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Post-Earthquake Assessment and Strengthening of a Cultural-Heritage Residential Masonry Building after the 2020 Zagreb Earthquake

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Abstract: After a long period of no excessive ground shaking in Croatia and the region of ex-Yugoslavia, an earthquake that woke up the entire region was the one that shook Croatia on 22 March 2020. More than 25,000 buildings were severely damaged. A process of reconstruction and strengthening of existing damaged buildings is underway. This paper presents proposed strengthening measures to be conducted on a cultural-historical building located in the city of Zagreb, which is under protection and located in zone A. After a detailed visual inspection and on-site experimental investigations, modeling of the existing and strengthened structure was performed in 3Muri. It is an old unreinforced masonry building typical not only for this region but for relevant parts of Europe (north, central, and east). The aim was to strengthen the building to Level 3 while respecting the ICOMOS recommendations and Venice Charter. Some non-completely conservative concessions had to be made, to fully retrofit the building as requested. The structural strengthening consisted of a series of organic interventions relying on—in the weakest direction—a new steel frame, new steel-ring frames, and FRCM materials, besides fillings the cracks. Such intervention resulted in increasing the ultimate load in the X and Y directions, respectively, more than 650 and 175% with reference to the unstrengthened structure. Good consistency was obtained between the numerical modeling, visual inspection, and on-site testing.

Keywords: masonry cultural heritage building; strengthening; pushover analysis; FRCM; 3MURI; earthquake; Venice Charter; ICOMOS recommendations; ambient testing; numerical modeling

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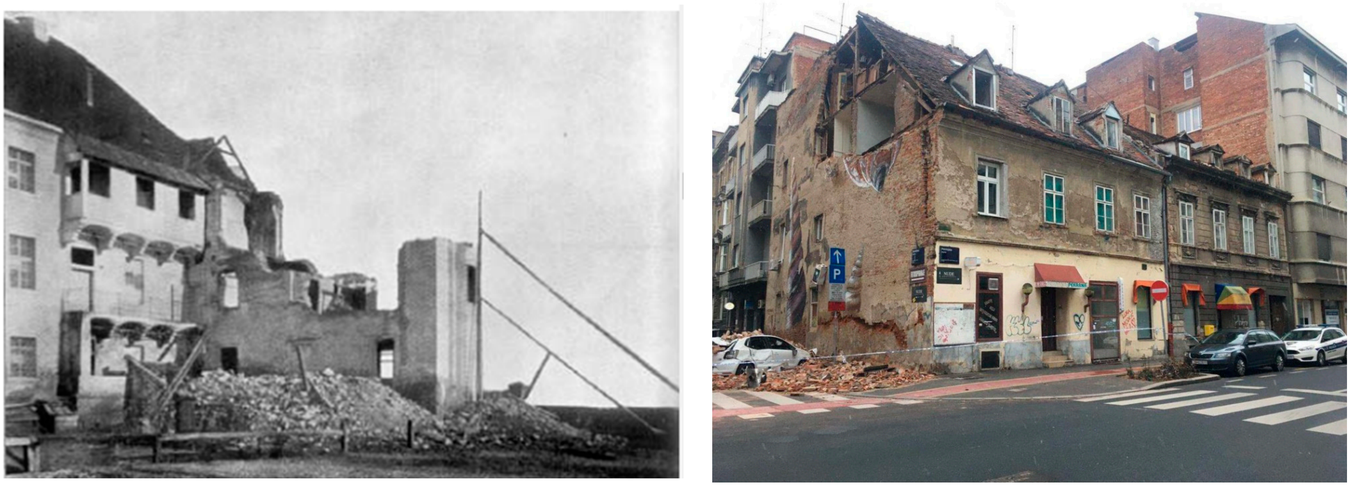
1. Introduction

Earthquakes are one of the natural hazards which can have a dreadful influence on existing buildings, population, economy, and community as a whole. Multiple consequences of the ground shaking can lead to the devastation of complete districts and even some small villages can be completely erased by destructive earthquakes, such as the case of the village of Onna in Italy in 2009. On 22 March 2020, around half past seven in the morning, an earthquake woke up the citizens of Zagreb. The earthquake was of magnitude M_L = 5.5, Mw = 5.3, and the intensity in the Zagreb Metropolitan area was measured as VII-VIII according to the Medvedev–Sponheuer–Karnik scale [1]. Additionally, the fact that this was a shallow earthquake having a depth of only 10 km influenced the degree of the building's damage. The measured peak ground acceleration (PGA) for the leading North-South direction was 0.22 g [2]. This main shock was followed by numerous aftershocks and the ground continued to shake for several months. It was 142 years before that Zagreb was shaken by an earthquake, known as the Great Zagreb earthquake, with an estimated magnitude (M_L = 6.3) [3], according to macroseismic observations, which had caused tremendous damage. In the building's assessment report, it was stated that 1758 residential buildings were damaged and out of that 27.6% were heavily damaged (Figure 1a). Numerous buildings of various usage (residential buildings, public buildings, medical facilities, historical monuments, etc.) were damaged by the 2020 Zagreb earthquake, having different levels of damage from minor damage to severe damage, and even partial collapse of buildings (Figure 1b). The most affected area was in the vicinity of the epicenter, the Lower Town of the city of Zagreb, constituted of cultural heritage buildings. Additionally, buildings constructed in the late 19th and 20th centuries, mainly unreinforced masonry structures were damaged to a great extent. It is estimated that up to 25,000 buildings were affected by this earthquake. Luckily, not many human fatalities were registered (one human life was lost) and this can be connected to the COVID-19 restrictions at that time.



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

Figure 1. Building devastation after: (**a**) 1880 Zagreb earthquake [4] Reproduced with permission from Zagreb City Museum, 2022.; (**b**) 2020 Zagreb earthquake (author's figures).

Based on the available statistical census data [5], the percentage of dwellings constructed before 1945 was 13.3%, from 1946 to 1970 was 29.1%, from 1971 to 1980 was 22.1%, from 1981 to 1990 was 16.8%, from 1991 to 2005 was 13.6%, and after 2006 was 5.1%. The first group can be subcategorized into a period before 1919 (7.63%) and from 1919 to 1945 (5.78%). The categorization of building according to construction age (six groups) is connected with the introduction, revision, and upgrading of seismic codes in ex-Yugoslavia which were applied to Bosnia and Herzegovina [6], Croatia, and other ex-Yugoslavian Republics. For the city of Zagreb, a very similar trend is observed for certain construction periods (13.3%; 30.4%; 17.8%; 14.7%; 15.2; and 8.8%, respectively) [7]. Unreinforced masonry structures (URM) mainly constructed between 1860 and 1920 make up the historical cores of Croatian cities, so does the city of Zagreb as well, where this percentage is 3.9. It was all until 1920 that the floors were made with timber flexible floors which were not well connected to the walls and the connection of the wall was poor and inadequate. Buildings constructed until 1948 were built during the period when there were no standards regarding seismic actions in this region. The first official document which had minimum seismic requirements was the PTP2 (1948) [8], having no limitation on the height of the various masonry building, which would be modified in 1964 regulations PTP-GuSP64 [9].

During the 19th century, the material which was mainly used for construction was adobe, stone, brick, timber, and steel profiles to a minimum extent. Family buildings (an example of the considered case study) in this period (until 1920) were constructed as URM structures with wooden floors, except for the basement ceilings which were very often constructed with iron beams and segmental brick vaults. This is the case as well for residential multistorey buildings. These structures usually have an adequate load-bearing capacity for standard loads except for seismic loads. It was in 1930 that the timber floors were starting to be replaced by the semi-prefabricated ribbed reinforced concrete floor, having a concrete layer of around 6.5 cm which could be considered as a rigid diaphragm [10]. Buildings with rigid floors had a smaller level of damage compared to the ones with wooden floors. After II World War, until 1964, monolithic reinforced concrete floors started to be introduced in the building's construction. After the 1963 Skopje earthquake, confined masonry was massively implemented in this region. This was as well an era of construction of residential buildings made of reinforced concrete load-bearing systems in line with the regulations for 1964 and then later stricter regulation regarding seismic actions enforced in 1981 after the 1979 Montenegro earthquake. Eurocodes were introduced in a step-by-step sequence firstly having a pre-standard status for the duration of 6 years (1992–1998) and then with a full EN label from 1998. The final implementation of Eurocode standards started in 2005 [7].

After the 2020 Zagreb earthquake, it was necessary to collect information about the type and level of damage. As there were no post-earthquake forms for Croatia, it was decided to use the Italian forms and adjust them to Croatia [11]. After conducting the preliminary inspection, each inspected building was marked with the appropriate color and label. Buildings were categorized into three usability categories marked with three colors. The green color indicated that the building is usable either with limitations (U1) or with recommendations (U2). Buildings marked with a yellow sticker indicated that they are temporarily unusable: either that a detailed inspection is required (PN1) or that short-term countermeasures are required (PN2). Unusable buildings either due to external risk (N1) or due to damage (N2) were marked with red color.

Several papers have been published regarding the rehabilitation of damaged buildings after the 2020 Zagreb earthquake. One building of the infantry barrack of Prince Rudolf (built from 1887 to 1889) located in the Lower Town of the city of Zagreb was researched by [12]. It is listed as one of the Protected Cultural Heritage buildings being located within the A protection zone of the Historical and Urban Entity of the city of Zagreb. The building is representative of a typical URM structure, where the load-bearing walls are made of Austro-Hungarian solid bricks of the old standard (14 × 6.5 × 29 cm). The thickness of the walls reduces from the basement to the upper floors. The ceiling floor is made of a brick vault while the other floors are made of wooden and steel beams. In order to increase the capacity of the brick load-bearing walls, fibre reinforced cementitious matrix (FRCM) strengthening system or concrete jacketing was proposed. Additionally, the removal of brick partition walls was proposed and replacement with a drywall system. In order to increase the stiffness of the wooden floors, a thin reinforced concrete compression slab was proposed [12]. Moreover, two additional strengthening ideas proposed a new steel equivalent system and seismic isolation. A step further was conducted in [13], where three possible ways of strengthening a URM structure used for education purposes built in the 19th century were elaborated. Strengthening was proposed solely by FRCM, shotcrete, and then their combination. The analyses were implemented through the application of the static nonlinear method in 3Muri software. Besides the increase of the load-bearing capacity, the estimation of the expected cost and environmental impact of each proposed remedial measure was investigated. The lowest cost and the highest capacity increase were obtained for strengthening the structure with shotcrete; however, its application was proven to be too invasive for cultural heritage structures [13]. The FRCM system had a much smaller emission of CO_2 than the shotcrete application.

A remarkable building located at Matice Hrvatske 2 Street in Zagreb, identified as one of the most significant structures in the whole of Croatia [14], was investigated in the paper [15]. The building was constructed in 1887 and during its life, several adaptations were conducted. It is an L-shaped corner building within an old masonry building block. The vertical loadbearing system is composed of Austro-Hungarian solid bricks connected with lime mortar. This structure is as well under heritage protection. The building was modeled as a stand-alone structure due to a lack of information regarding the adjacent buildings. Good consistency was obtained between the modeling and the noted damage after the 2020 Zagreb earthquake.

Our paper is structured in the way that in Section 2 a description of the methodology which has been applied is briefly explained and discussed. The intention was to emphasize the importance of the application of the ICOMOS recommendations [16] and the principles of the Venice Charter when reconstructing buildings of cultural heritage. Section 3 is dedicated to the selected case study building Ribnjak 44, a building located within the protection zone "A". The section opens with the historical information which was obtained about the buildings, and the defined classification of the building after the 2020 Zagreb earthquake according to the EMS-98 classification. In Section 3.3, a detailed explanation is provided about the conducted visual inspection of the buildings and crack pattern survey. Once the visual inspection was conducted, it was necessary to make on-site investigations and perform tests required for the determination of the in-situ compressive stress level as well as deformability characteristics of masonry (modulus of elasticity) and dynamic characteristics of the structure (Section 3.4). Section 4 is dedicated to the modeling of the existing structure, taking into account all the data provided in the previous sections. The nonlinear static analysis (pushover analysis) was performed in the 3Muri software. Once the results were obtained, it was concluded that the structure does not possess adequate capacity, requiring it to be strengthened. This leads to Section 5 which elaborates on various strengthening procedures and their modeling in the 3Muri software. An adequate choice of various strengthening procedures had to be selected in order to follow and respect the recommendations regarding material compatibility with existing materials, application of reversible methods as much as possible, and methods that are least invasive and most compatible with heritage values, matching the need for safety and durability. Section 6 compares the results of the unstrengthened and strengthened structure and indicates the effectiveness of the strengthening techniques. The paper closes with a conclusion and provides several recommendations and explains the contributions of the research to knowledge in the European context. It indicates some encountered challenges and provides suggestions on how to overcome these issues.

2. Applied Methodology

In order to propose and design a suitable intervention and strengthen the existing buildings, proper investigation and diagnosis are the key factors. The importance of investigations for the determination of structural diagnosis (including historical aspects, materials, and structures) is emphasized in the Venice Charter (1964). Investigations must take into account a variety of different aspects including the typology of the building, the type of masonry elements, connections between elements, characteristics of materials, etc. Each structure has unique characteristics and the investigations must be planned and executed to ensure that an adequate understanding of the structure is obtained [17]. The methodology can be divided into two phases. In the first phase, it is necessary to gather as much information as possible regarding the structure (historical information, description of the building, survey and description of the damage pattern, and in-situ and laboratory experimental tests). The second phase is devoted to the numerical analysis where it is necessary to select what is the most effective modeling type (block-based models, continuum homogeneous models, geometry-based models, or equivalent frame models) and type of analysis (linear or nonlinear). Once numerical modeling is conducted, based on the obtained results it is necessary to determine appropriate interventions for strengthening the structure. The use of this phased multidisciplinary procedure is essential for an in-depth understanding of structures. This kind of analysis allows the performance of knowledge-based structural analysis and thus defines with more confidence the strengthening interventions. It can as well prevent the execution of intrusive repair works. Its application gains special interest when structures are located in areas with high seismic risk [18].

