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Seismic Design of a Typical Mid-Rise Residential Building in Serbia Using Confined Masonry and Reinforced Concrete Frame Systems

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Abstract: Masonry has been widely used for the construction of residential buildings in Serbia and the majority of European countries. Confined masonry (CM) is a contemporary masonry technology that consists of load-bearing masonry walls enclosed in lightly reinforced horizontal and vertical reinforced concrete (RC) confining elements. CM has been widely used for the construction of low-rise and mid-rise residential buildings in Serbia and the region (Yugoslavia) since the 1960s. The design case study of a typical multi-family residential building located in Niš, Serbia (the third-largest urban center in the country), is discussed in this paper. This building was initially designed as a five-story CM structure in accordance with the 1981 Yugoslav seismic design code PTN-S, which was enforced in Serbia until 2019, when the Eurocode was adopted for official seismic design codes. Due to architectural constraints, the original design solution involving the CM system was not compliant with the code; hence, an alternative design using an RC-frame system with masonry infills was adopted. A comparison of two different design solutions provides insight into the different requirements of seismic design codes that have been used in the region. It is important to observe that seismic forces for RC structures determined in accordance with the PTN-S code are considerably lower compared to the ones determined according to EC 8-1, with the ratio ranging from 0.37 to 0.69. The seismic shear force according to Eurocode 8 is 1.46 times higher than the force that was used for seismic design according to the PTN-S code in the case of RC-frame structures. The results of an analysis of CM structures show that the seismic shear force in accordance with Eurocode 8 is almost 2.6 times higher than the force that was used for seismic design in accordance with the PTN-S code. The findings of this study are believed to be useful for understanding the difference in seismic design solutions for previous seismic design codes (which were used in the region for more than 40 years) and the present codes (Eurocodes).

Keywords: confined-masonry buildings; seismic design; residential buildings; Eurocode 8; masonry infills; Yugoslav seismic design code; reinforced concrete frame system

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1. Introduction

Masonry has been used for the construction of residential buildings in Serbia and other Balkan-region countries for many centuries. Solid clay bricks were used for the construction of unreinforced masonry (URM) buildings in Serbia from the second half of the 19th century until the 1950s. After WWII, reinforced concrete (RC) became prevalent for mid- and high-rise building construction, but masonry was still used for the construction of low-rise single-family buildings and mid-rise multi-family residential buildings. According to the 2011 Census of Serbia [1], more than 70% of housing units in Serbia were built between 1946 and 1990, that is, during the period when Serbia was a part of Yugoslavia (SFRY). Since the 1960s, confined masonry (CM) has evolved as a modern masonry construction technology;

hence, the URM construction practice in the country was gradually discontinued. At the same time, modular clay blocks with vertical perforations phased out the use of traditional solid clay bricks in masonry construction practice. CM buildings account for an important fraction of the building stock in Serbia and other countries in the region, and they are the focus of the study presented in this paper.

CM buildings have a load-bearing masonry wall system in which the walls are constructed first, one floor at a time, followed by the cast-in-place RC tie columns. Finally, RC tie beams are constructed on top of the walls, simultaneously with the floor/roof slab construction. The key structural components of a CM building, as shown in Figure 1, are as follows: (i) load-bearing masonry walls, (ii) horizontal and vertical RC confining elements (tie beams and tie columns), (iii) RC floor and roof slabs, and (iv) a foundation. RC confining elements, particularly tie columns, are the most important structural features of CM buildings. These tie columns provide confinement to masonry walls and protect them from extensive damage and collapse due to moderate-to-strong earthquakes.

Figure 1. Key components of a typical CM building from the Balkan region (Petrinja, Croatia).

CM construction has been practiced in the Balkan region, particularly within the territory of Yugoslavia [2], since 1964, when the first seismic design code, PTP-12 [3,4], was published. A subsequent version of the code, PTN-S, was issued in 1981 [5], and it was enforced within the territory of Yugoslavia until the 21st century, although the breakdown of the country occurred in 1991. In Serbia, North Macedonia, Bosnia and Herzegovina, and Montenegro, PTN-S was followed for more than 30 years, until the Eurocodes were adopted as the official codes for the design of building structures. For example, Eurocode 8—Part 1 (EC8-1) [6], pertaining to the seismic design of new structures, has recently been adopted in Serbia (SRPS EN 1998-1/NA:2018) [7], while Eurocode 6 (EC 6) [8] has been adopted as an official code for the design of masonry structures (SRPS EN 1996-1-1:2016). It should be noted that extensive experimental research studies on the seismic behavior of CM buildings characteristic of the local construction practice in SFRY have been conducted by Prof. Miha Tomaževič and his collaborators in Slovenia since the 1980s, e.g., [9,10]. An overview of seismic design regulations for buildings in accordance with the Yugoslav design code has been presented elsewhere [11].

The territory of Serbia is characterized by a moderate seismic hazard, and it has a history of numerous damaging earthquakes. In the last 15 years, a few damaging earthquakes hit the Balkan region and exposed CM buildings to the effects of ground-shaking. On 3 November 2010, a magnitude (M_L)-5.4 earthquake affected the city of Kraljevo, Serbia, and the neighboring villages. The earthquake caused two fatalities and affected more than 6000 buildings. It was estimated that 25% of all damaged buildings experienced severe damage and had to be vacated [12]. The majority of the damaged buildings were of URM construction, and the damage was attributed to flexible floor/roof diaphragms and the poor quality of materials and construction (among other reasons) [13,14]. By and large, modern CM buildings performed well, provided that the RC confining elements were adequate, particularly the tie columns. Figure 2 shows a damaged single-family residential building in a village near Kraljevo, in which shear failure of the RC tie columns at the ground-floor level was observed. The RC tie columns did not have transverse reinforcement (ties), and the modular clay blocks had horizontally aligned perforations, which is an unacceptable practice for load-bearing masonry construction.

Figure 2. Rural single-family CM building damaged due to the 2010 Kraljevo, Serbia, earthquake, showing a failure of the RC tie column at the ground-floor level.

On 26 November 2019, an earthquake with a magnitude (Mw) of 6.4 hit Albania, causing 51 fatalities and about 3000 injuries. Durrës, the second largest city in Albania, was the most severely affected. In general, mid-rise masonry buildings performed well in the earthquake (including URM and CM buildings), with the exception of buildings with precast hollow core concrete slab floors. CM construction had been practiced in Albania since the 1980s, but this technology was replaced with an RC-frame system at the beginning of the 21st century. CM buildings performed well in the earthquake, except for the buildings with a mixed system that comprised RC frames at the ground-floor level and a CM wall system in the upper parts of the buildings (Figure 3a) [15,16].

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

Figure 3. Damage to CM buildings in recent earthquakes in the Balkan region: (**a**) the collapse of a building with a mixed RC and CM system due to the November 2019 Albania earthquake and (**b**) the damage to a masonry building with URM walls at the ground-floor level and the CM walls above due to the December 2020 Petrinja, Croatia, earthquake. ((**a**): Red elipse shows damage of the loggia at the ground floor level; (**b**): Red elipse shows vertical tie-column elements are omitted. That is reason of the high level damage of CM structure, in this case).

On 29 December 2020, an earthquake with an epicenter close to Petrinja, Croatia (Mw = 6.4), caused seven fatalities, and 26 people were injured. The earthquake most severely affected older low-rise URM buildings with flexible wooden floors, while modern

CM buildings generally performed very well, in most cases without any damage. However, some single-family houses with URM walls at the ground-floor level and a CM system at the upper floors experienced damage at the ground-floor level, where vertical RC confining elements were missing (Figure 3b) [17].

This paper discusses the features of a 60-year-long CM construction practice in Serbia and the former Yugoslavia region and the relevant design code provisions. Subsequently, a design case study featuring a typical mid-rise multi-family residential building located in Niš, Serbia, is presented. The building is five stories high, and it was initially designed as a CM structure in accordance with the 1981 seismic design code PTN-S, which was followed within the territory of Yugoslavia (SFRY). The seismic analysis and design of buildings were also performed according to Eurocode 8, which is currently the official seismic design code in the country. A comparison of the analysis and design results for these two codes is presented and discussed. Since the building was ultimately designed and constructed as an RC building, the analysis and design of an RC building were also performed according to local codes. A comparison of the design solutions for CM and RC buildings is also presented in this paper.

2. An Overview of Seismic Design Provisions for CM- and RC-Frame Buildings

2.1. Seismic Analysis and Design Provisions for CM Buildings

2.1.1. A Summary of the Relevant Provisions per Local Seismic Design Codes and Eurocodes

In Serbia and its neighboring countries, the seismic design of masonry structures was governed through the applicable technical regulations, starting in 1949 [18] and ending in 1991, with the last code issued before Yugoslavia fell apart [19,20]. According to these codes, masonry buildings were classified as (i) URM buildings, (ii) CM buildings, or (iii) reinforced masonry buildings. Since 1949, URM buildings had been required to have horizontal RC confining elements at the floor and roof levels; hence, the main difference between URM and CM buildings is the provision of vertical confining elements in CM buildings. Reinforced masonry consists of masonry units and steel reinforcing bars that are embedded in mortar or cement-based grout so that all the materials act together in resisting forces. Note that the PTP-12 code permitted the construction of six- and five-story CM buildings in Seismic Zones VIII and IX, respectively, but the PTN-S code restricted the CM building height to four and three stories for the same Seismic Zones (VIII and IX).

Section 9.6 of EC8-1 code prescribes that the seismic design of CM buildings needs to be performed according to one of the following two approaches:

1. A prescriptive design approach called "Rules for simple buildings" for regular low-rise CM buildings (up to five floor levels) (Section 9.7 of EC8-1 code);
2. The engineered analysis and design approach requires the verification of seismic resistance of each structural element in a CM building, and the design resistance is to be determined based on the EC 6 requirements; this approach is to be followed when designing CM buildings that do not meet the "Rules for simple buildings".

The rules for simple buildings, as outlined in Section 9.7 of EC8-1 code prescribe the required amount of walls in each horizontal direction of the building plan, which is expressed as a percentage of the floor plan area ($p_{A,min}$), and is referred to as the wall index (*WI*) in this paper. For a specific building design, the *WI* value can be determined as a sum of the cross-sectional areas for all the CM walls in the direction of the considered earthquake action relative to the ground-floor area.

The required *WI* value depends on the number of stories and the seismic hazard level. It is described as a product of design-site acceleration (a_{gS}) and a correction factor, k. The k value ranges from 1.0 to 2.0, depending on the average wall length.

Table 9.3 of EC8-1 code contains *WI* values for all types of masonry buildings. The values in the table were developed with an assumed compressive strength for masonry units (e.g., modular clay blocks) of 5.0 MPa. For a three-story CM building, the minimum required *WI* values range from 2.0 to 4.0% when site acceleration increases from 0.07 to 0.15 g. Note that CM buildings taller than three stories cannot be designed according to these rules at sites with a_{gS} value greater than 0.15 g, but two-story CM buildings can be designed at all sites with a site acceleration of up to 0.2 g.

2.1.2. A Comparison of the Seismic Design Requirements for CM Buildings

Both codes, that is, PTN-S and EC8-1, contain comprehensive seismic design requirements related to CM buildings; however, there are differences in terms of five important requirements, as discussed next.

1. Spatial integrity

The spatial integrity of masonry buildings is critical for their satisfactory seismic performance, and it can be achieved through adequate connections between the floor/roof and wall structures; this requirement is prescribed in the PTN-S code (Cl. 95) and EC8-1 (9.5.1.2). In CM buildings, a system of interconnected (monolithically cast) horizontal and vertical RC confining elements (tie columns and tie beams) ensures their spatial integrity.

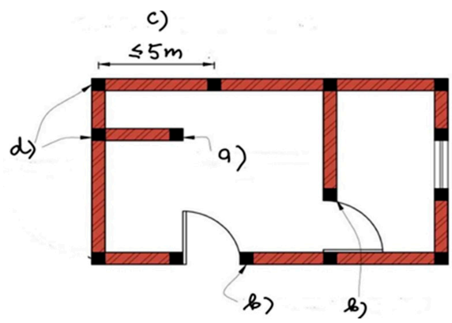
Diaphragm action (also known as "rigid diaphragm" action) is also related to the spatial integrity of masonry buildings, and it influences their seismic performance. The stiffness of a floor system governs the distribution of seismic forces among individual walls at a floor level. For example, solid RC slabs or semi-prefabricated floors consisting of masonry elements and small-size, monolithically cast RC beams (often referred to as "LMT" in Serbia) are most common in contemporary CM buildings. These floor structures are considered to act as rigid diaphragms that distribute horizontal seismic forces to individual walls at each floor level in proportion to their relative stiffnesses. The PTN-S code (Cl. 95) prescribes the rigid diaphragm requirement for all types of masonry buildings. Although EC8-1 (9.5.1.3) does not prescribe the type of floor system, it is required that the floor demonstrates "effective diaphragm action", which implicitly calls for a floor system that acts like a rigid diaphragm.

