diaphragm, and is subsequently transferred to individual walls. The *WI* concept is applicable to buildings with rigid diaphragms, in which a seismic shear force in specific wall is proportional to its stiffness relative to the sum of the stiffnesses of all walls aligned in the same direction. It should be noted that the *WI* concept cannot be applied for assessing seismic safety of wall structures subjected to out-of-plane seismic effects.

In low-rise masonry buildings, seismic behaviour of the walls is usually governed by shear. Shear stiffness of the wall is proportional to its cross-sectional area (length × thickness), hence it is possible to establish the required ratio between the amount of walls (sum of cross-sectional areas of all walls in one direction) and the floor plan area. The above explanation is based on the strength-based design approach, which gives conservative *WI* values. Alternatively, *WI* values can be determined based on the displacement-based design approach, which will likely give lower (and more realistic) *WI* values, however it requires knowledge of deformation-based nonlinear performance parameters for masonry walls.

Several research studies in countries like Mexico and Chile have confirmed a correlation between the *WI* and the extent of earthquake damage in masonry and RC wall structures [29]. Chilean researchers correlated the *WI* to the observed damage grade for more than 280 masonry buildings affected by the 1985 Llolleo, Chile earthquake (M 7.8) [30]. The surveyed buildings were of reinforced, CM, and hybrid masonry construction. The buildings were one- to four storeys high. It was concluded that a minimum *WI/n* value of 1.15% or higher was required in each direction to avoid earthquake damage in these buildings. Masonry buildings with a *WI/n* value in the range from 0.50 to 1.15% suffered moderate damage, while buildings with a *WI/n* value of less than 0.50% suffered heavy damage. A study on CM buildings affected by the 2010 Maule, Chile earthquake (M 8.8) showed that, in general, the buildings with a *WI/n* value of 0.9% and higher remained undamaged, while buildings with *WI/n* of 0.75% or less experienced severe damage at the MSK shaking intensity of VII or higher [31]. A study on 238 CM buildings damaged in the 2008 Wenchuan, China earthquake (M_s 8.0) was performed by Cai et al. [32]. The results showed that for seismic intensity zone VIII, which corresponds to acceleration 0.10 $k \times g$ according to Eurocode 8 (see Table 4), "safe" *WI/n* values range from the minimum of 1.2% for heavy damages to 1.7% for moderate damages. For seismic intensity zone IX, which corresponds to acceleration 0.20 $k \times g$, "safe" *WI/n* values range from the minimum of 2.0% for heavy damages to 2.5% for moderate damages. Note that these values are related to CM buildings, which are expected to remain safe at lower *WI/n* values compared to URM buildings. The same study included some URM buildings; however, the results were not reported in detail.

5. Building Damage in the 2010 Kraljevo, Serbia Earthquake

The most damaging earthquake in Serbia in the 21st century occurred on 3 November, 2010. The epicentre was close to the Sirča village, approximately 4 km north of Kraljevo, a city with population of approximately 68,000. The earthquake had magnitude (M_L) of 5.4 and focal depth of 13 km [33], it caused two fatalities and more than USD 100 million in damages [2]. It was estimated that approximately 6000 buildings experienced damage or collapse due to the earthquake; out of these, approximately 25% were found to be unsafe to occupy [34]. The earthquake caused collapse of significant number of single-family dwellings and damages of many other structures, including multi-family housing, educational and health facilities, heritage structures, etc. Unfortunately, acceleration records for the earthquake are not available.

Masonry buildings accounted for more than 90% of the building stock in the earthquake-affected area and were most severely affected by the earthquake. Based on the inspection of damaged buildings, it was noticed that severe damage and/or collapse of single-family URM dwellings (mostly one- or two-storey high) were due to inadequate design and construction, poor quality of materials, and/or construction of horizontal/vertical extensions [14].

Several multi-family URM buildings (3- to 5-storey high) were damaged in the earthquake and required repair and/or retrofit [35–37]. These buildings were constructed after the World War II (1945–1963), but before the first seismic code for the former Yugoslavia was published in 1964 [1]. Damage patterns observed in these buildings after the Kraljevo earthquake were discussed in a few publications [14,35]. 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 [14,38].

A rapid damage survey was performed immediately after the earthquake and it was focused on ensuring life safety of the occupants [39]. The second round of surveys was performed to establish damage states using two locally developed survey forms (the "long" and "short" ones). The damage classification was custom-developed for this earthquake and contained six damage grades (K1 to K6), ranging from slight damage to collapse [40]. The survey was performed by local experts, as well as civil engineers and academics from all universities in Serbia [34].

A study on 1193 residential masonry buildings damaged in the 2010 Kraljevo earthquake was performed using the information from the post-earthquake survey forms collected by the City of Kraljevo [34,41]. The authors classified residential building stock in the earthquake affected area into six types (BT1 to BT6) based on the architectural layouts, structural features, and construction date: BT1-traditional, stone foundation, wooden superstructure buildings (pre-1950s); BT2-masonry structures constructed using the old brick format (pre-1933); BT3-masonry structures constructed using the new brick size (post-1933); BT4-masonry structures with horizontal RC tie-beams (1963–1975); BT5–CM structures with horizontal and vertical RC confining elements (1975–1990), and BT6–CM structures with horizontal and vertical RC confining elements (1990–2010). URM multi-family buildings, which are the scope of this study, could be classified as BT4.