ICOMOS [16] has published and approved the Recommendations for the Analysis, Conservation and Structural Restoration of Architectural Heritage. To make a decision regarding which remedial measures will be selected and applied, it is required to apply a methodology (Figure 2) which is an iterative process between the tasks of data acquisition, structural behavior, and diagnosis and safety [19]. The correct interpretation of the diagnosis and safety from a qualitative and quantitate perspective is of the utmost importance as it will lead to the type and degree of remedial measures.

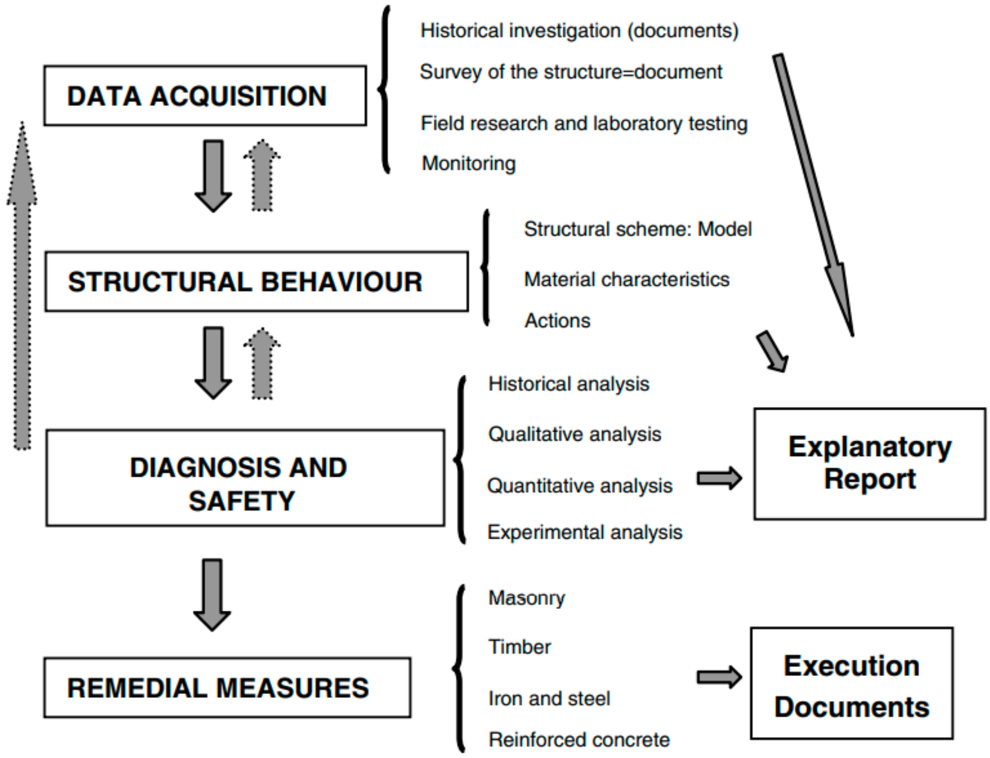


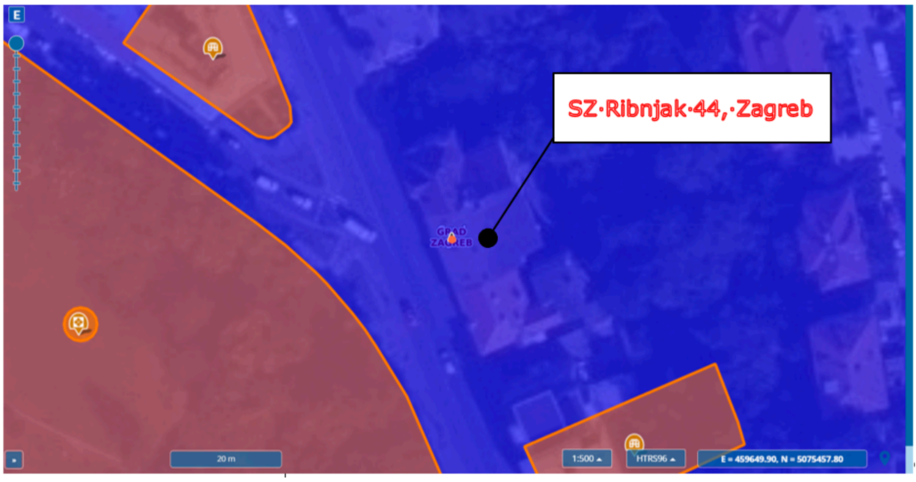
Figure 2. Flowchart with the methodology for structural interventions proposed by [16,19]. Reproduced with permission from Lourenço, P.B., 2022.

Comprehensive and in-depth knowledge of the structure and material characteristics, state of damage, and crack pattern with the causes is a prerequisite for adequate and correct rehabilitation of the structure. It is only once an accurate diagnosis of the structure's damage stated has been determined that adequate prevention and rehabilitation measures can be effectively conducted. Incorrect diagnosis may lead to inadequate strengthening recommendations, which may be more harmful [20]. This methodology has been used for the assessment of several buildings [17,19,21–26]. The research conducted by Ademović [17] and Ademović et al. [21] applied this methodology in the seismic assessment of a typical residential building constructed in Sarajevo (Bosnia and Herzegovina) in 1957. Iterative procedure in the view of various strengthening procedures was investigated in [22]. In this work, three strengthening proposals were given: the addition of new walls, the addition of new walls and a tie, and the addition of new wall and fiber reinforced polymers (FRP) on the ground floor. It was noted that the inclusion of ties provides a higher capacity of the global structure in relation to the strengthening by FRP. However, this did not solve the issue of localized damage on the ground floor, which was fixed with the inclusion of FRP. The mode of failure transformed from shear failure to bending damage, being a more tolerable failure mode for masonry structures. The recommended iterative process proposed in the ICOMOS methodology was implemented for the assessment of the historical structures of the Monastery of Jerónimos in Lisbon [19]. From the gathered results, it was proposed to conduct additional on-site experimental investigations and additional monitoring. This is all with the goal to obtain as much information as possible that leads to the application of the least invasive and most adequate strengthening measures. It has been emphasized that a correct and adequate choice of strengthening methodology and techniques in cultural heritage buildings requires an iterative approach. In [23], the Cathedral of Porto dating back to the middle of the 12th century was investigated. The absence of a suitable preliminary diagnosis was recouped with excessive multidisciplinary activity during the execution period. The work conducted by Lourenço et al. [24] presents all the above-mentioned steps for a building from the 1930s. The case study highlights the importance of suitable methodologies that allow a clear understanding of the behavior of complex structures; correct assessment of the need for structural strengthening; and analysis of the economic implications of different strengthening interventions in the specific case. The instrumental case of Julianos Church in Umm el-Jimal [25] elaborates on the influence of the local construction material and techniques that have been used for construction on the arrangement of complex roofing structures. The data obtained from comprehensive historical research is combined with structural engineering reconstruction leading to the generation of 2D and 3D digital reconstruction models. The archaeological complex of "Chokepukio" in Cusco and the church "San Pedro Apostol de Andahuaylillas" were assessed in the same city [26] with the application of a modern scientific approach organized in an iterative manner and phases. Combining aerial photogrammetry with in-situ operational modal analysis and IR thermography is beneficial as more precise information is obtained regarding geometry, material, and state-of-damage. This information is then used as input data for numerical modeling. In this specific case, equivalent static nonlinear analyses were conducted. The obtained failure patterns interpreted correctly can lead to less invasive strengthening interventions [26].

3. Case Study-Building in the Street Ribnjak

The building considered as a case study in this paper is located on the eastern side of Ribnjak street in Zagreb, in a building row. It was constructed at the end of 1904. In 1904, the cluster construction method was prescribed, together with semi-open and open-mode construction [27]. This residential building (Ribnjak 44) is located in the area of the protection zone of the cultural property of the Republic of Croatia, within the area of the cultural-historical complex of the city of Zagreb, Protection Zone—A (Figure 3a). This zone represents an area of extremely well-preserved and particularly valuable historical structures. By valorization, the protection zone 'A' has been established for urban settlements or their parts with pronounced urban-architectural, cultural-historical, landscape, or ambient values of emphasized significance for the narrower and wider picture of the city, with a preserved architectural structure of high monumental value. It is interesting to mention that the famous Villa Peroš is located at Ribnjak 46 which belongs to the built structure "in the group", which together with the buildings at house number 42 and 44 forms a series of one-story houses in accordance with the so-called cluster construction method. This represents a transitional form from the closed (Lower Town block) to the open way of building in the northern areas. More details about the history of Villa Peroš and damage after the 2020 Zagreb earthquake can be found in [28].

A complete preservation procedure of the historic urban structure, spatial and landscape features, and individual buildings are applied in this zone [29]. The register of all cultural assets that have been damaged in the Zagreb 2020 earthquake in the protection zone area A is shown in Figure 3b.



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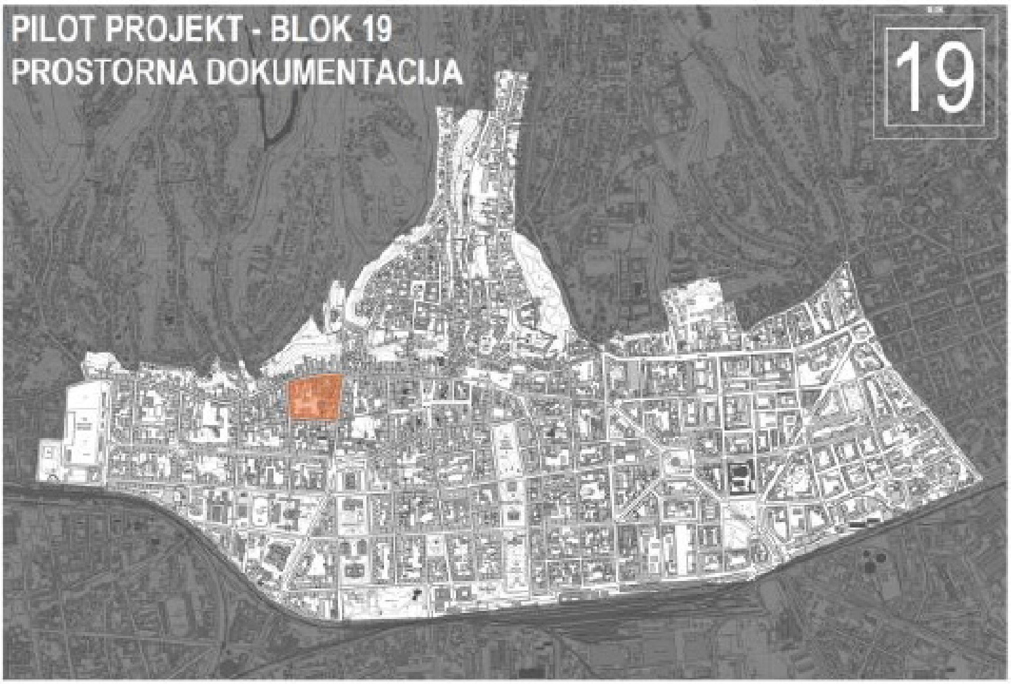


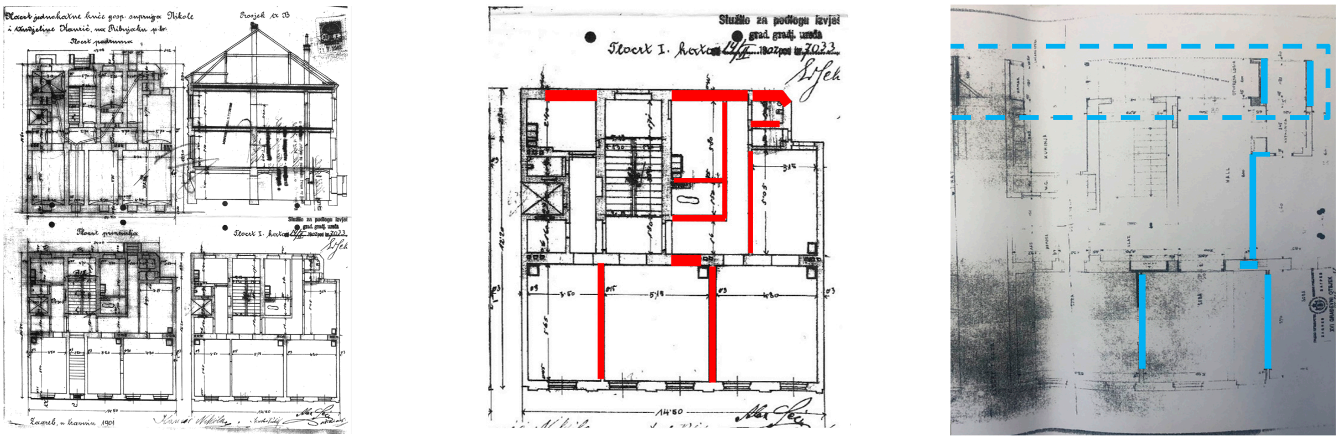
(**b**)

Figure 3. (**a**) Location of the elaborated building—Zone A; (**b**) register of cultural assets-damages in a plan. Data source: Geoportal of cultural assets, Ministry of culture and media, 2020. Prepared for the needs of the complete reconstruction of the historical urban area of the city of Zagreb—authors of the study, Urban model of the reconstruction, Faculty of Architecture—Department of Urban Planning, Spatial Planning and Landscape Architecture [30]. Reproduced with permission from Institute for Spatial Planning of the City of Zagreb, Program cjelovite obnove povijesne urbane cjeline grada Zagreba, prijedlog za javnu raspravu 22_ožujak 2022, 2022.

3.1. Historic Information

As the building under investigation is a building of cultural-historical importance, it is of the utmost importance to follow the procedure explained previously. Historic buildings, regardless of their importance (famous monuments or so-called "minor"), characterize a vital part of our cultural heritage [31] that have to be preserved as testimonies of our past for future generations.