2. Layout of RC tie columns

The locations and spacing of the RC tie columns in a CM building are very important, and they are prescribed by design codes. PTN-S (Cl. 97) prescribed that RC tie columns needed to be provided at the wall intersections and also at the free ends of the walls with a thickness greater than or equal to 190 mm. The maximum spacing of RC tie columns was limited to 5.0 m for walls with a 190 mm thickness, versus 6.0 m for 240 mm thick walls.

According to EC8-1 (9.5.3.4), RC tie columns should be placed (see Figure 4a) as follows:

(a) At the free edges of each structural wall element;
(b) At both sides of any wall opening with an area of more than 1.5 m. sq.;
(c) Within the wall if necessary to limit the spacing of vertical confining elements to 5 m;
(d) At the intersections of structural walls wherever the confining elements imposed via the above rules are at a distance larger than 1.5 m.



(**a**)

(**b**)

Figure 4. Confining elements in CM buildings: (**a**) a typical floor plan showing the locations of RC tie columns and (**b**) a discontinuous RC tie beam (an example of an inadequate construction practice). (Red elipse shows the discontinuity of horizontal confining elements what is serious structural mistake in this case of CM building).

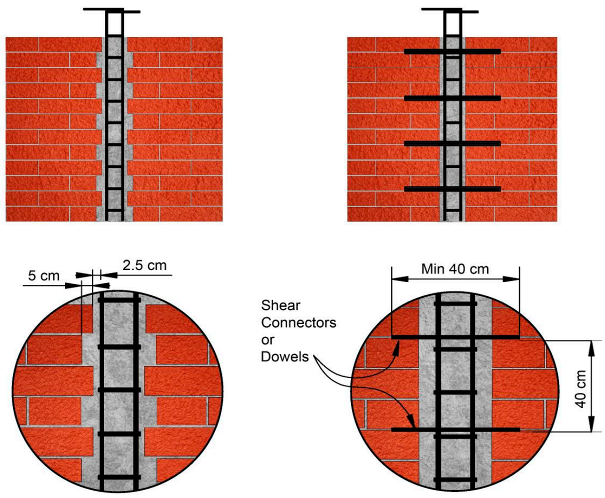
RC tie columns play an important role in controlling the damage to CM walls with openings. The results of experimental research studies and evidence from past earthquakes have confirmed the vulnerability of CM walls with openings. On the other hand, the good performance of CM wall with openings and RC tie columns was reported for the 2020 Petrinja, Croatia, earthquake (see Figure 1). The PTN-S code did not require a provision of RC tie columns for the openings that were less than 2.5 m wide (for Seismic Zones VIII and IX) or up to 3.5 m wide (for Seismic Zone IX). However, RC tie columns had to be provided at the ends of openings with a width of up to 30% larger than the original limit set for unconfined openings. According to EC8-1 (9.5.3.4.b), RC tie columns should be provided at the ends of openings with an area of 1.5 m.sq. It is important to consider a ratio of the opening area and the CM wall panel area (the masonry wall plus adjacent tie columns and tie beams). The openings with an area exceeding 10% of the CM wall panel area may be considered as large and need to be enclosed with RC tie columns. CM wall panels with unconfined openings should not be considered to contribute to the seismic resistance of a CM building.

3. Spacing of RC tie beams

RC tie beams need to be constructed at each floor level and are usually integrated with RC floor slabs. Discontinuous tie beams are not effective and should not be constructed (Figure 4b). The PTN-S code does not prescribe the spacing of tie beams, but it stipulates that RC tie beams must be provided in all walls with a thickness of 190 mm or more. According to EC8-1 (9.5.3.5), the maximum spacing of tie beams should not exceed 4 m.

4. Interface between masonry walls and RC tie columns

CM is a unique masonry construction technology due to the provision of RC confining elements, which are integral parts of CM walls. EC8-1 (9.5.3.1) prescribes that horizontal and vertical RC confining elements need to be joined together and anchored to the elements of the main structural system. An interface between these masonry walls and vertical RC confining elements can be achieved through toothing (Figure 5a). For example, bricks at the ends of CM walls are left in a zig-zag pattern by 25 to 50 mm to provide a "toothed" interface between the masonry and concrete. PTN-S (Cl.97) prescribes toothed connections between RC tie columns and masonry walls. In countries where clay or concrete blocks are used for CM construction, toothing cannot be provided due to vertical (as opposed to zig-zag) interfaces between the walls and adjacent tie columns. In such cases, horizontal reinforcement (dowels) at the wall ends can be extended through the tie columns (Figure 5b). The dowels are usually in the form of small-diameter wire embedded into the mortar bed joints.



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

Figure 5. Wall-to-tie-column interface: (**a**) toothing; (**b**) horizontal steel dowels.

5. Masonry material properties/strength

Since load-bearing walls are the main components of CM buildings, the minimum code requirements related to masonry materials significantly influence both the gravity and the lateral load resistance of individual CM walls and entire buildings.

The compressive strength of masonry elements (bricks or blocks) is very important because it significantly influences the magnitude of masonry compressive strength. According to the PTN-S code (Cl. 109), the minimum compressive strength of clay bricks (solid or hollow) (MO100-MO150) is in the range of 10–15 MPa, and for modular clay blocks (MO150), it is 15 MPa. On the other hand, EC8-1 prescribed a minimum compressive strength of 5.0 MPa, irrespective of the type of masonry element.

In terms of mortar properties, PTN-S prescribed the use of cement–lime–sand mortar for masonry construction at all sites located in Seismic Intensity Zones VII–IX. Mortar grades range from M25-M50, corresponding to the compressive strength of 2.5–5.0 MPa, depending on the type and strength of the masonry elements. On the other hand, EC8-1 prescribes a minimum mortar compressive strength of 5.0 MPa.

Based on the above-presented comparisons and requirements for the design of CM structures according to the PTN-S and EC8-1 codes, i.e., design approaches, the value of the wall index (WI), spatial integrity, the layout of RC tie columns, the spacing of RC tie beams, masonry walls' and RC tie columns' interface, masonry material properties/strength, and the geometrical properties of masonry walls, it can be concluded that recommendations and requirements according to the EC8-1 code are more demanding and strict, and the construction of masonry buildings incurs higher costs. The recommendations according to EC8-1 insist on regular structures, the quality of materials, and the quality of construction, i.e., on the elements related to the ultimate bearing capacity and serviceability of masonry structures. The consequence is that the buildings with CM structures designed according to EC8-1 are characterized by high safety in terms of seismic resistance and durability [21–23].

2.2. Seismic Analysis and Design Provisions for RC Buildings

Until 2022, the structural and seismic design of RC structures in Serbia had to be performed according to the requirements of local codes: the PTN-S code [5] for the seismic design of all new buildings and the PBAB 87 code [24] for the structural design of RC structures (without seismic design provisions). The PTN-S code was significantly more advanced compared to the previous code PTP-12, which was published in 1964 [4]. The development of the PTN-S code started in the 1970s, but its release was expedited due to the 1979 Montenegro earthquake, which significantly affected both masonry and RC structures. In particular, a few major hotels along the Montenegro coast were constructed with RC-frame structures, and they experienced extensive damage or collapse due to the non-ductile design and detailing of their structural elements. The soft-story collapse of the five-story "Slavija" Hotel in Budva, Montenegro, due to the 1979 earthquake is presented in Figure 6.

(**a**)

(**b**)

Figure 6. Collapse of the "Slavija" Hotel buildings in Budva due to the 1979 Montenegro earthquake: (**a**) a collapsed block and (**b**) column damage detail (credit: B. Petrović/SUZI-SAEE).

The PTN-S code was applicable to sites with a seismic intensity of VII to IX. The code specified an equivalent static analysis procedure for the majority of structures. A dynamic analysis procedure was prescribed for special structures, such as tall buildings with more than 25 floors, and irregular structures (e.g., buildings with flexible floors). Designed seismic forces for the equivalent static analysis were based on the building occupancy and seismic design parameters (importance, seismic intensity, soil type, and fundamental period). A coefficient accounting for the type of structural system and the expected ductility was also introduced (note that a similar coefficient did not exist in the 1964 code). The distribution of seismic forces along a building height considered the effect of higher vibration modes for buildings taller than five stories.

The PTN-S code contained specific seismic design and detailing requirements for RC-frame structures. It was required that the lateral load resistance in an RC structure had to be provided in both horizontal directions of a building plan. Another important requirement was related to the ductile design and detailing requirements for RC and steel structures, according to which nonlinear deformations had to be concentrated within plastic hinge zones, which had to be designed with an adequate capacity to sustain these deformations. The PTN-S code contained special provisions for the design and detailing of ductile RC-frame structures, in which plastic hinges were formed at the beam ends, while nonlinear deformations in the columns had to be avoided (the "weak beam-strong column" failure mechanism). As a rule, the stiffness of the beams was lower than the stiffness of the adjacent columns.

Longitudinal reinforcement in beams within the support regions of each span had to be provided at the top and bottom of the section. It was required that the reinforcement ratio for top reinforcement, μ', was at least 50% of the bottom reinforcement ratio, μ, because it was considered that such reinforcement distribution increased the ductility of potential plastic hinges in the beams ($\mu' \geq 0.5\ \mu$). The maximum spacing of transverse reinforcement (stirrups) in beams was limited to 20 cm, while within the support regions of each span (20% of the span length), closely spaced stirrups with a spacing not exceeding 10 cm were required. The stirrups had to be anchored through overlapping along the shorter-beam cross-sectional dimension. The seismic detailing requirements for beams according to the PTN-S code are presented in Figure 7.

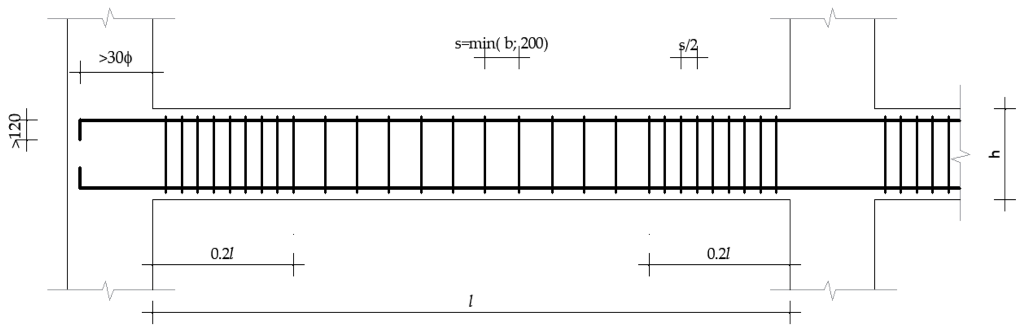


Figure 7. Seismic detailing requirements for beams in RC frames according to the PTN-S code.

Columns in RC-frame structures had to be designed such that the axial compression due to the specified gravity load had to be less than 35% of the concrete compression strength, that is, $\sigma_o/\beta_B \leq 0.35$, where $\sigma_o = P/F$ (P is the axial force, F is the cross-sectional area of the column, and $\beta_B = 0.7\ \beta_k$, where β_k is the cube compression strength of the cube).

The spacing of transverse reinforcement (ties) in columns had to not exceed 15 cm, while in the vicinity of hinges (at a distance equal to the highest of the following values: 1.5 times larger dimension of the cross section and 1/6 of the column height or 50 cm), the tie spacing was reduced by 50%. The anchorage of ties in the columns was achieved with overlapping along the entire length of the shorter side.

In high-rise buildings for which structural analyses were carried out using the dynamic procedure, transverse reinforcement had to be provided continuously through the beam–column joints.

The code also prescribed that longitudinal column reinforcement with a diameter larger than 20 mm had to be continued by means of welding; alternatively, the reinforcement had to be continued over two floors, but 50% of the total reinforcement could be spliced at each floor. The seismic detailing requirements for RC columns according to the PTN-S code are presented in Figure 8.