The authors used the post-earthquake damage survey data to classify building damage into the following four damage states (DS): DS1–slight damage, DS2–moderate damage, DS3–heavy damage, and DS4–collapse. They decided to merge some of the grades used in the original survey, e.g., K1 and K2, and also K4 and K5. The distribution of damage states across various building types is presented in Figure 4. It can be seen from the figure that the traditional constructions (BT1) suffered the most extensive damage (84% of the database entries were in the DS3 and DS4 states), while CM building types (BT5 and BT6) suffered only slight damage (90% and 96% of the database entries were in DS1 and DS2 states combined). It can be seen that the BT4 type buildings, which are relevant for this study, experienced more severe damage (26% of the entries were in DS3 and DS4 states) compared to the BT5 type (10%).

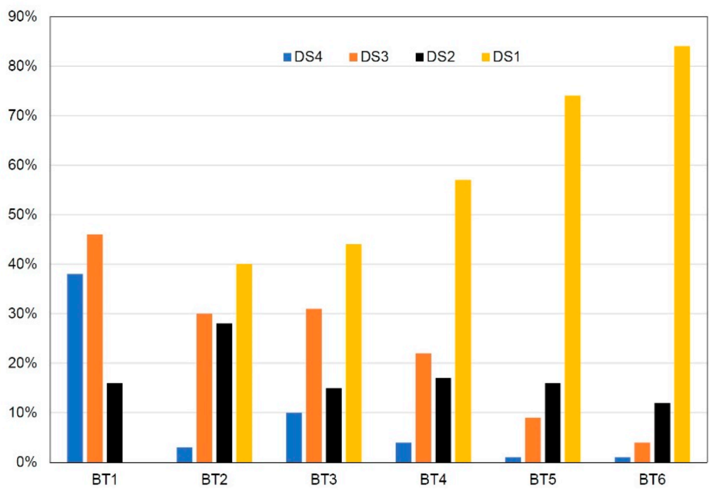


Figure 4. The distribution of damage states for various building types for 1193 buildings surveyed after the 2010 Kraljevo, Serbia earthquake (based on [34]).

Table 5 presents official damage classification used for building surveys after the 2010 Kraljevo earthquake (K1 to K6), as well as a simplified classification used in the study by Stojadinović et al. [41] (DS1 to DS4). Finally, a damage classification for masonry buildings according to the EMS-98 scale was also presented [42]. It can be seen from the table that damage states K4 and K5 based on the official damage classification correspond to damage grade DS3, according to the simplified classification [41]. As far as mapping to the EMS-98 scale is concerned, based on the review of 23 damaged buildings, which were the scope of this study damage states K4 and K5 correspond to Grade 3, according to EMS-98 [42].

Table 5. Damage classification for masonry buildings affected by the 2010 Kraljevo earthquake.

<table>
<tr><th>2010 Kraljevo Earthquake—Official Damage Classification [1]</th><th>Simplified Classification [2]</th><th>European Macroseismic Scale (EMS-98) [3]</th></tr>
<tr><td>K1: slight non-structural damage, including roofing, plaster cracking, damaged chimneys.</td><td rowspan="2">DS1: slight damage</td><td>Grade 1: negligible to slight damage (no structural damage, slight non-structural damage).
Hairline cracks in very few walls, fall of small pieces of plaster only.</td></tr>
<tr><td>K2: more widespread non-structural damage, collapse of chimneys, extensive plaster cracking; slight structural damage of loadbearing walls and extensive cracking of partition walls.</td><td>Grade 2: moderate damage (slight structural damage, moderate non-structural damage).
Cracks in many walls; fall of fairly large pieces of plaster; partial collapse of chimneys.</td></tr>
<tr><td>K3: moderate structural damage of roof structure, gable walls, widening of cracks in loadbearing walls.</td><td>DS2: moderate damage</td><td rowspan="2">Grade 3: substantial to heavy damage (moderate structural damage, moderate non-structural damage).
Large and extensive cracks in most walls; roof tiles detach; chimneys fracture at the roofline; failure of individual non-structural elements (partitions, gable walls).</td></tr>
<tr><td>K4: heavy structural damage or partial collapse of roof structure, severe damage/collapse of partition walls.</td><td rowspan="2">DS3: heavy damage</td></tr>
<tr><td>K5: very heavy damage and displacement of loadbearing structure (walls) which can be repaired.</td><td>Grade 4: very heavy damage (heavy structural damage, very heavy non-structural damage).
Serious failure of walls; partial structural failure of roofs and floors.</td></tr>
<tr><td>K6: severe damage or collapse of loadbearing structure, e.g., damage of all structural walls at specific levels and their partial collapse.</td><td>DS4: collapse</td><td>Grade 5: destruction (very heavy structural damage)—total or near total collapse.</td></tr>
</table>

[1] Serbian Chamber of Engineers 2010 [40]; [2] Stojadinović et al., 2017 [41]; [3] Grünthal 1998 [42].

Detailed engineering reports were developed for buildings, which experienced structural damage in the earthquake and had to be repaired and retrofitted. These buildings were mostly assigned damage grades K4 and K5, see Table 5. The engineering reports contained information related to the observed damage, seismic evaluation according to the PTN-S code, and drawings containing details of repair and seismic retrofitting solutions. Retrofitted buildings were intended to comply with the seismic design provisions of the PTN-S code. Note that similar reports were not prepared for buildings, which experienced minor damage (grades K1 to K3); hence, information related to similar buildings, which experienced minor damage was not available for this study.