First of all, it was necessary to obtain the archive drawings from the State Archives in Zagreb (Figure 4a–c). As can be seen from the drawings, the design was created in 1901. Since its construction, the building has undergone several reconstructions and upgrades. The first one was conducted in 1937 when several walls were demolished and a new lodge in the courtyard was constructed (Figure 4b,c). Demolition of the façade load-bearing walls looking at the courtyard additionally decreased the loadbearing capacity of the structure in the X direction. To make a wider open space, the construction of the new walls did not follow the position of the existing walls on the ground floor causing the change in load transfer. The reconstruction of the apartments was not synchronized causing a change in the global behavior of the structure. In 1998, several reconstructions were conducted on the first floor, while in 2015, in the apartments, several steel girders have been constructed. The demolishment of the longitudinal walls, as will be proved later on by the calculations, caused a lot of difficulties in the reconstruction process. A new lodge was constructed (see Figures 4c and 5b) founded on new steel columns. The constructed lodge is not adequately connected to the existing structure, the steel columns do not have adequate foundations, and this additionally modified the behavior of the structure.



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

Figure 4. Investigated buildings Ribnjak 44: (**a**) original drawings from the State Archives in Zagreb; (**b**) reconstruction-demolished walls shown in red color State Archives in Zagreb; (**c**) construction of new walls and a lodge [32]. Reproduced with permission from State Archives in Zagreb, 2022.

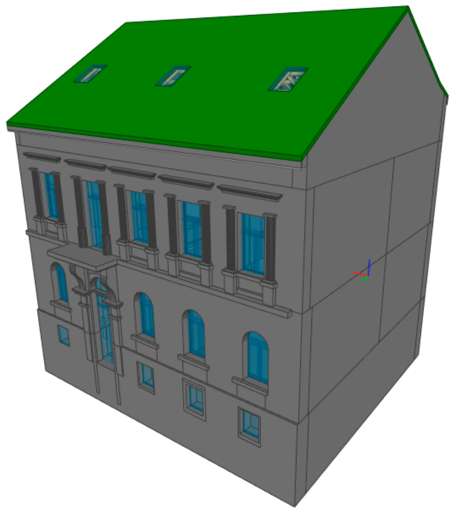
(**a**) (**b**)

Figure 5. Investigated buildings Ribnjak 44: (**a**) street view (building in the middle); (**b**) courtyard view [32]. Reproduced with permission from Projekt na kvadrat d.o.o., 2022.

The present state of the unreinforced masonry (URM) buildings is presented in Figure 5a,b. It is a row building (the yellow one) located between two buildings of the same height (on the north and the south side). The building has a regular shape with dimensions 14.5 × 17.20 m in plan, the floor plan of the building is 217 m^2, and the gross construction area is equal to 799.07 m^2. The structure consists of a basement, two storeys (ground floor and first floor), and an attic.

Due to the undergone changes, and the fact that not all these changes and upgrades have been documented, it was of the utmost importance to make a point cloud laser scanning of the building in order to obtain the correct geometry of all the elements of the buildings. Laser scanning imaging was used to form a point cloud of the exterior and the interior of the entire building. The 3D point cloud and 3D vector digital model of the existing building were constructed as presented in Figure 6a,b. The height of the basement floor is 3.48 m, the height of the ground floor is 4.10 m, and the height of the first floor is 4 m. The height of the attic varies and the one looking at the street is 1.1 m while the one towards the courtyard is 0.5 m (Figure 6b). The load-bearing walls are built with solid "old format" brick (29 × 14 × 6.5 cm). Load-bearing walls (longitudinal and transverse) have different thicknesses and range from 30 to 75 cm. The largest thickness is in the basement and the smallest is in the attic.

(**a**)



(**b**)

Figure 6. (**a**) 3D point cloud; (**b**) 3D vector digital model of the existing building. Reproduced with permission from GO2BIM d.o.o. Zagreb, 2022.

The ceilings of the basement are made of brick vaults, supported on load-bearing walls and brick arches, while the ceilings of the ground floor and first floor are made of wooden beams. One concrete beam with dimensions 45/50 cm was identified on the first floor above the newly constructed lodge.

3.2. Description of the Current State of the Building

Based on the preliminary (quick) assessment of the damage to the building, an assessment of the carrying capacity and stability was conducted along with the first risk assessments and the need to evacuate the building in the event of an emergency repeated earthquake. Inspection and assessment of damage to the building after the earthquake was carried out according to the EMS-98 classification for brick buildings. It was marked with a yellow label and PN1 (temporarily unusable) meaning that a detailed inspection is required. PN1 means that the building has moderate damage with no danger of collapse. The bearing capacity of the building is partially violated. It is not recommended to stay in the building, that is, citizens stay at their own risk in such a building. A shorter stay in the building is possible, with the advice of the structural experts related to the necessary measures and the restriction of stay (depending on the danger). A structural expert makes recommendations to eliminate hazards.

3.3. Visual Inspection of the Buildings and Crack Pattern Survey

A detailed visual inspection of the building was performed on 28 June 2021. During the inspection, minor to moderate damage was noted. There was no damage observed in the basement. The most pronounced damage to the structure was observed at a gable wall, as it partially collapsed (Figure 7a). On the roof, the rotation of the purlins towards the outer part of the wall was observed. This is visible in several purlins where the rotation of the purlin located at the corner is minimal, whereas the maximum rotation is observed on the seventh purlin (Figure 7b). The chimney collapsed and immediately after the earthquake the roofing has been rehabilitated.

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

(**d**) (**e**) (**f**)

Figure 7. (**a**) Partial collapse of a gable wall; (**b**) oration of purlin's; (**c**) damage in the spandrels; (**d**) cracks at the contact of the ceiling; (**e**) vertical crack at the contact of two walls; (**f**) vertical crack along the wall [32]. Reproduced with permission from Projekt na kvadrat d.o.o., 2022.

The interpretation of the crack pattern can be of substantial assistance in understanding the structure's state of damage, its possible causes, and the type of survey to be performed [33]. Cracks were observed on the largest number of lintels, both vertical and diagonal in one direction and cross (diagonal) cracks (Figure 7c). Moreover, minor diagonal and X-pattern cracks are visible in the walls at the lintels (above the doors) and parapets between the windows on the façade. These are minor cracks that indicate that there has been an exceedance of the shear strength, but since the width of the cracks is small, it enables the transfer of forces through friction. Additionally, vertical cracks were observed on the lintels near the openings. These are mostly minor cracks that are located at the top and bottom of the walls. The appearance of these smaller cracks in most cases is caused by arch action and indicates that the tensile strength has been exceeded. This represents characteristic damage to such structures. Some of the cracks were repaired after the earthquake. In any case, it is necessary to determine the exact width of the cracks and, depending on the degree of damage, the decision will be made regarding an appropriate rehabilitation method (injection of crack or installation of spiral reinforcement bars). The observed horizontal cracks, which were recorded at the bottom and top of the walls between the openings, occur mainly due to stress concentration along the corners of the opening. These are mostly minor cracks that can be rehabilitated with the injection process.

The floors of the ground and first floor of this URM building are made of light and flexible timber so the appearance of cracks in the ceiling structure was expected, as most probably there is no good connection between the floors and the wall and between the walls (Figure 7d,e). This is an additional parameter that makes these structures vulnerable to earthquakes. More severe damage was noted in wooden ceilings, load-bearing walls, and connection between the load-bearing walls and the ceiling. The cracks extend throughout the entire height of the wall at the connection of the two walls Figure 7e and along the height of the wall Figure 7f.

In this case, as the floor is flexible, the loads on the walls are transferred in relation to their surface, and not as per their stiffness as in the case of the structures with rigid floors which exhibit a "box" behavior [21]. The appearance of horizontal cracks located at the level of the floors and the gable wall was also observed. Generally, when it comes to very small movements (which is not the case here) sliding occurs on the surfaces between the wall and the floor structures, and in this case, it is a damage of a minor degree. In case the cracks are followed by dislocations in the value of several mm, it is a serious slip between the floor structure and the wall. In this case, it is necessary to carry out appropriate rehabilitation strengthening of the floor structure, and appropriate connection of the floor and the walls.

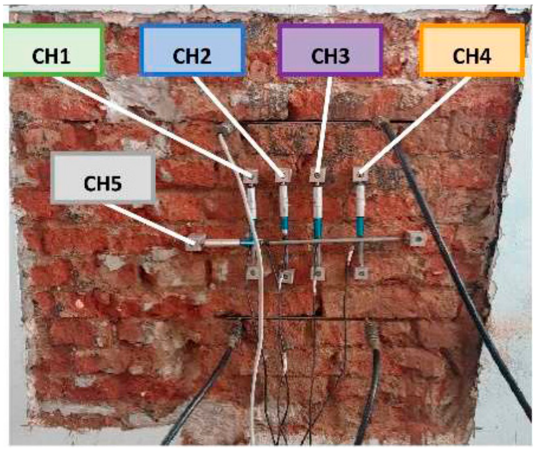
During the preparation for the flat-jack testing, the type of masonry and its units were determined, as well as the quality of mortar. As is usually the case in these types of structures, bricks are found to be in a good shape and of good quality, while the mortar is of very bad quality.

Damage was observed to the steel columns located on the western façade (courtyard side). One of the columns was displaced from its original position, and under the second column damage to the concrete part was observed. All the damages were photographed and described in detail which became a part of the report on the assessment of the existing state of the building structure which was conducted by this team [32].

3.4. Onsite Testing

Accurate diagnosis results from comprehensive on-site and laboratory experimental tests. In cultural-heritage buildings, the on-site tests should be non-destructive as much as possible, or at least minor destructive [31,33]. Identification of geometry, details, and materials of the structural elements (type of masonry units, types of floors, geometry, connections between elements, joints, etc.) was carried out in accordance with the provisions defined in points C.1 and C.2 of the standard EN 1998-3 [34].

To define the in-situ compressive stress level as well as deformability characteristics of masonry (modulus of elasticity), the flat-jack technique [35–37] was employed. Tests were carried out in the basement walls, with one wall in the longitudinal and the other in the transversal direction. A test layout is presented in Figure 8a and the measurement of the stress-strain relationship is shown in Figure 8b. It is seen that the load-bearing walls are built with “old format” solid bricks (29 × 14 × 6.5 cm), assembled with lime mortar which was a common material used in the late 19th and at the beginning of the 20th century.



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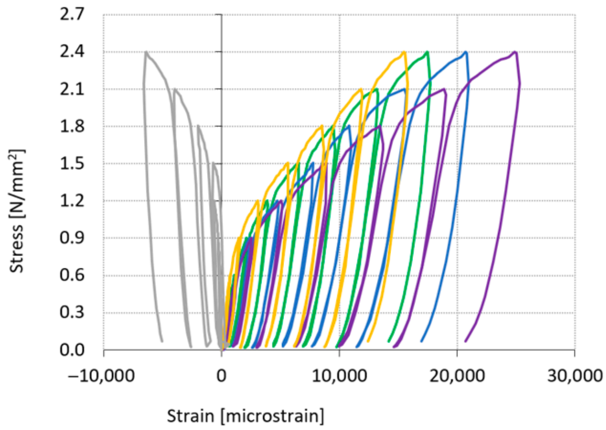


(**b**)

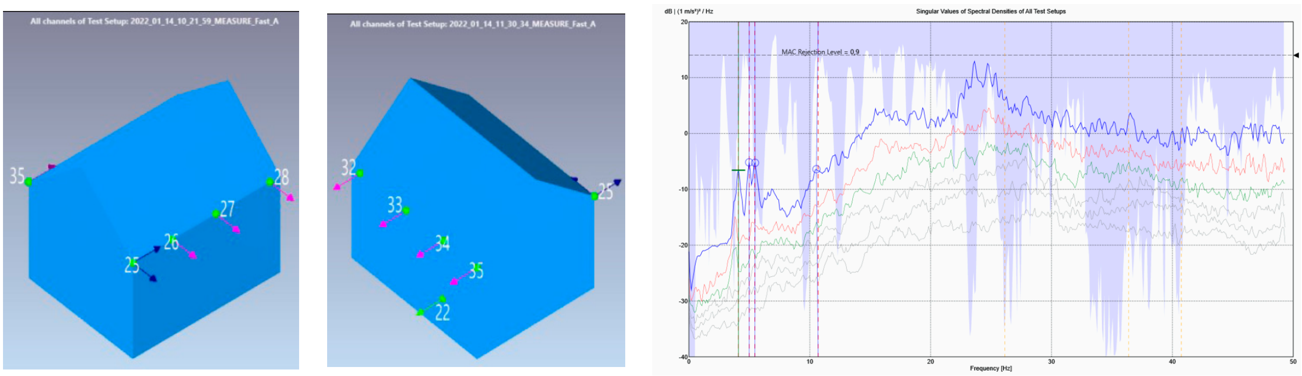
Figure 8. (**a**) 3D Sensors’ layout; (**b**) Stress-strain relationship measured at different sensors.

The tests were made in accordance with the ASTM C1197-14a [38]. The maximum attained stress values were in the range of 2.4 to 2.7 N/mm^2. Maximum values were limited also for the great deformability encountered in masonry. The modulus of elasticity at the first and second sampling positions were in fact 554 and 470 N/mm^2 at the beginning of the loading paths (between 0.00 and 0.30 N/mm^2), and tests were carried out almost until material plasticization or strong Young’s modulus reduction of masonry. The obtained values are rather low compared to the suggested values found in the literature, and this may be explained by the relatively thick joints and by a very deformable mortar. It has also to be stressed that maximum tests values do not correspond to the material strength, since the tests induced “compaction” in the material (seen in the hardening effect at the end of the second test) and that almost no cracks—indicating masonry crushing—were noticed throughout the tests. This once again emphasizes the importance of on-site testing and the fact that each building represents a case study of its own and has to be separately examined. The values that were selected as input in the numerical modeling were cautiously defined as f_m = 2.2 N/mm^2 for the compressive strength of masonry and the modulus of elasticity E = 500 N/mm^2.