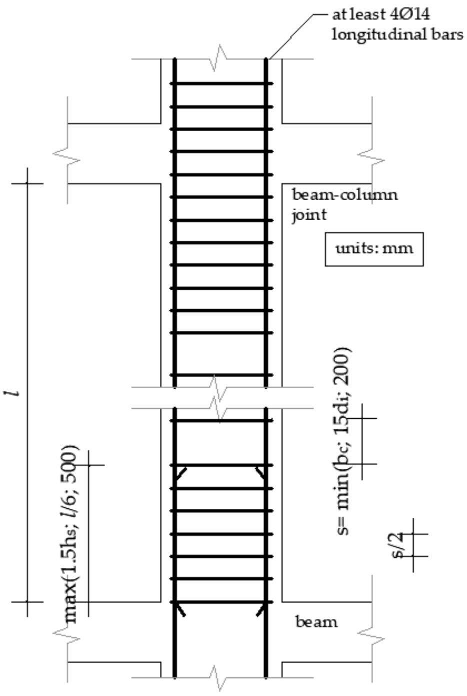


Figure 8. Seismic detailing requirements for columns in RC frames in accordance with the PTN-S code.

The code also prescribed that only lightweight masonry infills were permitted in RC-frame systems. However, when infills were expected to interact with the frame, it was necessary to provide horizontal reinforcement at the infill-to-frame interface in order to ensure the integral action of the entire system. In flexible structural systems, which were characterized by an interstory drift ratio exceeding h/300 (where h is the floor height in cm), experimental testing had to be performed to prove the stability of infills and the extent of damage due to seismic actions. The stability of infills had to be controlled in the out-of-plane direction.

2.3. A Comparison of the Ductility Requirements for RC-Frame Structures According to Different Codes

This section presents a comparison of the PTN-S code and EC8-1 from the perspective of ductility. The PTN-S code did not explicitly define the value of the behavior factor, q; hence, the level of design loading was the same for all RC-frame structures.

EC8-1 prescribes three ductility classes: low—DCL (q = 1.5–2.0), medium—DCM (q < 4.0), and high—DCH (q > 4.0). As a result, the design seismic loading in accordance with EC8-1 is two to four times higher than the one determined based on the code, depending on the ductility class. Note that, in the areas characterized by longer earthquake return periods, a higher ductility coefficient reduces the differences in seismic loading.

The choice of a ductility class and the value of the behavior factor, q, may significantly affect the level of the seismic design load. A lower design load implies the required higher ductility class, notable nonlinear deformations, and stricter requirements for structural details.

The consequence is a more obvious post-elastic deformation, especially in plastic hinge zones. This is the concept of ''lower standards'' for structural safety under seismic activity, characterized by lower construction costs corresponding to the economic prosperity of a society. A lower seismic loading level according to the PTN-S code, compared to EC8-1, should be addressed with more detailed structural requirements in order to provide the required ductility.

The PTN-S code prescribed general rules for the ductile behavior of a structure by limiting the axial compression in columns and through reinforcement-detailing rules.

The PTN-S code prescribed the same behavior factor (ductility coefficient K_p) for all structural systems, that is, the same structural requirements had to be satisfied, irrespective of ductility. K_p = 2.0 was assigned for buildings with flexible floors. The value was almost the same as the design loading for the high-ductility systems in accordance with EC8-1. The highest level of design loading in accordance with the PTN-S code (required for irregular structures) is equal to the lowest loading level in accordance with EC8-1 (permitted for regular structures).

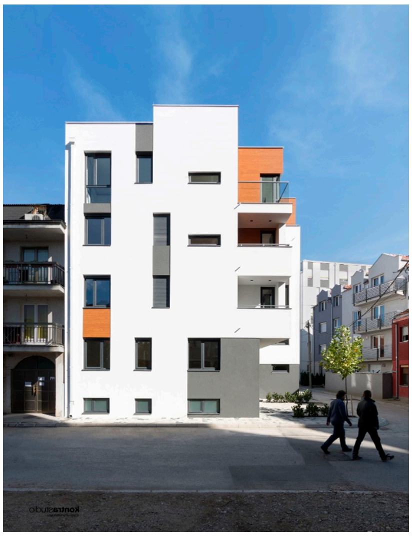
The easiest way to define the ductility level is in terms of the displacement ductility ratio, as the maximum displacement divided by the displacement at the first yield.

The lateral displacement at the top of a building is limited to H/600, in accordance with PTN-S, corresponding to the maximum interstory slope with a value of 1/300. For example, with a behavior factor of q = 10, the displacement of the top of a building is ten times higher, 1/30. When the allowed slope according to EC8-1 is 0.004–0.006, it means that, according to the PTN-S code, the allowed slope was five to eight times higher. Given that the values of seismic displacements depend on elastic stiffness, it means that the minimum required stiffness of the structure according to EC8-1 is a few times higher than according to the PTN-S code requirement. Unlike EC8-1, the PTN-S code prescribed only limits for elastic design displacement.

3. Design Case Study: Building Description and Consideration of a Structural System

3.1. Background

This study was motivated by a design project for a multi-family residential building located at Car Uroš Street No. 8 in Niš, Serbia (Figure 9). The residential building was erected in 2015 as a five-story building with a half-basement and a half-floor at the top. The lower four floors have a plan with a 14.50 m length and a 12.60 m width. The half-floor at the top has smaller plan dimensions (an 8.90 m length and a 5.20 m width). The typical floor height is 2.90 m, while the total building height is 15.30 m.



(**a**)

(**b**)

Figure 9. Case study building in Niš, Serbia: (**a**) south facade 1; (**b**) southeast facade.

The building was originally designed in 2015 as a CM structure, but the design calculations in accordance with the Serbian seismic design code PTN-S showed insufficient seismic resistance for multiple load-bearing walls. Note that the architectural design envisaged a wall thickness of 20 cm, which is less than the minimum values set by EC8-1 for masonry structures built in seismically active regions. Moreover, the building height (five stories) exceeded the height limitation set by the PTN-S code for CM buildings in Seismic Intensity Zone VIII (four stories). The architectural design envisaged an irregular building in both its plan (horizontal irregularity) and elevation (vertical irregularity). The building is irregular in elevation because it consists of two parts which are connected through a staircase, with the floor slabs offset vertically by approximately a ½-story height. Another tyoe of irregularity is due to a change in plan dimensions over the building height, that is, the presence of overhangs above the ground-floor level; see Figure 9. These irregularities contributed to increased seismic forces in the structural elements.

The results of the seismic design calculation for CM structures according to both codes (PTN-S and EC8-1) showed that the design solution was inadequate concerning the aspect of seismic resistance. Therefore, the structural system of this building was changed to an RC frame with masonry infills. The following sections of the paper present a seismic analysis and verification of seismic resistance for the key structural elements of the building according to the ultimate limit states (ULS) design approach.

3.2. Structural Systems Considered in the Study

Alongside two structural systems that were originally considered in the project, an additional two systems were considered in the present study in order to draw a comparison for seismic design results across different masonry building typologies and different codes (PTN-S and EC8-1). The following four models were considered in this study: (i) an RC frame with masonry infills (Type A); (ii) a load-bearing CM wall system (Type B), (iii) a load-bearing masonry wall system with horizontal confining elements (Type C), and (iv) a URM wall system (Type D).

The building was originally designed as a CM structure. The cross-sectional dimensions of the vertical RC confining elements ranged from 20 cm × 20 cm to 25 cm × 25 cm (square), while the horizontal RC confining elements had a width that matched the wall thickness (20 cm), but their depth was larger (25 cm). These horizontal elements were integrated with the floor system, which was in the form of a ribbed RC slab. Masonry walls were constructed using 20 and 25 cm modular clay blocks in cement–lime–sand mortar (mix composition 1:3:9). The masonry compressive strength of 5 MPa was considered in the design. The ground-floor layout is shown in Figure 10. The same mechanical and geometrical properties of masonry walls were considered in this study for Type C and D models.

The building was subsequently redesigned as an RC-frame system with column cross-sectional dimensions 25 cm × 40 cm and beams with similar dimensions. The concrete grade was C25/35. The infill walls were constructed using 20 and 25 cm modular clay blocks laid in cement–lime–sand mortar with a mix composition of 1:3:9. The compressive strength of masonry was assumed as 5 MPa. The infill walls, given the contemporary requirements for design in seismically active areas, did not make structural or load-bearing contributions. A comparison between the RC moment-resisting frame and the RC frame with infills that participate in the transfer of seismic forces is presented later. The floor and roof structures are ribbed RC slabs. A ground floor plan showing a layout of the RC columns and beams and a vertical section are shown in Figures 11a and 11b, respectively.

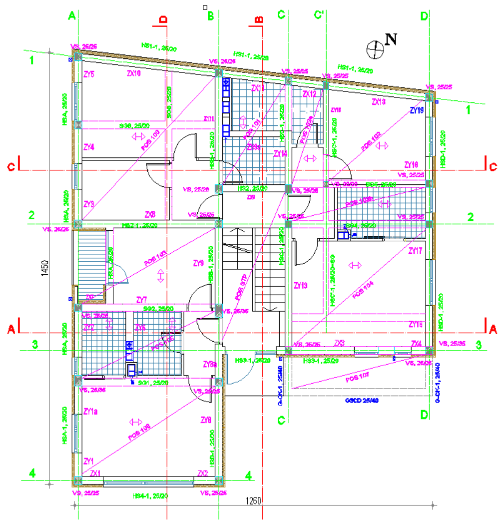
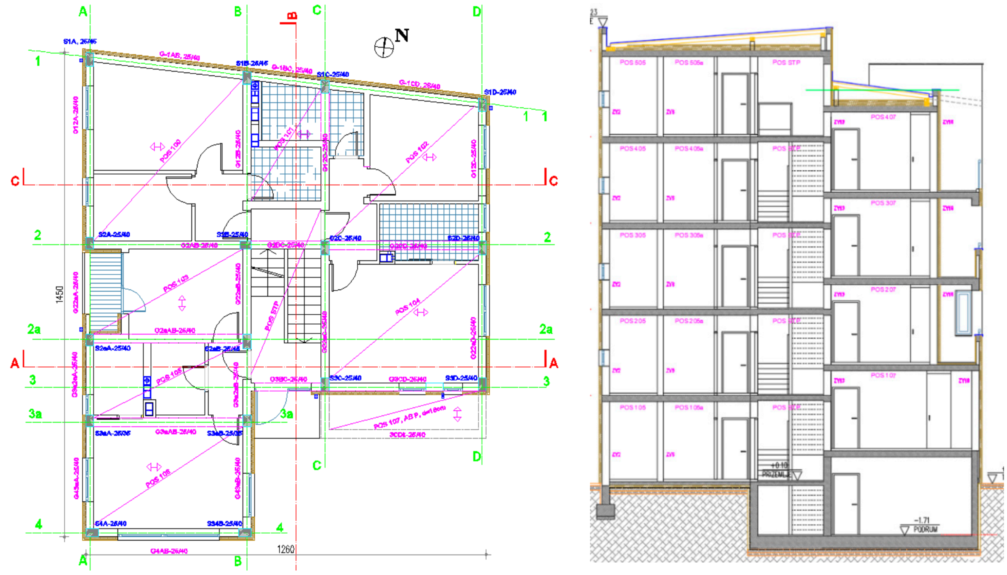


Figure 10. Ground-floor layout for the case study building—CM solution. (This is layout of the structural elements of the CM structure. Red color: Position of the vertical sections; Letters in green color: marked horizontal confining elements of the CM structure in X- and Y directions; construction directions: 1–1, 2–2, 3–3. . .A–A, B–B. . .D–D; Signs and letters in magenta color: position of vertical confining elements, marked walls of the CM structure in X- and Y directions, floors (POS 103, POS 105. . .)).



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

Figure 11. Case study building—RC-frame solution: (**a**) plan at the ground-floor level and (**b**) vertical section A–A. (This is RC frame structure. Letters and signs in blue color: RC columns of RC frame structure; magenta color: RC beams of RC frame structure and floors).

4. Seismic Analysis and Design

4.1. Input Parameters and Seismic Force Calculation

Equivalent static analysis was performed for a seismic assessment of different structural systems considered in the case study in accordance with the PTN-S and EC8-1 codes. The fact that this is a building with rigid diaphragms was taken into account, so seismic forces at the floor level were distributed to individual walls proportionally to their stiffness.

The load combination in accordance with the PTN-S code is

$$\Sigma G_k + \Sigma Q_k + \Sigma Q_s \tag{1}$$

where G_k is the self-weight of the structure, Q_k is the variable load, and Q_s is the seismic load.

The load combination in accordance with the EC8-1 code is

$$\Sigma G_k + 0.3\Sigma Q_k + \Sigma A_{Ed} \tag{2}$$

where A_{Ed} designates design loading due to earthquake action.