The authors reviewed information related to 23 damaged buildings located in the centre of Kraljevo (less than 10 km away from the epicentre). All buildings were multi-family residential buildings of URM construction. The selected buildings were typical for post-WWII residential construction in the former SFRY, for the period before 1964. Only 1 out of 23 buildings was constructed after 1964. The height of the original buildings ranged from two to five storeys, but majority of buildings (74%) were 3-storey high. Loadbearing masonry walls were typically constructed using solid clay bricks and cement:lime:sand mortar, and their thickness ranged from 25 cm (interior walls) to 38 cm (exterior walls). Floors and roofs were in the form of semi-prefabricated composite masonry and concrete systems, which consisted of horizontally aligned masonry units with cast-in-place RC topping and RC tie-beams (ring beams). This technology was used in the former SFRY and was known under the commercial name TM3.

Each building had a centrally located staircase, but elevators were not provided. Eleven buildings had vertical extensions, which consisted of additional (one to three) floors constructed on top of the existing building. The buildings had regular floor plans and a symmetrical wall layout. The average floor plan area was 321 m^2, but the values ranged from 143 to 860 m^2. The buildings also had regular elevations, without significant variations in stiffness between adjacent floors.

These buildings experienced both structural and nonstructural damage in the Kraljevo earthquake. Damage patterns were similar in all buildings. Structural damage was limited to the walls located at the bottom floors (usually ground floor) in the form of inclined cracks, typical for diagonal tension shear failure, which is caused by in-plane seismic effects. No visible out-of-plane damage was observed, which could be explained by relatively low wall slenderness and a limited seismic intensity. There was no visible damage at the wall-to-floor connections. Nonstructural damage was observed in partition walls at the upper floors, particularly in the extended portion of the building (top floor). Nonstructural damage was both in the form of in-plane and out-of-plane damage. Out-of-plane damage can be attributed to higher spectral accelerations at the top floor level and relatively high wall slenderness ratio.

Based on the survey data, seven buildings (30% of the total number) experienced very heavy damage (K5), while remaining 16 buildings experienced heavy damage (K4). These buildings satisfied the requirements for Simple Buildings prescribed by Eurocode 8, Part 1; hence, the *WI* values for each building were determined to assess adequacy of the design. Note that the *WI* values were determined for each horizontal direction of a floor plan, that is, longitudinal (X) and transverse (Y). Note that the damage rating was assigned to a specific building based on the overall extent of damage, irrespective of the fact that walls in only one direction experienced damage. Therefore, both *WI/n* values for a specific building were assigned the same damage state.

Figure 5 shows a histogram of normalised *WI* values (*WI/n*) for all buildings. Note that two *WI/n* values are assigned to each building, corresponding to X and Y directions; this results in 46 data points in total. It can be seen that more than 95% of all values are less than 2.9%, but the remaining values are relatively high (up to approximately 5.0%). Approximately 30% of all *WI/n* values are less than 1.5%, which is considered inadequate according to Eurocode 8, since the minimum required value of 2.5 % was prescribed for buildings of same height and earthquake intensity similar to the 2010 Kraljevo earthquake. Note that an average *WI/n* value for all buildings is approximately 2.0%.

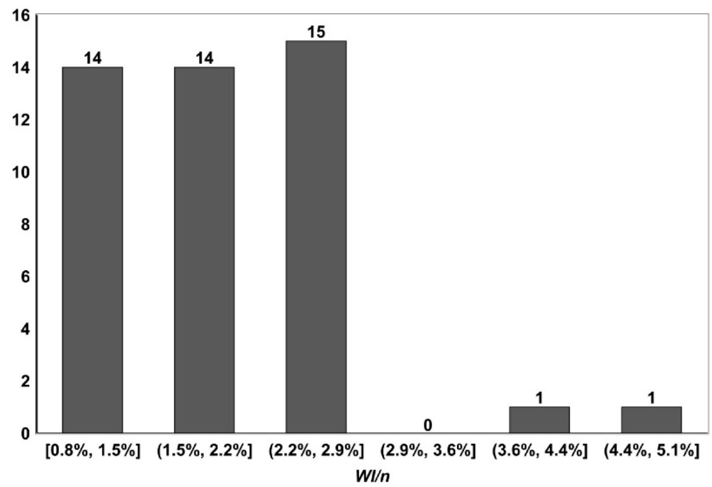


Figure 5. Histogram of *WI/n* values for 23 buildings (based on two *WI/n* values per building).

Figure 6 shows a relationship between the *WI/n* versus the number of storeys (*n*). It can be seen that the *WI/n* values are highest for 2-storey buildings and lowest for 5-storey buildings. This trend can be explained by the difference in architectural planning for two-storey buildings and other buildings considered in this study.