As the floors are made of wooden beams, it was necessary to determine their dimensions and direction. The direction of the beams was from east to west. Dimensions of the wooden beams on the first floor and the attic were 14 × 24 cm placed 0.80 m and 0.85 apart, respectively, with a 2.4 cm thick wooden plank over them. The built-in timber is of good quality. The load is carried in one direction by these types of floors. Several steel beams were noted at the ceiling of the first floor where the removal of the walls took place during the 2015 reconstruction. The dimensions of all openings (doors and windows) were accurately determined by the laser scanner technique.

Operational modal analysis (OMA) was used to define the dynamic features of the investigated structure (natural frequencies, mode shapes, and damping) as a means of verification of the structural behavior and integrity of a building. The obtained information was used for the calibration of the numerical model. OMA determines the modal properties of a structure based on vibration data collected when the structure is under its operating conditions. It is as well-known as output-only modal analysis, ambient modal identification, and in-operation modal analysis [39]. Data are elaborated with ARTEMIS

MODAL 7.2.0.0/2022 licensed software, using enhanced frequency domain decomposition (EFDD) method. This method enables the determination of the damping ratios which is not the case with the frequency domain decomposition (FDD). Additionally, the accuracy of dynamic identification is higher compared to FDD. It is able to estimate closely spaced modes with good accuracy [39]. The vibrations were measured on two different setups (1–2). Sensors are placed to identify global modes acquiring the vibrations mainly at the attic level, at the building corners, and on the long side. Reference sensors no.1 and 2 (in blue) are in the S–W corner of the building—ch. 1 dir. N-S (Y), ch. 2 dir. E–W (X) sensors no.3–6 moved from the South to the North side (Figure 9a). Sensor no.7 was used only in setup 2, at a lower level (Figure 9b). The singular value decomposition of the acquired data is shown in (Figure 9c).



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

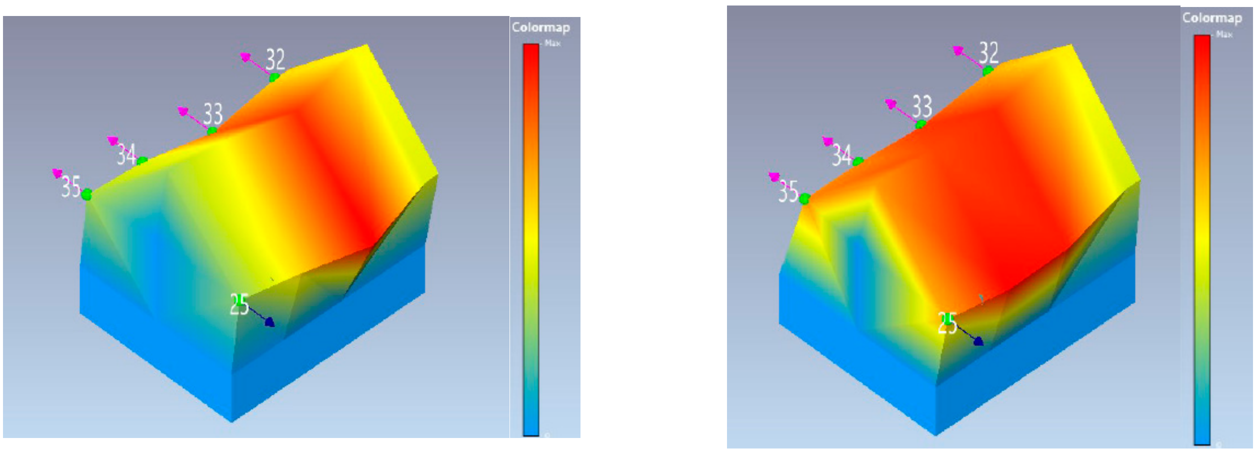
Figure 9. (**a**) Set up no. 1; (**b**) Set up no. 2; (**c**) Singular value decomposition of the acquired data.

The obtained measured data for the first two modes is given in Table 1.

Table 1. Identifies measured dynamic properties.

Mode	Frequency [Hz]	Damping [%]
1	4.09	3.23
2	4.99	2.52

The first two modes were out-of-plane modes, the first one in bending and the second one translational (Figure 10a,b) both in the Y direction.

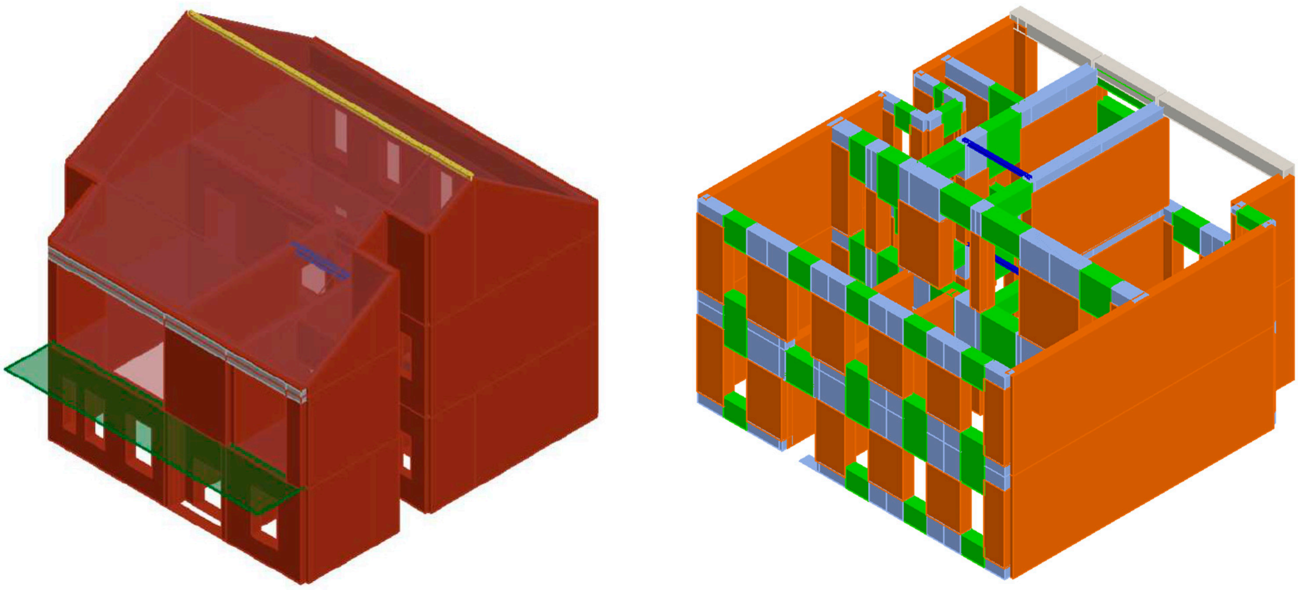


(**a**) (**b**)

Figure 10. (**a**) 1st mode; (**b**) 2nd mode.

4. Numerical 3D Model of the Existing Buildings

It was decided to construct a 3D model in 3Muri software [40] based on a macro-element approach. This software has been used by numerous researchers [17,21,41–51] and many others. The benefits of this approach are its simplicity, adequate precision, low computational complexity, and computational unpretentiousness [17,52,53]. The non-linear response is determined in masonry macro-elements, composed of piers and spandrels and rigid elements that connect the piers and the spandrels; for more details, see [40]. The 3D model of the building is presented in Figure 11a,b. It was decided to model the building as a single building because there was no information regarding the characteristics of the adjacent buildings and the location of their load-bearing walls. It is clear that this is a conservative approach, and if the structure was to be looked at as being a part of the aggregate there would be stiffness and resistance increase [54].



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

Figure 11. (**a**) The 3D model; (**b**) 3D equivalent frame of the elaborated building.

The non-linear static pushover analysis became a popular tool for the seismic assessment of existing and new structures. It provides adequate information on seismic demands imposed by the design ground motion on the structural system and its components. It is a performance-based methodology, which is based on constant gravity loads and an incremental increase of the horizontal force distribution on a structure. An envelope of all the responses derived from the non-linear dynamic analysis is obtained as a result that represents the structural behavior [21]. Both load distributions which are available in 3MURI (uniform pattern and modal pattern) were elaborated. It was seen that the modal pattern distribution (horizontal loads proportional to the first vibration mode shape) prevails. For the existing structures and the type of cracks, it was appropriate to choose the Turnsek–Cacovic law.

4.1. Geometry and Materials

The exact geometrical data was obtained through laser scanning and this was implemented in the model. Regarding the material characteristic, the input data used was the one obtained from the flat-jack tests, while the other data required for the input was calculated according to the suggested formulas provided in the literature and based on engineering judgment. Masonry mechanical properties that were taken in the analysis are given in Table 2. The damage conditions were treated as existing taking into account stiffness reduction.

Table 2. Masonry mechanical properties.

Mechanical Property	Abbreviation	Value
Compressive strength of masonry	f_m	2.2 N/mm^2
Specific weight of masonry	γ_m	18 kN/m^3
Modulus of elasticity	E	500 N/mm^2
Shear modulus	G	200 N/mm^2

Concrete class C20/25 was chosen for the concrete element with the mean values as proposed in Eurocode 2 [55]. Reinforcement for the tie beams was taken to be equal to B420 with a characteristic yield strength of 420 N/mm^2. During the visual inspection, it was noted that the timber was of good quality and the strength class was determined based on engineering judgment. The one-way timber floor with a single wood plank of 2.4 cm was selected for the floor structure having a strength class of C24 according to EN 338:2016 [56] meaning that the type of timber is softwood having a bending strength of 24 N/mm^2 in the major axis. This material is considered new. The newly constructed lodge is supported by steel columns made of structural steel S235 with a modulus of elasticity of E = 210,000 N/mm^2 and yield strength of 235 N/mm^2.

4.2. Conducted Analysis and Their Results

A new Law on Reconstruction of Earthquake-Damaged Buildings in the city of Zagreb, Krapina-Zagorje County, and Zagreb County [57] was enforced after the 2020 Zagreb earthquake. The law clearly stated which return periods have to be taken into account for certain locations and levels of strengthening. In this specific case, in order to check the state of the structure, the return period of 475 years representing the limit state of significant damage (SD) and the 95 years return period representing the limit state of damage limitation (DL) was examined. Regarding the strengthening procedure, Level 3 which envisaged strengthening the building to the return period of 225 years which corresponds to a probability of exceedance of 20% in 50 years was selected. So, in Table 3, PGA values for different limit states are presented.

Table 3. Peak ground accelerations for various return periods for the selected location.

Return Period [Years]	PGA
95	0.128 g
225	0.183 g
475	0.255 g

According to the latest geological research of the city of Zagreb, the soil is classified as soil type C. The seismic analysis was conducted in line with EN 1998-1 [58] and EN 1998-3 [34]. The building belongs to the importance class III (buildings whose seismic resistance is of importance in view of the consequences associated with a collapse) [58] and the recommended value of the importance factor is 1.2. According to the available data from the original drawings, conducted laser scanning, and on-site experimental tests, Knowledge Level 2 was assumed with the confidence factors (CF) equal to 1.2.

As in any calculations, first of all, static analyses are conducted. Due to very low masonry characteristics, many of the elements had already bending damage and did not possess adequate capacity to take over the gravity loads as indicated in Figure 12. According to the calculation, 32 out of 75 elements did not possess adequate vertical load resistance. This had a direct effect on the seismic capacity of the walls.

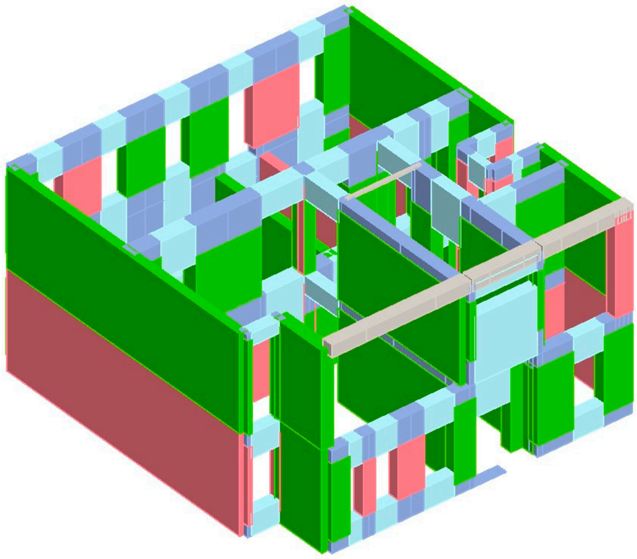


Figure 12. Damage stated due to static loads.

The second calculation which was conducted was the modal analysis which provides information regarding the eigenfrequencies and eigenmodes of the structure. Here, only the dominant global modes in the Y (mode 1) and X (mode 2) directions are given in Table 4.

Table 4. Modal analysis information.

Mode	Frequency [Hz]	Period [s]	Mx [%]	My [%]	Mz [%]
1	3.41	0.29317	0.08	74.58	0
2	2.49	0.40236	24.83	0	0

A rather good consistency is obtained between the measured and modeled eigenfrequency in the Y direction by which the valuation of the model was performed.

In order to correctly implement the nonlinear static pushover analysis, several parameters have to be correctly chosen. First of all, the correct choice of the seismic load pattern is required; secondly, the selection of the correct control node is required for the optimization of the numerical convergence. Finally, representative displacement is to be considered in the pushover curve [59]. This is of great importance when the structures are irregular and with flexible diaphragms, which is the case in this building. In this case, both uniform and modal load distribution were elaborated together with various levels of eccentricities leading to 12 pushover analyses in the X and 12 pushover analyses in the Y direction. The correct choice of the control node in-plane was a very sensitive task as the results are a function of different stiffnesses and strengths of masonry walls. The control node was selected in the wall that had the largest displacement, which would first collapse [59]. An average displacement of all nodes at the same level, weighted by the seismic nodal mass, was chosen instead of the displacement of the control node as proposed by [59]. This provides a generalized explanation of the structure behavior, independent from the different stiffnesses and strengths of the walls, identifying a single failure point.