An equivalent static seismic analysis was initially performed in accordance with the seismic design code PTN-S. Seismic forces were determined considering the following parameters: a building category coefficient of K_o = 1.0 (corresponding to Category I), a seismic intensity coefficient of K_s = 0.05 (Seismic Intensity Zone VIII), a dynamic response coefficient of K_d = 1.0, and a ductility coefficient of K_p = 1.6 for CM structures, K_p = 2.0 for unreinforced masonry, and K_p = 1.0 for RC-frame structures. The soil category was II.

A response spectra seismic analysis was completed for all models according to the requirements of EC8-1 discussed earlier in the paper. The Type 1 design spectra [25] for Niš were developed. The design ground acceleration for soil Type A was 0.1 g, while soil Type C was considered for the site. Spectral accelerations for the elastic design spectrum $S_d(T)$ according to EC8-1 were divided by the behavior factor q = 2.4 for CM structures, q = 2.4 for unreinforced structures, and q = 3.9 for RC-frame structures. The seismic hazard setting for the construction site led to the selection of the Type 1 spectrum. The design spectra for Niš, Serbia, based on the PTN-S and EC8-1 codes are presented in Figure 12.

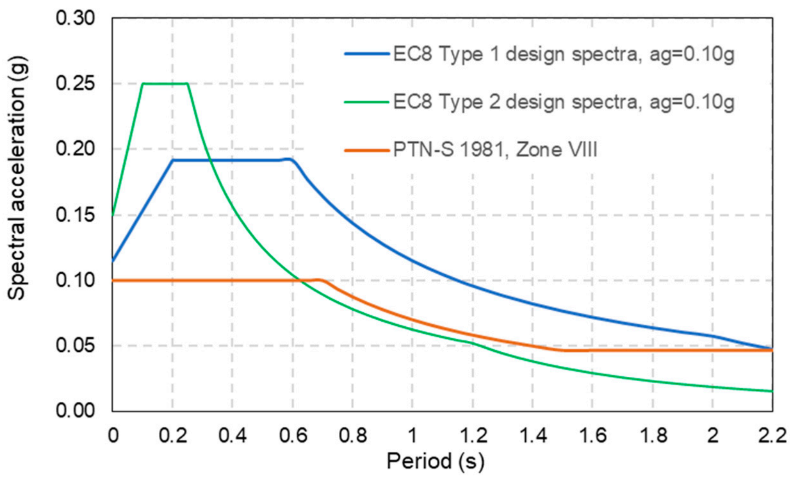


Figure 12. Design spectra: Type 1 and Type 2 for Niš, Serbia, according to the EC8-1 (a_g = 0.1 g, ground Type C) and PTN-S codes (zone VIII).

Seismic base shear forces for different types of structures, calculated in accordance with the PTN-S and EC8-1 design procedures, are summarized in Table 1. It is important to observe from the table that the seismic forces in accordance with the PTN-S code are considerably less compared to the ones determined according to EC 8-1, with the ratio ranging from 0.37 to 0.69.

Table 1. Seismic base shear forces multiplied by a partial safety factor.

Type of Structure	Code	Seismic Base Shear Force (kN)		Total Seismic Weight (t)
		Direction		
		X	Y	
A (RC frame with masonry infills)	PTN-S	476.31	483.07	1122.51
	EC 8-T1 (Type 1 spectra)	695.56	695.56	1122.51
	PTN-S/EC8-T1	0.68	0.69	1.00

Table 1. *Cont.*

Type of Structure	Code	Seismic Base Shear Force (kN) Direction X	Seismic Base Shear Force (kN) Direction Y	Total Seismic Weight (t)
B (CM)	PTN-S	553.56	553.56	1231.86
	EC 8-T1 (Type 1 spectra)	1445.40	1445.40	1231.86
	PTN-S/EC8-T1	0.38	0.38	1.00
C (URM with horizontal confining elements)	PTN-S	525.17	525.17	1173.16
	EC 8-T1 (Type 1 spectra)	1378.20	1378.20	1173.16
	PTN-S/EC8-T1	0.38	0.38	1.00
D (URM)	PTN-S	499.99	499.99	1121.51
	EC 8-T1 (Type 1 spectra)	1317.50	1317.50	1121.51
	PTN-S/EC8-T1	0.37	0.37	1.00

4.2. Numerical Models

For the purpose of seismic analysis, 3D numerical models for masonry structures (Types B, C, and D) and RC-frame structures (Type A) were developed for the case study building. Shell elements were used to model masonry walls. Floors were considered rigid diaphragms and were created as plate elements, while beams, columns, and confining elements were created as frame elements; see an isometric view in Figure 13. The foundation structure was simulated through fixed-base restraints.

It is important to note that the numerical modeling and analysis of the CM walls is a challenging task. The available numerical approaches for modeling CM walls based on the finite element method (FEM) can be classified into micro and macro models. Micro models are mainly used for parametric studies to investigate the influence of key design parameters on the seismic response of CM walls, i.e., the tie column size and reinforcement, the wall aspect ratio, the concrete-to-masonry interface, and the axial stress level, on the seismic behavior of CM walls [26–28]. However, their proper consideration requires sophisticated computational techniques, which are usually not practical for designers. Furthermore, although toothed wall-to-tie-column interface with tie columns has been shown to improve the seismic performance and post-peak behavior of CM walls [29,30], toothed interfaces are rarely used in Serbia due to the use of modular clay blocks for wall construction, in Serbia.

The modulus of elasticity of the masonry used to develop the wall element model was 4000 MPa (corresponding to a masonry characteristic compressive strength of 5.0 MPa).

The values of the concrete modulus of elasticity used to develop the RC-frame elements of the Type A building and the concrete confining elements of the Type B and D models was 21,000 MPa (the characteristic compressive strength of concrete is f_{bk} = 30 MPa).

The ''Tower'' finite element software package (TOWER-3D Model Builder 8.5-x64 Edition 8547) was used for numerical modeling and the seismic analysis of the four different types of building structures [31].

Analysis was performed using the Equivalent Frame Model (EFM) [32].

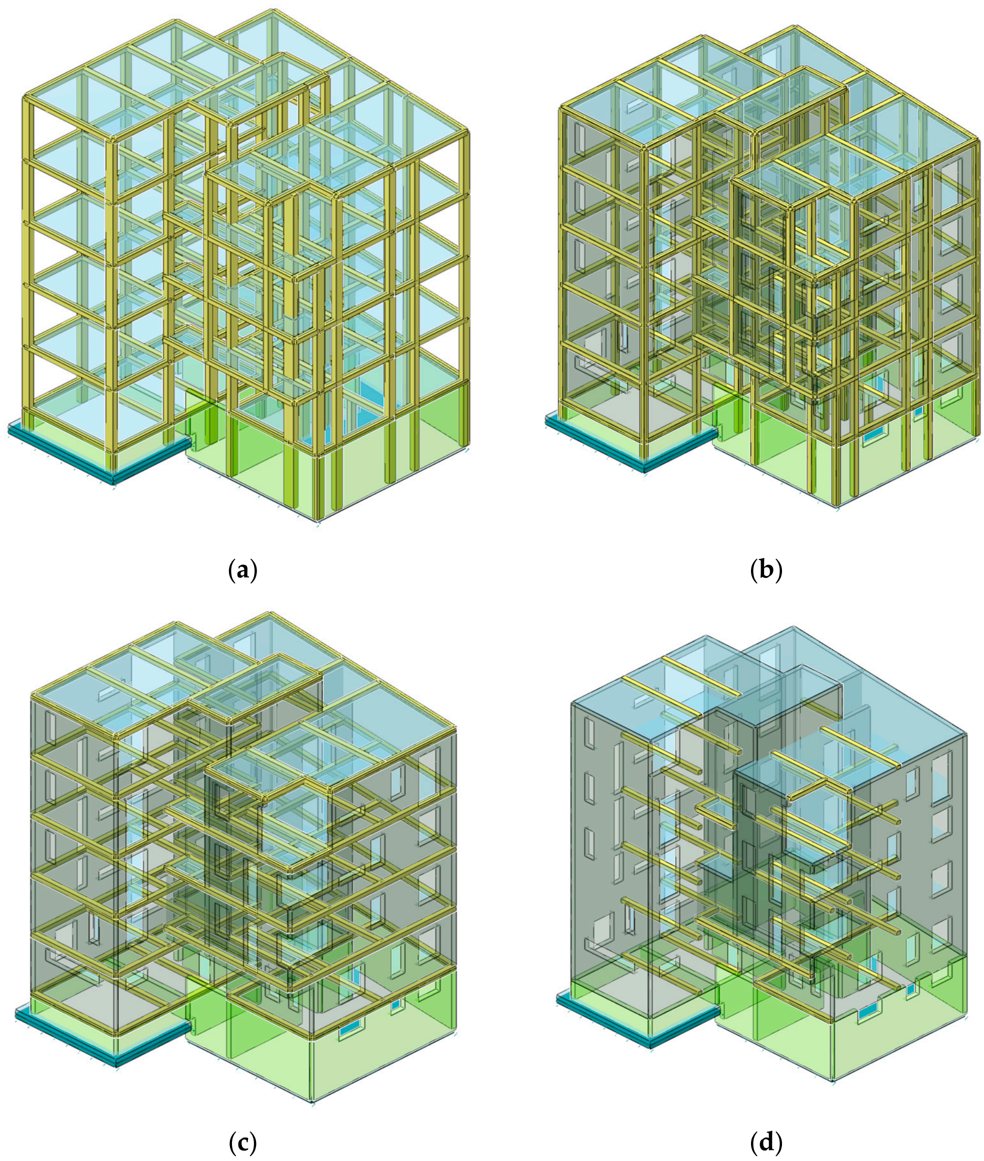


Figure 13. Numerical models for different structural systems: (**a**) Type A; (**b**) Type B; (**c**) Type C; and (**d**) Type D.

4.3. Modal Analysis—Dynamic Characteristics of the Models

In accordance with the PTN-S code, the fundamental period could be determined either via modal analysis or empirical formulas (however, no specific formulas were included in the code).

According to the EC8-1 code for buildings up to a 40 m height, the fundamental period can be determined according to the following approximate equations:

$$T_1 = C_1 H^{3/4} \tag{3}$$

where

$$C_1 = 0.075\sqrt{A_C} \tag{4}$$

and

$$A_C = \sum\left[A_i\left(0.2 + \left(\frac{l_{wi}}{H}\right)\right)^2\right] \tag{5}$$

Note that H denotes the overall building height, Ac is the total effective area of walls in the direction under consideration (expressed in m^2), A_i is the effective cross-sectional area of wall i at the ground-floor level (expressed in m^2), and l_{wi} is the length of a wall in the direction under consideration.

The modal analysis results identified the dynamic properties of different numerical models of the case study building. Table 2 presents the values of the fundamental period for all the building models in both directions (X and Y). A comparison of the fundamental period values for the building models showed that the fundamental period for Model A is higher compared to the other models because the RC-frame structure (Model A) is more flexible, and seismic loading acts only on the concrete frame. Infill walls have only a nonstructural function, and their contribution to the seismic resistance was not taken into account. The Model B structure has fundamental period values that are 10–15% lower compared to Models C and D. On the other hand, identical fundamental periods for the Type C and D structures—that is, an unreinforced masonry structure with horizontal confining elements and an unreinforced masonry structure without any confining elements, respectively—indicate that the effect of horizontal confining elements on the lateral stiffness is not significant for buildings with rigid diaphragms.

Table 2. Fundamental periods for the different types of case study building structures (sec).

	Model 1	**Model 2**	**Model 3**	**Model 4**
	A	B	C	D
X-direction	0.73	0.24	0.28	0.28
Y-direction	0.72	0.23	0.26	0.26

4.4. The Results: Seismic Demand and Overall Resistance

The results of the seismic analyses facilitated a comparison of the seismic requirements and load-bearing capacity of walls according to the PTN-S and EC8-1 codes. Seismic base shear forces for all building types, calculated on the basis of the PTN-S code and EC 8-1, are stated in Table 1. The PTN-S seismic base shear forces according to the PTN-S code are meaningfully lower than EC8-1 forces (Figure 12). A difference in magnitude and distribution of the seismic base shear forces for Models A, B, C, and D is presented in Figures 14–17, respectively. The difference was pronounced at the fourth floor (elevation: 11.0 m) for Model A with seismic intensity of VIII and IX, according to PTN-S and EC8-1, respectively (Figure 14). The seismic shear force according to EC8-1 is 1.46 times higher than the force that was used for seismic analysis, considering seismic intensity VIII. The results of the analysis for the masonry models (Types B, C, and D) show a notable difference up to the middle of the fifth floor (elevation: 13.0 m). In this case, the seismic shear force in accordance with EC8-1 was almost 2.6 times higher than the force determined in accordance with the PTN-S code.