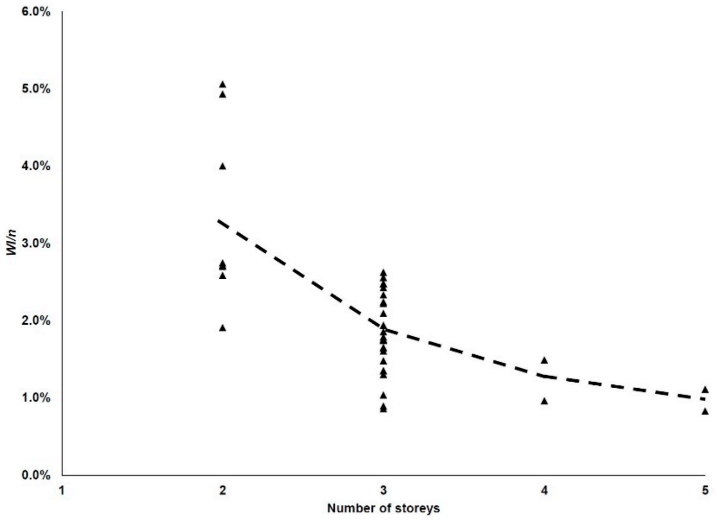


Figure 6. Distribution of *WI/n* values depending on the number of storeys (based on two *WI/n* values per building).

Figure 7 presents a relationship between the observed damage (grades K4 and K5) and the calculated *WI/n* values. It can be seen that the buildings, which experienced very heavy damage (grade K5), had *WI/n* values within the range of less than 1.0% up to approximately 2.5%. The average *WI/n* value for these buildings is 1.9%. It can be seen from the figure that the majority of data points are associated with buildings, which experienced heavy damage (grade K4). A large scatter of *WI/n* values can be observed for these buildings (from less than 1.0% up to 5.0%), but more than 90% values are less than 2.9%. Note that 7 (out of 32) *WI/n* values are higher than the minimum required value of 2.5% which was prescribed by Eurocode 8.

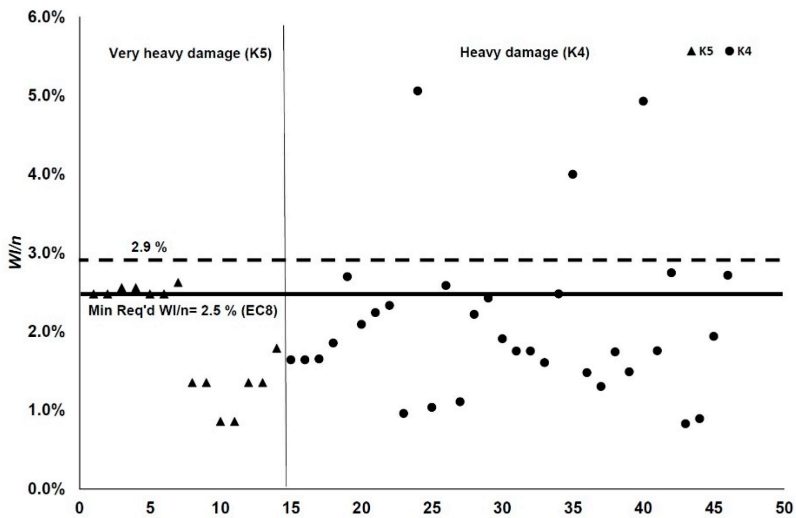
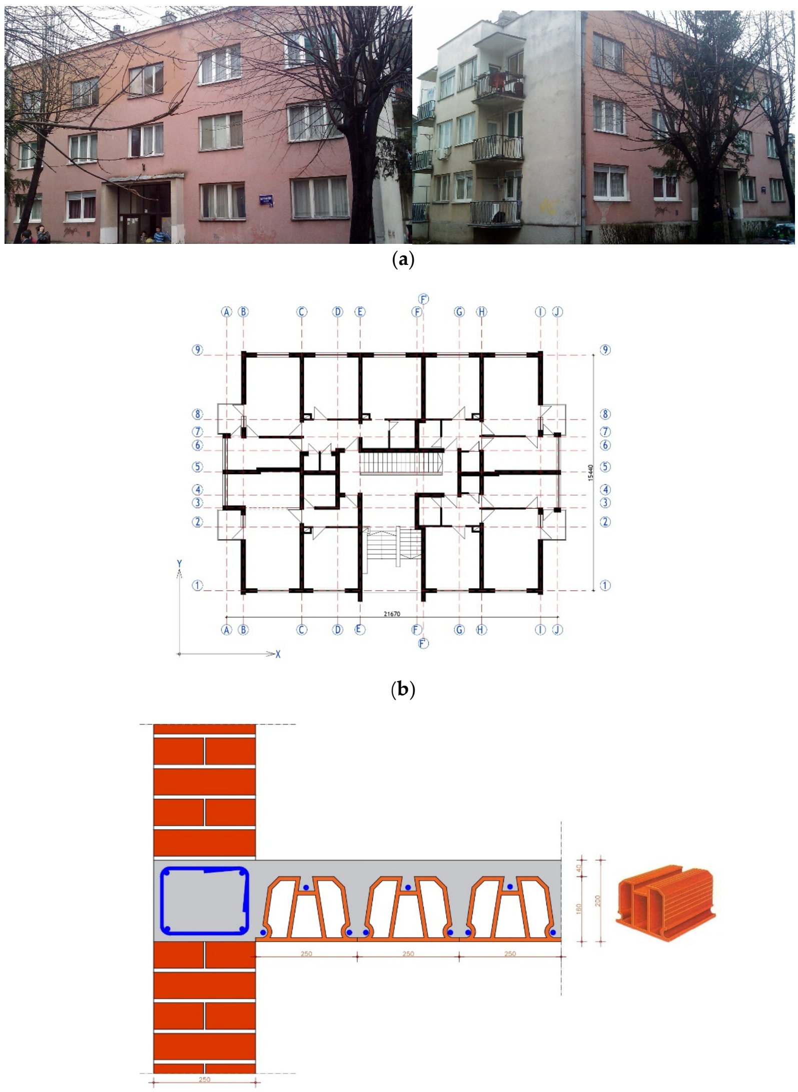


Figure 7. Relationship between the *WI/n* and damage grade for 23 buildings (based on two *WI/n* values per building).