As a result, capacity curves are obtained for all the cases as presented in Figure 13. The structure has rather inferior resistance in the X direction, the appearance of the first crack was observed already at 95.70 kN, while the maximum reached load was 204.17 kN. The structure has rather superior resistance in the Y direction, with the first crack occurring at the force 13.30 times larger, and the maximum load reached was equal to 1468.35 kN, being 7.19 times larger than in the X direction. This can be connected to a large number of openings and a much lower percentage of load-bearing walls in the X direction.

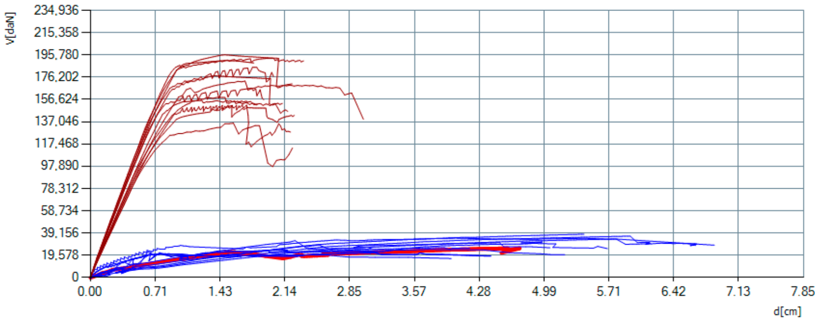


Figure 13. Capacity curves in X and Y directions.

The type of damage at the maximum displacement capacity for X and Y directions is presented in Figures 14 and 15a. The predominant damage in the walls is caused by bending (pink). A rather similar situation is observed in the Y direction; however, additionally, one of the walls experienced shear failures (orange color) and bending failure (red color) (Figure 15b). The bending damage above the windows in the lintels is clearly observed on the façade wall (Figure 15c) and the visible cracks on the walls due to bending have been presented in Figure 7f.

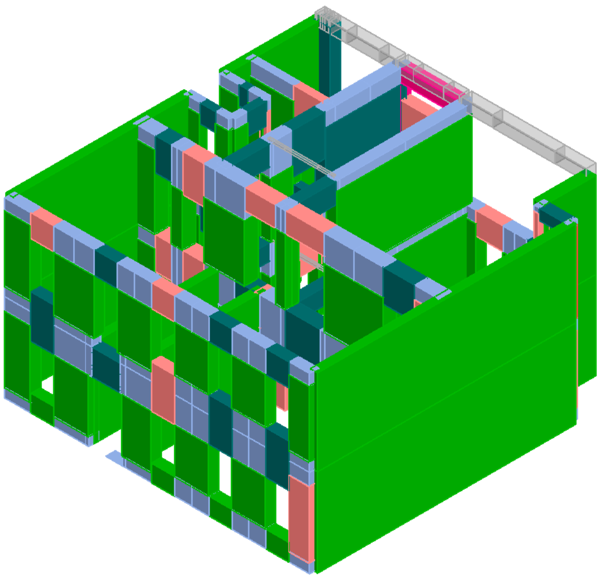
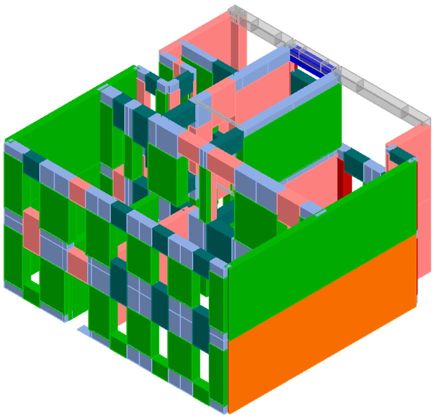


Figure 14. Damage at maximum displacement capacity for pushover in the X direction.

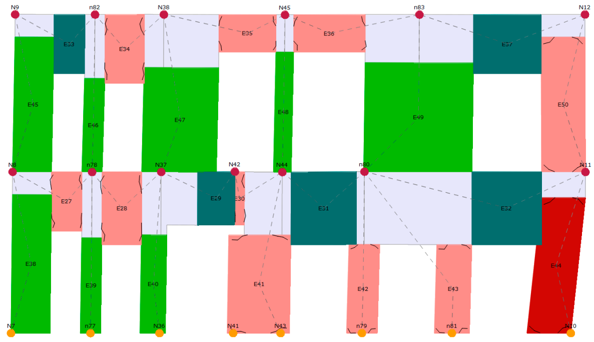
According to the obtained results, not even one of the twenty-four conducted analyses fulfilled the requirements. The parameter (α) is used for the seismic vulnerability evaluation. It is defined by Equation (1).

$$\alpha_{PGA} = PGA/a_{gR} \quad (1)$$

where: PGA represents the peak ground acceleration and a_{gR} limit capacity acceleration of the given building, which represents the ratio between the limit capacity acceleration of the given building and the reference PGA. For the representative analysis in the X and Y directions, these values were α(SD) = 0.373, α(DL) = 0.311 and α(SD) = 0.521, α(DL) = 0.771, respectively. This means that the limit state of significant damage (SD) for the return period of 475 years and the limit state of limited damage (DL) for the return of 95 years was exceeded requiring the strengthening of the existing structure.



(**a**)

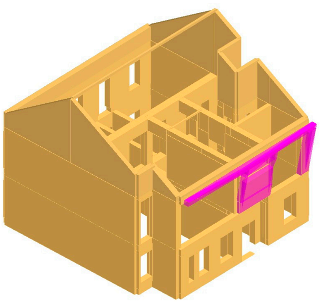


(**b**)

(**c**)

Figure 15. Damage at maximum displacement capacity for pushover in the Y direction; (**a**) shear failure of the wall: (**b**) bending failure of the wall; (**c**) bending damage above the windows in the lintels.

Besides checking the global in-plane behavior of masonry structures, it is as well necessary to check the out-of-plane behavior which is basically the first mode of failure in traditional URM structures, especially the ones with flexible diaphragms. The timber floors are usually not well connected to the walls and the connection of the walls is not constructed in a proper way, not providing a "box" behavior of the structure and leading to out-of-plane failures. As expected, the façade wall looking at the courtyard did not pass the out-of-plane verification (Figure 16). The failure mode is consistent with the 1st eigenfrequency mode.



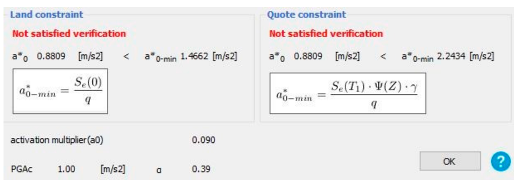


Figure 16. Local mechanism out-of-plane failure of the façade wall.

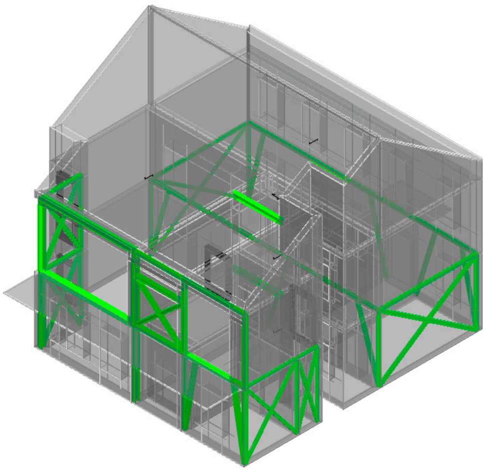
5. Numerical 3D Model of the Strengthened Buildings

As already mentioned in Subsection 4.2, Level 3 as defined in [57] was chosen as the adequate level for strengthening this masonry building. Modeling was conducted in

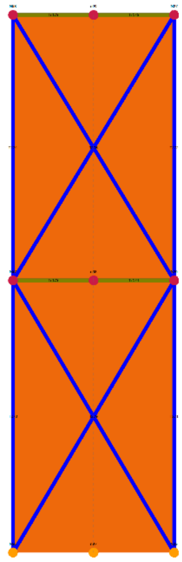
3Muri with the inclusion of the strengthening elements. Due to the very low mechanical characteristics of the masonry, several strengthening measures had to be taken in order to improve the seismic resistance of the existing building. First of all, it was necessary to add a new steel moment frame structure inside the existing masonry building with the goal to increase the horizontal resistance, as presented in Figure 17a. This has been identified as one of the most popular methods due to its several advantages such as its reversibility, which is one of the requirements provided in the Charter of Venice [60], fast and easy construction, strength and stiffness increase, and minimal spatial disruption. Additionally, with this kind of strengthening there is no significant increase in the building self-weight [61]. In their work, Karantoni and Sarantitis [62] investigated the influence of constructing steel and concrete frames every 2 m at the floor level on masonry walls. It was noted that this kind of strengthening procedure increases the shear strength and decreases the bending deflection. Steel frames can be connected or not connected to the existing masonry load-bearing structure. Papalou [63] investigated how the masonry structure behaves once the steel frames are connected to the masonry walls. Special care in the modeling procedure was devoted to the connections as they have been considered as hinges not permitting the transfer of the bending moment. Improvement of the seismic behavior is obtained once a close space connection of the steel frame with the masonry wall is created opposed to the situation if a gap is left between the steel structure and the load-bearing wall. Besides the bare frames in several locations, braced frames were envisaged as more rigid elements. In this way, the load-bearing of the whole structure has been increased as well as its ductility. This is foreseen in the location of full masonry walls without openings not affecting the physical and visual access Figure 17b.

During renovations, very often new perforations are created for new windows or doors, affecting the strength and stiffness of the wall and the seismic behavior of the whole structure. Recently, only several researchers conducted studies in this domain. Proença et al. [64] and Billi et al. [65] focused on experimental and numerical modeling of steel ring-frame around the openings. The inclusion of the steel frame affects the mode of failure. Once a steel ring-frame has been installed, the mechanism of solid masonry changes from a shear mechanism to a mixed shear-rocking mechanism with a spread cracking pattern [64]. In order to reestablish the wall's stiffness, it is necessary to use a steel profile with a very large moment of inertia. In-plane strength and ductility will be considerably increased only if a perfect connection between the wall and the steel frame is established [65]. A very comprehensive and in-depth experimental and numerical analysis in line with the work conducted by [64,65] was conducted by Oña Vera et al. [66]. It was shown that it is quite acceptable to select a steel profile with a smaller moment of inertia and their choice was (HEA140) which was capable of re-establishing the in-plane lateral load and the displacement capacity of the elaborated solid wall. Maximal wall stiffness that could be restored amounts to 50% of the original wall regardless of the steel profile size [66]. This was taken into account in the steel profile selection for strengthening the openings (Figure 17c).

The second measure consists of strengthening the walls with FRCM, a material that has been introduced recently for strengthening masonry and concrete structures. This material has pushed aside FRP as the usage of polymeric resins showed to be mechanically, physically, and heritage preservation incompatible with the existing masonry structures [67]. Contrary to FRP, the FRCM uses a thick layer of inorganic plaster as opposed to polymer resins which are used in FRP. The chemical and physical properties of FRCM composites are very similar to masonry, especially in the case of Basalt and AR-glass fibers. This material in the literature is known by other names such as fibre reinforced mortar (FRM) and textile reinforced mortar (TRM). The experimental investigation and numerical modeling of FRCM are still ongoing [68–74] with several applications in practice [75,76]. In our case, some walls were strengthened on both sides and some only on one side. The number of strengthened layers varied depending on the level of damage and capacity of the walls Figure 17d.



(**a**)



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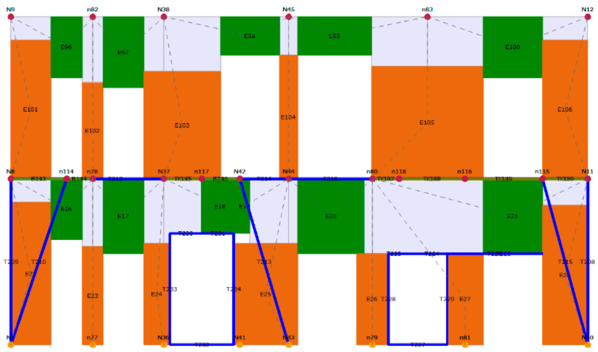

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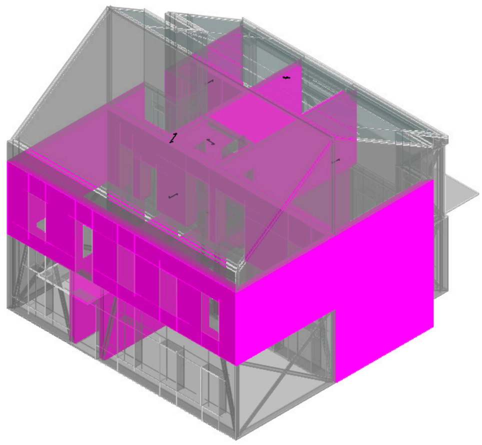
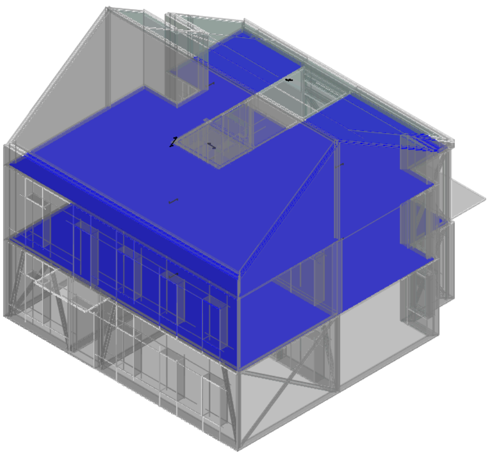
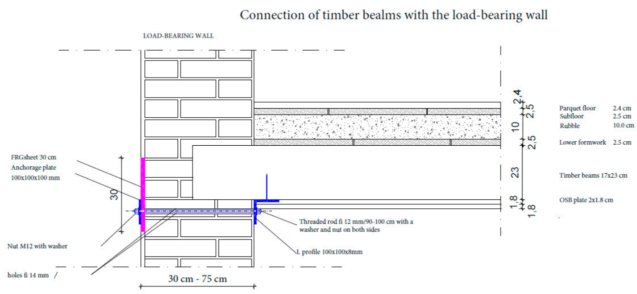


(**d**)



(**e**)



(**f**)

Figure 17. 3Muri 3D strengthened model (**a**) steel structure; (**b**) braced frame; (**c**) steel ring-frame around the doors; (**d**) Strengthening of the walls with FRCM; (**e**) Strengthening of the walls with OSB; (**f**) Connection of the timber beams with load-bearing walls.