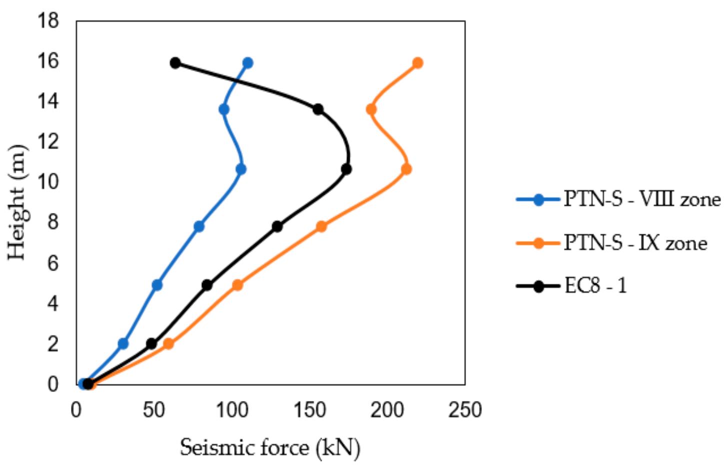


Figure 14. Magnitudes and distributions of seismic forces (Model A).

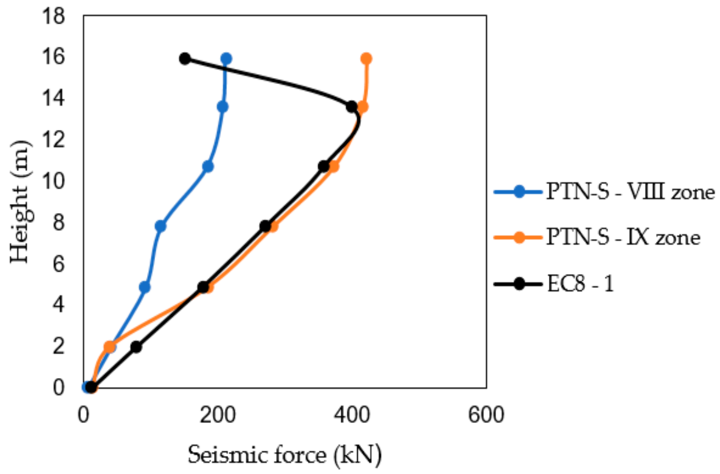


Figure 15. Magnitudes and distributions of seismic forces (Model B).

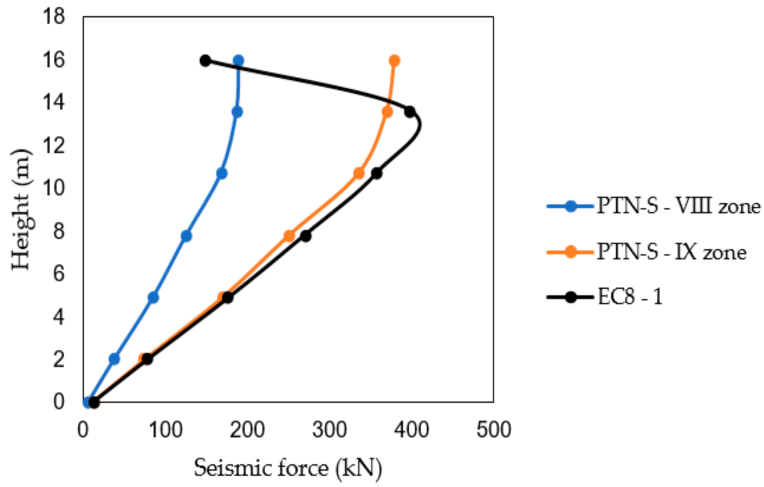


Figure 16. Magnitudes and distributions of seismic forces (Model C).

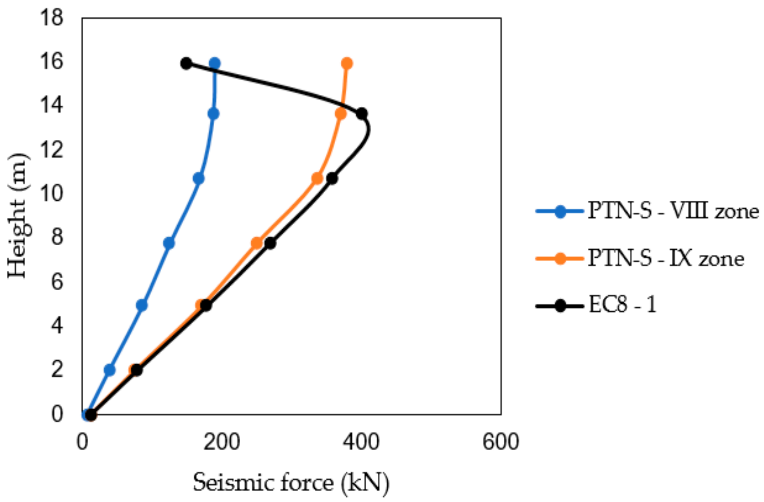


Figure 17. Magnitudes and distributions of seismic forces (Model D).

Based on the comparison of the charts showing magnitudes and distributions of seismic forces along the building height (Figures 14–17), the following observations can be made:

- The magnitudes and distributions of seismic forces along the height of the RC-frame structure (Type A) determined for the VIII and IX Seismic Zones, according to the

PTN-S code and EC8-1, were notably different (Figure 14). The design seismic forces according to EC8-1 for an RC-frame structure (the black line) were higher than those of the PTN-S code for Seismic Zone VIII but lower than those calculated in accordance with the PTN-S code for Seismic Zone IX;

- For the analyzed types of masonry structures (B, C, and D), the magnitudes and distributions of seismic forces determined in accordance with the PTN-S code for Seismic Zone IX and EC8-1 were almost identical.

Based on the above-mentioned findings, we can conclude that the seismic design requirements in accordance with the PTN-S code for Seismic Zone IX are similar to the seismic design requirements according to EC8-1. Therefore, the residential CM building stock in Niš constructed before 2019 in accordance with the PTN-S code satisfies the seismic requirements for Seismic Zone VIII according to EC8-1. It is important to note that, if it has to fulfill the EC8-1 requirements, the residential CM stock in Niš built in the last few decades must fulfill the PTN-S code's seismic requirements for Seismic Zone IX. From that point of view, we can say that the seismic resilience of the recently constructed CM building stock is overestimated compared to the EC8-1 seismic resilience requirements. If our aim is to have the residential CM building stock in Serbia satisfy the EC8-1 requirements related to seismic resilience, we should consider strengthening the existing buildings. This initiative should also be taken into consideration in other Balkan-region countries in order to enhance seismic safety.

Figure 18 shows a comparison of the magnitudes and distributions of seismic forces along the building height for all case study models in Seismic Zone VIII in accordance with the PTN-S code.

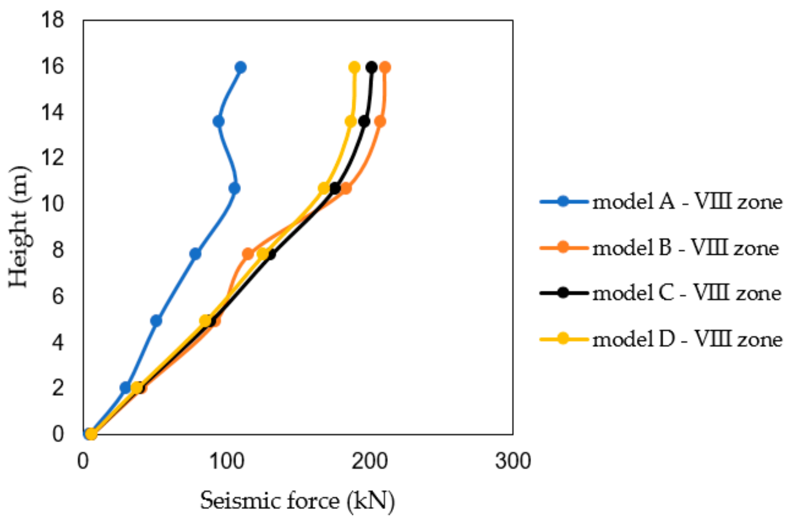


Figure 18. Magnitude and distribution of seismic forces for different structure types (A, B, C, and D), determined in accordance with the PTN-S code.

Figure 19 shows a comparison of the magnitudes and distributions of seismic forces along the building height for all the case study models in accordance with EC8-1.

It can be observed that seismic forces for different masonry models (B, C, and D), calculated in accordance with the PTN-S code are very limited (Figure 18). These variations cannot be noticed in Figure 19 since the functions of magnitudes and distributions are almost identical, which can be explained by the following facts:

- All three models (B, C, and D) are masonry structures;
- There was an insignificant difference in the total seismic weights for these models (see Table 1);
- The ground acceleration value was the same for all three models;
- The value of the behavior factor according to EC8-1 for all three models were the same, q = 2.4;

- The values of the fundamental period were very similar in both directions: Tx = 0.237–0.280 and Ty = 0.228–0.259.

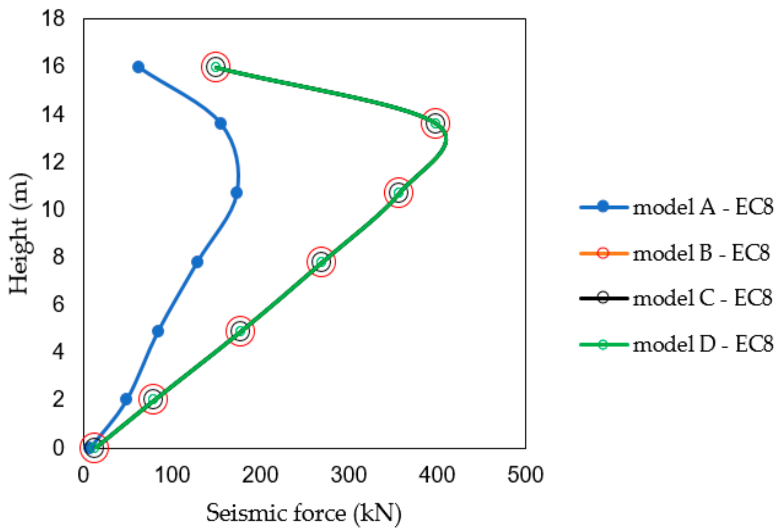


Figure 19. Magnitude and distribution of seismic forces for different structure types (A, B, C, and D) determined in accordance with EC 8-1.

4.5. The Results: Lateral Displacements

Besides the strength-related verification of structural elements, the design was also evaluated in terms of the maximum lateral displacements in both directions and the total drift ratio. These response parameters were determined for all structural models in both directions, and they are presented in Tables 3 and 4. The elastic displacements, Δ_e, the design displacements, Δ_d, and the total drift ratio, d, are presented in these tables. The total drift ratio was obtained as a product of the elastic displacement and behavior factor q. According to EC 8-1, a behavior factor of q = 3.9 was used for the RC-frame structure (Model A), while q = 2.4 was used for the masonry structures (Models B, C, and D). The displacement values were higher for the EC8-1 results compared to the PTN-S results. For example, d ranged from 0.038% to 0.153% for the case study models in accordance with the PTN-S code and from 0.051% to 0.178% in accordance with EC8-1. The drift values for RC-frame structures (Type A) were higher compared to the different types of masonry structures, which was expected due to the higher flexibility of RC-frame structures. The elastic displacement values of the models were similar in both directions. For example, the Δ_e value for Model A in the X-direction was 16.70 mm (Table 3), while for the same model in the Y-direction, the corresponding value was 16.45 mm (Table 4). Similar observations applied to lateral displacements in both directions for the masonry models (Types B, C, and D).

Table 3. The values of the displacement parameters for different structure types in the X-direction in accordance with the PTN-S code and EC 8-1.

		Type A	Type B	Type C	Type D
PTN-S	Δ_e (mm)	16.70	2.97	4.33	4.18
	Δ_d (mm)	25.05	4.46	6.49	6.27
	d (%)	0.153	0.027	0.039	0.038
EC8-1	Δ_e (mm)	19.49	3.96	5.76	5.56
	Δ_d (mm)	29.24	5.94	8.64	8.34
	d (%)	0.178	0.036	0.053	0.051

Table 4. The values of the displacement parameters for different structure types in the Y-direction in accordance with the PTN-S code and EC8-1.

		Type A	Type B	Type C	Type D
PTN-S	Δ_e (mm)	16.45	2.28	3.32	3.22
	Δ_d (mm)	24.68	3.42	4.98	4.83
	d (%)	0.150	0.021	0.030	0.029
EC8-1	Δ_e (mm)	17.39	3.11	4.51	4.37
	Δ_d (mm)	26.08	4.66	6.76	6.56
	d (%)	0.159	0.028	0.041	0.040

In general, the lateral displacements and drift values were very low, which is a common characteristic of masonry buildings.