6. Seismic Assessment a Typical URM Building Damaged in the 2010 Kraljevo Earthquake

6.1. Building Description

The case study building is located in the Njegoševa Street no. 2 in Kraljevo. The building was constructed around 1950 as a 3-storey residential building with a basement and a half-floor at the top, see Figure 8a). The lower three floors are 22.2 m in length and 16.0 m in width. The top floor has smaller plan dimensions (11.0 m length and 10.9 m width). The typical floor height is 2.8 m. Walls at the lower three floors are 25 cm thick and were constructed using solid clay bricks in cement:lime mortar, while the walls at the top floor were constructed using modular (multi-perforated) clay and concrete blocks. Thickness of walls constructed using modular clay blocks was 140 mm (with vertically aligned holes), but in some cases, partition walls were 70 mm thick because the holes were aligned horizontally (as shown in Figure 9c). Floor and roof structures were constructed using semi-prefabricated composite masonry and concrete system, as discussed above. Although the original construction documentation was not available, post-earthquake building survey confirmed the presence of RC tie-beams at each floor level. It is assumed that the building has RC strip footings. The floor plan at the ground floor level is shown in Figure 8b). Note that longitudinal walls are aligned in the N–S direction.

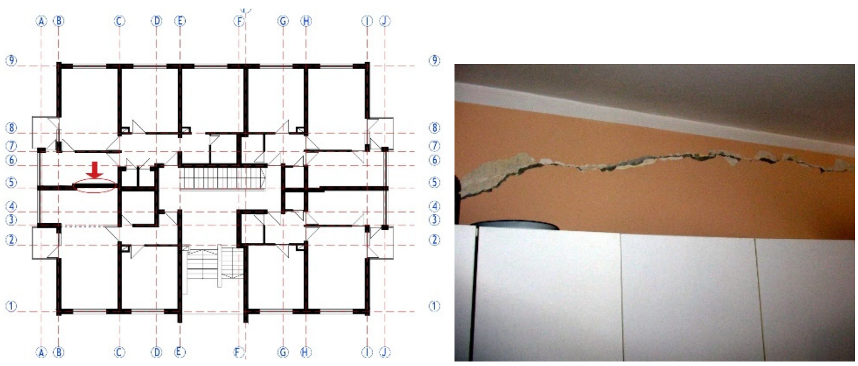


(a)

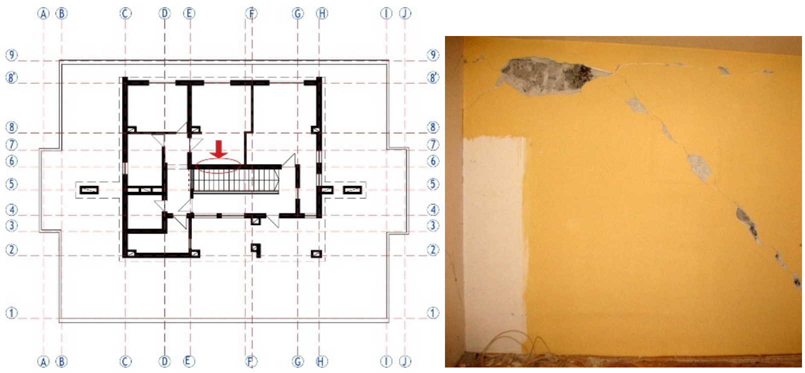
(b)

(c)

Figure 8. Case study building in Kraljevo, Serbia: (**a**) exterior views; (**b**) floor plan at ground floor level, and (**c**) floor system.



(**a**)



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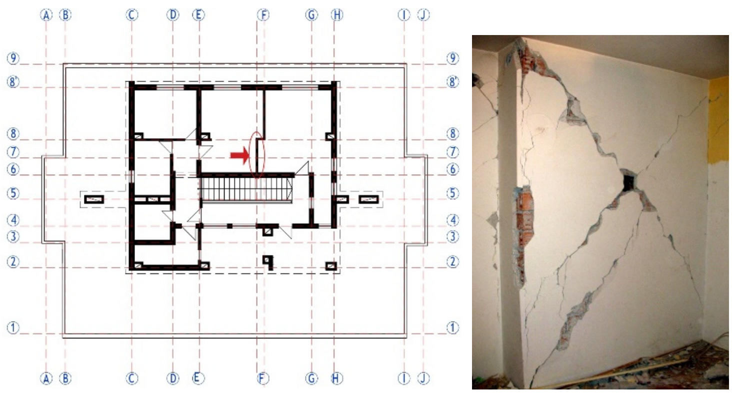

(**c**)

Figure 9. Earthquake damage patterns: (**a**) severe cracking in a longitudinal wall at the second floor level (gridline 5); (**b**) inclined cracking in a longitudinal wall at the top floor level (gridline 6 adjacent to the staircase) and (**c**) failure of a partition wall at the top floor level.