One of the main flaws of traditional timber floors is their low in-plane stiffness and lack of effective connections to the load-bearing masonry walls leading to out-of-plane local failures [77]. The decision on which kind of strengthening technique will be selected was guided by the fact that many researchers indicated the important decisive role of the in-plane stiffness of timber diaphragms, and emphasized that disproportionate stiffening of the floors could even have a negative effect on the seismic behavior of the existing building [78–81].

Strengthening and replacement of timber slabs with reinforced concrete slabs have been proven to be unproductive and even worsen the behavior of buildings made of low masonry quality as reported by on-site aftermath earthquake investigations in Italy [82–84]. Using a thin concrete layer would increase the in-plane stiffness if properly connected; however, additional weight would be added consequently increasing the seismic loads on the existing structure.

Gubana and Melotto [82] conducted comprehensive experimental testing of oriented strand board (OSB) panels and it was noted that there was an increase of the initial secant stiffness associated with the top displacement of about six times regardless of the panel orientation in relation to the joists. The initial stiffness of the floor increased seven times with the application of screws for fastening the OSB panels. The increase of strength was from six to nine times with the application of OSB panels which were connected with nails, while with the application of screws, the strength increased 15 times [82], both connection types being reversible and minimally invasive as requested per [60]. This all gives a good basis for the proposal of this kind of strengthening of the URM buildings in seismic-prone areas. In that respect, to improve the in-plane stiffness of the slabs, but still keep the same behavior of the structure, was the purpose to strengthen the timber slabs with OSB having a thickness of 18 mm and laid in two layers (Figure 17e). In order not to open the slab from the upper side as the apartments are all occupied and in usage, it was proposed to construct the strengthening from the lower side of the slab. This strengthening procedure has several benefits, from its low mass and thin thickness to irreversibility and minimum invasiveness, which is in agreement with the requirements for cultural-heritage buildings [60].

Connection of the existing timber slab with the walls will be conducted with the application of the L steel profiles as presented in Figure 17f. It is well known that increasing the in-plane stiffness of the floor has the largest impact on the improvement of the seismic behavior of the existing traditional masonry building. This enables the "box" behavior of the structure (to some degree) and the transfer of the loads to the walls according to their stiffness.

If the crack width is relatively small (e.g., less than 10 mm) and the wall thickness is relatively small, cracks that occur may be sealed with injection material. This was selected as a strengthening measure for walls with small cracks. Around the crack, it is necessary to remove the plaster (preferably traditional, not using electric tools, in order to avoid vibrations and their negative impact on the walls), the area around the crack and inside the cracks has to be cleaned, the cracks will be sealed with grouting material, and a new layer of plaster will be applied.

After several iterations, the final strengthening procedures were adopted.

Examples of the proposed strengthening measures are shown in Figures 18–20.

The capacity curves in the X and Y directions of the strengthened existing URM structure are presented in Figure 21. The first crack is noted at the value of 1152 kN in the X direction while the ultimate load reached 1345 kN. In the Y direction, the structure has a much larger capacity, and the occurrence of the first crack is noted at 2423 kN and the ultimate load reached 2549 kN.

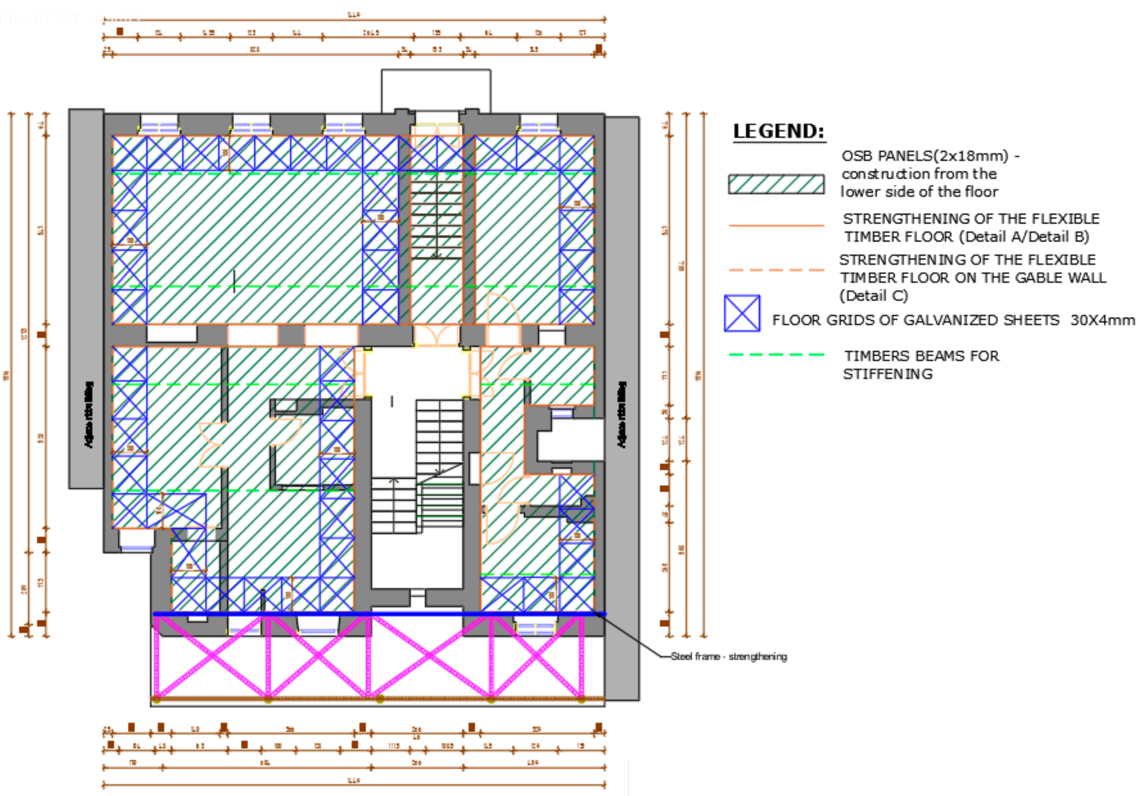


Figure 18. Strengthening measures of the floor at the ground floor level.

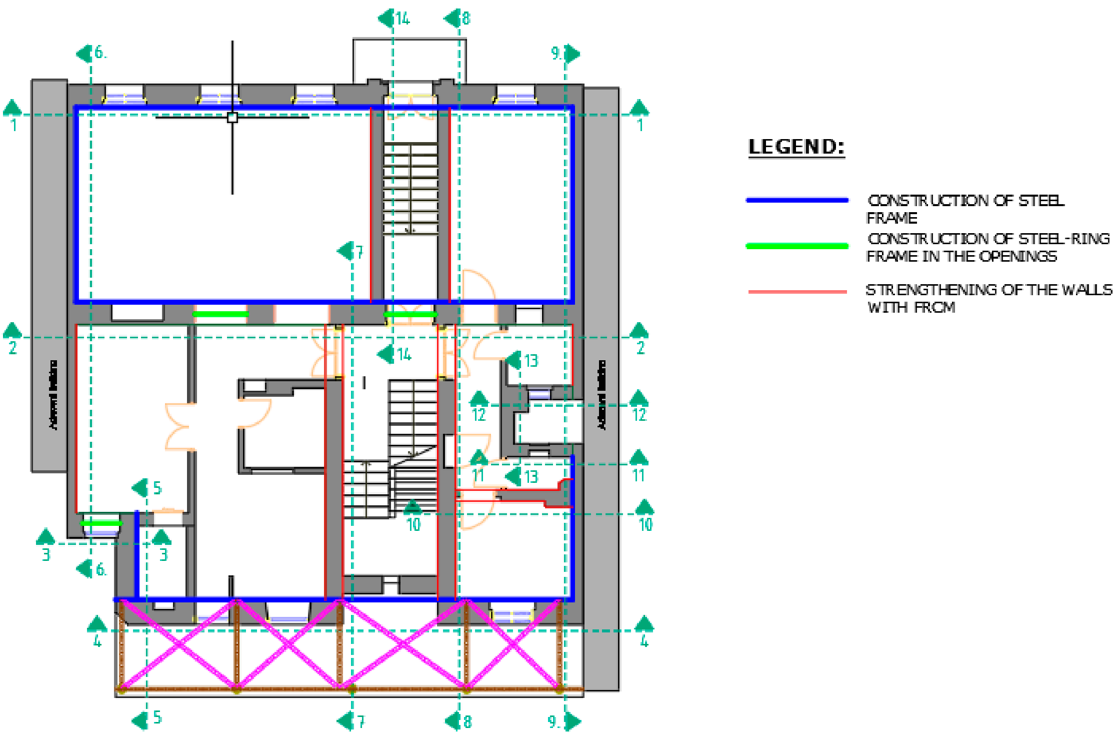


Figure 19. Strengthening measures of the walls at the ground floor level.

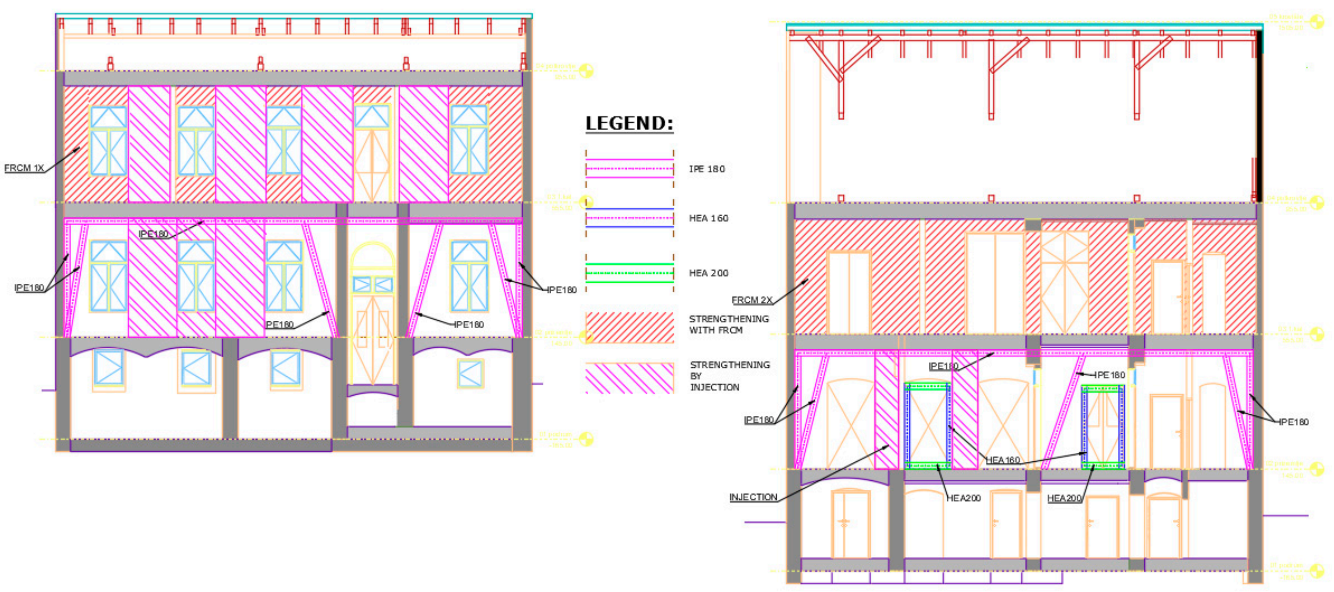


Figure 20. Cross-sections 1-1 and 2-2, Strengthening measures.

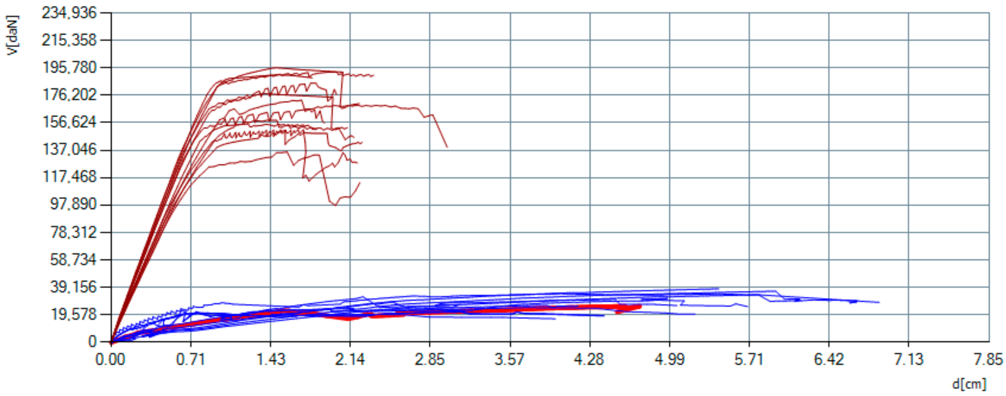


Figure 21. Capacity curves in the X and Y directions of the strengthened structure.

The modal parameters of the strengthened structure are presented in Table 5.

Table 5. Modal analysis information—strengthened structure.

Mode	Frequency [Hz]	Period [s]	Mx [%]	My [%]	Mz [%]
1	4.06	0.24648	0	82.47	0
2	2.87	0.34845	92.06	0	0

For the representative analysis in the X and Y directions, these values were α(SD) = 1.188, and α(SD) = 1.068, respectively. This means that the limit state of significant damage (SD) for the return period of 225 years was satisfied and that the strengthening methodologies are effective.