4.6. Seismic Safety Verification for the CM Structure

4.6.1. "Rules for Simple Buildings" (EC 8-1)

Although the building does not meet the requirements for simple buildings according to EC8-1 due to its irregularity and building height (five stories), its seismic safety was initially estimated according to the rules for simple masonry buildings according to EC8-1. The values of the wall index (*WI)* were 4.23% for the transverse X-direction and 4.31% for the longitudinal Y-direction, respectively. The total floor plan area is 141.32 m^2. The minimum required *WI* value of 4% has been prescribed for each horizontal direction for a four-story CM building in Seismic Zone VIII; this corresponds to an a_gS value of 0.1 g or higher, depending on the wall length parameter *k*.

The geometrical properties of the walls and their layout at the ground-floor level in the X- and Y-directions for the case CM structure (Type B) are presented in Figure 20 and Tables 5 and 6.

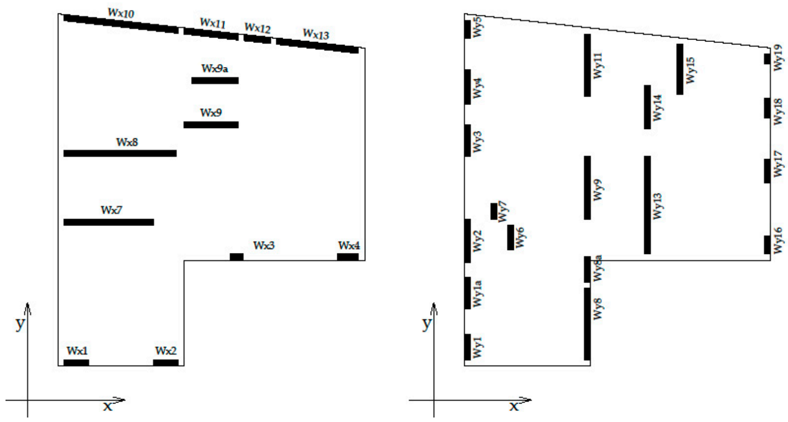


Figure 20. Layout of walls in both directions showing the wall labels.

Table 5. Geometrical properties of X-direction walls—CM structure (Type B).

	WX1	WX2	WX3	WX4	WX7	WZX8	WX9	WX9a	WX10	WX11	WX12	WX13
	(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**	**(11)**	**(12)**
L(m)	2.35	2.35	0.50	0.85	3.70	4.60	2.20	1.88	4.73	2.22	1.10	3.42
d(m)	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
A(m^2)	0.588	0.588	0.125	0.213	0.925	1.150	0.550	0.469	1.183	0.554	0.276	0.855
											$\sum$	7.474 m^2

Note: L—wall length; d—wall thickness; A—wall area.

Table 6. Geometrical properties of Y-direction walls—CM structure (Type B).

	WY1	WY1a	WY2	WY3	WY4	WY5	WY6	WY7	WY8	WY8a
	(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**
L(m)	0.70	0.95	1.45	0.95	1.20	0.63	1.00	0.62	2.95	1.05
d(m)	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
$A(m^2)$	0.175	0.238	0.363	0.238	0.300	0.158	0.250	0.155	0.738	0.263
	WY9	**WY11**	**WY13**	**WY14**	**WY15**	**WY16**	**WY17**	**WY18**	**WY19**	
	(11)	**(12)**	**(13)**	**(14)**	**(15)**	**(16)**	**(17)**	**(18)**	**(19)**	
L(m)	2.55	2.55	3.78	1.80	2.06	0.75	0.95	0.80	0.42	
d(m)	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	
$A(m^2)$	0.638	0.638	0.946	0.450	0.412	0.188	0.238	0.200	0.105	
								$\sum$	$6.688\ m^2$	

Note: L—wall length; d—wall thickness; A—wall area.

4.6.2. Seismic Strength Verification According to the EC8-1 Engineering Design Approach

According to the ultimate limit states (ULS) design approach, a structural element is considered safe if the following condition is fulfilled:

$$E_d < V_{Rd} \tag{6}$$

where E_d is the design value of the acting force, and V_{Rd} is the design value of resistance for that structural element. The design seismic forces according to Eurocode 8 can be determined using the following relations:

$$E_{Edx} + 0.3E_{Edy} 0.3E_{Edx} + E_{Edy} \tag{7}$$

The above equations show that, besides the forces acting in the principal direction (e.g., E_{dx}), it is required to consider the effect of forces in the other direction (e.g., $0.3E_{dy}$) to account for the possible direction of seismic action [33].

Based on the above-mentioned considerations, the seismic forces in each wall were determined, and seismic resistance was checked according to both codes. The effects of shear, compression, and bending were estimated for all the walls.

The characteristic masonry compressive strength was determined based on the adopted mortar and block compressive strengths in accordance with the PTN-S code, and the resulting value was 4 MPa. The relevant equations were presented by Blagojević et al.

According to EC8-1, the shear resistance of the wall was determined from the following equation:

$$N_{us} = L(f_{sko} + 0.4\sigma_d)d/\gamma_m \tag{8}$$

where f_{sko} is the characteristic shear strength for masonry, σ_d is the minimum compression stress determined due to loading effects, d is the wall thickness, and γ_m is the partial safety factor for masonry. Note that the f_{sko} values were prescribed in the code.

The equation for the design value's shear resistance was prescribed by EC6-1, as follows

$$V_{Rd} = \frac{f_{vk} t L_c}{\gamma_m} \tag{9}$$

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

$$f_{vk} = f_{vko} + 0.4\sigma_d \tag{10}$$

where f_{vko} is the characteristic initial shear strength of masonry, and σ_d is the design compression stress.

A verification of the seismic resistance of the CM structure was also performed, and the results are presented in Tables 7 and 8. The design value of the acting seismic base shear force, V_{Ed}, was compared with the design value of the shear resistance at the ground-floor level, V_{Rd}, which was taken as the sum of resistances for all walls oriented in the same direction (X or Y).

Table 7. The ratio of shear capacity and the design seismic shear force according to EC8-1 for walls in the X-direction (CM Model B).

	WX1	**WX2**	**WX3**	**WX4**	**WX7**	**WZX8**	**WX9**	**WX9a**	**WX10**	**WX11**	**WX12**	**WX13**
	(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**	**(11)**	**(12)**
V_{Rd} (kN)	89.71	89.71	19.12	30.44	178.99	233.60	93.66	74.40	240.21	88.52	40.28	157.69
V_{Ed} (kN)	122.60	122.60	5.53	19.67	169.02	188.64	79.94	69.82	238.94	95.55	31.33	160.20
V_{Rd}/V_{Ed}	0.73	0.73	3.46	1.55	1.06	1.24	1.17	1.06	1.005	0.93	1.28	0.98

Note: V_{Rd}—design value of shear resistance; V_{Ed}—design value of the acting shear force.

Table 8. The ratio of shear capacity and the design seismic base shear force according to EC8-1 for walls in the Y-direction (CM Model B).

	WY1	**WY1a**	**WY2**	**WY3**	**WY4**	**WY5**	**WY6**	**WY7**	**WY8**	**WY8a**
	(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**
V_{Rd} (kN)	24.03	32.12	49.88	32.12	40.74	21.86	34.21	21.72	121.80	36.80
V_{Ed} (kN)	15.56	30.42	66.24	30.42	47.77	12.10	32.18	11.31	155.35	32.41
V_{Rd}/V_{Ed}	1.54	1.06	0.75	1.06	0.85	1.81	1.06	1.92	0.78	1.14
	WY9	**WY11**	**WY13**	**WY14**	**WY15**	**WY16**	**WY17**	**WY18**	**WY19**	
	(11)	**(12)**	**(13)**	**(14)**	**(15)**	**(16)**	**(17)**	**(18)**	**(19)**	
V_{Rd} (kN)	101.00	101.00	166.80	65.24	60.48	25.57	31.80	26.93	15.93	
V_{Ed}(kN)	129.79	129.79	217.65	54.71	85.19	18.35	31.70	21.97	4.44	
V_{Rd}/V_{Ed}	0.78	0.78	0.77	1.19	0.71	1.39	1.00	1.23	3.59	

Note: V_{Rd}—design value of shear resistance; V_{Ed}—design value of the acting shear force.

A similar comparison of the walls in the X- and Y-directions in the CM building (Type B) showed that the majority of these walls did not have a satisfactory shear capacity, including the following walls: WX1, WX2, WX11, WX13, WY2, WY8, WY9, WY11, WY13, and WY15. The V_{Rd}/V_{Ed} ratio values for these walls were in the range of 0.73 to 0.98.

The V_{Rd}/V_{Ed} ratio values for the X-direction were generally lower than those for the Y-direction; this corresponds to the WI (calculations shown in Section 4.6.1). The case study building had a higher WI value for the longitudinal X-direction (5.29%) compared to the transverse Y-direction (4.73%).

According to the results obtained from the detailed analysis of the Type B model, and after the number of floors that were not compliant with the codes was taken into consideration, the RC-frame (Type A structure) proved to be a feasible choice.

4.6.3. The Distribution of Design Seismic Forces to Different Members of CM Buildings

A comparative analysis of the seismic resistance for a CM structure (Type B) according to the PTN-S and EC8-1 codes showed differences in shear resistance of a CM wall, as well as the distribution of vertical and horizontal seismic forces in masonry and RC parts of the walls. According to the PTN-S code, the seismic resistance of a CM wall depends on the mechanical properties of masonry blocks and mortar, i.e., on the masonry shear strength.

On the other hand, EC8-1 takes into consideration the mechanical properties of both the wall and the RC tie-columns, see Figure 21. A brief overview of the calculation procedure for CM walls according to EC8-1 is provided next.

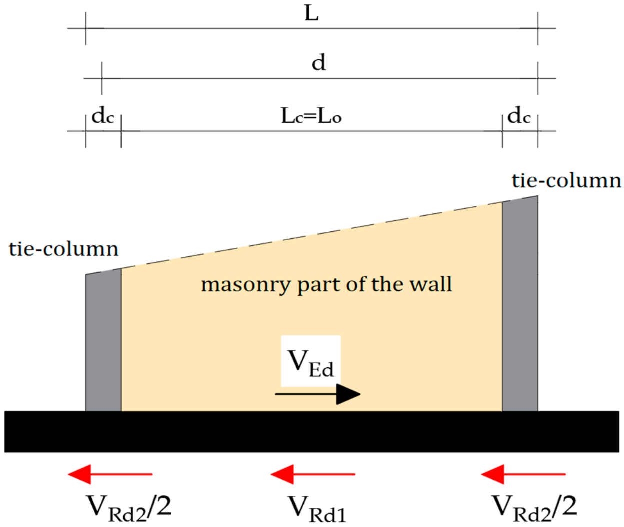


Figure 21. Design shear resistance model of a CM wall according to EC8-1.

Masonry walls with RC tie-columns have a higher shear strength than URM walls. The shear strength of the an URM wall with length L_c and thickness t, V_{Rd1}, is increased to account for the design shear strength of tie-column concrete, V_{Rd2}, while ignoring the effect of reinforcement in the tie-columns. The total shear strength can be determined as follows:

$$V_{Rd} = V_{Rd1} + V_{Rd2} = \frac{f_{vk} t L_c}{\gamma_m} + V_{Rd2} \tag{11}$$

The design shear strength of tie column concrete, V_{Rd2}, having dimensions of t/d_c, was determined according to the Eurocode 2:

$$V_{Rd2} = \left[v_{min} + k_1 \sigma_{cp}\right] \cdot A_c \tag{12}$$

$$v_{min} = 0.035 k^{1.5} f_{ck}^{0.5} \tag{13}$$

$$k = 1 + \sqrt{\frac{200}{d}},\ d[mm] \tag{14}$$

where:

f_{ck} is the characteristic compressive strength of concrete;
$k_1 = 0.15$;
$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$is the compressive stress in RC tie-column;
N_{Ed} is the axial compression force in tie-columns;
$f_{cd} = f_{ck}/\gamma_c$ is the design compressive strength of concrete;
A_c is the cross-sectional area of 2 tie columns;
$\gamma_c = 1.2$ is the partial safety coefficient for concrete (the incident action).

Seismic design of the case study buildings (Type B) considered the values of seismic forces in the CM walls, including masonry walls and tie-columns, which were obtained based on the above-described approach.