This is a loadbearing masonry structure typical for the post-World War II period. Loadbearing walls are of URM construction and there are rigid floor/roof diaphragms. Since the building was constructed around 1950, it is assumed that seismic effects were not considered in the original design. According to the official post-earthquake damage survey, the building experienced heavy damage (grade K4) in the 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. For example, wide cracks developed along the masonry-to-tie beam interface in a longitudinal wall along gridline 5 at the second floor level (see Figure 9a). The most extensive damage was observed in the extended portion of the building (top floor level). Wall along gridline 6 (adjacent to the staircase) experienced structural damage, in the form of a wide inclined crack, which extended into a wide horizontal crack at the masonry-to-tie beam interface (see Figure 9b). Extensive nonstructural damage, in the form of wide diagonal tension cracks, was observed in 70 mm thick partition walls constructed using modular clay blocks with horizontally aligned holes, as illustrated in Figure 9c).

6.2. Numerical Model

A 3D numerical model of the building was developed for seismic analysis purposes. The model simulates masonry walls as shell elements, while floor and roof slabs were modelled as plate elements; see Figure 10. The base supports are fixed. The modulus of elasticity of masonry of 2410 MPa was used for developing the wall element model, which was determined considering cement:lime:sand mortar and 10 MPa characteristic compressive strength for masonry units.

Dynamic properties of the model were determined by means of modal analysis. Due to a slight difference in seismic masses according to the PTN-S and Eurocode 8, there are slight differences in fundamental periods corresponding to these codes. Fundamental periods for longitudinal direction are 0.27 and 0.26 s for PTN-S and Eurocode 8, respectively (the difference is insignificant), while the corresponding values for transverse direction are the same (0.20 s). It can be observed that the fundamental period is smaller in transverse direction, as expected; this is due to larger amount of walls and the corresponding *WI* value, as discussed later in this section.

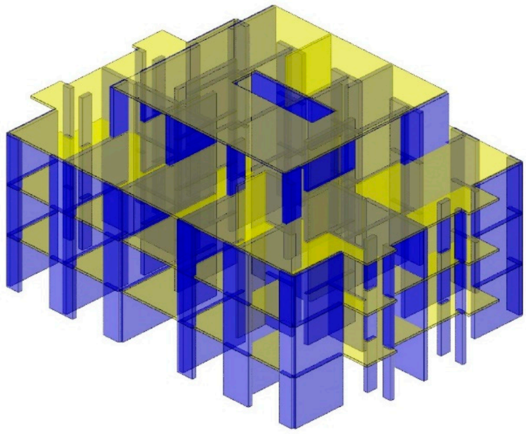

Figure 10. Numerical model for the case study building (isometric view).

6.3. Seismic Analysis According to PTN-S and Eurocode 8

Linear elastic equivalent static analysis for the case study building was performed according to the PTN-S and Eurocode 8, Part 1. Since this is a low-rise building with rigid diaphragms, seismic forces were distributed to individual walls in proportion to their respective shear stiffnesses. It is important to acknowledge the difference in load combinations prescribed by the two codes. The load combination according to the PTN-S code is as follows

$$\sum G_k + \sum Q_k + \sum Q_s$$

where G_k is the self-weight of structural elements and other permanent load; Q_k is variable load, and Q_s is loading due to earthquake action. The corresponding load combination according to Eurocode 8, Part 1 is as follows

$$\sum G_k + 0.3 \sum Q_k + \sum A_{Ed}$$

where A_{Ed} denotes loading due to earthquake effects. Note a significant difference in partial safety factor for variable load (Q_k) between the PTN-S code (1.0) and Eurocode 8 (0.3).

The building was analysed according to the PTN-S code, which was in effect in Serbia at the time of the earthquake. Seismic forces were determined considering the following parameters: building category coefficient K_o = 1.0, seismic intensity coefficient K_s = 0.05 (seismic intensity zone VIII), dynamic response coefficient K_d = 1.0, and ductility and damping coefficient K_p = 2.0. A detailed seismic analysis was also performed according to the requirements of Eurocode 8, Part 1, as discussed earlier in the paper (see Table 2). Both Type 1 and Type 2 design spectra for Kraljevo were considered, as shown in Figure 2.

The total horizontal seismic base shear forces according to the PTN-S and Eurocode 8, Part 1, are summarised in Table 6. It can be observed from the table that the PTN-S results in significantly less forces compared to the Eurocode 8, with the ratio ranging from 0.33 to 0.45. Note that, in both cases, the forces were multiplied by applicable partial safety factors.

Table 6. Total horizontal seismic force (multiplied by applicable partial safety factors).

	Total Seismic Force (kN)		Total Seismic Weight (kN)
Design Code	**Longitudinal (X) Direction**	**Transverse (Y) Direction**	
PTN-S	948.6	948.6	11,912.0
EC 8-T1 (Type 1 spectra)	2301.7	2127.2	11,416.4
PTN-S/EC8-T1	0.41	0.45	1.04
EC8-T2 (Type 2 spectra)	2851.4	2781.8	11,416.4
PTN-S/EC8-T2	0.33	0.34	1.04

6.4. Seismic Safety Verification According to the Rules for Simple Buildings (Eurocode 8, Part 1)

Since the building has a regular plan shape and wall layout, it was evaluated according to the Rules for Simple Buildings of Eurocode 8, Part 1 [20]. The *WI/n* values were calculated as 1.76% and 1.03% for transverse and longitudinal directions, respectively. Based on the seismic hazard parameters and the number of storeys, the minimum *WI/n* value of 2.5% is prescribed for each horizontal direction of a 3-storey URM building in seismic zone VIII, corresponding to $a_g S$ value of 0.2 g (as assumed in this study). The *WI/n* value for longitudinal direction (1.03%) appears to be significantly deficient according to Eurocode 8, Part 1, which prescribes the minimum value of 2.5% (corresponding to $a_g S \leq 0.15k \cdot g$ and $k \leq 1.33$). Interestingly, the building experienced damage in the longitudinal walls due to the 2010 Kraljevo earthquake, as discussed above.