6. Comparison of Results

In order to see the effectiveness of the proposed strengthening methods, it is feasible to compare the capacity curves of the unstrengthened and strengthened existing URM structure under consideration. With the application of all given strengthening procedures, it was possible to increase the seismic capacity of the building in both directions. The inferior X direction was now upgraded so that the occurrence of the first crack happened at a load that is 12.04 times larger than for the unstrengthened structure. The increase in initial stiffness in the X direction was 6.4 times, while in the Y direction it was 1.4 times. Additionally, the ultimate reached load in the X direction is more than 6.5 times larger compared to the unstrengthened structure. The ultimate load reached in the Y direction increased by 75% in relation to the unstrengthened structure. It is important to state that the global eigenmodes did not change significantly and this is beneficial, as there should not be a major difference in this aspect. It was observed that now the mode shape in the X direction is translational, while in the unstrengthened structure there were unequal movements of the nodes. In this way, better structural behavior was obtained. The strengthened structure passed all the seismic verification. The value of α was greater than 1 for all the conducted analyses and limit states. The effect of the strengthening is shown in Figure 22 for the two load distribution patterns.

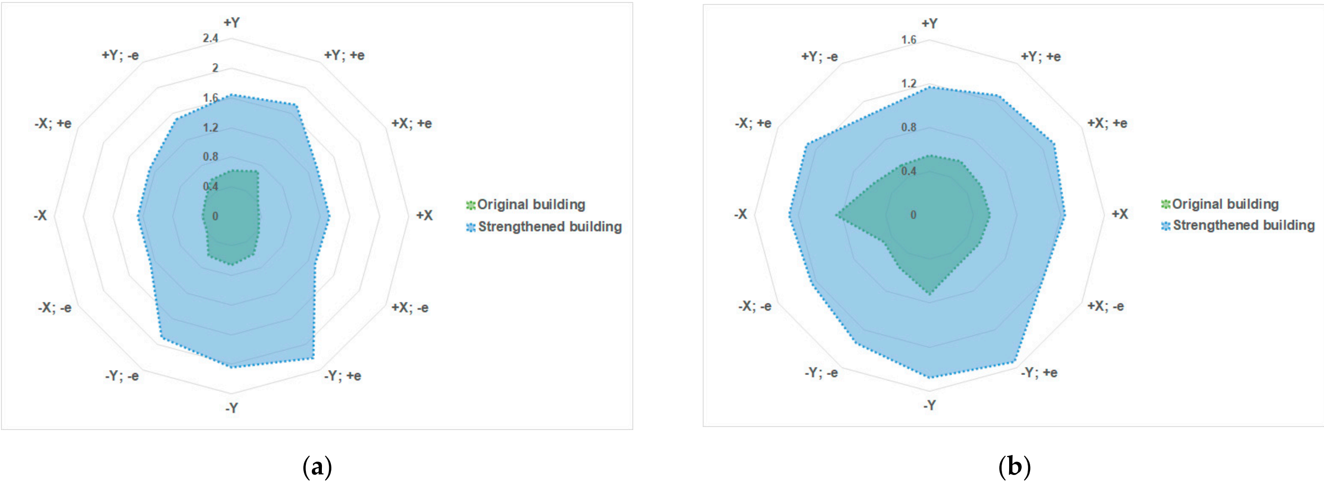


Figure 22. Comparison of α values for the original building and strengthened building for (**a**) uniform load distribution; (**b**) modal load distribution.

7. Conclusions

Earthquakes are one of the most devastating natural forces that cause damage to structures, human fatalities, and economic loss. Several variables affect the level of damage on different buildings, such as structural type, type of soil, maintenance, buildings' age, and many others. The aftermath of the 2020 Zagreb earthquake was more than 25,000 damaged buildings mainly located in the historic part of the city of Zagreb. An example of the strengthening procedures which follow the principles of ICOMOS and the Venice Charter have been suggested for the damaged building in Ribnjak 44 street, in order not to jeopardize the cultural character of the building, which would be done if invasive strengthening techniques were proposed. In this study, the aim was to strengthen the building to Level 3 (225 years return period) as per the Law on Reconstruction by Earthquake-Damaged Buildings in the city of Zagreb, Krapina-Zagorje County, and Zagreb County. The recommended strengthening procedures were the inclusion of a new steel frame, steel ring-frames, strengthening with FRCM, and injection of the cracks. The behavior of the buildings being located in a building row (clustered construction method) or as aggregates is affected by numerous parameters which leads to rather complex behavior. Due to the lack of information regarding the adjacent buildings, it was decided to model the building as a single building. In that respect, a perfect match between modeling and experimental data was not obtained; however, the obtained results were more than adequate and acceptable.

The study intended to show the behavior of a cultural-historical building exposed to the 2020 Zagreb earthquake. After a detailed visual inspection, a crack pattern was created, identification of damage was performed, and material characteristics and global behavior of the structure were determined by Ambiental investigations, leading to numerical modeling and proposed strengthening methods respecting the principles of ICOMOS and the Venice Charter.

During the modeling of the building, it was necessary to overcome several issues. First of all, it was necessary to select an adequate modeling strategy, which in this case was the equivalent frame model, substantially the one used by engineers in practice. The benefits are seen in its modeling simplicity, ease of application, and limited computational effort. The next challenge was how should the Ribnjak 44 building be modeled, as a single building or with the entire cluster taken into account. Due to a lack of information about the adjacent buildings, it was decided to model the structure as a single building, taking special care regarding the interpretation of the results and their comparison with the in-situ investigations. In order to overcome this issue, it would be feasible to conduct a simplified procedure for seismic vulnerability assessment of masonry building aggregates as proposed by [56] and model the entire cluster if time and cost permit such activity to be conducted.

Masonry construction is one of the most common construction typologies not only in Europe but in the world and even in zones of significant seismic hazard. Brickwork masonry structures with timber floors are widely geographically distributed throughout most urban and rural settlements in Europe with some specific features for each region. The paper provides a comprehensive example of the selected strengthening methods of a cultural-historic URM building in a seismically active area, avoiding overly invasive interventions that do not jeopardize the historical value of the architecture, which can find its application in different European countries.

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References

1. Stepinac, M.; Lourenço, P.B.; Atalić, J.; Kišiček, T.; Uroš, M.; Baniček, M.; Šavor Novak, M. Damage classification of residential buildings in historical downtown after the ML5.5 earthquake in Zagreb, Croatia in 2020. *Int. J. Disaster Risk Reduct.* **2021**, *56*, 102140. [CrossRef]
2. Uroš, M.; Todorić, M.; Crnogorac, M.; Atalić, J.; Šavor Novak, M.; Lakušić, S. *Potresno Inženjerstvo—Obnova Zidanih Zgrada*, 1st ed.; Građevinski fakultet Sveučilišta u Zagrebu: Zagreb, Croatia, 2021; pp. 1–594.

3. Torbar, J. *Report on the Zagreb Earthquake of November 9, 1880*; Book I; Jugoslovenska Akademija Znanosti i Umjetnosti: Zagreb, Croatia, 1882; pp. 1–144. (In Croatian)
4. Zagreb City Museum. Available online: https://www.mgz.hr/hr/dogadanja/tematska-vodstva-stalnim-postavom---potres-u-zagrebu-1880-godine,3324.html (accessed on 15 September 2022).
5. Croatian Bureau of Statistics. 2011. Available online: https://web.dzs.hr/default_e.htm (accessed on 28 October 2022).
6. Ademović, N.; Kalman Šipoš, T.; Hadzima-Nyarko, M. Rapid assessment of earthquake risk for Bosnia and Herzegovina. *Bull. Earthq. Eng.* **2020**, *18*, 1835–1863. [CrossRef]
7. Kalman Šipoš, T.; Hadzima-Nyarko, M. Seismic Risk of Croatian Cities Based on Building's Vulnerability. *Technical Gazette* **2018**, *25*, 1088–1094. [CrossRef]
8. PTP2 (1948) Provisional technical regulations (PTP) for loading of structures, part 2, no. 11730, 12 July 1948—PTP2. *Official Gazette of FNRY*, 17 June 1948; No. 61/48.
9. PTP-GuSP64 (1964) Provisional technical regulations for construction in seismic regions. *Official Gazette of SFRY*, 19 September 1964; No. 39/64.
10. Drysdale, R.G.; Hamid, A.A.; Baker, L.R. *Masonry Structures: Behaviour and Design*, 2nd ed.; The Masonry Society: Boulder, CO, USA, 1999; pp. 1–809.
11. Uroš, M.; Šavor Novak, M.; Atalić, J.; Sigmund, Z.; Baniček, M.; Demšić, M.; Hak, S. Post-earthquake damage assessment of buildings-Procedure for conducting building inspections. *Građevinar* **2021**, *56*, 1089–1115. [CrossRef]
12. Milić, M.; Stepinac, M.; Lulić, L.; Ivanišević, N.; Matorić, I.; Čačić Šipoš, B.; Endo, Y. Assessment and Rehabilitation of Culturally Protected Prince Rudolf Infantry Barracks in Zagreb after Major Earthquake. *Buildings* **2021**, *11*, 508. [CrossRef]
13. Moretić, A.; Stepica, M.; Lourenço, P.B. Seismic upgrading of cultural heritage-A case study using an educational building in Croatia from the historicism style. *Case Stud. Constr. Mater.* **2022**, *17*, e01183. [CrossRef]
14. Official Site of Matica Hrvatska. Available online: https://www.matica.hr (accessed on 30 October 2022).
15. Hafner, I.; Lazarević, D.; Kišiček, T.; Stepinac, M. Post- Earthquake Assessment of a Historical Masonry Building after the Zagreb Earthquake—Case Study. *Buildings* **2022**, *12*, 323. [CrossRef]
16. ICOMOS International Council on Monuments and Sites. Recommendations for the Analysis, Conservation and Structural Restoration of Architectural Heritage. Available online: https://www.icomos.org/en/about-the-centre/179-articles-en-francais/ressources/charters-and-standards/165-icomos-charter-principles-for-the-analysis-conservation-and-structural-restoration-of-architectural-heritage (accessed on 16 September 2022).
17. Ademović, N. Structural and Seismic Behavior of Typical Masonry Buildings from Bosnia and Herzegovina. Master's Thesis, University of Minho, Guimarães, Portugal, 2011.
18. Casarin, F. Structural Assessment and Seismic Vulnerability Analysis of a Complex Historical Building. Ph.D. Thesis, University of Padova, Padova, Italy, 2006.
19. Lourenço, P.B.; Krakowiak, K.J.; Fernandes, F.M.; Ramos, L.F. Failure analysis of Monastery of Jerónimos, Lisbon: How to learn from sophisticated numerical models. *Eng. Fail. Anal.* **2017**, *14*, 280–300. [CrossRef]
20. Binda, L.; Modena, C. *Lectures in Advanced Masters in Structural Analysis of Monuments and Historical Constructions*; Slides 2012; University of Padova: Padova, Italy, 2012; pp. 1–150.
21. Ademovic, N.; Hrasnica, M.; Oliveira, D.V. Pushover analysis and failure pattern of a typical masonry residential building in Bosnia and Herzegovina. *Eng. Struct.* **2013**, *50*, 13–29. [CrossRef]
22. Ademović, N.; Hrasnica, M. Seismic Assessment and Strengthening of an Existing Multi-Storey Masonry Building in Sarajevo, Bosnia and Herzegovina. In Proceedings of the 2nd European Conference on Earthquake Engineering and Seismology, Istanbul, Turkey, 25–29 August 2014.
23. Lourenço, P.B.; Melo, A.; Carneiro, M. Remedial measures for Cathedral of Porto: Apost-modern conservation approach. In *Structural Analysis of Historical Constructions IV*, 1st ed.; Modena, C., Lourenço, P.B., Roca, P., Eds.; A.A. Balkema Publishers: Leiden, The Netherlands, 2005; Volume 1, pp. 51–62.
24. Lourenço, P.B.; Lourenço, J.B.; Oliveira, D.V. Inspecção e reabilitação doSeminário Conciliar de Braga. In Proceedings of the BE 2004—Encontro Nacional de Betão Estrutural, Universidadedo Porto, Porto, Portugal, 17–19 November 2004; pp. 815–822.
25. Al Rabady, R.; Rababeh, S. Engineering the reconstruction of Hawrān's Ecclesiae during late antiquity: Case of Julianos church in Umm el-Jimal, Jordan. *Herit. Sci.* **2022**, *10*, 81. [CrossRef]
26. Aguilar, R. Applications of modern technologies for the seismic assessment of heritage constructions in Peru. In Proceedings of the 6th World Conference on Earthquake Engineering, Santiago, Chile, 9–13 January 2017; Paper No. 2445. pp. 1–11.
27. Kahle, D. Building Code for the City of Zagreb between 1850 and 1918. *Prostor* **2004**, *12*, 203–216.
28. Gazivoda, N.; Sironić, M. "Osobni dosjei" vile Frangeš i vile Peroš prije i nakon potresa 2020. godine u Zagrebu. *Rad. Inst. Povij. Umjet.* **2020**, *44*, 103–124. [CrossRef]
29. Odluka o izmjenama i dopunama Odluke o donošenju Prostornog plana Grada Zagreba. 2014. Available online: https://www1.zagreb.hr/zagreb/slglasnik.nsf/rest-akt/467ebf4469afb49ac1257d96004f65dd?Open (accessed on 3 July 2022).
30. Program Cjelovite Obnove Povijesne Urbane Cjeline Grada Zagreba, Prijedlog za Javnu Raspravu 22_ožujak 2022, Zavod za Prostorno Uređenje Grada Zagreba, in Croatian. Available online: https://www.zzpugz.hr/wp-content/uploads/2022/03/Program_obnove_prijedlog.pdf (accessed on 5 August 2022).