The distribution of vertical and horizontal seismic forces between masonry and RC tie-columns was studied for the wall (WX8) located at the ground floor of the building. The wall thickness is 25 cm, while the tie-columns have a square cross-section 25 cm × 25 cm (Figure 22a–c). Both structural elements, the tie-columns and the masonry wall, were modeled as plate elements. The depth of the tie-columns was increased in 5 cm increments up to the tie-columns' size of t/d_c = 25/50 cm. The width of the tie-columns remained equal to the thickness of the masonry wall (25 cm). The analysis results are presented in Tables 9 and 10 and Figures 23–25. A variation in the bending moment values, axial forces, and shear forces was observed at the interface of the wall and the foundation slab for alternative seismic action according to the load distribution on the structural elements of the CM structure defined in the EC8-1 code. It was observed that as the tie-column size and the stiffness increased, in forces V_{Ed}, M_{Ed}, and N_{Ed} of the seismic design of the masonry wall decreased. Of course, with the increase in the tie-column size with the same amount of reinforcement, the values of the reinforcement tensile strain were the same. With an increase in the tie-columns' size, their contribution to the redistribution of gravity and seismic loading increased. It is important to note that the variation in shear force in masonry wall, V_{Ed} was insignificant (see Table 9). The axial compression force showed a more prominent variation (both in masonry wall and tie-columns, see N_{ed} values in Tables 9 and 10), while the bending moment considerably changed with an increase in the tie-columns size (see M_{ed} values in Tables 9 and 10). With an increase in the depth of the tie-columns from 25 to 50 cm, the value of the bending moment acting on the masonry wall decreased from 134.76 to 30.5 kNm, see Table 9. As a result, the bending moment that was redistributed to the tie-columns increased from 3.43 to 10.16 kNm, but both values are rather low (see Table 10). As expected, with an increase in cross section of the tie-columns, the mass of CM wall increased, and therefore, the mass of the CM structure, and eventually the total seismic force, increased. As a result an increased amount of reinforcement in the tie-columns would be required. When tie-column cross-section becomes significantly large the confining elements will act as RC columns and RC beams of an RC frame structure.

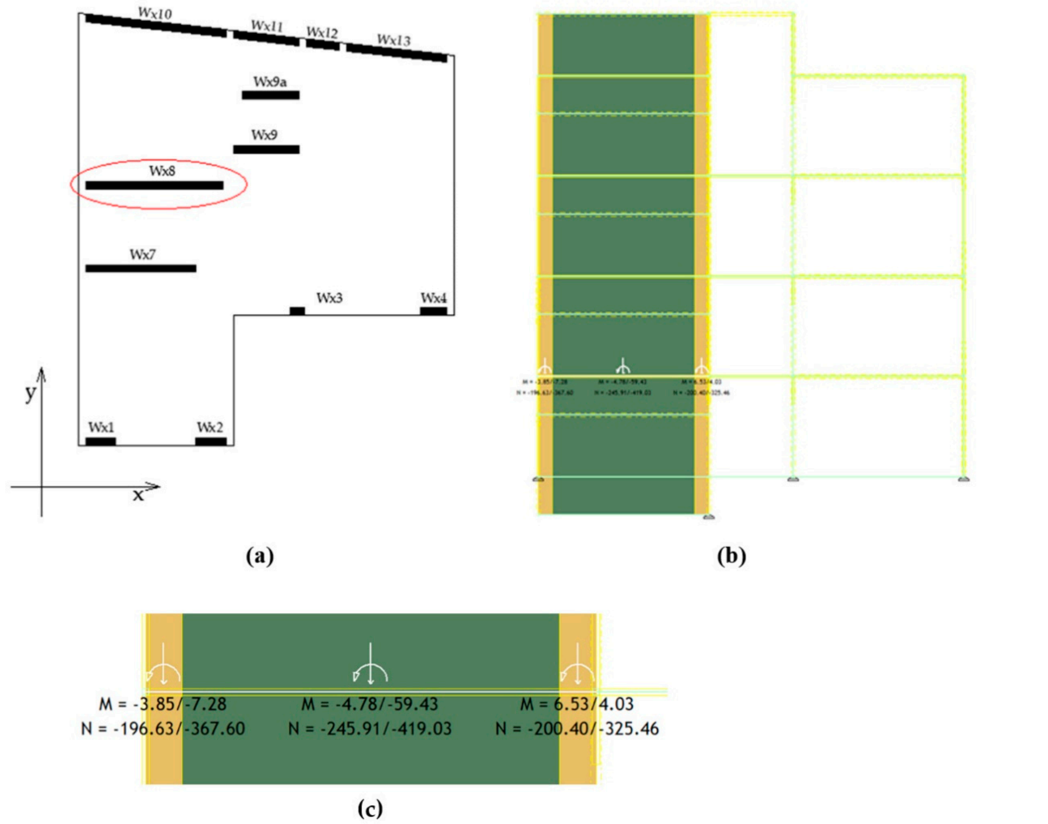


Figure 22. Redistribution of gravity and seismic forces in relation to the ground-level wall (WX8) of the case study CM building (Type B): (**a**) position of the CM wall WX8 (by red elipse is marked analyzed wall); (**b**) numerical model of the CM structure with tie-columns of the cross-sectional dimensions 25 cm × 40 cm; and (**c**) values of internal forces for the alternative seismic action.

Table 9. Values of the internal forces in masonry part of the CM wall (WX8) depending on the cross-sectional dimensions of the tie-columns.

Tie-Column Dimensions (cm)	25/25	25/30	25/35	25/40	25/45	25/50
	(1)	(2)	(3)	(4)	(5)	(6)
Bending Moment, M_{Ed} (kNm)	134.76	100.38	77.85	59.43	35.79	30.50
Shear Force, V_{Ed} (kN)	46.65	45.64	44.57	43.76	42.86	42.41
Axial Force, N_{Ed} (kN)	522.84	484.75	449.97	419.03	385.80	364.13

Table 10. Values of the internal forces in the tie-columns of the CM wall (WX8) depending on the cross-sectional dimensions of the tie-columns.

Tie-Column Dimensions (cm)	25/25	25/30	25/35	25/40	25/45	25/50
	(1)	(2)	(3)	(4)	(5)	(6)
Bending Moment, M_{Ed} (kNm)	3.43	4.62	5.90	7.28	8.68	10.16
Shear Force, V_{Ed} (kN)	3.38	3.41	3.83	4.29	4.81	5.37
Axial Force, N_{Ed} (kN)	274.21	308.71	339.28	367.60	392.14	415.41

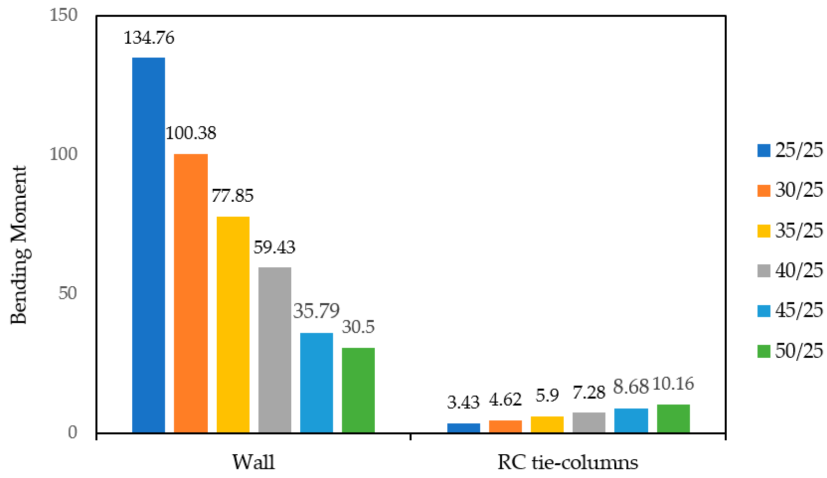


Figure 23. Redistribution of the bending moment in the structural elements of the CM wall (masonry and tie-columns) depending on the size of the tie-columns. (ranging from 25/25 cm to 50/25 cm).

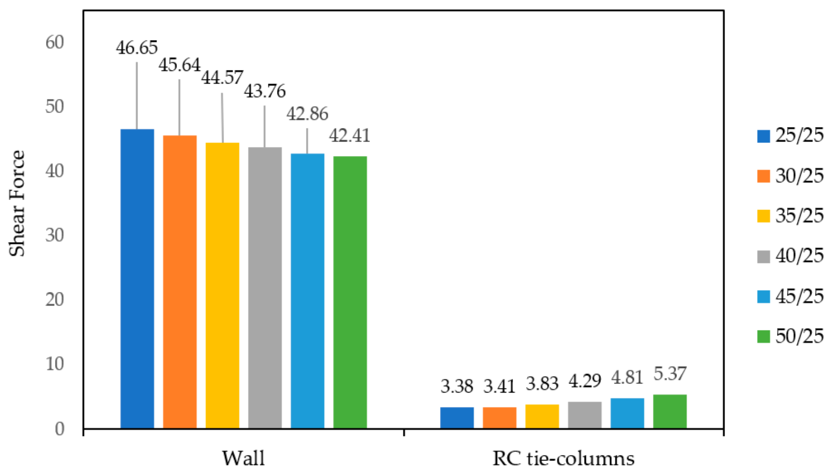


Figure 24. Redistribution of the shear force in the structural elements of the CM (masonry and tie-columns) depending on the size of the tie columns.

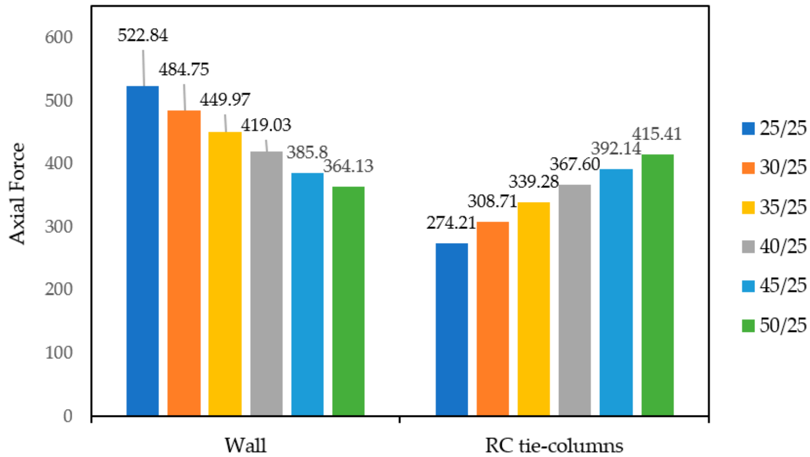


Figure 25. Redistribution of the axial force in the structural elements of the CM (masonry and tie-columns) depending on the of the tie-columns.

4.7. Seismic Analysis and Design of an Alternative Model: RC Frames with Masonry Infills

4.7.1. Numerical Models and Seismic Analysis

An additional model of the RC-frame structure for the case study building was developed to evaluate the effect of masonry infills on the seismic demand of the frame structure. The infills were modeled as equivalent diagonal struts that were pin-connected at the ends and were assumed to transfer compression forces between the opposite joints in a frame panel [34–36]. The masonry elements were modular clay blocks, and the wall thickness was 25 cm. The masonry properties were a 5.0 MPa compressive strength and a modulus of elasticity of 4000 MPa. The strut thickness was taken as equal to the wall thickness, while the width of the cross section was taken as equal to one-quarter of the strut length (0.25 r_{inf}); see Figure 26. Note that a smaller width (0.13 r_{inf}) was assigned to struts in walls with openings (windows and doors).