6.5. Verification of Lateral Load Resistance for Individual Walls

After the internal seismic forces in the walls were determined, lateral load resistance was verified according to both codes. The walls were evaluated for the effects of shear, plus combined axial load and bending. Since this is a low-rise building, the wall behaviour is shear-dominant.

Verification of lateral resistance for masonry walls was first performed according to the Ultimate Limit Stress Design approach prescribed by PTN-Z [24]. Since the mechanical properties of masonry were not known at the time of evaluation, the characteristic masonry compressive strength was determined based on the assumed mortar and brick compressive strengths according to PTN-Z, and the resulting f_k value was 2.4 MPa. The shear resistance of the wall was determined from the following equation:

$$N_{us} = L(f_{sko} + 0.4\sigma_d)d/\gamma_m \tag{2}$$

where f_{sko} is the characteristic shear strength for masonry, σ_o is the minimum compression stress determined due to loading effects, d is wall thickness, and γ_m is partial safety factor for masonry. Note that, in the absence of experimental data, f_{sko} values are prescribed by the code based on the type of masonry unit and mortar properties.

Note that the PTN-Z equation for shear resistance is somewhat similar to the equation prescribed by Eurocode 6 (Cl.6.2) [26], as follows

$$V_{Rd} = \frac{f_{vk} t L_c}{\gamma_m} \tag{3}$$

where f_{vk} is the characteristic shear strength for masonry, L_C is the wall length resisting shear, t is thickness of the wall resisting shear, and γ_m is partial safety factor for masonry. Note that f_{vk} can be determined from the following equation:

$$f_{vk} = f_{vko} + 0.4\sigma_d \tag{4}$$

where f_{vko} is characteristic initial compressive strength of masonry under zero compression stress, and σ_d is design compression stress.

A comparison of capacities calculated according to the two codes was made for a longitudinal wall along gridline 6 and transverse wall along gridline H, as shown in Figure 11. The capacities are summarised in Table 7. The following input parameters were used for verification of longitudinal wall:

$$\mathrm{PTN-Z:}\ \ f_{sko} = 0.3\ \mathrm{N/mm^2}\ \sigma_o + 0.034\ \mathrm{N/mm^2}\ d = 250\ \mathrm{mm}\ L = 5.62\ \mathrm{m}\ \gamma_m = 2.5$$

$$\mathrm{Eurocode\ 6:}\ \ f_{vko} = 0.3\ \mathrm{N/mm^2}\ \sigma_d + 0.72\ \mathrm{N/mm^2}\ t = 250\ \mathrm{mm}\ L_C = 5.62\ \mathrm{m}\ \gamma_m = 1.5$$

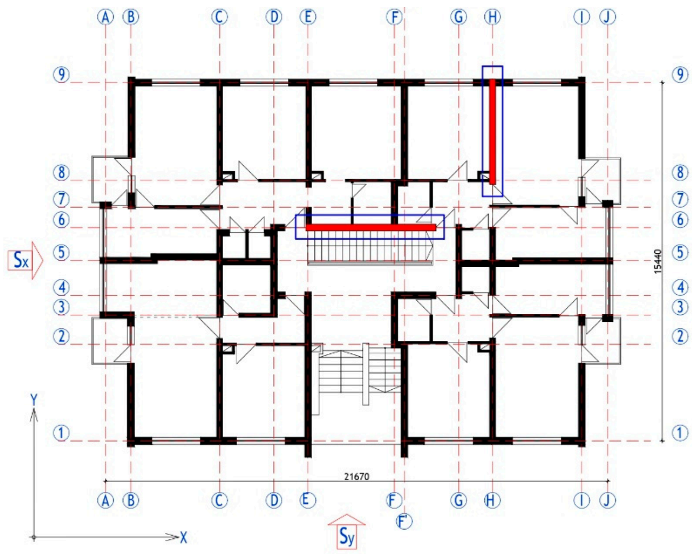

Figure 11. A plan view showing the walls used for the code comparison.

Table 7. Verification of shear capacity for longitudinal and transverse direction (selected walls).

	Longitudinal (X) Direction			Transverse (Y) Direction		
Design Code	**Shear Force E_d (kN)**	**Shear Capacity V_{rd} (kN)**	**E_d/V_{rd}**	**Shear Force E_d (kN)**	**Shear Capacity V_{rd} (kN)**	**E_d/V_{rd}**
PTN-S	181	177	1.02	68	177	0.38
EC8 (Type 1 spectra)	431	295	1.46	149	295	0.50
EC8 (Type 2 spectra)	534	295	1.81	195	295	0.66

It can be concluded from the table that, although there is a significant difference in the magnitude of seismic forces, the results show that the longitudinal wall is deficient because the ratio of seismic shear demand and capacity (E_d/V_{rd}) exceeds 1.0. On the other hand, the transverse wall meets the code requirements because the ratio is less than 1.0. These results are in line with the seismic performance of the building in the 2010 Kraljevo earthquake, because only the walls in longitudinal direction experienced damage.