31. Binda, L.; Saisi, A. *Knowledge of the Building, on Site Investigation and Connected Problems*; Cosenza, E., Ed.; Eurocode 8 Perspectives from the Italian Standpoint Workshop; Doppiavoce: Napoli, Italy, 2009; pp. 213–224.
32. Čizmar, D.; Ademović, N.; Toholj, M.; Radović, D. *Elaborat ocjene postojećeg stanja građevinske konstrukcije*; Report on the state of the structure; Projekt nakvadrat d.o.o.: Zagreb, Croatia, February 2022. (In Croatian)
33. Binda, L.; Cardani, G.; Cantini, L.; Tiraboschi, C. On site and laboratory detection of the quality of masonry in historic buildings. In Proceedings of the International Symposium on Studies on Historical Heritage, Antalya, Turkey, 17–21 September 2007; pp. 667–674, ISBN 978-975-461-433-6.
34. *EN 1998-3:2005+AC:2010*; Eurocode 8: Design of Structures for Earthquake Resistance—Part 3: Assessment and Retrofitting of Buildings. European Union: Brussels, Belgium, 2005. Available online: https://www.phd.eng.br/wp-content/uploads/2014/07/en.1998.3.2005.pdf (accessed on 10 September 2022).
35. Cescatti, E.; Dalla Benetta, M.; Modena, C.; Casarin, F. Analysis and evaluations of flat jack test on a wide existing masonry buildings sample. In *Brick and Block Masonry—Trends, Innovations and Challenges*, 1st ed.; Modena, C., da Porto, F., Valluzzi, M., Eds.; Taylor and Francis Group: London, UK, 2016; pp. 1485–1491.
36. Simões, A.; Bento, R.; Gago, A.; Lopes, M. Mechanical Characterization of Masonry Walls With Flat-Jack Tests. *Exp. Tech.* **2016**, *40*, 1163–1178. [CrossRef]
37. Gregorczyk, P.; Lourenço, P.B. A Review on Flat-Jack Testing. *Engenharia Civil.* **2000**, *9*, 39–50.
38. *ASTM C1197-14a*; Standard Test Method for in situ Measurement of Masonry Deformability Properties using the Flatjack Method. American Society for Testing and Materials: West Conshohocken, PA, USA, 2020.
39. Zahid, F.B.; Ong, Z.C.; Khoo, S.Y. A review of operational modal analysis techniques for in-service modal identification. *J. Braz. Soc. Mech. Sci. Eng.* **2020**, *42*, 398. [CrossRef]
40. S.T.A. DATA. 3 Muri Program 12.5.0. Available online: http://www.stadata.com (accessed on 15 September 2021).
41. Ademović, N.; Oliveira, D.V.; Lourenço, P.B. Seismic Evaluation and Strengthening of an Existing Masonry Building in Sarajevo, BiH. *Buildings* **2019**, *9*, 30. [CrossRef]
42. Formisano, A.; Ademovic, N. An overview on seismic analysis of masonry building aggregates. *Front. Built Environ.* **2022**, *8*, 966281. [CrossRef]
43. Ademović, N.; Oliveira, D.V. Seismic Assessment of a Typical Masonry Residential Building in Bosnia and Herzegovina. In Proceedings of the 15th World Conference on Earthquake Engineering, Lisboa, Portugal, 24–28 September 2012.
44. Simões, A.; Bento, R.; Cattari, S.; Lagomarsino, S. Seismic performance-based assessment of "Gaioleiro" buildings. *Eng. Struct.* **2014**, *80*, 486–500. [CrossRef]
45. Simões, A.; Milosevic, J.; Meireles, H.; Bento, R.; Cattari, S.; Lagomarsino, S. Fragility curves for old masonry building types in Lisbon. *Bull. Earthq. Eng.* **2015**, *13*, 3083–3105. [CrossRef]
46. Formisano, A.; Marzo, A. Simplified and refined methods for seismic vulnerability assessment and retrofitting of an Italian cultural heritage masonry building. *Comput Struct.* **2017**, *180*, 13–26. [CrossRef]
47. Chieffo, N.; Fasan, M.; Romanelli, F.; Formisano, A.; Mochi, G. Physics-Based Ground Motion Simulations for the Prediction of the Seismic Vulnerability of Masonry Building Compounds in Mirandola (Italy). *Buildings* **2021**, *11*, 667. [CrossRef]
48. Valluzzi, M.R.; Sbrogiò, L.; Saretta, Y. Intervention Strategies for the Seismic Improvement of Masonry Buildings Based on FME Validation: The Case of a Terraced Building Struck by the 2016 Central Italy Earthquake. *Buildings* **2021**, *11*, 404. [CrossRef]
49. Lagomarsino, S.; Degli Abbati, S.; Ottonelli, D.; Cattari, S. Integration of Modelling Approaches for the Seismic Assessment of Complex URM Buildings: The Podestà Palace in Mantua, Italy. *Buildings* **2021**, *11*, 269. [CrossRef]
50. Chieffo, N.; Formisano, A.; Mochi, G.; Mosoarca, M. Seismic vulnerability assessment and simplified empirical formulation for predicting the vibration periods of structural units in aggregate configuration. *Geosciences* **2021**, *11*, 287. [CrossRef]
51. Lamego, P.; Lourenço, P.B.; Sousa, M.L.; Marques, R. Seismic vulnerability and risk analysis of the old building stock at urban scale: Application to a neighbourhood in Lisbon. *Bull. Earthq. Eng.* **2017**, *15*, 2901–2937. [CrossRef]
52. Mouyiannou, A.; Rota, M.; Penna, A.; Magenes, G. Identification of suitable limit states from nonlinear dynamic analyses of masonry structures. *J. Earthq. Eng.* **2014**, *18*, 231–263. [CrossRef]
53. Aşıkoğlu, A.; Vasconcelos, G.; Lourenço, P.B.; Pantò, B. Pushover analysis of unreinforced irregular masonry buildings: Lessons from different modeling approaches. *Eng. Struct.* **2020**, *218*, 110830. [CrossRef]
54. Formisano, A.; Florio, G.; Landolfo, R.; Mazzolani, F.M. Numerical calibration of an easy method for seismic behaviour assessment on large scale of masonry building aggregates. *Adv. Eng. Softw.* **2015**, *80*, 116–138. [CrossRef]
55. *EN 1992-1-1*; 2015 Eurocode 2: Design of Concrete Structures, Buildings. European Union: Brussels, Belgium, 2004. Available online: https://www.phd.eng.br/wp-content/uploads/2015/12/en.1992.1.1.2004.pdf (accessed on 10 August 2022).
56. *EN 338:2016*; Structural Timber—Strength Classes. European Standard: Brussels, Belgium, 2016.
57. Law on the Reconstruction of Earthquake-Damaged Buildings in the City of Zagreb, Krapina-Zagorje County and Zagreb County (NN 102/2020). Available online: https://www.zakon.hr/z/2656/Zakon-o-obnovi-zgrada-o%C5%A1te%C4%87enih-potresom-na-podru%C4%8Dju-Grada-Zagreba,-Krapinsko-zagorske-%C5%BEupanije,-Zagreba%C4%8Dke-%C5%BEupanije,-Sisa%C4%8Dko-moslava%C4%8Dke-%C5%BEupanije-i-Karlova%C4%8Dke-%C5%BEupanije (accessed on 10 August 2022).
58. *EN 1998-1*; Eurocode 8: Design of Structures for Earthquake Resistance—Part 1: General Rules, Seismic Actions and Rules for Buildings. European Union: Brussels, Belgium, 2004. Available online: https://www.phd.eng.br/wp-content/uploads/2015/02/en.1998.1.2004.pdf (accessed on 10 August 2022).

59. Lagomarsino, S.; Cattari, S. *Seismic Performance of Historical Masonry Structures Through Pushover and Nonlinear Dynamic Analyses In Perspectives on European Earthquake Engineering and Seismology, Geotechnical, Geological and Earthquake Engineering*, 1st ed.; Ansal, A., Ed.; Springer: Cham, Switzerland, 2015; Volume 39. [CrossRef]
60. Charter of Venice. Decisions and resolutions. In Proceedings of the 2nd International Congress of Architects and Technicians of Historical Monuments, Venezia, Italy, 25–31 May 1964; Volume 5, pp. 25–31.
61. Giuriani, E. Consolidamento degli edifici storici [in Italian]. In *UTET Scienze Technique*; UTET: Torino, Italy, 2012; ISBN 978-8859807636.
62. Karantoni, F.; Sarantitis, D. Seismic behaviour of masonry buildings after alterations of the load bearing system. In Proceedings of the 3rd National Conference in Seismic Mechanics and Technical Seismology, Athens, Greece, 24 November 2008. (In Greek).
63. Papalou, A. Strengthening of masonry structures using steel frames. *Int. J. Eng. Technol.* **2013**, *2*, 50–56. [CrossRef]
64. Proença, J.; Gago, A.; Vilas Boas, A. Structural window frame for in-plane seismic strengthening of masonry wall buildings. *Int. J. Archit. Herit.* **2018**, *13*, 98–113. [CrossRef]
65. Billi, L.; Laudicina, F.; Salvatori, L.; Orlando, M.; Spinelli, P. Forming new steel-framed openings in load-bearing masonry walls: Design methods and nonlinear finite element simulations. *Bull. Earthq. Eng.* **2019**, *5*, 2647–2670. [CrossRef]
66. Oña, V.; Mónica, Y.; Metelli, G.; Barros, J.A.O.; Plizzari, G.A. Effectiveness of a steel ring-frame for the seismic strengthening of masonry walls with new openings. *Eng. Struct.* **2021**, *226*, 111341. [CrossRef]
67. Di Tommaso, A.; Focacci, F. *Strengthening Historical Monuments with FRP: A Design Criteria Review, Composites in Construction: A Reality*; ASCE: Reston, VA, USA, 2001; ISBN 0-7844-0596-4. [CrossRef]
68. Pantò, B.; Malena, M.; de Felice, G. Non-Linear Modeling of Masonry Arches Strengthened with FRCM. *Key Eng. Mater.* **2017**, *747*, 93–100. [CrossRef]
69. Scacco, J.; Ghiassi, B.; Milani, G.; Lourenço, P. A fast modeling approach for numerical analysis of unreinforced and FRCM reinforced masonry walls under out-of-plane loading. *Compos. B Eng.* **2020**, *180*, 107553. [CrossRef]
70. Aiello, M.A.; Cascardi, A.; Ombres, L.; Verre, S. Confinement of Masonry Columns with the FRCM-System: Theoretical and Experimental Investigation. *Infrastructures* **2020**, *5*, 101. [CrossRef]
71. Murgo, F.S.; Ferretti, F.; Mazzotti, C.A. discrete-cracking numerical model for the in-plane behavior of FRCM strengthened masonry panels. *Bull. Earthq. Eng.* **2021**, *19*, 4471–4502. [CrossRef]
72. Angiolilli, M.; Gregori, A.; Pathirage, M.; Cusatis, G. Fiber Reinforced Cementitious Matrix (FRCM) for strengthening historical stone masonry structures: Experiments and computations. *Eng. Struct.* **2020**, *224*, 111102. [CrossRef]
73. Angiolilli, M.; Gregori, A.; Cusatis, G. Simulating the Nonlinear Mechanical Behavior of FRCM-strengthened Irregular Stone Masonry Walls. *Int. J. Archit. Herit.* **2021**. [CrossRef]
74. Türkmen, Ö.S.; De Vries, B.T.; Wijte, S.N.M. Mechanical characterization and out-of-plane behaviour of Fabric Reinforced Cementitious Matrix (FRCM) overlay on clay brick masonry. *Civ. Eng. Des.* **2019**, *1*, 131–147. [CrossRef]
75. D'Ambrisi, A.; Focacci, F.; Luciano, R.; Alecci, V.; De Stefano, M. Carbon-FRCM materials for structural upgrade of masonry arch road bridges. *Compos. Part B.* **2015**, *75*, 355–366. [CrossRef]
76. Berardi, F.; Focacci, F.; Mantegazza, G.; Miceli, G. Rinforzo di un viadotto ferroviario con PBO-FRCM. In Proceedings of the 1 Convegno Nazionale Assocompositi, Milan, Italy, 25–26 May 2011. (In Italian).
77. Lourenço, P.B.; Mendes, N.; Ramos, L.F.; Oliveira, D.V. Analysis of masonry structures without box behavior. *Int. J. Archit. Herit.* **2011**, *5*, 369–382. [CrossRef]
78. Trutalli, D.; Marchi, L.; Scotta, R.; Pozza, L. Dynamic simulation of an irregular masonry building with different rehabilitation methods applied to timber floors. In Proceedings of the 6th International Conference Computational Methods in Structural Dynamics and Earthquake Engineering, National Technical University of Athens, Athens, Greece, 15–17 June 2017.
79. Borri, A.; Corradi, M. Structural engineers vs. conservators safety vs. preservation: Problems, doubts and proposals. In Proceedings of the 1st International Conference (TMM_CH) Transdisciplinary Multispectral Modelling and Cooperation for the Preservation of Cultural Heritage, Athens, Greece, 10–13 October 2018.
80. Gubana, A.; Melotto, M. Discrete-element analysis of floor influence on seismic response of masonry structures. *Proc. Inst. Civ. Eng. Struct. Build.* **2021**, *174*, 459–472. [CrossRef]
81. Trutalli, D.; Marchi, L.; Scotta, R.; Pozza, L. Seismic capacity of irregular unreinforced masonry buildings with timber floors. *Proc. Inst. Civ. Eng. Struct. Build.* **2021**, *174*, 473–490. [CrossRef]
82. Gubana, A.; Melotto, M. Experimental tests on wood-based in-plane strengthening solutions for the seismic retrofit of traditional timber floors. *Constr. Build. Mater.* **2018**, *191*, 290–299. [CrossRef]
83. Binda, L.; Gambarotta, L.; Lagomarsino, S.; Modena, C. A multilevel approach to the damage assessment and seismic improvement of masonry buildings in Italy. In *Seismic Damage to Masonry Buildings*, 1st ed.; Bernardini, A., Ed.; Balkema: Amsterdam, The Netherlands, 1999; pp. 170–195.
84. Valluzzi, M.R. On the vulnerability of historical masonry structures: Analysis and mitigation. *Mater. Struct.* **2007**, *40*, 723–743. [CrossRef]