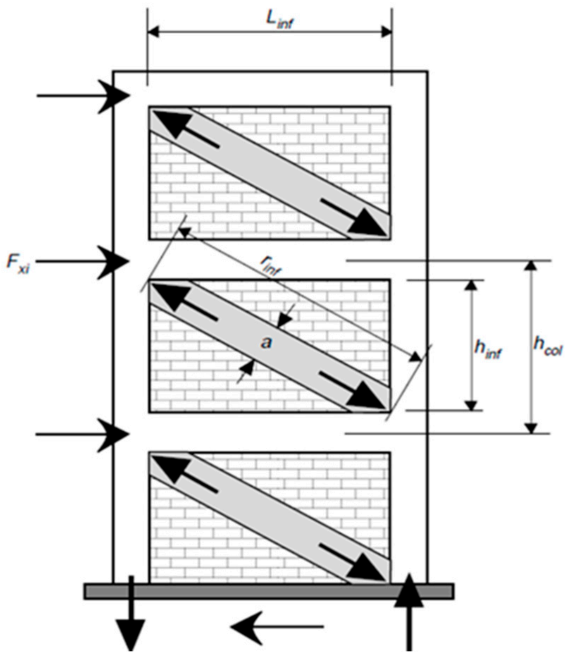
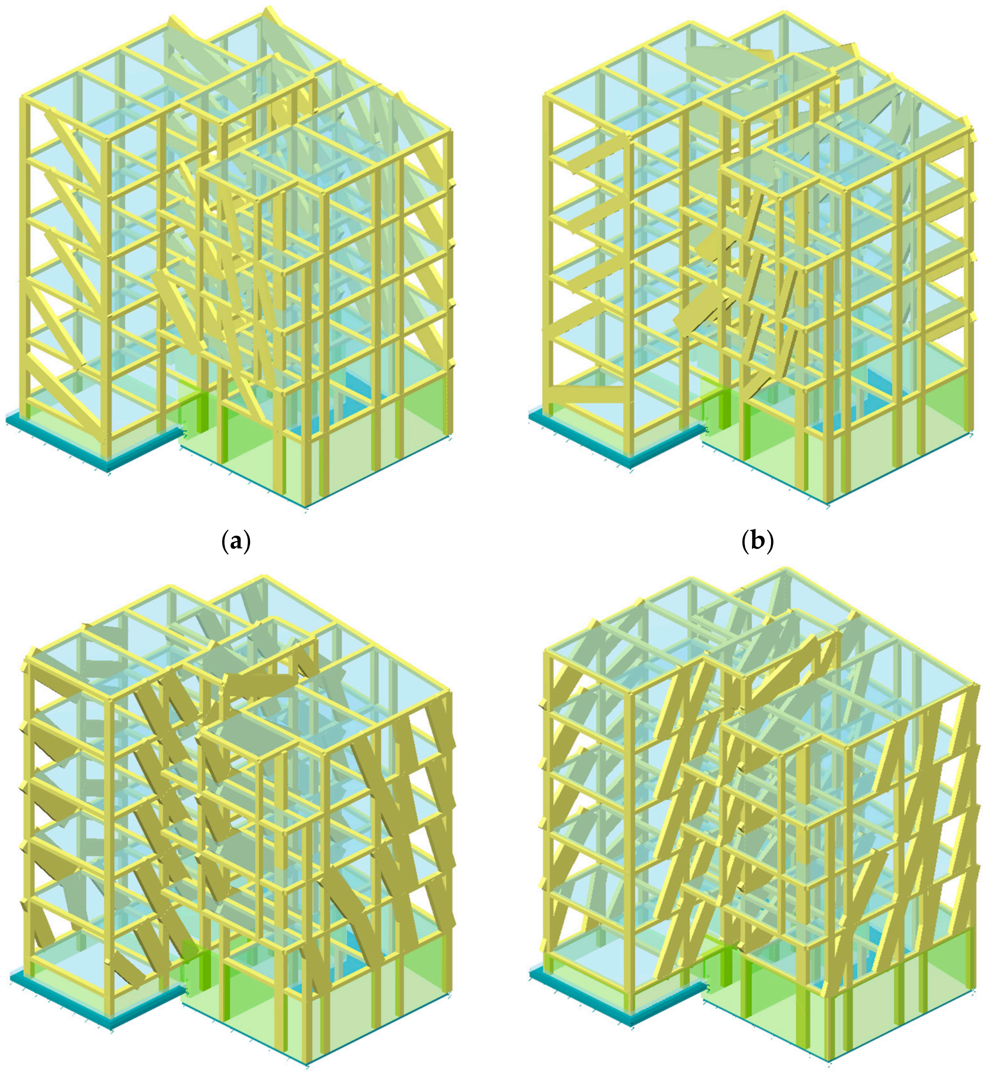


Figure 26. A typical RC frame with equivalent diagonal struts.

The analysis results for the RC-frame model with masonry infills (Figure 27a–d) subjected to seismic actions in the X- and Y-direction are presented in Table 11 and summarized below:

- The fundamental period for the RC-frame structure with masonry infills was 0.65 s, which was approximately 6% lower than that of the corresponding model for the bare RC frame (0.73 s);
- The maximum elastic horizontal displacement for the RC-frame model with infills according to the PTN-S code was 12.43 mm, which was less than that of the corresponding displacement for the bare frame model (16.70 mm);
- The maximum elastic horizontal displacement for the RC-frame model with infills according to the EC8-1 code was 15.43 mm, which was less than that of the corresponding displacement for the bare frame model (19.49 mm).



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

(**c**) (**d**)

Figure 27. Numerical models for RC frames with masonry infills: (**a**) X1 = X-direction (for seismic forces acting in + direction); (**b**) X2 = X-direction (for seismic forces acting in opposite direction); (**c**) Y1 = Y-direction (for seismic forces acting in + direction), and (**d**) Y2 = Y-direction (for seismic forces acting in opposite direction).

It is obvious that the analysis results for these two RC-frame models (with and without infills) were similar. It can be concluded that the bare RC-frame structure was very stiff and that the contribution of masonry infills was not significant in this case.

Table 11. Maximum displacements and fundamental periods for the RC-frame model with masonry infills according to the PTN-S and EC8-1 codes.

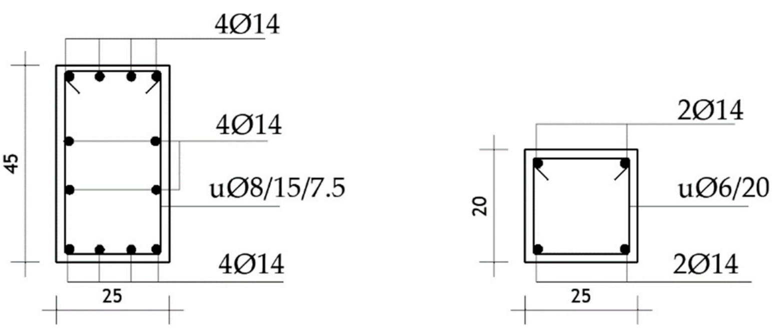
RC-Frame Structure Model with Masonry Infills					
		X1	**X2**	**Y1**	**Y2**
Fundamental period (s)		0.65	0.56	0.75	0.75
Displacements PTN-S	Δ_e (mm)	17.25	12.43	15.14	15.16
Displacements EC8-1	Δ_e (mm)	20.44	15.43	17.52	17.55

4.7.2. Design of an RC-Frame Structure Frame According to PTN-S and PBAB 87

Equivalent static seismic analysis of the RC-frame structure was conducted according to both the PTN-S and EC 8-1 codes. As already mentioned, the building was constructed as an RC-frame structure. The seismic design and detailing of the RC structural elements were performed according to the PTN-S code (special requirements for RC structures) and also the Yugoslav code for the design of RC structures, PBAB 87, which was issued in 1987 and was in effect until 2019, when Eurocode 2 [37] became the governing code for the design of RC structures in Serbia (along with other Eurocodes). The PBAB 87 code prescribed the design of RC structures according to the ULS design approach in terms of strength and serviceability, and it was considered comparable to Eurocode 2 to a certain extent.

The dimensions of RC structural elements (beams and columns) were selected while keeping in mind the building height, the ductility requirements (the stiffness of the beams was lower than the stiffness of the columns, which created conditions for the occurrence of nonlinear deformations at the beam ends), and the lateral displacement limits. The cross-sectional dimensions of the RC columns and beams are 25 cm × 40 cm. The concrete grade was MB 30 (corresponding to the design compressive strength of f_B = 20.5 MPa), while the steel class was GA240/360 (yield strength: 240 MPa).

The design loads for the design of the RC beams, due to the permanent and variable loads, were obtained by implementing partial safety coefficients of 1.6 and 1.8, respectively. The design loads for the RC columns, due to the permanent and variable loads, were calculated by implementing partial safety coefficients of 1.9 and 2.1, respectively. A combination of permanent, variable, and seismic loads was also considered, with the corresponding global safety coefficient of 1.3. The required amount of reinforcement was calculated based on the envelope of internal forces in the structural elements. Figure 28 shows reinforcement details for the column marked as S-2aB in Figure 11a, as well as reinforcement details for a typical vertical RC confining element.



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

Figure 28. Reinforcement details: (**a**) RC column (RC-frame solution); (**b**) vertical RC confining element (CM solution).

The cross-sectional dimensions of RC confining elements ranged from 20 cm × 20 cm to 25 cm × 20 cm, and they contain reinforcement determined according to PBAB 87. The longitudinal reinforcement in the horizontal RC confining elements amounted to 4∅12 bars, while the longitudinal reinforcement in the vertical RC confining elements amounted to 4∅14 bars (note that GA 240/360 steel was used). The transverse reinforcement in the RC confining elements was in the form of ∅6 bars at 20 cm spacing.

Based on experiences with similar projects, an average amount of reinforcement in all RC confining elements in a CM structure could be estimated at 20–25 kg per m^2 of the floor area. Similarly, it could be estimated that approximately 40–45 kg of steel per m^2 of the floor area is needed for an RC-frame structure. Based on the structural types considered in this study, the quantity of concrete necessary for the construction of all RC elements was also calculated, and it is presented in Table 12.

Table 12. Concrete quantity for the entire structure for structure types A, B, C, and D.

Quantity of Concrete for an Entire Structure (All RC Structural Elements)		
Type	Description	Quantity m^3
A	RC-frame structure	294.83
B	CM structure with horizontal and vertical RC confining elements	283.60
C	Masonry structure with horizontal RC confining elements	260.58
D	Unreinforced masonry structure (URM)	240.31

The difference in the quantity of concrete required for the construction of structural elements in different structural systems was rather small, but the difference in the quantity of reinforcement per m^2 of floor area was more significant.

5. Conclusions

This paper has presented a case study of alternative design solutions for a typical mid-rise residential building in the city Niš, Serbia, which was originally designed as a CM structure. The design of the CM structure, according to the codes standing in Serbia at the time of the design, showed that the seismic capacity of the structure was inadequate. Consequently, an RC-frame structure was designed and constructed. The seismic design was carried out in accordance with the PTN-S seismic design code, which was developed in the former Yugoslavia. A comparison of the analysis results for the PTN-S and EC 8-1 codes was presented. The following conclusions are of importance:

The case study building selected for this study is a common example of a mid-rise residential CM building in Serbia. Buildings of this type must be designed in compliance with applicable design codes, depending on the building size, the number of floors, and the presence of irregularities.

A comparison of the seismic analysis results was carried out in accordance with the PTN-S codes and EC 8-1 for four structural types: A, B, C, and D. This comparison showed that the seismic requirements according to the EC8-1 code are significantly higher compared to those of the PTN-S code.

The seismic design of the original solution (CM structure/Type B), performed in accordance with the PTN-S and EC 8-1 codes, did not meet the seismic design requirements. A seismic safety verification of individual CM walls showed that several walls did not have satisfactory shear capacity. Due to architectural constraints, wall lengths could not be increased, and it was not possible to add new walls.

An RC-frame structure offered a feasible alternative to the original design solution (a CM structure); this can be attributed to the building height (a mid-rise building) and the site with a moderate seismic hazard.

There was a similarity in terms of the seismic force values for the PTN-S code (Seismic Intensity Zone IX, in accordance with the related seismic zonation map) and the EC8-1 code for Niš, Serbia (based on the revised seismic hazard map for Serbia, which is correlated with EC8). The residential CM building stock in Niš constructed in accordance with the PTN-S code had to meet seismic requirements for Seismic Intensity Zone VIII. Since the seismic design requirements of EC8-1 are more stringent compared to those prescribed by the PTN-S code, it can be expected that the buildings constructed in Niš (and possibly in other sites in Serbia) before 2019 (when the Eurocodes were enforced in the country) do not meet the seismic design requirements of EC8-1 or other related Eurocodes. It is possible that many existing buildings may need to be retrofitted in order to comply with the current code requirements.

The redistribution of vertical and horizontal seismic forces in relation to the ground level wall (WX8) of the case study building (Type B—CM structure) was analyzed. It is obvious that, with the increase in the tie columns' size, the relative strength and stiffness of CM wall were increased, but the value of the shear forces, V_{Ed}, and the values of the M_{Ed} and N_{Ed} forces that act on the masonry wall were reduced. With the increase in the dimensions of the tie columns' cross section, their contribution to the redistribution of gravity and seismic loading increased. It is important to note that the seismic force variation was of a very low intensity. The value of the axial compression force was characterized by a more prominent variation, while the value of the bending moment was considerably changed with the increase in the cross section of the tie columns—that is, with the increase of the height of the cross section of the tie columns from 25 cm to 50 cm, the value of the bending moment acting on the masonry part of the wall decreased by approximately four times, and the value of the bending moment that was redistributed on the tie columns increased by approximately three times.

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