7. Conclusions

Low- and mid-rise URM buildings constitute a significant portion of the housing stock in Serbia and neighbouring European countries. Multi-family URM buildings (usually 3- to 5-storey high) are of particular concern due to higher occupancy and vulnerability to earthquake effects. Recent earthquakes in Croatia caused damage to these buildings and some of them had to be vacated after these earthquakes. This paper presents the results of a seismic assessment study of mid-rise URM buildings in the context of seismic codes from the former Yugoslavia, which were enforced at the time of the original design, and Eurocode 8, which is currently used for seismic design and assessment of buildings in Serbia and the region. A qualitative overview of the seismic design provisions contained in the PTN-S and Eurocode 8 has identified differences between the two codes.

One of the focal points of the study was the Wall Index (*WI*) provision for seismic design of simple masonry buildings according to Eurocode 8, Part 1, but in this paper, it was explored in the context of seismic assessment of existing buildings. The authors studied 23 low-rise URM buildings damaged in the 2010 Kraljevo, Serbia earthquake, to examine a relationship between the *WI* and the extent of earthquake damage. The following conclusions have been derived based on the study:

1. Normalised *WI/n* values for seven buildings that experienced very heavy structural damage (grade K5) were in a broad range, from 1.0 to 3.0%, but the average value was 1.9%. The majority of the buildings (16 out of 23) experienced heavy damage (grade K4) in the earthquake. A large scatter was observed (*WI* values ranged from 1.0 to 5.0%), but 90% values were less than 2.9%. It appears that there is no significant difference in *WI* values for buildings with damage grades K4 and K5. This could be explained by similar characteristics of these damage grades; hence, it would be reasonable to treat them as a single damage grade, as proposed by other researchers [41].
2. Based on the results of the study on 23 buildings, it can be concluded that more than 90% of buildings had *WI/n* value of 2.9% or less. This value is comparable to the minimum *WI/n* value of 2.5% prescribed by Eurocode 8 for simple buildings, at sites characterised by the same seismic hazard level (a_gxS < 0.2 g). Note that the Eurocode 8 *WI* limit is intended for two-storey buildings. The corresponding limit for three-storey buildings is not available because construction of these buildings is not permitted at sites characterised by high seismic hazard.
3. According to Eurocode 8 provisions, the required *WI* value for a specific building depends on the seismic hazard level, number of storeys, and the masonry construction technology (unreinforced, confined, etc.). The *WI* value also depends on the mechanical properties of masonry, but the fixed minimum value was considered in the current code. In order to facilitate the application of *WI* for seismic assessment of existing simple masonry buildings, a complete set of recommended *WI* values is needed for a range of seismic hazard parameters, building heights, combined with different masonry technologies and corresponding mechanical properties of masonry materials.

Provisions of past seismic design codes from former Yugoslavia have been applied for seismic assessment of a typical multi-family URM building, which was damaged in the 2010 Kraljevo, Serbia earthquake, and was included in the study on 23 buildings. The following findings have been derived based on this case study:

4. The total horizontal seismic force calculated based on the 1981 Yugoslav seismic code (PTN-S) is significantly smaller than the corresponding force calculated based on the Eurocode 8, Part 1: the difference for the Kraljevo site (seismic intensity zone VIII according to the PTN-S code) is by more than 100%, even after the partial safety factors were applied for both codes.

5. The difference in dynamic characteristics of the structure obtained from the two codes, such as mass and fundamental period, are insignificant. For example, modal mass obtained according to the PTN-S code is by only 4% higher than the corresponding mass obtained from Eurocode 8. The difference in modal mass values are due to different load factors for variable loading according to PTN-S (0.5) and Eurocode 8 (0.15).
6. The design did not meet the minimum *WI* requirements for simple masonry buildings based on Eurocode 8, Part 1, for walls in longitudinal direction. This finding is in line with the observed damage in this building after the 2010 Kraljevo earthquake.
7. The shear resistance check was performed according to the two codes for two selected walls—one in the longitudinal direction and the other in the transverse direction. The results showed that, in spite of the different magnitudes of internal forces, the conclusions were the same: the longitudinal wall under consideration was deficient in terms of its shear resistance, while the shear resistance of transverse wall was adequate.

The study performed in the paper has a few limitations. First, the sample (23 buildings) may be too small for a statistic analysis. Second, all buildings examined in the study experienced a similar extent of earthquake damage (damage grade). It would be desirable to consider buildings characterised by different damage grades, including undamaged buildings. Although the study was performed using limited data, it is believed that the results are relevant and contribute to a scarce knowledge base related to post-earthquake assessment of low- to mid-rise URM buildings in Serbia and neighbouring countries.

Author Contributions: Conceptualization, P.B. and S.B.; methodology, P.B. and S.B.; validation, R.C.; formal analysis, P.B.; investigation, S.B., P.B. and R.C.; resources, P.B. and S.B.; writing—original draft preparation, S.B. and P.B.; writing—review and editing, R.C.; visualization, P.B. and S.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 are grateful to the City Administration of the City of Kraljevo for providing information related to the buildings affected by the 2010 Kraljevo earthquake.

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

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